**ROYAL DECREE / , of adopting the**

**Earthquake-Resistant Construction Standard NCSR-23**

The Standing Committee on Earthquake Resistant Standards, an inter-ministerial collegiate body, created by Decree 3209/1974 of 30 August 1974, attached to the Ministry of Transport, Mobility and Urban Agenda and within the Directorate-General of the National Geographical Institute, as set out in Royal Decree 645/2020 of 7 July developing the basic organizational structure of the Ministry of Transport, Mobility and Urban Agenda, among its functions, is entrusted with updating the Earthquake-Resistance Regulations, as provided for in Article 2(B) of Royal Decree 518/1984 of 22 February 1984, which reorganises its composition.

Until now, the current regulations are included in the 'Resistant Construction Standard: General Section and Building (NCSE-02)' adopted by Royal Decree 997/2002, of 27 September, and in the 'Earthquake-Resistant Construction Standard: Bridges (NCSP-07)' approved by Royal Decree 637/2007, of 18 May 2007. The time elapsed since their adoption requires that they be revised and updated, both for technical and regulatory reasons, in order to adapt the regulations to the current state of knowledge on seismology and seismic engineering, as well as to the regulatory reinforcement in which it must operate.

The Standing Committee on Earthquake-Resistant Standards, in exercising its functions, has drawn up a new Earthquake-Resistant Construction Standard that replaces the aforementioned standards, incorporating the most relevant aspects of European regulations for the calculation of structures, in accordance with the procedures established in the Structural Eurocodes and expanding their content with more structural typologies. The new standard establishes the technical conditions to be met by building structures and civil engineering works, so that their behaviour, in the face of seismic phenomena, avoids serious consequences for the health and safety of people, avoids economic losses and promotes the maintenance of basic services for society in cases of high intensity earthquakes.

This Royal Decree complies with the principles of necessity, effectiveness, proportionality, legal certainty, transparency and efficiency established in Article 129 of Law 39/2015, of 1 October 2015, on the Common Administrative Procedure of Public Administrations.

It complies with the principles of necessity and effectiveness, since the application of the new Earthquake-Resistant Standard represents, with respect to the previous regulations, an advance in the knowledge and determination of seismic activity and also a clarification in the use of the concepts and requirements of earthquake resistance and ductility that must be required of structures in seismic areas. The project does not distort competition in the market, but favours competition by regulating aspects not covered by the current regulations. It is also in line with the principle of proportionality, since the standard contains the rules necessary to meet the need described above, without there being any other measures which are less restrictive of rights or imposing fewer obligations on the addressees, and with the principle of legal certainty given their integration into the legal order, in full consistency with the national and European regulations in force. The principle of transparency is guaranteed, since all the information procedures indicated in the Government’s Law 50/1997 of 27 November 1997 have been complied with, and this has been published on the Spanish Government’s transparency portal. Lastly, in application of the principle of efficiency, this standard does not entail an increase in additional administrative burdens or an increase in public expenditure.

This Royal Decree is issued under the provisions in Article 149.1(13) of the Spanish Constitution, which attributes to the State competence in matters of bases and coordination of the general planning of economic activity.

The drafting process for this Royal Decree completed the procedure set out in Law 50/1997 of 27 November 1997 of the Government, and in Directive (EU) 2015/1535 of the European Parliament and of the Council of 9 September 2015 laying down a procedure for the provision of information in the field of technical regulations and of rules on Information Society services, as well as Royal Decree 1337/1999 of 31 July 1999, governing the provision of information in the area of technical rules and regulations and regulations related to information society services.

This Royal Decree is adopted at the initiative of the Standing Committee on Earthquake-Resistant Standards.

By virtue thereof, at the proposal of the Minister for Transport, Mobility and Urban Agenda and after deliberation in the Council of Ministers on the day ...

I HEREBY DECREE

**Article 1. Object**

The Earthquake-Resistant Construction Standard, NCSR-23, is hereby adopted, which establishes the essential concepts and requirements to be met by structures located in seismic areas, in Spain, in addition to compliance with the rest of the specific regulations in force regarding structures.

The structures and constructions that may be subjected to the action of earthquakes must be projected, executed and documented considering the seismic action in accordance with the provisions of the six Annexes that constitute this Earthquake-Resistant Standard and which are:

Annex 1: General rules, seismic actions and rules for construction.
Annex 2: Bridges.
Annex 3: Assessment and seismic adequacy of buildings.
Annex 4: Silos, tanks and pipes.
Annex 5: Foundations, containment structures and geotechnical aspects.
Annex 6: Towers, masts and chimneys.

Alternatively, the author of the project and the optional management may, in use of their powers, under their responsibility and prior agreement of the owner, adopt other solutions that partially or totally depart from the procedures referred to in the preceding annexes (through different calculation systems, construction arrangements, etc.), provided that it is documented that the structure complies with the requirements of this Earthquake-Resistant Standard, achieving at least equivalent services to those that would be obtained by the application of the procedures of this Standard.

**Article 2. Scope of application**

The requirements for earthquake-resistant content of this Standard apply to all projects and construction works of new buildings, bridges, towers, masts, chimneys, silos, tanks, pipes, containment structures and their foundations, as well as to the geotechnical project.

Likewise, this Standard applies to the seismic evaluation of existing buildings, and also, where appropriate, to the seismic suitability thereof, in cases where significant renovation or structural rehabilitation works are carried out or when such assessment or adaptation is required. For the purposes of this Earthquake-Resistant Standard, seismic suitability covers both the reinforcement of undamaged structures and the reinforcement of structures damaged by an earthquake.

The conditions that may necessitate the seismic assessment of individual buildings – possibly leading to seismic adequacy – fall outside the subject matter and scope of this Standard.

As appropriate, this Earthquake-Resistant Standard may also be applied in addition to other structural types not explicitly included in its scope, where they do not exist for the same specific standards or provisions, and provided that they are not expressly excluded from its scope.

**Sole transitional provision. Application to designs and works.**

The provisions of this Royal Decree shall not apply in civil works to projects whose order of drafting or study, in the field of public administrations, or commission, in other cases, had been carried out prior to the entry into force of this Royal Decree, as well as to the works carried out in the development thereof, provided that they are initiated within a period not exceeding two years from that entry into force, unless the competent public administration, or, where appropriate, the promoter, agrees that it is mandatory.

The provisions of this Royal Decree must not be applicable in the field of building to projects that have requested municipal works license or request it within nine months of the entry into force of this Royal Decree, applying in this case Royal Decree 997/2002, of 27 September 2002, adopting the Earthquake-Resistant Construction Standard: General Part and Building (NCSE-02). Such works must begin within the maximum period of effectiveness of the said licence, in accordance with its regulatory regulations, and, failing that, within a period not exceeding six months from the date of granting of the said licence. Otherwise, the projects must be adapted to the provisions of this Royal Decree.

**Sole repealing provision. Repeal of regulations**

As of the entry into force of this Royal Decree, Royal Decree 997/2002 of 27 September 2002 adopting the Earthquake-Resistant Construction Standard are repealed: General Part and Building (NCSE-02), and Royal Decree 637/2007, of 18 May 2007, adopting the Earthquake-Resistant Construction Standard: Bridges (NCSP-07).

**First final provision. Attribution of powers**

This Royal Decree has a basic character and is issued under the provisions of Article 149.1(13) of the Spanish Constitution, which confers on the State exclusive competence over the basis and coordination of the general planning of economic activity.

**Second final provision. Implementing authority**

The holder of the Ministry of Transport, Mobility and Urban Agenda is empowered to issue the necessary provisions for the development and application of the provisions of this Royal Decree.

**Third final provision. Authorisation for the updating of Appendices E, F and G of Annex 1 to the Earthquake-Resistant Construction Standard.**

The holder of the Ministry of Transport, Mobility and Urban Agenda is authorised to update the hazard map defined in Appendices E and F to Annex 1 and the list of standards referred to in Appendix G to Annex 1, where such updates are intended to bring those contents into line with the progress of the technique or with Community legislation.

**Fourth final provision. Entry into force**

This Royal Decree shall enter into force on 2 January 2024.

# ANNEX 1.

# Project of earthquake-resistant structures

# General rules, seismic actions and rules for construction

## 1 General considerations

### 1.1 Object and field of application

#### 1.1.1 Object and field of application of Earthquake- -Resistant Construction Standard

(1) The Earthquake-Resistant Standard applies to the project and construction of buildings and civil engineering works in seismic regions Its objective is to ensure that, in the event of earthquakes:

- human lives are protected;
- damage is limited; and
- structures important for civil protection remain operational.

(2) Special structures such as nuclear power plants, liquefied natural gas plants, open-sea structures and large dams are excluded from the object and field of application of this Earthquake-Resistant Standard.

(3) The Earthquake-Resistant Standard contains only those provisions that, in addition to the other applicable regulations, have to be applied for the project of structures in seismic regions.

(4) The Earthquake-Resistant Standard is divided into several Annexes (see **1.1.2** and **1.1.3**).

#### 1.1.2 Object and field of application of Annex 1

(1) Annex 1 applies to buildings and civil-engineering works projects in seismic regions. It is divided into 10 chapters, some of which are dedicated specifically to building projects.

(2) Chapter **2** of this Annex contains the basic performance requirements and test criteria applicable to buildings and civil-engineering works in seismic regions.

(3) Chapter **3** of this Annex provides the rules for the representation of seismic actions and for their combination with other actions. Certain types of structures, discussed in Annexes 2 to 5, require the complementary rules set out in these annexes.

(4) Chapter **4** of this Annex contains the general project rules that specifically concern buildings.

(5) Chapters **5** to **9** of this Annex contain the specific rules on the various materials and structural elements that specifically concern buildings, as follows:

- Chapter **5**: Specific rules for concrete buildings.
- Chapter **6**: Specific rules for steel buildings.
- Chapter **7**: Specific rules for buildings of mixed steel-and-concrete structure.

- Chapter **8**: Specific rules for wooden buildings.

- Chapter **9**: Specific rules for brickwork buildings.

(6) Chapter **10** contains the fundamental requirements and other aspects related to the project and safety, related to the isolation at the base of the structures and, specifically, to the isolation at the base of the buildings.

NB Specific rules for bridge isolation are developed in Annex 2.

(7) Appendices A and B contain additional elements related to the spectrum of elastic response of displacements. and to the displacements to be considered for analysis by pushover analysis.

Appendix C contains additional indications related to the seismic dimensioning of slabs of mixed concrete and steel beams in the junctions (knots) between beams and pillars of bending porticoes.

Appendix D contains the specifications to be included in the project documentation, in the case of buildings, regarding verifying the response of the construction to the earthquake.

Appendix E contains a table with the values of the seismic hazard benchmarks, given for a mesh of points covering the entire national territory.

Appendix G contains a list of national and international standards for consultation or reference.

NB Appendix F contains, as an additional element, an image of the hazard map obtained from interpolating the data in Appendix E. This Appendix is for information purposes, and should not be used for the direct collection of seismic hazard benchmark values.

### 1.1.3 Other Annexes to the Earthquake-Resistant Construction Standard

(1) Other Annexes to the Earthquake-Resistant Standard, complementary to this Annex, include the following:

- Annex 2 contains the specific provisions relating to bridges;

- Annex 3 contains provisions for the assessment and seismic adequacy of existing buildings;

- Annex 4 contains the specific provisions concerning silos, tanks and pipes;

- Annex 5 contains the specific provisions concerning foundations, containment structures and geotechnical aspects;

- Annex 6 contains the specific provisions regarding towers, masts and chimneys.

## 1.2 Standards for reference and consultation.

(1) The specific regulations in force must be taken into account for the application of this Earthquake-Resistant Standard.

(2) The UNE standards cited in this Earthquake-Resistant Standard should be used in the version indicated in Appendix G.

## 1.3 Assumptions

(1) The following assumptions apply:

- the choice of structural system and structure calculation procedure has been made by duly qualified and experienced staff;
- implementation is carried out by staff with the appropriate skills and experience;
- adequate supervision and quality control is ensured during the project and execution of the work, i.e. in the project offices, in the brickwork, in the floors and on the site:
- construction materials and products are used as specified in the Structural Code;
- the structure must be properly maintained;
- the structure must be used according to the project assumptions.

(2) No change to the original project is assumed to take place in the structure during the construction phase or during its subsequent life, unless adequate justification and verification is provided to the property, validated or approved by the author of the project or, if this is not possible, by another properly qualified technician. Due to the specific nature of the seismic response, this applies even in the event of changes that result in an increase in strength or structural rigidity.

## 1.4 International system of units (I.S.)

(1) In the application of this Earthquake-Resistant Standard, the units of the International System (I.S.) must be used in accordance with the provisions of Royal Decree 2032/2009, of 30 December 2009, establishing the legal units of measure, and with Royal Decree 493/2020, of 28 April 2020, which modifies the previous one.

(2) The following units are recommended for calculations:

- forces and loads: kN, kN/m, kN/m$^2$
- density: kg/m$^3$, tonnes/m$^3$
- mass: kg, tonnes
- specific weight: kN/m$^3$
- voltages: N/mm$^2$ (= MN/m$^2$ or MPa), kN/m$^2$ (= kPa)
- moments: kNm
- acceleration: m/s$^2$, g (= 9.81 m/s$^2$)

## 1.5 Terms and definitions

### 1.5.1 Common terms

(1) The terms and definitions indicated in section **1.4** of Annex 18 to the Structural Code apply.

### 1.5.2 Other terms used in this Earthquake-Resistant Standard

(1) The following terms are used in this Earthquake-Resistant Standard with the following meanings:

**behaviour coefficient:**
Coefficient used in the calculation to reduce the forces obtained from a linear analysis, in order to take into account the nonlinear response of a structure, associated with material, structural system and calculation methods.

**dimensioning by capacity:**
Calculation method in which some elements of the structural system are chosen, sized and detailed to ensure the dissipation of energy in the face of large deformations, while all other structural elements are equipped with sufficient resistance so that the chosen means of energy dissipation can be maintained.

**dissipative structure:**
Structure capable of dissipating energy through ductile hysteresis behaviour or by other mechanisms.

**dissipation areas:**
Predefined parts of a dissipative structure in which the dissipative capacities are mainly located.

NB They are also called critical areas.

**dynamically independent unit:**
Structure or part of a structure that is directly subjected to the movement of the ground, and whose response is not affected by the response of adjacent units or structures.

**importance factor:**
Coefficient related to the consequences of structural breakage (failure).

**non-dissipative structure:**
Structure projected for a particular seismic calculation situation, without taking into account the nonlinear behaviour of the material.

**non-structural element:**
An architectural, mechanical or electrical element, system or component that, either due to the lack of resistance or rigidity or the way it is connected to the structure, is not considered in the seismic load-transmitting project in the seismic loads.

NB For the particular case of enclosures and partitions, see 4.2.2.

**primary seismic elements:**
Structural elements considered as part of the structural system that resists seismic action, modelled in the analysis for the seismic situation of calculation, and completely projected and detailed constructively to resist earthquakes in accordance with the rules of this Earthquake-Resistant Standard.

NB The terms 'primary seismic element' and 'primary seismic element' are used interchangeably.

**secondary seismic elements:**

Structural elements that are not considered as part of the system resistant to seismic action and whose resistance and rigidity to seismic actions is disregarded.

NB They are not required to meet all the rules of the Earthquake-Resistant Standard, but they are constructively projected and detailed to withstand gravitational loads when subjected to displacements caused by the seismic calculation situation.

**potentially active fault:**
Fault that meets any of the following requirements:

- Its average slip rate is equal to or greater than 1 mm/year.

- There is evidence of a rupture or cosmic deformation in the surface of the ground over the past 129 000 years (from the beginning of the Upper Pleistocene, to the present day).

- It has associated seismic activity attested by the instrumental seismological record, or deduced from historical, archaeological or geological information, always within the previous 129 000 years.

NOTE 1 In any case, the maximum magnitude attributed to this failure and its recurrence period must be taken into account, so that they are consistent with the period of return of the seismic action considered in the project.

NOTE 2 The terms 'potentially active fault' and 'seismically active fault' are used in this Standard without distinction.

## 1.6 Symbols

### 1.6.1 General considerations

(1) The symbols indicated in section **1.5** of Annex 18 to the Structural Code apply. For symbols relating to materials, as well as symbols not specifically related to earthquakes, the relevant regulatory provisions apply.

(2) For ease of use, other symbols related to seismic actions are defined when they appear. However, the symbols that appear most frequently in Annex 1 are further listed and defined in the sections **1.6.2** and **1.6.3.**

### 1.6.2 Other symbols used in chapters 2 and 3 of this Annex

$A_{\mathrm{Ed}}$ calculation value of seismic action (= $\gamma_{\mathrm{I}} \cdot A_{\mathrm{Ek}}$)

$A_{\mathrm{Ek}}$ seismic action characteristic value for the reference return period

$E_{\mathrm{d}}$ calculation value of the effects of the action

$M_w$ earthquake magnitude (moment magnitude scale is used)

$N_{\mathrm{SPT}}$ number of hits of the standard penetration test (*Standard Penetration Test, SPT*)

$P_{\mathrm{NCR}}$ reference probability to exceed, in 50 years, the reference seismic action for the non-collapse requirement

$Q$ variable action

$S_{\mathrm{e}}(T)$ elastic response spectrum of horizontal ground acceleration, also called 'elastic response spectrum'. For $T$ = 0 the spectral acceleration given by this spectrum is equal to the ground acceleration calculation value in a type-A terrain, multiplied by the ground coefficient $S$

$S_{ve}(T)$ vertical ground-acceleration elastic response spectrum

$S_{De}(T)$ elastic response spectrum of displacements

$S_d(T)$ calculation spectrum (for elastic analysis)

$S$ ground coefficient

$T$ vibration period of a linear system with a single degree of freedom

$T_s$ duration of the stationary part of the seismic movement

$T_{NCR}$ baseline seismic action return period for non-collapse requirement

$a_{gR}$ maximum ground reference acceleration on a type-A terrain

$a_g$ calculation value of ground acceleration in a type-A terrain

$a_{vg}$ calculation value of the acceleration of the ground in the vertical direction

$c_u$ shear strength without drainage

$d_g$ calculation value of ground displacement

$g$ gravitational acceleration

$q$ behaviour coefficient

$v_{s,30}$ mean value of wave propagation velocity $S$ in the upper 30 m of ground profile, for unit deformation less than or equal to $10^{-5}$

$\gamma_I$ importance factor

$\eta$ damping correction coefficient

$\xi$ viscous damping quotient (in percentage)

$\Psi_{2,i}$ combination coefficient for the quasi-permanent value of a variable action $i$

$\Psi_{E,i}$ combination coefficient for a variable action $i$, to be considered when determining the effects of the calculation seismic action

### 1.6.3 Other symbols used in chapter 4 of this Annex

$E_E$ effects of seismic action

$E_{Edx}$, $E_{Edy}$ calculation values of effects due to horizontal components ($x$ and $y$) of seismic action

$E_{Edz}$ calculation value of the effects due to the vertical component of the seismic action

$F_i$ horizontal seismic force on the floor $i$

$F_a$ horizontal seismic force acting on a non-structural element (appendix)

$F_b$ shear strength at the base of the building

$H$ height of the building from the foundation or from the top of a rigid basement

$L_{max.}$ $L_{min.}$ maximum and minimum dimensions of the building floor plan, measured in orthogonal directions

$R_d$ strength calculation value

$S_a$ seismic coefficients for non-structural elements

$T_1$ fundamental building-vibration period

$T_a$ fundamental period of vibration of a non-structural element (appendix)

$W_a$ weight of a non-structural element (appendix)

$D$ displacement

$d_r$ calculation value of the collapse between floors

$e_a$ accidental eccentricity of the mass of a floor relative to its nominal position

$h$ height between floors

$m_i$ floor mass $i$

$N$ number of floors on foundation or top of a rigid basement

$q_a$ behaviour coefficient of a non-structural element (appendix)

$q_d$ behaviour coefficient for displacement

$s_i$ mass displacement $m_i$ for the fundamental mode of a building

$z_i$ mass height $m_i$ regarding the seismic-action application level

$A$ ratio between ground acceleration calculation value and gravity acceleration

$\gamma_a$ importance factor of a non-structural element (appendix)

$\gamma_d$ resistance-reserve coefficient (over-resistance) for diaphragms

$\Theta$ sensitivity coefficient of collapse between floors

### 1.6.4 Other symbols used in chapter 5 of this Annex

$A_c$ section area of a concrete element

$A_{sh}$ total area of horizontal brackets in a beam-pillar knot

$A_{si}$ total area of steel reinforcement in each diagonal direction of a coupled beam

$A_{st}$ area of a transverse reinforcement element

$A_{sv}$ total area of vertical mesh reinforcement of a wall (wall screen)

$A_{sv,i}$ total area of the intermediate rounds between the corner reinforcements arranged in one direction of the cross section of a pillar

$A_w$ total horizontal cross-section area of a wall

$\Sigma A_{si}$ sum of the areas of all oblique reinforcements in both directions, in a concrete wall reinforced with oblique reinforcements to resist shear strength due to displacement

$\Sigma A_{sj}$ sum of the areas of all vertical reinforcements of the web of a wall or additional reinforcement, arranged in the boundary elements of the wall specifically to withstand the shear strength due to displacement

$\Sigma M_{Rb}$ sum of the calculation values of the resistant moments of the beams that converge in the junction (knot) in the relevant direction

$\Sigma M_{Rc}$ sum of the calculation values of the resistant moments of the pillars that converge in the union (knot) in the relevant direction

$D_o$ diameter of the core confined in a circular pillar

$M_{i,d}$ moment at the end of a beam or pillar for the determination of shear strength in capacity dimensioning

$M_{Rb,i}$ calculation value of the sturdy moment of a beam at its end *i*

$M_{Rc,i}$ calculation value of the sturdy moment of a pillar at its end *i*

$N_{Ed}$ axial force obtained from the analysis for the seismic calculation situation

$T_1$ fundamental period of a building in the relevant horizontal direction

$T_C$ period corresponding to the upper limit of the area of constant acceleration of the elastic spectrum

$V'_{Ed}$ shear strength of a wall obtained from the analysis for the seismic calculation situation

$V_{dd}$ resistance of the pin of the vertical reinforcements of a wall

$V_{Ed}$ calculation value of shear strength on a wall

$V_{Ed,max.}$ maximum shear strength at the end of a beam, obtained by capacity dimensioning

$V_{Ed,min.}$ minimum shear strength at the end of a beam, obtained by capacity dimensioning

$V_{fd}$ contribution of friction to wall resistance to shear strength due to displacement

$V_{id}$ contribution of inclined reinforcements to wall resistance to shear strength due to displacement

$V_{Rd,c}$ calculation value of shear strength of elements without shearing reinforcement, in accordance with the Structural Code (Annex 19)

$V_{Rd,S}$ calculation value of shearing stress resistance due to displacement

$b$ width of the lower wing of a beam

$b_c$ dimension of the cross-section of a pillar

$b_{eff}$ effective wing width of a traction beam on the face of a support pillar

$b_i$ distance between two consecutive reinforcements linked to a pillar by cross-arm or transverse tying

$b_o$ thickness of the core confined to a pillar or contour element of a wall (relating to the axis of the fences)

$b_w$ thickness of the parts of the confined area of a section of a wall, or width of the web of a beam

$b_{wo}$ thickness of the web of a wall

$d$ useful edge of a section

$d_{bL}$ diameter of longitudinal reinforcement

$d_{bw}$ diameter of a fence

$f_{cd}$ calculation value of the compressive strength of concrete

$f_{ctm}$ average value of the tensile strength of concrete

$f_{yd}$ calculation value of the elastic limit of steel

$f_{yd,h}$ calculation value of the elastic limit of the horizontal web reinforcement

$f_{yd,v}$ calculation value of the elastic limit of the vertical web reinforcement

$f_{yld}$ calculation value of the elastic limit of longitudinal reinforcements

$f_{ywd}$ calculation value of the elastic limit of transverse reinforcements

$h$ cross-section height

$h_c$ transverse raw edge

$h_f$ wing height

$h_{jc}$ distance between the layers furthest from the reinforcements of a pillar in a junction (knot) between beam and pillar

$h_{jw}$ distance between the upper and lower reinforcements of a beam

$h_o$ edge of the confined core (to the axis of the fences)

$h_s$ floor free height

$h_w$ height of a wall or edge of a cross-section of a beam

$k_D$ coefficient reflecting the ductility class in the calculation of the pillar length required for anchoring the reinforcements of a beam in a junction (knot), equal to 1 for DCH and 2/3 for class DCM[*]

$k_w$ coefficient reflecting the predominant mode of breakage (failure) in structural systems with walls

$l_{cl}$ free length of a beam or pillar

$l_{cr}$ critical area length

$l_i$ distance between the axis of two groups of oblique reinforcements in the section of the base of walls provided with inclined reinforcements to resist sliding due to shear strength

$l_w$ length of the cross-section of a wall

[*] NB: DCH (*Ductility Class – High*) refers to the high ductility class and DCM (*Ductility Class – Medium*) refers to the middle ductility class. Chapters 5, 6 and 7 of this Annex provide more information on ductility classes.

| | |
|---|---|
| $n$ | total number of longitudinal rounds linked laterally by fences or transverse states at the perimeter of the section of the pillar |
| $q_o$ | basic value of the behaviour coefficient |
| $s$ | separation of transverse reinforcements |
| $x_u$ | neutral fibre depth |
| $z.$ | internal mechanical arm |
| $\alpha$ | confinement efficiency coefficient, angle between the diagonal reinforcements and the axis of a coupled beam |
| $\alpha_o$ | predominant aspect ratio of the walls of a structural system |
| $\alpha_l$ | horizontal calculation of seismic action multiplier when the system's first plastic hinge is formed |
| $\alpha_u$ | horizontal calculation of seismic action multiplier when the overall plastic mechanism is formed |
| $\gamma_c$ | partial safety coefficient for concrete |
| $\gamma_{Rd}$ | model's uncertainty coefficient on the calculation value of resistances in estimating the effects of capacity dimensioning action, taking into account various sources of the resistance reserve (over-resistance) |
| $\gamma_s$ | partial coefficient for steel |
| $\varepsilon_{cu2}$ | final unit deformation of non-confined concrete |
| $\varepsilon_{cu2,c}$ | final deformation of confined concrete |
| $\varepsilon_{su,k}$ | characteristic value of the ultimate unit deformation of steel for passive reinforcements |
| $\varepsilon_{sy,d}$ | calculation value of the unit deformation of steel corresponding to the elastic limit |
| $\eta$ | coefficient of reduction of the compressive strength of concrete due to tensile deformations in the transverse direction |
| $\zeta$ | ratio $V_{Ed,min.}/V_{Ed,max.}$ between the minimum and maximum shear strength acting in the extreme section of a beam |
| $\mu_f$ | coefficient of friction of concrete on concrete under cyclic actions |
| $\mu_\phi$ | coefficient of ductility in curvatures |
| $\mu_\delta$ | coefficient of ductility to displacement |
| $\nu$ | axial force due to seismic calculation situation, normalised for $A_c f_{cd}$ |
| $\xi$ | relative depth of neutral axis |
| $\rho$ | amount of traction reinforcements |
| $\rho'$ | amount of compression reinforcements on beams |
| $\sigma_{cm}$ | average value of normal concrete stress |

$\rho_h$ amount of horizontal reinforcements of the web of a wall

$\rho_l$ total amount of longitudinal reinforcements

$\rho_{max.}$ maximum permissible amount of traction reinforcement in critical areas of primary seismic beams

$\rho_v$ amount of the vertical reinforcements of the web of a wall

$\rho_w$ amount of reinforcements at shear strength

$\omega_v$ mechanical amount of vertical web reinforcements

$\omega_{wd}$ mechanical volumetric size of confinement reinforcements

### 1.6.5 Other symbols used in Chapter 6 of this Annex

$L$ beam span

$M_{Ed}$ calculation value of the bending moment obtained from the analysis for the seismic calculation situation

$M_{pl,RdA}$ calculation value of the resistant plastic moment at the A end of an element

$M_{pl,RdB}$ calculation value of the resistant plastic moment at the B end of an element

$N_{Ed}$ calculation value of the axial force obtained from the analysis for the seismic calculation situation

$N_{Ed,E}$ axial force obtained from the seismic calculation action only

$N_{Ed,G}$ axial force due to non-seismic actions included in the combination of actions for the seismic calculation situation

$N_{pl,Rd}$ calculation value of the plastic tensile strength of the raw cross-section of an element, in accordance with the Structural Code (Annex 22)

$N_{Rd}(M_{Ed},V_{Ed})$ calculation value of the axial resistance of a pillar or diagonal element, according to the Structural Code (Annex 22), taking into account the interaction with the bending moment $M_{Ed}$ and with the shearing stress $V_{Ed}$ in the seismic situation

$R_d$ connection resistance in accordance with the Structural Code (Annex 22)

$R_{fy}$ plastic resistance of a bonded dissipative element, depending on the elastic limit of calculation of the material, as defined in the Structural Code (Annex 22)

$V_{Ed}$ calculation value of the shear strength obtained from the analysis for the seismic calculation situation

$V_{Ed,G}$ shear strength due to non-seismic actions included in the combination of actions for the seismic calculation situation

$V_{Ed,M}$ shear strength due to the application of plastic sturdy moments on both ends of a beam

$V_{pl,Rd}$ calculation value of the shear strength of an element in accordance with the Structural Code (Annex 22)

| | |
|---|---|
| $V_{wp,Ed}$ | calculation value of the shear strength on a web panel due to the effects of the seismic calculation action |
| $V_{wp,Rd}$ | calculation value of web panel shear strength in accordance with Structural Code (Annex 22) |
| $e$ | length of a stretch of seismic coupling |
| $f_y$ | nominal elastic limit of steel |
| $f_{y,max.}$ | upper value of the elastic limit of steel |
| $q$ | behaviour coefficient |
| $t_w$ | thickness of the web of a stretch of seismic coupling |
| $t_f$ | thickness of the wing of a stretch of seismic coupling |
| $\Omega$ | multiplier coefficient of axial force $N_{Ed,E}$, obtained from analysis for the seismic calculation situation, for the calculation of non-dissipative elements in porticoes with centred and off-centre triangulations, as per sections **6.7.4** and 6.8.3, respectively |
| $\alpha$ | quotient between the lowest calculation bending moment $M_{Ed,A}$ at one end of a seismic coupling stretch and the greater of the calculation bending moments $M_{Ed,B}$ at the end where the plastic hinge is formed, both moments taken in absolute value |
| $\alpha_l$ | multiplier coefficient of the seismic action of horizontal calculation to the formation of the first plastic hinge of the system |
| $\alpha_u$ | horizontal calculation of seismic action multiplier when the overall plastic mechanism is formed |
| $\gamma_M$ | partial safety coefficient for material properties |
| $\gamma_{ov}$ | material-resistance reserve coefficient |
| $\Delta$ | deflection of the beam in the middle of the vain with respect to the tangent to the axis of the beam at its end (see Figure 6.11) |
| $\gamma_{pb}$ | calculation value multiplier $N_{pl,Rd}$ of the tensile strength corresponding to the elastic limit of the compressed arm of a V triangulation, for estimating the effect of the unbalanced seismic action on the beam to which the joint is connected |
| $\gamma_s$ | partial steel-safety coefficient |
| $\theta_p$ | rotational capacity of the plastic hinge area |
| $\bar{\lambda}$ | dimensionless slenderness of an element as defined in the Structural Code (Annex 22) |

### 1.6.6 Other symbols used in Chapter 7 of this Annex

| | |
|---|---|
| $A_{pl}$ | horizontal plate area |
| $E_a$ | steel elasticity module |
| $E_{cm}$ | average value of the concrete elasticity module in accordance with the Structural Code |

(Annex 19)

$I_a$ inertia moment of the steel part area of a mixed section, with respect to the bending axis of the mixed section

$I_c$ inertia moment of the area of the concrete part of a mixed section, with respect to the bending axis of the mixed section

$I_{eq}$ equivalent inertia moment of the mixed section area

$I_s$ inertia moment of the reinforcements of a mixed section, with respect to the bending axis of the mixed section

$M_{pl,Rd,c}$ calculation value of the sturdy plastic moment of the pillar, calculated for its lower limit value and taking into account the concrete of the section and only the steel parts of the section classified as ductile

$M_{U,Rd,b}$ upper limit of the resistant plastic moment of the beam calculated taking into account the concrete of the section and all steel parts of the section including those not classified as ductile.

$V_{wp,Ed}$ calculation value of the shear strength on the web panel, calculated on the basis of the plastic resistance of adjacent dissipative areas in beams or connections

$V_{wp,Rd}$ calculation value of the shearing strength of mixed steel and concrete web panels, in accordance with the Structural Code (Annex 30).

$b$ wing width

$b_b$ width of the mixed beam (see Figure 7.3a) or supporting width of the concrete slab on the pillar (see Figure 7.7)

$b_e$ effective partial wing width on each side of the steel web

$b_{eff}$ effective overall width of the concrete wing

$b_o$ width (minimum dimension) of the confined concrete core (with reference to the axis of the fences)

$d_{bL}$ diameter of longitudinal reinforcements

$d_{bw}$ diameter of encirclement reinforcements

$f_{yd}$ calculation value of the strength of steel corresponding to the elastic limit

$f_{ydf}$ calculation value of steel strength corresponding to the elastic limit in the wing

$f_{ydw}$ calculation value of web reinforcement resistance

$h_b$ edge of the mixed beam

$h_c$ edge of the mixed pillar section

$k_r$ effectiveness coefficient of the shape of the ribs of a sheet of steel ribs

$k_t$ reduction coefficient of the calculation value of the shear strength of the connectors, in accordance with the Structural Code (Annex 30)

$I_{cl}$ free length of pillar

$l_{cr}$ critical area length

$n$ equivalence quotient between steel and concrete for short-term actions

$q$ behaviour coefficient

$r$ coefficients for reducing the rigidity of concrete for the calculation of the rigidity of the mixed pillars

$t_f$ wing thickness

$\gamma_c$ partial safety coefficient for concrete

$\gamma_M$ partial safety coefficient for material properties

$\gamma_{ov}$ material-resistance reserve coefficient

$\gamma_s$ partial safety-coefficient for steel

$\varepsilon_a$ total unit deformation of steel for ultimate limit state

$\varepsilon_{cu2}$ final unit deformation of non-confined concrete

$\eta$ minimum degree of connection, as defined in section **6.6.1.2** of Annex 30 to the Structural Code

### 1.6.7 Other symbols used in chapter 8 of this Annex

$E_o$ wood elasticity module for instant loads

$b$ width of wood section

$d$ diameter of fastening element

$h$ edge of wooden beams

$k_{mod}$ wood resistance modifier coefficient for instant loads, according to the Technical Building Code (Basic Document DB SE-M 'Structural safety: wood'; see modification factor by load duration and service class.).

$q$ behaviour coefficient

$\gamma_M$ partial safety coefficient for material properties

### 1.6.8 Other symbols used in chapter 9 of this Annex

$a_{g,urm}$ higher value of the on-site ground acceleration calculation value, for use in non-reinforced brickwork structures complying with the provisions of this Earthquake-Resistant Standard

$A_{min.}$ total cross-sectional area of the brickwork walls, required in each horizontal direction to apply the rules for 'single brickwork buildings'

$f_{b,min.}$ standard compressive strength of brickwork units, normal to course face (table)

$f_{bh,min.}$ normalised compressive strength of the brickwork units, parallel to the course face (table), on the wall plane

| | |
|---|---|
| $f_{m,min.}$ | minimum mortar resistance |
| $h$ | maximum free height of openings adjacent to the wall (wall screen) |
| $h_{ef}$ | effective wall height (wall screen) |
| $l$ | wall length (wall screen) |
| $n$ | number of floors above ground level |
| $p_{A,min.}$ | minimum value of the sum of the cross-sectional surfaces of the shearing-resistant walls in each direction, as a percentage of the total floor area |
| $p_{max.}$ | percentage of total area of concrete slag above level |
| $q$ | behaviour coefficient |
| $t_{ef}$ | effective wall thickness (wall screen) |
| $\Delta_{A,max.}$ | maximum difference in the surfaces of the horizontal sections of the shearing-resistant walls between adjacent floors of the 'brickwork simple buildings' |
| $\Delta_{m,max.}$ | maximum difference of masses between adjacent floors of the 'simple brickwork buildings' |
| $\gamma_m$ | partial safety coefficients of brickwork properties |
| $\gamma_s$ | partial safety coefficient of reinforcement steel |
| $\lambda_{min.}$ | quotient between the floor lengths of the short and long sides |

### 1.6.9 Other symbols used in chapter 10 of this Annex

| | |
|---|---|
| $K_{eff}$ | effective rigidity of the isolation system in the considered horizontal main direction, for a displacement equal to the displacement calculation value $d_{dc}$ |
| $K_V$ | total rigidity of the isolation system in the vertical direction |
| $K_{xi}$ | effective rigidity of a given unit $i$ in the direction $x$ |
| $K_{yi}$ | effective rigidity of a given unit $i$ in the direction $y$ |
| $T_{eff}$ | effective fundamental period of superstructure corresponding to horizontal translation, assumed superstructure as a rigid body |
| $T_f$ | fundamental period of the superstructure, supposed set at its base |
| $T_V$ | fundamental period of the superstructure in the vertical direction, the superstructure is assumed to be rigid body |
| $M$ | mass of the superstructure |
| $d_{dc}$ | calculation value of the displacement of the effective rigidity centre in the relevant direction |
| $d_{db}$ | calculation value of the total displacement of an isolation unit |
| $e_{tot,y}$ | total eccentricity in the direction $y$ |
| $f_j$ | horizontal forces at each level $j$ |
| $r_y$ | radius of torsion of isolation system |
| $(x_i,y_j)$ | coordinates of isolation unit i referring to the effective rigidity centre |

$\delta_i$ amplification coefficient

$\xi_{eff}$ effective damping

# 2 Performance requirements and verification criteria

## 2.1 Key requirements

(1) Structures in seismic regions must be designed and constructed in such a way that the following requirements are met, each with an appropriate degree of reliability.

- Non-collapse requirement

  The structure must be designed and constructed to withstand the seismic action calculation defined in chapter **3** without local or overall collapse, i.e. maintaining its structural integrity and residual bearing capacity after the earthquake. The seismic action calculation is expressed in terms of:

  a) the reference seismic action associated with a probability of excess reference, $P_{NCR}$, 10 % in 50 years, or a reference return period $T_{NCR}$ = 475 years; and

  b) the Importance Factor $\gamma_I$ (see Annex 18 to the Structural Code and points **(2)** and **(3)** of this section) to take into account the differentiation of reliability.

NB The probability value, $P_R$, of exceeding in $T_L$ years a specific level of seismic action, is related to the mean return period, $T_R$, of this seismic action level according to the equation $T_R = -T_L/\ln(1 - P_R)$. Thus, for a $T_L$ dated, the seismic action can be equally specified either by its average return period, $T_R$, or by its probability of surplus, $P_R$ in $T_L$ years.

- Damage limitation requirement

  The structure must be designed and constructed to withstand a seismic action that has a higher probability of occurrence than the seismic action calculation, without any associated damage or use limitations, the costs of which are disproportionately high compared to the cost of the structure itself. The seismic action to be considered for the 'damage limitation requirement' has a reference exceedance probability, $P_{DLR}$, of 10 % in 10 years, or a return period $T_{DLR}$ = 95 years. In the absence of more precise information, the coefficient of reduction applied to the seismic calculation action according to point **(2)** of section **4.4.3.2** can be used to obtain seismic action for checking the damage limitation requirement.

(2) The objective reliability for non-collapse and damage limitation requirements are laid down in national regulations for different types of buildings and civil engineering works, depending on the consequences of the failure.

NB: The Structural Code defines the reliability of concrete, steel and mixed structures, and the Technical Building Code that of structures, in the field of building, of wood, brickwork and foundations.

(3) Differentiation of reliability is established by classifying structures into different importance classes. Each importance class is assigned an importance factor $\gamma_I$. Whenever possible, this coefficient should be deducted in such a way that it corresponds to a greater or smaller value of the return period of ground movement (compared to the reference return period), suitable for the project of that specific category of structures (see point **(3)** of section **3.2.1**).

(4) The different levels of reliability are obtained by multiplying the reference seismic action by this importance factor or, when using linear analysis, the corresponding effects of the action. A detailed guide to the relevant importance classes and their corresponding importance factors is provided in the relevant annexes to this Earthquake-Resistant Standard.

NB In the majority of displacements, it can be deemed that the annual exceedance rate $H(a_{gR})$ of the maximum acceleration of the reference ground, $a_{gR}$, varies with $a_{gR}$ as follows: $H(a_{gR}) \sim k_0\, a_{gR}^{-k}$, with exponent value $k$ depending on seismicity, but generally being in the order of 3. Thus, if the seismic action is defined in terms of the maximum acceleration of the reference ground, $a_{gR}$, the value of the importance factor $\gamma_I$which multiplies to the reference seismic action to achieve the same probability of excess in $T_L$ years than in $T_{LR}$ years for which the reference seismic action is defined, can be calculated as: $\gamma_I \sim (T_{LR}/T_L)^{-1/k}$. Alternatively, the value of the importance factor $\gamma_I$ by which the reference seismic action has to be multiplied to obtain a probability value of excess seismic action, $P_L$, in $T^L$ years, (other than the probability of exceedance reference, $P_{LR}$, in the same period of $T_L$ years), can be estimated by: $\gamma_I \sim (P_L/P_{LR})^{-1/k}$.

## 2.2 Verification criteria

### 2.2.1 General considerations

(1) In order to meet the fundamental requirements set out in **2.1**, the following limit states should be checked (see **2.2.2** and **2.2.3**):

- last limit states;
- damage limitation states.

The ultimate limit states are those associated with collapse or with other forms of structural rupture (failure) that could endanger people’s safety.

Damage limitation states are those associated with the occurrence of damage, from which the specified service requirements are no longer met.

(2) In order to limit uncertainties and promote a good performance of structures in the face of seismic actions more severe than the seismic action calculation, a number of appropriate specific measures should also be taken (see **2.2.4**).

(3) In cases of low seismicity (see point **(4)** of section **3.2.1**), the fundamental requirements may be complied with by the application of simpler rules than those indicated in the corresponding annexes to this Earthquake-Resistant Standard.

(4) In cases of very low seismicity, it is not necessary to observe the provisions of this Earthquake-Resistant Standard (see point **(5)** of section **3.2.1** and the notes for the definition of very low seismicity cases).

(5) Chapter 9 gives rules for 'simple brickwork buildings'. By complying with these rules, the indicated 'brickwork simple buildings' are considered to satisfy the fundamental requirements of this Annex without analytical safety checks.

### 2.2.2 Ultimate limit status

(1) It must be verified that the structural system has the strength and energy dissipation capacity specified in the relevant annexes to this Earthquake-Resistant Standard.

(2) The strength and energy dissipation capacity to be assigned to the structure are related to the degree of utilisation of its nonlinear response. For practical purposes, the ratio between resistance and energy dissipation capacity is characterised by the values of the coefficient of behaviour, $q$, and the associated ductility classification, which are given in the corresponding annexes of this Earthquake-Resistant Standard. As a limit case, for the design of structures classified as not dissipative, energy dissipation by hysteresis is not taken into account and the behaviour coefficient cannot be taken, in general, greater than the value 1.5, considered to take into account the reserves of resistance. Similarly, for steel or mixed concrete and steel buildings classified as having a low dissipative structural performance, this limit value of the coefficient $q$, cannot be taken more than 1.5 (see Table 6.1 or Table 7.1, respectively). For dissipative structures, the coefficient of behaviour is taken higher from these limit values, taking into account the dissipation of energy by hysteresis that mainly occurs in specifically projected areas, called dissipative areas or critical areas.

NB The value of the behaviour coefficient $q$ must be limited for the dynamic stability limit state of the structure and for damages due to low cycle fatigue of certain structural details (especially connections). The most unfavourable limit condition must be applied when determining the values of the coefficient $q$. The values of the coefficient $q$ indicated in the annexes to this Standard are considered to comply with this requirement.

(3) The structure as a whole should be checked to ensure that it is stable against the seismic action calculation. Both rollover and displacement stability should be taken into account. The specific rules for checking the roll-over stability of structures are indicated in the appropriate annexes to this Earthquake-Resistant Standard.

(4) It must be verified that both the elements of the foundation and the ground of the foundation itself are capable of resisting, without substantial permanent deformations, the effects of the actions resulting from the response of the superstructure. When determining the reactions, due consideration should be given to the actual resistance that can be developed by the structural element transmitting the actions.

(5) In the analysis, account should be taken of the possible influence of the second-order effects on the values of the effects of the action.

(6) It should be verified that, under the seismic action calculation, the behaviour of non-structural elements does not pose any risk to people, nor does it have any detrimental effect on the response of structural elements. In sections **4.3.5** and **4.3.6** specific rules are given for buildings.

### 2.2.3 Damage limitation status

(1) An adequate degree of reliability against unacceptable damage must be ensured by complying with the limits, as defined in the relevant annexes to this Earthquake-Resistant Standard, for deformation or for other relevant limit values.

(2) In structures important for civil protection, the structural system should be checked to ensure that it has sufficient strength and rigidity in order to maintain the functioning of the vital services of the facilities in the face of an earthquake associated with an appropriate return period.

### 2.2.4 Specific measures

#### 2.2.4.1 Project

(1) As far as possible, the structures should have simple and regular shapes, both in flat and elevated, (see **4.2.3**). If necessary, this can be done by subdividing the structure by joints into dynamically independent units.

(2) In order to ensure global dissipative and ductile behaviour, fragile breakage or premature formation of unstable mechanisms should be avoided. To this end, when indicated in the relevant annexes of this Earthquake-Resistant Standard, the capacity dimensioning method should be used, which is used to establish the strength hierarchy of the various structural components and break modes (failure), necessary to ensure a suitable plastic mechanism and to avoid fragile breakage modes.

(3) Since the seismic behaviour of a structure largely depends on the behaviour of its critical areas or elements, the construction detail of the overall structure and of these particular areas or elements must be able to maintain, under cyclical conditions, the ability to transmit the necessary forces and to dissipate energy. In order to achieve this objective, the details of the joints between structural elements and areas where nonlinear behaviour is foreseeable should be carefully sized.

(4) The calculation should be based on an appropriate structural model which, where necessary, should take into account the influence of ground deformability, non-structural elements, and other aspects, such as the presence of adjacent structures.

#### 2.2.4.2 Cementing

(1) The rigidity of the foundation must be adequate to transmit to the ground, as uniformly as possible, the actions due to the superstructure.

(2) Except for bridges, a single type of foundation must generally be used for the same structure, unless the latter consists of dynamically independent units.

#### 2.2.4.3 Quality-system plan

(1) Project documents should indicate the sizes, construction details and material characteristics of the structural elements. Where appropriate, the project documents should also include the characteristics of the special devices to be used and the distances between structural and non-structural elements. Provisions for quality control should also be included. In the case of buildings, the documentation mentioned in Appendix D- *Specifications relating to project documents in the case of buildings*.

(2) Elements of particular structural importance that require special verification during construction must be identified in the plans of the project. In this case, the verification methods to be used should also be specified.

(3) In regions of high seismicity and in the case of structures of special importance, plans must be used that respond to a formal quality system and reflect the design, construction and use, in addition to the control procedures prescribed in the other applicable regulations.

# 3 Terrain conditions and seismic action

## 3.1 Terrain conditions

### 3.1.1 General considerations

(1) Appropriate studies should be carried out in order to classify the site according to the types listed in section **3.1.2**.

(2) In the section **4.2** of Annex 5, complementary technical criteria are given concerning the study and classification of the terrain.

(3) The construction location and the nature of the land underpinning it must normally be free from risks of land breakage, slope instability and permanent settling caused by liquefied or densification in the event of an earthquake. The possibility of the occurrence of such phenomena should be studied in accordance with the requirements of Chapter **4** of Annex 5.

(4) Depending on the type of importance of the structure and the particular conditions of the project, ground studies or geological studies should be carried out in order to determine the seismic action. Additional investigations to those required for dimensioning against non-seismic loads in the case of major class I buildings may be avoided in accordance with Table 4.3 (see section **4.2.5**). They can also be omitted for buildings of major class II according to Table 4.3, provided that there is a survey of the terrain up to a depth sufficient to allow the interpretation that the characteristics of the terrain do not worsen from that depth.

### 3.1.2 Identification of terrain types

(1) The average terrain types A, B, C and D described by stratigraphic profiles and parameter $v_{s,30}$ indicated in Table 3.1 and detailed below can be used to take into account the influence of local terrain conditions on seismic action. This can also be done taking into account, in addition, the influence of deep geology on seismic action, for which the formation located at a depth from which $v_s \geq 800$ m/s must be considered as rocky substrate.

**Table 3.1 - Terrain types**

| Medium type of terrain | $v_{s,30}$ (m/s) | Description |
|---|---|---|
| A | > 800 | Compact rock or cemented ground emerging or with a surface ground layer of less than 5 m thick. |
| B | 360 – 800 | In the tens of metres closest to the surface, predominance of dense granular soils or hard cohesive soils or the presence of thin layers of loose or cohesive granular soils. |
| C | 180 – 360 | In the tens of metres closest to the surface, predominance of granular soils of medium compactness or cohesive soils of firm or very firm consistency or presence of layers of fairly thick granular soils loose or cohesive soft. |
| D | < 180 | In the tens of metres closest to the surface, the predominance of layers of great thickness of loose or cohesive granular soils. |
| S1 | < 100 | Soils consisting of, or containing, a layer of at least 10 m thick, of clays or soft silts, of high plasticity (IP > 40) and with a high moisture content. |

| | | |
|---|---|---|
| S2 | | Soils consisting of silts, sands and liquefiable gravels or susceptible clays, or other ground profile not contained in types A-D or $S_1$. |

The terrain is classified according to its ability to amplify the seismic movement that occurs in the rock, which depends on the thickness of the surface soils and the average rate of propagation of transverse seismic waves. The terrain may be homogeneous or consist of several layers of the following types (from I to IV):

– Type I terrain layer: Compact rock or cemented ground, with propagation rate of transverse elastic waves $v_s$ > 800 m/s.

– Type II terrain layer: Highly altered or highly fractured rock, dense granular soils or hard cohesive soils, with propagation rate of transverse elastic waves 800 m/s ≥$v_s$ > 360 m/s.

– Type III terrain layer: Medium compact granular ground or cohesive ground of firm to very firm consistency, with propagation rate of transverse elastic waves 360 m/s ≥ $v_s$ > 180 m/s.

– Type IV terrain layer: Loose granular ground or soft cohesive ground, with propagation rate of transverse elastic waves $v_s$ ≤ 180 m/s.

The classification of the ground layer type (I to IV) is made by means of the velocity $v_s$of propagation of the transverse waves corresponding to a tangential deformation of $10^{-5}$ or less. Preferably $v_s$should be determined directly. In addition, static or dynamic penetration tests can be used in granular soils, in cohesive soils for simple compressive strength, and in rocks and soils the rate of propagation of longitudinal seismic waves.

Type I ground layers typically possess longitudinal elastic wave velocity $v_P$ > 2 000 m/s.

Type II terrain layers usually possess longitudinal elastic wave velocity $v_p$ > 1 000 m/s, granulars, hit in SPT $N$1.60 > 40 and resistance per static pentrometer tip $q_p$ > 15 MPa, and cohesive simple compressive strength $q_u$ > 500 kPa.

Type III ground layers usually possess, granulars, hit in SPT 40 ≥ $N_{1.60}$ > 15 and Strength per Static Pentrometer 15 MPa ≥ $q_p$ > 6 MPa, and cohesive simple compression strength 500 kPa ≥ $_{qu}$ > 150 kPa.

Type IV terrain layers usually have parameters N 1.60, $q_p$, $q_u$ lower than those indicated for other types.

In each actual terrain (from A to D), formed by $N$ layers of different types of terrain, the mean velocity of the transverse elastic waves $v_{s,30}$ is determined as set out in section 3.1.2(3).

(2) The location must be classified according to the mean velocity value of the shearing wave, $v_{s,30}$.

(3) The average speed of the shearing wave $v_{s,30}$ must be calculated according to the following equation:

$$v_{s,30} = \frac{30}{\sum_{i=1,N} \frac{h_i}{v_i}} \quad (3.1)$$

where $h_i$ and $v_i$ represent the thickness (in meters) and the speed of the shearing wave (at a deformation level of $10^{-5}$ or lower) of the *i*-th formation or layer, of a total of $N$, existing in the 30 first metres below the natural surface of the terrain.

(4) In locations with terrain conditions that respond to one of the two special types of terrain, $S_1$ or $S_2$, specific studies are required in order to define seismic action. For these types, particularly for $S_2$, the possibility of ground failure under seismic action should be taken into account.

NB Special attention should be paid if the tank is in land of type $S_1$. Such grounds typically have very low values of $v_s$, low internal dampening and an abnormally extensive linear behaviour range; this can lead to anomalous seismic amplification at the site and for ground-structure interaction purposes (see chapter **6** of Annex 5). In this case, a special study should be carried out to define the seismic action, in order to establish the dependence of the response spectrum of the thickness and the value of the $v_s$ of the soft clay/alluvial layer, as well as the contrast of rigidity between this layer and the underlying materials.

## 3.2 Seismic action

### 3.2.1 Seismic regions

(1) For the purposes of this Earthquake-Resistant Standard, a classification of the national territory is established by means of a sufficiently dense grid of points for which the reference values of seismic hazard parameters are given (see Appendix E).

(2) For most applications of this Earthquake-Resistant Standard, seismic hazard is defined by the following parameters:

– The maximum horizontal acceleration (or peak) of ground reference in terrain type A, $a_{gR}$.

– The contribution coefficient $K$, which takes into account the different contribution of the seismicity of the peninsula, the adjacent marine areas and the farthest (that of the Azores-Gibraltar area located in Gorringe-Herradura).

– The magnitude $M_w$, which should be considered in some cases, such as the selection of accelergrams recorded in real earthquakes or the duration assigned to artificial accelerographs or the number of equivalent cycles considered in liquefying calculations of silts, sands and gravels.

The reference peak (horizontal) acceleration in terrain type A, $a_{gR}$, at a point P of the territory is taken equal to:

a) the value indicated in Appendix E, *Peak reference horizontal acceleration values in ground type A and parameter K*, when the geodetic coordinates of point P match those of any of the points in the mesh defined in that section.

b) $a_{gR} = \left( \frac{a_{gR1}}{d_1} + \frac{a_{gR2}}{d_2} \right) / \left( \frac{1}{d_1} + \frac{1}{d_2} \right)$, when the P point is located on the meridians or parallels passing through the points of the mesh indicated in Appendix E, being a $a_{gR1}$ and $a_{gR2}$ the accelerations of the two mesh points closest to P, and $d_1$ and $d_2$, their respective distances to point P.

c) $a_{gR} = \left( \frac{a_{gR1}}{d_1} + \frac{a_{gR2}}{d_2} + \frac{a_{gR3}}{d_2} + \frac{a_{gR4}}{d_2} \right) / \left( \frac{1}{d_1} + \frac{1}{d_2} + \frac{1}{d_3} + \frac{1}{d_4} \right)$ in any other case; with $a_{gR1}$, $a_{gR2}$, $a_{gR3}$, $a_{gR4}$ being the accelerations of the four mesh points closest to P, and $d_1$, $d_2$, $d_3$ and $d_4$ their respective distances to point P.

The contribution coefficient *K* at each point P of the territory must be taken equal to:

a) the value indicated in Appendix E, where the geodetic coordinates of point P match those of any of the points in the mesh defined in that section;

b) $K = \left( \frac{K_1}{d_1} + \frac{K_2}{d_2} \right) / \left( \frac{1}{d_1} + \frac{1}{d_2} \right)$, when the point *P* is located on the meridians or parallels passing through the mesh points indicated in Appendix E; with $K_1$ and $K_2$ being the values of this coefficient at the two mesh points closest to P, and $d_1$ and $d_2$, their respective distances to point P.

c) $K = \left( \frac{K_1}{d_1} + \frac{K_2}{d_2} + \frac{K_3}{d_3} + \frac{K_4}{d_4} \right) / \left( \frac{1}{d_1} + \frac{1}{d_2} + \frac{1}{d_3} + \frac{1}{d_4} \right)$ in any other case; being $K_1$, $K_2$, $K_3$, $K_4$ the values of this coefficient at the mesh points closest to *P*, and $d_1$, $d_2$, $d_3$ and $d_4$ their respective distances to point P.

The value of magnitude $M_w$ of the earthquake to be considered for the definition of artificial accelergrams -point 3.2.3.1.2(2) of this Annex - and for liquefaction calculations - Table B.1 of Appendix B of Annex 5 - is $M_w = 6$ when *K* is less than or equal to 1.1 and $M_w = 8$ when *K* is greater than 1.1.

The corresponding annexes to this Earthquake-Resistant Standard give the complementary parameters, necessary for the specific types of structures.

(3) The maximum ground reference acceleration corresponds to the reference return period, $T_{NCR}$, of seismic action for the requirement of non-collapse or equivalent, to the reference probability, $P_{NCR}$, for that acceleration to be exceeded in 50 years (see point **(1)** of section **2.1**). This return period is assigned a factor of importance $\gamma_I$ equal to 1.0. For return periods other than the reference period (see important classes in points **(3)** and **(4)** of section **2.1**), the calculation value of the acceleration of the ground in a terrain type A, $a_g$, is equal to $\gamma_I$ times the maximum reference horizontal acceleration in terrain type A, ($a_g = \gamma_I \cdot a_{gR}$). (See note to **(4)** of section **2.1**).

(4) In cases of low seismicity, reduced or simplified earthquake-resistant calculation methods may be used for certain types or categories of structures. Cases of low seismicity are considered those in which the product $a_g \cdot S$ is not greater than 0.1 g (0.98 m/s$^2$).

(5) In cases of very low seismicity, it is not necessary to observe the provisions of this Earthquake-Resistant Standard. Cases of very low seismicity are considered cases in which the value of the maximum ground reference acceleration in a terrain type A, $a_{gR}$, is less than 0.04 g (0.39 m/s$^2$).

### 3.2.2 Basic representation of seismic action

#### 3.2.2.1 General considerations

(1) Within the scope of this Resistant Standard, the seismic motion of a given point on the surface is represented by an elastic response spectrum of ground acceleration, hereinafter referred to as the 'elastic response spectrum'.

(2) The form of the elastic response spectrum is taken as the same for the two seismic action levels introduced in point **(1)** of section **2.1** and in point **(1)** of section **2.2.1** for the non-collapse requirement (ultimate limit state – seismic calculation action) and for the damage limitation requirement.

(3) Horizontal seismic action is described by two orthogonal components, considered independent and represented by the same response spectrum.

(4) A unique form of the elastic response spectrum is adopted for the three components of the seismic action (see Figure 3.1 in section 3.2.2.2 (1)). The parameters defining this spectral form are listed in Table 3.2 (in section 3.2.2.2.(2)) for the horizontal components and in Table 3.3 (in section 3.2.2.3 (1)) for the vertical component.

(5) Considering the sources generating earthquakes in the national territory, the possibility of using more than one spectral form for the representation of seismic action is not considered

NB The contribution coefficient K, which seeks to take into account the contribution of different seismic sources at one site and in particular the contribution of the farthest seismicity (see point (2) of section 3.2.1), generates an appropriate variation of the spectral shape in the areas most affected by this seismicity.

(6) For Important Structures ($\gamma_I > 1.0$) topographic amplification effects must be taken into account.

NB Appendix A to Annex 5 provides information on the effects of topographic amplification.

(7) Representations can be used in the time domain of seismic motion (see **3.2.3**).

(8) For specific types of structures, the tolerance of spatial and temporal variation of ground movement may be required (see Annexes 2, 4 and 6).

### 3.2.2.2 Horizontal elastic response spectrum

(1) For the horizontal components of the seismic action, the elastic response spectrum $S_e(T)$ is defined by the following equations (see Figure 3.1):

$$0 \leq T \leq T_B : \; S_e(T) = a_g \cdot S \cdot \left[1 + \frac{T}{T_B} \cdot (\eta \cdot 2{,}5 - 1)\right] \tag{3.2}$$

$$T_B \leq T \leq T_C : \; S_e(T) = a_g \cdot S \cdot \eta \cdot 2{,}5 \tag{3.3}$$

$$T_C \leq T \leq T_D : \; S_e(T) = a_g \cdot S \cdot \eta \cdot 2{,}5 \left[\frac{T_C}{T}\right] \tag{3.4}$$

$$T_D \leq T \leq 4s: \; S_e(T) = a_g \cdot S \cdot \eta \cdot 2,5\left[\frac{T_C T_D}{T^2}\right] \tag{3.5}$$

Where

$S_e(T)$ is the spectrum of elastic response;

$T$ is the vibration period of a linear system with a degree of freedom;

$a_g$ is the calculation value of ground acceleration in a type-A terrain ($a_g = \gamma_I \cdot a_{gR}$);

$T_B$ is the lower limit of the period of the constant spectral acceleration span;

$T_C$ is the upper limit of the period of the constant spectral acceleration span;

$T_D$ is the value that defines the beginning of the constant displacement response span of the spectrum;

$S$ is the ground coefficient;

$\eta$ is the damping correction coefficient with reference value $\eta = 1$, for 5 % viscous damping, see point (**3**) of this section.

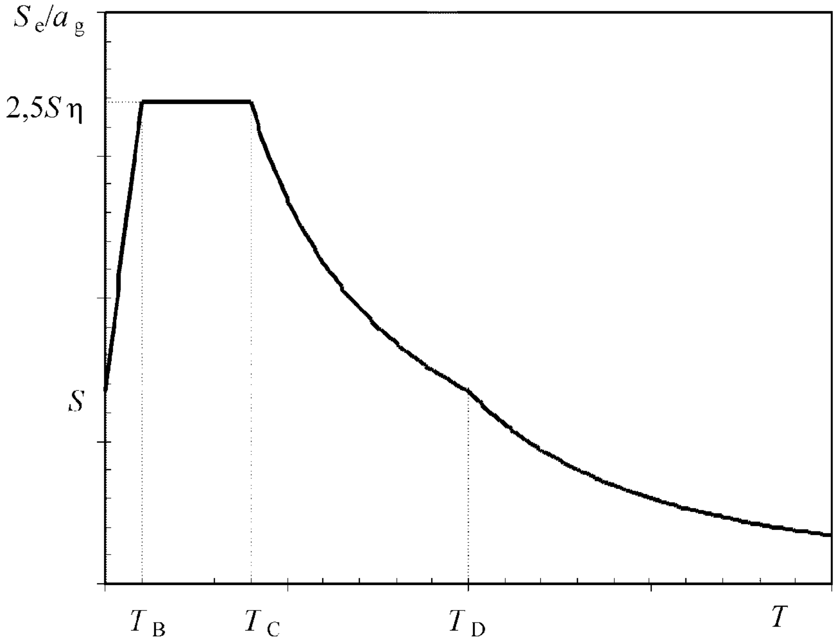



**Figure 3.1 – Form of the elastic response spectrum**

(2) Periods values $T_B$, $T_C$ and $T_D$ and the ground coefficient $S$ describing the shape of the elastic response spectrum depend on the terrain type.

Table 3.2 gives the parameter values $S$, $T_B$, $T_C$ and $T_D$ corresponding to terrain types A, B, C and D.

**Table 3.2 - Parameter values describing horizontal elastic response spectrum**

| Ground type | S | | $T_B$ (s) | $T_C$ (s) | $T_D$ (s) |
|---|---|---|---|---|---|
| A | 1 | | $\frac{T_c}{5}$ | $\frac{K}{4}$ | 2.0 |
| B<br><br>C | $a_g \leq 0.1\,g$:<br><br>$0.1\,g < a_g \leq 0.4\,g$:<br>$a_g > 0.4\,g$: | $S = C$<br><br>$S = C + 3.33\left(\frac{a_g}{g} - 0,1\right)(1.0 - C)$<br>$S = 1$ | $\frac{T_c}{5}$ | $\frac{KC}{4}$ | 2.0 |
| D | $a_g \leq 0.1\,g$:<br><br>$0.1\,g < a_g \leq 0.4\,g$:<br>$a_g > 0.4\,g$: | $S = 2$<br><br>$S = 2.33 - 3.33\,\frac{a_g}{g}$<br>$S = 1$ | $\frac{T_c}{5}$ | $\frac{K}{2}$ | 2.0 |

where $C = (800/v_{s,30})^{0.465}$ (con $v_{s,30}$ in m/s) and $K$ is set in section 3.2.1(2).

NB The values of $S$, $T_B$, $T_C$, and $T_D$ should be obtained by specific studies for terrain types $S_1$ and $S_2$.

(3) The value of the damping correction coefficient, $\eta$, can be determined by the equation

$$\eta = \sqrt{10/(5+\xi)} \geq 0,55 \tag{3.6}$$

where $\xi$ is the value of the viscous damping quotient of the structure, expressed as a percentage.

(4) If for special cases a viscous damping quotient different than 5 % has to be used, this value is indicated in the corresponding annex to this Earthquake-Resistant Standard.

(5) The elastic displacement response spectrum, $S_{De}(T)$, must be obtained by direct transformation of the elastic response spectrum of accelerations, $S_e(T)$, using the following equation:

$$S_{De}(T) = S_e(T)\left[\frac{T}{2\pi}\right]^2 \tag{3.7}$$

(6) Equation (3.7) must normally be applied to periods of vibration not exceeding 4.0 s. For structures with vibration periods greater than 4.0 s, a more complete definition of the elastic response spectrum in terms of displacement must be used.

NB A more complete definition of the elastic response spectrum in terms of displacements is presented in Appendix A, (especially suitable for the case where earthquakes that contribute most to seismic hazard, defined on site by probabilistic assessment, have a magnitude $M_w \geq 5.5$). For periods greater than 4.0 s the elastic response spectrum of accelerations can be deduced from the elastic response spectrum of displacements by reversing the equation (3.7).

### 3.2.2.3 Vertical elastic response spectrum

(1) The vertical component of the seismic action must be represented by an elastic response spectrum, $S_{ve}(T)$, which is deduced using equations (3.8) to (3.11).

$$0 \le T \le T_B : \; S_{ve}(T) = a_{vg} \cdot \left[1 + \frac{T}{T_B} \cdot (\eta \cdot 3{,}0 - 1)\right] \tag{3.8}$$

$$T_B \le T \le T_C : S_{ve}(T) = a_{vg} \cdot \eta \cdot 3{,}0 \tag{3.9}$$

$$T_C \le T \le T_D : \; S_{ve}(T) = a_{vg} \cdot \eta \cdot 3{,}0 \left[\frac{T_C}{T}\right] \tag{3.10}$$

$$T_D \le T \le 4s : S_{ve}(T) = a_{vg} \cdot \eta \cdot 3{,}0 \left[\frac{T_C \cdot T_D}{T^2}\right] \tag{3.11}$$

Table 3.4 gives the values of the parameters describing vertical spectra for terrain types A, B, C, and D. These values do not apply to special terrains $S_1$ and $S_2$.

**Table 3.3 -Parameter values describing vertical elastic response spectrum**

| $a_{vg}$ / $a_g$ | $T_{vB}$ / $T_B$ | $T_{vC}$ / $T_C$ | $T_{vD}$ / $T_D$ |
|---|---|---|---|
| 0.7 | 1.0 | 0.75 | 1.0 |

NB $T_{vB}$, $T_{vC}$ and $T_{vD}$, are the values of $T_B$, $T_C$ and $T_D$, respectively, to be used in equations (3.8) to (3.11) that define the shape of the vertical elastic response spectrum. In Table 3.4 the values are provided in relation to the values of $T_B$, $T_C$ and $T_D$ of the horizontal elastic response spectrum.

### 3.2.2.4 Calculation value of ground displacement

(1) Unless specific studies based on the available information indicate otherwise, the ground displacement calculation value, $d_g$, corresponding to the ground acceleration calculation value, can be estimated by means of the following equation:

$$d_g = 0{,}025 \cdot a_g \cdot S \cdot T_C \cdot T_D \tag{3.12}$$

with $a_g$, $S$, $T_C$ and $T_D$ as defined in section **3.2.2.2**.

### 3.2.2.5 Calculation spectrum for elastic analysis

(1) The ability of structural systems to withstand seismic actions in the nonlinear range generally allows their calculation to withstand seismic forces smaller than those corresponding to a linear elastic response.

(2) In order to avoid an explicitly inelastic structural analysis, the structure's ability to dissipate energy, mainly through the ductile behaviour of its elements or other mechanisms, is taken into account by carrying out an elastic analysis based on a narrow spectrum of response compared to that of elastic response, hereinafter referred to as 'calculation spectrum'. This reduction is achieved by introducing the behaviour coefficient $q$.

(3) The coefficient of behaviour $q$ is an approximation to the quotient between the seismic forces that the structure would experience if its response were completely elastic with a viscous damping of 5 %, and the seismic forces that with a conventional elastic analysis model can be considered in the calculation, still ensuring a satisfactory response of the structure. The various annexes to this Earthquake-Resistant Standard indicate the values of the behaviour coefficient $q$ for the various materials and structural systems according to the corresponding ductility classes, also considering the influence of a viscous damping of 5 %. The value of the behaviour coefficient $q$ may be different in the different horizontal directions of the structure, although the ductility classification must be the same in all directions.

(4) For the horizontal components of the seismic action the calculation spectrum, $S_d(T)$, should be defined by the following equations:

$$0 \leq T \leq T_B: \quad S_d(T) = a_g \cdot S \cdot \left[ \frac{2}{3} + \frac{T}{T_B} \cdot \left( \frac{2,5}{q} - \frac{2}{3} \right) \right] \tag{3.13}$$

$$T_B \leq T \leq T_C: \quad S_d(T) = a_g \cdot S \cdot \frac{2,5}{q} \tag{3.14}$$

$$T_C \leq T \leq T_D: \quad S_d(T) \begin{cases} = a_g \cdot S \cdot \dfrac{2,5}{q} \cdot \left[ \dfrac{T_C}{T} \right] \\ \geq \beta \cdot a_g \end{cases} \tag{3.15}$$

$$T_D \leq T: \quad S_d(T) \begin{cases} = a_g \cdot S \cdot \dfrac{2,5}{q} \cdot \left[ \dfrac{T_C T_D}{T^2} \right] \\ \geq \beta \cdot a_g \end{cases} \tag{3.16}$$

Where

$a_g$, $S$, $T_C$, $T_D$ are defined in **3.2.2.2**;

$S_d(T)$ is the calculation spectrum;

$q$ is the behaviour coefficient;

$\beta$ is the coefficient corresponding to the lower threshold of the horizontal calculation spectrum. $\beta$ is 0.2 for buildings and 0.1 for bridges.

(5) For the vertical component of seismic action, the calculation spectrum is given by equations (3.13) to (3.16), with the ground acceleration calculation value, $a_{vg}$, replacing $a_g$, taking $S$ equal to 1.0 and the other parameters as defined in section **3.2.2.3**.

(6) For the vertical component of the seismic action, generally speaking a behaviour coefficient, q, not greater than 1.5 should be adopted for all materials and structural systems'

(7) The adoption in the vertical direction of values of $q$ greater than 1.5, except in low ductility (DCL) structures, where it is not allowed, must be justified by an appropriate analysis.

(8) The calculation spectrum defined above might not be sufficient for calculating structures with isolation at the base or with energy dissipation systems.

### 3.2.3 Alternative representations of seismic action

#### 3.2.3.1 Representation in time domain

##### 3.2.3.1.1 General considerations

(1) Seismic motion can also be represented as the acceleration of the ground according to time, and by related magnitudes (speed and displacement).

(2) When a spatial model of the structure is required, the seismic motion must consist of three accelergrams acting simultaneously. The same accelergram cannot be used simultaneously in the two horizontal directions. It is possible to simplify in accordance with the provisions of the corresponding annexes to this Syndrome Resistant Standard.

(3) Depending on the nature of the application and the information actually available, the description of the seismic motion can be made by using artificial accelergrams (see **3.2.3.1.2**) and recorded or simulated accelergrams (see **3.2.3.1.3**).

##### 3.2.3.1.2 Artificial accelergrams

(1) Artificial accelergrams should be generated in such a way that their spectrum matches the elastic response spectra indicated in sections **3.2.2.2** and **3.2.2.3** for 5 % viscous damping ($\xi$ = 5%).

(2) The duration of the accelergrams should be consistent with the magnitude and other relevant characteristics of the earthquake that contribute to the determination of $a_g$.

(3) Where no specific data is available, the minimum duration $T_s$ of the stationary part of the accelergrams must be equal to 10 s.

(4) The set of artificial accelergrams must observe the following rules:

a) a minimum of three accelergrams must be used;

b) the mean of the acceleration spectral response values for the zero period (calculated from the individual accelergrams) must not be less than the value of $a_g \cdot S$ for the site in question;

c) in the range of periods between 0.2 $T_1$ and 2 $T_1$, where $T_1$ is the fundamental period of the structure in the direction in which the accelergram must be applied, no mean spectrum value for 5 % damping, calculated from all accelergrams or histories in the time domain, must be less than 90 % of the corresponding value of the elastic spectrum of response for 5 % damping.

##### 3.2.3.1.3 Recorded or simulated accelergrams

(1) Recorded accelergrams or accelerographs generated by a physical simulation of the source and trajectory mechanisms may be used, provided that the samples used are recognised as representative

of the seismogenetic characteristics of the sources and ground conditions of the site, and that their values are scaled to the value $a_g \cdot S$ corresponding to the area under consideration.

(2) For ground movement amplification analysis and for dynamic slope stability checks, see section **2.2** of Annex 5.

(3) The set of accelergrams to be used, recorded or simulated, must satisfy the point **(4)** of **3.2.3.1.2.**

### 3.2.3.2 Spatial model of seismic action

(1) For structures with special characteristics in which the hypothesis that they suffer the same excitation at all their support points cannot reasonably be hypothesised, spatial models of seismic action should be used (see point **(8)** of section **3.2.2.1**).

(2) Such spatial models must be consistent with the elastic response spectra used in the basic definition of seismic action, in accordance with sections **3.2.2.2** and **3.2.2.3**.

### 3.2.4 Combinations of seismic action with other actions

(1) The calculation value $E_d$ of the effects of the actions for the seismic calculation situation must be determined in accordance with section **6.4.3.4** of Annex 18 to the Structural Code.

NB In the case of bridges, the provisions of section 5.5 of Annex 2 must be taken into account.

(2) The inertia effects of the seismic calculation action should be assessed taking into account the existence of masses associated with all gravitational loads appearing in the following combination of actions:

$$\Sigma G_{k,j} \text{ "+" } \Sigma \psi_{E,i} \cdot Q_{k,i} \tag{3.17}$$

Where

$\psi_{E,i}$ is the combination coefficient for the variable action $i$ (for building, see **4.2.4**).

(3) The combination coefficients $\psi_{E,i}$ take into account the probability that the loads $Q_{k,i}$ will not act on the whole structure during the earthquake. These coefficients can also take into account a reduced participation of the masses in the movement of the structure, due to a non-rigid union between them.

(4) The values $\psi_{2,i}$ are indicated in the specific regulations in force and the values $\psi_{E,i}$ for buildings and other types of structures are indicated in the corresponding annexes to this Earthquake-Resistant Standard.

# 4 Buildings project

## 4.1 General considerations

### 4.1.1 Object and field of application

(1) Chapter **4** contains the general rules for the earthquake-resistant project of buildings, and should be applied in conjunction with chapters **2, 3** and **5** to **9**.

(2) Chapters **5** to **9** refer to the specific rules for the various materials and elements used in buildings.

(3) In chapter **10** the basic ideas are given for buildings with isolation at the base.

## 4.2 Characteristics of earthquake-resistant buildings

### 4.2.1 Basic principles of the project design

(1) In seismic regions, the characteristics of seismic hazard should be taken into account in the initial stages of the design of a building, so as to make it possible to achieve a structural system that, within acceptable costs, meets the fundamental requirements set out in section **2.1**.

(2) The principles governing the project design are:

- structural simplicity;
- uniformity, symmetry and redundancy;
- strength and bidirectional rigidity;
- torque strength and rigidity;
- diaphragm action at floor level;
- adequate foundation.

These principles are detailed in the following sections.

#### 4.2.1.1 Structural simplicity

(1) Structural simplicity, characterised by the existence of clear and direct trajectories for the transmission of seismic forces, is an important objective to pursue since the modelling, analysis, dimensioning, construction detail and construction of simple structures are subject to much less uncertainties and, consequently, the prediction of their seismic behaviour is much more reliable.

#### 4.2.1.2 Uniformity, symmetry and redundancy

(1) Floor uniformity is characterised by a regular distribution of structural and non-structural, elements, which allows a short and direct transmission of the inertia forces created in the distributed masses of the building. If necessary, uniformity can be achieved by subdividing the entire building into dynamically independent units by seismic joints, provided that these joints are sized to avoid collision between the individual units, in accordance with section **4.4.2.7**.

(2) The uniformity in the distribution of the structure and masses along the height of the building is also important, since it tends to eliminate the existence of sensitive areas where the concentration of tensions or high demands for ductility can prematurely cause collapse.

(3) A close relationship between mass distribution and the distribution of strength and rigidity eliminates the large eccentricities between mass and rigidity.

(4) If the building configuration is symmetrical or almost symmetrical, a symmetrical arrangement of the structural elements, which must be well distributed on the floor, is appropriate to achieve uniformity.

(5) The use of regularly distributed structural elements increases redundancy and allows for a more favourable redistribution of the effects of the actions and a dissipation of energy spread throughout the structure.

#### 4.2.1.3 Strength and bidirectional rigidity

(1) Horizontal seismic movement is a two-way phenomenon and, consequently, the structure of the building must be able to withstand horizontal actions in any direction.

(2) To satisfy the point **(1)**, the structural elements must be arranged following an orthogonal structural level pattern, ensuring similar strength and rigidity characteristics in the two main directions.

(3) The choice of rigidity characteristics of the structure, while tending to minimise the effects of seismic action (taking into account their specific characteristics for the site), should also limit the development of excessive displacements that could lead either to instabilities due to second order effects, or to major damage.

#### 4.2.1.4 Torque strength and rigidity

(1) In addition to lateral strength and rigidity, building structures must have adequate torque strength and rigidity in order to limit the development of torsion movements that tend to stress the different structural elements in a non-uniform manner. In this respect, the provisions in which the main earthquake-resistant elements are distributed near the periphery of the building have clear advantages.

#### 4.2.1.5 Diaphragm action at floor level

(1) In buildings, concrete slab (including roof) play a very important role in the overall seismic behaviour of the structure. These concrete slabs act as horizontal diaphragms that collect and transmit inertia forces to vertical structural systems, and ensure that these systems act together to withstand horizontal seismic action. The action of concrete slabs as diaphragms is particularly relevant in cases of complex and non-uniform arrangements of vertical structural systems, or where systems with different horizontal deformation characteristics work together (e.g. dual or mixed systems).

(2) The concrete slab systems and the cover must be fitted with rigidity and strength in their plane, as well as an effective connection to the vertical structural systems. Special care should be taken in cases of non-compact or very elongated floor configurations, as well as in cases of concrete slabs with large openings, especially if the latter are located in the vicinity of the main vertical structural elements, thereby frustrating the effective connection between the vertical and the horizontal structure.

(3) Diaphragms must have sufficient rigidity in their plane for the distribution of horizontal inertia forces to vertical structural systems according to the calculation assumptions (e.g. the rigidity of the diaphragm, see point **(4)** of section **4.3.1**), particularly when there are significant changes in rigidity or deviations between the vertical elements above and below the diaphragm.

#### 4.2.1.6 Adequate foundation

(1) In relation to seismic action, the project and construction of the foundations, as well as their connections with the superstructure, must ensure a uniform seismic excitement throughout the

building.
(2) For structures consisting of a discrete number of structural walls (screen walls) with different thicknesses and rigidities, a rigid foundation, drawer or alveolar type, containing a foundation slab and a top slab, must generally be chosen.

(3) For buildings with isolated foundation elements (shoes or piles), it is recommended to use a foundation slab or tie beams between these elements in the two main directions, respecting the criteria and rules of section **5.4.1.2** of Annex 5.

### 4.2.2 Primary and secondary seismic elements

(1) A number of structural elements (e.g. beams or pillars) that are not part of the building's system resistant to seismic action can be designated as 'secondary' seismic elements. The resistance and rigidity of these elements in the face of seismic actions must be disregarded. Such items do not need to meet the requirements of chapters **5** to **9**. However, both these elements and their connections must be designed and detailed constructively to continue supporting gravitational loads when subjected to displacements caused by the most unfavourable seismic calculation conditions. In the draft of these elements, attention should be paid to the second order effects (P-Δ effects).

(2) Chapters **5** to **9** provide complementary rules to those contained in the Structural Code (for concrete, steel and mixed structures) and to those included in the Technical Building Code (for wooden building structures and foundations), for the project and construction detail of secondary seismic elements.

(3) All structural elements not designated as secondary seismic elements are considered as primary seismic elements. They are taken as part of the system resistant to lateral forces and must be modelled in the structural calculation according to the section **4.3.1,** and project and detail constructively with respect to the seismic resistance according to the rules indicated in chapters **5** to **9**.

The enclosures and partitions of buildings must be classified and designated as structural elements, or as non-structural elements, in accordance with the following principles:

- Enclosures and partitions of buildings must be considered as non-structural elements when explicitly separated from the structure, in which case the solutions used to maintain their stability and functionality must be described.

- Regardless of their conditions of connection with the structure, non-structural elements will also be designated as those enclosures and partitions in which, due to their low rigidity or resistance, their participation in the building's system resistant to seismic action may be disregarded.

- Enclosures and partitions of buildings which are not separated from the structure and which, by reason of their rigidity and strength, may form part of the Earthquake-Resistant to seismic action of the building must be considered as structural elements (primary seismic elements). In this case, they should be included in the calculation model by, for example, introducing connecting rods of equivalent rigidity into the model and should be checked against the requests resulting from it. Where the enclosures and partitions of buildings are to be considered structural elements on the basis of the above, a performance coefficient q greater than 2 cannot be adopted.

It is prohibited to modify structural elements throughout the life of the building, including enclosures and partitions if they are classified as such, except as a result of a project supported by a competent technician.

Any change to the original project, including those involving an increase in the strength or rigidity of the modified elements, is prohibited, except as a result of a project justified by a competent technician.

(4) The total contribution to lateral rigidity of all secondary seismic elements must not exceed 15 % of that of all primary seismic elements.

(5) It is not allowed to designate structural elements as secondary seismic elements in order to change the classification of the structure from 'non-regular' to 'regular', as described in section **4.2.3**.

### 4.2.3 Criteria for structural regularity

#### 4.2.3.1 General considerations

(1) For the purposes of the seismic project, building structures are classified as regular or non-regular.

NB In building structures consisting of more than one dynamically independent unit, the appropriate categorisation and criteria in section **4.2.3** refer to each dynamically independent unit. In such structures, each 'dynamically independent singular unit' has the meaning of 'building' for the application of the section **4.2.3**.

(2) This differentiation has implications for the following aspects of seismic calculation:

- the structural model, which may be either a simplified flat model or a spatial model;
- the method of analysis, which may be either a simplified analysis by means of a response spectrum (lateral force method), or a modal analysis;
- the value of the behaviour coefficient, $q$, to be reduced for non-regular buildings in elevation (see **4.2.3.3**).

(3) In relation to the implications of structural regularity in analysis and calculation, the regularity characteristics of the building in floor and elevation are considered separately (Table 4.1).

**Table 4.1 - Consequences of Structural Regularity in Analysis and Earthquake-Resistant Calculation**

| Regularity | | Simplification allowed | | Behaviour coefficient |
|---|---|---|---|---|
| Floor | Elevation view | Model | Linear elastic analysis | (For linear analysis) |
| Yes | Yes | Plane | Lateral force[a] | Reference value |
| Yes | No | Plane | Modal | Reduced value |
| No | Yes | Spatial[b] | Lateral force[a] | Reference value |
| No | No | Spatial | Modal | Reduced value |

a If the condition of point **(2)** (a) of section **4.3.3.2.1** is also met.

b Under the specific conditions indicated in point **(8)** of section **4.3.3.1** a different flat model may be used in each horizontal direction, according to point **(8)** of section of **4.3.3.1**.

(4) In the sections **4.2.3.2** and **4.2.3.3** the criteria describing regularity flat and elevated are given. The rules that refer to modelling and analysis are specified in section **4.3**.

(5) The regularity criteria indicated in sections **4.2.3.2** and **4.2.3.3** must be considered as necessary conditions. It should be verified that the alleged regularity for the structure of the building is not altered by other features not included in these criteria.

(6) In chapters **5** to **9** the reference values of the behaviour coefficients are given.

(7) For non-regular buildings in elevation, the reduced values of the performance coefficient are obtained by multiplying the reference values by 0.8.

#### 4.2.3.2 Criteria of level regularity

(1) In order for a building to be categorised as regular level, it must satisfy all related conditions in the following points.

(2) In relation to lateral rigidity and mass distribution, the structure of the building should be approximately symmetrical level with respect to two orthogonal axes.

(3) The level configuration must be compact, that is, each concrete slab must be delimited by a convex polygonal line. If there are level shifts (corner chamfers or rearward-displaced alignments), level regularity can still be considered complied with the condition that such shifts do not affect the level rigidity in the concrete slab for each shift, the area between the perimeter of the floor and a convex polygonal line level wrapping does not exceed 5 % of the area of said floor.

(4) The rigidity of the level concrete slabs must be large enough compared to the lateral rigidity of the vertical structural elements, so that the deformation of the concrete slab must have a small effect on the distribution of forces between the vertical structural elements. In this respect, floor configurations in L, C, H, I and X must be carefully examined with regard to the rigidity of the lateral branches, which must be comparable to that of the central part in order to satisfy the rigid diaphragm condition. The application of this point should be considered for the assessment of the overall performance of the building.

(5) The slenderness $\lambda = L_{max.}/L_{min.}$ of the building in floor should not be greater than 4, where $L_{max.}$ and $L_{min.}$ are, respectively, the largest and least dimension in building floor, measured in orthogonal directions.

(6) For each level and for each direction, *x* and *y*, of the analysis the structural eccentricity $e_0$ and the radius of torsion must be in accordance with the following two conditions, which for the analysis direction *y* are expressed as:

$$e_{ox} \leq 0,30 \cdot r_x \qquad (4.1.a)$$

$$r_x \geq l_s \qquad (4.2b)$$

Where

$e_{ox}$ is the distance between the centre of rigidity and the centre of gravity, measured along the direction *x*, which is normal to the direction of analysis considered;

$r_x$ is the square root of the quotient between torsional rigidity and lateral rigidity in the direction $y$ ('radius of torsion'); and

$l_s$ is the radius of the mass of the level concrete slab (square root of the quotient between: (a) the polar level inertia moment of the concrete slab's mass with respect to the centre of gravity of the concrete slab, and (b) the mass of the concrete slab.

The definitions of the stiffness centre and radius of torsion, $r$, are given in the points **(7)** to **(9)** of this section.

(7) In one-storey buildings, the stiffness centre is defined as the centre of lateral stiffness of all primary seismic elements. The radius of torsion, $r$, is defined as the square root of the quotient between the overall torsional rigidity to the centre of lateral stiffness, and the overall lateral rigidity in one direction, taking into account all the primary seismic elements in this direction.

(8) In multi-storey buildings, it is only possible to define roughly the centre of rigidity and the radius of torsion. For the classification of structural level regularity and for the approximate analysis of torque effects, a simplified definition is possible if the following two conditions are met:

a) all lateral load resistant systems, such as cores, structural walls or porticoes, run without interruption from the foundation to the top of the building.
b) the deformations under horizontal loads of each of these resistant systems are not very different. This condition can be considered complied with in the case of portico systems and wall systems (screen walls). In general, this condition is not complied with in dual systems.

(9) In porticoes and slender wall systems (screen walls) in which bending deformations prevail, the positions of the rigidity centres and the radii of torsion can be calculated for all floors as those associated with the moments of inertia of the cross-sections of the vertical elements. If, in addition to bending deformations, shearing deformations are also significant, they can be considered by means of an equivalent moment of cross-sectional inertia.

### 4.2.3.3 Criteria for vertical regularity

(1) In order for a building to be classified as regular in elevation, it must satisfy all related conditions in the following points.

(2) All resistance systems of lateral loads, such as cores, structural walls or porticoes, must run without interruption from their foundations to the top of the building or, where there are shifts at different heights, to the top of the corresponding area of the building.

(3) Both the lateral rigidity and the mass of each floor must be kept constant or reduced gradually, without sudden changes, from the base to the top of each building.

(4) In porticoed buildings, the quotient between the actual strength of each floor and the strength required by the analysis must not vary disproportionately between adjacent floors. In this context, the special aspects of the porticoes that frame brickwork fillers are dealt with in section **4.3.6.3.2**.

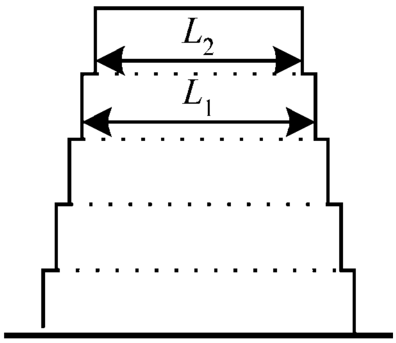
(5) Where there are shifts, the following additional conditions apply:

a) in the case of successive shifts that maintain axial symmetry, the shift of any floor should not be greater than 20 % of the size of the lower floor in the direction of the shift (see Figures 4.1a and 4.1.b).

b) in the case of a single shift within the lower 15 % of the total height of the main structural system, such shift should not be greater than 50 % of the lower floor dimension (see Figure 4.1.c). In this case, the structure of the lower part covering the vertical projection of the perimeter of the upper floors must be sized to withstand at least 75 % of the horizontal shear strength that would be developed in that area in a similar building, but without the elongation of the base.

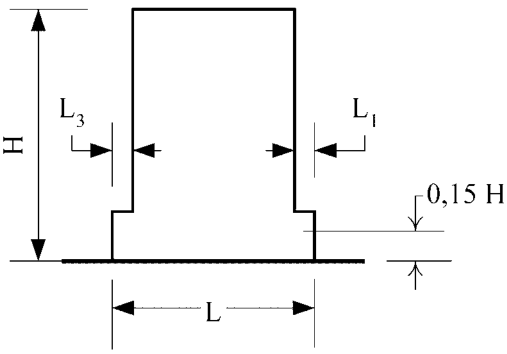
c) if the shifts do not maintain the symmetry, for each face, the sum of the shifts of all floors should not be greater than 30 % of the floor dimension of the first existing floor on the foundation or on the top of a rigid basement, and each of the shifts should not be greater than 10 % of the dimension of the lower floor (see Figure 4.1.d).

(a)



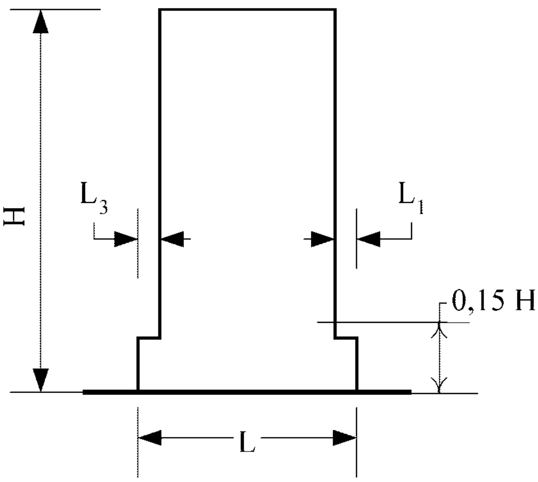

Criterion for (a): $\frac{L_1 - L_2}{L_1} \leq 0{,}20$

(b) (shift occurs above 0.15 H)



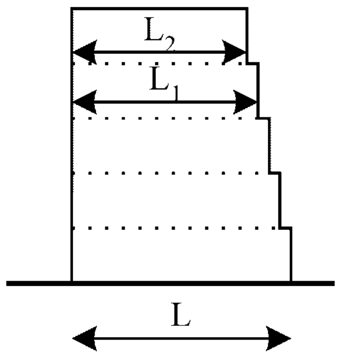

Criterion for (b): $\frac{L_3 + L_1}{L} \leq 0{,}20$

(c) (shift occurs below 0.15 H)




Criterion for (c): $\frac{L_3 + L_1}{L} \leq 0{,}50$

d)




Criterion for (d): $\frac{L - L_2}{L} \leq 0{,}30$

$$\frac{L_1 - L_2}{L_1} \leq 0{,}10$$

**Figure 4.1 - Criteria for regularity of buildings with shifts**

### 4.2.4 Combination coefficients for variable actions

(1) The combination coefficients $\psi_{2,i}$ (for quasi-permanent values of the variable action $Q_i$) for the building project (see **3.2.4**) should be those indicated in the Technical Building Code (section 4. Verifications based on partial coefficients of the Basic Structural Security Document DB-SE).

(2) The combination coefficients $\psi$E,i, indicated in point **(2)** of section **3.2.4** for the calculation of the effects of seismic actions, must be obtained from the following equation:

$$\Psi_{E,i} = \varphi \cdot \Psi_{2,i} \tag{4.2}$$

The values for $\varphi$ are related in Table 4.2.

**Table 4.2 - Values of $\varphi$ for calculating $\psi_{Ei}$**

| Variable type of action | Floor | $\varphi$ |
|---|---|---|
| Categories A-C* | Roof<br>Floors with related occupancies<br>Floors with independent occupancies | 1.0<br>0.8<br>0.5 |
| Categories D-F* and Archives | | 1.0 |
| * Categories defined in the Technical Building Code DB-SE-EA | | |

### 4.2.5 Classes of importance and importance factors

(1) The buildings are classified into 4 importance classes of depending on the consequences of their collapse for human life, on their importance for public safety and civil protection in the immediate aftermath of the earthquake, and on the social and economic consequences of the collapse.

(2) The importance classes are characterised by different importance factors $\gamma_I$, described in **(3)** of section **2.1**.

(3) The importance factor $\gamma_I$ = 1.0 is associated with an earthquake that has the reference return period indicated in point **(3)** of section **3.2.1**.

(4) The definitions of the importance classes are given in Table 4.3.

**Table 4.3 - Importance classes for buildings**

| Importance classes | Buildings |
|---|---|
| I | Buildings of minimum importance for public safety:<br>Buildings in which people stay for long durations, and with a negligible probability that their destruction by the earthquake could cause casualties, disrupt a primary service, or cause significant economic damage. These include:<br><br>• Agricultural or livestock buildings. |
| II | Buildings whose destruction by the earthquake could cause casualties, disrupt a service for the community or produce significant economic losses, without in any case being an essential service or causing catastrophic effects. This includes ordinary buildings, not corresponding to the other categories, among others:<br><br>• Single-family residential buildings<br>• Residential buildings<br>• Buildings for commercial use in which no more than 300 persons are expected to occupy<br>• Buildings intended for public performances which no more than 300 people are expected to occupy<br>• Office buildings where no more than 300 people are expected to occupy<br>• Buildings intended for industrial activities which do not host more than 300 persons, and which do not present a risk of major accidents involving dangerous substances<br>• Buildings intended for the parking vehicles open to the public |
| III | Buildings whose seismic resistance is paramount, considering the consequences associated with their destruction by the earthquake. These include:<br><br>• Buildings intended for public performances which more than 300 people are expected to occupy<br>• Buildings for commercial use in which an occupation of more than 300 persons is anticipated<br>• Buildings intended for industrial activities which host more than 300 persons, and which do not present a risk of major accidents involving dangerous substances<br>• Buildings classified as historical or artistic monuments, or of cultural or similar interest, by the competent bodies of the Public Administrations. |
| IV | Buildings whose integrity in the event of an earthquake is vital for civil protection, national defence needs, or whose destruction by the earthquake can disrupt essential services or lead to catastrophic effects. These include, but are not limited to:<br><br>• Establishments for teaching use at any of its levels (college, universities, etc.)<br>• Hospitals, health centres or facilities<br>• Buildings for disaster organisation and coordination of operations<br>• Buildings dedicated to basic communications facilities, radio, television, telephone and telegraph exchanges.<br>• Buildings for aid personnel and equipment, such as fire barracks, police, armed forces, |

| | |
|---|---|
| | machinery and ambulance fleets<br>• Buildings for essential service facilities for the population (water, electricity, fuels, etc.)<br>• Buildings and facilities of means of transport at railway stations, airports and ports<br>• Buildings and industrial installations at risk of major accidents involving hazardous substances |

NOTE 1 Importance classes I, II and III or IV correspond approximately to consequences classes CC1, CC2 and CC3, respectively, defined in Appendix B of Annex 18 to the Structural Code.

NOTE 2 Where, where applicable, this Earthquake-Resistant Standard applies to structural types not explicitly included in its field of application, a significant factor consistent with the reliability principles and requirements laid down therein must be selected for these structures.

(5) The value $\gamma_I$ for the different importance classes is:

Importance class I (moderate importance): $\gamma_I = 0.8$

Importance class II (normal importance): $\gamma_I = 1$

Importance class III (greater importance): $\gamma_I = 1.3$

Importance class IV (special importance): $\gamma_I = 1.4$

(6) For buildings hosting hazardous facilities or materials, the importance factor must be established in accordance with the criteria set out in Annex 4.

## 4.3 Structural analysis

### 4.3.1 Modelling

(1) The model of the building must adequately represent its distribution of rigidities and masses, so that all significant deformed and inertia forces are adequately taken into account for the seismic action under consideration. In the case of nonlinear analysis, the model must also adequately represent the distribution of resistances.

(2) The model should also take into account the contribution of the joining areas to the deformation of the building, e.g. the ends of beams and pillars in the porticoed structures. Non-structural elements that could influence the response of the primary seismic structure should also be taken into account.

(3) In general, the structure can be considered to consist of a certain number of systems resistant to vertical and lateral loads, connected by horizontal diaphragms.

(4) When the diaphragms made up of the concrete slab of the building can be considered rigid in its planes, the masses and moments of inertia of each floor can be concentrated in the centre of gravity.

NB The diaphragm can be considered rigid if, when modelled with its real flexibility in its plane, its horizontal displacements do not exceed at any point those resulting from the rigid diaphragm hypothesis in more than 10 % of the corresponding absolute horizontal displacements for the seismic calculation situation.

(5) For buildings that meet the criteria for regularity of floor (see **4.2.3.2**) or the conditions set out in point **(8)** of section **4.3.3.1**, the analysis can be carried out using two flat models, one for each main direction.

(6) In concrete buildings, in mixed concrete and steel buildings, and in brickwork buildings, the rigidities of the resistant elements should, in general, be assessed taking the effect of cracking into account. This rigidity must correspond to the beginning of the plasticisation of the reinforcement.

(7) Unless a more precise method of analysis of fissured elements is developed, the properties of bending and shearing elastic rigidity of concrete and brickwork elements can be taken equal to half the rigidity corresponding to the non-fissured elements.

(8) Filler walls that contribute significantly to the rigidity and lateral strength of the building must be taken into account. See section **4.3.6** for brickwork fillers with concrete, steel or mixed porticoed structure.

(9) The deformability of foundation should be taken into account in the model, provided that it can have an adverse global influence on the structural response.

NB The deformability of foundation (including ground-structure interaction) can always be taken into account, including where it has beneficial effects.

(10) The masses must be calculated from the gravitational loads that appear in the combination of actions indicated in section **3.2.4**. The combination coefficients $\psi_{Ei}$ are indicated in **(2)** of section **4.2.4**.

### 4.3.2 Accidental torque effects

(1) In order to take into account uncertainties in the location of masses and spatial variation of seismic motion, the calculated centre of gravity for each floor *i* should be considered as if it were displaced from its nominal position in each direction an accidental eccentricity:

$$e_{ai} = \pm 0,05 \cdot L_i \qquad (4.3)$$

Where

$e_{ai}$ is the accidental eccentricity of floor mass *i* with respect to its nominal position, applied in the same direction in all floors;

$L_i$ is the dimension of the floor, perpendicular to the direction of seismic action.

### 4.3.3 Methods of analysis

#### 4.3.3.1 General considerations

(1) Within the object and field of application of chapter **4**, the seismic effects and effects of the other actions considered in the seismic calculation situation can be determined assuming an elastic-linear behaviour of the structure.

(2) The reference method for determining seismic effects should be modal analysis by means of response spectrum, using an elastic-linear model of the structure and the calculation spectrum indicated in section **3.2.2.5**.

(3) Depending on the structural characteristics of the building, one of the following two types of elastic-linear analysis can be used:

a) the 'lateral force analysis method' for buildings that meet the conditions set out in **4.3.3.2**.

b) the 'modal analysis by means of response spectrum', which is applicable to all types of buildings (see **4.3.3.3**).

(4) As an alternative to a linear method, nonlinear methods such as:

c) nonlinear static analysis (pushover analysis),

d) nonlinear analysis in the time domain using accelerographs (dynamic).

provided that the conditions specified in points **(5)** and **(6)** of this section are met, and section **4.3.3.4**.

NB In chapter **10** are given the conditions under which linear methods (a) and (b), or nonlinear methods (c) and (d), can be used for buildings with isolation at the base. For uninsulated buildings at the base, the linear methods of point **(3)** of section **4.3.3.1**, as specified in section **4.3.3.2.1** can always be used. Nonlinear methods are permitted provided that all parameters used are justified in the calculation memory and the information necessary to allow independent verification is provided.

(5) Nonlinear analyses must be adequately justified in relation to seismic action (seismic input), the constituent model used, the method of interpreting the results of the analysis and the requirements to be met.

(6) Non-isolated base structures calculated by pushover nonlinear analysis, without using the behaviour coefficient $q$ (see **(1)**(d) of section **4.3.3.4.2.1**), must meet the requirements of point **(5)** of section **4.4.2.2**, as well as the rules of chapters **5** to **9** for dissipative structures.

(7) If the criteria for level regularity are met (see **4.2.3.2**), elastic-linear analysis can be developed using two flat models, one for each of the main horizontal directions.

(8) Depending on the importance class of the building, linear elastic analysis can be developed using two flat models, one for each of the main horizontal directions, even if the criteria for regularity on the floor of section **4.2.3.2** are not met, provided that all of the following particular regularity conditions are met:

a) the building must have well-distributed and relatively rigid partitions and enclosures;

b) the height of the building must not exceed 10 m;

c) the rigidity of the concrete slab in its plane must be quite large compared to the lateral rigidity of the vertical structural elements so that a rigid diaphragm behaviour can be assumed;

d) the centres of lateral rigidity and gravity must each be approximately on a vertical line and satisfy, in the two horizontal directions of analysis, the conditions $r_x^2 > l_s^2 + e_{ox}^2$, $r_y^2 > l_s^2 + e_{oy}^2$, where the turning radius $l_s$, radii of torsion $r_y$ and the natural eccentricities $e_{0x}$and $e_{0y}$are defined in point **(6)** of section **4.2.3.2**.

(9) In buildings that satisfy all the conditions of point **(8)** of this section with the exception of condition d), a linear elastic analysis can also be performed using two flat models, one for each of the main horizontal directions, but in these cases all seismic action effects resulting from the analysis must be multiplied by 1.25.

(10) Buildings that do not meet the criteria of points **(7)** to **(9)** of this section should be analysed using a spatial model.

(11) Whenever a spatial model is used, the seismic calculation action should be applied along all relevant horizontal directions (with respect to the structural configuration on the floor of the building) and its orthogonal horizontal directions. For buildings with sturdy elements in two perpendicular directions, these two directions should be deemed relevant directions.

#### 4.3.3.2 Lateral force analysis method

##### 4.3.3.2.1 General considerations

(1) This type of analysis can be applied to buildings whose response is not significantly affected by contributions from vibration modes superior to fundamental mode in each main direction.

(2) The requirement of point **(1)** of this section is considered to be complied with in buildings that meet the following two conditions:

a) in the two main directions they have fundamental vibration periods, $T_1$, less than the following values:

$$T_1 \leq \begin{cases} 4 \cdot T_C \\ 2{,}0\ \text{s} \end{cases} \tag{4.4}$$

where: $T_c$ is defined in section **3.2.2.2**;

b) meet the criteria for regularity in lifting indicated in section **4.2.3.3.**

##### 4.3.3.2.2 Shear strength at the base of the structure

(1) For each horizontal direction in which the building is analysed, the seismic shear strength at the base, $F_b$, should be determined using the following equation:

$$F_b = S_d(T_1) \cdot m \cdot \lambda \tag{4.5}$$

Where

$S_d(T_1)$ is the order of the calculation spectrum (see **3.2.2.5**) for the period $T_1$;

$T_1$ is the fundamental period of vibration of the building for the movement of translation in the direction considered;

$m$ is the total mass of the building on the foundation or on top of a rigid basement, calculated according to point **(2)** of section **3.2.4**;

$\lambda$ is the correction coefficient, the value of which is equal to $\lambda$ = 0.85 if $T_1 \leq 2\ T_C$ and the building has more than two floors or, in another case, $\lambda$ = 1.0.

NB The coefficient $\lambda$ takes into account the fact that in buildings with at least three floors and translational degrees of freedom in each horizontal direction, the effective modal mass of the first (fundamental) mode is less, 15 % on average, than the total mass of the building.

(2) For the determination of the fundamental vibration period, $T_1$, of the building, equations based on structural dynamics methods (e.g. Rayleigh method) can be used.

(3) For buildings up to 40 m high, the value $T_1$ (in s) can be approximated by the following equation:

$$T_1 = C_t \cdot H^{3/4} \tag{4.6}$$

Where

$C_t$ is 0.085 for bending-resistant steel spatial porticoes, 0.075 for flex-resistant concrete spatial porticoes and for steel porticoes with off-centre triangulations and 0.050 for other structures;

$H$ is the height of the building, in m; from the foundation or from the top of a rigid basement.

(4) Alternatively, for structures with shearing-resistant concrete or brickwork walls (walls), the value $C_t$ of equation (4.6) can be taken as

$$C_t = 0{,}075 / \sqrt{A_c} \tag{4.7}$$

Where

$$A_c = \Sigma \left[ A_i \cdot \left( 0{,}2 + (l_{wi} / H)^2 \right) \right] \tag{4.8}$$

And

$A_c$ is the total effective area of shearing resistant walls of the first floor of the building, in $m^2$;

$A_i$ is the effective cross section area of shearing-resistant wall, $i$, of the first floor of the building in the considered direction, in $m^2$;

$H$ has the same meaning as in point **(3)** of this section;

$l_{wi}$ is the length of the shearing resistant wall, $i$, of the first floor of the building in parallel to the applied forces, in m, with the restriction that $l_{wi}/H$ must not exceed 0.9.

(5) Alternatively, $T_1$ (in s) can be estimated using the following equation:

$$T_1 = 2 \cdot \sqrt{d} \tag{4.9}$$

Where

$d$ is the lateral elastic displacement, in m, of the upper part of the building, due to the gravitational loads applied in the horizontal direction.

#### 4.3.3.2.3 Distribution of horizontal seismic forces

(1) The geometries of the deformed ones corresponding to the fundamental mode in the horizontal directions of analysis of the building can be calculated using structural dynamics methods, or they can be approximated by horizontal displacements that increase linearly along the height of the building.

(2) For the two flat models, the effects of seismic action should be determined by applying horizontal forces $F_i$ to all floors of the building.

$$F_i = F_b \cdot \frac{s_i \cdot m_i}{\Sigma s_j \cdot m_j} \tag{4.10}$$

Where

$F_i$ is the horizontal force acting on the floor $i$;

$F_b$ is the seismic shear strength at the base, according to equation (4.5);

$s_i$, $s_j$ are mass displacements $m_i$, $m_j$ for fundamental mode deformed;

$m_i$, $m_j$ are the floor masses calculated according to point **(2)** of section **3.2.4**.

(3) When the fundamental mode deformed geometry is approached by horizontal displacements that are linearly increased with height, the horizontal forces $F_i$ must be obtained from the equation:

$$F_i = F_b \cdot \frac{z_i \cdot m_i}{\Sigma z_j \cdot m_j} \tag{4.11}$$

Where

$z_i$, $z_j$ are the heights of the masses $m_i$, $m_j$ regarding the level of application of seismic action (foundation or upper part of a rigid basement).

(4) The horizontal forces $F_i$, determined in accordance with this section, must be distributed among the resistant side load system assuming the rigid concrete slab in its plane.

#### 4.3.3.2.4 Torque effects

(1) If lateral rigidity and mass have a symmetrical level distribution, and unless the accidental eccentricity set out in point **(1)** of **4.3.2** is not taken into account by a more accurate method (e.g. point **(1)** of section **4.3.3.3.3**), accidental torsion effects can be considered by multiplying the effects of the resulting actions on each resistant element of the application of point **(4)** of section **4.3.3.2.3**, by a coefficient $\delta$ given by:

$$\delta = 1 + 0{,}6 \cdot \frac{x}{L_e} \tag{4.12}$$

Where

$x$ is the distance on the floor of the element considered to be the building's centre of gravity, measured perpendicular to the direction of the seismic action under consideration;

$L_e$ is the distance between the two most external lateral load resistant elements, measured perpendicularly to the direction of the seismic action under consideration.

(2) If the analysis is developed using two flat models, one for each main horizontal direction, the torsion effects can be determined by doubling the accidental eccentricity $e_{ai}$ of the equation (4.3) and applying the point **(1)** of this section, with the coefficient 0.6 of the equation (4.12) increased to 1.2.

#### 4.3.3.3 Modal analysis through response spectra

##### 4.3.3.3.1 General considerations

(1) This type of analysis should be applied to buildings that do not meet the conditions indicated in point **(2)** of section **4.3.3.2.1** to apply the lateral force analysis method.

(2) The response of all modes of vibration that contribute significantly to the overall response should be taken into account.

(3) The requirements specified in point **(2)** can be considered complied with if either of the following two propositions can be demonstrated:

- the sum of the effective modal masses for the modes considered represents at least 90 % of the total mass of the structure;

- are taken into account all modes with effective modal masses greater than 5 % of the total mass.

NB The effective modal mass $m_k$, corresponding to a mode $k$, is determined so that the shear strength in the base $F_{bk}$ associated with this mode, which acts in the direction of application of seismic action, can be expressed as $F_{bk} = S_d(T_k)\ m_k$. It can be shown that the sum of the effective modal masses (for all modes and for a given direction) is equal to the mass of the structure.

(4) When using a spatial model, the above conditions must be checked for each direction considered.

(5) If the requirements specified in point **(3)** cannot be met (e.g. in buildings with a significant contribution of torque modes), the minimum number $k$ modes to be taken into account in a spatial analysis must satisfy the following two conditions:

$$k \geq 3 \cdot \sqrt{n} \tag{4.13}$$

and

$$T_k \leq 0{,}20 \text{ s} \tag{4.14}$$

Where

$k$ is the number of modes considered;

$n$ is the number of floors above the foundation or the top of a rigid basement;

$T_k$ is the vibration period of the mode $k$.

#### 4.3.3.3.2 Combination of modal responses

(1) The response of two vibration modes $i$ y $j$ (including both translation and torque modes) can be considered independent of one another, if your periods $T_i$ and $T_j$ satisfy the following condition (with $T_j \leq T_i$):

$$T_j \leq 0{,}9 \cdot T_i \tag{4.15}$$

(2) Provided that all relevant modal responses (see points **(3)** to **(5)** of section**4.3.3.3.1**) can be considered as independent of each other, the maximum value $E_E$ of a seismic action effect can be taken as:

$$E_E = \sqrt{\Sigma E_{Ei}^{2}} \tag{4.16}$$

Where

$E_E$ is the effect of the seismic action considered (force, displacement, etc.);

$E_{Ei}$ is the value of such effect due to vibration mode $i$.

(3) If point **(1)** is not complied with, more precise procedures should be adopted for the combination of maximum modal responses, such as the 'Full Quadratic Combination').

#### 4.3.3.3.3 Torque effects

(1) Whenever a spatial model is used for analysis, the accidental torque effects indicated in point **(1)** of section **4.3.2** can be determined as the envelope of the effects resulting from the application of static loads, consisting of a series of torsion moments $M_{ai}$ with respect to the vertical axis of each floor $i$:

$$M_{ai} = e_{ai} \cdot F_i \tag{4.17}$$

Where

$M_{ai}$ is the torque moment applied to the floor $i$ with respect to its vertical axis;

$e_{ai}$ is the accidental eccentricity of floor mass $i$, according to equation (4.3), for all considered directions;

$F_i$ is the horizontal force acting on the floor i, obtained from the section **4.3.3.2.3** for all considered directions.

(2) The effects of the loads obtained in point **(1)** must be taken into account with positive and negative signs (the same criterion of signs for all floors).

(3) Provided that two separate flat models are used in the analysis, torque effects can be taken into account by applying the rules of point **(2)** of section **4.3.3.2.4** for the effects of the action calculated in accordance with section **4.3.3.3.2**.

### 4.3.3.4 Nonlinear methods

#### 4.3.3.4.1 General considerations

(1) The mathematical model used in elastic analysis should be extended to include the strength of structural elements and their postelastic behaviour.

(2) At least a two-line tension-deformation ratio must be used for each element. In reinforced concrete and brickwork buildings, the elastic rigidity of a two-line tension-deformation ratio must correspond to that of the cracked sections (see point **(7)** of section **4.3.1**). In ductile elements, for which excursions in the postelastic domain are expected during the response, the elastic rigidity of a bilinear relationship should be the drying rigidity corresponding to the plasticisation point. Trilineal force-deformation relationships are allowed to take into account rigidities before and after cracking.

(3) Zero rigidity may be assumed after plasticisation. If resistance degradation is expected, e.g. in brickwork walls or other fragile elements, such degradation has to be included in the force-deformation ratios of these elements.

(4) Unless otherwise specified, the properties of the elements must be based on the average values of the properties of the materials. For new structures, the average values of the properties of the material can be estimated from the corresponding characteristic values, based on the information provided in the specific regulations in force.

(5) According to section **3.2.4** gravitational loads must be applied to the appropriate elements of the mathematical model.

(6) When determining tension-deformation relationships for structural elements, the axillary forces due to gravitational loads must be taken into account. In vertical structural elements, the bending moments due to gravitational loads may be depreciated, unless they significantly influence the overall behaviour of the structure.

(7) The seismic action should be applied in the positive and negative directions, and the resulting maximum seismic effects should be used.

#### 4.3.3.4.2 Nonlinear static analysis (pushover)

##### 4.3.3.4.2.1 General considerations

(1) Incremental thrust analysis (pushover) is a nonlinear static analysis performed under constant gravitational loads and horizontal loads that increase monotonously. It can be applied to check the structural performance of newly designed or existing buildings, with the following objectives:

a) checking or reviewing the strength reserve quotient (over-resistance) values $\alpha_u/\alpha_1$ (see sections **5.2.2.2**, **6.3.2**, **7.3.2**);

b) estimation of expected plastic mechanisms and distribution of damage;

c) the assessment of the structural performance of existing or reinforced buildings for the objectives of Annex 3.

d) as an alternative to calculation based on an elastic-linear analysis using the behaviour coefficient *q*. In this case, the target displacement indicated in point **(1)** of section **4.3.3.4.2.6** must be used as the basis for the calculation.

(2) Buildings that do not meet the regularity criteria of **4.2.3.2** or criteria (a) to (e) of point **(8)** of section of section **4.3.3.1** should be analysed using a spatial model. Two independent analyses can be carried out, applying a single load direction for each analysis.

(3) For buildings that meet the regularity criteria of **4.2.3.2** or criteria (a) to (e) of point **(8)** of section **4.3.3.1**, the analysis can be carried out using two flat models, one for each main horizontal direction.

(4) For low-rise brickwork buildings, where the behaviour of structural walls is dominated by the shearing, each floor can be analysed independently.

(5) It is estimated that the requirements of point **(4)** are met if the number of floors is equal to or less than 3, and if the average aspect ratio (height to width) of the structural walls is less than 1.0.

#### 4.3.3.4.2.2 Side loads

(1) At least two vertical lateral load distributions must be applied:

- a 'uniform' pattern, based on lateral forces proportional to the masses, regardless of its height (uniform acceleration);

- a 'modal' pattern proportional to the lateral forces, consistent with the distribution of lateral forces in the considered direction, determined in the elastic analysis (according to **4.3.3.2** or **4.3.3.3**).

(2) Lateral loads should be applied in the position of the masses in the model. Accidental eccentricity according to point **(1)** of **4.3.2** must be taken into account.

#### 4.3.3.4.2.3 Capacity curve

(1) The ratio between the shear strength at the base and the control displacement (the 'capacity curve') must be determined by means of an pushover analysis for the control shift values between zero and the value corresponding to 150 % of the target displacement, as defined in section **4.3.3.4.2.6**.

(2) The control displacement can be taken at the centre of gravity of the building roof. The highest part of an installation chamber (penthouse), or similar, should not be considered as the roof.

#### 4.3.3.4.2.4 Resistance reserve coefficient (over-resistance)

(1) When the resistance reserve ratio ($\alpha_u/\alpha_1$) is determined by pushover analysis, the lower value of the resistance reserve quotient obtained for the two lateral load distributions must be used.

#### 4.3.3.4.2.5 Plastic mechanism

(1) The plastic mechanism should be determined for the two applied lateral load distributions. The plastic mechanisms must be in accordance with the mechanisms on which the behaviour coefficient $q$ used in the calculation is based.

#### 4.3.3.4.2.6 Target displacement

(1) Target displacement should be defined as the seismic demand deduced from the elastic response spectrum of the section **3.2.2.2**, in terms of displacement of an equivalent system of one degree of freedom.

NB Appendix B provides a procedure for determining target displacement from the elastic response spectrum.

#### 4.3.3.4.2.7 Procedure for estimating torque effects

(1) Incremental thrust analysis developed with the force patterns specified in section **4.3.3.4.2.2** may significantly underestimate rigid/strength side deformations of a torsion-flexible structure, i.e. of a structure in which the first vibration mode is mainly influenced by torque. The same applies for rigid/strength side deformations in a direction of a structure in which the second vibration mode is mainly influenced by torque. For such structures, the rigid/strength side displacements should be increased, compared to those corresponding to a torsion-balanced structure.

NB The rigid/most resistant side on the floor is the one that under the action of lateral static forces parallel to it develops minor horizontal displacements to those on the opposite side. For torque flexible structures dynamic shifts on the stiffer/harder side can increase considerably due to the influence of a mode where torsion predominates.

(2) The requirement specified in point **(1)** of this section is considered complied with if the amplification coefficient applied to rigid/resistant side displacements is based on the results of a modal elastic analysis of the spatial model.

(3) If, for the analysis of structures that are level regular, two flat models are used, the torsion effects can be estimated according to the sections **4.3.3.2.4** or **4.3.3.3.3**.

#### 4.3.3.4.3 Nonlinear analysis in the time domain

(1) The response of the structure over time can be obtained through the direct numerical integration of the differential equations of motion, using the accelergrams defined in section **3.2.3.1** to represent the motion of the ground.

(2) Structural element models must conform to points **(2)** a **(4)** of section **4.3.3.4.1** and must be supplemented by rules describing the behaviour of the elements under postelastic discharge and recharging cycles. These rules must reflect in a real way the dissipation of energy in the elements, beyond the range of the amplitudes of displacement expected in the seismic calculation situation.

(3) If the response is obtained from at least seven nonlinear analyses in the time domain with ground motions in accordance with the section **3.2.3.1**, the average of the values of the responses obtained for all these analyses must be used as the calculation value of the effect of the action $E_d$ for the corresponding subsection **4.4.2.2**. Otherwise, use as $E_d$ the most unfavourable value obtained for the response among all analyses.

### 4.3.3.5 Combination of the effects of seismic action components

#### 4.3.3.5.1 Horizontal components of seismic action

(1) In general, horizontal components of seismic action (see point **(3)** of **3.2.2.1**) should be considered to act simultaneously.

(2) The combination of the horizontal components of the seismic action can be taken into account as follows:

a) the structural response for each component should be assessed separately, using the combination rules for the modal responses indicated in section **4.3.3.3.2**;

b) the maximum value of each effect of the action on the structure, due to the two horizontal components of the seismic action, can then be estimated by the rule of the square root of the sum of the squares of the responses due to each horizontal component;

c) rule (b) generally provides an estimate of the safety side of the probable values of other effects of an action simultaneously with the maximum value obtained in (b). More accurate models can be used to estimate the probable simultaneous values of more than one effect of the action due to the two horizontal components of the seismic action.

(3) As an alternative to rules (b) and (c) of point **(2)** of this section, the effects of the action due to the combination of the horizontal components of the seismic action can be calculated using the following two combinations:

a) $E_{\mathrm{Edx}}$ "+" 0.30 $E_{\mathrm{Edy}}$ (4.18)

b) 0.30 $E_{\mathrm{Edx}}$ "+" $E_{\mathrm{Edy}}$ (4.19)

Where

"+" means 'combined with';

$E_{\mathrm{Edx}}$ represents the effects of the action due to the application of the seismic action in the direction of the chosen axis as horizontal axis $x$ of the structure;

$E_{\mathrm{Edy}}$ it represents the effects of action due to the application of the same seismic action in the direction of the orthogonal horizontal axis, $y$, of the structure.

(4) 'If the structural system or the classification of regularity in height of the building is different in the different horizontal directions, the value of the coefficient of performance, $q$, may also be different.

(5) In the above combinations, the sign of each component that is most unfavourable to the particular effect to be considered must be taken.

(6) When using a nonlinear static analysis (pushover) and a spatial model is applied, the combination rules of points **(2)** and **(3)** of this section must apply, considering as $E_{\mathrm{dx}}$ the forces and deformations due to the application of the target displacement in the direction $x$, and as $E_{\mathrm{dy}}$ the forces and deformations due to the application of the target displacement in the direction $y$. The internal forces resulting from the combination must not exceed the corresponding capacities.

(7) When performing a nonlinear analysis in the time domain and using a spatial model of the structure, the accelerographs acting simultaneously should be considered acting in both horizontal directions.

(8) For buildings that meet the criteria of regularity on the floor and where the independent triangulated screens or systems in the two main horizontal directions are the only primary seismic elements (see **4.2.2**), it can be assumed that the seismic action acts independently and without having to consider the combinations **(2)** and **(3)** of this section, along the two orthogonal horizontal axes of the structure.

#### 4.3.3.5.2 Vertical component of seismic action

(1) If $a_{vg}$ is greater than 0.25 g (2.5 m/s$^2$), the vertical component of the seismic action, as defined in section **3.2.2.3**, must be taken into account in the following cases:

- for horizontal or almost horizontal structural elements with lamps of 20 m or greater;
- for horizontal or almost horizontal overhangs of a length exceeding 5 m;
- for horizontal or almost horizontal prestressed elements;
- for beams supporting pillars;
- for structures with isolation at the base.

(2) The analysis to determine the effects due to the vertical component of the seismic action can be based on a partial model of the structure, including the elements on which the vertical component is considered to act (e.g. those related in the previous points), and which takes into account the rigidity of the adjacent elements.

(3) It is necessary to take into account the effects due to the vertical component only for the elements considered (e.g. those related in point **(1)** of this section) and for its directly associated bearing elements or infrastructures.

(4) If the horizontal components of the seismic action are also relevant to these elements, the rules set out in **(2)** of section **4.3.3.5.1** may be applied, extending them to the three components of the seismic action. Alternatively, the following three combinations can be used to calculate the effects of the action:

a) $E_{Edx}$ "+" 0.30 $E_{Edy}$ "+" 0.30 $E_{Edz}$ (4.20)

b) 0.30 $E_{Edx}$ "+" $E_{Edy}$ "+" 0.30 $E_{Edz}$ (4.21)

c) 0.30 $E_{Edx}$ "+" 0.30 $E_{Edy}$ "+" $E_{Edz}$ (4.22)

Where

"+" means 'combined with';

$E_{Edx}$ and $E_{Edy}$ have the same meaning as point **(3)** of section **4.3.3.5.1**;

$E_{Edz}$ represents the effects of the action due to the application of the vertical component of the seismic calculation action, as defined in point **(6)** of section **3.2.2.5**.

(5) If a nonlinear static analysis (pushover) is developed, the vertical component of seismic action may be disregarded.

### 4.3.4 Calculation of displacement

(1) If a linear analysis is carried out, the displacements induced by the seismic calculation action should be calculated according to the elastic deformations of the structural system using the following simplified equation:

$$d_s = q_d \, d_e \tag{4.23}$$

Where

$d_s$ is the displacement of a point of the structural system, induced by the seismic action calculation;

$q_d$ is the behaviour coefficient for displacement, which is assumed equal to $q$, unless otherwise specified;

$d_e$ is the displacement of the same point of the structural system, as determined by a linear analysis based on the calculation response spectrum, according to section **3.2.2.5**.

The value of $d_s$ does not need to be greater than that deducted from the elastic spectrum.

NB In general, $q_d$ is greater than $q$ if the fundamental period of the structure is less than $T_C$ (see Figure B.2).

(2) When determining the displacements $d_e$, torque effects obtained from seismic action should be taken into account.

(3) For both static and dynamic nonlinear analyses, the displacements are those obtained directly from the analysis, without any subsequent modification.

### 4.3.5 Non-structural elements

#### 4.3.5.1 General considerations

(1) Non-structural elements (appendices) of buildings (e.g. parapets, hatches, antennas, mechanical equipment and supplementary installations, curtain walls, partitions, enclosures, railings, etc.) which could, in the event of failure, cause damage to people or affect the main structure of the building or the services of critical facilities, must be checked, together with their supports, connections and fastenings or anchorages, to resist the seismic calculation action.

(2) In the case of non-structural elements of high importance or of a particularly dangerous nature, seismic analysis should be based on a realistic model of the relevant structures and on the use of appropriate response spectra deduced from the response of the structural support elements belonging to the main earthquake-resistant system.

(3) In all other cases, simplifications of this procedure, duly justified (e.g. those indicated in **(2)** of section **4.3.5.2**, are allowed).

#### 4.3.5.2 Checks

(1) Non-structural elements, as well as their connections and attachments or anchorages, should be checked for the seismic calculation situation (see **3.2.4**).

NB The transmission of forces to the structure due to the anchoring of non-structural elements and their influence on structural behaviour should be taken into account. The requirements for anchorages in concrete are indicated in section **2.7** of Annex 19 to the Structural Code.

(2) The effects of seismic action can be determined by applying to the non-structural element a horizontal force $F_a$, which is defined as follows:

$$F_a = (S_a \cdot W_a \cdot \gamma_a) / q_a \qquad (4.24)$$

Where

$F_a$ is the horizontal seismic force, acting in the centre of gravity of the non-structural element, in the most unfavourable direction;

$W_a$ is the weight of the element:

$S_a$ is the seismic coefficient applicable to non-structural elements, see point **(3)** of this section;

$\gamma_a$ is the importance factor of the element, see section **4.3.5.3**;

$q_a$ is the coefficient of behaviour of the element, see Table 4.4.

(3) Seismic coefficient $S_a$ can be obtained as follows:

$$S_a = \alpha \cdot S \cdot [3(1 + z/H) / (1 + (1 - T_a/T_1)^2)\text{-}0.5] \qquad (4.25)$$

Where

$\alpha$ is the quotient between the calculation value of the ground acceleration in a terrain type A, $a_g$, and the acceleration of gravity $g$;

$S$ is the ground coefficient;

$T_a$ is the fundamental period of vibration of the non-structural element;

$T_1$ is the fundamental period of vibration of the building in the appropriate direction;

$z$. is the height of the non-structural element above the level of application of the seismic action (foundation or higher part of a rigid basement); and

$H$ is the height of the building measured from the foundation or from the top of a rigid basement.

The seismic coefficient value $S_a$ cannot be taken less than $\alpha \cdot S$.

#### 4.3.5.3 Importance factors

(1) The importance factor $\gamma_a$ should not be less than 1.5 for the following non-structural elements:

- machinery anchorage elements and equipment needed for vital safety systems;

- tanks and containers containing toxic or explosive substances that are considered hazardous to the safety of the general public.

(2) In all other cases, it can be assumed that the importance factor $\gamma_a$ of the non-structural elements $\gamma_a$ = 1.0.

#### 4.3.5.4 Behaviour coefficients

(1) Table 4.4 gives the upper-limit values of the behaviour coefficient $q_a$ for non-structural elements.

**Table 4.4 - Values of $q_a$ for non-structural elements**

| Type of non-structural element | $q_a$ |
|---|---|
| - Parapets in cantilever or ornamentations<br>- Signalling and advertising panels<br>- Chimneys, masts and deposits placed on pillars acting as non-braced brackets at a length greater than half of their total height | 1.0 |
| - Outside and interior walls<br>- Tablets and façades<br>- Chimneys, masts and deposits placed on pillars that act as non-braced brackets at a length less than half their total height, or triangulated or subject to the structure on or above their centre of gravity.<br>- Anchoring elements for floor-supported book cabinets and shelves.<br>- Anchoring elements for false ceilings (suspended) and light fastening devices. | 2.0 |

### 4.3.6 Additional measures for brickwork-filled porticoes

#### 4.3.6.1 General considerations

(1) Paragraphs **4.3.6.1** to **4.3.6.3** apply to dual concrete porticoes or systems equivalent to DCH-class porticoes (see chapter **5**) and to bend-resistant porticoes of steel or mixed concrete and DCH steel (see chapters **6** and **7**), which have non-reinforced brickwork fillers interacting with the structure that meet all of the following conditions:

a) they are built after the concrete slab of the concrete porticoes or the assembly of the steel portico;

b) are in contact with the portico (i.e. without special separation joints), but without structural connection to it (through straps, tapes, poles or shearing connectors);

c) they are considered, in principle, as non-structural elements.

(2) Although the scope of sections **4.3.6.1** to **4.3.6.3** is limited pursuant to point **(1)** of this section, these sections provide good practice criteria, the monitoring of which may be positive for concrete, steel or mixed DCM or DCL class structures with brick fillers. In particular, for panels that could be vulnerable to breakage by exiting your plane, the placement of tethers can reduce the danger caused by falling brick

(3) The provisions of point **(2)** of section **1.3** concerning a possible future modification of the structure should also apply to fillers.

(4) In the case of wall systems, or dual concrete equivalent to walls, as well as for triangulated steel or mixed concrete and steel systems, interaction with brickwork fillers may be neglected.

(5) If the fillers of reinforced factories are part of the structural system, the analysis and calculation must be carried out in accordance with the criteria and rules indicated for factories confined in chapter **9**.

(6) The requirements and criteria indicated in section **4.3.6.2** are considered complied with if the rules indicated in **4.3.6.3** and **4.3.6.4** are followed, as well as the special rules of chapters **5** to **7**.

#### 4.3.6.2 Requirements and criteria

(1) The consequences of the level irregularity caused by the fillings should be taken into account.

(2) Account should be taken of the consequences of the lifting irregularity caused by the fillings.

(3) The large uncertainties related to the behaviour of the fillers (i.e. the variability of their mechanical properties and their anchorage to the surrounding portico, their possible modification during the period of use of the building, as well as the uneven degree of damage suffered during the earthquake itself) should be taken into account.

(4) Possible adverse local effects due to portico-filler interaction should be taken into account; for example, the shear strength breakage of the pillars induced by the action of the diagonal cranks of the fillers (see chapters **5** to **7**).

#### 4.3.6.3 Irregularities due to brickwork fillings

##### 4.3.6.3.1 Level irregularities

(1) Very irregular, asymmetrical or non-uniform level arrangements of fillers (taking into account the size of openings and perforations in the filler panels) should be avoided.

(2) In the case of pronounced level irregularities due to the asymmetrical arrangement of the fillers (e.g. the existence of fills mainly along two consecutive sides of the building), spatial models should be used for the analysis of the structure. Fillers must be included in the model and a sensitivity analysis relating to the position and properties of the fillers must be developed (e.g. ignoring one in three or four panels on a flat portico, especially on the more flexible sides). Special attention should be paid to checking the structural elements of the flexible level sides (i.e. the ones furthest from the one where the fillers are concentrated), in the face of the effects of any torsion response caused by the fillers.

(3) Fillers with more than one significant opening or drilling (e.g. a door and a window, etc.) must be ignored in the models developed to carry out the analysis according to point **(2)** of this section.

(4) When brickwork fillers are not regularly distributed, but also not in such a way as to constitute a pronounced level irregularity, these irregularities can be taken into account by multiplying by a coefficient of 2.0 the effects of the additional eccentricity deducted in accordance with sections **4.3.3.2.4** and **4.3.3.3.3**

##### 4.3.6.3.2 Vertical irregularities

(1) If there are significant vertical irregularities (e.g. drastic reduction of fillers of one or more floors compared to others), the effects of seismic action on the vertical elements of the respective floors should be increased.

(2) If a more precise model is not used, the point **(1)** is considered complied with if the calculated effects of the seismic action are increased by a magnification coefficient, $\eta$, defined as follows:

$$\eta = \left(1 + \Delta V_{\mathrm{Rw}} / \Sigma V_{\mathrm{Ed}}\right) \leq q \tag{4.26}$$

Where

$\Delta V_{\mathrm{Rw}}$ is the total reduction of resistance in the brickwork walls of the floor considered, compared to the upper floors with more fillers.

$\Sigma V_{\mathrm{Ed}}$ is the sum of the shear strength of seismic origin that act on all the primary vertical seismic elements of the floor under consideration.

(3) If equation (4.26) leads to an increase coefficient, $\eta$, less than 1.1, there is no need to modify the effects of the action.

#### 4.3.6.4 Limitation of damage to fillers

(1) Except in low seismicity areas (see **(4)** of section **3.2.1**); for structural systems referred to in point **(1)** of section **4.3.6.1**, belonging to all types of ductility, DCL, DCM or DCH, appropriate measures must be taken to prevent fragile breakage and premature disintegration of filler walls (in particular brickwork panels with openings or made of easily fragmentable materials), as well as partial or total collapse by output of their plan of slender panels of factories. Special attention should be paid to brickwork panels with a ratio of slenderness (a quotient between the lowest value between length or height, and thickness) greater than 15.

(2) Lightweight wire meshes well anchored to one side of the wall, the wall straps attached to the pillars and placed within the horizontal joints (bed joints), and the concrete poles and transverse straps to the panels that embrace the entire thickness of the wall are examples of measurements, in accordance with point (1) of this section, to improve the performance and integrity of brick fillers, both in their plane and outside their plane.

(3) If there are large openings or perforations in any of the filler panels, its edges must be reinforced with sturdy elements.

## 4.4 Safety checks

### 4.4.1 General considerations

(1) For safety checks, the relevant limit states (see **4.4.2** and **4.4.3**) and specific measures (see **2.2.4**) should be considered.

(2) In buildings of important classes other than Class IV (see Table 4.3) the checks prescribed in sections **4.4.2** and **4.4.3** can be considered complied with if the following two conditions are met:

a) the total shear strength at the base due to the seismic calculation situation, determined with a performance coefficient equal to the value applicable to low dissipation structures (see point **(2)** of section **2.2.2**) is lower than that produced by combinations of other relevant actions for which the building is calculated by linear elastic analysis. This requirement concerns the shear strength over the entire structure at the base level of the building (foundation or top of a rigid basement);

b) account is taken of the specific measures described in section **2.2.4** with the exception of the provisions in points **(2)** and **(3)** of section **2.2.4.1**.

(3) In the case of low dissipative structures (see **2.2.2(2)**), it is not necessary to apply the requirements of ductility, capacity dimensioning or resistance reserve (over-resistance) of section 4.4.2.

### 4.4.2 Ultimate limit status

#### 4.4.2.1 General considerations

(1) The requirement of non-collapse (ultimate limit state) in the seismic calculation situation is considered complied with if the following conditions relating to strength, ductility, equilibrium, foundation stability and seismic joints are met.

#### 4.4.2.2 Resistance conditions

(1) For all structural elements, including connections, and for relevant non-structural elements, the following ratio must be complied with:

$$E_d \leq R_d \tag{4.27}$$

Where

$E_d$ is the calculation value of the effect of the action due to the seismic calculation situation (see **6.4.3.4** of Annex 18 to the Structural Code) including, if necessary, the second order effects (see point **(2)** of this section). The redistribution of the bending moments in accordance with the Structural Code (Annexes 19, 22 and 30) is permitted.

$R_d$ is the calculation value of the corresponding strength of the element, calculated according to the specific rules for the material concerned (in terms of the characteristic values of the material properties, $f_k$, and partial safety coefficient $\gamma_M$) and according to the mechanical models associated with the specific type of structural system, as indicated in chapters **5** to **9** of this document, and in the specific regulations in force.

(2) The effects of the second order (P-Δ effects) do not need to be taken into account if the following condition is met on all floors:

$$\theta = \frac{P_{tot} \cdot d_r}{V_{tot} \cdot h} \leq 0,10 \tag{4.28}$$

Where

$\theta$ is the sensitivity coefficient of the collapse between floors;

$P_{tot}$ is the total gravitational charge from the floor considered upwards, for the seismic calculation situation;

$d_r$ is the calculation value of the collapse between floors, evaluated as the difference between the mean lateral displacement, $d_s$, of the top and bottom of the floor under consideration, and calculated in accordance with section **4.3.4**;

$V_{tot}$ is the total seismic shear strength of the floor; and

$h$ is the height between floors.

(3) If 0.1 $\theta \leq$ 0.2, the second order effects can be taken into account by increasing the corresponding effects of the seismic action by a coefficient equal to 1/ (1 – $\theta$).

(4) The value of the coefficient $\theta$ must not exceed 0.3.

(5) If the effects of the calculation action, $E_d$, are obtained by a nonlinear analysis method (see **4.3.3.4**), the point **(1)** of this section must be applied in terms of forces only for fragile elements. In dissipative areas that are designed and detailed constructively to have ductility, the resistance condition, equation (4.27), must be complied with in terms of deformation of the elements (e.g. rotation of plastic hinges or rotation of the wing of the beam) with the appropriate partial coefficients of safety of the materials applied to the deformation capacities of the elements (see also points **(2)** and **(4)** of section **5.7** of Annex 19 to the Structural Code).

(6) Fatigue resistance does not need to be tested in the seismic calculation situation.

#### 4.4.2.3 Global and local ductility conditions

(1) It must be verified that both the structural elements and the structure as a whole have the appropriate ductility, taking into account the exploitation of the expected ductility, which depends on the structural system chosen and the coefficient of performance.

(2) The specific material requirements defined in chapters **5** to **9** including, where indicated, capacity dimensioning arrangements, should be met in order to obtain the strength hierarchy of the different structural components necessary to ensure the intended plastic bearing configuration, and to avoid fragile breakage modes.

(3) In multi-storey buildings, the formation of a soft floor plastic mechanism should be prevented, since such a mechanism may require excessive demands of local ductility on soft floor pillars.

(4) Unless otherwise specified in chapters **5** to **8**, to meet the requirements of point **(3)** in porticoed buildings with two or more floors, including those of equivalent porticoes as defined in point **(1)** of section **5.1.2**, the following condition must be met at all knots of the primary or secondary seismic beams with the primary seismic pillars:

$$\sum M_{Rc} \geq 1,3 \sum M_{Rb} \qquad (4.29)$$

Where

$\Sigma M_{Rc}$ is the sum of the calculation values of the resistant moments of the pillars that converge in the knot. In equation (4.29) the minimum value of the sturdy moments of the pillars must be used within the range of the axillary forces of the pillars for the seismic calculation situation;

$\Sigma M_{Rb}$ is the sum of the calculation values of the resistant moments of the beams that converge in the union. When using partial resistance junctions, the sturdy moments of these junctions are taken into account for the calculation of $\Sigma M_{Rb}$.

NB A rigorous interpretation of the equation (4.29) requires the calculation of the moments in the centre of the knot. These moments correspond to the development of the calculation values of the resistant moments of the pillars or beams on the outer faces of the knot, plus an appropriate increase due to the sharp forces applied on the joining faces. However, the loss of precision is minimal, and the simplification achieved is considerable if the increase due to the shearing is disregarded. This approach is therefore considered acceptable.

(5) Equation (4.29) must be complied with in two orthogonal vertical planes of curvature, which, in buildings arranged in two orthogonal directions, are defined by these two directions. It must be complied with for the two directions (positive and negative) of the moment action of the beams around the knot, with the moments of the pillars always opposing the moments of the beams. If the structural system is a portico or an equivalent portico in only one of the two main horizontal directions of the structural system, then the equation (4.29) must be complied with just in the vertical plane containing that direction.

(6) The rules of points **(4)** and **(5)** of this section do not apply to the upper floor of multi-storey buildings.

(7) In chapters **5** to **7** capacity dimensioning rules are given to avoid fragile break modes.

(8) The requirements of points **(1)** and **(2)** are deemed to be met if all of the following conditions are met:

a) the plastic mechanisms obtained by pushover analysis are satisfactory;

b) the demands for deformation and global ductility, per floor and local, obtained from pushover analyses (with different lateral load distributions) do not exceed the corresponding capacities;

c) the fragile elements remain in the elastic domain.

#### 4.4.2.4 Equilibrium condition

(1) The structure of the building must be stable, including roll-over stability and displacement, in the seismic calculation situation specified in section **6.4.3.4** of Annex 18 to the Structural Code.

(2) In special cases the balance can be checked by energy balance methods or by methods that take into account geometric non-linearities, with the seismic action defined as described in section **3.2.3.1**.

#### 4.4.2.5 Horizontal diaphragm resistance

(1) Diaphragms and diagonal triangulations arranged in horizontal planes must be able to transmit, with a sufficient reserve of strength, the effects of the seismic calculation action to the resistant lateral load systems to which they are attached.

(2) The requirement in section (1) of this section is considered complied with if for the relevant strength checks, the effects on the diaphragm of seismic action obtained in the analysis are multiplied by a resistance reserve coefficient, $\gamma_d$. The value of $\gamma_d$ for fragile break modes, such as for shearing stress on concrete diaphragms, is 1.3, and for ductile break modes is 1.1.

(3) In the section **5.10**, dimensioning arrangements are given for concrete diaphragms.

#### 4.4.2.6 Strength of the foundations

(1) The foundation system must comply with the specific regulations in force and with the requirements of Chapter 5 of Annex 5.

(2) The effects of the action on the elements of the foundation should be obtained on the basis of capacity dimensioning considerations, taking into account the development of possible resistance

reserves, but it is not necessary that these effects exceed those corresponding to the response of the structure in the seismic calculation situation, inherent in the hypothesis of an elastic behaviour ($q$ = 1.0).

(3) If the effects on foundation have been determined using the behaviour coefficient $q$ applied to low dissipative structures (see point **(2)** of section **2.2.2**) no design per capacity according to point **(2)** is required.

(4) The point **(2)** of this section is considered complied with for foundations of isolated vertical elements (walls or pillars) if the calculation values of the effects of the action $E_{Fd}$ on the foundations are deduced as follows:

$$E_{Fd} = E_{F,G} + \gamma_{Rd} \Omega E_{F,E} \quad (4.30)$$

Where

$\gamma_{Rd}$ is the resistance reserve coefficient, taken equal to 1.0 for $q \leq 3$, or equal to 1.2 in other cases;

$E_{F,G}$ is the effect of non-seismic actions, included in the combination of actions for the seismic calculation situation (see **6.4.3.4** of Annex 18 to the Structural Code);

$E_{F,E}$ is the effect of the action obtained from the seismic action calculation; and where

$\Omega$ is the value of $(R_{di}/E_{di}) \leq q$ of the dissipative area or element $i$ structure which has the greatest influence on the effect $E_F$ which is considered;

$R_{di}$ is the calculation value of the area or element strength $i$; and

$E_{di}$ is the calculation value of the effect of the action on the area or element $i$, for the seismic calculation situation.

(5) For foundations of structural walls or bending-resistant portico pillars, $\Omega$ is the minimum quotient value $M_{Rd}/M_{Ed}$ in the two main orthogonal directions, in the lowest cross section where a plastic hinge can be formed in the vertical element, in the seismic calculation situation.

(6) For portico pillar foundations with centred triangulations, $\Omega$ is the minimum ratio value $N_{pl,Rd}/N_{Ed}$ on all voltage diagonals of the triangulated portico (see**(1)** of section **6.7.4**).

(7) For foundations of portico pillars with off-centre triangulations, Ω is the minimum of the following two values: the minimum quotient value $V_{pl,Rd}/V_{Ed}$ of all short stretches of seismic energy dissipation, and minimum quotient value $M_{pl,Rd}/M_{Ed}$ of all intermediate and long couplings of the triangulated portico (see section **(1)** of section **6.8.3**).

(8) For current foundations of more than one vertical element (foundation beams, running shoes, foundation walls etc.), the point **(2)** is considered complied with if the value of $\Omega$ used in equation (4.30) is obtained from the vertical element with the highest horizontal shear strength in the seismic calculation situation or, alternatively, if in equation (4.30) you take the value $\Omega$ = 1 with the value of the resistance reserve coefficient $\gamma_{Rd}$ increased to 1.4.

#### 4.4.2.7 Condition of seismic joint

(1) Buildings must be protected from collisions with adjacent structures, or between structurally independent units of the same building, caused by an earthquake.

(2) The point **(1)** is considered complied with:

a) for buildings, or structurally independent units not belonging to the same property, if the distance of the property boundary to the potential impact points is not less than the maximum horizontal displacement of the building to the corresponding level, calculated according to equation (4.23);

b) for buildings, or structurally independent units belonging to the same building plot, if the distance between them is not less than the square root of the sum of the squares of the maximum horizontal displacements of the two buildings or units at the corresponding level, calculated according to equation (4.23).

(3) If the heights of the floors of the building or of the independent units being calculated are the same as those of the adjacent building or unit, the minimum distance specified above may be reduced by a coefficient of 0.7.

### 4.4.3 Damage limitation

#### 4.4.3.1 General considerations

(1) The 'damage limitation requirement' is considered to have been complied with if, in the event of a seismic action with a probability of occurrence greater than the seismic action calculation corresponding to the 'non-collapse requirement' in accordance with points **(1)** of section **2.1** and **(3)** of section **3.2.1**, the collapse between floors are limited in accordance with section **4.4.3.2**.

(2) In the case of buildings that are important for civil protection or that contain sensitive equipment, additional damage limitation checks may be required.

#### 4.4.3.2 Limitation of collapse between floors

(1) Unless otherwise specified in chapters **5** to **9**, the following limits should be observed:

a) for buildings having non-structural elements of fragile materials attached to the structure:

$$d_r \nu \leq 0,005 h \tag{4.31}$$

b) for buildings having non-structural ductile elements

$$d_r \nu \leq 0,0075 h \tag{4.32}$$

c) for buildings that have non-structural elements linked so that they do not interfere with structural deformations or do not have non-structural elements:

$$d_r \nu \leq 0,010 h \tag{4.33}$$

Where

$d_r$ is the calculation value of the collapse between floors, as defined in point **(2)** of section **4.4.2.2**;

$h$ is the height of the floor;

$\nu$ is the reduction coefficient that considers the shortest return period of seismic action associated with the damage limitation requirement.

(2) The reduction coefficient $\nu$ may also depend on the importance class of the building. For use, the hypothesis is implicit that the elastic response spectrum of seismic action under which the 'damage limitation requirement' is to be met (see point **(1)** of section **3.2.2.1**) has the same shape as the elastic response spectrum of the seismic calculation action corresponding to the 'non-collapse requirement' according to the points **(1)** of section **2.1** and **(3)** of section **3.2.1**.

The values of $\nu$ are 0.4 for classes III and IV and $\nu$ = 0.5 for importance classes I and II.

# 5 Specific rules for concrete buildings

## 5.1 General considerations

### 5.1.1 Object and field of application

(1) Chapter **5** applies to the project of reinforced concrete buildings in seismic regions, henceforth called concrete buildings. Both in-situ and prefabricated concrete buildings are dealt with.

(2) Reticular concrete slabs or flat slabs on isolated pillars, and concrete porticoes with flat beams (understood as those in which the beam width $b_w$, is greater than the width of the pillar $b_c$ measured perpendicularly to the beam axis) can only be used as part of the earthquake-resistant system (primary seismic elements) in classified areas of low seismicity. Flat beams must in any case comply that $b_w \leq \min \{b_c + h_w; 2b_c\}$, where $h_w$ is the edge of the beam.

(3) For the project of concrete buildings, the provisions of Annex 19 of the Structural Code apply. The following rules are complementary to those indicated in these regulations.

### 5.1.2 Terms and definitions

(1) The following terms are used in chapter **5** with the following meanings:

**critical area:**
Region of a primary seismic element where the most unfavourable combination of the effects (M, N, V, T) of the actions occurs and where plastic hinges can be formed.

NB In concrete buildings, critical areas are dissipative areas. The length of the critical area is defined for each structural element in the corresponding section of this chapter.

**beam:**
Structural element subject mainly to transverse loads and reduced axial stress calculation value of $\nu_d = N_{Ed}/A_c f_{cd}$ not greater than 0.1 (positive compression).

NB In general, the beams are horizontal.

**pillar:**
Structural element that withstands axial compression gravitational loads or is subjected to reduced axis calculation force of $\nu_d = N_{Ed}/A_c f_{cd}$ greater than 0.1.

NB In general, the pillars are vertical.

**wall:**
Structural element supporting other elements and with a floor cross-section proportion of length to thickness, $l_w/b_w$, greater than 4.

**ductile wall:**
Fixed wall at its base, in such a way that the relative rotation of the base is avoided with respect to the rest of the structural system, and that it is projected and detailed to dissipate energy by means of an area of plastic hinge to bend free of large openings or perforations, just above its base.

**large wall lightly reinforced:**
Large cross-dimensional wall, i.e. whose length $l_w$ is at least equal to 4.0 m or two-thirds of its height

$h_w$, whichever is smaller, so it is expected to exhibit limited cracking and limited fragile and inelastic behaviour in the seismic calculation situation

NB This type of wall is designed to transform seismic energy into potential energy, by means of transient elevation (scaling) of structural masses, and into energy dissipated on the ground, by rotation as rigid solid, etc. Due to their dimensions, their lack of fixation at the base, or the recessing with transverse walls of large dimensions that prevent the formation of plastic hinges at the base, they cannot be effectively projected for the dissipation of energy by means of plastic signs at the base.

**coupled wall:**
Structural element composed of two or more simple walls regularly connected by beams of adequate ductility ('coupled beams'), capable of reducing by at least 25 % the sum of the bending moments at the base of the walls, if they work separately.

**screen system:**
Structural system in which vertical and lateral loads are resisted by vertical, isolated or coupled structural walls whose shear strength at the base of the building exceeds 65 % of the total shear strength of the entire structural system

NOTE 1 In this definition, and in those that follow, the percentage of shear strength can be replaced by the percentage of shear strength in the seismic calculation situation.

NOTE 2 If most of the shear strength of the system walls is provided by coupled walls, the system can be considered as a system of coupled walls.

**portico system:**
Structural system in which both vertical and lateral loads are resisted by spatial porticoes whose shear strength at the base of the building is greater than 65 % of the total shear strength of the total structural system.

**dual system:**
Structural system in which vertical loads are mainly supported by spatial and lateral porticoes are resisted in part by the portico system and partly by structural walls, isolated or coupled.

**dual system equivalent to portico:**
Dual system in which the shear strength of the portico system at the base of the building is more than 50 % of the total shear strength of the entire structural system.

**dual system equivalent to wall:**
Dual system in which the shear strength of the walls at the base of the building is more than 50 % of the total seismic resistance of the entire structural system.

**flexible torque system (or core system):**
Dual or wall system lacking minimal torque rigidity (see points **(4)** and **(6)** of section **5.2.2.1**).

NOTE 1 An example of this is a structural system composed of flexible porticoes combined with concentrated walls near the centre of the building in plan.

NOTE 2 This definition does not apply to systems consisting of several heavily perforated walls around vertical facilities and services. For these systems, the most appropriate definition of the overall structural configuration should be chosen in each case.

**inverted pendulum systems:**
A system in which 50 % or more of its mass is located in the upper third of the height of the structure, or in which energy dissipation takes place mainly at the base of an isolated element of the building.

NB The porticoes of a floor with the upper ends of the pillars connected along the two main directions of the building and where the value of the reduced axial load of the pillar, $\nu_d$, does not exceed 0.3, do not belong to this category.

## 5.2 Dimensioning principles

### 5.2.1 Energy dissipation capacity and ductility classes

(1) The project of concrete earthquake-resistant buildings should provide adequate energy dissipation capacity without substantially reducing its overall resistance to vertical and horizontal loads. To do so, the criteria and requirements of chapter **2** are applied. In the seismic calculation situation, adequate strength of all structural elements should be provided, and nonlinear deformation demands in critical areas should be proportional to the global ductility assumed in the calculations.

(2) Alternatively, concrete buildings can be dimensioned for low dissipation capacity and low ductility, applying only the rules of Annex 19 of the Structural Code for the seismic calculation situation and without considering the specific provisions indicated in this chapter, provided that the requirements set out in section **5.3** are met. In the case of buildings without base isolation (see chapter **10**), dimensioning by this alternative, called ductility class L or low, is recommended only in cases of low seismicity (see point **(4)** of section **3.2.1**).

(3) Concrete earthquake-resistant buildings where the point **(2)** does not apply must be dimensioned in such a way as to exhibit energy dissipation capacity and overall ductile behaviour. Global ductile behaviour is ensured if the demand for ductility extends over a large number of elements and to locations at all levels of the structure. For this purpose, ductile break modes (such as bending) must precede fragile (such as shearing) modes with sufficient reliability.

(4) Concrete buildings designed according to point **(3)** are classified into two types of ductility: DCM [M or medium ductility class (medium)] and DCH [H or high ductility class (high)], depending on their hysteresis dissipation capacity. Both classes correspond to buildings designed, sized and detailed according to specific earthquake-resistant provisions, allowing the structure to develop stable mechanisms associated with high hysteretic dissipation of energy under repeated alternative loads, without suffering fragile breaks.

(5) In order to provide the appropriate amount of ductility in the ductility classes M and H, the specific provisions for any structural element corresponding to each of the classes must be met (see **5.4** to **5.6**). Depending on the different ductilities available in each class, different values of the behaviour coefficient *q* are used for each class (see **5.2.2.2**).

No geographical limitations are established in the use of the ductility classes M and H.

### 5.2.2 Types of structures and behaviour coefficients

#### 5.2.2.1 Types of structures

(1) Concrete buildings should be classified as belonging to one of the following types of structures (see **5.1.2**), according to their behaviour under horizontal seismic actions:

a) portico system;

b) dual system (equivalent to portico or to wall);

c) ductile wall system (coupled or decoupled);

d) system of lightly reinforced large walls;

e) inverted pendulum system;

f) flexible torque system (or core system).

(2) Except for those classified as flexible torque (or core) systems, concrete buildings can be classified within a given type of structure for one of the horizontal directions and within another type for the other horizontal direction.

(3) A system of walls must be classified as having large walls of lightly reinforced dimensions if, in the relevant horizontal direction, it is composed of at least two walls with a length of not less than 4.0 m or 2 $h_w/3$, taking the lowest value, which collectively support at least 20 % of the gravitational charge due to the upper part in the seismic calculation situation, and if you have a fundamental period $T_1$ (assuming the fixed base with respect to rotation) less than or equal to 0.5 s. It would be sufficient to have only one wall that satisfies the above conditions in one of the two directions if it is ensured that:

(a) the base value of the behaviour coefficient $q_o$ in this direction is divided by 1.5 with respect to the value indicated in Table 5.1; and

(b) there are at least two walls that meet the above conditions in the orthogonal direction.

(4) The first four types of systems (i.e. portico, dual and wall systems of both types) must have minimal torque rigidity that satisfies equation (4.1b) in both horizontal directions.

(5) For portico or wall systems where vertical elements are well distributed on the level, the requirement of point **(4)** can be considered complied with without any analytical check.

(6) Gantry, dual or wall systems without the minimum torque rigidity indicated in point **(4)** must be classified as torque flexible systems (core systems).

(7) If a structural system cannot be classified as large walls lightly reinforced according to the point **(3)**, then all its walls must be projected and detailed as ductile walls.

#### 5.2.2.2 Behaviour coefficients for horizontal seismic actions

(1) The maximum value of the behaviour coefficient $q$, entered in point **(3)** of section**3.2.2.5** to consider the energy dissipation capacity, must be obtained for each calculation direction as follows:

$$q = q_0 k_w \geq 1,5 \tag{5.1}$$

Where

$q_o$ is the base value of the performance coefficient, function of the type of structural system and its regularity in height (see point **(2)** of this section);

$k_W$ is the coefficient that reflects the predominant break mode in structural systems with walls (see point **(11)** of this section).

(2) For regular buildings in height according to section **4.2.3.3**, the base values of $q_o$ for the different

types of structures are indicated in Table 5.1.

**Table 5.1 - Base values of behaviour coefficient $q_o$, for regular systems at height**

| STRUCTURAL TYPE | DCM | DCH |
|---|---|---|
| Gantry system, dual, with coupled walls | 3.0 $\alpha_u/\alpha_l$ | 4.5 $\alpha_u/\alpha_l$ |
| Decoupled-wall system | 3.0 | 4.0 $\alpha_u/\alpha_l$ |
| Core system | 2.0 | 3.0 |
| Inverted pendulum systems | 1.5 | 2.0 |

(3) For non-regular buildings in height, the value $q_o$ must be reduced by 20 % (see point **(7)** of section **4.2.3.1** and Table 4.1).

(4) $\alpha_l$ y $\alpha_u$ are defined as follows:

$\alpha_l$ is the value by which the horizontal calculation seismic action is multiplied, so that the bending resistance is first achieved in any element of the structure, while the rest of the calculation actions remain constant;

$\alpha_u$ is the value by which the seismic action of horizontal calculation is multiplied, so that plastic hinges are formed in a number of sections sufficient for the development of the overall instability of the structure, while the rest of the calculation actions remain constant. The coefficient $\alpha_u$ can be obtained by a nonlinear static global analysis (pushover).

(5) When the multiplier coefficient $\alpha_u/\alpha_l$ has not been evaluated by explicit calculation, the following approximate values can be used for regular buildings on the floor:

a) Portico systems or dual systems equivalent to portico.

- One-storey buildings: $\alpha_u/\alpha_l$ = 1.1;
- Several floors, one-bay porticoes: $\alpha_u/\alpha_l$ = 1.2;
- various floors, porticoes of several bays or dual structures equivalent to portico: $\alpha_u/\alpha_l$ = 1.3.

b) Wall systems or dual systems equivalent to wall.

- wall systems with only two walls decoupled by horizontal direction: $\alpha_u/\alpha_l$ = 1.0;
- other decoupled wall systems: $\alpha_u/\alpha_l$ = 1.1;
- dual wall equivalent systems, or coupled wall systems: $\alpha_u/\alpha_l$ = 1.2.

(6) For buildings that are not regular in plan (see **4.2.3.2**), the approximate value of $\alpha_u/\alpha_l$ that can be used -when the relevant calculations have not been carried out for evaluation- is the average of (a) 1.0 and (b) the value indicated in point (5) of this section.

(7) Values of $\alpha_u/\alpha_l$ higher than those indicated in points **(5)** and **(6)** of this section can be used, provided that they are confirmed by a nonlinear static global analysis (pushover).

(8) The maximum value of $\alpha_u/\alpha_l$ that can be used in the calculation is 1.5, even if the analysis

mentioned in point **(7)** of this section results in higher values.

(9) The value of $q_o$ indicated for inverted pendulum systems can be increased, provided that greater energy dissipation capacity is ensured in the critical area of the structure.

(10) For other cases, it is not allowed to increase the values of $q_o$.

(11) The coefficient $k_w$, which reflects the predominant fracture mode in structural systems with walls, should be taken as follows:

$$k_w = \begin{cases} 1.00\text{, for portico-equivalent portico systems and dual systems } (1+ \alpha_o)/3 < \\ 1\text{, but not less than 0.5, for wall systems, wall equivalent systems and} \\ \text{torque flexible systems (core systems)} \end{cases} \tag{5.2}$$

where $\alpha_o$ is the predominant aspect ratio of the walls of the structural system.

(12) If aspect ratios $h_{wi}/l_{wi}$ of all the walls $i$ of a structural system do not differ significantly, the predominant aspect ratio $\alpha_o$ can be calculated as follows:

$$\alpha_o = \sum h_{wi} / \sum l_{wi} \tag{5.3}$$

Where

$h_{wi}$ is the height of wall $i$; and

$l_{wi}$ is the length of the wall section $i$.

(13) In lightly reinforced large wall systems, energy dissipation cannot be relied on by plastic hinges, so they must be sized as structures belonging to the DCM ductility class.

### 5.2.3 Dimensioning criteria

#### 5.2.3.1 General considerations

(1) The dimensioning principles described in section **5.2.1** and chapter **2** must be applied to earthquake-resistant structural elements of concrete buildings as specified in sections **5.2.3.2** to **5.2.3.7**.

(2) The dimensioning criteria indicated in sections **5.2.3.2** to **5.2.3.7** are considered complied with if the conditions set out in sections **5.4** to **5.7** are met.

#### 5.2.3.2 Local resistance condition

(1) All critical areas of the structure must meet the requirements set out in point **(1)** of section **4.4.2.2.**

#### 5.2.3.3 Capacity-dimensioning criterion

(1) Fragile break mechanisms or any other unwanted type (e.g. the concentration of plastic hinges in buildings of one or more floors, the shearing of structural elements, breaking the junctions between beam and pillar, plasticisation in foundations or any other element that needs to remain elastic) should be avoided by calculating the effects of the calculation actions on the selected areas from the equilibrium conditions, assuming that plastic hinges and over strengths that may appear due to them have formed in the adjacent areas.

(2) Earthquake-resistant pillars of porticoed concrete structures or portico equivalents must satisfy the capacity dimensioning requirements set out in point **(4)** of section **4.4.2.3**, with the following exceptions:

a) in flat porticoes with at least four pillars of approximately equal cross-section, it is not necessary to satisfy the condition (4.29) on all pillars, but simply in three out of four.

b) on the lower floor of two-storey buildings, if the value of reduced axial load $\nu_d$ does not exceed 0.3 in any pillar.

(3) Forging reinforcement parallel to the beam within the effective wing width specified in point **(3)** of **5.4.3.1.1**, must be assumed to contribute to the bending capabilities of the beam involved in the calculation of $\Sigma M_{Rb}$ in equation (4.29), if this reinforcement is anchored beyond the beam section on the nude face.

#### 5.2.3.4 Local ductility condition

(1) In order to achieve the necessary global ductility of the structure, the areas where plastic hinges can be formed, areas defined below for each type of element of a building, must have a great plastic rotation capacity.

(2) Point **(1)** is considered complied with under the following conditions:

a) sufficient ductility vis-à-vis curvature is available in all critical areas of the earthquake-resistant elements, including the ends of the pillars (depending on the potential for formation of plastic hinges in the pillars) (see point **(3)** of this section);

b) the local buckling of compressed steel in areas of potential formation of plastic hinges of earthquake-resistant elements has been adequately prevented. The application rules to be considered are indicated in **5.4.3** and **5.5.3**;

c) appropriate grades of steel and concrete have been adopted to ensure local ductility, namely:

- steel used in critical areas of earthquake-resistant elements must have a high and uniform plastic deformation (see points **(1)** of section **5.3.2**, **(3)** of section **5.4.1.1**, and **(3)** of section **5.5.1.1**);

- the ratio between breakage tension and plastic limit of steel used in critical areas of seismic elements must be significantly greater than the unit. The steel frame conforming to the requirements of points **(1)** of section **5.3.2**, **(3)** of section **5.4.1.1** o **(3)** of section **5.5.1.1**, as appropriate;

- concrete used in primary seismic elements must have an adequate compressive strength and a unit breakage deformation that exceeds, with an appropriate margin, the deformation corresponding to the maximum compression. It can be assumed that a concrete meeting the requirements of points **(1)** of section **5.4.1.1** or (**1**) of section **5.5.1.1**, as appropriate, satisfies these requirements.

(3) Unless more precise data are available, and unless section **(4)** of this section applies, the point **(2)** (a) is considered to be complied with if the coefficient of ductility in curvatures, $\mu_\phi$, of those areas (defined as the ratio between curvature at 85 % of the resistant moment after breakage and curvature at the elastic boundary, provided that the boundary unit deformations of concrete and steel are not exceeded, $\varepsilon_{cu}$ and $\varepsilon_{su,k}$) is at least the same as the following coefficients:

$$\mu_\phi = 2q_o - 1 \quad \text{if } T_1 \geq T_C \tag{5.4}$$

$$\mu_\phi = 1 + 2(q_o - 1)T_C / T_1 \quad \text{if } T_1 < T_C \tag{5.5}$$

where $q_o$ is the corresponding base value of the behaviour coefficient of Table 5.1 and $T_1$ is the fundamental period of the building, both taken in the vertical plane in which bending occurs, and $T_C$ is the upper limit of the period in the constant acceleration stretch of the spectrum, according to point**(2)** of section **3.2.2.2**.

NB The equations (5.4) and (5.5) are based on the ratio between $\mu_\phi$ and the displacement ductility coefficient, $\mu_\delta$: $\mu_\phi = 2\mu_\delta - 1$, which is normally a conservative approach for concrete elements, and in the following ratio between $\mu_\delta$ and $q$: $\mu_\delta = q$ if $T_1 \geq T_C$, $\mu_\delta = 1 + (q - 1)\ T_C/T_1$ si $T_1 < T_C$ (see also chapter B5 in Appendix B). The value of $q_o$ is used instead of $q$, because $q$ must be less than $q_o$ in irregular buildings, recognising that greater lateral resistance is needed to protect them. However, local ductility demands may be higher than those corresponding to $q_o$ values, so a reduction of ductility capacity in curvatures cannot be guaranteed.

(4) In areas of earthquake-resistant areas with type-S longitudinal steel framing as defined in Articles 34 and 35 of the Structural Code, the ductility coefficient in curvatures, $\mu_\phi$, must be equal to at least 1.5 times the value obtained from equations (5.4) or (5.5), whichever applies.

#### 5.2.3.5 Hyperestaticity of the structure

(1) A high degree of hyperstatism, accompanied by a redistribution capacity, should be sought to achieve a more widespread energy dissipation and increase the total dissipated energy. Consequently, lower performance coefficients should be assigned to structural systems with a lower degree of hyperestaticity (see Table 5.1). The necessary redistribution capacity must be achieved through the local ductility rules indicated in **5.4** to **5.6**.

#### 5.2.3.6 Secondary seismic elements and resistances

(1) A limited number of structural elements can be designated as secondary seismic elements, according to section **4.2.2**.

(2) The rules for the project and the construction details of the secondary seismic elements are indicated in the section **5.7**.

(3) Resistances or stabilising effects that have not been explicitly taken into account in calculations can improve both strength and energy dissipation (e.g. membrane reactions of slabs caused by the

mismatch of structural walls).

(4) Non-structural elements can also contribute to energy dissipation, provided they are evenly distributed throughout the structure. However, appropriate measures should be taken to avoid possible local adverse effects due to the interaction between structural and non-structural systems (see **5.9**).

(5) Special rules are given in sections **4.3.6** and **5.9** for porticoed bays filled with brickwork.

#### 5.2.3.7 Specific additional measures

(1) Due to the random nature of seismic action and uncertainties about the cyclic postelastic behaviour of concrete structures, the overall uncertainty is substantially greater than in the case of non-seismic actions. Therefore, measures should be taken to reduce uncertainties related to structural configuration, analysis, resistance and ductility.

(2) There can be significant uncertainties about resistance due to geometric errors. To minimise them, the following rules must apply:

a) minimum dimensions for structural elements (see **5.4.1.2** and **5.5.1.2**) should be respected to reduce sensitivity to geometric errors;

b) the ratio between the minimum and maximum dimensions of linear elements should be limited in order to minimise the risk of lateral instability of these elements (see **5. 4.1.2** and point**(2)** of section **5.5.1.2.1**);

c) movement between floors should be limited to reduce P-Δ effects on the pillars (see points **(2)** a **(4)** of section **4.4.2.2**).

d) the upper reinforcement of the beams, in their extreme cross-sections, should be extended by a significant percentage along the entire length of the beams (see point **(5)** of section **5.4.3.1.2**, and point **(5)** of section **5.5.3.1.3**) in order to consider the uncertainty in determining the location of the beams inflection points;

e) changes in the sign of the moments, not foreseen by the calculations, must be taken into account by having a minimum reinforcement on the appropriate faces of the beams (see **5.5.3.1.3**).

(3) To minimise uncertainties related to ductility, the following rules should be observed:

a) each primary seismic element must be given an appropriate minimum local ductility, regardless of the ductility class adopted in the calculation (see **5.4** and **5.5**).

b) a minimum amount of tensile reinforcement must be provided to avoid brittle cracking (see **5.4.3** and **5.5.5**).

c) an appropriate limitation of the reduced axial force calculation value (see point **(3)** of section **5.4.3.2.1**, point **(2)** of section **5.4.3.4.1**, point **(3)** of section **5.5.3.2.1** and point **(2)** of section **5.5.3.4.1**) to reduce the consequences of coating loss and to avoid major uncertainties about available ductilities that occur for high axial load levels.

### 5.2.4 Safety checks

(1) For ultimate limit state checks in structural concrete, the partial safety coefficients for material

properties, $\gamma_c$ and $\gamma_s$, should take into account the possible degradation of material strength due to cyclic deformations.

(2) When due to local ductility provisions, the ratio between the residual resistance after degradation and the initial resistance is approximately equal to the ratio between the values of $\gamma_M$ for accidental and fundamental load combinations, the values of the partial safety coefficients, $\gamma_c$ and $\gamma_s$, adopted for persistent and transient calculation situations must be applied in the calculation seismic situation.

NB The values to be assigned to the coefficients $\gamma_c$ and $\gamma_s$ in situations of persistent, transient and accidental calculation can be found in Annex 19 of the Structural Code.

## 5.3 Dimensioning according to Annex 19 to the Structural Code

### 5.3.1 General considerations

(1) Seismic dimensioning is permitted for the low ductility class (DCL) following Annex 19 of the Structural Code without any other requirements except those indicated in **5.3.2**, only in cases of low seismicity (see point **(4)** of section **3.2.1**).

### 5.3.2 Materials

(1) In primary seismic elements (see **4.2.2**), steel must be used for passive reinforcement of type S or SD as defined in Articles 34 and 35 of the Structural Code.

### 5.3.3 Behaviour coefficient

(1) In the calculation of seismic actions a behaviour coefficient *q* maximum of 1.5 can be used, regardless of the type of structural system and the regularity in height.

## 5.4 Dimensioning for medium ductility class (DCM)

### 5.4.1 Geometric and material restrictions

#### 5.4.1.1 Materials

(1) In primary seismic elements, the characteristic strength concrete $f_{ck}$ less than 25 N/mm$^2$ must not be used, in accordance with Article 33.4 of the Structural Code.

(2) Except in the case of fences and transverse states, in critical areas of primary seismic elements only corrugated rounds should be used as steel for passive reinforcement.

(3) In critical areas of primary seismic elements, reinforcement steel must be used for passive reinforcement of type S or SD according to Articles 34 and 35 of the Structural Code.

(4) Electro-welded meshes can be used if they meet the requirements specified in points **(2)** and **(3)** above.

#### 5.4.1.2 Geometric restrictions

##### 5.4.1.2.1 Beams

(1) The eccentricity of the axis of the beam with respect to the axis of the pillar with which it links should be limited in order to achieve an effective transfer of cyclical moments from a primary earthquake-resistant beam to a pillar.

(2) To meet the requirement of point **(1)**, the distance between the axis of the centres of gravity of the two elements must be limited to less than $b_c/4$, where $b_c$ is the largest of the dimensions of the cross section of the normal pillars to the longitudinal axis of the beam.

(3) In order to take advantage of the favourable effect of pillar compression on the adhesion of horizontal trusses passing through the knot, the width $b_w$ of a primary earthquake-resistant beam must satisfy the following equation:

$$b_w \leq \text{mín.}\left|b_c + h_w;\ 2b_c\right| \tag{5.6}$$

where $h_w$ is the edge of the beam and $b_c$ has been defined in point (2) of this section.

#### 5.4.1.2.2 Pillars

(1) The minimum cross-sectional dimensions of the primary earthquake-resistant pillars should not be less than 200 mm.

(2) Unless $\theta \leq 0.1$ (see **4.4.2.2(2)**), the cross-sectional dimensions of primary earthquake-resistant pillars must not be smaller than:

— the 20th of the greatest distance between the turning point of the deformed one and the ends of the pillar, for bending in a plane parallel to the considered dimension of the pillar;

— 250 mm.

#### 5.4.1.2.3 Ductile walls

(1) The thickness of the web, $b_{wo}$ (in meters), must satisfy the following equation:

$$b_{wo} \geq \text{máx.}\left|0,15, h_s / 20\right| \tag{5.7}$$

where $h_s$ is the floor free height in meters.

(2) For the thickness of confined contour elements, additional criteria specified in point **(10)** of section **5.4.3.4.2** are applied.

#### 5.4.1.2.4 Large walls lightly reinforced

(1) The condition given in point **(1)** of section **5.4.1.2.3** is also applied in the case of lightly reinforced large walls.

#### 5.4.1.2.5 Specific rules for beams supporting isolated vertical elements

(1) Structural walls must not be supported (in whole or in part) by beams or slabs.

(2) For primary earthquake-resistant beams supporting isolated pillars, the following conditions must be met:

a) there should be no eccentricity between the axis of the pillar and that of the beam;

b) the beam must have at least two direct supports, such as walls or pillars.

### 5.4.2 Effects of the calculation action

#### 5.4.2.1 General considerations

(1) Except in the case of primary earthquake-resistant ductile walls, for which section **5.4.2.4**applies, the calculation values of the bending moments and axillary forces must be obtained from the dimensioning of the structure in the seismic calculation situation, according to section **6.4.3.4** of Annex 18 to the Structural Code, taking into account, in addition, the second order effects in accordance with section **4.4.2.2** of this Annex and the capacity dimensioning criteria indicated in point **(2)** of section **5.2.3.3**. A redistribution of bending moments is permitted in accordance with Annex 19 of the Structural Code. The calculation values of the shear strength of beams, pillars, ductile walls and lightly reinforced primary earthquake-resistant walls are obtained, respectively, according to sections **5.4.2.2**, **5.4.2.3**, **5.4.2.4 and 5.4.2.5** of this Annex.

#### 5.4.2.2 Beams

(1) In primary earthquake-resistant beams, the shear strength of calculation should be determined according to the capacity dimensioning criterion, depending on the equilibrium situation of the low beam: a) transverse charge acting on it in the seismic calculation situation; and b) moments at the end $M_{i,d}$ (with $i$ = 1.2 indicating the ends of the beam), associated with the formation of plastic balls for the positive and negative directions of the seismic charge. Plastic hinges must be assumed to form either at the ends of the beam or (if formed there first) on the vertical elements connected to the knots into which the ends of the beams are inserted (see Figure 5.1).

(2) Point **(1)** must be applied as follows:

a) in the extreme section $i$, two shearing stress values must be calculated, i.e. maximum $V_{Ed,max,i}$ and minimum $V_{Ed,min,i}$ corresponding to the maximum positive value and the maximum negative value of the moments at the end $M_{i,d}$ which can be developed at ends 1 and 2 of the beam;

b) the moments at the end $M_{i,d}$ that appears in points **(1)** and **(2)**(a) can be calculated by:

$$M_{i,d} = \gamma_{Rd} M_{Rb,i} \min\left(1, \frac{\sum M_{Rc}}{\sum M_{Rb}}\right) \quad (5.8)$$

Where

$\gamma_{Rd}$ is the coefficient that considers the possible reserve of resistance due to hardening by deformation of the steel and which, in the case of beams of DCM ductility class, can be taken as 1.0;

$M_{Rb,i}$ is the calculation value of the bending resistance of the beam at its end $i$, in the sense of seismic bending moment under the considered sense of seismic action;

$\Sigma M_{Rc}$ and $\Sigma M_{Rb}$ they represent the sum of the calculation values of the sturdy moments of the pillars and the sum of the calculation values of the resistant moments of the beams confluent in the knot, respectively (see point **(4)** of section **4.4.2.3**). The value of $\Sigma M_{Rc}$ must correspond to the axillary forces of the pillar in the seismic calculation situation, for the considered sense of the seismic action.

c) at the end of a beam where it is supported by another beam, instead of meeting a vertical element, the moment at the end $M_{i,d}$ can be taken as equal to the actuating moment in the extreme section of the beam in the seismic calculation situation.

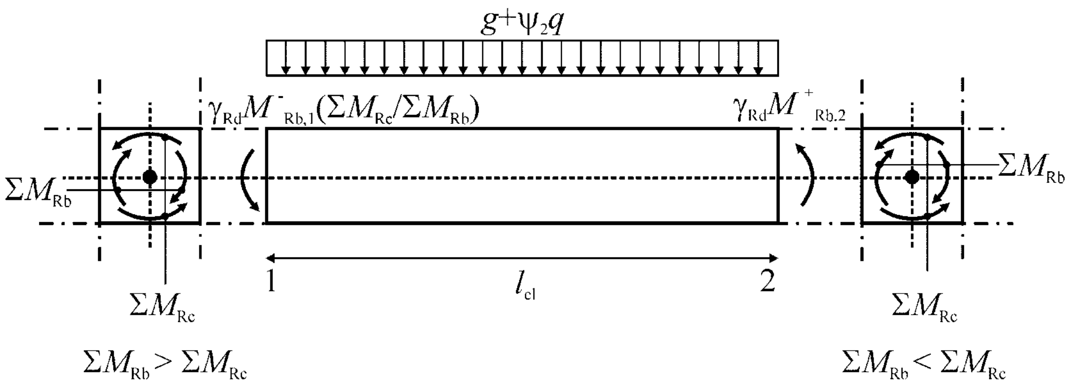



**Figure 5.1 – Capacity dimensioning values of shear strength on beams**

### 5.4.2.3 Pillars

(1) In primary earthquake-resistant pillars, the calculation values of the shear strength should be determined according to the capacity dimensioning criterion, depending on the equilibrium situation of the pillar under the moments, at the end $M_{i,d}$ (with $i$ = 1.2 indicating the ends of the pillar), corresponding to the formation of plastic bearings for the positive and negative directions of the seismic load. Plastic hinges must be assumed to form at the ends of the beams or (if formed there first) on the vertical elements connected to the knots into which the ends of those pillars are inserted (see Figure 5.2).

(2) The moments at the end $M_{i,d}$ that appear in the section **(1)** can be calculated by:

$$M_{i,d} = \gamma_{Rd} M_{Rc,i} \min\left(1, \frac{\sum M_{Rb}}{\sum M_{Rc}}\right) \quad (5.9)$$

Where

$\gamma_{Rd}$ is the coefficient that considers the possible reserve of strength due to steel deformation hardening and the confinement of concrete in the compression area of the section, which is taken as 1.1;

$M_{Rc,i}$ is the calculation value of the sturdy moment of the pillar at its end $i$, in the sense of seismic bending moment under the considered sense of seismic action;

$\Sigma M_{Rc}$ andthey are defined in point **(2)** of section **5.4.2.2**.

$\Sigma M_{Rb}$

(3) The values of $M_{Rc,i}$ and of $\Sigma M_{Rc}$ must correspond to the axial force(s) of the pillar in the seismic calculation situation for the considered sense of seismic action.

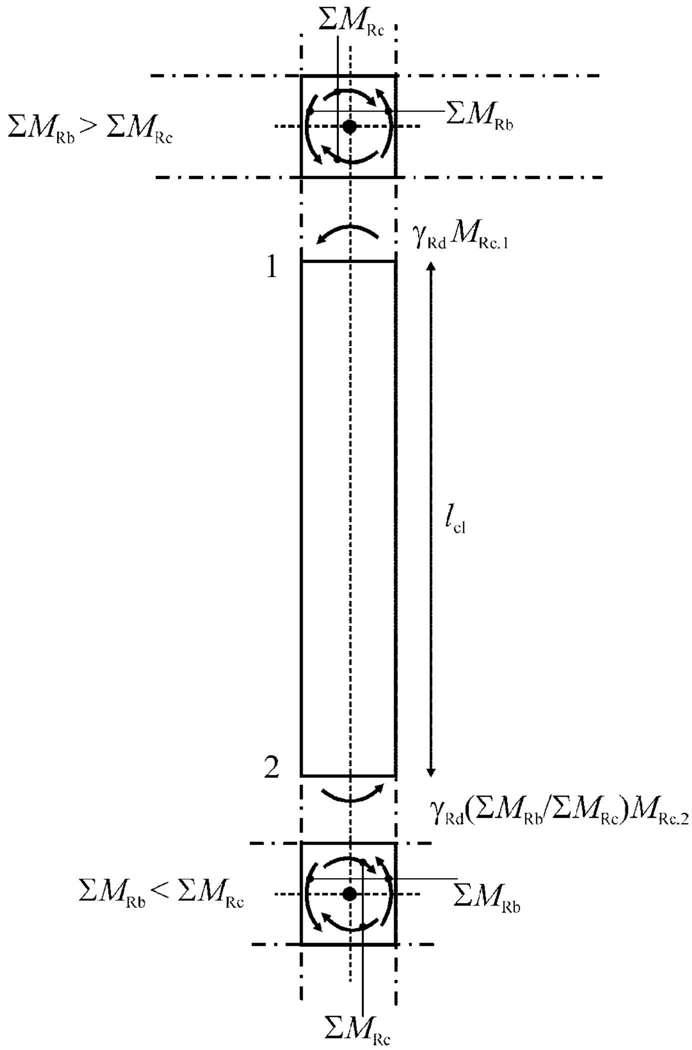



**Figure 5.2 – Capacity dimensioning values for shear strength in pillars**

#### 5.4.2.4 Special provisions for ductile walls

(1) Uncertainties in the calculation and postelastic dynamic effects should be taken into account, at least by an appropriate simplified method. If a more precise method is not available, the rules indicated in the following sections of the calculation envelopes for bending moments, as well as for the coefficients of increase of the shear strength, can be used.

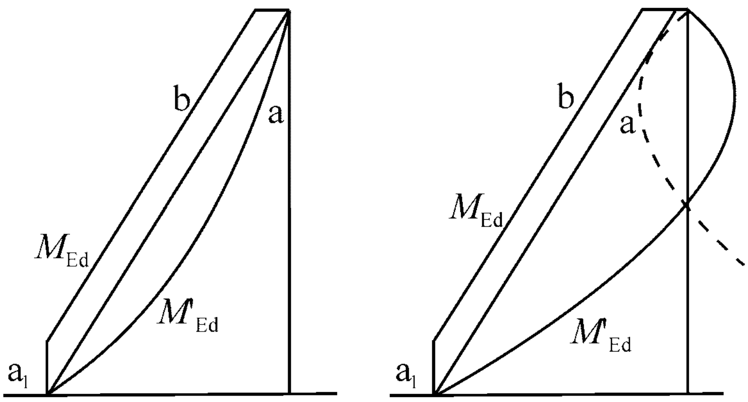
(2) A redistribution of the effects of seismic action of up to 30 % between primary earthquake-resistant walls is permitted, provided that there is no reduction in the total demand for resistance. Shearing forces should be redistributed along with the ending moments, so that the ratio between these ending moments and the shear strength is not significantly affected in the walls considered in isolation. In walls subjected to major variations in the axillary forces, such as in coupled walls, the moments and shearing must be redistributed from the poorly compressed walls to the walls subjected to high axillary compressions.

(3) In coupled walls, a redistribution of the seismic action, up to 20 %, between coupling beams of different floors is allowed, provided that the seismic axial force at the base of each wall considered in isolation (the resulting from the shear strength on the coupling beams) is not affected.

(4) Uncertainties relating to the distribution of moments along the height of slender primary earthquake-resistant walls (with a ratio between height and length $h_w/l_w$ greater than 2.0 should be considered).

(5) The requirements of point **(4)** can be considered complied with by applying, regardless of the type of analysis used, the following simplified procedure:

The calculation bending-moment diagram along the height of the wall must be obtained by means of an envelope of the calculated bending-moment diagram (determined from the structural calculation), moved vertically (translation of the tractions). The enveloping curve can be assumed linear if the structure does not show significant discontinuities of mass, rigidity or strength along its height (see Figure 5.3). The translation of tractions must be consistent with the inclination of the crank taken for checking the USL of the shearing, with a possible distribution in crank fan near the base, and with the concrete slab acting as straps.




Legend

a Bending-moment diagram calculated by analysis

b Calculation envelope

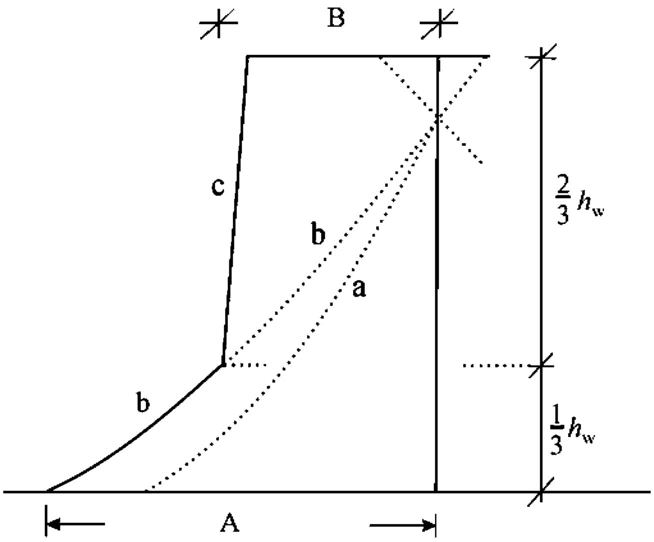
$a_1$ Translation of tractions

**Figure 5.3 – Calculation envelope for ending moments on slender walls (left: wall systems, right: dual systems)**

(6) Possible increases in the shear strength at the base of the primary earthquake-resistant wall after plasticisation should be taken into account.

(7) The requirement specified in point **(6)** can be considered complied with if the shear strength are increased by 50 % compared to those obtained in the calculation.

(8) In dual systems containing slender walls, the calculation envelope of the ending moments in Figure 5.4 must be used to take into account uncertainties in higher order effects.




Legend

a Diagram of the shear strength calculated by analysis

b Diagram of increased shear strength

c Calculation envelope

A $V_{\text{wall,base}}$

B $V_{\text{wall,top}} \geq V_{\text{wall,base}}/2$

**Figure 5.4 – Computing envelope of shear strength on slender walls of dual systems**

### 5.4.2.5 Special provisions for large, lightly reinforced walls

(1) The shear strength $V'_{Ed}$, obtained from the calculation should be increased to ensure that bending plasticisation precedes the formation of the final cutter state.

(2) The requirement indicated in point **(1)** of this section is considered complied with if, on each wall floor, the calculation value of the shear strength $V_{Ed}$ is obtained from the shear strength calculated in the analysis, $V'_{Ed}$, through the following equation:

$$V_{Ed} = V'_{Ed}\,\frac{q+1}{2} \tag{5.10}$$

(3) When performing the USL check of the bending wall with axial stress, consideration should be given to the additional dynamic axillary forces developed in large walls due to the take-off from the ground (mismatch) or the opening and closing of horizontal cracks.

(4) Unless more accurate calculation results are available, the dynamic component of the wall axial force indicated in point **(3)** can be taken as 50 % of the axial force on the wall due to the gravitational loads present in the seismic calculation situation. This force should be taken with a positive or negative sign, whichever is more unfavourable.

(5) If the value of the behaviour coefficient, $q$, does not exceed 2.0, the effect of dynamic axial stress

referred to in points **(3)** and **(4)**. can be disregarded.

### 5.4.3 USL check and construction details

#### 5.4.3.1 Beams

##### 5.4.3.1.1 Bending and shear strength

(1) Bending and shearing resistances must be calculated in accordance with Annex 19 to the Structural Code.

(2) The upper reinforcements in the extreme cross-sections of the primary earthquake-resistant beams in T or L must be placed primarily within the width of the web. Only part of this reinforcement can be placed outside, but within the effective wing width $b_{eff}$.

(3) Effective wing width $b_{eff}$ can be assumed as follows:

a) for primary earthquake-resistant beams meeting outer pillars, effective wing width $b_{eff}$, in the absence of cross beams, is taken as the width $b_c$ of the pillar (see Figure 5.5 b), or equal to this increased width in 2 $h_f$ on each side (see Figure 5.5a) if there is a transverse beam of similar width;

b) for primary earthquake-resistant beams that converge into inner pillars, the widths indicated in (a) can be increased by 2 $h_f$ on each side of the beam (see Figures 5.5c and 5.5d).

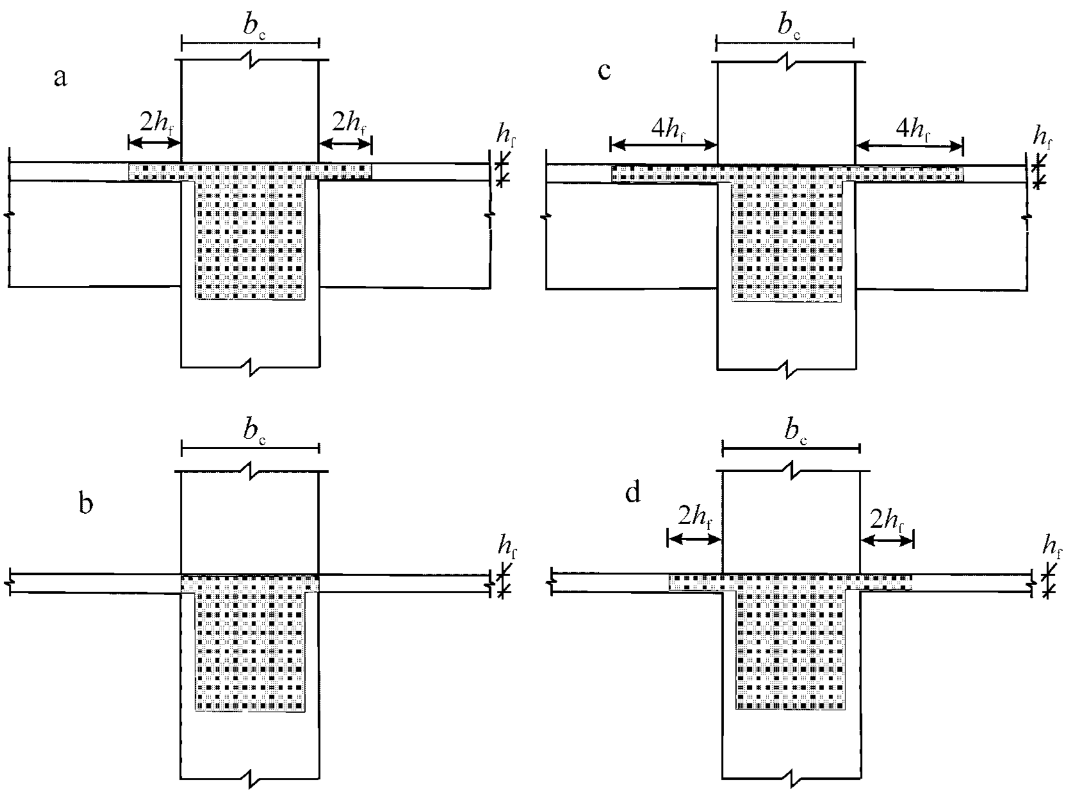



**Figure 5.5 – Effective wing width $b_{eff}$ for beams meeting pillars**

#### 5.4.3.1.2 Construction details relating to local ductility

(1) Critical areas should be considered those areas of a primary earthquake-resistant beam located within a distance $l_{cr} = h_w$ (where $h_w$ is the edge of the beam) measured from an extreme section where the beam is connected to a beam-pillar knot, as well as those within the same distance $l_{cr} = h_f$, considered on both sides of any other section susceptible to plasticising in the seismic calculation situation.

(2) In primary earthquake-resistant beams that support discontinuing vertical elements (interrupted), areas within a distance 2 $h_w$ measured on both sides of the supported vertical element should be considered as critical areas.

(3) In order to satisfy the local ductility criterion in the critical areas of primary earthquake-resistant beams, the value of the coefficient of ductility in curvatures $\mu_\phi$ must be at least equal to that indicated in point **(3)** of section **5.2.3.4**.

(4) The requirement specified in point **(3)** is considered complied with if the following conditions are met in both wings of a beam:

a) in the compression area there is a reinforcement not less than half of that arranged in the traction area, in addition to any reinforcement necessary for balance, in addition to the compression reinforcement necessary to check the ultimate limit state in the seismic calculation situation.

b) the amount of the $\rho$ tensile reinforcement does not exceed a maximum value $\rho_{max.}$ equal to:

$$\rho_{\text{máx.}} = \rho' + \frac{0,0018}{\mu_\varphi \varepsilon_{\text{sy,d}}} \cdot \frac{f_{\text{cd}}}{f_{\text{yd}}} \tag{5.11}$$

with the amounts of the tensile and compressive reinforcements $\rho$ and $\rho'$, normalised with respect to $bd$, with $b$ being the width of the beam compression wing. If the traction area includes a slab, the amount of slab reinforcement parallel to the beam within the effective wing width defined in point **(3)** of section **5.4.3.1.1** is included in $\rho$.

(5) Throughout the entire primary earthquake-resistant beam, the amount of $\rho$ tensile reinforcement should not be less than the following minimum value $\rho_{min.}$:

$$\rho_{\text{min.}} = 0,5 \left( \frac{f_{\text{ctm}}}{f_{\text{yk}}} \right) \tag{5.12}$$

(6) Within critical areas of primary earthquake-resistant beams, fences must be provided that comply with the following conditions:

a) the diameter $d_{bw}$ of the fences (in millimetres) should not be less than 6;

b) the separation, $s$, between fences (in millimetres) must not be greater than:

$$s = \min.\left\{ h_w / 4;\ 24d_{bw};\ 225;\ 8d_{bL} \right\} \qquad (5.13)$$

Where

$d_{bL}$ is the minimum diameter of the longitudinal reinforcement (in millimetres); and

$h_w$ is the edge of the beam (in millimetres);

c) the first fence must not be more than 50 mm from the end of the beam (see Figure 5.6).

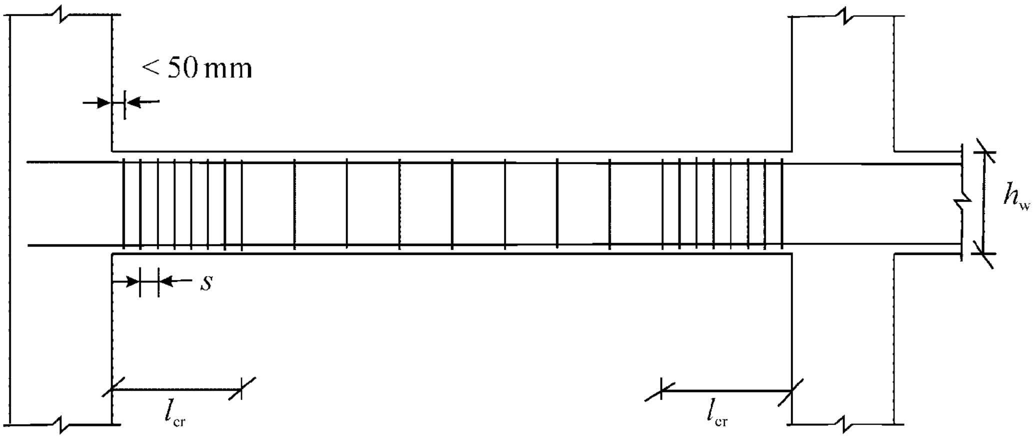



**Figure 5.6 – Transverse reinforcement in critical beam areas**

### 5.4.3.2 Pillars

#### 5.4.3.2.1 Resistances

(1) The bending and shearing resistances must be calculated according to Annex 19 of the Structural Code, using the value of the axial force obtained from the dimensioning in the seismic calculation situation.

(2) Biaxial bending can be taken into account in a simplified way, performing the test separately in each direction with the simple bending shear strength reduced by 30 %.

(3) In primary earthquake-resistant pillars, the reduced axial stress value $\nu_d$ should not exceed 0.65.

#### 5.4.3.2.2 Construction details relating to the local ductility of primary earthquake-resistant pillars

(1) The total amount of longitudinal reinforcement $\rho_l$ must not be less than 0.01 or greater than 0.04. In symmetrical cross-sections a symmetrical reinforcement must be provided ($\rho = \rho'$).

(2) At least one intermediate round between the corners along each side of the pillar should be arranged to ensure the integrity of the junctions between beam and pillar.

(3) Areas within a distance $l_{cr}$, measured from the two extreme sections of a primary earthquake-resistant pillar, should be considered critical areas.

(4) In the absence of more accurate information, the critical area length $l_{cr}$ (in meters) can be calculated using the following equation:

$$l_{cr} = \text{máx.}\left\{ h_c;\ l_{cl} / 6;\ 0{,}45 \right\} \tag{5.14}$$

Where

$h_c$ is the largest cross-sectional dimension of the pillar (in metres); and

$l_{cl}$ is the free length of the pillar (in meters).

(5) If $l_c/h_c < 3$, the entire height of the primary earthquake-resistant pillar should be considered as a critical area and should therefore be assembled as such.

(6) In the critical area of the base of the primary earthquake-resistant pillars, a value of the coefficient of ductility in curvatures must be equal to that indicated in point **(3)** of section **5.2.3.4**.

(7) If, for the specified value of $\mu_{\phi}$, a final unit deformation of the non-confined concrete greater than $\varepsilon_{cu2} = 0.0035$ is required in any area of the cross-section, the loss of strength due to the loss of concrete coating should be compensated by proper confinement of the concrete core, considering the properties of the confined concrete indicated in section **3.1.9** of Annex 19 of the Structural Code.

(8) The requirements specified in points **(6)** and **(7)** of this section are assumed to be complied with if:

$$\alpha\omega_{wd} \geq 30\mu_{\phi}\nu_d \cdot \varepsilon_{sy,d} \cdot \frac{b_c}{b_o} - 0{,}035 \tag{5.15}$$

Where

$\omega_{wd}$ mechanical volumetric quantity of confinement fences in critical areas

$$\left[\omega_{wd} = \frac{\text{volume of confinement fences}}{\text{concrete core volume}} \cdot \frac{f_{yd}}{f_{cd}}\right]$$

$\mu_{\phi}$ is the required value of the coefficient of ductility in curvatures;

$\nu_d$ is the calculation value of reduced axial force ($\nu_d = N_{Ed}/A_c \cdot f_{cd}$);

$\varepsilon_{sy,d}$ is the calculation value of the unit deformation of steel corresponding to the elastic limit;

$h_c$ is the transverse raw edge;

$h_o$ is the edge of the confined core (to the axis of the fences);

$b_c$ is the transverse gross width;

$b_o$ is the width of the confined core (up to the axis of the fences)

$\alpha$ is the confinement efficiency coefficient, equal to $\alpha = \alpha_n \cdot \alpha_s$, with:

a) for rectangular cross-sections:

$$\alpha_n = 1 - \sum_n b_i^2 / 6 b_o h_o \tag{5.16a}$$

$$\alpha_s = (1 - s / 2b_o)(1 - s / 2h_o) \tag{5.17a}$$

Where

$n$ is the total number of longitudinal rounds laterally bound by fences or transverse states; and

$b_i$ is the distance between two consecutive linked reinforcements (see Figure 5.7; also for $b_o$, $h_o$, s).

b) for circular cross-sections with circular fences and a confined core diameter $D_o$ (up to the axis of the fences):

$$\alpha_n = 1 \tag{5.16b}$$

$$\alpha_s = (1 - s / 2D_o)^2 \tag{5.17b}$$

c) for circular cross-sections with helical confinement reinforcement:

$$\alpha_n = 1 \tag{5.16c}$$

$$\alpha_s = (1 - s / 2D_o) \tag{5.17c}$$

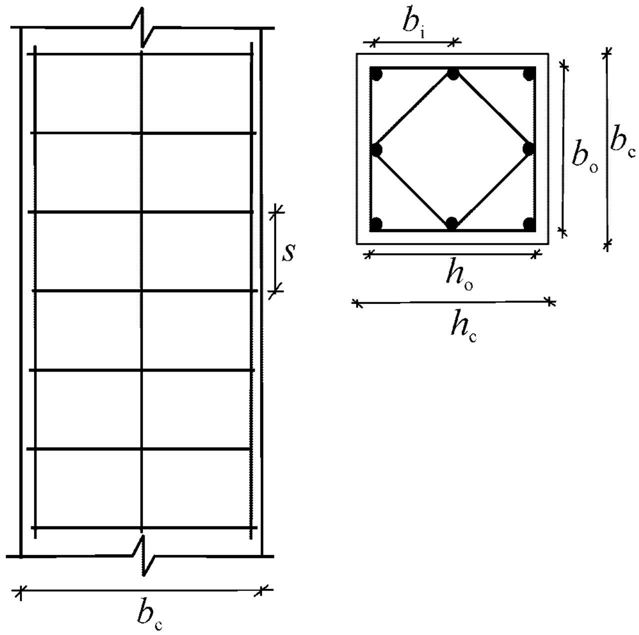



**Figure 5.7 – Concrete core confining**

(9) A minimum value of $\omega_{wd}$ equal to 0.08 must be provided in the critical area of the base of the primary earthquake-resistant pillars.

(10) In the critical areas of the primary earthquake-resistant pillars, cross-sectional encirclements and states of at least 6 mm diameter must be arranged, with a separation such that a minimum ductility is ensured, and the local buckling of the longitudinal reinforcements is avoided. The pattern of placement of the fences should be such that the cross sections of the pillar benefit from the triaxial compression conditions produced by those fences.

(11) The minimum conditions set out in point **(10)** are considered to be complied with if the following conditions are met:

a) the separation, $s$, between fences (in millimetres) does not exceed:

$$s = \min\left\{ b_o / 2;\ 175; 8d_{bL} \right\} \tag{5.18}$$

Where

$b_o$ is the minimum dimension of the concrete core, up to the axis of the fences (in millimetres); and

$d_{bL}$ is the minimum diameter of the longitudinal reinforcement (in millimetres).

b) the distance between two consecutive longitudinal rounds bound by fences or transverse states does not exceed 200 mm. In addition, based on the provisions of section 9.5.3 (6) of Annex 19 to the Structural Code, it must be complied with that any longitudinal bar is properly fastened by the corner of a fence or by a transverse tying when it is more than 150 mm from another that is fastened.

(12) The transverse reinforcement in the critical areas of the base of primary earthquake-resistant pillars can be determined according to Annex 19 of the Structural Code, if the reduced axial load in the seismic calculation situation is less than 0.2 and the value of the behaviour coefficient $q$ used for the calculation does not exceed 2.0.

#### 5.4.3.3 Joints between beam and pillar

(1) The horizontal confinement reinforcement in primary earthquake-resistant beam junctions with pillars must not be less than that specified in points **(8)** to **(11)** of section **5.4.3.2.2** for critical areas of the pillars, with the exception of the case indicated in the following point.

(2) If the beams concur on the four sides of the knot and their width is at least three-quarters of the parallel dimension of the cross section of the pillar, the separation of the horizontal confinement reinforcement in the knot may be increased to twice that specified in point **(1)** of this section, but not exceeding 150 mm.

(3) Between the corner reinforcements of the pillar, at least one vertical intermediate round must be arranged on either side of the junction knot between primary earthquake-resistant beams and pillars.

#### 5.4.3.4 Ductile walls (screen walls)

##### 5.4.3.4.1 Bending and shear strength

(1) Bending and shearing resistances should be calculated in accordance with Annex 19 to the Structural Code, using the value of the axial force obtained from the calculation in the seismic calculation situation, unless otherwise specified in the following points.

(2) In primary earthquake-resistant walls the value of the normalised axial load, $\nu_d$, must not exceed 0.4.

(3) The vertical reinforcement of the web should be taken into account in the calculation of the bending resistance of the sections of the wall.

(4) Sections of composite walls consisting of connected or interlinked segments (sections in L, T, U, I, or the like) must be considered integral units, consisting of a web or souls parallel to or approximately parallel to the direction of the acting seismic shear strength, and in a normal or approximately normal wing or wings in that direction. In order to calculate the bending resistance, the effective width of the wing on each side of a web must be assumed to extend from the face of the web to a minimum distance of:

a) the actual width of the wing;

b) half the distance to an adjacent web of the wall; and

c) 25 % of the total height of the wall above the level considered.

##### 5.4.3.4.2 Construction details relating to local ductility

(1) The height of the critical area, $h_{cr}$, above (and, if applicable, below) the base of the wall can be estimated as follows:

$$h_{cr} = \max [l_w\ h_w /6] \tag{5.19a}$$

but with:

$$h_{cr} \leq \begin{cases} 2 \cdot l_w \\ \begin{cases} h_s & \text{for n} \leq 6 \text{ floors} \\ 2 \cdot h_s & \text{for n} \geq 7 \text{ floors} \end{cases} \end{cases} \tag{5.19b}$$

where $h_s$ is the free height of the floor, and where the base is defined as the level of foundation or underrun in basement floors with rigid diaphragms and perimeter walls (see point **(5)** of section **5.8.1**).

(2) In critical wall areas, a value of the coefficient of ductility in curvatures, $\mu_\phi$, which is at least equal to that calculated by equations (5.4) and (5.5) of point **(3)** of section **5.2.3.4**, where the base value of the behaviour coefficient, $q_o$, in such equations is replaced by the product of $q_o$ times the value of the ratio $M_{Ed}/M_{Rd}$ at the base of the wall in the seismic calculation situation, in which $M_{Ed}$ is the calculation value of the bending moment obtained from the analysis, and $M_{Rd}$ is the calculation value of the bending resistance.

(3) Unless a more precise method is used, the value of $\mu_\phi$ specified in point **(2)** of this section can be achieved by confinement reinforcement within the border areas of the cross section, called contour elements, the extent of which must be determined according to the point **(6)** of this section. The amount of confinement reinforcement must be calculated according to the points **(4)** and **(5)** below.

(4) In the case of walls with rectangular cross-sections, the volumetric mechanical amount of the required confinement reinforcement, $\omega_{wd}$, in the contour elements must satisfy the following equation, with the values of $\mu_\phi$ indicated in point **(2)** above:

$$\alpha\omega_{wd} \geq 30\mu_\varphi (\nu_d + \omega_\nu)\, \varepsilon_{sy,d} \frac{b_c}{b_o} - 0{,}035 \tag{5.20}$$

whose parameters are defined in **(8)** point of section 5.4.3.2.2, except $\omega_v$, which is the mechanical amount of the vertical web reinforcement ($\omega_v = \rho_v f_{yd,v}/f_{cd}$).

(5) In the case of walls with stiffeners or wings, or with a section composed of several rectangular parts (sections in T, L, I, U, etc.), the volumetric mechanical amount of the confinement reinforcement in the contour elements can be determined as follows:

a) the axis, $N_{Ed}$, and the total area of the vertical web reinforcement, $A_{sv}$, should be normalised to $h_c\ b_c\ f_{cd}$, where the width of the stiffener or compressed wing is taken equal to the width of cross section $b_c$ ($\nu_d = N_{Ed} / h_c b_c f_{cd}$, $\omega_v = (A_{sv} / h_c b_c)\, f_{yd} / f_{cd}$). The depth of the neutral fibre $x_u$ corresponding to the ultimate curvature situation after the loss of the coating (splitting) of the concrete located outside the confined core of the contour elements can be estimated as:

$$x_u = (\nu_d + \omega_\nu) \frac{l_w b_c}{b_o} \tag{5.21}$$

where $b_o$ is the width of the core confined in the contour elements, stiffeners or wings. If the value of $x_u$ of equation (5.21) is not greater than the edge of the stiffener or wing after the loss of concrete coating, the volumetric mechanical amount of the confinement reinforcement in the stiffener or wing is determined as in point (a) of this section (i.e. according to equation (5.20), the point **(4)** of section **5.4.3.4.2**) with $\nu_d$, $\omega_v$, $b_c$ and $b_o$ referring to the edge of the stiffener or wing.

b) if the value of $x_u$ is greater than the width of the stiffener or wing after the loss of concrete coating, the general method based on: 1) Definition of the coefficient of ductility in curvatures such as $\mu_\phi = \phi_u / \phi_y$, 2) the calculation of $\phi_u$ as $\varepsilon_{cu2,c}/x_u$ and of $\phi_y$ as $\varepsilon_{sy} / (d - x_y)$; 3) Balancing section for neutral fibre depth estimation $x_u$ and $x_y$; and 4) the values of the strength and ultimate unit deformation of confined concrete, $f_{ck,c}$ and $\varepsilon_{cu2,c}$ depending on the effective lateral confinement voltage (see section **3.1.9** of Annex 19 of the Structural Code). The confinement reinforcement and the length of the confined walls must be dimensioned accordingly.

(6) The reinforcement indicated in points **(3)** to **(5)** of this section must be extended vertically over the height $h_{cr}$ of the critical area, as defined in point **(1)** of section **5.4.3.4.2** and horizontally over a length $l_c$ measured from the extreme compression fibre of the wall to the point where the unconfined concrete could be peeled due to high unit deformations by compression. If more accurate data are not available, the ultimate unit compression deformation of the non-confined concrete at which the peeling is expected can be taken equal to $\varepsilon_{cu2}$ = 0.0035. The confined contour element can be limited to a distance $x_u\,(1 - \varepsilon_{cu2}/\varepsilon_{cu2,c})$ from the fence axis close to the extreme compression fibre, with $x_u$ being the depth of the confined compression area, corresponding to the ultimate curvature estimated by equilibrium (see equation (5.21) for a width of the confined compression area, $b_o$, constant) and the ultimate unit deformation of confined concrete, $\varepsilon_{cu2,c}$, estimated based on section **3.1.9** of Annex 19 of the Structural Code as $\varepsilon_{cu2,c} = 0.0035 + 0.1\,\alpha\omega_{wd}$ (see Figure 5.8). The length $l_c$ of the confined contour element must not be less than $0.15 \cdot l_w$ or $1.50 \cdot b_w$

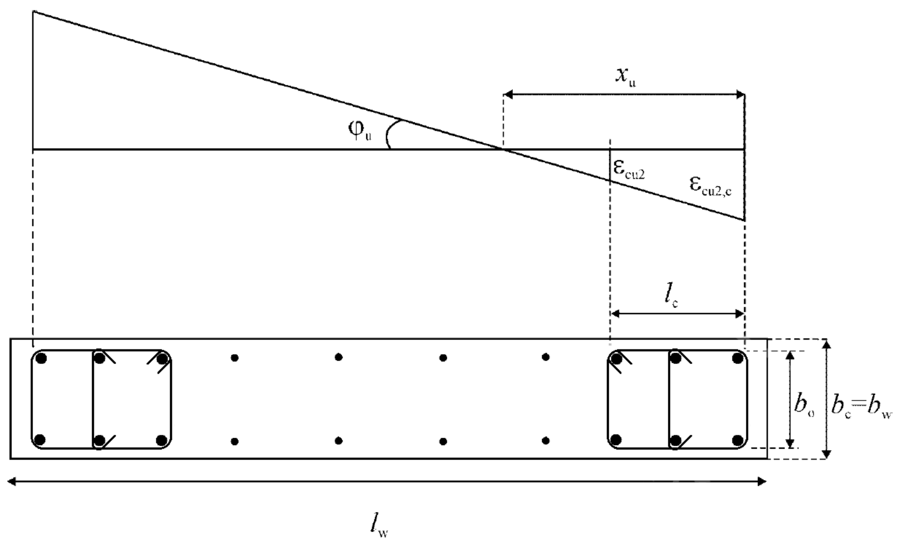



**Figure 5.8 – Confined contour element of a wall end with free edge. (Top: unit deformations for the ultimate curvature. Bottom: cross-section of the wall; the horizontal reinforcement of the web is not represented in the Figure)**

(7) It is not necessary to have a boundary element confined on wall wings that has a thickness $b_f \geq h_s/15$ and thickness $l_f \geq h_s/5$, where $h_s$ designates the floor free height (see Figure 5.9). However, confined contour elements may be needed at the end of this type of wings, due to out-of-plane bending of the wall.

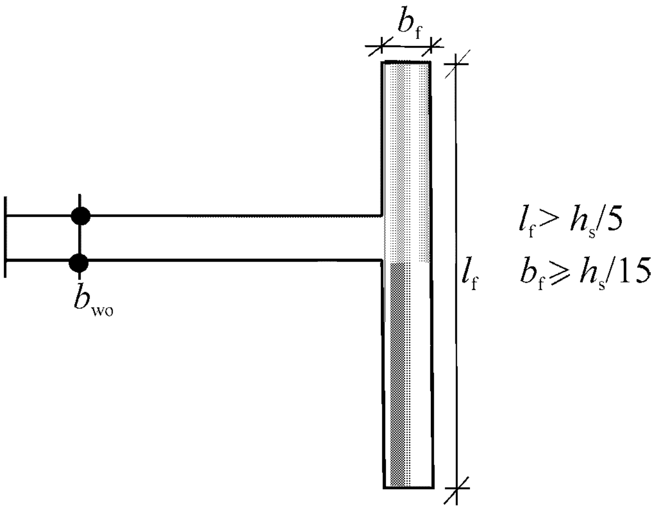



**Figure 5.9 – Confined contour elements not needed at the end of walls with a large transverse stiffener**

(8) The amount of longitudinal reinforcement in the contour elements must not be less than 0.005.

(9) The provisions of points **(9)** to **(11)** of section **5.4.3.2.2** are also applied within the boundary elements of walls. Overlapping fences must be used, so that any longitudinal round is linked by a fence or a transverse tying, using hooks bent at 135° and length 10 $d_{bw}$.

(10) The thickness $b_w$ of the confined parts of the wall section (contour elements) must not be less than 200 mm. In addition, if its length does not exceed the highest value between $2b_w$ and 0.2 $l_w$, $b_w$ should be no less than $h_s/15$, where $h_s$ designates the floor free height. If this length exceeds the maximum value between $2b_w$ and 0,2 $l_w$, $b_w$ should be no less than $h_s$ /10 (see Figure 5.10).

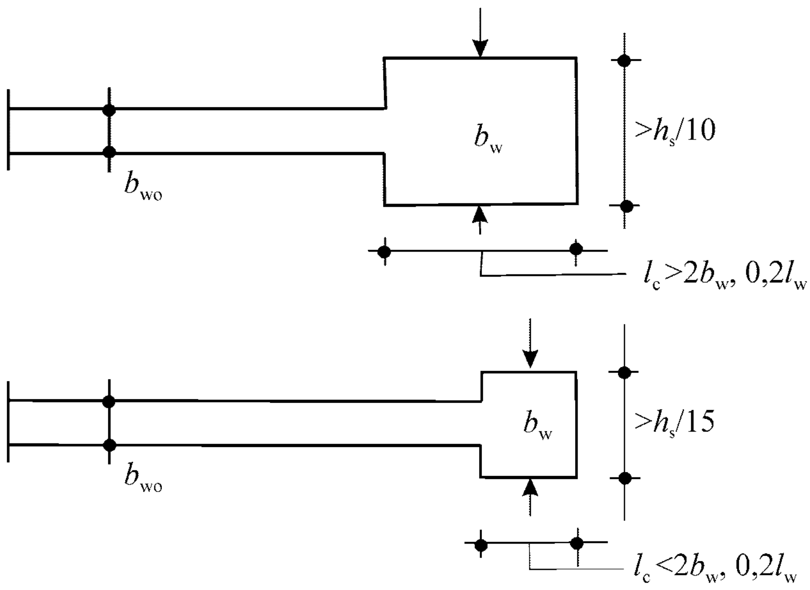



**Figure 5.10 – Minimum thickness of confined contour elements**

(11) In areas of the wall above the critical area, only the relevant rules of Annex 19 of the Structural Code apply to vertical, horizontal and transverse reinforcements. However, in those parts of the section in which the compression unit deformation, $\varepsilon_c$, is greater than 0.002 in the seismic calculation situation, a minimum amount must be provided for the vertical reinforcement of 0.005.

(12) The cross-sectional truss of the contour elements indicated in points **(4)** to **(10)** of this section can be determined by considering only the requirements of Annex 19 to the Structural Code if any of the following conditions are met:

a) the calculation value of reduced axial force, $\nu_d$, is not greater than 0.15; or,

b) the value of $\nu_d$ is not greater than 0.20 and the coefficient $q$ used for the calculation has been reduced by 15 %.

#### 5.4.3.5 Large walls lightly reinforced

##### 5.4.3.5.1 Bending resistance

(1) The USL bending with axial stress should be checked assuming horizontal fissuring, in accordance with the specific provisions indicated in Annex 19 of the Structural Code, including the hypothesis of flat sections.

(2) Normal stresses in concrete should be limited in order to prevent instability outside the wall.

(3) The requirement specified in point **(2)** of this section can be complied with by using the rules relating to the second order effects indicated in Annex 19 to the Structural Code, supplemented by other rules relating to normal stresses in concrete, if necessary.

(4) When considering the dynamic axial stress of the points **(3)** and **(4)** of section **5.4.2.5** in the check of the USL bending with axial stress, the ultimate unit deformation, $\varepsilon_{cu2,c}$, of unconfined concrete can be increased to 0.005. A higher value may be taken for confined concrete, in accordance with section **3.1.9** of Annex 19 to the Structural Code, provided that the loss of the coating (splitting) of

unconfined concrete is considered.

#### 5.4.3.5.2 Shear strength

(1) Due to the safety margin provided by the increase of the calculation values of the shear strength in the points **(1)** and **(2)** of section **5.4.2.5** and the fact that the response (including possible tilted cracking) is controlled with respect to deformation, provided that the value of $V_{Ed}$ indicated in point **(2)** of section **5.4.2.5** is less than or equal to the calculation value of the shear strength $V_{Rd,c}$ indicated in section **6.2.2** of Annex 19 to the Structural Code, no minimum amount of shearing reinforcement is required in the web, $\rho_{w,min}$.

NB The value of $\rho_{w,min.}$ is the minimum value for walls indicated in Annex 19 of the Structural Code.

(2) Provided that the condition $V_{Ed} \leq V_{Rd,c}$ is not met, the web shearing reinforcement in accordance with Annex 19 of the Structural Code must be calculated on the basis of a variable tilt lattice model or a model of cranks and straps as more appropriate for the particular geometry of the wall.

(3) If a model of cranks and straps is used, the width of the crank must take into account the presence of openings, and must not be greater than the lower value between 0.25 $l_w$ and 4 $b_{wo}$.

(4) The USL of shear strength due to sliding in horizontal construction joints must be checked in accordance with section **6.2.5** of Annex 19 of the Structural Code, with an anchor length of the reinforcements that cross the interface acting as edge beams, increased by 50 % compared to that required by Annex 19 of the Structural Code.

#### 5.4.3.5.3 Construction details relating to local ductility

(1) The vertical reinforcements necessary for the testing of the USL bending with axial stress, or to satisfy any minimum reinforcement requirement, must be linked by a fence or a transverse tying with diameter not less than 6 mm or one third of the diameter of the vertical round, $d_{bL}$. The cross-sectional fences or states must have a vertical separation not greater than the lowest value between 100 mm or 8 $d_{bL}$.

(2) The reinforcements necessary for checking the flex USL with axial stress, and coerced laterally by fences or transverse states according to point **(1)** above, must be concentrated in contour elements at the ends of the cross section. These elements must extend along the length $l_w$ of the wall, a distance not less than the greater value between $b_w$ and 3 $b_w$ $\sigma_{cm}$ / $f_{cd}$, where $\sigma_{cm}$ is the mean value of the concrete tension in the compression area, in the USL bending with axial force. The diameter of the vertical rounds must not be less than 12 mm on the lower floor of the building, or on any floor where the length $l_w$ of the wall is reduced, compared to that of the floor below, by more than one third of the floor height $h_s$. On all other floors, the diameter of the vertical rounds must not be less than 10 mm.

(3) In order to avoid changing behavioural mode from one controlled by bending to one controlled by shearing, the amount of the vertical reinforcement located in the section of the wall must not unnecessarily exceed the amount required for the verification of the bending USL with axial stress or that required for the integrity of the concrete.

(4) Continuous horizontal or vertical steel tying systems must be provided: (a) along all intersections of walls or connections with wings or stiffeners; (b) at all levels of concrete slab, and (c) around openings in walls. At a minimum, these binding systems must comply with section **9.10** of Annex 19 to the Structural Code.

## 5.5 Dimensioning for high ductility class (DCH)

### 5.5.1 Geometric and material restrictions

#### 5.5.1.1 Materials

(1) In primary seismic elements, the characteristic strength concrete $f_{ck}$ less than 25 N/mm$^2$ must not be used, in accordance with Article 33.4 of the Structural Code.

(2) The requirement specified in **(2)** of section **5.4.1.1** applies to this section.

(3) In critical areas of primary seismic elements, steel must be used for SD-type passive reinforcement as defined in Articles 34 and 35 of the Structural Code. In addition, the higher characteristic value (95 % percentile) of the actual resistance corresponding to the elastic limit, $f_{yk,0.95}$, should not exceed the nominal value by more than 25 %.

#### 5.5.1.2 Geometric restrictions

##### 5.5.1.2.1 Beams

(1) The width of primary earthquake-resistant beams should not be less than 200 mm.

(2) The ratio between width and height of the web of the primary earthquake-resistant beams must satisfy the equation (5.40b) of Annex 19 to the Structural Code.

(3) Point **(1)** of section **5.4.1.2.1** applies.

(4) Point **(2)** of section **5.4.1.2.1** applies.

(5) Point **(3)** of section **5.4.1.2.1** applies.

##### 5.5.1.2.2 Pillars

(1) The minimum dimension of the section of primary earthquake-resistant pillars should not be less than 250 mm.

(2) Point **(1)** of section **5.4.1.2.2** applies.

##### 5.5.1.2.3 Ductile walls

(1) The provisions apply to simple walls, as well as to the individual components of primary earthquake-resistant walls coupled, under effects of action on their plane, with total recessing and anchoring in their bases, on suitable basements or foundations, so that the wall cannot rotate at its base. In this respect, walls supported by slabs or beams are not allowed (see also **5.4.1.2.5**).

(2) Point **(1)** of section **5.4.1.2.3** applies.

(3) The additional requirements relating to the thickness of confined contour elements of primary sister-resistant walls, as specified in points **(8)** and **(9)** of section **5.5.3.4.5**.

(4) In primary earthquake-resistant walls, unorganised openings must be avoided, arranged irregularly to form coupled walls unless their influence is either taken into account in the calculation, dimensioning and construction details, or is negligible.

#### 5.5.1.2.4 Specific rules for beams that support discontinuous vertical elements

(1) Point **(1)** of section **5.4.1.2.5** applies.

(2) Point **(2)** of section **5.4.1.2.5** applies.

### 5.5.2 Effects of the calculation action

#### 5.5.2.1 Beams

(1) Point **(1)** of section **5.4.2.1** applies for the calculation values of bending moments and axillary forces.

(2) Point **(1)** of section **5.4.2.2** applies.

(3) The point **(2)** of section **5.4.2.2**, with the value $\gamma_{Rd}$ = 1.2 in equation (5.8) is applied.

#### 5.5.2.2 Pillars

(1) The point **(1)** of section **5.4.2.1** (which also refers to the capacity dimensioning requirements of point **(2)** of section **5.2.3.3**) applies for the calculation values of bending moments and axial forces.

(2) Point **(1)** of section **5.4.2.3** applies.

(3) Point **(2)** of section **5.4.2.3** applies, with the value $\gamma_{Rd}$ = 1.3 in the equation (5.9).

(4) Point **(3)** of section **5.4.2.3** applies.

#### 5.5.2.3 Joints between beam and pillar

(1) The horizontal shear strength that acts around the core of a junction between beams and primary earthquake-resistant pillars should be determined taking into account the lousy conditions under seismic load, i.e. the capacity dimensioning conditions for the concurrent beams in the knot and the lower values of the shear strength compatible in the rest of the concurrent elements.

(2) Simplified equations can be used to determine the actuating shearing stress on the concrete core of the junctions as follows:

a) for inner beam-pillar knots:

$$V_{jhd} = \gamma_{Rd}(A_{s1} + A_{s2})f_{yd} - V_C \qquad (5.22)$$

b) for outer beam-pillar knots:

$$V_{jhd} = \gamma_{Rd} \cdot A_{s1} \cdot f_{yd} - V_C \qquad (5.23)$$

Where

$A_{s1}$ is the area of the upper reinforcement of the beam;

$A_{s2}$ is the area of the lower reinforcement of the beam;

$V_C$ is the shear strength on the pillar above the knot, obtained from the calculation in the seismic calculation situation;

$\gamma_{Rd}$ is a coefficient that represents the possible reserve of resistance due to hardening by deformation of the steel, which should not be less than 1.2.

(3) The actuating sharp forces in the junctions should correspond to the terrible direction of the seismic action, which influences when choosing the values $A_{s1}$, $A_{s2}$ and $V_C$ which are used in equations (5.22) and (5.23).

### 5.5.2.4 Ductile walls

#### 5.5.2.4.1 Special provisions for slender walls in their plan

(1) Point **(1)** of section **5.4.2.4** applies.

(2) Point **(2)** of section **5.4.2.4** applies.

(3) Point **(3)** of section **5.4.2.4** applies.

(4) Point **(4)** of section **5.4.2.4** applies.

(5) Point **(5)** of section **5.4.2.4** applies.

(6) Point **(6)** of section **5.4.2.4** applies.

(7) The requirements of point **(6)** are considered fulfilled if the following simplified procedure is applied, incorporating the capacity dimensioning criterion:

The calculation values of the shear strength, $V_{Ed}$, must be obtained from the equation:

$$V_{Ed} = \varepsilon \cdot V'_{Ed} \qquad (5.24)$$

Where

$V'_{Ed}$ is the shear strength obtained from the calculation;

$\varepsilon$ is the increase coefficient, calculated from equation (5.25) and being not less than 1.5:

$$\varepsilon = q \cdot \sqrt{\left(\frac{\gamma_{Rd}}{q} \cdot \frac{M_{Rd}}{M_{Ed}}\right)^2 + 0,1\left(\frac{S_e(T_C)}{S_e(T_1)}\right)^2} \leq q \qquad (5.25)$$

Where

$q$ is the behaviour coefficient used in the calculation;

$M_{Ed}$ is the calculation value of the bending moment at the base of the wall;

$M_{Rd}$ is the calculation value of the bending resistance at the base of the wall;

$\gamma_{Rd}$ is a coefficient representing the possible reserve of resistance due to hardening by deformation of the steel; if no more accurate data is available, you can take $\gamma_{Rd}$ equal to 1.2;

$T_1$ is the fundamental period of vibration of the building in the direction of shear strength $V_{Ed}$;

$T_C$ is the upper limit of the constant spectral acceleration stretch (see **3.2.2**);

$S_e(T)$ is the order of the elastic response spectrum (see **3.2.2**).

(8) The provisions of point **(8)** of section **5.4.2.4** apply to slender walls of DCH ductility class.

**5.5.2.4.2 Special provisions for low walls**

(1) For those primary earthquake-resistant walls with a height-to-length ratio $h_w/l_w$ not greater than 2.0, there is no need to modify the bending moments resulting from the calculation. Shearing increases due to dynamic effects may also be disregarded.

(2) The shear strength $V'_{Ed}$ obtained from the analysis must be increased according to the following equation:

$$V_{Ed} = \gamma_{Rd} \cdot \left( \frac{M_{Rd}}{M_{Ed}} \right) \cdot V'_{Ed} \leq q \cdot V'_{Ed} \tag{5.26}$$

(see section **(7)** of section**5.2.4.1** for definitions and values of variables.)

**5.5.3 USL check and construction details**

**5.5.3.1 Beams**

**5.5.3.1.1 Bending resistance**

(1) The bending resistance must be calculated in accordance with Annex 19 to the Structural Code.

(2) Point **(2)** of section **5.4.3.1.1** applies.

(3) Point **(3)** of section **5.4.3.1.1** applies.

**5.5.3.1.2 Shear strength**

(1) Calculations and checks of shear strength must be performed in accordance with Annex 19 to the Structural Code, unless otherwise specified in the following points.

(2) In critical areas of primary earthquake-resistant pillars, the $\theta$ of cranks in a lattice model should be 45°.

(3) As regards the arrangement of the shearing frame within the critical areas of the ends of a primary earthquake-resistant beam where the beam is integrated into the pillar, the following cases should be differentiated, depending on the algebraic value of the ratio $\zeta = V_{Ed,min.}/V_{Ed,max.}$ between the

minimum and maximum values of the active shear strength, according to point **(3)** of section **5.5.2.1**.

a) If $\zeta \geq -0.5$, the shearing strength provided by the reinforcement must be calculated in accordance with Annex 19 to the Structural Code.

b) If $\zeta < -0.5$, i.e. when an almost complete investment of shear strength is expected, then:

i) if $|V_E|_{máx.} \leq (2+\zeta) \cdot f_{ctd} \cdot b_w \cdot d$ (5.27)

where $f_{ctd}$ is the calculation value of the tensile strength of concrete, the rule a) of this point applies.

ii. if $|V_E|_{máx.}$ exceeds the limit value of equation (5.27), two-way tilted reinforcement must be arranged either at ± 45° relative to the axis of the beam, or along the two diagonals in height of the beam, and half of $|V_E|_{máx.}$ must be resisted by fences and the other half by inclined reinforcement;

- in this case, the check is done by the condition:

$$0,5\, V_{Emáx.} \leq 2A_s \cdot f_{yd} \cdot \mathrm{sen}\alpha \qquad (5.28)$$

Where

$A_s$ is the area of the reinforcement inclined in a direction that crosses the potential slip plane (i.e. the extreme cross section of the beam);

$\alpha$ is the angle between the tilted reinforcement and the axis of the beam (normally $\alpha$ = 45°, or as $\alpha \approx (d\text{-}d')/l_b$).

#### 5.5.3.1.3 Construction details relating to local ductility

(1) Critical areas should be considered those parts of a primary earthquake-resistant beam located within a distance $l_{cr}$ = 1,5 $h_w$, measured from an extreme cross-section where the beam connects to a beam-pillar knot, as well as those within the same distance $l_{cr}$ = 1.5 $h_w$, considered on both sides of any other cross-section susceptible to lamination in the seismic calculation situation.

(2) Point **(2)** of section **5.4.3.1.2** applies.

(3) Point **(3)** of section **5.4.3.1.2** applies.

(4) Point **(4)** of section **5.4.3.1.2** applies.

(5) In order to meet the necessary ductility conditions, the following conditions must be met throughout the length of the primary earthquake-resistant beam:

a) the point **(5)** of **5.4.3.1.2** must be complied with;

b) at least two high-adhesion rounds with $d_b$ = 14 mm must be arranged on both the upper and lower sides of the beam, which must be extended along the entire length of the beam;

c) a quarter of the maximum section of the upper reinforcement on the supports should be extended along the entire length of the beam.

(6) Point **(6)** of section **5.4.3.1.2** is applied, replacing equation (5.13) with the following:

$$s = \text{mín.}\left\{ h_w / 4;\ 24d_{bw};\ 175;\ 6d_{bL} \right\} \tag{5.29}$$

### 5.5.3.2 Pillars

#### 5.5.3.2.1 Resistances

(1) Point **(1)** of section **5.4.3.2.1** applies.

(2) Point **(2)** of section **5.4.3.2.1** applies.

(3) In primary earthquake-resistant pillars, the reduced axial stress value $\nu_d$ should not exceed 0.55.

#### 5.5.3.2.2 Construction details relating to local ductility

(1) Point **(1)** of section **5.4.3.2.2** applies.

(2) Point **(2)** of section **5.4.3.2.2** applies.

(3) Point **(3)** of section **5.4.3.2.2** applies.

(4) In the absence of more accurate data, the critical area length $l_{cr}$ (in meters) can be calculated as follows:

$$l_{cr} = \text{máx.}\left\{ 1,5h_c;\ l_{cl} / 6;\ 0,6 \right\} \tag{5.30}$$

Where

$h_c$ is the largest cross-sectional dimension of the pillar (in metres); and

$l_{cl}$ is the free length of the pillar (in meters).

(5) Point **(5)** of section **5.4.3.2.2** applies.

(6) Point **(6)** of section **5.4.3.2.2** applies.

(7) The construction details of critical areas above the base of the pillar must be based on a minimum value of the ductility coefficient in curvatures, $\mu_\phi$ (see **5.2.3.4**), obtained from point **(3)** of section **5.2.3.4**. If the pillar is protected against the appearance of plastic hinges by the capacity dimensioning procedure of point **(4)** of section **4.4.2.3** [i.e. It satisfies the equation (4.29)], the value $q_o$ of equations (5.4) and (5.5) can be replaced by 2/3 of the value of $q_o$ which is applied in a direction parallel to the transverse edge of the pillar.

(8) Point **(7)** of section **5.4.3.2.2** applies.

(9) The requirements of points **(6)**, **(7)** and **(8)** of this section are considered complied with if point **(8)** of section **5.4.3.2.2** with the values of $\mu_\phi$ specified in said points **(6)** and **(7)**.

(10) The minimum value of $\omega_{wd}$ to be arranged within the critical area is 0.12 at the base of the pillar, or 0.08 in all critical areas of the pillar above the base.

(11) Point **(10)** of section **5.4.3.2.2** applies.

(12) The minimum conditions indicated in point **(11)** are considered to be complied with if the following requirements are met:

a) the diameter $d_{bw}$ of the fences is at least equal to:

$$d_{bw} \geq 0,4 \cdot d_{bL,máx.} \cdot \sqrt{f_{ydL} / f_{ydw}} \quad (5.31)$$

b) the separation $s$ of the fences (in millimetres) is not greater than:

$$s = \text{mín.} \left| b_0 / 3;\ 125;\ 6 d_{bL} \right| \quad (5.32)$$

Where

$b_o$ (in millimetres) is the minimum dimension of the concrete core (inside the fences): and

$d_{bL}$ is the minimum diameter of longitudinal rounds (in millimetres).

c) the distance between consecutive longitudinal rounds bound by fences or transverse states does not exceed 150 mm.

(13) On the two lower floors of the buildings, fences must also be arranged, in accordance with points **(11)** and **(12)** of this section, beyond the critical areas along an additional length equal to half the length of these areas.

(14) The amount of longitudinal reinforcement at the base of the pillar of the lower floor of the building (i.e. where the pillar connects with the foundation) must not be less than that arranged at the top of the building.

#### 5.5.3.3 Joints between beam and pillar

(1) The diagonal compression induced in the knot by the crank mechanism must not exceed the compressive strength of the concrete in the presence of unit deformations by transverse tensile.

(2) In the absence of a more precise model, the requirement indicated in point **(1)** above can be complied with the help of the following rules:

a) in inner beam-pillar knots, the equation must be complied with:

$$V_{jhd} \leq \eta f_{cd} \sqrt{1 - \frac{\nu_d}{\eta}} b_j h_{jc} \tag{5.33}$$

Where

$\eta$ = 0.61(1-$f_{ck}$/250)

$h_{jc}$ is the distance between extreme layers of the pillar reinforcement;

$b_j$ is defined in equation (5.34);

$\nu_d$ is the reduced axial stress on the pillar above the knot;

$f_{ck}$ is expressed in MPa.

b) in outer beam-pillar knots:

$V_{jhd}$ must be less than 80 % of the given value on the right side of equation (5.33), where:

$V_{jhd}$ is defined by equations (5.22) and (5.23), respectively;

and the effective width of knot, $b_j$, is:

a) if $b_c > b_w : b_j = \text{mín.} \{b_c; (b_w + 0{,}5 \cdot h_c)\}$ (5.34a)

b) if $b_c < b_w : b_j = \text{mín.} \{b_w; (b_c + 0{,}5 \cdot h_c)\}$ (5.34b)

(3) Adequate confinement (both horizontal and vertical) of the knot must be provided in order to limit the maximum diagonal tensile tension of the concrete, max. $\sigma_{ct}$, to $f_{ctd}$. In the absence of a more precise model, this condition can be complied with by placing horizontal fences with a diameter of not less than 6 mm inside the knot, so that:

$$\frac{A_{sh} \cdot f_{ywd}}{b_j \cdot h_{jw}} \geq \frac{\left(\frac{V_{jhd}}{b_j \cdot h_{jc}}\right)^2}{f_{ctd} + \nu_d f_{cd}} - f_{ctd} \tag{5.35}$$

Where

$A_{sh}$ is the total area of horizontal fences;

$V_{jhd}$ is defined in equations (5.22) and (5.23);

$h_{jw}$ is the distance between the upper and lower reinforcements of the beam;

$h_{jc}$ is the distance between the extreme layers of the pillar reinforcement;

$b_j$ is defined in equation (5.34);

$\nu_d$ is the calculation value of reduced axial stress of the pillar, above the junction ( $\nu_d=N_{Ed}/A_c \cdot f_{cd}$);

$f_{ctd}$ is the calculation value of the tensile strength of concrete, according to Annex 19 of the Structural Code.

(4) As an alternative to the rule indicated in point **(3)** of this section, the integrity of the knot after diagonal cracking can be secured by horizontal fence reinforcement. For this purpose, the following total horizontal fence area must be arranged in the knot:

a) in inner knots:

$$A_{sh} f_{ywd} \geq \gamma_{Rd} \left( A_{s1} + A_{s2} \right) f_{yd} \left( 1 - 0,8 \nu_d \right) \quad (5.36a)$$

b) in outer knots:

$$A_{sh} f_{ywd} \geq \gamma_{Rd} A_{s2} f_{yd} \left( 1 - 0,8 \nu_d \right) \quad (5.36b)$$

where $\gamma_{Rd}$ is equal to 1,2 (see point **(2)** of section **5.5.2.3**) and the reduced axial force calculation value, $\nu_d$, refers to the part of the pillar above the knot, in equation (5.36a), or to the part of the pillar under the knot, in equation (5.36b).

(5) Horizontal fences calculated according to the points **(3)** and **(4)** of this section must be distributed evenly within the height $h_{jw}$ between the upper and lower reinforcements of the beam. In outer knots, the fences must enclose the ends of the beam's reinforcements bent towards the knot.

(6) Appropriate vertical reinforcement must be fitted to the pillar passing through the knot, so that:

$$A_{sv,i} \geq (2/3) \cdot A_{sh} \left( h_{jc} / h_{jw} \right) \quad (5.37)$$

where $A_{sh}$ is the total area of horizontal fences, required according to the points **(3)** and **(4)** of this section, and $A_{sv,I}$ is the total area of the intermediate rounds placed on the relevant faces of the pillar, between the rounds of the pillar corner (including the rounds that are part of the longitudinal reinforcement of the pillars).

(7) Point **(1)** of section **5.4.3.3** applies.

(8) Point **(2)** of section **5.4.3.3** applies.

(9) Point **(3)** of section **5.4.3.3** applies.

### 5.5.3.4 Ductile walls

#### 5.5.3.4.1 Bending resistance

(1) Bending resistance should be evaluated and tested in the same way as in pillars, under the most unfavourable axial stress for the seismic calculation situation.

(2) In primary seismic walls, the reduced axial stress value, $\nu_d$, must not exceed 0.35.

#### 5.5.3.4.2 Diagonal compression rupture of the web due to shear strength

(1) The value of $V_{Rd,max.}$ can be calculated as follows:

a) outside the critical area:

according to Annex 19 of the Structural Code, with an internal mechanical arm length, $z$, equal to 0,8 $l_w$ and an inclination of the compression crank with respect to the vertical, given by such $\theta$, equal to 1.0.

b) within the critical area:

40 % of the value outside the critical area.

#### 5.5.3.4.3 Rupture by diagonal traction of the web due to shear strength

(1) Web reinforcement calculation for shearing USL check should take into account the value of shearing ratio $\alpha_s = M_{Ed}/(V_{Ed}\ l_w)$. The maximum value of $\alpha_s$ must be used on a given building floor for checking the shearing USL of that floor.

(2) If $\alpha_s \geq 2.0$, the provisions set out in **(1)** to **(7)** of section **6.2.3** of Annex 19 to the Structural Code apply, with the values of z and of such $\theta$ taken as in point **(1)**(a) of section **5.5.3.4.2**.

(3) If $\alpha_s < 2.0$, the following provisions apply:

a) horizontal web reinforcements must satisfy the following condition (see section **6.2.3** of Annex 19 to the Structural Code):

$$V_{Ed} \leq V_{Rd,c} + 0,75 \rho_h f_{yd,h} b_{wo} \alpha_s l_w \tag{5.38}$$

Where

$\rho_h$ is the horizontal reinforcement amount [$\rho_h = A_h / (b_{wo} \cdot s_h)$];

$f_{yd,h}$ is the calculation value of the elastic limit of the horizontal web reinforcement;

$V_{Rd,c}$ is the calculation value of the shear strength of the elements without shearing reinforcement.

In the critical wall area, $V_{Rd,c}$ must be 0 if the axis $N_{Ed}$ is traction.

b) vertical reinforcement must be arranged in the web, anchored and spliced along the entire height of the wall in accordance with Annex 19 to the Structural Code, so that the condition is met:

$$\rho_h f_{yd,h} b_{wo} z \leq \rho_v f_{yd,v} b_{wo} z + \min. N_{Ed} \tag{5.39}$$

Where

$\rho_v$ is the amount of the vertical reinforcement [$\rho_v = A_v / (b_{wo} \cdot s_h)$];

$f_{yd,v}$ is the calculation value of the elastic limit of the vertical web reinforcement;

and where the axial stress $N_{Ed}$ is taken as positive when it is compression.

(4) Horizontal web reinforcements must be fully anchored at the ends of the section of the wall, within the confined core of the contour elements, e.g. by 90° or 135° hooks, or by straight extension if practicable.

(5) Alternatively, it may be assumed that horizontal web reinforcement contributes to the confinement of the boundary elements of the wall, provided that it is formed by elongated fences, properly closed using hooks bent at 135° and of length 10 $d_{bw}$, as provided for in section 5.6.1. (2). In any case, the provisions of section 5.4.3.4.2 must be complied with.

#### 5.5.3.4.4 Breakage by shear strength due to displacement

(1) In the plans of shear strength by potential displacement of critical areas (e.g. in construction joints) the following condition must be met:

$$V_{Ed} \leq V_{Rd,S}$$

where $V_{Rd,S}$ is the calculation value of the shearing stress resistance due to displacement.

(2) The value of $V_{Rd,S}$ can be taken as follows:

$$V_{Rd,S} = V_{dd} + V_{id} + V_{fd} \tag{5.40}$$

with

$$V_{dd} = \text{mín.} \begin{cases} 1,3 \cdot \Sigma A_{sj} \cdot \sqrt{f_{cd} \cdot f_{yd}} \\ 0,25 \cdot f_{yd} \cdot \Sigma A_{sj} \end{cases} \tag{5.41}$$

$$V_{id} = \Sigma A_{si} \cdot f_{yd} \cdot \cos\varphi \tag{5.42}$$

$$V_{fd} = \text{mín.} \begin{cases} \mu_f \cdot \left[ \left( \Sigma A_{sj} \cdot f_{yd} + N_{Sd} \right) \cdot \xi + M_{Ed} / z \right] \\ 0,5\eta \cdot f_{cd} \cdot \xi \cdot l_w \cdot b_{wo} \end{cases} \tag{5.43}$$

Where

$V_{dd}$ is the strength of the pin of the vertical reinforcements;

$V_{id}$ is the shear strength of inclined reinforcements (at an angle $\varphi$ to the potential slip plane, e.g. construction joints);

$V_{fd}$ is the resistance due to friction;

$\mu_f$ is the coefficient of concrete-concrete friction under cyclic actions, which must be equal to 0.6 for smooth surfaces and 0.7 for rough surfaces as defined in section **6.2.5** of Annex 19 to the Structural Code;

z. is the length of the internal mechanical arm;

$\xi$ is the relative depth of the neutral fibre;

$\Sigma A_{sj}$ is the sum of the areas of the vertical web reinforcements and the additional reinforcements placed specifically on the contour elements to provide resistance against sliding;

$\Sigma A_{si}$ is the sum of the areas of all inclined reinforcements in the two directions, for this purpose rounds of large diameter are recommended;

$$\eta = 0{,}6\ \left(1\ - f_{ck}\,(\mathrm{MPa})\,/\,250\right) \tag{5.44}$$

$N_{Ed}$ is assumed positive when it is compression.

(3) In the case of low walls, the following must be complied with:

a) at the base of the wall $V_{id}$ should be greater than $V_{Ed}/2$;

b) in higher levels $V_{id}$ should be greater than $V_{Ed}/4$.

(4) The inclined reinforcements must be fully anchored to both sides of the potential sliding surfaces, and must cross all sections of the wall at a distance of $0.5 \cdot l_w$ or $0.5 \cdot h_w$, whichever is lower, above the base critical section.

(5) Inclined reinforcements produce an increase in bending resistance at the base of the wall, to be taken into account when calculating the shearing acting $V_{Ed}$ according to the capacity dimensioning criterion (see points **(6)** and **(7)** of section **5.5.2.4.1**, and point **(2)** of section **5.5.2.4.2**). Two alternative methods can be used:

a) the increase of bending resistance $\Delta M_{Rd}$ to be used in calculating $V_{Ed}$ can be estimated as:

$$\Delta M_{Rd} = \frac{1}{2} \cdot \Sigma A_{si} \cdot f_{yd} \cdot \mathrm{sen}\ \varphi \cdot l_i \tag{5.45}$$

Where

$l_i$ is the distance between the axles of the two groups of oblique reinforcements, situated at an angle of $\pm\ \phi$ relative to the potential slip plane, measured in the base section;

and the rest of the symbols are the same as those in the equation (5.42).

b) an actuating shearing $V_{Ed}$ can be calculated without taking into account the effect of inclined reinforcements. In equation (5.42), $V_{id}$ is taken as the net shear strength of the inclined reinforcements (i.e. the actual shear strength, reduced by the increase of the actuating shearing). This net shear strength of the tilted reinforcements with respect to the slip can be estimated as:

$$V_{id} = \Sigma A_{si} \cdot f_{yd} \cdot \left[ \cos\varphi - 0{,}5 \cdot l_i \cdot \mathrm{sen}\,\varphi / \left( \alpha_s \cdot l_w \right) \right] \quad (5.46)$$

#### 5.5.3.4.5 Construction details relating to local ductility

(1) Point **(1)** of section **5.4.3.4.2** applies.

(2) Point **(2)** of section **5.4.3.4.2** applies.

(3) Point **(3)** of section **5.4.3.4.2** applies.

(4) Point **(4)** of section **5.4.3.4.2** applies.

(5) Point **(5)** of section **5.4.3.4.2** applies.

(6) Point **(6)** of section **5.4.3.4.2** applies.

(7) Point **(8)** of section **5.4.3.4.2** applies.

(8) Point **(10)** of section **5.4.3.4.2** applies.

(9) If the wall is connected to a wing with thickness $b_f \geq h_s/15$ and width $l_f \geq h_s/5$ (where $h_s$ designates the floor's free height), and the confined contour element needs to extend beyond the wing into the web, an additional length of up to $3b_{wo}$, then the thickness $b_w$ of the web contour element must only meet the provisions indicated in point **(1)** of section **5.4.1.2.3** for $b_{wo}$ (Figure 5.11).

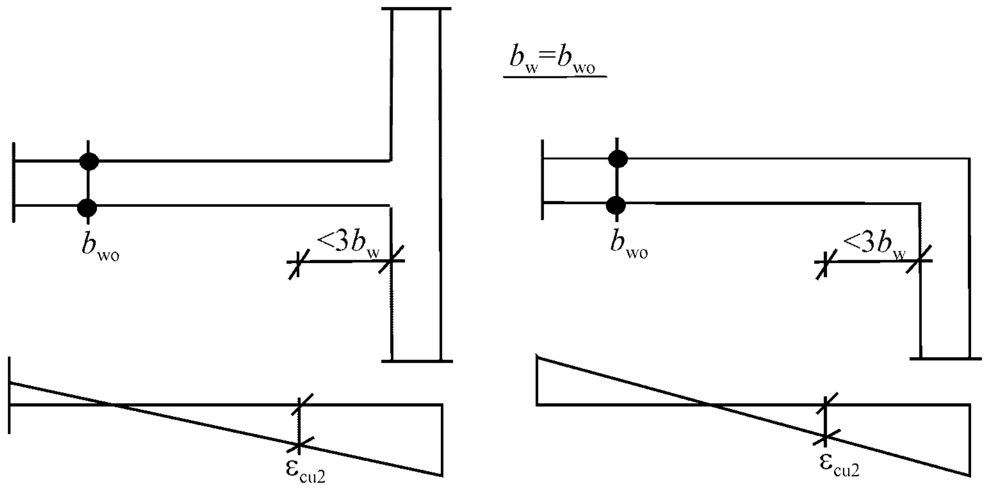



**Figure 5.11 – Minimum thickness of contour elements confined to DCH ductility class walls with large wings**

(10) Within the wall contour elements, the requirements specified in point **(12)** of section **5.5.3.2.2,** and $\omega_{wd}$ must have a minimum value of 0.12. Superimposed fences must be overlapped, so that any other longitudinal round is linked by a fence or an tying system.

(11) Above critical areas, contour elements must be arranged along a height equal to one more floor, with at least half of the confinement reinforcement required for the critical area.

(12) Point **(1)** of section **5.4.3.4.2** applies.

(13) Premature cracking by shearing the web of the walls should be avoided by providing a minimum amount of web reinforcement: $\rho_{h,min.} = \rho_{v,min.} = 0.002$.

(14) The web reinforcement must be arranged as two round orthogonal meshes with the same adhesion characteristics, one on each side of the wall. The meshes must be connected by transverse states, approximately 500 mm apart.

(15) The web reinforcement must have a diameter not less than 8 mm, but not greater than one eighth of the web width, $b_{wo}$. The separation must not be greater than the lowest value between 250 mm or 25 times the diameter of the rounds.

(16) A minimum amount of vertical reinforcement fully anchored through the cold joints must be arranged in order to counteract the unfavourable effects in the event of cracking in such joints, and associated uncertainties. This minimum amount of reinforcement, $\rho_{min.}$, necessary to restore the shearing strength of non-fissured concrete, is:

$$\rho_{min.} \geq \begin{cases} \left(1,3 \cdot f_{ctd} - \dfrac{N_{Ed}}{A_w}\right) / \left(f_{yd} \cdot \left(1 + 1,5\sqrt{f_{cd} / f_{yd}}\right)\right) \\ 0,0025 \end{cases} \tag{5.47}$$

where $A_w$ is the total horizontal cross section area of the wall, and $N_{Ed}$ should be positive in case of compression.

#### 5.5.3.5 Coupling elements for coupled walls

(1) The coupling of walls by means of slabs should not be taken into account insofar as it is not effective.

(2) The provisions of section **5.5.3.1** may only apply to coupling beams if any of the following conditions are met:

a) cracking in both diagonal directions is unlikely. An acceptable application rule is:

$$V_{Ed} \leq f_{ctd} b_w d \tag{5.48}$$

b) the prevalence of the manner bending-break is ensured. An acceptable rule of application is: $l/h \geq 3$.

(3) If none of the conditions of point **(2)** are met, the resistance to seismic actions must be ensured by having a reinforcement along the two diagonals of the beam, in accordance with the following conditions (see Figure 5.12):

a) compliance with the following equation must be ensured:

$$V_{Ed} \leq 2 \cdot A_{si} \cdot f_{yd} \cdot \text{sen}\,\alpha \tag{5.49}$$

Where

$V_{Ed}$ is the calculation value of the shear strength in the coupling element ($V_{Ed} = 2 \cdot M_{Ed} / l$);

$A_{si}$ is the total area of the reinforcements in each diagonal direction;

$\alpha$ is the angle between the diagonal reinforcements and the axis of the beam.

b) the diagonal reinforcement must be arranged in pillar-type elements, with lateral dimensions equal to at least 0.5 $b_w$; its anchorage length must be 50 % longer than that required by Annex 19 to the Structural Code.

c) fences around these pillar-type elements must be arranged to prevent the buckling of longitudinal reinforcements. For fences the provisions of **(12)** of section **5.5.3.2.2** apply.

d) longitudinal and transverse reinforcements must be arranged on both sides of the beam, complying with the minimum requirements specified in Annex 19 of the Structural Code for large edge beams. The longitudinal reinforcement must not be anchored in coupled walls and can only be extended within them a distance of 150 mm.

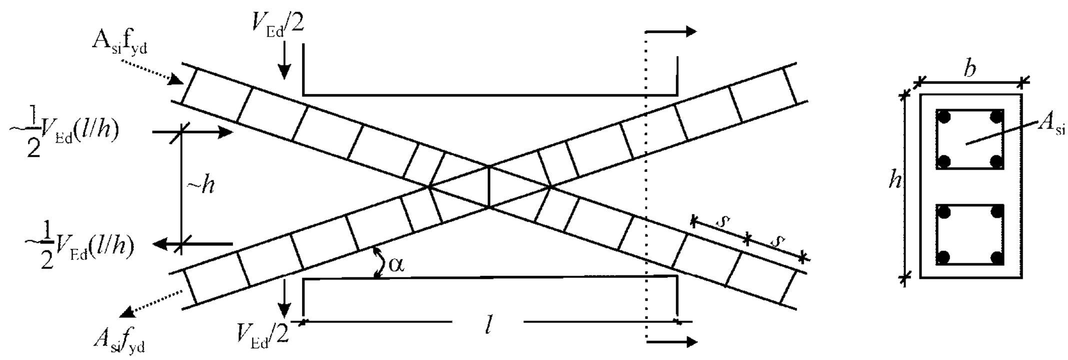



**Figure 5.12 – Coupling beams with diagonal reinforcement**

## 5.6 Provisions for anchorages and fittings

### 5.6.1 General considerations

(1) For the construction details of the reinforcements, the section **8** of Annex 19 of the Structural Code applies, together with the additional rules indicated in the following sections. Alternatively, reinforcement anchorage and splice lengths can be obtained in accordance with Article 49.5 of the Structural Code (in this case, point **(3)** of section **5.6.1** and point **(2)** of section **5.6.2.1** can be obtained).

(2) In cases where fences are used as transverse reinforcement on beams, pillars or walls, closed

brackets with hooks bent at 135° and length 10 $d_{bw}$ should be used.

(3) In high ductility class structures, DCH, the anchoring length of beam or pillar reinforcements anchored in the junctions between beams and pillars should be measured from a point on the round at a distance 5 $d_{bL}$ within the knot from the face of the junction, to take into account the extent of the plasticised area due to postelastic cyclic deformations (for example of a beam, see Figure 5.13a).

### 5.6.2 Anchoring of the reinforcements

#### 5.6.2.1 Pillars

(1) When calculating the anchor length or the flap of the pillar reinforcements contributing to the bending resistance of critical area elements, the ratio between the required reinforcement area and the actually arranged, $A_{s,req}/A_{s,prov}$, should be assumed equal to 1.

(2) If, in the seismic calculation situation, the axial stress on a pillar is traction, the anchor lengths must be increased by 50 % compared to the values specified in Annex 19 to the Structural Code.

#### 5.6.2.2 Beams

(1) The part of the longitudinal reinforcement of the beams that bends in the knots for anchoring, must always be placed within the corresponding fences existing on the pillars.

(2) To prevent adhesion breakage, the diameter of the longitudinal trusses of beams passing through beam-pillar junctions, $d_{bL}$, should be limited by following the following equations:

a) for inner beam-pillar knots:

$$\frac{d_{bL}}{h_c} \leq \frac{7,5 \cdot f_{ctm}}{\gamma_{Rd} \cdot f_{yd}} \cdot \frac{1 + 0,8 \cdot \nu_d}{1 + 0.75 k_D \cdot \rho' / \rho_{máx.}} \tag{5.50a}$$

b) for outer beam-pillar knots:

$$\frac{d_{bL}}{h_c} \leq \frac{7,5 \cdot f_{ctm}}{\gamma_{Rd} \cdot f_{yd}} \cdot \left(1 + 0,8 \cdot \nu_d\right) \tag{5.50b}$$

Where

$h_c$ is the width of the pillar parallel to the reinforcements;

$f_{ctm}$ is the average value of the tensile strength of concrete;

$f_{yd}$ is the calculation value of the elastic limit of steel;

$\nu_d$ is the calculation value of reduced axial stress in the pillar, adopting its minimum value for the seismic calculation situation ($\nu_d = N_{Ed}/f_{cd} \cdot A_c$);

$k_D$ is the coefficient reflecting the ductility class, equal to 1 for the ductility class DCH and 2/3 for the ductility class DCM;

$\rho'$ is the amount of compression reinforcement of the beam passing through the junction;

$\rho_{max.}$ is the maximum permissible amount for the traction reinforcement (see point **(4)** of section **5.4.3.1.2** and point **(4)** of section **5.5.3.1.3**);

$\gamma_{Rd}$ is the uncertainty coefficient of the model for the strength calculation values, taken as 1,2 or 1.0, respectively, for DCH and DCM ductility classes (due to the reserve of resistance attributable to steel deformation hardening of the longitudinal beam reinforcements).

The equations (5.50a) and (5.50b) above do not apply to diagonal reinforcements that cross the knots.

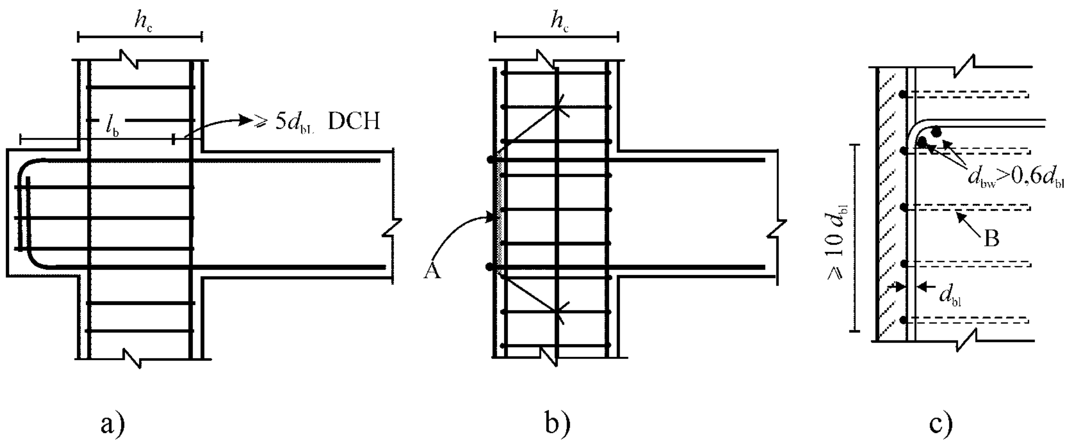
(3) If the requirement of point **(2)** as regards the outer beam-pillar knots cannot be met, because the edge, $h_c$, of the pillar in parallel to the reinforcement is too small, the following additional measures can be taken, to ensure the anchoring of the longitudinal reinforcement of the beams:

a) the beam or slab may be extended horizontally in the form of external projections (see Figure 5.13a);

b) cross-arm or anchor plates welded at the ends of the reinforcement may be used (see Figure 5.13b);

c) pins with a minimum length of 10 $d_{bL}$ and cross-arms grouped next to the elbow of the pins can be added (see Figure 5.13c).

(4) Upper or lower reinforcements passing through inner knots must end in the elements converging in the knot, at a distance not less than $l_{cr}$ (length of the critical area of the element, see point **(1)** of section **5.4.3.1.2** and point **(1)** of section **5.5.3.1.3**) of the face of the knot.




Legend

A Anchor plate

B Fences around the reinforcements of the pillar

**Figure 5.13 – Additional measures for anchorages in outer beam-pillar knots**

### 5.6.3 Frame splicing

(1) Welded-flap splicing should not occur within the critical areas of the structural elements.

(2) Mechanical connectors may be spliced on pillars and walls if these devices have been properly tested under conditions compatible with the selected ductility class.

(3) The transverse reinforcement to be arranged within the flap length must be dimensioned in accordance with Annex 19 of the Structural Code. In addition, the following requirements must be met:

a) if the reinforcements, anchored and continuous, are arranged in a plane parallel to the transverse reinforcement, the sum of the areas of the overlapping reinforcements, $\Sigma A_{sL}$, should be used in the calculation of the transverse reinforcement;

b) if the reinforcements, anchored and continuous, are arranged in a normal plane to the transverse reinforcement, the area of the transverse reinforcement should be calculated based on the area of the largest round of the overlapping longitudinal reinforcements, $A_{sL}$;

c) the separation $s$ of the transverse reinforcement in the overlap area (in millimetres) must not be greater than:

$$s = \text{mín.}\ \{h/4;\ 100\} \tag{5.51}$$

where $h$ is the smaller dimension of the section.

(4) The required area of transverse reinforcement, $A_{st}$, within the overlap area of the longitudinal reinforcement of the spliced pillars in the same area (as defined in Annex 19 to the Structural Code), or the longitudinal reinforcement of the contour elements in walls, can be calculated through the following formula:

$$A_{st} = s\left(d_{bl} / 50\right)\left(f_{yld} / f_{ywd}\right) \tag{5.52}$$

Where

$A_{st}$ is the area of a branch of the transverse reinforcement;

$d_{bL}$ is the diameter of the round object of the splicing;

$s$ is the separation of the transverse reinforcement;

$f_{yld}$ is the calculation value of the elastic limit of longitudinal reinforcements;

$f_{ywd}$ is the calculation value of the elastic limit of the transverse reinforcements.

## 5.7 Dimensioning and construction details of secondary seismic elements

(1) Section **5.7** applies to those elements designated as secondary seismic elements, subject to significant deformations in the seismic calculation situation (e.g. nerves in a slab are not subject to these requirements). These elements must be dimensioned and detailed so that they maintain their

ability to withstand the gravitational loads present in the seismic calculation situation, when subjected to the maximum deformations of that situation.

(2) The maximum deformations in the seismic calculation situation must be dimensioned in accordance with section **4.3.4** and should take into account the effects P-Δ (second order), according to the points **(2)** and **(3)** of section **4.4.2.2**. These deformations must be determined by analysing the structure in the seismic calculation situation, in which the contribution of secondary seismic elements to lateral rigidity is disregarded and the primary seismic elements are modelled with their fissured rigidity, bending and shearing.

(3) Secondary seismic elements are assumed to satisfy the requirements of point **(1)** of this section if the ending moments and shear strength, calculated on the basis of: a) the deformations indicated in point **(2)**; and (b) their fissured bending and shearing rigidity do not exceed the calculation values of the bending and shearing resistances, $M_{Rd}$ and $V_{Rd}$, respectively, determined in accordance with Annex 19 to the Structural Code.

## 5.8 Concrete foundation elements

### 5.8.1 Object and field of application

(1) The following points apply to the calculation of concrete foundation elements such as shoes, tie beams, foundation beams, foundation slabs, foundation walls, cleavages and piles, as well as the connections between or between these elements and vertical concrete elements. The calculation of these elements must follow the rules of section **5.4** of Annex 5.

(2) If the effects of the calculation action for the design of dissipative structure foundation elements are obtained based on capacity dimensioning considerations, according to point **(2)** of section **4.4.2.6**, it is not expected to dissipate energy in these elements in the seismic calculation situation. These elements can be calculated by following the rules indicated in point **(1)** of section **5.3.2**.

(3) If the effects of the calculation action for foundation elements in dissipative structures are obtained from the analysis for the seismic calculation situation without taking into account the considerations on capacity dimensioning indicated in point **(2)** of section **4.4.2.6**, the dimensioning of these elements must follow the rules applicable to the elements of the superstructure of the selected ductility class. For tie beams and foundation beams, it is necessary that the shear strength are obtained on the basis of capacity dimensioning considerations, according to section **5.4.2.2** for DCM ductility class buildings, or according to the points **(2)** or **(3)** of section **5.5.2.1** in the case of DCH ductility class buildings.

(4) If the effects of the calculation action for foundation elements have been obtained using a performance coefficient value q less than or equal to the upper limit of $q$ for low dissipative behaviour (1.5 in concrete buildings, or between 1.5 and 2.0 for steel or mixed steel and concrete buildings, in accordance with Note 1 to Table 6.1 or Note 1 to Table 7.1, respectively), the dimensioning of these elements can follow the rules of point **(1)** of section **5.3.2** (see also point **(3)** of section **4.4.2.6**).

(5) In drawer-type infrastructures of dissipative structures, consisting of: a concrete slab acting as a rigid diaphragm at basement-roof level; a foundation slab or a tie-up of binding beams or foundation beams at foundation level, and (c) peripheral or interior foundation walls, calculated in accordance with point **(2)** of this section, it is expected that the pillars and beams (including those on the basement-roof) must remain elastic under the seismic calculation situation, and can be calculated according to point **(1)** of section **5.3.2**. The shearing walls must be dimensioned for the development of plastic hinges at the basement-roof slab level. In this way, the critical area should be considered to extend to a height $h_{cr}$ above the basement-roof slab and to a depth $h_{cr}$ below it (see point **(1)** of section

**5.4.3.4.2** and point **(1)** of section **5.5.3.4.5**). Any reduction of the cross section of the wall below the roof slab of the basement, within the height of the floor or floors affected by the critical area, is expressly prohibited. In any case, it is recommended that the dimensions of the walls be uniform throughout their height, reaching the foundation without major changes in their cross section, in order to ensure an adequate transmission of the loads and that the potential formation of the plastic hinge occurs at the intended location. In addition, the entire free height of such walls within the basement should be sheared assuming that the wall develops its bending resistance reserve $\gamma_{Rd} \cdot M_{Rc,i}$ (with $\gamma_{Rd}$ = 1.1 for DCM ductility class $\gamma_{Rd}$ = 1.2 for DCH ductility class) at basement-roof level and with zero moment at foundation level.

### 5.8.2 Tie beams and foundation beams

(1) Short attachment pillars between the upper face of a footprint or a cleavage and the plane of the tie beams or foundation beams should be avoided. To do this, such a plane must be below the top face of the footprint or the pile cap.

(2) The check must assume that axillary forces on the binding beams or in bonding areas of the foundation slabs, in accordance with points **(6)** and **(7)** of section **5.4.1.2** of Annex 5, they act in conjunction with the effects of the action obtained according to points **(2)** or **(3)** of section **4.4.2.6** for the seismic calculation situation and taking into account second order effects.

(3) The cross-section of binding beams and foundation beams must have a width of at least $b_{w,min.}$ = 0.4 m and a edge of, at least, $h_{w,min}$ = 0.4 m.

(4) The foundation slabs arranged according to section **(2)** of section **5.4.1.2** of Annex 5 for horizontal connection of isolated or stapled shoes must have a thickness of, at least, $t_{min.}$ = 0.2 m and a reinforcement amount of at least $\rho_{s,min.}$= 0.2 % in each face and direction.

(5) The tethering beams and foundation beams must, throughout their entire length, have a longitudinal reinforcement amount $\rho_{b,min}$ of, at least, 0.4%, at the top and bottom.

### 5.8.3 Connections of vertical elements with beams or foundation walls

(1) The common area (knot) of a beam or foundation wall and a vertical element must meet the requirements of points **5.4.3.3** or **5.5.3.3**, as if it were a junction between beam and pillar.

(2) If a beam or foundation wall of a DCH ductility class structure is calculated for action effects obtained on the basis of capacity dimensioning considerations, according to point **(2)** of section **4.4.2.6**, the horizontal shear strength, $V_{jhd}$, in the knot is obtained based on the calculation results according to the points **(2)**, **(4)**, **(5)** and **(6)** of section **4.4.2.6**.

(3) If a beam or foundation wall of a DCH ductility class structure is not calculated according to the capacity dimensioning method of points **(4)**, **(5)** and **(6)** of section **4.4.2.6** (see point **(3)** of section **5.8.1**), the horizontal shear strength, $V_{jhd}$, in the knot area is determined according to equations (5.22) and (5.23) of **(2)** of section **5.5.2.3**, for beam-pillar knots.

(4) In DCM ductility class structures, the connection of beams or foundation walls with vertical elements can follow the rules indicated in section **5.4.3.3**.

(5) Pins or anchor hooks located at the bottom of the longitudinal reinforcement of the vertical elements must be oriented in such a way as to produce compressions in the connection area.

### 5.8.4 Concrete piles and pile caps *in situ*

(1) The upper part of the pilot, at a distance from the lower face of the pile cap of twice the dimension of the cross-section of that pile, as well as the areas located at a distance of up to $2d$ on each side of an interface between two layers of ground with significantly different shearing rigidities (ratio between shearing modules greater than 6), must be constructively detailed as areas of potential formation of plastic hinges. To do this, transverse and confinement reinforcement must be arranged following the rules for critical areas of pillars of the corresponding ductility class or, at least, of DCM ductility class.

(2) When applying the requirement specified in point **(3)** of **5.8.1** to the dimensioning of piles of dissipative structures, such piles must be constructively calculated and detailed for the potential formation of plastic hinges on your head. To do this, the length by which it is required to increase the transverse and confinement reinforcement at the top of the pilot is increased by 50 %, according to point **(1)** above. In addition, a shear strength calculation value equal to at least that obtained based on points **(4)** to **(8)** of section **4.4.2.6**.

(3) The piles necessary to withstand tensile stresses or those assumed to be recessed at the top must be anchored in the cleavage in such a way as to allow the development of the calculation value of the resistance to mismatch of that pile, or the calculation value of the tensile strength of the pile reinforcement, whichever is lower. If the part of these piles embedded in the pile cap is concreted before the branching itself, pins must be arranged at the interface in which the connection occurs.

## 5.9 Local effects due to brickwork or concrete fillers

(1) Due to the particular vulnerability of floor filler walls at ground level, an earthquake-induced irregularity is expected to occur and, therefore, appropriate measures must be taken. If a more precise method is not used, the entire length of the ground floor pillars should be considered as the critical length, and should be confined accordingly.

(2) If the height of the fillers is less than the free length of the adjacent pillars, the following measures must be taken:

a) the entire height of the pillars is considered as a critical area, and must be assembled with the quantity and pattern of fences required for critical areas.

b) the consequences of reducing the ratio of the shearing section of those pillars should be considered appropriately. To do this, sections **5.4.2.3** and **5.5.2.2** must be applied for the calculation of the acting shear strength, depending on the ductility class. In this calculation, the free length of the pillar, $l_{cl}$, must be taken equal to the length of the pillar that is not in contact with the fillers, and the moment $M_{i,d}$ in the section of the pillar at the top of the filler wall must be taken equal to $\gamma_{Rd} \cdot M_{Rc,i}$ con $\gamma_{Rd}$ = 1.1 for DCM ductility class and 1.3 for DCH ductility class, with $M_{Rc,i}$ being the calculation value of the bending resistance of the pillar;

c) the transverse reinforcement to withstand this shear strength must be arranged along the length of the pillar that is not in contact with the fillers, and extended along a length $h_c$ (dimension of the cross section of the pillar in the plane of the filler wall) within the part of the pillar that is in contact with such fillers;

d) if the length of the pillar that is not in contact with the fillers is less than 1.5 h $h_c$, the shear strength must be resisted by the diagonal reinforcements.

(3) Where fillers are spread along the entire free length of adjacent pillars, and there are brickwork walls only on one side of the pillar (e.g. corner pillars), the entire length of the pillar must be considered as a critical area and must be assembled with the quantity and pattern of fences required for critical areas.

(4) The length, $l_c$, of the pillars on which the stress is exerted due to the diagonal connecting rod of the filler must be checked by shearing for the lower of the values of the following two shearing stresses: the horizontal component of the connecting rod force of the filler, assumed to be equal to the horizontal shearing strength of the panel, estimated according to the shearing strength of the horizontal joints; or b) the shear strength calculated in accordance with section **5.4.2.3** or **5.5.2.2**, depending on the ductility class, assuming the bending resistance reserve capacity of the pillar, $\gamma_{Rd} \cdot M_{Rc,i}$, develops at the two ends of the contact length, $l_c$. The contact length must be assumed to be equal to the total vertical width of the diagonal connecting rod of the filler Unless a more accurate estimation of this width is made, taking into account the elastic properties and geometry of the filler and of the pillar, it can be assumed that the width of the connecting rod is a fixed fraction of the length of the diagonal of the panel.

## 5.10 Provisions for concrete diaphragms

(1) A solid reinforced concrete slab can serve as a diaphragm if its thickness is not less than 70 mm and is reinforced in the two horizontal directions with at least the minimum reinforcement specified in Annex 19 to the Structural Code.

(2) A prefabricated concrete floor slab or cover with a concrete compression layer *in situ* may be considered as a diaphragm if that layer:

a) meets the requirements of section 5.11.3.5;

b) is reinforced in both horizontal directions with at least the minimum reinforcement specified in Annex 19 to the Structural Code;

c) its reinforcement is connected to the beams or walls supporting the concrete slab;

d) is concreted on a clean and rough substrate, or connected to that substrate through shearing connectors; and

e) is sized in such a way as to provide the rigidity and strength required for diaphragms.

(3) The seismic project must include USL testing of reinforced concrete diaphragms in DCH ductility class structures with the following properties:

- irregular geometries or shapes divided into floor, diaphragms with hollows and incoming;
- large and irregular openings in diaphragm;
- irregular mass or rigidity distribution (e.g. incoming or outgoing cases);
- basements with walls only on a part of its perimeter or only in part of the floor area at ground level.

(4) The effects of actions on reinforced concrete diaphragms can be estimated by modelling the diaphragms as wide beams, as flat plating or as models of cranks and straps, resting on elastic

supports.

(5) The calculation values of the effects of the shares must be obtained taking into account section **4.4.2.5**.

(6) The values for calculating resistances must be obtained in accordance with Annex 19 to the Structural Code.

(7) In the case of flexible torsion systems (core systems) or structural wall systems of ductility class DCH, it must be checked that horizontal forces are transmitted from the diaphragms to the cores or into the walls. In this respect, the following provisions apply:

a) the calculation value of the shearing voltage at the interface between the diaphragm and a core or wall must be limited to $1.5f_{ctd}$, in order to control cracking;

b) adequate resistance to breakage by shearing stress due to displacement should be ensured, assuming the crank angle is 45°. Additional reinforcement which contributes to the shear strength of the interface between diaphragms and cores or walls must be fitted; the anchorage of these reinforcements must follow the provisions indicated in section **5.6**.

## 5.11 Prefabricated concrete structures

### 5.11.1 General considerations

#### 5.11.1.1 Object and field of application. Types of structures

(1) Section **5.11** applies to the seismic project of concrete structures partially or entirely constructed with prefabricated elements.

(2) Unless otherwise specified (see point **(4)** of section **5.11.1.3**), all provisions of Chapter **5** of this Annex and section **10** of Annex 19 to the Structural Code apply.

(3) Section **5.11** covers the following types of structures, defined in sections **5.1.2** and **5.2.2.1**:

- portico systems;

- wall systems (screen walls);

- dual systems (mixed prefabricated structures and prefabricated or monolithic walls).

(4) In addition, they also cover:

- wall systems panel (transverse wall structures)

- alveolar structures (alveolar systems of prefabricated monolithic parts).

#### 5.11.1.2 Evaluation of prefabricated structures

(1) When modelling the prefabricated structures, the following evaluations should be carried out:

a) identification of the different roles of the structural elements, among the following:

- elements that withstand only gravitational loads, e.g. articulated pillars arranged around a reinforced concrete core;
- elements resisting both gravitational and seismic loads, e.g. porticoes or walls;
- elements that provide an adequate connection between structural elements, e.g. concrete slabs or covered with diaphragm function.

b) ability to comply with the seismic resistance provisions of sections **5.1** to **5.10**:

- prefabricated systems capable of meeting these requirements;
- prefabricated systems combined with pillars or walls manufactured *in situ*, in order to meet these requirements;
- prefabricated systems that do not meet the above conditions and therefore need additional dimensioning criteria and to which lower performance coefficients should be assigned.

c) identification of non-structural elements, which may be:

- completely decoupled structure; or
- partially resisting deformation of structural elements.

d) identification of the effect of connections on the energy dissipation capacity of the structure:

- connections sufficiently far away from critical areas (as defined in **(1)** of section **5.1.2**), which do not affect the energy dissipation capacity of the structure [see section **5.11.2.1.1** and, for example, Figure 5.14(a)].
- connections located within critical areas but oversized appropriately with respect to the rest of the structure, so that in the seismic calculation situation they remain elastic, while the inelastic response occurs in other critical areas [see section **5.11.2.1.2** and, for example, Figure 5.14(b)].
- connections within critical areas and with considerable ductility [see **5.11.2.1.3** and e.g. Figures 5.14(c) and 5.14(d)].

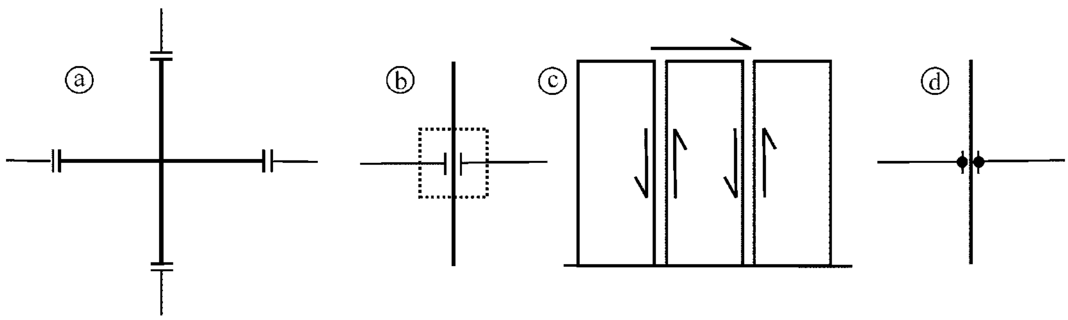



**Figure 5.14 – (a) connection located outside critical areas;**
**(b) oversized connection with plastic hinges displaced to the outside of the connection;**
**(c) ductile shearing connections of large panels, located within critical areas (e.g. ground floor concrete slab);**
**and (d) ductile continuity connections located within critical areas of the porticoes**

#### 5.11.1.3 Dimensioning criteria

##### 5.11.1.3.1 Local resistance

(1) In prefabricated elements and their connections, the possible degradation of the response due to postelastic cyclic deformations must be taken into account. Normally, such response degradation is covered by the partial safety coefficients of the material for steel and concrete (see points **(1)** and **(2)** of section **5.2.4**). If this is not the case, the calculation value of the strength of the prefabricated connections under monotonous load must be appropriately reduced for checks for the seismic calculation situation.

##### 5.11.1.3.2 Energy dissipation

(1) In prefabricated concrete structures, the main energy dissipation mechanism should be plastic hinges within critical areas.

(2) In addition to energy dissipation by plastic hinges within critical areas, prefabricated structures can also dissipate energy by means of plastic shearing mechanisms along the junctions, provided that the following two conditions are met:

a) the force-response ratio must not be significantly degraded during seismic action; and

b) possible instabilities should be avoided in an appropriate manner.

(3) The three ductility classes indicated in chapter **5** for on-site manufactured structures also apply in the case of prefabricated systems. Only point **(2)** of section **5.2.1** and section **5.3** of chapter **5** apply to the DCL ductility class prefabricated building project.
The DCL ductility class will only be considered in cases of low seismicity. In the case of panel wall structures, the DCM ductility class must be selected.

(4) The shearing energy dissipation capacity can be taken into account, especially in prefabricated wall systems (screen walls), using the values of the local sliding ductility coefficients, $\mu_s$, in the choice of the global behaviour coefficient $q$.

##### 5.11.1.3.3 Additional specific provisions

(1) Section **5.11** only covers regular prefabricated structures (see **4.2.3**). However, the verification of prefabricated elements of non-regular structures may be based on the provisions of this subsection-

(2) All vertical elements must be extended to the foundation level, without interruption.

(3) The uncertainties related to resistances are those indicated in point **(2)** of section **5.2.3.7**.

(4) The uncertainties related to ductility are those indicated in point **(3)** of section **5.2.3.7**.

#### 5.11.1.4 Behaviour coefficients

(1) For prefabricated structures complying with the provisions of section **5.11**, the value of the behaviour coefficient $q_p$ can be calculated according to the following equation, unless specific studies authorise other values:

$$q_p = k_p \cdot q \tag{5.53}$$

where

$q$ is the coefficient of behaviour according to equation (5.1);

$k_p$ is the reduction coefficient, which depends on the energy dissipation capacity of the prefabricated structure (see point **(2)** of this section), which adopts the following values:

$$k_p = \begin{cases} 1.00 \text{ for structures with connections according to 5.11.2.1.1, 5.11.2.1.2 or 5.11.2.1.3} \\ 0.5 \text{ for structures with other connections} \end{cases}$$

(2) For prefabricated structures that do not meet the dimensioning provisions indicated in section **5.11**, the value of the behaviour coefficient $q_p$ must not exceed 1.5.

#### 5.11.1.5 Analysis of transitional situations

(1) During the execution of a structure in which temporary triangulations (arrangements) are provided, it is not necessary to consider seismic actions as calculation situations. However, whenever the earthquake could cause the sinking of some part of the structure resulting in a serious risk to human life, temporal triangulations should be explicitly sized for a properly reduced seismic action.

(2) In the absence of specific studies, it can be assumed that this action is equal to a fraction $A_p$ = 15 % of the calculation action as defined in chapter **3**.

### 5.11.2 Connections of prefabricated elements

#### 5.11.2.1 General provisions

##### 5.11.2.1.1 Connections away from critical areas

(1) The connections of prefabricated elements considered to be remote from critical areas must be located at a distance, from the extreme face of the nearest critical area, equal to at least the largest of the dimensions of the cross-sectional sections of the element in which that area is located.

(2) Such connections must be sized to: a) a shear strength obtained by capacity dimensioning of sections **5.4.2.2** and **5.4.2.3**, with a coefficient to take into account the strength reserve due to steel deformation hardening, $\gamma_{Rd}$, equal to 1.1 for the DCM ductility class or 1.2 for the DCH ductility class; and (b) a bending moment equal to at least the acting moment obtained from the calculation and 50 % of the resistant moment, $M_{Rd}$, on the extreme face of the nearest critical area, multiplied by the coefficient $\gamma_{Rd}$.

##### 5.11.2.1.2 Oversized junctions

(1) The effects of the oversized connection calculation action must be obtained by following the capacity dimensioning rules of sections **5.4.2.2** for beams and **5.4.2.3** for pillars, depending on bending resistance reserves in the extreme sections of critical areas, equal to $\gamma_{Rd} \cdot M_{Rd}$, with the coefficient $\gamma_{Rd}$ taken equal to 1.20 for DCM ductility class and 1.35 for DCH ductility class.

(2) The reinforcement ends of the oversized connection must be anchored completely before the extreme sections of the critical area.

(3) The reinforcement of the critical area must be anchored completely outside the oversized connection.

#### 5.11.2.1.3 Energy dissipating connections

(1) These connections must meet the local ductility criterion indicated in section **5.2.3.4** and in the relevant points of section **5.4.3** and **5.5.3**.

(2) Alternatively, it must be demonstrated by inelastic cyclic tests of an appropriate number of test pieces representative of the connection, that this connection has a stable cyclic deformation and energy dissipation capacity equal to that of a monolithic connection with the same strength and in accordance with the local ductility provisions specified in sections **5.4.3** or **5.5.3**.

(3) Tests must be carried out on representative specimens, following an appropriate deformation cycle history, including at least three complete cycles at the amplitude corresponding to $q_p$, according to point **(3)** of section **5.2.3.4**

### 5.11.2.2 Assessment of resistance of connections

(1) The calculation value of the strength of the connections between precast concrete elements must be obtained in accordance with the provisions of section **6.2.5** and the section **10** of Annex 19 to the Structural Code, using the partial material safety coefficients indicated in points **(2)** and **(3)** of section **5.2.4**. In the event that these provisions do not sufficiently cover the type of connection considered, their strength must be assessed by means of appropriate experimental studies.

(2) In the assessment of the resistance to shearing stress due to the displacement of a connection, friction resistance should be disregarded under external compressive forces (opposed to the internal forces due to the gag effect of the reinforcements crossing the junction).

(3) Welding steel reinforcements into energy dissipation connections can be taken into account structurally when all of the following conditions are met:

a) only weldable steels are used;

b) welding materials, techniques and personnel ensure a loss of local ductility of less than 10 % of the ductility coefficient that would be achieved if the connection were made by a means other than welding.

(4) It must be demonstrated, analytically and experimentally, that steel elements (profiles or rounds) fixed to concrete elements and with the mission of contributing to seismic resistance withstand a history of deformation cyclic loads imposed at the target level of ductility, as specified in point **(2)** of section **5.11.2.1.3**.

### 5.11.3 Element

#### 5.11.3.1 Beams

(1) The appropriate provisions of section **10** of Annex 19 to the Structural Code and sections **5.4.2.1**, **5.4.3.1**, **5.5.2.1** and **5.5.3.1** of this Annex 1 apply, in addition to those set out in **5.11**.

(2) Prefabricated beams simply supported must be structurally connected to pillars or walls. The connection must ensure the transmission of horizontal forces in the seismic calculation situation, without considering friction.

(3) In addition to the relevant provisions set out in section **10** of Annex 19 to the Structural Code, the tolerance and margins to consider the loss of coating (removing) of the supports must also be sufficient for the foreseeable displacement of the bearing element (see **4.3.4**).

#### 5.11.3.2 Pillars

(1) The relevant provisions of sections **5.4.3.2** and **5.5.3.2** apply, in addition to the rules set out in section **5.11**.

(2) Pillar-pillar connections within critical areas are allowed only in the DCM ductility class.

(3) For prefabricated portico systems with articulated beam-pillar connections, the pillars must be fixed completely against translation and rotation at the base, in foundations dimensioned in accordance with section**5.11.2.1.2**.

#### 5.11.3.3 Joints between beam and pillar

(1) The junctions between beam and monolithic pillar [see Figure 5.14(a)] must follow the relevant provisions of sections **5.4.3.3** and **5.5.3.3**.

(2) The strength and ductility of the end connections of the beams to the pillars [see Figures 5.14(b) and 5.14(d)] must be checked specifically in accordance with section **5.11.2.2.1**.

#### 5.11.3.4 Large-sized prefabricated panel walls

(1) Section **10** of Annex 19 of the Structural Code applies, with the following modifications:

a) the minimum total amount of reinforcement refers to the actual area of the concrete cross-section, and must include the vertical reinforcements of the web and the contouring elements;

b) single mesh reinforcement in the form of a single layer is not allowed;

c) minimum confinement of concrete close to the ends of all prefabricated panels, as specified in sections **5.4.3.4.2** or **5.5.3.4.5** for pillars, over a square side section $b_w$, where $b_w$ designates the thickness of the panel.

(2) The part of the panel wall between a vertical joint and an aperture less than 2.5 $b_w$ of that joint must be dimensioned and detailed in accordance with sections **5.4.3.4.2** or **5.5.3.4.5**, depending on the ductility class.

(3) The degradation of the strength of the connections must be avoided.

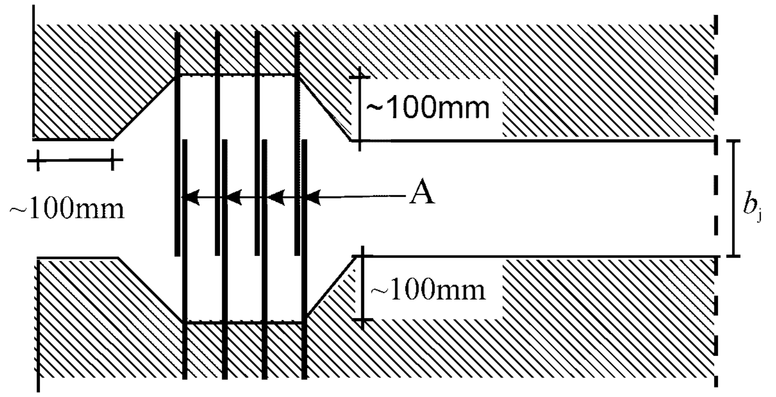
(4) For this purpose, all vertical joints must be rough or fitted, and must be checked by shearing.

(5) Horizontal joints subjected to compression along their entire length may not be jagged. If they are partially in compression and partially in traction, they must have notches along their entire length.

(6) The following additional rules apply for checking horizontal connections of walls built with large-sized prefabricated panels:

a) the total tensile force produced by the axillary actions (with respect to the wall) must be resisted by the vertical reinforcement located along the traction area of the panel, and anchored in the body of the upper and lower panels. The continuity of this reinforcement must be ensured by ductile welding carried out within the horizontal joint or, preferably, by special staples arranged for this purpose (Figure 5.15).

b) in horizontal connections which are partly in compression and partially in traction (under the seismic calculation situation), the shearing strength check (see **5.11.2.2**) must be carried out only along the compression part. In such case, the value of the axial stress, $N_{Ed}$, must be replaced by the value of the total compressive force, $F_c$, acting in the compression area.




Legend

A Reinforcement flap welding

**Figure 5.15 – Traction reinforcement that may be required at the ends of the walls**

(7) In order to improve local ductility along vertical connections of large panels, the following additional rules must be respected:

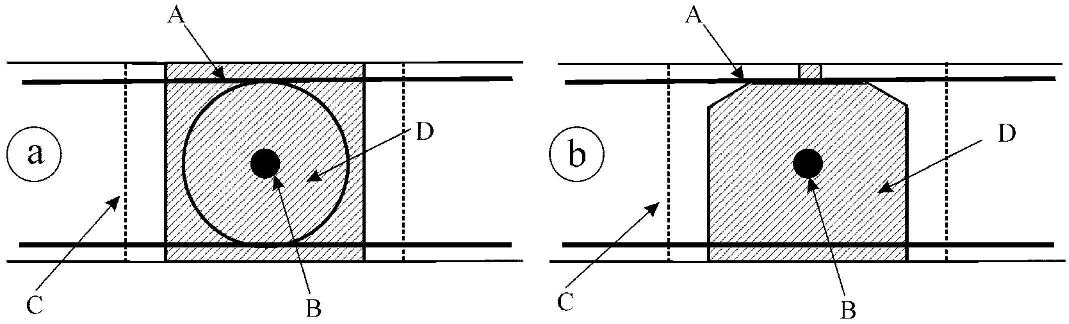
a) a minimum reinforcement must be provided through the connections equal to 0.10 % for fully compressed connections, and equal to 0.25 % for connections partially in compression and partly in traction;

b) the amount of reinforcement through the connections must be limited, in order to avoid a sudden loss of rigidity when exceeding the peak of the behavioural diagram. If not adequately justified, the amount of reinforcement must not exceed 2 %;

c) such reinforcement must be distributed over the entire length of the connection. In the DCM ductility class this reinforcement can be concentrated in three bands (top, half and bottom);

d) measures must be taken to ensure the continuity of the reinforcement through the panel-panel connections. For this purpose, steel reinforcements must be anchored in vertical connections either with U-bars or, in the case of junctions with at least one free face, by welding through the connection (see Figure 5.16);

e) to ensure continuity throughout the connection after cracking, a longitudinal reinforcement of minimum size $\rho_{c,min.}$ =1 % within the connection filler mortar (see Figure 5.16).




Legend

A Reinforcement that exceeds the connection

B Longitudinal connection reinforcement

C Teeth (notches)

D Filling mortar between panels

**Figure 5.16 – Cross-section of vertical connections between large, prefabricated panels, (a) junction with two free sides; B) junction with a free face**

(8) As a result of the energy dissipation capability along vertical (and partly along horizontal) connections of large panels, the walls constructed with such prefabricated panels are exempt from meeting the requirements set out in sections **5.4.3.4.2** and **5.5.3.4.5**, concerning confinement of contour elements.

### 5.11.3.5 Diaphragms

(1) In addition to the provisions of section **10** of Annex 19 to the Structural Code, concerning slabs, and those indicated in section **5.10** of this Annex, the following dimensioning rules also apply in the case of diaphragm concrete slabs made of prefabricated elements.

(2) When the rigid diaphragm condition of the point **(4)** of section **4.3.1** is not complied with, the deformability in the floor concrete slab as well as the connections with the vertical elements must be taken into account in the model.

(3) The rigid behaviour of the diaphragm is improved if the joints in the diaphragm are placed only on its supports. A suitable coating layer of reinforced concrete *in situ* can greatly improve the rigidity of the diaphragm. The thickness of this coating layer must not be less than 40 mm if the lamp between supports is less than 8 m, or not less than 50 mm for larger lamps; its mesh reinforcement must be connected to the sturdy vertical elements above and below.

(4) The tensile forces must be resisted by steel straps arranged at least along the perimeter of the diaphragm, as well as along some of the joints of the prefabricated slabs. If a manufactured coating layer *in situ* is used, this additional reinforcement must be placed on this coating layer.

(5) In any case, these straps must form a continuous reinforcement system throughout the entire diaphragm and must be properly connected to each lateral stress-resistant element.

(6) The sharp forces acting in the plane along the slab-slab or slab-beam connections must be calculated in accordance with section **4.4.2.5**. The calculation value of the resistance must be obtained according to section **5.11.2.2**.

(7) The primary seismic elements above and below the diaphragm must be properly connected to the diaphragm. For this purpose, any horizontal joint must always be properly assembled. Friction

forces due to external compression forces should not be considered for this purpose.

# 6 Specific rules for steel buildings

## 6.1 General considerations

### 6.1.1 Object and field of application

(1) For the project of steel buildings the provisions of the Structural Code (Annexes 22 to 29) apply. The following rules complement these regulations.

(2) For buildings with mixed steel and concrete structures the chapter **7** applies.

### 6.1.2 Dimensioning principles

(1) Earthquake-resistant steel buildings must be designed according to one of the following principles (see Table 6.1):

- principle (a) Minimally dissipative structural behaviour;
- principle (b) Dissipative structural behaviour.

**Table 6.1 - Dimensioning principles, structural ductility classes and upper limit of behaviour coefficient reference values**

| Dimensioning principle | Structural ductility class | Range of behaviour coefficient reference values $q$ |
|---|---|---|
| Principle (a)<br>Minimally dissipative structural behaviour | DCL (low) | ≤ 1.5 |
| Principle (b)<br>Dissipative structural behaviour | DCM (medium) | ≤ 4<br>Also limited by the values in Table 6.2 |
| | DCH (high) | Limited only by the values in Table 6.2 |

The value for the upper limit of $q$ for a low dissipative behaviour is 1.5.

No geographical limitations are established on the choice of dimensioning principle and the DCM and DCH ductility class for steel structures.

(2) In principle a) the effects of the action can be obtained by using an overall elastic calculation, without considering a significant nonlinear behaviour of the material. When using the calculation spectrum defined in section **3.2.2.5**, the upper limit of the reference value of the behaviour coefficient $q$ must be taken equal to 1.5 (see point **(1)** above). In the case of height irregularity, the behaviour coefficient $q$ should be corrected according to point **(7)** of section **4.2.3.1**, but it is not necessary to take it less than 1.5.

3) In principle (a), where the upper limit of the reference value of $q$ is taken equal to 1.5, the primary seismic elements of the structure may belong to any of the four classes of cross-sections 1, 2, 3 or 4, as defined in section 5.5.2 (1) of Annex 22 to the Structural Code.

(4) In principle (a) the strength of the elements and connections must be assessed according to the Structural Code (Annexes 22 to 29) without any additional requirements. For buildings that are not

seismically isolated (see chapter **10**), dimensioning according to principle (a) is recommended only for cases of low seismicity (see point **(4)** of section **3.2.1**).

(5) Principle (b) takes into account the ability of some parts of the structure (dissipative areas) to resist earthquake actions by their behaviour outside the elastic domain. When using the calculation spectrum defined in **3.2.2.5**, the reference value of the behaviour coefficient $q$ can be taken greater than the upper-limit value set out in Table 6.1 for a less dissipative structural behaviour. The upper-limit value of $q$ depends on the ductility class and the type of structure (see **6.3**). When this principle (b) is adopted, the requirements specified in sections **6.2** to **6.11** must be met.

(6) Structures sized according to principle (b) must belong to the DCM or DCH ductility classes. These classes correspond to a greater capacity of the structure to dissipate energy through plastic mechanisms. Depending on the ductility class, the specific requirements relating to one or more of the following aspects must be met: class of steel profiles and rotational capacity of connections.

### 6.1.3 Safety checks

(1) For the ultimate limit state checks in structural steel, the partial safety coefficient for the material properties $\gamma_s = \gamma_M$ it should take into account the possible degradation of resistance due to cyclic deformations.

When due to local ductility provisions, the ratio between the residual resistance after degradation and the initial resistance is approximately equal to the ratio between the values of $\gamma_M$ for accidental and fundamental load combinations, the partial safety coefficient $\gamma_s$ adopted for persistent and transitory calculation situations must be applied in the calculation seismic situation.

(2) In the capacity dimensioning checks specified in sections **6.5** to **6.8**, consideration should be given to the possibility that the actual elastic limit of steel is greater than the nominal elastic limit, using a resistance reserve coefficient of the material $\gamma_{ov}$ (see point **(3)** of section **6.2**).

## 6.2 Materials

(1) Structural steel must comply with the provisions of the Structural Code.

(2) The distribution in the structure of the material properties, such as the elastic limit and hardness, must be such that the dissipative areas are formed where it was intended in the dimensioning.

NB The plasticisation of the dissipative areas usually occurs before the exit of the elastic domain of the other areas during the earthquake.

(3) The requirement indicated in point **(2)** above can be met if the steel elastic limit of the dissipative areas and the dimensioning of the structure meet any of the following conditions (a), (b) or (c):

a) upper value of elastic limit, $f_{y,max.}$, of the steel of the dissipative areas satisfies the equation: $f_{y,max.} \leq 1.1\ \gamma_{ov} f_y$

where

$\gamma_{ov}$ is the resistance reserve coefficient used in the calculation $\gamma_{ov} = 1.25$; and

$f_y$ is the nominal elastic limit specified for the type of steel in question.

NB For steels of type S235 and with $\gamma_{ov}$ = 1.25, this method produces a maximum of $f_{y,max.}$ = 323 N/mm$^2$.

b) the dimensioning of the structure is carried out according to a single type of steel and a single nominal elastic limit $f_y$ for steel of both dissipative and non-dissipative areas; a higher value is specified $f_{y,max.}$ for dissipative area steel; the nominal value $f_y$ of specified steels for non-dissipative areas and junctions exceeds the maximum elastic limit value $f_{y,max.}$ of the dissipative areas.

NB This condition normally leads to the use of S355 type steels in non-dissipative elements and junctions (dimensioned on the basis of the $f_y$ of S235 type steels) and the use of S235 type steels for dissipative elements and junctions where the maximum elastic limit value of S235 steels is limited to $f_{y,max.}$ = 355 N/mm$^2$.

c) the real elastic limit, $f_{y,act}$, of the steel of each dissipative area is determined by measurements, and the resistance reserve coefficient is obtained for each dissipative area according to equation $\gamma_{ov,act} = f_{y,act}/f_y$, with $f_y$ being the nominal elastic limit of steel from dissipative areas.

NB This condition is applicable when using known steels from the stock, in the assessment of existing buildings, or when the safety side assumptions used in dimensioning relating to the elastic limit are confirmed by measurements made before construction.

(4) If condition (b) of **(3)** above is complied with, the resistance reserve coefficient, $\gamma_{ov}$, can be taken equal to 1.0 in the dimensioning checks for structural elements defined in sections **6.5** to **6.8**. In checking equation (6.1) for connections, the value of $\gamma_{ov}$ to be used is the same as in condition (a) of point **(3)**.

(5) If condition (c) of point **(3)** previously is complied with, the resistance reserve coefficient, $\gamma_{ov}$, equal to the maximum between the values $\gamma_{ov,act}$ obtained in the checks specified in sections **6.5** to **6.8**.

(6) For dissipative areas, the value of the elastic limit $f_{y,max}$ considered in the application of the conditions indicated in point **(3)** must be specified and annotated in the planes.

(7) The hardness of steels and welds must satisfy the requirements for seismic action with the quasi-permanent value of the operating temperature (see Annex 28 to the Structural Code).

(8) The required hardness of steels and welds and the minimum operating temperature adopted in combination with the seismic action must be defined in the project specifications.

(9) In bolted junctions of primary seismic elements, high-strength screws of categories 8.8 or 10.9 must be used.

(10) The control of the properties of the materials must be carried out in accordance with section **6.11**.

## 6.3 Types of structures and behaviour coefficients

### 6.3.1 Types of structures

(1) Steel buildings must be classified within one of the following types of structures, depending on the behaviour of their primary resistant structure under seismic actions (see Figures 6.1 to 6.8):

a) Pending-resistant porticoes: structures in which resistance to horizontal forces occurs mainly by elements acting at bending.

b) Porticoes with centred triangulations: structures in which resistance to horizontal forces occurs mainly by elements subjected to axillary forces.

c) Porticoes with off-centre triangulations: structures in which resistance to horizontal forces is produced mainly by elements subjected to axillary forces, but in which the eccentricity of geometry is such that energy can be dissipated in seismic couplings by cyclic bending or cyclic shearing.

d) Inverted pendulum structures: structures defined in section **5.1.2**, in which the dissipative areas are located at the base of the pillars.

e) Structures with concrete cores or concrete walls: structures in which resistance to horizontal forces occurs mainly by such nuclei or walls (screen walls).

f) Bending-resistant porticoes, combined with centred triangulations.

g) Bending-resistant porticoes, combined with fillers.

(2) In bending-resistant porticoes, the dissipative areas should be located mainly in the plastic hinges formed on the beams or in the junctions between beams and abutments, so that the energy is dissipated by cyclic bending. These dissipative areas can also be located on the pillars:

- at the base of the portico;

- at the top of the pillars of the last floor in multi-storey buildings;

- on the top and bottom parts of the building pillars of a floor in which $N_{Ed}$ on the pillars meets the condition: $N_{Ed}/N_{pl,Rd} < 0.3$.

(3) In porticoes with centred triangulations, the dissipative areas should be located mainly in the diagonal tensions.

Triangulations (bracing) may belong to one of the following categories:

- active diagonal tensile triangulations, in which the resistance to horizontal forces can be secured by means of the diagonal tensions only, disregarding the diagonal compressions;

- V triangulations, in which the resistance to horizontal forces can be secured by both the tensile diagonals and the diagonal compressions. The point of intersection of these diagonals is located on a horizontal element that must be continuous.

K Triangulations, where the intersection of diagonals is located on a pillar (see Figure 6.9) cannot be used.

(4) On porticoes with off-centre triangulations, configurations must be used to ensure that all seismic couplings are active, as shown in Figure 6.4.

(5) The inverted pendulum structures can be considered as bend-resistant porticoes assuming that

the earthquake-resistant structures have more than one pillar on each resistant plane and that the following condition of limitation of the axial stress in each of the pillars is complied with: $N_{Ed} < 0.3\ N_{pl,Rd}$.

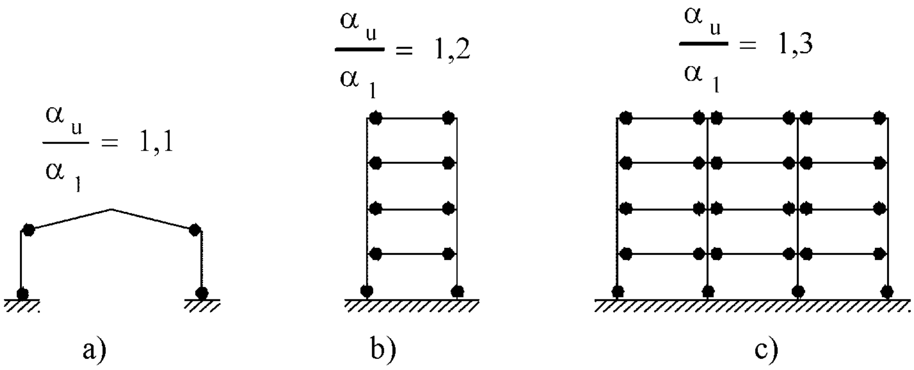



**Figure 6.1 – Flex-resistant porticoes (dissipative areas in beams and at the bottom of pillars) Default values for $\alpha_u/\alpha_1$ (see section 6.3.2.(3) and Table 6.2)**

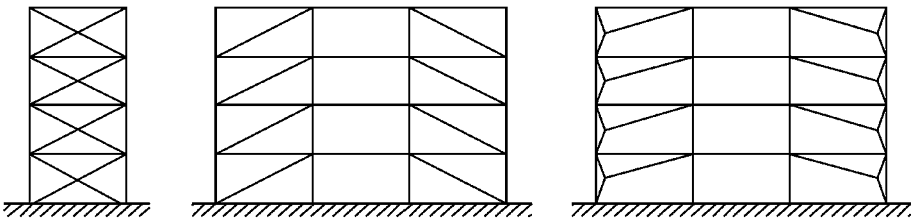


**Figure 6.2 - Porticoes with diagonal centred triangulations (dissipative areas only at diagonal tensions)**

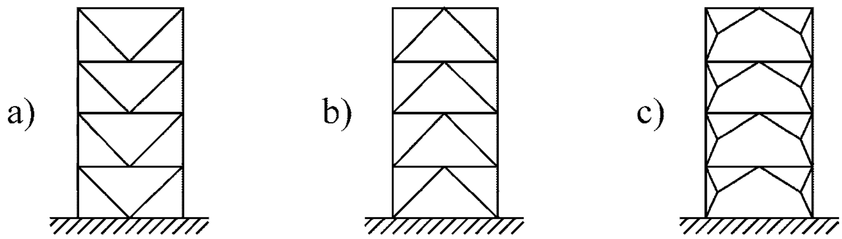



**Figure 6.3 – Porticoes with V-centred triangulations (dissipative areas at diagonal tensions and diagonal compressions)**

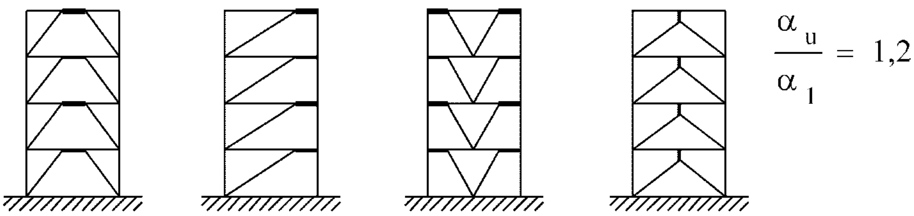



**Figure 6.4 - Porticoes with off-centre triangulations (dissipative areas in bending and shearing couplings).**

**Default values for $\alpha_u/\alpha_1$**
**(see point (3) of section 6.3.2 and Table 6.2)**

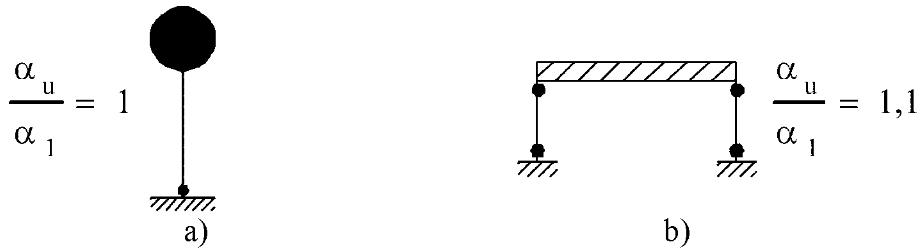



**Figure 6.5 – Inverted pendulum structures:**
**a) dissipative areas at the base of the pillar;**
**b) pillar dissipative zones ($N_{Ed}/N_{pl,Rd}$ < 0.3).**
**Default values for $\alpha_u/\alpha_1$ (see section 6.3.2.(3) and Table 6.2)**

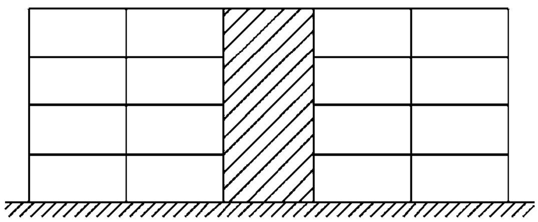


**Figure 6.6 – Structures with concrete cores or concrete walls**

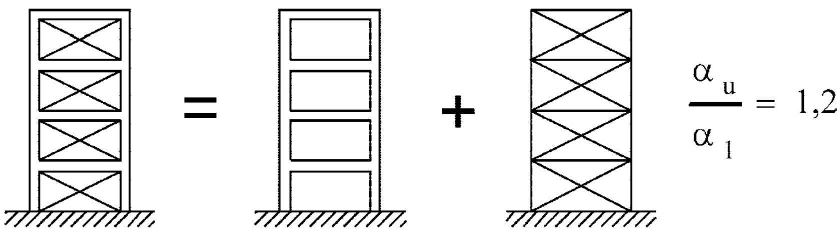



**Figure 6.7 - Bending-resistant porticoes combined with centred triangulations**
**(dissipative areas on the bending portico and on the diagonals at traction)**
**Default values for $\alpha_u/\alpha_1$ (see section 6.3.2.(3) and Table 6.2)**

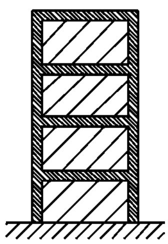


**Figure 6.8 - Bending-resistant porticoes combined with fillers**

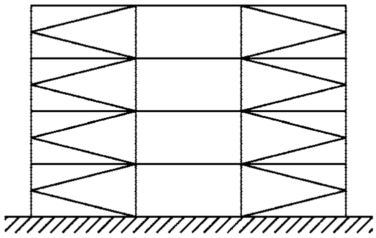


**Figure 6.9 - Portico with K triangulations (not allowed)**

### 6.3.2 Behaviour coefficients

(1) The behaviour coefficient $q$, introduced in the section **3.2.2.5**, takes into account the energy dissipation capacity of the structure. For regular structural systems a behaviour coefficient $q$ must be taken with the upper limits of the reference values specified in Table 6.2, assuming that the rules indicated in sections **6.5** to **6.11** are met.

**Table 6.2 - Upper limits of behaviour coefficient reference values for regular height systems**

| | STRUCTURAL TYPE | Ductility class DCM | Ductility class DCH |
|---|---|---|---|
| a) | Bending-resistant porticoes | 4 | $5\ \alpha_u/\alpha_1$ |
| b) | Porticoes with centred triangulations | | |
| | Diagonal triangulations | 4 | 4 |
| | V triangulations | 2 | 2.5 |
| c) | Porticoes with off-centre triangulations | 4 | $5\ \alpha_u/\alpha_1$ |
| d) | Inverted pendulum structures | 2 | $2\ \alpha_u/\alpha_1$ |
| e) | Structures with concrete cores or concrete walls | See chapter **5** | |
| f) | Bending porticoes combined with concentric bracing | 4 | $4\ \alpha_u/\alpha_1$ |
| g) | Bending-resistant porticoes combined with fillers | 2 | 2 |
| | Non-connected concrete or brickwork fillers, in contact with the structure | See chapter **7** | |
| | Connected reinforced concrete fillers<br>Bend resistant portico isolated fillers (see bending resistant porticoes) | 4 | $5\ \alpha_u/\alpha_1$ |

(2) If the building is irregular in height (see **4.2.3.3**) the upper-limit values of $q$ indicated in Table 6.2 must be reduced by 20 % (see **(7)** of section **4.2.3.1** and Table 4.1).

(3) For regular buildings on the floor, if no calculations are performed to determine $\alpha_u/\alpha_1$, the approximate default values of the ratio $\alpha_u/\alpha_1$ indicated in Figures 6.1 to 6.8 can be used. The parameters $\alpha_l$ and $\alpha_u$ are defined as follows:

- $\alpha_1$ is the value by which the seismic action of horizontal calculation is multiplied, in order to first reach the plastic resistance in some element of the structure, while the rest of the calculation actions remain constant;
- $\alpha_u$ is the value by which the seismic action of horizontal calculation is multiplied, so that the plastic hinges are formed in a sufficient number of sections for the development of an

overall instability of the structure, while the rest of the calculation actions remain constant. The coefficient $\alpha_u$ can be obtained by a nonlinear static global analysis (pushover).

(4) For non-regular buildings on the floor (see **4.2.3.2**), the approximate value of $\alpha_u/\alpha_1$ which can be used, if no calculations are performed for evaluation, is equal to the mean between (a) 1.0; and (b) the value given in Figures 6.1 to 6.8.

(5) Values of $\alpha_u/\alpha_1$ greater than those specified in points **(3)** and **(4)** above are allowed, provided that they can be confirmed with the calculation of $\alpha_u/\alpha_1$ by nonlinear static global analysis (pushover).

(6) The maximum value of $\alpha_u/\alpha_1$ that can be used in dimensioning is equal to 1.6, even if the analysis mentioned in point **(5)** above provides higher potential values.

## 6.4 Structural analysis

(1) The dimensioning of the concrete slab with diaphragm function must be in accordance with section **4.4.2.5**.

(2) Unless otherwise specified in this chapter (e.g. for porticoes with centred triangulations, see points **(1)** and **(2)** of section **6.7.2**) the structure analysis can be performed assuming that all elements of the earthquake-resistant structure are active.

## 6.5 Dimensioning criteria and construction detail rules common to all types of structures with dissipative structural behaviour

### 6.5.1 General considerations

(1) The dimensioning criteria indicated in section **6.5.2** must be applied to the earthquake-resistant parts of the sized structures according to the principle of dissipative structural behaviour.

(2) The dimensioning criteria indicated in section **6.5.2** are considered complied with if the construction details rules indicated in sections **6.5.3** to **6.5.5** are observed.

### 6.5.2 Dimensioning rules for dissipative structures

(1) Structures with dissipative areas should be sized so that plasticisation, local buckling or any other phenomenon due to hysteresis behaviour does not affect the overall stability of the structure.

NB The coefficients $q$ indicated in Table 6.2 are considered to meet this requirement (see point **(2)** of section **2.2.2**).

(2) Dissipative areas must have adequate strength and ductility. Resistance must be checked in accordance with the Structural Code (Annexes 22 to 29).

(3) Dissipative areas can be located in structural elements or connections.

(4) If the dissipative areas are located in the structural elements, the non-dissipative parts and the connections of the dissipative parts with the rest of the structure must have a sufficient reserve of strength to allow the development of cyclic plasticisation in the dissipative parts.

(5) When the dissipative areas are located in the junctions, the connected elements must have a

sufficient reserve of resistance to allow the development of cyclic plasticisation in such junctions.

### 6.5.3 Dimensioning rules for dissipative elements to compression or bending

(1) Sufficient local ductility must be ensured for elements dissipating energy in compression or bending, limiting the ratio between width and thickness, $b/t$, according to the section classes specified in **5.5** of Annex 22 of the Structural Code.

(2) The requirements for the section classes of energy dissipating steel elements are indicated in Table 6.3 according to the ductility class and behaviour coefficient $q$ used in dimensioning.

**Table 6.3 - Requirements for section classes of dissipative elements depending on ductility classes and reference behaviour coefficient**

| Ductility class | behaviour coefficient reference value $q$ | Required class of the section |
|---|---|---|
| DCM | $1.5 < q \leq 2$ | Class 1, 2 or 3 |
| | $2 < q \leq 4$ | Class 1 or 2 |
| DCH | $q > 4$ | Class 1 |

### 6.5.4 Dimensioning rules for tensile parts or elements

(1) In the case of elements or parts of elements subject to traction, the ductility requirement specified in section **(3)** of section **6.2.3** of Annex 22 to the Structural Code must be met.

### 6.5.5 Dimensioning rules for junctions in dissipative areas

(1) The dimensioning of the junctions must be such that it limits the location of plastic unit deformations, the level of residual stresses, and that it avoids construction defects.

(2) Non-dissipative connections of dissipative elements made with full penetration butt welds can be deemed as meeting the resistance reserve criterion.

(3) For junctions with angled welds or bolted junctions, the following equation must be complied with:

$$R_d \geq 1,1\, \gamma_{ov}\, R_{fy} \tag{6.1}$$

where

$R_d$ is the resistance of the connection, according to the Structural Code (Annexes 22 to 29);

$R_{fy}$ is the plastic resistance of the bonded dissipative element, based on the calculation value of the elastic limit of the material, as defined in the Structural Code (Annexes 22 to 29);

$\gamma_{ov}$ is the resistance reserve coefficient (see point **(2)** of section **6.1.3**, and section **6.2**).

(4) $B$ and $C$ according to section **3.4.1** of Annex 26 to the Structural Code, and screwed traction junctions of category $E$, according to section **3.4.2** of that Annex. Shearing junctions with suitable screws are also allowed. Friction surfaces must belong to Class A or B in accordance with Article 93.8 of the Structural Code.

(5) For shearing screw junctions, the calculation value of the shearing strength of the screws must be more than 1.2 times the calculation value of the resistant capacity.

(6) The adequacy of the dimensioning should be confirmed by experimental evidence, so that the strength and ductility of the elements and their junctions under cyclic load must, in turn, be confirmed by experimental evidence, in order to satisfy the specific requirements defined in the sections **6.6** to **6.9** for each type of structure and class of structural ductility. This applies to junctions of partial or total resistance in or adjacent to dissipative areas.

(7) Experimental evidence can be based on existing data. Otherwise, tests must be carried out.

## 6.6 Dimensioning and construction details rules for bending-resistant porticoes

### 6.6.1 Dimensioning criteria

(1) The bending-resistant porticoes must be sized so that the plastic hinges are formed in the beams or in the connections of the beams with the pillars, but not in the pillars, according to section **4.4.2.3**. This requirement applies neither on the basis of the structure, nor on the upper floor of multi-storey buildings, nor in single-storey buildings.

(2) Depending on the location of the dissipative areas, the point **(4)** or point **(5)** of section **6.5.2**.

(3) The required spherical formation pattern must be achieved by observing the rules indicated in sections **4.4.2.3**, **6.6.2**, **6.6.3**, and **6.6.4**.

### 6.6.2 Beams

(1) It must be checked that the beams have sufficient resistance to breakage by bending by bending outside the plane and by lateral buckling in accordance with the Structural Code (Annexes 22 to 29), assuming the formation of a plastic hinge at one end of the beam. The end of the beam to be considered is the most requested in the seismic calculation situation.

(2) For the plastic hinges of the beams, it must be checked that neither the total resistant plastic moment nor the rotational capacity is reduced by compression or shear strength. To do this, the following conditions must be checked at the points where the formation of hinges is expected:

$$\frac{M_{Ed}}{M_{pl,Rd}} \leq 1,0 \tag{6.2}$$

$$\frac{N_{Ed}}{N_{pl,Rd}} \leq 0,15 \tag{6.3}$$

$$\frac{V_{Ed}}{V_{pl,Rd}} \leq 0,5 \tag{6.4}$$

where

$$V_{Ed} = V_{Ed,G} + V_{Ed,M} \tag{6.5}$$

$N_{Ed}$ is the calculation value of the axial force;

$M_{Ed}$ is the calculation value of the bending moment;

$V_{Ed}$ is the calculation value of the shear strength;

$N_{pl,Rd}$, $M_{pl, Rd}$, $V_{pl,Rd}$ are the values of calculation of resistances, in accordance with the Structural Code;

$V_{Ed,G}$ is the calculation value of the shear strength due to non-seismic actions;

$V_{Ed,M}$ is the calculation value of the shear strength due to the application of plastic moments $M_{pl,Rd,A}$ and $M_{pl,Rd,B}$ with opposite signs at ends A and B of the beam.

NB $V_{Ed,M} = (M_{pl,Rd,A} + M_{pl,Rd,B}) / L$ is the most unfavourable condition, corresponding to a span beam $L$ and dissipative areas at both ends.

(3) For sections belonging to cross-sectional class 3, equations (6.2) to (6.5) must be checked by replacing $N_{pl,Rd}$, $M_{pl,Rd}$, $V_{pl,Rd}$ by $N_{el,Rd}$, $M_{el, Rd}$, $V_{el,Rd}$.

(4) If equation (6.3) is not met, the requirement specified in point **(2)** above is considered complied with if the provisions of section **6.2.9.1** of Annex 22 to the Structural Code are met.

### 6.6.3 Pillars

(1) The pillars must be checked for compression, considering the most unfavourable combination of axial force and ending moments. In the checks, $N_{Ed}$, $M_{Ed}$, $V_{Ed}$ must be obtained by:

$$\begin{aligned} N_{Ed} &= N_{Ed,G} + 1,1\gamma_{ov}\,\Omega N_{Ed,E} \\ M_{Ed} &= M_{Ed,G} + 1,1\gamma_{ov}\,\Omega M_{Ed,E} \\ V_{Ed} &= V_{Ed,G} + 1,1\gamma_{ov}\,\Omega V_{Ed,E} \end{aligned} \tag{6.6}$$

where

$N_{Ed,G}$ ($M_{Ed,G}$, $V_{Ed,G}$) are the compression forces (bending moment and shear strength, respectively) in the pillar, due to the non-seismic actions included in the combination of actions for the seismic calculation situation;

$N_{Ed,E}$ ($M_{Ed,E}$, $V_{Ed,E}$) are the compression forces (bending moment and shear strength, respectively) in the pillar due to the seismic action calculation;

$\gamma_{ov}$ is the resistance reserve coefficient (see point **(2)** of section **6.1.3** and point **(3)** of section **6.2**);

$\Omega$ is the minimum value of $\Omega_i = M_{pl,Rd,i} / M_{Ed,i}$ of all pillars in which dissipative areas exist; $M_{Ed,i}$ is the calculation value of the bending moment in the beam $i$ for the calculation seismic situation and $M_{pl,Rd,i}$ is the corresponding plastic moment.

(2) On pillars forming plastic hinges as indicated in point **(1)** of section **6.6.1**, the check must take

into account that in these plastic hinges the moment acting is equal to $M_{pl,Rd}$.

(3) The test of the strength of the pillars must be carried out in accordance with section **6** of Annex 22 to the Structural Code.

(4) The shearing stress of the pillar $V_{Ed}$, resulting from the structural calculation, must satisfy the following equation:

$$\frac{V_{Ed}}{V_{pl,Rd}} \leq 0,5 \tag{6.7}$$

(5) The transmission of forces from the beams to the pillars must comply with the dimensioning rules indicated in section **6** of Annex 26 of the Structural Code.

(6) The shear strength of web panels reinforced in the connections between beam and pillar (see Figure 6.10) must satisfy the following equation:

$$\frac{V_{wp,Ed}}{V_{wp,Rd}} \leq 1,0 \tag{6.8}$$

where

$V_{wp,Ed}$ is the calculation value of the shear strength on the web panel due to the effects of the actions, taking into account the plastic resistance of adjacent dissipative areas in beams or connections;

$V_{wp,Rd}$ is the web panel shear strength according to section **6.2.6.1** of Annex 26 of the Structural Code. It is not necessary to take into account the effect of the stresses of the axial stress and of the bending moment on the plastic shear strength.

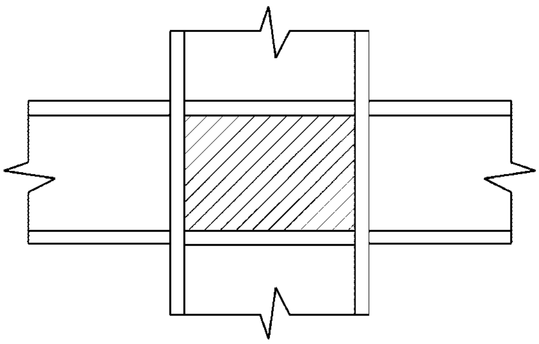


**Figure 6.10 - Web panel reinforced by wings and stiffeners**

(7) It must also be checked that the shear strength of the web panels is in accordance with section **5** of Annex 25 to the Structural Code:

$$V_{wp,Ed} < V_{wb,Rd} \tag{6.9}$$

where

$V_{wb,Rd}$ is the shear strength of the web panel.

### 6.6.4 Beam-to-pillar connections

(1) If the structure is dimensioned to dissipate energy in the beams, the beam-to-pillar connections

should be sized for the required strength reserve degree (see **6.5.5**), taking into account the sturdy moment $M_{pl,Rd}$ and the shear strength ($V_{Ed,G} + V_{Ed,M}$) evaluated in section **6.6.2**.

(2) Dissipative or partial resistance semi-rigid junctions are permitted if the following requirements are met:

a) the connection has a rotational capability consistent with global deformations;

b) the elements converging in the connections have been shown to be stable in the ultimate limit state (USL);

c) the effect of connection deformation on global displacement has been taken into account using a nonlinear static global analysis (pushover) or nonlinear analysis in the time domain.

(3) The dimensioning of the connection must be such that the rotation capacity of the plastic spherical area, $\theta_p$, is not less than 35 mrad for DCH ductility class structures, and 25 mrad for DCM ductility class structures with $q$ > 2. The rotation, $\theta_p$, is defined as:

$$\theta_p = \delta / \; 0,5L \tag{6.10}$$

where (see Figure 6.11)

$\delta$ is the arrow of the beam halfway through the bay;

$L$ is the beam span.

The rotational capacity of the plastic spherical, $\theta_p$, under cyclic load, must be ensured, without a degradation of strength or stiffness greater than 20 %. This requirement is valid regardless of the intended location of the dissipative areas.

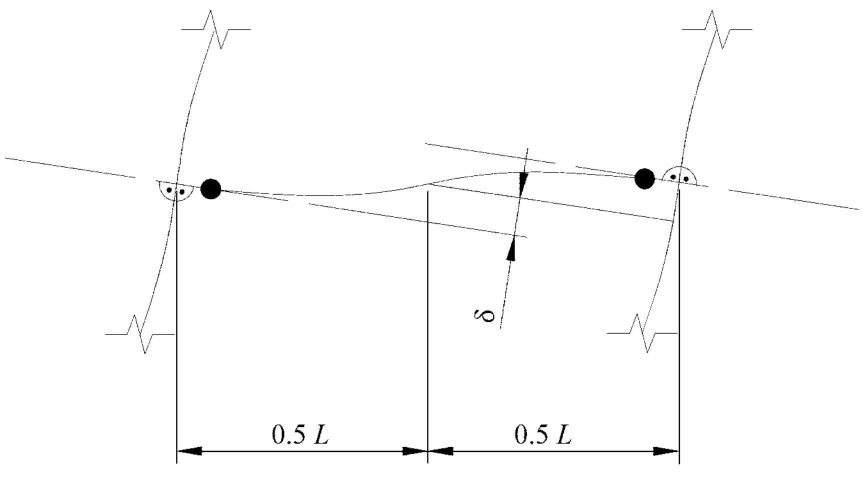



**Figure 6.11 - Beam Arrow For Calculation of $\boldsymbol{\theta}_p$**

(4) In the experiments carried out to evaluate $\theta_p$, the shearing strength of the web panel between pillars must satisfy equation (6.8) and the cutter deformation of the panel between pillars must not contribute more than 30 % to the plastic rotation capacity, $\theta_p$.

(5) The elastic deformation of the pillar must not be included in the assessment of $\theta_p$.

(6) Where partial resistance connections are used, the capacity dimensioning of the pillar must be

derived from the plastic capacity of the junctions.

## 6.7 Dimensioning rules and construction details for porticoes with triangulations (bracing) centred

### 6.7.1 Dimensioning criteria

(1) The portico with centred triangulations should be sized so that the plasticisation of the diagonals in traction takes place before the rupture of the connections and before the plasticisation or the buckling of the beams or pillars.

(2) The diagonal elements of the bracing should be placed in such a way that the structure shows, under stress inversions, similar load-arrow characteristics in each floor in opposite directions of the same direction of bracing.

(3) To do this, the following rule must be met in each floor:

$$\frac{\left| A^{+} - A^{-} \right|}{A^{+} + A^{-}} \leq 0{,}05 \tag{6.11}$$

where $A^+$ and $A^-$ are the horizontal projection areas of the cross-sections of the diagonals at traction, when horizontal seismic actions have a positive or negative direction, respectively (see Figure 6.12).

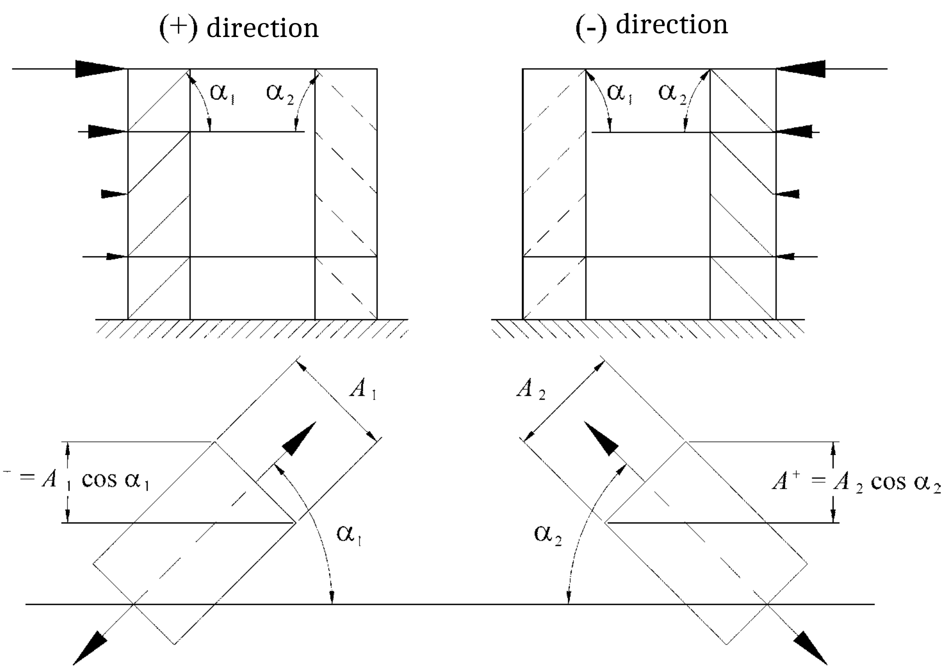



**Figure 6.12 - Example of application of section 6.7.1 (3)**

### 6.7.2 Analysis

(1) Under conditions of gravitational loads, such loads should be considered to be resisted exclusively by beams and pillars, without taking into account the elements of bracing (triangulations).

(2) In an elastic analysis for seismic action, diagonals should be taken into account as follows:

- in structures with diagonal triangulations, only diagonals in traction should be taken into account;

- in structures with V triangulations, both diagonals in traction and diagonals in compression should be taken into account.

(3) In the analysis of any type of concentric bracing, it is possible to take into account both the diagonal tensions and the diagonal compressions provided that the following conditions are met:

a) a nonlinear static global analysis (pushover) or nonlinear analysis in the time domain is used;

b) in the diagonal behaviour model, pre-buckling and post-buckling situations are taken into account; and

c) the basic information that justifies the model used to represent the behaviour of the diagonals is provided.

### 6.7.3 Diagonal elements

(1) In porticoes with triangulations in X, the dimensionless slenderness, $\bar{\lambda}$, defined in Annex 22 of the Structural Code must be limited to $1.3 < \bar{\lambda} \leq 2.0$.

NB The limit of 1.3 is defined to avoid overloading of the pillars in the pre-buckling phase (when both the diagonal compressions and the diagonal tensions are active) beyond the effects of the action obtained from an analysis in the last state where only the diagonal of traction is considered active.

(2) In porticoes with diagonal triangulations where diagonals are not arranged as diagonal triangulations in X (see, for example, Figure 6.12), the dimensionless slenderness, $\bar{\lambda}$, must be less than or equal to 2.0.

(3) In porticoes with V triangulations the dimensionless slenderness, $\bar{\lambda}$, must be less than or equal to 2.0.

(4) In structures up to two floors, there is no limitation for $\bar{\lambda}$.

(5) The plastic resistance, $N_{pl,Rd}$, of the diagonals' crude cross-section must be such that $N_{pl,Rd} \geq N_{Ed}$.

(6) In porticoes with V triangulations, the compressed diagonals must be dimensioned for compressive strength, in accordance with the Structural Code (Annexes 22 to 29).

(7) The connections of the diagonals to any other element must comply with the dimensioning rules of section **6.5.5**.

(8) In order to satisfy a homogeneous dissipative behaviour of the diagonals, it must be checked that the maximum resistance reserve, $\Omega_i$, defined in point **(1)** of section **6.7.4**, does not differ from the

minimum value,, by more than 25 %.

(9) Semi-rigid and/or partial resistance dissipative connections are permitted if the following conditions are met:

a) the connections have an elongation capacity consistent with global deformations;

b) the effect of deformation of connections on global displacement is taken into account using a nonlinear static global analysis (pushover) or a nonlinear analysis in the time domain.

### 6.7.4 Beams and pillars

(1) Beams and pillars under axillary forces must meet the following minimum strength requirements:

$$N_{\mathrm{pl,Rd}}(M_{\mathrm{Ed}}) \geq N_{\mathrm{Ed,G}} + 1,1\gamma_{\mathrm{ov}}\ \Omega . N_{\mathrm{Ed,E}} \tag{6.12}$$

where

$N_{\mathrm{pl,Rd}}$ ($M_{\mathrm{Ed}}$) is the calculation value of the buckling resistance of the beam or pillar in accordance with the Structural Code, taking into account the interaction of the buckling resistance with the bending moment $M_{\mathrm{Ed}}$, defined as its calculation value for the seismic calculation situation;

$N_{\mathrm{Ed,G}}$ is the axial force on the beam or in the pillar due to the non-seismic actions included in the combination of actions for the seismic calculation situation;

$N_{\mathrm{Ed,E}}$ is the axial force on the beam or on the pillar due to the seismic action calculation;

$\gamma_{\mathrm{ov}}$ is the resistance reserve coefficient (see point **(2)** of section **6.1.3** and point **(3)** of section **6.2**);

$\Omega$ is the minimum value of $\Omega_{\mathrm{i}} = N_{\mathrm{pl,Rd,i}}/N_{\mathrm{Ed,i}}$ over all diagonals of the portico system with triangulations, where:

$N_{\mathrm{pl,Rd,i}}$ is the calculation value of the diagonal resistance *i*;

$N_{\mathrm{Ed,i}}$ is the calculation value of the axial force in the same diagonal *i* in the seismic calculation situation.

(2) For porticoes with V-triangulations, the beams must be sized in such a way as to withstand:

- all non-seismic actions without considering intermediate supports given by diagonals;

- the vertical effect of the unbalanced seismic action applied to the beam by triangulations after the buckling of the diagonal in compression. This action effect is calculated using $N_{\mathrm{pl,Rd}}$ for traction triangulation and $\gamma_{\mathrm{pb}} N_{\mathrm{pl,Rd}}$ for bracing compression (triangulation). The value of $\gamma_{\mathrm{pb}}$ is 0,3

NB The coefficient $\gamma_{\mathrm{pb}}$ is used for estimating the post-bracing resistance of the diagonal compressions.

(3) In porticoes with diagonal triangulations in which the tensile and compressive diagonals do not intersect (e.g. diagonals in Figure 6.12), the dimensioning must take into account the tensile and

compressive forces developed on the pillars adjacent to the compressed diagonals and corresponding to the compressive forces on these diagonals, equal to the value of their bracing resistance.

## 6.8 Dimensioning rules and construction details for porticoes with off-centred triangulations

### 6.8.1 Dimensioning criteria

(1) Porches with off-centre triangulations should be sized so that the elements or parts of the specific elements called seismic couplings are capable of dissipating energy by forming plastic bending mechanisms or plastic shearing mechanisms.

(2) The structural system must be sized in such a way as to achieve a homogeneous dissipative behaviour of the entire set of seismic couplings.

NB The following rules are intended to ensure that plasticisation in couplings, including effects of deformation hardening on plastic hinges or shearing panels, must occur before laminating or breaking elsewhere.

(3) Seismic couplings can be horizontal or vertical elements (see Figure 6.4).

### 6.8.2 Seismic couplings

(1) The web of a coupling (seismic coupling tram) must be of constant thickness, without reinforcement plates and without holes or deliveries.

(2) Seismic couplings are classified into three categories, depending on the type of plastic mechanism developed:

- short couplings, dissipating energy by plasticisation, mainly by shearing;
- long couplings, dissipating energy by plasticisation, mainly by bending;
- intermediate couplings, in which the plastic mechanism involves bending and shearing.

(3) For sections in I, the following parameters are used to define the calculation values of resistances and category limits:

$$M_{p,link} = f_y b t_f (d - t_f) \tag{6.13}$$

$$V_{p,link} = (f_y / \sqrt{3}) t_w (d - t_f) \tag{6.14}$$

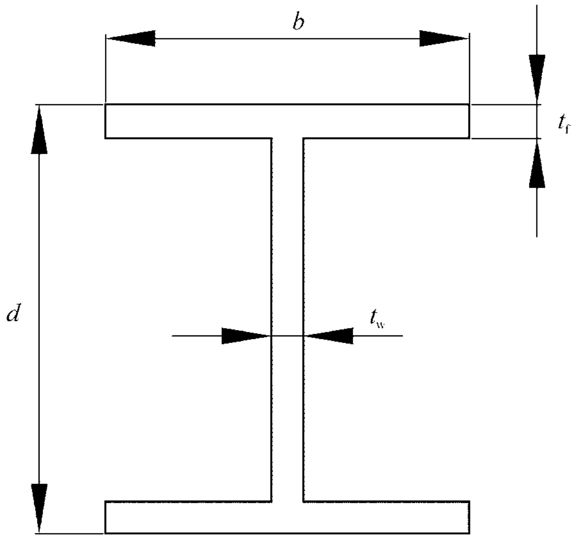



**Figure 6.13 - Definition of symbols for sections in I of couplings**

(4) Si $N_{Ed}/N_{pl,Rd} \leq 0.15$, the coupling strength calculation value must meet the following two conditions at the two ends of that coupling:

$$V_{Ed} \leq V_{p,link} \tag{6.15}$$

$$M_{Ed} \leq M_{p,link} \tag{6.16}$$

where

$N_{Ed}$, $M_{Ed}$, $V_{Ed}$ are the calculation values of the effects of the action, respectively, the calculation value of the axial force, the calculation value of the bending moment and the calculation value of the shearing, at the two ends of the coupling.

(5) If $N_{Ed}/N_{Rd} > 0.15$, equations (6.15) and (6.16) must be complied with the following reduced values, $V_{p,link,r}$ and $M_{p,link,r}$, used instead of $V_{p,link}$ and $M_{p,link}$:

$$V_{p,link,r} = V_{p,link} \left[ 1 - \left( N_{Ed} / N_{pl,Rd} \right)^2 \right]^{0,5} \tag{6.17}$$

$$M_{p,link,r} = M_{p,link} \left[ 1 - \left( N_{Ed} / N_{pl,Rd} \right) \right] \tag{6.18}$$

(6) If $N_{Ed}/N_{Rd} \geq 0,15$, the length $e$ of the coupling must not exceed:

$$e \leq 1,6\ M_{p,link} / V_{p,link} \quad \text{cuando } R < 0,3 \tag{6.19}$$

or

$$e \leq (1{,}15 - 0{,}5R)\ 1{,}6 M_{p,link} / V_{p,link} \quad \text{cuando } R \geq 0{,}3 \tag{6.20}$$

where $R = N_{Ed} \cdot t_w \cdot (d - t_f) / (V_{Ed} \cdot A)$, with $A$ being the gross area of the coupling.

(7) In order to achieve an overall dissipative behaviour of the structure, it must be checked that each of the values of the ratios $\Omega_i$ defined in point **(1)** of section **6.8.3** do not exceed the minimum value $\Omega$ resulting from point **(1)** of section **6.8.3** by more than 25 % of that minimum value.

(8) In the case where equal moments can appear simultaneously at both ends of the coupling [see Figure 6.14(a)], these couplings can be classified according to length $e$. For sections in I, the categories are:

- short couplings $e < e_s = 1.6\ M_{p,link} / V_{p,link}$ (6.21)
- long couplings $e > e_L = 3{,}0\ M_{p,link} / V_{p,link}$ (6.22)
- intermediate couplings $e_s < e < e_L$ (6.23)

(9) In the case where only one plastic hinge is formed at one end of the coupling (see Figure 6.14b), the value of the length $e$ defines the categories of these couplings. For sections in I, the categories are:

- short couplings $e \leq e_s = 0.8\ (1+\alpha)\ M_{p,link} / V_{p,link}$ (6.24)
- long couplings $e > e_L = 1.5\ (1+\alpha)\ M_{p,link} / V_{p,link}$ (6.25)
- intermediate couplings $e_s < e < e_L$ (6.26)

where $\alpha$ is the ratio between the minor of the bending moments at one end of the coupling in the seismic calculation situation, $M_{Ed,A}$, and the greater of the bending moments at the end where the plastic spherical, $M_{Ed,B}$, is taken in absolute values.

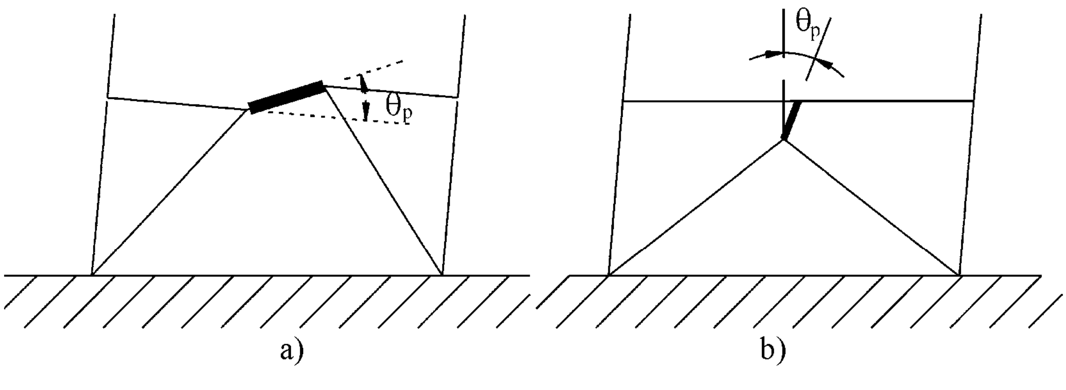



**Figure 6.14 - a) equal moments at the ends of the coupling;**
**b) different moments at the ends of the coupling**

(10) The rotation angle $\theta_p$ between the coupling and the element outside the coupling, as defined in point **(3)** of section **6.6.4**, must be consistent with the overall deformations. It must not exceed the following values:

- short couplings $\theta_p \leq \theta_{pR}$ = 0.08 radians (6.27)

- long couplings $\theta_p \leq \theta_{pR}$ = 0.02 radians (6.28)

- intermediate couplings $\theta_p \leq \theta_{pR}$ = value determined by linear interpolation of the above values (6.29)

(11) Complete web stiffeners must be arranged on both sides of the coupling web at the ends of the diagonal triangulations of the coupling. These stiffeners must have a combined width not less than ($b_f$ – $2t_w$) and a thickness not less than the greater value between 0.75 $t_w$ and 10 mm.

(12) Couplings must have intermediate web stiffeners, as follows:

a) short couplings must have a separation between intermediate web stiffeners not greater than ($30\ t_w - d/5$) for a rotation angle of the coupling $\theta_p$ of 0.08 radians, or ($52\ t_w - d/5$) for coupling rotation angles $\theta_p$ 0.02 radians or less. A linear interpolation must be used for values of $\theta_p$ between 0.08 and 0.02 radians;

b) long couplings must have an intermediate web stiffener, at a distance of 1.5 times *b,* measured from each end of the coupling where the plastic hinge would be formed;

c) intermediate couplings must have an intermediate web stiffener meeting the requirements of points (a) and (b) above;

d) no intermediate web stiffeners are required in length couplings $e$ greater than 5 $M_p/V_p$;

e) intermediate web stiffeners must be complete. For edge couplings $d$ less than 600 mm, stiffeners are required only on one side of the coupling web. The thickness of the one-sided stiffeners must not be less than the highest value between $t_w$ and 10 mm, and its width must be no less than $(b/2) - t_w$. For edge couplings $d$ equal to or greater than 600 mm, similar stiffeners must be placed on both sides of the web.

(13) Angle welds connecting an stiffener to the coupling web must have a suitable strength calculation value to withstand a stress of $\gamma_{ov} f_y A_{st}$, where $A_{st}$ is the area of the stiffener. The calculation value of the strength of the angle welds linking the stiffener to the wings must be adequate to withstand a force of $\gamma_{ov} f_y A_{st}/4$.

(14) Lateral supports must be arranged on the upper and lower wings of the coupling at its ends. The end side supports of couplings must have a calculation value of the axial resistance sufficient to provide lateral support for forces of 6 % of the expected nominal axial resistance of the coupling, calculated as $f_y\, b\, t_f$.

(15) In beams where there is a seismic coupling, the shear strength of the web panels outside the coupling must be checked in accordance with section **5** of Annex 25 to the Structural Code.

### 6.8.3 Elements not containing seismic couplings

(1) Elements not containing seismic couplings, such as pillars and diagonal elements when horizontal couplings are used in beams, and also beams when using vertical couplings must be checked with compression, considering the most unfavourable combination of axial stress and ending moments:

$$N_{Rd}(M_{Ed}, V_{Ed}) \geq N_{Ed,G} + 1,1\gamma_{ov}\ \Omega N_{Ed,E} \tag{6.30}$$

where

$N_{Rd}(M_{Ed},V_{Ed})$ is the calculation value of the axial resistance of the pillar or diagonal element according to the Structural Code, taking into account the interaction with the bending moment $M_{Ed}$ and the shearing moment $V_{Ed}$, taken equal to their calculation value for the seismic calculation situation;

$N_{Ed,G}$ is the compression force in the pillar or diagonal element due to the non-seismic actions included in the combination of actions for the seismic calculation situation;

$N_{Ed,E}$ is the compression stress on the pillar or diagonal element due to the seismic action calculation;

$\gamma_{ov}$ is the resistance reserve coefficient (see point **(2)** of section **6.1.3** and point **(3)** of section **6.2**);

$\Omega$ is a multiplier equal to the minimum of the following values:

The minimum value of $\Omega_i$ = 1,5 $V_{p,link,i}$ / $V_{Ed,i}$ between all short couplings;

The minimum value of $\Omega_i$ = 1,5 $M_{p,link,i}$ / $M_{Ed,i}$ between all intermediate and long couplings;

where

$V_{Ed,i}$, $M_{Ed,i}$ are the calculation values of the shear strength and the bending moment in the coupling *i* in the seismic calculation situation;

$V_{p,link,i}$, $M_{p,link,i}$ are the calculation values of the plastic shearing and bending resistances of the coupling *i*, as indicated in point **(3)** of section **6.8.2**.

### 6.8.4 Connections of seismic couplings

(1) If the structure is designed to dissipate energy in the seismic couplings, the connections of such couplings or of the elements containing them, must be dimensioned for action purposes, $E_d$, calculated as follows:

$$E_d \geq E_{d,G} + 1,1\gamma_{ov}\ \Omega_i E_{d,E} \tag{6.31}$$

where

$E_{d,G}$ is the effect of the action on the connection due to the non-seismic actions included in the combination of actions for the seismic calculation situation;

$E_{d,E}$ is the effect of the action on the connection due to the seismic action calculation;

$\gamma_{ov}$ is the resistance reserve coefficient (see point **(2)** of section **6.1.3** and point **(3)** of section **6.2**);

$\Omega_i$ is the resistance reserve coefficient for coupling, obtained according to point **(1)** of section **6.8.3**.

(2) In the case of semi-rigid connections or partial resistance, it can be assumed that energy dissipation occurs only in such connections. This is admissible if all of the following conditions are met:

a) the connections have sufficient rotational capacity for the corresponding deformation demands;

b) the elements converging in the connections have been shown to be stable in USL;

c) the effect of connection deformations on global displacement has been taken into account.

(3) Where partial resistance connections are used for seismic couplings, the capacity dimensioning of other elements of the structure must be obtained from the plastic capacity of the coupling connections.

## 6.9 Dimensioning rules for inverted pendulum structures

(1) In inverted pendulum structures (defined in point (d) of section **6.3.1**), the pillars must be checked under compression considering the most unfavourable combination of axial force and ending moments.

(2) In the checks, $N_{Ed}$, $M_{Ed}$, $V_{Ed}$ must be obtained as in section **6.6.3**.

(3) The dimensionless slenderness of the pillars must be limited to $\bar{\lambda} \leq 1.5$.

(4) The sensitivity coefficient of the relative floor collapse as defined in section **4.4.2.2** must be limited to $\theta \leq 0.20$.

## 6.10 Dimensioning rules for steel structures with concrete cores or walls and for bending-resistant porticoes combined with centred triangulations or fillers

### 6.10.1 Structures with concrete cores or walls

(1) Steel elements must be checked in accordance with this Chapter and the Structural Code (Annexes 22 to 29), while the concrete elements must be dimensioned according to chapter **5 of this Annex**.

(2) Elements in which there is interaction between steel and concrete must be checked in accordance with chapter **7**.

### 6.10.2 Bending-resistant porticoes combined with centred triangulations

(1) Dual structures with bending-resistant porticoes and porticoes with triangulations (arrangements) acting in the same direction must be dimensioned using a single coefficient *q*. Horizontal stresses must be distributed among the different porticoes according to their elastic rigidities.

(2) Bending resistant porticoes and triangulation porticoes must comply with sections **6.6**, **6.7** and **6.8**.

### 6.10.3 Bending-resistant porticoes combined with fillers

(1) Bend-resistant porticoes in which the filler walls are effectively connected to the steel structure must be dimensioned according to chapter **7**.

(2) Bend-resistant porticoes in which the filler walls are structurally disconnected from the steel portico on the side and top sides should be dimensioned as steel structures.

(3) Bend-resistant porticoes in which the filler walls are in contact with, but not effectively connected to, the steel structure must comply with the following rules:

a) filler walls must be evenly distributed in height so as not to locally increase the demand for ductility in the elements of the structure. If this is not verified, the building must be regarded as irregular in height;

b) the interaction between the portico and the filler must be taken into account. Internal forces on beams and pillars due to the action of diagonal cranks on fillers should also be taken into account. To do this, you can use the rules indicated in section **5.9**;

c) steel porticoes must be checked in accordance with the rules of this chapter, while reinforced concrete or brickwork filler walls must be sized in accordance with Annex 19 to the Structural Code and chapters **5** or **9 of this Annex**.

## 6.11 Project and construction control

(1) The control of the project and the construction must ensure that the actual structure corresponds to the one planned.

(2) To this end, in addition to the provisions indicated in the Structural Code (Annexes 22 to 29), the following requirements must be met:

a) the drawings made for the manufacture and construction must indicate the details of the connections, sizes and qualities of the screws and welds, as well as the steel types of the elements, indicating the maximum permissible elastic limit of the steel, $f_{y,max}$, to be used by the constructor in the dissipative areas;

b) the conformity of the materials with section **6.2** must be checked;

c) the control of tightening of the screws and the quality of the welds must follow the rules indicated in the Structural Code;

d) during construction it must be ensured that the actual elastic limit value of used steel does not exceed the value of $f_{y,max}$ for dissipative areas indicated on the planes by more than 10 %.

(3) If any of the above conditions are not met, the necessary corrections or justifications must be provided to meet the requirements of this Annex and to ensure the safety of the structure.

# 7 Specific rules for steel and concrete mixed-structure buildings

## 7.1 General considerations

### 7.1.1 Object and field of application

(1) For the project of buildings with a mixed structure of steel and concrete, the provisions of Annex 30 of the Structural Code apply. The following rules complement those set out in that regulation.

(2) The provisions of chapters **5** and **6 of this Annex** apply, except when modified by those indicated in this Chapter.

### 7.1.2 Dimensioning principles

(1) Earthquake-resistant buildings constructed with mixed structures should be designed according to one of the following principles (see Table 7.1):

- Principle (a) Minimally dissipative structural behaviour.
- Principle (b) Dissipative structural behaviour, with mixed dissipative areas.
- Principle (c) Dissipative structural behaviour, with steel dissipative areas.

**Table 7.1 - Dimensioning principles, structural ductility classes and upper limits of behaviour coefficient reference values**

| Dimensioning principle | Structural ductility class | Range of behaviour coefficient reference values $q$ |
|---|---|---|
| Principle (a)<br>Minimally dissipative structural behaviour | DCL (low) | $\leq 1.5$ |
| Principles (b) or (c)<br>Dissipative structural behaviour | DCM (medium) | $\leq 4$<br>Also limited by the values in Table 7.2 |
| | DCH (high) | Only limited by the values in Table 7.2 |

The upper-limit value of $q$ for minimally dissipative behaviour is 1.5.

No geographical limitations are established for the choice of dimensioning principles and DCM and DCH ductility classes for mixed structures.

(2) In principle (a) the effects of the action can be obtained by using an elastic analysis, without taking into account the nonlinear behaviour of the material, but considering the reduction of the inertia moment due to the cracking of the concrete in part of the beam beams, in accordance with the general rules for structural calculation defined in section **7.4** and the specific rules indicated in sections **7.7** to **7.11** depending on each type of structure. When using the calculation spectrum defined in section **3.2.2.5**, the upper limit of the reference value of the behaviour coefficient $q$ must be taken equal to 1.5 (see point **(1)** above). In case of irregularity in height, the upper limit of the performance coefficient value $q$ should be corrected as indicated in point **(7)** of section **4.2.3.1**, but it is not necessary to take a value less than 1.5.

(3) In principle (a) the strength of the elements and junctions must be assessed in accordance with the Structural Code (Annexes 22 to 32, concerning steel and mixed structures), without any additional requirements. For buildings that are not isolated at the base (see chapter **10**) dimensioning according to principle (a) is recommended only in cases of low seismicity (see point **(4)** of **3.2.1**).

(4) Principles (b) and (c) consider the ability of parts of the structure (dissipative areas) to resist seismic actions by inelastic behaviour. When using the calculation spectrum defined in **3.2.2.5**, the upper limit of the behaviour coefficient reference value $q$ must be taken with a value greater than the upper value set out in Table 7.1 for low dissipative structural behaviour. The upper-limit value of $q$ is function of ductility class and structure type (see **7.3**). When adopting principles (b) or (c) the requirements set out in sections **7.2** to **7.12**. must be met.

(5) In principle (c), structures are not supposed to benefit from mixed behaviour in dissipative areas. The application of principle (c) is conditioned by strict compliance with measures that prevent the participation of concrete in the strength of the dissipative areas. In principle (c) the mixed structure is dimensioned according to Annex 30 of the Structural Code for non-seismic actions and in accordance with chapter **6** of this Annex, to resist seismic action. The measures that prevent the participation of concrete are indicated in the section **7.7.5**.

(6) The dimensioning rules for mixed dissipative structures (principle b) aim at the development of reliable local plastic mechanisms (dissipative areas) and a global plastic mechanism that dissipates as much energy as possible under the seismic action calculation. For each structural element or type of structure considered in this chapter, the rules that allow achieving this objective are indicated in the sections **7.5** to **7.11**, with reference to the so-called particular criteria. These criteria aim to achieve the development of an overall mechanical behaviour for which dimensioning provisions can be established.

(7) Structures sized according to principle (b) must belong to the structural ductility classes DCM or DCH. These classes correspond to a greater capacity of the structure to dissipate energy through plastic mechanisms. A structure belonging to a given ductility class must meet specific requirements in one or more of the following: class of steel profiles, rotational capacity of connections and construction details.

### 7.1.3 Safety checks

(1) Point **(1)** of section **5.2.4** and point **(1)** of section **6.1.3** apply.

(2) Point **(2)** of section **5.2.4** applies.

(3) Point **(3)** of section **5.2.4** applies.

(4) In the capacity dimensioning checks relevant to the parts of the steel structure, the point **(3)** of section **6.2**.

## 7.2 Materials

### 7.2.1 Concrete

(1) In the dissipative areas, the characteristic strength $f_{ck}$ of the prescribed concrete must not be less than 25 N/mm$^2$, in accordance with Article 33.4 of the Structural Code. If the characteristic strength $f_{ck}$ of concrete is greater than 40 N/mm$^2$, the dimensioning is not within the field of application of this Annex.

### 7.2.2 Steel for passive reinforcement

(1) For the DCM ductility class, the steel for passive reinforcement considered in the plastic resistance of the dissipative areas must be of type S or SD, as defined in Articles 34 and 35 of the Structural Code. For the DCH ductility class, the steel for passive reinforcement considered in the plastic resistance of the dissipative areas must be SD type.

(2) In areas of non-dissipative structures subject to high stresses, type S or SD steel must be used (according to Article 34 of the Structural Code). This requirement applies to both round and electro-welded meshes.

(3) In areas with high stresses only corrugated rounds are allowed as steel for passive reinforcement, except for closed brackets or transverse states.

(4) Electro-welded meshes which do not meet the ductility criteria of point **(1)** above must not be used in dissipative areas. If such meshes are used, a ductile reinforcement must be arranged by doubling the mesh, and its resistant capacity must be considered in the capacity analysis.

### 7.2.3 Structural steel

(1) The requirements are those specified in section **6.2**.

## 7.3 Types of structures and behaviour coefficients

### 7.3.1 Types of structures

(1) Mixed steel and concrete structures should be classified within one of the following types of structures, depending on the behaviour of their primary resistant structure under seismic actions:

a) <u>Mixed bending-resistant porticoes</u>, are structures with the same definition and limitations as those indicated in **(1)**(a) of section **6.3.1**, but where beams and pillars may be either structural steel or mixed steel and concrete (see Figure 6.1);

b) <u>mixed porticoes with centred triangulations</u>, are structures with the same definition and limitations as those indicated in point **(1)**(b) of section **6.3.1** and Figures 6.2 and 6.3. Beams and pillars can be either structural steel, or mixed steel and concrete. Triangulations must be of structural steel;

c) <u>mixed porticoes with off-centre triangulations</u>, are structures with the same definition and limitations as those indicated in point **(1)**(c) of section **6.3.1** and in Figure 6.4. Elements not containing couplings may be either structural steel or mixed steel and concrete. The couplings must be of structural steel, except for those of the slab. Energy dissipation should occur only through bending or shearing plasticisation of such couplings;

d) <u>inverted pendulum structures</u>, are structures with the same definition and limitations as those indicated in point **(1)**(d) of section **6.3.1** (see Figure 6.5);

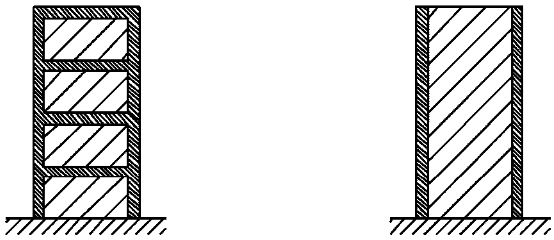
e) <u>mixed portico systems</u>, are structures that behave, basically, like reinforced concrete walls. Mixed structural systems may belong to one of the following types of structure:

- type 1 corresponds to a steel or mixed portico working alongside concrete fillers connected to the steel structure [see Figure 7.1(a)];

- type 2 is a reinforced concrete wall in which embedded steel profiles, connected to the concrete structure, are used as vertical edge reinforcement [see Figure 7.1(b)];

- type 3 is a system in which steel or mixed beams are used to couple two or more reinforced or mixed concrete walls (see Figure 7.2).

f) <u>steel-plate mixed shearing walls</u>, are structures consisting of a continuous vertical steel plate along the entire height of the building, embedded in reinforced concrete on one or both sides of the plate, and with structural or mixed steel edge elements.



a) b)

**Figure 7.1 - Mixed structural systems. Mixed walls:**

**a) type 1 - steel structure or bend-resistant mixed porticoes with connected concrete fill panels;**

**b) type 2 - mixed walls reinforced with embedded vertical steel profiles**

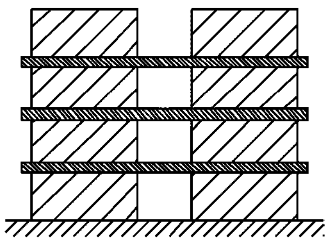


**Figure 7.2 - Mixed structural systems.**
**Type 3 - concrete or mixed walls coupled by steel or mixed beams**

(2) In all types of mixed structural systems energy dissipation occurs in vertical steel profiles and vertical wall reinforcements. In type 3 mixed structural systems, energy dissipation occurs also in coupling beams.

(3) If in mixed structural systems the wall elements are not connected to the steel structure, chapters **5** and **6** are applied.

### 7.3.2 Behaviour coefficients

(1) The behaviour coefficient $q$, defined in the section **3.2.2.5**, considers the energy dissipation capacity of the structure. For regular structural systems, performance coefficients $q$ must be taken

considering the upper limit of the reference value indicated in Table 6.2 or Table 7.2, provided that the rules of sections **7.5** to **7.11** are complied with.

**Table 7.2 - Upper limits of performance coefficient reference values for regular height systems**

| STRUCTURAL TYPOLOGY | Ductility class | |
|---|---|---|
| | DCM | DCH |
| (a), (b), (c) and (d) | See Table 6.2 | |
| e) Mixed structural systems | 3 $\alpha_u/\alpha_1$ | 4 $\alpha_u/\alpha_1$ |
| Mixed walls (type 1 and type 2)<br>Concrete or mixed walls coupled by steel beams or mixed beams (type 3) | 3 $\alpha_u/\alpha_1$ | 4.5 $\alpha_u/\alpha_1$ |
| f) Steel-plate mixed shearing walls | 3 $\alpha_u/\alpha_1$ | 4 $\alpha_u/\alpha_1$ |

(2) If the building is irregular in height (see **4.2.3.3**), the values of $q$ listed in Table 6.2 and Table 7.2 must be reduced by 20 % (see point (7) of section **4.2.3.1** and Table 4.1).

(3) For regular buildings on the floor, if no specific calculations are carried out to assess $\alpha_u/\alpha_1$ (see point **(3)** of section **6.3.2**), the approximate default values of the ratio $\alpha_u/\alpha_1$ presented in Figures 6.1 to 6.8 can be used. For mixed structural systems, the default value can be taken as $\alpha_u/\alpha_1$ = 1.1. In the case of mixed steel plate shearing walls, the default value can be taken as $\alpha_u/\alpha_1$ = 1,2.

(4) For non-regular storeyed buildings (see **4.2.3.2**), the approximate value of $\alpha_u/\alpha_1$ which can be used when no explicit calculations are performed for evaluation is equal to the mean between: (a) 1.0 and (b) the value indicated in point **(3)** above.

(5) Values of u / 1 $\alpha_u/\alpha_1$ higher than those indicated in points **(3)** and **(4)** of this section are allowed, assuming they have been calculated by a nonlinear static global analysis (pushover).

(6) The maximum value of $\alpha_u/\alpha_1$ that can be used in the calculation is 1.6, even if the analysis mentioned in **(5)** above indicates higher potential values.

## 7.4 Structural analysis

### 7.4.1 Object and field of application

(1) The following rules apply to the dimensioning of the structure under seismic action by the method of lateral force and spectral modal analysis.

### 7.4.2 Rigidity of the sections

(1) The rigidity of the mixed sections in which the concrete is compressed must be obtained using an equivalence coefficient, $n$

$$n = E_a / E_{cm} = 7 \tag{7.1}$$

(2) For mixed beams with compressed slab, the inertia of the section, $I_1$, must be obtained taking

into account the effective width of the slab defined in section **7.6.3**.

(3) The rigidity of the mixed sections in which the concrete is drawn must be obtained on the assumption that the concrete is cracked, and that only the steel parts of the section are active.

(4) For mixed beams with drawn slab, the inertia of the section, $I_2$, must be obtained taking into account the effective width of the slab defined in section **7.6.3**.

(5) The structure must be analysed taking into account the presence of compression concrete in certain areas and concrete traction in others; the distribution of these areas is indicated in the sections **7.7** to **7.11** for the different types of structures.

## 7.5 Dimensioning criteria and construction detail rules for dissipative structural behaviour, common to all types of structures

### 7.5.1 General considerations

(1) The dimensioning criteria indicated in section **7.5.2** must be applied to the earthquake-resistant parts of the projected structures following the principle of a dissipative structural behaviour.

(2) The dimensioning criteria indicated in **7.5.2** are considered complied with if the rules indicated in **7.5.3**, **7.5.4** and **7.6** to **7.11**. are complied with.

### 7.5.2 Dimensioning criteria for dissipative structures

(1) Structures with dissipative areas should be sized in such a way that plasticisation, local buckling or any other phenomenon due to hysteresis behaviour in those areas does not affect the overall stability of the structure.

NB The coefficients $q$ indicated in Table 7.2 are considered to comply with this requirement (see point **(2)** of section **2.2.2**).

(2) Dissipative areas must have adequate ductility and resistance. Resistance must be determined in accordance with the Structural Code (Annexes 22 to 29) and Chapter 6 for principle (c) (see **7.1.2**) and according to Annex 30 to the Structural Code and Chapter **7** for principle (b) (see **7.1.2**). Ductility is achieved by complying with the rules of construction details.

(3) Dissipative areas can be located in structural elements or connections.

(4) If the dissipative areas are located in the structural elements, the non-dissipative parts and the connections between the dissipative areas and the rest of the structure must have a sufficient reserve of strength to allow the development of cyclic plasticisation in those dissipative parts.

(5) When the dissipative areas are located in the connections, the connected elements must have a sufficient reserve of resistance to allow the development of cyclic plasticisation in those connections.

### 7.5.3 Plastic resistance of dissipative areas

(1) In the dimensioning of mixed steel and concrete structures, two plastic resistances are used for dissipative areas: a lower-limit plastic resistance (subscript: *pl*, *Rd*) and an upper-limit plastic resistance (subscript: *U, Rd*).

(2) The lower-limit plastic resistance of the dissipative areas is that used in the dimensioning checks

relating to the sections of the dissipative elements; for example, $M_{Ed} < M_{pl,Rd}$. The lower-limit plastic strength of the dissipative areas is obtained taking into account the concrete of the section and only those steel components of that section classified as ductile.

(3) The upper-limit plastic resistance of the dissipative areas is used in the dimensioning by capacity of the elements adjacent to the dissipative area: for example, in checking the capacity dimensioning of point **(4)** of section **4.4.2.3**; the calculation values of the resistant moments of the beams are the upper-limit plastic resistances, $M_{U,Rd,b}$, while those of the pillars are the lower-limit plastic strengths, $M_{pl,Rd,c}$.

(4) The upper-limit plastic strength is obtained taking into account the concrete component and all the steel components present in the section, including those that are not classified as ductile.

(5) The effects of the action directly related to the resistance of the dissipative areas should be determined on the basis of the upper limit strength of the mixed dissipative sections; for example, the calculation value of the shear strength at the end of a mixed dissipative beam should be determined based on the upper-limit plastic moment of the mixed section.

### 7.5.4 Construction detail rules for mixed connections in dissipative areas

(1) Dimensioning should limit the location of plastic deformations and high residual stresses, as well as prevent construction defects.

(2) The integrity of the concrete in compression should be maintained during the earthquake, and the plasticisation should be limited to the steel profiles.

(3) The plasticisation of the reinforcements on a slab must only be allowed if the beams are dimensioned according to point **(8)** of section **7.6.2**.

(4) Section **6.5** is applied for the dimensioning of welds and screws.

(5) The local dimensioning of the required reinforcements in the concrete of the junction area must be justified by models that satisfy the balance (e.g. Appendix C for slabs).

(6) The points **(6)** and **(7)** of section **6.5.5.**

(7) In the fully embedded web panels of the beam-pillar connections, the strength of the panel area can be obtained as the sum of the concrete and steel panel contributions, if the following conditions are met:

a) the aspect ratio $h_b/h_c$ of the panel area is:

$$0,6 < h_b / h_c < 1,4 \quad (7.2)$$

b)

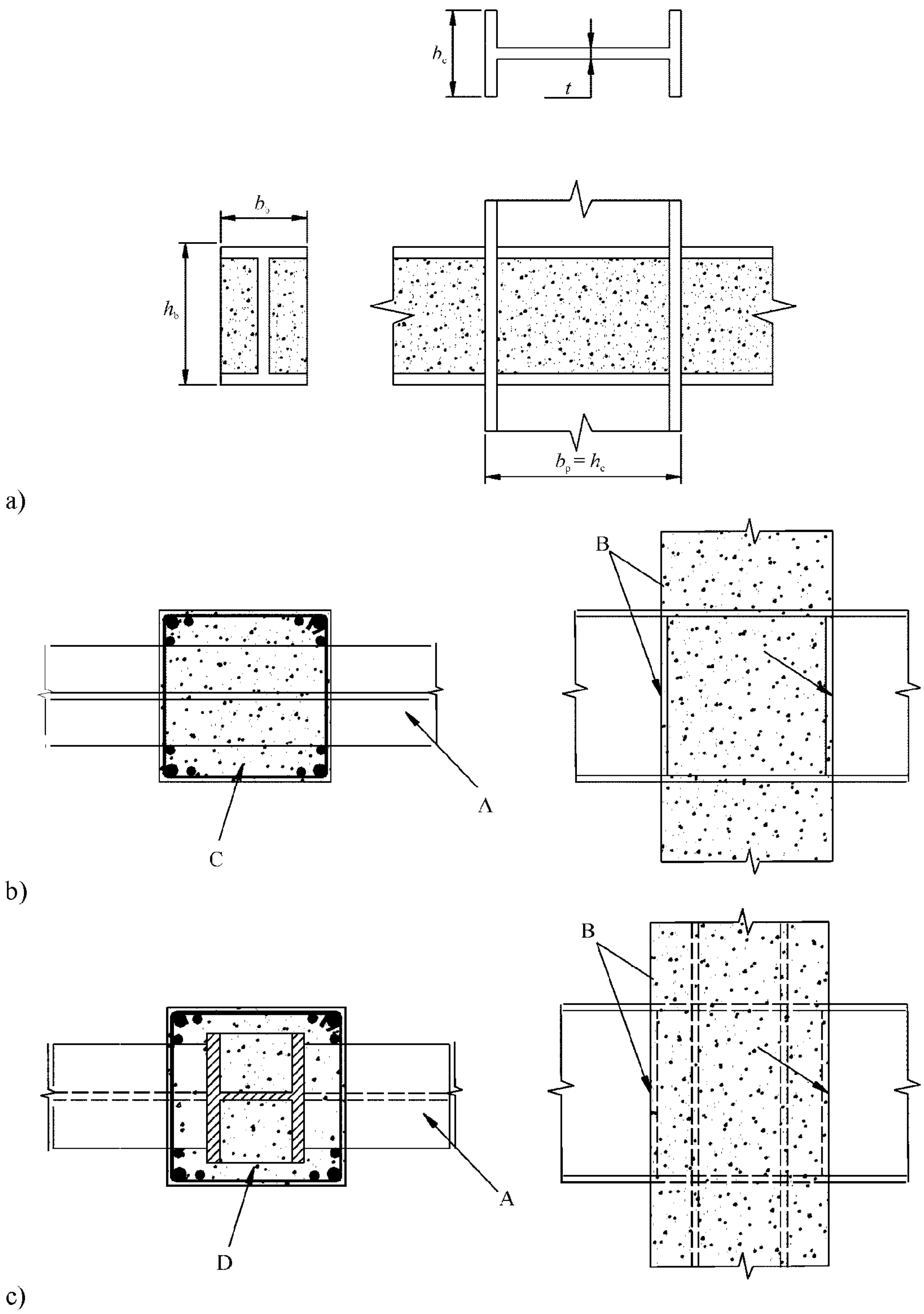
$$V_{wp,Ed} < 0,8V_{wp,Rd} \quad (7.3)$$

where

$V_{wp,Ed}$ is the calculation value of the shear strength on the panel due to the effects of the action, taking into account the plastic resistance of the adjacent mixed dissipative areas in beams or connections;

$V_{wp,Rd}$ is the shearing strength of mixed steel and concrete panel in accordance with Annex 30 to the Structural Code;

$h_b$, $h_c$ are defined in Figure 7.3a).




Legend

A Steel beam

B Face-support plates

C Reinforced concrete pillar

D Embedded mixed pillar

**Figure 7.3 – Connections between beam and pillar**

(8) In partially embedded stiffened panels, an assessment similar to that of point **(7)** above is permitted if, in addition to the requirements of point **(9)**, one of the following conditions is met:

a) in the partially embedded stiffened panel, straight rounds of the type defined in **(4)** of **7.6.5** and complying with points **(5)** and **(6)** of section **7.6.5** with maximum separation $s_1 = c$; these couplings are oriented perpendicularly to the longest side of the panel and no other panel reinforcement is required; or

b) no reinforcement, as long as $h_b/b_b < 1.2$ and $h_c/b_c < 1,2$

where $h_b$, $b_b$, $b_c$ and $h_c$ are those defined in Figure 7.3.a).

(9) Where a steel or mixed dissipative beam is connected to a reinforced concrete pillar, as shown in Figure 7.3b), a vertical reinforcement for the pillar must be provided with a calculation value of the axial resistance at least equal to the shearing strength of the coupling beam, near the stiffener or support plate of the faces adjacent to the dissipative area. The use of vertical reinforcement intended for other purposes is permitted as part of the required vertical reinforcement. The presence of face support plates or plates is required, which must be complete stiffeners with a combined width not less than ($b_b$ – 2 $t$); its thickness must not be less than 0.75$t$ or 8 mm; $b_b$ and $t$ are, respectively, the width of the beam wing and the panel thickness (see Figure 7.3).

(10) When a steel or mixed dissipative beam is connected fully embedded to a mixed pillar, as shown in Figure 7.3(c), the beam-pillar connection may be dimensioned either as a connection between beam and steel pillar or as a connection between beam and mixed pillar. In the latter case, the vertical reinforcements of the pillar can be sized according to the point **(9)** above, or by distributing the shearing strength of the beam between the steel profile of the pillar and the reinforcement of that pillar. In both cases, support plates of faces are required, as described in point **(9)**.

(11) The vertical reinforcement of the pillar specified in points **(9)** and **(10)** above must be confined by a transverse reinforcement that meets the requirements for the elements defined in section **7.6**.

## 7.6 Rules for elements

### 7.6.1 General considerations

(1) Mixed elements that are primary seismic elements must comply with Annex 30 of the Structural Code and with the additional rules indicated in this Chapter 7.

(2) The earthquake-resistant structure is dimensioned as a global plastic mechanism that includes local dissipative areas; this global mechanism identifies the elements in which dissipative areas are located and, indirectly, elements without dissipative areas.

(3) In the case of tensile elements or parts thereof, the ductility requirement set out in point **(3)** of section **6.2.3** of Annex 22 to the Structural Code must be met.

(4) Sufficient local ductility of the elements dissipating energy under compression or bending must be ensured, limiting the ratios between the width and thickness of its walls. Steel dissipative areas and non-embedded steel parts of mixed elements must meet the requirements of point **(1)** of section **6.5.3** and Table 6.3. The dissipative areas of embedded mixed elements must comply with the requirements of Table 7.3. The limits given for the in-flight wings of the fully or partially embedded elements can be smoothed if special construction details are provided, as described in point **(9)** of section **7.6.4** and in

points **(4)** to **(6)** of section **7.6.5**.

**Table 7.3 - Relationship between the coefficient of behaviour and the slenderness limits of the profile plates in the dissipative areas of the composite mixed structures**

| **Structure ductility class** | **DCM** | | **DCH** |
|---|---|---|---|
| behaviour coefficient reference value ($q$) | $q \leq 1.5 - 2$ | $1.5 - 2 < q < 4$ | $q > 4$ |
| Section in H or I partially embedded<br>Section in H or I fully embedded<br>Limits $c/t_f$ for wings in flight: | $20\ \varepsilon$ | $14\ \varepsilon$ | $9\ \varepsilon$ |
| Rectangular section filled<br>Limits $h/t$: | $52\ \varepsilon$ | $38\ \varepsilon$ | $24\ \varepsilon$ |
| Circular section filled<br>Limits $d/t$: | $90\ \varepsilon^2$ | $85\ \varepsilon^2$ | $80\ \varepsilon^2$ |

where

$\varepsilon =$ $(f_y/235)^{0.5}$;
$c/t_f$ is defined in Figure 7.8;
$d/t$ and $h/t$ are the relationships between the maximum external dimension and the thickness of the steel profile wall.

(5) In sections **7.6.2**, **7.6.4**, **7.6.5** and **7.6.6**, more specific construction details rules for mixed elements are given.

(6) In the dimensioning of any type of mixed pillar, either only the strength of the steel profile or the combined strengths of the steel profile and the enclosing concrete or concrete filler can be taken into account.

(7) The dimensioning of pillars in which it is assumed that the strength of the elements comes only from the steel profile can be carried out in accordance with the provisions of chapter **6**. In the case of dissipative pillars, the capacity dimensioning rules of points **(4)** and **(5)** of section **7.5.2** and **(3)** of section **7.5.3**.

(8) For fully embedded pillars, with mixed behaviour, the minimum cross section dimensions $b$, $h$ or $d$ must not be less than 250 mm.

(9) The strength of non-dissipative mixed pillars, including shear strength, must be determined in accordance with the rules indicated in Annex 30 of the Structural Code.

(10) For pillars, when wrapping concrete or concrete filler is supposed to contribute to the axial resistance or bending of the element, the dimensioning rules of sections **7.6.4** to **7.6.6** apply. These rules ensure a total transfer of the shearing between the concrete and the steel parts of a section, and protect the dissipative areas against premature inelastic breakage.

(11) For earthquake-resistant dimensioning, the calculation value of the shear strength given in Table 6.6 of Annex 30 to the Structural Code must be multiplied by a reduction coefficient of 0.5.

(12) Where, for reasons of capacity dimensioning, the total composite strength of a pillar is used, a total transfer of the shearing between the steel and the reinforced concrete parts must be ensured. If the transfer of the shearing through adhesion and friction is insufficient, shearing connectors must be arranged to ensure total composite action.

(13) Where a pillar is subjected mainly to axillary forces, a sufficient shearing transfer must be provided to ensure that the concrete and steel parts share the loads applied to the pillar in the connections to the beams and the bracing elements (triangulations).

(14) The pillars are not usually dimensioned to be dissipative, except at their base in some types of structures. However, due to uncertainties in behaviour, confinement reinforcement is required in critical areas as specified in section **7.6.4**.

(15) Sections **5.6.2.1** and **5.6.3**, relating to anchoring and splices in the calculation of reinforced concrete pillars, also apply to the reinforcements of the mixed pillars.

### 7.6.2 Mixed steel beams with a slab

(1) The objective of dimensioning in this section is to maintain the integrity of the concrete slab during the seismic event, while plasticising occurs at the bottom of the steel profile or in the slab reinforcements.

(2) If it is not intended to benefit from the mixed character of the beam section for energy dissipation, section **7.7.5** should be applied.

(3) Beams designed to behave as mixed elements in dissipative areas of the earthquake-resistant structure must be calculated for a total or partial shearing connection, in accordance with Annex 30 of the Structural Code. The minimum degree of connection, $\eta$, as defined in section **6.6.1.2** of that Annex, must not be less than 0.8, and the total strength of the shearing connectors within any negative moment region must not be less than the plastic strength of the reinforcement.

(4) The calculation value of the strength of the connectors in dissipative areas is obtained from the calculation value of the resistance indicated in Annex 30 of the Structural Code, multiplied by a reduction coefficient of 0.75.

(5) A full shearing connection is required when using non-ductile connectors.

(6) When using a ribbed steel sheet with the nerves transverse to the supporting beams, the reduction coefficient $k_t$ of the calculation value of the shear strength of the connectors, as indicated in Annex 30 of the Structural Code, must be further reduced by multiplying it by the efficiency coefficient for ribbed sheet $k_r$ indicated in Figure 7.4.

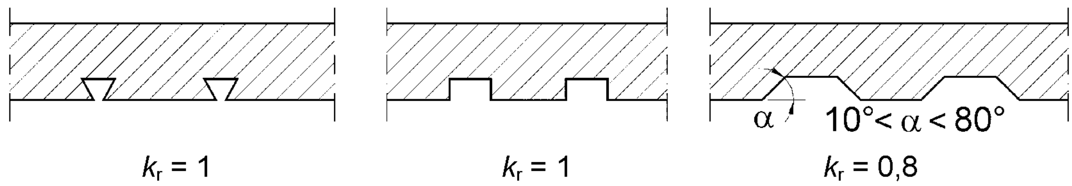



**Figure 7.4 – Values of the coefficient of efficiency for ribbed sheet**

(7) To achieve ductility in plastic hinges, the ratio $x/d$ of the distance $x$ between the upper

compression fibre of concrete and the plastic neutral fibre, and the *d* edge of the mixed section, must comply with the following equation:

$$x / d < \varepsilon_{cu2} / (\varepsilon_{cu2} + \varepsilon_{a}) \quad (7.4)$$

where

$\varepsilon_{cu2}$ is the ultimate unit deformation of non-confined concrete (see Annex 19 to the Structural Code);

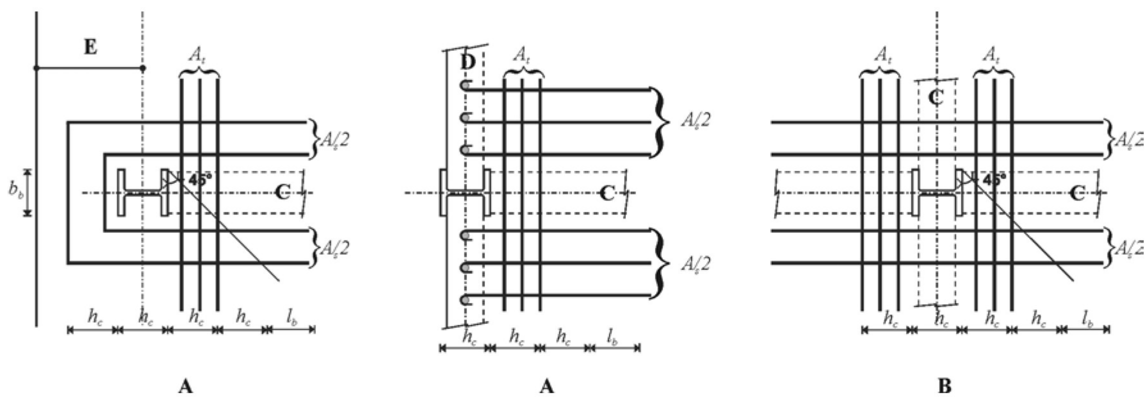
$\varepsilon_{a}$ is the total unit deformation corresponding to the ultimate limit state.

(8) The rule indicated in point **(7)** above is considered complied with when the ratio *x*/*d* of a section is less than the limits indicated in Table 7.4.

**Table 7.4 - Limit values of *x*/*d* for beam ductility with slab**

| **Ductility class** | ***q*** | **$f_y$ (N/mm$^2$)** | **Upper limit *x*/*d*** |
|---|---|---|---|
| DCM | $1.5 < q \leq 4$ | 355 | 0.27 |
| | $1.5 < q \leq 4$ | 235 | 0.36 |
| DCH | $q > 4$ | 355 | 0.20 |
| | $q > 4$ | 235 | 0.27 |

(9) The dissipative areas of the beams must be fitted with ductile steel reinforcements for the slab, known as 'seismic reinforcement' (see Figure 7.5), specifically arranged, in the area of the joining of the beam and the pillar. Its dimensioning and the symbols used in Figure 7.5 are specified in Appendix C.




Legend

A Outside knot

B Inner knot

C Steel beam

D Façade steel beam

E Cantilevered reinforced concrete band

**Figure 7.5 – Configuration of 'Seismic Armours'**

### 7.6.3 Effective slab width

(1) The total effective width $b_{eff}$ of the concrete wing associated with each steel web must be taken as the sum of the partial effective widths $b_{e1}$ and $b_{e2}$ of the existing wing portion on each side of the web axis (Figure 7.6). These effective partial widths on each side must be taken as the $b_e$ indicated in Table 7.5, but no greater than the actual available widths $b_1$ and $b_2$ defined in point **(2)** below.

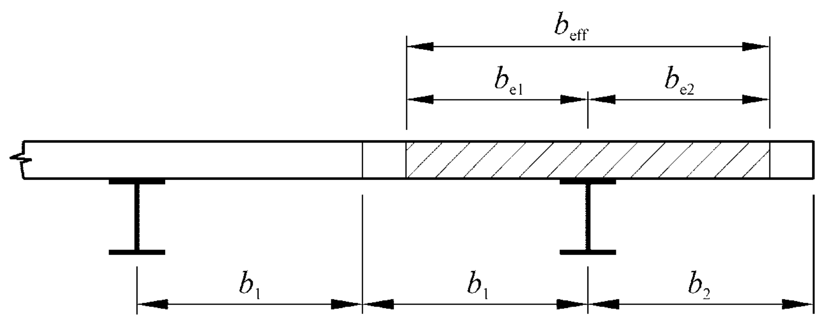
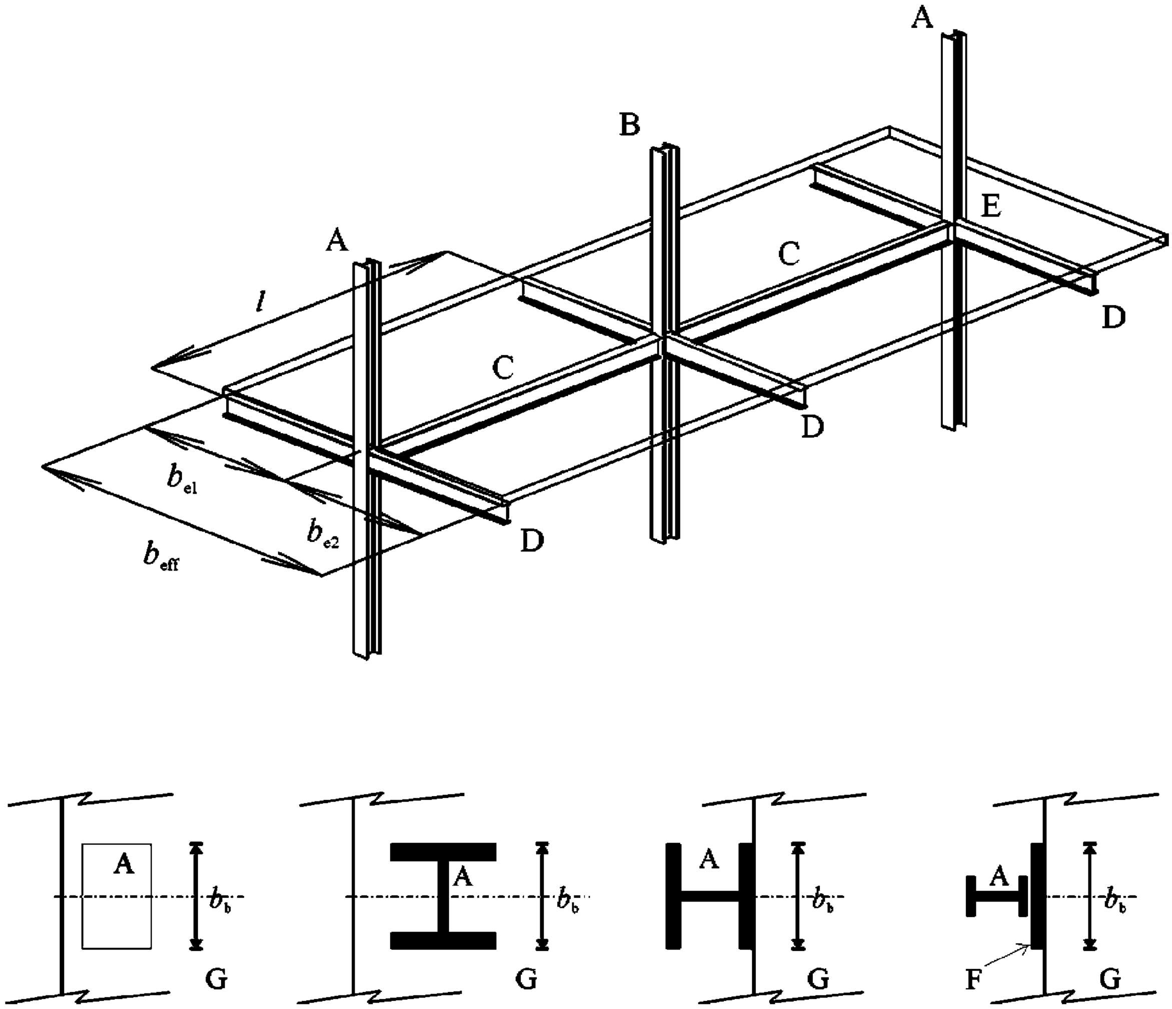



**Figure 7.6 – Definition of effective widths $b_e$ and $b_{eff}$**

(2) The actual width $b$ of each portion must be taken as half the distance from the web to the adjacent web, unless there is a free edge, in which case the actual width is the distance from the web to that free edge.

(3) The effective partial width $b_e$ of the slab used in determining the elastic and plastic properties of the mixed sections in $T$ composed of a steel section connected to that slab is defined in Table 7.5 and Figure 7.7. These values are valid for beams in position C of Figure 7.5 if the dimensioning of the slab reinforcement and the connection of the slab to the steel beams are in accordance with Appendix C. In Table 7.5, those moments that induce compression on the slab are considered positive, while those that induce traction are considered negative. The symbols $b_b$, $h_c$, $b_e$, $b_{eff}$ and $l_e$ used in tables 7.5 I and 7.5 II are defined in Figures 7.5, 7.6 and 7.7. $b_b$ is the supporting width of the slab concrete on the pillar in the horizontal direction perpendicular to the beam for which the effective width is calculated; such support width may include additional plates or devices aimed at increasing support capacity.




Legend

A External pillar
B Internal pillar
C Longitudinal beam
D Transverse beam or steel façade beam
E Concrete cantilever
F Extended support
G Concrete slab

**Figure 7.7 – Definition of elements in bending-resistant portico structures**

**Table 7.5 I - Partial effective width $b_e$ for elastic analysis of structure**

| $b_e$ | Cross-shearing element | $b_e$ for $M_{Rd}$ (Elastic) |
|---|---|---|
| On internal pillar | Present or not present | For negative M: 0.005 $l$<br>For positive M: 0.0375 $l$ |
| On external pillar | Present | |
| On external pillar | Not present or reinforcement not anchored | For negative M: 0<br>For positive M: 0.025 $l$ |

**Table 7.5 II - Partial effective width $b_e$ slab for resistant plastic moment evaluation**

| Sign of the bending moment M | Placement | Cross-shearing element | $b_e$ for $M_{Rd}$ (Plastic) |
|---|---|---|---|
| Negative M | Internal pillar | Seismic reinforcement | $0.1\ l$ |
| Negative M | External pillar | All configurations with the reinforcement anchored to the façade beam or to the cantilever concrete band | $0.1\ l$ |
| Negative M | External pillar | All configurations with the reinforcements not anchored to the façade beam or to the cantilever concrete band | 0.0 |
| Positive M | Internal pillar | Seismic reinforcement | $0.075\ l$ |
| Positive M | External pillar | Steel transverse beam with connectors. Concrete slab to the outer face of a pillar with an H-section, with the main axis oriented according to Figure 7.5 or extending beyond (concrete edge band). Seismic reinforcement | $0.075\ l$ |
| Positive M | External pillar | No steel transverse beam or steel transverse beam with connectors. Concrete slab to the outer face and a pillar with an H section, with the main axis oriented according to Figure 7.5 or extending beyond (edge band).<br>Seismic reinforcement | $b_b\ /2 + 0.7\ h_c\ /2$ |
| Positive M | External pillar | Other configurations. Seismic reinforcement | $b_b\ /2 \leq b_{e,max.}$<br>$b_{e,max.} = 0.05\ l$ |

### 7.6.4 Fully embedded mixed pillars

(1) In dissipative structures, there are critical areas at both ends of all pillar-free lengths, in bending-resistant portico structures, and in the portion of the pillars adjacent to the couplings, in porticoes with off-centre triangulations. The lengths $l_{cr}$ of these critical areas (in meters) are specified in equation (5.14) for the DCM ductility class, or in equation (5.30) for the DCH ductility class, where $h_c$ in those equations represents the edge of the mixed section (in meters).

(2) In order to meet the demands of plastic rotation and to compensate for the loss of strength due to the loss of coating (concrete) loss of concrete, the following equation must be met within the critical areas defined above:

$$\alpha\, w_{wd} \geq 30 \cdot m_f \cdot \nu_d \cdot \varepsilon_{sy,d} \cdot \frac{b_c}{b_o} - 0{,}035 \qquad (7.5)$$

in which the variables are those defined in point **(8)** of section **5.4.3.2.2** and the calculation value of reduced axial force, $\nu_d$, is defined as:

$$\nu_d = N_{Ed} / N_{pl,Rd} = N_{Ed} / \left( A_a f_{yd} + A_c f_{cd} + A_s f_{sd} \right) \qquad (7.6)$$

where

$A_a$ is the area of the steel profile;

$A_c$ is the concrete area;

$A_s$ is the area of reinforcement;

$f_{yd}$ is the calculation value of the elastic limit of steel;

$f_{cd}$ is the calculation value of the compressive strength of concrete;

$f_{sd}$ is the calculation value of the elastic limit of the steel of the reinforcements.

(3) The separation $s$ (in millimetres) of the confinement brackets in critical areas must not exceed:

$s$ = min. ($b_o$ /2, 260, 9 $d_{bL}$) for DCM ductility class (7.7)

$s$ = min. ($b_o$ /2, 175, 8 $d_{bL}$) for DCH ductility class (7.8)

or at the bottom of the lower floor, for the ductility class DCH

$s$ = min. ($b_o$ /2, 150, 6 $d_{bL}$) for DCM ductility class (7.9)

where

$b_o$ is the minimum dimension of the concrete core (up to the axis of the fences, in millimetres);

$d_{bL}$ is the minimum diameter of the longitudinal reinforcements (in millimetres).

(4) The diameter of the fences, $d_{bw}$ (in millimetres) must be at least:

$d_{bw}$ = 6 for DCM ductility class (7.10)

$d_{bw}$ = max. (0.35 $d_{bL,max.}$ $[f_{ydL} / f_{ydw}]^{0.5}$, 6) for the DHC ductility class (7.11)

where

$d_{bL,max.}$ is the maximum diameter of the longitudinal reinforcements (in millimetres).

(5) In critical areas, the distance between the longitudinal reinforcements braced by coated pins or transverse states must not exceed 250 mm for the DCM ductility class or 200 mm for the ductility class DCH.

(6) On the two lower floors of a building, fences must be arranged in accordance with points **(3), (4)** and **(5)**, beyond the critical areas, for an additional length equal to half the length of those critical areas.

(7) For mixed dissipative pillars, the shearing strength must be determined solely on the basis of the structural steel profile.

(8) The ratio between structure ductility class and permissible slenderness ($c/t_f$) for wings in flight

in dissipative areas is indicated in Table 7.3.

(9) Confinement fences can delay local tampering in dissipative areas. The limits for wing slenderness indicated in Table 7.3 may be increased if the fences are arranged with a longitudinal separation, $s$, less than the wing flight, c, i.e.: $s/c < 1.0$. For $s/c$ 0.5, the limits indicated in Table 7.3 can be increased by up to 50 %. For values $0.5 < s/c < 1.0$, linear interpolation can be used.

(10) The diameter $d_{bw}$ of confinement fences used to prevent wing buckling must not be less than

$$d_{bw} = \left[ \left( b \cdot t_f / 8 \right) \left( f_{ydf} / f_{ydw} \right) \right]^{0,5} \tag{7.12}$$

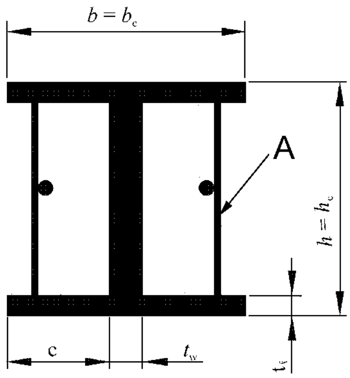
where $b$ and $t_f$ are the edge and thickness of the wing, respectively, and $f_{ydf}$ and $f_{ydw}$ are the calculation values of the wing and reinforcement resistances, respectively.

### 7.6.5 Partially embedded elements

(1) In dispensing areas where the energy is dissipated through plastic bending of a mixed section, the longitudinal separation of the transverse reinforcement, $s$, must meet the requirements of point **(3)** of section **7.6.4** along a length greater than or equal to $l_{cr}$ for dissipative areas at the ends of an element or 2 $l_{cr}$ for dissipative areas in the element.

(2) For the dissipative elements, the shear strength must be determined solely on the basis of the structural steel section, unless special construction details are provided to mobilise the shearing strength of the concrete section.

(3) The ratio between structure ductility class and permissible slenderness ($c/t_f$) for wings in flight in dissipative areas is indicated in Table 7.3.




Legend

A Additional straight rounds (couplings)

**Figure 7.8 – Detail of transverse assembly, with additional straight rounds (couplings) welded to the wings**

(4) Round straight soldiers inside the wings, as shown in Figure 7.8, in addition to the reinforcement required in Annex 30 to the Structural Code, may delay local buckling in the dissipative areas. In this case, the limits indicated in Table 7.3 for wing slenderness can be increased if the reinforcements are

arranged with a longitudinal separation, $s_1$, less than the distances exceeding the wing, c, i.e.: $s_1/c < 1.0$. Para $s_1/c < 0.5$, the limits indicated in Table 7.3 can be increased by up to 50 %. For values $0.5 < s_1/c < 1.0$, linear interpolation can be used.

Additional straight rounds (couplings) must also comply with the rules of the points **(5)** and **(6)** below.

(5) The diameter, $d_{bw}$, of the additional straight rounds referred to in point ( **4)** above, must be at least 6 mm. When transverse straight rounds are used to delay local wing buckling, as also described in point **(4)**, $d_{bw}$ must not be less than the value obtained in equation (7.12).

(6) The additional straight rounds referred to in point **(4)** above must be welded at both ends of the wings, and the weld capacity must not be less than the plastic tensile strength of the straight rounds. A concrete-free coating of at least 20 mm but not exceeding 40 mm must be provided for these rounds.

(7) The dimensioning of partially embedded mixed elements can take into account the strength of the structural steel profile only, or the composite strength of the steel section together with the wrapping concrete.

(8) The dimensioning of partially embedded mixed elements in which only the structural steel section is assumed to contribute to the strength of the element can be carried out in accordance with the provisions of chapter **6**, but the capacity dimensioning provisions indicated in points **(4)** and **(5)** of section **7.5.2** and **(3)** of section **7.5.3**.

### 7.6.6 Filled mixed pillars

(1) The ratio between structure ductility class and permissible slenderness $d/t$ or $h/t$ is indicated in Table 7.3.

(2) The shearing strength of dissipative pillars must be determined by reference to the structural steel section or according to the section of the reinforced concrete, with the hollow steel section considered exclusively as shearing.

(3) In the case of non-dissipative elements, the shear strength of the pillar must be determined in accordance with Annex 30 to the Structural Code.

## 7.7 Dimensioning and construction details rules for bending-resistant porticoes

### 7.7.1 Particular criteria

(1) Point **(1)** of section **6.6.1** applies.

(2) The mixed beams must be dimensioned to be ductile, so that the integrity of the concrete is maintained.

(3) Depending on the location of the dissipative areas, the points **(4)** or **(5)** of section **7.5.2**.

(4) The required spherical formation scheme must be obtained by complying with the rules indicated in sections **4.4.2.3**, **7.7.3**, **7.7.4** and **7.7.5**.

### 7.7.2 Analysis

(1) The analysis of the structure must be carried out according to the section properties defined in section **7.4**.

(2) In beams, two different bending rigidities must be taken into account: $EI_1$ for the part of the

spans subjected to positive bending (non-fissured section) and $EI_2$ for the part of the spans subjected to negative bending (fissured section).

(3) Alternatively, the analysis can be performed considering, for the whole beam, an equivalent inertia, $I_{eq}$, constant for every span:

$$I_{eq} = 0{,}6\ I_1 + 0{,}4\ I_2 \tag{7.13}$$

(4) For mixed pillars, bending rigidity is obtained from:

$$(EI)_c = 0{,}9(EI_a + rE_{cm}I_c + EI_s) \tag{7.14}$$

where

$E$ andare the steel and concrete elasticity modules, respectively;
$E_{cm}$

$r$ is the reduction coefficient that depends on the type of cross-section of the pillar. The value of $r$ must be justified based on the fissured inertia value. Alternatively, $r$ = 0.5 can be adopted;

$I_a$, $I_c$ anddesignate the inertia of the steel, concrete and reinforcement section, respectively.
$I_s$

### 7.7.3 Rules for beams and pillars

(1) The dimensioning of mixed beams with T-section must be in accordance with section **7.6.2**. Partially embedded beams must conform to section **7.6.5**.

(2) The beams should be checked for buckling by bending outside the plane and lateral buckling, according to Annex 30 of the Structural Code, assuming the formation of a negative plastic moment at one end of the beam.

(3) Point **(2)** of section **6.6.2** applies.

(4) Mixed lattices should not be used as dissipative beams.

(5) Point **(1)** of section **6.6.3** applies.

(6) On pillars where plastic hinges are formed as indicated in point **(1)** of section **7.7.1** the check must assume that $M_{pl,Rd}$ is reached in these plastic hinges.

(7) The following equation should be applied for all mixed pillars:

$$N_{Ed} / N_{pl,Rd} < 0{,}30 \tag{7.15}$$

(8) The strength checks of the pillars must be carried out in accordance with section **6.7** of Annex 30 of the Structural Code.

(9) The shearing stress of the pillar $V_{Ed}$ (obtained from analysis) must be limited according to equation (6.4).

#### 7.7.4 Connections between beam and pillar

(1) The provisions indicated in section **6.6.4** apply.

#### 7.7.5 Condition for disregarding the mixed character of beams with slabs

(1) The plastic strength of a mixed beam section with slab (lower or upper-limit plastic resistance in dissipative areas) can be obtained by taking into account only the steel section (dimension according to principle c), as defined in **7.1.2**), provided that the slab is completely disconnected from the steel structure in a circular area of diameter $2b_{\text{eff}}$ around a pillar, with $b_{\text{eff}}$ being the highest value of the effective widths of beams connected to that pillar.

(2) For the purposes of point **(1)**, 'totally disconnected' means that there is no contact between the slab and any vertical face of any steel element (e.g. pillars, shearing connectors, splicing plates, corrugated wing, steel board nailed to the steel profile wing).

(3) In partially embedded beams, the contribution of concrete between the wings of the steel profile must be taken into account.

### 7.8 Dimensioning and construction details rules for mixed porticoes with cantered triangulations

#### 7.8.1 Particular criteria

(1) Point **(1)** of section 6.7.1. applies.

(2) The pillars and beams must be either structural steel or mixed steel.

(3) Triangulations (arrangements) must be of structural steel.

(4) Point **(2)** of section **6.7.1** applies.

#### 7.8.2 Analysis

(1) The provisions indicated in section **6.7.2** apply.

#### 7.8.3 Diagonal elements

(1) The provisions indicated in section **6.7.3** apply.

#### 7.8.4 Beams and pillars

(1) The provisions indicated in section **6.7.4** apply.

### 7.9 Dimensioning and construction details rules for mixed porticoes with off-centre triangulations

#### 7.9.1 Particular criteria

(1) Mixed porticoes with off-centre triangulations should be dimensioned so that the dissipative action occurs mainly through the shearing plasticisation of the couplings. All other elements should remain in an elastic state and the rupture of the junctions should be avoided.

(2) Pillars, beams and triangulations (bracing) must be structural or mixed steel.

(3) Triangulations, pillars and beam sections outside the ductile couplings must be dimensioned to remain in an elastic state under the maximum forces that may be generated by the coupling of the beam in a state of total plasticisation and cyclic deformation hardening.

(4) Point **(2)** of section **6.8.1** applies.

### 7.9.2 Analysis

(1) The analysis of the structure is based on the section properties defined in section**7.4.2**.

(2) In beams, two different bending rigidities are considered: $EI_1$ for the part of the spans subjected to positive bending (non-fissured section) and $EI_2$ for the part of the spans subjected to negative bending (fissured section).

### 7.9.3 Couplings

(1) Couplings must be composed of steel profiles, if possible mixed with slabs. They may not be embedded.

(2) The rules on seismic couplings and their stiffeners as indicated in section**6.8.2** apply. Couplings must be of short or intermediate length, with a maximum length*e*:

- in structures where two plastic hinges would be formed at the ends of the coupling

$$e = 2M_{p,link} / V_{p,link} \tag{7.16}$$

- in structures where only one plastic hinge would be formed at one end of the coupling

$$e < M_{p,link} / V_{p,link} \tag{7.17}$$

Point **(3)** of section **6.8.2** provides the definitions of $M_{p,link}$ and $V_{p,link}$. For $M_{p,link}$, only steel components of the coupling section are considered in the evaluation, disregarding the concrete slab.

(3) When seismic couplings converge into a reinforced concrete pillar or embedded pillar, support plates or plates must be arranged on the two sides of the coupling, on the face of the pillar and in the final section of the coupling. These support plates must conform to section **7.5.4**.

(4) The dimensioning of the connections between beam and pillar adjacent to the dissipative couplings must be in accordance with section **7.5.4**.

(5) The connections must comply with the requirements of the connections of steel porticoes with off-centre triangulations, according to section **6.8.4**.

### 7.9.4 Elements not containing seismic couplings

(1) Elements that do not contain seismic couplings must comply with the rules indicated in section **6.8.3**, taking into account the combined strength of steel and concrete in the case of mixed elements

and the relevant rules for elements of **7.6** and Annex 30 of the Structural Code.

(2) Where a coupling is adjacent to a fully embedded mixed pillar, a transverse reinforcement conforming to the requirements of **7.6.4** must be provided above and below the coupling connection.

(3) In the case of mixed triangulation under traction, only the structural steel cross-section must be taken into account in the assessment of the strength of such bracing.

## 7.10 Dimensioning rules and construction details for structural systems consisting of mixed shearing walls (screen walls) of reinforced concrete and structural steel elements

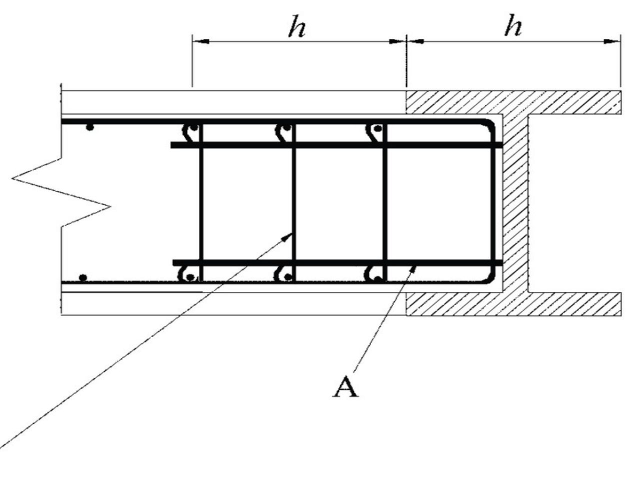
### 7.10.1 Particular criteria

(1) The provisions of this section apply to mixed structural systems belonging to one of the three types defined in point (**e**) of section **7.3.1**.

(2) Structural systems of Types 1 and 2 must be sized to behave as shearing walls and dissipate energy in vertical steel profiles and vertical reinforcement. The fillers must be connected to the contour elements to prevent their separation.

(3) In the Type 1 structural system the shear strength of the floor must be transmitted by horizontal shearing on the wall and at the interface between the wall and the beams.

(4) The type 3 structural system must be sized to dissipate energy on the shearing walls and the coupling beams.

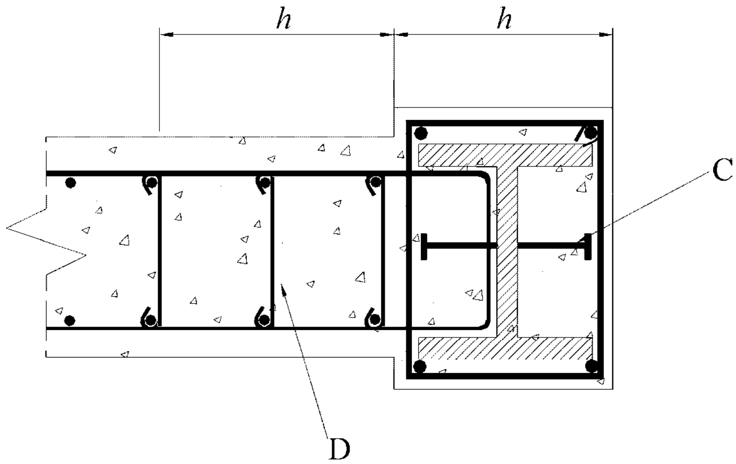



Legend

A Bars welded to the pillar

B Transverse reinforcement

**Figure 7.9a – Construction details of partially embedded mixed contour elements (construction details of cross-arms are for DCH ductility class)**

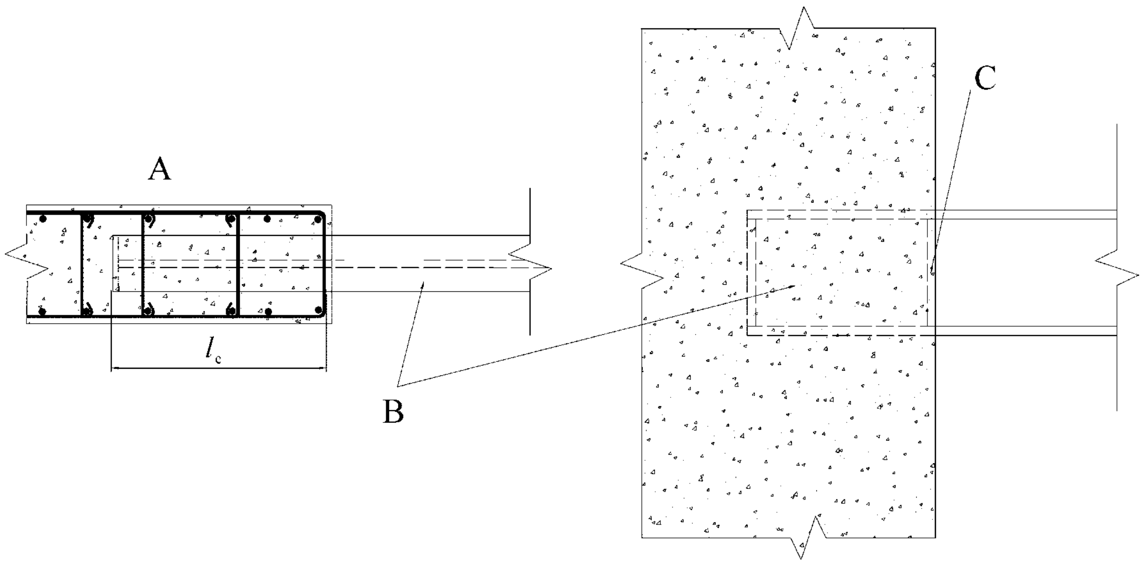



Legend

C Shearing connectors

D Transverse tying

**Figure 7.9b – Construction details of fully embedded mixed contour elements (construction details of cross-arms are for DCH ductility class)**




Legend

A Additional wall reinforcement, in the embedded steel beam

B Steel-coupling beam

C Confinement plate

**Figure 7.10 – Construction details of a coupling beam that meets a wall (construction details are for the DCH ductility class)**

### 7.10.2 Analysis

(1) The analysis of the structure should be based on the section properties defined in chapter **5** for concrete walls and section **7.4.2** for mixed beams.

(2) In structural systems of Types 1 or 2, where vertical structural steel profiles, wholly or partially embedded, act as contour elements of reinforced concrete fillers, the analysis should be carried out on the assumption that the effects of seismic action on these vertical contouring elements are exclusively axillary stresses.

(3) These axillary forces should be determined on the assumption that the shear strength is transmitted through the reinforced concrete wall and that all forces of gravity and overturning are transmitted, in turn, by the shearing wall acting together with the vertical contouring elements.

(4) In type 3 structural systems, if mixed coupling beams are used, the points **(2)** and **(3)** of section **7.7.2**.

### 7.10.3 Construction detail rules for mixed DCM ductility class walls

(1) Reinforced concrete filler panels in Type 1 and reinforced concrete walls in Types 2 and 3 must meet the requirements of chapter **5** for DCM ductility class.

(2) Partially embedded steel profiles used as contour elements of reinforced concrete panels must belong to a cross-section class related to the structure performance coefficient as indicated in Table 7.3.

(3) Fully embedded structural steel profiles used as contour elements in reinforced concrete panels must be dimensioned according to section **7.6.4**.

(4) Partially embedded structural steel profiles used as contour elements of reinforced concrete panels must be dimensioned according to section **7.6.5**.

(5) Shearing connector bolts or binding reinforcement (welded, anchored through holes in steel elements or anchored around the steel element) must be arranged to convey vertical and horizontal shear strength between the structural steel of the contour elements and the reinforced concrete.

### 7.10.4 Construction details rules for DCM ductility class coupling beams

(1) The coupling beams must have a recessed length in the reinforced concrete wall sufficient to withstand the most unfavourable combination of sharp moments and forces generated by bending and shearing resistances of the coupling beam. Embedding length, $l_e$, should be considered to begin within the first layer of the confinement reinforcement in the boundary element of the wall (see Figure 7.10). This embedding length, $l_e$, should not be less than 1.5 times the height of the coupling beam.

(2) The dimensioning of connections between beam and wall must be in accordance with section **7.5.4**.

(3) The vertical wall reinforcements defined in points **(9)** and **(10)** of section **7.5.4**, with a calculation value of the axial resistance equal to the shearing strength of the coupling beam, must be placed over the embedding length of the beam, with two thirds of the steel located on the first half of that embedding length. This wall reinforcement must extend a distance of at least once the anchorage length above and below the wings of the coupling beam. It is allowed to use vertical reinforcement

with other targets such as vertical contour elements as part of the required vertical reinforcement. The reinforcement must comply with section **7.6**.

### 7.10.5 Additional construction details rules for the DCH ductility class

(1) A cross-sectional reinforcement must be provided for the confinement of mixed contour elements, partially or fully embedded. The reinforcement must extend a distance $2h$ within the concrete walls, where $h$ is the edge of the contour element in the plane of the wall [see Figures 7.9a and 7.9b].

(2) The requirements for portico couplings with off-centre triangulations apply also for coupling beams.

## 7.11 Dimensioning and construction details rules for mixed shearing walls with steel plate

### 7.11.1 Particular criteria

(1) Mixed shearing walls with steel plate must be dimensioned so that they laminate under the effect of the shear strength of the steel plate.

(2) The steel plate must be stiffened by enclosing concrete on one or two sides and fastening to the enclosing reinforced concrete to prevent the buckling of the steel plate.

### 7.11.2 Analysis

(1) The analysis of the structure must be based on the properties of the section and the materials defined in sections **7.4.2** and **7.6**.

### 7.11.3 Construction details rules

(1) It should be verified that:

$$V_{\mathrm{Ed}} < V_{\mathrm{Rd}} \tag{7.18}$$

with the shear strength given by:

$$V_{\mathrm{Rd}} = A_{\mathrm{pl}} \times f_{\mathrm{yd}} / \sqrt{3} \tag{7.19}$$

Where

$f_{\mathrm{yd}}$ is the calculation value of the elastic limit of plate steel; and

$A_{\mathrm{pl}}$ is the horizontal area of the plate.

(2) The connections between the plate and the contour elements (pillars and beams), as well as the connections between the plate and the surrounding concrete, should be dimensioned so that the plastic resistance of the plate can be fully developed.

(3) The steel plate must be connected continuously and along all its edges to the structural steel frame and to the contour elements with welds or bolts, so that the plastic resistance of the shearing plate develops.

(4) The contour elements must be dimensioned to meet the requirements of section **7.10**.

(5) The thickness of the steel must not be less than 200 mm when available on one side only, or 100 mm on each side when available on both sides.

(6) The minimum amount of reinforcement in both directions should not be less than 0.25 %.

(7) The openings in the steel plate must be stiffened as required by the analysis.

## 7.12 Dimensioning and construction control

(1) For the control of dimensioning and construction, the section **6.11** applies.

# 8 Specific rules for wooden buildings

## 8.1 General considerations

### 8.1.1 Object and field of application

(1) For the project of wooden buildings the Basic Document DB SE-M is applied 'Structural security: wood' of the Technical Building Code. The following rules complement those set out in that standard.

### 8.1.2 Definitions

(1) The following terms are used in this chapter with the following meanings:

**static ductility:**
Relationship between breakage deformation and deformation at the limit of elastic behaviour, evaluated in quasi-static cyclic tests (see point **(3)** of section **8.3**).

**semi-rigid junctions**
Junctions with significant flexibility, the influence of which has to be taken into account in the structural calculation according to the Basic Document DB SE-M (e.g. pin-type unions).

**rigid junctions:**
Junctions with negligible flexibility according to Basic Document DB SE-M (e.g. glued solid wood junctions).

**pin-type junctions:**
Junctions with pin-type mechanical fasteners (keys, staples, strips, pins, screws, etc.) loaded perpendicular to their axles.

**carpenter junctions:**
Junctions where loads are transmitted through pressure areas and without mechanical fastening elements (e.g. slanted gluing, dovetail, half wood).

### 8.1.3 Dimensioning principles

(1) Earthquake-resistant wooden buildings should be sized according to one of the following principles:

a) dissipative structural behaviour;

b) Minimally dissipative structural behaviour.

(2) Principle (a) takes into account the ability of some parts of the structure (dissipative areas) to withstand earthquake actions outside its elastic domain. When using the calculation spectrum defined in **3.2.2.5**, the behaviour coefficient $q$ can be taken greater than 1.5. The value of $q$ depends on the ductility class (see **8.3**).

(3) Structures sized according to principle (a) must belong to the DCM or DCH ductility classes. A structure belonging to a given ductility class must meet the specific requirements in one or more of the following fields: type of structure, type of connections and ductile rotation capacity of these.

(4) Dissipative areas should be located in junctions and connections, while wooden elements should be considered to have elastic behaviour.

(5) The properties of the dissipative areas must be determined by tests either in single junctions, or in complete structures or in parts thereof, in accordance with UNE-EN 12512.

(6) In principle (b), the effects of the action are calculated by an overall elastic analysis, without taking into account the nonlinear behaviour of the material. When using the calculation spectrum defined in section **3.2.2.5**, the behaviour coefficient $q$ must not be taken more than 1.5. The strength of the elements and connections must be sized in accordance with the specific regulations in force, without any additional requirements. This principle is called DCL (low) ductility class and is appropriate only for certain types of structures (see Table 8.1).

## 8.2 Materials and properties of dissipative areas

(1) The relevant provisions of Basic Document DB SE-M 'Structural security: wood' of the Technical Building Code. With regard to the properties of steel elements, the provisions of the Structural Code apply (Annexes 22 to 29).

(2) When using the principle of dissipative structural behaviour, the following rules apply:

a) in junctions considered as dissipative areas, only mechanical materials and fasteners may be used that provide adequate performance against low cycle fatigue;

b) glued junctions should be considered as non-dissipative areas;

c) carpenter junctions can only be used when they can provide sufficient energy dissipation capacity, without presenting risks of brittle shearing break or traction perpendicular to the fibre. The decision on its use should be based on the appropriate test results.

(3) Point **(2)**(a) above is considered complied with if point **(3)** of section **8.3** is complied with.

(4) For material of screen panels on shearing walls and diaphragms, the point **(2)**(a) above is considered complied with if the following conditions are met:

a) agglomeration boards have a density of at least 650 kg/m$^3$;

b) plywood panels have a minimum thickness of 9 mm;

c) agglomerate or fibre panels have a minimum thickness of 13 mm.

(5) The steel material for connections must meet the following conditions:

a) all connecting elements made of steel must comply with the requirements of the Structural Code (Annexes 22 to 29);

b) compliance with point **(3)** of section **8.3** of the ductility properties of the connections in the lattice and between the material of the screen panels and the wooden portico in the DCM or DCH ductility class structures (see **8.3**) must be checked, by means of cyclic tests, under the appropriate combination of the connected parts and the fasteners.

## 8.3 Ductility classes and behaviour coefficients

(1) Depending on their ductility behaviour and energy dissipation capacity under seismic actions, wood buildings should be classified into one of the three classes of ductility, DCL, DCM or DCH, indicated in Table 8.1, where the corresponding upper-limit values of the performance coefficients are also shown. There are no geographical limitations to the use of the DCM and DCH ductility classes.

**Table 8.1 - Dimensioning principles, types of structures and upper-limit values of behaviour coefficients for the three ductility classes**

| Dimensioning principle and ductility class | *q* | Examples of structures |
|---|---|---|
| Low Energy Dissipation Capacity – DCL | 1.5 | Cantilevered structures; beams; arches with two or three joints: lattice attached with connectors. |
| Medium energy dissipation capacity – DCM | 2 | Wall panels glued with diaphragms, connected with nails and screws; lattice ties with pins and bolts; mixed structures consisting of a wooden portico (which resist horizontal forces) and a filler without bearing capacity. |
| | 2.5 | Hyperstatic porticoes with pins and bolts (see point **(3)** of section **8.1.3**). |
| High-energy dissipation capacity – DCH | 3 | Wall panels nailed with glued diaphragms, connected with nails and screws; lattice with nailed junctions. |
| | 4 | Hyperstatic porticoes with pins and bolts (see point **(3)** of section **8.1.3**). |
| | 5 | Wall panels nailed with nailed diaphragms, connected with nails and screws. |

(2) If the building is irregular in height (see **4.2.3.3**) the values of $q$ indicated in Table 8.1 must be reduced by 20 %, but there is no need to take them less than $q$ = 1,5 (see point **(7)** of section **4.2.3.1** and table 4.1).

(3) In order for the indicated values of the behaviour coefficient to be used, dissipative areas must be able to deform plastically for at least three complete inversion cycles, with a static ductility ratio of 4 for DCM ductility class structures and 6 for DCH ductility class structures, without a reduction of their strength greater than 20 %.

(4) The provisions of point **(3)** above and points **(2)**(a) and **(5)**(b) of section **8.2** can be considered complied with in the dissipative areas of all types of structures if the following conditions are met:

a) in wood-wood and steel-wood junctions with bolts, screwed and studded, the minimum thickness of the connected parts is $10{\cdot}d$ and the diameter of the fastening element, $d$, does not exceed 12 mm;

b) in shearing walls and diaphragms, the screen panels are lined with wood, with a minimum thickness of $4d$, where the nail diameter, $d$, does not exceed 3.1 mm.

If the above requirements are not met, but a minimum thickness of $8d$ and $3d$ elements is ensured for cases (a) and (b), respectively, the lower upper-limit values of the performance coefficient $q$, indicated in Table 8.2.

**Table 8.2 - Types of structures and reduced upper-limit values of behaviour coefficients**

| Types of structures | behaviour coefficient *q* |
|---|---|
| Hyperstatic porticoes with pins and bolted junctions | 2.5 |
| Wall panels nailed with nailed diaphragms | 4.0 |

(5) For structures that have different and independent properties in the two horizontal directions, the values of $q$ to be used in calculating the effects of seismic action in each main direction must correspond to the properties of the structural system in that direction, and may be different.

## 8.4 Structural analysis

(1) The displacement in the junctions of the structure should be taken into account in the calculation.

(2) A value of module $E_o$ should be used for instant loading (10 % higher than short term).

(3) Ground-floor diaphragms can be considered rigid in the structural model without further testing, if the following two conditions are met:

a) the construction details rules for horizontal diaphragms indicated in section **8.5.3** apply;

and

b) their openings do not significantly affect the overall rigidity in the plane of the concrete slab.

## 8.5 Construction details rules

### 8.5.1 General considerations

(1) The construction details rules indicated in sections **8.5.2** and **8.5.3** apply to earthquake-resistant parts of structures sized according to the principle of dissipative structural behaviour (DCM and DCH ductility classes).

(2) The structures with dissipative areas should be sized so that these areas are located, mainly, in those parts of the structure where the plasticisation, the local buckling or any other phenomenon due to hysteretic behaviour do not affect the overall stability of the structure.

### 8.5.2 Construction details rules for connections

(1) Compression elements and their connections (e.g. carpenter junctions), which may fail due to deformations caused by load reversals, should be dimensioned in such a way as to avoid separation and remain in their original position.

(2) Screws and pins should be tightened and adjusted into the holes. Large screws and pins ($d$ > 16 mm) must not be used in wood-wood and steel-wood connections, except in combination with wooden connectors.

(3) Pins, smooth nails and staples should not be used without additional provisions to prevent removal.

(4) In the case of a traction perpendicular to the vein, additional provisions should be made to prevent displacement (e.g. nailed metal or plywood plates).

### 8.5.3 Construction detail rules for horizontal diaphragms

(1) For horizontal diaphragms under seismic actions, the Basic Document DB SE-M applies 'Structural security: wood' of the Technical Building Code, with the following modifications:

a) the increase coefficient of 1.2 should not be used for the strength of the fasteners at the edges of sheets;

b) when the panels are on the three-pocket, the increase coefficient of 1.5 should not be used for the separation of nails along the discontinuous edges of the panels;

c) the distribution of shear strength in the diaphragms should be assessed taking into account the position in the floor of the vertical elements resisting lateral loads.

(2) All edges of screen panels not converging into portico elements should be supported and connected to transverse reinforcements located between the wooden beams. Reinforcements should also be provided on horizontal diaphragms above vertical elements resistant to lateral loads (e.g. walls).

(3) The continuity of the beams, including brushes, should be ensured in areas where the diaphragm is altered by openings or gaps.

(4) If there are no intermediate transverse reinforcements along the entire beam height, the height-to-width ratio ($h/b$) of the wooden beams must be less than 4.

(5) If $a_{\mathrm{g}} \cdot S \geq 0.2 \cdot \mathrm{g}$, the separation of mechanical fastening elements in the areas of discontinuity should be reduced by 25 %, but not less than the minimum separation indicated in the specific regulations in force (see chapters 8 and 10 of DB SE-M of the Technical Building Code).

(6) When concrete slabs are considered rigid in the plane for structural calculation, there should be no change in the direction of the beam beams on the supports, where horizontal forces are transmitted to the vertical elements (e.g. shearing walls).

## 8.6 Safety checks

(1) The strength values of the wood material should be determined taking into account the values of $k_{\mathrm{mod}}$ for instantaneous loads, as set out in DB SE-M 'Structural safety: wood' of the Technical Building Code.

(2) For USL checks of structures sized according to the principle of non-dissipative structural behaviour (DCL ductility class), for material properties for fundamental load combinations, partial safety coefficients $\gamma_{\mathrm{M}}$ of DB SE-M are applied.

(3) For USL checks of structures sized according to the principle of dissipative structural behaviour (DCM or DCH ductility classes), partial safety coefficients $\gamma_{\mathrm{M}}$ are applied for material properties for accidental combinations of DB SE-M loads.

(4) To ensure the development of cyclic plastic deformation in the dissipative areas, the remaining

structural elements and connections must be sized with a sufficient strength reserve. This resistance reserve requirement applies in particular to:

- anchoring straps and any connection to massive support elements located below;

- the connections between horizontal diaphragms and vertical elements resisting lateral load.

(5) Carpenter junctions do not present a risk of fragile break if the test of the shearing voltage according to the DB SE-M is carried out with an additional partial safety coefficient of 1.3.

## 8.7 Dimensioning and construction control

(1) The provisions indicated in the DB SE-M of the Technical Building Code apply.

(2) The following structural elements must be identified in the project plans and specifications should be provided for their specific control during construction:

- anchoring straps and any bindings to foundation elements;

- diagonal-traction steel lattice used in triangulations (arrangements);

- the connections between horizontal diaphragms and vertical elements resisting lateral load;

- the connections between the screen panels and the wooden porticoes that converge on the horizontal and vertical diaphragms.

(3) The specific control of the construction must understand the properties of the material and the precision in the execution.

# 9 Specific rules for brickwork buildings

## 9.1 Object and field of application

(1) This chapter applies to the project of non-reinforced, confined and reinforced brickwork buildings in seismic areas.

(2) For the brick building project, the provisions of the Basic Document DB SE-F ‘Structural security: brickwork’ of the Technical Building Code. The following rules complement those set out in that regulation.

## 9.2 Rigging materials and models

### 9.2.1 Types of brickwork units

(1) Brickwork units must be robust enough to prevent local fragile breakage. This condition is met for the 'solid' and 'perforated' parts groups indicated in Table 4.1 of the Basic Document DB SE-F 'Structural security: brickwork'.

### 9.2.2 Minimum resistance of brickwork units

(1) Except in cases of low seismicity, the standard compressive strength of the brickwork units, calculated in accordance with UNE-EN 772-1, must not be less than the following values:

- normal to course seat face (table): $f_{b,min}$ = 5 N/mm$^2$
- parallel to the course seat face (table), on the wall plane: $f_{bh,min}$ = 2 N/mm$^2$

### 9.2.3 Mortar

(1) Except in cases of low seismicity, minimum resistance is required for mortar, $f_{m,mind}$ = 5 N/mm$^2$ for non-reinforced brickwork and $f_{m,min}$ = 10 N/mm$^2$ for reinforced factories.

### 9.2.4 Rigging (joining) the brickwork

(1) Three alternative classes of joints are considered:

a) fully filled with mortar joints;

b) joints without filler;

c) non-filled joints with mechanical assembly between the brickwork units.

In areas where the application of the earthquake-resistant Standard is necessary, only joints of type (a) are permitted.

## 9.3 Types of construction and behaviour coefficients

(1) Depending on the type of brickwork used for the earthquake-resistant elements, brickwork buildings must be classified into one of the following types of construction:

a) non-reinforced brickwork construction;

b) confined brickwork construction;

c) reinforced brickwork construction.

NOTE 1 Construction with brickwork systems providing improved ductility of the structure is included (see Note to Table 9.1).

NOTE 2 Structures with brickwork fillers are not covered by this chapter.

(2) Due to its low tensile strength and low ductility, in case of structural function, the non-reinforced brickwork that complies exclusively with the provisions indicated in the Basic Document DB SE-F 'Structural safety: brickwork' of the Technical Building Code is considered to offer a low dissipation capacity (DCL ductility class) and therefore only allowed for use in low seismicity areas (see point **(4)** of section **3.2.1** of this Annex), if it is also met that the effective wall thickness, $t_{ef}$, is not less than a minimum value, $t_{ef,min}$. With $t_{ef,min}$ = 350 mm, for natural stone brickwork, and $t_{ef,min}$ = 240 mm for brick brickwork.

(3) Due to the reasons given in point **(2)** above, the non-reinforced brickwork that satisfies the provisions of this Annex cannot be used if the value of $a_g \cdot S$ exceeds a certain upper value, $a_{g,urm}$ = 0.20 g, which is consistent with the values adopted in sections **9.2.2** and **9.2.3** for the standard compressive strength of brickwork units, $f_{b,min}$, and $f_{bh,min}$. and for the minimum mortar resistance, $f_{m,min}$.

(4) For types (a) to (c), the permissible value ranges of the upper limit of the behaviour coefficient $q$ are indicated in Table 9.1.

**Table 9.1 - Construction types and upper-limit values of the behaviour coefficient**

| Construction type | behaviour coefficient $q$ |
|---|---|
| Non-reinforced brickwork, exclusively DB-compliant SE-F, Brickwork of the Technical Building Code. (only in cases of low seismicity) | 1.5 |
| Non-reinforced brickwork according to this Annex | 1.5 |
| Confined brickwork | 2.0 |
| Reinforced brickwork | 2.5 |

NB For the performance coefficient q, a value other than that specified in Table 9.1 can only be adopted if there is a certificate issued by a national or international notified body to justify it.

(5) If the building is not adjusted in height (see **4.2.3.3**) the values of $q$ indicated in Table 9.1 must be reduced by 20 %, but without needing to be taken as lower than $q$ = 1.5 (see point **(7)** of section **4.2.3.1** and Table 4.1).

## 9.4 Structural analysis

(1) The structural model for the building project should represent the rigidity properties of the total system.

(2) The rigidity of the structural elements should be assessed in the light of their shearing and flex flexibility and, if relevant, also their flexibility against axial force. Non-fissured elastic rigidity or,

preferably and more realistically, fissured rigidity can be used for calculation in order to consider the influence of cracking on deformations and to obtain a better approximation to the slope of the first branch of a force-deformation bilinear model for the structural element.

(3) In the absence of an accurate assessment of the rigidity properties, supported by an appropriate calculation, the fissured rigidity at bending and shearing may be taken equal to half of the non-fissured elastic rigidity of the raw section.

(4) In the structural model, the brickwork enclosures that make up lintels and forks in the spans of a wall can be considered as coupling beams between two wall elements, if they are continuously attached to them and are connected both to the tie beam of the concrete slab and to the loader located below.

(5) If the structural model takes into account coupling beams, a portico calculation can be used to determine the effects of actions on vertical and horizontal structural elements.

(6) The base shearing in the walls, as obtained by the linear analysis described in chapter **4**, can be redistributed between the walls, assuming that:

a) the overall balance is complied with (i.e. the same total base cutter and the same position of the resulting force are achieved);

b) the shearing on any wall is neither reduced by more than 25 % nor increased by more than 33 %; and

c) the consequences of redistribution for the diaphragm(s) are taken into account.

## 9.5 Dimensioning criteria and building rules

### 9.5.1 General considerations

(1) The brickwork buildings must be composed of concrete slabs and walls, which are joined in two orthogonal horizontal directions and in the vertical direction.

(2) The connection between the concrete slabs and the walls must be made by means of steel straps or reinforced concrete perimeter beams.

(3) Any type of concrete slab can be used, provided that the general requirements of continuity and effective operation of the diaphragm are respected.

(4) Shearing walls should be arranged in at least two orthogonal directions.

(5) Shearing walls must meet certain geometric requirements, described below and the values of which are given in Table 9.2:

a) the effective thickness of shearing walls, $t_{ef}$ cannot be less than a minimum value, $t_{ef,min.}$;

b) the ratio $(h_{ef}/t_{ef})$ between the effective wall height and its effective thickness cannot exceed a maximum value, $(h_{ef} / t_{ef})_{max.}$; and

c) the ratio of wall length, $l$, with respect to the highest free height, $h$, of the gaps adjacent to the wall, can not be less than a minimum value, $(l/h)_{min.}$

**Table 9.2 - Geometric requirements for shearing walls**

| Type of brickwork | $t_{ef,min.}$ (mm) | $(h_{ef}/t_{ef})_{max.}$ | $(l/h)_{min.}$ |
|---|---|---|---|
| Non-reinforced, with natural stone units | 350 | 9 | 0.5 |
| Not reinforced, with any other type of units | 240 | 12 | 0.4 |
| Not reinforced, with any other type of units, in cases of low seismicity | 170 | 15 | 0.35 |
| Confined | 240 | 15 | 0.3 |
| Reinforced | 240 | 15 | No restriction |
| The symbols used have the following meanings:<br>$t_{ef}$ effective thickness or wall calculation (see DB SE-F of the Technical Building Code.);<br>$h_{ef}$ effective wall height or calculation (see DB SE-F of the Technical Building Code.);<br>$h$ maximum free height of openings adjacent to the wall;<br>$l$ wall length. | | | |

(6) Shearing walls that do not comply with the minimum geometric requirements of the point **(5)** above can be considered as secondary seismic elements. These walls must comply with the points **(1)** and **(2)** of section **9.5.2**.

### 9.5.2 Additional requirements for non-reinforced brickwork conforming to this Annex

(1) Horizontal concrete beams or, alternatively, steel straps must be arranged on the wall plane at the level of each concrete slab and in any case with a vertical separation of not more than 4 m. These beams or braces must form continuous tying elements, physically attached to each other.

NB It is essential to arrange continuous beams or straps along the entire perimeter.

(2) Horizontal concrete beams must have longitudinal reinforcement with a cross-section not less than 200 mm$^2$.

### 9.5.3 Additional requirements for confined brickwork

(1) Horizontal and vertical confinement elements must be joined together and anchored to the elements of the main structural system.

(2) In order to achieve an effective connection between the confinement elements and the brickwork, the concrete of the confinement elements must be executed after the brickwork has been built.

(3) The cross-sectional dimensions of horizontal and vertical confinement elements may not be less than 150 mm. In double-leaf walls, the thickness of the confinement elements must ensure the connection between the two sheets and their effective confinement.

(4) Vertical confinement elements must be affixed:

- on the free edges of each structural element of the wall;
- on both sides of any opening in the wall with an area greater than 1.5 $m^2$;
- inside the wall if necessary, in order not to exceed the 4 m separation between the confinement elements;
- at structural wall intersections, where the confinement elements required by the above rules are at a distance greater than 1.5 m.

(5) Horizontal confinement elements must be placed in the wall plane at the level of each concrete slab and in any case with a vertical separation of not more than 4 m.

(6) The cross-sectional area of the longitudinal reinforcement may not be less than 300 $mm^2$ or less than 1 % of the cross-sectional area of the confinement element.

(7) Abutments of a diameter of not less than 6 mm and with a separation of not more than 150 mm must be arranged around the longitudinal reinforcement.

(8) The steel of the reinforcement must be of type S or SD, in accordance with Article 34 of the Structural Code.

(9) Flap splices may not have a length less than 60 times the diameter of the rounds.

### 9.5.4 Additional requirements for the reinforced brickwork

(1) The horizontal reinforcement must be located at the horizontal joints or in the appropriate slots of the units, with a vertical separation not exceeding 600 mm.

(2) The brickwork units with holes must accommodate the necessary reinforcement in the lintels and parapets.

(3) Steel reinforcements of a diameter of not less than 4 mm, bent around the vertical rounds at the edges of the wall, must be used.

(4) The minimum percentage of horizontal reinforcement in the wall, normalised with respect to the gross area of the section, must not be less than 0.05 %.

(5) High percentages of horizontal reinforcement leading to compression break in units should be avoided prior to steel plasticisation.

(6) The vertical reinforcement spread over the wall, as a percentage of the gross area of the horizontal section of the wall, must not be less than 0.08 %.

(7) The vertical reinforcement must be placed in holes, cavities or holes in the units.

(8) Vertical reinforcements with a cross-section not less than 200 $mm^2$must be fitted:

- at the two free ends of each wall;
- at each wall intersection;

- inside the wall, so as not to exceed the separation of 5 m between these reinforcements.

(9) The points **(7)**, **(8)** and **(9)** of section **9.5.3** are applied.

(10) Parapets and lintels must be regularly linked to the brickwork of adjacent walls by means of horizontal reinforcement.

## 9.6 Safety check

(1) The safety of buildings against collapse should be explicitly checked, except for buildings that meet the rules set out in **9.7.2** for 'simple brickwork buildings”.

(2) For verifying safety against collapse, the calculation value of the strength of each structural element should be assessed in accordance with the specific regulations in force.

(3) In ultimate limit state checks for the seismic calculation situation, the partial safety coefficients $\gamma_m$ must be used for brickwork properties and $\gamma_s$ for reinforcement steel. A value of $\gamma_s = 1$ is adopted; $\gamma_m$ = max. [2/3 of the value given in the DB-SE-F; 1,5]

## 9.7 Rules for 'simple brickwork buildings'

### 9.7.1 General considerations

(1) Buildings belonging to importance classes I or II and complying with sections **9.2**, **9.5** and **9.2.7** can be classified as 'simple brickwork buildings'.

(2) For such buildings, the explicit safety check according to section**9.6** is not mandatory.

### 9.7.2 Rules

(1) Depending on the product $a_g \cdot S$ on site and type of construction, the allowed number of floors above ground, $n$, must be arranged in two orthogonal directions with a minimum area of the cross section, $A_{min.}$, in each of these directions. The minimum cross section area is expressed as a minimum percentage, $p_{A,min.}$, of the total concrete slab area per floor.

NB The values to be assigned to $n$ and $p_{A,min.}$ are indicated in Table 9.3. In any case, the limitations on the use of non-reinforced brickwork indicated in point**(3)** of section **9.3** must be respected.

**Table 9.3 - Maximum allowable number of floors above ground and minimum area of shearing walls for 'simple brickwork buildings'**

| Local acceleration $a_g \cdot S$ | | < 0.07 g | < 0.11 g | < 0.15 g | < 0.19 g |
|---|---|---|---|---|---|
| Construction type | Number of floors (n)** | Minimum sum of cross-sectional areas of horizontal shearing walls in each direction, as a percentage of the total concrete slab area of each floor ($p_{A,min.}$) | | | |
| Non-reinforced brickwork | 1 | 2.0 % | 2.5 % | 3.5 % | n/a |
| | 2 | 2.5 % | 3.0 % | 5.0 % | n/a |
| | 3 | 3.0 % | 5.0 % | n/a | n/a |
| | 4 | 5.0 % | n/a* | n/a | n/a |
| Confined brickwork | 1 | 2.0 % | 2.5 % | 3.0 % | 3.5 % |
| | 2 | 2.0 % | 2.5 % | 3.5 % | 4.0 % |
| | 3 | 2.5 % | 3.5 % | 4.0 % | n/a |
| | 4 | 4.0 % | 5.0 % | n/a | n/a |
| Reinforced brickwork | 1 | 1.5 % | 2.0 % | 2.5 % | 3.0 % |
| | 2 | 2.0 % | 2.5 % | 3.5 % | 4.0 % |
| | 3 | 2.5 % | 3.5 % | 4.0 % | n/a |
| | 4 | 3.5 % | 4.0 % | 5.0 % | n/a |
| * n/a means 'not acceptable'. | | | | | |
| ** The cover space on complete floors is not considered in the number of floors. | | | | | |

(2) The floor configuration of the building must comply with the following conditions:

a) the floor must be approximately rectangular;

b) the ratio between the length of the major side and that of the minor side in floor must not be less than a minimum value, $\lambda_{min} = 0.25$;

c) the area of the projections of the recesses and recesses of the rectangular shape must not be greater than a percentage, $p_{max} = 15\ \%$, of the total area of the concrete slab above the considered level.

(3) The shearing walls of the building must comply with all of the following conditions:

a) the building must be stiffened by shearing walls, distributed almost symmetrically on the floor in two orthogonal directions;

b) a minimum of two parallel walls must be located in two orthogonal directions, the length of each wall being greater than 30 % of the length of the building in the direction of the wall concerned;

c) at least for walls in one direction, the distance between them must be greater than 75 % of the length of the building in the other direction;

d) at least 75 % of vertical loads must be supported by shearing walls;

e) the shearing walls must be continuous from the top to the bottom of the building.

(4) In cases of low seismicity (see point **(4)** of section **3.2.1**) the wall length required in point **(3)**(b) above can be provided by the cumulative length of the shearing walls (see point **(5)** of section **9.5.1**) according to an axis, separated by apertures. In this case, at least one of the shearing walls in each direction must have a length, $l$, not less than that corresponding to twice the minimum value of $l/h$ defined in point **(5)**(c) of section **9.5.1**.

(5) Between two successive floors, the differences in mass and in the cross-sectional area of the horizontal shearing walls, in both horizontal directions, must be limited to maximum values of $\Delta_{m,max.}$ = 20% and $\Delta_{A,max.}$ = 20 %, respectively.

(6) For non-reinforced brickwork buildings the walls in one direction must be attached to the walls in the orthogonal direction with a maximum separation of 7 m.

# 10 Insulation of the base

## 10.1 Object and field of application

(1) This chapter covers the project of seismically isolated structures in which the isolation system, located below the main mass of the structure, aims to reduce the seismic response of the system that resists lateral force.

(2) The reduction of the seismic response of the system resisting lateral force can be obtained by increasing the fundamental period of the seismically isolated structure, modifying the shape of the fundamental mode and increasing damping, or by a combination of these effects. The isolation system may consist of linear or nonlinear springs or dampers.

(3) This chapter gives specific rules regarding the isolation of the base of buildings.

(4) This chapter does not cover passive energy dissipation systems that are not arranged over a simple interface, but distributed across several floors or levels of the structure.

## 10.2 Definitions

(1) The following terms are used in this chapter with the following meanings:

**isolation system:**
Set of components used to provide seismic isolation, arranged over the isolation interface.

NB They are usually located below the main mass of the structure.

**isolation interface:**
Surface separating the infrastructure and the superstructure and where the isolation system is located.

NB The placement of the isolation interface at the base of the structure is the most common mode in buildings, tanks and silos. In bridges, the isolation system is usually combined with the supports and the isolation interface is located between the board and the pillars or brackets.

**insulating units:**
Elements constituting the isolation system.

The devices considered in this chapter consist of elastomeric laminated supports, elastoplastic devices, viscous or friction dampers, pendulums, and other devices whose behaviour meets point **(2)** of section **10.1**. Each unit provides one or a combination of the following functions:

- a vertical bearing capacity combined with increased lateral flexibility and high vertical rigidity;
- energy dissipation, either hysteretic or viscous;
- an ability to regain its position;
- a lateral coercion (sufficient elastic rigidity) under non-seismic side service loads.

**infrastructure:**

Part of the structure located under the isolation interface, including foundation.

NB The lateral flexibility of the infrastructure(s) is generally negligible compared to that of the isolation system, but is not always the case (e.g. bridges).

**superstructure:**
Part of the structure that is isolated and is located over the isolation interface.

**total isolation:**
The superstructure is completely isolated if, in the seismic calculus situation, it remains within the elastic domain. Otherwise, the superstructure is partially isolated.

**effective rigidity centre:**
Calculated rigidity centre on the upper face of the isolation interface, i.e. including the flexibility of the insulating units and that of the infrastructure(s).

NB In buildings, deposits and similar structures, the flexibility of the infrastructure may be disregarded in the determination of this centre, which then coincides with the rigidity centre of the isolation units.

**calculation displacement (of the isolation system according to one main direction):**
Maximum horizontal displacement of the effective rigidity centre between the top of the infrastructure and the bottom of the superstructure, which occurs in the seismic calculation situation.

**total calculation displacement (of an isolation unit according to one main direction):**
Maximum horizontal displacement at the unit site, including displacement due to calculation displacement and overall torque rotation relative to the vertical axis.

**effective calculation rigidity (of the isolation system in one main direction):**
Ratio between the value of the total horizontal force transmitted through the isolation interface, when the calculation displacement occurs in the same direction and the absolute value of that calculation displacement (drying rigidity).

NB Effective rigidity is usually obtained by iterative dynamic calculation.

**effective period:**
Fundamental period, in the direction considered, of a system with a single degree of freedom having the mass of the superstructure and a rigidity equal to the effective rigidity of the isolation system.

**effective damping (of the isolation system in a main direction):**
Effective viscous damping value corresponding to the energy dissipated by the isolation system during the cyclic response in the calculation displacement.

## 10.3 Key requirements

(1) The fundamental requirements set out in section **2.1** and in the corresponding parts of this Earthquake-Resistant Standard must be complied with, depending on the type of structure considered.

(2) Greater reliability is required for isolation devices. For this purpose, a coefficient of increase must be applied to the seismic displacements of each unit. The value of this coefficient in the case of buildings must be $\gamma_x$=1.2.

## 10.4 Compliance criteria

(1) In order to meet the fundamental requirements, the limit states defined in **(1)** of section **2.2.1** must be checked.

(2) In the limit state of damage, all ducts that cross the joints around the isolated structure must remain within the elastic domain.

(3) In buildings, for the state of damage limitation, the movement between floors in the infrastructure and in the superstructure must be limited, according to section **4.4.3.2**.

(4) In the ultimate limit state, the ultimate capacity of the isolation devices in terms of strength and deformability must not be exceeded, with the corresponding partial safety coefficients (see section **(6)** of section **10.10**).

(5) Only total isolation is considered in this chapter.

(6) Although in certain cases it may be acceptable for the infrastructure to have inelastic behaviour, in this chapter it is considered to remain in the elastic domain.

(7) In the ultimate limit state, isolation devices can reach their ultimate capacity, while the superstructure and infrastructure remain in the elastic domain. Capacity dimensioning and construction details to ensure ductility are therefore not necessary either in the superstructure or in the infrastructure.

(8) In the ultimate limit state, gas pipes or other dangerous lines crossing the joints separating the superstructure from the terrain or surrounding constructions must be sized to safely allow the relative displacement between the isolated superstructure and the surrounding terrain(s), taking into account the coefficient defined in point **(2)** of section **10.3**.

## 10.5 General dimensioning provisions

### 10.5.1 General provisions regarding devices

(1) Sufficient space should be provided between the superstructure and the infrastructure, as well as any other measures necessary to enable the inspection, maintenance and replacement of the devices during the structure's lifetime.

(2) If necessary, devices must be protected from potentially dangerous effects such as fire and chemical or biological attack.

(3) The materials used in the dimensioning and construction of the devices must comply with the appropriate existing standards.

### 10.5.2 Control of undesired movements

(1) In order to minimise the effects of torque, the effective rigidity centre and buffer centre of the isolation system must be as close as possible to the projection of the centre of gravity on the isolation interface.

(2) In order to minimise differences in the behaviour of insulating devices, the compression stress induced in them by permanent actions must be as uniform as possible.

(3) The devices must be attached to the superstructure and infrastructure.

(4) The isolation system should be sized so that potential shocks and torque movements are controlled by appropriate measures.

(5) The requirements of point **(4)** concerning shocks are considered to be complied with if the effects of such potential shocks are avoided by appropriate devices (shock absorbers, shock absorption devices, etc.).

### 10.5.3 Control of differential seismic motions of the terrain

(1) Structural elements located above and below the isolation interface must be sufficiently rigid in both directions, horizontal and vertical, so as to minimise the effects of differential seismic ground displacements. This does not apply to bridges or elevated structures, where piles and pillars located under the isolation interface can be deformable.

(2) Point **(1)** is deemed complied with in buildings if all of the following conditions are met:

a) a rigid diaphragm is arranged above and below the isolation system, consisting of a reinforced concrete slab or a tie-beam plating, dimensioned taking into account all appropriate local and global buckling modes. This rigid diaphragm is not necessary if the structure consists of rigid structures in drawer;

b) the devices forming the isolation system are fixed at both ends of the rigid diaphragms defined above, either directly or, if not possible, by vertical elements whose relative horizontal displacement in the seismic calculation situation must be less than 1/20 of the relative displacement of the isolation system.

### 10.5.4 Control of movements relative to the surrounding terrain and buildings

(1) Sufficient space should be provided between the isolated superstructure and the nearby terrain or constructions, to allow its displacement in all directions in the seismic calculation situation.

### 10.5.5 Design of the project of isolated buildings at its base

(1) The principles of the design of the project for isolated buildings at its base must be based on the indications in chapter **2** and section **4.2**, with the additional provisions indicated in this chapter.

## 10.6 Seismic action

(1) It should be assumed that the two horizontal components and the vertical component of the seismic action act simultaneously.

(2) Each component of the seismic action is defined in **3.2**, in terms of the elastic spectrum for the applicable local terrain conditions and for the ground acceleration calculation value $a_g$.

(3) Site-specific spectra that include effects from nearby sources should also be taken into account in buildings of class IV if the building is located less than 15 km from the nearest potentially active fault with a magnitude $M_w \geq 6{,}5$ These spectra must not be taken less than the normalised spectra defined in point **(2)** of this section.

(4) Combinations of seismic action components for buildings are indicated in section **4.3.3.5**.

(5) If time domain analysis is required, a set of at least three accelergrams must be used, which must meet the requirements of sections **3.2.3.1** and **3.2.3.2**.

## 10.7 Behaviour coefficient

(1) Unless otherwise indicated, in **(5)** of section **10.10**, the value of the behaviour coefficient should be taken equal to $q$ = 1.

## 10.8 Insulation-system properties

(1) The values of the physical and mechanical properties of the isolation system to be used in the calculation should be the most unfavourable to be achieved during the lifetime of the structure. They should reflect, where relevant, the influence of:

- load application speed;
- magnitude of simultaneous vertical charge;
- the magnitude of the simultaneous horizontal charge in the transverse direction;
- temperature;
- the change of properties over the expected lifetime.

(2) Accelerations and inertia forces induced by the earthquake must be assessed taking into account the maximum value of rigidity and damping and friction coefficients.

(3) Displacements must be assessed taking into account the minimum value of rigidity and damping and friction coefficients.

(4) In buildings of importance class I or II, the mean values of the physical and mechanical properties may be used, provided that the extreme values (maximum and minimum) do not differ by more than 15 % of the average values.

## 10.9 Structural analysis

### 10.9.1 General considerations

(1) The dynamic response of the structural system must be analysed in terms of accelerations, inertia forces and displacements.

(2) In buildings, torque effects should be taken into account, including the effects of accidental eccentricity as defined in **4.3.2**.

(3) The model of the isolation system must reflect with sufficient precision the spatial distribution of the isolation units so that the translation in both horizontal directions, the corresponding overturning effects and the rotation with respect to the vertical axis are adequately taken into account. The characteristics of the different types of units used in the isolation system must be adequately reflected.

### 10.9.2 Equivalent linear analysis

(1) If the conditions of point **(5)** of this section are met, the isolation system can be modelled by an

equivalent linear visco-elastic behaviour if the system is composed of devices such as elastomeric laminated supports, or by a hysteretic bilinear behaviour if the system consists of elastoplastic devices.

(2) If an equivalent linear model is used, the effective rigidity of each insulating unit (i.e. the stiffness drying value for the calculation value of the total displacement, $d_{db}$), respecting the provisions of point **(1)** of section **10.8**. The effective rigidity of the isolation system, $K_{eff}$, is the sum of the effective rigidities of the insulating units.

(3) If an equivalent linear model is used, the energy dissipation of the isolation system must be expressed in terms of an equivalent viscous damping, the 'effective damping' ($\xi_{eff}$). The dissipation of energy in the supports must be expressed from measurements of the energy dissipated in cycles with a frequency in the range of the natural frequencies of the modes considered. For higher modes, outside this range, the modal damping ratio of the entire structure must be that of the fixed-base superstructure.

(4) When the effective rigidity or effective damping of certain insulating units depends on the displacement calculation value, $d_{dc}$, an iterative procedure must be applied, until the difference between the assumed values and those calculated for $d_{dc}$ does not exceed 5 % of the assumed value.

(5) The behaviour of the isolation system can be considered as equivalent linear if all of the following conditions are met:

a) the effective rigidity of the isolation system defined in point **(2)** of this section is at least equal to 50 % of the effective rigidity for a displacement of 0.2 $d_{dc}$;

b) the effective damping ratio of the isolation system as defined in point **(3)** of this section does not exceed 30 %;

c) the force/displacement characteristics of the isolation system do not vary by more than 10 % depending on the speed of application of the load or due to vertical loads;

d) the increase of the recovery force in the isolation system for displacements between 0.5 $d_{dc}$ and $d_{dc}$ is at least equal to 2.5 % of the total gravity load above the isolation system;

(6) If the behaviour of the isolation system is considered as equivalent linear and the seismic action is defined through the elastic spectrum specified in point **(2)** of section **10.6**, a buffer correction must be carried out in accordance with point **(3)** of section **3.2.2.2**.

### 10.9.3 Simplified linear analysis

(1) The simplified linear analysis method considers two horizontal dynamic translations and superimposes the static effects of torsion. The superstructure is assumed to be a rigid solid moving above the isolation system, subject to the conditions of points **(2)** and **(3)** of this section. The effective period of translation is then:

$$T_{eff} = 2\pi \sqrt{\frac{M}{K_{eff}}} \tag{10.1}$$

Where

$M$ is the mass of the superstructure;

$K_{eff}$ is the effective horizontal rigidity of the isolation system, as defined in point **(2)** of section **10.9.2**.

(2) The torque movement around the vertical axis may be disregarded in the evaluation of effective horizontal rigidity and simplified linear analysis if, in each of the two main horizontal directions, the total eccentricity (including accidental eccentricity) between the rigidity centre of the isolation system and the vertical projection of the centre of gravity of the superstructure does not exceed 7.5 % of the length of the transverse superstructure to the horizontal direction considered. This is a condition for the application of the simplified linear analysis method.

(3) The simplified method can be applied to isolation systems with behaviour equivalent to linear damped, if they also meet all of the following conditions:

a) the distance from the site to the potentially active fault with a nearest magnitude $M_w \geq 6.5$ is greater than 15 km;

b) the largest of the floor dimensions of the superstructure is not greater than 50 m;

c) the infrastructure is sufficiently rigid to minimise the effects of differential terrain movements;

d) all devices are located above the elements of the infrastructure supporting the vertical loads;

e) effective period, $T_{eff}$, satisfies the following condition:

$$3T_f \leq T_{eff} \leq 3\ s \tag{10.2}$$

where $T_f$ is the fundamental period of the superstructure if its fixed base is assumed (estimated by a simplified equation).

(4) In order to apply the simplified method to isolation systems with a damped linear performance in buildings, all of the following conditions must be met in addition to the point **(3)** of this section:

a) the system resisting the lateral loads of the superstructure must be arranged on a regular and symmetrical basis along the two main axes on the floor of the structure;

b) the rotation by balancing at the base of the infrastructure must be disregarded;

c) the ratio between the vertical and horizontal rigidity of the isolation system must satisfy the following equation:

$$\frac{K_v}{K_{eff}} \geq 150 \tag{10.3}$$

d) the fundamental period in the vertical direction, $T_v$, must not be greater than 0.1 s, where:

$$T_V = 2\pi \sqrt{\frac{M}{K_V}} \tag{10.4}$$

(5) The displacement of the centre of rigidity due to seismic action must be obtained for each horizontal direction from the following equation:

$$d_{dc} = \frac{M\, S_e(T_{eff}, \xi_{eff})}{K_{eff,min}} \tag{10.5}$$

where $S_e(T_{eff}, \xi_{eff})$ is the spectral acceleration defined in section **3.2.2.2**, taking into account the appropriate value of effective damping according to point **(3)** of section **10.9.2**.

(6) The horizontal stresses applied at each level of the superstructure must be calculated, for each horizontal direction, by the following equation:

$$f_j = m_j\, S_e(T_{eff}, \xi_{eff}) \tag{10.6}$$

where $m_j$ is the mass at the level *j*.

(7) The system of forces considered in the point **(6)** induces the torsion effects due to the combination of natural and accidental eccentricities.

(8) If the condition of the point **(2)** of this section is complied with to negate the torque movement around the vertical axis, the torque effects on each individual isolation unit can be taken into account by amplifying in each direction the effects of the action defined in points **(5)** and **(6)** by a coefficient, $\delta_i$, given (for action in the direction *x*) by:

$$\delta_{xi} = 1 + \frac{e_{tot,y}}{r_y^2} y_i \tag{10.7}$$

Where

*and* is the horizontal direction transverse to the direction *x* considered;

$(x_i, y_i)$ are the coordinates of the isolation unit *i* referring to the effective stiffness centre;

$e_{tot,y}$ is the total eccentricity in the direction *y*;

$r_y$ is the torsion radius of the isolation system, given by the following equation:

$$r_y^2 = \sum \left( x_i^2 K_{yi} + y_i^2 K_{xi} \right) / \sum K_{xi} \tag{10.8}$$

with $K_{xi}$ and $K_{yi}$ being the effective rigidities of a given unit i in the directions *x* and *y*, respectively.

(9) The effects of torsion on the superstructure must be estimated in accordance with section**4.3.3.2.4**.

### 10.9.4 Simplified linear modal analysis

(1) If the performance of the devices can be considered as equivalent linear, but all the conditions indicated in points **(2)**, **(3)** nor – in the case this is applicable – **(4)** of section **10.9.3**, a modal analysis may be carried out in accordance with section **4.3.3.3**.

(2) If all conditions of point **(3)** and – in the case this is applicable – **(4)** of section **10.9.3** are met, a simplified analysis can be used considering horizontal displacements and torque around the vertical axis, and assuming that infrastructures and superstructures behave rigidly. In this case, the total eccentricity (including accidental eccentricity as defined in point **(1)** of the subsection **4.3.2**) of the superstructure mass must be taken into account in the analysis. The displacements at each point of the structure must then be calculated by combining translational and rotational displacements. This particularly affects the evaluation of the effective rigidity of each isolation unit. Inertia forces and times for checking insulating units and infrastructures and superstructures must be taken into account.

### 10.9.5 Analysis in the time domain

(1) If an isolation system cannot be represented by an equivalent linear model (i.e. if the conditions indicated in point **(5)** of section **10.9.2**) are not met, the seismic response should be assessed by means of a time domain analysis, using a law of behaviour of the devices that can adequately reproduce the behaviour of the system in the deformation range and speeds foreseen in the seismic calculation situation.

### 10.9.6 Non-structural elements

(1) In buildings, non-structural elements should be analysed in accordance with section **4.3.5**, taking due account of the dynamic effects of isolation (see points **(2)** and **(3)** of section **4.3.5.1**).

## 10.10 Safety checks in the ultimate limit state

(1) The infrastructure must be checked under the effect of the inertia forces applied directly to it, as well as under the forces and moments transmitted to it by the isolation system.

(2) The ultimate limit state of the infrastructure and superstructure should be checked using the values of $\gamma_M$ defined in the relevant chapters of this Resistant Standard.

(3) In buildings, safety checks concerning balance and strength in infrastructure and superstructure must be carried out in accordance with section **4.4**. There is no need to respect capacity dimensioning, global or local ductility conditions.

(4) In buildings, structural elements of infrastructure and superstructure can be dimensioned as non-dissipative. For concrete, steel or mixed steel and concrete buildings, the DCL ductility class can be adopted and section **5.3**, points **(2)**, **(3)** and **(4)** of section **6.1.2**, or points **(2)** and **(3)** of section **7.1.2**, respectively.

(5) In buildings, the strength condition of the structural elements of the superstructure can be complied with by taking into account the effects of seismic action divided by a performance coefficient not greater than 1.5.

(6) Taking into account the possible buckling break of the devices and using the corresponding values of $\gamma_M$, the strength of the isolation system should be assessed taking into account the coefficient $\gamma_x$ defined in point **(2)** of section **10.3**.

(7) Depending on the type of device concerned, the strength of the insulating units must be assessed at the ultimate limit state in terms of:

a) forces, taking into account the maximum possible vertical and horizontal forces in the seismic calculation situation, including rollover effects;

b) relative horizontal total displacement between the top and bottom sides of the unit. The total horizontal displacement must include the deformation due to the seismic action calculation and the effects of retraction, fluency, temperature and treading (if the superstructure is prestressed).

## Appendix A

### Elastic-response spectrum of displacements

**A.1** For long vibration period structures, seismic action can be represented in the form of a displacement response spectrum, $S_{De}(T)$ as illustrated in Figure A.1.

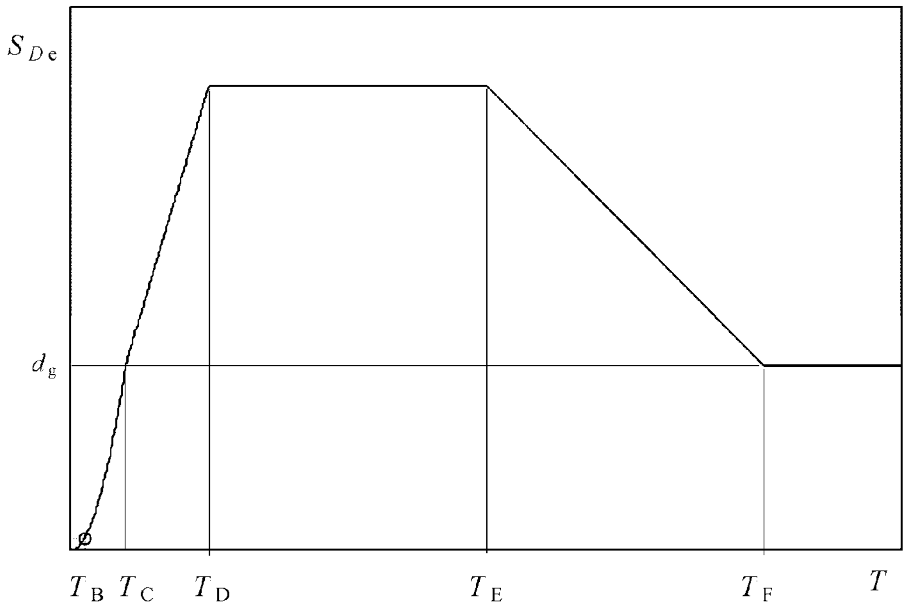



**Figure A.1 – Elastic displacement response spectrum**

**A.2** Until the control period $T_E$, the spectral ordering is obtained from equations (3.2) to (3.5), replacing $S_e(T)$ by $S_{De}(T)$ by means of equation (3.7). For vibration periods beyond $T_E$, the ordered displacement elastic response spectrum is obtained from equations (A.1) and (A.2).

$$T_E \leq T \leq T_F : \; S_{De}(T) = 0{,}025 a_g \cdot S \cdot T_C \cdot T_D \left[ 2{,}5\eta + \left( \frac{T - T_E}{T_F - T_E} \right) (1 - 2{,}5\eta) \right] \tag{A.1}$$

$$T \geq T_F : \; S_{De}(T) = d_g \tag{A.2}$$

where $S$, $T_C$ and $T_D$ are indicated in table 3.2, $\eta$ is obtained from equation (3.6) and $d_g$ is obtained from equation (3.12). Control periods $T_E$ and $T_F$ are presented in table A.1.

**Table A.1 - Additional control periods for displacement spectrum**

| Type of terrain | $T_E$ (s) | $T_F$ (s) |
|---|---|---|
| A | 4.5 | 10.0 |

| | | |
|---|---|---|
| B | 5.0 | 10.0 |
| C | 6.0 | 10.0 |
| D | 6.0 | 10.0 |
| E | 6.0 | 10.0 |

## Appendix B

## Determining target displacement for nonlinear static analysis (pushover analysis)

### B.1 General considerations

The target displacement is determined from the elastic response spectrum (see **3.2.2.2**). The capacity curve, which represents the ratio between the shear strength at the base and the displacement of the control knot, is determined according to section **4.3.3.4.2.3**.

The following ratio is assumed between the normalised forces and the $\bar{F}_i$ and the normalised displacements $\Phi_i$:

$$\bar{F}_i = m_i \Phi_i \tag{B.1}$$

where $m_i$ is the level mass $i$.

Displacements are normalised so that $\Phi_n$ = 1, where n is the control knot (usually, $n$ designates the roof level). Consequently, $\bar{F}_n = m_n$.

### B.2 Transformation into an equivalent system of one degree of freedom

The mass of a single degree of freedom equivalent system, $m^*$, is determined as follows:

$$m^* = \sum m_i \Phi_i = \sum \bar{F}_i \tag{B.2}$$

And the conversion coefficient is obtained from:

$$\Gamma = \frac{m^*}{\sum m_i \Phi_i^2} = \frac{\sum \bar{F}_i}{\sum \left( \frac{\bar{F}_i^2}{m_i} \right)} \tag{B.3}$$

The force $F^*$ and the displacement $d^*$ of an equivalent system of one degree of freedom are calculated as follows:

$$F^* = \frac{F_b}{\Gamma} \tag{B.4}$$

$$d^* = \frac{d_n}{\Gamma} \quad \text{(B.5)}$$

where $F_b$ and $d_n$ are, respectively, the shear strength at the base and the displacement of the control knot of a system with several degrees of freedom.

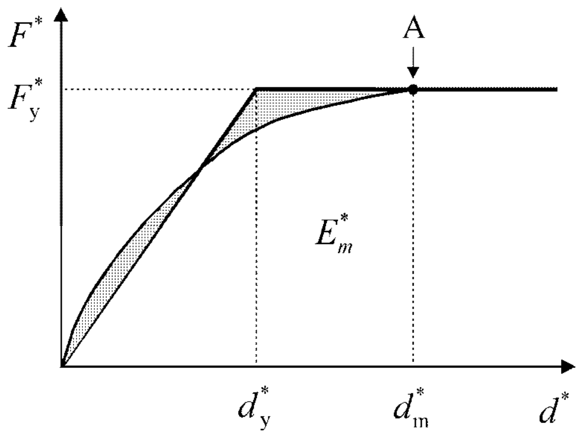
## B.3 Determining the perfect elastoplastic force-displacement ratio

The stress corresponding to the elastic limit $F_y^*$, which also represents the ultimate resistance of the idealised system, is equal to the shear strength at the base in the formation of the plastic mechanism. The initial rigidity of the idealised system is determined in such a way that the areas under the actual and idealised force-deformation curves are equal (see Figure B.1).

Based on this hypothesis, the displacement corresponding to the elastic limit of the idealised system of one degree of freedom $d_y^*$ is given by:

$$d_y^* = 2\left(d_m^* - \frac{E_m^*}{F_y^*}\right) \quad \text{(B.6)}$$

where $E_m^*$ is the actual deformation energy just at the time of the formation of the plastic mechanism.




Legend

A Plastic mechanism

**Figure B.1 – Determining the idealised force-displacement elastoplastic ratio**

## B.4 Determining the idealised system period equivalent to a single degree of freedom

The period $T^*$ of the idealised system equivalent to a single degree of freedom is determined by:

$$T^* = 2\pi \sqrt{\frac{m^* d_y^*}{F_y^*}} \quad \text{(B.7)}$$

## B.5 Determining the target displacement for the system equivalent to a single degree of freedom

The target displacement of the structure with period $T^*$ and unlimited elastic behaviour is given by:

$$d_{et}^* = S_e(T^*) \left[ \frac{T^*}{2\pi} \right]^2 \quad \text{(B.8)}$$

where $S_e(T^*)$ is the order of the elastic acceleration response spectrum in the period $T^*$.

For the determination of the target displacement $d_t^*$, different equations should be used for structures in the short period range and for structures in the medium and long period ranges, as indicated below. The limit period between the short and medium period range is $T_C$ (see Figure 3.1 and Tables 3.2 and 3.3).

a) $T^* < T_C$ (short period range)

If $F_y^* / m^* \geq S_e(T^*)$ the answer is elastic, and then

$$d_t^* = d_{et}^* \quad \text{(B.9)}$$

If $F_y^* / m^* < S_e(T^*)$ the answer is nonlinear, and

$$d_t^* = \frac{d_{et}^*}{q_u} \left( 1 + (q_u - 1) \frac{T_C}{T^*} \right) \geq d_{et}^* \quad \text{(B.10)}$$

where $q_u$ is the ratio between accelerations in the structure, with unlimited elastic behaviour, $S_e(T^*)$, and in the structure with limited resistance, $F_y^* / m^*$.

$$q_u = \frac{S_e(T^*) m^*}{F_y^*} \quad \text{(B.11)}$$

$d_t^*$ need not exceed 3 $d_{et}^*$.

b) $T^* \geq T_C$ (range of medium and long periods)

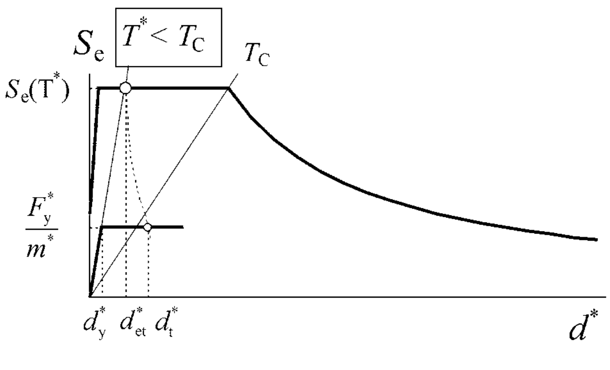
$$d_t^* = d_{et}^* \tag{B.12}$$

$d_t^*$ need not exceed 3 $d_{et}^*$.

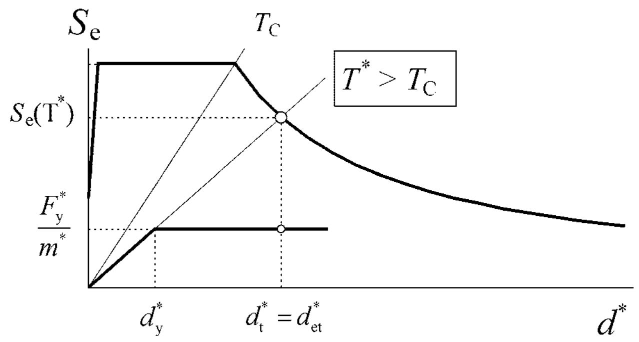
The ratio between the different magnitudes can be seen in Figures B.2 (a) and (b). The figures are represented in acceleration/displacement format. The period $T^*$ is represented by the radial line running from the origin of the coordinate system to the point of the elastic response spectrum defined by the $d_{et}^* = S_e(T^*)(T^*/2\pi)^2$ y $S_e(T^*)$. coordinates

**Iterative procedure (optional)**

If the target displacement $d_t^*$ determined in the 4th step (Chapter B.5) is very different from the displacement $d_m^*$ (Figure B.1) used in determining the idealised force-shift ratio elastoplastic in step 2 (Chapter B.3), an iterative procedure can be applied in which steps 2 to 4 are repeated using $d_t^*$ in the 2nd step (and the corresponding value of $F_y^*$), instead of $d_m^*$.




a) Short period range




b) Range of medium and long periods

**Figure B.2 – Determining target displacement for the system equivalent to a single degree of freedom**

## B.5 Determining target displacement for a system with various degrees of freedom

The target displacement of the system with various degrees of freedom is given by:

$$d_t = \Gamma d_t^* \tag{B.13}$$

The target displacement corresponds to the control knot.

**Appendix C**

**Dimensioning of the concrete slab of mixed steel and concrete beams in beam-pillar junctions of bending-resistant porticoes**

## C.1 General considerations

(1) This Appendix deals with the dimensioning of the slab and its connection to the steel structure, in bending-resistant porticoes in which the beams are mixed beams with T-section consisting of a steel profile with concrete compression head.

(2) The Appendix has been developed and experimentally validated for flex-resistant mixed porticoes with rigid connections and formation of plastic hinges on the beams. The equations indicated in this Appendix have not been validated for connections with partial resistance in which deformations are more localised in the junctions.

(3) Plastic hinges at the ends of a beam on a bending-resistant mixed portico should be ductile. According to this Appendix, two requirements must be met to ensure that a high bending ductility is obtained:

- premature buckling of the steel part should be avoided;
- should avoid premature crushing of slab concrete.

(4) The first condition imposes an upper limit on the cross section area $A_s$ of the longitudinal reinforcements in the effective width of the slab. The second condition imposes a lower limit on the cross section area $A_T$ of the cross-sectional reinforcement on the front of the pillar.

## C.2 Rules for the prevention of premature steel profile buckling

(1) Point **(4)** of section **7.6.1** applies.

## C.3 Rules for preventing premature crushing of concrete

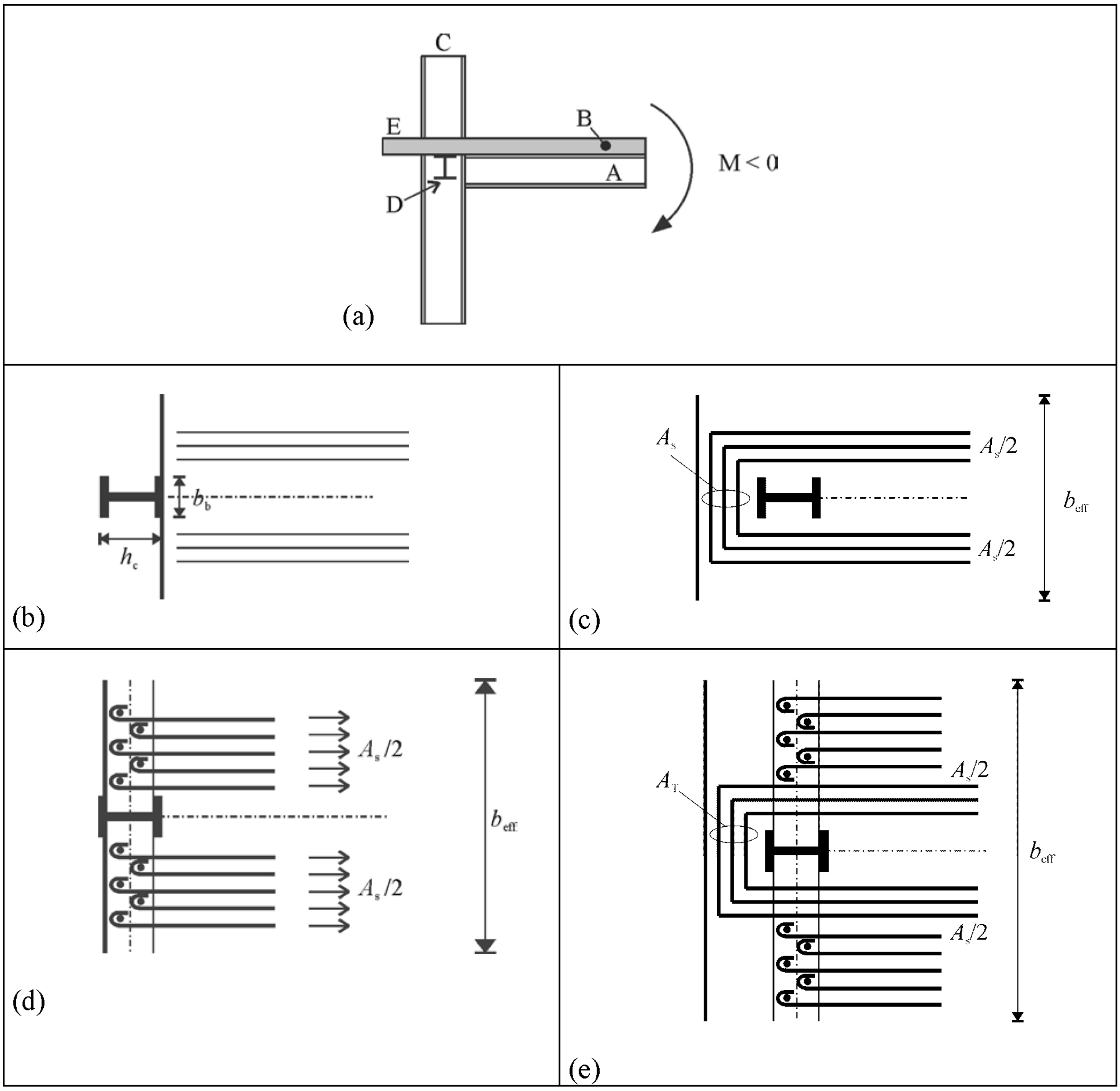
### C.3.1 External pillar. Bending of the pillar in the direction perpendicular to the façade; bending moment applied to the negative beam: *M* < 0

#### C.3.1.1 There is no steel façade beam; there is no cantilever concrete band [Figure C.1(b)]

(1) When there is no steel façade beam or cantilevered concrete band, the resistant moment (capacity) of the junction must be taken as the plastic moment of the steel beam exclusively.

#### C.3.1.2 There is no steel façade beam; there is cantilevered concrete band [Figure C.1(c)]

(1) When there is a cantilever concrete band, but no steel façade beam, Annex 30 of the Structural Code is applied for the calculation of the resistant moment of the union.




Legend

(a) Elevation view

(b) There is no cantilever concrete band; there is no steel façade beam (see **C.3.1.1**)

(c) There is cantilevered concrete band; there is no steel façade beam – (see **C.3.1.2**)

(d) There is no cantilever concrete band; steel façade beam exists – (see **C.3.1.3**)

(e) There is cantilevered concrete band; steel façade beam exists – (see **C.3.1.4**)

A Main beam

B Slab

C External pillar

D Steel façade beam

E Cantilever concrete band

**Figure C.1 – Configurations of external mixed beam-pillar junctions subjected to a negative bending moment according to a direction perpendicular to the façade**

#### C.3.1.3 There is steel façade beam; slab extending to the outer face of the pillar; there is no cantilever concrete band [Figure C.1(d)]

(1) Where there is a steel façade beam but not a cantilever concrete band, the strength of the junction may include the contribution of the slab reinforcements, assuming that the requirements set out in points **(2)** to **(7)** of this section.

(2) The slab reinforcements must be effectively anchored to the shearing connectors of the steel façade beam.

(3) The steel façade beam must be attached to the pillar.

(4) The cross-section area of steel reinforcements, $A_s$, should be such that the plasticisation of the steel reinforcements occurs before the breakage of the connectors and the façade beams.

(5) The cross-section area of steel reinforcements, $A_s$, and connectors must be arranged at a length equal to the effective width defined in **7.6.3** and Table 7.5 II.

(6) Connectors must be such that:

$$n \cdot P_{Rd} \geq 1,1\, F_{Rds} \tag{C.1}$$

Where

$n$ is the number of connectors within the effective width;

$P_{Rd}$ is the calculation value of the resistance of a connector;

$F_{Rds}$ is the calculation value of the resistance of the reinforcements within the effective width: $F_{Rds} = A_s \cdot f_{yd}$;

$f_{yd}$ is the calculation value of the elastic limit of steel of the slab reinforcement.

(7) The façade beam must be checked at bending, shearing and torsion under horizontal stress $F_{Rds}$ applied to connectors.

#### C.3.1.4 There is steel façade beam and cantilevered concrete band [Figure C.1(e)]

(1) When there is a steel façade beam and a cantilever concrete band, the resistant moment (capacity) of the union may include the contribution of: (a) the force transmitted through the steel façade beam, as described in section **C.3.1.3** (see point **(2)** of this section); and (b) the force transmitted through the mechanism described in Annex 30 to the Structural Code (see point **(3)** of this section).

(2) The part of the capacity due to the cross-sectional area of the reinforcements anchored to the transverse façade beam can be calculated according to section **C.3.1.3**, assuming that the requirements indicated in points **(2)** to **(7)** of section **C.3.1.3**

(3) The part of the capacity due to the cross-section area of the anchored trusses within the cantilever concrete band can be calculated according to section **C.3.1.2**.

### C.3.2 External pillar. Bending of the pillar in the direction perpendicular to the façade; bending moment applied to the positive beam: *M* > 0

#### C.3.2.1 There is no steel façade beam; slab extending to the inner face of the pillar [Figure C.2(b-c)]

(1) When the concrete slab is limited to the inner face of the pillar, the strength of the junction can be calculated based on the transfer of stress by direct compression (support) of the concrete on the face of the pillar. This capacity can be calculated from the compression force calculated according to point **(2)** of this section, assuming that the confinement reinforcement on the slab satisfies the point **(4)** of this section.

(2) The maximum value of the force transmitted to the slab can be taken as:

$$F_{\mathrm{Rd1}} = b_{\mathrm{b}} d_{\mathrm{eff}} f_{\mathrm{cd}} \tag{C.2}$$

Where

$d_{\mathrm{eff}}$ is the overall edge of the slab in the case of solid slabs or the thickness of the slab above the ribs of the ribbed sheet for mixed slabs;

$b_{\mathrm{b}}$ is the supporting width of the slab concrete on the pillar (see Figure 7.7).

(3) It is necessary to confine the concrete next to the wing of the pillar. The area of the cross-section of the confinement reinforcement must satisfy the following equation:

$$A_{\mathrm{T}} \geq 0,25 d_{\mathrm{eff}} b_{\mathrm{b}} \frac{0,15l - b_{\mathrm{b}}}{0,15l} \frac{f_{\mathrm{cd}}}{f_{\mathrm{yd,T}}} \tag{C.3}$$

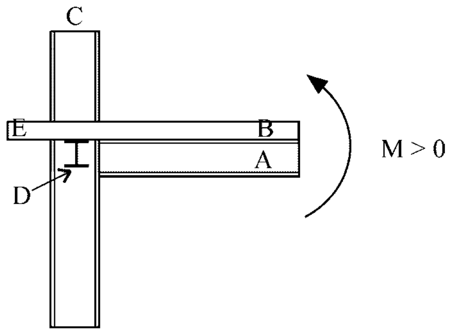
Where

$l$ is the beam span, as defined in point **(3)** of **7.6.3** and Figure 7.7;

$f_{\mathrm{yd,T}}$ is the calculation value of the elastic limit of the transverse reinforcement on the slab.

The cross section area, $A_{\mathrm{T}}$, of this reinforcement must be distributed evenly over a beam length equal to $b_{\mathrm{b}}$. The distance of the first round of reinforcement to the face of the pillar must not exceed 30 mm.

(4) The cross-section area, $A_{\mathrm{T}}$, of steel defined in point **(3)** can be arranged partially or entirely by means of reinforcement arranged for other purposes, for example for bending resistance of the slab.



(a)

Legend

(a) Elevation view

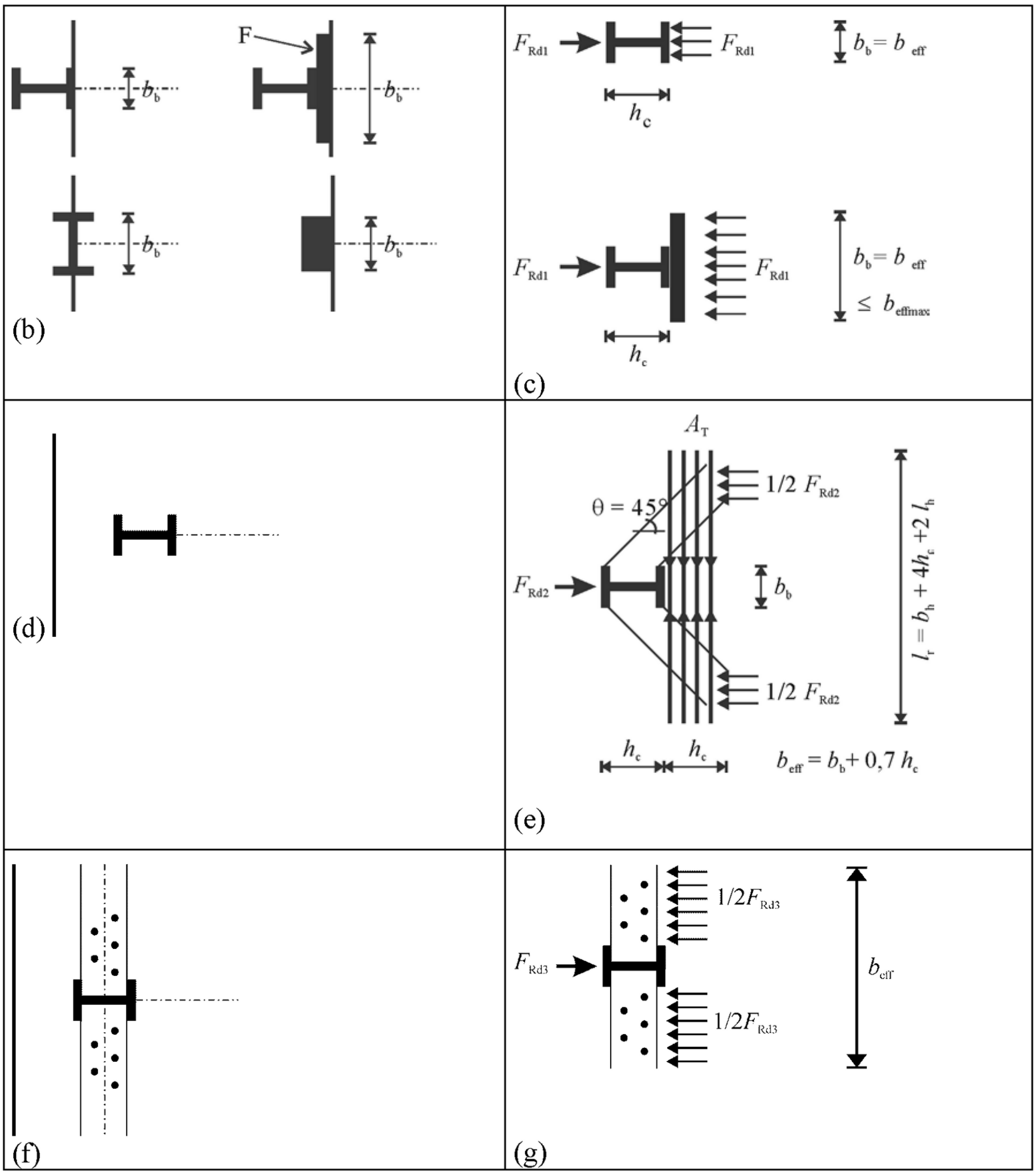
A Main beam

B Slab

C External pillar

D Steel façade beam

E Cantilever concrete band




Legend

(b) There is no cantilever concrete band – there is no steel façade beam – see section **C.3.2.1**

(c) Mechanism 1

(d) Slab extending to the outer face of the pillar or beyond as a cantilever concrete band; there is no steel façade beam (see **C.3.2.2)**

(e) Mechanism 2

(f) Slab extending to the outer face of the pillar or beyond as a cantilever concrete band; there is a steel façade beam (see **C.3.2.3)**

(g) Mechanism 3

F Additional device attached to the pillar to ensure support

**Figure C.2 - Configurations of external mixed beam-pillar junctions subjected to positive bending moments according to a direction perpendicular to the façade and possible transfer of forces from the slab**

#### C.3.2.2 There is no steel façade beam; slab extending to the outer face of the pillar or beyond as cantilever concrete band [Figure C.2 (c-d-e)]

(1) When there is no steel façade beam, the strength of the junction can be calculated from the compression force developed by combining the following two mechanisms:

<u>mechanism 1</u>: direct compression on the pillar. The calculation value of the force transferred through this mechanism must not exceed the value obtained from the following equation:

$$F_{Rd1} = b_b d_{eff} f_{cd} \tag{C.4}$$

<u>mechanism 2</u>: compressed concrete cranks inclined towards the faces of the pillar. If the angle of inclination is equal to 45°, the calculation value of the force transferred by this mechanism must not exceed the value obtained from the following equation:

$$F_{Rd2} = 0,7 h_c d_{eff} f_{cd} \tag{C.5}$$

Where

$h_c$ is the edge of the steel profile of the pillar.

(2) The total cross-section area of steel anchorage straps, $A_T$, must satisfy the following equation [see Figure C.2(e)]:

$$A_T \geq 0,5 \frac{F_{Rd2}}{f_{yd,T}} \tag{C.6}$$

(3) The steel area $A_T$ must be distributed along a beam length equal to $h_c$ and be fully anchored. The required length of the reinforcements is $L = b_b + 4\ h_c + 2\ l_b$, where $l_b$ is the anchoring length of such reinforcements according to Annex 19 of the Structural Code.

(4) The strength of the junction can be calculated from the calculation value of the maximum compression force that can be transmitted:

$$F_{Rd1} + F_{Rd2} = b_{eff} d_{eff} f_{cd} \tag{C.7}$$

Where

$b_{eff}$ is the effective width of the slab in the junction, as can be seen from section **7.6.3** and Table 7.5 II. In this case $b_{eff} = 0.7\ h_c + b_b$.

#### C.3.2.3 There is steel façade beam; slab extending to the outer face of the pillar or beyond as cantilever concrete band [Figure C.2 (c-e-f-g)]

(1) When steel façade beam exists, a third stress transfer mechanism $F_{Rd3}$ is activated to compression, involving the steel façade beam:

$$F_{Rd3} = n \cdot P_{Rd} \tag{C.8}$$

Where

$n$ is the number of connectors within the effective width, obtained from section **7.6.3** and Table 7.5 II;

$P_{Rd}$ is the calculation value of the resistance of a connector.

(2) Section **C.3.2.2** applies.

(3) The calculation value of the maximum compression force that can be transmitted is $b_{eff}\, d_{eff}\, f_{cd}$. It is transmitted if the following equation is complied with:

$$F_{Rd1} + F_{Rd2} + F_{Rd3} > b_{eff} d_{eff} f_{cd} \tag{C.9}$$

The 'complete' mixed plastic sturdy moment is obtained by choosing the $n$ connector number so that adequate $F_{Rd3}$ force is achieved. The maximum efficient width corresponds to the width $b_{eff}$ defined in section **7.6.3** and Table 7.5 II. In this case, $b_{eff} = 0.15\, l$.

### C.3.3 Internal pillar

#### C.3.3.1 There is no transverse beam [see Figure C.3(b-c)]

(1) Where there is no transverse beam, the strength of the junction can be calculated from the compression force developed by combining the following two mechanisms:

<u>mechanism 1</u>: direct compression on the pillar. The calculation value of the force transferred through this mechanism must not exceed the value obtained from the following equation:

$$F_{Rd1} = b_b d_{eff} f_{cd} \tag{C.10}$$

<u>mechanism 2</u>: compressed concrete cranks inclined 45° towards the faces of the pillar. The calculation value of the force transferred through this mechanism must not exceed the value obtained from the following equation:

$$F_{Rd2} = 0,7 h_c d_{eff} f_{cd} \tag{C.11}$$

(2) The total cross-section area of steel straps, $A$T, required for the development of mechanism 2, must satisfy the following equation:

$$A_T \geq 0,5 \frac{F_{Rd2}}{f_{yd,T}} \tag{C.12}$$

(3) This cross-sectional area, $A_T$, should be placed on each side of the pillar to consider the change in the bending moments sign.

(4) The calculation value of the compression force developed by the combination of the two mechanisms is:

$$F_{Rd1} + F_{Rd2} = (0{,}7\, h_c + b_b)\, d_{eff} f_{cd} \tag{C.13}$$

(5) The total effect of the action that takes place on the slab due to the ending moments on the opposite faces of the pillar, and which needs to be transmitted to the pillar through the combination of mechanisms 1 and 2, is the sum of the tensile force $F_{st}$ in the reinforcements parallel to the beam on the face of the pillar where the moment is negative and of the compressive force $F_{sc}$ in concrete on the face of the pillar where the moment is positive:

$$F_{st} + F_{sc} = A_s f_{yd} + b_{eff} d_{eff} f_{cd} \tag{C.14}$$

Where

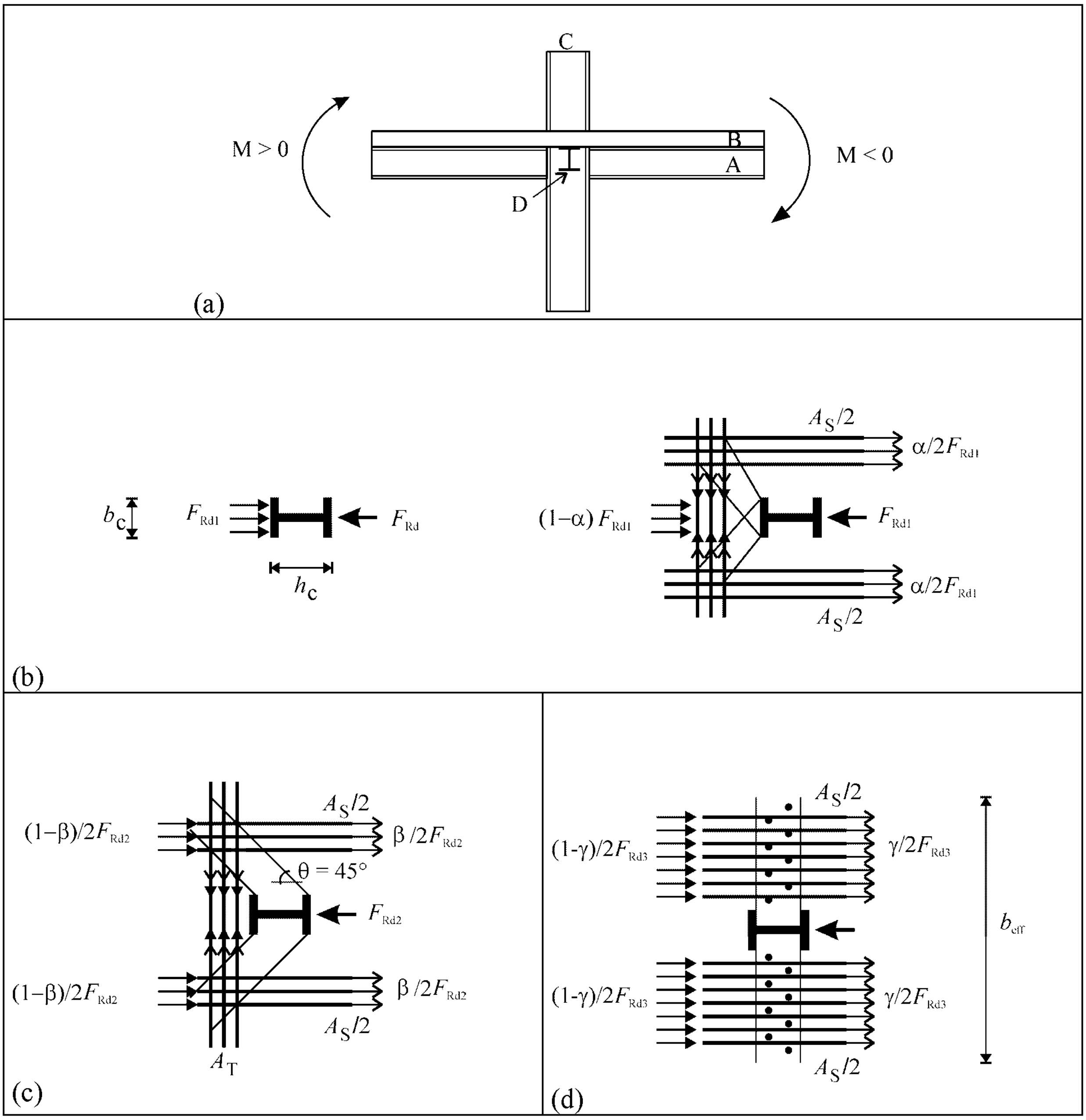
$A_s$ is the cross-section area of the reinforcements within the effective width subjected to negative moment, $b_{eff}$, specified in **7.6.3** and Table 7.5 II; and

$b_{eff}$ is the effective width subjected to positive moment, as specified in section **7.6.3** and Table 7.5 II. In this case, $b_{eff}$ = 0.15 $l$.

(6) In the dimensioning that seeks to achieve plasticisation in the wing of the pillar of the steel profile without crushing the concrete of the slab, the following condition must be respected:

$$1{,}2\,(F_{sc} + F_{st}) \leq F_{Rd1} + F_{Rd2} \tag{C.15}$$

If the above condition is not met, the junction's ability to transmit the forces from the slab to the pillar must be increased either by the presence of a transverse beam (see **C.3.3.2**), or by increasing the direct compression of the concrete on the pillar by means of additional devices (see **C.3.2.1**).




Legend

(a) Elevation view

(b) Mechanism 1

(c) Mechanism 2

(d) Mechanism 3

A Main beam

B Slab

C Internal pillar

D Transverse beam

**Figure C.3 – Possible transfer of slab forces in an internal mixed beam-pillar junction with and without transverse beam, subjected to a positive bending moment on one side and negative on the other**

#### C.3.3.2 There is transverse beam [Figure C.3(d)]

(1) When transverse beam exists, a third stress transfer mechanism $F_{Rd3}$ is activated to compression, involving the steel transverse beam.

$$F_{Rd3} = n \cdot P_{Rd} \tag{C.16}$$

Where

$n$ is the number of connectors in the effective width obtained using section **7.6.3** and Table 7.5 II;

$P_{Rd}$ is the calculation value of the resistance of a connector.

(2) Point **(2)** of section **C.3.3.1** applies to anchor straps.

(3) The calculation value of the compressive force, obtained by combination of the three mechanisms, is:

$$F_{Rd1} + F_{Rd2} + F_{Rd3} = \left(0,7\ h_c + b_b\right) d_{eff}\ f_{cd} + n \cdot P_{Rd} \tag{C.17}$$

where $n$ is the number of connectors in $b_{eff}$ for negative moment or positive moment, as defined in **7.6.3** and Table 7.5 II, taking the highest of the values obtained for the two beams confluent in the pillar.

(4) The point **(5)** of section **C.3.3.1** is applied for the calculation of the total effect of the action, $F_{st}$ + $F_{sc}$, developed on the slab due to the bending moments on the opposite faces of the pillar.

(5) In the dimensioning which seeks to achieve plasticisation on the lower side of the steel profile without crushing the concrete of the slab, the following condition must be respected:

$$1,2\ \left(F_{sc} + F_{st}\right) \leq F_{Rd1} + F_{Rd2} + F_{Rd3} \tag{C.18}$$

# Appendix D

## Specifications for project documents in the case of buildings

## D.1 Project documentation

Except in the case of constructions located in regions of very low seismicity, the project documentation must include, in addition, within the structural report, a specific chapter dedicated to the verification of the response of the construction to the earthquake, containing at least the following sections:

*a) Supporting report*
This will contain a list of the project criteria (based on the sections of Annex 1 referred to below). In particular:

- Basic parameters
  The classification adopted for the site (section 3.1.2) must be specified and the documentation on which it is based must be listed. The materiality class of the construction (section 4.2.5 for buildings) and the method of analysis (section 4.3.3) will also be defined.

- Definition of the resistant system
  The measures taken to comply with the basic project principles (section 4.2.1) must be indicated.
  Construction elements designated as primary and secondary seismic elements (section 4.2.2), as well as non-structural elements. The classification must be explicitly justified on the basis of the contribution of each system to rigidity in relation to horizontal actions in each direction concerned, explicitly indicating the measures taken to avoid interaction between structural and non-structural elements'.

- Classification of the resistant system
  Specify how the resistant system has been classified according to the types described in sections 5.2.2 (for reinforced concrete structures), 6.3.1 (metal structures, 7.3.1 (mixed structures), 8.3 (wooden structures) and 9.3 (brickwork structures).

- behaviour coefficients $q_o$ and $q$
  The basic value of the performance coefficient $q_o$ and the performance coefficient value $q$ which is introduced in section 3.2.2.5, adopted as a function of the resistant system and the class of ductility selected in sections 5.2.2 (for reinforced concrete structures), 6.3 (metal structures), 7.3 (mixed structures), 8.3 (wood structures) and 9.3 (brickwork structures) must be indicated.

- Calculation model
  In reinforced concrete, mixed and brickwork structures, the degree of reduction in rigidity due to the cracking of the elements must be explicitly indicated and justified.

- Checks
  Those relating to the size of the joint must be related between independent structural blocks or adjoining plots.
  It will also relate those referring to the stability of the enclosures and partitions against the normal actions to their plane.

*b) Calculation report*

In addition to the checks in the ultimate limit state USL and ELS service limit state, a hierarchy of checks must be established using capacity criteria (compared to the maximum forces that the united elements can transmit, unless these exceed those obtained in an overall analysis without reduction of forces by ductility). This will apply to:

- Foundations

- The junctions between prefabricated elements

- The pillars, compared to the requests induced by the brickwork cloths with which they are in contact (considering the local effect of the cloths that do not occupy the entire height of the pillar or which may result from its breakage). The check will include the possibility of sliding the construction joints between pillars and beams

- In the same sense, all knots must be checked according to the condition of equation (4.29) and similar

*c) Final report of work*

Any changes made to the initial project, including those involving an increase in the strength or rigidity of the modified elements, must be justified by a project carried out by a Competent Technician.

## Appendix E

## Reference peak horizontal acceleration values in soil type A, agR and parameter K (contribution coefficient)

| Long. | Lat. | K | $a_{gR}$ | Long. | Lat. | K | $a_{gR}$ | Long. | Lat. | K | $a_{gR}$ | Long. | Lat. | K | $a_{gR}$ |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| -3.0 | 35.2 | 1.0 | 0.155 | -5.4 | 36.2 | 1.1 | 0.093 | -4.9 | 36.4 | 1.0 | 0.135 | -3.5 | 36.5 | 1.0 | 0.171 |
| -2.9 | 35.2 | 1.0 | 0.150 | -5.3 | 36.2 | 1.1 | 0.097 | -4.8 | 36.4 | 1.0 | 0.142 | -3.4 | 36.5 | 1.0 | 0.178 |
| -2.8 | 35.2 | 1.2 | 0.143 | -5.2 | 36.2 | 1.1 | 0.103 | -4.7 | 36.4 | 1.0 | 0.147 | -3.3 | 36.5 | 1.0 | 0.185 |
| -3.0 | 35.3 | 1.0 | 0.160 | -5.1 | 36.2 | 1.1 | 0.107 | -4.6 | 36.4 | 1.0 | 0.148 | -3.2 | 36.5 | 1.0 | 0.191 |
| -2.9 | 35.3 | 1.0 | 0.156 | -5.0 | 36.2 | 1.1 | 0.119 | -4.5 | 36.4 | 1.0 | 0.147 | -3.1 | 36.5 | 1.0 | 0.192 |
| -2.8 | 35.3 | 1.0 | 0.149 | -4.9 | 36.2 | 1.1 | 0.128 | -4.4 | 36.4 | 1.0 | 0.144 | -3.0 | 36.5 | 1.0 | 0.193 |
| -3.0 | 35.4 | 1.0 | 0.165 | -4.8 | 36.2 | 1.0 | 0.133 | -4.3 | 36.4 | 1.0 | 0.141 | -2.9 | 36.5 | 1.0 | 0.193 |
| -2.9 | 35.4 | 1.0 | 0.160 | -4.7 | 36.2 | 1.0 | 0.135 | -4.2 | 36.4 | 1.0 | 0.138 | -2.8 | 36.5 | 1.0 | 0.193 |
| -2.8 | 35.4 | 1.0 | 0.155 | -4.6 | 36.2 | 1.0 | 0.134 | -4.1 | 36.4 | 1.0 | 0.134 | -2.7 | 36.5 | 1.0 | 0.193 |
| -5.4 | 35.8 | 1.1 | 0.093 | -6.8 | 36.3 | 1.3 | 0.160 | -4.0 | 36.4 | 1.0 | 0.133 | -2.6 | 36.5 | 1.0 | 0.190 |
| -5.3 | 35.8 | 1.1 | 0.095 | -6.7 | 36.3 | 1.3 | 0.159 | -3.9 | 36.4 | 1.0 | 0.134 | -2.5 | 36.5 | 1.0 | 0.185 |
| -5.2 | 35.8 | 1.1 | 0.099 | -6.6 | 36.3 | 1.3 | 0.149 | -3.8 | 36.4 | 1.0 | 0.134 | -2.4 | 36.5 | 1.0 | 0.174 |
| -6.2 | 35.9 | 1.3 | 0.111 | -6.5 | 36.3 | 1.3 | 0.138 | -3.7 | 36.4 | 1.0 | 0.137 | -2.3 | 36.5 | 1.0 | 0.151 |
| -5.9 | 35.9 | 1.2 | 0.090 | -6.4 | 36.3 | 1.3 | 0.129 | -3.6 | 36.4 | 1.0 | 0.142 | -2.2 | 36.5 | 1.0 | 0.134 |
| -5.6 | 35.9 | 1.1 | 0.088 | -6.3 | 36.3 | 1.3 | 0.120 | -3.5 | 36.4 | 1.0 | 0.154 | -2.1 | 36.5 | 1.0 | 0.122 |
| -5.4 | 35.9 | 1.1 | 0.091 | -6.2 | 36.3 | 1.3 | 0.116 | -3.4 | 36.4 | 1.0 | 0.171 | -2.0 | 36.5 | 1.0 | 0.107 |
| -5.3 | 35.9 | 1.1 | 0.095 | -6.1 | 36.3 | 1.2 | 0.099 | -3.3 | 36.4 | 1.0 | 0.184 | -1.9 | 36.5 | 1.0 | 0.094 |
| -5.2 | 35.9 | 1.1 | 0.099 | -6.0 | 36.3 | 1.2 | 0.097 | -3.2 | 36.4 | 1.0 | 0.189 | -1.8 | 36.5 | 1.0 | 0.086 |
| -6.4 | 36.0 | 1.3 | 0.136 | -5.9 | 36.3 | 1.2 | 0.097 | -3.1 | 36.4 | 1.0 | 0.191 | -1.7 | 36.5 | 1.0 | 0.081 |
| -6.2 | 36.0 | 1.3 | 0.118 | -5.8 | 36.3 | 1.2 | 0.088 | -3.0 | 36.4 | 1.0 | 0.193 | -1.6 | 36.5 | 1.0 | 0.077 |
| -6.1 | 36.0 | 1.2 | 0.099 | -5.7 | 36.3 | 1.1 | 0.086 | -2.9 | 36.4 | 1.0 | 0.193 | -6.9 | 36.6 | 1.3 | 0.165 |
| -6.0 | 36.0 | 1.2 | 0.095 | -5.6 | 36.3 | 1.1 | 0.087 | -2.8 | 36.4 | 1.0 | 0.192 | -6.8 | 36.6 | 1.3 | 0.159 |
| -5.9 | 36.0 | 1.2 | 0.094 | -5.5 | 36.3 | 1.1 | 0.091 | -2.7 | 36.4 | 1.0 | 0.189 | -6.7 | 36.6 | 1.3 | 0.156 |
| -5.8 | 36.0 | 1.2 | 0.090 | -5.4 | 36.3 | 1.1 | 0.095 | -2.6 | 36.4 | 1.0 | 0.183 | -6.6 | 36.6 | 1.3 | 0.146 |
| -5.7 | 36.0 | 1.2 | 0.086 | -5.3 | 36.3 | 1.1 | 0.099 | -2.5 | 36.4 | 1.0 | 0.172 | -6.5 | 36.6 | 1.3 | 0.136 |
| -5.6 | 36.0 | 1.1 | 0.086 | -5.2 | 36.3 | 1.1 | 0.104 | -2.4 | 36.4 | 1.0 | 0.149 | -6.4 | 36.6 | 1.3 | 0.127 |
| -5.5 | 36.0 | 1.1 | 0.087 | -5.1 | 36.3 | 1.1 | 0.113 | -2.0 | 36.4 | 1.0 | 0.096 | -6.3 | 36.6 | 1.3 | 0.119 |
| -5.4 | 36.0 | 1.1 | 0.090 | -5.0 | 36.3 | 1.1 | 0.124 | -1.6 | 36.4 | 1.0 | 0.078 | -6.2 | 36.6 | 1.3 | 0.114 |
| -5.3 | 36.0 | 1.1 | 0.094 | -4.9 | 36.3 | 1.0 | 0.133 | -6.9 | 36.5 | 1.3 | 0.166 | -6.1 | 36.6 | 1.2 | 0.099 |
| -5.2 | 36.0 | 1.1 | 0.100 | -4.8 | 36.3 | 1.0 | 0.140 | -6.8 | 36.5 | 1.3 | 0.160 | -6.0 | 36.6 | 1.2 | 0.095 |
| -4.9 | 36.0 | 1.1 | 0.121 | -4.7 | 36.3 | 1.0 | 0.139 | -6.7 | 36.5 | 1.3 | 0.158 | -5.9 | 36.6 | 1.1 | 0.092 |
| -6.5 | 36.1 | 1.3 | 0.139 | -4.6 | 36.3 | 1.0 | 0.139 | -6.6 | 36.5 | 1.3 | 0.147 | -5.8 | 36.6 | 1.1 | 0.090 |
| -6.4 | 36.1 | 1.3 | 0.133 | -4.5 | 36.3 | 1.0 | 0.140 | -6.5 | 36.5 | 1.3 | 0.136 | -5.7 | 36.6 | 1.1 | 0.093 |
| -6.3 | 36.1 | 1.3 | 0.120 | -4.4 | 36.3 | 1.0 | 0.138 | -6.4 | 36.5 | 1.3 | 0.128 | -5.6 | 36.6 | 1.1 | 0.103 |
| -6.2 | 36.1 | 1.3 | 0.117 | -4.3 | 36.3 | 1.0 | 0.136 | -6.3 | 36.5 | 1.3 | 0.120 | -5.5 | 36.6 | 1.1 | 0.115 |
| -6.1 | 36.1 | 1.2 | 0.099 | -4.1 | 36.3 | 1.0 | 0.130 | -6.2 | 36.5 | 1.3 | 0.114 | -5.4 | 36.6 | 1.1 | 0.129 |
| -6.0 | 36.1 | 1.2 | 0.096 | -3.9 | 36.3 | 1.0 | 0.129 | -6.1 | 36.5 | 1.2 | 0.099 | -5.3 | 36.6 | 1.0 | 0.134 |
| -5.9 | 36.1 | 1.2 | 0.092 | -3.7 | 36.3 | 1.0 | 0.132 | -6.0 | 36.5 | 1.2 | 0.096 | -5.2 | 36.6 | 1.0 | 0.133 |
| -5.8 | 36.1 | 1.2 | 0.089 | -3.4 | 36.3 | 1.0 | 0.158 | -5.9 | 36.5 | 1.2 | 0.090 | -5.1 | 36.6 | 1.0 | 0.130 |
| -5.7 | 36.1 | 1.1 | 0.085 | -2.9 | 36.3 | 1.0 | 0.187 | -5.8 | 36.5 | 1.1 | 0.090 | -5.0 | 36.6 | 1.0 | 0.134 |
| -5.6 | 36.1 | 1.1 | 0.085 | -2.6 | 36.3 | 1.0 | 0.161 | -5.7 | 36.5 | 1.1 | 0.089 | -4.9 | 36.6 | 1.0 | 0.137 |
| -5.5 | 36.1 | 1.1 | 0.086 | -2.4 | 36.3 | 1.0 | 0.131 | -5.6 | 36.5 | 1.1 | 0.097 | -4.8 | 36.6 | 1.0 | 0.144 |
| -5.4 | 36.1 | 1.1 | 0.092 | -6.9 | 36.4 | 1.3 | 0.165 | -5.5 | 36.5 | 1.1 | 0.103 | -4.7 | 36.6 | 1.0 | 0.150 |
| -5.3 | 36.1 | 1.1 | 0.094 | -6.8 | 36.4 | 1.3 | 0.160 | -5.4 | 36.5 | 1.1 | 0.112 | -4.6 | 36.6 | 1.0 | 0.153 |
| -5.2 | 36.1 | 1.1 | 0.101 | -6.7 | 36.4 | 1.3 | 0.159 | -5.3 | 36.5 | 1.1 | 0.118 | -4.5 | 36.6 | 1.0 | 0.155 |
| -5.1 | 36.1 | 1.1 | 0.107 | -6.6 | 36.4 | 1.3 | 0.150 | -5.2 | 36.5 | 1.1 | 0.124 | -4.4 | 36.6 | 1.0 | 0.156 |
| -5.0 | 36.1 | 1.1 | 0.116 | -6.5 | 36.4 | 1.3 | 0.137 | -5.1 | 36.5 | 1.0 | 0.125 | -4.3 | 36.6 | 1.0 | 0.157 |
| -4.9 | 36.1 | 1.1 | 0.123 | -6.4 | 36.4 | 1.3 | 0.128 | -5.0 | 36.5 | 1.0 | 0.131 | -4.2 | 36.6 | 1.0 | 0.158 |
| -4.8 | 36.1 | 1.1 | 0.129 | -6.3 | 36.4 | 1.3 | 0.120 | -4.9 | 36.5 | 1.0 | 0.136 | -4.1 | 36.6 | 1.0 | 0.155 |
| -6.7 | 36.2 | 1.3 | 0.158 | -6.2 | 36.4 | 1.3 | 0.114 | -4.8 | 36.5 | 1.0 | 0.143 | -4.0 | 36.6 | 1.0 | 0.154 |
| -6.6 | 36.2 | 1.3 | 0.148 | -6.1 | 36.4 | 1.2 | 0.099 | -4.7 | 36.5 | 1.0 | 0.148 | -3.9 | 36.6 | 1.0 | 0.154 |
| -6.5 | 36.2 | 1.3 | 0.138 | -6.0 | 36.4 | 1.2 | 0.095 | -4.6 | 36.5 | 1.0 | 0.151 | -3.8 | 36.6 | 1.0 | 0.157 |
| -6.4 | 36.2 | 1.3 | 0.130 | -5.9 | 36.4 | 1.2 | 0.097 | -4.5 | 36.5 | 1.0 | 0.152 | -3.7 | 36.6 | 1.0 | 0.165 |
| -6.3 | 36.2 | 1.3 | 0.120 | -5.8 | 36.4 | 1.1 | 0.089 | -4.4 | 36.5 | 1.0 | 0.152 | -3.6 | 36.6 | 1.0 | 0.178 |
| -6.2 | 36.2 | 1.3 | 0.116 | -5.7 | 36.4 | 1.1 | 0.087 | -4.3 | 36.5 | 1.0 | 0.151 | -3.5 | 36.6 | 1.0 | 0.181 |
| -6.1 | 36.2 | 1.2 | 0.099 | -5.6 | 36.4 | 1.1 | 0.092 | -4.2 | 36.5 | 1.0 | 0.148 | -3.4 | 36.6 | 1.0 | 0.183 |
| -6.0 | 36.2 | 1.2 | 0.098 | -5.5 | 36.4 | 1.1 | 0.095 | -4.1 | 36.5 | 1.0 | 0.142 | -3.3 | 36.6 | 1.0 | 0.188 |
| -5.9 | 36.2 | 1.2 | 0.097 | -5.4 | 36.4 | 1.1 | 0.100 | -4.0 | 36.5 | 1.0 | 0.141 | -3.2 | 36.6 | 1.0 | 0.191 |
| -5.8 | 36.2 | 1.2 | 0.089 | -5.3 | 36.4 | 1.1 | 0.105 | -3.9 | 36.5 | 1.0 | 0.143 | -3.1 | 36.6 | 1.0 | 0.193 |
| -5.7 | 36.2 | 1.1 | 0.084 | -5.2 | 36.4 | 1.1 | 0.112 | -3.8 | 36.5 | 1.0 | 0.143 | -3.0 | 36.6 | 1.0 | 0.194 |
| -5.6 | 36.2 | 1.1 | 0.088 | -5.1 | 36.4 | 1.1 | 0.122 | -3.7 | 36.5 | 1.0 | 0.147 | -2.9 | 36.6 | 1.0 | 0.194 |
| -5.5 | 36.2 | 1.1 | 0.087 | -5.0 | 36.4 | 1.0 | 0.129 | -3.6 | 36.5 | 1.0 | 0.157 | -2.8 | 36.6 | 1.0 | 0.193 |

| Long. | Lat. | K | $a_{gR}$ | Long. | Lat. | K | $a_{gR}$ | Long. | Lat. | K | $a_{gR}$ | Long. | Lat. | K | $a_{gR}$ |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| -2.7 | 36.6 | 1.0 | 0.192 | -2.1 | 36.7 | 1.0 | 0.151 | -1.7 | 36.8 | 1.0 | 0.095 | -1.5 | 36.9 | 1.0 | 0.085 |
| -2.6 | 36.6 | 1.0 | 0.191 | -2.0 | 36.7 | 1.0 | 0.131 | -1.6 | 36.8 | 1.0 | 0.086 | -1.4 | 36.9 | 1.0 | 0.079 |
| -2.5 | 36.6 | 1.0 | 0.189 | -1.9 | 36.7 | 1.0 | 0.112 | -1.5 | 36.8 | 1.0 | 0.080 | -1.3 | 36.9 | 1.0 | 0.074 |
| -2.4 | 36.6 | 1.0 | 0.184 | -1.8 | 36.7 | 1.0 | 0.098 | -1.4 | 36.8 | 1.0 | 0.075 | -7.4 | 37.0 | 1.3 | 0.167 |
| -2.3 | 36.6 | 1.0 | 0.174 | -1.7 | 36.7 | 1.0 | 0.088 | -7.3 | 36.9 | 1.3 | 0.164 | -7.3 | 37.0 | 1.3 | 0.165 |
| -2.2 | 36.6 | 1.0 | 0.152 | -1.6 | 36.7 | 1.0 | 0.081 | -7.2 | 36.9 | 1.3 | 0.164 | -7.2 | 37.0 | 1.3 | 0.164 |
| -2.1 | 36.6 | 1.0 | 0.134 | -1.5 | 36.7 | 1.0 | 0.077 | -7.1 | 36.9 | 1.3 | 0.162 | -7.1 | 37.0 | 1.3 | 0.160 |
| -2.0 | 36.6 | 1.0 | 0.118 | -1.4 | 36.7 | 1.0 | 0.073 | -7.0 | 36.9 | 1.3 | 0.161 | -7.0 | 37.0 | 1.3 | 0.156 |
| -1.9 | 36.6 | 1.0 | 0.102 | -7.1 | 36.8 | 1.3 | 0.168 | -6.9 | 36.9 | 1.3 | 0.157 | -6.9 | 37.0 | 1.3 | 0.151 |
| -1.8 | 36.6 | 1.0 | 0.091 | -7.0 | 36.8 | 1.3 | 0.164 | -6.8 | 36.9 | 1.3 | 0.155 | -6.8 | 37.0 | 1.3 | 0.149 |
| -1.7 | 36.6 | 1.0 | 0.083 | -6.9 | 36.8 | 1.3 | 0.158 | -6.7 | 36.9 | 1.3 | 0.149 | -6.7 | 37.0 | 1.3 | 0.145 |
| -1.6 | 36.6 | 1.0 | 0.078 | -6.8 | 36.8 | 1.3 | 0.156 | -6.6 | 36.9 | 1.3 | 0.140 | -6.6 | 37.0 | 1.3 | 0.134 |
| -1.5 | 36.6 | 1.0 | 0.074 | -6.7 | 36.8 | 1.3 | 0.150 | -6.5 | 36.9 | 1.2 | 0.138 | -6.5 | 37.0 | 1.2 | 0.120 |
| -7.0 | 36.7 | 1.3 | 0.168 | -6.6 | 36.8 | 1.3 | 0.141 | -6.4 | 36.9 | 1.2 | 0.120 | -6.4 | 37.0 | 1.2 | 0.120 |
| -6.9 | 36.7 | 1.3 | 0.162 | -6.5 | 36.8 | 1.3 | 0.135 | -6.3 | 36.9 | 1.2 | 0.120 | -6.3 | 37.0 | 1.2 | 0.117 |
| -6.8 | 36.7 | 1.3 | 0.158 | -6.4 | 36.8 | 1.2 | 0.126 | -6.2 | 36.9 | 1.2 | 0.105 | -6.2 | 37.0 | 1.2 | 0.102 |
| -6.7 | 36.7 | 1.3 | 0.155 | -6.3 | 36.8 | 1.2 | 0.122 | -6.1 | 36.9 | 1.2 | 0.095 | -6.1 | 37.0 | 1.2 | 0.094 |
| -6.6 | 36.7 | 1.3 | 0.145 | -6.2 | 36.8 | 1.2 | 0.115 | -6.0 | 36.9 | 1.1 | 0.093 | -6.0 | 37.0 | 1.1 | 0.096 |
| -6.5 | 36.7 | 1.3 | 0.136 | -6.1 | 36.8 | 1.2 | 0.096 | -5.9 | 36.9 | 1.1 | 0.099 | -5.9 | 37.0 | 1.1 | 0.100 |
| -6.4 | 36.7 | 1.3 | 0.127 | -6.0 | 36.8 | 1.2 | 0.095 | -5.8 | 36.9 | 1.1 | 0.104 | -5.8 | 37.0 | 1.1 | 0.108 |
| -6.3 | 36.7 | 1.2 | 0.119 | -5.9 | 36.8 | 1.1 | 0.098 | -5.7 | 36.9 | 1.1 | 0.114 | -5.7 | 37.0 | 1.1 | 0.112 |
| -6.2 | 36.7 | 1.2 | 0.113 | -5.8 | 36.8 | 1.1 | 0.101 | -5.6 | 36.9 | 1.0 | 0.120 | -5.6 | 37.0 | 1.1 | 0.116 |
| -6.1 | 36.7 | 1.2 | 0.097 | -5.7 | 36.8 | 1.1 | 0.110 | -5.5 | 36.9 | 1.0 | 0.126 | -5.5 | 37.0 | 1.0 | 0.119 |
| -6.0 | 36.7 | 1.2 | 0.095 | -5.6 | 36.8 | 1.0 | 0.122 | -5.4 | 36.9 | 1.0 | 0.132 | -5.4 | 37.0 | 1.0 | 0.124 |
| -5.9 | 36.7 | 1.1 | 0.095 | -5.5 | 36.8 | 1.0 | 0.132 | -5.3 | 36.9 | 1.0 | 0.139 | -5.3 | 37.0 | 1.0 | 0.130 |
| -5.8 | 36.7 | 1.1 | 0.094 | -5.4 | 36.8 | 1.0 | 0.139 | -5.2 | 36.9 | 1.0 | 0.143 | -5.2 | 37.0 | 1.0 | 0.132 |
| -5.7 | 36.7 | 1.1 | 0.102 | -5.3 | 36.8 | 1.0 | 0.143 | -5.1 | 36.9 | 1.0 | 0.144 | -5.1 | 37.0 | 1.0 | 0.140 |
| -5.6 | 36.7 | 1.1 | 0.114 | -5.2 | 36.8 | 1.0 | 0.146 | -5.0 | 36.9 | 1.0 | 0.146 | -5.0 | 37.0 | 1.0 | 0.147 |
| -5.5 | 36.7 | 1.0 | 0.129 | -5.1 | 36.8 | 1.0 | 0.144 | -4.9 | 36.9 | 1.0 | 0.151 | -4.9 | 37.0 | 1.0 | 0.156 |
| -5.4 | 36.7 | 1.0 | 0.138 | -5.0 | 36.8 | 1.0 | 0.142 | -4.8 | 36.9 | 1.0 | 0.154 | -4.8 | 37.0 | 1.0 | 0.168 |
| -5.3 | 36.7 | 1.0 | 0.139 | -4.9 | 36.8 | 1.0 | 0.144 | -4.7 | 36.9 | 1.0 | 0.161 | -4.7 | 37.0 | 1.0 | 0.180 |
| -5.2 | 36.7 | 1.0 | 0.142 | -4.8 | 36.8 | 1.0 | 0.147 | -4.6 | 36.9 | 1.0 | 0.171 | -4.6 | 37.0 | 1.0 | 0.190 |
| -5.1 | 36.7 | 1.0 | 0.140 | -4.7 | 36.8 | 1.0 | 0.153 | -4.5 | 36.9 | 1.0 | 0.183 | -4.5 | 37.0 | 1.0 | 0.198 |

| | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| -5.0 | 36.7 | 1.0 | 0.138 | -4.6 | 36.8 | 1.0 | 0.158 | -4.4 | 36.9 | 1.0 | 0.195 | -4.4 | 37.0 | 1.0 | 0.209 |
| -4.9 | 36.7 | 1.0 | 0.140 | -4.5 | 36.8 | 1.0 | 0.164 | -4.3 | 36.9 | 1.0 | 0.209 | -4.3 | 37.0 | 1.0 | 0.227 |
| -4.8 | 36.7 | 1.0 | 0.145 | -4.4 | 36.8 | 1.0 | 0.170 | -4.2 | 36.9 | 1.0 | 0.230 | -4.2 | 37.0 | 1.0 | 0.245 |
| -4.7 | 36.7 | 1.0 | 0.151 | -4.3 | 36.8 | 1.0 | 0.178 | -4.1 | 36.9 | 1.0 | 0.232 | -4.1 | 37.0 | 1.0 | 0.246 |
| -4.6 | 36.7 | 1.0 | 0.154 | -4.2 | 36.8 | 1.0 | 0.188 | -4.0 | 36.9 | 1.0 | 0.238 | -4.0 | 37.0 | 1.0 | 0.252 |
| -4.5 | 36.7 | 1.0 | 0.158 | -4.1 | 36.8 | 1.0 | 0.197 | -3.9 | 36.9 | 1.0 | 0.236 | -3.9 | 37.0 | 1.0 | 0.254 |
| -4.4 | 36.7 | 1.0 | 0.161 | -4.0 | 36.8 | 1.0 | 0.205 | -3.8 | 36.9 | 1.0 | 0.236 | -3.8 | 37.0 | 1.0 | 0.258 |
| -4.3 | 36.7 | 1.0 | 0.163 | -3.9 | 36.8 | 1.0 | 0.209 | -3.7 | 36.9 | 1.0 | 0.236 | -3.7 | 37.0 | 1.0 | 0.260 |
| -4.2 | 36.7 | 1.0 | 0.167 | -3.8 | 36.8 | 1.0 | 0.212 | -3.6 | 36.9 | 1.0 | 0.233 | -3.6 | 37.0 | 1.0 | 0.255 |
| -4.1 | 36.7 | 1.0 | 0.170 | -3.7 | 36.8 | 1.0 | 0.208 | -3.5 | 36.9 | 1.0 | 0.213 | -3.5 | 37.0 | 1.0 | 0.230 |
| -4.0 | 36.7 | 1.0 | 0.175 | -3.6 | 36.8 | 1.0 | 0.205 | -3.4 | 36.9 | 1.0 | 0.188 | -3.4 | 37.0 | 1.0 | 0.183 |
| -3.9 | 36.7 | 1.0 | 0.181 | -3.5 | 36.8 | 1.0 | 0.196 | -3.3 | 36.9 | 1.0 | 0.183 | -3.3 | 37.0 | 1.0 | 0.166 |
| -3.8 | 36.7 | 1.0 | 0.186 | -3.4 | 36.8 | 1.0 | 0.192 | -3.2 | 36.9 | 1.0 | 0.182 | -3.2 | 37.0 | 1.0 | 0.160 |
| -3.7 | 36.7 | 1.0 | 0.188 | -3.3 | 36.8 | 1.0 | 0.192 | -3.1 | 36.9 | 1.0 | 0.185 | -3.1 | 37.0 | 1.0 | 0.157 |
| -3.6 | 36.7 | 1.0 | 0.188 | -3.2 | 36.8 | 1.0 | 0.194 | -3.0 | 36.9 | 1.0 | 0.186 | -3.0 | 37.0 | 1.0 | 0.158 |
| -3.5 | 36.7 | 1.0 | 0.187 | -3.1 | 36.8 | 1.0 | 0.194 | -2.9 | 36.9 | 1.0 | 0.185 | -2.9 | 37.0 | 1.0 | 0.162 |
| -3.4 | 36.7 | 1.0 | 0.189 | -3.0 | 36.8 | 1.0 | 0.194 | -2.8 | 36.9 | 1.0 | 0.186 | -2.8 | 37.0 | 1.0 | 0.167 |
| -3.3 | 36.7 | 1.0 | 0.190 | -2.9 | 36.8 | 1.0 | 0.192 | -2.7 | 36.9 | 1.0 | 0.186 | -2.7 | 37.0 | 1.0 | 0.173 |
| -3.2 | 36.7 | 1.0 | 0.194 | -2.8 | 36.8 | 1.0 | 0.191 | -2.6 | 36.9 | 1.0 | 0.187 | -2.6 | 37.0 | 1.0 | 0.177 |
| -3.1 | 36.7 | 1.0 | 0.195 | -2.7 | 36.8 | 1.0 | 0.189 | -2.5 | 36.9 | 1.0 | 0.187 | -2.5 | 37.0 | 1.0 | 0.180 |
| -3.0 | 36.7 | 1.0 | 0.194 | -2.6 | 36.8 | 1.0 | 0.189 | -2.4 | 36.9 | 1.0 | 0.187 | -2.4 | 37.0 | 1.0 | 0.183 |
| -2.9 | 36.7 | 1.0 | 0.193 | -2.5 | 36.8 | 1.0 | 0.189 | -2.3 | 36.9 | 1.0 | 0.188 | -2.3 | 37.0 | 1.0 | 0.183 |
| -2.8 | 36.7 | 1.0 | 0.192 | -2.4 | 36.8 | 1.0 | 0.189 | -2.2 | 36.9 | 1.0 | 0.186 | -2.2 | 37.0 | 1.0 | 0.183 |
| -2.7 | 36.7 | 1.0 | 0.191 | -2.3 | 36.8 | 1.0 | 0.188 | -2.1 | 36.9 | 1.0 | 0.183 | -2.1 | 37.0 | 1.0 | 0.184 |
| -2.6 | 36.7 | 1.0 | 0.190 | -2.2 | 36.8 | 1.0 | 0.184 | -2.0 | 36.9 | 1.0 | 0.173 | -2.0 | 37.0 | 1.0 | 0.182 |
| -2.5 | 36.7 | 1.0 | 0.190 | -2.1 | 36.8 | 1.0 | 0.174 | -1.9 | 36.9 | 1.0 | 0.148 | -1.9 | 37.0 | 1.0 | 0.171 |
| -2.4 | 36.7 | 1.0 | 0.188 | -2.0 | 36.8 | 1.0 | 0.150 | -1.8 | 36.9 | 1.0 | 0.120 | -1.8 | 37.0 | 1.0 | 0.143 |
| -2.3 | 36.7 | 1.0 | 0.184 | -1.9 | 36.8 | 1.0 | 0.127 | -1.7 | 36.9 | 1.0 | 0.103 | -1.7 | 37.0 | 1.0 | 0.115 |
| -2.2 | 36.7 | 1.0 | 0.174 | -1.8 | 36.8 | 1.0 | 0.107 | -1.6 | 36.9 | 1.0 | 0.093 | -1.6 | 37.0 | 1.0 | 0.101 |

| Long. | Lat. | K | $a_{gR}$ | Long. | Lat. | K | $a_{gR}$ | Long. | Lat. | K | $a_{gR}$ | Long. | Lat. | K | $a_{gR}$ |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| -1.5 | 37.0 | 1.0 | 0.091 | -1.7 | 37.1 | 1.0 | 0.131 | -2.0 | 37.2 | 1.0 | 0.171 | -2.3 | 37.3 | 1.0 | 0.158 |
| -1.4 | 37.0 | 1.0 | 0.084 | -1.6 | 37.1 | 1.0 | 0.110 | -1.9 | 37.2 | 1.0 | 0.184 | -2.2 | 37.3 | 1.0 | 0.161 |
| -1.3 | 37.0 | 1.0 | 0.078 | -1.5 | 37.1 | 1.0 | 0.097 | -1.8 | 37.2 | 1.0 | 0.179 | -2.1 | 37.3 | 1.0 | 0.164 |
| -1.2 | 37.0 | 1.0 | 0.074 | -1.4 | 37.1 | 1.0 | 0.089 | -1.7 | 37.2 | 1.0 | 0.150 | -2.0 | 37.3 | 1.0 | 0.173 |
| -7.5 | 37.1 | 1.3 | 0.161 | -1.3 | 37.1 | 1.0 | 0.083 | -1.6 | 37.2 | 1.0 | 0.122 | -1.9 | 37.3 | 1.0 | 0.187 |
| -7.4 | 37.1 | 1.3 | 0.160 | -1.2 | 37.1 | 1.0 | 0.078 | -1.5 | 37.2 | 1.0 | 0.105 | -1.8 | 37.3 | 1.0 | 0.186 |
| -7.3 | 37.1 | 1.3 | 0.159 | -1.1 | 37.1 | 1.0 | 0.074 | -1.4 | 37.2 | 1.0 | 0.095 | -1.7 | 37.3 | 1.0 | 0.166 |
| -7.2 | 37.1 | 1.3 | 0.156 | -7.5 | 37.2 | 1.3 | 0.153 | -1.3 | 37.2 | 1.0 | 0.088 | -1.6 | 37.3 | 1.0 | 0.137 |
| -7.1 | 37.1 | 1.3 | 0.156 | -7.4 | 37.2 | 1.3 | 0.151 | -1.2 | 37.2 | 1.0 | 0.084 | -1.5 | 37.3 | 1.0 | 0.114 |
| -7.0 | 37.1 | 1.3 | 0.155 | -7.3 | 37.2 | 1.3 | 0.152 | -1.1 | 37.2 | 1.0 | 0.079 | -1.4 | 37.3 | 1.0 | 0.101 |
| -6.9 | 37.1 | 1.3 | 0.148 | -7.2 | 37.2 | 1.3 | 0.150 | -1.0 | 37.2 | 1.0 | 0.076 | -1.3 | 37.3 | 1.0 | 0.094 |
| -6.8 | 37.1 | 1.3 | 0.146 | -7.1 | 37.2 | 1.3 | 0.146 | -7.4 | 37.3 | 1.3 | 0.142 | -1.2 | 37.3 | 1.0 | 0.089 |
| -6.7 | 37.1 | 1.3 | 0.138 | -7.0 | 37.2 | 1.3 | 0.143 | -7.3 | 37.3 | 1.3 | 0.141 | -1.1 | 37.3 | 1.0 | 0.086 |
| -6.6 | 37.1 | 1.3 | 0.128 | -6.9 | 37.2 | 1.3 | 0.138 | -7.2 | 37.3 | 1.3 | 0.140 | -1.0 | 37.3 | 1.0 | 0.083 |
| -6.5 | 37.1 | 1.2 | 0.121 | -6.8 | 37.2 | 1.3 | 0.135 | -7.1 | 37.3 | 1.3 | 0.135 | -0.8 | 37.3 | 1.0 | 0.077 |
| -6.4 | 37.1 | 1.2 | 0.120 | -6.7 | 37.2 | 1.3 | 0.132 | -7.0 | 37.3 | 1.3 | 0.130 | -0.6 | 37.3 | 1.0 | 0.072 |
| -6.3 | 37.1 | 1.2 | 0.109 | -6.6 | 37.2 | 1.3 | 0.125 | -6.9 | 37.3 | 1.3 | 0.125 | -7.6 | 37.4 | 1.3 | 0.141 |
| -6.2 | 37.1 | 1.2 | 0.101 | -6.5 | 37.2 | 1.2 | 0.118 | -6.8 | 37.3 | 1.3 | 0.120 | -7.5 | 37.4 | 1.3 | 0.136 |
| -6.1 | 37.1 | 1.2 | 0.094 | -6.4 | 37.2 | 1.2 | 0.114 | -6.7 | 37.3 | 1.3 | 0.115 | -7.4 | 37.4 | 1.3 | 0.132 |
| -6.0 | 37.1 | 1.1 | 0.094 | -6.3 | 37.2 | 1.2 | 0.107 | -6.6 | 37.3 | 1.3 | 0.111 | -7.3 | 37.4 | 1.3 | 0.128 |
| -5.9 | 37.1 | 1.1 | 0.096 | -6.2 | 37.2 | 1.2 | 0.100 | -6.5 | 37.3 | 1.2 | 0.107 | -7.2 | 37.4 | 1.3 | 0.124 |
| -5.8 | 37.1 | 1.1 | 0.100 | -6.1 | 37.2 | 1.2 | 0.093 | -6.4 | 37.3 | 1.2 | 0.103 | -7.1 | 37.4 | 1.3 | 0.120 |
| -5.7 | 37.1 | 1.1 | 0.104 | -6.0 | 37.2 | 1.1 | 0.093 | -6.3 | 37.3 | 1.2 | 0.099 | -7.0 | 37.4 | 1.3 | 0.116 |
| -5.6 | 37.1 | 1.1 | 0.107 | -5.9 | 37.2 | 1.1 | 0.093 | -6.2 | 37.3 | 1.2 | 0.095 | -6.9 | 37.4 | 1.3 | 0.112 |
| -5.5 | 37.1 | 1.1 | 0.113 | -5.8 | 37.2 | 1.1 | 0.095 | -6.1 | 37.3 | 1.2 | 0.091 | -6.8 | 37.4 | 1.3 | 0.108 |
| -5.4 | 37.1 | 1.0 | 0.117 | -5.7 | 37.2 | 1.1 | 0.098 | -6.0 | 37.3 | 1.1 | 0.091 | -6.7 | 37.4 | 1.3 | 0.103 |
| -5.3 | 37.1 | 1.0 | 0.123 | -5.6 | 37.2 | 1.1 | 0.101 | -5.9 | 37.3 | 1.1 | 0.091 | -6.6 | 37.4 | 1.3 | 0.099 |
| -5.2 | 37.1 | 1.0 | 0.127 | -5.5 | 37.2 | 1.1 | 0.104 | -5.8 | 37.3 | 1.1 | 0.092 | -6.5 | 37.4 | 1.2 | 0.095 |
| -5.1 | 37.1 | 1.0 | 0.128 | -5.4 | 37.2 | 1.1 | 0.107 | -5.7 | 37.3 | 1.1 | 0.094 | -6.4 | 37.4 | 1.2 | 0.093 |
| -5.0 | 37.1 | 1.0 | 0.136 | -5.3 | 37.2 | 1.1 | 0.112 | -5.6 | 37.3 | 1.1 | 0.097 | -6.3 | 37.4 | 1.2 | 0.091 |
| -4.9 | 37.1 | 1.0 | 0.154 | -5.2 | 37.2 | 1.0 | 0.117 | -5.5 | 37.3 | 1.1 | 0.097 | -6.2 | 37.4 | 1.2 | 0.090 |
| -4.8 | 37.1 | 1.0 | 0.170 | -5.1 | 37.2 | 1.0 | 0.121 | -5.4 | 37.3 | 1.1 | 0.099 | -6.1 | 37.4 | 1.2 | 0.088 |
| -4.7 | 37.1 | 1.0 | 0.181 | -5.0 | 37.2 | 1.0 | 0.126 | -5.3 | 37.3 | 1.1 | 0.103 | -6.0 | 37.4 | 1.1 | 0.088 |
| -4.6 | 37.1 | 1.0 | 0.189 | -4.9 | 37.2 | 1.0 | 0.135 | -5.2 | 37.3 | 1.1 | 0.106 | -5.9 | 37.4 | 1.1 | 0.089 |
| -4.5 | 37.1 | 1.0 | 0.197 | -4.8 | 37.2 | 1.0 | 0.143 | -5.1 | 37.3 | 1.0 | 0.110 | -5.8 | 37.4 | 1.1 | 0.090 |
| -4.4 | 37.1 | 1.0 | 0.210 | -4.7 | 37.2 | 1.0 | 0.152 | -5.0 | 37.3 | 1.0 | 0.116 | -5.7 | 37.4 | 1.1 | 0.091 |
| -4.3 | 37.1 | 1.0 | 0.231 | -4.6 | 37.2 | 1.0 | 0.165 | -4.9 | 37.3 | 1.0 | 0.120 | -5.6 | 37.4 | 1.1 | 0.093 |
| -4.2 | 37.1 | 1.0 | 0.242 | -4.5 | 37.2 | 1.0 | 0.181 | -4.8 | 37.3 | 1.0 | 0.124 | -5.5 | 37.4 | 1.1 | 0.095 |
| -4.1 | 37.1 | 1.0 | 0.248 | -4.4 | 37.2 | 1.0 | 0.197 | -4.7 | 37.3 | 1.0 | 0.129 | -5.4 | 37.4 | 1.1 | 0.095 |
| -4.0 | 37.1 | 1.0 | 0.251 | -4.3 | 37.2 | 1.0 | 0.219 | -4.6 | 37.3 | 1.0 | 0.136 | -5.3 | 37.4 | 1.1 | 0.097 |
| -3.9 | 37.1 | 1.0 | 0.258 | -4.2 | 37.2 | 1.0 | 0.226 | -4.5 | 37.3 | 1.0 | 0.145 | -5.2 | 37.4 | 1.1 | 0.100 |
| -3.8 | 37.1 | 1.0 | 0.260 | -4.1 | 37.2 | 1.0 | 0.237 | -4.4 | 37.3 | 1.0 | 0.157 | -5.1 | 37.4 | 1.1 | 0.105 |
| -3.7 | 37.1 | 1.0 | 0.260 | -4.0 | 37.2 | 1.0 | 0.246 | -4.3 | 37.3 | 1.0 | 0.171 | -5.0 | 37.4 | 1.0 | 0.108 |
| -3.6 | 37.1 | 1.0 | 0.260 | -3.9 | 37.2 | 1.0 | 0.255 | -4.2 | 37.3 | 1.0 | 0.190 | -4.9 | 37.4 | 1.0 | 0.112 |
| -3.5 | 37.1 | 1.0 | 0.237 | -3.8 | 37.2 | 1.0 | 0.260 | -4.1 | 37.3 | 1.0 | 0.210 | -4.8 | 37.4 | 1.0 | 0.115 |
| -3.4 | 37.1 | 1.0 | 0.186 | -3.7 | 37.2 | 1.0 | 0.260 | -4.0 | 37.3 | 1.0 | 0.228 | -4.7 | 37.4 | 1.0 | 0.119 |
| -3.3 | 37.1 | 1.0 | 0.162 | -3.6 | 37.2 | 1.0 | 0.258 | -3.9 | 37.3 | 1.0 | 0.243 | -4.6 | 37.4 | 1.0 | 0.124 |
| -3.2 | 37.1 | 1.0 | 0.152 | -3.5 | 37.2 | 1.0 | 0.235 | -3.8 | 37.3 | 1.0 | 0.250 | -4.5 | 37.4 | 1.0 | 0.129 |
| -3.1 | 37.1 | 1.0 | 0.146 | -3.4 | 37.2 | 1.0 | 0.189 | -3.7 | 37.3 | 1.0 | 0.252 | -4.4 | 37.4 | 1.0 | 0.135 |
| -3.0 | 37.1 | 1.0 | 0.143 | -3.3 | 37.2 | 1.0 | 0.161 | -3.6 | 37.3 | 1.0 | 0.246 | -4.3 | 37.4 | 1.0 | 0.140 |
| -2.9 | 37.1 | 1.0 | 0.147 | -3.2 | 37.2 | 1.0 | 0.148 | -3.5 | 37.3 | 1.0 | 0.223 | -4.2 | 37.4 | 1.0 | 0.149 |
| -2.8 | 37.1 | 1.0 | 0.149 | -3.1 | 37.2 | 1.0 | 0.143 | -3.4 | 37.3 | 1.0 | 0.179 | -4.1 | 37.4 | 1.0 | 0.160 |
| -2.7 | 37.1 | 1.0 | 0.153 | -3.0 | 37.2 | 1.0 | 0.140 | -3.3 | 37.3 | 1.0 | 0.157 | -4.0 | 37.4 | 1.0 | 0.173 |
| -2.6 | 37.1 | 1.0 | 0.158 | -2.9 | 37.2 | 1.0 | 0.139 | -3.2 | 37.3 | 1.0 | 0.145 | -3.9 | 37.4 | 1.0 | 0.192 |
| -2.5 | 37.1 | 1.0 | 0.161 | -2.8 | 37.2 | 1.0 | 0.140 | -3.1 | 37.3 | 1.0 | 0.140 | -3.8 | 37.4 | 1.0 | 0.213 |
| -2.4 | 37.1 | 1.0 | 0.163 | -2.7 | 37.2 | 1.0 | 0.142 | -3.0 | 37.3 | 1.0 | 0.136 | -3.7 | 37.4 | 1.0 | 0.227 |
| -2.3 | 37.1 | 1.0 | 0.166 | -2.6 | 37.2 | 1.0 | 0.145 | -2.9 | 37.3 | 1.0 | 0.134 | -3.6 | 37.4 | 1.0 | 0.226 |
| -2.2 | 37.1 | 1.0 | 0.170 | -2.5 | 37.2 | 1.0 | 0.147 | -2.8 | 37.3 | 1.0 | 0.136 | -3.5 | 37.4 | 1.0 | 0.200 |
| -2.1 | 37.1 | 1.0 | 0.176 | -2.4 | 37.2 | 1.0 | 0.149 | -2.7 | 37.3 | 1.0 | 0.140 | -3.4 | 37.4 | 1.0 | 0.165 |
| -2.0 | 37.1 | 1.0 | 0.179 | -2.3 | 37.2 | 1.0 | 0.151 | -2.6 | 37.3 | 1.0 | 0.148 | -3.3 | 37.4 | 1.0 | 0.149 |
| -1.9 | 37.1 | 1.0 | 0.180 | -2.2 | 37.2 | 1.0 | 0.154 | -2.5 | 37.3 | 1.0 | 0.152 | -3.2 | 37.4 | 1.0 | 0.141 |
| -1.8 | 37.1 | 1.0 | 0.166 | -2.1 | 37.2 | 1.0 | 0.159 | -2.4 | 37.3 | 1.0 | 0.156 | -3.1 | 37.4 | 1.0 | 0.136 |

| Long. | Lat. | K | $a_{gR}$ | Long. | Lat. | K | $a_{gR}$ | Long. | Lat. | K | $a_{gR}$ | Long. | Lat. | K | $a_{gR}$ |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| -3.0 | 37.4 | 1.0 | 0.133 | -3.9 | 37.5 | 1.0 | 0.150 | -5.0 | 37.6 | 1.1 | 0.099 | -6.3 | 37.7 | 1.2 | 0.076 |
| -2.9 | 37.4 | 1.0 | 0.132 | -3.8 | 37.5 | 1.0 | 0.157 | -4.9 | 37.6 | 1.1 | 0.102 | -6.2 | 37.7 | 1.2 | 0.077 |
| -2.8 | 37.4 | 1.0 | 0.132 | -3.7 | 37.5 | 1.0 | 0.167 | -4.8 | 37.6 | 1.0 | 0.105 | -6.1 | 37.7 | 1.2 | 0.077 |
| -2.7 | 37.4 | 1.0 | 0.137 | -3.6 | 37.5 | 1.0 | 0.178 | -4.7 | 37.6 | 1.0 | 0.109 | -6.0 | 37.7 | 1.1 | 0.078 |
| -2.6 | 37.4 | 1.0 | 0.146 | -3.5 | 37.5 | 1.0 | 0.169 | -4.6 | 37.6 | 1.0 | 0.112 | -5.9 | 37.7 | 1.1 | 0.078 |
| -2.5 | 37.4 | 1.0 | 0.158 | -3.4 | 37.5 | 1.0 | 0.149 | -4.5 | 37.6 | 1.0 | 0.115 | -5.8 | 37.7 | 1.1 | 0.079 |
| -2.4 | 37.4 | 1.0 | 0.163 | -3.3 | 37.5 | 1.0 | 0.140 | -4.4 | 37.6 | 1.0 | 0.116 | -5.7 | 37.7 | 1.1 | 0.079 |
| -2.3 | 37.4 | 1.0 | 0.165 | -3.2 | 37.5 | 1.0 | 0.136 | -4.3 | 37.6 | 1.0 | 0.118 | -5.6 | 37.7 | 1.1 | 0.080 |
| -2.2 | 37.4 | 1.0 | 0.166 | -3.1 | 37.5 | 1.0 | 0.132 | -4.2 | 37.6 | 1.0 | 0.121 | -5.5 | 37.7 | 1.1 | 0.080 |
| -2.1 | 37.4 | 1.0 | 0.169 | -3.0 | 37.5 | 1.0 | 0.130 | -4.1 | 37.6 | 1.0 | 0.123 | -5.4 | 37.7 | 1.1 | 0.082 |
| -2.0 | 37.4 | 1.0 | 0.178 | -2.9 | 37.5 | 1.0 | 0.128 | -4.0 | 37.6 | 1.0 | 0.126 | -5.3 | 37.7 | 1.1 | 0.086 |
| -1.9 | 37.4 | 1.0 | 0.190 | -2.8 | 37.5 | 1.0 | 0.129 | -3.9 | 37.6 | 1.0 | 0.130 | -5.2 | 37.7 | 1.1 | 0.088 |
| -1.8 | 37.4 | 1.0 | 0.188 | -2.7 | 37.5 | 1.0 | 0.133 | -3.8 | 37.6 | 1.0 | 0.131 | -5.1 | 37.7 | 1.1 | 0.092 |
| -1.7 | 37.4 | 1.0 | 0.174 | -2.6 | 37.5 | 1.0 | 0.142 | -3.7 | 37.6 | 1.0 | 0.133 | -5.0 | 37.7 | 1.1 | 0.094 |
| -1.6 | 37.4 | 1.0 | 0.145 | -2.5 | 37.5 | 1.0 | 0.151 | -3.6 | 37.6 | 1.0 | 0.137 | -4.9 | 37.7 | 1.1 | 0.097 |
| -1.5 | 37.4 | 1.0 | 0.124 | -2.4 | 37.5 | 1.0 | 0.155 | -3.5 | 37.6 | 1.0 | 0.134 | -4.8 | 37.7 | 1.1 | 0.100 |
| -1.4 | 37.4 | 1.0 | 0.110 | -2.3 | 37.5 | 1.0 | 0.157 | -3.4 | 37.6 | 1.0 | 0.129 | -4.7 | 37.7 | 1.0 | 0.104 |
| -1.3 | 37.4 | 1.0 | 0.102 | -2.2 | 37.5 | 1.0 | 0.159 | -3.3 | 37.6 | 1.0 | 0.127 | -4.6 | 37.7 | 1.0 | 0.107 |
| -1.2 | 37.4 | 1.0 | 0.096 | -2.1 | 37.5 | 1.0 | 0.162 | -3.2 | 37.6 | 1.0 | 0.126 | -4.5 | 37.7 | 1.0 | 0.109 |
| -1.1 | 37.4 | 1.0 | 0.093 | -2.0 | 37.5 | 1.0 | 0.173 | -3.1 | 37.6 | 1.0 | 0.124 | -4.4 | 37.7 | 1.0 | 0.110 |
| -1.0 | 37.4 | 1.0 | 0.089 | -1.9 | 37.5 | 1.0 | 0.189 | -3.0 | 37.6 | 1.0 | 0.123 | -4.3 | 37.7 | 1.0 | 0.111 |
| -0.9 | 37.4 | 1.0 | 0.087 | -1.8 | 37.5 | 1.0 | 0.191 | -2.9 | 37.6 | 1.0 | 0.123 | -4.2 | 37.7 | 1.0 | 0.113 |
| -0.8 | 37.4 | 1.0 | 0.085 | -1.7 | 37.5 | 1.0 | 0.181 | -2.8 | 37.6 | 1.0 | 0.125 | -4.1 | 37.7 | 1.0 | 0.114 |
| -0.7 | 37.4 | 1.0 | 0.082 | -1.6 | 37.5 | 1.0 | 0.159 | -2.7 | 37.6 | 1.0 | 0.133 | -4.0 | 37.7 | 1.0 | 0.117 |
| -0.6 | 37.4 | 1.0 | 0.080 | -1.5 | 37.5 | 1.0 | 0.136 | -2.6 | 37.6 | 1.0 | 0.141 | -3.9 | 37.7 | 1.0 | 0.117 |
| -0.4 | 37.4 | 1.0 | 0.074 | -1.4 | 37.5 | 1.0 | 0.122 | -2.5 | 37.6 | 1.0 | 0.145 | -3.8 | 37.7 | 1.0 | 0.119 |
| -7.5 | 37.5 | 1.3 | 0.121 | -1.3 | 37.5 | 1.0 | 0.113 | -2.4 | 37.6 | 1.0 | 0.150 | -3.7 | 37.7 | 1.0 | 0.120 |
| -7.4 | 37.5 | 1.3 | 0.117 | -1.2 | 37.5 | 1.0 | 0.107 | -2.3 | 37.6 | 1.0 | 0.152 | -3.6 | 37.7 | 1.0 | 0.120 |
| -7.3 | 37.5 | 1.3 | 0.113 | -1.1 | 37.5 | 1.0 | 0.103 | -2.2 | 37.6 | 1.0 | 0.154 | -3.5 | 37.7 | 1.0 | 0.119 |
| -7.2 | 37.5 | 1.3 | 0.109 | -1.0 | 37.5 | 1.0 | 0.100 | -2.1 | 37.6 | 1.0 | 0.157 | -3.4 | 37.7 | 1.0 | 0.118 |
| -7.1 | 37.5 | 1.3 | 0.105 | -0.9 | 37.5 | 1.0 | 0.097 | -2.0 | 37.6 | 1.0 | 0.163 | -3.3 | 37.7 | 1.0 | 0.115 |
| -7.0 | 37.5 | 1.3 | 0.100 | -0.8 | 37.5 | 1.0 | 0.095 | -1.9 | 37.6 | 1.0 | 0.175 | -3.2 | 37.7 | 1.0 | 0.113 |
| -6.9 | 37.5 | 1.3 | 0.092 | -0.7 | 37.5 | 1.0 | 0.093 | -1.8 | 37.6 | 1.0 | 0.189 | -3.1 | 37.7 | 1.0 | 0.110 |

| | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| -6.8 | 37.5 | 1.3 | 0.091 | -0.6 | 37.5 | 1.0 | 0.090 | -1.7 | 37.6 | 1.0 | 0.193 | -3.0 | 37.7 | 1.0 | 0.106 |
| -6.7 | 37.5 | 1.3 | 0.090 | -0.5 | 37.5 | 1.0 | 0.086 | -1.6 | 37.6 | 1.0 | 0.183 | -2.9 | 37.7 | 1.0 | 0.108 |
| -6.6 | 37.5 | 1.3 | 0.089 | -0.4 | 37.5 | 1.0 | 0.082 | -1.5 | 37.6 | 1.0 | 0.165 | -2.8 | 37.7 | 1.0 | 0.118 |
| -6.5 | 37.5 | 1.2 | 0.089 | -0.3 | 37.5 | 1.0 | 0.078 | -1.4 | 37.6 | 1.0 | 0.143 | -2.7 | 37.7 | 1.0 | 0.132 |
| -6.4 | 37.5 | 1.2 | 0.087 | -7.5 | 37.6 | 1.3 | 0.103 | -1.3 | 37.6 | 1.0 | 0.129 | -2.6 | 37.7 | 1.0 | 0.138 |
| -6.3 | 37.5 | 1.2 | 0.086 | -7.4 | 37.6 | 1.3 | 0.100 | -1.2 | 37.6 | 1.0 | 0.121 | -2.5 | 37.7 | 1.0 | 0.142 |
| -6.2 | 37.5 | 1.2 | 0.084 | -7.3 | 37.6 | 1.3 | 0.098 | -1.1 | 37.6 | 1.0 | 0.115 | -2.4 | 37.7 | 1.0 | 0.144 |
| -6.1 | 37.5 | 1.2 | 0.084 | -7.2 | 37.6 | 1.3 | 0.097 | -1.0 | 37.6 | 1.0 | 0.112 | -2.3 | 37.7 | 1.0 | 0.149 |
| -6.0 | 37.5 | 1.1 | 0.085 | -7.1 | 37.6 | 1.3 | 0.093 | -0.9 | 37.6 | 1.0 | 0.110 | -2.2 | 37.7 | 1.0 | 0.151 |
| -5.9 | 37.5 | 1.1 | 0.085 | -7.0 | 37.6 | 1.3 | 0.082 | -0.8 | 37.6 | 1.0 | 0.108 | -2.1 | 37.7 | 1.0 | 0.153 |
| -5.8 | 37.5 | 1.1 | 0.086 | -6.9 | 37.6 | 1.3 | 0.082 | -0.7 | 37.6 | 1.0 | 0.105 | -2.0 | 37.7 | 1.0 | 0.156 |
| -5.7 | 37.5 | 1.1 | 0.088 | -6.8 | 37.6 | 1.3 | 0.081 | -0.6 | 37.6 | 1.0 | 0.102 | -1.9 | 37.7 | 1.0 | 0.161 |
| -5.6 | 37.5 | 1.1 | 0.089 | -6.7 | 37.6 | 1.3 | 0.081 | -0.5 | 37.6 | 1.0 | 0.097 | -1.8 | 37.7 | 1.0 | 0.177 |
| -5.5 | 37.5 | 1.1 | 0.090 | -6.6 | 37.6 | 1.3 | 0.081 | -0.4 | 37.6 | 1.0 | 0.092 | -1.7 | 37.7 | 1.0 | 0.192 |
| -5.4 | 37.5 | 1.1 | 0.090 | -6.5 | 37.6 | 1.2 | 0.081 | -0.3 | 37.6 | 1.0 | 0.087 | -1.6 | 37.7 | 1.0 | 0.195 |
| -5.3 | 37.5 | 1.1 | 0.093 | -6.4 | 37.6 | 1.2 | 0.081 | -0.2 | 37.6 | 1.0 | 0.082 | -1.5 | 37.7 | 1.0 | 0.188 |
| -5.2 | 37.5 | 1.1 | 0.097 | -6.3 | 37.6 | 1.2 | 0.082 | -7.6 | 37.7 | 1.3 | 0.088 | -1.4 | 37.7 | 1.0 | 0.173 |
| -5.1 | 37.5 | 1.1 | 0.100 | -6.2 | 37.6 | 1.2 | 0.082 | -7.5 | 37.7 | 1.3 | 0.087 | -1.3 | 37.7 | 1.0 | 0.152 |
| -5.0 | 37.5 | 1.1 | 0.104 | -6.1 | 37.6 | 1.2 | 0.084 | -7.4 | 37.7 | 1.3 | 0.085 | -1.2 | 37.7 | 1.0 | 0.139 |
| -4.9 | 37.5 | 1.0 | 0.107 | -6.0 | 37.6 | 1.1 | 0.083 | -7.3 | 37.7 | 1.3 | 0.085 | -1.1 | 37.7 | 1.0 | 0.132 |
| -4.8 | 37.5 | 1.0 | 0.110 | -5.9 | 37.6 | 1.1 | 0.084 | -7.2 | 37.7 | 1.3 | 0.083 | -1.0 | 37.7 | 1.0 | 0.127 |
| -4.7 | 37.5 | 1.0 | 0.114 | -5.8 | 37.6 | 1.1 | 0.084 | -7.1 | 37.7 | 1.3 | 0.080 | -0.9 | 37.7 | 1.0 | 0.126 |
| -4.6 | 37.5 | 1.0 | 0.117 | -5.7 | 37.6 | 1.1 | 0.085 | -7.0 | 37.7 | 1.3 | 0.078 | -0.8 | 37.7 | 1.0 | 0.122 |
| -4.5 | 37.5 | 1.0 | 0.121 | -5.6 | 37.6 | 1.1 | 0.085 | -6.9 | 37.7 | 1.3 | 0.076 | -0.7 | 37.7 | 1.0 | 0.119 |
| -4.4 | 37.5 | 1.0 | 0.125 | -5.5 | 37.6 | 1.1 | 0.086 | -6.8 | 37.7 | 1.3 | 0.076 | -0.6 | 37.7 | 1.0 | 0.114 |
| -4.3 | 37.5 | 1.0 | 0.127 | -5.4 | 37.6 | 1.1 | 0.088 | -6.7 | 37.7 | 1.3 | 0.076 | -0.5 | 37.7 | 1.0 | 0.110 |
| -4.2 | 37.5 | 1.0 | 0.131 | -5.3 | 37.6 | 1.1 | 0.090 | -6.6 | 37.7 | 1.3 | 0.075 | -0.4 | 37.7 | 1.0 | 0.104 |
| -4.1 | 37.5 | 1.0 | 0.136 | -5.2 | 37.6 | 1.1 | 0.093 | -6.5 | 37.7 | 1.2 | 0.075 | -0.3 | 37.7 | 1.0 | 0.097 |
| -4.0 | 37.5 | 1.0 | 0.143 | -5.1 | 37.6 | 1.1 | 0.096 | -6.4 | 37.7 | 1.2 | 0.075 | -0.2 | 37.7 | 1.0 | 0.090 |

| Long. | Lat. | K | $a_{gR}$ | Long. | Lat. | K | $a_{gR}$ | Long. | Lat. | K | $a_{gR}$ | Long. | Lat. | K | $a_{gR}$ |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| -0.1 | 37.7 | 1.0 | 0.082 | -1.4 | 37.8 | 1.0 | 0.194 | -2.7 | 37.9 | 1.0 | 0.091 | -4.0 | 38.0 | 1.0 | 0.092 |
| -7.5 | 37.8 | 1.3 | 0.082 | -1.3 | 37.8 | 1.0 | 0.185 | -2.6 | 37.9 | 1.0 | 0.102 | -3.9 | 38.0 | 1.0 | 0.094 |
| -7.4 | 37.8 | 1.3 | 0.081 | -1.2 | 37.8 | 1.0 | 0.174 | -2.5 | 37.9 | 1.0 | 0.116 | -3.8 | 38.0 | 1.0 | 0.095 |
| -7.3 | 37.8 | 1.3 | 0.081 | -1.1 | 37.8 | 1.0 | 0.164 | -2.4 | 37.9 | 1.0 | 0.128 | -3.7 | 38.0 | 1.0 | 0.094 |
| -7.2 | 37.8 | 1.3 | 0.080 | -1.0 | 37.8 | 1.0 | 0.159 | -2.3 | 37.9 | 1.0 | 0.135 | -3.6 | 38.0 | 1.0 | 0.094 |
| -7.1 | 37.8 | 1.3 | 0.077 | -0.9 | 37.8 | 1.0 | 0.150 | -2.2 | 37.9 | 1.0 | 0.139 | -3.5 | 38.0 | 1.0 | 0.092 |
| -7.0 | 37.8 | 1.3 | 0.074 | -0.8 | 37.8 | 1.0 | 0.146 | -2.1 | 37.9 | 1.0 | 0.143 | -3.4 | 38.0 | 1.0 | 0.089 |
| -6.9 | 37.8 | 1.3 | 0.072 | -0.7 | 37.8 | 1.0 | 0.141 | -2.0 | 37.9 | 1.0 | 0.147 | -3.3 | 38.0 | 1.0 | 0.084 |
| -6.8 | 37.8 | 1.3 | 0.071 | -0.6 | 37.8 | 1.0 | 0.135 | -1.9 | 37.9 | 1.0 | 0.152 | -3.2 | 38.0 | 1.0 | 0.079 |
| -6.7 | 37.8 | 1.3 | 0.071 | -0.5 | 37.8 | 1.0 | 0.130 | -1.8 | 37.9 | 1.0 | 0.157 | -3.1 | 38.0 | 1.0 | 0.075 |
| -6.6 | 37.8 | 1.3 | 0.070 | -0.4 | 37.8 | 1.0 | 0.122 | -1.7 | 37.9 | 1.0 | 0.164 | -3.0 | 38.0 | 1.0 | 0.074 |
| -6.5 | 37.8 | 1.3 | 0.070 | -0.3 | 37.8 | 1.0 | 0.112 | -1.6 | 37.9 | 1.0 | 0.175 | -2.9 | 38.0 | 1.0 | 0.073 |
| -6.4 | 37.8 | 1.2 | 0.068 | -0.2 | 37.8 | 1.0 | 0.102 | -1.5 | 37.9 | 1.0 | 0.189 | -2.8 | 38.0 | 1.0 | 0.075 |
| -6.3 | 37.8 | 1.2 | 0.068 | -0.1 | 37.8 | 1.0 | 0.092 | -1.4 | 37.9 | 1.0 | 0.202 | -2.7 | 38.0 | 1.0 | 0.078 |
| -6.2 | 37.8 | 1.2 | 0.067 | 0.0 | 37.8 | 1.0 | 0.081 | -1.3 | 37.9 | 1.0 | 0.210 | -2.6 | 38.0 | 1.0 | 0.084 |
| -6.1 | 37.8 | 1.2 | 0.067 | -7.4 | 37.9 | 1.3 | 0.080 | -1.2 | 37.9 | 1.0 | 0.216 | -2.5 | 38.0 | 1.0 | 0.093 |
| -6.0 | 37.8 | 1.2 | 0.068 | -7.3 | 37.9 | 1.3 | 0.080 | -1.1 | 37.9 | 1.0 | 0.212 | -2.4 | 38.0 | 1.0 | 0.104 |
| -5.9 | 37.8 | 1.1 | 0.068 | -7.2 | 37.9 | 1.3 | 0.076 | -1.0 | 37.9 | 1.0 | 0.205 | -2.3 | 38.0 | 1.0 | 0.115 |
| -5.8 | 37.8 | 1.1 | 0.069 | -7.1 | 37.9 | 1.3 | 0.074 | -0.9 | 37.9 | 1.0 | 0.198 | -2.2 | 38.0 | 1.0 | 0.126 |
| -5.7 | 37.8 | 1.1 | 0.069 | -7.0 | 37.9 | 1.3 | 0.071 | -0.8 | 37.9 | 1.0 | 0.192 | -2.1 | 38.0 | 1.0 | 0.134 |
| -5.6 | 37.8 | 1.1 | 0.070 | -6.9 | 37.9 | 1.3 | 0.070 | -0.7 | 37.9 | 1.0 | 0.185 | -2.0 | 38.0 | 1.0 | 0.141 |
| -5.5 | 37.8 | 1.1 | 0.070 | -6.8 | 37.9 | 1.3 | 0.069 | -0.6 | 37.9 | 1.0 | 0.177 | -1.9 | 38.0 | 1.0 | 0.147 |
| -5.4 | 37.8 | 1.1 | 0.073 | -6.7 | 37.9 | 1.3 | 0.067 | -0.5 | 37.9 | 1.0 | 0.167 | -1.8 | 38.0 | 1.0 | 0.151 |
| -5.3 | 37.8 | 1.1 | 0.077 | -6.6 | 37.9 | 1.3 | 0.066 | -0.4 | 37.9 | 1.0 | 0.158 | -1.7 | 38.0 | 1.0 | 0.158 |
| -5.2 | 37.8 | 1.1 | 0.081 | -6.5 | 37.9 | 1.3 | 0.064 | -0.3 | 37.9 | 1.0 | 0.139 | -1.6 | 38.0 | 1.0 | 0.166 |
| -5.1 | 37.8 | 1.1 | 0.085 | -6.4 | 37.9 | 1.2 | 0.065 | -0.2 | 37.9 | 1.0 | 0.121 | -1.5 | 38.0 | 1.0 | 0.179 |
| -5.0 | 37.8 | 1.1 | 0.088 | -6.3 | 37.9 | 1.2 | 0.063 | -0.1 | 37.9 | 1.0 | 0.106 | -1.4 | 38.0 | 1.0 | 0.197 |
| -4.9 | 37.8 | 1.1 | 0.091 | -6.2 | 37.9 | 1.2 | 0.062 | 0.0 | 37.9 | 1.0 | 0.092 | -1.3 | 38.0 | 1.0 | 0.218 |
| -4.8 | 37.8 | 1.1 | 0.095 | -6.1 | 37.9 | 1.2 | 0.062 | 0.1 | 37.9 | 1.0 | 0.081 | -1.2 | 38.0 | 1.0 | 0.227 |
| -4.7 | 37.8 | 1.1 | 0.098 | -6.0 | 37.9 | 1.2 | 0.062 | -7.3 | 38.0 | 1.3 | 0.077 | -1.1 | 38.0 | 1.0 | 0.233 |
| -4.6 | 37.8 | 1.0 | 0.101 | -5.9 | 37.9 | 1.2 | 0.061 | -7.2 | 38.0 | 1.3 | 0.074 | -1.0 | 38.0 | 1.0 | 0.227 |
| -4.5 | 37.8 | 1.0 | 0.102 | -5.8 | 37.9 | 1.1 | 0.061 | -7.1 | 38.0 | 1.3 | 0.071 | -0.9 | 38.0 | 1.0 | 0.225 |
| -4.4 | 37.8 | 1.0 | 0.103 | -5.7 | 37.9 | 1.1 | 0.061 | -7.0 | 38.0 | 1.3 | 0.069 | -0.8 | 38.0 | 1.0 | 0.225 |
| -4.3 | 37.8 | 1.0 | 0.105 | -5.6 | 37.9 | 1.1 | 0.061 | -6.9 | 38.0 | 1.3 | 0.067 | -0.7 | 38.0 | 1.0 | 0.221 |
| -4.2 | 37.8 | 1.0 | 0.106 | -5.5 | 37.9 | 1.1 | 0.062 | -6.8 | 38.0 | 1.3 | 0.065 | -0.6 | 38.0 | 1.0 | 0.216 |
| -4.1 | 37.8 | 1.0 | 0.107 | -5.4 | 37.9 | 1.1 | 0.064 | -6.7 | 38.0 | 1.3 | 0.063 | -0.5 | 38.0 | 1.0 | 0.208 |
| -4.0 | 37.8 | 1.0 | 0.108 | -5.3 | 37.9 | 1.1 | 0.066 | -6.6 | 38.0 | 1.3 | 0.061 | -0.4 | 38.0 | 1.0 | 0.198 |
| -3.9 | 37.8 | 1.0 | 0.109 | -5.2 | 37.9 | 1.1 | 0.070 | -6.5 | 38.0 | 1.3 | 0.061 | -0.3 | 38.0 | 1.0 | 0.183 |
| -3.8 | 37.8 | 1.0 | 0.110 | -5.1 | 37.9 | 1.1 | 0.073 | -6.4 | 38.0 | 1.3 | 0.058 | -0.2 | 38.0 | 1.0 | 0.158 |
| -3.7 | 37.8 | 1.0 | 0.110 | -5.0 | 37.9 | 1.1 | 0.077 | -6.3 | 38.0 | 1.3 | 0.057 | -0.1 | 38.0 | 1.0 | 0.133 |
| -3.6 | 37.8 | 1.0 | 0.110 | -4.9 | 37.9 | 1.1 | 0.082 | -6.2 | 38.0 | 1.2 | 0.055 | 0.0 | 38.0 | 1.0 | 0.107 |
| -3.5 | 37.8 | 1.0 | 0.109 | -4.8 | 37.9 | 1.1 | 0.086 | -6.1 | 38.0 | 1.2 | 0.055 | 0.1 | 38.0 | 1.0 | 0.089 |
| -3.4 | 37.8 | 1.0 | 0.108 | -4.7 | 37.9 | 1.1 | 0.090 | -6.0 | 38.0 | 1.2 | 0.055 | -7.1 | 38.1 | 1.3 | 0.070 |
| -3.3 | 37.8 | 1.0 | 0.106 | -4.6 | 37.9 | 1.1 | 0.093 | -5.9 | 38.0 | 1.2 | 0.055 | -7.0 | 38.1 | 1.3 | 0.068 |
| -3.2 | 37.8 | 1.0 | 0.102 | -4.5 | 37.9 | 1.0 | 0.095 | -5.8 | 38.0 | 1.2 | 0.055 | -6.9 | 38.1 | 1.3 | 0.066 |
| -3.1 | 37.8 | 1.0 | 0.097 | -4.4 | 37.9 | 1.0 | 0.096 | -5.7 | 38.0 | 1.2 | 0.056 | -6.8 | 38.1 | 1.3 | 0.064 |
| -3.0 | 37.8 | 1.0 | 0.092 | -4.3 | 37.9 | 1.0 | 0.097 | -5.6 | 38.0 | 1.2 | 0.056 | -6.7 | 38.1 | 1.3 | 0.062 |
| -2.9 | 37.8 | 1.0 | 0.092 | -4.2 | 37.9 | 1.0 | 0.098 | -5.5 | 38.0 | 1.1 | 0.058 | -6.6 | 38.1 | 1.3 | 0.060 |
| -2.8 | 37.8 | 1.0 | 0.099 | -4.1 | 37.9 | 1.0 | 0.099 | -5.4 | 38.0 | 1.1 | 0.059 | -6.5 | 38.1 | 1.3 | 0.058 |
| -2.7 | 37.8 | 1.0 | 0.114 | -4.0 | 37.9 | 1.0 | 0.100 | -5.3 | 38.0 | 1.1 | 0.060 | -6.4 | 38.1 | 1.3 | 0.054 |
| -2.6 | 37.8 | 1.0 | 0.128 | -3.9 | 37.9 | 1.0 | 0.101 | -5.2 | 38.0 | 1.1 | 0.061 | -6.3 | 38.1 | 1.3 | 0.053 |
| -2.5 | 37.8 | 1.0 | 0.136 | -3.8 | 37.9 | 1.0 | 0.101 | -5.1 | 38.0 | 1.1 | 0.064 | -6.2 | 38.1 | 1.3 | 0.052 |
| -2.4 | 37.8 | 1.0 | 0.140 | -3.7 | 37.9 | 1.0 | 0.102 | -5.0 | 38.0 | 1.1 | 0.066 | -6.1 | 38.1 | 1.2 | 0.052 |
| -2.3 | 37.8 | 1.0 | 0.143 | -3.6 | 37.9 | 1.0 | 0.102 | -4.9 | 38.0 | 1.1 | 0.069 | -6.0 | 38.1 | 1.2 | 0.052 |
| -2.2 | 37.8 | 1.0 | 0.146 | -3.5 | 37.9 | 1.0 | 0.102 | -4.8 | 38.0 | 1.1 | 0.072 | -5.9 | 38.1 | 1.2 | 0.052 |
| -2.1 | 37.8 | 1.0 | 0.149 | -3.4 | 37.9 | 1.0 | 0.100 | -4.7 | 38.0 | 1.1 | 0.076 | -5.8 | 38.1 | 1.2 | 0.052 |
| -2.0 | 37.8 | 1.0 | 0.151 | -3.3 | 37.9 | 1.0 | 0.096 | -4.6 | 38.0 | 1.1 | 0.079 | -5.7 | 38.1 | 1.2 | 0.053 |
| -1.9 | 37.8 | 1.0 | 0.156 | -3.2 | 37.9 | 1.0 | 0.090 | -4.5 | 38.0 | 1.1 | 0.083 | -5.6 | 38.1 | 1.2 | 0.053 |
| -1.8 | 37.8 | 1.0 | 0.165 | -3.1 | 37.9 | 1.0 | 0.086 | -4.4 | 38.0 | 1.0 | 0.086 | -5.5 | 38.1 | 1.2 | 0.053 |
| -1.7 | 37.8 | 1.0 | 0.177 | -3.0 | 37.9 | 1.0 | 0.082 | -4.3 | 38.0 | 1.0 | 0.088 | -5.4 | 38.1 | 1.2 | 0.055 |
| -1.6 | 37.8 | 1.0 | 0.189 | -2.9 | 37.9 | 1.0 | 0.081 | -4.2 | 38.0 | 1.0 | 0.090 | -5.3 | 38.1 | 1.2 | 0.055 |
| -1.5 | 37.8 | 1.0 | 0.197 | -2.8 | 37.9 | 1.0 | 0.084 | -4.1 | 38.0 | 1.0 | 0.091 | -5.2 | 38.1 | 1.1 | 0.056 |

| Long. | Lat. | K | $a_{gR}$ | Long. | Lat. | K | $a_{gR}$ | Long. | Lat. | K | $a_{gR}$ | Long. | Lat. | K | $a_{gR}$ |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| -5.1 | 38.1 | 1.1 | 0.057 | -6.4 | 38.2 | 1.3 | 0.053 | -0.2 | 38.2 | 1.0 | 0.200 | -1.7 | 38.3 | 1.0 | 0.121 |
| -5.0 | 38.1 | 1.1 | 0.059 | -6.3 | 38.2 | 1.3 | 0.051 | -0.1 | 38.2 | 1.0 | 0.178 | -1.6 | 38.3 | 1.0 | 0.140 |
| -4.9 | 38.1 | 1.1 | 0.061 | -6.2 | 38.2 | 1.3 | 0.050 | 0.0 | 38.2 | 1.0 | 0.143 | -1.5 | 38.3 | 1.0 | 0.159 |
| -4.8 | 38.1 | 1.1 | 0.062 | -6.1 | 38.2 | 1.3 | 0.050 | 0.1 | 38.2 | 1.0 | 0.108 | -1.4 | 38.3 | 1.0 | 0.174 |
| -4.7 | 38.1 | 1.1 | 0.064 | -6.0 | 38.2 | 1.3 | 0.050 | 0.2 | 38.2 | 1.0 | 0.090 | -1.3 | 38.3 | 1.0 | 0.179 |
| -4.6 | 38.1 | 1.1 | 0.066 | -5.9 | 38.2 | 1.3 | 0.050 | 0.3 | 38.2 | 1.0 | 0.078 | -1.2 | 38.3 | 1.0 | 0.183 |
| -4.5 | 38.1 | 1.1 | 0.069 | -5.8 | 38.2 | 1.2 | 0.050 | -7.3 | 38.3 | 1.2 | 0.072 | -1.1 | 38.3 | 1.0 | 0.180 |
| -4.4 | 38.1 | 1.1 | 0.070 | -5.7 | 38.2 | 1.2 | 0.050 | -7.2 | 38.3 | 1.2 | 0.069 | -1.0 | 38.3 | 1.0 | 0.181 |
| -4.3 | 38.1 | 1.0 | 0.073 | -5.6 | 38.2 | 1.2 | 0.050 | -7.1 | 38.3 | 1.2 | 0.066 | -0.9 | 38.3 | 1.0 | 0.184 |
| -4.2 | 38.1 | 1.0 | 0.076 | -5.5 | 38.2 | 1.2 | 0.050 | -7.0 | 38.3 | 1.3 | 0.063 | -0.8 | 38.3 | 1.0 | 0.188 |
| -4.1 | 38.1 | 1.0 | 0.077 | -5.4 | 38.2 | 1.2 | 0.050 | -6.9 | 38.3 | 1.3 | 0.061 | -0.7 | 38.3 | 1.0 | 0.190 |
| -4.0 | 38.1 | 1.0 | 0.080 | -5.3 | 38.2 | 1.2 | 0.051 | -6.8 | 38.3 | 1.3 | 0.058 | -0.6 | 38.3 | 1.0 | 0.186 |
| -3.9 | 38.1 | 1.0 | 0.084 | -5.2 | 38.2 | 1.2 | 0.051 | -6.7 | 38.3 | 1.3 | 0.057 | -0.5 | 38.3 | 1.0 | 0.189 |
| -3.8 | 38.1 | 1.0 | 0.086 | -5.1 | 38.2 | 1.2 | 0.052 | -6.6 | 38.3 | 1.3 | 0.056 | -0.4 | 38.3 | 1.0 | 0.194 |
| -3.7 | 38.1 | 1.0 | 0.086 | -5.0 | 38.2 | 1.2 | 0.053 | -6.5 | 38.3 | 1.3 | 0.054 | -0.3 | 38.3 | 1.0 | 0.197 |
| -3.6 | 38.1 | 1.0 | 0.086 | -4.9 | 38.2 | 1.1 | 0.054 | -6.4 | 38.3 | 1.3 | 0.050 | -0.2 | 38.3 | 1.0 | 0.195 |
| -3.5 | 38.1 | 1.0 | 0.084 | -4.8 | 38.2 | 1.1 | 0.055 | -6.3 | 38.3 | 1.3 | 0.049 | -0.1 | 38.3 | 1.0 | 0.186 |
| -3.4 | 38.1 | 1.0 | 0.079 | -4.7 | 38.2 | 1.1 | 0.056 | -6.2 | 38.3 | 1.3 | 0.047 | 0.0 | 38.3 | 1.0 | 0.156 |
| -3.3 | 38.1 | 1.0 | 0.074 | -4.6 | 38.2 | 1.1 | 0.058 | -6.1 | 38.3 | 1.3 | 0.048 | 0.1 | 38.3 | 1.0 | 0.118 |
| -3.2 | 38.1 | 1.0 | 0.071 | -4.5 | 38.2 | 1.1 | 0.058 | -6.0 | 38.3 | 1.3 | 0.048 | 0.2 | 38.3 | 1.0 | 0.097 |
| -3.1 | 38.1 | 1.0 | 0.069 | -4.4 | 38.2 | 1.1 | 0.059 | -5.9 | 38.3 | 1.3 | 0.048 | -7.4 | 38.4 | 1.2 | 0.074 |
| -3.0 | 38.1 | 1.0 | 0.068 | -4.3 | 38.2 | 1.1 | 0.061 | -5.8 | 38.3 | 1.3 | 0.048 | -7.3 | 38.4 | 1.2 | 0.071 |
| -2.9 | 38.1 | 1.0 | 0.067 | -4.2 | 38.2 | 1.0 | 0.063 | -5.7 | 38.3 | 1.3 | 0.047 | -7.2 | 38.4 | 1.2 | 0.068 |
| -2.8 | 38.1 | 1.0 | 0.068 | -4.1 | 38.2 | 1.0 | 0.065 | -5.6 | 38.3 | 1.3 | 0.047 | -7.1 | 38.4 | 1.2 | 0.065 |
| -2.7 | 38.1 | 1.0 | 0.070 | -4.0 | 38.2 | 1.0 | 0.067 | -5.5 | 38.3 | 1.3 | 0.046 | -7.0 | 38.4 | 1.3 | 0.061 |
| -2.6 | 38.1 | 1.0 | 0.074 | -3.9 | 38.2 | 1.0 | 0.069 | -5.4 | 38.3 | 1.2 | 0.047 | -6.9 | 38.4 | 1.3 | 0.059 |
| -2.5 | 38.1 | 1.0 | 0.079 | -3.8 | 38.2 | 1.0 | 0.072 | -5.3 | 38.3 | 1.2 | 0.047 | -6.8 | 38.4 | 1.3 | 0.058 |
| -2.4 | 38.1 | 1.0 | 0.086 | -3.7 | 38.2 | 1.0 | 0.075 | -5.2 | 38.3 | 1.2 | 0.047 | -6.7 | 38.4 | 1.3 | 0.055 |
| -2.3 | 38.1 | 1.0 | 0.094 | -3.6 | 38.2 | 1.0 | 0.075 | -5.1 | 38.3 | 1.2 | 0.047 | -6.6 | 38.4 | 1.3 | 0.054 |
| -2.2 | 38.1 | 1.0 | 0.103 | -3.5 | 38.2 | 1.0 | 0.072 | -5.0 | 38.3 | 1.2 | 0.047 | -6.5 | 38.4 | 1.3 | 0.053 |
| -2.1 | 38.1 | 1.0 | 0.114 | -3.4 | 38.2 | 1.0 | 0.069 | -4.9 | 38.3 | 1.2 | 0.047 | -6.4 | 38.4 | 1.3 | 0.051 |
| -2.0 | 38.1 | 1.0 | 0.127 | -3.3 | 38.2 | 1.0 | 0.066 | -4.8 | 38.3 | 1.2 | 0.049 | -6.3 | 38.4 | 1.3 | 0.050 |
| -1.9 | 38.1 | 1.0 | 0.137 | -3.2 | 38.2 | 1.0 | 0.065 | -4.7 | 38.3 | 1.2 | 0.050 | -6.2 | 38.4 | 1.3 | 0.047 |

| | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| -1.8 | 38.1 | 1.0 | 0.143 | -3.1 | 38.2 | 1.0 | 0.064 | -4.6 | 38.3 | 1.1 | 0.049 | -6.1 | 38.4 | 1.3 | 0.045 |
| -1.7 | 38.1 | 1.0 | 0.152 | -3.0 | 38.2 | 1.0 | 0.063 | -4.5 | 38.3 | 1.1 | 0.052 | -6.0 | 38.4 | 1.3 | 0.046 |
| -1.6 | 38.1 | 1.0 | 0.160 | -2.9 | 38.2 | 1.0 | 0.063 | -4.4 | 38.3 | 1.1 | 0.051 | -5.9 | 38.4 | 1.3 | 0.045 |
| -1.5 | 38.1 | 1.0 | 0.175 | -2.8 | 38.2 | 1.0 | 0.064 | -4.3 | 38.3 | 1.1 | 0.054 | -5.8 | 38.4 | 1.3 | 0.045 |
| -1.4 | 38.1 | 1.0 | 0.197 | -2.7 | 38.2 | 1.0 | 0.065 | -4.2 | 38.3 | 1.1 | 0.055 | -5.7 | 38.4 | 1.3 | 0.044 |
| -1.3 | 38.1 | 1.0 | 0.208 | -2.6 | 38.2 | 1.0 | 0.067 | -4.1 | 38.3 | 1.0 | 0.055 | -5.6 | 38.4 | 1.3 | 0.044 |
| -1.2 | 38.1 | 1.0 | 0.216 | -2.5 | 38.2 | 1.0 | 0.070 | -4.0 | 38.3 | 1.0 | 0.057 | -5.5 | 38.4 | 1.3 | 0.043 |
| -1.1 | 38.1 | 1.0 | 0.219 | -2.4 | 38.2 | 1.0 | 0.074 | -3.9 | 38.3 | 1.0 | 0.058 | -5.4 | 38.4 | 1.3 | 0.042 |
| -1.0 | 38.1 | 1.0 | 0.222 | -2.3 | 38.2 | 1.0 | 0.079 | -3.8 | 38.3 | 1.0 | 0.058 | -5.3 | 38.4 | 1.3 | 0.042 |
| -0.9 | 38.1 | 1.0 | 0.225 | -2.2 | 38.2 | 1.0 | 0.085 | -3.7 | 38.3 | 1.0 | 0.060 | -5.2 | 38.4 | 1.2 | 0.042 |
| -0.8 | 38.1 | 1.0 | 0.227 | -2.1 | 38.2 | 1.0 | 0.092 | -3.6 | 38.3 | 1.0 | 0.060 | -5.1 | 38.4 | 1.2 | 0.042 |
| -0.7 | 38.1 | 1.0 | 0.225 | -2.0 | 38.2 | 1.0 | 0.101 | -3.5 | 38.3 | 1.0 | 0.058 | -5.0 | 38.4 | 1.2 | 0.043 |
| -0.6 | 38.1 | 1.0 | 0.223 | -1.9 | 38.2 | 1.0 | 0.112 | -3.4 | 38.3 | 1.0 | 0.058 | -4.9 | 38.4 | 1.2 | 0.044 |
| -0.5 | 38.1 | 1.0 | 0.218 | -1.8 | 38.2 | 1.0 | 0.128 | -3.3 | 38.3 | 1.0 | 0.057 | -4.8 | 38.4 | 1.2 | 0.044 |
| -0.4 | 38.1 | 1.0 | 0.212 | -1.7 | 38.2 | 1.0 | 0.142 | -3.2 | 38.3 | 1.0 | 0.057 | -4.7 | 38.4 | 1.2 | 0.043 |
| -0.3 | 38.1 | 1.0 | 0.205 | -1.6 | 38.2 | 1.0 | 0.154 | -3.1 | 38.3 | 1.0 | 0.058 | -4.6 | 38.4 | 1.2 | 0.045 |
| -0.2 | 38.1 | 1.0 | 0.189 | -1.5 | 38.2 | 1.0 | 0.173 | -3.0 | 38.3 | 1.0 | 0.058 | -4.5 | 38.4 | 1.2 | 0.046 |
| -0.1 | 38.1 | 1.0 | 0.163 | -1.4 | 38.2 | 1.0 | 0.191 | -2.9 | 38.3 | 1.0 | 0.060 | -4.4 | 38.4 | 1.1 | 0.046 |
| 0.0 | 38.1 | 1.0 | 0.127 | -1.3 | 38.2 | 1.0 | 0.200 | -2.8 | 38.3 | 1.0 | 0.060 | -4.3 | 38.4 | 1.1 | 0.046 |
| 0.1 | 38.1 | 1.0 | 0.099 | -1.2 | 38.2 | 1.0 | 0.202 | -2.7 | 38.3 | 1.0 | 0.060 | -4.2 | 38.4 | 1.1 | 0.047 |
| 0.2 | 38.1 | 1.0 | 0.083 | -1.1 | 38.2 | 1.0 | 0.196 | -2.6 | 38.3 | 1.0 | 0.061 | -4.1 | 38.4 | 1.1 | 0.048 |
| -7.2 | 38.2 | 1.2 | 0.071 | -1.0 | 38.2 | 1.0 | 0.198 | -2.5 | 38.3 | 1.0 | 0.064 | -4.0 | 38.4 | 1.0 | 0.048 |
| -7.1 | 38.2 | 1.2 | 0.068 | -0.9 | 38.2 | 1.0 | 0.203 | -2.4 | 38.3 | 1.0 | 0.067 | -3.9 | 38.4 | 1.0 | 0.049 |
| -7.0 | 38.2 | 1.3 | 0.066 | -0.8 | 38.2 | 1.0 | 0.210 | -2.3 | 38.3 | 1.0 | 0.070 | -3.8 | 38.4 | 1.0 | 0.049 |
| -6.9 | 38.2 | 1.3 | 0.062 | -0.7 | 38.2 | 1.0 | 0.214 | -2.2 | 38.3 | 1.0 | 0.074 | -3.7 | 38.4 | 1.0 | 0.049 |
| -6.8 | 38.2 | 1.3 | 0.061 | -0.6 | 38.2 | 1.0 | 0.216 | -2.1 | 38.3 | 1.0 | 0.079 | -3.6 | 38.4 | 1.0 | 0.049 |
| -6.7 | 38.2 | 1.3 | 0.057 | -0.5 | 38.2 | 1.0 | 0.215 | -2.0 | 38.3 | 1.0 | 0.084 | -3.5 | 38.4 | 1.0 | 0.049 |
| -6.6 | 38.2 | 1.3 | 0.056 | -0.4 | 38.2 | 1.0 | 0.212 | -1.9 | 38.3 | 1.0 | 0.092 | -3.4 | 38.4 | 1.0 | 0.049 |
| -6.5 | 38.2 | 1.3 | 0.054 | -0.3 | 38.2 | 1.0 | 0.208 | -1.8 | 38.3 | 1.0 | 0.103 | -3.3 | 38.4 | 1.0 | 0.049 |

| Long. | Lat. | K | $a_{gR}$ | Long. | Lat. | K | $a_{gR}$ | Long. | Lat. | K | $a_{gR}$ | Long. | Lat. | K | $a_{gR}$ |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| -3.2 | 38.4 | 1.0 | 0.050 | -4.8 | 38.5 | 1.2 | 0.040 | -6.8 | 38.6 | 1.3 | 0.056 | 0.2 | 38.6 | 1.0 | 0.128 |
| -3.1 | 38.4 | 1.0 | 0.052 | -4.7 | 38.5 | 1.2 | 0.039 | -6.7 | 38.6 | 1.3 | 0.054 | 0.3 | 38.6 | 1.0 | 0.121 |
| -3.0 | 38.4 | 1.0 | 0.053 | -4.6 | 38.5 | 1.2 | 0.041 | -6.6 | 38.6 | 1.3 | 0.053 | 0.4 | 38.6 | 1.0 | 0.113 |
| -2.9 | 38.4 | 1.0 | 0.055 | -4.5 | 38.5 | 1.2 | 0.042 | -6.5 | 38.6 | 1.3 | 0.053 | 0.5 | 38.6 | 1.0 | 0.101 |
| -2.8 | 38.4 | 1.0 | 0.055 | -4.4 | 38.5 | 1.2 | 0.041 | -6.4 | 38.6 | 1.3 | 0.052 | 0.6 | 38.6 | 1.0 | 0.083 |
| -2.7 | 38.4 | 1.0 | 0.056 | -4.3 | 38.5 | 1.1 | 0.042 | -6.3 | 38.6 | 1.3 | 0.051 | 1.2 | 38.6 | 1.0 | 0.040 |
| -2.6 | 38.4 | 1.0 | 0.057 | -4.2 | 38.5 | 1.1 | 0.042 | -6.2 | 38.6 | 1.3 | 0.048 | 1.3 | 38.6 | 1.0 | 0.040 |
| -2.5 | 38.4 | 1.0 | 0.059 | -4.1 | 38.5 | 1.1 | 0.042 | -6.1 | 38.6 | 1.3 | 0.044 | 1.4 | 38.6 | 1.0 | 0.040 |
| -2.4 | 38.4 | 1.0 | 0.061 | -4.0 | 38.5 | 1.1 | 0.042 | -6.0 | 38.6 | 1.3 | 0.042 | 1.5 | 38.6 | 1.0 | 0.040 |
| -2.3 | 38.4 | 1.0 | 0.063 | -3.9 | 38.5 | 1.0 | 0.042 | -5.9 | 38.6 | 1.3 | 0.040 | 1.6 | 38.6 | 1.0 | 0.040 |
| -2.2 | 38.4 | 1.0 | 0.067 | -3.8 | 38.5 | 1.0 | 0.042 | -5.8 | 38.6 | 1.3 | 0.038 | 1.7 | 38.6 | 1.0 | 0.040 |
| -2.1 | 38.4 | 1.0 | 0.071 | -3.7 | 38.5 | 1.0 | 0.042 | -5.7 | 38.6 | 1.3 | 0.036 | -7.2 | 38.7 | 1.3 | 0.066 |
| -2.0 | 38.4 | 1.0 | 0.076 | -3.6 | 38.5 | 1.0 | 0.042 | -5.6 | 38.6 | 1.3 | 0.038 | -7.1 | 38.7 | 1.3 | 0.063 |
| -1.9 | 38.4 | 1.0 | 0.082 | -3.5 | 38.5 | 1.0 | 0.042 | -4.7 | 38.6 | 1.2 | 0.037 | -7.0 | 38.7 | 1.3 | 0.059 |
| -1.8 | 38.4 | 1.0 | 0.090 | -3.4 | 38.5 | 1.0 | 0.043 | -4.6 | 38.6 | 1.2 | 0.038 | -6.9 | 38.7 | 1.3 | 0.057 |
| -1.7 | 38.4 | 1.0 | 0.101 | -3.3 | 38.5 | 1.0 | 0.044 | -4.5 | 38.6 | 1.2 | 0.038 | -6.8 | 38.7 | 1.3 | 0.055 |
| -1.6 | 38.4 | 1.0 | 0.114 | -3.2 | 38.5 | 1.0 | 0.044 | -4.4 | 38.6 | 1.2 | 0.036 | -6.7 | 38.7 | 1.3 | 0.054 |
| -1.5 | 38.4 | 1.0 | 0.132 | -3.1 | 38.5 | 1.0 | 0.045 | -4.3 | 38.6 | 1.2 | 0.036 | -6.6 | 38.7 | 1.3 | 0.052 |
| -1.4 | 38.4 | 1.0 | 0.149 | -3.0 | 38.5 | 1.0 | 0.046 | -4.2 | 38.6 | 1.1 | 0.036 | -6.5 | 38.7 | 1.3 | 0.051 |
| -1.3 | 38.4 | 1.0 | 0.157 | -2.9 | 38.5 | 1.0 | 0.048 | -4.1 | 38.6 | 1.1 | 0.037 | -6.4 | 38.7 | 1.3 | 0.050 |
| -1.2 | 38.4 | 1.0 | 0.159 | -2.8 | 38.5 | 1.0 | 0.049 | -4.0 | 38.6 | 1.1 | 0.036 | -6.3 | 38.7 | 1.3 | 0.049 |
| -1.1 | 38.4 | 1.0 | 0.167 | -2.7 | 38.5 | 1.0 | 0.051 | -3.9 | 38.6 | 1.1 | 0.037 | -6.2 | 38.7 | 1.3 | 0.047 |
| -1.0 | 38.4 | 1.0 | 0.169 | -2.6 | 38.5 | 1.0 | 0.052 | -3.8 | 38.6 | 1.0 | 0.037 | -6.1 | 38.7 | 1.3 | 0.044 |
| -0.9 | 38.4 | 1.0 | 0.173 | -2.5 | 38.5 | 1.0 | 0.054 | -3.7 | 38.6 | 1.0 | 0.037 | -6.0 | 38.7 | 1.3 | 0.041 |
| -0.8 | 38.4 | 1.0 | 0.173 | -2.4 | 38.5 | 1.0 | 0.056 | -3.6 | 38.6 | 1.0 | 0.038 | -5.9 | 38.7 | 1.3 | 0.039 |
| -0.7 | 38.4 | 1.0 | 0.171 | -2.3 | 38.5 | 1.0 | 0.059 | -3.5 | 38.6 | 1.0 | 0.038 | -3.2 | 38.7 | 1.0 | 0.035 |
| -0.6 | 38.4 | 1.0 | 0.168 | -2.2 | 38.5 | 1.0 | 0.062 | -3.4 | 38.6 | 1.0 | 0.039 | -3.1 | 38.7 | 1.0 | 0.036 |
| -0.5 | 38.4 | 1.0 | 0.167 | -2.1 | 38.5 | 1.0 | 0.066 | -3.3 | 38.6 | 1.0 | 0.039 | -3.0 | 38.7 | 1.0 | 0.036 |
| -0.4 | 38.4 | 1.0 | 0.166 | -2.0 | 38.5 | 1.0 | 0.070 | -3.2 | 38.6 | 1.0 | 0.040 | -2.9 | 38.7 | 1.0 | 0.036 |
| -0.3 | 38.4 | 1.0 | 0.166 | -1.9 | 38.5 | 1.0 | 0.075 | -3.1 | 38.6 | 1.0 | 0.041 | -2.8 | 38.7 | 1.0 | 0.038 |
| -0.2 | 38.4 | 1.0 | 0.167 | -1.8 | 38.5 | 1.0 | 0.082 | -3.0 | 38.6 | 1.0 | 0.041 | -2.7 | 38.7 | 1.0 | 0.039 |
| -0.1 | 38.4 | 1.0 | 0.165 | -1.7 | 38.5 | 1.0 | 0.090 | -2.9 | 38.6 | 1.0 | 0.042 | -2.6 | 38.7 | 1.0 | 0.041 |
| 0.0 | 38.4 | 1.0 | 0.152 | -1.6 | 38.5 | 1.0 | 0.100 | -2.8 | 38.6 | 1.0 | 0.043 | -2.5 | 38.7 | 1.0 | 0.043 |

| | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 0.1 | 38.4 | 1.0 | 0.126 | -1.5 | 38.5 | 1.0 | 0.110 | -2.7 | 38.6 | 1.0 | 0.044 | -2.4 | 38.7 | 1.0 | 0.046 |
| 0.2 | 38.4 | 1.0 | 0.107 | -1.4 | 38.5 | 1.0 | 0.125 | -2.6 | 38.6 | 1.0 | 0.046 | -2.3 | 38.7 | 1.0 | 0.050 |
| 0.3 | 38.4 | 1.0 | 0.093 | -1.3 | 38.5 | 1.0 | 0.140 | -2.5 | 38.6 | 1.0 | 0.049 | -2.2 | 38.7 | 1.0 | 0.053 |
| 0.4 | 38.4 | 1.0 | 0.083 | -1.2 | 38.5 | 1.0 | 0.148 | -2.4 | 38.6 | 1.0 | 0.052 | -2.1 | 38.7 | 1.0 | 0.057 |
| -7.3 | 38.5 | 1.2 | 0.071 | -1.1 | 38.5 | 1.0 | 0.152 | -2.3 | 38.6 | 1.0 | 0.055 | -2.0 | 38.7 | 1.0 | 0.061 |
| -7.2 | 38.5 | 1.2 | 0.067 | -1.0 | 38.5 | 1.0 | 0.159 | -2.2 | 38.6 | 1.0 | 0.058 | -1.9 | 38.7 | 1.0 | 0.066 |
| -7.1 | 38.5 | 1.3 | 0.064 | -0.9 | 38.5 | 1.0 | 0.163 | -2.1 | 38.6 | 1.0 | 0.061 | -1.8 | 38.7 | 1.0 | 0.072 |
| -7.0 | 38.5 | 1.3 | 0.061 | -0.8 | 38.5 | 1.0 | 0.163 | -2.0 | 38.6 | 1.0 | 0.065 | -1.7 | 38.7 | 1.0 | 0.078 |
| -6.9 | 38.5 | 1.3 | 0.059 | -0.7 | 38.5 | 1.0 | 0.160 | -1.9 | 38.6 | 1.0 | 0.071 | -1.6 | 38.7 | 1.0 | 0.084 |
| -6.8 | 38.5 | 1.3 | 0.056 | -0.6 | 38.5 | 1.0 | 0.160 | -1.8 | 38.6 | 1.0 | 0.077 | -1.5 | 38.7 | 1.0 | 0.090 |
| -6.7 | 38.5 | 1.3 | 0.055 | -0.5 | 38.5 | 1.0 | 0.160 | -1.7 | 38.6 | 1.0 | 0.084 | -1.4 | 38.7 | 1.0 | 0.097 |
| -6.6 | 38.5 | 1.3 | 0.054 | -0.4 | 38.5 | 1.0 | 0.158 | -1.6 | 38.6 | 1.0 | 0.090 | -1.3 | 38.7 | 1.0 | 0.105 |
| -6.5 | 38.5 | 1.3 | 0.053 | -0.3 | 38.5 | 1.0 | 0.156 | -1.5 | 38.6 | 1.0 | 0.097 | -1.2 | 38.7 | 1.0 | 0.117 |
| -6.4 | 38.5 | 1.3 | 0.052 | -0.2 | 38.5 | 1.0 | 0.153 | -1.4 | 38.6 | 1.0 | 0.106 | -1.1 | 38.7 | 1.0 | 0.133 |
| -6.3 | 38.5 | 1.3 | 0.051 | -0.1 | 38.5 | 1.0 | 0.148 | -1.3 | 38.6 | 1.0 | 0.117 | -1.0 | 38.7 | 1.0 | 0.148 |
| -6.2 | 38.5 | 1.3 | 0.048 | 0.0 | 38.5 | 1.0 | 0.141 | -1.2 | 38.6 | 1.0 | 0.131 | -0.9 | 38.7 | 1.0 | 0.155 |
| -6.1 | 38.5 | 1.3 | 0.045 | 0.1 | 38.5 | 1.0 | 0.132 | -1.1 | 38.6 | 1.0 | 0.142 | -0.8 | 38.7 | 1.0 | 0.146 |
| -6.0 | 38.5 | 1.3 | 0.043 | 0.2 | 38.5 | 1.0 | 0.122 | -1.0 | 38.6 | 1.0 | 0.148 | -0.7 | 38.7 | 1.0 | 0.154 |
| -5.9 | 38.5 | 1.3 | 0.041 | 0.3 | 38.5 | 1.0 | 0.110 | -0.9 | 38.6 | 1.0 | 0.156 | -0.6 | 38.7 | 1.0 | 0.156 |
| -5.8 | 38.5 | 1.3 | 0.042 | 0.4 | 38.5 | 1.0 | 0.098 | -0.8 | 38.6 | 1.0 | 0.155 | -0.5 | 38.7 | 1.0 | 0.158 |
| -5.7 | 38.5 | 1.3 | 0.041 | 0.5 | 38.5 | 1.0 | 0.085 | -0.7 | 38.6 | 1.0 | 0.157 | -0.4 | 38.7 | 1.0 | 0.157 |
| -5.6 | 38.5 | 1.3 | 0.040 | 1.3 | 38.5 | 1.0 | 0.040 | -0.6 | 38.6 | 1.0 | 0.157 | -0.3 | 38.7 | 1.0 | 0.154 |
| -5.5 | 38.5 | 1.3 | 0.039 | 1.6 | 38.5 | 1.0 | 0.040 | -0.5 | 38.6 | 1.0 | 0.156 | -0.2 | 38.7 | 1.0 | 0.150 |
| -5.4 | 38.5 | 1.3 | 0.039 | -7.4 | 38.6 | 1.2 | 0.073 | -0.4 | 38.6 | 1.0 | 0.155 | -0.1 | 38.7 | 1.0 | 0.146 |
| -5.3 | 38.5 | 1.3 | 0.039 | -7.3 | 38.6 | 1.2 | 0.071 | -0.3 | 38.6 | 1.0 | 0.153 | 0.0 | 38.7 | 1.0 | 0.141 |
| -5.2 | 38.5 | 1.3 | 0.039 | -7.2 | 38.6 | 1.3 | 0.067 | -0.2 | 38.6 | 1.0 | 0.149 | 0.1 | 38.7 | 1.0 | 0.136 |
| -5.1 | 38.5 | 1.2 | 0.038 | -7.1 | 38.6 | 1.3 | 0.063 | -0.1 | 38.6 | 1.0 | 0.144 | 0.2 | 38.7 | 1.0 | 0.129 |
| -5.0 | 38.5 | 1.2 | 0.038 | -7.0 | 38.6 | 1.3 | 0.059 | 0.0 | 38.6 | 1.0 | 0.139 | 0.3 | 38.7 | 1.0 | 0.123 |
| -4.9 | 38.5 | 1.2 | 0.039 | -6.9 | 38.6 | 1.3 | 0.057 | 0.1 | 38.6 | 1.0 | 0.133 | 0.4 | 38.7 | 1.0 | 0.116 |

| Long. | Lat. | K | $a_{gR}$ | Long. | Lat. | K | $a_{gR}$ | Long. | Lat. | K | $a_{gR}$ | Long. | Lat. | K | $a_{gR}$ |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 0.5 | 38.7 | 1.0 | 0.103 | -7.2 | 38.9 | 1.3 | 0.063 | -6.0 | 39.0 | 1.3 | 0.038 | -1.0 | 39.1 | 1.0 | 0.118 |
| 1.2 | 38.7 | 1.0 | 0.040 | -7.1 | 38.9 | 1.3 | 0.061 | -2.4 | 39.0 | 1.0 | 0.037 | -0.9 | 39.1 | 1.0 | 0.131 |
| 1.3 | 38.7 | 1.0 | 0.040 | -7.0 | 38.9 | 1.3 | 0.058 | -2.3 | 39.0 | 1.0 | 0.039 | -0.8 | 39.1 | 1.0 | 0.141 |
| 1.4 | 38.7 | 1.0 | 0.040 | -6.9 | 38.9 | 1.3 | 0.055 | -2.2 | 39.0 | 1.0 | 0.041 | -0.7 | 39.1 | 1.0 | 0.148 |
| 1.5 | 38.7 | 1.0 | 0.040 | -6.8 | 38.9 | 1.3 | 0.053 | -2.1 | 39.0 | 1.0 | 0.043 | -0.6 | 39.1 | 1.0 | 0.153 |
| 1.6 | 38.7 | 1.0 | 0.040 | -6.7 | 38.9 | 1.3 | 0.051 | -2.0 | 39.0 | 1.0 | 0.046 | -0.5 | 39.1 | 1.0 | 0.153 |
| -7.3 | 38.8 | 1.3 | 0.068 | -6.6 | 38.9 | 1.3 | 0.048 | -1.9 | 39.0 | 1.0 | 0.047 | -0.4 | 39.1 | 1.0 | 0.153 |
| -7.2 | 38.8 | 1.3 | 0.064 | -6.5 | 38.9 | 1.3 | 0.047 | -1.8 | 39.0 | 1.0 | 0.052 | -0.3 | 39.1 | 1.0 | 0.152 |
| -7.1 | 38.8 | 1.3 | 0.062 | -6.4 | 38.9 | 1.3 | 0.046 | -1.7 | 39.0 | 1.0 | 0.056 | -0.2 | 39.1 | 1.0 | 0.148 |
| -7.0 | 38.8 | 1.3 | 0.059 | -6.3 | 38.9 | 1.3 | 0.044 | -1.6 | 39.0 | 1.0 | 0.062 | -0.1 | 39.1 | 1.0 | 0.134 |
| -6.9 | 38.8 | 1.3 | 0.056 | -6.2 | 38.9 | 1.3 | 0.042 | -1.5 | 39.0 | 1.0 | 0.067 | 0.0 | 39.1 | 1.0 | 0.131 |
| -6.8 | 38.8 | 1.3 | 0.054 | -6.1 | 38.9 | 1.3 | 0.041 | -1.4 | 39.0 | 1.0 | 0.074 | 0.1 | 39.1 | 1.0 | 0.128 |
| -6.7 | 38.8 | 1.3 | 0.052 | -6.0 | 38.9 | 1.3 | 0.039 | -1.3 | 39.0 | 1.0 | 0.081 | 0.2 | 39.1 | 1.0 | 0.124 |
| -6.6 | 38.8 | 1.3 | 0.051 | -5.9 | 38.9 | 1.3 | 0.038 | -1.2 | 39.0 | 1.0 | 0.091 | 0.3 | 39.1 | 1.0 | 0.111 |
| -6.5 | 38.8 | 1.3 | 0.050 | -2.5 | 38.9 | 1.0 | 0.036 | -1.1 | 39.0 | 1.0 | 0.105 | 0.4 | 39.1 | 1.0 | 0.090 |
| -6.4 | 38.8 | 1.3 | 0.048 | -2.4 | 38.9 | 1.0 | 0.039 | -1.0 | 39.0 | 1.0 | 0.124 | 1.1 | 39.1 | 1.0 | 0.040 |
| -6.3 | 38.8 | 1.3 | 0.047 | -2.3 | 38.9 | 1.0 | 0.041 | -0.9 | 39.0 | 1.0 | 0.138 | 1.2 | 39.1 | 1.0 | 0.040 |
| -6.2 | 38.8 | 1.3 | 0.044 | -2.2 | 38.9 | 1.0 | 0.044 | -0.8 | 39.0 | 1.0 | 0.148 | 1.3 | 39.1 | 1.0 | 0.040 |
| -6.1 | 38.8 | 1.3 | 0.042 | -2.1 | 38.9 | 1.0 | 0.047 | -0.7 | 39.0 | 1.0 | 0.157 | 1.4 | 39.1 | 1.0 | 0.040 |
| -6.0 | 38.8 | 1.3 | 0.040 | -2.0 | 38.9 | 1.0 | 0.050 | -0.6 | 39.0 | 1.0 | 0.160 | 1.5 | 39.1 | 1.0 | 0.040 |
| -5.9 | 38.8 | 1.3 | 0.039 | -1.9 | 38.9 | 1.0 | 0.053 | -0.5 | 39.0 | 1.0 | 0.162 | 1.6 | 39.1 | 1.0 | 0.040 |
| -2.7 | 38.8 | 1.0 | 0.035 | -1.8 | 38.9 | 1.0 | 0.057 | -0.4 | 39.0 | 1.0 | 0.162 | 1.7 | 39.1 | 1.0 | 0.040 |
| -2.6 | 38.8 | 1.0 | 0.037 | -1.7 | 38.9 | 1.0 | 0.063 | -0.3 | 39.0 | 1.0 | 0.142 | 2.7 | 39.1 | 1.0 | 0.040 |
| -2.5 | 38.8 | 1.0 | 0.039 | -1.6 | 38.9 | 1.0 | 0.070 | -0.2 | 39.0 | 1.0 | 0.140 | 2.9 | 39.1 | 1.0 | 0.040 |
| -2.4 | 38.8 | 1.0 | 0.042 | -1.5 | 38.9 | 1.0 | 0.077 | -0.1 | 39.0 | 1.0 | 0.142 | 3.2 | 39.1 | 1.0 | 0.040 |
| -2.3 | 38.8 | 1.0 | 0.045 | -1.4 | 38.9 | 1.0 | 0.083 | 0.0 | 39.0 | 1.0 | 0.139 | -7.2 | 39.2 | 1.3 | 0.058 |
| -2.2 | 38.8 | 1.0 | 0.048 | -1.3 | 38.9 | 1.0 | 0.090 | 0.1 | 39.0 | 1.0 | 0.134 | -7.1 | 39.2 | 1.3 | 0.054 |
| -2.1 | 38.8 | 1.0 | 0.051 | -1.2 | 38.9 | 1.0 | 0.098 | 0.2 | 39.0 | 1.0 | 0.129 | -7.0 | 39.2 | 1.3 | 0.051 |
| -2.0 | 38.8 | 1.0 | 0.055 | -1.1 | 38.9 | 1.0 | 0.110 | 0.3 | 39.0 | 1.0 | 0.120 | -6.9 | 39.2 | 1.3 | 0.048 |
| -1.9 | 38.8 | 1.0 | 0.059 | -1.0 | 38.9 | 1.0 | 0.128 | 0.4 | 39.0 | 1.0 | 0.101 | -6.8 | 39.2 | 1.3 | 0.046 |
| -1.8 | 38.8 | 1.0 | 0.065 | -0.9 | 38.9 | 1.0 | 0.144 | 1.1 | 39.0 | 1.0 | 0.040 | -6.7 | 39.2 | 1.3 | 0.044 |
| -1.7 | 38.8 | 1.0 | 0.072 | -0.8 | 38.9 | 1.0 | 0.152 | 1.2 | 39.0 | 1.0 | 0.040 | -6.6 | 39.2 | 1.3 | 0.042 |
| -1.6 | 38.8 | 1.0 | 0.078 | -0.7 | 38.9 | 1.0 | 0.157 | 1.3 | 39.0 | 1.0 | 0.040 | -6.5 | 39.2 | 1.3 | 0.041 |

| | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| -1.5 | 38.8 | 1.0 | 0.083 | -0.6 | 38.9 | 1.0 | 0.163 | 1.4 | 39.0 | 1.0 | 0.040 | -6.4 | 39.2 | 1.3 | 0.039 |
| -1.4 | 38.8 | 1.0 | 0.089 | -0.5 | 38.9 | 1.0 | 0.147 | 1.5 | 39.0 | 1.0 | 0.040 | -6.3 | 39.2 | 1.3 | 0.038 |
| -1.3 | 38.8 | 1.0 | 0.096 | -0.4 | 38.9 | 1.0 | 0.153 | 1.6 | 39.0 | 1.0 | 0.040 | -2.1 | 39.2 | 1.0 | 0.037 |
| -1.2 | 38.8 | 1.0 | 0.105 | -0.3 | 38.9 | 1.0 | 0.151 | -7.3 | 39.1 | 1.3 | 0.062 | -2.0 | 39.2 | 1.0 | 0.039 |
| -1.1 | 38.8 | 1.0 | 0.117 | -0.2 | 38.9 | 1.0 | 0.148 | -7.2 | 39.1 | 1.3 | 0.060 | -1.9 | 39.2 | 1.0 | 0.041 |
| -1.0 | 38.8 | 1.0 | 0.133 | -0.1 | 38.9 | 1.0 | 0.144 | -7.1 | 39.1 | 1.3 | 0.056 | -1.8 | 39.2 | 1.0 | 0.045 |
| -0.9 | 38.8 | 1.0 | 0.148 | 0.0 | 38.9 | 1.0 | 0.142 | -7.0 | 39.1 | 1.3 | 0.054 | -1.7 | 39.2 | 1.0 | 0.047 |
| -0.8 | 38.8 | 1.0 | 0.155 | 0.1 | 38.9 | 1.0 | 0.137 | -6.9 | 39.1 | 1.3 | 0.051 | -1.6 | 39.2 | 1.0 | 0.051 |
| -0.7 | 38.8 | 1.0 | 0.159 | 0.2 | 38.9 | 1.0 | 0.132 | -6.8 | 39.1 | 1.3 | 0.048 | -1.5 | 39.2 | 1.0 | 0.056 |
| -0.6 | 38.8 | 1.0 | 0.154 | 0.3 | 38.9 | 1.0 | 0.124 | -6.7 | 39.1 | 1.3 | 0.046 | -1.4 | 39.2 | 1.0 | 0.063 |
| -0.5 | 38.8 | 1.0 | 0.156 | 0.4 | 38.9 | 1.0 | 0.110 | -6.6 | 39.1 | 1.3 | 0.044 | -1.3 | 39.2 | 1.0 | 0.070 |
| -0.4 | 38.8 | 1.0 | 0.155 | 1.1 | 38.9 | 1.0 | 0.040 | -6.5 | 39.1 | 1.3 | 0.042 | -1.2 | 39.2 | 1.0 | 0.080 |
| -0.3 | 38.8 | 1.0 | 0.155 | 1.2 | 38.9 | 1.0 | 0.040 | -6.4 | 39.1 | 1.3 | 0.041 | -1.1 | 39.2 | 1.0 | 0.095 |
| -0.2 | 38.8 | 1.0 | 0.151 | 1.3 | 38.9 | 1.0 | 0.040 | -6.3 | 39.1 | 1.3 | 0.040 | -1.0 | 39.2 | 1.0 | 0.113 |
| -0.1 | 38.8 | 1.0 | 0.147 | 1.4 | 38.9 | 1.0 | 0.040 | -6.2 | 39.1 | 1.3 | 0.039 | -0.9 | 39.2 | 1.0 | 0.126 |
| 0.0 | 38.8 | 1.0 | 0.142 | 1.5 | 38.9 | 1.0 | 0.040 | -6.1 | 39.1 | 1.3 | 0.037 | -0.8 | 39.2 | 1.0 | 0.131 |
| 0.1 | 38.8 | 1.0 | 0.137 | 1.6 | 38.9 | 1.0 | 0.040 | -2.3 | 39.1 | 1.0 | 0.037 | -0.7 | 39.2 | 1.0 | 0.137 |
| 0.2 | 38.8 | 1.0 | 0.131 | -7.2 | 39.0 | 1.3 | 0.061 | -2.2 | 39.1 | 1.0 | 0.039 | -0.6 | 39.2 | 1.0 | 0.141 |
| 0.3 | 38.8 | 1.0 | 0.124 | -7.1 | 39.0 | 1.3 | 0.059 | -2.1 | 39.1 | 1.0 | 0.039 | -0.5 | 39.2 | 1.0 | 0.143 |
| 0.4 | 38.8 | 1.0 | 0.114 | -7.0 | 39.0 | 1.3 | 0.056 | -2.0 | 39.1 | 1.0 | 0.041 | -0.4 | 39.2 | 1.0 | 0.144 |
| 0.5 | 38.8 | 1.0 | 0.097 | -6.9 | 39.0 | 1.3 | 0.054 | -1.9 | 39.1 | 1.0 | 0.044 | -0.3 | 39.2 | 1.0 | 0.143 |
| 0.6 | 38.8 | 1.0 | 0.077 | -6.8 | 39.0 | 1.3 | 0.051 | -1.8 | 39.1 | 1.0 | 0.048 | -0.2 | 39.2 | 1.0 | 0.141 |
| 1.1 | 38.8 | 1.0 | 0.040 | -6.7 | 39.0 | 1.3 | 0.049 | -1.7 | 39.1 | 1.0 | 0.051 | -0.1 | 39.2 | 1.0 | 0.139 |
| 1.2 | 38.8 | 1.0 | 0.040 | -6.6 | 39.0 | 1.3 | 0.046 | -1.6 | 39.1 | 1.0 | 0.055 | 0.0 | 39.2 | 1.0 | 0.135 |
| 1.3 | 38.8 | 1.0 | 0.040 | -6.5 | 39.0 | 1.3 | 0.044 | -1.5 | 39.1 | 1.0 | 0.060 | 0.1 | 39.2 | 1.0 | 0.129 |
| 1.4 | 38.8 | 1.0 | 0.040 | -6.4 | 39.0 | 1.3 | 0.043 | -1.4 | 39.1 | 1.0 | 0.066 | 0.2 | 39.2 | 1.0 | 0.115 |
| 1.5 | 38.8 | 1.0 | 0.040 | -6.3 | 39.0 | 1.3 | 0.042 | -1.3 | 39.1 | 1.0 | 0.074 | 0.3 | 39.2 | 1.0 | 0.099 |
| 1.6 | 38.8 | 1.0 | 0.040 | -6.2 | 39.0 | 1.3 | 0.040 | -1.2 | 39.1 | 1.0 | 0.084 | 0.4 | 39.2 | 1.0 | 0.080 |
| 1.7 | 38.8 | 1.0 | 0.040 | -6.1 | 39.0 | 1.3 | 0.039 | -1.1 | 39.1 | 1.0 | 0.100 | 0.5 | 39.2 | 1.0 | 0.066 |

| Long. | Lat. | K | $a_{gR}$ | Long. | Lat. | K | $a_{gR}$ | Long. | Lat. | K | $a_{gR}$ | Long. | Lat. | K | $a_{gR}$ |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1.2 | 39.2 | 1.0 | 0.040 | -7.1 | 39.4 | 1.3 | 0.049 | -0.9 | 39.5 | 1.0 | 0.089 | 2.5 | 39.6 | 1.0 | 0.040 |
| 1.3 | 39.2 | 1.0 | 0.040 | -7.0 | 39.4 | 1.3 | 0.046 | -0.8 | 39.5 | 1.0 | 0.096 | 2.6 | 39.6 | 1.0 | 0.040 |
| 1.4 | 39.2 | 1.0 | 0.040 | -6.9 | 39.4 | 1.3 | 0.044 | -0.7 | 39.5 | 1.0 | 0.101 | 2.7 | 39.6 | 1.0 | 0.042 |
| 1.5 | 39.2 | 1.0 | 0.040 | -6.8 | 39.4 | 1.3 | 0.042 | -0.6 | 39.5 | 1.0 | 0.103 | 2.8 | 39.6 | 1.0 | 0.044 |
| 1.6 | 39.2 | 1.0 | 0.040 | -6.7 | 39.4 | 1.3 | 0.040 | -0.5 | 39.5 | 1.0 | 0.104 | 2.9 | 39.6 | 1.0 | 0.044 |
| 1.7 | 39.2 | 1.0 | 0.040 | -6.6 | 39.4 | 1.3 | 0.039 | -0.4 | 39.5 | 1.0 | 0.104 | 3.0 | 39.6 | 1.0 | 0.044 |
| 2.6 | 39.2 | 1.0 | 0.040 | -6.5 | 39.4 | 1.3 | 0.038 | -0.3 | 39.5 | 1.0 | 0.102 | 3.1 | 39.6 | 1.0 | 0.045 |
| 2.7 | 39.2 | 1.0 | 0.040 | -1.9 | 39.4 | 1.0 | 0.038 | -0.2 | 39.5 | 1.0 | 0.101 | 3.2 | 39.6 | 1.0 | 0.043 |
| 2.8 | 39.2 | 1.0 | 0.040 | -1.8 | 39.4 | 1.0 | 0.041 | -0.1 | 39.5 | 1.0 | 0.098 | 3.3 | 39.6 | 1.0 | 0.040 |
| 2.9 | 39.2 | 1.0 | 0.040 | -1.7 | 39.4 | 1.0 | 0.043 | 0.0 | 39.5 | 1.0 | 0.095 | 3.4 | 39.6 | 1.0 | 0.040 |
| 3.0 | 39.2 | 1.0 | 0.040 | -1.6 | 39.4 | 1.0 | 0.046 | 0.1 | 39.5 | 1.0 | 0.090 | 3.5 | 39.6 | 1.0 | 0.040 |
| 3.1 | 39.2 | 1.0 | 0.040 | -1.5 | 39.4 | 1.0 | 0.050 | 0.2 | 39.5 | 1.0 | 0.082 | -7.6 | 39.7 | 1.1 | 0.059 |
| 3.2 | 39.2 | 1.0 | 0.040 | -1.4 | 39.4 | 1.0 | 0.055 | 0.3 | 39.5 | 1.0 | 0.070 | -7.5 | 39.7 | 1.1 | 0.055 |
| 3.4 | 39.2 | 1.0 | 0.040 | -1.3 | 39.4 | 1.0 | 0.062 | 2.2 | 39.5 | 1.0 | 0.040 | -7.4 | 39.7 | 1.1 | 0.050 |
| -7.4 | 39.3 | 1.2 | 0.060 | -1.2 | 39.4 | 1.0 | 0.070 | 2.3 | 39.5 | 1.0 | 0.040 | -7.3 | 39.7 | 1.1 | 0.047 |
| -7.3 | 39.3 | 1.3 | 0.058 | -1.1 | 39.4 | 1.0 | 0.081 | 2.4 | 39.5 | 1.0 | 0.040 | -7.2 | 39.7 | 1.1 | 0.045 |
| -7.2 | 39.3 | 1.3 | 0.056 | -1.0 | 39.4 | 1.0 | 0.095 | 2.5 | 39.5 | 1.0 | 0.040 | -7.1 | 39.7 | 1.1 | 0.043 |
| -7.1 | 39.3 | 1.3 | 0.052 | -0.9 | 39.4 | 1.0 | 0.107 | 2.6 | 39.5 | 1.0 | 0.040 | -7.0 | 39.7 | 1.1 | 0.042 |
| -7.0 | 39.3 | 1.3 | 0.048 | -0.8 | 39.4 | 1.0 | 0.115 | 2.7 | 39.5 | 1.0 | 0.040 | -6.9 | 39.7 | 1.2 | 0.040 |
| -6.9 | 39.3 | 1.3 | 0.045 | -0.7 | 39.4 | 1.0 | 0.120 | 2.8 | 39.5 | 1.0 | 0.040 | -6.8 | 39.7 | 1.2 | 0.039 |
| -6.8 | 39.3 | 1.3 | 0.043 | -0.6 | 39.4 | 1.0 | 0.122 | 2.9 | 39.5 | 1.0 | 0.040 | -6.7 | 39.7 | 1.2 | 0.038 |
| -6.7 | 39.3 | 1.3 | 0.042 | -0.5 | 39.4 | 1.0 | 0.123 | 3.0 | 39.5 | 1.0 | 0.040 | -1.7 | 39.7 | 1.0 | 0.039 |
| -6.6 | 39.3 | 1.3 | 0.040 | -0.4 | 39.4 | 1.0 | 0.123 | 3.1 | 39.5 | 1.0 | 0.040 | -1.6 | 39.7 | 1.0 | 0.042 |
| -6.5 | 39.3 | 1.3 | 0.039 | -0.3 | 39.4 | 1.0 | 0.121 | 3.2 | 39.5 | 1.0 | 0.040 | -1.5 | 39.7 | 1.0 | 0.045 |
| -6.4 | 39.3 | 1.3 | 0.038 | -0.2 | 39.4 | 1.0 | 0.119 | 3.3 | 39.5 | 1.0 | 0.040 | -1.4 | 39.7 | 1.0 | 0.048 |
| -2.0 | 39.3 | 1.0 | 0.038 | -0.1 | 39.4 | 1.0 | 0.116 | 3.4 | 39.5 | 1.0 | 0.040 | -1.3 | 39.7 | 1.0 | 0.051 |
| -1.9 | 39.3 | 1.0 | 0.040 | 0.0 | 39.4 | 1.0 | 0.114 | 3.5 | 39.5 | 1.0 | 0.040 | -1.2 | 39.7 | 1.0 | 0.055 |
| -1.8 | 39.3 | 1.0 | 0.042 | 0.1 | 39.4 | 1.0 | 0.109 | 3.6 | 39.5 | 1.0 | 0.040 | -1.1 | 39.7 | 1.0 | 0.059 |
| -1.7 | 39.3 | 1.0 | 0.045 | 0.2 | 39.4 | 1.0 | 0.098 | -7.5 | 39.6 | 1.2 | 0.057 | -1.0 | 39.7 | 1.0 | 0.062 |
| -1.6 | 39.3 | 1.0 | 0.049 | 0.3 | 39.4 | 1.0 | 0.083 | -7.4 | 39.6 | 1.2 | 0.052 | -0.9 | 39.7 | 1.0 | 0.065 |
| -1.5 | 39.3 | 1.0 | 0.054 | 2.2 | 39.4 | 1.0 | 0.040 | -7.3 | 39.6 | 1.2 | 0.049 | -0.8 | 39.7 | 1.0 | 0.069 |
| -1.4 | 39.3 | 1.0 | 0.060 | 2.4 | 39.4 | 1.0 | 0.040 | -7.2 | 39.6 | 1.2 | 0.047 | -0.7 | 39.7 | 1.0 | 0.072 |
| -1.3 | 39.3 | 1.0 | 0.067 | 2.5 | 39.4 | 1.0 | 0.040 | -7.1 | 39.6 | 1.2 | 0.045 | -0.6 | 39.7 | 1.0 | 0.073 |

| | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| -1.2 | 39.3 | 1.0 | 0.076 | 2.6 | 39.4 | 1.0 | 0.040 | -7.0 | 39.6 | 1.2 | 0.043 | -0.5 | 39.7 | 1.0 | 0.073 |
| -1.1 | 39.3 | 1.0 | 0.089 | 2.7 | 39.4 | 1.0 | 0.040 | -6.9 | 39.6 | 1.2 | 0.042 | -0.4 | 39.7 | 1.0 | 0.072 |
| -1.0 | 39.3 | 1.0 | 0.106 | 2.8 | 39.4 | 1.0 | 0.040 | -6.8 | 39.6 | 1.2 | 0.040 | -0.3 | 39.7 | 1.0 | 0.071 |
| -0.9 | 39.3 | 1.0 | 0.118 | 2.9 | 39.4 | 1.0 | 0.040 | -6.7 | 39.6 | 1.3 | 0.039 | -0.2 | 39.7 | 1.0 | 0.069 |
| -0.8 | 39.3 | 1.0 | 0.125 | 3.0 | 39.4 | 1.0 | 0.040 | -1.8 | 39.6 | 1.0 | 0.038 | -0.1 | 39.7 | 1.0 | 0.067 |
| -0.7 | 39.3 | 1.0 | 0.130 | 3.1 | 39.4 | 1.0 | 0.040 | -1.7 | 39.6 | 1.0 | 0.040 | 0.0 | 39.7 | 1.0 | 0.065 |
| -0.6 | 39.3 | 1.0 | 0.132 | 3.2 | 39.4 | 1.0 | 0.040 | -1.6 | 39.6 | 1.0 | 0.042 | 0.1 | 39.7 | 1.0 | 0.062 |
| -0.5 | 39.3 | 1.0 | 0.134 | 3.3 | 39.4 | 1.0 | 0.040 | -1.5 | 39.6 | 1.0 | 0.045 | 0.2 | 39.7 | 1.0 | 0.058 |
| -0.4 | 39.3 | 1.0 | 0.134 | 3.4 | 39.4 | 1.0 | 0.040 | -1.4 | 39.6 | 1.0 | 0.049 | 0.3 | 39.7 | 1.0 | 0.054 |
| -0.3 | 39.3 | 1.0 | 0.132 | -7.6 | 39.5 | 1.2 | 0.063 | -1.3 | 39.6 | 1.0 | 0.053 | 0.4 | 39.7 | 1.0 | 0.049 |
| -0.2 | 39.3 | 1.0 | 0.130 | -7.4 | 39.5 | 1.2 | 0.055 | -1.2 | 39.6 | 1.0 | 0.058 | 0.6 | 39.7 | 1.0 | 0.038 |
| -0.1 | 39.3 | 1.0 | 0.128 | -7.3 | 39.5 | 1.2 | 0.052 | -1.1 | 39.6 | 1.0 | 0.063 | 2.2 | 39.7 | 1.0 | 0.040 |
| 0.0 | 39.3 | 1.0 | 0.126 | -7.2 | 39.5 | 1.2 | 0.049 | -1.0 | 39.6 | 1.0 | 0.069 | 2.3 | 39.7 | 1.0 | 0.040 |
| 0.1 | 39.3 | 1.0 | 0.121 | -7.1 | 39.5 | 1.2 | 0.046 | -0.9 | 39.6 | 1.0 | 0.075 | 2.4 | 39.7 | 1.0 | 0.040 |
| 0.2 | 39.3 | 1.0 | 0.110 | -7.0 | 39.5 | 1.3 | 0.044 | -0.8 | 39.6 | 1.0 | 0.080 | 2.5 | 39.7 | 1.0 | 0.040 |
| 0.3 | 39.3 | 1.0 | 0.093 | -6.9 | 39.5 | 1.3 | 0.043 | -0.7 | 39.6 | 1.0 | 0.084 | 2.6 | 39.7 | 1.0 | 0.040 |
| 2.4 | 39.3 | 1.0 | 0.040 | -6.8 | 39.5 | 1.3 | 0.041 | -0.6 | 39.6 | 1.0 | 0.086 | 2.7 | 39.7 | 1.0 | 0.041 |
| 2.6 | 39.3 | 1.0 | 0.040 | -6.7 | 39.5 | 1.3 | 0.039 | -0.5 | 39.6 | 1.0 | 0.086 | 2.8 | 39.7 | 1.0 | 0.042 |
| 2.7 | 39.3 | 1.0 | 0.040 | -6.6 | 39.5 | 1.3 | 0.038 | -0.4 | 39.6 | 1.0 | 0.086 | 2.9 | 39.7 | 1.0 | 0.043 |
| 2.8 | 39.3 | 1.0 | 0.040 | -1.9 | 39.5 | 1.0 | 0.037 | -0.3 | 39.6 | 1.0 | 0.085 | 3.0 | 39.7 | 1.0 | 0.042 |
| 2.9 | 39.3 | 1.0 | 0.040 | -1.8 | 39.5 | 1.0 | 0.039 | -0.2 | 39.6 | 1.0 | 0.083 | 3.1 | 39.7 | 1.0 | 0.040 |
| 3.0 | 39.3 | 1.0 | 0.040 | -1.7 | 39.5 | 1.0 | 0.041 | -0.1 | 39.6 | 1.0 | 0.080 | 3.2 | 39.7 | 1.0 | 0.040 |
| 3.1 | 39.3 | 1.0 | 0.040 | -1.6 | 39.5 | 1.0 | 0.044 | 0.0 | 39.6 | 1.0 | 0.078 | 3.3 | 39.7 | 1.0 | 0.040 |
| 3.2 | 39.3 | 1.0 | 0.040 | -1.5 | 39.5 | 1.0 | 0.047 | 0.1 | 39.6 | 1.0 | 0.074 | 3.4 | 39.7 | 1.0 | 0.040 |
| 3.3 | 39.3 | 1.0 | 0.040 | -1.4 | 39.5 | 1.0 | 0.051 | 0.2 | 39.6 | 1.0 | 0.067 | 3.5 | 39.7 | 1.0 | 0.040 |
| 3.4 | 39.3 | 1.0 | 0.040 | -1.3 | 39.5 | 1.0 | 0.056 | 0.3 | 39.6 | 1.0 | 0.060 | 3.6 | 39.7 | 1.0 | 0.040 |
| 3.5 | 39.3 | 1.0 | 0.040 | -1.2 | 39.5 | 1.0 | 0.063 | 2.2 | 39.6 | 1.0 | 0.040 | 4.1 | 39.7 | 1.0 | 0.040 |
| -7.3 | 39.4 | 1.2 | 0.055 | -1.1 | 39.5 | 1.0 | 0.071 | 2.3 | 39.6 | 1.0 | 0.040 | 4.2 | 39.7 | 1.0 | 0.040 |
| -7.2 | 39.4 | 1.3 | 0.052 | -1.0 | 39.5 | 1.0 | 0.080 | 2.4 | 39.6 | 1.0 | 0.040 | 4.3 | 39.7 | 1.0 | 0.040 |

| Long. | Lat. | K | $a_{gR}$ | Long. | Lat. | K | $a_{gR}$ | Long. | Lat. | K | $a_{gR}$ | Long. | Lat. | K | $a_{gR}$ |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 4.4 | 39.7 | 1.0 | 0.040 | -0.5 | 39.9 | 1.0 | 0.055 | 2.8 | 40.0 | 1.0 | 0.040 | -1.6 | 40.2 | 1.0 | 0.044 |
| -7.1 | 39.8 | 1.1 | 0.042 | -0.4 | 39.9 | 1.0 | 0.056 | 2.9 | 40.0 | 1.0 | 0.040 | -1.5 | 40.2 | 1.0 | 0.047 |
| -7.0 | 39.8 | 1.1 | 0.041 | -0.3 | 39.9 | 1.0 | 0.055 | 3.0 | 40.0 | 1.0 | 0.040 | -1.4 | 40.2 | 1.0 | 0.048 |
| -6.9 | 39.8 | 1.1 | 0.039 | -0.2 | 39.9 | 1.0 | 0.054 | 3.1 | 40.0 | 1.0 | 0.040 | -1.3 | 40.2 | 1.0 | 0.049 |
| -6.8 | 39.8 | 1.2 | 0.038 | -0.1 | 39.9 | 1.0 | 0.053 | 3.2 | 40.0 | 1.0 | 0.040 | -1.2 | 40.2 | 1.0 | 0.051 |
| -1.7 | 39.8 | 1.0 | 0.039 | 0.0 | 39.9 | 1.0 | 0.052 | 3.3 | 40.0 | 1.0 | 0.040 | -1.1 | 40.2 | 1.0 | 0.049 |
| -1.6 | 39.8 | 1.0 | 0.042 | 0.1 | 39.9 | 1.0 | 0.050 | 3.4 | 40.0 | 1.0 | 0.040 | -1.0 | 40.2 | 1.0 | 0.047 |
| -1.5 | 39.8 | 1.0 | 0.045 | 0.2 | 39.9 | 1.0 | 0.049 | 3.7 | 40.0 | 1.0 | 0.040 | -0.9 | 40.2 | 1.0 | 0.046 |
| -1.4 | 39.8 | 1.0 | 0.047 | 0.3 | 39.9 | 1.0 | 0.047 | 3.8 | 40.0 | 1.0 | 0.040 | -0.8 | 40.2 | 1.0 | 0.045 |
| -1.3 | 39.8 | 1.0 | 0.050 | 0.4 | 39.9 | 1.0 | 0.044 | 3.9 | 40.0 | 1.0 | 0.040 | -0.7 | 40.2 | 1.0 | 0.044 |
| -1.2 | 39.8 | 1.0 | 0.052 | 0.5 | 39.9 | 1.0 | 0.041 | 4.0 | 40.0 | 1.0 | 0.040 | -0.6 | 40.2 | 1.0 | 0.044 |
| -1.1 | 39.8 | 1.0 | 0.054 | 0.6 | 39.9 | 1.0 | 0.038 | 4.1 | 40.0 | 1.0 | 0.040 | -0.5 | 40.2 | 1.0 | 0.044 |
| -1.0 | 39.8 | 1.0 | 0.056 | 0.7 | 39.9 | 1.0 | 0.034 | 4.2 | 40.0 | 1.0 | 0.040 | -0.4 | 40.2 | 1.0 | 0.044 |
| -0.9 | 39.8 | 1.0 | 0.058 | 2.4 | 39.9 | 1.0 | 0.040 | 4.3 | 40.0 | 1.0 | 0.040 | -0.3 | 40.2 | 1.0 | 0.044 |
| -0.8 | 39.8 | 1.0 | 0.059 | 2.5 | 39.9 | 1.0 | 0.040 | 4.4 | 40.0 | 1.0 | 0.040 | -0.2 | 40.2 | 1.0 | 0.044 |
| -0.7 | 39.8 | 1.0 | 0.061 | 2.6 | 39.9 | 1.0 | 0.040 | -7.1 | 40.1 | 1.0 | 0.041 | -0.1 | 40.2 | 1.0 | 0.044 |
| -0.6 | 39.8 | 1.0 | 0.061 | 2.7 | 39.9 | 1.0 | 0.040 | -7.0 | 40.1 | 1.0 | 0.039 | 0.0 | 40.2 | 1.0 | 0.044 |
| -0.5 | 39.8 | 1.0 | 0.062 | 2.8 | 39.9 | 1.0 | 0.040 | -1.9 | 40.1 | 1.0 | 0.038 | 0.1 | 40.2 | 1.0 | 0.044 |
| -0.4 | 39.8 | 1.0 | 0.062 | 2.9 | 39.9 | 1.0 | 0.040 | -1.8 | 40.1 | 1.0 | 0.040 | 0.2 | 40.2 | 1.0 | 0.043 |
| -0.3 | 39.8 | 1.0 | 0.062 | 3.0 | 39.9 | 1.0 | 0.040 | -1.7 | 40.1 | 1.0 | 0.042 | 0.3 | 40.2 | 1.0 | 0.042 |
| -0.2 | 39.8 | 1.0 | 0.060 | 3.1 | 39.9 | 1.0 | 0.040 | -1.6 | 40.1 | 1.0 | 0.044 | 0.4 | 40.2 | 1.0 | 0.041 |
| -0.1 | 39.8 | 1.0 | 0.059 | 3.2 | 39.9 | 1.0 | 0.040 | -1.5 | 40.1 | 1.0 | 0.046 | 0.5 | 40.2 | 1.0 | 0.040 |
| 0.0 | 39.8 | 1.0 | 0.057 | 3.3 | 39.9 | 1.0 | 0.040 | -1.4 | 40.1 | 1.0 | 0.048 | 0.6 | 40.2 | 1.0 | 0.039 |
| 0.1 | 39.8 | 1.0 | 0.055 | 3.4 | 39.9 | 1.0 | 0.040 | -1.3 | 40.1 | 1.0 | 0.051 | 0.7 | 40.2 | 1.0 | 0.038 |
| 0.2 | 39.8 | 1.0 | 0.052 | 3.5 | 39.9 | 1.0 | 0.040 | -1.2 | 40.1 | 1.0 | 0.051 | 0.8 | 40.2 | 1.0 | 0.035 |
| 0.3 | 39.8 | 1.0 | 0.049 | 3.6 | 39.9 | 1.0 | 0.040 | -1.1 | 40.1 | 1.0 | 0.049 | 0.9 | 40.2 | 1.0 | 0.033 |
| 0.4 | 39.8 | 1.0 | 0.046 | 3.7 | 39.9 | 1.0 | 0.040 | -1.0 | 40.1 | 1.0 | 0.048 | 3.7 | 40.2 | 1.0 | 0.040 |
| 0.5 | 39.8 | 1.0 | 0.042 | 3.8 | 39.9 | 1.0 | 0.040 | -0.9 | 40.1 | 1.0 | 0.047 | 3.8 | 40.2 | 1.0 | 0.040 |
| 2.3 | 39.8 | 1.0 | 0.040 | 3.9 | 39.9 | 1.0 | 0.040 | -0.8 | 40.1 | 1.0 | 0.047 | 3.9 | 40.2 | 1.0 | 0.040 |
| 2.4 | 39.8 | 1.0 | 0.040 | 4.0 | 39.9 | 1.0 | 0.040 | -0.7 | 40.1 | 1.0 | 0.047 | 4.0 | 40.2 | 1.0 | 0.040 |
| 2.5 | 39.8 | 1.0 | 0.040 | 4.1 | 39.9 | 1.0 | 0.040 | -0.6 | 40.1 | 1.0 | 0.047 | 4.1 | 40.2 | 1.0 | 0.040 |
| 2.6 | 39.8 | 1.0 | 0.040 | 4.2 | 39.9 | 1.0 | 0.040 | -0.5 | 40.1 | 1.0 | 0.047 | 4.2 | 40.2 | 1.0 | 0.040 |
| 2.7 | 39.8 | 1.0 | 0.040 | 4.3 | 39.9 | 1.0 | 0.040 | -0.4 | 40.1 | 1.0 | 0.047 | 4.3 | 40.2 | 1.0 | 0.040 |
| 2.8 | 39.8 | 1.0 | 0.040 | 4.4 | 39.9 | 1.0 | 0.040 | -0.3 | 40.1 | 1.0 | 0.047 | -7.1 | 40.3 | 1.0 | 0.041 |
| 2.9 | 39.8 | 1.0 | 0.040 | -1.9 | 40.0 | 1.0 | 0.037 | -0.2 | 40.1 | 1.0 | 0.047 | -7.0 | 40.3 | 1.0 | 0.040 |
| 3.0 | 39.8 | 1.0 | 0.041 | -1.8 | 40.0 | 1.0 | 0.039 | -0.1 | 40.1 | 1.0 | 0.047 | -1.9 | 40.3 | 1.0 | 0.039 |
| 3.1 | 39.8 | 1.0 | 0.040 | -1.7 | 40.0 | 1.0 | 0.041 | 0.0 | 40.1 | 1.0 | 0.046 | -1.8 | 40.3 | 1.0 | 0.040 |
| 3.2 | 39.8 | 1.0 | 0.040 | -1.6 | 40.0 | 1.0 | 0.043 | 0.1 | 40.1 | 1.0 | 0.045 | -1.7 | 40.3 | 1.0 | 0.042 |
| 3.3 | 39.8 | 1.0 | 0.040 | -1.5 | 40.0 | 1.0 | 0.046 | 0.2 | 40.1 | 1.0 | 0.044 | -1.6 | 40.3 | 1.0 | 0.044 |
| 3.4 | 39.8 | 1.0 | 0.040 | -1.4 | 40.0 | 1.0 | 0.048 | 0.3 | 40.1 | 1.0 | 0.043 | -1.5 | 40.3 | 1.0 | 0.045 |
| 3.5 | 39.8 | 1.0 | 0.040 | -1.3 | 40.0 | 1.0 | 0.051 | 0.4 | 40.1 | 1.0 | 0.042 | -1.4 | 40.3 | 1.0 | 0.046 |
| 3.7 | 39.8 | 1.0 | 0.040 | -1.2 | 40.0 | 1.0 | 0.051 | 0.5 | 40.1 | 1.0 | 0.040 | -1.3 | 40.3 | 1.0 | 0.047 |
| 3.9 | 39.8 | 1.0 | 0.040 | -1.1 | 40.0 | 1.0 | 0.050 | 0.6 | 40.1 | 1.0 | 0.039 | -1.2 | 40.3 | 1.0 | 0.049 |
| 4.1 | 39.8 | 1.0 | 0.040 | -1.0 | 40.0 | 1.0 | 0.049 | 0.7 | 40.1 | 1.0 | 0.036 | -1.1 | 40.3 | 1.0 | 0.049 |
| 4.2 | 39.8 | 1.0 | 0.040 | -0.9 | 40.0 | 1.0 | 0.049 | 0.8 | 40.1 | 1.0 | 0.033 | -1.0 | 40.3 | 1.0 | 0.047 |
| 4.3 | 39.8 | 1.0 | 0.040 | -0.8 | 40.0 | 1.0 | 0.049 | 2.9 | 40.1 | 1.0 | 0.040 | -0.9 | 40.3 | 1.0 | 0.045 |
| 4.4 | 39.8 | 1.0 | 0.040 | -0.7 | 40.0 | 1.0 | 0.050 | 3.0 | 40.1 | 1.0 | 0.040 | -0.8 | 40.3 | 1.0 | 0.044 |
| -7.0 | 39.9 | 1.1 | 0.040 | -0.6 | 40.0 | 1.0 | 0.050 | 3.1 | 40.1 | 1.0 | 0.040 | -0.7 | 40.3 | 1.0 | 0.043 |
| -6.9 | 39.9 | 1.1 | 0.038 | -0.5 | 40.0 | 1.0 | 0.051 | 3.2 | 40.1 | 1.0 | 0.040 | -0.6 | 40.3 | 1.0 | 0.042 |
| -1.8 | 39.9 | 1.0 | 0.037 | -0.4 | 40.0 | 1.0 | 0.051 | 3.3 | 40.1 | 1.0 | 0.040 | -0.5 | 40.3 | 1.0 | 0.042 |
| -1.7 | 39.9 | 1.0 | 0.040 | -0.3 | 40.0 | 1.0 | 0.051 | 3.7 | 40.1 | 1.0 | 0.040 | -0.4 | 40.3 | 1.0 | 0.042 |
| -1.6 | 39.9 | 1.0 | 0.042 | -0.2 | 40.0 | 1.0 | 0.050 | 3.8 | 40.1 | 1.0 | 0.040 | -0.3 | 40.3 | 1.0 | 0.042 |
| -1.5 | 39.9 | 1.0 | 0.045 | -0.1 | 40.0 | 1.0 | 0.049 | 3.9 | 40.1 | 1.0 | 0.040 | -0.2 | 40.3 | 1.0 | 0.043 |
| -1.4 | 39.9 | 1.0 | 0.047 | 0.0 | 40.0 | 1.0 | 0.049 | 4.0 | 40.1 | 1.0 | 0.040 | -0.1 | 40.3 | 1.0 | 0.043 |
| -1.3 | 39.9 | 1.0 | 0.050 | 0.1 | 40.0 | 1.0 | 0.048 | 4.1 | 40.1 | 1.0 | 0.040 | 0.0 | 40.3 | 1.0 | 0.043 |
| -1.2 | 39.9 | 1.0 | 0.051 | 0.2 | 40.0 | 1.0 | 0.046 | 4.2 | 40.1 | 1.0 | 0.040 | 0.1 | 40.3 | 1.0 | 0.043 |
| -1.1 | 39.9 | 1.0 | 0.052 | 0.3 | 40.0 | 1.0 | 0.045 | 4.3 | 40.1 | 1.0 | 0.040 | 0.2 | 40.3 | 1.0 | 0.042 |
| -1.0 | 39.9 | 1.0 | 0.053 | 0.4 | 40.0 | 1.0 | 0.043 | 4.4 | 40.1 | 1.0 | 0.040 | 0.3 | 40.3 | 1.0 | 0.042 |
| -0.9 | 39.9 | 1.0 | 0.053 | 0.5 | 40.0 | 1.0 | 0.040 | -7.0 | 40.2 | 1.0 | 0.039 | 0.4 | 40.3 | 1.0 | 0.042 |
| -0.8 | 39.9 | 1.0 | 0.054 | 0.6 | 40.0 | 1.0 | 0.038 | -1.9 | 40.2 | 1.0 | 0.038 | 0.5 | 40.3 | 1.0 | 0.041 |
| -0.7 | 39.9 | 1.0 | 0.054 | 2.6 | 40.0 | 1.0 | 0.040 | -1.8 | 40.2 | 1.0 | 0.040 | 0.6 | 40.3 | 1.0 | 0.040 |
| -0.6 | 39.9 | 1.0 | 0.055 | 2.7 | 40.0 | 1.0 | 0.040 | -1.7 | 40.2 | 1.0 | 0.042 | 0.7 | 40.3 | 1.0 | 0.039 |

| Long. | Lat. | K | $a_{gR}$ | Long. | Lat. | K | $a_{gR}$ | Long. | Lat. | K | $a_{gR}$ | Long. | Lat. | K | $a_{gR}$ |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 0.8 | 40.3 | 1.0 | 0.038 | 0.4 | 40.5 | 1.0 | 0.044 | 0.1 | 40.7 | 1.0 | 0.041 | -1.3 | 40.9 | 1.0 | 0.039 |
| 0.9 | 40.3 | 1.0 | 0.036 | 0.5 | 40.5 | 1.0 | 0.045 | 0.2 | 40.7 | 1.0 | 0.042 | -1.2 | 40.9 | 1.0 | 0.039 |
| 1.1 | 40.3 | 1.0 | 0.030 | 0.6 | 40.5 | 1.0 | 0.045 | 0.3 | 40.7 | 1.0 | 0.043 | -1.1 | 40.9 | 1.0 | 0.039 |
| -2.0 | 40.4 | 1.0 | 0.038 | 0.7 | 40.5 | 1.0 | 0.045 | 0.4 | 40.7 | 1.0 | 0.044 | -1.0 | 40.9 | 1.0 | 0.038 |
| -1.9 | 40.4 | 1.0 | 0.039 | 0.8 | 40.5 | 1.0 | 0.044 | 0.5 | 40.7 | 1.0 | 0.047 | -0.9 | 40.9 | 1.0 | 0.037 |
| -1.8 | 40.4 | 1.0 | 0.041 | 0.9 | 40.5 | 1.0 | 0.043 | 0.6 | 40.7 | 1.0 | 0.046 | 0.2 | 40.9 | 1.0 | 0.037 |
| -1.7 | 40.4 | 1.0 | 0.042 | 1.0 | 40.5 | 1.0 | 0.041 | 0.7 | 40.7 | 1.0 | 0.048 | 0.3 | 40.9 | 1.0 | 0.040 |
| -1.6 | 40.4 | 1.0 | 0.043 | 1.1 | 40.5 | 1.0 | 0.039 | 0.8 | 40.7 | 1.0 | 0.046 | 0.4 | 40.9 | 1.0 | 0.043 |
| -1.5 | 40.4 | 1.0 | 0.044 | 1.2 | 40.5 | 1.0 | 0.036 | 0.9 | 40.7 | 1.0 | 0.046 | 0.5 | 40.9 | 1.0 | 0.047 |
| -1.4 | 40.4 | 1.0 | 0.045 | 1.3 | 40.5 | 1.0 | 0.033 | 1.0 | 40.7 | 1.0 | 0.048 | 0.6 | 40.9 | 1.0 | 0.048 |
| -1.3 | 40.4 | 1.0 | 0.046 | 1.4 | 40.5 | 1.0 | 0.031 | 1.1 | 40.7 | 1.0 | 0.047 | 0.7 | 40.9 | 1.0 | 0.050 |
| -1.2 | 40.4 | 1.0 | 0.047 | -6.8 | 40.6 | 1.0 | 0.038 | 1.2 | 40.7 | 1.0 | 0.046 | 0.8 | 40.9 | 1.0 | 0.051 |
| -1.1 | 40.4 | 1.0 | 0.048 | -6.7 | 40.6 | 1.0 | 0.035 | 1.3 | 40.7 | 1.0 | 0.045 | 0.9 | 40.9 | 1.0 | 0.053 |
| -1.0 | 40.4 | 1.0 | 0.046 | -2.0 | 40.6 | 1.0 | 0.038 | 1.4 | 40.7 | 1.0 | 0.044 | 1.0 | 40.9 | 1.0 | 0.052 |
| -0.9 | 40.4 | 1.0 | 0.044 | -1.9 | 40.6 | 1.0 | 0.040 | 1.8 | 40.7 | 1.0 | 0.036 | 1.1 | 40.9 | 1.0 | 0.052 |
| -0.8 | 40.4 | 1.0 | 0.043 | -1.8 | 40.6 | 1.0 | 0.040 | -6.8 | 40.8 | 1.0 | 0.042 | 1.2 | 40.9 | 1.0 | 0.053 |
| -0.7 | 40.4 | 1.0 | 0.042 | -1.7 | 40.6 | 1.0 | 0.041 | -6.7 | 40.8 | 1.0 | 0.040 | 1.3 | 40.9 | 1.0 | 0.054 |
| -0.6 | 40.4 | 1.0 | 0.041 | -1.6 | 40.6 | 1.0 | 0.041 | -6.6 | 40.8 | 1.0 | 0.038 | 1.4 | 40.9 | 1.0 | 0.055 |
| -0.5 | 40.4 | 1.0 | 0.040 | -1.5 | 40.6 | 1.0 | 0.042 | -2.0 | 40.8 | 1.0 | 0.036 | 1.5 | 40.9 | 1.0 | 0.056 |
| -0.4 | 40.4 | 1.0 | 0.041 | -1.4 | 40.6 | 1.0 | 0.044 | -1.9 | 40.8 | 1.0 | 0.037 | 1.6 | 40.9 | 1.0 | 0.057 |
| -0.3 | 40.4 | 1.0 | 0.041 | -1.3 | 40.6 | 1.0 | 0.045 | -1.8 | 40.8 | 1.0 | 0.040 | 1.7 | 40.9 | 1.0 | 0.055 |
| -0.2 | 40.4 | 1.0 | 0.041 | -1.2 | 40.6 | 1.0 | 0.046 | -1.7 | 40.8 | 1.0 | 0.040 | 1.8 | 40.9 | 1.0 | 0.052 |
| -0.1 | 40.4 | 1.0 | 0.042 | -1.1 | 40.6 | 1.0 | 0.047 | -1.6 | 40.8 | 1.0 | 0.041 | 1.9 | 40.9 | 1.0 | 0.050 |
| 0.0 | 40.4 | 1.0 | 0.042 | -1.0 | 40.6 | 1.0 | 0.046 | -1.5 | 40.8 | 1.0 | 0.041 | 2.0 | 40.9 | 1.0 | 0.048 |
| 0.1 | 40.4 | 1.0 | 0.042 | -0.9 | 40.6 | 1.0 | 0.043 | -1.4 | 40.8 | 1.0 | 0.041 | 2.1 | 40.9 | 1.0 | 0.047 |
| 0.2 | 40.4 | 1.0 | 0.042 | -0.8 | 40.6 | 1.0 | 0.041 | -1.3 | 40.8 | 1.0 | 0.042 | 2.2 | 40.9 | 1.0 | 0.045 |
| 0.3 | 40.4 | 1.0 | 0.042 | -0.7 | 40.6 | 1.0 | 0.040 | -1.2 | 40.8 | 1.0 | 0.042 | 2.3 | 40.9 | 1.0 | 0.043 |
| 0.4 | 40.4 | 1.0 | 0.043 | -0.6 | 40.6 | 1.0 | 0.038 | -1.1 | 40.8 | 1.0 | 0.042 | 2.4 | 40.9 | 1.0 | 0.042 |
| 0.5 | 40.4 | 1.0 | 0.043 | -0.2 | 40.6 | 1.0 | 0.038 | -1.0 | 40.8 | 1.0 | 0.041 | 2.6 | 40.9 | 1.0 | 0.040 |
| 0.6 | 40.4 | 1.0 | 0.042 | -0.1 | 40.6 | 1.0 | 0.039 | -0.9 | 40.8 | 1.0 | 0.039 | -6.9 | 41.0 | 1.0 | 0.046 |
| 0.7 | 40.4 | 1.0 | 0.042 | 0.0 | 40.6 | 1.0 | 0.040 | -0.8 | 40.8 | 1.0 | 0.038 | -6.8 | 41.0 | 1.0 | 0.045 |
| 0.8 | 40.4 | 1.0 | 0.041 | 0.1 | 40.6 | 1.0 | 0.042 | 0.0 | 40.8 | 1.0 | 0.035 | -6.7 | 41.0 | 1.0 | 0.043 |
| 0.9 | 40.4 | 1.0 | 0.039 | 0.2 | 40.6 | 1.0 | 0.043 | 0.1 | 40.8 | 1.0 | 0.037 | -6.6 | 41.0 | 1.0 | 0.041 |
| 1.0 | 40.4 | 1.0 | 0.037 | 0.3 | 40.6 | 1.0 | 0.044 | 0.2 | 40.8 | 1.0 | 0.040 | -6.5 | 41.0 | 1.0 | 0.039 |
| 1.1 | 40.4 | 1.0 | 0.034 | 0.4 | 40.6 | 1.0 | 0.045 | 0.3 | 40.8 | 1.0 | 0.042 | -6.4 | 41.0 | 1.0 | 0.037 |
| 1.3 | 40.4 | 1.0 | 0.029 | 0.5 | 40.6 | 1.0 | 0.046 | 0.4 | 40.8 | 1.0 | 0.044 | -1.6 | 41.0 | 1.0 | 0.037 |
| -6.9 | 40.5 | 1.0 | 0.040 | 0.6 | 40.6 | 1.0 | 0.046 | 0.5 | 40.8 | 1.0 | 0.046 | -1.5 | 41.0 | 1.0 | 0.037 |
| -6.8 | 40.5 | 1.0 | 0.037 | 0.7 | 40.6 | 1.0 | 0.046 | 0.6 | 40.8 | 1.0 | 0.048 | -1.4 | 41.0 | 1.0 | 0.039 |
| -2.0 | 40.5 | 1.0 | 0.038 | 0.8 | 40.6 | 1.0 | 0.046 | 0.7 | 40.8 | 1.0 | 0.049 | -1.3 | 41.0 | 1.0 | 0.039 |
| -1.9 | 40.5 | 1.0 | 0.040 | 0.9 | 40.6 | 1.0 | 0.046 | 0.8 | 40.8 | 1.0 | 0.049 | 0.2 | 41.0 | 1.0 | 0.033 |
| -1.8 | 40.5 | 1.0 | 0.041 | 1.0 | 40.6 | 1.0 | 0.045 | 0.9 | 40.8 | 1.0 | 0.050 | 0.3 | 41.0 | 1.0 | 0.036 |
| -1.7 | 40.5 | 1.0 | 0.042 | 1.1 | 40.6 | 1.0 | 0.044 | 1.0 | 40.8 | 1.0 | 0.050 | 0.4 | 41.0 | 1.0 | 0.040 |
| -1.6 | 40.5 | 1.0 | 0.043 | 1.2 | 40.6 | 1.0 | 0.042 | 1.1 | 40.8 | 1.0 | 0.050 | 0.5 | 41.0 | 1.0 | 0.044 |
| -1.5 | 40.5 | 1.0 | 0.044 | 1.3 | 40.6 | 1.0 | 0.039 | 1.2 | 40.8 | 1.0 | 0.051 | 0.6 | 41.0 | 1.0 | 0.047 |
| -1.4 | 40.5 | 1.0 | 0.044 | -6.8 | 40.7 | 1.0 | 0.040 | 1.3 | 40.8 | 1.0 | 0.050 | 0.7 | 41.0 | 1.0 | 0.051 |
| -1.3 | 40.5 | 1.0 | 0.046 | -6.7 | 40.7 | 1.0 | 0.038 | 1.4 | 40.8 | 1.0 | 0.049 | 0.8 | 41.0 | 1.0 | 0.053 |
| -1.2 | 40.5 | 1.0 | 0.047 | -2.0 | 40.7 | 1.0 | 0.038 | 1.5 | 40.8 | 1.0 | 0.050 | 0.9 | 41.0 | 1.0 | 0.054 |
| -1.1 | 40.5 | 1.0 | 0.048 | -1.9 | 40.7 | 1.0 | 0.040 | 1.6 | 40.8 | 1.0 | 0.049 | 1.0 | 41.0 | 1.0 | 0.055 |
| -1.0 | 40.5 | 1.0 | 0.047 | -1.8 | 40.7 | 1.0 | 0.040 | 1.7 | 40.8 | 1.0 | 0.047 | 1.1 | 41.0 | 1.0 | 0.056 |
| -0.9 | 40.5 | 1.0 | 0.044 | -1.7 | 40.7 | 1.0 | 0.041 | 1.8 | 40.8 | 1.0 | 0.044 | 1.2 | 41.0 | 1.0 | 0.056 |
| -0.8 | 40.5 | 1.0 | 0.042 | -1.6 | 40.7 | 1.0 | 0.041 | 2.3 | 40.8 | 1.0 | 0.037 | 1.3 | 41.0 | 1.0 | 0.057 |
| -0.7 | 40.5 | 1.0 | 0.040 | -1.5 | 40.7 | 1.0 | 0.042 | -7.0 | 40.9 | 1.0 | 0.047 | 1.4 | 41.0 | 1.0 | 0.058 |
| -0.6 | 40.5 | 1.0 | 0.040 | -1.4 | 40.7 | 1.0 | 0.042 | -6.9 | 40.9 | 1.0 | 0.045 | 1.5 | 41.0 | 1.0 | 0.059 |
| -0.5 | 40.5 | 1.0 | 0.039 | -1.3 | 40.7 | 1.0 | 0.042 | -6.8 | 40.9 | 1.0 | 0.043 | 1.6 | 41.0 | 1.0 | 0.060 |
| -0.4 | 40.5 | 1.0 | 0.039 | -1.2 | 40.7 | 1.0 | 0.045 | -6.7 | 40.9 | 1.0 | 0.042 | 1.7 | 41.0 | 1.0 | 0.059 |
| -0.3 | 40.5 | 1.0 | 0.039 | -1.1 | 40.7 | 1.0 | 0.045 | -6.6 | 40.9 | 1.0 | 0.039 | 1.8 | 41.0 | 1.0 | 0.058 |
| -0.2 | 40.5 | 1.0 | 0.040 | -1.0 | 40.7 | 1.0 | 0.044 | -6.5 | 40.9 | 1.0 | 0.038 | 1.9 | 41.0 | 1.0 | 0.058 |
| -0.1 | 40.5 | 1.0 | 0.041 | -0.9 | 40.7 | 1.0 | 0.041 | -1.8 | 40.9 | 1.0 | 0.037 | 2.0 | 41.0 | 1.0 | 0.057 |
| 0.0 | 40.5 | 1.0 | 0.041 | -0.8 | 40.7 | 1.0 | 0.039 | -1.7 | 40.9 | 1.0 | 0.037 | 2.1 | 41.0 | 1.0 | 0.057 |
| 0.1 | 40.5 | 1.0 | 0.042 | -0.7 | 40.7 | 1.0 | 0.038 | -1.6 | 40.9 | 1.0 | 0.040 | 2.2 | 41.0 | 1.0 | 0.055 |
| 0.2 | 40.5 | 1.0 | 0.042 | -0.1 | 40.7 | 1.0 | 0.038 | -1.5 | 40.9 | 1.0 | 0.040 | 2.3 | 41.0 | 1.0 | 0.053 |
| 0.3 | 40.5 | 1.0 | 0.043 | 0.0 | 40.7 | 1.0 | 0.039 | -1.4 | 40.9 | 1.0 | 0.040 | 2.4 | 41.0 | 1.0 | 0.051 |

| Long. | Lat. | K | $a_{gR}$ | Long. | Lat. | K | $a_{gR}$ | Long. | Lat. | K | $a_{gR}$ | Long. | Lat. | K | $a_{gR}$ |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 2.5 | 41.0 | 1.0 | 0.049 | 1.9 | 41.2 | 1.0 | 0.067 | -6.0 | 41.4 | 1.0 | 0.038 | -1.9 | 41.5 | 1.0 | 0.042 |
| 2.6 | 41.0 | 1.0 | 0.047 | 2.0 | 41.2 | 1.0 | 0.069 | -5.8 | 41.4 | 1.0 | 0.035 | -1.8 | 41.5 | 1.0 | 0.041 |
| 2.8 | 41.0 | 1.0 | 0.044 | 2.1 | 41.2 | 1.0 | 0.071 | -2.8 | 41.4 | 1.0 | 0.034 | -1.7 | 41.5 | 1.0 | 0.041 |
| -7.0 | 41.1 | 1.0 | 0.049 | 2.2 | 41.2 | 1.0 | 0.074 | -2.5 | 41.4 | 1.0 | 0.039 | -1.6 | 41.5 | 1.0 | 0.040 |
| -6.9 | 41.1 | 1.0 | 0.046 | 2.3 | 41.2 | 1.0 | 0.076 | -2.4 | 41.4 | 1.0 | 0.040 | -1.5 | 41.5 | 1.0 | 0.042 |
| -6.8 | 41.1 | 1.0 | 0.045 | 2.4 | 41.2 | 1.0 | 0.076 | -2.3 | 41.4 | 1.0 | 0.041 | 0.7 | 41.5 | 1.0 | 0.035 |
| -6.7 | 41.1 | 1.0 | 0.044 | 2.5 | 41.2 | 1.0 | 0.076 | -2.2 | 41.4 | 1.0 | 0.041 | 0.8 | 41.5 | 1.0 | 0.037 |
| -6.6 | 41.1 | 1.0 | 0.042 | 2.6 | 41.2 | 1.0 | 0.074 | -2.1 | 41.4 | 1.0 | 0.040 | 0.9 | 41.5 | 1.0 | 0.040 |
| -6.5 | 41.1 | 1.0 | 0.041 | 2.7 | 41.2 | 1.0 | 0.070 | -2.0 | 41.4 | 1.0 | 0.041 | 1.0 | 41.5 | 1.0 | 0.043 |
| -6.4 | 41.1 | 1.0 | 0.039 | 2.8 | 41.2 | 1.0 | 0.066 | -1.9 | 41.4 | 1.0 | 0.040 | 1.1 | 41.5 | 1.0 | 0.046 |
| -6.3 | 41.1 | 1.0 | 0.037 | 2.9 | 41.2 | 1.0 | 0.061 | -1.8 | 41.4 | 1.0 | 0.041 | 1.2 | 41.5 | 1.0 | 0.049 |
| 0.3 | 41.1 | 1.0 | 0.032 | 3.0 | 41.2 | 1.0 | 0.059 | -1.7 | 41.4 | 1.0 | 0.041 | 1.3 | 41.5 | 1.0 | 0.052 |
| 0.4 | 41.1 | 1.0 | 0.036 | 3.3 | 41.2 | 1.0 | 0.048 | -1.6 | 41.4 | 1.0 | 0.040 | 1.4 | 41.5 | 1.0 | 0.055 |
| 0.5 | 41.1 | 1.0 | 0.040 | -6.8 | 41.3 | 1.0 | 0.048 | -1.5 | 41.4 | 1.0 | 0.039 | 1.5 | 41.5 | 1.0 | 0.059 |
| 0.6 | 41.1 | 1.0 | 0.045 | -6.7 | 41.3 | 1.0 | 0.047 | -1.4 | 41.4 | 1.0 | 0.039 | 1.6 | 41.5 | 1.0 | 0.063 |
| 0.7 | 41.1 | 1.0 | 0.049 | -6.6 | 41.3 | 1.0 | 0.045 | 0.6 | 41.4 | 1.0 | 0.034 | 1.7 | 41.5 | 1.0 | 0.067 |
| 0.8 | 41.1 | 1.0 | 0.052 | -6.5 | 41.3 | 1.0 | 0.044 | 0.7 | 41.4 | 1.0 | 0.037 | 1.8 | 41.5 | 1.0 | 0.072 |
| 0.9 | 41.1 | 1.0 | 0.054 | -6.4 | 41.3 | 1.0 | 0.042 | 0.8 | 41.4 | 1.0 | 0.040 | 1.9 | 41.5 | 1.0 | 0.078 |
| 1.0 | 41.1 | 1.0 | 0.056 | -6.3 | 41.3 | 1.0 | 0.041 | 0.9 | 41.4 | 1.0 | 0.044 | 2.0 | 41.5 | 1.0 | 0.084 |
| 1.1 | 41.1 | 1.0 | 0.057 | -6.2 | 41.3 | 1.0 | 0.039 | 1.0 | 41.4 | 1.0 | 0.048 | 2.1 | 41.5 | 1.0 | 0.089 |
| 1.2 | 41.1 | 1.0 | 0.058 | -6.1 | 41.3 | 1.0 | 0.038 | 1.1 | 41.4 | 1.0 | 0.051 | 2.2 | 41.5 | 1.0 | 0.092 |
| 1.3 | 41.1 | 1.0 | 0.059 | -6.0 | 41.3 | 1.0 | 0.037 | 1.2 | 41.4 | 1.0 | 0.054 | 2.3 | 41.5 | 1.0 | 0.094 |
| 1.4 | 41.1 | 1.0 | 0.060 | -2.5 | 41.3 | 1.0 | 0.035 | 1.3 | 41.4 | 1.0 | 0.057 | 2.4 | 41.5 | 1.0 | 0.096 |
| 1.5 | 41.1 | 1.0 | 0.061 | -1.8 | 41.3 | 1.0 | 0.040 | 1.4 | 41.4 | 1.0 | 0.059 | 2.5 | 41.5 | 1.0 | 0.098 |
| 1.6 | 41.1 | 1.0 | 0.061 | -1.7 | 41.3 | 1.0 | 0.041 | 1.5 | 41.4 | 1.0 | 0.062 | 2.6 | 41.5 | 1.0 | 0.100 |
| 1.7 | 41.1 | 1.0 | 0.062 | -1.6 | 41.3 | 1.0 | 0.040 | 1.6 | 41.4 | 1.0 | 0.065 | 2.7 | 41.5 | 1.0 | 0.100 |
| 1.8 | 41.1 | 1.0 | 0.062 | -1.5 | 41.3 | 1.0 | 0.040 | 1.7 | 41.4 | 1.0 | 0.068 | 2.8 | 41.5 | 1.0 | 0.101 |
| 1.9 | 41.1 | 1.0 | 0.063 | 0.5 | 41.3 | 1.0 | 0.033 | 1.8 | 41.4 | 1.0 | 0.070 | 2.9 | 41.5 | 1.0 | 0.102 |
| 2.0 | 41.1 | 1.0 | 0.063 | 0.6 | 41.3 | 1.0 | 0.036 | 1.9 | 41.4 | 1.0 | 0.074 | 3.0 | 41.5 | 1.0 | 0.100 |
| 2.1 | 41.1 | 1.0 | 0.064 | 0.7 | 41.3 | 1.0 | 0.040 | 2.0 | 41.4 | 1.0 | 0.080 | 3.1 | 41.5 | 1.0 | 0.096 |
| 2.2 | 41.1 | 1.0 | 0.065 | 0.8 | 41.3 | 1.0 | 0.045 | 2.1 | 41.4 | 1.0 | 0.084 | 3.2 | 41.5 | 1.0 | 0.091 |
| 2.3 | 41.1 | 1.0 | 0.065 | 0.9 | 41.3 | 1.0 | 0.049 | 2.2 | 41.4 | 1.0 | 0.087 | 3.3 | 41.5 | 1.0 | 0.085 |
| 2.4 | 41.1 | 1.0 | 0.065 | 1.0 | 41.3 | 1.0 | 0.053 | 2.3 | 41.4 | 1.0 | 0.089 | 3.4 | 41.5 | 1.0 | 0.080 |
| 2.5 | 41.1 | 1.0 | 0.063 | 1.1 | 41.3 | 1.0 | 0.056 | 2.4 | 41.4 | 1.0 | 0.090 | 3.5 | 41.5 | 1.0 | 0.075 |
| 2.6 | 41.1 | 1.0 | 0.060 | 1.2 | 41.3 | 1.0 | 0.058 | 2.5 | 41.4 | 1.0 | 0.091 | 3.6 | 41.5 | 1.0 | 0.070 |
| 2.7 | 41.1 | 1.0 | 0.056 | 1.3 | 41.3 | 1.0 | 0.059 | 2.6 | 41.4 | 1.0 | 0.091 | 3.7 | 41.5 | 1.0 | 0.064 |
| 2.8 | 41.1 | 1.0 | 0.053 | 1.4 | 41.3 | 1.0 | 0.061 | 2.7 | 41.4 | 1.0 | 0.091 | 3.8 | 41.5 | 1.0 | 0.057 |
| 3.0 | 41.1 | 1.0 | 0.048 | 1.5 | 41.3 | 1.0 | 0.063 | 2.8 | 41.4 | 1.0 | 0.090 | -6.6 | 41.6 | 1.0 | 0.045 |
| -6.7 | 41.2 | 1.0 | 0.045 | 1.6 | 41.3 | 1.0 | 0.065 | 2.9 | 41.4 | 1.0 | 0.089 | -6.4 | 41.6 | 1.0 | 0.044 |
| -6.6 | 41.2 | 1.0 | 0.044 | 1.7 | 41.3 | 1.0 | 0.067 | 3.0 | 41.4 | 1.0 | 0.088 | -6.3 | 41.6 | 1.0 | 0.044 |
| -6.5 | 41.2 | 1.0 | 0.043 | 1.8 | 41.3 | 1.0 | 0.068 | 3.1 | 41.4 | 1.0 | 0.085 | -6.2 | 41.6 | 1.0 | 0.043 |
| -6.4 | 41.2 | 1.0 | 0.041 | 1.9 | 41.3 | 1.0 | 0.071 | 3.2 | 41.4 | 1.0 | 0.081 | -6.1 | 41.6 | 1.0 | 0.043 |
| -6.3 | 41.2 | 1.0 | 0.039 | 2.0 | 41.3 | 1.0 | 0.074 | 3.3 | 41.4 | 1.0 | 0.075 | -6.0 | 41.6 | 1.0 | 0.043 |
| -6.2 | 41.2 | 1.0 | 0.037 | 2.1 | 41.3 | 1.0 | 0.079 | 3.4 | 41.4 | 1.0 | 0.070 | -5.9 | 41.6 | 1.0 | 0.041 |
| -1.8 | 41.2 | 1.0 | 0.039 | 2.2 | 41.3 | 1.0 | 0.082 | 3.5 | 41.4 | 1.0 | 0.063 | -5.8 | 41.6 | 1.0 | 0.038 |
| -1.7 | 41.2 | 1.0 | 0.039 | 2.3 | 41.3 | 1.0 | 0.084 | 3.7 | 41.4 | 1.0 | 0.050 | -3.2 | 41.6 | 1.0 | 0.037 |
| -1.6 | 41.2 | 1.0 | 0.038 | 2.4 | 41.3 | 1.0 | 0.084 | -6.3 | 41.5 | 1.0 | 0.043 | -3.1 | 41.6 | 1.0 | 0.040 |
| 0.4 | 41.2 | 1.0 | 0.033 | 2.5 | 41.3 | 1.0 | 0.085 | -6.2 | 41.5 | 1.0 | 0.043 | -3.0 | 41.6 | 1.0 | 0.041 |
| 0.5 | 41.2 | 1.0 | 0.036 | 2.6 | 41.3 | 1.0 | 0.084 | -6.1 | 41.5 | 1.0 | 0.043 | -2.9 | 41.6 | 1.0 | 0.042 |
| 0.6 | 41.2 | 1.0 | 0.040 | 2.7 | 41.3 | 1.0 | 0.082 | -6.0 | 41.5 | 1.0 | 0.042 | -2.8 | 41.6 | 1.0 | 0.043 |
| 0.7 | 41.2 | 1.0 | 0.045 | 2.8 | 41.3 | 1.0 | 0.079 | -5.9 | 41.5 | 1.0 | 0.040 | -2.7 | 41.6 | 1.0 | 0.044 |
| 0.8 | 41.2 | 1.0 | 0.049 | 2.9 | 41.3 | 1.0 | 0.075 | -5.8 | 41.5 | 1.0 | 0.037 | -2.6 | 41.6 | 1.0 | 0.045 |
| 0.9 | 41.2 | 1.0 | 0.053 | 3.0 | 41.3 | 1.0 | 0.072 | -3.1 | 41.5 | 1.0 | 0.035 | -2.5 | 41.6 | 1.0 | 0.045 |
| 1.0 | 41.2 | 1.0 | 0.056 | 3.1 | 41.3 | 1.0 | 0.071 | -2.8 | 41.5 | 1.0 | 0.040 | -2.4 | 41.6 | 1.0 | 0.045 |
| 1.1 | 41.2 | 1.0 | 0.058 | 3.2 | 41.3 | 1.0 | 0.066 | -2.7 | 41.5 | 1.0 | 0.041 | -2.3 | 41.6 | 1.0 | 0.046 |
| 1.2 | 41.2 | 1.0 | 0.059 | 3.3 | 41.3 | 1.0 | 0.061 | -2.6 | 41.5 | 1.0 | 0.042 | -2.2 | 41.6 | 1.0 | 0.046 |
| 1.3 | 41.2 | 1.0 | 0.060 | 3.5 | 41.3 | 1.0 | 0.049 | -2.5 | 41.5 | 1.0 | 0.043 | -2.1 | 41.6 | 1.0 | 0.047 |
| 1.4 | 41.2 | 1.0 | 0.061 | -6.5 | 41.4 | 1.0 | 0.044 | -2.4 | 41.5 | 1.0 | 0.043 | -2.0 | 41.6 | 1.0 | 0.047 |
| 1.5 | 41.2 | 1.0 | 0.062 | -6.4 | 41.4 | 1.0 | 0.043 | -2.3 | 41.5 | 1.0 | 0.043 | -1.9 | 41.6 | 1.0 | 0.046 |
| 1.6 | 41.2 | 1.0 | 0.063 | -6.3 | 41.4 | 1.0 | 0.042 | -2.2 | 41.5 | 1.0 | 0.044 | -1.8 | 41.6 | 1.0 | 0.043 |
| 1.7 | 41.2 | 1.0 | 0.064 | -6.2 | 41.4 | 1.0 | 0.041 | -2.1 | 41.5 | 1.0 | 0.043 | -1.7 | 41.6 | 1.0 | 0.044 |
| 1.8 | 41.2 | 1.0 | 0.065 | -6.1 | 41.4 | 1.0 | 0.040 | -2.0 | 41.5 | 1.0 | 0.043 | -1.6 | 41.6 | 1.0 | 0.043 |

| Long. | Lat. | K | $a_{gR}$ | Long. | Lat. | K | $a_{gR}$ | Long. | Lat. | K | $a_{gR}$ | Long. | Lat. | K | $a_{gR}$ |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| -1.5 | 41.6 | 1.0 | 0.041 | 1.5 | 41.7 | 1.0 | 0.052 | 1.0 | 41.8 | 1.0 | 0.041 | -2.5 | 41.9 | 1.0 | 0.051 |
| 0.8 | 41.6 | 1.0 | 0.036 | 1.6 | 41.7 | 1.0 | 0.055 | 1.1 | 41.8 | 1.0 | 0.042 | -2.4 | 41.9 | 1.0 | 0.057 |
| 0.9 | 41.6 | 1.0 | 0.038 | 1.7 | 41.7 | 1.0 | 0.060 | 1.2 | 41.8 | 1.0 | 0.044 | -2.3 | 41.9 | 1.0 | 0.057 |
| 1.0 | 41.6 | 1.0 | 0.040 | 1.8 | 41.7 | 1.0 | 0.065 | 1.3 | 41.8 | 1.0 | 0.046 | -2.2 | 41.9 | 1.0 | 0.062 |
| 1.1 | 41.6 | 1.0 | 0.042 | 1.9 | 41.7 | 1.0 | 0.072 | 1.4 | 41.8 | 1.0 | 0.048 | -2.1 | 41.9 | 1.0 | 0.061 |
| 1.2 | 41.6 | 1.0 | 0.045 | 2.0 | 41.7 | 1.0 | 0.080 | 1.5 | 41.8 | 1.0 | 0.051 | -2.0 | 41.9 | 1.0 | 0.059 |
| 1.3 | 41.6 | 1.0 | 0.047 | 2.1 | 41.7 | 1.0 | 0.088 | 1.6 | 41.8 | 1.0 | 0.054 | -1.9 | 41.9 | 1.0 | 0.055 |
| 1.4 | 41.6 | 1.0 | 0.050 | 2.2 | 41.7 | 1.0 | 0.094 | 1.7 | 41.8 | 1.0 | 0.058 | -1.8 | 41.9 | 1.0 | 0.050 |
| 1.5 | 41.6 | 1.0 | 0.054 | 2.3 | 41.7 | 1.0 | 0.099 | 1.8 | 41.8 | 1.0 | 0.062 | -1.7 | 41.9 | 1.0 | 0.046 |
| 1.6 | 41.6 | 1.0 | 0.058 | 2.4 | 41.7 | 1.0 | 0.102 | 1.9 | 41.8 | 1.0 | 0.068 | -1.6 | 41.9 | 1.0 | 0.042 |
| 1.7 | 41.6 | 1.0 | 0.063 | 2.5 | 41.7 | 1.0 | 0.107 | 2.0 | 41.8 | 1.0 | 0.074 | 0.0 | 41.9 | 1.0 | 0.034 |
| 1.8 | 41.6 | 1.0 | 0.069 | 2.6 | 41.7 | 1.0 | 0.111 | 2.1 | 41.8 | 1.0 | 0.082 | 0.3 | 41.9 | 1.0 | 0.037 |
| 1.9 | 41.6 | 1.0 | 0.077 | 2.7 | 41.7 | 1.0 | 0.114 | 2.2 | 41.8 | 1.0 | 0.090 | 0.4 | 41.9 | 1.0 | 0.039 |
| 2.0 | 41.6 | 1.0 | 0.084 | 2.8 | 41.7 | 1.0 | 0.114 | 2.3 | 41.8 | 1.0 | 0.097 | 0.5 | 41.9 | 1.0 | 0.040 |
| 2.1 | 41.6 | 1.0 | 0.090 | 2.9 | 41.7 | 1.0 | 0.114 | 2.4 | 41.8 | 1.0 | 0.104 | 0.6 | 41.9 | 1.0 | 0.041 |
| 2.2 | 41.6 | 1.0 | 0.094 | 3.0 | 41.7 | 1.0 | 0.112 | 2.5 | 41.8 | 1.0 | 0.109 | 0.7 | 41.9 | 1.0 | 0.042 |
| 2.3 | 41.6 | 1.0 | 0.097 | 3.1 | 41.7 | 1.0 | 0.108 | 2.6 | 41.8 | 1.0 | 0.114 | 0.8 | 41.9 | 1.0 | 0.043 |
| 2.4 | 41.6 | 1.0 | 0.100 | 3.2 | 41.7 | 1.0 | 0.104 | 2.7 | 41.8 | 1.0 | 0.117 | 0.9 | 41.9 | 1.0 | 0.043 |
| 2.5 | 41.6 | 1.0 | 0.102 | 3.3 | 41.7 | 1.0 | 0.099 | 2.8 | 41.8 | 1.0 | 0.117 | 1.0 | 41.9 | 1.0 | 0.044 |
| 2.6 | 41.6 | 1.0 | 0.105 | 3.4 | 41.7 | 1.0 | 0.092 | 2.9 | 41.8 | 1.0 | 0.115 | 1.1 | 41.9 | 1.0 | 0.045 |
| 2.7 | 41.6 | 1.0 | 0.107 | 3.5 | 41.7 | 1.0 | 0.086 | 3.0 | 41.8 | 1.0 | 0.112 | 1.2 | 41.9 | 1.0 | 0.047 |
| 2.8 | 41.6 | 1.0 | 0.109 | 3.6 | 41.7 | 1.0 | 0.081 | 3.1 | 41.8 | 1.0 | 0.109 | 1.3 | 41.9 | 1.0 | 0.048 |
| 2.9 | 41.6 | 1.0 | 0.109 | 3.7 | 41.7 | 1.0 | 0.077 | 3.2 | 41.8 | 1.0 | 0.105 | 1.4 | 41.9 | 1.0 | 0.051 |
| 3.0 | 41.6 | 1.0 | 0.108 | 3.8 | 41.7 | 1.0 | 0.072 | 3.3 | 41.8 | 1.0 | 0.101 | 1.5 | 41.9 | 1.0 | 0.054 |
| 3.1 | 41.6 | 1.0 | 0.104 | -9.2 | 41.8 | 1.0 | 0.061 | 3.4 | 41.8 | 1.0 | 0.096 | 1.6 | 41.9 | 1.0 | 0.057 |
| 3.2 | 41.6 | 1.0 | 0.099 | -8.9 | 41.8 | 1.0 | 0.071 | 3.5 | 41.8 | 1.0 | 0.090 | 1.7 | 41.9 | 1.0 | 0.060 |
| 3.3 | 41.6 | 1.0 | 0.091 | -8.7 | 41.8 | 1.0 | 0.073 | 3.6 | 41.8 | 1.0 | 0.084 | 1.8 | 41.9 | 1.0 | 0.064 |
| 3.4 | 41.6 | 1.0 | 0.086 | -8.3 | 41.8 | 1.0 | 0.069 | 3.7 | 41.8 | 1.0 | 0.079 | 1.9 | 41.9 | 1.0 | 0.068 |
| 3.5 | 41.6 | 1.0 | 0.081 | -8.1 | 41.8 | 1.0 | 0.063 | -9.5 | 41.9 | 1.0 | 0.039 | 2.0 | 41.9 | 1.0 | 0.074 |
| 3.6 | 41.6 | 1.0 | 0.077 | -7.8 | 41.8 | 1.0 | 0.054 | -9.2 | 41.9 | 1.0 | 0.064 | 2.1 | 41.9 | 1.0 | 0.081 |
| 3.7 | 41.6 | 1.0 | 0.073 | -7.6 | 41.8 | 1.0 | 0.050 | -9.1 | 41.9 | 1.0 | 0.066 | 2.2 | 41.9 | 1.0 | 0.089 |
| -6.5 | 41.7 | 1.0 | 0.045 | -7.3 | 41.8 | 1.0 | 0.050 | -9.0 | 41.9 | 1.0 | 0.068 | 2.3 | 41.9 | 1.0 | 0.097 |
| -6.4 | 41.7 | 1.0 | 0.044 | -7.1 | 41.8 | 1.0 | 0.047 | -8.9 | 41.9 | 1.0 | 0.070 | 2.4 | 41.9 | 1.0 | 0.105 |
| -6.3 | 41.7 | 1.0 | 0.044 | -6.5 | 41.8 | 1.0 | 0.044 | -8.8 | 41.9 | 1.0 | 0.073 | 2.5 | 41.9 | 1.0 | 0.113 |
| -6.2 | 41.7 | 1.0 | 0.044 | -6.4 | 41.8 | 1.0 | 0.044 | -8.7 | 41.9 | 1.0 | 0.073 | 2.6 | 41.9 | 1.0 | 0.117 |
| -6.1 | 41.7 | 1.0 | 0.044 | -6.3 | 41.8 | 1.0 | 0.043 | -8.6 | 41.9 | 1.0 | 0.073 | 2.7 | 41.9 | 1.0 | 0.119 |
| -6.0 | 41.7 | 1.0 | 0.043 | -6.2 | 41.8 | 1.0 | 0.042 | -8.2 | 41.9 | 1.0 | 0.064 | 2.8 | 41.9 | 1.0 | 0.118 |
| -5.9 | 41.7 | 1.0 | 0.041 | -6.1 | 41.8 | 1.0 | 0.042 | -8.1 | 41.9 | 1.0 | 0.062 | 2.9 | 41.9 | 1.0 | 0.115 |
| -5.8 | 41.7 | 1.0 | 0.039 | -6.0 | 41.8 | 1.0 | 0.041 | -8.0 | 41.9 | 1.0 | 0.061 | 3.0 | 41.9 | 1.0 | 0.113 |
| -3.1 | 41.7 | 1.0 | 0.041 | -5.9 | 41.8 | 1.0 | 0.040 | -7.9 | 41.9 | 1.0 | 0.058 | 3.1 | 41.9 | 1.0 | 0.110 |
| -3.0 | 41.7 | 1.0 | 0.042 | -5.8 | 41.8 | 1.0 | 0.038 | -7.8 | 41.9 | 1.0 | 0.055 | 3.2 | 41.9 | 1.0 | 0.106 |
| -2.9 | 41.7 | 1.0 | 0.044 | -3.1 | 41.8 | 1.0 | 0.040 | -7.7 | 41.9 | 1.0 | 0.052 | 3.3 | 41.9 | 1.0 | 0.102 |
| -2.8 | 41.7 | 1.0 | 0.045 | -3.0 | 41.8 | 1.0 | 0.042 | -7.6 | 41.9 | 1.0 | 0.050 | 3.4 | 41.9 | 1.0 | 0.098 |
| -2.7 | 41.7 | 1.0 | 0.045 | -2.9 | 41.8 | 1.0 | 0.043 | -7.5 | 41.9 | 1.0 | 0.049 | 3.5 | 41.9 | 1.0 | 0.093 |
| -2.6 | 41.7 | 1.0 | 0.046 | -2.8 | 41.8 | 1.0 | 0.044 | -7.4 | 41.9 | 1.0 | 0.050 | 3.6 | 41.9 | 1.0 | 0.087 |
| -2.5 | 41.7 | 1.0 | 0.047 | -2.7 | 41.8 | 1.0 | 0.045 | -7.3 | 41.9 | 1.0 | 0.049 | 3.7 | 41.9 | 1.0 | 0.080 |
| -2.4 | 41.7 | 1.0 | 0.048 | -2.6 | 41.8 | 1.0 | 0.047 | -7.2 | 41.9 | 1.0 | 0.048 | 3.8 | 41.9 | 1.0 | 0.073 |
| -2.3 | 41.7 | 1.0 | 0.049 | -2.5 | 41.8 | 1.0 | 0.049 | -7.1 | 41.9 | 1.0 | 0.047 | -9.6 | 42.0 | 1.0 | 0.038 |
| -2.2 | 41.7 | 1.0 | 0.052 | -2.4 | 41.8 | 1.0 | 0.050 | -6.6 | 41.9 | 1.0 | 0.044 | -9.5 | 42.0 | 1.0 | 0.042 |
| -2.1 | 41.7 | 1.0 | 0.051 | -2.3 | 41.8 | 1.0 | 0.054 | -6.5 | 41.9 | 1.0 | 0.043 | -9.4 | 42.0 | 1.0 | 0.049 |
| -2.0 | 41.7 | 1.0 | 0.050 | -2.2 | 41.8 | 1.0 | 0.055 | -6.4 | 41.9 | 1.0 | 0.043 | -9.3 | 42.0 | 1.0 | 0.058 |
| -1.9 | 41.7 | 1.0 | 0.049 | -2.1 | 41.8 | 1.0 | 0.055 | -6.3 | 41.9 | 1.0 | 0.043 | -9.2 | 42.0 | 1.0 | 0.064 |
| -1.8 | 41.7 | 1.0 | 0.047 | -2.0 | 41.8 | 1.0 | 0.053 | -6.2 | 41.9 | 1.0 | 0.042 | -9.1 | 42.0 | 1.0 | 0.067 |
| -1.7 | 41.7 | 1.0 | 0.045 | -1.9 | 41.8 | 1.0 | 0.051 | -6.1 | 41.9 | 1.0 | 0.042 | -9.0 | 42.0 | 1.0 | 0.068 |
| -1.6 | 41.7 | 1.0 | 0.043 | -1.8 | 41.8 | 1.0 | 0.048 | -6.0 | 41.9 | 1.0 | 0.040 | -8.9 | 42.0 | 1.0 | 0.069 |
| -1.5 | 41.7 | 1.0 | 0.041 | -1.7 | 41.8 | 1.0 | 0.045 | -5.9 | 41.9 | 1.0 | 0.039 | -8.8 | 42.0 | 1.0 | 0.070 |
| 0.9 | 41.7 | 1.0 | 0.038 | -1.6 | 41.8 | 1.0 | 0.042 | -3.1 | 41.9 | 1.0 | 0.041 | -8.7 | 42.0 | 1.0 | 0.071 |
| 1.0 | 41.7 | 1.0 | 0.040 | -1.5 | 41.8 | 1.0 | 0.039 | -3.0 | 41.9 | 1.0 | 0.042 | -8.6 | 42.0 | 1.0 | 0.071 |
| 1.1 | 41.7 | 1.0 | 0.042 | -1.4 | 41.8 | 1.0 | 0.036 | -2.9 | 41.9 | 1.0 | 0.044 | -8.3 | 42.0 | 1.0 | 0.066 |
| 1.2 | 41.7 | 1.0 | 0.044 | 0.4 | 41.8 | 1.0 | 0.034 | -2.8 | 41.9 | 1.0 | 0.045 | -8.2 | 42.0 | 1.0 | 0.063 |
| 1.3 | 41.7 | 1.0 | 0.046 | 0.5 | 41.8 | 1.0 | 0.035 | -2.7 | 41.9 | 1.0 | 0.048 | -8.1 | 42.0 | 1.0 | 0.062 |
| 1.4 | 41.7 | 1.0 | 0.048 | 0.9 | 41.8 | 1.0 | 0.040 | -2.6 | 41.9 | 1.0 | 0.049 | -8.0 | 42.0 | 1.0 | 0.060 |

| Long. | Lat. | K | $a_{gR}$ | Long. | Lat. | K | $a_{gR}$ | Long. | Lat. | K | $a_{gR}$ | Long. | Lat. | K | $a_{gR}$ |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| -7.9 | 42.0 | 1.0 | 0.058 | 2.0 | 42.0 | 1.0 | 0.079 | -2.5 | 42.1 | 1.0 | 0.061 | -9.3 | 42.2 | 1.0 | 0.059 |
| -7.8 | 42.0 | 1.0 | 0.056 | 2.1 | 42.0 | 1.0 | 0.085 | -2.4 | 42.1 | 1.0 | 0.064 | -9.2 | 42.2 | 1.0 | 0.064 |
| -7.7 | 42.0 | 1.0 | 0.053 | 2.2 | 42.0 | 1.0 | 0.092 | -2.3 | 42.1 | 1.0 | 0.069 | -9.1 | 42.2 | 1.0 | 0.066 |
| -7.6 | 42.0 | 1.0 | 0.050 | 2.3 | 42.0 | 1.0 | 0.101 | -2.2 | 42.1 | 1.0 | 0.069 | -9.0 | 42.2 | 1.0 | 0.067 |
| -7.5 | 42.0 | 1.0 | 0.049 | 2.4 | 42.0 | 1.0 | 0.111 | -2.1 | 42.1 | 1.0 | 0.066 | -8.9 | 42.2 | 1.0 | 0.068 |
| -7.4 | 42.0 | 1.0 | 0.050 | 2.5 | 42.0 | 1.0 | 0.117 | -2.0 | 42.1 | 1.0 | 0.061 | -8.8 | 42.2 | 1.0 | 0.069 |
| -7.3 | 42.0 | 1.0 | 0.048 | 2.6 | 42.0 | 1.0 | 0.120 | -1.9 | 42.1 | 1.0 | 0.055 | -8.7 | 42.2 | 1.0 | 0.070 |
| -7.2 | 42.0 | 1.0 | 0.047 | 2.7 | 42.0 | 1.0 | 0.119 | -1.8 | 42.1 | 1.0 | 0.050 | -8.6 | 42.2 | 1.0 | 0.070 |
| -7.1 | 42.0 | 1.0 | 0.046 | 2.8 | 42.0 | 1.0 | 0.118 | -1.7 | 42.1 | 1.0 | 0.045 | -8.5 | 42.2 | 1.0 | 0.070 |
| -7.0 | 42.0 | 1.0 | 0.045 | 2.9 | 42.0 | 1.0 | 0.115 | -1.6 | 42.1 | 1.0 | 0.042 | -8.4 | 42.2 | 1.0 | 0.068 |
| -6.9 | 42.0 | 1.0 | 0.044 | 3.0 | 42.0 | 1.0 | 0.113 | -1.5 | 42.1 | 1.0 | 0.040 | -8.3 | 42.2 | 1.0 | 0.064 |
| -6.8 | 42.0 | 1.0 | 0.044 | 3.1 | 42.0 | 1.0 | 0.109 | -1.4 | 42.1 | 1.0 | 0.039 | -8.2 | 42.2 | 1.0 | 0.062 |
| -6.7 | 42.0 | 1.0 | 0.044 | 3.2 | 42.0 | 1.0 | 0.106 | -0.7 | 42.1 | 1.0 | 0.039 | -8.1 | 42.2 | 1.0 | 0.061 |
| -6.6 | 42.0 | 1.0 | 0.043 | 3.3 | 42.0 | 1.0 | 0.102 | -0.6 | 42.1 | 1.0 | 0.040 | -8.0 | 42.2 | 1.0 | 0.060 |
| -6.5 | 42.0 | 1.0 | 0.043 | 3.4 | 42.0 | 1.0 | 0.099 | -0.5 | 42.1 | 1.0 | 0.041 | -7.9 | 42.2 | 1.0 | 0.060 |
| -6.4 | 42.0 | 1.0 | 0.042 | 3.5 | 42.0 | 1.0 | 0.094 | -0.4 | 42.1 | 1.0 | 0.041 | -7.8 | 42.2 | 1.0 | 0.059 |
| -6.3 | 42.0 | 1.0 | 0.041 | 3.6 | 42.0 | 1.0 | 0.088 | -0.3 | 42.1 | 1.0 | 0.042 | -7.7 | 42.2 | 1.0 | 0.057 |
| -6.2 | 42.0 | 1.0 | 0.041 | 3.7 | 42.0 | 1.0 | 0.080 | -0.2 | 42.1 | 1.0 | 0.043 | -7.6 | 42.2 | 1.0 | 0.054 |
| -6.1 | 42.0 | 1.0 | 0.041 | -9.5 | 42.1 | 1.0 | 0.045 | -0.1 | 42.1 | 1.0 | 0.044 | -7.5 | 42.2 | 1.0 | 0.052 |
| -6.0 | 42.0 | 1.0 | 0.040 | -9.4 | 42.1 | 1.0 | 0.051 | 0.0 | 42.1 | 1.0 | 0.046 | -7.4 | 42.2 | 1.0 | 0.049 |
| -5.9 | 42.0 | 1.0 | 0.039 | -9.3 | 42.1 | 1.0 | 0.059 | 0.1 | 42.1 | 1.0 | 0.047 | -7.3 | 42.2 | 1.0 | 0.049 |
| -3.3 | 42.0 | 1.0 | 0.038 | -9.2 | 42.1 | 1.0 | 0.064 | 0.2 | 42.1 | 1.0 | 0.049 | -7.2 | 42.2 | 1.0 | 0.047 |
| -3.2 | 42.0 | 1.0 | 0.040 | -9.1 | 42.1 | 1.0 | 0.066 | 0.3 | 42.1 | 1.0 | 0.050 | -7.1 | 42.2 | 1.0 | 0.046 |
| -3.1 | 42.0 | 1.0 | 0.041 | -9.0 | 42.1 | 1.0 | 0.068 | 0.4 | 42.1 | 1.0 | 0.052 | -7.0 | 42.2 | 1.0 | 0.046 |
| -3.0 | 42.0 | 1.0 | 0.043 | -8.9 | 42.1 | 1.0 | 0.069 | 0.5 | 42.1 | 1.0 | 0.053 | -6.9 | 42.2 | 1.0 | 0.045 |
| -2.9 | 42.0 | 1.0 | 0.045 | -8.8 | 42.1 | 1.0 | 0.070 | 0.6 | 42.1 | 1.0 | 0.054 | -6.8 | 42.2 | 1.0 | 0.045 |
| -2.8 | 42.0 | 1.0 | 0.048 | -8.7 | 42.1 | 1.0 | 0.071 | 0.7 | 42.1 | 1.0 | 0.055 | -6.7 | 42.2 | 1.0 | 0.044 |
| -2.7 | 42.0 | 1.0 | 0.050 | -8.6 | 42.1 | 1.0 | 0.071 | 0.8 | 42.1 | 1.0 | 0.056 | -6.6 | 42.2 | 1.0 | 0.043 |
| -2.6 | 42.0 | 1.0 | 0.052 | -8.5 | 42.1 | 1.0 | 0.071 | 0.9 | 42.1 | 1.0 | 0.058 | -6.5 | 42.2 | 1.0 | 0.043 |
| -2.5 | 42.0 | 1.0 | 0.059 | -8.4 | 42.1 | 1.0 | 0.069 | 1.0 | 42.1 | 1.0 | 0.059 | -6.4 | 42.2 | 1.0 | 0.042 |
| -2.4 | 42.0 | 1.0 | 0.063 | -8.3 | 42.1 | 1.0 | 0.066 | 1.1 | 42.1 | 1.0 | 0.060 | -6.3 | 42.2 | 1.0 | 0.041 |
| -2.3 | 42.0 | 1.0 | 0.066 | -8.2 | 42.1 | 1.0 | 0.063 | 1.2 | 42.1 | 1.0 | 0.062 | -6.2 | 42.2 | 1.0 | 0.041 |
| -2.2 | 42.0 | 1.0 | 0.066 | -8.1 | 42.1 | 1.0 | 0.062 | 1.3 | 42.1 | 1.0 | 0.064 | -6.1 | 42.2 | 1.0 | 0.040 |
| -2.1 | 42.0 | 1.0 | 0.065 | -8.0 | 42.1 | 1.0 | 0.061 | 1.4 | 42.1 | 1.0 | 0.067 | -6.0 | 42.2 | 1.0 | 0.040 |
| -2.0 | 42.0 | 1.0 | 0.062 | -7.9 | 42.1 | 1.0 | 0.060 | 1.5 | 42.1 | 1.0 | 0.071 | -3.2 | 42.2 | 1.0 | 0.041 |
| -1.9 | 42.0 | 1.0 | 0.056 | -7.8 | 42.1 | 1.0 | 0.058 | 1.6 | 42.1 | 1.0 | 0.074 | -3.1 | 42.2 | 1.0 | 0.043 |
| -1.8 | 42.0 | 1.0 | 0.050 | -7.7 | 42.1 | 1.0 | 0.056 | 1.7 | 42.1 | 1.0 | 0.078 | -3.0 | 42.2 | 1.0 | 0.045 |
| -1.7 | 42.0 | 1.0 | 0.045 | -7.6 | 42.1 | 1.0 | 0.053 | 1.8 | 42.1 | 1.0 | 0.082 | -2.9 | 42.2 | 1.0 | 0.049 |
| -1.6 | 42.0 | 1.0 | 0.041 | -7.5 | 42.1 | 1.0 | 0.051 | 1.9 | 42.1 | 1.0 | 0.087 | -2.8 | 42.2 | 1.0 | 0.053 |
| -1.5 | 42.0 | 1.0 | 0.038 | -7.4 | 42.1 | 1.0 | 0.049 | 2.0 | 42.1 | 1.0 | 0.092 | -2.7 | 42.2 | 1.0 | 0.057 |
| -1.4 | 42.0 | 1.0 | 0.036 | -7.3 | 42.1 | 1.0 | 0.049 | 2.1 | 42.1 | 1.0 | 0.097 | -2.6 | 42.2 | 1.0 | 0.061 |
| -0.7 | 42.0 | 1.0 | 0.034 | -7.2 | 42.1 | 1.0 | 0.047 | 2.2 | 42.1 | 1.0 | 0.103 | -2.5 | 42.2 | 1.0 | 0.064 |
| 0.0 | 42.0 | 1.0 | 0.039 | -7.1 | 42.1 | 1.0 | 0.046 | 2.3 | 42.1 | 1.0 | 0.112 | -2.4 | 42.2 | 1.0 | 0.069 |
| 0.1 | 42.0 | 1.0 | 0.040 | -7.0 | 42.1 | 1.0 | 0.046 | 2.4 | 42.1 | 1.0 | 0.119 | -2.3 | 42.2 | 1.0 | 0.069 |
| 0.2 | 42.0 | 1.0 | 0.041 | -6.9 | 42.1 | 1.0 | 0.045 | 2.5 | 42.1 | 1.0 | 0.124 | -2.2 | 42.2 | 1.0 | 0.067 |
| | | | | | | | | | | | | | | | |
| 0.3 | 42.0 | 1.0 | 0.043 | -6.8 | 42.1 | 1.0 | 0.045 | 2.6 | 42.1 | 1.0 | 0.123 | -2.1 | 42.2 | 1.0 | 0.063 |
| 0.4 | 42.0 | 1.0 | 0.045 | -6.7 | 42.1 | 1.0 | 0.044 | 2.7 | 42.1 | 1.0 | 0.121 | -2.0 | 42.2 | 1.0 | 0.057 |
| 0.5 | 42.0 | 1.0 | 0.046 | -6.6 | 42.1 | 1.0 | 0.044 | 2.8 | 42.1 | 1.0 | 0.119 | -1.9 | 42.2 | 1.0 | 0.053 |
| 0.6 | 42.0 | 1.0 | 0.047 | -6.5 | 42.1 | 1.0 | 0.043 | 2.9 | 42.1 | 1.0 | 0.116 | -1.8 | 42.2 | 1.0 | 0.049 |
| 0.7 | 42.0 | 1.0 | 0.048 | -6.4 | 42.1 | 1.0 | 0.042 | 3.0 | 42.1 | 1.0 | 0.114 | -1.7 | 42.2 | 1.0 | 0.046 |
| 0.8 | 42.0 | 1.0 | 0.048 | -6.3 | 42.1 | 1.0 | 0.042 | 3.1 | 42.1 | 1.0 | 0.110 | -1.6 | 42.2 | 1.0 | 0.044 |
| 0.9 | 42.0 | 1.0 | 0.049 | -6.2 | 42.1 | 1.0 | 0.041 | 3.2 | 42.1 | 1.0 | 0.107 | -1.5 | 42.2 | 1.0 | 0.043 |
| 1.0 | 42.0 | 1.0 | 0.050 | -6.1 | 42.1 | 1.0 | 0.041 | 3.3 | 42.1 | 1.0 | 0.103 | -1.4 | 42.2 | 1.0 | 0.043 |
| 1.1 | 42.0 | 1.0 | 0.051 | -6.0 | 42.1 | 1.0 | 0.040 | 3.4 | 42.1 | 1.0 | 0.099 | -1.3 | 42.2 | 1.0 | 0.043 |
| 1.2 | 42.0 | 1.0 | 0.052 | -5.9 | 42.1 | 1.0 | 0.039 | 3.5 | 42.1 | 1.0 | 0.095 | -1.2 | 42.2 | 1.0 | 0.043 |
| 1.3 | 42.0 | 1.0 | 0.054 | -3.2 | 42.1 | 1.0 | 0.041 | 3.6 | 42.1 | 1.0 | 0.088 | -1.1 | 42.2 | 1.0 | 0.044 |
| 1.4 | 42.0 | 1.0 | 0.056 | -3.1 | 42.1 | 1.0 | 0.042 | 3.7 | 42.1 | 1.0 | 0.077 | -1.0 | 42.2 | 1.0 | 0.044 |
| 1.5 | 42.0 | 1.0 | 0.059 | -3.0 | 42.1 | 1.0 | 0.045 | 3.8 | 42.1 | 1.0 | 0.065 | -0.9 | 42.2 | 1.0 | 0.044 |
| 1.6 | 42.0 | 1.0 | 0.062 | -2.9 | 42.1 | 1.0 | 0.048 | -9.7 | 42.2 | 1.0 | 0.037 | -0.8 | 42.2 | 1.0 | 0.045 |
| 1.7 | 42.0 | 1.0 | 0.065 | -2.8 | 42.1 | 1.0 | 0.051 | -9.6 | 42.2 | 1.0 | 0.041 | -0.7 | 42.2 | 1.0 | 0.046 |
| 1.8 | 42.0 | 1.0 | 0.069 | -2.7 | 42.1 | 1.0 | 0.054 | -9.5 | 42.2 | 1.0 | 0.046 | -0.6 | 42.2 | 1.0 | 0.046 |
| 1.9 | 42.0 | 1.0 | 0.074 | -2.6 | 42.1 | 1.0 | 0.059 | -9.4 | 42.2 | 1.0 | 0.053 | -0.5 | 42.2 | 1.0 | 0.047 |

| Long. | Lat. | K | $a_{gR}$ | Long. | Lat. | K | $a_{gR}$ | Long. | Lat. | K | $a_{gR}$ | Long. | Lat. | K | $a_{gR}$ |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| -0.4 | 42.2 | 1.0 | 0.048 | -7.6 | 42.3 | 1.0 | 0.055 | 1.2 | 42.3 | 1.0 | 0.102 | -6.2 | 42.4 | 1.0 | 0.040 |
| -0.3 | 42.2 | 1.0 | 0.049 | -7.5 | 42.3 | 1.0 | 0.052 | 1.3 | 42.3 | 1.0 | 0.108 | -6.1 | 42.4 | 1.0 | 0.039 |
| -0.2 | 42.2 | 1.0 | 0.050 | -7.4 | 42.3 | 1.0 | 0.050 | 1.4 | 42.3 | 1.0 | 0.113 | -3.1 | 42.4 | 1.0 | 0.040 |
| -0.1 | 42.2 | 1.0 | 0.052 | -7.3 | 42.3 | 1.0 | 0.048 | 1.5 | 42.3 | 1.0 | 0.116 | -3.0 | 42.4 | 1.0 | 0.042 |
| 0.0 | 42.2 | 1.0 | 0.053 | -7.2 | 42.3 | 1.0 | 0.048 | 1.6 | 42.3 | 1.0 | 0.118 | -2.9 | 42.4 | 1.0 | 0.045 |
| 0.1 | 42.2 | 1.0 | 0.055 | -7.1 | 42.3 | 1.0 | 0.047 | 1.7 | 42.3 | 1.0 | 0.121 | -2.8 | 42.4 | 1.0 | 0.048 |
| 0.2 | 42.2 | 1.0 | 0.056 | -7.0 | 42.3 | 1.0 | 0.046 | 1.8 | 42.3 | 1.0 | 0.124 | -2.7 | 42.4 | 1.0 | 0.050 |
| 0.3 | 42.2 | 1.0 | 0.058 | -6.9 | 42.3 | 1.0 | 0.045 | 1.9 | 42.3 | 1.0 | 0.126 | -2.6 | 42.4 | 1.0 | 0.056 |
| 0.4 | 42.2 | 1.0 | 0.059 | -6.8 | 42.3 | 1.0 | 0.045 | 2.0 | 42.3 | 1.0 | 0.131 | -2.5 | 42.4 | 1.0 | 0.059 |
| 0.5 | 42.2 | 1.0 | 0.061 | -6.7 | 42.3 | 1.0 | 0.044 | 2.1 | 42.3 | 1.0 | 0.135 | -2.4 | 42.4 | 1.0 | 0.061 |
| 0.6 | 42.2 | 1.0 | 0.063 | -6.6 | 42.3 | 1.0 | 0.043 | 2.2 | 42.3 | 1.0 | 0.138 | -2.3 | 42.4 | 1.0 | 0.062 |
| 0.7 | 42.2 | 1.0 | 0.065 | -6.5 | 42.3 | 1.0 | 0.042 | 2.3 | 42.3 | 1.0 | 0.139 | -2.2 | 42.4 | 1.0 | 0.061 |
| 0.8 | 42.2 | 1.0 | 0.067 | -6.4 | 42.3 | 1.0 | 0.042 | 2.4 | 42.3 | 1.0 | 0.137 | -2.1 | 42.4 | 1.0 | 0.060 |
| 0.9 | 42.2 | 1.0 | 0.069 | -6.3 | 42.3 | 1.0 | 0.041 | 2.5 | 42.3 | 1.0 | 0.132 | -2.0 | 42.4 | 1.0 | 0.059 |
| 1.0 | 42.2 | 1.0 | 0.071 | -6.2 | 42.3 | 1.0 | 0.040 | 2.6 | 42.3 | 1.0 | 0.127 | -1.9 | 42.4 | 1.0 | 0.058 |
| 1.1 | 42.2 | 1.0 | 0.073 | -6.1 | 42.3 | 1.0 | 0.040 | 2.7 | 42.3 | 1.0 | 0.122 | -1.8 | 42.4 | 1.0 | 0.057 |
| 1.2 | 42.2 | 1.0 | 0.076 | -6.0 | 42.3 | 1.0 | 0.039 | 2.8 | 42.3 | 1.0 | 0.120 | -1.7 | 42.4 | 1.0 | 0.057 |
| 1.3 | 42.2 | 1.0 | 0.079 | -3.3 | 42.3 | 1.0 | 0.038 | 2.9 | 42.3 | 1.0 | 0.116 | -1.6 | 42.4 | 1.0 | 0.058 |
| 1.4 | 42.2 | 1.0 | 0.084 | -3.2 | 42.3 | 1.0 | 0.040 | 3.0 | 42.3 | 1.0 | 0.113 | -1.5 | 42.4 | 1.0 | 0.058 |
| 1.5 | 42.2 | 1.0 | 0.088 | -3.1 | 42.3 | 1.0 | 0.042 | 3.1 | 42.3 | 1.0 | 0.110 | -1.4 | 42.4 | 1.0 | 0.059 |
| 1.6 | 42.2 | 1.0 | 0.092 | -3.0 | 42.3 | 1.0 | 0.045 | 3.2 | 42.3 | 1.0 | 0.106 | -1.3 | 42.4 | 1.0 | 0.061 |
| 1.7 | 42.2 | 1.0 | 0.096 | -2.9 | 42.3 | 1.0 | 0.049 | 3.3 | 42.3 | 1.0 | 0.102 | -1.2 | 42.4 | 1.0 | 0.063 |
| 1.8 | 42.2 | 1.0 | 0.101 | -2.8 | 42.3 | 1.0 | 0.053 | 3.4 | 42.3 | 1.0 | 0.097 | -1.1 | 42.4 | 1.0 | 0.064 |
| 1.9 | 42.2 | 1.0 | 0.106 | -2.7 | 42.3 | 1.0 | 0.058 | 3.5 | 42.3 | 1.0 | 0.090 | -1.0 | 42.4 | 1.0 | 0.065 |
| 2.0 | 42.2 | 1.0 | 0.112 | -2.6 | 42.3 | 1.0 | 0.061 | 3.6 | 42.3 | 1.0 | 0.079 | -0.9 | 42.4 | 1.0 | 0.064 |
| 2.1 | 42.2 | 1.0 | 0.117 | -2.5 | 42.3 | 1.0 | 0.065 | 3.7 | 42.3 | 1.0 | 0.065 | -0.8 | 42.4 | 1.0 | 0.063 |
| 2.2 | 42.2 | 1.0 | 0.122 | -2.4 | 42.3 | 1.0 | 0.065 | 3.8 | 42.3 | 1.0 | 0.052 | -0.7 | 42.4 | 1.0 | 0.063 |
| 2.3 | 42.2 | 1.0 | 0.126 | -2.3 | 42.3 | 1.0 | 0.064 | -9.7 | 42.4 | 1.0 | 0.039 | -0.6 | 42.4 | 1.0 | 0.064 |
| 2.4 | 42.2 | 1.0 | 0.128 | -2.2 | 42.3 | 1.0 | 0.062 | -9.6 | 42.4 | 1.0 | 0.044 | -0.5 | 42.4 | 1.0 | 0.065 |
| 2.5 | 42.2 | 1.0 | 0.128 | -2.1 | 42.3 | 1.0 | 0.058 | -9.5 | 42.4 | 1.0 | 0.049 | -0.4 | 42.4 | 1.0 | 0.066 |
| 2.6 | 42.2 | 1.0 | 0.125 | -2.0 | 42.3 | 1.0 | 0.056 | -9.4 | 42.4 | 1.0 | 0.054 | -0.3 | 42.4 | 1.0 | 0.068 |
| 2.7 | 42.2 | 1.0 | 0.122 | -1.9 | 42.3 | 1.0 | 0.053 | -9.3 | 42.4 | 1.0 | 0.059 | -0.2 | 42.4 | 1.0 | 0.069 |
| 2.8 | 42.2 | 1.0 | 0.118 | -1.8 | 42.3 | 1.0 | 0.051 | -9.2 | 42.4 | 1.0 | 0.063 | -0.1 | 42.4 | 1.0 | 0.071 |
| 2.9 | 42.2 | 1.0 | 0.117 | -1.7 | 42.3 | 1.0 | 0.049 | -9.1 | 42.4 | 1.0 | 0.064 | 0.0 | 42.4 | 1.0 | 0.073 |
| 3.0 | 42.2 | 1.0 | 0.114 | -1.6 | 42.3 | 1.0 | 0.049 | -9.0 | 42.4 | 1.0 | 0.065 | 0.1 | 42.4 | 1.0 | 0.075 |
| 3.1 | 42.2 | 1.0 | 0.110 | -1.5 | 42.3 | 1.0 | 0.048 | -8.9 | 42.4 | 1.0 | 0.066 | 0.2 | 42.4 | 1.0 | 0.077 |
| 3.2 | 42.2 | 1.0 | 0.106 | -1.4 | 42.3 | 1.0 | 0.049 | -8.8 | 42.4 | 1.0 | 0.066 | 0.3 | 42.4 | 1.0 | 0.079 |
| 3.3 | 42.2 | 1.0 | 0.103 | -1.3 | 42.3 | 1.0 | 0.050 | -8.7 | 42.4 | 1.0 | 0.067 | 0.4 | 42.4 | 1.0 | 0.082 |
| 3.4 | 42.2 | 1.0 | 0.099 | -1.2 | 42.3 | 1.0 | 0.051 | -8.6 | 42.4 | 1.0 | 0.068 | 0.5 | 42.4 | 1.0 | 0.086 |
| 3.5 | 42.2 | 1.0 | 0.093 | -1.1 | 42.3 | 1.0 | 0.051 | -8.5 | 42.4 | 1.0 | 0.068 | 0.6 | 42.4 | 1.0 | 0.092 |
| 3.6 | 42.2 | 1.0 | 0.085 | -1.0 | 42.3 | 1.0 | 0.052 | -8.4 | 42.4 | 1.0 | 0.066 | 0.7 | 42.4 | 1.0 | 0.100 |
| 3.7 | 42.2 | 1.0 | 0.072 | -0.9 | 42.3 | 1.0 | 0.052 | -8.3 | 42.4 | 1.0 | 0.062 | 0.8 | 42.4 | 1.0 | 0.110 |
| -9.6 | 42.3 | 1.0 | 0.042 | -0.8 | 42.3 | 1.0 | 0.053 | -8.2 | 42.4 | 1.0 | 0.060 | 0.9 | 42.4 | 1.0 | 0.118 |
| -9.5 | 42.3 | 1.0 | 0.048 | -0.7 | 42.3 | 1.0 | 0.053 | -8.1 | 42.4 | 1.0 | 0.059 | 1.0 | 42.4 | 1.0 | 0.122 |
| -9.4 | 42.3 | 1.0 | 0.054 | -0.6 | 42.3 | 1.0 | 0.054 | -8.0 | 42.4 | 1.0 | 0.059 | 1.1 | 42.4 | 1.0 | 0.125 |
| -9.3 | 42.3 | 1.0 | 0.060 | -0.5 | 42.3 | 1.0 | 0.055 | -7.9 | 42.4 | 1.0 | 0.058 | 1.2 | 42.4 | 1.0 | 0.126 |
| -9.2 | 42.3 | 1.0 | 0.063 | -0.4 | 42.3 | 1.0 | 0.056 | -7.8 | 42.4 | 1.0 | 0.058 | 1.3 | 42.4 | 1.0 | 0.127 |
| -9.1 | 42.3 | 1.0 | 0.065 | -0.3 | 42.3 | 1.0 | 0.058 | -7.7 | 42.4 | 1.0 | 0.057 | 1.4 | 42.4 | 1.0 | 0.130 |
| -9.0 | 42.3 | 1.0 | 0.066 | -0.2 | 42.3 | 1.0 | 0.059 | -7.6 | 42.4 | 1.0 | 0.056 | 1.5 | 42.4 | 1.0 | 0.131 |
| -8.9 | 42.3 | 1.0 | 0.067 | -0.1 | 42.3 | 1.0 | 0.060 | -7.5 | 42.4 | 1.0 | 0.053 | 1.6 | 42.4 | 1.0 | 0.131 |
| -8.8 | 42.3 | 1.0 | 0.068 | 0.0 | 42.3 | 1.0 | 0.062 | -7.4 | 42.4 | 1.0 | 0.051 | 1.7 | 42.4 | 1.0 | 0.132 |
| -8.7 | 42.3 | 1.0 | 0.069 | 0.1 | 42.3 | 1.0 | 0.063 | -7.3 | 42.4 | 1.0 | 0.049 | 1.8 | 42.4 | 1.0 | 0.133 |
| -8.6 | 42.3 | 1.0 | 0.068 | 0.2 | 42.3 | 1.0 | 0.065 | -7.2 | 42.4 | 1.0 | 0.049 | 1.9 | 42.4 | 1.0 | 0.134 |
| -8.5 | 42.3 | 1.0 | 0.068 | 0.3 | 42.3 | 1.0 | 0.067 | -7.1 | 42.4 | 1.0 | 0.048 | 2.0 | 42.4 | 1.0 | 0.136 |
| -8.4 | 42.3 | 1.0 | 0.066 | 0.4 | 42.3 | 1.0 | 0.069 | -7.0 | 42.4 | 1.0 | 0.047 | 2.1 | 42.4 | 1.0 | 0.137 |
| -8.3 | 42.3 | 1.0 | 0.063 | 0.5 | 42.3 | 1.0 | 0.071 | -6.9 | 42.4 | 1.0 | 0.046 | 2.2 | 42.4 | 1.0 | 0.139 |
| -8.2 | 42.3 | 1.0 | 0.061 | 0.6 | 42.3 | 1.0 | 0.074 | -6.8 | 42.4 | 1.0 | 0.045 | 2.3 | 42.4 | 1.0 | 0.139 |
| -8.1 | 42.3 | 1.0 | 0.060 | 0.7 | 42.3 | 1.0 | 0.079 | -6.7 | 42.4 | 1.0 | 0.044 | 2.4 | 42.4 | 1.0 | 0.138 |
| -8.0 | 42.3 | 1.0 | 0.060 | 0.8 | 42.3 | 1.0 | 0.083 | -6.6 | 42.4 | 1.0 | 0.044 | 2.5 | 42.4 | 1.0 | 0.135 |
| -7.9 | 42.3 | 1.0 | 0.059 | 0.9 | 42.3 | 1.0 | 0.087 | -6.5 | 42.4 | 1.0 | 0.042 | 2.6 | 42.4 | 1.0 | 0.129 |
| -7.8 | 42.3 | 1.0 | 0.059 | 1.0 | 42.3 | 1.0 | 0.092 | -6.4 | 42.4 | 1.0 | 0.042 | 2.7 | 42.4 | 1.0 | 0.124 |
| -7.7 | 42.3 | 1.0 | 0.057 | 1.1 | 42.3 | 1.0 | 0.097 | -6.3 | 42.4 | 1.0 | 0.041 | 2.8 | 42.4 | 1.0 | 0.120 |

| Long. | Lat. | K | $a_{gR}$ | Long. | Lat. | K | $a_{gR}$ | Long. | Lat. | K | $a_{gR}$ | Long. | Lat. | K | $a_{gR}$ |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 2.9 | 42.4 | 1.0 | 0.116 | -1.4 | 42.5 | 1.0 | 0.074 | -8.5 | 42.6 | 1.0 | 0.065 | 0.8 | 42.6 | 1.0 | 0.143 |
| 3.0 | 42.4 | 1.0 | 0.114 | -1.3 | 42.5 | 1.0 | 0.077 | -8.4 | 42.6 | 1.0 | 0.063 | 0.9 | 42.6 | 1.0 | 0.141 |
| 3.1 | 42.4 | 1.0 | 0.110 | -1.2 | 42.5 | 1.0 | 0.079 | -8.3 | 42.6 | 1.0 | 0.060 | 1.0 | 42.6 | 1.0 | 0.139 |
| 3.2 | 42.4 | 1.0 | 0.106 | -1.1 | 42.5 | 1.0 | 0.080 | -8.2 | 42.6 | 1.0 | 0.059 | 1.1 | 42.6 | 1.0 | 0.136 |
| 3.3 | 42.4 | 1.0 | 0.101 | -1.0 | 42.5 | 1.0 | 0.080 | -8.1 | 42.6 | 1.0 | 0.058 | 1.2 | 42.6 | 1.0 | 0.133 |
| 3.4 | 42.4 | 1.0 | 0.094 | -0.9 | 42.5 | 1.0 | 0.079 | -8.0 | 42.6 | 1.0 | 0.057 | 1.3 | 42.6 | 1.0 | 0.131 |
| 3.5 | 42.4 | 1.0 | 0.084 | -0.8 | 42.5 | 1.0 | 0.076 | -7.9 | 42.6 | 1.0 | 0.057 | 1.4 | 42.6 | 1.0 | 0.130 |
| 3.6 | 42.4 | 1.0 | 0.071 | -0.7 | 42.5 | 1.0 | 0.075 | -7.8 | 42.6 | 1.0 | 0.057 | 1.5 | 42.6 | 1.0 | 0.130 |
| -9.6 | 42.5 | 1.0 | 0.044 | -0.6 | 42.5 | 1.0 | 0.076 | -7.7 | 42.6 | 1.0 | 0.057 | 1.6 | 42.6 | 1.0 | 0.130 |
| -9.5 | 42.5 | 1.0 | 0.049 | -0.5 | 42.5 | 1.0 | 0.078 | -7.6 | 42.6 | 1.0 | 0.057 | 1.7 | 42.6 | 1.0 | 0.130 |
| -9.4 | 42.5 | 1.0 | 0.053 | -0.4 | 42.5 | 1.0 | 0.079 | -7.5 | 42.6 | 1.0 | 0.057 | 1.8 | 42.6 | 1.0 | 0.131 |
| -9.3 | 42.5 | 1.0 | 0.058 | -0.3 | 42.5 | 1.0 | 0.081 | -7.4 | 42.6 | 1.0 | 0.056 | -9.8 | 42.7 | 1.0 | 0.037 |
| -9.2 | 42.5 | 1.0 | 0.060 | -0.2 | 42.5 | 1.0 | 0.084 | -7.3 | 42.6 | 1.0 | 0.055 | -9.7 | 42.7 | 1.0 | 0.041 |
| -9.1 | 42.5 | 1.0 | 0.062 | -0.1 | 42.5 | 1.0 | 0.086 | -7.2 | 42.6 | 1.0 | 0.055 | -9.6 | 42.7 | 1.0 | 0.045 |
| -9.0 | 42.5 | 1.0 | 0.063 | 0.0 | 42.5 | 1.0 | 0.088 | -7.1 | 42.6 | 1.0 | 0.053 | -9.5 | 42.7 | 1.0 | 0.047 |
| -8.9 | 42.5 | 1.0 | 0.064 | 0.1 | 42.5 | 1.0 | 0.090 | -7.0 | 42.6 | 1.0 | 0.051 | -9.4 | 42.7 | 1.0 | 0.049 |
| -8.8 | 42.5 | 1.0 | 0.065 | 0.2 | 42.5 | 1.0 | 0.093 | -6.9 | 42.6 | 1.0 | 0.049 | -9.3 | 42.7 | 1.0 | 0.051 |
| -8.7 | 42.5 | 1.0 | 0.066 | 0.3 | 42.5 | 1.0 | 0.096 | -6.8 | 42.6 | 1.0 | 0.047 | -9.2 | 42.7 | 1.0 | 0.053 |
| -8.6 | 42.5 | 1.0 | 0.067 | 0.4 | 42.5 | 1.0 | 0.101 | -6.7 | 42.6 | 1.0 | 0.044 | -9.1 | 42.7 | 1.0 | 0.055 |
| -8.5 | 42.5 | 1.0 | 0.066 | 0.5 | 42.5 | 1.0 | 0.109 | -6.6 | 42.6 | 1.0 | 0.043 | -9.0 | 42.7 | 1.0 | 0.057 |
| -8.4 | 42.5 | 1.0 | 0.064 | 0.6 | 42.5 | 1.0 | 0.120 | -6.5 | 42.6 | 1.0 | 0.041 | -8.9 | 42.7 | 1.0 | 0.059 |
| -8.3 | 42.5 | 1.0 | 0.061 | 0.7 | 42.5 | 1.0 | 0.130 | -6.4 | 42.6 | 1.0 | 0.040 | -8.8 | 42.7 | 1.0 | 0.061 |
| -8.2 | 42.5 | 1.0 | 0.059 | 0.8 | 42.5 | 1.0 | 0.134 | -6.3 | 42.6 | 1.0 | 0.040 | -8.7 | 42.7 | 1.0 | 0.063 |
| -8.1 | 42.5 | 1.0 | 0.059 | 0.9 | 42.5 | 1.0 | 0.135 | -6.2 | 42.6 | 1.0 | 0.039 | -8.6 | 42.7 | 1.0 | 0.064 |
| -8.0 | 42.5 | 1.0 | 0.058 | 1.0 | 42.5 | 1.0 | 0.135 | -3.0 | 42.6 | 1.0 | 0.037 | -8.5 | 42.7 | 1.0 | 0.064 |
| -7.9 | 42.5 | 1.0 | 0.058 | 1.1 | 42.5 | 1.0 | 0.134 | -2.9 | 42.6 | 1.0 | 0.039 | -8.4 | 42.7 | 1.0 | 0.061 |
| -7.8 | 42.5 | 1.0 | 0.057 | 1.2 | 42.5 | 1.0 | 0.133 | -2.8 | 42.6 | 1.0 | 0.043 | -8.3 | 42.7 | 1.0 | 0.059 |
| -7.7 | 42.5 | 1.0 | 0.057 | 1.3 | 42.5 | 1.0 | 0.132 | -2.7 | 42.6 | 1.0 | 0.046 | -8.2 | 42.7 | 1.0 | 0.057 |
| -7.6 | 42.5 | 1.0 | 0.056 | 1.4 | 42.5 | 1.0 | 0.132 | -2.6 | 42.6 | 1.0 | 0.049 | -8.1 | 42.7 | 1.0 | 0.057 |
| -7.5 | 42.5 | 1.0 | 0.055 | 1.5 | 42.5 | 1.0 | 0.133 | -2.5 | 42.6 | 1.0 | 0.053 | -8.0 | 42.7 | 1.0 | 0.056 |
| -7.4 | 42.5 | 1.0 | 0.053 | 1.6 | 42.5 | 1.0 | 0.134 | -2.4 | 42.6 | 1.0 | 0.060 | -7.9 | 42.7 | 1.0 | 0.056 |
| -7.3 | 42.5 | 1.0 | 0.051 | 1.7 | 42.5 | 1.0 | 0.135 | -2.3 | 42.6 | 1.0 | 0.066 | -7.8 | 42.7 | 1.0 | 0.057 |
| -7.2 | 42.5 | 1.0 | 0.050 | 1.8 | 42.5 | 1.0 | 0.136 | -2.2 | 42.6 | 1.0 | 0.069 | -7.7 | 42.7 | 1.0 | 0.057 |
| -7.1 | 42.5 | 1.0 | 0.050 | 1.9 | 42.5 | 1.0 | 0.136 | -2.1 | 42.6 | 1.0 | 0.071 | -7.6 | 42.7 | 1.0 | 0.058 |
| -7.0 | 42.5 | 1.0 | 0.048 | 2.0 | 42.5 | 1.0 | 0.136 | -2.0 | 42.6 | 1.0 | 0.073 | -7.5 | 42.7 | 1.0 | 0.059 |
| -6.9 | 42.5 | 1.0 | 0.047 | 2.1 | 42.5 | 1.0 | 0.136 | -1.9 | 42.6 | 1.0 | 0.075 | -7.4 | 42.7 | 1.0 | 0.059 |
| -6.8 | 42.5 | 1.0 | 0.046 | 2.2 | 42.5 | 1.0 | 0.135 | -1.8 | 42.6 | 1.0 | 0.078 | -7.3 | 42.7 | 1.0 | 0.059 |
| -6.7 | 42.5 | 1.0 | 0.045 | 2.3 | 42.5 | 1.0 | 0.135 | -1.7 | 42.6 | 1.0 | 0.079 | -7.2 | 42.7 | 1.0 | 0.060 |
| -6.6 | 42.5 | 1.0 | 0.043 | 2.4 | 42.5 | 1.0 | 0.135 | -1.6 | 42.6 | 1.0 | 0.080 | -7.1 | 42.7 | 1.0 | 0.057 |
| -6.5 | 42.5 | 1.0 | 0.042 | 2.6 | 42.5 | 1.0 | 0.131 | -1.5 | 42.6 | 1.0 | 0.082 | -7.0 | 42.7 | 1.0 | 0.055 |
| -6.4 | 42.5 | 1.0 | 0.041 | 2.7 | 42.5 | 1.0 | 0.128 | -1.4 | 42.6 | 1.0 | 0.083 | -6.9 | 42.7 | 1.0 | 0.051 |
| -6.3 | 42.5 | 1.0 | 0.040 | 2.8 | 42.5 | 1.0 | 0.124 | -1.3 | 42.6 | 1.0 | 0.086 | -6.8 | 42.7 | 1.0 | 0.047 |
| -6.2 | 42.5 | 1.0 | 0.040 | 2.9 | 42.5 | 1.0 | 0.121 | -1.2 | 42.6 | 1.0 | 0.088 | -6.7 | 42.7 | 1.0 | 0.044 |
| -6.1 | 42.5 | 1.0 | 0.039 | 3.0 | 42.5 | 1.0 | 0.118 | -1.1 | 42.6 | 1.0 | 0.089 | -6.6 | 42.7 | 1.0 | 0.042 |
| -3.2 | 42.5 | 1.0 | 0.035 | 3.1 | 42.5 | 1.0 | 0.114 | -1.0 | 42.6 | 1.0 | 0.091 | -6.5 | 42.7 | 1.0 | 0.040 |
| -3.1 | 42.5 | 1.0 | 0.037 | 3.2 | 42.5 | 1.0 | 0.109 | -0.9 | 42.6 | 1.0 | 0.092 | -6.4 | 42.7 | 1.0 | 0.039 |
| -3.0 | 42.5 | 1.0 | 0.039 | 3.3 | 42.5 | 1.0 | 0.102 | -0.8 | 42.6 | 1.0 | 0.093 | -6.3 | 42.7 | 1.0 | 0.038 |
| -2.9 | 42.5 | 1.0 | 0.041 | 3.4 | 42.5 | 1.0 | 0.091 | -0.7 | 42.6 | 1.0 | 0.091 | -2.9 | 42.7 | 1.0 | 0.039 |
| -2.8 | 42.5 | 1.0 | 0.044 | 3.5 | 42.5 | 1.0 | 0.077 | -0.6 | 42.6 | 1.0 | 0.092 | -2.8 | 42.7 | 1.0 | 0.042 |
| -2.7 | 42.5 | 1.0 | 0.048 | 3.6 | 42.5 | 1.0 | 0.065 | -0.5 | 42.6 | 1.0 | 0.095 | -2.7 | 42.7 | 1.0 | 0.045 |
| -2.6 | 42.5 | 1.0 | 0.052 | 3.7 | 42.5 | 1.0 | 0.054 | -0.4 | 42.6 | 1.0 | 0.098 | -2.6 | 42.7 | 1.0 | 0.047 |
| -2.5 | 42.5 | 1.0 | 0.057 | -9.6 | 42.6 | 1.0 | 0.044 | -0.3 | 42.6 | 1.0 | 0.101 | -2.5 | 42.7 | 1.0 | 0.051 |
| -2.4 | 42.5 | 1.0 | 0.062 | -9.5 | 42.6 | 1.0 | 0.048 | -0.2 | 42.6 | 1.0 | 0.103 | -2.4 | 42.7 | 1.0 | 0.056 |
| -2.3 | 42.5 | 1.0 | 0.066 | -9.4 | 42.6 | 1.0 | 0.050 | -0.1 | 42.6 | 1.0 | 0.106 | -2.3 | 42.7 | 1.0 | 0.063 |
| -2.2 | 42.5 | 1.0 | 0.067 | -9.3 | 42.6 | 1.0 | 0.053 | 0.0 | 42.6 | 1.0 | 0.109 | -2.2 | 42.7 | 1.0 | 0.069 |
| -2.1 | 42.5 | 1.0 | 0.068 | -9.2 | 42.6 | 1.0 | 0.056 | 0.1 | 42.6 | 1.0 | 0.112 | -2.1 | 42.7 | 1.0 | 0.074 |
| -2.0 | 42.5 | 1.0 | 0.069 | -9.1 | 42.6 | 1.0 | 0.059 | 0.2 | 42.6 | 1.0 | 0.116 | -2.0 | 42.7 | 1.0 | 0.078 |
| -1.9 | 42.5 | 1.0 | 0.069 | -9.0 | 42.6 | 1.0 | 0.061 | 0.3 | 42.6 | 1.0 | 0.122 | -1.9 | 42.7 | 1.0 | 0.081 |
| -1.8 | 42.5 | 1.0 | 0.070 | -8.9 | 42.6 | 1.0 | 0.063 | 0.4 | 42.6 | 1.0 | 0.131 | -1.8 | 42.7 | 1.0 | 0.085 |
| -1.7 | 42.5 | 1.0 | 0.071 | -8.8 | 42.6 | 1.0 | 0.064 | 0.5 | 42.6 | 1.0 | 0.138 | -1.7 | 42.7 | 1.0 | 0.087 |
| -1.6 | 42.5 | 1.0 | 0.072 | -8.7 | 42.6 | 1.0 | 0.065 | 0.6 | 42.6 | 1.0 | 0.142 | -1.6 | 42.7 | 1.0 | 0.087 |
| -1.5 | 42.5 | 1.0 | 0.073 | -8.6 | 42.6 | 1.0 | 0.066 | 0.7 | 42.6 | 1.0 | 0.143 | -1.5 | 42.7 | 1.0 | 0.088 |

| Long. | Lat. | K | $a_{gR}$ | Long. | Lat. | K | $a_{gR}$ | Long. | Lat. | K | $a_{gR}$ | Long. | Lat. | K | $a_{gR}$ |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| -1.4 | 42.7 | 1.0 | 0.089 | -6.5 | 42.8 | 1.0 | 0.038 | -8.4 | 42.9 | 1.0 | 0.056 | -9.1 | 43.0 | 1.0 | 0.050 |
| -1.3 | 42.7 | 1.0 | 0.091 | -6.4 | 42.8 | 1.0 | 0.037 | -8.3 | 42.9 | 1.0 | 0.056 | -9.0 | 43.0 | 1.0 | 0.051 |
| -1.2 | 42.7 | 1.0 | 0.094 | -3.0 | 42.8 | 1.0 | 0.037 | -8.2 | 42.9 | 1.0 | 0.055 | -8.9 | 43.0 | 1.0 | 0.052 |
| -1.1 | 42.7 | 1.0 | 0.096 | -2.9 | 42.8 | 1.0 | 0.040 | -8.1 | 42.9 | 1.0 | 0.055 | -8.8 | 43.0 | 1.0 | 0.053 |
| -1.0 | 42.7 | 1.0 | 0.100 | -2.8 | 42.8 | 1.0 | 0.042 | -8.0 | 42.9 | 1.0 | 0.055 | -8.7 | 43.0 | 1.0 | 0.053 |
| -0.9 | 42.7 | 1.0 | 0.104 | -2.7 | 42.8 | 1.0 | 0.044 | -7.9 | 42.9 | 1.0 | 0.055 | -8.6 | 43.0 | 1.0 | 0.054 |
| -0.8 | 42.7 | 1.0 | 0.108 | -2.6 | 42.8 | 1.0 | 0.047 | -7.8 | 42.9 | 1.0 | 0.056 | -8.5 | 43.0 | 1.0 | 0.054 |
| -0.7 | 42.7 | 1.0 | 0.112 | -2.5 | 42.8 | 1.0 | 0.050 | -7.7 | 42.9 | 1.0 | 0.057 | -8.4 | 43.0 | 1.0 | 0.054 |
| -0.6 | 42.7 | 1.0 | 0.115 | -2.4 | 42.8 | 1.0 | 0.054 | -7.6 | 42.9 | 1.0 | 0.058 | -8.3 | 43.0 | 1.0 | 0.054 |
| -0.5 | 42.7 | 1.0 | 0.119 | -2.3 | 42.8 | 1.0 | 0.060 | -7.5 | 42.9 | 1.0 | 0.059 | -8.2 | 43.0 | 1.0 | 0.054 |
| -0.4 | 42.7 | 1.0 | 0.125 | -2.2 | 42.8 | 1.0 | 0.068 | -7.4 | 42.9 | 1.0 | 0.060 | -8.1 | 43.0 | 1.0 | 0.054 |
| -0.3 | 42.7 | 1.0 | 0.130 | -2.1 | 42.8 | 1.0 | 0.076 | -7.3 | 42.9 | 1.0 | 0.063 | -8.0 | 43.0 | 1.0 | 0.054 |
| -0.2 | 42.7 | 1.0 | 0.133 | -2.0 | 42.8 | 1.0 | 0.082 | -7.2 | 42.9 | 1.0 | 0.061 | -7.9 | 43.0 | 1.0 | 0.054 |
| -0.1 | 42.7 | 1.0 | 0.137 | -1.9 | 42.8 | 1.0 | 0.086 | -7.1 | 42.9 | 1.0 | 0.058 | -7.8 | 43.0 | 1.0 | 0.055 |
| 0.0 | 42.7 | 1.0 | 0.142 | -1.8 | 42.8 | 1.0 | 0.089 | -7.0 | 42.9 | 1.0 | 0.055 | -7.7 | 43.0 | 1.0 | 0.056 |
| 0.1 | 42.7 | 1.0 | 0.148 | -1.7 | 42.8 | 1.0 | 0.091 | -6.9 | 42.9 | 1.0 | 0.050 | -7.6 | 43.0 | 1.0 | 0.057 |
| 0.2 | 42.7 | 1.0 | 0.153 | -1.6 | 42.8 | 1.0 | 0.092 | -6.8 | 42.9 | 1.0 | 0.045 | -7.5 | 43.0 | 1.0 | 0.058 |
| 0.3 | 42.7 | 1.0 | 0.154 | -1.5 | 42.8 | 1.0 | 0.093 | -6.7 | 42.9 | 1.0 | 0.041 | -7.4 | 43.0 | 1.0 | 0.059 |
| 0.4 | 42.7 | 1.0 | 0.152 | -1.4 | 42.8 | 1.0 | 0.095 | -6.6 | 42.9 | 1.0 | 0.039 | -7.3 | 43.0 | 1.0 | 0.060 |
| 0.5 | 42.7 | 1.0 | 0.150 | -1.3 | 42.8 | 1.0 | 0.097 | -6.5 | 42.9 | 1.0 | 0.036 | -7.2 | 43.0 | 1.0 | 0.059 |
| 0.6 | 42.7 | 1.0 | 0.147 | -1.2 | 42.8 | 1.0 | 0.101 | -3.0 | 42.9 | 1.0 | 0.038 | -7.1 | 43.0 | 1.0 | 0.056 |
| 0.7 | 42.7 | 1.0 | 0.146 | -1.1 | 42.8 | 1.0 | 0.105 | -2.9 | 42.9 | 1.0 | 0.040 | -7.0 | 43.0 | 1.0 | 0.052 |
| 0.8 | 42.7 | 1.0 | 0.142 | -1.0 | 42.8 | 1.0 | 0.110 | -2.8 | 42.9 | 1.0 | 0.042 | -6.9 | 43.0 | 1.0 | 0.047 |
| 0.9 | 42.7 | 1.0 | 0.138 | -0.9 | 42.8 | 1.0 | 0.117 | -2.7 | 42.9 | 1.0 | 0.044 | -6.8 | 43.0 | 1.0 | 0.042 |
| 1.0 | 42.7 | 1.0 | 0.135 | -0.8 | 42.8 | 1.0 | 0.126 | -2.6 | 42.9 | 1.0 | 0.047 | -6.7 | 43.0 | 1.0 | 0.039 |
| 1.1 | 42.7 | 1.0 | 0.132 | -0.7 | 42.8 | 1.0 | 0.136 | -2.5 | 42.9 | 1.0 | 0.050 | -6.6 | 43.0 | 1.0 | 0.037 |
| 1.2 | 42.7 | 1.0 | 0.127 | -0.6 | 42.8 | 1.0 | 0.145 | -2.4 | 42.9 | 1.0 | 0.053 | -3.0 | 43.0 | 1.0 | 0.038 |
| 1.3 | 42.7 | 1.0 | 0.124 | -0.5 | 42.8 | 1.0 | 0.157 | -2.3 | 42.9 | 1.0 | 0.058 | -2.9 | 43.0 | 1.0 | 0.040 |
| 1.4 | 42.7 | 1.0 | 0.122 | -0.4 | 42.8 | 1.0 | 0.164 | -2.2 | 42.9 | 1.0 | 0.065 | -2.8 | 43.0 | 1.0 | 0.042 |
| -9.8 | 42.8 | 1.0 | 0.038 | -0.3 | 42.8 | 1.0 | 0.170 | -2.1 | 42.9 | 1.0 | 0.074 | -2.7 | 43.0 | 1.0 | 0.044 |
| -9.7 | 42.8 | 1.0 | 0.042 | -0.2 | 42.8 | 1.0 | 0.175 | -2.0 | 42.9 | 1.0 | 0.083 | -2.6 | 43.0 | 1.0 | 0.047 |
| -9.6 | 42.8 | 1.0 | 0.044 | -0.1 | 42.8 | 1.0 | 0.179 | -1.9 | 42.9 | 1.0 | 0.088 | -2.5 | 43.0 | 1.0 | 0.050 |
| -9.5 | 42.8 | 1.0 | 0.046 | 0.0 | 42.8 | 1.0 | 0.185 | -1.8 | 42.9 | 1.0 | 0.090 | -2.4 | 43.0 | 1.0 | 0.053 |
| -9.4 | 42.8 | 1.0 | 0.048 | 0.1 | 42.8 | 1.0 | 0.189 | -1.7 | 42.9 | 1.0 | 0.092 | -2.3 | 43.0 | 1.0 | 0.057 |
| -9.3 | 42.8 | 1.0 | 0.050 | 0.2 | 42.8 | 1.0 | 0.193 | -1.6 | 42.9 | 1.0 | 0.094 | -2.2 | 43.0 | 1.0 | 0.063 |
| -9.2 | 42.8 | 1.0 | 0.051 | 0.3 | 42.8 | 1.0 | 0.188 | -1.5 | 42.9 | 1.0 | 0.096 | -2.1 | 43.0 | 1.0 | 0.070 |
| -9.1 | 42.8 | 1.0 | 0.053 | 0.4 | 42.8 | 1.0 | 0.165 | -1.4 | 42.9 | 1.0 | 0.099 | -2.0 | 43.0 | 1.0 | 0.079 |
| -9.0 | 42.8 | 1.0 | 0.054 | 0.5 | 42.8 | 1.0 | 0.151 | -1.3 | 42.9 | 1.0 | 0.103 | -1.9 | 43.0 | 1.0 | 0.086 |
| -8.9 | 42.8 | 1.0 | 0.056 | 0.6 | 42.8 | 1.0 | 0.145 | -1.2 | 42.9 | 1.0 | 0.108 | -1.8 | 43.0 | 1.0 | 0.090 |
| -8.8 | 42.8 | 1.0 | 0.057 | 0.7 | 42.8 | 1.0 | 0.139 | -1.1 | 42.9 | 1.0 | 0.114 | -1.7 | 43.0 | 1.0 | 0.093 |
| -8.7 | 42.8 | 1.0 | 0.058 | 0.8 | 42.8 | 1.0 | 0.133 | -1.0 | 42.9 | 1.0 | 0.124 | -1.6 | 43.0 | 1.0 | 0.096 |
| -8.6 | 42.8 | 1.0 | 0.060 | 0.9 | 42.8 | 1.0 | 0.128 | -0.9 | 42.9 | 1.0 | 0.136 | -1.5 | 43.0 | 1.0 | 0.100 |
| -8.5 | 42.8 | 1.0 | 0.060 | 1.0 | 42.8 | 1.0 | 0.124 | -0.8 | 42.9 | 1.0 | 0.153 | -1.4 | 43.0 | 1.0 | 0.104 |
| -8.4 | 42.8 | 1.0 | 0.059 | 1.1 | 42.8 | 1.0 | 0.121 | -0.7 | 42.9 | 1.0 | 0.172 | -1.3 | 43.0 | 1.0 | 0.109 |
| -8.3 | 42.8 | 1.0 | 0.057 | 1.2 | 42.8 | 1.0 | 0.117 | -0.6 | 42.9 | 1.0 | 0.184 | -1.2 | 43.0 | 1.0 | 0.116 |
| -8.2 | 42.8 | 1.0 | 0.057 | 1.3 | 42.8 | 1.0 | 0.114 | -0.5 | 42.9 | 1.0 | 0.198 | -1.1 | 43.0 | 1.0 | 0.126 |
| -8.1 | 42.8 | 1.0 | 0.056 | 1.4 | 42.8 | 1.0 | 0.112 | -0.4 | 42.9 | 1.0 | 0.206 | -1.0 | 43.0 | 1.0 | 0.143 |
| -8.0 | 42.8 | 1.0 | 0.056 | 1.5 | 42.8 | 1.0 | 0.110 | -0.3 | 42.9 | 1.0 | 0.213 | -0.9 | 43.0 | 1.0 | 0.167 |
| -7.9 | 42.8 | 1.0 | 0.056 | -9.8 | 42.9 | 1.0 | 0.039 | -0.2 | 42.9 | 1.0 | 0.223 | -0.8 | 43.0 | 1.0 | 0.204 |
| -7.8 | 42.8 | 1.0 | 0.056 | -9.7 | 42.9 | 1.0 | 0.042 | -0.1 | 42.9 | 1.0 | 0.236 | -0.7 | 43.0 | 1.0 | 0.236 |
| -7.7 | 42.8 | 1.0 | 0.057 | -9.6 | 42.9 | 1.0 | 0.043 | 0.6 | 42.9 | 1.0 | 0.143 | -9.8 | 43.1 | 1.0 | 0.038 |
| -7.6 | 42.8 | 1.0 | 0.059 | -9.5 | 42.9 | 1.0 | 0.045 | 0.7 | 42.9 | 1.0 | 0.135 | -9.7 | 43.1 | 1.0 | 0.040 |
| -7.5 | 42.8 | 1.0 | 0.060 | -9.4 | 42.9 | 1.0 | 0.047 | 0.8 | 42.9 | 1.0 | 0.127 | -9.6 | 43.1 | 1.0 | 0.043 |
| -7.4 | 42.8 | 1.0 | 0.061 | -9.3 | 42.9 | 1.0 | 0.049 | 0.9 | 42.9 | 1.0 | 0.122 | -9.5 | 43.1 | 1.0 | 0.044 |
| -7.3 | 42.8 | 1.0 | 0.064 | -9.2 | 42.9 | 1.0 | 0.050 | 1.0 | 42.9 | 1.0 | 0.117 | -9.4 | 43.1 | 1.0 | 0.046 |
| -7.2 | 42.8 | 1.0 | 0.063 | -9.1 | 42.9 | 1.0 | 0.051 | -9.8 | 43.0 | 1.0 | 0.039 | -9.3 | 43.1 | 1.0 | 0.047 |
| -7.1 | 42.8 | 1.0 | 0.060 | -9.0 | 42.9 | 1.0 | 0.052 | -9.7 | 43.0 | 1.0 | 0.041 | -9.2 | 43.1 | 1.0 | 0.048 |
| -7.0 | 42.8 | 1.0 | 0.056 | -8.9 | 42.9 | 1.0 | 0.053 | -9.6 | 43.0 | 1.0 | 0.043 | -9.1 | 43.1 | 1.0 | 0.049 |
| -6.9 | 42.8 | 1.0 | 0.052 | -8.8 | 42.9 | 1.0 | 0.054 | -9.5 | 43.0 | 1.0 | 0.045 | -9.0 | 43.1 | 1.0 | 0.050 |
| -6.8 | 42.8 | 1.0 | 0.047 | -8.7 | 42.9 | 1.0 | 0.055 | -9.4 | 43.0 | 1.0 | 0.046 | -8.9 | 43.1 | 1.0 | 0.050 |
| -6.7 | 42.8 | 1.0 | 0.043 | -8.6 | 42.9 | 1.0 | 0.056 | -9.3 | 43.0 | 1.0 | 0.047 | -8.8 | 43.1 | 1.0 | 0.051 |
| -6.6 | 42.8 | 1.0 | 0.040 | -8.5 | 42.9 | 1.0 | 0.056 | -9.2 | 43.0 | 1.0 | 0.049 | -8.7 | 43.1 | 1.0 | 0.052 |

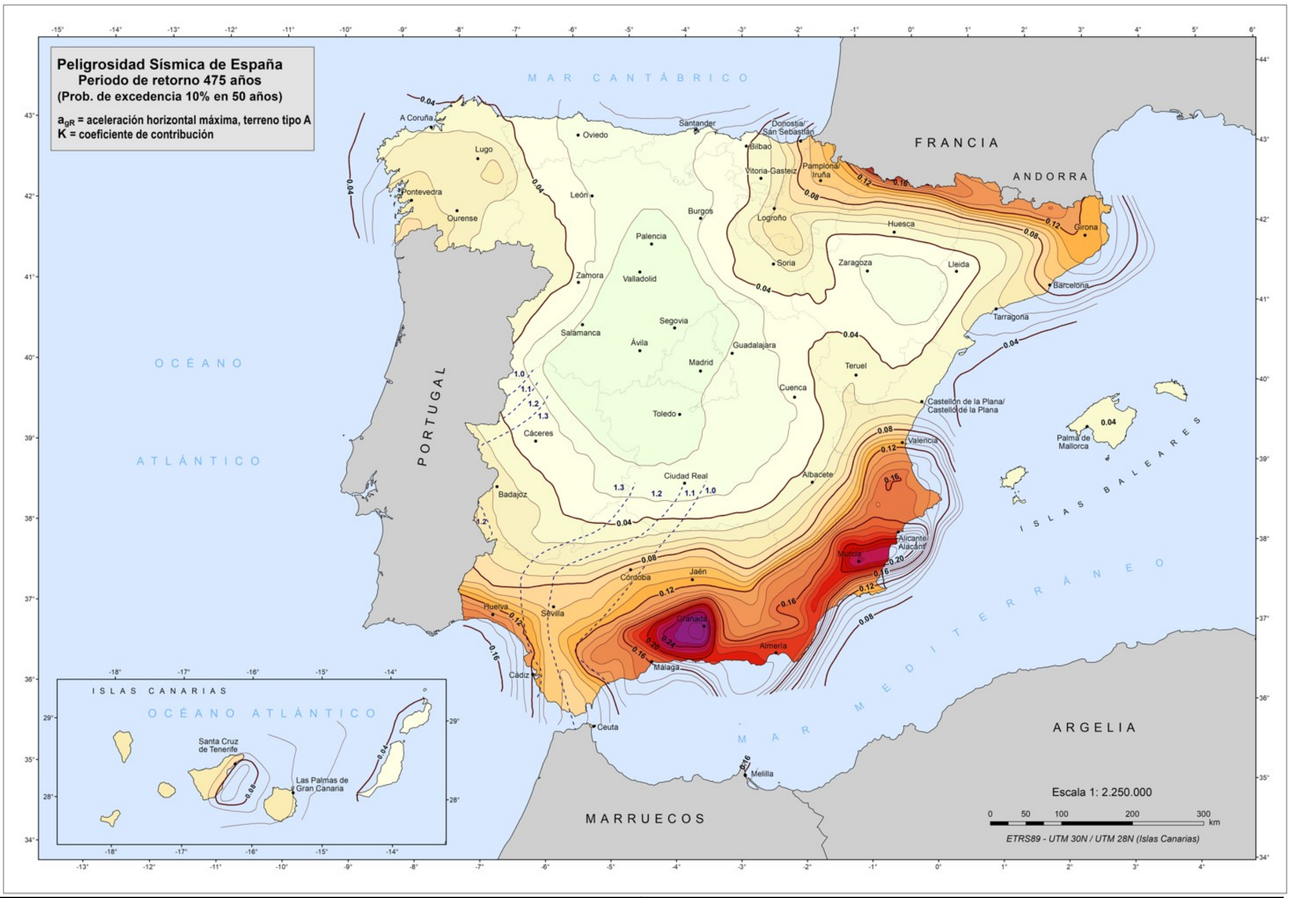
| Long. | Lat. | K | $a_{gR}$ | Long. | Lat. | K | $a_{gR}$ | Long. | Lat. | K | $a_{gR}$ | Long. | Lat. | K | $a_{gR}$ |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| -8.6 | 43.1 | 1.0 | 0.052 | -7.5 | 43.2 | 1.0 | 0.055 | -2.5 | 43.3 | 1.0 | 0.050 | -9.0 | 43.5 | 1.0 | 0.047 |
| -8.5 | 43.1 | 1.0 | 0.053 | -7.4 | 43.2 | 1.0 | 0.054 | -2.4 | 43.3 | 1.0 | 0.056 | -8.9 | 43.5 | 1.0 | 0.047 |
| -8.4 | 43.1 | 1.0 | 0.053 | -7.3 | 43.2 | 1.0 | 0.052 | -2.3 | 43.3 | 1.0 | 0.064 | -8.8 | 43.5 | 1.0 | 0.047 |
| -8.3 | 43.1 | 1.0 | 0.053 | -7.2 | 43.2 | 1.0 | 0.051 | -2.2 | 43.3 | 1.0 | 0.072 | -8.7 | 43.5 | 1.0 | 0.047 |
| -8.2 | 43.1 | 1.0 | 0.053 | -7.1 | 43.2 | 1.0 | 0.048 | -2.1 | 43.3 | 1.0 | 0.077 | -8.6 | 43.5 | 1.0 | 0.047 |
| -8.1 | 43.1 | 1.0 | 0.053 | -7.0 | 43.2 | 1.0 | 0.045 | -2.0 | 43.3 | 1.0 | 0.082 | -8.5 | 43.5 | 1.0 | 0.047 |
| -8.0 | 43.1 | 1.0 | 0.053 | -6.9 | 43.2 | 1.0 | 0.041 | -1.9 | 43.3 | 1.0 | 0.086 | -8.4 | 43.5 | 1.0 | 0.047 |
| -7.9 | 43.1 | 1.0 | 0.053 | -6.8 | 43.2 | 1.0 | 0.039 | -1.8 | 43.3 | 1.0 | 0.091 | -8.3 | 43.5 | 1.0 | 0.047 |
| -7.8 | 43.1 | 1.0 | 0.054 | -6.7 | 43.2 | 1.0 | 0.036 | -1.7 | 43.3 | 1.0 | 0.097 | -8.2 | 43.5 | 1.0 | 0.046 |
| -7.7 | 43.1 | 1.0 | 0.055 | -3.0 | 43.2 | 1.0 | 0.038 | -1.6 | 43.3 | 1.0 | 0.103 | -8.1 | 43.5 | 1.0 | 0.045 |
| -7.6 | 43.1 | 1.0 | 0.056 | -2.9 | 43.2 | 1.0 | 0.040 | -1.5 | 43.3 | 1.0 | 0.107 | -8.0 | 43.5 | 1.0 | 0.045 |
| -7.5 | 43.1 | 1.0 | 0.056 | -2.8 | 43.2 | 1.0 | 0.042 | -1.4 | 43.3 | 1.0 | 0.113 | -7.9 | 43.5 | 1.0 | 0.046 |
| -7.4 | 43.1 | 1.0 | 0.057 | -2.7 | 43.2 | 1.0 | 0.044 | -1.3 | 43.3 | 1.0 | 0.124 | -7.8 | 43.5 | 1.0 | 0.045 |
| -7.3 | 43.1 | 1.0 | 0.057 | -2.6 | 43.2 | 1.0 | 0.046 | -9.6 | 43.4 | 1.0 | 0.040 | -7.7 | 43.5 | 1.0 | 0.045 |
| -7.2 | 43.1 | 1.0 | 0.055 | -2.5 | 43.2 | 1.0 | 0.050 | -9.5 | 43.4 | 1.0 | 0.042 | -7.6 | 43.5 | 1.0 | 0.043 |
| -7.1 | 43.1 | 1.0 | 0.053 | -2.4 | 43.2 | 1.0 | 0.054 | -9.4 | 43.4 | 1.0 | 0.043 | -7.5 | 43.5 | 1.0 | 0.043 |
| -7.0 | 43.1 | 1.0 | 0.048 | -2.3 | 43.2 | 1.0 | 0.060 | -9.3 | 43.4 | 1.0 | 0.045 | -7.4 | 43.5 | 1.0 | 0.042 |
| -6.9 | 43.1 | 1.0 | 0.044 | -2.2 | 43.2 | 1.0 | 0.066 | -9.2 | 43.4 | 1.0 | 0.046 | -7.3 | 43.5 | 1.0 | 0.042 |
| -6.8 | 43.1 | 1.0 | 0.041 | -2.1 | 43.2 | 1.0 | 0.073 | -9.1 | 43.4 | 1.0 | 0.047 | -7.2 | 43.5 | 1.0 | 0.041 |
| -6.7 | 43.1 | 1.0 | 0.038 | -2.0 | 43.2 | 1.0 | 0.081 | -9.0 | 43.4 | 1.0 | 0.047 | -7.1 | 43.5 | 1.0 | 0.040 |
| -3.0 | 43.1 | 1.0 | 0.038 | -1.9 | 43.2 | 1.0 | 0.086 | -8.9 | 43.4 | 1.0 | 0.048 | -7.0 | 43.5 | 1.0 | 0.040 |
| -2.9 | 43.1 | 1.0 | 0.040 | -1.8 | 43.2 | 1.0 | 0.091 | -8.8 | 43.4 | 1.0 | 0.048 | -6.9 | 43.5 | 1.0 | 0.038 |
| -2.8 | 43.1 | 1.0 | 0.042 | -1.7 | 43.2 | 1.0 | 0.096 | -8.7 | 43.4 | 1.0 | 0.048 | -2.8 | 43.5 | 1.0 | 0.033 |
| -2.7 | 43.1 | 1.0 | 0.044 | -1.6 | 43.2 | 1.0 | 0.100 | -8.6 | 43.4 | 1.0 | 0.048 | -2.7 | 43.5 | 1.0 | 0.036 |
| -2.6 | 43.1 | 1.0 | 0.047 | -1.5 | 43.2 | 1.0 | 0.105 | -8.5 | 43.4 | 1.0 | 0.049 | -2.6 | 43.5 | 1.0 | 0.042 |
| -2.5 | 43.1 | 1.0 | 0.050 | -1.4 | 43.2 | 1.0 | 0.111 | -8.4 | 43.4 | 1.0 | 0.049 | -2.5 | 43.5 | 1.0 | 0.049 |
| -2.4 | 43.1 | 1.0 | 0.053 | -1.3 | 43.2 | 1.0 | 0.121 | -8.3 | 43.4 | 1.0 | 0.049 | -2.4 | 43.5 | 1.0 | 0.055 |
| -2.3 | 43.1 | 1.0 | 0.057 | -1.2 | 43.2 | 1.0 | 0.138 | -8.2 | 43.4 | 1.0 | 0.049 | -2.3 | 43.5 | 1.0 | 0.060 |
| -2.2 | 43.1 | 1.0 | 0.062 | -9.6 | 43.3 | 1.0 | 0.041 | -8.1 | 43.4 | 1.0 | 0.049 | -2.2 | 43.5 | 1.0 | 0.064 |
| -2.1 | 43.1 | 1.0 | 0.069 | -9.5 | 43.3 | 1.0 | 0.043 | -8.0 | 43.4 | 1.0 | 0.048 | -2.1 | 43.5 | 1.0 | 0.068 |
| -2.0 | 43.1 | 1.0 | 0.077 | -9.4 | 43.3 | 1.0 | 0.044 | -7.9 | 43.4 | 1.0 | 0.049 | -2.0 | 43.5 | 1.0 | 0.072 |
| -1.9 | 43.1 | 1.0 | 0.085 | -9.3 | 43.3 | 1.0 | 0.046 | -7.8 | 43.4 | 1.0 | 0.048 | -1.9 | 43.5 | 1.0 | 0.077 |
| -1.8 | 43.1 | 1.0 | 0.091 | -9.2 | 43.3 | 1.0 | 0.047 | -7.7 | 43.4 | 1.0 | 0.048 | -1.8 | 43.5 | 1.0 | 0.083 |
| -1.7 | 43.1 | 1.0 | 0.095 | -9.1 | 43.3 | 1.0 | 0.047 | -7.6 | 43.4 | 1.0 | 0.047 | -1.7 | 43.5 | 1.0 | 0.090 |
| -1.6 | 43.1 | 1.0 | 0.099 | -9.0 | 43.3 | 1.0 | 0.048 | -7.5 | 43.4 | 1.0 | 0.046 | -9.4 | 43.6 | 1.0 | 0.043 |
| -1.5 | 43.1 | 1.0 | 0.103 | -8.9 | 43.3 | 1.0 | 0.048 | -7.4 | 43.4 | 1.0 | 0.045 | -9.3 | 43.6 | 1.0 | 0.044 |
| -1.4 | 43.1 | 1.0 | 0.109 | -8.8 | 43.3 | 1.0 | 0.049 | -7.3 | 43.4 | 1.0 | 0.044 | -9.2 | 43.6 | 1.0 | 0.045 |
| -1.3 | 43.1 | 1.0 | 0.116 | -8.7 | 43.3 | 1.0 | 0.049 | -7.2 | 43.4 | 1.0 | 0.043 | -9.1 | 43.6 | 1.0 | 0.045 |
| -1.2 | 43.1 | 1.0 | 0.126 | -8.6 | 43.3 | 1.0 | 0.050 | -7.1 | 43.4 | 1.0 | 0.042 | -9.0 | 43.6 | 1.0 | 0.045 |
| -1.1 | 43.1 | 1.0 | 0.141 | -8.5 | 43.3 | 1.0 | 0.050 | -7.0 | 43.4 | 1.0 | 0.041 | -8.9 | 43.6 | 1.0 | 0.046 |
| -1.0 | 43.1 | 1.0 | 0.163 | -8.4 | 43.3 | 1.0 | 0.050 | -6.9 | 43.4 | 1.0 | 0.039 | -8.8 | 43.6 | 1.0 | 0.045 |
| -9.6 | 43.2 | 1.0 | 0.042 | -8.3 | 43.3 | 1.0 | 0.050 | -2.9 | 43.4 | 1.0 | 0.037 | -8.7 | 43.6 | 1.0 | 0.045 |
| -9.5 | 43.2 | 1.0 | 0.044 | -8.2 | 43.3 | 1.0 | 0.050 | -2.8 | 43.4 | 1.0 | 0.038 | -8.6 | 43.6 | 1.0 | 0.045 |
| -9.4 | 43.2 | 1.0 | 0.045 | -8.1 | 43.3 | 1.0 | 0.050 | -2.7 | 43.4 | 1.0 | 0.041 | -8.5 | 43.6 | 1.0 | 0.044 |
| -9.3 | 43.2 | 1.0 | 0.046 | -8.0 | 43.3 | 1.0 | 0.051 | -2.6 | 43.4 | 1.0 | 0.046 | -8.4 | 43.6 | 1.0 | 0.043 |
| -9.2 | 43.2 | 1.0 | 0.047 | -7.9 | 43.3 | 1.0 | 0.051 | -2.5 | 43.4 | 1.0 | 0.053 | -8.3 | 43.6 | 1.0 | 0.043 |
| -9.1 | 43.2 | 1.0 | 0.048 | -7.8 | 43.3 | 1.0 | 0.051 | -2.4 | 43.4 | 1.0 | 0.060 | -8.2 | 43.6 | 1.0 | 0.042 |
| -9.0 | 43.2 | 1.0 | 0.049 | -7.7 | 43.3 | 1.0 | 0.051 | -2.3 | 43.4 | 1.0 | 0.066 | -8.1 | 43.6 | 1.0 | 0.042 |
| -8.9 | 43.2 | 1.0 | 0.049 | -7.6 | 43.3 | 1.0 | 0.052 | -2.2 | 43.4 | 1.0 | 0.071 | -8.0 | 43.6 | 1.0 | 0.042 |
| -8.8 | 43.2 | 1.0 | 0.050 | -7.5 | 43.3 | 1.0 | 0.050 | -2.1 | 43.4 | 1.0 | 0.075 | -7.9 | 43.6 | 1.0 | 0.042 |
| -8.7 | 43.2 | 1.0 | 0.050 | -7.4 | 43.3 | 1.0 | 0.049 | -2.0 | 43.4 | 1.0 | 0.079 | -7.8 | 43.6 | 1.0 | 0.042 |
| -8.6 | 43.2 | 1.0 | 0.051 | -7.3 | 43.3 | 1.0 | 0.048 | -1.9 | 43.4 | 1.0 | 0.084 | -7.7 | 43.6 | 1.0 | 0.042 |
| -8.5 | 43.2 | 1.0 | 0.051 | -7.2 | 43.3 | 1.0 | 0.046 | -1.8 | 43.4 | 1.0 | 0.091 | -7.6 | 43.6 | 1.0 | 0.042 |
| -8.4 | 43.2 | 1.0 | 0.051 | -7.1 | 43.3 | 1.0 | 0.044 | -1.7 | 43.4 | 1.0 | 0.104 | -7.5 | 43.6 | 1.0 | 0.042 |
| -8.3 | 43.2 | 1.0 | 0.051 | -7.0 | 43.3 | 1.0 | 0.042 | -1.6 | 43.4 | 1.0 | 0.112 | -7.4 | 43.6 | 1.0 | 0.041 |
| -8.2 | 43.2 | 1.0 | 0.052 | -6.9 | 43.3 | 1.0 | 0.039 | -9.7 | 43.5 | 1.0 | 0.034 | -7.3 | 43.6 | 1.0 | 0.041 |
| -8.1 | 43.2 | 1.0 | 0.052 | -6.8 | 43.3 | 1.0 | 0.037 | -9.6 | 43.5 | 1.0 | 0.039 | -7.2 | 43.6 | 1.0 | 0.040 |
| -8.0 | 43.2 | 1.0 | 0.052 | -3.0 | 43.3 | 1.0 | 0.038 | -9.5 | 43.5 | 1.0 | 0.041 | -7.1 | 43.6 | 1.0 | 0.039 |
| -7.9 | 43.2 | 1.0 | 0.052 | -2.9 | 43.3 | 1.0 | 0.039 | -9.4 | 43.5 | 1.0 | 0.044 | -7.0 | 43.6 | 1.0 | 0.039 |
| -7.8 | 43.2 | 1.0 | 0.053 | -2.8 | 43.3 | 1.0 | 0.041 | -9.3 | 43.5 | 1.0 | 0.045 | -2.7 | 43.6 | 1.0 | 0.030 |
| -7.7 | 43.2 | 1.0 | 0.054 | -2.7 | 43.3 | 1.0 | 0.043 | -9.2 | 43.5 | 1.0 | 0.046 | -2.6 | 43.6 | 1.0 | 0.033 |
| -7.6 | 43.2 | 1.0 | 0.054 | -2.6 | 43.3 | 1.0 | 0.046 | -9.1 | 43.5 | 1.0 | 0.046 | -2.5 | 43.6 | 1.0 | 0.037 |

| Long. | Lat. | K | $a_{gR}$ | Long. | Lat. | K | $a_{gR}$ | Long. | Lat. | K | $a_{gR}$ | Long. | Lat. | K | $a_{gR}$ |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| -2.4 | 43.6 | 1.0 | 0.041 | -15.5 | 27.8 | 1.0 | 0.060 | -15.4 | 28.2 | 1.0 | 0.059 | -13.8 | 28.6 | 1.0 | 0.040 |
| -2.3 | 43.6 | 1.0 | 0.044 | -15.4 | 27.8 | 1.0 | 0.057 | -14.4 | 28.2 | 1.0 | 0.040 | -13.7 | 28.6 | 1.0 | 0.040 |
| -2.2 | 43.6 | 1.0 | 0.049 | -15.3 | 27.8 | 1.0 | 0.052 | -14.3 | 28.2 | 1.0 | 0.040 | -18.1 | 28.7 | 1.0 | 0.068 |
| -2.1 | 43.6 | 1.0 | 0.053 | -17.9 | 27.9 | 1.0 | 0.068 | -14.2 | 28.2 | 1.0 | 0.040 | -18.0 | 28.7 | 1.0 | 0.068 |
| -2.0 | 43.6 | 1.0 | 0.058 | -15.8 | 27.9 | 1.0 | 0.065 | -14.1 | 28.2 | 1.0 | 0.040 | -17.9 | 28.7 | 1.0 | 0.068 |
| -9.0 | 43.7 | 1.0 | 0.044 | -15.7 | 27.9 | 1.0 | 0.065 | -14.0 | 28.2 | 1.0 | 0.040 | -17.8 | 28.7 | 1.0 | 0.068 |
| -8.9 | 43.7 | 1.0 | 0.043 | -15.6 | 27.9 | 1.0 | 0.064 | -13.9 | 28.2 | 1.0 | 0.040 | -17.7 | 28.7 | 1.0 | 0.068 |
| -8.8 | 43.7 | 1.0 | 0.042 | -15.5 | 27.9 | 1.0 | 0.062 | -16.9 | 28.3 | 1.0 | 0.067 | -14.1 | 28.7 | 1.0 | 0.040 |
| -8.7 | 43.7 | 1.0 | 0.041 | -15.4 | 27.9 | 1.0 | 0.059 | -16.8 | 28.3 | 1.0 | 0.067 | -14.0 | 28.7 | 1.0 | 0.040 |
| -8.6 | 43.7 | 1.0 | 0.041 | -15.3 | 27.9 | 1.0 | 0.053 | -16.7 | 28.3 | 1.0 | 0.067 | -13.9 | 28.7 | 1.0 | 0.040 |
| -8.5 | 43.7 | 1.0 | 0.040 | -17.3 | 28.0 | 1.0 | 0.068 | -16.6 | 28.3 | 1.0 | 0.068 | -13.8 | 28.7 | 1.0 | 0.040 |
| -8.4 | 43.7 | 1.0 | 0.039 | -17.2 | 28.0 | 1.0 | 0.068 | -16.5 | 28.3 | 1.0 | 0.070 | -13.7 | 28.7 | 1.0 | 0.040 |
| -8.3 | 43.7 | 1.0 | 0.039 | -17.1 | 28.0 | 1.0 | 0.068 | -16.4 | 28.3 | 1.0 | 0.075 | -18.0 | 28.8 | 1.0 | 0.068 |
| -8.2 | 43.7 | 1.0 | 0.040 | -16.8 | 28.0 | 1.0 | 0.068 | -16.3 | 28.3 | 1.0 | 0.088 | -17.9 | 28.8 | 1.0 | 0.067 |
| -8.1 | 43.7 | 1.0 | 0.040 | -16.7 | 28.0 | 1.0 | 0.068 | -14.3 | 28.3 | 1.0 | 0.040 | -17.8 | 28.8 | 1.0 | 0.067 |
| -8.0 | 43.7 | 1.0 | 0.040 | -16.6 | 28.0 | 1.0 | 0.071 | -14.2 | 28.3 | 1.0 | 0.040 | -17.7 | 28.8 | 1.0 | 0.067 |
| -7.9 | 43.7 | 1.0 | 0.041 | -16.5 | 28.0 | 1.0 | 0.082 | -14.1 | 28.3 | 1.0 | 0.040 | -14.0 | 28.8 | 1.0 | 0.040 |
| -7.8 | 43.7 | 1.0 | 0.041 | -16.4 | 28.0 | 1.0 | 0.094 | -14.0 | 28.3 | 1.0 | 0.040 | -13.9 | 28.8 | 1.0 | 0.040 |
| -7.7 | 43.7 | 1.0 | 0.041 | -15.9 | 28.0 | 1.0 | 0.069 | -13.9 | 28.3 | 1.0 | 0.040 | -13.8 | 28.8 | 1.0 | 0.040 |
| -7.6 | 43.7 | 1.0 | 0.041 | -15.8 | 28.0 | 1.0 | 0.067 | -13.8 | 28.3 | 1.0 | 0.040 | -13.7 | 28.8 | 1.0 | 0.040 |
| -7.5 | 43.7 | 1.0 | 0.040 | -15.7 | 28.0 | 1.0 | 0.066 | -17.9 | 28.4 | 1.0 | 0.068 | -13.6 | 28.8 | 1.0 | 0.040 |
| -7.4 | 43.7 | 1.0 | 0.040 | -15.6 | 28.0 | 1.0 | 0.065 | -17.8 | 28.4 | 1.0 | 0.068 | -17.9 | 28.9 | 1.0 | 0.067 |
| -7.3 | 43.7 | 1.0 | 0.040 | -15.5 | 28.0 | 1.0 | 0.063 | -17.0 | 28.4 | 1.0 | 0.068 | -17.8 | 28.9 | 1.0 | 0.067 |
| -7.2 | 43.7 | 1.0 | 0.039 | -15.4 | 28.0 | 1.0 | 0.059 | -16.9 | 28.4 | 1.0 | 0.067 | -17.7 | 28.9 | 1.0 | 0.067 |
| -7.1 | 43.7 | 1.0 | 0.039 | -15.3 | 28.0 | 1.0 | 0.054 | -16.8 | 28.4 | 1.0 | 0.067 | -13.9 | 28.9 | 1.0 | 0.040 |
| -2.5 | 43.7 | 1.0 | 0.029 | -14.5 | 28.0 | 1.0 | 0.040 | -16.7 | 28.4 | 1.0 | 0.067 | -13.8 | 28.9 | 1.0 | 0.040 |
| -2.4 | 43.7 | 1.0 | 0.031 | -14.4 | 28.0 | 1.0 | 0.040 | -16.6 | 28.4 | 1.0 | 0.067 | -13.7 | 28.9 | 1.0 | 0.040 |
| -2.3 | 43.7 | 1.0 | 0.034 | -14.3 | 28.0 | 1.0 | 0.040 | -16.5 | 28.4 | 1.0 | 0.068 | -13.6 | 28.9 | 1.0 | 0.040 |
| -2.2 | 43.7 | 1.0 | 0.037 | -14.2 | 28.0 | 1.0 | 0.040 | -16.4 | 28.4 | 1.0 | 0.070 | -13.5 | 28.9 | 1.0 | 0.040 |
| -8.6 | 43.8 | 1.0 | 0.037 | -17.3 | 28.1 | 1.0 | 0.068 | -16.3 | 28.4 | 1.0 | 0.078 | -13.9 | 29.0 | 1.0 | 0.040 |
| -8.5 | 43.8 | 1.0 | 0.037 | -17.2 | 28.1 | 1.0 | 0.068 | -16.2 | 28.4 | 1.0 | 0.090 | -13.8 | 29.0 | 1.0 | 0.040 |
| -8.4 | 43.8 | 1.0 | 0.037 | -17.1 | 28.1 | 1.0 | 0.068 | -16.1 | 28.4 | 1.0 | 0.096 | -13.7 | 29.0 | 1.0 | 0.040 |
| -8.3 | 43.8 | 1.0 | 0.037 | -16.8 | 28.1 | 1.0 | 0.068 | -14.2 | 28.4 | 1.0 | 0.040 | -13.6 | 29.0 | 1.0 | 0.040 |
| -8.2 | 43.8 | 1.0 | 0.038 | -16.7 | 28.1 | 1.0 | 0.068 | -14.1 | 28.4 | 1.0 | 0.040 | -13.5 | 29.0 | 1.0 | 0.040 |
| -8.1 | 43.8 | 1.0 | 0.039 | -16.6 | 28.1 | 1.0 | 0.071 | -14.0 | 28.4 | 1.0 | 0.040 | -13.4 | 29.0 | 1.0 | 0.040 |
| -8.0 | 43.8 | 1.0 | 0.039 | -16.5 | 28.1 | 1.0 | 0.081 | -13.9 | 28.4 | 1.0 | 0.040 | -13.9 | 29.1 | 1.0 | 0.040 |
| -7.9 | 43.8 | 1.0 | 0.040 | -16.4 | 28.1 | 1.0 | 0.094 | -13.8 | 28.4 | 1.0 | 0.040 | -13.8 | 29.1 | 1.0 | 0.040 |
| -7.8 | 43.8 | 1.0 | 0.040 | -15.8 | 28.1 | 1.0 | 0.068 | -17.9 | 28.5 | 1.0 | 0.068 | -13.7 | 29.1 | 1.0 | 0.040 |
| -7.7 | 43.8 | 1.0 | 0.040 | -15.7 | 28.1 | 1.0 | 0.066 | -17.8 | 28.5 | 1.0 | 0.068 | -13.6 | 29.1 | 1.0 | 0.040 |
| -7.6 | 43.8 | 1.0 | 0.040 | -15.6 | 28.1 | 1.0 | 0.065 | -17.7 | 28.5 | 1.0 | 0.068 | -13.5 | 29.1 | 1.0 | 0.040 |
| -7.5 | 43.8 | 1.0 | 0.039 | -15.5 | 28.1 | 1.0 | 0.063 | -16.5 | 28.5 | 1.0 | 0.067 | -13.4 | 29.1 | 1.0 | 0.040 |
| -7.4 | 43.8 | 1.0 | 0.039 | -15.4 | 28.1 | 1.0 | 0.059 | -16.4 | 28.5 | 1.0 | 0.068 | -13.7 | 29.2 | 1.0 | 0.040 |
| -7.3 | 43.8 | 1.0 | 0.039 | -15.3 | 28.1 | 1.0 | 0.054 | -16.3 | 28.5 | 1.0 | 0.071 | -13.6 | 29.2 | 1.0 | 0.040 |
| | | | | -14.5 | 28.1 | 1.0 | 0.040 | -16.2 | 28.5 | 1.0 | 0.078 | -13.5 | 29.2 | 1.0 | 0.040 |
| -18.1 | 27.6 | 1.0 | 0.067 | -14.4 | 28.1 | 1.0 | 0.040 | -16.1 | 28.5 | 1.0 | 0.083 | -13.4 | 29.2 | 1.0 | 0.040 |
| -18.0 | 27.6 | 1.0 | 0.067 | -14.3 | 28.1 | 1.0 | 0.040 | -14.2 | 28.5 | 1.0 | 0.040 | -13.3 | 29.2 | 1.0 | 0.040 |
| -17.9 | 27.6 | 1.0 | 0.067 | -14.2 | 28.1 | 1.0 | 0.040 | -14.1 | 28.5 | 1.0 | 0.040 | -13.5 | 29.3 | 1.0 | 0.040 |
| -18.2 | 27.7 | 1.0 | 0.067 | -14.1 | 28.1 | 1.0 | 0.040 | -14.0 | 28.5 | 1.0 | 0.040 | -13.4 | 29.3 | 1.0 | 0.040 |
| -18.1 | 27.7 | 1.0 | 0.067 | -14.0 | 28.1 | 1.0 | 0.040 | -13.9 | 28.5 | 1.0 | 0.040 | | | | |
| -18.0 | 27.7 | 1.0 | 0.067 | -17.3 | 28.2 | 1.0 | 0.068 | -13.8 | 28.5 | 1.0 | 0.040 | | | | |
| -17.9 | 27.7 | 1.0 | 0.067 | -17.2 | 28.2 | 1.0 | 0.068 | -13.7 | 28.5 | 1.0 | 0.040 | | | | |
| -15.7 | 27.7 | 1.0 | 0.059 | -17.1 | 28.2 | 1.0 | 0.068 | -18.0 | 28.6 | 1.0 | 0.068 | | | | |
| -15.6 | 27.7 | 1.0 | 0.058 | -16.9 | 28.2 | 1.0 | 0.068 | -17.9 | 28.6 | 1.0 | 0.068 | | | | |
| -15.5 | 27.7 | 1.0 | 0.056 | -16.8 | 28.2 | 1.0 | 0.067 | -17.8 | 28.6 | 1.0 | 0.068 | | | | |
| -15.4 | 27.7 | 1.0 | 0.052 | -16.7 | 28.2 | 1.0 | 0.068 | -17.7 | 28.6 | 1.0 | 0.068 | | | | |
| -18.2 | 27.8 | 1.0 | 0.068 | -16.6 | 28.2 | 1.0 | 0.069 | -16.4 | 28.6 | 1.0 | 0.067 | | | | |
| -18.1 | 27.8 | 1.0 | 0.068 | -16.5 | 28.2 | 1.0 | 0.074 | -16.3 | 28.6 | 1.0 | 0.068 | | | | |
| -18.0 | 27.8 | 1.0 | 0.068 | -16.4 | 28.2 | 1.0 | 0.085 | -16.2 | 28.6 | 1.0 | 0.069 | | | | |
| -17.9 | 27.8 | 1.0 | 0.068 | -16.3 | 28.2 | 1.0 | 0.096 | -16.1 | 28.6 | 1.0 | 0.070 | | | | |
| -15.8 | 27.8 | 1.0 | 0.064 | -15.7 | 28.2 | 1.0 | 0.067 | -14.1 | 28.6 | 1.0 | 0.040 | | | | |
| -15.7 | 27.8 | 1.0 | 0.063 | -15.6 | 28.2 | 1.0 | 0.065 | -14.0 | 28.6 | 1.0 | 0.040 | | | | |
| -15.6 | 27.8 | 1.0 | 0.062 | -15.5 | 28.2 | 1.0 | 0.063 | -13.9 | 28.6 | 1.0 | 0.040 | | | | |

# Appendix F

## Seismic hazard map, return period Tr=475 years

Seismic hazard map, obtained from the hazard data, peak reference horizontal acceleration, $a_{gR}$, in soil type A and parameter K, from Appendix E.




| | |
|---|---|
| Peligrosidad Sísmica de España<br>Periodo de retorno 475 años<br>(Prob. de excedencia 10% en 50 años)<br>$a_{gR}$ = aceleración horizontal máxima, terreno tipo A<br>K = coeficiente de contribución | Spanish Seismic Hazards<br>Return period 475 years<br>(Prob. of exceedance 10% in 50 years)<br>$a_{gR}$ = maximum horizontal acceleration, terrain type A<br>K = contribution coefficient |
| Escala 1:2.250.000 | Scale 1:2,250,000 |
| *ETRS89 - UTM 30N / UTM 28N (Islas Canarias)* | *ETRS89 – UTM 30N / UTM 28N (Canary Islands)* |

**Appendix G**

**List of UNE standards**

## G.1 Introduction

The paragraphs of this Seismic Resistance Standard establish a series of verifications of the processes included in its scope and of the conformity of the products which, in certain cases, are referred to UNE-EN or UNE-EN ISO standards. These reference standards are included here.

For the purposes of this Seismic Resistance Standard, it should be understood that the UNE-EN or UNE-EN ISO standards cited in the same, always refer to the versions listed in this Appendix, except in the case of harmonised UNE-EN standards that are a transposition of EN standards whose reference has been published in the Official Journal of the European Union, (in the framework of the application of Regulation No 305/2011 of the European Parliament and of the Council of 9 March 2011 laying down harmonised conditions for the marketing of construction products), in which case the citation shall relate to the latest Commission Communication containing such reference. In the case of standards referenced in harmonised standards, the version included in the harmonised UNE-EN standards cited above should be applied.

## G.2 Reference and consultation standards

### G.2.1 UNE/EN standards

| | |
|---|---|
| UNE-EN 772-1:2011+A1:2016 | *Methods of test for masonry units. Part 1: Determination of compressive strength.* |
| UNE-EN 1337-2:2006 | *Structural bearings. Part 2: Sliding elements.* |
| UNE-EN 1337-3:2005 | *Structural bearings. Part 3: Elastomeric bearings.* |
| UNE-EN 1991-4:2011<br>UNE-EN 1991-4:2011/AC:2013 | *Eurocode 1: Actions on structures. Part 4: Silos and tanks.* |
| UNE-EN 1997-1:2016 | *Eurocode 7: Geotechnical design Part 1: General rules.* |
| UNE-EN 12512:2002+A1:2006 | *Timber structures. Test methods. Cyclic testing of joints made with mechanical fasteners.* |
| UNE-EN 13084-2:2008 | *Free-standing chimneys. Part 2: Concrete chimneys.* |
| UNE-EN 13084-7:2013 | *Free-standing chimneys. Part 7: Product specifications of cylindrical steel fabrications for use in single wall steel chimneys and steel liners.* |
| UNE-EN 15129:2019 | *Anti-seismic devices* |

### G.2.2 UNE-EN ISO standards

| | |
|---|---|
| UNE-EN ISO 17892-4:2019 | *Geotechnical investigation and testing. Laboratory testing of soil. Part 4: Determination of particle size distribution.* |
| UNE-EN ISO 17892-12:2019 | *Geotechnical investigation and testing. Laboratory testing of soil. Part 12: Determination of liquid and plastic limits.* |
| UNE-EN ISO 22476-3:2006 | *Geotechnical investigation and testing. Field testing. Part 3: Standard penetration test.* |
| UNE-EN ISO 22476-12:2010 | *Geotechnical investigation and testing. Field testing. Part 12: Mechanical cone penetration test (CPTM).* |

# APPENDIX 2

# Design of seismically resistant structures

# Bridges

## 1 Introduction

### 1.1 Object and field of application

#### 1.1.1 Purpose and scope of Annex 2

(1) The scope of the Seismic Resistance Standard is defined in section **1.1.1** of Annex 1, and the scope of this Annex is defined in this section. The other Annexes of the Seismic Resistance Standard are listed in section **1.1.3** of Annex 1.

(2) Within the scope set out in Annex 1, this Annex contains the performance requirements, compliance criteria and application rules to be considered, applicable to the design of seismically resistant bridges.

(3) This Annex first covers the seismic design of bridges where the horizontal seismic actions are resisted mainly by the bending of the piers or by the abutments; i.e., bridges consisting of vertical or quasi-vertical pier systems supporting the traffic deck superstructure. It is also applicable to the seismic design of arch or cable-stayed bridges, although the provisions of this Annex should not be considered to cover these cases completely.

(4) Suspension bridges, timber and factory bridges, movable and floating bridges are not included in the scope of this Annex.

(5) This Annex contains only those provisions which, together with the relevant requirements contained in other parts of the Seismic Resistance Standard, as well as in the other applicable regulations, shall be respected for the design of bridges in seismic regions. In cases of low seismicity, simplified design criteria may be established (see point **(1)** of section **2.3.7**).

(6) This Annex covers the following topics:

- Basic requirements and compliance criteria.
- Seismic actions.
- Analysis.
- Resistance testing.
- Construction details.

This Annex also includes a specific chapter on seismic isolation, with provisions covering the application of this method of seismic protection to bridges.

(7) Appendix G contains the rules for the calculation of stresses using the capacity design criteria.

(8) Appendix J contains rules concerning variations in the design properties of seismic isolators and how these variations may be taken into account in the calculations.

NOTE 1 Appendix A provides information about the probabilities of occurrence of the reference earthquake, as well as recommendations for the selection of the applicable calculated seismic action during the construction phase.

NOTE 2 Appendix B provides information on the relationship between displacement ductility and curvature ductility of plastic hinges in concrete piers.

NOTE 3 Appendix C provides information for the estimation of the effective rigidity of ductile reinforced concrete members.

NOTE 4 Appendix D provides information for modelling and analysis of the spatial variability of seismic motion.

NOTE 5 Appendix E provides information on probable material properties and deformation capacities of plastic hinges for non-linear analyses.

NOTE 6 Appendix F provides information and advice on the added mass of entrained water for submerged piers.

NOTE 7 Appendix H provides advice and information on non-linear static (incremental thrust) analysis.

NOTE 8 Appendix JJ provides information on the λ coefficients of the current types of isolators.

NOTE 9 Appendix K contains test requirements for validation of the design properties of seismic isolators.

NOTE 10 Appendices A, B, C, D, E, F, H, JJ and K of this Annex are not regulatory.

## 1.2 Standards for reference and consultation

(1) The specifications in section**1.2** of Annex 1 apply.

## 1.3 Assumptions

(1) The specifications in section**1.3** of Annex 1 apply.

## 1.4 International system of units (I.S.)

(1) The specifications in section**1.4** of Annex 1 apply.

## 1.5 Terms and definitions

### 1.5.1 General considerations

(1) For the purposes of this Annex, the following definitions apply.

### 1.5.2 Common terms

(1) The terms and definitions of section **1.4** of Annex 18 of the Structural Code apply.

### 1.5.3 Other terms used in this Annex

**dimensioning by capacity**
Calculation procedure used when designing structures with ductile behaviour in order to ensure the hierarchy of strengths of the different structural elements necessary to achieve the intended configuration of plastic hinges and to avoid brittle failure modes.

**ductile elements**
Elements capable of dissipating energy through the formation of plastic hinges.

**ductile structure**
Structure which, under the action of strong seismic movements, is capable of dissipating a significant amount of the energy generated by the earthquake by the formation of an intended configuration of plastic hinges, or by other mechanisms.

**limited ductile behaviour**
Seismic behaviour of bridges under calculated seismic action, without significant energy dissipation in the plastic hinges.

**active connection**
Connection established by seismic couplings.

**seismic isolation**
The provision of special isolation devices to bridge structures in order to reduce the seismic response (forces and/or displacements).

**spatial variability (of seismic action)**
A situation where the seismic motion is different at different bridge supports and therefore the seismic action cannot be based on the usual characterisation of the motion at a single point.

**seismic behaviour**
Behaviour of the bridge under the calculated earthquake which, depending on the characteristics of the overall force-displacement relationship of the structure, may be either ductile or limited ductility/essentially elastic.

**seismic couplings**
Movement limiting elements through which all or part of the seismic action can be transmitted. Used in combination with supporting devices, they can be arranged with an appropriate clearance so that they are only activated in the event that the design value of the seismic displacement is exceeded.

**minimum delivery distance**
A safety measure consisting of the establishment of a minimum distance between the inner edge of the supported element and the outer edge of the supporting element. The minimum delivery is intended to ensure that the support maintains its function in the event of extreme seismic displacements.

**seismic displacement calculation value**
Displacements induced by the calculated seismic action.

**total calculation displacement in the calculated seismic situation**
Displacement used to determine the appropriate joint clearances or widths for the protection of

critical or most important elements. It includes the calculated value of the seismic displacement, the displacement due to the long-term effect of permanent and quasi-permanent actions, and an appropriate fraction of the displacement due to thermal motions.

## 1.6 Symbols

### 1.6.1 General considerations

(1) The symbols indicated in section **1.5** of Annex 18 to the Structural Code apply. For symbols relating to materials, as well as for symbols not specifically related to earthquakes, the provisions of the specific regulations in force apply (see point **(1)** of section **1.2** of Annex 1).

(2) The symbols which appear most frequently in this Annex are listed and defined in the following sections. For ease of use, other symbols relating to seismic actions are defined in the text where they are used.

### 1.6.2 Other symbols used in Chapters 2 and 3 of Annex 2

$d_E$ seismic displacement calculation value (due only to calculated seismic action)
$d_{Ee}$ seismic displacement calculated by linear analysis
$d_G$ long-term deformation due to permanent and quasi-permanent actions
$d_g$ calculation value of ground displacement, according to section **3.2.2.4** of Annex 1
$d_i$ ground displacement of assembly B at the location of support *i*
$d_{ri}$ displacement of the floor at the location of support *i*, with respect to a reference support 0
$d_T$ displacement due to thermal action
$d_u$ last displacement (with non-linear behaviour)
$d_y$ displacement corresponding to the yield stress
$A_{Ed}$ calculated seismic action
$F_{Rd}$ calculated seismic action resisting force value
$L_g$ distance from which the seismic ground motions can be considered to be totally independent
$L_i$ distance between support *i* and reference support 0
$L_{i-1,i}$ distance between two consecutive supports, *i*-1 and *i*
$R_i$ reaction force at the base of the pier *i*
$S_a$ average response spectrum at the site
$S_i$ site response spectrum
$T_{eff}$ effective period of the isolation system
$\gamma_I$ importance coefficient
$\Delta d_i$ displacement of the ground at the location of an intermediate support *i*, with respect to the adjacent supports *i*-1 and *i*+1
$\mu_d$ displacement ductility coefficient
$\psi_2$ combination coefficient for the quasi-permanent value of thermal action

### 1.6.3 Other symbols used in Chapter 4 of Annex 2

$d_a$ average of the transverse displacements of all pier crowns under the transverse seismic action, or under the action of a transverse load of similar distribution
$d_i$ displacement of the *i*th nodal point
$d_m$ asymptotic value of the spectrum for the *m*th motion in the long-period range, expressed in terms of displacements
$e$ $e_a + e_d$

$e_a$ accidental mass eccentricity (= 0.03 $L$ or 0.03 $B$)
$e_d$ additional eccentricity reflecting the dynamic effect of the simultaneous translational and rotational vibrations (= 0.05 $L$ or 0.05 $B$)
$e_o$ theoretical eccentricity
$g$ gravitational acceleration
$h$ edge of the cross-section in the direction of deflection of the plastic hinge
$k_m$ effect of the $m$th independent movement
$r_i$ reduction coefficient of the required local force reduction in the ductile element $i$
$r_{min.}$ minimum value of $r_i$
$r_{max.}$ maximum value of $r_i$
$A_{Ed}$ seismic action calculation
$A_{Ex}$ seismic action in the $x$ direction
$A_{Ey}$ seismic action in the $y$ direction
$A_{Ez}$ seismic action in the $z$ direction
$B$ width of the deck
$E$ maximum probable value of the effect of an action
$E_i$ response in mode $i$
$F$ horizontal load calculated according to the fundamental mode method
$G$ total effective weight of the structure, equal to the weight of the deck plus the weights of the upper half of the piers
$G_i$ concentrated weight at the $i$th nodal point
$K$ system rigidity
$L$ total length of the continuous deck
$L_s$ distance between the plastic hinge and the point of zero moment
$M$ total mass
$M_{Ed,i}$ maximum value of the calculated moment for the calculated seismic situation, at the predicted location of the plastic hinge of ductile member $i$
$M_{Rd,i}$ last calculated moment resisting force of the plastic hinge of ductile member $i$
$M_i$ equivalent static moment with respect to the vertical axis passing through the centre of gravity of the deck
$Q_{k,l}$ characteristic value of traffic load
$R_d$ strength calculation value
$S_d(\mathrm{T})$ spectral acceleration of the calculated response spectrum
$T$ period of the fundamental mode of vibration in the considered direction
$X$ horizontal longitudinal axis of the bridge
$Y$ horizontal transverse axis of the bridge
$Z$ vertical axis
$\alpha_s$ shear rate of the pier
$\Delta_d$ maximum difference between the transverse displacements of the crowns of all piers, resulting from the transverse seismic action or from the action of a transverse load of similar distribution
$\eta_k$ normalised axial force (= $N_{Ed}/(A_c f_{ck})$)
$\theta_{p,d}$ calculation value of the rotational capacity of the plastic hinge
$\theta_{p,E}$ plastic hinge rotational demand
$\xi$ viscous damping index or ratio
$\psi_{2,i}$ combination coefficient for the quasi-permanent value of the variable action $i$

### 1.6.4 Other symbols used in Chapter 5 of Annex 2

$d_{Ed}$ relative transverse displacement of the ends of the ductile element under consideration
$f_{ck}$ characteristic value of the concrete strength

| | |
|---|---|
| $f_{ctd}$ | estimated value of the tensile strength of the concrete |
| $f_{sd}$ | reduced reinforcement stress to limit cracking |
| $f_{sy}$ | value calculation of the strength of the reinforcement of the nodes |
| $z_b$ | internal mechanical arm of the end sections of the beams |
| $z_c$ | internal mechanical arm of the plastic hinge cross-section of the column |
| $A_C$ ($V_C$, $M_C$, $N_C$) | effects obtained from the application of capacity dimensioning |
| $A_c$ | transverse area of concrete |
| $A_{Ed}$ | calculated seismic action (seismic action only) |
| $A_{Sd}$ | action corresponding to the calculated seismic action situation |
| $A_{sx}$ | transverse area of horizontal reinforcement at the node |
| $A_{sz}$ | transverse area of vertical reinforcement at the node |
| $E_d$ | calculated stress or calculated value of the effect of action in the calculated seismic situation |
| $G_k$ | characteristic value of the permanent load |
| $M_o$ | moment of over-resistance (reserve resistance) of the plastic hinge |
| $M_{Ed}$ | design value of the bending moment at the design seismic situation |
| $M_{Rd}$ | design value of the bending resistance of the ductile section |
| $N_{Ed}$ | axial load in design seismic situation |
| $N_{cG}$ | axial load on the column due to permanent or quasi-permanent actions in the calculated seismic situation |
| $N_{jz}$ | vertical axial load on a node |
| $Q_{1k}$ | characteristic value of traffic load |
| $Q_2$ | quasi-permanent value of long-term actions |
| $P_k$ | characteristic value of the prestress load after all losses |
| $R_d$ | calculated value of the section resistance |
| $R_{df}$ | calculated value of the maximum frictional or frictional force of a sliding support device |
| $T_{Rc}$ | resultant force of the tensioned reinforcement of the column |
| $V_{E,d}$ | calculated value of shear force |
| $V_{jx}$ | calculated value of the horizontal shear force at a node |
| $V_{jz}$ | calculated value of the vertical shear force at a node |
| $V_{1bC}$ | shear force of beam adjacent to the tensile face of the column |
| $\gamma_M$ | partial material safety coefficient |
| $\gamma_o$ | coefficient of over-resistance (reserve strength) |
| $\gamma_{of}$ | amplification coefficient for friction due to ageing effects |
| $\gamma_{Bd,Bdl}$ | additional partial safety factor against fragile failure modes |
| $\rho_x$ | geometric value of horizontal reinforcement at a node |
| $\rho_y$ | geometric value of closed abutments, in the transverse direction of the connecting panel (perpendicular to the plane of actions) |
| $\rho_z$ | geometric value of vertical reinforcement at a node |
| $\psi_{21}$ | combination coefficient |
| $\Delta A_{sx}$ | area of the horizontal reinforcement of a node, located outside the node |
| $\Delta A_{xz}$ | area of the vertical reinforcement of a node, located outside the node |

### 1.6.5 Other symbols used in Chapter 6 of Annex 2

| | |
|---|---|
| $a_g$ | calculated value of ground acceleration on a type A terrain (see **3.2.2.2** of Annex 1) |
| $b$ | centre-to-centre dimension of the cross-section of the confined core, perpendicular to the direction of confinement under consideration, measured to the axes of the perimeter frames |
| $b_{min.}$ | smallest dimension of the concrete core |
| $d_{bL}$ | diameter of longitudinal reinforcement |

$d_{eg}$ effective displacement (relative between abutment and deck), due to the spatial variation of the seismic displacement of the soil
$d_{es}$ effective seismic displacement of the support, due to deformation of the structure
$d_g$ calculated value of maximum ground displacement, as specified in section **3.2.2.4** of Annex 1
$f_t$ tensile strength
$f_y$ yield strength of steel
$f_{ys}$ yield strength of longitudinal reinforcement
$f_{yt}$ yield strength of forks
$l_m$ minimum length of the support that is capable of safely transmitting the vertical reaction
$l_{ov}$ minimum overlap length
$s$ spacing of spiral frames or spiral pitch in the direction of the element axis
$s_L$ maximum separation (longitudinal)
$s_T$ separation between the trusses or additional cross ties, in the direction of the axis of the member
$s_t$ transverse separation
$v_g$ calculated ground velocity
$v_s$ ground propagation velocity of transverse waves S, for small shear deformations
$A_c$ concrete gross section area
$A_{cc}$ area of the confined concrete core of the section
$A_{sp}$ transverse area of a truss or spiral
$A_{sw}$ total transverse area of trusses and forks in the direction transverse to the direction of confinement
$A_t$ transversal area of a fork
$D_i$ inner diameter
$D_{sp}$ diameter of the spiral or circular frame
$E_d$ total earth thrust acting on the abutment under seismic conditions as defined in Annex 5
$F_{Rd}$ calculated value of the resistance
$L_h$ calculated length of plastic hinges
$L_{eff}$ effective length of the deck
$Q_d$ weight of the section of deck attached to a pier or abutment, or the lesser of the weights of two sections of deck, one on each side of an intermediate separation joint
$S$ soil coefficient as described in section **3.2.2.2** of Annex 1
$T_C$ corner period of the elastic response spectrum as described in section **3.2.2.2** of Annex 1
$a_g$ calculation value of ground acceleration in a type-A terrain
$\gamma_I$ importance coefficient
$\gamma_s$ transverse soil seismic deformation in free field
$\delta$ parameter quotient function $f_t / f_y$
$\mu_\Phi$ coefficient of ductility in curvature required
$\Sigma A_s$ sum of the transverse areas of the longitudinal reinforcement embraced by the fork
$\rho_L$ geometric value of the longitudinal reinforcement
$\rho_w$ geometric value of the transverse reinforcement
$\omega_{wd}$ mechanical value of the confining reinforcement.

### 1.6.6 Other symbols used in Chapter 7 and Appendices J, JJ and K to Annex 2

$a_g$ calculation value of ground acceleration in a type-A terrain
$a_{gR}$ maximum reference ground acceleration on a type A terrain
$d$ calculated displacement value
$d_b$ isolator displacement
$d_{bd}$ calculated value of the isolator displacement, corresponding to the calculated

| | |
|---|---|
| | displacement of the isolation system $d_{cd}$ |
| $d_{bi}$ | isolator displacement *i* |
| $d_{bi,a}$ | calculated value of the increased displacement of isolator *i* |
| $d_{bi,d}$ | calculated value of the displacement of isolator *i* |
| $d_{cd}$ | calculated value of the displacement calculation of the isolation system |
| $d_{cf}$ | calculated value of the isolation system displacement, obtained from the fundamental mode method analysis |
| $d_{d,m}$ | displacement of the centre of rigidity, derived from the analysis |
| $d_{G,i}$ | initial displacement at isolator *i* |
| $d_{id}$ | displacement of the superstructure at the location of the substructure and isolator *i* |
| $d_m$ | displacement capacity of the isolation system |
| $d_{max.}$ | maximum displacement |
| $d_{m,i}$ | maximum total displacement on each isolator *i* |
| $d_n$, $d_p$ | respectively, minimum negative and minimum positive displacements obtained in the tests |
| $d_{rm}$ | residual displacement of the isolation system |
| $d_y$ | displacement corresponding to the yield stress |
| $e_x$ | eccentricity in the longitudinal direction of the bridge |
| $r$ | radius of gyration of the mass of the deck with respect to the vertical axis passing through its centre of gravity |
| sign $(\dot{d}_b)$ | vector sign speed $\dot{d}_b$ |
| $t_e$ | thickness of all elastomeric layers |
| $v$ | displacement velocity of a viscous isolator |
| $v_{max.}$ | maximum displacement velocity of a viscous isolator |
| $x_i$, $y_i$ | planimetric coordinates of pier *i* |
| $A_b$ | effective transverse area of an elastomer or elastomeric support |
| $E_D$ | energy dissipated per cycle of amplitude equal to the calculated value of the displacement of the isolation system $d_{cd}$ |
| $E_{Di}$ | energy dissipated in isolator *i*, per amplitude cycle equal to the calculated value of the displacement of the isolation system $d_{cd}$ |
| $E_E$ | calculated seismic loads |
| $E_{EA}$ | internal seismic loads obtained from the analysis |
| $F_{max.}$ | maximum force corresponding to the calculated displacement value |
| $F_n$, $F_p$ | respectively, minimum negative and maximum positive forces measured in tests of devices with hysteresis or frictional behaviour, or negative and positive forces, corresponding to $d_n$ and $d_p$, measured in tests of devices with viscoelastic behaviour |
| $F_y$ | force corresponding to the yield strength, obtained by monotonic loading |
| $F_0$ | force corresponding to the zero displacement, obtained by cyclic loading |
| $G_b$ | transverse modulus of elasticity of the elastomer or elastomeric support |
| $G_g$ | conventional apparent transverse modulus of elasticity of the elastomer, in accordance with UNE-EN 1337-3 |
| *HDRB* | High Dumping Rubber Bearing |
| $H_i$ | pier height *i* |
| $K_{bi}$ | effective isolator rigidity *i* |
| $K_e$ | elastic rigidity of a bilinear hysteretic insulator subjected to a monotonic load |
| $K_L$ | rigidity of the lead core of a lead-rubber device |
| $K_p$ | post-elastic rigidity of a bilinear hysteretic isolator |
| $K_{eff}$ | effective rigidity of the isolation system in the main horizontal direction considered, for a displacement equal to the calculated value of the displacement $d_{cd}$ |
| $K_{eff,i}$ | mixed rigidity of the isolators and the corresponding pier *i* |

| | |
|---|---|
| $K_{fi}$ | rotational rigidity of the pier foundation *i* |
| $K_R$ | rigidity of the rubber or lead-rubber support device |
| $K_{ri}$ | rotational rigidity of the foundation of pier *i* |
| $K_{si}$ | rigidity at displacement of the pier *i* |
| $K_{ti}$ | translational rigidity of pier foundation *i* |
| $K_{xi}$, $K_{yi}$ | effective mixed rigidity of an isolation device and pier *i* |
| *LRB* | Lead Rubber Bearing |
| $M_d$ | mass of the superstructure |
| $N_{Sd}$ | axial load acting through the isolator |
| *PTFE* | Polytetrafluorothylene |
| $Q_G$ | permanent axial load of the isolator |
| $R_b$ | radius of the spherical sliding surface |
| $S$ | soil coefficient of the elastic response spectrum, according to section **3.2.2.2** of Annex 1 |
| $T_C$, $T_D$ | corner periods of the elastic response spectrum, according to section **7.4.1(1)** of this Annex ~~standard~~ and section **3.2.2.2** of Annex 1 |
| $T_{eff}$ | effective period of the isolation system |
| $T_{min.\,b}$ | minimum temperature of the bearing apparatus for seismic calculation |
| $V_d$ | maximum shear stress transmitted through the isolation interface |
| $V_f$ | maximum shear stress estimated by the fundamental mode design method |
| UBDP | Upper Bound Design Properties of Isolators |
| LBDP | Lower Bound Design Properties of Isolators |
| $\alpha_b$ | exponent of the velocity of the viscous damper |
| $\gamma_I$ | bridge importance coefficient |
| $\Delta F_{Ed}$ | additional vertical load due to seismic effects of overturning |
| $\Delta F_m$ | load increment between displacements $d_m/2$ and $d_m$ |
| $\mu_d$ | dynamic friction coefficient |
| $\xi$ | equivalent viscous critical damping ratio |
| $\xi_b$ | contribution of the isolators to effective damping |
| $\xi_{eff}$ | effective damping of the isolation system |
| $\psi_{fi}$ | combination coefficient |

# 2 Basic requirements and compliance criteria

## 2.1 Calculated seismic action

(1) The design philosophy of this Annex is to achieve, with appropriate reliability for the calculated seismic action ($A_{Ed}$), the collapse-free requirement described in section **2.2.2**, as well as in Annex 1, section **2.1(1)**.

(2) Unless otherwise specified in this Annex, the elastic response spectrum corresponding to the calculated seismic action is applied in accordance with sections **3.2.2.2**, **3.2.2.3** and **3.2.2.4** of Annex 1. For the application of the equivalent linear method described in section **4.1.6** (using the behaviour coefficient $q$), the spectrum must be the design spectrum, according to section **3.2.2.5** of Annex 1.

(3) The calculated seismic action, $A_{Ed}$, is expressed as a function of:

a) the reference seismic action, $A_{Ek}$, associated with a reference exceedance probability, $P_{NCR}$, of 10% in 50 years or a reference return period, $T_{NCR}$ = 475 years; and

b) the significance factor $\gamma_I$, in order to take into account the different degrees of reliability (see

Annex 18 of the Structural Code).

$$A_{\mathrm{Ed}} = \gamma_{\mathrm{I}} A_{\mathrm{Ek}} \tag{2.1}$$

NOTE 1 See section 2.1 and section 3.2.1(3) of Annex 1.

NOTE 2 Appendix A provides information on the reference seismic action and on the choice of the calculated seismic action for the construction phase.

(4) Bridges are classified according to their importance in terms of the consequences of their failure for human life and for the maintenance of communications, especially in the immediate post-earthquake period, as well as the economic consequences of collapse.

The classes of importance of bridges are defined according to the criteria of seismic actions according to the intended use of the structure and the damage that may be caused by its destruction. For these purposes only, the following categories are differentiated:

I) Importance class I: Bridges of moderate importance
This includes bridges which, in the opinion of the competent public administration, have a negligible probability that their destruction could cause casualties, interrupt a primary service or cause significant economic damage to third parties.

Railway bridges are not considered to be included in this class.

II) Importance class II: Bridges of normal importance
Bridges whose destruction is likely to cause casualties or to interrupt a service necessary for the community or to cause significant economic loss, provided that it is not an essential service, nor is it likely to cause catastrophic effects, as judged by the competent public administration.

This class includes all railway bridges which are not considered to be of major importance.

III) Importance class III: Bridges of major importance
Bridges whose destruction is likely to interrupt an essential service after the earthquake or to result in catastrophic effects, as judged by the competent public administration.

In the case of railway bridges, they are defined as follows:

- Bridges located on the main access lines to large urban centres, including all those corresponding to the main suburban network of these centres.

- Bridges located on lines with heavy traffic connecting important population centres. In general, these will be the lines integrated in the main network (type A), with a speed equal to or greater than 200 km/h.

- Bridges located on high-speed lines.

- Bridges located on lines constituting the only railway connection between two regions or large urban centres.

- Bridges supporting other vital services for the population (electricity and water mains, etc.).

NOTE Importance classes I, II and III correspond approximately to consequence classes CC1, CC2 and CC3, respectively, as defined in section **B.3.1** of Annex 18 of the Structural Code.

(5) The importance classes are characterised by different importance coefficients $\gamma_I$ as described in paragraph **2.1(3)**, as well as in section **2.1(3)** of Annex 1.

(6) The importance coefficient $\gamma_I$ = 1.0 is associated with a seismic action having the reference return period specified in paragraph **2.1(3)**, as well as in section **3.2.1** of Annex 1.

Importance class I (bridges of moderate importance): $\gamma_I$ set by the competent public administration.
Importance class II (bridges of normal importance): $\gamma_I$ =1
Importance class III (bridges of major importance): $\gamma_I$ =1.3.

## 2.2 Basic requirements

### 2.2.1 General considerations

(1) The project must aim to meet the following two basic requirements.

### 2.2.2 Non-collapse, no collapse (latest limit state)

(1) After the occurrence of the calculated seismic action, the bridge must maintain its structural integrity and adequate residual strength, although some parts of the bridge may suffer considerable damage.

(2) In the pier, bending plastification (i.e. the formation of plastic hinges) is allowed to occur in specific sections. Where the bridge does not have any seismic isolation, such bending plastification is generally necessary in areas of high seismicity in order to reduce the calculated seismic action to a level that corresponds to a reasonable increase in additional construction cost compared to that of a bridge not designed to resist earthquakes.

(3) In general, the bridge deck shall be sized to avoid damage other than local damage to secondary components such as expansion joints, continuity slabs (see section **2.3.2.2(4)**) or parapets.

(4) Where the calculated seismic action has a significant probability of exceedance during the service life of the bridge, the design should seek a structure that will tolerate the damage. Those parts of the bridge that are susceptible to damage because of their contribution to energy dissipation under the calculated seismic action shall be sized so that, after its occurrence, the bridge can be used for emergency traffic and are readily repairable.

(5) The calculated seismic action may be considered as an accidental action when the calculated seismic action, in accordance with paragraph **1.4.3.5** and section **4.1.1(2)** of Annex 18 of the Structural Code, has a low probability of being exceeded during the design life of the bridge. In any case, the requirements of **(3)** and **(4)** above must be fulfilled.

### 2.2.3 Limitation of damage (limited status of service)

(1) A seismic action with a high probability of occurrence can only cause minor damage to secondary elements and those parts of the bridge designed to contribute to energy dissipation. All other parts of the bridge shall remain undamaged.

## 2.3 Compliance criteria

### 2.3.1 General considerations

(1) In order to meet the basic requirements set out in section **2.2**, the project must satisfy the criteria listed in the following sections. In general, the criteria that explicitly seek to satisfy the requirement of absence of collapse (**2.2.2**) cover at the same time, implicitly, the requirement of damage limitation (**2.2.3**).

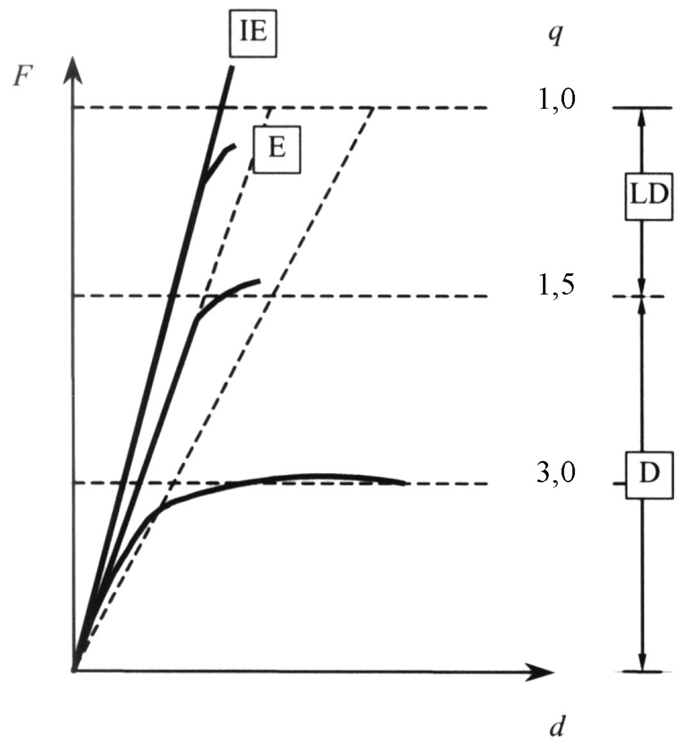
(2) Conformity with the criteria set out in this Annex implies that all the basic requirements of **2.2** can be considered to be satisfied.

(3) The compliance criteria depend on the behaviour under the calculated seismic action for which the bridge is designed. This performance may be chosen in accordance with the provisions of section **2.3.2**.

### 2.3.2 Predicted seismic behaviour

#### 2.3.2.1 General considerations

(1) The bridge must be designed so that its behaviour under calculated seismic action is either ductile or limited ductility/essentially elastic, depending on the seismicity at the site, whether a seismic isolation system is adopted in the design, or any other constraints that may prevail. This behaviour (ductile or limited ductility) is characterised by the overall load-displacement relationship of the structure shown schematically in Figure 2.1 (see also Table 4.1).




Legend:

*q* Behaviour coefficient

IE Ideally elastic

E Essentially elastic

LD Limited ductility

D Ductile

**Figure 2.1 – Seismic behaviour**

### 2.3.2.2 Ductile behaviour

(1) In areas of moderate to high seismicity it is usually preferable, for both economic and safety reasons, to design the bridge for ductile behaviour, i.e. to provide it with reliable means to dissipate a significant amount of the energy input in the event of severe earthquakes. This is achieved either by designing it to form an intended configuration of plastic hinges or by using isolation devices, in accordance with the provisions of Chapter **7**. The following points in this section refer to ductile behaviour achieved by means of plastic bending spherical plain bearings.

(2) Bridges with ductile behaviour must be designed in such a way that, through the formation of plastic bending hinges, a stable partial or total plastic mechanism can develop in the structure. These hinges are normally formed in the pier and act as primary energy dissipation elements.

(3) The location of the plastic hinges should, as far as possible, be chosen at points accessible for inspection and repair.

(4) The bridge deck must remain in the elastic range. However, the formation of plastic hinges (in bending with respect to the transverse axis) in flexible ductile concrete slabs is permitted, provided that the upper continuity of the slabs is ensured between spans of simply supported precast concrete beams.

(5) Plastic hinges must not be formed in reinforced concrete cross-sections where the standard axial force, $\eta_k$, as defined in section **5.3(4)**, is greater than 0.6.

(6) This Annex does not contain rules on the ductility of prestressed or post-tensioned elements. Consequently, the formation of plastic hinges in such members under calculated seismic action shall be avoided.

(7) Plastic bending hinges need not necessarily form in every pier. However, the optimum post-elastic seismic behaviour of a bridge is achieved if plastic hinges develop approximately simultaneously in as many pier elements as possible.

(8) The ability of the structure to form plastic hinges is necessary to ensure energy dissipation and consequently ductile behaviour (see section **4.1.6(2)**).

NOTE The deformation of bridges supported exclusively by simple low damping elastomeric bearings is predominantly elastic and does not generally lead to ductile behaviour (see section **4.1.6(11)** ).

(9) The overall load-displacement relationship shall exhibit a significant plateau for the load above the yield point, and shall ensure energy dissipation by hysteresis in at least five cycles of inelastic deformation (see Figures 2.1, 2.2 and 2.3).

NOTE Elastomeric bearings used on some supports, combined with a monolithic support on other piers, may result in the resisting force increasing as displacements increase after plastic hinges have formed in the other bearing elements. However, the rate of increase of the resisting force should be appreciably reduced after the formation of the plastic hinges.

(10) Bearing members (piers or abutments) connected to the deck by means of sliding or flexible bearing elements (sliding bearings or flexible elastomeric bearings) shall, in general, remain within the elastic range.

#### 2.3.2.3 Limited ductility behaviour

(1) In structures with limited ductility behaviour, a plastification zone with a significant reduction of the secant stiffness need not appear under calculated seismic action. In terms of load-displacement characteristics, the formation of a load plateau is not required even if the deviation from the ideal elastic behaviour provides some energy dissipation by hysteresis. Such behaviour corresponds to a value of the behaviour coefficient $q \leq 1.5$ and should be referred to in this Annex as "limited ductility".

NOTE Values of $q$ in the range $1 \leq q \leq 1.5$ are mainly attributed to the inherent margin between the calculated strength and the probable strength, for the design seismic situation.

(2) For bridges where the seismic response may be dominated by the effects of higher modes of vibration (e.g. cable-stayed bridges) or where the construction details of the plastic hinges arranged to provide ductility may be unreliable (e.g. due to high axial force or low shear-span shear-span ratio), a performance coefficient $q = 1$, corresponding to elastic behaviour, is recommended.

### 2.3.3 Resistance tests

(1) For bridges designed for ductile behaviour, the areas where plastic hinges are expected to form must be checked for adequate flexural strength to resist the effects of the calculated seismic action, as specified in section **5.5**. The shear resistance of the plastic hinges, as well as the shear and bending resistance of the other zones, must be designed to resist the 'capacity design effects' specified in section **2.3.4** (see also **5.3**).

(2) For bridges designed for limited ductility behaviour, it shall be verified that all sections have adequate strength to resist the effects of calculated seismic action as described in section **5.5** (see **5.6.2**).

### 2.3.4 Dimensioning by capacity

(1) Bridges with ductile behaviour must be designed by capacity to ensure that an appropriate strength hierarchy is established in the various structural elements. This is to ensure that the intended plastic hinge configuration will be achieved and that brittle failure modes will be avoided.

(2) Compliance with **(1)** above is to be achieved by using the "capacity sizing effects" in the design of all elements that are intended to remain in the elastic mode to resist all brittle failure modes. Such effects are obtained, as specified in section **5.3**, by establishing the equilibrium conditions of the designed plastic mechanism, when all bending spherical plain bearings have developed an upper bound of their bending resistance (overstrength).

(3) For bridges with limited ductility behaviour, the application of the capacity sizing method is not required.

### 2.3.5 Provisions for achieving ductility

#### 2.3.5.1 General requirements

(1) The intended plastic hinges must be provided with adequate ductility to ensure the overall ductility required by the structure.

NOTE The definitions of global and local ductility, given in sections **2.3.5.2** and **2.3.5.3**, are intended to establish the theoretical basis of ductile behaviour. In general, they are not required for the practical verification of ductility, which is carried out in accordance with section **2.3.5.4**.

### 2.3.5.2 Global ductility

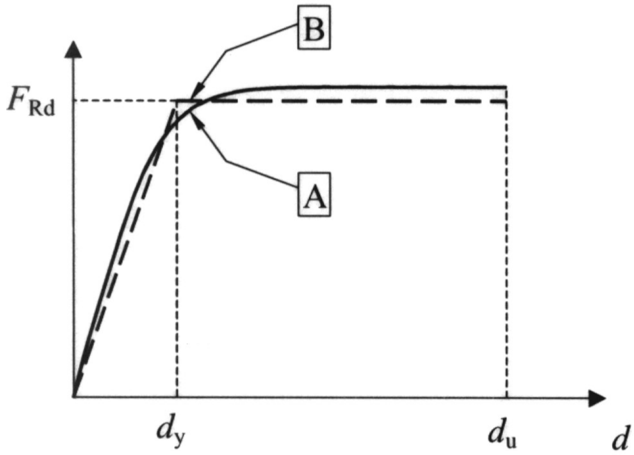
(1) The calculated value of the ductility coefficient of the structure (displacement ductility coefficient) is defined, from an equivalent one degree of freedom system with an ideal perfect elastoplastic force-displacement diagram as shown in Figure 2.2, as the quotient of the latest limit state displacement ($d_u$) and the elastic limit state displacement ($d_y$), both measured at the centre of gravity: i.e., $\mu_d = d_u/d_y$.

(2) When an equivalent linear analysis is carried out, it is assumed that the yield strength of the perfect global elastoplastic load-displacement diagram is equal to the calculated value of the resistance, $F_{Rd}$. The displacement corresponding to the yield point, which defines the elastic branch, is chosen to be as close as possible to the design force-displacement curve (for a monotonic load).

(3) The ultimate displacement, $d_u$, is defined as the maximum displacement that satisfies the following condition: The structure shall be capable of resisting at least five complete cycles of deformation up to the latest displacement:

- without initiation of failure of the confining reinforcement in reinforced concrete sections, or local buckling effects in steel sections; and

- without a drop in the strength of the ductile steel members, or a drop of more than 20% in the ultimate strength of the ductile reinforced concrete members (see Figure 2.3).

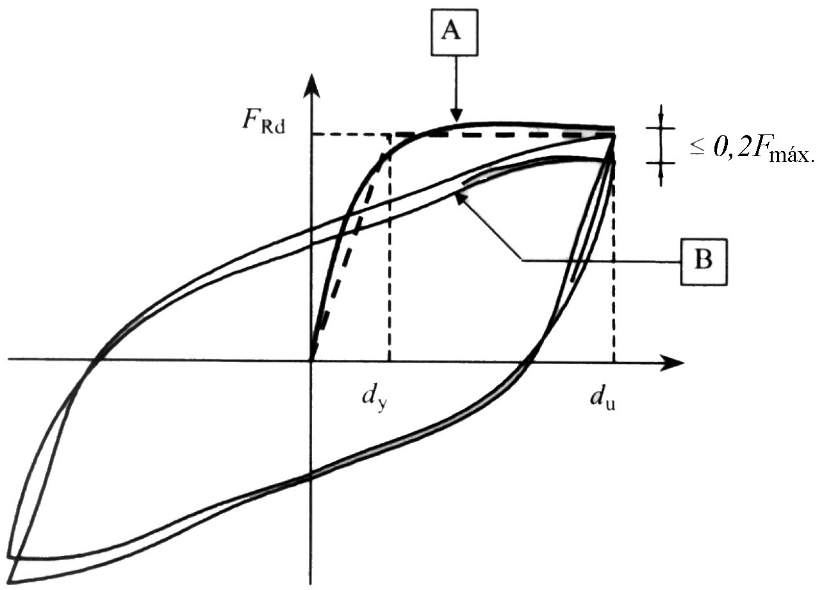



Legend:

A Calculation

B Elastoplastic

**Figure 2.2 – Overall force-displacement diagram (monotonic load)**




Legend:

A Monotonic load

B Fifth cycle

**Figure 2.3 – Force-displacement cycles (reinforced concrete)**

### 2.3.5.3 Local ductility in plastic hinges

(1) The overall ductility of the structure depends on the local ductility available at the plastic hinges (see Figure 2.4). This can be expressed in terms of the ductility coefficient at cross-section curvatures:

$$\mu_\Phi = \Phi_u \,/\, \Phi_y \tag{2.2}$$

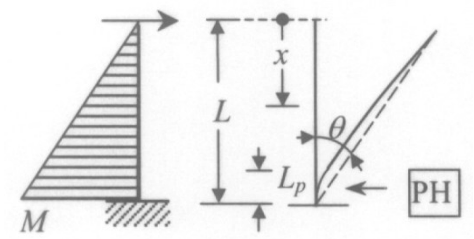
or in terms of the rotational ductility coefficient for the ultimate curvature at the end where the hinge is formed, which depends on the plastic rotational capacity, $\theta_{p,u} = \theta_u - \theta_y$, of the hinge:

$$\mu_\theta = \theta_u \,/\, \theta_y = 1 + (\theta_u - \theta_y)\,/\,\theta_y = 1 + \theta_{p,u}\,/\,\theta_y \tag{2.3}$$

The amplitude of rotation corresponding to a length L is given by the chord rotation, defined as the angle between the end section of the plastic hinge and the zero moment section, as shown in Figure 2.4.

NOTE 1 For concrete elements, the relationship between $\theta_p$, $\Phi_u$, $\Phi_y$, $L$ and $L_p$ is given by equation (E.16b) in section **E 3.2**.

NOTE 2 The length, $L_p$, of the plastic hinges for concrete members shall be obtained as a function of the geometry and other characteristics of the member under consideration, by means of equation (E.19) given in section **E3.2(5)**.




Legend:
PH Plastic hinge

$$\theta = \frac{1}{L}\int_0^L \Phi\, x\, dx$$

**Figure 2.4 – Rotation of the line**

(2) In the above expressions, the ultimate deflections shall be in accordance with those given in section **2.3.5.2(3)**.

NOTE Appendix B gives for a simple case the relationship between the ductility in bending of a plastic hinge and the global ductility coefficient in displacements. This relationship is not acceptable for ductility testing.

### 2.3.5.4 Ductility testing

(1) Compliance with the specific rules detailed in Chapter 6 can be considered as ensuring the availability of adequate local ductility and global ductility.

(2) When a non-linear static or dynamic analysis is carried out, the rotational ductility demands must be checked against the ductility capacities of the plastic hinges (see **4.2.4.4**).

(3) For bridges with limited ductility behaviour, the provisions of section **6.5** shall be applied.

### 2.3.6 Joints. Displacement control. Construction details

#### 2.3.6.1 Effective rigidity. Calculated seismic displacement

(1) Where equivalent linear analysis methods are applied, the stiffness of each element must be chosen to correspond to its secant stiffness for the maximum calculated stresses under the calculated seismic action. For elements containing plastic hinges, this corresponds to the secant stiffness at the theoretical yield stress (see Figure 2.5).

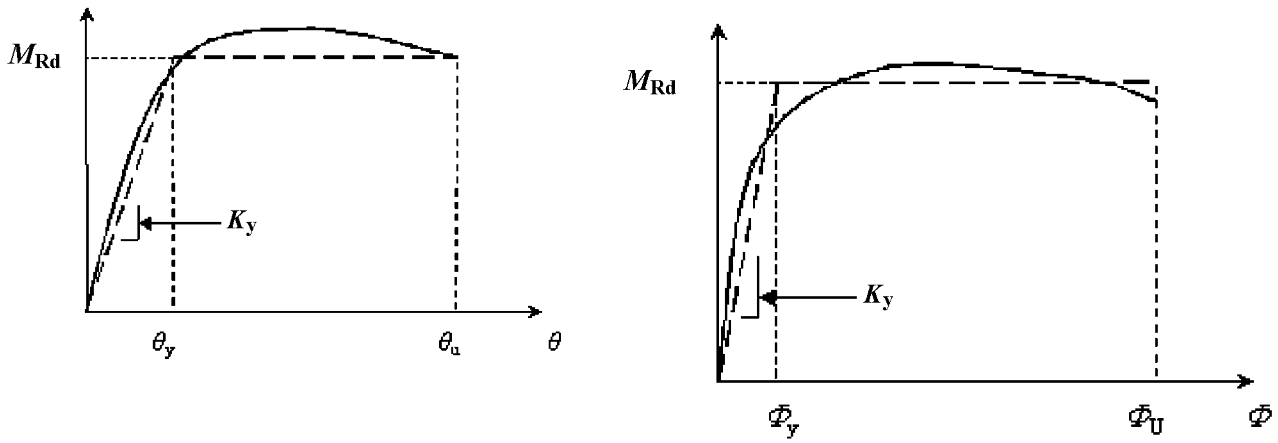



**Figure 2.5 - Moment-deflection diagrams for plastic hinges (left: moment-rotation relationship of a plastic hinge for structural steel; right: moment-curvature relationship in a cross-section of the hinge for reinforced concrete)**

(2) For reinforced concrete bridge elements designed for ductile behaviour, and unless a more accurate method of estimation is used, the effective bending stiffness to be considered for the calculated seismic action in a linear analysis (static or dynamic) can be estimated as follows.

- For reinforced concrete piers, a value obtained on the basis of the secant stiffness at the theoretical yield point.

- For reinforced or prestressed concrete decks, the stiffness of the uncracked concrete cross-sections.

NOTE Appendix C gives indications for the estimation of the effective stiffness of reinforced concrete members.

(3) For bridges designed for limited ductility behaviour, either the rules of **(2)** may be applied, or the bending stiffness of the uncracked concrete sections may be taken for the whole structure.

(4) For both ductile and limited ductility bridges, the significant reduction in the torsional stiffness of the concrete decks compared to the torsional stiffness of the uncracked deck must be taken into account. Unless a more precise calculation is made, the following percentages of the torsional stiffness of the uncracked cross-section can be used:

- for open sections or slabs: the torsional rigidity can be ignored;

- for prestressed caissons: 50% of the rigidity of the uncracked gross section;

- for reinforced concrete caissons: 30% of the rigidity of the uncracked gross section.

(5) For both ductile and limited ductility bridges, the displacements obtained from an analysis carried out in accordance with **(2)** and **(3)** shall be multiplied by the quotient of (a) the bending stiffness of the member used in the analysis and (b) the value of the bending stiffness corresponding to the stress level resulting from the analysis.

NOTE It should be noted that in case of an equivalent linear analysis (see section **4.1.6(1)**), an overestimation of the effective stiffness produces results that are on the safe side with respect to the effects of seismic action. In such a case, it is only necessary to correct the displacements obtained from the analysis, based on the bending stiffness corresponding to the resulting moment level. On the other hand, if the initially assumed effective stiffness is significantly less than that corresponding to the actual stresses obtained in the analysis, the analysis must be repeated using a better approximation of the effective stiffness.

(6) If calculated by a linear seismic analysis based on the design response spectrum, defined in accordance with the provisions of section **3.2.2.5** of Annex 1, the design seismic displacements, $d_E$, must be derived from the displacements, $d_{Ee}$, obtained in that analysis, as follows:

$$d_E = \pm\, \eta \mu_d d_{Ee} \tag{2.4}$$

where

$\eta$ is the damping correction coefficient specified in section **3.2.2.2(3)** of Annex 1, calculated from the values of the viscous damping ratio, $\xi$, given in section **4.1.3(1)**.

(7) When the displacements $d_{Ee}$ are derived from a linear elastic analysis based on the elastic response spectrum defined in accordance with the provisions of section **3.2.2.2** of Annex 1 ($q$ = 1.0), the calculated value of the displacement, $d_E$, must be taken equal to $d_{Ee}$.

(8) The coefficient of ductility at displacements should be assumed to be equal to the following values:

- when the fundamental period, $T$, in the horizontal direction considered is $T \geq T_o = 1.25T_C$, where $T_C$ is the corner period defined in accordance with the provisions of section **3.2.2.2** of Annex 1, then:

$$\mu_d = q \tag{2.5}$$

- si $T < T_o$, then

$$\mu_d = (q - 1)\frac{T_o}{T} + 1 \leq 5q - 4 \tag{2.6}$$

where $q$ is the value of the performance coefficient considered in the analysis from which the value of $d_{Ee.}$ is derived.

NOTE Expression (2.6) provides a smooth transition between the "equal displacement" rule applicable for $T \geq T_o$ and the short period span (not usual in the case of bridges), where the assumption of a low value of $q$ is convenient. For very small periods ($T$ < 0.033 s), $q$ = 1 (see also section **4.1.6(9)**) should be considered, resulting in $\mu_d$ = 1.

(9) When using non-linear time domain analysis, the deformation characteristics of the plasticised elements must approximate their actual post-elastic behaviour, both with respect to the loading and unloading branches of the hysteresis cycles and to potential degradation effects (see **4.2.4.4**).

### 2.3.6.2 Joints.

(1) The joints between the supporting and supported elements must be designed to ensure structural integrity and to avoid misalignment in case of very large seismic displacements.

(2) Unless otherwise specified in this Annex, bracing, connectors and anchorage devices to prevent uplift, used to ensure structural integrity, shall be designed in accordance with the criteria for capacity design (see **5.3**, **6.6.2.1**, **6.6.3.1** and **6.6.3.2**).

(3) At movable connections of new bridges, appropriate delivery lengths shall be provided between the supporting and supported elements in order to avoid uplift at the support line (see **6.6.4**).

(4) When rehabilitating existing bridges, an active connection between the load-bearing and supported elements (see sections **6.6.1(3)** and **6.6.3.1(1)**) may be used as an alternative to the provision of delivery lengths.

### 2.3.6.3 Displacement control. Construction details

(1) In addition to ensuring the overall ductility demand, the structural and non-structural details of the bridge and its components must be arranged so that their performance is adapted to the displacements corresponding to the calculated seismic situation.

(2) Gaps or voids must be provided to protect critical or major structural elements. The width of these gaps must be appropriate to the total value of the displacement corresponding to the calculated seismic situation, $d_{Ee}$, determined as follows:

$$d_{Ed} = d_E + d_G + \psi_2\, d_T \qquad (2.7)$$

where the different displacements in the above expression must be combined with the most unfavourable sign and have the following meaning:

$d_E$ is the calculated value of the seismic displacement, determined in accordance with section **2.3.6.1**;

$d_G$ is the long-term displacement due to permanent and quasi-permanent actions (e.g. post-tensioning, shrinkage and creep in concrete slabs);

$d_T$ is the displacement due to thermal action; and

$\psi_2$ = 0.5 is the combination coefficient for the quasi-permanent value of thermal action.

Where significant, second order effects must be taken into account in the determination of the total displacement value for the calculated seismic situation.

(3) The relative design seismic displacement between two independent sections of a bridge, $d_E$, can be estimated as the square root of the sum of the squares of the values of the calculated value of the seismic displacement determined for each section in accordance with section **2.3.6.1**.

(4) Large shocks, caused by unplanned impacts between the main structural members, must be prevented by ductile/resilient elements or by special energy absorbing devices (buffers). These elements must have a clearance or play at least equal to the total value of the displacement corresponding to the calculated seismic situation, $d_{Ed}$.

(5) Construction details of secondary structural elements (e.g. expansion joints of deck and sidewalls or abutment laps) that are expected to be damaged as a result of the calculated seismic action must be designed for a foreseeable failure mode and to allow for permanent repairs. The clearances shall be set at appropriate percentages of the calculated value of the seismic displacement and movements due to thermal actions, $p_E$ and $p_T$, respectively, after allowing for any long-term creep and shrinkage effects, so as to avoid damage caused by frequent earthquakes. The appropriate values of these percentages can be selected on the basis of a cost-effectiveness analysis of the measures taken to prevent damage.

In the absence of an explicit optimisation study, the following values should be adopted:
$p_E = 0.4$ (for the calculated value of the seismic displacement); $p_T = 0.5$ (for movement due to thermal action).

NOTE At railway bridge joints, differential transverse displacements may have to be either avoided or limited to appropriate values in order to prevent derailments.

### 2.3.7 Simplified criteria

(1) Simplified criteria for low seismicity can be established.

NOTE 1 Low seismicity cases are considered to be those where the $a_g{\cdot}S$ product is not greater than 0.1 g (0.98 m/s$^2$). See section **3.2.1(4)** of Annex 1.

NOTE 2 In areas of low seismicity and for bridges of class I or II importance and with a length of less than 15 m, the calculation of seismic actions is permitted by assimilating them to the equivalent static force corresponding to a uniform horizontal acceleration equal to $a_g{\cdot}S$. In this case the construction provisions shall be those relating to limited ductility.

## 2.4 Design of the project

(1) It is important to consider the implications of seismic action during the conceptual stage of bridge design, even for cases of low to moderate seismicity.

(2) In cases of low seismicity, the type of expected seismic behaviour of the bridge must be decided (see **2.3.2**). If limited ductility (or essentially elastic) behaviour is chosen, the simplified criteria according to section **2.3.7** can be applied.

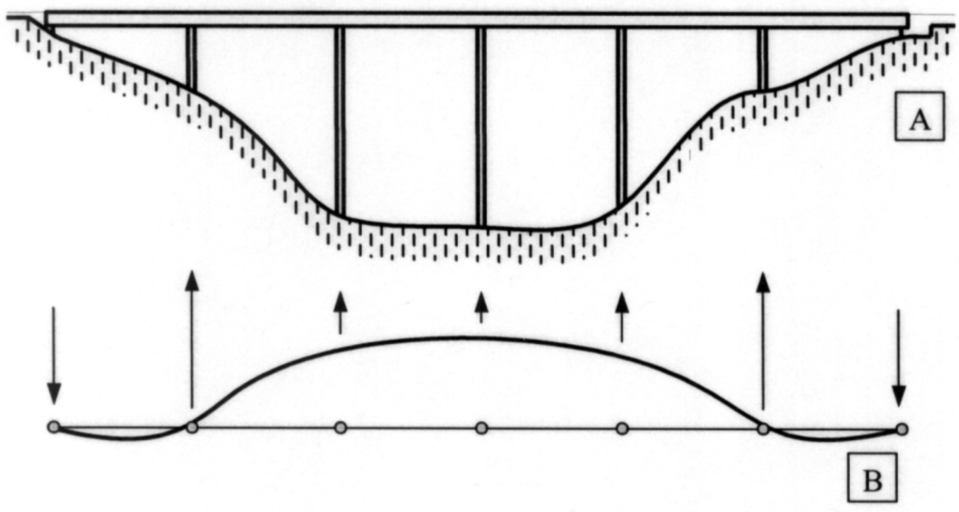
(3) In cases of moderate or high seismicity, it is generally desirable to choose ductile behaviour. Its application should be decided either by the adoption of a reliable plastic mechanism or by the use of seismic isolation at the base and/or energy dissipation devices. Where ductile behaviour is chosen, the provisions of **(4)** to **(8)** shall be observed.

(4) The number of supporting elements (piers and abutments) to be used to resist the seismic forces in the longitudinal and transverse directions must be decided. In general, bridges with a continuous deck perform better under seismic actions than bridges with many expansion joints. Optimal seismic

behaviour in the post-elastic range is achieved if the plastic hinges develop at approximately the same time in as many piers as possible. However, the number of piles resistant to seismic action may have to be smaller than the total number of piers. This is achieved by the use of sliding or flexible supports between the deck and some piers in the longitudinal direction, in order to reduce the stresses caused by forced or imposed deformations of the deck due to thermal, shrinkage and other non-seismic actions.

(5) A balance must be maintained between the strength and flexibility requirements of horizontal supports. High flexibility reduces the magnitude of lateral forces induced by calculated seismic action, but increases movement at joints and movable supports, and may result in high second order effects.

(6) In the case of bridges with a continuous deck where the transverse rigidity of the abutments and adjacent piers is very high compared to that of the other piers (as may occur in steep valleys), it may be preferable to use transverse sliding or elastomeric bearings on the short piers or on the abutments, in order to avoid an unfavourable distribution of the transverse seismic action between the piers and abutments, as shown in Figure 2.6.




Legend:

A Elevation

B Plan

**Figure 2.6 - Unfavourable distribution of transverse seismic action**

(7) The location of areas for energy dissipation should be chosen to ensure accessibility for inspection and repair. Such locations shall be clearly indicated in the relevant project documents.

(8) The location of areas where seismic damage is possible or expected to occur, other than those indicated in **(7)** shall be identified and the difficulty of repairs shall be minimised.

(9) For exceptionally long bridges, or bridges crossing inhomogeneous soil formations, the number and location of intermediate expansion joints shall be determined.

(10) For bridges crossing potentially active tectonic faults, the value of the probable discontinuity of ground displacement on both sides of the fault must be estimated and provision made for its absorption, either by adjusting the flexibility of the structure or by providing appropriate joints.

(11) The liquefaction potential of the foundation soil shall be investigated in accordance with the relevant provisions of Annex 5.

# 3 Seismic action

## 3.1 Definition of seismic action

### 3.1.1 General considerations

(1) The complexity of the model chosen to describe the seismic action must be appropriate to the considered seismic motion being described and the importance of the structure, as well as comparable to the sophistication of the model used in the bridge analysis.

(2) In this chapter, only the motion transmitted by the ground to the structure is considered for the quantification of the seismic action. However, earthquakes can induce permanent ground displacements ranging from ground failure to fault rupture. Such displacements can lead to imposed deformations with severe consequences for bridges. This type of hazard must be assessed by specific studies. Their consequences must be minimised by appropriate measures, such as the choice of an appropriate structural system. This chapter does not deal with the effects of tsunamis or tidal waves.

### 3.1.2 Application of motion components

(1) In general, only the three translational components of the seismic action need to be considered for the design of bridges. When the response spectrum method is applied, the bridge can be analysed separately for each of the translational components, in the longitudinal, transverse and vertical directions, of the seismic action. In this case, the seismic action is represented by three single-component actions, one for each direction, quantified in accordance with the provisions of section **3.2**. The effects of these actions must be combined as specified in section **4.2.1.4**.

(2) When a non-linear analysis is carried out in the time domain, the bridge must be analysed under the simultaneous action of the different components.

(3) The seismic action is applied to the interface or contact surface between the structure and the ground. If springs are used to represent the soil rigidity, either for isolated footings or for deep foundations such as piles, shafts (caissons), etc., (see Annex 5), the motion is applied at the end of the spring that models the soil.

## 3.2 Quantification of components

### 3.2.1 General considerations

(1) Each component of the seismic motion must be quantified in terms of a response spectrum or a time domain representation (compatible with each other), as set out in Chapter **3** of Annex 1, which also contains the basic definitions.

### 3.2.2 Location-dependent elastic response spectrum

#### 3.2.2.1 Horizontal component

(1) The horizontal component shall be established in accordance with section **3.2.2.2** of Annex 1 according to the type of terrain under the foundation of the bridge supports. Where more than one type of terrain corresponds to these supports, section **3.3** applies.

#### 3.2.2.2 Vertical component

(1) Where it is necessary to take into account the vertical component of the seismic motion (see **4.1.7**), the on-site elastic response spectrum of this component must be taken in accordance with section **3.2.2.3** of Annex 1.

#### 3.2.2.3 Near-field effects

(1) Where the site is located within 10 km, measured horizontally, of a known potentially active fault that could give rise to an earthquake of moment magnitude greater than 6.5, site-specific spectra that consider near-field effects should be used.

### 3.2.3 Time domain representation

(1) At least three pairs of horizontal time domain components of the ground motion must be used when carrying out a non-linear time domain analysis. The pairs should be chosen from earthquakes with magnitudes, source distances and mechanisms consistent with those defining the calculated seismic action.

(2) Where the required number of appropriate pairs of ground motion records is not available, the lack of recorded motions may be replaced by suitably modified records or artificial accelerograms.

(3) Compatibility with the relevant elastic response spectrum at 5 % damping with respect to the critical, corresponding to the calculated seismic action, should be established by modulating the amplitude of the motions as follows:

a) For each earthquake, defined by a pair of horizontal motions, the spectrum must be determined by the SRSS rule, i.e. taking for each period the square root of the sum of the squares of the spectral ordinates corresponding to the spectrum of each component for a damping of 5% with respect to the critical.

b) The spectrum corresponding to the set of earthquakes must be constructed by averaging the values of the spectral ordinates of each of the spectra obtained for each earthquake using the rule of the square root of the sum of the squares in the previous step.

c) The above spectrum, corresponding to the set of earthquakes, must be scaled so that in the period range from 0.2 $T_1$ to 1.5 $T_1$, where $T_1$ is the natural period of the fundamental mode of the structure in the case of a ductile bridge or the effective period ($T_{eff}$) of the isolation system in the case of a bridge with seismic isolation (see **7.2**) is not less than 1.3 times the elastic response spectrum with 5% damping with respect to the critical one, corresponding to the calculated seismic action.

d) The scaling factor derived in the previous step must be applied to each of the seismic motion components.

(4) When the spectrum obtained from the square root rule of the sum of the squares of the components of a recorded accelerogram gives accelerations which, when divided by the values corresponding to the elastic response spectrum for the calculated seismic action, show a large variation in the range of periods specified in consideration **c** of **(3)** above, modifications may be made to the recorded accelerogram, so that the spectrum obtained from the above rule of the square root of the sum of the squares of the modified components will more closely approximate the elastic response spectrum for the calculated seismic action.

(5) The components of each pair of records in the time domain must be applied simultaneously.

(6) When all three components of the motion records are used for a non-linear time domain analysis, the horizontal pairs of components may be scaled according to **(3)**, independently of the scaling of the vertical components. The latter scaling must be performed such that the average of the relevant

spectral ordinates of the ensemble are not more than 10 % less than those of the elastic response spectrum with 5 % damping relative to the critical, corresponding to the vertical seismic action, in the period range from $0.2T_v$ to $1{,}5T_v$, where $T_v$ is the period corresponding to the lower mode in which the response to the vertical component prevails over the response to the horizontal components (e.g. in terms of mobilised mass).

(7) It is also permitted to use pairs of horizontal ground motion records in combination with vertical records of different seismic motions, compatible with the requirements of **(1)** above. Independent scaling of the pairs of horizontal records and vertical records must be carried out as specified in **(6)**.

(8) Modification of the vertical component of the record defined in **(6)** and **(7)** is permitted using the method specified in **(4)**.

### 3.2.4 On-site calculation spectrum for linear analysis

(1) Both ductile and ductility-limited structures must be designed by linear analysis using a reduced response spectrum, called the design spectrum, as specified in section **3.2.2.5** of Annex 1.

## 3.3 Spatial variability of seismic action

(1) For bridge sections with a continuous deck, spatial variability is to be considered when at least one of the following two conditions is satisfied:

- The soil properties vary along the length of the bridge such that more than one type of terrain (as defined in section **3.1.1** of Annex 1) corresponds to the different deck supports.
- The soil properties are approximately uniform along the length of the bridge, but the length of the continuous deck exceeds a limiting length, $L_{\lim} = L_g / 1.5$, where the length $L_g$ is defined in **(6)** below.

(2) The model describing the spatial variability must take into account, even if only in a simplified form, the propagation properties of the seismic waves, as well as the progressive loss of correlation between the motions at different points due to the random heterogeneity of the ground, which implies complex reflections and refractions of the waves. The model must also consider, even if only in a simplified form, the additional increase in correlation loss due to differences in the mechanical properties of the soil along the bridge, which also changes the frequency content of the waves from one support to another.

NOTE Models of the spatial variability of earthquake motions, as well as appropriate methods of analysis, are presented in Appendix D.

(3) Unless a more accurate assessment is made, the simplified method given in **(4)** to **(7)** may be used.

(4) The inertial response shall be taken into account by one of the methods specified in Chapter **4** (see **4.2.1**, **4.2.3** and **4.2.4**), using a single seismic excitation for the whole structure (e.g. a single response spectrum or corresponding sets of compatible accelerograms), calculated for the most severe type of terrain under the bridge supports.

(5) The spatial variation of the seismic action can be estimated by pseudo-static responses to a set of displacements applied at the foundation of the bridge deck supports. These sets should reflect the likely configurations of the spatial variability of the free-field seismic motion, and should be chosen to induce the maximum values of the effects of the seismic action under study.

(6) The requirements of **(5)** above are considered to be satisfied by applying each of the following two sets of horizontal displacements, the effects of which need not be combined, separately in each horizontal direction of analysis, to the corresponding support foundations or to the soil end of the spring of interest representing the soil rigidity:

a) Set A

Set A consists of the relative displacements:

$$d_{ri} = \varepsilon_r L_i \leq d_g \sqrt{2}$$

with $$\varepsilon_r = \frac{d_g \sqrt{2}}{L_g}$$

applied simultaneously with the same sign (+ or -) to all bridge supports (*1* to *n*) in the horizontal direction considered (see Figure 3.1).

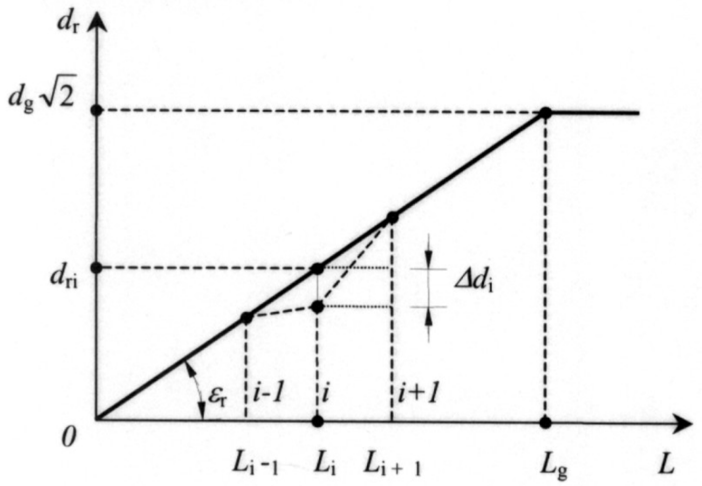



**Figure 3.1 – Set of displacements A**

where

$d_g$ is the calculated value of the ground displacement corresponding to the terrain type of support *i*, as specified in section **3.2.2.4** of Annex 1;

$L_i$ is the distance (projection on the horizontal plane) of support *i* to a reference support *i* = 0, which may be conveniently chosen as one of the end supports;

$L_g$ is the distance beyond which the ground motions can be considered totally independent, i.e. uncorrelated. The value to be assigned to $L_g$ is given in Table 3.1, depending on the type of terrain.

**Table 3.1 - Distance beyond which the soil movements can be considered as totally independent**

| Type of terrain | A | B | C | D |
|---|---|---|---|---|
| $L_g$ (m) | 600 | 500 | 400 | 300 |

b) Set B

Set B covers the influence of soil displacements occurring in opposite directions in adjoining piers. This is taken into account by assuming displacements $\Delta d_i$ of any intermediate support $i$ (> 1) with respect to its adjoining supports, $i$-1 and $i$+1, considered as fixed (see Figure 3.1).

$$\Delta d_i = \pm \beta_r \varepsilon_r L_{\alpha v,i}$$

Where

$L_{\alpha\nu,i}$ is the average of the distances $L_{i-1,i}$ and $L_{i,i+1}$, between the intermediate support $i$ and its contiguous supports $i$-1 and $i$+1, respectively. For extreme supports (0 and $n$), $L_{\alpha\nu,0} = L_{0.1}$ y $L_{\alpha\nu,n} = L_{n-1,n}$;

$\beta_r$ is a coefficient that takes into account the magnitude of soil displacements having opposite directions on adjacent supports.

$\beta_r$ = 0.5 when all three supports rest on the same soil type;

$\beta_r$ = 1.0 when the soil type of one of the supports is different from that of the other two.

$\varepsilon_r$ has the same meaning as in set A above. If a change of terrain type occurs between two supports, the maximum value of $\varepsilon_r$ should be used.

Set B consists of the following configuration of imposed absolute displacements, with opposite signs on adjacent supports, $i$ and $i$+1, for $i$ = 0 to $n$--1 (see figure 3.2).

$$d_i = \pm \Delta d_i /2$$

$$d_{i+1} = \pm \Delta d_{i+1}/2$$

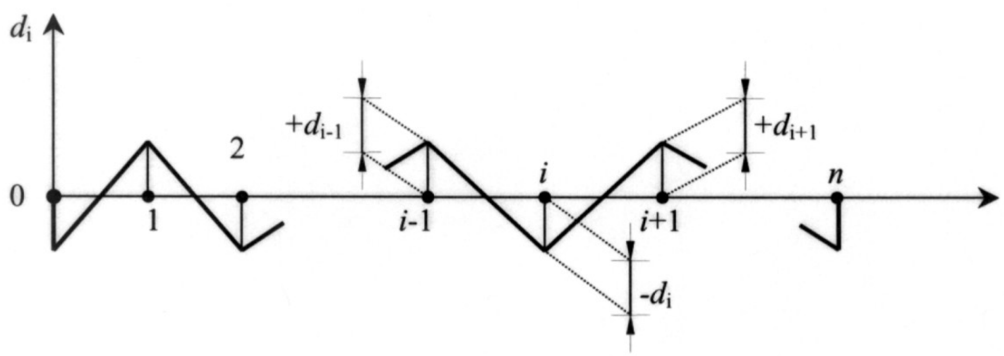



**Figure 3.2 – Set of displacements B**

(7) In each horizontal direction, the most severe effects resulting from the pseudo-static analyses of **(5)** and **(6)** must be combined with the relevant effects of the inertial response of **(4)** by using the square root of the sum of the squares rule (SRSS). The result of this combination determines the

responses of the analysis in the considered direction. For the combination of the effects of the different components of the seismic action, the rules of section **4.2.1.4** are applied.

(8) When a time domain analysis is developed, the seismic motions applied to each support shall reflect with sufficient reliability the likely spatial variability of the seismic action.

NOTE Indications for generating seismic motion samples that reflect the likely spatial variability are given in Section D.2.

# 4 Analysis

## 4.1 Modelling

### 4.1.1 Dynamic degrees of freedom

(1) The bridge model and the choice of dynamic degrees of freedom must represent the mass and stiffness distribution so that all significant deformation modes and inertial forces are mobilised under the calculated seismic excitation.

(2) In some cases it is sufficient to use two different models in the analysis, one to model the response in the longitudinal direction of the bridge, and one for the transverse direction. Cases where it is necessary to consider the vertical component of the seismic action are defined in section **4.1.7**.

### 4.1.2 Masses

(1) The average values of the permanent masses and the values of the quasi-permanent masses corresponding to the variable actions are to be considered.

(2) The distributed masses can be concentrated at nodes, according to the chosen degrees of freedom.

(3) For calculation purposes, the mean values of the permanent actions should be taken equal to their characteristic values.

(4) The quasi-permanent values of the variable actions should be taken equal to $\psi_{2.1}\, Q_{k,1}$, where $Q_{k,1}$ is the characteristic value of the traffic load.

For the purpose of the application of this Seismic Resistance Standard, the combination coefficients, $\psi_{2,1}$, for the seismic situation shall be adopted according to the following values:

- Footbridges and pedestrian bridges, $\psi_{2.1} = 0$.

- Road or rail bridges with normal traffic, $\psi_{2.1} = 0$.

- Bridges with high traffic (only for uniform overload):

  - Road bridges, $\psi_{2.1} = 0.2$.

  - Railway bridges, $\psi_{2.1} = 0.3$.

NOTE 1 Road bridges with high traffic conditions can be considered as comparable to those of motorways and other roads of national importance. Railway bridges with high traffic conditions can be considered as comparable to those on intercity rail links and high speed lines.

NOTE 2 Without prejudice to the above, the verification shall additionally be carried out considering the combinations of actions provided for in the specific regulations in force.

(5) When the pier is immersed in water, and unless a more precise assessment of the hydrodynamic interaction is made, it may be estimated that this effect acts in both horizontal directions, considering an added mass of entrained water per unit length of the immersed pier. The hydrodynamic influence on the vertical seismic action can be ignored.

NOTE Appendix F provides a procedure for the calculation of the added mass of entrained water in the horizontal directions for submerged piers.

### 4.1.3 Damping of the structure and rigidity of the elements

(1) When a response spectrum analysis is performed, the following values of the viscous damping ratio can be assumed, $\xi$, depending on the material of the elements in which most of the deformation energy is dissipated during the seismic response. In general, this will be the case for the piers.

| | |
|---|---|
| Welded steel | 0.02 |
| Bolted steel | 0.04 |
| Reinforced concrete | 0.05 |
| Prestressed concrete | 0.02 |

NOTE When the structure consists of different elements, *i*, with different viscous damping rates, $\xi_i$, the effective viscous damping of the structure, $\xi_{eff}$, can be estimated as:

$$\xi_{eff} = \frac{\sum \xi_i E_{di}}{\sum E_{di}}$$

where $E_{di}$ is the deformation energy induced by seismic action in the component *i*. The effective damping rates can be conveniently estimated independently for each mode on the basis of the relevant value of $E_{di}$.

(2) The stiffness of the elements can be estimated in accordance with section **2.3.6.1**.

(3) In concrete decks consisting of precast concrete beams and cast-in-place slabs, the continuity slabs (see section **2.3.2.2(4)**) shall be included in the seismic analysis model taking into account their relative eccentricity to the deck axis and a reduced value of their flexural stiffness. Unless this stiffness is estimated as a function of the rotation of the relevant plastic hinges, a value of 25% of the flexural stiffness of the uncracked concrete cross-section may be taken.

(4) For second order effects, section **2.4(5)** and section **5.4(1)** apply. Significant second order effects may occur for bridges with slender piers and for special bridges such as arch bridges and cable-stayed bridges.

### 4.1.4 Soil modelling

(1) For the seismic analysis of the overall system, the load-bearing elements transmitting the seismic action of the soil to the deck must be assumed, in general, to be solid with the foundation terrain (see section **3.1.2(3)**). The effects of soil-structure interaction may be considered, in accordance with Annex 5, using appropriately defined impedances or representative soil springs.

(2) Soil-structure interaction effects must always be taken into account for piers where, under the action of a horizontal unit load applied in a given direction at the top of the pier, the flexibility of the soil contributes more than 20 % to the total displacement of the top of the pier.

(3) The effects of soil-structure interaction on piles or caissons shall be determined as specified in section **5.4.2** of Annex 5, taking into account the provisions of section **6.4.2**.

(4) In cases where it is difficult to estimate reliably the mechanical properties of the soil, the analysis shall be carried out using the estimated maximum and minimum probable values. The maximum estimates of soil stiffness shall be used to determine the stresses and the minimum estimates for bridge displacements.

### 4.1.5 Torsional effects

(1) Rotational movements of the bridge around a vertical axis should only be considered for skewed bridges (skew angle $\varphi > 20°$) and for bridges with a $B/L$ ratio > 2.0.

NOTE Such bridges tend to rotate about the vertical axis, even when the vertical axis theoretically coincides with the centre of rigidity ($L$ is the total length of the continuous deck and $B$ is the width of the deck).

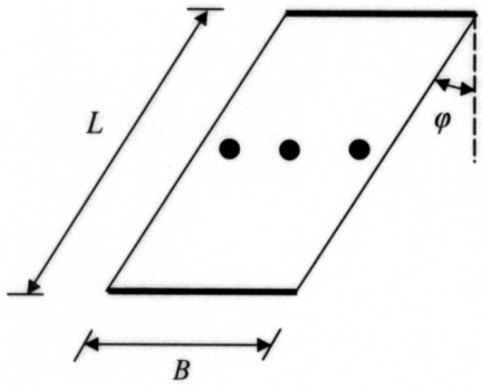



**Figure 4.1 - Bypassed bridge**

(2) In general, strongly skewed bridges ($\varphi > 45°$) should be avoided in areas of high seismicity. If this is not possible and the bridge is supported on the abutments by braces, the actual horizontal rigidity of the braces should be accurately modelled, taking into account the concentration of vertical reactions near the obtuse angles. Alternatively, an increased accidental eccentricity may be used.

(3) When the fundamental mode method (see **4.2.2**) is used for the calculation of skewed bridges, it must be considered that the following equivalent static torsional moment acts on the vertical axis passing through the centre of gravity of the deck:

$$M_t = \pm F e \qquad (4.1)$$

where

$F$ is the horizontal force calculated according to expression (4.12);

$e = e_a + e_d$

$e_a =$ 0.03 $L$ or 0.03 $B$, is the accidental eccentricity of the mass; and

$e_d$ = 0.05 $L$ or 0.05 $B$, is an additional eccentricity reflecting the dynamic effect of simultaneous translational and rotational vibrations.

For the calculation of $e_a$ and $e_d$, the dimensions $L$ or $B$, transverse to the direction of excitation, must be used.

(4) When using a full dynamic model (spatial model), the dynamic part of the torsional excitation is taken into account, if the centre of gravity is displaced, by introducing the accidental eccentricity $e_a$ in the most unfavourable direction and direction. However, the rotation effects can also be estimated by using the static torsional moment of expression (4.1).

(5) The torsional or rotational resistance of a bridge structure should not depend on the torsional rigidity of a single pier. For single span bridges, the supporting devices must be sized to resist the effects of rotation.

### 4.1.6 Behavioural coefficients for linear analysis

(1) The reference design method of this Annex is a response spectrum analysis with the design spectrum defined in section **3.2.2.5** of Annex 1 (see section **3.2.4(1)**). The behavioural coefficient is defined globally for the whole structure and reflects its ductility capacity, i.e. the ability of the ductile elements to resist, with acceptable damage but without failure, seismic actions in the post-elastic range. The available levels of ductility are specified in section **2.3.2**. The ability of ductile elements to develop plastic bending hinges is an essential requirement for the application of the behavioural coefficient values $q$ given for ductile behaviour in Table 4.1.

NOTE The linear analysis method with consideration of sufficiently conservative global force reduction coefficients (behavioural coefficients as defined in table 4.1) is generally considered a reasonable compromise between the uncertainties intrinsic to the seismic problem and the corresponding permissible errors on the one hand, and the effort required for analysis and calculation on the other hand.

(2) This capacity demand of the ductile elements required to develop flexural plastic hinges is considered to be assured when following the construction detailing rules of Chapter **6** and developing the design according to the capacity dimensioning criteria as specified in section **5.3**.

(3) The maximum values of the behavioural coefficient $q$ that can be used for the two horizontal seismic components are specified in Table 4.1, depending on the post-elastic behaviour of the ductile elements in which the main energy dissipation takes place. If a bridge has several types of ductile elements, the behavioural coefficient $q$ corresponding to the group with the largest contribution to the seismic resistance should be used. Different values of $q$ may be used in each of the two horizontal directions.

NOTE The use of behavioural coefficient values lower than the maximum permissible value specified in Table 4.1 normally leads to reduced ductility demands, which usually implies a reduction in damage potential. The use of such a reduced coefficient is therefore at the discretion of the designer or owner.

**Table 4.1 - Maximum values of behavioural coefficient $q$**

| Type of ductile elements | Seismic behaviour | |
|---|---|---|
| | **Limited ductility** | **Ductile** |
| Reinforced concrete piers: | | |
| Vertical piers working in bending | 1.5 | 3.5 $\lambda(\alpha_s)$ |

| Type of ductile elements | Seismic behaviour | |
|---|---|---|
| | Limited ductility | Ductile |
| Inclined piers working in bending | 1.2 | $2.1\,\lambda(\alpha_s)$ |
| Steel piers: | | |
| Vertical piers working in bending | 1.5 | 3.5 |
| Inclined piers working in bending | 1.2 | 2.0 |
| Piers with centred triangulations | 1.5 | 2.5 |
| Piers with off-centred triangulations | - | 3.5 |
| Abutments rigidly connected to the deck: | | |
| In general | 1.5 | 1.5 |
| Structures embedded in the terrain (see **4.1.6(9), (10)**) | 1.0 | 1.0 |
| Arches | 1.2 | 2.0 |

* $\alpha_s = L_s / h$ is the shear rate of the pier, where $L_s$ is the distance between the plastic hinge and the point of zero moment, and $h$ is the cross-sectional depth in the direction of the plastic hinge.

For $\alpha_s \geq 3$ $\quad \lambda(\alpha_s) = 1.0$

$3 > \alpha_s \geq 1.0$ $\quad \lambda(\alpha_s) = \sqrt{\frac{\alpha_s}{3}}$

NOTE For piers of rectangular shape, when the compression zone under seismic action in the overall direction considered is triangular in shape, the minimum value of the $\alpha_s$ values corresponding to the two sides of the section shall be used.

(4) For all bridges with a regular seismic behaviour as specified in section **4.1.8**, and assuming that the construction detailing requirements set out in Chapter **6** are satisfied, the values of the coefficient $q$ detailed for ductile behaviour in Table 4.1 may be adopted without any special check of the available ductility. Where only the requirements specified in section **6.5** are met, the values of the coefficient $q$ detailed in Table 4.1 for limited ductility behaviour may be used, regardless of the regularity or irregularity of the bridge, without any special check of the available ductility.

(5) For ductile reinforced concrete elements, the values of the coefficient $q$ specified in Table 4.1 are valid when the reduced axial force $\eta_k$, as defined in section **5.3(4)**, does not exceed 0.30. If 0.30 < $\eta_k$ ¿ 0.60, even in a single ductile element, the value of the behavioural coefficient must be reduced to:

$$q_r = q - \frac{\eta_k - 0{,}3}{0{,}3}(q - 1) \geq 1{,}0 \tag{4.2}$$

A value of $q_r$ = 1.0 (elastic behaviour) shall be taken for bridges where the seismic-resisting system contains elements with $\eta_k$ ¿ 0.6.

(6) The values of the coefficient $q$ given in Table 4.1 can be adopted for ductile behaviour only if the locations of all plastic hinges of interest are accessible for inspection and repair. Otherwise, the values in Table 4.1 should be multiplied by 0.6; however, it is not necessary to consider final $q$ values less than 1.0.

NOTE The term 'accessible' used in the previous point means 'accessible even with reasonable difficulty'. The footing of a pier foundation located in a backfill, even at a significant depth, is considered to be "accessible". On the other hand, the foot of a pier foundation shaft immersed in deep water or the heads of piles under a large pile cap shall not be considered as accessible.

(7) Where energy dissipation is intended to take place in plastic hinges in piers designed for ductile behaviour, in locations which are not accessible, the final value of the coefficient $q$ used must not be less than 2.1 for vertical piers and 1.5 for inclined piers (see also section **5.4.2(5)** of Annex 5).

(8) Section **2.3.2.2(4)** regarding the formation of plastic hinges in the deck applies.

NOTE In this case the potential formation of plastic hinges in the secondary elements of the deck (continuity slabs) is allowed, but shall not be relied upon to increase the value of $q$.

(9) Bridge structures whose mass essentially follows the horizontal seismic behaviour of the ground ('embedded in the terrain' structures), do not experience a significant amplification of the horizontal ground acceleration. Such structures are characterised by a very low value of the natural period in the two horizontal directions ($T$ ¿ 0.03 s). The inertial response of these structures in the horizontal directions can be evaluated by calculating, directly from the design ground acceleration and with $q$ = 1, the horizontal inertial forces. This category includes abutments connected to the deck by means of flexible elements.

(10) Bridge structures consisting of an essentially horizontal deck rigidly connected to both abutments (either monolithically or by means of fixed supporting devices or connectors) may be considered as falling into the category of **(9)** above, regardless of the value of the natural period, if the abutments are embedded in rigid natural terrain formations for at least 80% of their lateral surface. If these conditions are not met, then the interaction with the terrain at the abutment location must be included in the model, using realistic soil rigidity parameters. If $T$ > 0.03 s, then the design spectrum defined in section **3.2.2.5** of Annex 1 with $q$ = 1.50 shall be used.

(11) When the main part of the calculated seismic action is resisted by elastomeric bearings, the flexibility of these bearings leads to an almost elastic behaviour of the system. Bridges of this type must be designed in accordance with Chapter **7**.

NOTE In general, no plastic hinges will develop in the piers that are connected to the deck in the direction considered by flexible elements. A similar situation will occur in each of the piers that have very low rigidity compared to that of the other piers (see section **2.3.2.2(7)** and the NOTE following **(9)**). Such elements have a negligible contribution to resisting seismic actions and consequently do not affect the value of the coefficient $q$ (see section **4.1.6(3)**).

(12) The behavioural coefficient for the analysis in the vertical direction must always be taken to be equal to 1.0.

### 4.1.7 Vertical component of seismic action

(1) In general, the effects of the vertical component of the seismic action on the piers need not be taken into account, except in areas of high seismicity if the piers are subject to high bending stresses due to the permanent vertical actions of the deck, or when the bridge is located within 5 km of an active seismotectonic fault, in which case the vertical seismic action is determined in accordance with the provisions of section **3.2.2.3**.

(2) The effects of the vertical seismic component acting upwards on prestressed concrete decks must always be taken into account.

(3) The effects of the vertical seismic component on supporting devices and connectors must always be taken into account.

(4) The estimation of the effects of the vertical component can be carried out using the fundamental mode method and the flexible deck model (see **4.2.2.4**).

### 4.1.8 Regular and irregular seismic behaviour of ductile bridges

(1) Designating as $M_{Ed,i}$ the maximum value of the design bending moment at the location of the expected plastic hinge in ductile member *i*, as derived from the analysis for the calculated seismic situation, and as $M_{Rd,i}$ to the ultimate design resisting moment of the same section with its actual reinforcement under the concurrent action of the effects of the non-seismic actions in the calculated seismic situation, the reduction coefficient of the local force $r_i$ associated with element *i*, subjected to the specific seismic action, is then defined as:

$$r_i = q \frac{M_{Ed,i}}{M_{Rd,i}} \quad (4.3)$$

NOTE 1 Since $M_{Ed,i} \leq Mrd,i,_{Rd,i}$, it follows that $r_1 \leq q$.

NOTE 2 When in a regular bridge the maximum value of $r_i$ of all the ductile elements, $r_{max.}$, is significantly less than $q$, the design cannot take full advantage of the maximum permissible values of $q$. When $r_{max.}$ = 1.0 the bridge responds elastically to the design earthquake under consideration.

(2) A bridge must be considered to have a regular seismic behaviour in the horizontal direction under consideration, when the following condition is satisfied:

$$\rho = \frac{r_{máx.}}{r_{mín.}} \leq \rho_o \quad (4.4)$$

where

$r_{min.}$ is the minimum value of $r_i$;

$r_{max.}$ is the maximum value of $r_i$ among all ductile elements*i*; and

$\rho_o$ is a limit value selected to ensure that sequential plastification of ductile elements will not result in unacceptably high ductility demands on an element. The value $\rho_o$ = 2.0 is adopted.

(3) One or more ductile elements (piers) may be exempted from the above calculation of $r_{min.}$ and $r_{max.}$ if their total contribution to the shear force does not exceed 20% of the total seismic shear force in the horizontal direction considered.

(4) Bridges not satisfying expression (4.4) must be considered to have irregular seismic behaviour in the horizontal direction considered. Such bridges must be calculated either by using a reduced *q* value:

$$q_r = q\frac{\rho_0}{\rho} \geq 1,0 \qquad (4.5)$$

or on the basis of the results of a non-linear analysis as specified in section **4.1.9**.

### 4.1.9 Non-linear analysis of irregular bridges

(1) In bridges with irregular seismic behaviour, sequential plastification of the ductile elements (piers) can cause substantial deviations of the results of the equivalent linear analysis performed under the assumption of a global force reduction coefficient $q$ (behavioural coefficient) compared to those of the non-linear response of the bridge structure. The deviations are mainly due to the following causes:

- The first appearing plastic hinges usually develop the maximum post-elastic deformations, which can lead to a concentration of unacceptable ductility demands on these hinges.

- After the formation of the first plastic hinges (usually in the stiffest elements), the distribution of stiffnesses and thus forces may change with respect to those predicted by the equivalent linear analysis. This can lead to a substantial change of the assumed configuration of plastic hinges.

(2) In general, the actual response of irregular bridges to the calculated seismic action can be estimated by means of a non-linear dynamic analysis in the time domain, carried out in accordance with the provisions of section **4.2.4**.

(3) As specified in section **4.2.5**, an approximate value of the non-linear response may also be obtained by a combination of an equivalent linear analysis with a non-linear static analysis (incremental thrust analysis).

## 4.2 Methods of analysis

### 4.2.1 Linear dynamic analysis. Response spectrum method

#### 4.2.1.1 Definition and field of application

(1) The response spectrum analysis is an elastic calculation of the maximum dynamic responses of all significant modes of the structure, using the ordinates of the design spectrum at the site (see section **3.2.2.5** of Annex 1). The global response is obtained by a statistical combination of the maximum modal contributions. Such an analysis can be applied in all cases where a linear analysis is permitted.

(1) The effects of seismic action must be determined from a suitable discrete linear model (full dynamic model), idealised according to the laws of mechanics and the principles of structural analysis, and compatible with an associated idealisation of the seismic action. In general, this model is a spatial model.

#### 4.2.1.2 Significant modes

(1) All modes that have a significant contribution to the total response of the structure must be taken into account.

(2) For bridges where the total mass $M$ can be considered as a sum of the 'effective modal masses', $M_i$, the criterion in **(1)** above is considered satisfied if the sum of the effective modal masses of the considered modes, $(\Sigma M_i)_c$, constitutes at least 90% of the total mass of the bridge.

(3) If after considering all modes with $T$ ¿ 0.033 s the condition in **(2)** is not satisfied, the number of modes considered may be considered acceptable, provided that the following two conditions are satisfied:

- $(\Sigma M_i)_c/M \geq 0.70$
- The final values of the effects of seismic action are multiplied by $M/(\Sigma M_i)_c$.

#### 4.2.1.3 Combination of modal responses

(1) In general, the probable maximum value $E$ of an effect of a seismic action (stress, displacement, etc.) should be taken equal to the square root of the sum of the squares of the modal responses $E_i$ (SRSS rule):

$$E = \sqrt{\sum E_i^2} \tag{4.6}$$

It must be assumed that this effect of the action acts in both directions (with signs + and -).

(2) When two modes have very close natural periods, the SRSS rule (expression (4.6)) is not conservative and more precise rules must be applied. Two natural periods $T_i$, $T_j$, can be considered very close if they satisfy the condition:

$$\frac{0,1}{0,1+\sqrt{\xi_i \xi_j}} \leq \rho_{ij} = T_i / T_j \leq 1 + 10\sqrt{\xi_i \xi_j} \tag{4.7}$$

where $\xi_i$ and $\xi_j$ are the viscous damping rates of modes $i$ and $j$, respectively (see **(3)**).

(3) For any two modes satisfying expression (4.7), the full quadratic combination (CQC) method can be used instead of the SRSS rule:

$$E = \sqrt{\Sigma_i \Sigma_j E_i r_{ij} E_j} \tag{4.8}$$

with $i = 1 \ldots .n$, $j = 1 \ldots n$

In expression (4.8), $r_{ij}$ is the correlation coefficient:

$$r_{ij} = \frac{8\sqrt{\xi_i \xi_j}(\xi_i + \rho_{ij}\xi_j)\rho_{ij}^{3/2}}{(1+\rho_{ij}^2)^2 + 4\xi_i \xi_j \rho_{ij}(1+\rho_{ij}^2) + 4(\xi_i^2 + \xi_j^2)\rho_{ij}^2} \tag{4.9}$$

where

$\xi_i$ and $\xi_j$ are the viscous damping rates *i* corresponding to modes *i* and *j*,respectively.

NOTE Expression (4.9) gives $r_{ij} = r_{ji}$. When $T_i = T_j$, then $\xi_i = \xi_j$ and $r_{ij} = 1$.

#### 4.2.1.4 Combination of the components of seismic action

(1) The probable maximum effect of action *E* due to the simultaneous action of the seismic action components along the horizontal *X* and *Y* axes and the vertical *Z* axis can be estimated in accordance with the provisions of section **4.3.3.5.2(4)** of Annex 1, i.e. by applying the square root of the sum of the squares rule (SRSS) to the maximum effects of the action, $E_x$, $E_y$ and $E_z$, due to the independently calculated seismic actions along each axis.

$$E = \sqrt{E_x^2 + E_y^2 + E_z^2} \tag{4.10}$$

(2) Again in accordance with of section **4.3.3.5.2(4)** of Annex 1, the probable maximum effect of the action, *E*, can be taken as the worst of the effects calculated by expressions (4.18) to (4.22) of Annex 1.

### 4.2.2 Fundamental mode method

#### 4.2.2.1 Definition

(1) In the fundamental mode method the equivalent static seismic forces are derived from the inertia forces corresponding to the fundamental mode and the natural period of the structure in the direction under consideration, taking the appropriate ordinate of the design spectrum at the site. The method also includes simplifications concerning the geometric shape of the first mode and the estimation of the fundamental period.

(2) Depending on the specific characteristics of the bridge, this method can be applied by using three different types of models, namely:

- rigid deck model
- flexible deck model
- single pier model

(3) For the combination of the seismic action components, the rules of section **4.2.1.4** should be applied.

#### 4.2.2.2 Field of application

(1) The method can be applied in all cases where the dynamic behaviour of the structure can be sufficiently approximated by a single dynamic degree of freedom model. This condition is considered to be satisfied in the following cases:

(a) In the longitudinal direction of approximately straight bridges with continuous deck, when the seismic forces are resisted by piers whose total mass is less than 20 % of the total deck mass.

(b) In the transverse direction of case (a), if the structural system is approximately symmetric about the centre of the deck, i.e. when the theoretical eccentricity,$e_o$, between the centre of rigidity of the supporting members and the centre of gravity of the deck does not exceed 5 % of the deck length (*L*).

(c) In the case of piers supporting simply supported piers, if no significant interaction between the piers is expected and if the total mass of each pier is less than 20% of the portion of the deck gravitating on it.

#### 4.2.2.3 Rigid deck model

(1) This model can be applied only when, under seismic action, the deformation of the deck in a horizontal plane is negligible compared to the horizontal displacements of the piers' heads. This condition is always fulfilled in the longitudinal direction of approximately straight bridges with a continuous deck. The deck can be assumed rigid in the transverse direction either if $L/B \leq 4.0$ or if the following condition is satisfied:

$$\frac{\Delta_d}{d_a} \leq 0,20 \tag{4.11}$$

where

$L$ is the overall length of the continuous deck;

$B$ is the width of the deck; and

$\Delta_d$ and $d_a$ are, respectively, the maximum and mean differences of the transverse displacements of all the pier heads due to the calculated seismic action or to the action of a similarly distributed transverse load.

(2) The seismic effects must be determined by applying an equivalent static horizontal force, $F$, to the deck, given by the expression:

$$F = M\,S_d\,(T) \tag{4.12}$$

where

$M$ is the total effective mass of the structure, equal to the mass of the deck plus the mass of the upper half of the piers;

$S_d(T)$ is the spectral acceleration corresponding to the design spectrum (see section **3.2.2.5** of Annex 1), for the fundamental period $T$ of the bridge, estimated as:

$$T = 2\pi\sqrt{\frac{M}{K}} \tag{4.13}$$

where $K = \Sigma K_i$ is the rigidity of the system, equal to the sum of the stiffnesses of the resisting elements.

(3) In the transverse direction, the force $F$ can be distributed along the deck proportionally to the distribution of the effective masses.

#### 4.2.2.4 Flexible deck model

(1) The flexible deck model must be used when the expression (4.11) is not satisfied.

(2) Unless a more precise calculation is made, the fundamental period of the structure in the horizontal direction under consideration may be estimated by the Rayleigh quotient, using a generalised single degree of freedom system, as follows:

$$T = 2\pi \sqrt{\frac{\sum M_i d_i^2}{g \sum M_i d_i}} \tag{4.14}$$

where

$M_i$ is the mass concentrated at the *i*th node;

$d_i$ is the displacement in the direction under consideration when the structure is subjected to forces $gM_i$, acting on all nodes in the horizontal direction under consideration.

(3) The seismic effects must be determined by the application, at all nodes, of horizontal forces $F_i$ given by:

$$F_i = \frac{4\pi^2}{gT^2} S_d(T) d_i M_i \tag{4.15}$$

where

$T$ is the period of the fundamental mode of vibration for the horizontal direction considered;

$M_i$ is the mass concentrated at the *i*th nodal point;

$d_i$ is the displacement of the *i*th nodal point, calculated from an approximate modal warping of the first mode (can be taken equal to the values determined in **(2)** above);

$S_d(T)$ is the spectral acceleration of the calculation spectrum (see **3.2.2.5** of Annex 1); and

$g$ is the gravitational acceleration.

#### 4.2.2.5 Torsional effects in the transverse direction (rotation about the vertical axis)

(1) When using the rigid or flexible deck model in the transverse direction of a bridge, the torsional effects can be estimated by applying a static torsional moment, $M_t$, in accordance with the provisions of expression (4.1) in section **4.1.5(3)**. The eccentricity shall can be estimated as follows:

$$e = e_o + e_a \tag{4.16}$$

where

$e_o$ is the theoretical eccentricity (see case (b) in section **4.2.2.2(1)**);

$e_a =$ 0.05 $L$ is an additional eccentricity that takes into account the effects of accidental and dynamic amplifications.

(2) The force $F$ can be determined either from expression (4.12) or as $\Sigma F_i$ from expression (4.15). The moment $M_t$ can be distributed among the bearing elements using the rigid deck model.

#### 4.2.2.6 Single pier model

(1) In some cases, the seismic action in the transverse direction of the bridge is resisted mainly by the piers, with no significant interaction between adjacent piers. In such cases, the effects of the seismic action acting on the *i*th pier can be approximately evaluated by applying an equivalent static force on the pier:

$$F_i = M_i \, S_d(T_i) \tag{4.17}$$

where

$M_i$ is the effective mass attributed to the pier $i$; and

$$T_i = 2\pi \sqrt{\frac{M_i}{K_i}} \tag{4.18}$$

is the fundamental period of the said pier, considered independently of the rest of the bridge.

(2) This simplification can be applied as a first approximation in preliminary analyses when for all contiguous piers, $i$ and $i+1$, for the results of expression (4.18) the following condition is satisfied:

$$0.90 \leq T_i / T_{i+1} \leq 1.10 \tag{4.19}$$

Otherwise, a redistribution of the effective masses attributed to each pier is necessary to satisfy the above condition.

### 4.2.3 Alternative linear methods

#### 4.2.3.1 Time series analysis

(1) In a time series analysis, the calculated seismic action must be taken as the mean value of the extreme response calculated for each accelerogram of the set of records in the considered time domain. For the selection of these records, the provisions of section **3.2.3** apply.

### 4.2.4 Non-linear dynamic analysis in the time domain

#### 4.2.4.1 General considerations

(1) The time-dependent response of the structure must be obtained by direct numerical integration of the non-linear differential equations of motion. The seismic excitation must consist of ground motions in the time domain (accelerograms, see **3.2.3**). The effects for the calculated seismic situation of gravity loads and other quasi-permanent actions as well as second order effects must be taken into account.

(2) Unless otherwise stated in this standard, this method can only be used in combination with a standardised response spectrum analysis to obtain an insight into the post-elastic response and a comparison between the required and available local ductility demands. Generally, the results of the

non-linear analysis should not be used to lower the requirements obtained in a response spectrum analysis. However, in case of uneven bridges (see **4.1.8**) or bridges with seismic isolation devices (see chapter **7**), the lower values estimated from a rigorous time domain analysis can be replaced by the results of the response spectrum analysis.

#### 4.2.4.2 Soil movements and combination of calculation

(1) The provisions of section **3.2.3** apply.

(2) The provisions of section **5.5(1)** and section **4.1.2** apply.

#### 4.2.4.3 Effects of the calculated action

(1) Where a non-linear dynamic analysis is carried out for at least seven independent pairs of horizontal ground motions, the average of each isolated response may be used as the calculated value of the effects of the action, except as otherwise stated in this Standard. Where fewer than seven non-linear dynamic analyses are carried out for the corresponding independent pairs of excitatory motions, the maximum responses of the ensemble shall be used as the effects of the calculated actions.

#### 4.2.4.4 Ductile structures

(1) **Objectives**

The main objectives of a non-linear time domain analysis of a ductile bridge are the following:

- The identification of the actual formation configuration of a plastic hinge.
- The estimation and verification of the probable post-elastic deformation demands on the plastic hinges, and the estimation of the displacement demands.
- The determination of strength requirements for the prevention of non-ductile failure modes in the superstructure and for soil testing.

(2) **Requirements**

For a ductile structure subjected to high local ductility demands, the achievement of the above objectives requires the following:

(a) A realistic identification of the extent of the structure that remains elastic. Such identification shall be based on the probable values of the elastic stresses and strains of the materials.

(b) In areas of plastic hinges, the stress-strain diagrams for both concrete and reinforcement or structural steel shall reflect the likely post-elastic behaviour, taking into account the confinement of the concrete, where relevant, and the effects of strain hardening and/or local buckling for the steel. The shape of hysteresis loops shall be adequately modelled, taking into account strength and stiffness degradation and hysteresis losses, if evidenced by appropriate laboratory tests.

(c) Verification that the deformation demands are reliably less than the plastic hinge capacities shall be carried out by comparing the plastic hinge rotational demands, $\theta_{p,E}$, with the corresponding design rotational capacities, $\theta_{p,d}$, as follows:

$$\theta_{p,E} \leq \theta_{p,d} \quad (4.20)$$

The calculated values of the plastic rotational capacities, $\theta_{p,d}$, shall be derived from the relevant test results or determined from the ultimate curvatures by dividing the probable value $\theta_{p,u}$ by a coefficient $\gamma_{R,p}$, reflecting local defects in the structure, model uncertainties and/or scatter in the relevant test results, as follows:

$$\theta_{p,d} = \frac{\theta_{p,u}}{\gamma_{R,p}} \quad (4.21)$$

The same condition shall be checked for the other deformation and capacity demands of the dissipative zones of steel structures (e.g. the elongation of the tensile elements of diagonals and the shear deformation of the shear panels of off-centre triangulations).

NOTE Appendix E provides information for the estimation of $\theta_{pd}$ and for the estimation of $\gamma_{R,p}$.

(d) It is not necessary to check the flexural strength of the members, as this is inherent in the non-linear analysis procedure carried out in accordance with (a) above. However, it shall be verified that no significant plastification occurs in the deck (see sections **5.6.3.6(1)** and **(2)**).

(e) The testing of members for non-ductile failure modes (shear stress in members and at nodes adjacent to plastic hinges), as well as for foundation failure, shall be carried out in accordance with the relevant rules of Chapter **5**. The response to capacity sizing shall be taken as the structural response resulting from the non-linear analysis multiplied by $\gamma_{Bdl}$, as specified in requirement (b) of section **5.6.2(2)**. These values shall not exceed the calculated resistances $R_d$ (= $R_k/\gamma_M$) of the corresponding sections, i.e:

$$\max. E_d \leq R_d \quad (4.22)$$

#### 4.2.4.5 Seismic isolation bridges

(1) The objective of the analysis is, in this case, the realistic assessment of force and displacement demands:

- taking appropriate account of the effect of the variability of the isolator properties, and

- ensuring that the isolated structure remains essentially elastic.

(2) The provisions of Chapter **7** apply

### 4.2.5 Non-linear static analysis (incremental pushover analysis)

(1) The incremental pushover analysis is a non-linear static analysis in which the structure is subjected to constant vertical (gravity) loads and uniformly increasing horizontal forces representing the effect of a horizontal seismic component. Second order effects must be taken into account. Horizontal loads are increased until the ultimate displacement corresponding to the fundamental mode at a reference point is reached.

(2) The main objectives of the analysis are the following:

- The estimation of the sequence and final configuration of the formation of the plastic hinges;
- The estimation of the redistribution of forces following the formation of plastic hinges;
- The evaluation of the force-displacement diagram of the structure ("capacity curve") and of the deformation demands of the plastic hinges up to the maximum displacement caused by the seismic action.

(3) The method can be applied to the entire bridge structure or to individual bridge components.

(4) The requirements of section **4.2.4.4(2)** apply, with the exception of requirement (b) of section **4.2.4.4(2)** for the modelling of the shape of hysteresis loops.

NOTE 1 A procedure for the application of this method is recommended in appendix H.

NOTE 2 It should be noted that a non-linear static analysis using incremental pushover, as described in Appendix H, leads to realistic results for structures whose response to horizontal seismic action in the direction considered can be reasonably approximated by a generalised one-degree-of-freedom system. Assuming that the influence of the masses of the piers is small, the above condition is always fulfilled in the longitudinal direction of approximately straight bridges. The condition is also fulfilled in the transverse direction when the distribution of the rigidity of the piers along the bridge provides a more or less uniform lateral support to a relatively stiff deck. This is most commonly the case for bridges where the height of the piers decreases towards the abutments or does not vary significantly. However, when the bridge has an exceptionally stiff pier with no plastic deformation capacity, located between a group of regular piers, the system cannot approximate a single degree of freedom in the transverse direction, and analysis by incremental thrusts may not lead to realistic results. A similar exception occurs on long bridges when very stiff piers are placed between groups of very regular ones, or on bridges where the mass of some of the piers has a significant effect on the dynamic behaviour in one or both directions. Such an irregular arrangement can be avoided, for example, by placing slip joints between the deck and the pier(s) causing the irregularity. If this is not possible or desirable, then a non-linear analysis in the time domain should be used.

# 5 Resistance testing

## 5.1 General considerations

(1) The provisions of this Chapter apply to earthquake resistant systems of bridges designed by an equivalent linear method taking into account a ductile or limited ductility behaviour of the structure (see **4.1.6**). For bridges fitted with isolation devices, Chapter **7** should be applied. For checks based on non-linear analysis results, section **4.2.4** applies. In the last two cases, section **5.2.1** applies.

## 5.2 Materials and resistance calculation value

### 5.2.1 Materials

(1) In bridges designed for ductile behaviour with $q > 1.5$, concrete elements in which plastic hinges may form must be reinforced with steel type SD, in accordance with Articles 34 and 35 of the Structural Code.

(2) Concrete elements of bridges designed for ductile behaviour, in which no plastic hinges can form (as a consequence of capacity design), as well as all concrete elements of bridges designed for limited ductility behaviour ($q \leq 1.5$) or all concrete elements of bridges with seismic isolation in accordance with Chapter **7**, may be reinforced with steel type S, as specified in Articles 34 and 35 of the Structural Code.

(3) The structural steel members of all bridges must comply with the provisions of Section **6.2** of Annex 1.

### 5.2.2 Calculated value of resistance

(1) The calculated value of the resistance of the element shall be determined in accordance with sections **5.2.4**, **6.1.3** or **7.1.3**, as appropriate, of Annex 1.

## 5.3 Dimensioning by capacity

(1) For structures designed for ductile behaviour, the stresses of the FC capacity design $F_C$ ($V_C$, $M_C$, $N_C$) must be calculated by analysing the expected plastic mechanism, under:

a) non-seismic actions for the calculated seismic situation; and

b) the level of seismic action in the direction considered (see **(6)**) for which all intended bending spherical plain bearings have developed bending moments equal to a certain upper fraction of their bending resistance, called the moment of over-resistance (reserve resistance) $M_o$.

(2) It is not necessary to adopt stresses for capacity design as large as those resulting for the calculated seismic situation (see section **5.5**) in the direction considered, with the effects of the seismic action multiplied by the behavioural coefficient $q$ used in the analysis for the calculated seismic action.

(3) The moment of over-resistance of a section is to be calculated as:

$$M_o = \gamma_o M_{Rd} \tag{5.1}$$

Where

$\gamma_o$ is the over-resistance coefficient;

$M_{Rd}$ is the bending resistance of the ductile section in the selected direction and direction, based on the actual section geometry including reinforcement where relevant, and the material properties (with $\gamma_M$ values corresponding to the fundamental design situations). When determining $M_{Rd}$, the biaxial bending moment under: (a) the effects arising from non-seismic actions for the calculated seismic situation and (b) the other effects of the seismic action corresponding to the calculated seismic action with the selected direction and direction.

(4) The coefficient of over-resistance (reserve strength) is intended to reflect the variability of the strength properties of the materials and the ratio of the ultimate strength to the yield strength. The following values shall be adopted for $\gamma_o$:

- For concrete elements: $\gamma_o = 1.35$;
- For steel elements: $\gamma_o = 1.25$.

In the case of reinforced concrete sections with special confining reinforcement as specified in section **6.2.1**, and with the value of the reduced axial force:

$$\eta_k = N_{Ed} / (A_c f_{ck}) \tag{5.2}$$

greater than 0.1, the value of the coefficient of over-resistance must be multiplied by $1+2(\eta_k\text{-}0.1)^2$

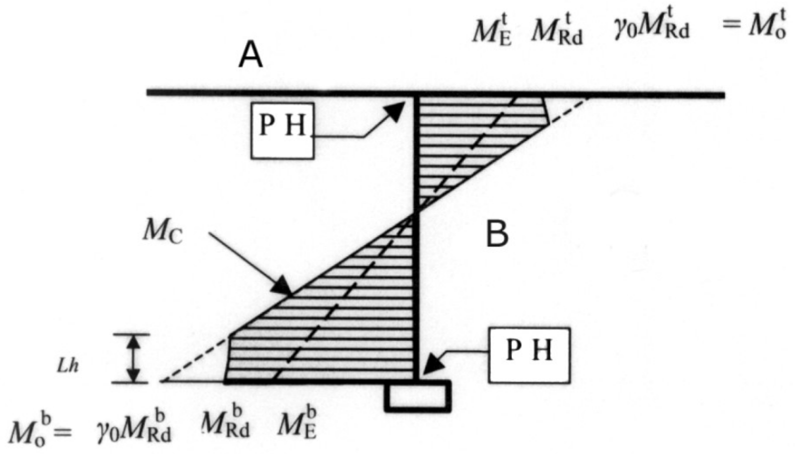
Where

$N_{Ed}$ is the value of the axial force at the location of the plastic hinge, corresponding to the calculated seismic situation, positive if compressive;

$A_c$ is the transverse area; and

$f_{ck}$ is the characteristic value of the concrete resistance.

(5) Over the entire length of the members in which plastic hinges develop, the bending moment of the capacity dimensioning, $M_c$, in the vicinity of the hinge (see Figure 5.1) must not be assumed to be greater than the corresponding bending resistance of the nearest hinge section $M_{Rd}$, calculated as specified in section **5.6.3.1**.




Legend:

A Deck

B Pier

PH Plastic hinge

**Figure 5.1 - Bending moments of capacity dimensioning, $M_c$, within the length of the elements containing the plastic hinges**

NOTE 1 The $M_{Rd}$ diagrams shown in Figure 5.1 correspond to a pier with variable cross-section (increasing towards the base). In the case of constant cross-section with uniform reinforcement, $M_{Rd}$ is also constant.

NOTE 2 For $L_h$ see section **6.2.1.5**.

(6) In general, the capacity design forces corresponding to the seismic action acting (with signs + and -) in each of the longitudinal and transverse directions must be calculated separately. A procedure and appropriate simplifications are given in Appendix G.

(7) When sliding bearings participate in the plastic collapse mechanism, their capacity must be assumed to be equal to $\gamma_{of} R_{df}$, where:

$\gamma_{of} = 1.30$ is the amplification coefficient for friction due to ageing effects; and

$R_{df}$ is the maximum design frictional force that can be transmitted by the bearing apparatus.

(8) For bridges with elastomeric bearings and designed for ductile behaviour, the elements resisting the shear forces transmitted by the bearings and where no plastic hinge formation is foreseen, must be calculated as follows: the effects of the capacity design must be determined from the maximum bearing deflection corresponding to the calculated value of the deck displacement and a rigidity of the bearings increased by 30 %.

## 5.4 Second order effects

(1) In linear analyses, approximate methods can be used to estimate the influence of second order effects on the critical sections (plastic hinges), also taking into account the cyclic nature of the seismic action, where it has a significant unfavourable effect.

The approximate method for estimating the second order effects due to seismic action is to assume that the increase in bending moments in the plastic hinge section due to the second order effects is:

$$\Delta M = \frac{1+q}{2} d_{\mathrm{Ee}} N_{\mathrm{Ed}} \tag{5.3}$$

where $N_{\mathrm{Ed}}$ is the axial force and $d_{\mathrm{Ee}}$ is the relative transverse displacement of the ends of the ductile member under consideration, both determined for the calculated seismic situation.

## 5.5 Combination of seismic and other actions

(1) The calculated value $E_{\mathrm{d}}$ of the effects of the actions in the calculated seismic situation must be determined in accordance with the provisions of section **6.4.3.4** of Annex 18 of the Structural Code (see section **3.2.4(1)** of Annex 1), in conjunction with the provisions of **(2)** to **(4)** of this section.
In general, and where **(3)** below does not apply, $E_{\mathrm{d}}$ shall be determined in accordance with the following expression:

$$Ed = \sum_{j \geq 1} G_{k,j} + P_k + A_{Ed} + \psi_{2,1} Q_{k,1} + Q_2 \tag{5.4}$$

Where

| | |
|---|---|
| "+" | means "to be combined with"; |
| $G_{\mathrm{k}}$ | are the permanent loads with their characteristic values; |
| $P_{\mathrm{k}}$ | is the characteristic value of the prestressing action, net of all losses; |
| $A_{\mathrm{Ed}}$ | is the calculated seismic action; |
| $Q_{\mathrm{k,1}}$ | is the characteristic value of the traffic load; |
| $\psi_{2,1}$ | is the combination coefficient for the traffic loads, as specified in section **4.1.2(4)**; and |

$Q_2$ is the quasi-permanent value for long duration actions (e.g. terrain thrust, hydrodynamic thrust, water currents, etc.).

NOTE Long duration actions are considered concurrent with the calculated seismic action.

(2) It is not necessary to combine the effects of seismic action with those of actions due to imposed deformations (caused by temperature, shrinkage, settlement of supports, residual movements of the terrain due to seismic faults).

(3) An exception to the rule in **(2)** above is the case of bridges where the seismic action is resisted by laminated elastomeric bearings (see also section **6.6.2.3(4)**). In this case, elastic behaviour of the system must be assumed and the effects of actions due to imposed deformations must be taken into account.

NOTE In the case of **(3)**, the displacement due to creep does not normally induce additional stresses in the system and can therefore be neglected. Creep also reduces the effective stresses induced in the structure by the imposed long-term deformations (e.g. by shrinkage).

(4) Wind and snow actions must be neglected in the determination of the $E_d$ design stress for the calculated seismic situation (expression (5.4)).

## 5.6 Testing the resistance of concrete sections

### 5.6.1 Calculated value of resistance

(1) Where the resistance of a section depends on the effects of a multi-component action (e.g. bending moments, uniaxial or biaxial, and axial force), the ultimate limit state conditions specified in sections **5.6.2** and **5.6.3** may be satisfied by considering separately the extreme value (maximum or minimum) of each component of the effect of the action, with the concurrent values of all other components of the effect of the action.

### 5.6.2 Structures with limited ductility behaviour

(1) The following condition must be satisfied for the bending resistance of the sections:

$$E_d \leq R_d \tag{5.5}$$

Where

$E_d$ is the calculated stress corresponding to the calculated seismic situation, including second order effects; and

$R_d$ is the bending resistance of the section, as specified in section **6.1** of Annex 19 of the Structural Code and in section **5.6.1(1)** of this Annex.

(2) Testing of the shear resistance of concrete elements shall be carried out as specified in section **6.2** of Annex 19 of the Structural Code, with the following additional rules:

a) the design forces must be determined in accordance with section **5.5(1)**, where the effect of the seismic action $A_{Ed}$ must be multiplied by the behavioural coefficient $q$ used in the linear analysis.

b) The shear resistances $V_{Rd,c}$, $V_{Rd,s}$ y $V_{Rd,max}$, deduced in accordance with section **6.2** of Annex 19 of the Structural Code, must be divided by an additional partial safety factor against brittle failure, $\gamma_{Bd1}$ = 1.25.

### 5.6.3 Structures with ductile behaviour

#### 5.6.3.1 Flexural resistance of plastic hinges

(1) The following condition must be satisfied:

$$M_{Ed} \leq M_{Rd} \quad (5.6)$$

where

$M_{Ed}$ is the requesting bending moment, as derived from the analysis, for the calculated seismic situation, including second order effects; and

$M_{Rd}$ is the last moment of resistance of the section, as defined in section **5.6.1(1)**.

(2) The longitudinal reinforcement of the member containing the hinge must remain constant and fully active throughout the length $L_h$ shown in Figure 5.1 and specified in section **6.2.1.5**.

#### 5.6.3.2 Flexural resistance of sections outside the plastic hinges area

(1) The following condition must be satisfied:

$$M_C \leq M_{Rd} \quad (5.7)$$

where

$M_C$ is the moment of the capacity dimensioning defined in section **5.3**; and

$M_{Rd}$ is the bending resistance of the section, as specified in section **6.1** of Annex 19 of the Structural Code, taking into account the interaction of the other components of the effects of the calculated action (axial force and, where applicable, bending moment in the orthogonal direction).

NOTE As a consequence of section **5.3(5)**, the cross-section and longitudinal reinforcement of the plastic hinge section shall not be affected by the testing of the capacity design.

#### 5.6.3.3 Shear resistance of elements outside the plastic hinges area

(1) Testing of shear resistance shall be carried out in accordance with section **6.2** of Annex 19 of the Structural Code, with the following additional rules:

a) The calculated action stresses must be assumed to be equal to the capacity design stresses, as specified in section **5.3**.

b) The values of the shear resistance $V_{Rd,c}$, $V_{Rd,s}$ y $V_{Rd,max}$, deducted in accordance with section **6.2** of Annex 19 of the Structural Code, must be divided by an additional partial safety factor against brittle failure, $\gamma_{Bd}$. For the value of $\gamma_{Bd}$, one of the following two alternatives must be adopted:

Alternative 1:
$$1 \le \gamma_{Bd} = \gamma_{Bd1} + 1 - \frac{qV_{Ed}}{V_{C,o}} \le \gamma_{Bd1} \quad (5.8a)$$

Alternative 2: $1 \le \gamma_{Bd} = \gamma_{Bd1}$ (5.8b)

Where

$\gamma_{Bd1}$ is defined in section **5.6.2(2)**;

$V_{Ed}$ is the ultimate shear stress for the calculated seismic situation defined by the combination of section **5.5(1)**; and

$V_{C,o}$ is the capacity dimensioning shear force determined in accordance with section **5.3** without taking into account the limitation of section **5.3(2)**.

NOTE As shown in Figure 5.2, alternative 2 is more conservative. The choice between both alternatives 1 and 2 for use is up to the Project Author.

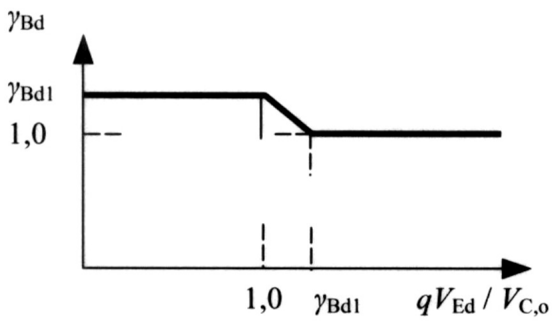



**Figure 5.2 – Alternative expressions (5.8a), (5.8b)**

(2) Unless a more precise calculation is made, for circular concrete sections of radius $r$ where the longitudinal reinforcement is distributed in a circle of radius $r_s$, a usable depth may be used:

$$d_e = r + \frac{2r_s}{\pi} \quad (5.9)$$

instead of $d$ in the corresponding expressions for shear resistance. The value of the effective arm $z$ can be assumed equal to: $z = 0.9\, d_e$.

### 5.6.3.4 Shear resistance of plastic hinges

(1) As specified in section **5.6.3.3(1)** applies.

(2) The angle $\theta$ between the compression rod and the main tensile reinforcement shall be assumed to be 45°.

(3) Instead of the dimensions $b_w$ and $d$ of the cross-section, the dimensions with respect to the axis of the perimeter frame of the confined concrete core must be taken.

(4) Section **5.6.3.3(2)** can be applied, using the dimensions of the confined concrete core.

(5) For elements with shear index $\alpha_s$ < 2.0 (see Table 4.1 for the definition of $\alpha_s$), testing of the pier against diagonal tension and creep failure shall be carried out in accordance with sections **5.5.3.4.3** and **5.5.3.4.4** of Annex 1, respectively. In these tests, the effects of the capacity design shall be taken as effects of the calculated actions.

### 5.6.3.5 Testing of nodes adjacent to plastic hinges

#### 5.6.3.5.1 General considerations

(1) Any node between a vertical ductile pier and the deck or a foundation element adjacent to a plastic hinge in the pier must be sized in shear to resist the effects of the capacity design of the plastic hinge in the direction considered. In the following sections, the piers are identified by the subscript "c" (for 'column or columnar'), while any other element that is embedded in the same node is considered as a "beam" and is identified by the subscript with "b".

(2) For any solid vertical pier of depth $h_c$ and width $b_c$ in the direction perpendicular to the bending direction of the plastic hinge, the effective width of the node must be assumed as follows:

- when the pier is embedded in a slab or in a transverse rib of a hollow-core slab:

$$b_j = b_c + 0.5\ h_c \tag{5.10}$$

- when the pier is embedded directly in a longitudinal web of width $b_w$ ($b_w$ is parallel to $b_c$):

$$b_j = \text{min.}\ (b_w;\ b_c + 0.5\ h_c) \tag{5.11}$$

- for circular piers of diameter $d_c$, the above definitions apply assuming $b_c = h_c = 0.9\ d_c$.

#### 5.6.3.5.2 Forces and stresses at the nodes

(1) The vertical design shear at the node, $V_{jz}$, must be assumed as:

$$V_{jz} = \gamma_o T_{Rc} - V_{b1C} \tag{5.12}$$

where

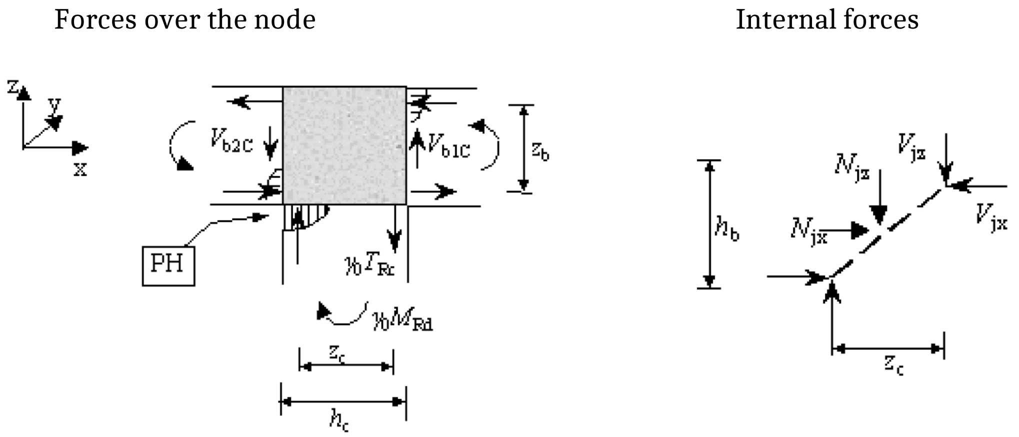
$T_{Rc}$ is the resultant force in the tensile reinforcement of the pier, corresponding to the bending resistance, $M_{Rd}$, of the plastic hinges defined in section **5.3(3)**, and $\gamma_o$ is the over-resistance coefficient, according to section **5.3(3)** and **(4)** (capacity design); and

$V_{1bC}$ is the shear force of the 'beam' adjacent to the tensile face of the column, corresponding to the effects of the capacity dimensioning of the plastic hinge.

(2) The horizontal design shear of the node, $V_{jx}$, can be determined as follows (see Figure 5.3):

$$V_{jx} = V_{jz} \frac{z_c}{z_b} \tag{5.13}$$

where $z_c$ and $z_b$ are, respectively, the mechanical arms of the plastic hinge and the end sections of the "beam", where $z_c$ and $z_b$ can be assumed to be equal to 0.9 times the edges of the corresponding effective sections (see **5.6.3.3** and **5.6.3.4**).




Legend:

PH Plastic hinge

**Figure 5.3 – Node forces**

(3) Testing against shear shall be carried out at the centre of the node where, in addition to $V_{jz}$ and $V_{jx}$, the influence of the following axial forces can be taken into account:

- vertical axial force at the node $N_{jz}$:

$$N_{jz} = \frac{b_c}{2b_j} N_{cG} \tag{5.14}$$

where

$N_{cG}$ is the axial force of the pier subjected to the non-seismic actions for the calculated seismic situation.

- horizontal force $N_{jx}$: equal to the effects of the axial force of the capacity design on the 'beam', including the effects of longitudinal prestressing net of all losses, if these axial forces are actually effective over the full width $b_j$ of the node;

- horizontal force $N_{jy}$: in the transverse direction, equal to the effect of the transverse prestressing net of all losses, effective inside the edge $h_c$ if such prestressing is provided for.

(4) The following average nominal stresses are used for testing the node:

Tangential tensions:

$$v_j = v_x = v_z = \frac{V_{jx}}{b_j z_c} = \frac{V_{jz}}{b_j z_b} \tag{5.15}$$

Axial stresses:

$$n_z = \frac{N_{jz}}{b_j h_c} \tag{5.16}$$

$$n_x = \frac{N_{jx}}{b_j h_b} \tag{5.17}$$

$$n_y = \frac{N_{jy}}{h_b h_c} \tag{5.18}$$

NOTE As indicated in section **5.3(6)**, the capacity dimensioning and consequently the corresponding testing of the nodes must be carried out considering the two signs of the seismic action (+ and -). It should also be noted that for bent nodes (e.g. on the end pier of a portal frame structure composed of multiple piers bending in the transverse direction of the bridge), the signs of $M_{Rd}$ and $V_{b1C}$ may be opposite to those shown in Figure 5.3 and that $N_{jk}$ may be a tensile stress.

#### 5.6.3.5.3 Testing

(1) If the mean shear stress at the node, $\nu_j$, does not exceed the shear cracking capacity of the node $\nu_{j,cr}$ given by expression (5.19), then the minimum reinforcement shall be provided as specified in **(6)**.

$$v_j \le v_{j,cr} = f_{ctd}\sqrt{\left(1+\frac{n_x}{f_{ctd}}\right)\left(1+\frac{n_z}{f_{ctd}}\right)} \le 1,50 f_{ctd} \tag{5.19}$$

where

$f_{ctd} = f_{ctk\,0.05}/\gamma_c$ is the calculated value of the tensile resistance of the concrete.

(2) The diagonal compression induced at the node by the diagonal compression connecting rod mechanism must not exceed the compressive strength of the concrete in the presence of transverse tensile deformations, also taking into account confining pressures and reinforcement.

(3) Unless a more accurate model exists, the requirement of **(2)** above is considered to be satisfied if the following condition is met:

$$v_j \le v_{j,Rd} = 0,50\alpha_c v f_{cd} \tag{5.20}$$

where

$$\nu = 0.6\,(1 - (f_{ck} / 250)) \qquad \text{(with } f_{ck} \text{ in MPa)} \tag{5.21}$$

The coefficient $\alpha_c$ in expression (5.20) takes into account the effects of any confining pressure ($n_{jy}$) and/or reinforcement ($\rho_y$) in the transverse direction $y$, on the compressive resistance of the diagonal connecting rod:

$$\alpha_c = 1 + 2\,(n_{jy} + \rho_y f_{sd})\,/\,f_{cd} \leq 1.5 \qquad (5.22)$$

where

$\rho_y$ = $A_{sy}/(h_c h_b)$ is the amount of reinforcement of any closed abutment in the direction transverse to the node in the plane of action (orthogonal to the plane of action); and

$f_{sd}$ = 300 MPa is a reduced stress of such reinforcement, for reasons of crack limitation.

(4) Reinforcement, both horizontal and vertical, shall be provided at the node in adequate quantity to resist the design shear stress. This requirement can be satisfied by providing horizontal and vertical reinforcement amounts, $\rho_x$ and $\rho_z$, respectively, such that:

$$\rho_x = \frac{v_j - n_x}{f_{sy}} \qquad (5.23)$$

$$\rho_z = \frac{v_j - n_z}{f_{sy}} \qquad (5.24)$$

where

$\rho_x = \frac{A_{sx}}{b_j h_b}$ is the amount of reinforcement of the node in the plane of action in the horizontal direction;

$\rho_z = \frac{A_{sz}}{b_j h_c}$ is the amount of reinforcement of the node in the plane of action in the vertical direction; and

$f_{sy}$ is the design yield strength of the knot reinforcement.

(5) The nodal reinforcement amounts $\rho_x$ and $\rho_y$ must not exceed the maximum value:

$$\rho_{máx} = \frac{\nu f_{cd}}{2 f_{sy}} \qquad (5.25)$$

where $\nu$ is given by expression (5.21).

(6) At the level of the node in the plane of action, a minimum amount of shear reinforcement must be provided in the form of closed abutments in both horizontal directions. The minimum amount required is:

$$\rho_{\text{min.}} = \frac{f_{\text{ctd}}}{f_{\text{sy}}} \tag{5.26}$$

#### 5.6.3.5.4 Arrangement of reinforcement

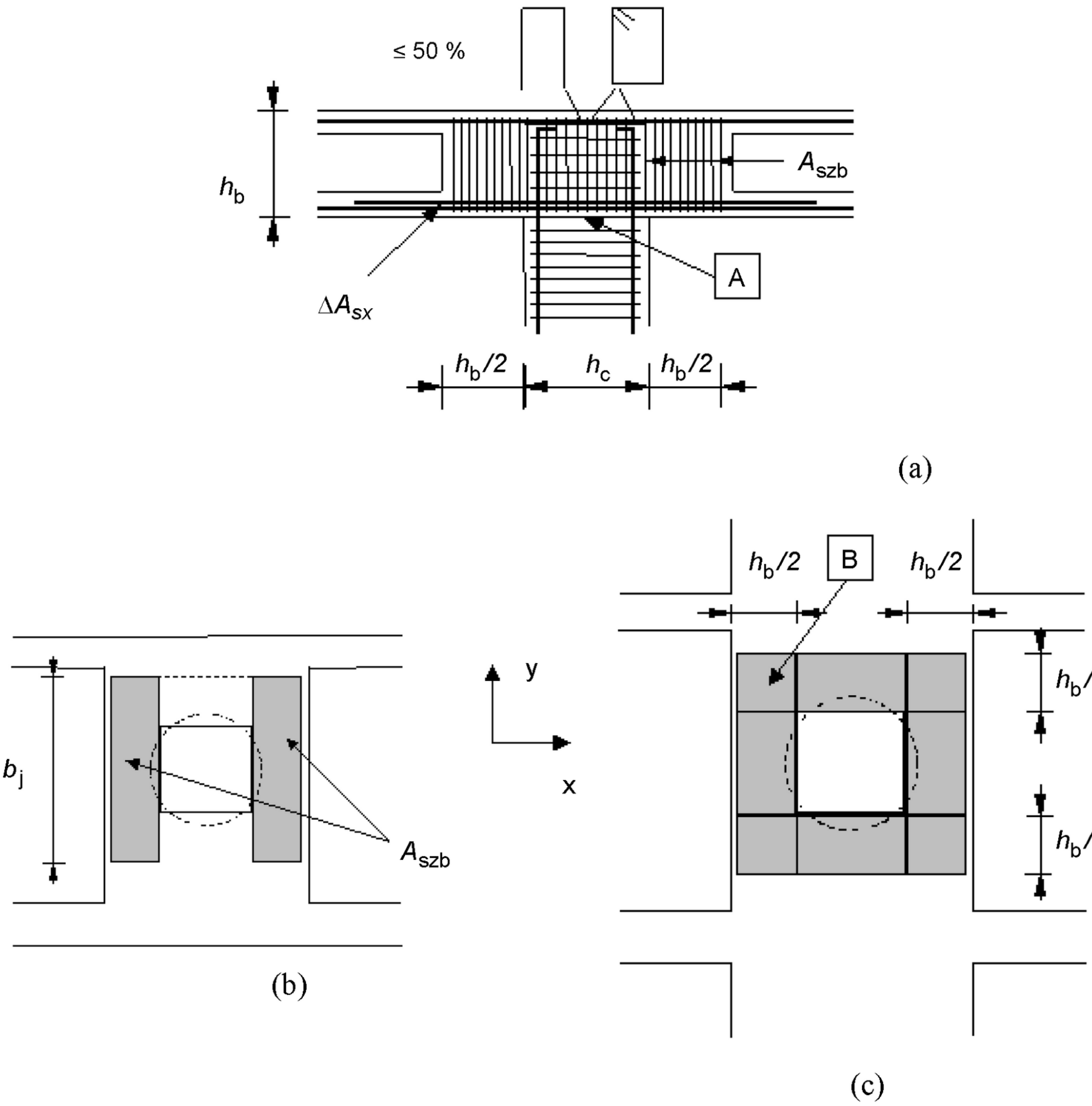
(1) The vertical abutments shall enclose the longitudinal reinforcement of the 'beam' located on the opposite side of the pier. The horizontal abutments shall enclose the vertical reinforcement of the pier as well as the horizontal bars of the 'beam' embedded in the node. It is recommended that the abutments / pier frames be extended inside the node.

(2) Up to 50 % of the total amount of vertical stirrups required at the junction may be U-bars enclosing the longitudinal reinforcement of the 'beam' on the opposite side of the pier (see Figure 5.4).

(3) In order to complete the required area of horizontal reinforcement of the node, $A_{sx}$, up to 50% of the longitudinal reinforcement, top and bottom, of the 'beam' may be taken into account, provided they are continuous through the node and are adequately anchored beyond the node.

(4) The longitudinal (vertical) reinforcement of the pier shall be introduced as far as possible into the interior of the 'beam', until exactly reaching the layers of reinforcement of the 'beam' located on the opposite side of the pier-beam interface. In the direction of the plastic hinge, the reinforcement of the two tensile zones of the pier shall be anchored by means of a rectangular pin directed towards the centre of the pier.

(5) When the required reinforcement area, $A_{sz}$ and/or $A_{sx}$, according to expressions (5.24) and (5.23), is so high as to make node construction difficult, then the alternative arrangement described in **(6)** and **(7)** can be applied (see Figure 5.4).




Legend:

A 'Beam' - pier interface

B The abutments placed in the common areas are considered in both directions.

**Figure 5.4 - Alternative nodal reinforcement arrangements;**
**(a) vertical section in the xz-plane;**
**(b) plan view of the plastic hinges formed in the x-direction;**
**(c) plan view of the plastic hinges in the x and y directions**

(6) Vertical abutments can be placed inside the body of the node to a constructively acceptable amount $\rho_{1z} \geq \rho_{min.}$. The remainder of the area $\Delta A_{sz} = (p_z - p_{1z})\ b_j h_c$, shall be located on each side of the 'beam', over the full width $b_j$ of the node and no further than 0.5 $h_b$ from the corresponding face of the pier.

(7) The horizontal reinforcement inside the body of the node may be reduced in $\Delta A_{sx} \leq \Delta A_{sz}$, if the amount of horizontal reinforcement remaining inside the body of the node satisfies the expression (5.26). The tensile reinforcement of the top and bottom fibres of the "beam" on both sides of the pier shall then be increased by an amount $\Delta A_{sx}$ above that required for the reinforcement in the corresponding sections of the "beam" for flexural testing under capacity sizing. To meet this requirement, additional reinforcement shall be placed over the full width $b_j$ of the node; such reinforcement shall be adequately anchored to be fully effective at a distance $h_b$ from the face of the pier.

#### 5.6.3.6 Testing the deck

(1) Testing should be carried out to ensure that no significant plasticisation occurs on the deck. Testing must be carried out:

- for bridges with limited ductility behaviour: for the most unfavourable effects of the calculated action, as specified in section **5.5**;

- for bridges with ductile behaviour: for the effects of capacity design, determined in accordance with section **5.3**;

(2) When the horizontal component of the seismic action in the transverse direction of the bridge is taken into account, the plastification of the deck due to bending in a horizontal plane is considered significant if the reinforcement of the top slab of the deck plastifies up to a distance from its end equal to 10% of the width of the top slab, or up to the junction of the top slab with a web, whichever limit is closer to the end of the top slab.

(3) When testing the deck on the basis of capacity sizing effects for seismic action acting in the transverse direction of the bridge, the significant reduction in the torsional stiffness of the deck, with increases in torsional moments, shall be taken into account. Unless a more precise calculation is carried out, the values specified in section **2.3.6.1(4)** may be accepted for bridges with limited ductility behaviour, or 70% of these values for bridges with ductile behaviour.

## 5.7 Testing of the resistance of steel and composite members

### 5.7.1 Steel piers

#### 5.7.1.1 General considerations

(1) Section **5.6.1(1)** applies for testing the pier against the effects of multi-component actions.

(2) Energy dissipation is only allowed in the piers, not in the deck.

(3) For bridges designed with ductile behaviour, the provisions of sections **6.5.2**, **6.5.4 and 6.5.5** of Annex 1 for dissipative structures apply.

(4) The provisions of section **6.5.3** of Annex 1 apply. However, class 3 cross-sections are only permitted when $q \leq 1.5$.

(5) The provisions of section **6.9** of Annex 1 apply to all bridge piers.

#### 5.7.1.2 Piers as bending-resistant portal frames

(1) For bridges designed for ductile behaviour, the design axial forces $N_{Ed}$ and design shear forces $V_{Ed}$ exerted on the piers consisting of bending-resisting frames shall be assumed to be equal respectively to the effects of the capacity design action, $N_c$ and $V_c$, as specified in section **5.3**.

(2) The design of the cross-sections of the plastic hinges of both beams and piers of the pier must satisfy the provisions of sections **6.6.2**, **6.6.3** and **6.6.4** of Annex 1, taking the values of $N_{Ed}$ and $V_{Ed}$ as specified in **(1)**.

#### 5.7.1.3 Piers as portal frames with centred triangulations

(1) The provisions of Annex 1 apply with the following modifications for bridges designed for ductile behaviour:

- The calculated values for axial shear force shall be in accordance with section **5.3**, considering the force exerted on all diagonals as corresponding to the overstrength $\gamma_o N_{pl,Rd}$ of the weakest diagonal (see **5.3** for $\gamma_o$);
- The second part of the expression (6.12) in section **6.7.4** of Annex 1 is to be replaced by the action for capacity sizing $N_{Ed} = N_C$.

#### 5.7.1.4 Piers as off-centre triangulated gantries

(1) The provisions of section **6.8** of Annex 1 apply.

### 5.7.2 Steel or composite decking

(1) On bridges designed for ductile behaviour ($q > 1.5$), the deck must be tested for capacity sizing purposes as specified in section **5.3**. For bridges designed for limited ductility behaviour ($q \leq 1.5$), testing of the deck must be carried out using the calculated action effects obtained in the analysis, according to expression (5.4). Testing may be carried out in accordance with the relevant rules of Annex 29 of the Structural Code or Annex 32 of the Structural Code, applicable to steel or composite decks, respectively.

## 5.8 Cementing

### 5.8.1 General considerations

(1) Bridge foundation systems must be designed to meet the general requirements set out in section **5.1** of Annex 5. Bridge foundations should not be used intentionally as sources of hysteresis energy dissipation and should therefore be designed to remain elastic under calculated seismic action as far as possible.

(2) Where necessary, soil-structure interaction should be assessed using the relevant provisions of Chapter **6** of Annex 5.

### 5.8.2 Effects of calculated actions

(1) For the purpose of testing the resistance, the effects of calculated actions on foundations shall be determined as specified in **(2)** to **(4)**.

(2) Bridges with limited ductility behaviour ($q \leq 1.5$) and bridges with seismic isolation devices:

The effects of the calculated actions should be as obtained from expression (5.4), taking the seismic effects obtained from the linear analysis of the structure for the calculated seismic situation as specified in section **5.5**, with the results of the analysis for the calculated seismic action multiplied by the coefficient $q$ employed (i.e. effectively using $q = 1$).

(3) Bridges with ductile behaviour ($q > 1{,}5$):

The effects of the design actions must be obtained by applying to the piers the design method based on the capacity sizing criteria as specified in section **5.3**.

(4) The provisions of requirement **(e)** in section **4.2.4.4(2)** apply to bridges calculated by non-linear analysis.

### 5.8.3 Resistance testing

(1) Testing of the resistance of foundations shall be carried out as specified in sections **5.4.1** (shallow foundations) and **5.4.2** (piles and piers) of Annex 5.

# 6 Construction details

## 6.1 General considerations

(1) The rules of this chapter apply only to bridges designed for ductile behaviour and are intended to ensure a minimum level of ductility in bending/rotation in plastic hinges.

(2) In section **6.5** the rules for the construction details of critical sections and non-ductility elements specific to bridges with limited ductility behaviour are specified.

(3) In general, no plastic hinges are allowed to form in the deck. Consequently, there is no need to apply any special construction detailing rules other than those applied for the design of bridges for non-seismic actions.

## 6.2 Concrete piers

### 6.2.1 Confinement

#### 6.2.1.1 General requirements

(1) The ductile behaviour of the compressed zone of the concrete inside the potential plastic hinge regions must be ensured.

(2) In regions of potential plastic hinges, where the reduced or normalised axial force (see section **5.3(3)**) exceeds the limit:

$$\eta_k = N_{Ed}/A_c f_{ck} > 0.08 \tag{6.1}$$

confinement of the compression zone shall be provided, in accordance with the provisions of section **6.2.1.4**, except for the case specified in **(3)**.

(3) No confinement is required in the piers if, under the ultimate limit state or collapse conditions, a ductility in terms of curvatures $\mu_\Phi$ = 13 for bridges with ductile behaviour, or $\mu_\Phi$ = 7 for bridges with limited ductility, can be achieved with a maximum unit deflection of the compressed concrete not exceeding the value:

$$\varepsilon_{cu2} = 0.35\ \% \tag{6.2}$$

NOTE The condition in **(3)** above can be achieved in cross-section piers containing flanges, when the compressed zone extends over a sufficient area of the flanges.

(4) In cases of deep compressed zones the confinement shall extend at least to the depth where the compressive unit strain exceeds $0.5\varepsilon_{cu2}$.

(5) The mechanical amount of transverse confining reinforcement is defined by:

$$\omega_{wd} = \rho_w \cdot f_{yd}/f_{cd} \tag{6.3}$$

where

(a) For rectangular sections,

$\rho_w$ is the geometric amount of confining reinforcement, defined as:

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

where

$A_{sw}$ is the total area of the trusses or forks in the direction of confinement considered;

$s_L$ is the spacing between trusses or forks in the longitudinal direction;

$b$ is the dimension of the confined concrete core perpendicular to the direction of confinement under consideration, measured between the outer parts of the perimeter frame.

(b) In circular sections:

The geometric confinement amount for circular sections $\rho_w$ relative to the confined concrete core is used:

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

where

$A_{sp}$ is the area of the spiral or frame;

$D_{sp}$ is the diameter of the spiral or frame;

$s_L$ is the interval between the reinforcement.

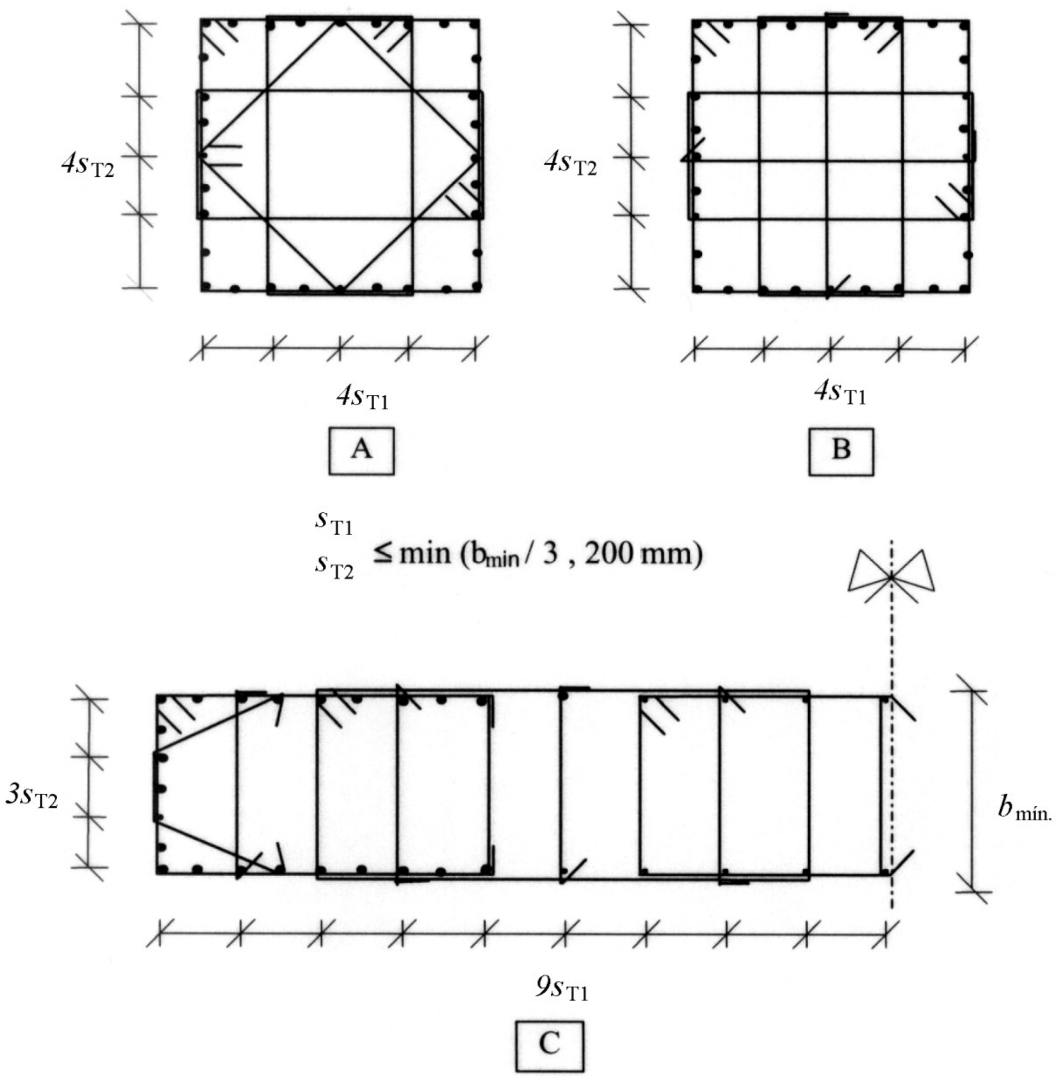
#### 6.2.1.2 Rectangular sections

(1) The spacing, $s_L$, between frames or transverse ties in the longitudinal direction must satisfy both of the following conditions:

- $s_L \leq 6$ times the diameter of the longitudinal reinforcement, $d_{bL}$;
- $s_L \leq 1/5$ of the smallest dimension of the confined concrete core, measured in relation to the axis of the frames.

(2) The transverse distance, $s_T$, between the two branches of the frames or between the additional transverse ties must not exceed 1/3 of the smallest dimension, $b_{min.}$, of the concrete core, measured in relation to the axis of the frames, nor 200 mm (see Figure 6.1a).

(3) It must be assumed that reinforcement inclined at an angle $\alpha > 0$ with respect to the transverse direction for which $\rho_w$ is calculated, contributes to the total area $A_{sw}$ of expression (6.4) with its area multiplied by $\cos\alpha$.




Legend:

A 4 overlapping frames

B 3 overlapping frames plus transverse ties

C Overlapping frames plus cross ties

**Figure 6.1.a - Typical construction details of confinement in rectangular section concrete piers using overlapping rectangular frames and transverse ties**

### 6.2.1.3 Circular sections

(1) The spacing of the frames or the pitch of the spiral, $s_L$, must satisfy both of the following conditions:

$s_L \leq$ 6 times the diameter of the longitudinal reinforcement, $d_{bL}$;

$s_L \leq$ 1/5 of the diameter of the confined concrete core, measured with respect to the axis of the frames.

#### 6.2.1.4 Necessary confining reinforcement

(1) The confinement is provided either by rectangular frames and/or cross ties, or by circular or spiral frames.

(2) The minimum amount of confinement reinforcement must be determined as follows:

- for rectangular frames and cross ties:

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

Where

$$\omega_{w,req} = \frac{A_c}{A_{cc}} \lambda\eta_k + 0.13 \frac{f_{yd}}{f_{cd}} (\rho_L\text{-}0.01) \tag{6.7}$$

Where

$A_c$ is the area of the gross concrete section

$A_{cc}$ is the area (of the core) of confined concrete, corresponding to the section measured with respect to the axis of the frames;

$\omega_{w,min.}$, $\lambda$ are the coefficients specified in Table 6.1; and

$\rho_L$ is the geometric amount of longitudinal reinforcement.

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

**Table 6.1 - Minimum values of $\lambda$ and $\omega_{w,min.}$**

| Seismic behaviour | $\lambda$ | $\omega_{w,min.}$ |
|---|---|---|
| Ductile | 0.37 | 0.18 |
| Limited ductility | 0.28 | 0.12 |

- for circular or spiral frames:

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

(3) Where rectangular frames and cross ties are used, the minimum reinforcement condition shall be satisfied in each of the two transverse directions.

(4) Interlocking spirals/frames are quite efficient for confining approximately rectangular sections. The distance between the centres of the interlocking spirals/frames must not exceed 0.6 $D_{sp}$, where $D_{sp}$ is the diameter of the spiral/frame (see Figure 6.1b).

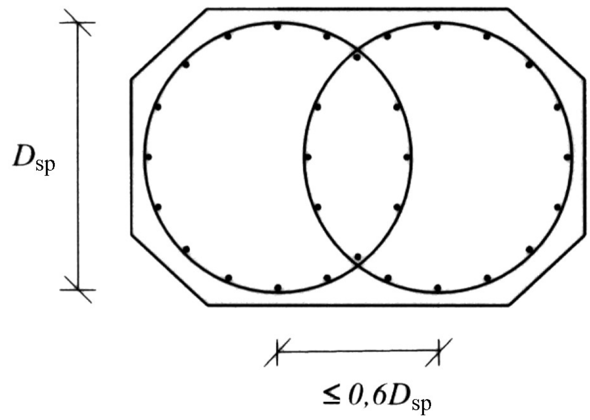



**Figure 6.1b – Typical construction details for the confinement of concrete piers using interlocking spirals/frames**

### 6.2.1.5 Size of the confinement - Length of potential plastic hinges

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

- edge of the pier in the plane of bending (perpendicular to the axis of the hinge);
- distance from the point of maximum moment to the point where the design moment is less than 80% of the maximum.

(2) When the reduced axial deflection is $0.6 \geq \eta_k > 0.3$, the design length of the potential plastic hinges, determined as in **(1)**, must be increased by 50 per cent.

(3) The design length of the plastic hinges ($L_h$), as defined above, shall be used exclusively for the construction details of the plastic hinge reinforcement. It shall not be used to estimate the rotation of the plastic hinges.

(4) Where confining reinforcement is required, the amount specified in section **6.2.1.4** shall be placed along the full length of the plastic hinge. Outside this length, the transverse reinforcement may be gradually reduced to the amount required by other criteria. The amount of transverse reinforcement along an additional length, $L_h$, adjacent to the theoretical end of the plastic hinge, shall not be less than 50% of the amount required in the plastic hinge.

### 6.2.2 Buckling of compressed longitudinal reinforcement

(1) Buckling of longitudinal reinforcement in areas where buckling can potentially occur must be avoided, even after several cycles in the post-elastic domain.

(2) To satisfy the requirement of **(1)** above, all main longitudinal reinforcement shall be restrained against outward buckling by arranging transverse reinforcement (frames or transverse ties) perpendicular to the longitudinal members and with a (longitudinal) spacing $s_L$ not exceeding $\delta d_{bL}$, where $d_{bL}$ is the diameter of the longitudinal members. The coefficient $\delta$ depends on the ratio $f_t$ / $f_y$ between the tensile resistance $f_{tk}$ and the yield strength $f_{yk}$ of the transverse reinforcement, in terms of characteristic values, according to the following relation:

$$5 \leq \delta = 2.5\ (f_{tk}/f_{yk}) + 2.25 \leq 6 \tag{6.9}$$

(3) Along the straight contours of the cross-section, anchorages shall be provided to the longitudinal reinforcement in one of the following ways:

a) by means of a perimeter clevis supported by transverse ties placed alternately on the longitudinal members, with a transverse (horizontal) $s_t$ spacing of not more than 200 mm. The transverse ties shall have side rails bent at 135° at one end, and at 135° or 90° at the other end. Cross-ties with 135° bends at both ends may consist of two pieces spliced together by overlapping. If $\eta_k$ > 0.30, transverse ties with 90° turnbuckles are not permitted. If the cross ties have unequal pins at both ends, these pins must be alternated in the adjoining cross ties, both horizontally and vertically. In large cross-sections, the perimeter cross-tie may be spliced using an appropriate overlap length combined with pins;

b) by means of overlapping closed clevises, arranged so that every corner bar, and at least alternatively every second internal longitudinal reinforcement, is linked by a clevis. The transverse (horizontal) spacing $s_T$ of the clevises shall not exceed 200 mm.

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

$$\min\left(\frac{A_t}{s_L}\right) = \frac{\sum A_s f_{ys}}{1{,}6\ f_{yt}}\ (\mathrm{mm}^2/\mathrm{m}) \tag{6.10}$$

Where

$A_t$ is the area of a clevis, in mm$^2$;

$s_L$ is the spacing of the clevises about the element axis, in m;

$A_s$ is the sum of the areas of the longitudinal reinforcement restrained by the clevises, in mm$^2$;

$f_{yt}$ is the yield strength of the clevises; and

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

### 6.2.3 Other rules

(1) Due to the potential risk of loss of cover in the area of plastic hinges, the confining transverse reinforcement shall be anchored by means of a 135° bent pin (unless a 90° bent pin is used in accordance with section **6.2.2(3)**), surrounding a longitudinal bar and with a suitable length (minimum 10 diameters) inside the concrete core.

(2) A similar anchorage or butt weld joint is required for the overlap of spirals or clevises inside the potential plastic hinge areas. In this case the overlaps of successive spirals or frames, when located along the perimeter of the member, shall be staggered as specified in section **8.7.2** of Annex 19 of the Structural Code.

(3) No overlapping or welding of longitudinal reinforcement is permitted within the plastic hinge zone. For mechanical couplers, see section **5.6.3(2)** of Annex 1.

### 6.2.4 Hollow piers

(1) In cases of low seismicity it is not necessary to apply the rules in **(2)** to **(4)**.

NOTE For cases of low seismicity, the notes in section **2.3.7(1)** apply.

(2) Unless adequate justification is provided, the slenderness or ratio $b/h$ between the free length of the walls, $b$, and the wall thickness, $h$, shall not exceed 8 in the area of plastic hinges (of length $L_h$ in accordance with section **6.2.1.5**) of hollow piers of box section, single or multi-cell.

(3) For cylindrical hollow piers, the limitation of **(2)** above applies to the quotient $D_i/h$, where $D_i$ is the internal diameter.

(4) For single or multi-cell box section piers, and where the value of the quotient $\eta_k$ defined in expression (6.1) does not exceed 0,20, testing of the confining reinforcement, as specified in section **6.2.1**, is not required, provided that the requirements of section **6.2.2** are met.

## 6.3 Steel piers

(1) For bridges designed for ductile behaviour, the construction detail rules of sections **6.5**, **6.6**, **6.7** and **6.8** of Annex 1 shall apply, with the modifications of section **5.7** of this Annex.

## 6.4 Foundations

### 6.4.1 Surface foundations

(1) Shallow foundations, such as footings, slabs, caissons, piers, etc., must not enter a plastic regime under calculated seismic action and therefore do not require special reinforcement details.

### 6.4.2 Pile foundations

(1) When using the capacity dimensioning method (see section **5.3**), it is not possible to avoid the location of ball joints in the piles, the integrity of the pile and the ductile behaviour must be ensured. In this case the following rules apply.

(2) The following pile locations shall be treated as potential locations of plastic hinges:

a) At the level of the pile heads adjacent to the pile cap, when the rotation of the pile cap around a horizontal axis transverse to the direction of seismic action is constrained by the high rigidity of the pile group for this degree of freedom.

b) At the depth at which the maximum bending moment develops in the pile. This depth shall be estimated by an analysis taking into account the effective bending stiffness of the pile (see **2.3.6.1**), the lateral stiffness of the soil and the rotational stiffness of the pile group at the pile cap height.

c) At the level of the contact surfaces between the different soil layers with markedly different shear deformability due to soil-pile kinematic interaction (see **5.4.2(1)** of Annex 5).

(3) In locations of type (a) in **(2)** above, confining reinforcement shall be provided in the amount specified in section **6.2.1.4**, along a vertical length equal to 3 times the pile diameter.

(4) Unless a more precise analysis is carried out, longitudinal reinforcement, as well as confining reinforcement, in the same amount for each of these types as required at the pile head, shall be provided along a length of twice the pile diameter on each side of the point of maximum moment at locations of type (b) in **(2)** above, and on each side of the contact surface at locations of type (c) in **(2)** above.

## 6.5 Structures with limited ductility behaviour

### 6.5.1 Testing of the ductility of critical sections

(1) The following rules shall be applied to critical sections of structures designed for limited ductility behaviour (with $q \leq 1.5$) in order to ensure a minimum of limited ductility.

NOTE These rules shall apply even in areas of low seismicity. For the definition of low seismicity zones, see NOTE 1 to section **2.3.7(1)**.

(2) A section is considered critical, i.e. location of a potential plastic hinge, when:

$$M_{Rd} \,/\, M_{Ed} < 1.30 \qquad (6.11)$$

Where

$M_{Ed}$ is the maximum requesting bending moment of the section for the calculated seismic situation;

$M_{Rd}$ is the minimum bending resistance of the section for the same calculated seismic situation.

(3) Where possible, the locations of potential plastic hinges shall be accessible for inspection.

(4) Unless confining reinforcement is not required in accordance with section **6.2.1.1(3)**, concrete elements must be provided with the confining reinforcement required in section **6.2.1.4** for constructions of limited ductility (see table 6.1). In such cases, buckling of the longitudinal reinforcement must also be prevented, in accordance with the provisions of section **6.2.2**.

### 6.5.2 Prevention of brittle failures of specific components without ductility

(1) Non-ductile structural components, such as fixed bearings, end supports and anchorages of cables and stay cables, as well as other non-ductile connections, must be sized using either the seismic action stresses multiplied by the coefficient $q$ used in the analysis or the stresses obtained according to the capacity sizing criteria. The latter should be calculated from the resistance of the relevant ductile elements (e.g. cables), and a coefficient of over-resistance (reserve resistance) of at least 1.3.

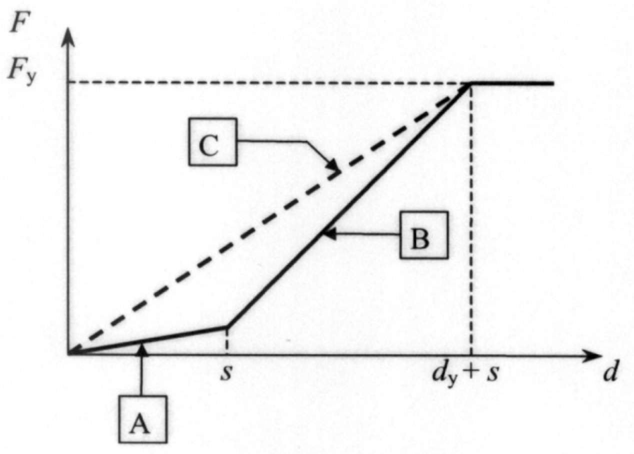
(2) Testing may be omitted if it can be demonstrated that the integrity of the structure is not affected by the failure of such joints. This demonstration should also address the possibility of sequential failure, such as may occur in the stay cables of cable-stayed bridges.

## 6.6 Seismic bearings and couplings

### 6.6.1 General requirements

(1) Non-seismic horizontal actions on the deck must be transmitted to the supporting elements (abutments or piers) through the structural connections, which may be monolithic or by means of support devices. For non-seismic actions, the support devices must be tested in accordance with the specific regulations in force or, failing that, with the specific technical documents that the author of the project, under his responsibility, considers most appropriate. Likewise, the provisions of the UNE-EN 1337-2 and UNE-EN 1337-3 standards must be complied with.

(2) In general, the calculated seismic action must be transmitted through the supporting apparatus. However, seismic couplings (as specified in section **6.6.3**) may be used to transmit the full calculated seismic action, provided that the effects of the dynamic impact are mitigated and taken into account in the design. Seismic couplings shall generally allow for bridge displacements due to the remaining non-seismic actions without transmitting significant loads. Where seismic couplings are used, the connection between the deck and the substructure must be adequately taken into account in the model of the structure. As a minimum, a linear approximation of the force-displacement relationship of the connected structure must be used (see Figure 6.2).




Legend:

$s$ Connector clearance

$d_y$ Maximum elastic displacement of the load-bearing element

A Rigidity of the bearing apparatus

B Rigidity of the supporting member

C Linear approximation of the curve

**Figure 6.2 – Force-displacement relationship for a structure with seismic couplings**

NOTE Certain types of seismic couplings may not be applicable to bridges subject to large non-seismic horizontal actions, or to bridges with special displacement limitations, such as railway bridges.

(3) The structural integrity of the bridge must be assured in the face of extreme seismic displacements. For fixed bearings, this requirement must be established either by capacity dimensioning of the normal bearings (see **6.6.2.1**), or by fitting additional connectors as a second line of defence (see section **6.6.2.1(2)** and section **6.6.3.1(2)**, case (b)). For mobile connections, appropriate delivery (seat) lengths shall be established in accordance with section **6.6.4**. In cases of bridge rehabilitation, seismic couplings may be used as an alternative.

(4) All types of seismic bearings and couplings must be accessible for inspection and maintenance and easily replaceable.

### 6.6.2 Supporting devices

#### 6.6.2.1 Stationary supporting appliances

(1) Except where the conditions of **(2)** are met, the effects of the calculated seismic action on fixed bracing shall be determined using the capacity sizing criteria.

(2) Fixed bracing may be sized only for the effects of the calculated seismic situation obtained from the analysis, provided that it can be replaced without difficulty and seismic couplings are added as a second line of defence.

#### 6.6.2.2 Mobile supporting devices

(1) The movable supporting device shall accept, without damage, the total calculated seismic displacement in the calculated seismic situation determined in accordance with section **2.3.6.3(2)**.

#### 6.6.2.3 Elastomeric supporting devices

(1) Elastomeric bearings may be used in one of the following configurations:

a) on isolated bearing elements, to accommodate the imposed deflections and resist only the non-seismic horizontal actions, while the calculated seismic action is resisted by structural connections (monolithic or by fixed devices) of the deck to other bearing elements (piers or abutments);

b) on all or some isolated bearing elements, with the same function as in (a) above, in combination with seismic couplings designed to resist the seismic action;

c) on all load-bearing members to resist both seismic and non-seismic actions.

(2) Elastomeric bearing devices used in configurations (a) and (b) of **(1)** above shall be designed to resist the maximum shear deformation due to the calculated seismic action in accordance with section **7.6.2(5)**.

(3) Significant damage to the elastomeric bearing devices described in **(2)** above is acceptable if the conditions specified in section **2.2.2(5)** are met. However, bridges whose elastomeric devices are susceptible to damage must retain their functionality of transferring vertical loads between the deck and the support to allow the movement of emergency vehicles.

(4) The seismic behaviour of bridges, where the elastomeric devices on all supports fully resist the calculated seismic action (configuration **(c)** in **(1)** above), is controlled by the high flexibility of the supports. These bridges and their devices must be sized in accordance with Chapter **7**.

### 6.6.3 Seismic couplings, anchorage devices to prevent uplift, and impact transmission units.

#### 6.6.3.1 Seismic couplings

(1) Seismic couplings may consist of shear keys, stops, and/or bolts or tie wires. Frictional joints are not considered as effective joints.

(2) Seismic couplings are required in the following cases.

a) In combination with elastomeric bearing devices, where the connectors are designed to transmit the calculated seismic action.

b) Combined with fixed devices not designed to the capacity design criteria.

c) In the longitudinal direction, at the location of movable end supports, between the deck and the abutment or pier of bridges under restoration, if the minimum delivery length requirements of section **6.6.4** are not met.

d) Between adjoining deck sections, at intermediate separation joints (located within the span).

(3) The calculated actions for the seismic couplings in the above items are to be determined as follows.

- In cases (a), (b) and (c) of **(2)** above, with the capacity sizing criteria (the horizontal resistance of the supporting devices must be assumed to be zero).

- In case (d) of the same **(2)**, and unless a more precise analysis is carried out taking into account the dynamic interaction of the contiguous sections of the deck, the connectors can be sized for an action equal to 1.5 $\alpha_g S M_d$, where $\alpha_g$ is the acceleration of the design soil in a terrain type A, S is the soil coefficient of section **3.2.2.2** of Annex 1, and $M_d$ is the mass of the deck section attached to a pier or abutment, or the lesser of the masses of the two parts of the deck on either side of an intermediate separation joint.

(4) Connectors shall be provided with adequate clearances or margins to allow them to remain inactive:

- under calculated seismic action in cases (c) and (d) of **(2)**,

- under any non-seismic action in case (a) of **(2)**.

(5) Where seismic couplings are used, means shall be provided to reduce the impact effects.

#### 6.6.3.2 Anchorage devices to prevent uplifting

(1) Anchorage devices to prevent uplift shall be provided on all supports where the total vertical reaction due to the calculated seismic action opposes and exceeds by a percentage, $p_H$, the compressive (downward) reaction due to the permanent load.

The values to be assigned to $p_H$ are as follows:

- $p_H$ = 80 % for bridges designed for ductile behaviour, where the vertical reaction due to the calculated seismic action is determined by the design criteria by capacity;

- $p_H$ = 50 % for bridges designed for limited ductility behaviour, where the vertical reaction due to the calculated seismic action is determined from an analysis for the calculated seismic action only (including the contribution of the vertical seismic component).

(2) The requirement of **(1)** above refers to the total vertical reaction of the deck per line of supports and does not apply to isolated devices of the same bearing element. However, no debonding of isolated devices may occur for the calculated seismic situation in accordance with section **5.5**.

#### 6.6.3.3 Shock transmission units

(1) Shock transmission units (STU) are devices that establish constraints as a function of the speed of the relative displacement between the deck and the supporting element (pier or abutment), as follows:

- For low speed movements ($v < v_1$), such as those due to temperature effects, or creep and shrinkage of the deck, the movement is practically free (with very low reaction).

- For high speed movements ($v > v_2$), such as those due to seismic or braking actions, the movement is blocked and the device acts practically as a rigid joint.

- The units may also have a force limiting function, which limits the force they transmit (for $v > v_2$) to a certain upper threshold, $F_{max.}$, beyond which movement occurs.

NOTE The properties and dimensioning of the impact transmission units (STU) are given in Standard UNE-EN 15129 (anti-seismic devices). The order of magnitude of the velocities mentioned above is $v_1 \cong 0.1$ mm/s, $v_2 \cong 1.0$ mm/s.

(2) The complete description of the laws defining the behaviour of the units used (force-displacement and force-velocity relationships) must be available during the design phase (supplied by the manufacturer of the units), including any influence of environmental factors on this behaviour (mainly temperature, ageing, cumulative travel). All parameter values necessary for the definition of the behaviour of the units (including the values of $v_1$, $v_2$ and $F_{max.}$ for the cases mentioned in point **(1)**), as well as the geometrical data and the calculated value of the resistance, $F_{Rd}$, of the units and their connections, must also be available. This information shall be based on appropriate official test results, or on a European Assessment Document (EAD).

(3) Where impact transmission units without the force limiting function are used to resist seismic forces, they shall have the following calculated value of resistance, $F_{Rd}$:

- for ductile bridges: $F_{Rd}$ shall not be less than the corresponding reaction for capacity design purposes;

- for bridges with limited ductility: $F_{Rd}$ shall not be less than the reaction obtained in the analysis due to the calculated seismic action, multiplied by the coefficient $q$ used.

The devices must provide sufficient displacement capacity for all low velocity actions and must maintain their capacity in terms of reaction force in their displaced state.

(4) Where impact transmission units acting as force limiters are used to resist seismic forces, the devices shall have a displacement capacity sufficient to absorb the calculated value of the relative displacement, $d_{Ed}$, corresponding to the calculated seismic situation determined in accordance with section **2.3.6.3(2)**, or in accordance with section **7.6.2(2)** for bridges with seismic isolation.

(5) All impact transmission units shall be accessible for inspection and for maintenance/replacement.

### 6.6.4 Minimum clearances

(1) A minimum clearance must be provided for supports where the relative displacement between the bearing and supported elements is designed for seismic conditions.

(2) The clearance length should be such as is necessary to ensure that the bearing function of the support is maintained under extreme seismic displacements.

(3) The minimum clearance length, $l_{ov}$, at the location of an abutment end support can be estimated as follows:

$$l_{ov} = l_m + d_{eg} + d_{es} \tag{6.12}$$

$$d_{eg} = \varepsilon_e L_{eff} \le 2d_g \tag{6.13}$$

$$\varepsilon_e = 2\frac{d_g}{L_g} \quad (6.14)$$

Where

$l_m$ is the minimum length of the support capable of guaranteeing the totally safe transmission of the vertical reaction, never less than 400 mm;

$d_{eg}$ is the effective relative displacement of the two parts due to the spatial variation of the seismic ground displacement. When the bridge site is within 5 km of a known seismically active fault capable of causing an earthquake of magnitude $M_w \geq 6.5$, and unless a specific seismological investigation is available, twice the value of $d_{eg}$ obtained from expression (6.13) shall be taken as the value of $d_{eg}$;

$d_g$ is the calculated value of ground displacement as specified in section **3.2.2.4** of Annex 1;

$L_g$ is the distance parameter specified in section **3.3(6)**;

$L_{eff}$ is the effective length of the deck, taken as the distance between the deck joint in question and the nearest point where there is a rigid connection of the deck to the substructure. If the deck is rigidly connected to a group of more than one pier, $L_{eff}$ must then be taken as the distance between the support and the centre of the group of piers. In this context, "rigid connection" means a connection of the deck or deck section to an element of the substructure, either monolithically, or by means of fixed support devices, seismic couplings, or impact transmission units without force limiting function;

$D_{en}$ is the effective seismic displacement of the support due to deformation of the structure, estimated as follows.

- For decks attached to piers, either monolithically or by means of fixed support devices acting as full seismic couplings:

$$d_{es} = d_{Ed} \tag{6.15a}$$

- where $d_{Ed}$ is the total calculated value of the longitudinal displacement corresponding to the ultimate design earthquake in the seismic situation, determined in accordance with the expression (2.7) in section **2.3.6.3**.

- For decks connected to the piers or to an abutment by seismic couplings with a clearance equal to $s$:

$$d_{es} = d_{Ed} + s \tag{6.15b}$$

(4) If there is an intermediate separation joint between two deck sections, $l_{ov}$ shall be estimated by taking the square root of the sum of the squares of the calculated values for each of the two deck sections in accordance with **(3)** above. At the location of an end support of a deck section on an intermediate pier, the minimum clearance $l_{ov}$ shall be taken as the value estimated in accordance with **(3)** above, plus the maximum displacement of the top of the pier, $d_E$, for the calculated seismic situation.

## 6.7 Concrete abutments and retaining walls

### 6.7.1 General requirements

(1) All critical structural elements of abutments must be designed to remain essentially elastic under the calculated seismic action. The design of the foundation must be in accordance with section **5.8**. Depending on the structural function of the horizontal connection between the abutment and the deck, the provisions of sections **6.7.2** and **6.7.3** apply.

NOTE For controlled damage to side walls or abutment laps, see section **2.3.6.3(5)**.

### 6.7.2 Flexible union between abutments and deck

(1) In abutments flexibly connected to the deck, the deck is supported by sliding or elastomeric bearings. Elastomeric bearings (or seismic couplings, if present) may be designed to contribute to the seismic resistance of the deck, but not to that of the abutments.

(2) The seismic design of such abutments shall take into account the following actions, which are assumed to act in phase.

a) The thrusts of the terrain, including the seismic effects calculated in accordance with Annex 5, Chapter **7**.

b) Inertia forces acting on the mass of the abutment and on the mass of the existing backfill on its foundation. In general, these effects can be determined on the basis of the calculated value of the acceleration corresponding to the ground surface at the location, $a_g S$.

c) The actions transmitted by the supporting devices, determined by the capacity design criteria if a ductile behaviour of the bridge has been assumed, according to section **5.3(7)** and section **5.3(8)**. If the bridge is designed for $q$ = 1.0, then the reactions on the supporting devices, resulting from the seismic analysis, must be considered.

(3) Where the terrain thrusts assumed in **(2)**(a) above are calculated in accordance with Annex 5 on the basis of an acceptable displacement of the abutment, provision shall be made to allow for such displacement by calculating the clearance between the deck and the abutment wall bounding the deck joint. In this case, it must also be ensured that the displacement assumed for calculating the actions in **(2)**(a) above can actually occur before potential failure of the stirrup itself. This requirement can be considered to be satisfied if the abutment body is dimensioned by increasing the seismic part of the actions in **(2)**(a) above by 30 %.

### 6.7.3 Abutments rigidly connected to the deck

(1) The connection of the abutment to the deck is considered rigid if it is either monolithic, or if it is made through fixed support devices or connectors designed to resist seismic action. Such abutments have an important contribution to the seismic resistance, both in the longitudinal and transverse direction.

(2) The mechanical analysis model shall incorporate the effect of the interaction between the soil and the abutments, using either realistic values of the relevant soil stiffness parameters or values corresponding to the upper and lower stiffness limits.

(3) Where both piers and abutments provide the seismic resistance of the bridge, the use of estimated values of the upper and lower limits of soil stiffness is recommended in order to achieve safety-side results for both piers and abutments.

(4) A partial safety factor $q$ = 1.5 should be used in the analysis of the bridge.

(5) The following actions in the longitudinal direction shall be taken into account.

a) the inertia forces acting on the mass of the structure, which can be estimated using the fundamental mode method (see **4.2.2**).

b) the static thrusts of the terrain acting on the two abutments ($E_o$).

c) the additional seismic ground pressures

$$\Delta E_d = E_d - E_o \tag{6.16}$$

Where

$E_d$ is the total terrain thrust acting on the abutment due to the calculated seismic action, in accordance with Annex 5. It is assumed that the thrusts $\Delta E_d$ act on both abutments in the same direction.

(6) The connection of the deck to the abutment (including fixed supporting devices or connectors, if any) shall be calculated for the effects of the action resulting from the above points. The reactions on the passive side may be taken into account as specified in **(8)**.

(7) In order that the damage to the soil or embankment behind an abutment rigidly connected to the deck is kept within acceptable limits, the calculated value of the seismic displacement shall not exceed a limiting value, $d_{\text{lim}}$, dependent on the bridge importance class (see Table 6.2).

**Table 6.2. - Limit values for the calculated value of seismic displacement for abutments rigidly connected to the deck**

| Bridge importance class | Limit displacement $d_{\text{lim}}$ (mm) |
|---|---|
| III | 30 |
| II | 60 |
| I | No limit |

(8) The reaction of the soil towards the fill, caused by the movement of the abutment and any monolithic fins attached to it, is assumed to act on the following surfaces:

- in the longitudinal direction, on the back of the abutment moving against the soil or backfill;

- in the transverse direction, on the back of those wings that move against the backfill.

These reactions can be estimated on the basis of the horizontal soil moduli corresponding to the specific geotechnical conditions.

The relevant abutment shall be designed to resist this soil reaction in addition to the static thrusts of the terrain.

(9) When an abutment is buried in rigid formations of the natural terrain for more than 80% of its height, it may be considered as fully embedded in the terrain. In this case a coefficient $q$ = 1 shall be used and the inertia forces shall be determined on the basis of the calculated value of the acceleration corresponding to the ground surface at the location, $a_g S$ (i.e. without spectral amplification).

### 6.7.4 Underground frame-bridges with high overloads

(1) For buried frame-bridges with a large thickness of infill above the top slab (exceeding 50% of the span), the assumptions for the inertial seismic response used in section **6.7.3** may not be applicable, because they lead to unrealistic results. In such a case, the inertial part of the response shall be neglected and the response shall be calculated based on the kinematic compatibility for the calculated seismic action between the frame structure and the free-field seismic deformation of the surrounding terrain.

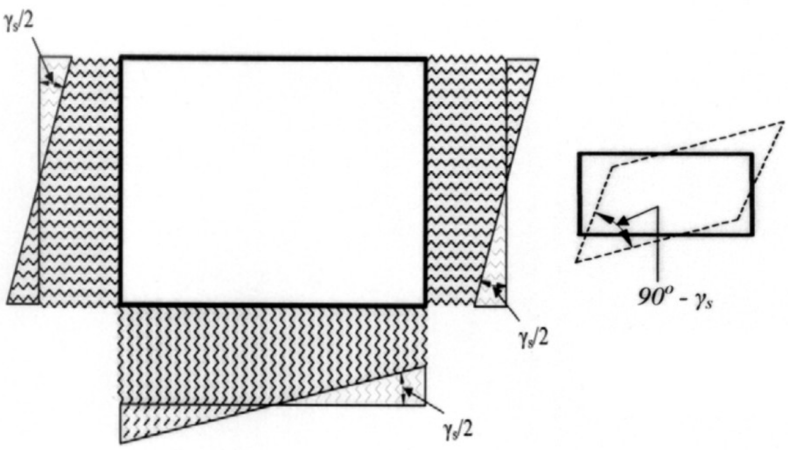
(2) For this purpose, the free-field seismic ground deformation can be assumed as a uniform tangential deformation field (see Figure 6.3), of value:

$$\gamma_s = \frac{v_g}{v_s} \tag{6.17}$$

where

$v_g$ is the maximum ground velocity (see **(3)** below);

$v_s$ is the velocity of the transverse shear wave in the ground, compatible with the tangential deformation corresponding to the acceleration at ground level. This value can be estimated from the value $v_{s,max.}$ given for small deformations in Table 4.1 of Annex 5.




Legend:

$\gamma_s$ Free-field tangential ground deformation

**Figure 6.3 - Kinematic response of frame-bridges**

(3) In the absence of specific data, the maximum ground velocity shall be estimated from the calculated value of the ground level acceleration, $a_g$, in a terrain type A, using the relationship:

$$v_g = \frac{S T_C a_g}{2\pi} \tag{6.18}$$

where $S$ y $T_C$ are in accordance with section **3.2.2.2** of Annex 1.

### 6.7.5 Retaining walls

(1) Independent retaining walls must be designed in accordance with section **6.7.2(2)** and **(3)**, without considering any action from the supporting devices.

# 7 Seismic isolation bridges

## 7.1 General considerations

(1) This chapter covers the design of bridges with a special isolation system to reduce their response to horizontal seismic action. The isolation elements are placed on an isolation interface, normally located under the deck and above the crown of the piers/abutments.

(2) Reduction of the response can be achieved:

- by lengthening the fundamental period of the structure (effect of the period phase shift in the response spectrum), which reduces the forces, but increases the displacements;

- by increasing the damping, which reduces displacements and may reduce forces;

- (preferably) by a combination of the above two effects.

## 7.2 Definitions

**isolating or isolation system:**
Set of components used to provide seismic isolation, located at the isolation interface level.

**isolation units or isolators:**
Each of the components constituting the isolation system. Each unit provides only one, or a combination, of the following functions:

- ability to resist vertical loading, combined with high lateral flexibility and high vertical rigidity;
- energy dissipation (hysteretic, viscous, frictional);
- ability to return laterally to the initial position;
- horizontal restraint (sufficient elastic rigidity) under horizontal, non-seismic service loads.

**substructure(s):**
Art(s) of the structure(s) located below the isolation interface, usually consisting of piers and abutments. In general, the horizontal flexibility of substructures should be taken into account.

**superstructure:**
The part of the structure above the isolation interface. In bridges, this part usually consists of the deck.

**effective centre of rigidity:**
Centre C of rigidity defined at the top of the isolation interface, considering the superstructure as a rigid body, but taking into account the flexibilities of the isolators and the substructure(s).

**calculated value of the displacement ($d_{cd}$) of the isolation system in a principal direction:**
Maximum horizontal displacement (with respect to the ground) of the centre of rigidity of the superstructure, which originates under the calculated seismic action.
calculated value of the displacement (dbi) of the isolator i:

**calculated value of the displacement ($d_{bi}$) of the isolator *i*:**
Displacement of the superstructure with respect to the substructure at the position of the isolator, corresponding to the calculated value of the displacement of the isolator system.

**calculated major displacement ($d_{bi,a}$) of the isolator *i*:**
Calculated value of the displacement of the isolator, multiplied by the amplification coefficient $\gamma_{IS}$ according to section **7.6.2**.

**maximum total displacement of the isolator *i*:**
Sum of the calculated value of the increased displacement, the horizontal displacement due to permanent actions, the long-term deformations of the superstructure (post-tensioning, shrinkage and creep of the concrete decks) and 50% of the displacement due to movements caused by temperature variations.

**effective rigidity of the isolation system in a principal direction:**
Ratio of the value of the total horizontal force transmitted through the isolation interface, concurrent with the calculated value of the displacement in the same direction, to the absolute value of the calculated value of the displacement (secant stiffness).

**effective period:**
Fundamental period in the considered direction of a single degree of freedom system having the mass of the superstructure and a stiffness equal to the effective stiffness of the isolation system as specified in section **7.5.4**.

**effective damping of the isolation system:**
The value of the viscous damping ratio corresponding to the energy dissipated by the isolation system during a cyclic response to the calculated value of the displacement.

**simple low damping elastomeric bearings:**
Low damping laminated elastomeric bearings according to UNE-EN 1337-3, not subject to UNE-EN 15129 (anti-seismic devices) (see section **7.5.2.3.3(5)**).

**special elastomeric bearings:**
Laminated elastomeric bearings with high damping successfully tested according to the requirements of Standard UNE-EN 15129 (anti-seismic devices) (see section **7.5.2.3.3(7)**).

## 7.3 Basic requirements and compliance criteria

(1) The basic requirements set out in section **2.2** must be satisfied.

(2) The seismic response of the superstructure and substructures to the calculated seismic situations must be assumed to be of limited ductility ($q \leq 1.5$).

(3) The bridge is considered to satisfy the basic requirements if it is designed as specified in sections **7.4** and **7.5** and complies with sections **7.6** and **7.7**.

(4) Increased reliability is required for the resistance and integrity of the isolation system, due to the critical role of its displacement capability for the safety of the bridge. This reliability is considered to be achieved if the isolation system is designed in accordance with the requirements of section **7.6.2**.

(5) The design properties must be validated by qualification and prototype tests for all types of isolators, with the exceptions of simple low-damping elastomeric bearings in accordance with sections **7.5.2.3.3(5)** and **(6)** and flat sliding bearings in accordance with section **7.5.2.3.5(5)**.

NOTE Appendix K is intended to provide recommendations for prototype tests in cases where the Standard UNE-EN 15129 ("Anti-seismic devices") does not provide detailed requirements for type tests.

## 7.4 Seismic action

### 7.4.1 Calculation spectrum

(1) The spectrum used must not be less than the elastic response spectrum detailed in section **3.2.2.2** of Annex 1 for non-isolated structures (see also **3.2.2.5(8)** of Annex 1).

NOTE The value assigned in section **3.2.2.2**. of Annex 1 is adopted for $T_D$. However, particular attention must be paid to the fact that the safety of structures with seismic isolation depends mainly on the displacement demands imposed on the isolation system, which are directly proportional to the value of the period $T_D$.

### 7.4.2 Time domain representation.

(1) The provisions of section **3.2.3** apply.

## 7.5 Analysis and modelling procedures

### 7.5.1 General considerations

(1) The following analysis procedures applicable to seismically isolated bridges with the application conditions detailed in section **7.5.3** are given below.

a) Spectral analysis based on the fundamental mode

b) Multi-mode spectral analysis

c) Non-linear time domain analysis

(2) In addition to the conditions specified in section **7.5.3**, the following prerequisites are added for the application of methods (a) and (b) of **(1)** above:

- The normally non-linear force-displacement relationship of the isolation system must be approximated with sufficient accuracy by the effective rigidity $K_{eff}$, i.e. by the secant value of the stiffness corresponding to the calculated value of the displacement (see Figure 7.1). This representation should be based on successive approximations of the calculated value of the displacement ($d_{cd}$).

- The energy dissipation of the isolation system should be expressed as the 'effective damping' ($\xi_{eff}$), in terms of an equivalent viscous damping.

(3) The normal methods of linear dynamic analysis, specified in section **4.2**, may be applied if the isolation system consists exclusively of simple low-damping elastomeric bearings (with an equivalent viscous damping ratio of about 0.05). Elastomeric bearings can be considered as linear elastic elements that deform in shear (and possibly compression). Their damping may be assumed to be equal to the overall viscous damping of the structure (see also section **7.5.2.3.3(2)**). The whole structure shall remain essentially elastic.

### 7.5.2 Calculation properties of the isolation system

#### 7.5.2.1 General considerations

(1) All isolators must comply with UNE-EN 15129 (anti-seismic devices) or be in conformity with a European Technical Assessment (ETA) issued for the same. For the case of isolators whose prototype tests are not fully covered by this standard, the requirements given in Appendix **K** of this Annex may be applied.

NOTE See references above, as well as section **7.5.2.4(5)**, **(6)** and **(7)**, in relation to single low-damping elastomeric bearings used as specified in section **7.5.2.3.3(4)**, **(5)** and **(6)**, and in relation to PTFE (polytetrafluoroethylene) lubricated flat sliding bearings, used in accordance with section **7.5.2.3.5(5)**.

#### 7.5.2.2 Rigidity in the vertical direction

(1) Isolators resisting vertical loads must be sufficiently stiff in the vertical direction.

(2) The requirement of **(1)** above is considered to be satisfied if the horizontal displacement of the centre of gravity of the structure, due to the vertical flexibility of the isolators, is less than 5% of the calculated value of the displacement $d_{cd}$. Testing is not required if simple sliding or elastomeric supports with low damping are used as load-bearing elements for vertical loads at the isolation interface.

#### 7.5.2.3 Calculated properties in horizontal directions

##### 7.5.2.3.1 General considerations

(1) The calculated properties of the isolators depend on their behaviour, which may be one or a combination of those described in sections **7.5.2.3.2** to **7.5.2.3.5**.

#### 7.5.2.3.2 Hysteretic behaviour

(1) The force-displacement relationship of the isolator in the horizontal direction can be approximated by a bilinear function, as shown in Figure 7.1 for an isolator *i* (index *i* has been omitted).

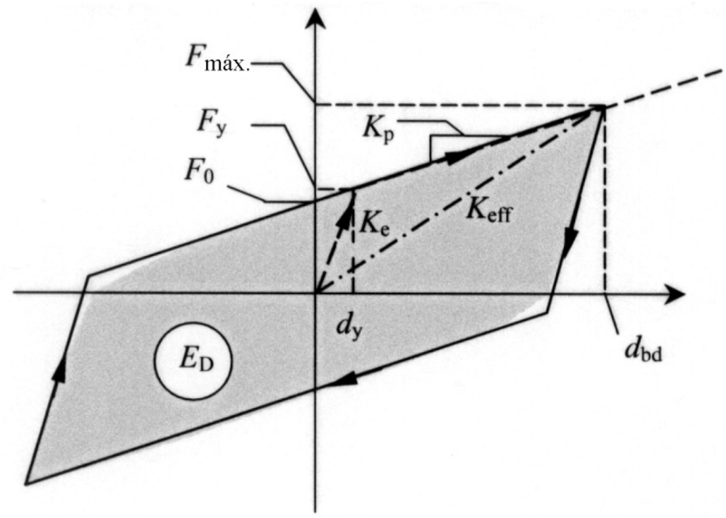



**Figure 7.1 – Bilinear approximation of the force-displacement hysteretic behaviour**

(2) The parameters of the bilinear approximation are as follows:

$d_y$ = displacement corresponding to the yield stress;

$d_{bd}$ = calculated value of the displacement of the isolator, corresponding to the calculated displacement, $d_{cd}$, of the isolator system;

$E_D$ = energy dissipated per cycle corresponding to the calculated value of the displacement $d_{bd}$, equal to the area enclosed by the actual hysteresis loop = 4 ($F_y\ d_{bd}$ - $F_{max.} d_y$);

$F_y$ = force corresponding to the yield stress, obtained by monotonic loading;

$F_o$ = force corresponding to the null displacement, obtained by cyclic loads = $F_y$ - $K_p\ d_y$;

$F_{max.}$ = maximum force, corresponding to the calculated value of the displacement $d_{bd}$;

$K_e$ = elastic rigidity obtained by monotonic loading = $F_y$ / $d_y$, also equal to the unloading stiffness at cyclic loading;

$K_p$ = post-elastic (tangent) rigidity = ($F_{max.} - F_y$) / ($d_{bd} - d_y$).

#### 7.5.2.3.3 Behaviour of elastomeric bearings

(1) Elastomeric bearings considered in this standard are laminated rubber bearings consisting of rubber layers reinforced with fully bonded steel plates. With regard to damping, a distinction is made between low-damping and high-damping elastomeric bearings.

(2) Low-damping elastomeric bearings are those with an equivalent viscous damping ratio, $\xi$, of less than 0.06. Such bearings have a cyclic behaviour similar to hysteretic behaviour with very slender hysteresis loops. Their behaviour must be approximated by that of a linear elastic element with an equivalent elastic rigidity in the horizontal direction equal t $G_b A_b / t_e$, where $G_b$ is the transverse modulus of elasticity of the elastomer (see section **7.5.2.4(5)**), $A_b$ is the effective horizontal area and $t_e$ is the total elastomer thickness.

(3) Highly damped elastomeric bearings exhibit significant hysteresis loops, corresponding to an equivalent viscous damping ratio, $\xi$, typically between 0.10 and 0.20. Their behaviour should be considered as linear hysteretic.

(4) From the point of view of the special tests required to assess their seismic performance, elastomeric bearings are differentiated in this standard between simple low-damping devices and special elastomeric bearings.

(5) Low-damping devices that comply with the provisions of UNE-EN 1337-3 are defined as simple low-damping elastomeric bearings.

(6) Simple low-damping elastomeric bearings may be used as isolators without special tests to determine their seismic performance.

(7) Special elastomeric bearings are high damping elastomeric bearings tested by special tests according to the requirements of UNE-EN 15129 (anti-seismic devices).

(8) The design properties of elastomeric bearings used in this chapter shall cover both roughness and non-roughness conditions of the bearings.

NOTE Elastomeric bearings show some roughness if they have previously (i.e. before testing) been subjected to one or more cycles of high shear deformation. Roughened devices show a significant drop in shear rigidity in successive cycles. It happens, however, that after a certain time (a few months), the original (initial) tangential rigidity of the bearings is practically recovered. This effect is particularly noticeable for high damping and low shear modulus supports and must be taken into account by the use of an appropriate range of design parameters (see section **K.2.1** and requirement R4 in section **K.2.3.3**).

(9) Lead rubber bearings (LRB) consist of low damping elastomeric bearings with a cylindrical lead core. The plasticisation of the lead core gives these devices a remarkable hysteretic behaviour. This hysteretic behaviour can be represented by the bilinear approximation shown in figure 7.1, with the following parameters:

- Elastic rigidity: $K_e = K_L + K_R$

  where $K_R$ and $K_L$ are, respectively, the shear stiffnesses of the elastomeric and lead parts of the device;

- Post-elastic rigidity: $K_p = K_R$;

- Load corresponding to the yield stress: $F_y = F_{Ly}\ (1 + K_R/K_L)$

  where $F_{Ly}$ is the yield load of the lead core.

NOTE 1 If $K_R << K_L$, then $K_e \cong K_L$ and $F_y \cong F_{Ly}$.

NOTE 2 Lead rubber bearings (LRB) shall be in accordance with the provisions of Standard UNE-EN 15129 (anti-seismic devices).

#### 7.5.2.3.4 Fluid viscous dampers

(1) The reaction of viscous fluid dampers is proportional to $v^{\alpha_b}$, where $v = \dot{d}_b = \frac{d}{dt}(d_b)$ is the velocity of the motion. This reaction is zero for the maximum displacement $d_{max.} = d_{bd}$ and therefore does not contribute to the effective rigidity of the isolation system. The force-displacement relationship of a fluid viscous damper is shown in Figure 7.2 (for sinusoidal motion), depending on the value of the exponent $\alpha_b$.

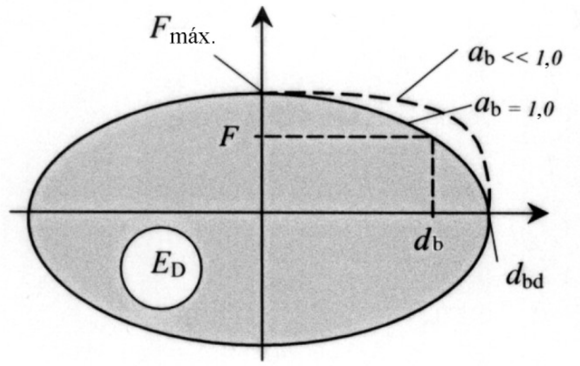



**Figure 7.2 – Law of viscous force-displacement behaviour**

$d_b = d_{bd} \sin(\omega t)$, with $\omega = 2\pi/T_{eff}$

$F = Cv^{\alpha_b} = F_{max.}(\cos(\omega t))^{\alpha_b}$

$F_{max.} = C(d_{bd}\omega)^{\alpha_b}$

$E_D = \lambda(\alpha_b)\ F_{max.}\ d_{bd}$

$$\lambda(\alpha_b) = 2^{2+\alpha_b} \frac{\Gamma^2(1 + 0{,}5\alpha_b)}{\Gamma(2 + \alpha_b)}$$

$\Gamma$ ( ) = is the gamma function

NOTE In some cases of viscous devices (fluid dampers) with low $\alpha_b$ values, the combination of the viscous element with a linear spring placed in series (reflecting the compressibility of the fluid) is necessary to satisfactorily match the force-velocity relationship with the test results to obtain $E_D$. However, this has only a small influence on the energy ($E_D$) dissipated by the device.

### 7.5.2.3.5 Frictional behaviour

(1) Sliding devices with a flat sliding surface limit the force transmitted to the superstructure to:

$$F_{\text{máx.}} = \mu_d N_{Sd} \text{signo}(\dot{d}_b) \tag{7.1}$$

Where

$\mu_d$ is the coefficient of friction or dynamic friction;

$N_{Sd}$ is the normal force transmitted by the device, and

sign ($\dot{d}_b$) is the sign of the velocity vector $\dot{d}_b$

$d_b$ is the relative displacement between the two sliding surfaces.

Such devices may, however, give rise to significant permanent displacements. Consequently, they must be used in combination with other devices having an adequate recovery capability (see **7.7.1**).

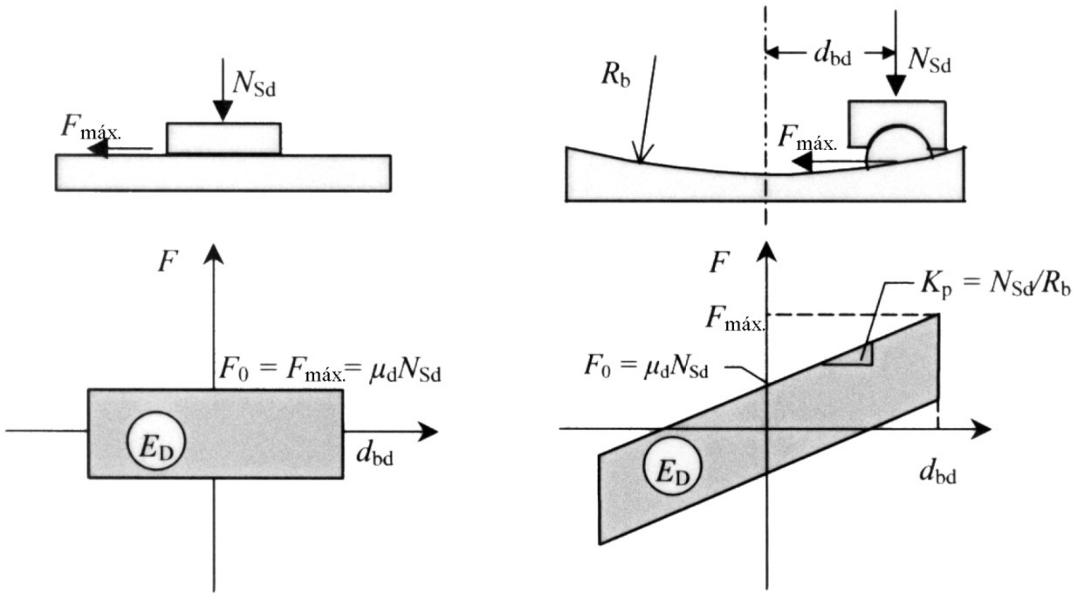



**Figure 7.3 – Frictional behaviour expressed by the force-displacement relationship**

(2) Sliding bearing devices with a spherical sliding surface of radius $R_b$ provide for a displacement $d_b$ a restoring force equal to $N_{Sd}d_b/R_b$. For these devices the force-displacement relationship is as follows:

$$F_{\text{máx.}} = \frac{N_{Sd}}{R_b} d_{bd} + \mu_d N_{Sd} \, signo(\dot{d}_{bd}) \tag{7.2}$$

NOTE Expression (7.2) provides a sufficient approximation when $d_b/R_b \leq 0.25$.

(3) In the above two cases, the energy dissipated per cycle $E_D$ (see Figure 7.3) for the calculated value of the displacement $d_{bd}$ becomes:

$$E_D = 4\ \mu_d N_{Sd} d_{bd} \tag{7.3}$$

(4) The dynamic friction coefficient, $\mu_d$, depends mainly on:

- the composition of the sliding surfaces;
- the use, or not, of lubrication;
- the pressure of the bearing device on the sliding surface in the calculated seismic situation;
- the sliding velocity.

it shall be determined by appropriate tests.

NOTE Appendix K gives information on the tests that can be carried out for the determination of the dynamic friction coefficient. It should be noted that, in the calculated seismic situation, for PTFE surfaces in their initial state, without the minimum additive and lubricated, sliding on polished stainless steel surfaces, the dynamic friction coefficient can be very low ($\leq$ 0.01) for the range of speeds corresponding to the seismic motions and for the normal range of the bearing device pressures on the sliding surface.

(5) On the assumption that the equivalent damping of the isolation system is evaluated ignoring any contribution from these elements, sliding bearings with a flat lubricated PTFE sliding surface allowing sliding in both horizontal directions, in accordance with UNE-EN 1337-2, are not subject to special tests to determine seismic behaviour, nor are elastomeric bearings containing lubricated PTFE sliding elements that allow sliding in one horizontal direction, while in the other direction they behave as simple low-damping elastomeric bearings, in accordance with UNE-EN 1337-2 and UNE-EN 1337-3.

#### 7.5.2.4 Variability of the properties of isolators

(1) The nominal design properties (DP) of the isolators must either be validated in general, according to the provisions of UNE-EN 15129 (anti-seismic devices), or be included in a European Assessment Document (EAD), with the exception of the special cases of simple low damping elastomeric bearings, according to section **7.5.2.3.3(5)** and **(6)**, and sliding bearings, in accordance with section **7.5.2.3.5(5)**, for which items **(4)**, **(5)** and **(6)** below apply.

NOTE See also NOTE under section **7.5.2.1(1)**.

(2) The rated properties of the isolators, and hence of the isolation system, can be affected by ageing, temperature, load history (roughness), contamination, and accumulated travel (wear). This variability must be accounted for in accordance with Appendix J by using the following two sets of appropriately established design properties of the isolation system:

- upper bound design properties (UBDP); and
- lower bound design properties (LBDP).

(3) In general, and regardless of the method of analysis, two analyses should be carried out: one using the UBDP properties, which leads to the maximum forces in the substructure and deck, and one using the LBDP properties, which leads to the maximum displacements of the isolation system and deck.

(4) Multi-mode spectral analysis or time domain analysis may be performed based on the set of nominal design properties only if the design displacements, $d_{cd}$, obtained from the fundamental mode analysis performed according to section **7.5.4** from the UBDP and LBDP properties, do not differ by more than ± 15 % from those of the design properties.

(5) The nominal design properties of simple low-damping elastomeric bearings according to section **7.5.2.3.3(5)** and **(6)**, may be assumed as follows:

- Transverse modulus of elasticity: $G_b = \alpha\, G_g$,

NOTE The value of $\alpha$ is normally between 1.1 and 1.4. The appropriate value is best determined by testing the device.

- where $G_g$ is the value of the "conventional apparent transverse modulus of elasticity" according to UNE-EN 1337-3;
- equivalent viscous damping: $\xi_{eff} = 0.05$

(6) The variability of the design properties of simple low-damping elastomeric bearings due to ageing and temperature can be limited to the value of $G_b$ and assumed as follows:

- LBDP $G_{b,min.} = G_b$
- UBDP as a function of the 'minimum support temperature for seismic calculation' $T_{min.,b}$ (see section **J.1(2)**), as follows:
  - when $T_{min.,b} \geq 0$ ºC

    $G_{b,max.} = 1.2\, G_b$

  - when $T_{min.,b} < 0$ ºC

    the value of $G_{b,max.}$ shall correspond to $T_{min,b.}$

NOTE In the absence of relevant test results, the value of $G_{b,max.}$ For $T_{min.,b} < 0$ °C can be obtained from $G_b$ adjusted for temperature and ageing according to the values for $\lambda_{max.}$ corresponding to $K_p$ specified in tables JJ.1 and JJ.2.

(7) The values of the friction parameters of the sliding elements whose contribution to energy dissipation is ignored, according to section **7.5.2.3.5(5)**, shall be taken as specified in UNE-EN 1337-2.

### 7.5.3 Conditions for the application of the methods of analysis

(1) The method of spectral analysis based on the fundamental mode may be applied if all of the following conditions are met:

a) The distance between the bridge site and the nearest known seismically active fault exceeds 10 km.

b) the ground conditions at the site correspond to one of the terrain types A, B, C or E in section **3.1.1** of Annex 1.

c) the effective damping ratio does not exceed 0.30.

(2) Multimodal spectral analysis may be applied if both conditions b and c of **(1)** above are met.

(3) Non-linear time domain analysis can be applied for the design of any bridge with seismic isolation.

### 7.5.4 Spectral analysis based on the fundamental mode

(1) The rigid deck model shall be used in all cases (see **4.2.2.3**).

(2) The shear stress transmitted through the isolation interface in each principal direction must be calculated considering the superstructure as a single degree of freedom system and using:

- the effective rigidity of the isolation system, $K_{eff}$;
- the effective damping of the isolation system, $\xi_{eff}$;
- the mass of the superstructure, $M_d$;
- the spectral acceleration $S_e$ ($T_{eff}$, $\eta_{eff}$) (see section **3.2.2.2** of Annex 1), corresponding to the effective period $T_{eff}$, with $\eta_{eff} = \eta(\xi_{eff})$.

The values of these parameters shall be calculated as follows:

- Effective rigidity:

$$K_{eff} = \Sigma\, K_{eff,i} \tag{7.4}$$

where $K_{eff,i}$ is the combined rigidity of the isolator and the corresponding substructure (pier) $i$.

- Effective damping:

$$\xi_{eff} = \frac{1}{2\pi}\left[\frac{\sum E_{D,i}}{K_{eff}\, d_{cd}^{2}}\right] \tag{7.5}$$

where

$\Sigma E_{D,i}$ is the sum of the energies dissipated by all the isolators $i$ in a complete deformation cycle for the calculated value of the displacement, $d_{cd}$.

- Effective period:

$$T_{\text{eff}} = 2\pi\sqrt{\frac{M_{\text{d}}}{K_{\text{eff}}}} \tag{7.6}$$

(3) This leads to the results shown in Table 7.1 and Figure 7.4.

**Table 7.1 - Spectral acceleration $S_e$ and calculated value of displacement $d_{cd}$**

| $T_{\text{eff}}$ | $S_{\text{e}}$ | $d_{\text{cd}}$ |
|---|---|---|
| $T_{\text{C}} \leq T_{\text{eff}} < T_{\text{D}}$ | $2.5 \frac{T_{\text{C}}}{T_{\text{eff}}} a_{\text{g}} S \eta_{\text{eff}}$ | $\frac{T_{\text{eff}}}{T_{\text{C}}} d_{\text{C}}$ |
| $T_{\text{D}} \leq T_{\text{eff}} \leq 4$ s | $2.5 \frac{T_{\text{C}} T_{\text{D}}}{T_{\text{eff}}^2} a_{\text{g}} S \eta_{\text{eff}}$ | $\frac{T_{\text{D}}}{T_{\text{C}}} d_{\text{C}}$ |

where

$$a_{\text{g}} = \gamma_{\text{I}} a_{\text{g,R}} \tag{7.6}$$

and

$$d_{\text{C}} = \frac{0,625}{\pi^2} a_g S \eta_{\text{eff}} T_{\text{C}}^2 \tag{7.8}$$

The value of $\eta_{\text{eff}}$ must be taken from the expression:

$$\eta_{\text{eff}} = \sqrt{\frac{0,10}{0,05 + \xi_{\text{eff}}}} \geq 0,40 \tag{7.9}$$

Maximum shear stress:

$$V_{\text{d}} = M_{\text{d}} S_{\text{e}} = K_{\text{eff}} d_{\text{cd}} \tag{7.10}$$

where

$S$, $T_{\text{C}}$ and $T_{\text{D}}$ are the parameters of the calculation spectrum dependent on the terrain type, as specified in paragraph **7.4.1(1)** of this Standard and in section **3.2.2.2** of Annex 1;

$a_{\text{g}}$ is the calculated value of ground acceleration in a terrain type A, corresponding to the bridge importance category;

$\gamma_{\text{I}}$ is the bridge importance coefficient; and

$a_{\text{g,R}}$ is the calculated value of the reference acceleration at ground level (corresponding to the reference return period).

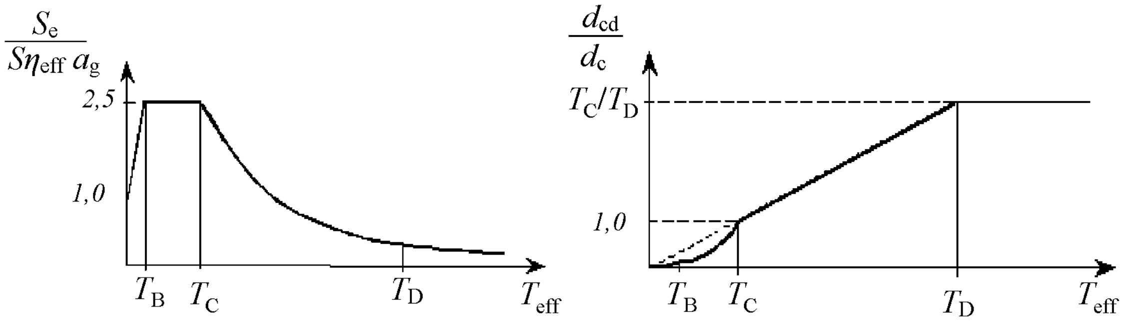


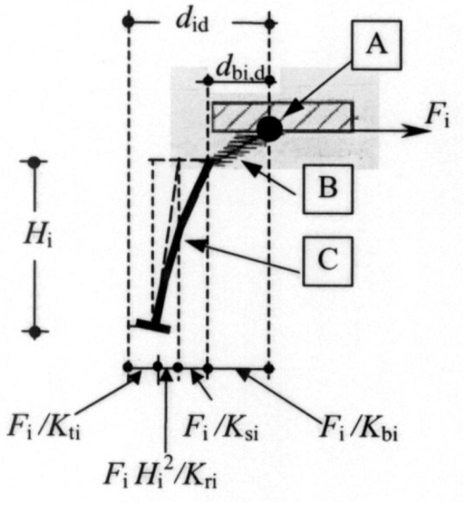

**Figure 7.4 – Acceleration and displacement spectra**

NOTE 1 Up to a period of 4 s the elastic response spectrum in section **3.2.2.2(1)** of Annex 1 applies. For $T_{eff}$ values greater than 4 s the elastic displacement response spectrum of Appendix A of Annex 1 must be used and the elastic acceleration response spectrum can be derived by inverting expression (3.7) of Annex 1. However, bridges with seismic isolation with $T_{eff}$ > 4 s require special attention, due to their inherently low rigidity against any horizontal action.

NOTE 2 For a pier of height $H_i$ with displacement stiffness $K_{si}$ (kN/m), supported by a foundation with translation stiffness $K_{ti}$ (kN/m), rotation stiffness $K_{fi}$ (kNm/rad), and supporting an isolator i with effective stiffness $K_{bi}$ (kN/m), the combined stiffness $K_{eff,i}$ is (see figure 7.5N):

$$\frac{1}{K_{eff,i}} = \frac{1}{K_{bi}} = \frac{1}{K_{ti}} = \frac{1}{K_{si}} = \frac{H_i^2}{K_{fi}} \tag{7.11N}$$

The flexibility of the isolator and its relative displacement $d_{bi} = \frac{F_i}{K_{bi}}$ are usually much larger than the other components of the superstructure displacement. For this reason, the effective damping of the system depends only on the sum of the energies dissipated by the isolators, $\Sigma E_{Di}$, and the relative displacement of the isolator is almost equal to the displacement of the superstructure at that point ($d_{bi}/d_{id} = K_{eff,i}/K_{bi} \cong 1$).




Legend:

A Superstructure

B Isolator *i*

C Pier *i*

**Figure 7.5N – Combined rigidity of the pier and the isolator *i***

(4) In essentially non-linear systems, $K_{eff}$ and $\xi_{eff}$ depend on the calculated value of the displacement $d_{cd}$ (see $d_{bd}$ in Figure 7.1). Successive approximations of $d_{cd}$ must be made to limit the deviations between the assumed and calculated values to within ± 5 %.

(5) To determine the effects of seismic action on the isolation system and the substructures in the main transverse direction (let us call it the *y*-direction), the influence of the in-plane eccentricity in the longitudinal direction $e_x$ (between the effective centre of stiffness and the centre of gravity of the deck), on the displacement of the superstructure $d_{id}$ at the position of pier *i*, shall be evaluated as follows:

$$d_{id} = \delta_i \, d_{cd} \tag{7.12}$$

$$\delta_i = 1 + \frac{e_x}{r r_x} x_i \tag{7.13}$$

with

$$r_x^2 = \frac{\sum (x_i^2 K_{yi} + y_i^2 K_{xi})}{\sum K_{yi}} \tag{7.14}$$

where

$e_x$ is the eccentricity in the longitudinal direction;

$r$ is the radius of gyration of the mass of the deck relative to the vertical axis passing through its centre of gravity;

$x_i$ and $y_i$ are the coordinates of the pier *i* with respect to the effective centre of rigidity;

$K_{yi}$ and $K_{xi}$ are the combined effective stiffnesses of the isolator and pier *i*, in the *y* and *x* directions, respectively.

NOTE For straight bridges, normally $y_i << x_i$. In such cases, the term $y_i^2 K_{si}$ can be omitted in the expression (7.14).

(6) For the combination of the seismic action components, section **4.2.1.4(4)** must be applied.

### 7.5.5 Multi-mode spectral analysis

(1) The model of the isolation system must reflect with sufficient accuracy:

- the spatial distribution of the isolators and the relevant overturning effects, and

- the translation of the superstructure in the two horizontal directions and its rotation with respect to the vertical axis.

(2) The model of the superstructure must reflect with sufficient accuracy its deformation in plan. Accidental eccentricity of the mass need not be considered.

(3) The model of the substructures shall reflect with sufficient accuracy the distribution of their stiffness properties and at least the rotational stiffness of the foundation. Where the pier has a significant mass and height, or if it is immersed in water, its mass distribution must also be modelled adequately.

(4) The effective damping given by expression (7.5) can be applied only to modes having periods greater than 0.8 $T_{eff}$. For all remaining modes, the damping ratio for the structure without seismic isolation shall be used, unless a more accurate estimate of the appropriate damping ratio is made.

(5) For the combination of the horizontal components of the seismic action, section **4.2.1.4(2)** should be applied.

(6) Both the displacement of the centre of rigidity of the isolation system ($d_{cd}$) and the total shear stress ($V_d$) transmitted through the isolation interface in each of the two horizontal directions, resulting from the calculation, are subject to lower limits as follows:

$$\rho_d = \frac{d_{cd}}{d_{cf}} \geq 0,80 \tag{7.15}$$

$$\rho_V = \frac{V_d}{V_f} \geq 0,80 \tag{7.16}$$

where

$d_{cf}$, $V_f$ are, respectively, the calculated value of the displacement and shear stress transmitted through the isolation interface, calculated using the spectral analysis based on the fundamental mode described in section **7.5.4**. The limitations of section **7.5.3(1)** do not apply to testing (7.15) and (7.16).

(7) In case the conditions of **(6)** above are not fulfilled, the relevant effects on the isolation system, deck and substructures shall be multiplied by:

$\frac{0,80}{\rho_d}$ for seismic displacements; or by (7.17)

$\frac{0,80}{\rho_V}$ for seismic forces and moments (7.18)

(8) It is not necessary to apply the constraints in **(6)** above and the relevant corrections in **(7)** if the bridge cannot be approximated (even coarsely) to a single degree of freedom model. Such cases may occur for:

- bridges with high piers, the mass of which has a significant influence on the displacement of the deck;

- bridges with a significant eccentricity, $e_x$, in the horizontal direction between the centre of gravity of the deck and the effective centre of rigidity ($e_x$ > 0,10 $L$).

In these cases, it is recommended that the constraints and corrections in **(6)** and **(7)** be applied in each direction to the displacements and forces deduced from the fundamental mode of the real bridge model in the same direction.

### 7.5.6 Time domain analysis

(1) Section **7.5.5(1)**, **(2)**, **(3)**, **(6)**, **(7)** and **(8)** apply, taking as values of $d_{cd}$ and $V_d$ in expressions (7.15) and (7.16) the corresponding effects of the calculated action, according to section **4.2.4.3(1)**.

### 7.5.7 Vertical component of seismic action

(1) The effects of the vertical component of the seismic action can be determined from a linear analysis using the response spectrum, independently of the method used for the calculation of the response to the horizontal seismic action. For the combination of the effects of the actions, section **4.2.1.4** applies.

## 7.6 Testing

### 7.6.1 Calculated seismic situation

(1) The calculated seismic situation is described by the combination of actions of expression (5.4) in section **5.5(1)**.

(2) The effects of the calculated seismic action on the isolation system are to be taken according to section **7.6.2**, and those on the superstructure and substructure according to section **7.6.3**.

### 7.6.2 Isolation system

(1) The necessary higher reliability of the isolation system (see section **7.3(4)**) must be established by dimensioning each isolator for a calculated value of the increased displacement $d_{bi,a}$:

$$d_{bi,a} = \gamma_{IS}\, d_{bi,d} \tag{7.19}$$

where $\gamma_{IS}$ is an amplification coefficient that is applied only on the calculated value of the displacement $d_{bi,d}$ of each isolator $i$, determined by one of the procedures specified in section **7.5**, and takes the value $\gamma_{IS}$ = 1.50.

If the spatial variability of the seismic action is taken into account by the simplified method of section **3.3(4), (5), (6) and (7)**, the increased design displacements must be estimated by the rule of section **3.3(7)**, where the displacements $d_{bi,d}$ due to the inertial response, calculated according to one of the methods in section **7.5**, must be aggregated according to expression (7.19) above, while those displacements corresponding to the spatial variability determined according to section **3.3(5) and (6)** need not be aggregated.

(2) The maximum total displacement of each isolator in each direction $d_{m,i}$ must be verified from expression (7.19a) by adding to the calculated value of the above increased seismic displacement the initial displacement $d_{G,i}$ potentially induced by:

a) permanent actions;

b) the long-term deformations (post-tensioning, shrinkage and creep of the concrete decks) of the superstructure; and

c) 50 % of the thermal action.

$$d_{m,i} \geq d_{G,i} + d_{bi,a} \tag{7.19a}$$

NOTE An additional condition for the displacement capacity $d_{m,i}$ of the isolators is given in section **7.7.1(4)**.

(3) All components of the isolation system must be capable of operating, without significant change in their isolation properties, up to their displacement capacity $d_{m,i}$ in the relevant direction.

(4) The calculated value of the resistance of each load-bearing element of the isolation system, including its anchorage, must exceed the force acting on the element for the maximum total displacement. It must also exceed the design force caused by the wind load on the structure in the relevant direction.

NOTE The maximum reaction of hydraulic viscous dampers (see **7.5.2.3.4**) corresponding to the increased displacement $d_{bi,a}$ can be estimated by multiplying the reaction resulting from the analysis by $\gamma_m^{\alpha_b/2}$, with $\alpha_b$ as defined in section **7.5.2.3.4**.

(5) Isolators consisting of simple low-damping elastomeric bearings shall be tested for the effects of the action set out in **(1)** to **(4)**, as specified in the relevant rules of UNE-EN 1337-3 as follows. The calculated value of the maximum total shear deformation at the support shall be calculated as the sum of:

a) the calculated value of the shear deformation due to vertical compression;

b) the shear deformation corresponding to the calculated value of the total horizontal displacement; and

c) the shear deformation corresponding to the calculated value of the total angular rotation of the supports in seismic situation

of the supports in the calculated seismic situation, without multiplying this sum by any amplification coefficient. This deformation shall not exceed the value of $\varepsilon_{u,d}$ according to the equation (2) of section **5.3.3** of Standard UNE-EN 1337-3. Testing for buckling and creep stability shall be carried out in accordance with the relevant rules of section **5.3.3.6** of UNE-EN 1337-3.

NOTE The value to be assigned to the partial safety coefficient $\gamma_m$, in the ratio for $\varepsilon_{u,d}$, for the determination of the calculated resistance of simple low damping elastomeric bearings in the calculated seismic situation is $\gamma_m$ = 1.00.

(6) For simple low-damping elastomeric bearings, in addition to the testing in **(5)** above, the following condition shall be checked:

$$\varepsilon_{q,d} \leq 2,0 \tag{7.20}$$

where $\varepsilon_{q,d}$ is the tangential deflection calculated according to expression (10) of section **5.3.3.3** of UNE-EN 1337-3. In this context, $v_{x,d}$ and $v_{y,d}$ shall be taken equal, respectively, to the maximum total relative displacements in the $x$ and $y$ directions, as specified in **(2)** above.

(7) For the calculated seismic situation, no uplift of the isolators supporting vertical forces is permitted under the calculated seismic action specified in section **7.4**.

(8) The sliding elements mentioned in section **7.5.2.3.5(5)** shall be sized, in accordance with the provisions of UNE-EN 1337-2, for the calculated value of the seismic displacement, as specified in point **(1)** above.

### 7.6.3 Substructures and superstructure

(1) The $E_{EA}$ internal seismic forces exerted only by the calculated seismic action on the substructures and superstructure must be derived from the results of an analysis carried out as specified in section **7.5**.

(2) The calculated seismic forces $E_E$ due to the calculated seismic action only, can be derived from the $E_{EA}$ forces defined in **(1)** above, after dividing these loads by the coefficient $q$ corresponding to limited ductile / essentially elastic behaviour, i.e. , $F_E = F_{EA}/ q$ with $q$ = 1.50.

(3) Testing shall be carried out to ensure that all members of the structure have an essentially elastic behaviour in accordance with the rules in sections **5.6.2** and **6.5**.

(4) The effects of the calculated action for foundations must be in accordance with section **5.8.2(2)**.

(5) The horizontal design forces of the load-bearing elements (piers or abutments) supporting the sliding bearings described in section **7.5.2.3.5(5)**, shall be derived from the maximum friction values, in accordance with the relevant provisions of UNE-EN 1337-2.

(6) In the case of **(5)** above and when the same supporting element also supports fluid viscous dampers, then:

a) the calculated seismic horizontal force of the supporting member in the direction of action of the damper shall be increased by the value of the maximum seismic force of the damper (see expression (7.21));

b) for non-seismic design situations, the design horizontal seismic force under the actions of the imposed deformations (temperature variation) shall be increased by the value of the damper reaction, estimated as 10 % of the maximum seismic damper force used in (a) above.

(7) When a single-mode or multi-mode spectral modal analysis is carried out for isolation systems consisting of a combination of elastomeric bearings and fluid viscous dampers supported on the same bearing element(s), the phase difference between the maximum values of the elastic and viscous elements may be taken into account by the approximation given below. The seismic forces shall be determined as the most unfavourable of those corresponding to the following characteristic states:

a) the state corresponding to the maximum displacement as given by expression (7.10). The damping forces are then zero;

b) to the state corresponding to the maximum velocity and zero displacement for which the maximum damping forces must be determined assuming that the maximum velocity is equal to:

$$v_{\text{máx.}} = 2\Pi d_{\text{bd}} / T_{\text{eff}} \tag{7.21}$$

where $d_{bd}$ is the maximum damper displacement corresponding to the calculated value of the displacement $d_{cd}$ of the isolation system;

c) the state corresponding to the maximum inertia force exerted on the superstructure, to be estimated as follows:

$$F_{\text{máx.}} = \left(f_1 + 2\xi_b f_2\right) S_e M_d \tag{7.22}$$

where $S_e$ is determined from Table 7.1, with $K_{eff}$ according to expression (7.4), without any contribution from the rigidity coming from the dampers, and

$$f_1 = \cos\left[\arctan\left(2\xi_b\right)\right] \tag{7.23a}$$

$$f_2 = \sin\left[\arctan\left(2\xi_b\right)\right] \tag{7.23b}$$

where $\xi_b$ is the contribution of the dampers to the effective damping $\xi_{eff}$ from expression (7.5).

In this state, the displacement reaches $f_1 d_{cd}$ and the velocity of the dampers reaches $v = f_2\, v_{max.}$.

(8) For non-seismic situations due to imposed deformation actions (temperature variation, etc.) in isolation systems consisting of a combination of viscous fluid dampers and elastomeric bearings, as in **(7)**, without sliding elements, the horizontal design force acting on the bearing elements supporting both the supporting devices and the dampers shall be determined assuming that the reactions of the dampers are zero.

## 7.7 Special requirements for the isolation system

### 7.7.1 Lateral recovery capacity

(1) The isolation system must have the capacity for lateral self-recovery in both principal directions, in order to avoid the accumulated gradual increase of displacements. This capability is available when the system has small residual displacements relative to its displacement capacity, $d_m$.

(2) The requirements of **(1)** above are considered to be satisfied in a direction when the displacement $d_0$, defined below, satisfies the following condition in the direction under consideration:

$$\frac{d_{cd}}{d_0} \geq \delta \qquad (7.24)$$

Where

$d_{cd}$ is the calculated value of the displacement of the isolation system in the direction under consideration, as defined in section **7.2**;

$d_0$ is the maximum residual displacement for which the isolation system can remain in static equilibrium in the direction considered, using the system properties defined in this section and in **(5)** below. No limitation due to the displacement capacity of the isolators (unlimited capacity) shall therefore be considered. For a system with bilinear behaviour according to section **7.5.2.3.2** or systems that can be approximated to them, $d_0$ is obtained as:

$$d_0 = F_0/K_p \qquad (7.25)$$

$\Delta$ is a numerical value.

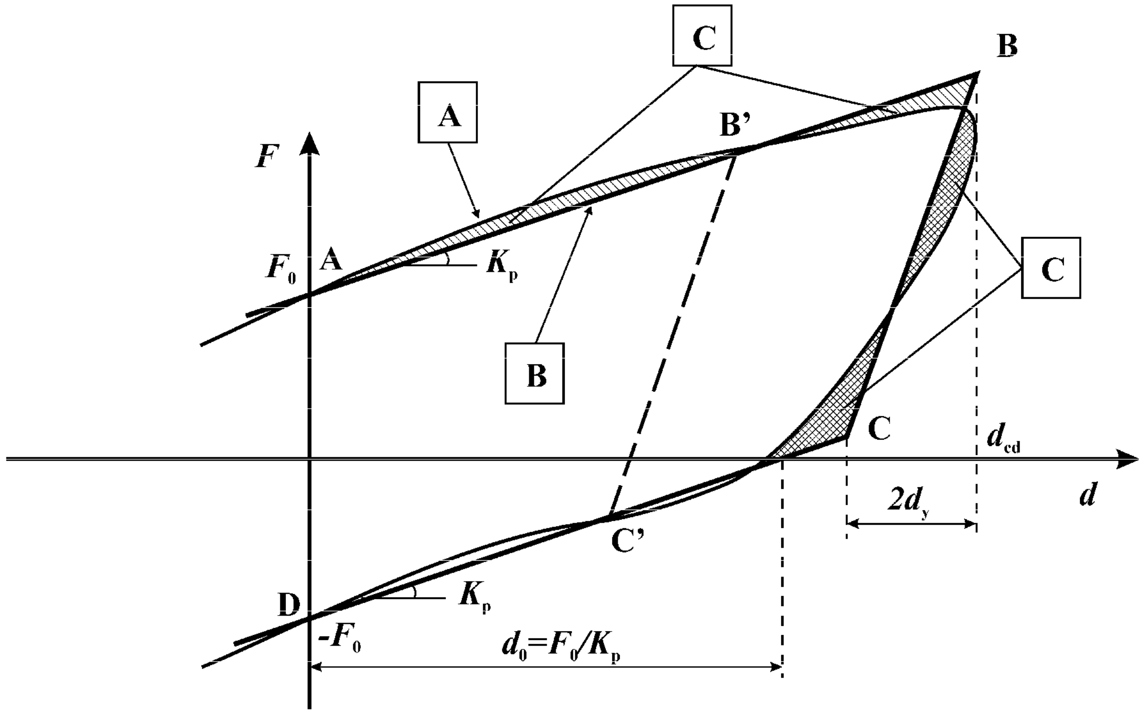
NOTE 1 The value assigned to the ratio $\delta$ is $\delta$ = 0.50 (see also figure 7.8 and NOTE 2 to section **7.7.1(4)**).

NOTE 2 For systems with approximately bilinear hysteretic behaviour (see Figure 7.6) the properties of the equivalent bilinear system shall be determined as follows: the value of the force for zero displacement, $F_0$ and the estimated value of the calculated displacement $d_{cd}$, are retained. The straight lines for the loading branch AB and the unloading branch BC are defined as approximately the corresponding branches of the real loop, keeping the areas equal.

NOTE 3 For systems with bilinear hysteretic behaviour according to section **7.5.2.3.2** or systems that can be approximated to them, the displacement $d_0 = f_0/K_p$ depends on the properties of the isolation system considered, independent of its displacement capacity. Therefore, in Figure 7.6 the systems with the loops ABCD and AB'C'D have the same $d_0$. The value of $d_0$ is positive when the post-elastic rigidity $K_p$ is positive, negative when $K_p$ is negative and $\infty$ when $K_p$ is zero. Systems with negative $K_p$ should not be used.

NOTE 4 For systems with sliding devices, with spherical sliding surfaces (see section **7.5.2.3.5(2)**), $d_0 = \mu_d R_b$.

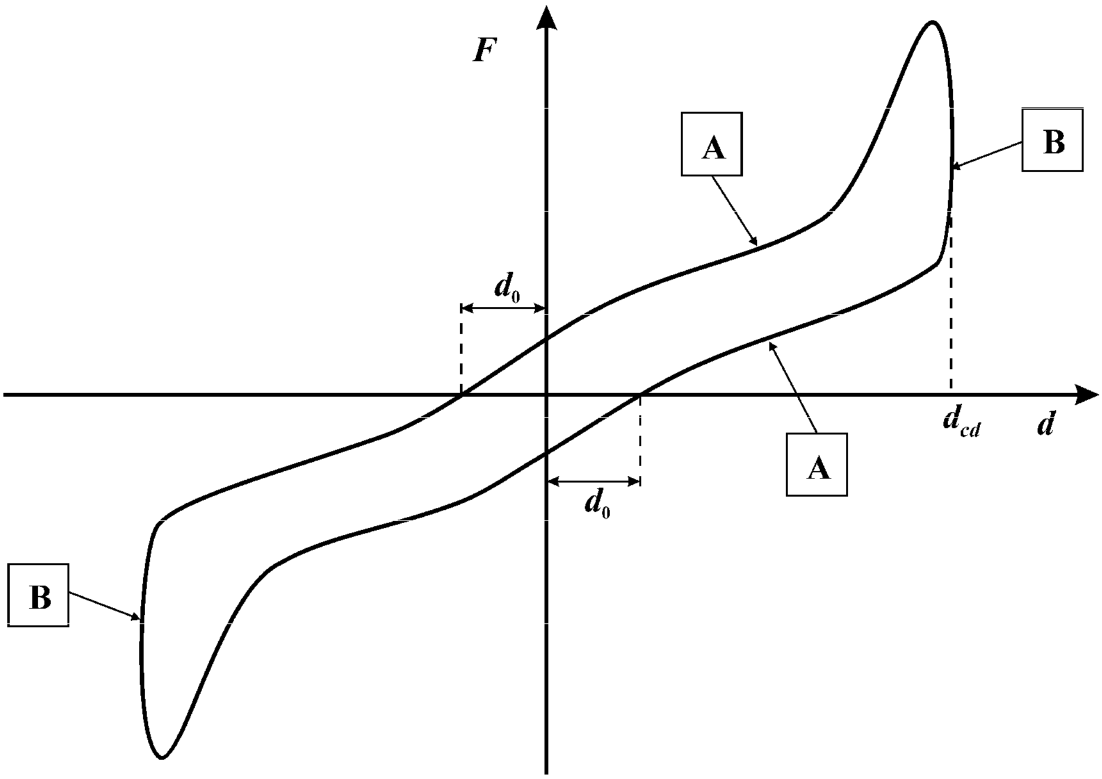
NOTE 5 For systems with hysteretic behaviour that cannot be approximated by a bilinear relationship (see Figure 7.7), the value of $d_0$can be defined as the intersection of the post-elastic branches with the axis of displacements. The displacement corresponding to the yield stress, $d_y$, can be assumed to be zero to increase reliability.




Legend:

| | |
|---|---|
| $F$ | Force |
| $d$ | Displacement |
| A | Actual force-displacement relationship |
| B | Bilinear approximation model (ABCD) |
| C | Equal areas |

**Figure 7.6 – Definition of the equivalent bilinear model for assessing resilience**




Legend:

*F* Force

*d* Displacement

$\boxed{A}$ Post-elastic branch

$\boxed{B}$ Elastic branch

**Figure 7.7 – Hysteretic systems that cannot be approximated by the bilinear model**

(3) Systems which do not satisfy condition (7.24) in a given direction may be considered to satisfy requirement **(1)** if they have sufficient displacement capacity to assume, with adequate reliability, the accumulation of residual displacements in that direction during the lifetime of the structure.

(4) The condition in **(3)** is considered to be fulfilled when the following conditions are satisfied for all isolators:

$$d_{m,i} \geq d_{G,i} + \gamma_{du}\, d_{bi,d}\, \rho_d \tag{7.26a}$$

where

$$\rho_d = 1 + 1{,}35\,\frac{1 - (d_y / d_{cd})^{0,6}}{1 + 80(d_{cd} / d_0)^{1,5}} \tag{7.26b}$$

and is represented in Figure 7.8

and

$d_{m,i}$ is the displacement capacity of the isolator *i* in the direction considered, i.e. the maximum displacement that the isolator can assume in that direction;

$d_{bi,d}$ is the calculated displacement of the isolator $i$ in the direction considered, according to section **7.6.2(1)**;

$d_{G,i}$ is the initial non-seismic displacement of the isolator $i$ according to section **7.6.2(2)**;

$d_y$ is the displacement corresponding to the yield stress of the equivalent bilinear system, determined in accordance with **(2)** above. For sliding systems, $d_y$ can be assumed to be zero. If there is uncertainty about the magnitude of $d_y$, it must be assumed to be zero;

$\gamma_{du}$ is a numerical coefficient representing the uncertainties in the estimation of the computational displacements.

NOTE 1 The value assigned to $\gamma_{du}$ is $\gamma_{du}$ = 1.20.

NOTE 2 The second term of the expression $\rho_d$ for in (7.26b) reflects the accumulation of residual displacements under the sequence of earthquake events occurring before the computational earthquake. For systems with $d_{cd}/d_0 \geq 0.50$, the residual displacement accumulation is negligible (see Figure 7.8). For systems with $d_{cd}/d_0 < \delta$ the maximum $d_{m,i}$ value should be obtained either from expression (7.26a) or from expression (7.19a), whichever is larger.

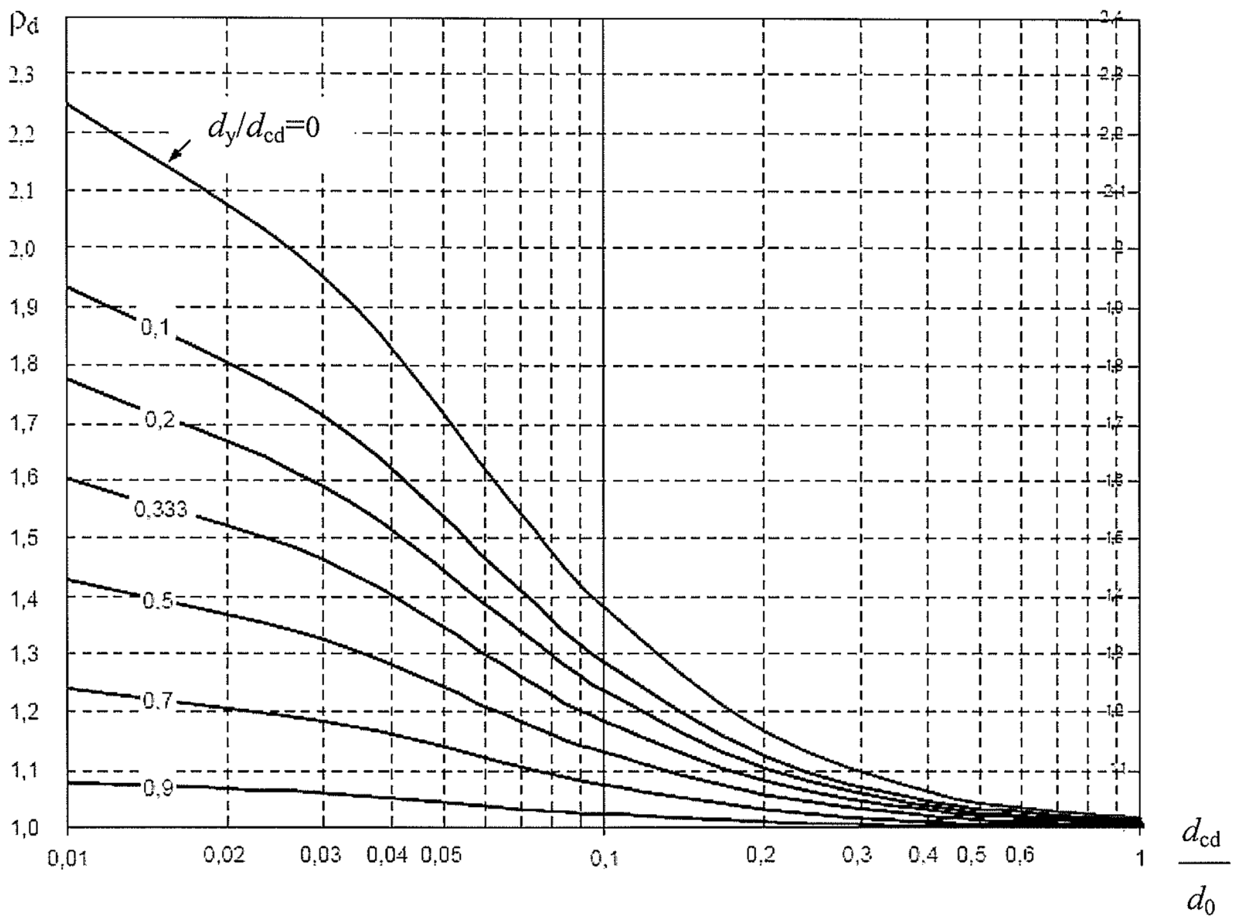



**Figure 7.8 – Representation of ρd according to the expression (7.26b)**

(5) The same properties of the isolators under dynamic conditions shall be used for the estimation of both $d_{cd}$ and $d_0$. The lateral recovery conditions (7.24) and (7.26) do not consider the effects of velocity variation on the isolator forces.

### 7.7.2 Lateral restraint at the isolation interface level

(1) The isolation system must provide sufficient lateral restraint to displacement at the isolation interface to satisfy any relevant requirements of the other specific regulations in force (see section 1.2), with respect to limiting displacements/deformations under the in-service performance criteria.

NOTE This requirement is normally critical for braking action on railway bridges.

(2) Where fungible triangulations (bracing) (fuse systems) are placed at the location of certain supports in the final configuration of the bridge to limit the displacement between the deck and the substructures, corresponding to the serviceability state, their elastic capacity shall not exceed 40 % of the design seismic force transmitted through the isolation interface of the isolated structure, in the same supports and direction. If this requirement is not met, the serviceability requirements (with the exception of fatigue) of the applicable regulations depending on the material (Annex 21, Annex 29 or Annex 32 of the Structural Code) must instead be met for the elements of the bridge structure subjected to the load for which the restraint bracing is designed when this load is increased so that the relevant reaction reaches the elastic capacity of this bracing.

NOTE Chapter **5** of UNE-EN 15129 provides specifications for rigid connection devices that can be used to provide lateral restraint at the isolation interface.

(3) Where impact transmission units with force limiting function (see **6.6.3.3**) are used to provide restraint to displacement in the service state, such impact transmission units shall be included in the model, in the tests and in the test method of the isolation system.

### 7.7.3 Inspection and maintenance

(1) All isolators shall be accessible for inspection and maintenance.

(2) An inspection and maintenance schedule must be drawn up for the isolation system and for all components passing through the isolation interface.

(3) The repair, replacement or refurbishment of any isolator or component crossing the isolation interface should be carried out under the direction of the entity responsible for bridge maintenance and should be detailed in an appropriate report.

## Appendix A

## Probabilities related to the reference seismic action - Recommendations for the selection of the calculated seismic action during the construction phase

### A.1 Reference seismic action

(1) The reference calculated seismic action can be defined by choosing an acceptably low probability ($p$) of being exceeded during the design life ($t_L$) of the structure. Consequently, the return period ($T_R$) of the earthquake is given by the expression:

$$T_R = 1/ (1 - (1 - p)^{1/t_L} \tag{A.1}$$

(2) The reference seismic action (corresponding to $\gamma_I = 1.0$) normally reflects an earthquake with a reference return period, $T_{NCR}$, of 475 years. Such an earthquake has a probability of exceedance between 0.10 and 0.19 for a design lifetime between 50 and 100 years, respectively. This level of calculated action is applicable to most bridges considered to be of medium importance.

### A.2 Calculated seismic action for the construction phase

(1) Assuming that $t_c$ is the duration of the construction phase of a bridge and $p$ is the acceptable probability of exceedance of the design earthquake during the construction phase, the return period $T_{Rc}$ is given by the expression (A.1), with $t_c$ in place of $t_L$. For the relatively small values normally associated with $t_c$ ($t_c$ ¿ 5 years), the expression (A.1) can be approximated by the following simpler relationship:

$$T_{Rc} \simeq \frac{t_c}{p} \tag{A.2}$$

It is recommended that the value of $p$ does not exceed 0.05.

(2) The calculated value of the ground acceleration $a_{gc}$ corresponding to a return period $T_{Rc}$ depends on the seismicity of the area. In many cases, the following relationship provides an acceptable approximation:

$$\frac{a_{gc}}{a_{g,R}} = \left( \frac{T_{Rc}}{T_{NCR}} \right)^k \tag{A.3}$$

where

$a_{gR}$ is the maximum acceleration at the reference ground level, corresponding to the return period $T_{NCR}$.

The value of the exponent k depends on the seismicity of the area. Values in the range 0.30 - 0.40 can normally be used.

(3) The robustness of all partial bridge structures should be ensured during the construction phases, irrespective of the calculated seismic actions.

## Appendix B

### Recommendations for the determination of the relationship between the displacement ductility and curvature ductility coefficients of plastic hinges in concrete piers

(1) Assuming that:

- the horizontal displacement at the centre of gravity of the deck is due only to the deformation of a perfectly embedded corbelled pier of length $L$, that
- the mass of the pier is negligible compared to the mass of the deck, and that
- $L_p$ is the length of the plastic hinge that develops at the base of the pier,

the required ductility coefficient in terms of curvatures for the hinge, $\mu_\Phi$, corresponding to a ductility coefficient in displacements of the structure $\mu_d$, as defined in section **2.3.5.2**, is:

$$\mu_\Phi = \frac{\Phi_u}{\Phi_y} = 1 + \frac{\mu_d - 1}{3\lambda(1 - 0{,}5\lambda)} \qquad \text{(B.1)}$$

where

$\lambda = L_p / L$

(2) In reinforced concrete sections (where the bending ductility coefficient is used as a measure of the ductility of the plastic hinge), the value of the ratio $\lambda$ is influenced by effects such as the penetration of the tensile reinforcement deformation into the adjoining element, inclined cracking due to bending-shear interaction, etc. The value of $L_p$ given in section **E.3.2(5)** can be used.

(3) When a considerable part of the displacement of the deck is due to the deformation of other elements that remain elastic after the formation of the plastic hinge, the required bending ductility coefficient,$\mu_{\Phi d}$, is given by the expression:

$$\mu_{\Phi d} = 1 + f(\mu_\Phi - 1) \qquad \text{(B.2)}$$

where

$f = d_{tot} / d_p$ is the quotient between the total displacement of the deck, $d_{tot}$, and the displacement $d_p$ due to the pier deformation only; and

$\mu_\Phi$ is calculated according to expression (B.1).

NOTE If, between the deck and the pier, the seismic action is transmitted through flexible elastomeric bearings inducing, for example, a value of $f$ = 5, and assuming that a certain value of $\mu_\Phi$ is required, e.g. $\mu_\Phi$ = 15, the required value of $\mu_{\Phi d}$ in case of fixed connection between the deck and the pier would be as high as 71, according to equation (B.2), which is certainly unattainable. Consequently, it is evident that the high flexibility of the elastomeric bearings used in the same force path as the rigid pier imposes a practically elastic overall behaviour of the system.

## Appendix C

## Recommendations for the estimation of the effective rigidity of ductile reinforced concrete members

## C.1 General considerations

(1) The effective rigidity of ductile concrete members used in linear seismic analysis should be equal to the secant stiffness at the theoretical yield stress. Unless otherwise demonstrated by calculation, one of the following approximate methods may be used to determine the secant rigidity at the theoretical yield stress.

## C.2 Method 1

(1) The effective moment of inertia, $J_{eff}$, of a constant cross-section pier can be estimated as follows:

$$J_{eff} = 0.08\, J_{un} + J_{cr} \tag{C.1}$$

where

$J_{un}$ is the moment of inertia of the gross section of the uncracked pier;

$J_{cr}$ is the moment of inertia of the cracked section when the tensile reinforcement is at yield stress. This moment can be estimated from the expression:

$$J_{cr} = M_y / (E_c \cdot \Phi_y) \tag{C.2}$$

where $M_y$ and $\Phi_y$ are, respectively, the moment corresponding to the yield stress and the curvature of the section, and $E_c$ is the elastic modulus of the concrete.

(2) These expressions have been obtained from a parametric analysis on a simplified non-linear model of a corbelled pier with either rectangular hollow or hollow solid circular cross-sections.

## C.3 Method 2

(1) The effective rigidity can be estimated from the ultimate design moment, $M_{Rd}$, and the yield stress curvature, $\Phi_y$, of the plastic hinge section as follows:

$$E_c J_{eff} = \nu M_{Rd} / \Phi_y \tag{C.3}$$

where

$\nu = 1.20$ is a correction coefficient reflecting the stiffening effect of the non-cracked part of the pier.

The curvature $\Phi_y$ corresponding to the yield stress can be determined as follows:

$$\Phi_y = (\varepsilon_{sy} - \varepsilon_{cy})/d_s \tag{C.4}$$

where

$d_s$ is the useful edge of the section with respect to the centre of the tensile reinforcement;

$\varepsilon_{sy}$ is the deflection corresponding to the yield stress of the reinforcement;

$\varepsilon_{cy}$ is the deformation in compression of the concrete, corresponding to the beginning of plastification of the tensile reinforcement.

The value of $\varepsilon_{cy}$ can be estimated by an analysis of the section based on $\varepsilon_{sy}$ and the actual force, $N_{Ed}$, acting in the calculated seismic situation.

(2) The adoption of the following values for the curvature at the yield point gives in general a satisfactory approximation:

for rectangular sections: $$\Phi_y = 2.1\ \varepsilon_{sy}/d \tag{C.5}$$

for circular sections: $$\Phi_y = 2.4\ \varepsilon_{sy}/d \tag{C.6}$$

where $d$ is the usable depth of the section.

(3) The analysis developed from a value of $E_c J_{eff}$ based on an assumed value of $M_{Rd}$ only needs to be corrected if the finally required value of the bending capacity, $M_{Rd,req}$, is significantly higher than the assumed value of $M_{Rd}$. If $M_{Rd,req} < M_{Rd}$, the correction can be made by multiplying the displacements resulting from the first analysis by the quotient $M_{Rd} / M_{Rd,req}$.

## Appendix D

## Spatial variability of seismic motion: model and recommended methods of analysis

## D.1 Description of the model

(1) The spatial variability can be described by a vector of zero-mean random processes. Under the assumption of stationarity, this vector is fully defined by its symmetric, $n \times n$-dimensional matrix of auto and intercorrelation functions of the power spectral density:

$$G(\omega) = \begin{bmatrix} G_{11}(\omega) & G_{12}(\omega) \ldots G_{1n}(\omega) \\ & G_{22}(\omega) \ldots G_{2n}(\omega) \\ & \ldots \quad \ldots \\ & G_{nm}(\omega) \end{bmatrix} \tag{D.1}$$

where $n$ is the number of supports.

It is useful to introduce the following dimensionless function of complex values, called *coherence function*:

$$\gamma_{ij}(\omega) = \frac{G_{ij}(\omega)}{\sqrt{G_{ii}(\omega) G_{jj}(\omega)}} \tag{D.2}$$

Its modulus is bounded by the values 0 and 1.0, and provides a measure of the linear statistical dependence of two processes on supports $i$ and $j$, whose distance is $d_{ij}$.

(2) The coherence function can be expressed as follows:

$$\gamma_{ij}(\omega) = \gamma_{ij,1}(\omega) \cdot \gamma_{ij,2}(\omega) \cdot \gamma_{ij,3}(\omega) = \exp\left[-\left(\frac{\alpha\omega d_{ij}}{v_s}\right)^2\right] \cdot \exp\left[i\frac{\omega d_{ij}^L}{v_{app}}\right] \cdot \exp\left[i\theta_{ij}(\omega)\right] \tag{D.3}$$

where

$v_s$ is the velocity of the transverse shear wave;

$\alpha$ is a constant;

$v_{app}$ is the so-called apparent wave velocity;

$d_{ij}^L$ is the distance between supports $i$ and $j$, projected along the direction of wave propagation; and

$\theta_{ij}(\omega)$ is a frequency-dependent phase angle.

(3) The factors $\gamma_{ij,1}(\omega)$, $\gamma_{ij,2}(\omega)$ and $\gamma_{ij,3}(\omega)$ take into account, respectively, the loss of correlation due to reflections / refractions in the propagation medium, the existence of a finite limit to the propagation velocity of the waves and their angle of incidence at the surface, and the different ground conditions at the locations of the two supports. The difference between the soil properties at the locations of the two supports is taken into account in the model by considering two soil columns, representing the two soil profiles, excited at their base by a stationary white noise of intensity $G_0$. The ground columns are characterised, respectively, by the transfer functions $H_i(\omega)$ and $H_j(\omega)$, able to provide the desired spectral content and intensity of the surface motion, at locations *i* and *j*.

$$G_{ii}(\omega) = G_0 \left| H_i(\omega) \right|^2 \tag{D.4}$$

(4) The power density spectrum at the site must be compatible with the elastic response spectrum given in section **3.2.2.2** of Annex 1.

It is also shown that:

$$\theta_{ij}(\omega) = \tan^{-1}\left\{ \frac{\mathrm{Im}\left[ H_i(\omega) H_j(-\omega) \right]}{\mathrm{Re}\left[ H_i(\omega) H_j(-\omega) \right]} \right\} \tag{D.5}$$

## D.2 Sample generation

(1) For the purpose of structural analysis, it may be necessary to generate samples from the vector of random processes described in section **D.1**. For this purpose, the matrix $\mathbf{G}(\omega)$ is first decomposed into the product:

$$\mathbf{G}(\omega) = \boldsymbol{L}(\omega) \boldsymbol{L}^{*T}(\omega) \tag{D.6}$$

of the matrix $\mathbf{L}(\omega)$ by the transpose of its complex conjugate. If the Cholesky decomposition is used, $\mathbf{L}(\omega)$is a lower triangular matrix.

Thus, a sample of the accelerogram in the generic support *i* can be obtained from the series:

$$a_i(t) = 2 \sum_{j=1}^{i} \sum_{k=1}^{N} \left| L_{ij}(\omega_k) \right| \sqrt{\Delta\omega} \cos\left[ \omega_k t - \theta_{ij}(\omega_k) + \phi_{jk} \right] \tag{D.7}$$

where

$N$ is the total number of frequencies $\omega_k$ at which the significant bandwidth of $L_{ij}(\omega)$ is discretised;

$\Delta\omega =$ $\omega_{max.}/N$, and the angles $\phi_{jk}$ are, for each element *j*, a set of *N* independent random variables, uniformly distributed between 0 and $2\pi$.

Samples generated according to expression (D.7) are characterised by both the desired frequency content and the degree of correlation assigned.

## D.3 Methods of analysis

### D.3.1 General considerations

(1) Based on sections **D.1** and **D.2**, the options described in sections **D.3.2** to **D.3.4** are useful for determining the structural response to spatially varying ground motions.

### D.3.2 Linear random vibration analysis

(1) A linear random vibration analysis is carried out by developing using a modal analysis of frequency-dependent transfer matrices with an excitation given by the matrix $\mathbf{G}(\omega)$.

(2) The elastic effects of the action are assumed to be equivalent to the mean values obtained from the probability distribution of the largest maximum value of the response, for the duration consistent with the earthquake considered for the definition of $a_g$.

(3) The calculated values are determined by dividing the elastic effects by the appropriate behavioural coefficient $q$, and compliance with the relevant rules in the normative part of this standard ensures a ductile response.

### D.3.3 Time domain analysis with correlated motion samples

(1) Linear time domain analyses can be carried out using sample accelerograms generated, as indicated in section **D.2**, from power spectra compatible with the elastic response spectra in the supports.

(2) The number of accelerograms used should be adequate to obtain stable estimators of the mean of the maximum responses of interest. The effects of the elastic actions are assumed to be equivalent to the mean values of the previous maximum responses. The calculated values are determined by dividing the effects of the elastic actions by the appropriate behavioural coefficient $q$, and compliance with the relevant rules in the normative part of this standard ensures the ductile response.

(3) Non-linear time domain analysis may be carried out using sample accelerograms generated, as indicated in section **D.2**, from power spectra compatible with the elastic response spectra in the supports. The number of accelerograms used should be adequate to obtain stable estimators of the mean of the maximum responses of interest.

(4) The calculated values of the $E_d$ action effects are assumed to be equivalent to the mean values of the previous maxima. The comparison between the design stress $E_d$ and the resistance $R_d$ is carried out as specified in Annex 1.

### D.3.4 Response spectrum for one excitation on several supports

#### D.3.4.1 General considerations

(1) Solutions for the elastic response of a structure subjected to a multi-support excitation can be found in the specialised technical literature. In the following, only the broad outlines of this analysis

will be presented.

### D.3.4.2 Linear response for multi-support excitation

(1) The equations of motion for a discretised linear system of $n$ degrees of freedom subjected to the motions of $m$ supports can be written as:

$$\begin{bmatrix} M & M_C \\ M_C^T & M_g \end{bmatrix} \begin{bmatrix} \ddot{x} \\ \ddot{u} \end{bmatrix} + \begin{bmatrix} C & C_C \\ C_C^T & C_g \end{bmatrix} \begin{bmatrix} \dot{x} \\ \dot{u} \end{bmatrix} + \begin{bmatrix} K & K_C \\ K_C^T & K_g \end{bmatrix} \begin{bmatrix} x \\ u \end{bmatrix} = \begin{bmatrix} 0 \\ F \end{bmatrix} \tag{D.8}$$

where

**x** is the $n$ - dimensional vector of all displacements in the unbound degrees of freedom;

**u** is the $m$ - dimensional vector of the indicated displacements of the supports;

**M**, **C** and **K** are, respectively, the $n \times n$ - dimensional matrices of mass, damping and rigidity, associated to the unbound degrees of freedom;

$\mathbf{M_g}$, $\mathbf{C_g}$ and $\mathbf{K_g}$ are, respectively, the $m \times m$ - dimensional matrices of mass, damping and rigidity, associated with the degrees of freedom of the supports;

$\mathbf{M_c}$, $\mathbf{C_c}$ and $\mathbf{K_c}$ are the coupled matrices, of dimension $n \times m$; and

**F** is the $m$ - dimensional vector of the reaction forces in the degrees of freedom of the supports.

(2) The total response is broken down into:

$$\mathbf{x} = \mathbf{x}^s + \mathbf{x}^d \tag{D.9}$$

where $\mathbf{x}^s$, called the pseudo-static component, is the solution of the expression (D.8) without the inertia and damping terms, i.e:

$$\mathbf{x}^s = -\mathbf{K}^{-1}\mathbf{K}_c\mathbf{u} = \mathbf{R}\mathbf{u} \tag{D.10}$$

Substituting expressions (D.9) and (D.10) into expression (D.8), we obtain the differential equation for the dynamic component, which is of the form:

$$\mathbf{M}\ddot{\mathbf{x}}^d + \mathbf{C}\dot{\mathbf{x}}^d + \mathbf{K}\mathbf{x}^d \simeq -\left(\mathbf{MR} + \mathbf{M}_c\right)\ddot{\mathbf{u}} \tag{D.11}$$

after eliminating the comparatively negligible term $-\left(\mathbf{CR} + \mathbf{C}_c\right)\dot{\mathbf{u}}$.

(3) Let, respectively, $\mathbf{\Phi}$, $\omega_i$ and $\xi_i$ be the modal shape matrix, the modal frequencies and the corresponding damping rates of the embedded base structure. By introducing $\mathbf{x}^d = \mathbf{\Phi y}$ in the expression (D.11), the uncoupled modal equations are obtained:

$$\ddot{y}_i + 2\xi_i\omega_i\dot{y}_i + \omega_i^2 y_i = \sum_{k=1}^{m} \beta_{ki}\ddot{u}_k(t) \qquad i = 1,\ldots,n \tag{D.12}$$

where the modal participation factor has the form:

$$\beta_{ki} = \frac{\boldsymbol{\varphi}_i^T(\mathbf{Mr}_k + \mathbf{M}_c\mathbf{i}_k)}{\boldsymbol{\varphi}_i^T\mathbf{M}\boldsymbol{\varphi}_i} \tag{D.13}$$

where $\mathbf{r}_k$ is the $k$-th column of the matrix $\mathbf{R}$, and $i_k$ is the $k$-th column of the identity $n \times n$ matrix.

(4) It is convenient to define a normalised modal response $s_{ki}(t)$, representing the response of a single degree-of-freedom oscillator with the frequency and damping rate of the $i$th mode, subjected to the acceleration $\ddot{u}_k(t)$ at the base:

$$\ddot{s}_{ki} + 2\xi_i\omega_i\dot{s}_{ki} + \omega_i^2 s_{ki} = \ddot{u}_k(t) \tag{D.14}$$

Clearly, one has:

$$y_i(t) = \sum_{k=1}^{m} \beta_{ki}s_{ki}(t) \tag{D.15}$$

(5) Any generic response of the variable of interest, $z(t)$ (displacement of a node, internal force, etc.), can be expressed by a linear function of the nodal displacement $\mathbf{x}(t)$:

$$z(t) = \mathbf{q}^T\mathbf{x}(t) = \mathbf{q}^T\left[\mathbf{x}^s(t) + \mathbf{x}^d(t)\right] \tag{D.16}$$

Substituting the expressions obtained for $\mathbf{x}^s$ and $\mathbf{x}^d$, one arrives at:

$$z(t) = \sum_{k=1}^{m} a_k u_k(t) + \sum_{k=1}^{m}\sum_{i=1}^{n} b_{ki}s_{ki}(t) \tag{D.17}$$

where:

$$a_k(t) = \mathbf{q}^T\mathbf{r}_k \qquad b_{ki} = \mathbf{q}^T\boldsymbol{\varphi}_i\beta_{ki} \tag{D.18}$$

#### D.3.4.3 Spectral response

(1) Using the underlying theory of random vibrations together with a model as described in section **D.1** for the support motions $\mathbf{u}(t)$, the standard deviation of the response of the variable of interest, $z(t)$, can be determined directly in terms of standard deviations of the excitation processes $\mathbf{u}(t)$ and of the normalised modal responses $\mathbf{s}(t)$, as well as of the correlation between the input and output values.

(2) Moreover, taking into account the relationship between the power spectral densities of the excitation processes, $\mathbf{G}_{\ddot{u}\ddot{u}}(\omega)$ [1], and the above standard deviations and correlations, as well as the relationships between the power spectral density of the response process and the response spectrum, the following expression for the mean value of the maximum response (i.e., the effect of the elastic action)[2] is derived:

$$\mu_{z_{\text{máx.}}} = \sqrt{\sum_{k=1}^{m}\sum_{l=1}^{m} a_k a_l \rho_{u_k u_l} u_{k,\text{máx.}} u_{l,\text{máx.}} + \sum_{k=1}^{m}\sum_{l=1}^{m}\sum_{i=1}^{n}\sum_{j=1}^{n} b_{ki} b_{lj} \rho_{s_{ki} s_{lj}} D_k(\omega_i, \xi_i) D_l(\omega_j, \xi_j)} \qquad \text{(D.19)}$$

where $u_{k,\text{max.}}$ and $u_{l,\text{max.}}$ are the maximum ground displacements at the location of supports $k$ and $l$, compatible with the respective local elastic response spectra, as defined in section **3.2.2.4** of Annex 1; $D_k(\omega_i, \xi_i)$ and $D_l(\omega_j, \xi_j)$ are the values of the elastic response spectra in displacements at supports $k$ and $l$ for the frequencies and damping rates of the considered modes, compatible with the respective local elastic response spectrum as defined in section **3.2.2.2** of Annex 1.

(3) The correlation coefficients, $\rho_{u_k u_l}$, between the maximum ground displacements, and $\rho_{s_{ki} s_{lj}}$ between the normalised modal responses, are given by:

$$\begin{aligned}
\rho_{u_k u_l} &= \frac{1}{\sigma_{u_k}\sigma_{u_l}} \int_{-\infty}^{\infty} G_{u_k u_l}(\omega)\, d\omega \\
\rho_{s_{ki} s_{lj}} &= \frac{1}{\sigma_{s_{ki}}\sigma_{s_{lj}}} \int_{-\infty}^{\infty} H_i(\omega)\, H_j(-\omega)\, G_{\ddot{u}_k \ddot{u}_l}(\omega)\, d\omega \\
\sigma^2_{u_k} &= \int_{-\infty}^{\infty} G_{u_k u_k}(\omega)\, d\omega \\
\sigma^2_{s_{ki}} &= \int_{-\infty}^{\infty} \left|H_i(\omega)\right|^2 G_{\ddot{u}_k \ddot{u}_k}(\omega)\, d\omega
\end{aligned} \qquad \text{(D.20)}$$

where $G_{u_k u_l}(\omega)$ is the $kl$ term of the matrix of power spectral densities of the ground displacement processes, related to the corresponding term for the acceleration processes by the relation $\mathbf{G}_{uu}(\omega) = \frac{1}{\omega^4}\mathbf{G}_{\ddot{u}\ddot{u}}(\omega)$; $H_i(\omega)$ is the frequency transfer function of the normalised modal shift, given by:

(D.21)

[1] )$\mathbf{G}_{\ddot{u}\ddot{u}}(\omega)$ denotes the matrix of the power spectral densities of the acceleration process at ground level which, for simplicity of notation, in section **D.1** is simply denoted $\mathbf{G}(\omega)$.

[2] )In expression (D.19) a contribution has been omitted which takes into account the correlation between the ***u*** and ***s***-terms, i.e. $\rho_{u_k s_{lj}}$. Numerical analysis shows that this contribution is negligible and can be disregarded.

$$H_i(\omega) = \frac{1}{\omega_i^2 - \omega^2 + i2\xi_i\omega_i\omega} \tag{D.21}$$

(4) In order to evaluate the integrals of expression (D.20), the power spectral densities should be associated with response spectra representing the information assumed to be available to the user of this method. The following expression can be used to relate the response and power spectrum at any station:

$$G_{\ddot{u}\ddot{u}}(\omega) = \omega^2\left(\frac{2\xi\omega}{\pi} + \frac{4}{\pi\tau}\right)\left[\frac{D(\omega,\xi)}{2,5}\right]^2 \qquad \omega \geq 0 \tag{D.22}$$

where $\tau$ is the duration of the stationary part of the ground motion, which is taken to be consistent with the earthquake considered for the definition of $a_g$.

(5) In practice, when local ground conditions differ from one support to another, the effect of this difference tends to predominate over the other two phenomena that generate correlation losses. Numerical analyses also show that the consideration in the coherence function of the third term $\gamma_{ij,3}(\omega)$, has little influence on the results and can therefore be considered approximately null. Based on these considerations and taking into account the approximate nature of the response spectrum method described, a significant simplification consists of considering a diagonal matrix $\mathbf{G}(\omega)$, i.e. considering the structure subjected to a vector of independent ground motion processes, each characterised by its own power spectral density function. Consequently, the expression (D.19) is simplified by:

$$u_{z_{máx.}} = \sqrt{\sum_{k=1}^{m} a_k^2 u_{k,máx.}^2 + \sum_{k=1}^{m}\sum_{i=1}^{n}\sum_{j=1}^{n} b_{ki} b_{kj} \rho_{s_{ki}s_{kj}} D_k(\omega_i,\xi_i) D_k(\omega_j,\xi_j)} \tag{D.23}$$

## Appendix E

## Recommendations on probable material properties and deformation capacities of plastic hinges for non-linear analyses

# E.1 General considerations

(1) This Appendix ~~Annex~~ provides recommendations for the selection of probable material properties and for the estimation of the deformation capacities of plastic hinges. Both cases are intended for use in non-linear analysis only, in accordance with sections **4.2.4** and **4.2.5**.

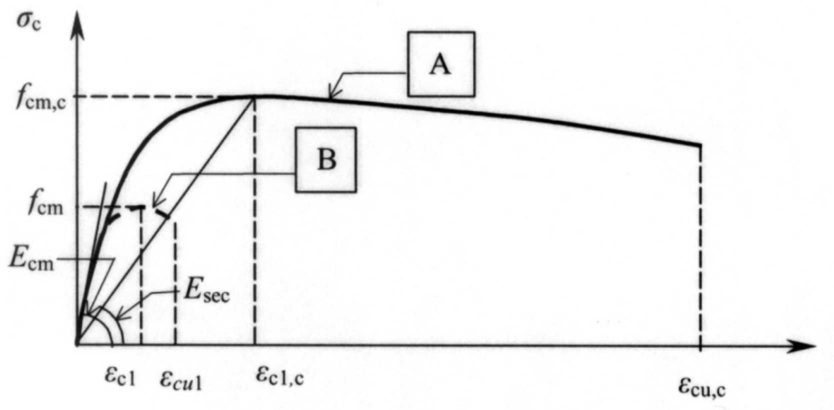
# E.2 Probable properties of materials

## E.2.1 Concrete

(1) The mean values $f_{cm}$ and $E_{cm}$, according to Table A19.3.1 of Annex 19 of the Structural Code, should be used.

(2) For unconfined concrete, the stress-strain relationship for non-linear analysis specified in section **3.1.5(1)** of Annex 19 of the Structural Code should be used, with the values of the deflections, $\varepsilon_{c1}$ and $\varepsilon_{cu1}$, given in Table A19.3.1 of the same standard.

(3) For confined concrete, the following method may be used as an alternative to section **3.1.9** of Annex 19 of the Structural Code (see Figure E.1):




Legend:

A Confined concrete

B Unconfined concrete

**Figure E.1 – Stress-strain relationship for confined concrete**

NOTE This property model for confined concrete is compatible with the values $\Phi_u$ and $L_p$ given in expressions (E.18) and (E.19), respectively.

(a) Concrete stress σc:

$$\frac{\sigma_c}{f_{cm,c}} = \frac{xr}{r - 1 + x^r} \tag{E.1}$$

where

$$x = \frac{\varepsilon_c}{\varepsilon_{c1,c}} \tag{E.2}$$

$$r = \frac{E_{cm}}{E_{cm} - E_{sec}} \tag{E.3}$$

secant modulus for latest load:

$$E_{sec} = \frac{f_{cm,c}}{\varepsilon_{c1,c}} \tag{E.4}$$

latest shear load:

$$f_{cm,c} = f_{cm}\lambda_c \tag{E.5}$$

$$\lambda_c = 2,254\sqrt{1 + 7,94\frac{\sigma_e}{f_{cm}}} - \frac{2\sigma_e}{f_{cm}} - 1,254 \tag{E.6}$$

deformation for latest load:

$$\varepsilon_{c1,c} = 0,002\left[1 + 5\left(\frac{f_{cm,c}}{f_{cm}} - 1\right)\right] \tag{E.7}$$

(b) Effective confining stress $\sigma_e$:

$\sigma_e$ is the effective confining stress acting in the two transverse directions, 2 and 3, ($\sigma_e = \sigma_{e2} = \sigma_{e3}$). This stress can be estimated as follows, based on the amount of confining reinforcement $\rho_w$, as defined in sections **6.2.1.2** or **6.2.1.3** and its probable yield strength $f_{ym}$:

- For circular or spiral frames:

$$\sigma_e = \frac{1}{2}\alpha\rho_w f_{ym} \tag{E.8}$$

- For rectangular frames or clevises:

$$\sigma_e = \alpha \rho_w f_{ym} \quad \text{(E.9)}$$

where $\alpha$ is the confinement efficiency coefficient (see section **5.4.3.2.2** of Annex 1).

For bridge piers confined in accordance with the construction detailing rules of section **6.2.1** and with a minimum dimension $b_{min.}$. ≅ 1.0 m, the value $\alpha \cong 1{,}0$ may be adopted.

NOTE If, in the case of orthogonal frames, the values of $\rho_w$ are not equal in the two transverse directions ($\rho_{w2}$ ¿ $\rho_{w3}$), the effective confining load can be estimated as: $\sigma_e = \sqrt{\sigma_{e2}\sigma_{e3}}$ .

(c) Latest deformation of the concrete $\varepsilon_{cu}$:

This deformation should correspond to the first rupture of the confining frame. Unless otherwise justified, it can be assumed as follows:

$$\varepsilon_{cu,c} = 0{,}004 + \frac{1{,}4 \rho_s f_{ym} \varepsilon_{su}}{f_{cm,c}} \quad \text{(E.10)}$$

where

$\rho_s = \rho_w$ for circular spirals or frames;

$\rho_s = 2\ \rho_w$ for orthogonal frames; and

$\varepsilon_{su} = \varepsilon_{um}$ is the average value of the elongation of the reinforcing steel for the maximum load (see section **3.2.2.2** of Annex 19 of the Structural Code).

### E.2.2 Passive reinforcement steel

(1) In the absence of relevant project-specific steel information, the following values may be used:

$$\frac{f_{ym}}{f_{yk}} = 1{,}15 \quad \text{(E.11)}$$

$$\frac{f_{tm}}{f_{tk}} = 1{,}20 \quad \text{(E.12)}$$

$$\varepsilon_{su} = \varepsilon_{uk} \quad \text{(E.13)}$$

### E.2.3 Structural steel

(1) In the absence of relevant project-specific steel information, the following values may be used:

$$\frac{f_{ym}}{f_{yn}} = 1{,}25 \tag{E.14}$$

$$\frac{f_{um}}{f_{un}} = 1{,}30 \tag{E.15}$$

where $f_{yn}$ and $f_{un}$ are, respectively, the nominal values of yield strength and ultimate tensile breaking load.

## E.3 Rotational capacity of plastic hinges

### E.3.1 General considerations

(1) In general, the rotational capacity of plastic hinges, $\theta_{p,u}$ (see requirement **c** of section **4.2.4.4(2)**), should be assessed on the basis of laboratory tests which satisfy the conditions of section **2.3.5.2(3)**, and which have been carried out on similar components. This applies to the deformation capacities of tensile members or plastic shear mechanisms used in off-centre structural steel triangulations.

(2) The similarity referred to above refers to the following aspects of parts or components, where applicable:

- geometry of the element;
- loading rate;
- relationships between the effects originating from the actions (bending moment, axial force, shear force);
- configuration of reinforcement (longitudinal and transverse reinforcement, including confining reinforcement), for reinforced concrete parts;
- local and/or shear buckling conditions of steel members.

(3) In the absence of specific justification based on actual data, the reduction coefficient $\gamma_{R,p}$ of expression (4.21) can be assumed as γR,p = 1.40.

### E.3.2 Reinforced concrete

(1) In the absence of appropriate laboratory test results, such as those mentioned in section **E.3.1**, the plastic rotation capacity $\theta_{p,u}$ and the total chord rotation $\theta_u$ of the plastic hinges (see Figure 2.4) can be estimated as follows, based on the ultimate curvature $\Phi_u$ and the length of the plastic hinge $L_p$ (see Figure E.2):

$$\theta_u = \theta_y + \theta_{p,u} \tag{E.16a}$$

$$\theta_{p,u} = (\Phi_u - \Phi_y) L_p \left(1 - \frac{L_p}{2L}\right) \tag{E.16b}$$

where

$L$ is the distance between the end section of the plastic hinge and the point of zero moment in the pier;

$\Phi_y$ is the curvature corresponding to the yield stress.

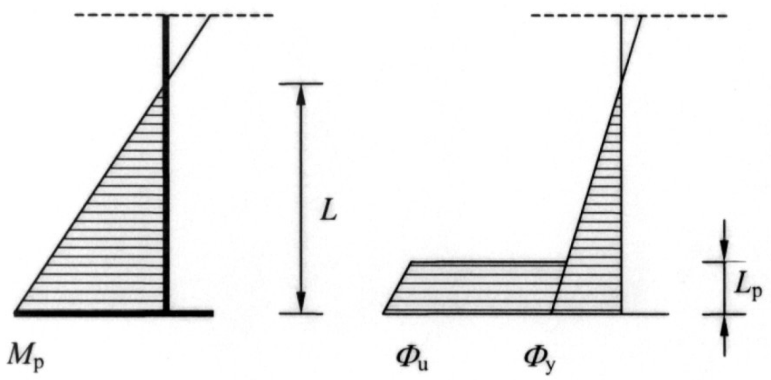


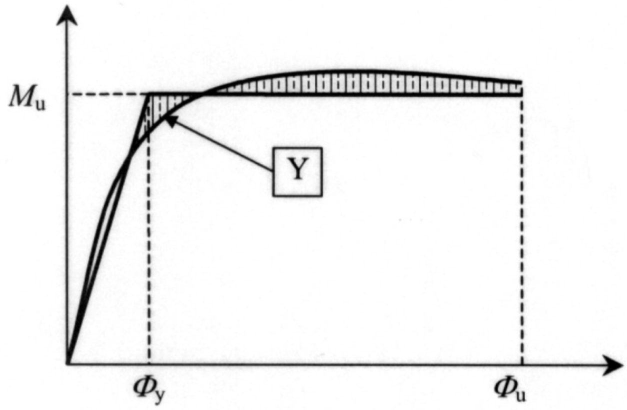

**Figure E.2 – $\Phi_y$ and $\Phi_u$**

For a linear variation of the bending moment, the rotation corresponding to the yield stress, $\theta_y$, can be taken as:

$$\theta_y = \frac{\Phi_y L}{3} \tag{E.17}$$

(2) Both $\Phi_y$ and $\Phi_u$ should be determined by means of a moment-curvature analysis of the section, under the axial load corresponding to the design seismic combination (see also **(4)**). When $\varepsilon_c$ ¿ $\varepsilon_{cu,1}$, only the confined concrete core section should be taken into account.

(3) $\Phi_y$ should be evaluated by idealising the actual M-Φ diagram by a bilinear diagram of equal area from the point where the first plastification of the reinforcement is exceeded, as shown in Figure E.3.




Legend:

Y Plastification of the first reinforcement

**Figure E.3 – Definition of $\boldsymbol{\Phi}_y$**

(4) The ultimate curvature $\Phi_u$ at the location of the plastic hinge of the element should be taken as:

$$\Phi_u = \frac{\varepsilon_s - \varepsilon_s}{d} \tag{E.18}$$

where

$d$ is the effective edge of the section;

$\varepsilon_s$ y $\varepsilon_c$ are, respectively, the deformations of the reinforcement and of the concrete (compressive deformations, negative), which are derived from the condition that one or both of them have reached the following ultimate values:

- $\varepsilon_{cu,1}$ for the unconfined concrete compression deflection (see Table A19.3.1 of Annex 19 of the Structural Code);
- $\varepsilon_{cu,c}$ for the compressive deflection of confined concrete (see section **E.2.1(3)(c)** or section **3.1.9(2)** of Annex 19 of the Structural Code);
- $\varepsilon_{su}$ for the tensile deflection of the reinforcement (see section **E.2.1(3)(c)**).

(5) For the case of a plastic hinge formed at the joints of the top or bottom of a pier with the deck or the foundation (footing or pile cap), with a longitudinal reinforcement of characteristic yield strength $f_{yk}$ (in MPa) and reinforcement diameter $d_{bL}$, the length $L_p$ of the plastic hinge can be assumed to be:

$$L_p = 0.10\,L + 0.015\,f_{yk}d_{bL} \tag{E.19}$$

where L is the distance between the plastic hinge and zero moment sections under the effect of seismic action.

(6) The above estimate of the plastic rotation capacity is valid for piers with shear rate:

$$\alpha_s = \frac{L}{d} \geq 3,0 \tag{E.20}$$

For values 1.0 $\leq \alpha_s \leq$ 3.0, the plastic rotation capacity should be multiplied by the reduction factor:

$$\lambda(\alpha_s) = \sqrt{\frac{\alpha_s}{3}} \tag{E.21}$$

## Appendix F

## Recommendations for determining the added mass of entrained water for submerged piers

(1) Unless otherwise justified by calculation, the total effective mass in the horizontal direction of a submerged pier should be considered equal to the sum of:

- the actual mass of the pier (without increase due to hydrostatic effect);
- the mass of water eventually contained inside the pier (for hollow piers);
- the added mass of water $m_a$ of the external water entrained per unit length of the submerged pier.

(2) For piers with a circular cross-section of radius $R$, $m_a$ can be estimated as:

$$m_a = \rho \pi R^2 \tag{F.1}$$

where $\rho$ is the density of water.

(3) For piers of elliptical section (see figure F.1), of axes $2a_x$ and $2a_y$, subjected to a horizontal seismic action forming an angle $\theta$ with the x-axis of the section, $m_a$ can be estimated as:

$$m_a = \rho \pi \left(a_y^2 \cos^2\theta + a_x^2 \sin^2\theta\right) \tag{F.2}$$

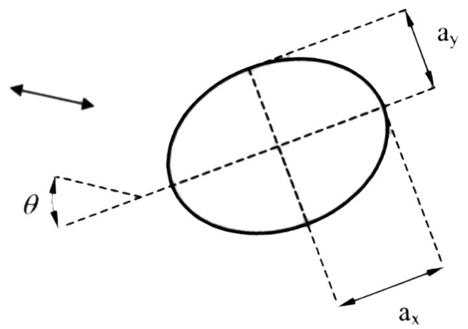



**Figure F.1 - Definition of the dimensions of an elliptical pier cross-section**

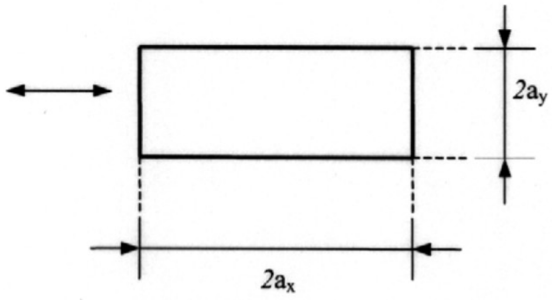



**Figure F.2 - Definition of the dimensions of a rectangular pier cross-section**

(4) For piers with a rectangular cross-section of dimensions $2a_x$ y $2a_y$ and for a symmetric action in the x-direction (see figure F.2), $m_a$ can be estimated as:

$$m_a = k\rho\pi a_y^2 \tag{F.3}$$

where the value of $k$ is taken from Table F.1 (linear interpolation is allowed).

**Table F.1 - Dependence of the added mass coefficient for rectangular piers with cross-section aspect ratio**

| $a_y/a_x$ | $k$ |
|---|---|
| 0.1 | 2.23 |
| 0.2 | 1.98 |
| 0.5 | 1.70 |
| 1.0 | 1.51 |
| 2.0 | 1.36 |
| 5.0 | 1.21 |
| 10.0 | 1.14 |
| ∞ | 1.00 |

# Appendix G

## Calculation of stresses for capacity dimensioning

### G.1 General procedure

(1) In general, the following procedure must be applied separately for each of the two horizontal components of the calculated seismic action with + or - signs:

(2) Step 1:

The calculated value of the ultimate resisting moment, $M_{Rd,h}$, of the intended plastic hinge sections corresponding to the selected horizontal direction of the seismic action ($A_E$) is determined with the sign considered (+ or -). The resistances must be based on the actual dimensions of the cross-sections and the final amount of longitudinal reinforcement. The calculation must consider the interaction with the axial force and, eventually, with the bending moment in the orthogonal direction, both obtained from the analysis for the combination corresponding to the calculated seismic situation given in expression (5.4) of section **5.5**.

(3) Step 2:

The variation $\Delta A_C$, of the effects of the actions of the plastic mechanism corresponding to the increase of the moments of the plastic hinges, ($\Delta M_h$), is determined from (a): the values due to the permanent actions ($M_{Gh}$), to (b): the moments of over-resistance of the sections:

$$\Delta M_h = \gamma_o M_{Rd,h} - M_{G,h} \quad \text{(G.1)}$$

where $\gamma_o$ is the coefficient of over-resistance (reserve resistance) specified in section **5.3**.

(4) The effects $\Delta A_C$ can be estimated, in general, from the equilibrium conditions, as long as reasonable approximations concerning the compatibility of deformations are acceptable.

(5) Step 3:

Finally, the stresses of the capacity dimensioning, $A_C$, must be obtained by superimposing the variation $\Delta A_C$ on the stresses due to the permanent actions $A_G$:

$$A_C = A_G + \Delta A_C \quad \text{(G.2)}$$

## G.2 Simplifications

(1) Simplifications of the general procedure specified in section **G.1** are permitted, as long as section **G.1(4)** is satisfied.

(2) When at the location of the plastic hinge, the bending moment due to permanent actions is negligible compared to the moment of over-resistance of the section ($M_{G,h} << \gamma_o M_{Rd,h}$), step 2 of section **G.1(3)** may be replaced by a direct estimation of the forces $\Delta A_C$ from the $A_E$ forces of the calculated seismic action. This is usually the case in the transverse direction of the piers, or in both directions when the piers are hinged to the deck. In such cases, the capacity sizing shear stress of pier '*i*' can be estimated as follows:

$$V_{C,i} = \Delta V_i = \frac{\gamma_o M_{Rd,h,1}}{M_{E,i}} V_{E,i} \tag{G.3}$$

and the capacity design effects on the deck and abutments can be estimated from the relation:

$$\Delta A_C \cong \frac{\Sigma V_{C,i}}{\Sigma V_{E,i}} A_E \tag{G.4}$$

# Appendix H

# Recommendations for non-linear static analysis (incremental pushover analysis)

## H.1 Analysis directions, reference points and maximum displacements

(1) The non-linear static analysis specified in section **4.2.5** should be carried out in the following two horizontal directions:

- the longitudinal direction *x*, defined by the centres of the two end sections of the deck;
- the transverse direction *y*, which should be assumed orthogonal to the longitudinal direction.

(2) The reference point should be the centre of gravity of the deformed deck.

(3) In each of the two horizontal directions, *x* and *y*, defined in **(1)** above, a non-linear static analysis should be carried out as specified in section **4.2.5**, until the following maximum displacements of the reference point are reached:

- in the *x*-direction (longitudinal):

$$d_{T,x} = d_{E,x} \tag{H.1}$$

- in the *y*-direction (transverse):

$$d_{T,y} = d_{E,y} \tag{H.2}$$

where

$d_{E,x}$ is the displacement in the *x*-direction at the centre of gravity of the deformed deck, resulting from an equivalent linear multi-modal spectral analysis (as specified in **4.2.1.3**), assuming $q$ = 1.0, due to $E_x$ "+" 0.3 $E$y. The spectral analysis should be carried out considering the effective rigidity of the ductile elements as specified in section **2.3.6.1**.

$d_{E,y}$ is the displacement in the *y*-direction at the same point, calculated in a similar way as above for $d_{Ex}$.

## H.2 Load distribution

(1) The horizontal load increments, $\Delta F_{ij}$, assumed to act on the concentrated masses $M_i$ in the studied direction at each load step $j$, should be taken equal to:

$$\Delta F_{i,j} = \Delta \alpha_j \, g \, M_i \, \zeta_i \tag{H.3}$$

where

$\Delta \alpha_j$ is the horizontal force increment, normalised to the weight $gM_i$ applied at step $j$; and

$\zeta_i$ is the shape factor defining the load distribution along the structure.

(2) Unless a better approximation is used, the following two distributions should be considered:

a) *constant distribution along the deck*, where

for the deck:

$$\zeta_i = 1 \tag{H.4}$$

and for the piers connected to the deck:

$$\zeta_i = \frac{z_i}{z_P} \tag{H.5}$$

where

$z_i$ is the height of point *i* above the foundation of each pier, and

$z_p$ is the total height of the pier *P* (distance between the terrain and the deck axis).

b) *distribution proportional to the deformation of the first mode*, where

$\zeta_i$ is proportional to the component, in the horizontal direction considered, of the modal displacement of the first mode at point *i*, in the same direction. The mode having the largest participation factor in the direction considered should be taken as the first mode in that direction. In the case of piers, especially, the following approximation can be considered as an alternative:

$$\zeta_i = \zeta_{TP} \frac{z_i}{z_P} \tag{H.6}$$

where $\zeta_{T,P}$ is the value of $\zeta$ corresponding to the node connecting the deck to the pier *P*.

## H.3 Deformation demands

(1) For each plastic hinge the deformation demands should be tested using the expression (4.20), where $\theta_{Ed}$ represents the maximum rotational demand when the maximum displacement is reached (see requirement (c) in section **4.2.4.4(2)**).

(2) In each direction, the total deformation corresponding to the first load step when the two terms of expression (4.20) become equal for any plastic hinge, defines the ultimate design deformation state of the bridge. If, in this state, the displacement of the reference point is less than the maximum displacement in the relevant direction, the design is considered unsatisfactory and should therefore be modified.

NOTE 1 Increasing the longitudinal reinforcement of the critical sections of plastic hinges, within the limits allowed by the construction, leads firstly to a consequent increase of the effective rigidity of the ductile elements (according to section **2.3.6.1**) and thus to a reduction of the maximum displacement according to section **H.1(3)** on the one hand, and of the deformation demands $\theta_{Ed}$ of section **H.3(1)** on the other hand. In general, the increase of the cross-section dimensions of ductile elements leads to a reduction of the deformation demands as well as to an increase of the deformation capacities of the elements.

NOTE 2 A design procedure for ductile members following these criteria involves only strain/displacement testing (no resistance testing). However, testing of non-ductile (shear) failures of both ductile and non-ductile members is carried out by means of resistance checks, in accordance with requirement **(e)** in section **4.2.4.4(2)**.

(3) In the longitudinal direction of an essentially straight bridge, the displacements of all the pier heads attached to the deck are nearly equal to the displacement of the reference point. In this case, the deformation demands of the plastic hinges can be evaluated directly from the maximum displacement.

## H.4 Testing the deck

(1) Testing should be carried out to ensure that, in accordance with section **5.6.3.6(2)** and **(3)**, no significant plastification occurs in the deck before the maximum displacement is reached (see requirement **(d)** in section **4.2.4.4(2)**).

(2) Lifting should be avoided in all devices on the same support before the target displacement is reached. Lifting of individual devices on the same support before the target displacement is reached is acceptable if it has no detrimental effect on the devices.

## H.5 Testing of non-ductile failure modes and foundation terrain

(1) In accordance with requirement **(e)** of **section 4.2.4.4(2)**, all elements should be tested for non-ductile (shear) failure modes, considering as calculated actions the distribution of forces corresponding to the maximum displacement. The same applies for testing of the foundation terrain.

# Appendix J

## Variations in the design properties of isolators

### J.1 Factors giving rise to changes or variations in design properties

(1) The determination of the design properties in terms of the upper and lower boundaries (UBDP and LBDP), required for the calculation of the isolation systems in accordance with section **7.5.2.4**, shall be established by evaluating the influence on each property of the following factors:

- $f_1$: ageing (including corrosion);
- $f_2$: temperature (minimum design temperature of the isolator, $T_{min.,b}$);
- $f_3$: contamination;
- $f_4$: cumulative travel (wear).

In general, the design properties of the cyclic response affected by the above factors are as follows (see Figures 7.1 and 7.3):

- the post-elastic rigidity $K_p$,
- the force corresponding to zero displacement, $F_o$.

(2) The minimum temperature of the isolator for the calculated seismic situation, $T_{min,b}$, shall correspond to the climatic conditions at the bridge site.

The method for determining the value of the minimum temperature of the isolator, in the calculated seismic situation, is as follows:

$$T_{min.,b} = T_{av} - \psi_2 \,(T_{av} - T_{min.}) + \Delta T_1$$

where

| | |
|---|---|
| $T_{min}$ | is the minimum value of the shaded temperature at the bridge location with a negative exceedance probability of 0,02. |
| $T_{av}$ | is the mean temperature at the bridge location and can be replaced by the mean between the maximum and minimum temperature. |
| $\psi_2 = 0.50$ | is the combination coefficient for the thermal action in the calculated seismic situation. |
| $\Delta T_1 = T_{e,min.} - T_{min.}$ | is the difference between the minimum uniform temperature component on a bridge, $T_{e,min.}$, and the minimum shaded air temperature (with an annual probability of being exceeded of 0.02) $T_{min.}$. |

## J.2 Assessment of variation

(1) In general, the effect on each calculation property of each of the factors $f_i$ ($i$ = 1 to 4) listed in section **J.1**, shall be assessed by comparing: (a) the maximum and minimum values (*max.DP*$_{fi}$ and *min.DP*$_{fi}$) of the design property, resulting from the influence of the factor $f_i$, with (b) respectively the maximum and minimum nominal values of the same property (*max.DP*$_{nom}$ and *min.DP*$_{nom}$), obtained from its measurement by prototype tests. The following indices shall be established to assess the influence of each factor $f_i$ on the calculation property under investigation:

$$\lambda_{máx.,fi} = \frac{máx.DP_{fi}}{máx.DP_{nom}} \tag{J.2}$$

$$\lambda_{mín.,fi} = \frac{mín.DP_{fi}}{mín.DP_{nom}} \tag{J.3}$$

NOTE 1 Appendix K provides recommendations on prototype (or type) tests for cases where the Standard UNE-EN 15129 (Anti-seismic devices) does not include detailed requirements for such tests.

NOTE 2 The values of the recommended $\lambda$ coefficients for commonly used isolators (i.e. for special elastomeric bearings, lead-core rubber bearings, sliding isolators and hydraulic viscous dampers) are given in Appendix **JJ**.

(2) The effective upper bound design property (UBDP) used in the dimensioning shall be estimated as follows:

$$\text{UBDP} = max.DP_{nom}.\lambda_{U,f1}.\lambda_{U,f2} \ldots \lambda_{U,f5} \tag{J.4}$$

with the modification coefficients:

$$\lambda_{U,f1} = 1 + (\lambda_{max.,fi} - 1)\,\psi_{fi} \tag{J.5}$$

where the combination coefficients $\psi_{fi}$ take into account the small probability of the simultaneous occurrence of the maximum adverse effects of all factors, and whose values should be considered according to Table J.2:

**Table J.2 - Combination coefficients $\psi_{fi}$**

| Importance class | $\psi_{fi}$ |
|---|---|
| III | 0.90 |
| II | 0.70 |
| I | 0.60 |

(3) In general, a similar format to expressions (J.4) and (J.5) should be used, together with $\lambda_{\mathrm{min.,fi}}$, for the effective lower bound design property (LBDP) (and corresponding modification coefficients $\lambda_{\mathrm{L,fi}}$). However, for the normally used elastomeric and friction bearings, it can be assumed in general that:

$$\lambda_{\mathrm{min,\,fi}} = 1 \tag{J.6}$$

and therefore:

$$\mathrm{LBDP} = min.DP_{\mathrm{nom}} \tag{J.7}$$

(4) For hydraulic dampers and in the absence of specific tests, it can be assumed that:

UBDP = $max.DP_{nom}$

LBDP = $min.DP_{nom}$

# Appendix JJ

## Recommended λ coefficients for current types of isolators

### JJ.1 $\lambda_{max.}$ values for elastomeric supports

(1) Unless other values are justified by appropriate tests, the $\lambda_{max.}$ values specified in the following tables JJ.1 to JJ.4 can be used for the estimation of the upper bound design properties (UBDP).

**Table JJ.1 - $f_1$ - Ageing**

| Component | $\lambda_{max.,f1}$ for | |
|---|---|---|
| | $K_p$ | $F_o$ |
| LDRB | 1.1 | 1.1 |
| HDRB1 | 1.2 | 1.2 |
| HDRB2 | 1.3 | 1.3 |
| Lead core | - | 1.0 |

With the following designation for rubber components:

LDRB: Low damping rubber backing with shear modulus greater than 0.5 MPa for 100% shear deformation;

HDRB1: High damping rubber bearing with $\xi_{eff} \leq 0.15$ and shear modulus, for 100% shear deformation, greater than 0.5 MPa;

HDRB2: High damping rubber backing with $\xi_{eff} > 0.15$ or shear modulus, for 100% shear strain, less than or equal to 0.5 MPa;

Lead core: Lead core for lead core rubber bearings (LRB).

**Table JJ.2 - $f_2$ - Temperature**

| Calculated temperature $T_{min.,b}$ (°C) | $\lambda_{max,f2}$ for | | | | | |
|---|---|---|---|---|---|---|
| | $K_p$ | | | $F_o$ | | |
| | LDRB | HDRB1 | HDRB2 | LDRB | HDRB1 | HDRB2 |
| 20 | 1.0 | 1.0 | 1.0 | 1.0 | 1.0 | 1.0 |
| 0 | 1.1 | 1.1 | 1.2 | 1.3 | 1.3 | 1.3 |
| - 10 | 1.1 | 1.2 | 1.4 | 1.4 | 1.4 | 1.4 |
| - 30 | 1.3 | 1.4 | 2.0 | 1.5 | 2.0 | 2.5 |

$T_{min,b}$ is the minimum temperature of the isolator for the calculated seismic situation corresponding to the bridge location (see Chapter **J.1(2)**).

**Table JJ.3 - $f_3$ - Contamination**

| |
|---|
| $\lambda_{max.,f3}$ = 1.0 |

**Table JJ.4 - $f_4$ - Cumulative travel**

| | |
|---|---|
| Rubber | $\lambda_{max.,f4}$ = 1.0 |
| Lead core | To be established by test |

## JJ.2 Values of $\lambda_{max.}$ for sliding isolators

(1) Unless different values are justified by appropriate test results, the $\lambda_{max.}$ values specified in the tables, JJ.5 to JJ.8 below, may be used for the estimation of the maximum force for zero displacement, $F_o$, corresponding to the upper bound design property (UBDP). The values given for unlubricated polytetrafluoroethylene (PTFE) elements may be considered applicable also for friction pendulum bearing devices.

**Table JJ.5 - $f_1$ - Ageing**

| Component | $\lambda_{max.,f}$ | | | | | |
|---|---|---|---|---|---|---|
| | PTFE Unlubricated | | PTFE lubricated | | Bimetallic interfaces | |
| Environment | Sealed | Unsealed | Sealed | Unsealed | Sealed | Unsealed |
| Normal | 1.1 | 1.2 | 1.3 | 1.4 | 2.0 | 2.2 |
| Severe | 1.2 | 1.5 | 1.4 | 1.8 | 2.2 | 2.5 |

The values in table JJ.5 refer to the following conditions:

- Stainless steel skid plates are assumed.
- Unsealed conditions are assumed, to allow exposure of the sliding surfaces to water and salt.
- The severe environment includes marine and industrial conditions.

Values for bimetallic mating surfaces apply to stainless steel to bronze mating surfaces.

**Table JJ.6 - $f_2$ - Temperature**

| Calculated temperature | $\lambda_{max.,f2}$ | | |
|---|---|---|---|
| $T_{min,b}$ (°C) | PTFE unlubricated | PTFE lubricated | Bimetallic interfaces |
| 20 | 1.0 | 1.0 | To be established by tests |
| 0 | 1.1 | 1.3 | |
| - 10 | 1.2 | 1.5 | |
| - 30 | 1.5 | 3.0 | |

**Table JJ.7 - $f_3$ - Contamination**

| Facility | $\lambda_{max,f3}$ | | |
|---|---|---|---|
| | PTFE unlubricated | PTFE lubricated | Bimetallic interfaces |
| Sealed with stainless steel surface facing downwards | 1.0 | 1.0 | 1.0 |
| Sealed with stainless steel surface upwards | 1.1 | 1.1 | 1.1 |
| Unsealed with stainless steel surface downwards | 1.2 | 3.0 | 1.1 |

The values in table JJ.7 refer to the following conditions:

- The sealing of the supporting devices is supposed to offer protection against contamination for all service conditions.

**Table JJ.8 - $f_4$ - Cumulative travel**

| | $\lambda_{max,,f4}$ | | |
|---|---|---|---|
| Cumulative travel (km) | PTFE unlubricated | PTFE lubricated | Bimetallic interfaces |
| $\leq$ 1.0 | 1.0 | 1.0 | To be established by tests |
| 1.0 < and $\leq$ 2 | 1.2 | 1.0 | To be established by tests |

**Appendix K**

**Recommended tests for validation of calculated properties of seismic isolators**

## K.1 Field of application

(1) This Appendix is intended to provide recommendations on prototype (or type) tests for cases where the Standard UNE-EN 15129 ('Anti-seismic devices') does not include detailed requirements for such tests.

(2) The tests described in this Annex can be used to validate the range of values of the deformation characteristics as well as the damping values of the isolators used in the design and analysis of seismically isolated bridges. These tests are not intended for use as quality control tests.

(3) The prototype tests specified in section **K.2** are intended to establish or validate the range of nominal design properties of the isolators envisaged in the design. In general, these tests may be project specific. However, the available results of tests carried out on specimens of similar type and size and with similar values for the design parameters are accepted.

(4) The purpose of the tests in section **K.3** is to justify the properties of isolators which are not normally project specific.

## K.2 Prototype tests

### K.2.1 General considerations

(1) Tests should be carried out on a minimum of two specimens. The specimens should not be subjected to any lateral or vertical loading prior to the prototype test.

(2) In general, full-size specimens should be used. The competent authority may authorise certain tests to be carried out on scaled-down specimens only when existing testing facilities do not have the capacity to test full-size specimens.

(3) Where full-scale specimens are used, they should be of the same type and material, with similar geometry to full-scale specimens, and should be manufactured by the same process and with the same quality control.

### K.2.2 Sequence of tests

(1) The following sequence of tests should be performed, for the prescribed number of cycles and with a vertical load equal to the average permanent load, on all isolators of the same type and size:

$T_1$ Three complete cycles of alternating loading between the plus and minus of the maximum thermal displacement, at a test speed of not less than 0.1 mm/min.

$T_2$ Twenty complete cycles of alternating loading at plus or minus the calculated non-seismic response, at an average test frequency of 0.5 Hz. Following the cyclic test, the load should be maintained for 1 min on the specimen.

$T_3$ Five complete alternating cycles, up to the calculated value of the increased seismic displacement.

$T_4$ Fifteen complete alternating cycles to the calculated value of the increased displacement, starting from the initial displacement (**7.6.2(2)**). The cycles may be applied in three groups of five cycles each, with each group separated by a dead time to allow for cooling of the specimen.

$T_5$ Repeat test $T_2$ but with the number of cycles reduced to three.

$T_6$ If an isolator is also a vertical load-bearing element, then it should also be tested for one complete alternating cycle up to the full calculated seismic displacement, under the following vertical loads:

$1.2\ Q_G + |\Delta F_{Ed}|$

$0.8\ Q_G - |\Delta F_{Ed}|$

where

$Q_G$ is the permanent load, and

$\Delta F_{Ed}$ is the additional vertical load due to overturning seismic effects, determined from the maximum response for the calculated seismic action.

(2) Tests $T_3$, $T_4$ and $T_6$ should be carried out at a frequency equal to the inverse of the effective period of the isolation system. Exceptions to this rule are permitted for isolators that are not dependent on the rate or speed of load application (the speed of load application has the primary effect of viscous or frictional heating of the specimen). The force-displacement characteristics of an isolator are considered to be independent of the rate of load application if there is less than 15% difference for any of the values of $F_o$ and $K_p$ defining the hysteresis loop (see Figure 7.1), when tested for three complete alternating cycles up to the calculated value of the displacement and at frequencies in the range of 0.2 to 2 times the inverse of the effective period of the isolation system.

## K.2.3 Determination of the characteristics of the isolators

### K.2.3.1 Force-displacement characteristics

(1) The effective rigidity of an isolator should be calculated for each load cycle as follows:

$$K_{eff} = \frac{F_p - F_n}{d_p - d_n} \qquad \text{(K.1)}$$

where

$d_p$ y $d_n$ are, respectively, the maximum positive and negative displacements of the test; and

$F_p$ and $F_n$ are, respectively, the maximum positive and negative forces for isolators with hysteretic and frictional behaviour or, respectively, the positive and negative loads corresponding to $d_p$ and $d_n$ for isolators with viscoelastic behaviour.

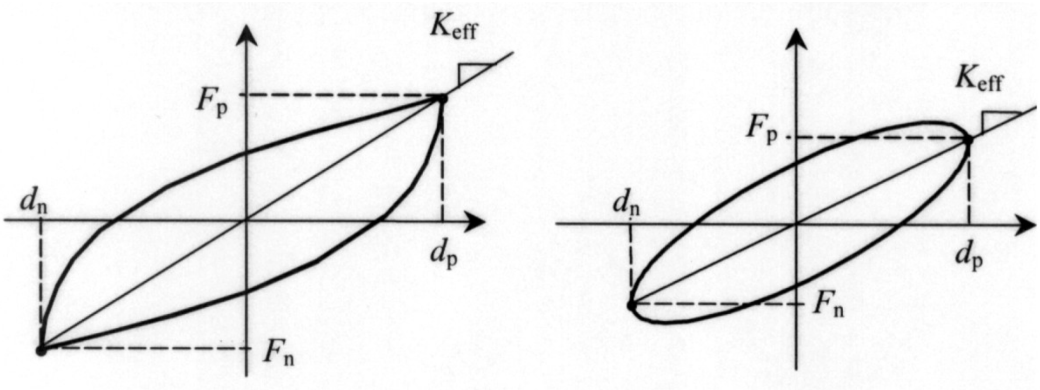



**Figure K1 – Force-displacement test diagrams (on the left: hysteretic or frictional behaviour; on the right: viscoelastic behaviour)**

#### K.2.3.2 Damping characteristics

(1) The energy dissipated per cycle, $E_{Di}$, by an isolator $i$, should be determined for each load cycle as the area of the corresponding hysteresis loop of the five complete alternating cycles up to the total calculated displacement defined in test $T_3$ of section **K.2.2**.

#### K.2.3.3 Adequacy of the system

(1) The performance of the tested specimens should be considered adequate if the following requirements are satisfied:

$R_1$ except for fluid viscous dampers, the force-displacement plots of all tests listed in section **K.2.2** should have a positive ability to resist increasing forces.

$R_2$ in test $T_1$ of section **K.2.2**, the maximum measured force should not exceed the calculated value by more than 5 %.

$R_3$ in tests $T_2$ and $T_5$ in section **K.2.2**, the maximum measured displacement should not exceed 110 % of the calculated value.

$R_4$ in test $T_3$ of section **K.2.2**, the maximum and minimum values of the effective $K_{effi}$ rigidity of the isolator $i$ (and the corresponding force-displacement diagrams) and the energy dissipated per cycle, $E_{Di}$, should be determined as the maximum and minimum of the average of each of the four pairs of consecutive test cycles, respectively. These nominal properties should be within the range of nominal properties assumed in the design.

$R_5$ in the $T_4$ test in section **K.2.2**, the ratio between the minimum and maximum effective stiffness, measured over each of the 15 cycles, should not be less than 0.7.

$R_6$ in the $T_4$ test in section **K.2.2**, the ratio of min.$E_D$/max.$E_D$ for each of the 15 cycles should not be less than 0.7.

$R_7$ all isolators resisting vertical loads should remain stable (i.e. with positive incremental rigidity), during test $T_6$ of section **K.2.2**.

$R_8$ At the conclusion of the tests, all tested specimens should be inspected for evidence of significant deterioration that could constitute grounds for rejection, such as (where applicable):

- Lack of adhesion between rubber and steel.
- Failure of the laminate to lay.
- Cracks in the rubber surface wider or deeper than 70% of the rubber cover thickness.
- Flaking of material on more than 5% of the bonding area.
- Lack of adhesion of polytetrafluoroethylene (PTFE) to metal on more than 5% of the

bonding area.
- Marking of stainless steel plates by marks deeper or wider than 0.5 mm and over a length exceeding 20 mm.
- Permanent deformation.
- Leakage.

## K.3 Other tests

### K.3.1 Wear and fatigue tests

(1) These tests should take into account the influence of accumulated travel due to displacements caused by thermal and traffic loads over a service life of at least 30 years.

(2) For bridges of normal length (up to about 200 m) and unless a different value is justified by the calculation, the minimum cumulative travel may be taken as 2 000 m.

### K.3.2 Low temperature tests

(1) If the isolators are intended for use in low temperature zones, with a minimum temperature for the seismic design of the isolator, $T_{min.,b}$ < 0 °C (see section **J.1(2)**), then a test at this temperature, consisting of five complete alternating cycles up to the calculated value of the displacement with the remaining conditions as specified in test $T_3$ in section **K.2.2**, should be carried out. The specimen should be kept below freezing temperature for at least two days prior to the test. The results should be evaluated as specified in requirement $R_4$ of section **K.2.3.3(1)**.

(2) In the tests of section **K.3.1**, 10 % of the run should be carried out under temperature $T_{min.,b}$.

# ANNEX 3

# Design of seismically resistant structures

# Evaluation and seismic adequacy of buildings.

## 1 General considerations

### 1.1 Object and field of application

(1) The purpose and scope of the Seismic Resistance Standard is defined in section **1.1.1** of Annex 1 and the purpose and scope of this Annex is defined in **(2)**, **(4)** and **(5)** below. Section **1.1.3** of Annex 1 indicates the additional parts of the Seismic Resistance Standard.

(2) The purpose and scope of Annex 3 is as follows:

- To provide criteria for the assessment of the seismic resistance performance of existing buildings considered in isolation.

- To describe an approach for the selection of necessary remedial measures.

- To establish criteria for the design of seismic retrofit measures (i.e. design, structural analysis including intervention measures, final sizing of structural elements and their connections to existing structural elements).

NOTE For the purpose of this Annex, seismic retrofitting covers both the strengthening of undamaged structures and the strengthening of earthquake damaged structures.

(3) When designing the intervention on a structure, in order to provide it with adequate resistance against seismic actions, it shall also be checked for non-seismic load combinations.

(4) Reflecting the basic requirements of Annex 1, this Annex covers the seismic assessment and retrofitting of buildings made of the most common structural materials: concrete, steel and masonry.

NOTE Appendices A, B and C contain additional information related to the assessment of reinforced concrete, steel and composite and masonry buildings, respectively, and their upgrading where necessary. These appendices are not of a regulatory nature.

(5) Although the provisions of this standard apply to all categories of buildings, the seismic assessment and upgrading of monuments and historic buildings often requires different types of provisions and different approaches, depending on the nature of the monuments.

(6) Taking into account that existing buildings:

i) reflect the state of knowledge at the time of their construction,

ii) possibly contain significant hidden errors,

iii) may have been subject to previous earthquakes or other accidental actions with unknown effects, the structural assessment and possible intervention is generally subject to a different degree of uncertainty (level of knowledge) than the design of new structures. Different sets of safety factors for materials and structures are therefore required, as well as different analysis procedures depending on the completeness and reliability of the available information.

## 1.2 Standards for consultation

(1) The provisions of section **1.2** of Annex 1 apply.

## 1.3 Assumptions

(1) The provisions of section **1.3** of Annex 1 apply

(2) The provisions of this standard assume that data collection and testing are carried out by experienced personnel and that the technician responsible for the assessment, possible seismic retrofit design and execution of the works, has appropriate experience for the type of structures to be strengthened or repaired.

(3) Inspection procedures, checklists and other data collection procedures must be documented and archived and referred to in the project documents.

## 1.4 International system of units (I.S.)

(1) The provisions of section **1.4** of Annex 1 apply

## 1.5 Terms and definitions

(1) See section **1.5** of Annex 1.

## 1.6 Symbols

### 1.6.1 General considerations

(1) See section **1.6** of Annex 1.

(2) The additional symbols used in this Annex are defined in the text when they appear.

### 1.6.2 Symbols used in Appendix A

| | |
|---|---|
| $b$ | width of loops on steel liners |
| $b_o$ and $h_o$ | dimensions of the confined concrete core measured from the axis of the frames |
| $b_i$ | spacing between longitudinal reinforcement axes |
| $c$ | concrete cover of the reinforcement |
| $d$ | effective cross-section depth (depth to the tensile reinforcement) |
| $d'$ | mechanical cover of the compression reinforcement |

| | |
|---|---|
| $d_{bL}$ | diameter of tensile reinforcement |
| $f_c$ | compressive strength of concrete (MPa) |
| $f_{cc}$ | compressive strength of confined concrete |
| $f_{cd}$ | calculated value of the resistance of concrete |
| $f_{ctm}$ | mean tensile strength of concrete |
| $f_{fdd,e}$ | calculated value for the effective peel resistance of FRP (fibre-reinforced polymer) |
| $f_{fu,W}(R)$ | ultimate resistance of the FRP sheet wrapped around a corner with radius $R$, expression (A.25) |
| $f_y$ | estimated mean value of the yield strength of the steel |
| $f_{yd}$ | calculated value of the yield strength of the reinforcement (longitudinal) |
| $f_{yj,d}$ | calculated value of yield strength of the liner |
| $f_{yw}$ | yield strength of transverse or confining reinforcement |
| $h$ | cross-section height |
| $k_b$ | $= \sqrt{1,5 \cdot (2 - w_f / s_f) / (1 + w_f / 100\ \text{mm})}$ FRP (fibre-reinforced polymer) strip/ply overlap coefficient |
| $n$ | number of overlapping reinforcements along the perimeter $p$ |
| $p)$ | length of the perimeter line inscribed in the pier section along the inside face of the longitudinal reinforcements |
| $s$ | spacing of the abutment axes |
| $s_f$ | spacing of the axes of the FRP (fibre-reinforced polymer) strips (= $w_f$ for FRP sheets) |
| $t_f$ | thickness of the FRP (fibre-reinforced polymer) sheet |
| $t_j$ | thickness of the liner |
| $x$ | compressed block depth |
| $w_f$ | width of FRP (fibre-reinforced polymer) strip/sheet |
| $z$ | internal mechanical arm of a section |
| $A_c$ | transverse area of the pier |
| $A_f$ | $= t_f \cdot w_f$ - sin $\beta$: horizontally projected transverse area of FRP (fibre-reinforced polymer) strip/sheet with thickness $t_f$, width $w_f$ and angle $\beta$ |
| $A_s$ | transverse area of longitudinal steel reinforcement |
| $A_{sw}$ | transverse area of the abutment |
| $E_f$ | FRP (fibre-reinforced polymer) modulus |
| $L_v$ | $= M/V$ shear span at the end of the element |
| $N$ | axial force (positive for compression) |
| $V_{R,c}$ | shear resistance of an element without web reinforcement |
| $V_{R,max.}$ | shear resistance determined by the exhaustion of the compressive strength in the diagonal compression linkage |

| | |
|---|---|
| $V_W$ | contribution of the transverse reinforcement to the shear resistance |
| $\alpha$ | containment efficiency factor |
| $\gamma_{el}$ | coefficient, greater than 1.0 for primary seismic elements and equal to 1.0 for secondary seismic elements |
| $\gamma_{fd}$ | partial FRP (fibre-reinforced polymer) debonding coefficient |
| $\delta$ | angle between the diagonal and the axis of a pier |
| $\varepsilon_{cu}$ | ultimate unit deflection of concrete |
| $\varepsilon_{ju}$ | ultimate unit deflection of FRP (fibre-reinforced polymer) |
| $\varepsilon_{su,w}$ | ultimate unit deflection of the confining reinforcement |
| $\theta$ | angle of inclination of the connecting rods in the shear force calculation |
| $\theta_y$ | chord rotation in the yield stress for a concrete element |
| $\theta_u$ | ultimate chord rotation for a concrete element |
| $v$ | $= N/bhf_c$ ($b$ width of the compressed block) |
| $\rho_d$ | amount of diagonal reinforcement |
| $\rho_f$ | volumetric amount of FRP (fibre-reinforced polymer) |
| $\rho_s$ | geometric amount of steel reinforcement |
| $\rho_{sx}$ | $= A_{sx}/b_w s_h$ = amount of transverse reinforcement parallel to the $x$-direction of the loads ($s_h$ = spacing between abutments) |
| $\rho_{tot}$ | total amount of longitudinal reinforcement |
| $\rho_{sw}$ | volumetric amount of confinement reinforcement |
| $\rho_w$ | amount of transverse reinforcement |
| $\varphi_u$ | ultimate curvature at the end section |
| $\varphi_y$ | yield stress curvature at the end section |
| $\omega, \omega'$ | mechanical value of tensile and compression reinforcement |

### 1.6.3 Symbols used in Appendix B

| | |
|---|---|
| $b_{cp}$ | width of the cover plate |
| $b_f$ | flange width |
| $d_c$ | edge of the pier |
| $d_z$ | depth of panel area between continuity plates |
| $e$ | distance between the plastic hinge and the abutment face |
| $f_c$ | compressive strength of concrete |
| $f_{ct}$ | tensile strength of concrete |
| $f_{uw}$ | tensile strength of welds |
| $f_{ywh}$ | yield strength of transverse reinforcement |
| $f_{y,pl}$ | nominal yield strength of each flange |
| $l_{cp}$ | length of the cover plate |
| $t_{cp}$ | thickness of the cover plate |
| $t_f$ | thickness (of the flange) |
| $t_{hw}$ | web thickness |

| | |
|---|---|
| $w_z$ | width of the panel area between the flanges of the pier |
| $A_g$ | gross section area |
| $A_{hf}$ | gusset flange area |
| $A_{pl}$ | area of each flange |
| $B_S$ | width of the flat brace (triangulation element) made of steel |
| $B$ | width of mixed section |
| $E$ | Young's modulus of the beam |
| $E_B$ | modulus of elasticity of RC (reinforced concrete) panel |
| $F_t$ | seismic shear stress of the foundation |
| $S$ | gantry height |
| $H_c$ | gantry storey height |
| $K_\varphi$ | rotational rigidity of the connection |
| $I$ | moment of inertia |
| $L$ | beam span |
| $M_{pb,Rd}$ | plastic moment of beam |
| $N_d$ | axial load calculated value |
| $N_y$ | yield strength of the steel triangulation element |
| $S_x$ | modulus of elasticity of the beam (with respect to its strong axis) |
| $T_C$ | panel thickness |
| $V_{pl,Rd,b}$ | shear stress in the plastic hinge of a beam |
| $Z_b$ | plastic modulus of the beam |
| $Z_e$ | effective plastic modulus of the cross-section at the location of the plastic hinge |
| $\rho_w$ | amount of transverse reinforcement |

### 1.7 SI Units

(1) See section **1.7** of Annex 1.

## 2 Performance requirements and compliance criteria

### 2.1 Key requirements

(1) The fundamental requirements relate to the level of damage of the structure, defined in this standard by three limit states (LS), namely: near collapse (NC), significant damage (SD) and damage limitation (DL). These limit states are to be characterised as follows:

Near Collapse (NC) limit state: The structure is severely damaged, with low residual stiffness and lateral resistance, but the vertical elements are still capable of supporting vertical loads. Most of the non-structural elements have collapsed. Significant permanent relative displacements occur. The structure is close to collapse and would probably not withstand another earthquake, even one of moderate intensity.

Significant Damage limit state (SD): The structure is significantly damaged, with some residual lateral rigidity and resistance, and the vertical elements are capable of supporting vertical loads. The non-structural elements are damaged, although the partitions and infills have not failed outside their midplane. Moderate permanent relative displacements occur. The structure can withstand aftershocks of moderate intensity. Repair of the structure may not be cost-effective.

Damage Limitation (DL) limit state: The structure is only slightly damaged, with structural elements not having undergone significant plastification and retaining their resistance and rigidity properties.

Non-structural elements, such as partitions and infills, may show widespread cracking, but are economically feasible to repair. Permanent relative displacements are negligible. The structure does not require any repair measures.

NOTE The definition of the limit state of proximity to collapse given in this annex is closer to the actual collapse of the building than that given in Annex 1 and corresponds to the maximum exploitation of the deformation capacity of the structural elements. The limit state associated with the requirement of non-collapse (absence of collapse) in Annex 1 is practically equivalent to that defined here as the limit state of significant damage.

(2) For buildings of importance class I and II, only the SD (significant damage) limit state should be considered. For buildings of importance class III and IV, the SD (significant damage), DL (damage limitation) and NC (near collapse) limit states should be considered.

(3) Appropriate levels of protection are achieved by selecting, for each of the limit states, a return period for the seismic action.

The protection normally considered appropriate for ordinary new buildings is obtained by selecting the following values for the return periods:

- Near Collapse (NC) limit state: 2475 years, corresponding to an exceedance probability of 2 % in 50 years.
- Significant Damage limit state (SD): 475 years, corresponding to an exceedance probability of 10 % in 50 years.
- Damage Limitation Limit State (DL): 225 years, corresponding to an exceedance probability of 20 % in 50 years.

## 2.2 Compliance criteria

### 2.2.1 General considerations

(1) Compliance with the requirements of section **2.1** is achieved by adopting the seismic action, method of analysis, verification procedures and construction details contained in this Annex, as appropriate for the different structural materials within its scope (i.e. concrete, steel, masonry).

(2) Except when the $q$-ratio approach is used (see **(4)**), compliance is tested with the full (unreduced, elastic) seismic action as defined in sections **2.1** and **4.2** for the appropriate return period.

(3) For the verification of structural members a distinction is made between 'ductile' and 'brittle'. Except when using the $q$-ratio approach, the former are to be verified by checking that the demands do not exceed the corresponding capacities in terms of deflections. The latter should be verified by checking that the demands do not exceed the corresponding capacities in terms of resistances.

NOTE For the classification of components/mechanisms as 'ductile' or 'brittle', information can be found in the appendices for the corresponding materials.

(4) Alternatively, a $q$-ratio approach can be used, where a seismic action reduced by a $q$-ratio is used, as described in section **4.2(3)**. In the safety checks, all structural elements must be checked by verifying that the demands due to the reduced seismic action do not exceed the corresponding capacities in terms of resistances, evaluated in accordance with **(5)**.

(5) For the calculation of the capacities of ductile or brittle elements, when these are compared with the demands for safety verification according to **(3)** and **(4)**, the average values of the existing material properties must be used as obtained directly from *in situ* tests and from additional sources of information, suitably divided by the confidence coefficients defined in section **3.5**, taking into account the level of knowledge achieved. For new or added materials the nominal properties should be used.

(6) Some of the existing structural elements may be designated as ‘secondary seismic’ in accordance with the definitions in section **4.2.2(1)**, **(2)** and **(3)** of Annex 1. 'Secondary seismic' elements must be verified against the same compliance criteria as 'primary seismic' elements, but using less conservative estimates of their capacity than those used for elements considered as 'primary seismic'.

(7) In the calculation of the resistant capacities of brittle ‘primary seismic’ elements, the material resistances must be divided by the partial coefficient of the material. The same values considered in the design of new buildings are adopted for the partial material coefficients.

NOTE Sections **5.2.4(3)**, **6.1.3(1)**, **7.1.3(1)** and **9.6(3)** of Annex 1 refer to the values of the partial safety factors for steel, concrete, structural steel and masonry for use in the design of new buildings.

### 2.2.2 Near Collapse (NC) limit state

(1) The demands should be based on the calculated seismic action relevant for this limit state. For ductile and brittle elements, the demands should be evaluated based on the results of the analyses. If a linear method of analysis is used, the demands on brittle elements should be modified according to section **4.5.1(1)**.

(2) The capacities must be based on appropriate definitions of ultimate deformations for ductile elements and ultimate resistances for brittle elements.

(3) The $q$-ratio approach (see section **2.2.1(4)** and section **4.2(3)**) is not normally suitable for testing this limit state.

NOTE The values of $q$ = 1.5 and 2.0 quoted in section **4.2(3)** for reinforced concrete and steel structures respectively, as well as the higher values of $q$ that can be justified with respect to local and total available ductility in accordance with the relevant provisions of Annex 1, correspond to compliance with the limit state of significant damage. If this approach is chosen for testing the near collapse limit state, then section **2.2.3(3)** can be applied, with a value of the coefficient $q$ approximately one third higher than those given in section **4.2(3)**.

### 2.2.3 Significant Damage (SD) limit state

(1) The demands should be based on the calculated seismic action relevant for this limit state. For ductile and brittle elements, the demands should be evaluated based on the results of the analyses. In case a linear method of analysis is used, the demands on brittle elements should be modified according to section **4.5.1(1)**.

(2) Except where the $q$-ratio approach is used, capacities should be assessed on the basis of representative damage deformations for ductile members and conservative estimates of the resistances of brittle members.

(3) In the $q$-ratio approach (see section **2.2.1(4)** and section **4.2(3)**), the demands must be based on the reduced seismic action and the capacities must be evaluated as for non-seismic design situations.

### 2.2.4 Damage Limitation (DL) limit state

(1) The demands should be based on the calculated seismic action relevant for this limit state.

(2) Except when using the $q$-ratio approach, capacities should be assessed based on the yield strengths of all structural elements, both ductile and brittle. The capacities of infills should be assessed on the basis of the average relative displacement capacity between floors for the infills.

(3) In the $q$-ratio approach (see section **2.2.1(4)** and section **4.2(3)**) demands and capacities are to be compared in terms of the mean and relative displacement between floors.

# 3 Information for structural evaluation

## 3.1 General and historical information

(1) In the assessment of the earthquake resistance of existing structures, input data should be collected from a variety of sources, including:

- the available documentation of the building in question;
- relevant generic data sources (e.g. contemporary standards and codes);
- field investigations; and
- in many cases, *in situ* and/or laboratory measurements and tests, as described in more detail in sections **3.2** and **3.4**.

(2) In order to minimise uncertainties, data obtained from different sources should be compared.

## 3.2 Required input data

(1) In general, the information for the structural assessment shall cover points (a) to (i) below:

a) Identification of the structural system and its compliance with the compliance criteria of section **4.2.3** of Annex 1. The information shall be collected from *in situ* surveys or from original project drawings, if available. In this case, information on possible structural changes since construction shall also be collected.

b) Identification of the type of foundations of the building.

c) Identification of the terrain conditions according to the classification in section **3.1** of Annex 1.

d) Information about the overall dimensions and properties of the cross-sections of the building elements, as well as the mechanical properties and the condition of the materials.

e) Information about identifiable defects of materials and inadequate construction details.

f) Information on the seismic design criteria used for the initial design, including the value of the force reduction coefficient (q coefficient), if applicable.

g) Description of the current and/or intended use of the building (identifying its class of significance as described in section **4.2.5** of Annex 1).

h) Re-evaluation of the imposed actions taking into account the use of the building.

i) Information about the type and extent of previous and current structural damage, if any, including previous remedial actions.

(2) Depending on the quantity and quality of the information collected in the previous sections, different types of analysis and different values of the confidence coefficients should be adopted, as indicated in section **3.3**.

## 3.3 Levels of knowledge

### 3.3.1 Definition of levels of knowledge

(1) In order to choose the type of admissible analysis and the appropriate values of the confidence coefficients, the following three levels of knowledge are defined:

KL1: Limited knowledge

KL2: Normal knowledge

KL3: Complete knowledge

(2) The factors that determine the appropriate level of knowledge (i.e. KL1, KL2 or KL3) are:

i) *geometry*: the geometric properties of the structural system and of those non-structural elements (e.g. masonry infill panels) that could affect the response of the structure;

ii. *constructional details*: including the amount and the cut-out of passive reinforcement in the reinforced concrete, the connections between the steel elements, the connection between the slabs working as diaphragms with the structure resisting lateral forces, the rigging and mortar joints in the masonry and the nature of all reinforcement elements in the masonry;

iii) *materials*: the mechanical properties of the materials.

(3) The level of knowledge achieved determines the permitted method of analysis (see **4.4**), as well as the values to be adopted for the confidence coefficients (CF). Procedures for obtaining the required data are given in section **3.4**.

(4) The relationship between the levels of knowledge and the applicable methods of analysis and the confidence coefficients are illustrated in Table 3.1. The terms 'visual', 'comprehensive', 'limited', 'extended' and 'full' are defined in section **3.4**.

**Table 3.1 - Levels of knowledge and corresponding methods of analysis (LF: lateral force method; MRS: modal analysis by response spectra) and confidence coefficients (CF)**

<table>
<tr><th>Level of knowledge</th><th>Geometry</th><th>Construction details</th><th>Materials</th><th>Analysis</th><th>CF</th></tr>
<tr><td>KL1</td><td rowspan="3">From the original general plans of the construction project with a visual inspection of samples<br>or<br>from a comprehensive inspection</td><td>Simulated project according to the original practice<br>and<br>limited in situ inspection</td><td>Default values according to the standards applicable at the time of construction<br>and<br>limited in situ tests</td><td>LF-MRS</td><td>$CF_{KL1} = 1.35$</td></tr>
<tr><td>KL2</td><td>from incomplete original detailed construction drawings with limited in situ inspection<br>or<br>Froman extended in situ inspection</td><td>From the original project specifications with limited in situ tests<br>or<br>from extended in situ testing</td><td>All</td><td>$CF_{KL2} = 1.20$</td></tr>
<tr><td>KL3</td><td>From the original detailed drawings of the construction project with limited in situ inspection<br>or<br>from a full in situ inspection</td><td>From the original test reports with limited in situ testing<br>or<br>from full in situ tests</td><td>All</td><td>$CF_{KL3} = 1.00$</td></tr>
</table>

### 3.3.2 KL1 Limited knowledge

(1) KL1 corresponds to the following state of knowledge:

i) *geometry*: the overall geometry of the structure and the dimensions of the elements are known either (a) by inspection or (b) from the original general construction design drawings used both in the original construction and in any subsequent modifications. In case (b), a sufficient sample of the measurements of both the overall geometry and the dimensions of the elements shall be tested *in situ*; if there are significant deviations from the original construction design drawings, a more complete dimensional inspection shall be carried out.

ii. *construction details*: structural construction details are not known from detailed design drawings and can be assumed based on a simulation of the design according to normal practice at the time of construction; in this case, limited inspections should be carried out on the most critical elements to check that the assumed assumptions correspond to the real situation. Otherwise, a more extensive *in situ* inspection is required.

iii) *materials*: no direct information on the mechanical properties of the construction materials is available either from the original design specifications or from the original test reports. Default values should be taken according to the standards at the time of construction, together with limited *in situ* testing of the most critical elements.

(2) The information collected must be sufficient to carry out local checks of the capacity of the elements and to establish a linear structural analysis model.

(3) Structural assessment based on a limited state of knowledge must be carried out by means of linear analysis methods, either static or dynamic (see **4.4**).

### 3.3.3 KL2: Normal knowledge

(1) KL2 corresponds to the following state of knowledge:

i) *geometry*: the geometry of the overall structure and the dimensions of the elements are known either (a) from an extended inspection or (b) from general plans of the construction project used both in the original construction and in any subsequent modifications. In case of (b), a sufficient sample of measurements of both the overall geometry and dimensions of the elements shall be tested *in situ*; if there are significant discrepancies from the general construction design drawings, a more complete inspection of the dimensions is required.

ii. *construction details*: structural construction details are known either from extended *in situ* inspections or from incomplete detailed design drawings. In the latter case, limited *in situ* inspections should be carried out on the most critical elements to check that the available information corresponds to the actual situation.

iii) *materials*: information on the mechanical properties of building materials is available either from extended in situ tests or from original design specifications. In the latter case, limited *in situ* tests shall be carried out.

(2) The information collected shall be sufficient to perform local verifications of the capacity of the element and to establish a linear or non-linear structural model.

(3) Structural assessment based on this state of knowledge can be performed by both linear and non-linear analysis methods, either static or dynamic (see **4.4**).

### 3.3.4 KL3: Complete knowledge

(1) KL3 corresponds to the following state of knowledge:

i) *geometry*: the geometry of the complete structure and the dimensions of the elements are known either (a) from a complete inspection or (b) from a complete set of general construction design drawings used both in the original construction and in any subsequent modifications. In case of (b), a sufficiently large sample of measurements of both the overall geometry and the dimensions of the components shall be tested *in situ*; in the case of significant deviations from the overall design drawings, a more complete dimensional inspection shall be carried out.

ii. *construction details*: structural construction details are known either from complete *in situ* inspections or from a complete set of detailed design drawings. In the latter case, *in situ* inspections limited to the most critical elements should be carried out to check that the available information corresponds to the actual situation.

iii) *materials*: information on the mechanical properties of the construction materials is available either from complete *in situ* tests or from original test reports. In the latter case, limited *in situ* testing should be carried out.

(2) Section **3.3.3(2)** applies.

(3) Section **3.3.3(3)** applies.

## 3.4 Identification of the level of knowledge

### 3.4.1 Geometry

#### 3.4.1.1 General plans of the construction project

(1) The general plans of the construction project are those documents that describe the geometry of the structure, allowing the identification of the structural elements and their dimensions, as well as the structural system that resists both vertical and lateral actions.

#### 3.4.1.2 Detailed design drawings of the construction project

(1) Detailed design drawings are those documents that describe the geometry of the structure, allowing the identification of the structural elements and their dimensions, as well as the structural system resisting both vertical and lateral actions. These drawings also contain information about the construction details (as specified in section **3.3.1 (2)**).

#### 3.4.1.3 Visual inspection.

(1) A visual inspection is a procedure used to check the correspondence between the actual geometry of the structure and the available general construction project drawings. Measurements of the geometry shall be made on a sample of selected elements. Possible structural changes that may occur during or after construction shall be subject to inspection according to section **3.4.1.4**.

#### 3.4.1.4 Comprehensive inspection

(1) A comprehensive inspection is a procedure that allows the generation of drawings describing the geometry of the structure, allowing the identification of the structural elements and their dimensions, as well as the structural system resisting both vertical and lateral actions.

### 3.4.2 Construction details

(1) Reliable, non-destructive methods can be adopted for inspections, as specified below.

#### 3.4.2.1 Project simulation

(1) A project simulation is a procedure to determine the amount and arrangement of passive reinforcement, both longitudinal and transverse, in all elements that contribute to the vertical and lateral resistance of the building. The project must be carried out on the basis of the regulatory documents and the state of the art at the time of construction.

#### 3.4.2.2 Limited *in situ* inspection

(1) A limited *in situ* inspection is a procedure used for testing the correspondence between the actual construction details of the structure and the available detailed design drawings or the results of the design simulation in section **3.4.2.1**. This involves carrying out inspections as described in section **3.4.4(1)**.

#### 3.4.2.3 Extended *in situ* inspection

(1) An extended *in situ* inspection is a procedure used when the original detailed design drawings are not available. This involves carrying out inspections as described in section **3.4.4(1)**.

#### 3.4.2.4 Complete *in situ* inspection

(1) A full *in situ* inspection is a procedure used when detailed drawings of the original project are not available and when a higher level of knowledge is sought. This involves carrying out inspections as described in section **3.4.4(1)**.

### 3.4.3 Materials

#### 3.4.3.1 Destructive and non-destructive testing

(1) The use of non-destructive testing methods (e.g. sclerometer testing, etc.) should be considered; however, these tests shall not be used in isolation, but in conjunction with destructive tests.

#### 3.4.3.2 Limited *in situ* testing

(1) A limited *in situ* testing programme is a procedure used to supplement the information on material properties obtained from the regulations in force at the time of construction or from the original project specifications, or from the original test reports. This involves carrying out tests as described in section **3.4.4(1)**. However, if the test values are lower than the default values according to the regulations in force at the time of construction, extended *in situ* tests are required.

#### 3.4.3.3 Extended *in situ* testing

(1) An extended *in situ* test programme is a procedure used to obtain information when the original project specifications and test reports are not available. This involves carrying out tests as described in section **3.4.4(1)**.

#### 3.4.3.4 Full *in situ* testing

(1) A full *in situ* testing programme is a procedure used to obtain information when neither the original project specifications nor the test reports are available and a higher level of knowledge is sought. This involves carrying out inspections as described in section **3.4.4(1)**.

### 3.4.4 Definition of inspection and testing levels

(1) The classification of inspection and testing levels depends on the percentage of structural elements whose construction details have to be checked, as well as on the number of material samples per storey to be taken for testing.

NOTE Table 3.2 gives, for ordinary situations, the minimum requirements for the number of inspections and tests to be carried out.

**Table 3.2 - Minimum requirements for different levels of inspection and testing**

| | Inspection (of construction details) | Testing (of materials) |
|---|---|---|
| | For each type of primary element (beam, column, wall): | |
| Level of inspection and testing | Percentage of elements to be checked for constructional details | Material samples per floor |
| Limited | 20 | 1 |
| Extended | 50 | 2 |
| Complete | 80 | 3 |

## 3.5 Confidence coefficients

(1) To determine the properties of existing materials for the capacity calculation, when the capacity is to be compared with the demand for safety verification, the average values obtained from *in situ* tests and from additional sources of information should be divided by the confidence coefficient, CF, given in Table 3.1 for the appropriate level of knowledge (see section **2.2.1(5)**).

(2) To determine the properties to be considered for the calculation of the capacity in terms of strength (resistance) of ductile components determining the effects of actions on brittle components/mechanisms, for use in section **4.5.1(1)(b)**, the mean value of the existing material properties, obtained from *in situ* tests and from additional sources of information, must be multiplied by the confidence coefficient, CF, given in Table 3.1 for the appropriate level of knowledge.

# 4 Evaluation

## 4.1 General considerations

(1) Testing is a quantitative procedure to check whether an existing building, damaged or not, can satisfy the appropriate limit state for the seismic action under consideration, as specified in section **2.1**.

(2) This standard is intended for the assessment of individual buildings, in order to decide on the need to intervene on their structure and to design the seismic retrofitting measures that may be necessary. It is not intended to assess the vulnerability of populations or groups of buildings to seismic risk for various purposes (e.g. to determine insurance risk premiums, to set priorities for risk mitigation, etc.).

(3) The assessment procedure should be carried out using the general methods of analysis specified in section **4.3** of Annex 1, and modified in this Annex 3 to adapt to the specific problems encountered in the assessment.

(4) Wherever possible, the method used should incorporate information from previous earthquakes on the performance of buildings of the same or similar type.

## 4.2 Seismic action and combination of seismic loads

(1) The basic models for the definition of seismic motion are those presented in sections **3.2.2** and **3.2.3** of Annex 1.

(2) Reference is made in particular to the elastic response spectrum specified in section **3.2.2.2** of Annex 1, scaled to the calculated values of the terrain acceleration established for the verification of the various limit states. The alternative representations permitted in section **3.2.3** of Annex 1 in terms of artificial or recorded accelerograms may also be applied.

(3) In the $q$-coefficient approach (see section **2.2.1(4)**), the calculation spectrum for the linear analysis is obtained from section **3.2.2.5** of Annex 1. A value of $q$ = 1.5 or 2.0 may be adopted for reinforced concrete and steel structures respectively, regardless of the type of structure. Higher values of $q$ may be adopted if adequately justified with respect to the available local and global ductilities assessed according to the relevant provisions of Annex 1.

(4) The calculated seismic action must be combined with other appropriate variable and permanent actions in accordance with section **3.2.4** of Annex 1.

## 4.3 Structural modelling

(1) A model of the structure must be established based on the information collected according to section **3.2**. The model must allow the effects of actions on all structural elements to be determined for the combination of seismic loads given in section **4.2**.

(2) All provisions of Annex 1 dealing with modelling (section **4.3.1** of Annex 1) and accidental torsional effects (section **4.3.2** of Annex 1) must be applied without modification.

(3) The resistance and rigidity of secondary seismic elements (see section **2.2.1(6)**) to lateral actions can generally be neglected in the analysis.

(4) However, it is advisable to take secondary seismic elements into account in the overall structural model if a non-linear analysis is used. The choice of the elements to be considered as secondary seismic elements may be changed in view of the results of a preliminary analysis. In no case shall the selection of these elements be such as to change the classification of the structure from irregular to regular according to the definitions in section **4.2.3** of Annex 1.

(5) The average values of the material properties must be used in the structural model.

## 4.4 Methods of analysis

### 4.4.1 General considerations

(1) The effects of the seismic actions, to be combined with the effects of the other permanent and variable loads according to the seismic load combination in section **4.2(4)**, can be evaluated using one of the following methods:

- lateral force analysis (linear);
- modal analysis by response spectra (linear) [or spectral modal calculation];
- non-linear static analysis (incremental thrust, pushover) [or incremental thrust method];
- non-linear dynamic analysis in the time domain;
- behavioural coefficient $q$ approach.

(2) Except for the $q$-ratio approach in section **2.2.1(4)** and section **4.2(3)**, the seismic action to be used shall be that corresponding to the elastic response spectrum (i.e. without reduction by the behavioural coefficient $q$) in section **3.2.2.2** of Annex 1 or its equivalent alternative representations in section **3.2.3** of Annex 1.

(3) In the $q$-coefficient approach of section **2.2.1(4)**, the seismic action is defined in section **4.2(3)**.

(4) Section **4.3.3.1(5)** of Annex 1 applies.

(5) The methods of analysis listed above are applicable within the framework of the conditions specified in sections **4.4.2** to **4.4.5**, with the exception of masonry structures for which it is necessary to use procedures which take into account the peculiarities of this type of construction.

NOTE Further information on these procedures can be found in the relevant appendix on materials.

### 4.4.2 Lateral force analysis

(1) The conditions for this method (also called "analysis with a system of equivalent static forces") to be applicable are given in section **4.3.3.2.1** of Annex 1, with the addition of the following:

Designating $\rho_i = D_i/C_i$ to the ratio of the demand $D_i$, obtained from the analysis under the combination of seismic loads, to the corresponding capacity $C_i$ for the $i$-th primary "ductile" element of the structure (bending moment in portal frames or shear walls, axial force in a triangulation (bracing) element of a braced portal frame, etc. ) and designating $\rho_{max.}$ and $\rho_{min.}$ as the maximum and minimum values of $\rho_1$, respectively, on all 'ductile' primary elements of the structure such that $\rho_i > 1$, the ratio $\rho_{max.}/\rho_{min.}$ does not exceed a maximum acceptable value between 2 and 3. In the vicinity of beam-pier connections, it is necessary to evaluate the ratio $\rho_i$ only in sections where plastic hinges are expected to form on the basis of the comparison of the sum of the bending capacities of the beams with that of the piers. Section **4.3(5)** applies to the calculation of the capacities $C_i$. For the determination of the capacities $C_i$ with respect to the bending moment of the vertical members, the value of the axial force can be taken equal to that due to the vertical loads only.

NOTE 1 The value assigned to this limit of $\rho_{max.}/\rho_{min.}$ is 2.5.

NOTE 2 As an additional condition, the capacity $C_i$ of 'brittle' members or mechanisms shall be greater than the corresponding demand $D_i$, evaluated according to section **4.5.1(1)**, **(2)** and **(3)**. However, imposing this as a criterion for the applicability of linear analysis is redundant since, according to sections **2.2.2(2)**, **2.2.3(2)** and **2.2.4(2)**, this condition will ultimately be met for all elements of the structure assessed or subjected to seismic retrofit, irrespective of the method of analysis.

(2) The method is to be applied as described in sections **4.3.3.2.2**, **4.3.3.2.3** and **4.3.3.2.4** of Annex 1, except that the ordinate of the response spectrum in expression (4.5) must be that of the elastic spectrum $S_e(T_1)$ rather than that of the design spectrum $S_d(T_1)$.

### 4.4.3 Response spectrum analysis

(1) The conditions for the applicability of this method are given in section **4.3.3.3.1** of Annex 1, to which the conditions specified in section **4.4.2** must be added.

(2) The method must be applied as described in sections **4.3.3.3.2** and **4.3.3.3.3** of Annex 1, using the elastic response spectrum $S_e(T_1)$.

### 4.4.4 Non-linear static analysis

#### 4.4.4.1 General considerations

(1) Non-linear static analysis (by incremental thrust) is a non-linear static analysis under constant gravity loads and monotonically increasing horizontal loads.

(2) Buildings that do not meet the criteria of section **4.3.3.4.2.1(2)** and **(3)** of Annex 1, in terms of their regularity in plan, must be analysed using a spatial model.

(3) For buildings meeting the regularity criteria of section **4.2.3.2** of Annex 1, the analysis can be carried out using two plan models, one for each main horizontal direction of the building.

#### 4.4.4.2 Lateral loads

(1) At least two vertical distributions of lateral loads shall be applied:

- a ‘uniform’ distribution, based on lateral forces that are proportional to the masses regardless of height (uniform response acceleration);

- a 'modal' distribution, proportional to the lateral forces and consistent with the distribution of lateral forces determined in the elastic analysis.

(2) Lateral loads shall be applied at the location of the masses in the model. Accidental eccentricity shall be taken into account.

#### 4.4.4.3 Capacity curve

(1) The relationship between the shear stress at the base and the control displacement (the 'capacity curve') shall be determined in accordance with sections **4.3.3.4.2.3(1)** and **(2)** of Annex 1.

#### 4.4.4.4 Target displacement

(1) The target displacement is defined in section **4.3.3.4.2.6(1)** of Annex 1.

NOTE The target displacement may be determined in accordance with Appendix B of Annex 1.

#### 4.4.4.5 Procedure for the estimation of torsional effects and higher-order modes

(1) The procedure given in section **4.3.3.4.2.7(1)** to **(3)** of Annex 1 applies for the estimation of torsional effects.

(2) For buildings not complying with the criteria in section **4.3.3.2.1(2)(a)** of Annex 1, the contributions to the response of vibration modes of higher order than the fundamental mode shall be taken into account in each principal direction.

NOTE The requirements in **(2)** may be satisfied either by non-linear analysis in the time domain in accordance with section **4.4.5**, or by special versions of the non-linear static analysis procedure which can take into account the effects of higher order modes on the overall response measurements (such as relative displacement between floors) and then be translated into estimates of local deformation demands (such as rotations of the plastic hinges of the elements).

### 4.4.5 Non-linear time domain analysis

(1) The procedure given in section **4.3.3.4.3(1)** to **(3)** of Annex 1 applies.

### 4.4.6 Approach according to the coefficient *q*

(1) In the *q*-coefficient approach, the method shall be applied as described in sections **4.3.3.2** or **4.3.3.3** of Annex 1, as appropriate.

#### 4.4.7 Combination of the components of seismic action

(1) The two horizontal components of the seismic action must be combined in accordance with section **4.3.3.5.1** of Annex 1.

(2) The vertical component of the seismic action is to be taken into account in the cases specified in section **4.3.3.5.2** of Annex 1 and, where appropriate, combined with the horizontal components as specified in the same section.

#### 4.4.8 Additional measures for masonry infilled structures

(1) The provisions of section **4.3.6** of Annex 1 apply where applicable.

#### 4.4.9 Combination coefficients for variable actions

(1) The provisions of section **4.2.4** of Annex 1 apply.

#### 4.4.10 Importance classes and importance coefficients

(1) The provisions of section **4.2.5** of Annex 1 apply.

### 4.5 Safety checks

#### 4.5.1 Linear analysis methods (lateral or modal force analysis by means of response spectra)

(1) The 'brittle' elements/mechanisms must be verified by taking into account the demands calculated from the equilibrium conditions, in terms of the effects of the actions produced by the ductile elements on the brittle element/mechanism. In this calculation, each effect of the action produced by the ductile element on the brittle element/mechanism considered must be taken as equal to:

(a) the value $D$ obtained from the analysis, if the capacity $C$ of the ductile element, evaluated using average values of the material properties, satisfies the relation $\rho = D/C \leq 1$;

(b) the capacity of the ductile element, evaluated using average values of the material properties multiplied by the confidence coefficients as defined in section **3.5**, taking into account the level of knowledge achieved, if $\rho = D/C > 1$, with $D$ y $C$ as defined in (a) above.

(2) In **(1)(b)** above, the capacities of the beam sections around the concrete beam-pier connections shall be calculated from expression (5.8) of Annex 1 and those of the pier sections around these connections from expression (5.9), using, on the right hand side of the expression, the value $\gamma_{Rd} = 1$ and the mean values of the material properties multiplied by the confidence coefficients as defined in section **3.5**.

(3) For the calculation of the force demands on the 'brittle' shear mechanism of the walls by applying **(1)(b)** above, the expression (5.26) of Annex 1 can be applied with $\gamma_{Rd} = 1$ and using as $M_{Rd}$ the capacity in terms of bending moment at the base, evaluated using the mean values of the material properties multiplied by the confidence coefficients as defined in section **3.5**.

(4) In **(1)** to **(3)** above, the bending moment capacities $C_i$ of the vertical elements may be based on the value of the axial force due to vertical loads only.

(5) The capacity value of both ductile and brittle components and mechanisms to be compared with the demand for safety checks shall be in accordance with section **2.2.1(5)**.

NOTE Information for the assessment of the capacity of components and mechanisms can be found in the relevant material appendices A, B and C.

### 4.5.2 Non-linear analysis methods (static or dynamic)

(1) The demands on the 'ductile' and 'brittle' elements must be those obtained from the analysis carried out in accordance with sections **4.4.4** or **4.4.5**, using the mean value of the material properties.

(2) Section **4.5.1(5)** applies.

NOTE Information for the assessment of the capacity of components and mechanisms can be found in the relevant material appendices A, B and C.

### 4.5.3 Approach according to the coefficient *q*

(1) The demand and capacity values for ductile and brittle elements shall be in accordance with section **2.2.1(4)** and section **2.2.3(3)**.

## 4.6 Summary of criteria for safety analysis and safety checks

(1) Table 4.3 summarises:

- The values of material properties to be adopted in the assessment of demand and element capacities for all types of analysis.

- The criteria to be followed for the safety verification of ductile and brittle elements for all types of analysis.

**Table 4.3 - Material property values and criteria for safety analysis and checks**

<table>
<tr><th colspan="2"></th><th colspan="2">Linear Model (LM)</th><th colspan="2">Non-Linear Model</th><th colspan="2">Approach according to the coefficient q</th></tr>
<tr><th colspan="2"></th><th>Demand</th><th>Capacity</th><th>Demand</th><th>Capacity</th><th>Demand</th><th>Capacity</th></tr>
<tr><td rowspan="7">Type of element or mechanism</td><td rowspan="4">Ductile</td><td colspan="2">Acceptability of the linear model (for testing the values of $\rho_i = D_i/C_i$):</td><td rowspan="7">From the analysis. The mean values of the properties in the model are used.</td><td rowspan="4">In terms of resistance. The mean values of the properties divided by the CFs are used.</td><td rowspan="4">From the analysis</td><td rowspan="7">In terms of resistance. The mean values of the properties divided by the CFs and the partial safety coefficients are used.</td></tr>
<tr><td>From the analysis. The mean values of the properties in the model are used.</td><td>In terms of resistance. The mean values of the properties in the model are used.</td></tr>
<tr><td colspan="2">Checks (if LM is accepted)</td></tr>
<tr><td>From the analysis</td><td>In terms of deformation. The mean values of the properties divided by the CFs are used.</td></tr>
<tr><td rowspan="3">Fragile</td><td colspan="2">Checks (if LM is accepted)</td><td rowspan="3">In terms of resistance. The mean values of the properties divided by the CFs are used.</td><td rowspan="3">According to the relevant chapter of Annex 1</td></tr>
<tr><td>If $\rho_i \leq 1$: from analysis</td><td rowspan="2">In terms of resistance. The mean values of the properties divided by the CFs and by the partial safety factor are used</td></tr>
<tr><td>If $\rho_i > 1$: from the equilibrium with the resistance of the ductile elements/mechanisms. The mean values of the properties multiplied by the CFs are used.</td></tr>
</table>

# 5 Decisions concerning intervention in the structure

## 5.1 Criteria for intervention on a structure

### 5.1.1 Introduction

(1) Intervention decisions should be made on the basis of the conclusions of the assessment of the structure and/or the nature and extent of the damage.

NOTE As in the design of new structures, optimal decisions are sought, taking into account social aspects, such as interruption of use or occupation during the intervention.

(2) This Annex describes the technical aspects of the relevant criteria.

### 5.1.2 Technical criteria.

(1) The selection of the type, technique, scope and urgency of the intervention should be based on the structural information collected during the assessment of the building.

(2) The following aspects should be taken into account:

a) All identified major local defects must be adequately remedied.

b) In the case of very irregular buildings (both in terms of rigidity and distribution of reserve resistance (over-resistance)), structural regularity shall be improved as much as possible both in plan and elevation.

c) The required regularity and resistance characteristics can be achieved by modifying the resistance and/or rigidity of an appropriate number of existing elements, or by introducing new structural elements.

d) Where necessary, local ductility must be effectively increased.

e) The increase in resistance after intervention must not reduce the overall available ductility.

f) Specifically for masonry structures: non-ductile lintels shall be replaced, inadequate connections between floors and walls shall be improved and horizontal thrusts acting on walls outside their median plane shall be eliminated.

### 5.1.3 Type of intervention

(1) An intervention can be selected from the following types, given as indicative:

a) Local or global modification of damaged or undamaged elements (repair, reinforcement or complete replacement), taking into account the rigidity, resistance and/or ductility of these elements.

b) Addition of new structural elements (e.g. triangulations or infill walls; reinforced concrete, steel or timber strapping in masonry constructions; etc.).

c) Modification of the structural system (removal of some structural joints; widening of joints, elimination of vulnerable elements; modification to obtain more regular and/or more ductile arrangements)[3)].

d) Addition of a new structural system to resist all or part of the seismic action.

e) Possible transformation of existing non-structural elements into structural elements.

f) Introduction of passive protection devices through dissipative triangulations or base isolation.

[3] )This is, for example, the case where vulnerable piers with respect to shear forces or soft floors (diaphanous floors) are transformed into more ductile arrangements; similarly, when the resistance reserve irregularities in the elevation, or eccentricities in the plan, are reduced by modifying the structural system.

g) Reduction of masses.

h) Restriction or change in use of the building.

i) Partial demolition.

(2) One type of intervention may be chosen, or a combination of two or more. In all cases, the effect of structural modifications on the foundation shall be taken into account.

(3) If isolation of the foundation is adopted, the provisions contained in Chapter **10** of Annex 1 must be followed.

#### 5.1.4 Non-structural elements

(1) Decisions regarding the repair or strengthening of non-structural elements must be taken whenever, in addition to functional requirements, the seismic behaviour of these elements may endanger the lives of the occupants or affect the value of the assets contained in the building.

(2) In such cases, the total or partial collapse of these elements shall be prevented by:

a) appropriate connections to structural members (see section **4.3.5** of Annex 1);

b) increasing the resistance of non-structural members (see section **4.3.5** of Annex 1);

c) the use of means of anchorage to prevent the possible fall or detachment of parts of these elements.

(3) The possible consequences of these provisions on the behaviour of the structural elements must be taken into account.

#### 5.1.5 Justification for the type of intervention selected

(1) In all cases, the documents relating to the seismic retrofit project must include a justification of the type of intervention selected and a description of its expected effect on the response of the structure.

(2) This justification shall be made available to the Owner.

## 6 Intervention project on the structure

### 6.1 Procedure of the seismic retrofit project

(1) The seismic retrofit project procedure should include the following steps:

a) Conceptual design.

b) Analysis.

c) Verifications.

(2) The conceptual design should cover the following:

i) Selection of techniques and/or materials, as well as the type and configuration of the intervention.

ii. Preliminary estimation of the dimensions of additional structural elements.

iii) Preliminary estimation of the modified rigidity of the elements subject to seismic retrofitting.

(3) The structural analysis methods specified in section **4.4** must be used, taking into account the modified characteristics of the building.

(4) The safety checks are to be carried out in general in accordance with section **4.5** for existing, modified and new structural elements. For existing materials, the average values from *in situ* tests and additional sources of information, modified by the confidence factor CF as specified in section **3.5**, are to be used for the safety verification. However, for new or added materials the nominal properties unmodified by the confidence factor CF should be used.

NOTE Information on the capacities of existing and new structural members can be found in the relevant appendices A, B or C related to materials.

(5) Where the structural system (including both existing and new structural members) can be made to comply with the requirements of Annex 1, the verifications may be carried out in accordance with the provisions of that Annex.

# Appendix A

# Recommendations for reinforced concrete structures

## A.1 Purpose and scope

(1) This appendix contains specific information for the seismic resistance assessment of reinforced concrete buildings in their existing state, and for their seismic retrofitting, where necessary.

## A.2 Identification of geometry, construction details and materials

### A.2.1 General considerations

(1) The following aspects should be carefully examined:

i. The physical condition of the reinforced concrete elements and the presence of any degradation due to carbonation, corrosion of steel, etc.

ii. The continuity of the load paths between the lateral load resisting elements.

### A.2.2 Geometry

(1) The data collected should include the following elements:

i. Identification of the resistance systems to lateral loads in both directions.

ii. Orientation of the unidirectional floors.

iii. Beam, column and wall depths and widths.

iv. Width of the flanges of T-beams.

v. Possible eccentricities between the axes of beams and piers at the connections.

### A.2.3 Construction details

(1) The data collected should include the following elements:

i. Amount of longitudinal reinforcement in beams, piers and walls.

ii. Amount and detailing of confinement reinforcement in critical areas and at beam-column connections.

iii. Amount of reinforcement in floor slabs that increase the negative bending resistance of T-beams.

iv. Seating lengths and support conditions of horizontal elements.

v. Thickness of concrete cover.

vi. Overlapping splices of longitudinal reinforcement.

### A.2.4 Materials

(1) The data collected should include the following elements:

i. Concrete resistance.

ii. Elastic limit, ultimate resistance and ultimate deformation of steel.

## A.3 Capacity models for assessment

### A.3.1 Introduction

(1) The provisions of this section apply to both primary and secondary seismic elements.

(2) Classification of components/mechanisms:

i. 'ductile': beams, columns and walls subjected to simple or composite bending;

ii. 'brittle': shear mechanisms of beams, columns, walls and connections.

### A.3.2 Beams, columns and walls subjected to simple or composite bending

#### A.3.2.1 Introduction

(1) The deflection capacity of beams, columns and walls, which is verified in accordance with sections **2.2.2(2)**, **2.2.3(2)** and **2.2.4(2)**, is defined in terms of chord rotation $\theta$, i.e. the angle between the tangent to the axis at the plasticised end and the chord connecting that end to the shear span ($L_v = M/V$ = moment/shear stress at the end section), i.e. with the inflection point of the deflection. The chord rotation is also equal to the ratio of the relative displacement of the element, i.e. the deflection at the end of the shear span with respect to the tangent to the axis at the plasticised end, divided by the shear span.

#### A.3.2.2 Near Collapse (NC) limit state

(1) The value of the total capacity (the elastic plus the inelastic part) of the chord rotation in exhaustion, $\theta_u$, of the concrete elements under the action of cyclic loads can be calculated from the following expression:

$$\theta_{um} = \frac{1}{\gamma_{el}} 0{,}016 \cdot (0{,}3^{\nu}) \left[ \frac{\text{máx.}(0{,}01; \omega')}{\text{máx.}(0{,}01; \omega)} f_c \right]^{0{,}225} \left( \text{mín.}\left( 9; \frac{L_V}{h} \right) \right)^{0{,}35} 25^{\left( \alpha \rho_{sx} \frac{f_{yw}}{f_c} \right)} (1{,}25^{100 \rho_d}) \qquad \text{(A.1)}$$

where

$\gamma_{e1}$ is equal to 1.5 for primary seismic elements and 1.0 for secondary seismic elements (as defined in section **2.2.1(6)**);

$h$ is the height of the transverse section;

$L_v$ $= M/V$ is the shear moment/stress ratio at the extreme section;

$\nu$ $= N/bhf_c$ ($b$ width of the compressed block, $N$ positive axial force in compression);

$\omega, \omega'$ is the mechanical amount of longitudinal tensile (including web reinforcement) and compressive reinforcement, respectively;

$f_c$ y $f_{yw}$ are the concrete compressive strength (in MPa) and the yield strength of the stirrups (in MPa), respectively, obtained directly as average values from *in situ* tests, and from additional sources of information, divided by the appropriate confidence factors as defined in section **3.5(1)** and Table 3.1, taking into account the level of knowledge attained;

$\rho_{sx} = A_{sx}/b_w s_h =$ is the amount of transverse reinforcement parallel to the $x$ direction of loading ($s_h$ = spacing between abutments);

$\rho_d$ is the amount of diagonal reinforcement (if any) in each diagonal direction;

$\alpha$ is the confinement effectiveness coefficient, which can be taken equal to:

$$\alpha = \left( 1 - \frac{s_h}{2b_o} \right) \left( 1 - \frac{s_h}{2h_o} \right) \left( 1 - \frac{\sum b_i^2}{6h_o b_o} \right) \qquad \text{(A.2)}$$

where

$b_o$ and $h_o$ are the dimensions of the confined concrete core;

$b_i$ is the distance between the axes of the longitudinal reinforcement (with index $i$) laterally restrained effectively at the perimeter of the transverse section (by means of a stirrup corner or by means of a transverse tie).

For walls, the value given by expression (A.1) is multiplied by 0.58.

If cold drawn reduced ductility steel is used, the total chord rotation capacity quoted above is divided by 1.6.

(2) The value of the plastic part of the chord rotation capacity of concrete elements under cyclic loading can be calculated from the following expression:

$$\theta_{um}^{pl} = \theta_{um} - \theta_y = \frac{1}{\gamma_{el}} 0{,}0145 \cdot (0{,}25^{\nu}) \left[ \frac{\text{máx.}(0{,}01;\omega')}{\text{máx.}(0{,}01;\omega)} \right]^{0{,}3} \cdot f_c^{\,0{,}2} \cdot \left( \text{mín.}\left( 9; \frac{L_V}{h} \right) \right)^{0{,}35} 25^{\left( \alpha \rho_{sx} \frac{f_{yw}}{f_c} \right)} (1{,}275^{100 \rho_d}) \tag{A.3}$$

where the chord rotation at the yield point, $\theta_y$, should be calculated according to section **A.3.2.4**, $\gamma_{el}$ is equal to 1.8 for primary seismic elements and 1.0 for secondary seismic elements, and the other variables are defined as in expression (A.1).

For walls, the value given by expression (A.3) is multiplied by 0.6.

If cold-formed steel of reduced ductility is used, the plastic part of the chord rotation capacity is divided by 2.

(3) For elements without earthquake resistant construction details, the values given by expressions (A.1) and (A.3) are divided by 1.2.

(4) **(1)** and **(2)** apply to elements with corrugated longitudinal reinforcement (high bond) without overlapping in the vicinity of the end zone where plastification is expected. If the corrugated longitudinal reinforcement has straight overlapping ends starting at the end section of the member - as is often the case for columns and walls with overlapping joints starting at the slab level - expressions (A.1) and (A.3) should be applied with the value of the amount of compression reinforcement, $\omega'$, twice the value applied outside the overlapping joint. Furthermore, if the overlap length $l_o$ is less than $l_{ou,min}$, the value of the plastic part of the chord rotation capacity given in **(2)** should be multiplied by $l_o/l_{ou,min}$, while the value of the chord rotation at the yield point, $\theta_y$, added to the above to obtain the total chord rotation capacity, should take into account the effect of the overlap in accordance with section **A.3.2.4(3)**. The value of $l_{ou,min}$ is $l_{ou,min} = d_{bL} f_{yL}/[(1.05 + 14.5\, \alpha_1 \rho_{sx} f_{yw}/f_c)\sqrt{f_c}]$,

where

$d_{bL}$ is the diameter of the overlapping reinforcement;

$f_{yL}$ is the average value of the yield strength of the overlapping reinforcement (in MPa) from *in situ* tests and from additional sources of information, multiplied by the corresponding confidence factor as defined in section **3.5** and Table 3.1, taking into account the level of knowledge attained (see section **3.5(2)**);

$f_c$, $f_{yw}$ y $\rho_{sx}$ as defined in **(1)**, and

$\alpha_1 = (1 - s_h/(2b_o))(1 - s_h/(2h_o))\, n_{restr}/n_{tot}$, with

- $n_{restr}$: number of overlapping longitudinal reinforcements laterally restrained effectively by means of an abutment corner or by a transverse tie; and

- $n_{tot}$: total number of overlapping longitudinal reinforcement around the perimeter of the transverse section.

(5) For elements with smooth longitudinal reinforcement without overlap in the vicinity of the

extreme zone where plastification is expected, the total chord rotation capacity can be taken equal to the value calculated according to **(1)** multiplied by 0.8, while the plastic part of the chord rotation capacity can be taken equal to the value calculated according to **(2)** multiplied by 0.75 (these factors include the reduction coefficient 1.2 of **(3)** to take into account the lack of constructional details for earthquake resistance). If the longitudinal reinforcement is overlapped starting at the end section of the member and standard hooks and an overlap length $l_o$ of at least 15 $d_{bL}$ are placed at the ends of the member, the chord rotation capacity of the member can be calculated as follows:

- In expressions (A.1) and (A.3) the shear span $L_V$ (ratio $M/V$ - moment/shear - at the end section) is reduced by the overlap length $l_o$, when the ultimate condition is controlled by the area just after the end of the overlap.

- The total chord rotation capacity can be taken equal to the value calculated according to **(1)** and **(3)** multiplied by 0.019 (10 + min(40, $l_o/d_{bL}$)), while the plastic part of the chord rotation capacity can be taken equal to that calculated according to **(2)** and **(3)** multiplied by 0.019 min(40, $l_o/d_{bL}$).

(6) An alternative expression can be used for the assessment of the ultimate chord rotation capacity:

$$\theta_{um} = \frac{1}{\gamma_{el}}\left(\theta_y + (\varphi_u - \varphi_y)L_{pl}\left(1 - \frac{0,5L_{pl}}{L_V}\right)\right) \quad \text{(A.4)}$$

where

$\theta_y$ is the chord rotation in the elastic limit defined according to expressions (A.10) and (A.11);

$\varphi_u$ is the ultimate curvature at the extreme section;

$\varphi_y$ is the yield stress curvature at the extreme section.

The value of the length $L_{p1}$ of the plastic hinge depends on how the improvement of the resistance and deformation capacity of the concrete due to confinement is taken into account in the calculation of the ultimate curvature of the end section $\phi_u$.

(7) If the ultimate curvature of the end section $\phi_u$, under cyclic loading, is calculated with:

(a) the ultimate unit deflection of the longitudinal reinforcement, $\varepsilon_{su}$, taken as:

- 5 % for S-type steel (in accordance with the value given in Article 34 of the Structural Code for the total elongation under maximum load, $\varepsilon_{max}$),

- 6 % for steel type SD, and

(b) the confinement model defined in section **3.1.9** of Annex 19 of the Structural Code, with effective lateral confining stress $\sigma_2$ taken equal to $\alpha\rho_{sx}f_{yw}$, where $\rho_{sx,}f_{yw}$ and $\alpha$ and α have been defined in **(1)**,

then, for elements with earthquake-resisting construction details and no overlap of longitudinal reinforcement in the vicinity of the section where plastic deformation is expected, $L_{p1}$ can be calculated from the following expression:

$$L_{pl} = 0,1L_V + 0,17h + 0,24\frac{d_{bL}f_y(\text{MPa})}{\sqrt{f_c(\text{MPa})}} \tag{A.5}$$

where $h$ is the height of the member and $d_{bL}$ is the (mean) diameter of the tensile reinforcement.

(8) If the ultimate curvature of the end section, $\phi_u$, under cyclic loading is calculated with:

(a) the ultimate unit strain of the longitudinal reinforcement, $\varepsilon_{su}$, taken as in **(7)(a)**; and

(b) a confinement model that represents, better than the model in section **3.1.9** of Annex 19 of the Structural Code, the enhancement of $\phi_u$, due to confinement under cyclic loading, namely a model in which:

- the resistance of the confined concrete is evaluated from:

$$f_{cc} = f_c\left[1 + 3,7\left(\frac{\alpha\rho_{sx}f_{yw}}{f_c}\right)^{0,86}\right] \tag{A.6}$$

- the unit strain corresponding to the resistance $f_{cc}$ is taken to increase the $\varepsilon_{c2}$ value of the unconfined concrete strain as: (A.7):

$$\varepsilon_{cc} = \varepsilon_{c2}\left[1 + 5\left(\frac{f_{cc}}{f_c} - 1\right)\right] \tag{A.7}$$

- and the ultimate strain of the extreme fibre in the compression zone is taken to be equal to: (A.8):

$$\varepsilon_{cu} = 0,004 + 0,5\frac{a\rho_{sx}f_{yw}}{f_{cc}} \tag{A.8}$$

Where

$\alpha$, $f_{yw}$ and $\rho_{sx}$ are defined in **(1)** and **(7)**, and $f_{cc}$ is the resistance of the confined concrete,

then for elements with earthquake-resistant construction details and without overlap of longitudinal reinforcement near the section where plastification is expected, $L_{p1}$ can be calculated from the following expression:

$$L_{pl} = \frac{L_V}{30} + 0,2h + 0,11\frac{d_{bL}f_y(\text{MPa})}{\sqrt{f_c(\text{MPa})}} \tag{A.9}$$

(9) If the confinement model of section **3.1.9** of Annex 19 of the Structural Code is adopted in the calculation of the ultimate curvature of the extreme section, $\phi_u$, and the value of $L_{pl}$ of expression (A.5)

is used in expression (A.4), then the coefficient $\gamma_{el}$ defined therein can be taken equal to 2 for primary seismic elements and 1.0 for secondary seismic elements. If, instead, the confinement model given by expressions (A.6) to (A.8) is used together with expression (A.9), then the value of the coefficient $\gamma_{el}$ can be taken equal to 1.7 for primary seismic elements and 1.0 for secondary seismic elements.

NOTE The values of the total chord rotation capacity, calculated according to **(1)** and **(2)** above (taking into account **(3)** to **(5)**) are usually very similar. Expression (A.1) is more convenient when calculations and demands are based on total rope rotations, while expression (A.3) is more appropriate for those cases where calculations and demands are based on the plastic part of the rope rotations; furthermore, **(4)** gives the chord rotation capacity of elements with corrugated longitudinal reinforcement and overlapping straight ends starting at the end section only in terms of expression (A.3). Expression (A.4) with $\gamma_{el}$ = 1 gives values quite similar to those obtained when used with point **(7)** or **(8)**, but the differences with respect to the predictions of points **(1)** or **(2)** are larger. The dispersion of the test results with respect to those of the expression (A.4) for $\gamma_{el}$ = 1 used with point **(8)** is less important than when used with point **(7)**. This is reflected in the different values of $\gamma_{e1}$ specified in **(1)**, **(2)** and **(9)**, for the primary seismic elements, since $\gamma_{el}$ is intended to convert mean values to average values minus one standard deviation. Finally, in **(3)** to **(5)** the effects of the lack of seismic-resisting construction details and of the overlap joint in the plastic hinge zone are specified only in connection with expressions (A.1) and (A.3).

(10) Existing walls complying with the definition of 'lightly reinforced large walls' in Annex 1 may be verified in accordance with Annex 19 of the Structural Code.

### A.3.2.3 Significant Damage (SD) limit state

(1) The chord rotation capacity corresponding to significant damage $\theta_{SD}$ can be assumed to be equal to ¾ of the ultimate chord rotation $\theta_u$ given in section **A.3.2.2**.

### A.3.2.4 Damage Limitation (DL) limit state

(1) The capacity for this limit state used in the verifications is the bending moment in plastification with the calculated value of the axial load.

(2) In the case where the verification is done in terms of deformations, the corresponding capacity is given by the chord rotation at the yield point $\theta_y$, evaluated as follows:

For beams and columns:

$$\theta_y = \phi_y \frac{L_V + a_V z}{3} + 0{,}0014\left(1 + 1{,}5\frac{h}{L_V}\right) + \frac{\varepsilon_y}{d - d'}\frac{d_{bL} f_y}{6\sqrt{f_c}} \qquad \text{(A.10a)}$$

For rectangular section walls, T-section walls or walls with contour elements slightly thicker than the web:

$$\theta_y = \phi_y \frac{L_V + a_V z}{3} + 0{,}0013 + \frac{\varepsilon_y}{d - d'}\frac{d_{bL} f_y}{6\sqrt{f_c}} \qquad \text{(A.11a)}$$

or the alternative (and equivalent) expressions for beams and columns:

$$\theta_y = \phi_y \frac{L_V + a_V z}{3} + 0{,}0014\left(1 + 1{,}5\frac{h}{L_V}\right) + \phi_y \frac{d_{bL} f_y}{8\sqrt{f_c}} \qquad \text{(A.10b)}$$

and for rectangular section walls, T-section walls or walls with contour elements slightly thicker than the thickness of the web:

$$\theta_y = \phi_y \frac{L_V + a_V z}{3} + 0{,}0013 + \phi_y \frac{d_{bL} f_y}{8\sqrt{f_c}} \quad \text{(A.11b)}$$

Where

$\phi_y$ yield stress curvature of the end section,

$a_V z$ displacement of the bending moment diagram (see section **9.2.1.3(2)** of Annex 19 to the Structural Code) with

$z$ internal mechanical member length, taken equal to $d - d'$ for beams, piers, T-section walls and walls with contour elements slightly thicker than the web, or 0.8 $h$ for walls with rectangular cross-section;

$a_V = 1$ if shear cracking is expected to precede bending plastification at the end section (i.e. when the bending moment at plastification at the end section, $M_y$, exceeds the product of $L_v$ times the shear resistance of the element considered without shear reinforcement, $V_{R,c}$, taken in accordance with section **6.2.2(1)** of Annex 19 of the Structural Code); otherwise (i.e. if $M_y < L_V V_{R,c}$) $a_V = 0$;

$f_y$ and $f_c$ yield strength of steel and resistance of concrete, respectively, as defined in expression (A.1), both in MPa;

$\varepsilon_y$ equal to $f_y/E_s$;

$d$ and $d'$ depths of tensile and compressive reinforcement respectively; and

$d_{bL}$ diameter (mean) of the tensile reinforcement.

The first term in expressions (A.10) and (A.11) takes into account the contribution of bending, the second term represents the contribution of shear deformation and the third term represents the slippage of the reinforcement anchorage.

NOTE The two alternative sets of expressions: (A.10a) and (A.11a) on the one hand, and (A.10b) and (A.11b) on the other hand, are practically equivalent. Expressions (A.10a) and (A.11a) are more rational, but expressions (A.10b) and (A.11b) are more practical and their use may in general be more convenient, as the calculation of $\phi_y$ can be difficult and error-prone.

(3) **(1)** and **(2)** apply to elements with non-overlapping longitudinal reinforcement in the vicinity of the extreme region where plastic deformation is expected. If the longitudinal reinforcement is corrugated with overlapping straight ends starting at the extreme section of the member (as in the case of columns and walls with lap joints starting at the slab level), the bending moment at plastification $M_y$ and the curvature at the yield point $\phi_y$ in expressions (A.10) and (A.11) should be calculated with an amount of compressive reinforcement equal to twice that used outside the lap joint. If the length of the straight overlap $l_o$ is less than $l_{oy,min} = 0.3\ d_{bL} f_{yL}/\sqrt{f_c}$, where $d_{bL}$ is the diameter of the overlapping reinforcement, $f_{yL}$ (in MPa) is the mean value of the yield strength of the steel of the overlapping reinforcement obtained from *in situ* tests and from additional sources of information, multiplied by the confidence factor as defined in section **3.5** and in Table 3.1, taking into account the level of knowledge achieved (see section **3.5(2)**) and $f_c$ (in MPa) is as defined in the expression (A.1):

- $M_y$ and $\phi_y$ should be calculated with the yield stress, $f_y$, multiplied by $l_o/l_{oy,\,min.}$;

- the unit strain at the yield stress, $\varepsilon_y$, in the last term of expressions (A.10a) and (A.11a) should be multiplied by $l_o/l_{oy,\,min.}$;

- the second term in expressions (A.10) and (A.11) should be multiplied by the ratio of the value of the bending moment in plastification $M_y$, modified to take account of the lap joint, to the bending moment in plastification outside the lap joint;

- in order to determine whether or not the term $\alpha_V\, z$ contributes to the first term in expressions (A.10) and (A.11) with $\alpha_V = 1$, the product $L_V V_{R,c}$ is compared with the bending moment in plastification $M_y$ modified to take into account the effect of the overlap.

(4) **(1)** and **(2)** can also be considered to apply to elements with plain longitudinal reinforcement, even if their ends, terminated by standard hooks, are overlapped at the end section of the element (such as in columns and walls with a lap joint starting at floor level), provided that the lap length $l_o$ is at least equal to $15d_{bL}$.

(5) If the verification is in terms of deflections, the deflection demands should be obtained from an analysis of the structural model, in which the rigidity of each element is taken to be equal to the mean value of $M_y L_y/3\theta_y$ at the two ends of the element. In this calculation, the shear span at the end section, $L_V$, can be taken equal to half the length of the element.

### A.3.3 Beams, columns and walls: shear forces

#### A.3.3.1 Near Collapse (NC) limit state

(1) The cyclic shear resistance, $V_R$, decreases with the plastic part of the ductility demand, expressed in terms of the ductility coefficient of the transverse deflection of the shear span or chord rotation at the end of the element: $\mu_\Delta^{pl} = \mu_\Delta$-1. For this purpose, $\mu_\Delta^{pl}$ can be calculated as the ratio of the plastic part of the chord rotation, $\theta$, to the chord rotation at the yield point, $\theta_y$, calculated according to section **A.3.2.4(2)** to **(4)**.

The following expression can be used for the shear resistance, controlled by the abutments, taking into account the reduction quoted above (units: MN and metres):

$$V_R = \frac{1}{\gamma_{el}}\left[\frac{h-x}{2L_V}\,\text{mín.}\left(N; 0,55A_c f_c\right) + \left(1 - 0,05\,\text{mín.}\left(5; \mu_\Delta^{pl}\right)\right)\cdot\right.$$
$$\left.\cdot\left[0,16\,\text{máx.}(0,5; 100\rho_{tot})\left(1 - 0,16\,\text{mín.}\left(5; \frac{L_V}{h}\right)\right)\sqrt{f_c}A_c + V_w\right]\right] \qquad \text{(A.12)}$$

where

$\gamma_{el}$ is equal to 1.5 for primary seismic elements and 1.0 for secondary seismic elements (as defined in section **2.2.1(6)**);

$h$ is the edge of the transverse section (equal to diameter $D$ in circular sections);

$x$ is the depth of the compressed block;

$N$ is the axial compressive force (positive if compressive and zero if tensile);

$L_V$ $= M/V$ is the ratio between the bending moment and the shear force at the end section;

$A_c$ is the transverse area of the transverse section, taken equal to $b_w d$ for a cross-section with a rectangular web of width $b_w$ and usable depth $d$, or equal to $\pi D_c^2/4$ (where $D_c = D - 2c - 2d_{bw}$ is the diameter of the concrete core within the confining reinforcement, with $D$ and $c$ defined in b) below and $d_{bw}$ is the diameter of the transverse reinforcement) for circular sections;

$f_c$ is the concrete compressive resistance, defined by expression (A.1); for primary seismic elements $f_c$ should be further divided by the partial concrete coefficient according to section **5.2.4** of Annex 1;

$\rho_{tot}$ is the total amount of longitudinal reinforcement;

$V_w$ is the contribution of the transverse reinforcement to the shear resistance, taken equal to:

a) For rectangular web transverse sections of width (thickness) $b_w$:

$$V_w = \rho_w b_w z f_{yw} \tag{A.13}$$

where

$\rho_w$ is the amount of transverse reinforcement;

$z$ is the length of the internal mechanical link, as specified in section **A.3.2.4(2)**; and

$f_{yw}$ is the yield strength of the transverse reinforcement as defined by expression (A.1); for primary seismic elements, $f_{yw}$ should further be divided by the partial coefficient for steel in accordance wsith section **5.2.4** of Annex 1;

b) for circular transverse sections:

$$V_w = \frac{\pi}{2}\frac{A_{sw}}{s} f_{yw} (D - 2c) \tag{A.14}$$

where

$D$ is the diameter of the section;

$A_{sw}$ is the transverse area of a circular abutment;

$s$ is the spacing between the axes of the abutments;

$f_{yw}$ is as defined in (a) above; and

$c$ is the concrete cover.

(2) The shear resistance of a concrete wall, $V_R$, cannot be taken to be greater than the value corresponding to the exhaustion of the compressive strength of the web, $V_{R,max.}$, which under cyclic loading can be calculated from the following expression (units: MN and metres):

$$V_{R,máx.} = \frac{0{,}85\left(1-0{,}06\,\text{mín.}\left(5;\ \mu_{\Delta}^{pl}\right)\right)}{\gamma_{el}}\left(1+1{,}8\,\text{mín.}\left(0{,}15;\ \frac{N}{A_c f_c}\right)\right)\left(1+0{,}25\,\text{máx.}\left(1{,}75;\ 100\rho_{tot}\right)\right)\left(1-0{,}2\,\text{mín.}\left(2;\ \frac{L_V}{h}\right)\right)\sqrt{f_c}\, b_w\, z \quad \text{(A.15)}$$

where $\gamma_{e1}$ = 1.15 for primary seismic elements and 1.0 for secondary seismic elements; $f_c$ is in MPa, $b_w$ and $z$ are in metres and $V_{R,max.}$ is in MN and the rest of the variables are defined in **(1)**.

The shear resistance under cyclic loading controlled by the depletion of the web compressive strength prior to bending plastification is obtained from expression (A.15) for $\mu_{\Delta}^{pl} = 0$.

(3) If in a concrete column the shear slenderness $L_V/h$, at the end section with the maximum value of the two moments at the ends, is less than or equal to 2.0, its shear resistance, $V_R$, should not be taken to be greater than the value corresponding to the exhaustion of the compressive strength of the web along the diagonal of the column after bending plastification, $V_{R,max.}$, which, under cyclic loading, can be calculated from the following expression (units: MN and metres):

$$V_{R,máx.} = \frac{\frac{4}{7}\left(1-0{,}02\ \text{mín.}\left(5;\ \mu_{\Delta}^{pl}\right)\right)}{\gamma_{el}}\left(1+1{,}35\frac{N}{A_c f_c}\right)\left(1+0{,}45\left(100\rho_{tot}\right)\right)\sqrt{\text{mín.}\left(40;\ f_c\right)}\, b_w\, z \sin 2\delta \quad \text{(A.16)}$$

where

$\delta$ is the angle between the diagonal and the axis of the column ($\tan\delta = h/2L_V$);

and all other variables are defined in **(3)**.

(4) The minimum value of the shear resistance, calculated in accordance with Annex 19 of the Structural Code or by expressions (A.12) to (A.16), should be used in the assessment.

(5) Average material properties determined from *in situ* tests and from additional sources of information should be used in the calculations.

(6) For primary seismic elements, the average material resistances, in addition to being divided by the appropriate confidence factors based on the level of knowledge, should also be divided by the partial material coefficients in accordance with section **5.2.4** of Annex 1.

#### A.3.3.2 Significant Damage (SD) and Damage Limitation (DL) limit states

(1) Testing for the exceedance of these two limit states is not required, unless these two limit states are the only ones to be tested. In this case, section **A.3.3.1** applies.

### A.3.4 Beam-column connections

#### A.3.4.1 Near Collapse (NC) limit state

(1) The shear demand at the connections is assessed in accordance with section **5.5.2.3** of Annex 1.

(2) The shear capacity of the joints is assessed in accordance with section **5.5.3.3** of Annex 1.

(3) Section **A.3.3.1(5)** and **(6)** applies to connections of primary seismic elements with other elements.

#### A.3.4.2 Significant Damage (SD) and Damage Limitation (DL) limit states

(1) Verification of the exceedance of these two limit states is not necessary, unless these two limit states are the only ones to be checked. In this case, section **A.3.4.1** applies.

## A.4 Capacity models for reinforcement

### A.4.1 General considerations

(1) The rules for element resistance and deformation capacities given in the following sections for reinforced elements refer to the capacities in the near collapse limit state of sections **A.3.3.2** and **A.3.3.1** before the application of the overall $\gamma_{el}$ coefficient. The $\gamma_{el}$ coefficients specified in sections **A.3.3.2** and **A.3.3.1** should be applied on the resistance and deformation capacities of the element subject to seismic retrofitting, determined in accordance with the following sections.

(2) The partial coefficients for new steel and concrete used for seismic retrofitting are given in section **5.2.4** of Annex 1 and for new structural steel used for seismic retrofitting are given in section **6.1.3(1)** of Annex 1.

### A.4.2 Concrete liner

#### A.4.2.1 Introduction

(1) Concrete liners are applied to columns and walls to achieve one of the following objectives:

- increase the bearing capacity;
- increase the bending and/or shear resistance;
- increase the deformation capacity;
- improve the resistance of poor lap joints.

(2) The thickness of the liners should allow for the placement of both longitudinal and transverse reinforcement with adequate cover.

(3) Where liners are intended to increase the bending resistance, the longitudinal reinforcement should continue into the adjacent storey through holes through the slab, while through horizontal holes in the beams, horizontal braces should be placed in the node. These braces may be omitted in the case of fully confined interior nodes.

(4) When only increases in shear resistance and deformation capacity are considered, together with a possible improvement of the overlapping connection, the liners should be finished (both concrete and reinforcement) with a gap of about 10 mm in relation to the floor slab.

### A.4.2.2 Improved resistance, rigidity and deformation capacity

(1) In assessing the resistance and deformation capacities of lined elements, the following simplifying assumptions can be made:

- the lined element behaves monolithically, with complete mixed action between old and new concrete;

- the fact that the axial load is originally applied only on the old column is not taken into account and the full axial load is assumed to act on the lined element;

- it is assumed that the properties of the concrete in the liner are applied to the entire cross-section of the element.

(2) It can be assumed that the following relationships hold between the values of $V_R$, $M_y$, $\theta_y$ and $\theta_u$ calculated under the above considerations and the values $V_R^*$, $M_y^*$, $\theta_y^*$ and $\theta_u^*$ to be adopted in the capacity verifications:

- For $V_R^*$:

$$V_R^* = 0,9 V_R \tag{A.17}$$

- For $M_y$*:

$$M_y^* = M_y \tag{A.18}$$

- For $\theta_y$*:

$$\theta_y^* = 1,05\,\theta_y \tag{A.19a}$$

- For $\theta_u$*:

$$\theta_u^* = \theta_u \tag{A.20}$$

(3) The values of $M_y$*, $\theta_y$* and $\theta_u$* of the lined element, to be compared with the demand in the safety verifications, should be calculated on the basis of: (a) the average value of the resistance of the existing steel obtained directly from *in situ* tests and from additional sources of information, divided by the appropriate confidence coefficient from section **3.5**, taking into account the level of knowledge achieved; and (b) the nominal resistance of the added concrete and added reinforcement.

(4) The value of $V_R$* of the lined element, to be compared with the demands in safety checks, should be calculated as a function of: (a) the average value of the resistance of the existing steel obtained directly from *in situ* tests and from additional sources of information, divided by the appropriate confidence coefficient of section **3.5**, taking into account the level of knowledge achieved; and (b) the nominal resistance of the added concrete and added reinforcement. In primary seismic elements, the average value of the resistance of the existing steel and the nominal resistance of the added materials should be divided by the partial coefficients of the steel and concrete in accordance with section **5.2.4** of Annex 1.

(5) The value of $M_y$* for lined elements where the effects of actions are exerted on brittle components/mechanisms, for use in section **4.5.1(1)(b)**, should be calculated as a function of: (a) the average value of the resistance of the existing steel obtained directly from *in situ* tests and from additional sources of information, multiplied by the appropriate confidence coefficient from section **3.5**, according to the level of knowledge attained; and (b) the nominal resistance of the added concrete and added reinforcement (see section **3.5(2)**).

## A.4.3 Steel liner

### A.4.3.1 Introduction

(1) Steel liners are mainly applied to columns in order to: increase the shear resistance and improve the strength of poor lap joints. They can also be considered to increase ductility due to the confinement they provide.

(2) Steel liners around rectangular columns usually consist of four angles to which continuous steel plates or thicker horizontal steel plates (clips) are welded. The angles can either be bonded to the concrete using epoxy resins or simply laid in contact with the column along the entire height without gaps. The clips can be pre-heated before welding to provide some positive confinement to the column.

#### A.4.3.2 Shear resistance

(1) It can be assumed that the liner's contribution to the shear resistance is in addition to the existing resistance, provided that the steel liner remains within the elastic range. This condition is necessary to allow the liner to control the opening of internal cracks and to ensure the integrity of the concrete, thus allowing the original shear resistance mechanism to continue to function.

(2) If only 50 % of the yield strength of the liner steel is used, the expression for the additional shear stress $V_j$ supported by the liner is:

$$V_j = 0,5\,h\,\frac{2t_j b}{s} f_{yj,d} \cdot (\cot\theta + \cot\beta) \cdot \mathrm{sen}\beta \qquad \text{(A.21)}$$

where

$h$ is the edge of the transverse section;

$t_j$ is the thickness of the steel clamps or, if applicable, of the continuous steel plates;

$b$ is the width of the clamps;

$s$ is the spacing between clamps ($b/s$ = 1, in case of continuous steel plates);

$\theta$ is the angle of inclination of the connecting rods;

$\beta$ is the angle between the axes of the clamps and the axis of the member ($\beta$ = 90° in case of continuous steel plates); and

$f_{yj,d}$ is the calculated value of the yield strength of the liner steel, equal to its nominal resistance divided by the partial coefficient of the structural steel in accordance with section **6.1.3(1)** of Annex 1.

#### A.4.3.3 Fixing of overlapping joints

(1) Steel liners can provide effective fixation in overlap joint areas to improve cyclic deformation capacity. To achieve this result it is necessary

- the length of the liner is at least 50% longer than the length of the joint area;

- that the liner is pressed against the column faces by at least two rows of bolts on each side perpendicular to the direction of loading;

- that where the joint takes place at the base of the pier, one row of bolts should be located at the top of the joint area and another at 1/3 of that area, starting at the base.

### A.4.4 Fibre-reinforced polymer (FRP) cladding and wrapping

#### A.4.4.1 Introduction

(1) The main uses of externally bonded FRP (fibre-reinforced polymers) used for seismic retrofitting of existing reinforced concrete elements are as follows:

- Improving the shear resistance capacity of columns and walls by applying externally bonded FRP, with the fibres in the direction of the confining reinforcement.

- Improvement of the available ductility at the ends of the elements by additional confinement in the form of FRP liners, with the fibres oriented along the perimeter.

- Prevention of overlap joint failure by improving the confinement of the overlap, with the fibres again along the perimeter.

(2) The effect of jacketed and wrapped elements on the bending resistance at the end section and on the value of chord rotation at the yield point, $\theta_y$, can be neglected ($\theta_y$ can be calculated according to section **A.3.2.4(2 to 4)**, taking $l_{oy,min.}$ equal to $0.2 d_{bL} f_{yL}/\sqrt{f_c}$, in section **A.3.2.4(4)**).

#### A.4.4.2 Shear resistance

(1) It is possible to improve the shear resistance capacity of brittle elements in beams, columns or shear walls by applying FRP strips or sheets. These can be applied by completely wrapping the element or by gluing them to the sides and bottom of the beam (U-shaped strips or sheets), or by gluing them only to the sides.

(2) The total shear resistance capacity, controlled by the abutments and FRP, is evaluated as the sum of a contribution from the existing concrete element, evaluated according to Annex 1, and a contribution, $V_f$, from the FRP.

(3) The total shear resistance capacity cannot be taken to be greater than the maximum shear resistance of the concrete element, $V_{R,max}$, controlled by the diagonal web compression. The value of $V_{R,max.}$ can be calculated in accordance with Annex 19 of the Structural Code. For concrete walls and columns with a shear slenderness, $L_V/h$, less than or equal to 2, the value of $V_{R,max.}$ is the lower of the value given in Annex 19 of the Structural Code and the value calculated according to section **A.3.3.1(2 and 3)**, respectively, under non-elastic cyclic loading.

(4) For elements with rectangular cross-section, the contribution of FRP to the shear resistance capacity can be evaluated as follows:

- for a full FRP shell, or for the case of U-shaped FRP strips or sheets,

$$V_{Rd,f} = 0,9\,d \cdot f_{fdd,e} \cdot 2 \cdot t_f \cdot \left(\frac{w_f}{s_f}\right)^2 \cdot (\cot\theta + \cot\beta) \cdot \mathrm{sen}\,\beta \quad \text{(A.22)}$$

- in the case of strips or sheets glued to the sides,

$$V_{\mathrm{Rd,f}} = 0,9\,d \cdot f_{\mathrm{fdd,e}} \cdot 2 \cdot t_{\mathrm{f}} \cdot \frac{\mathrm{sen}\beta}{\mathrm{sen}\theta} \cdot \frac{w_{\mathrm{f}}}{s_{\mathrm{f}}} \tag{A.23}$$

where

$d$ is the usable edge;

$\theta$ is the angle of inclination of the connecting rod;

$f_{\mathrm{fdd,e}}$ is the calculated value of the effective peel resistance of the FRP, which depends on the configuration of the reinforcement according to **(5)** for full FRP wraps, or item **(6)** for FRP U-shaped wraps, or **(7)** for FRP glued on sides;

$t_{\mathrm{f}}$ is the thickness of the FRP strip, sheet or web (over a single side);

$\beta$ is the angle between the (resistance) direction of the fibre in the FRP strip, sheet or web, and the axis of the element;

$w_{\mathrm{f}}$ is the width of the FRP web or sheet, measured orthogonally to the fibre (resistance) direction (for sheets: $w_{\mathrm{f}}$ = min. $(0.9d,h_{\mathrm{w}}) \cdot \sin(\theta + \beta) / \sin\theta$); and

$s_{\mathrm{f}}$ is the distance between FRP webs (= $w_{\mathrm{f}}$ for sheets), measured orthogonally to the fibre (resistance) direction.

(5) For fully wrapped (i.e. closed) or properly anchored (in the compression zone) liners, the calculated value of the effective peel resistance of the FRP can be taken from expressions (A.22) and (A.23) as:

$$f_{\mathrm{fdd,e,W}} = f_{\mathrm{fdd}} \cdot \left[1 - k\frac{L_{\mathrm{e}}\mathrm{sen}\beta}{2z}\right] + \frac{1}{2}\left(f_{\mathrm{fu,W}}(R) - f_{\mathrm{fdd}}\right) \cdot \left[1 - \frac{L_{\mathrm{e}}\mathrm{sen}\beta}{z}\right] \tag{A.24}$$

where

$z = 0.9\,d$ is the internal mechanical arm,

$k = \left|1 - \frac{2}{\pi}\right|$, and:

$$f_{\mathrm{fdd}} = \frac{1}{\gamma_{\mathrm{fd}}}\sqrt{0,6\frac{E_{\mathrm{f}}\,f_{\mathrm{ctm}}\,k_{\mathrm{b}}}{t_{\mathrm{f}}}} \quad \text{(units in N, mm)} \tag{A.25}$$

is the calculated value of the push-off resistance, with:

$\gamma_{\mathrm{fd}}$ partial coefficient for FRP push-off. Take the value $\gamma_{\mathrm{fd}}$ = 1,5;

$E_f$ is the modulus of the FRP sheets/plates;

$f_{ctm}$ is the average tensile strength of the concrete;

$k_b = \sqrt{1,5 \cdot (2 - w_f / s_f)/(1 + w_f / 100 \text{ mm})}$ is the cover coefficient,

in which:

$w_f$, $s_f$, $t_f$ are defined in **(4)**;

$f_{fu,W}(R)$ is the ultimate resistance of the FRP strip or sheet around the corner with radius $R$, given by:

$$f_{fu,W}(R) = f_{fdd} + \langle \eta_R \cdot f_{fu} - f_{fdd} \rangle \quad \text{(A.26)}$$

where the term between '-' should be taken into account only if it is positive and where the coefficient $\eta_R$ depends on the radius $R$ and the width of the beam $b_w$ according to:

$$\eta_R = 0,2 + 1,6 \frac{R}{b_w} \qquad 0 \leq \frac{R}{b_w} \leq 0,5 \quad \text{(A.27)}$$

$L_e$ is the effective bond length:

$$L_e = \sqrt{\frac{E_f \cdot t_f}{\sqrt{4 \cdot \tau_{max.}}}} \qquad \text{(unidades: N, mm)} \quad \text{(A.28)}$$

with:

$\tau_{max.}$ = $1.8 f_{ctm} k_b$ = maximum bond stress.

(6) For U-shaped (i.e. open) liners, the calculated value of the effective push-off resistance of the FRP can be obtained from expressions (A.22) and (A.23), as follows:

$$f_{fdd,e,U} = f_{fdd} \cdot \left[ 1 - k \frac{L_e \text{ sen}\beta}{z} \right] \quad \text{(A.29)}$$

where all variables are defined in **(5)**.

(7) For side bonded sheets/strips, the calculated value of the effective push-off resistance of the FRP can be obtained from the expressions (A.22) and (A.23), according to:

$$f_{\mathrm{fdd,e,S}} = f_{\mathrm{fdd}} \cdot \frac{z_{\mathrm{rid,eq}}}{z} \cdot \left(1 - \sqrt{k \frac{L_{\mathrm{eq}}}{z_{\mathrm{rid,eq}}}}\right)^2 \tag{A.30}$$

where

$$z_{\mathrm{rid,eq}} = z_{\mathrm{rid}} + L_{\mathrm{eq}}, \quad z_{\mathrm{rid}} = z - L_{\mathrm{e}} \cdot \mathrm{sen}\beta, \quad L_{\mathrm{eq}} = \frac{u_1}{\varepsilon_{\mathrm{fdd}}} \cdot \mathrm{sen}\beta \tag{A.31}$$

with:

$\varepsilon_{\mathrm{fdd}} = f_{\mathrm{fdd}}/E_{\mathrm{f}}$, and

$u_1 = k_{\mathrm{b}}/3$.

(8) For circular cross-section elements with a diameter $D$, the FRP contribution is evaluated with:

$$V_{\mathrm{f}} = 0,5\ A_{\mathrm{c}} \cdot \rho_{\mathrm{f}} \cdot E_{\mathrm{f}} \cdot \varepsilon_{\mathrm{f,ed}} \tag{A.32}$$

where

$A_{\mathrm{c}}$ transverse area of the column;

$\rho_{\mathrm{f}}$ equal to 4 $t_{\mathrm{f}}/D$, is the volumetric amount of FRP; and

$\varepsilon_{\mathrm{f,ed}}$ = 0.004.

(9) For members with their critical region fully wrapped with an FRP liner over a span at least equal to the member edge $h$, the cyclic shear resistance, $V_{\mathrm{R}}$, can be considered to decrease with the plastic part of the chord rotation ductility demand at the member end: $\mu_{\Delta}^{\mathrm{pl}} = \mu_{\Delta} - 1$, according to expression (A.12), by adding to $V_{\mathrm{W}}$ (i.e. to the contribution of the transverse reinforcement to the shear resistance) the contribution of the FRP liner. The contribution of the FRP liner to $V_{\mathrm{W}}$ can be calculated by assuming that the FRP stress reaches the calculated value of the FRP ultimate resistance, $f_{\mathrm{u,fd}}$, at the extreme tensile fibres and decreases linearly to zero along the serviceable edge $d$:

$$V_{\mathrm{w,f}} = 0,5 \rho_{\mathrm{f}} b_{\mathrm{w}} z f_{\mathrm{u,fd}} \tag{A.33}$$

where

$\rho_{\mathrm{f}}$ equals $2t_{\mathrm{f}}/b_{\mathrm{w}}$, is the geometrical amount of FRP;

$z$ is the length of the internal mechanical arm, taken equal to $d$; and

$f_{\mathrm{u,fd}}$ is the calculated value of the FRP ultimate resistance, equal to the FRP ultimate resistance, $f_{\mathrm{u,f}}$, divided by the partial coefficient $\gamma_{\mathrm{fd}}$ of the FRP, where $\gamma_{\mathrm{fd}}$ = 1.5.

### A.4.4.3 Containment action

(1) The improvement of the deformation capacity is achieved by the confinement of the concrete by means of FRP liners. These are applied around the element to be reinforced in the region of the potential plastic hinge.

(2) The required magnitude of the confining pressure depends on the ratio $I_\chi = \mu_{\phi,tar}/\mu_{\phi,ava}$, between the ductility in terms of target curvature $\mu_{\phi,tar}$ and the ductility in terms of available curvature $\mu_{\phi,ava}$ and can be evaluated as:

$$f_l = 0{,}4\ I_\chi^2 \frac{f_c \cdot \varepsilon_{cu}^2}{\varepsilon_{ju}^{1{,}5}} \tag{A.34}$$

where

$f_c$ is the resistance of the concrete, defined by the expression (A.1);

$\varepsilon_{cu}$ is the ultimate unit strain of the concrete; and

$\varepsilon_{ju}$ is the adopted ultimate unit deflection of the FRP liner, which is smaller than the ultimate unit deflection of the FRP, $\varepsilon_{fu}$.

(3) In the case of circular cross-sections wrapped in continuous sheets (not in webs), the confining pressure exerted by the FRP sheet is equal to $f_l = ½\ \rho_f\ E_f\ \varepsilon_{ju}$, where $E_f$ is the modulus of elasticity of the FRP and $\rho_f$ is the geometric size of the FRP liner relative to its thickness: $t_f = \rho_f\ D/4$, where $D$ is the diameter of the liner around the cross-section.

(4) In the case of rectangular cross-sections where the corners have been rounded to a radius $R$ to allow FRP wrapping around them (see figure A.1), the confining pressure applied by the FRP liner is evaluated as: $f'_l = k_s f_l$, with $k_s = 2R/D$ and $f_l = 2\ E_f\ \varepsilon_{ju}\ t_f/D$, where $D$ is the largest section width.

(5) In the case of wrap applied by strips with spacings $s_f$, the confining pressure applied by the FRP strips is evaluated as: $f'_l = k_g f_l$, with $k_g = (1 - s_f/2D)^2$.

(6) In the case of rectangular section elements with rounded corners as in figure A.1, an alternative to **(2)** and **(4)** is to calculate the total chord rotation, or its plastic part, by means of the expressions (A.1) or (A.3), respectively, adding $\alpha\rho_f f_{f,e}/f_c$ to the exponent of the term due to confinement (i.e. the power of base 25 in the penultimate term of the expressions (A.1) and (A.3)), with

(a) $\rho_f = 2t_f/b_w$, the amount of FRP parallel to the loading direction;

(b) $f_{f,e}$, effective stress given by the following expression:

$$f_{f,e} = \text{mín.}\left(f_{u,f};\ \varepsilon_{u,f} E_f\right)\left[1 - \text{mín.}\left[0{,}5;\ 0{,}7\,\text{mín.}\left(f_{u,f};\ \varepsilon_{u,f} E_f\right)\frac{\rho_f}{f_c}\right]\right] \tag{A.35}$$

where $f_{u,f}$ and $E_f$ are the resistance and modulus of elasticity of the FRP, and $\varepsilon_{u,f}$ a limit unit strain equal to 0.015 for CFRP (carbon fibre-reinforced polymer) or AFRP (aramid fibre-reinforced polymer) and 0.02 for GFRP (glass fibre-reinforced polymer); and

(c) $\alpha$, the confinement effectiveness coefficient given by:

$$\alpha = 1 - \frac{(b-2R)^2 + (h-2R)^2}{3bh} \tag{A.36}$$

where $R$ is the radius of the rounded corner of the section, and $b$, $h$ the dimensions of the section (see figure A.1).

(7) Point **(6)** applies to elements with corrugated reinforcement (high bond) or longitudinal plain reinforcement, with or without anti-seismic construction details, provided that the extreme zone is wrapped in FRP up to a distance from the extreme zone sufficient to ensure that the yield moment $M_y$ in the unwrapped part is not exceeded before the bending resistance $\gamma_{Rd}M_y$ is reached in the extreme section. To account for the increase in the bending resistance of the extreme section due to confinement by FRP, $\gamma_{Rd}$ should be at least equal to 1.3.

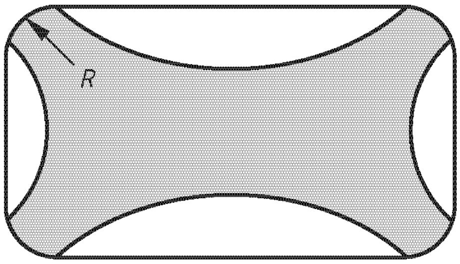



**Figure A.1 - Effective confinement area in an FRP-wrapped section**

### A.4.4.4 Fixing of overlapping joints

(1) Slippage of the overlapping joints can be prevented by applying a lateral pressure $\sigma_1$ by means of FRP liners. For circular columns, with diameter $D$, the required thickness can be estimated as:

$$t_f = \frac{D(\sigma_1 - \sigma_{sw})}{2E_f \cdot 0{,}001} \tag{A.37}$$

where $\sigma_{sw}$ is the confining stress due to the abutments for a deflection of 0.001 ($\sigma_{sw}$ = 0.001 $\rho_w E_s$), or the active pressure of the mortar injected between the FRP and the column, if any, while $\sigma_1$ represents the confining stress along the span $L_s$ of lap joints, as given by:

$$\sigma_l = \frac{A_s f_{yL}}{\left[\frac{p}{2n} + 2(d_{bL} + c)\right] L_s} \tag{A.38}$$

where

$A_s$ is the area of each spliced longitudinal bar;

$f_{yL}$ is the yield strength of the longitudinal steel reinforcement, taken equal to the average value obtained from *in situ* tests and additional sources of information, appropriately multiplied by the confidence coefficient, CF, given in Table 3.1 for the appropriate level of knowledge (see section **2.2.1(5)**);

*p)* is the perimeter, in the transverse section of the column, along the inside of the longitudinal steel reinforcement;

$n$ is the number of spliced reinforcement along $p$;

$d_{bL}$ is the (largest) diameter of the longitudinal steel reinforcement; and

$c$ is the thickness of the concrete cover.

(2) In the case of rectangular columns, the above expressions can be used by substituting $D$ by $b_w$, the section width, and reducing the effectiveness of the FRP liner by the coefficient defined in section **A.4.4.3(4)**.

(3) For rectangular section members with overlapping longitudinal reinforcement over a span $l_0$ from the end section of the member, the application of section **A.3.2.2(4)** is an alternative to **(1)** and **(2)** for the calculation of the effect of the FRP sheathing over a span exceeding the overlap length by at least 25%:

a) taking into account in expression (A.3) the confinement due to the transverse reinforcement only (the base power 25 in the penultimate term); and

b) calculating $l_{ou,min.}$ as: $l_{ou,min.} = d_{bL} f_{yL}/[(1{,}05 + 14{,}5\ \alpha_{1,f}\, \rho_f f_{f,e}/f_c)\sqrt{f_c}$ as a function of FRP only, with $\alpha_{l,f} = \alpha(4/n_{tot})$ and $\rho_f, f_{f,e}, \alpha, n_{tot}$ as defined in section **A.4.4.3(6)**.

# Appendix B

# Recommendations for steel and composite structures

## B.1 Purpose and scope

(1) This appendix contains information for the seismic resistance assessment of steel-framed and composite buildings in their present state and for their seismic retrofitting, where necessary. Seismic retrofitting may be local or global.

## B.2 Identification of geometry, construction details and materials

### B.2.1 General considerations

(1) The following aspects should be carefully examined:

i. The current physical condition of the base metal and connector materials, including the presence of deformations.

ii. The current physical condition of the primary and secondary seismic elements, including the presence of any degradation.

### B.2.2 Geometry

(1) The data collected should include the following items:

i. Identification of systems resistant to lateral forces.

ii. Identification of horizontal diaphragms.

iii. Original shape and dimensions of transverse cross-sections.

iv. Existing transverse section area, static moment, moment of inertia and torsional properties of critical cross-sections.

### B.2.3 Construction details

(1) The data collected should include the following items:

i. Dimensions and thickness of additional connected materials, including joint cover plates, triangulation elements (bracing) and stiffeners.

ii. Quantities of longitudinal and transverse reinforcement steel and dowels in beams, columns and composite walls.

iii. Adequate quantity and construction details of confining reinforcement in critical areas.

iv. Configuration and as-built properties of intermediate, continuity, and end connections.

### B.2.4 Materials

(1) The data collected should include the following items:

i. Concrete resistance.

ii. Elastic limit, strain hardening, ultimate strength and elongation of the steel.

(2) Where possible, areas of reduced stress, such as flange ends at beam-column ends and at the outer edges of slabs, should be selected for inspection.

(3) To assess the material properties, samples should be taken from the web plates of hot-rolled sections for elements designed as dissipative.

(4) To characterise the material properties of non-dissipative elements and/or connections, wing specimens should be used.

(5) Gamma radiography, ultrasonic testing through building façades or endoscopic examination through drilled holes are viable test methods where accessibility is limited or for mixed elements.

(6) The quality of base and filler materials should be demonstrated on the basis of chemical and metallurgical data.

(7) Charpy V-notch hardness testing should be used to demonstrate that heat-affected zones, if any, and the surrounding material have adequate resistance to brittle fracture.

(8) Destructive and/or non-destructive testing (liquid penetrant, magnetic particle, acoustic emission) and ultrasonic or tomographic methods may be used.

## B.3 Requirements concerning the geometry and materials of new or modified parts

### B.3.1 Geometry

(1) Steel sections of new parts should satisfy the slenderness ratio width/thickness limitations based on section classifications according to Chapters **6** and **7** of Annex 1.

(2) Straight transverse rounds increase the rotational capacities of existing or new column beams even with slender flanges and webs. Such transverse bars should be welded between the flanges according to section **7.6.5** of Annex 1.

(3) The transverse straight bars in **(2)** should be spaced like the transverse abutments used in concrete embedded elements.

### B.3.2 Materials

#### B.3.2.1 Structural steel

(1) For new parts or for the replacement of existing structural members, steel complying with section **6.2** of Annex 1 should be used.

(2) When the resistance and rigidity of structural members are assessed for each limit state, the effects of mixed action should be taken into account.

(3) The resistance of the full-thickness flanges of columns should be based on the reduced resistance as follows:

$$f_u = 0{,}90 \cdot f_y \tag{B.1}$$

(4) The thickness of the member should comply with the requirements of Table A28.2.1 of Annex 28 of the Structural Code, depending on the Charpy energy on V-notched specimen and other relevant parameters.

(5) Welding consumables (filler materials) should comply with the requirements of section **4.2** of Annex 26 of the Structural Code.

(6) In wide flange sections the specimens should be cut from the flange/web joint zones. This is a zone (k-zone) of potentially reduced Charpy hardness due to the slow cooling process during fabrication.

#### B.3.2.2 Steel for passive reinforcement

(1) The new steel for passive reinforcement, both in dissipative and non-dissipative zones in new or modified elements, should be of type SD according to Articles 34 and 35 of the Structural Code.

#### B.3.2.3 Concrete

(1) New concrete of new or modified components should be in accordance with requirement **(1)** of section **7.2.1** of Annex 1.

## B.4 Seismic adequacy of systems

### B.4.1 General considerations

(1) Overall seismic retrofit strategies should be able to increase the capacity of lateral force resisting systems and horizontal diaphragms and/or reduce the demand imposed by seismic actions.

(2) The structural system subject to seismic retrofit should satisfy the following requirements:

i. Evenness in mass distribution, rigidity and resistance, to avoid detrimental torsional effects and/or soft floor mechanisms (in diaphanous floors).

ii. Sufficient masses and rigidity to avoid highly flexible structures that may cause significant non-structural damage and significant P-Δ effects.

iii. Continuity and redundancy between elements, so as to ensure a uniform and clear load path and the prevention of brittle failures.

(3) Comprehensive interventions should include one or more of the following strategies:

i. Stiffening and strengthening of the structure and its foundation system.

ii. Improving the ductility of the structure.

iii. Mass reduction.

iv. Seismic isolation.

v. Additional damping.

(4) For all structural systems, stiffening, strengthening and ductility enhancement can be achieved using the strategies provided in sections **B.5** and **B.6**.

(5) Mass reduction can be achieved by one of the following measures:

i. Replacement of heavy cladding systems with lighter systems.

ii. Removal of unused equipment and stored loads.

iii. Replacement of masonry partitioning with lighter weight systems.

iv. Removal of one or more floors.

(6) Base isolation should not be used for structures with fundamental periods greater than 1.0 s. Such periods should be calculated by eigenvalue analysis (eigenvalues).

(7) Base isolation in new buildings should be designed in accordance with Annex 1.

(8) Re-evaluation of the foundation system (after seismic retrofitting) should be done in accordance with section **4.4.2.6** of Annex 1. If a linear analysis is used, the values of $\Omega$ in section **4.4.2.6(4)** of Annex 1 will normally be less than 1.0.

### B.4.2 Bending-resistant portal frames

(1) The mixed action between steel beams and concrete slabs should be improved by using shear connectors, and embedding the beams and columns in reinforced concrete, to increase the overall rigidity in all limit states.

(2) The length of the dissipative zones should be consistent with the location of the hinge given in the first line of Table B.6.

(3) Bending-resistant frames may be seismically retrofitted with semi-rigid and/or partially rigid, steel or composite connections.

(4) The fundamental period of frames with semi-rigid connections can be calculated as follows:

$$T = 0{,}085 \cdot H^{\left(0{,}85 - m/180\right)} \quad \text{if } 5 < m < 18 \text{ (semi-rigid).} \tag{B.2}$$

$$T = 0{,}085 \cdot H^{3/4} \quad \text{if } m \geq 18 \text{ (rigid)} \tag{B.3}$$

where $H$ is the height of the portal frame in metres and $m$ is a parameter defined as follows:

$$m = \frac{\left(K_{\varphi}\right)_{\mathrm{con}}}{\left(EI/L\right)_{\mathrm{b}}} \tag{B.4}$$

where

$K_{\varphi}$ is the rotational rigidity of the connection;

$I$ is the moment of inertia of the beam;

$L$ is the span of the beam;

$E$ is the Young's modulus of the beam.

(5) In addition to the horizontal force distribution given in section **4.3.3.2.3** of Annex 1 and section **4.4.4.2(1)** of this Annex, the following force distribution ($F_{x,y}$) should be used in the (linear) lateral force method and in the non-linear static analysis (by incremental thrusts) to detect the occurrence of all limit states:

$$F_{x,i} = \frac{W_{x,i} \cdot h_{x,i}^{\delta}}{\sum W_{x,i} \cdot h_{x,i}^{\delta}} \cdot F_b \tag{B.5}$$

where $F_b$ is the seismic shear stress at the base and $\delta$ is given by:

$$\delta = \begin{cases} 1{,}0 & \text{if} \quad T \leq 0{,}50 \text{ s} \\ 0{,}50 \cdot T + 0{,}75 & \text{if} \quad 0{,}50 < T < 2{,}50 \text{ s} \\ 2{,}0 & \text{if} \quad T > 2{,}50 \text{ s} \end{cases} \tag{B.6}$$

### B.4.3 Triangulated (braced) portal frames

(1) Off-centred triangulated frames and braced frames should be preferred for seismic adequacy over centrally triangulated frames.

(2) Portal frames with jibs are systems in which the triangulations (braces) are connected to a dissipative zone rather than to the beam-column connection.

(3) Aluminium or stainless steel may, only if their use is validated by testing, be used for dissipative zones in portal frames with centred or off-centre triangulation or jibs.

(4) In seismic retrofitting, steel, concrete and/or composite walls can be used to improve the ductile response and prevent beam-pier node instability. Their design and the design of their connection with steel elements should comply with Annex 1.

(5) Low yield strength steel may be used in the steel panels, which should be welded in the workshop and bolted on site.

(6) Triangulation may be introduced in bending-resistant frames in order to increase their lateral rigidity.

## B.5 Seismic evaluation and retrofitting of elements

### B.5.1 General requirements

(1) Beams should develop their full plastic moment without buckling of the flange or web at the significant damage (SD) limit state. The buckling should be limited to the near collapse (NC) limit state.

(2) No bending or axial plastification or buckling should occur in columns for the damage (DL) limit state and significant damage (SD) limit state.

(3) Diagonal bracing should support the plastic deformations and dissipate the energy through successive cycles of plastification and buckling. In the damage limitation (DL) limit state, buckling should be avoided.

(4) Steel plates should be welded to the flanges and/or webs to reduce slenderness.

(5) The bending capacity $M_{pb,Rd}$ of the beam at the plastic hinge should be calculated as follows:

$$M_{pb,Rd,b} = Z_e \cdot f_{yb} \tag{B.7}$$

where

$Z_e$ is the effective plastic modulus of the section at the plastic hinge, calculated from the actual measured dimensions of the section; and

$f_{yb}$ is the yield strength of the beam steel; for existing steel $f_{yb}$ can be taken as the mean value obtained from *in situ* tests and from additional sources of information, appropriately multiplied by the confidence coefficient, CF, given in Table 3.1 for the appropriate level of knowledge (see section **3.5(2)**); for new steel $f_{yb}$ may be taken as the nominal value multiplied by the reserve strength coefficient $\gamma_{ov}$ for the beam steel, determined in accordance with Annex 1, section **6.2(3)**, **(4)** and **(5)**.

(6) The bending demand $M_{cf,Ed}$ of the critical section at the column face is evaluated as follows:

$$M_{cf,Ed} = M_{pl,Rd,b} + V_{pl,Rd,b} \cdot e \tag{B.8}$$

where

$M_{pl,Rd,b}$ is the plastic moment of the beam at the plastic hinge of the beam;

$V_{pl,Rd,b}$ is the shear force at the hinge plastic hinge of the beam;

$E$ is the distance between the plastic hinge of the beam and the face of the column.

(7) The bending demand $M_{cc,Ed}$ of the critical section at the column axis can be calculated as follows:

$$M_{cc,Ed} = M_{pl,Rd,b} + V_{pl,Rd,b} \cdot \left( e + \frac{d_c}{2} \right) \tag{B.9}$$

where $d_c$ is the column depth.

## B.5.2 Deformation capacities of the elements

(1) The inelastic deformation capacities of the structural elements in the three limit states can be taken as indicated in the following paragraphs.

(2) The inelastic deformation capacities of beam nodes with columns may be taken as given in Table B.6 (section **B.6.2.1**), provided that the connected elements fulfil the requirements given in the first five lines of that table.

(3) For beams and columns in bending, the inelastic deflection capacity should be expressed in terms of plastic rotation at the end of the member, as a multiple of the chord rotation at the yield point, $\theta_y$, at the end in question. For beams and columns subjected to a dimensionless axial load $\nu$ not greater than 0.30, the inelastic deformation capacities at the three limit states can be taken equal to those given in Table B.1.

**Table B.1 - Plastic rotational capacity at the end of beams or columns subjected to a dimensionless axial load $\nu$ not greater than 0.30**

| | Limit state | | |
|---|---|---|---|
| **Transverse section class** | **DL** | **SD** | **NC** |
| 1 | 1.0 $\theta_y$ | 6.0 $\theta_y$ | 8.0 $\theta_y$ |
| 2 | 0.25 $\theta_y$ | 2.0 $\theta_y$ | 3.0 $\theta_y$ |

(4) For triangulations in compression the inelastic deflection capacity should be expressed in terms of the axial deflection of the triangulation, as a multiple of the axial deflection of the triangulation under buckling load, $\Delta_c$. For triangulations in compression (except for portal frame triangulations with off-centre triangulation) the inelastic deformation capacities in the three limit states can be taken according to Table B.2.

**Table B.2 - Axial deflection capacity of triangulations in compression (except triangulations of frames with off-centre triangulations)**

| | Limit state | | |
|---|---|---|---|
| **Transverse section class** | **DL** | **SD** | **NC** |
| 1 | 0.25 $\Delta_c$ | 4.0 $\Delta_c$ | 6.0 $\Delta_c$ |
| 2 | 0.25 $\Delta_c$ | 1.0 $\Delta_c$ | 2.0 $\Delta_c$ |

(5) For triangulations in tension the inelastic deflection capacity should be expressed in terms of the axial deflection of the triangulation, as a multiple of the axial deflection of the triangulation for the tensile plastification load, $\Delta_t$. For tensile triangulations (except for portal frame triangulations with off-centre triangulation) with class 1 and 2 transverse cross-sections, the inelastic deformation capacities in the three limit states can be taken according to Table B.3:

**Table B.3 - Axial bending capacity of triangulations in tension (except portal frame triangulations with off-centre triangulations)**

| Limit state | | |
|---|---|---|
| **DL** | **SD** | **NC** |
| 0.25 $\Delta_t$ | 7.0 $\Delta_t$ | 9.0 $\Delta_t$ |

(6) For beams and columns in tension, the inelastic deflection capacity should be expressed in terms of the axial deflection of the member, as a multiple of its axial deflection for the tensile plastification load, $\Delta_t$. For beams or columns in tension (except for portal frame triangulations with off-centre triangulation) with class 1 or 2 cross-sections, the inelastic deflection capacities in the three limit states can be taken according to Table B.4.

**Table B.4 - Axial deflection capacity of beams or columns in tension (except beams or columns of portal frames with off-centre triangulations)**

| Limit state | | |
|---|---|---|
| **DL** | **SD** | **NC** |
| 0.25 $\Delta_t$ | 3.0 $\Delta_t$ | 5.0 $\Delta_t$ |

## B.5.3 Beams

### B.5.3.1 Insufficient stability

(1) Beams with span-to-height ratios between 15 and 18 should be preferred to improve energy absorption. Therefore, intermediate supports should be used in seismic retrofitting to shorten large spans.

(2) Wings with insufficient stability should be restrained against lateral buckling. Lateral restraint of the top flange is not required if the mixed action with the slab is reliable. If this is not the case, the mixed action should be improved by fulfilling the requirements of section **B.5.3.5**.

### B.5.3.2 Insufficient resistance

(1) Steel plates should be added to the flanges of beams to increase insufficient bending capacity. It is not necessary to add steel to the top flange if the mixed action with the slab is reliable. Alternatively, structural steel beams with insufficient bending capacity should be embedded in reinforced concrete.

(2) Longitudinal reinforcement that may be added to increase insufficient bending capacity should be of type SD, in accordance with Articles 34 and 35 of the Structural Code.

(3) Beams subject to seismic retrofitting due to insufficient resistances should meet the requirements of the medium ductility class (MDC) of Annex 1.

(4) To improve the insufficient shear capacity, steel plates should be added to the web of beams with H-sections (double T-sections), or to the wall of hollow sections.

### B.5.3.3 Repair of buckled and fractured flanges

(1) Buckled and/or fractured flanges should be reinforced or replaced with new plates.

(2) Buckled bottom and/or top flanges should be repaired by adding web stiffeners over their full height, on both web faces of the beams, in accordance with **(3)** below, and by heat straightening of the buckled flange, or by removing and replacing it with a similar plate in accordance with **(4)** and **(5)** below.

(3) The web stiffeners should be placed at the edge and in the centre of the buckled flange, respectively; the thickness of the stiffener should be equal to that of the web of the beam.

(4) The new plates should either be welded in the same position as the original flange (i.e. directly to the web of the beam), or welded to the existing flange. In both cases the added plates should be oriented longitudinally in the rolling direction.

(5) Special shoring should be provided for the flange plates during cutting and replacement operations.

(6) Instead of welding a thick plate to the flange, it would be preferable to embed the steel beam in reinforced concrete.

### B.5.3.4 Weakening of the beams

(1) The ductility of steel beams can be improved by weakening the flange of the beam at the desired locations in order to move the dissipative zones away from the connections.

(2) These reduced beam sections (RBS) behave as a fuse that protects the beam-column connections against premature fracture. The reduced beam sections should be able to develop in each limit state the minimum rotations specified in Table B.5.

**Table B.5 - Required rotational capacity of reduced beam sections, RBS (in radians)**

| DL | SD | NC |
|---|---|---|
| 0.010 | 0.025 | 0.040 |

(3) The rotations in Table B.5 are considered to be achieved if the calculation of the reduced beam cross-sections is carried out by the following process:

i. The distance of the beginning of the RBS from the face of the column, *a*, and the length of the flange reduction, *b*, are calculated as follows:

$$a = 0,60\, b_f \tag{B.10}$$

$$b = 0,75\, d_b \tag{B.11}$$

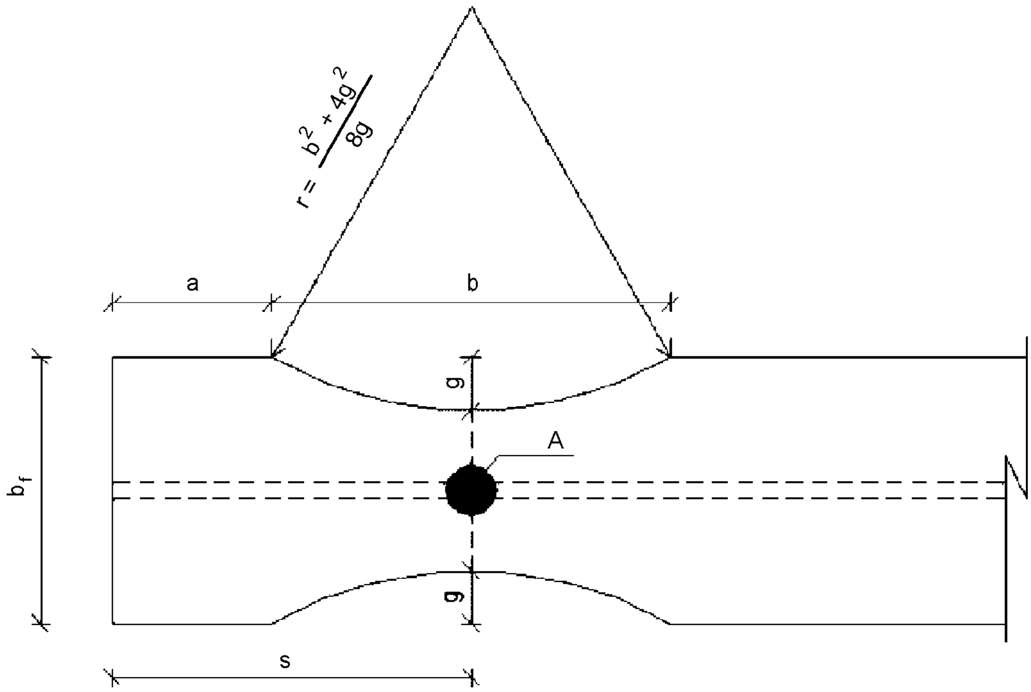
where

$b_f$ is the flange width;

$d_b$ is the edge of the beam.

ii. The distance of the intended plastic hinge section at the centre of the RBS, *s*, from the face of the column is calculated as:

$$s = a + \frac{b}{2} \tag{B.12}$$




Legend:
A = Plastic hinge

**Figure B.1 - Geometry of the flange reduction for reduced beam sections (RBS)**

iii. The depth of the flange cutout, $g$, is determined on each side; this depth should not be greater than $0.25 \cdot b_f$. As a first approximation, the following can be taken:

$$g = 0{,}20\ b_f \tag{B.13}$$

iv. The plastic modulus ($Z_{RBS}$) and plastic moment ($M_{pl,Rd,RBS}$) of the plastic hinge section at the centre of the RBS are calculated:

$$Z_{RBS} = Z_b - 2 \cdot g \cdot t_f \cdot (d_b - t_f) \tag{B.14}$$

$$M_{pl,Rd,RBS} = Z_{RBS} \cdot f_{yb} \tag{B.15}$$

where $Z_b$ is the plastic modulus of the beam and $f_{yb}$ is defined in section **B.5.1(5)**.

v. The shear force ($V_{pl,RBS}$) in the plastic hinge forming section is calculated from the equilibrium of the part of the beam ($L'$) between the two predicted plastic hinges (Figure B.2). For a uniform gravity load $w$ acting on the beam in the calculated seismic situation:

$$V_{pl,RBS} = \frac{2M_{pl,Rd,RBS}}{L'} + \frac{wL'}{2} \tag{B.16}$$

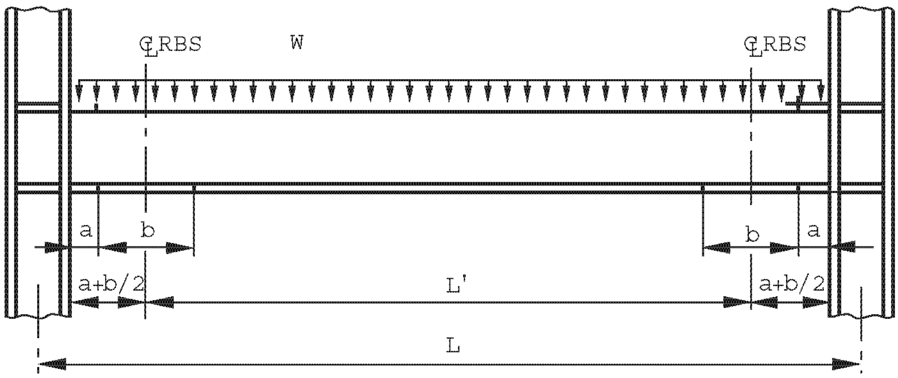
The different distributions of gravity loads along the beam should be taken into account in (the last term of) expression (B.16).

vi. Calculate the beam plastic moment away from the RBS, $M_{pl,Rd,b}$, as follows:

$$M_{pl,Rd,b} = Z_b \cdot f_{yb} \tag{B.17}$$

where $Z_b$ and $f_{yb}$ are defined in step (iv) above.

vii. It is verified that $M_{pl,Rd,b}$ is greater than the bending moment that develops at the face of the column when a plastic hinge is formed at the centre of the RBS: $M_{cf,Ed} = M_{pl,Rd,RBS} + V_{pl,RBS} \cdot e$. If it is not, the depth of cut $g$ is increased and steps (iv) to (vi) are repeated. The length $b$ should be chosen so that $M_{cf,Ed}$ is between 85% and 100% of $M_{pl,Rd,b}$.




Legend:

$w$ = Uniform gravity load in the calculated seismic situation

$L'$ = Distance between the centres of the RBS cuts

$L$ = Distance between the centre lines of the columns

**Figure B.2 - Part of a typical portal frame with reduced beam cross-sections (RBS)**

viii. Testing of the width to thickness ratios in the RBS is carried out to avoid buckling. The flange width should be measured at the ends of the middle two thirds of the reduced beam section.

ix. The radius, ($r$) of the cut-outs in both the upper flange and lower flange along the length $b$ of the reduced beam section is calculated:

$$r = \frac{b^2 + 4g^2}{8g} \tag{B.18}$$

x. It is tested that the manufacturing process ensures both adequate surface roughness (i.e. between 10 and 15 µm) on the finished cuts, as well as the absence of grind marks.

### B.5.3.5 Mixed elements

(1) The calculation of the capacity of composite beams should take into account the degree of shear connection between the steel element and the slab.

(2) Shear connectors between steel beams and composite slabs should not be used within dissipative

zones. They should be removed from existing composite beams.

(3) Shear connectors should be attached to the flanges by means of arc spot welds, but without full penetration into the flange. Rivet or bolt fixings should be avoided.

(4) Testing should be carried out to ensure that the maximum tensile deflections due to the presence of composite slabs do not cause tearing of the flanges.

(5) Beams embedded in concrete should be provided with abutments.

### B.5.4 Columns

#### B.5.4.1 Insufficient stability

(1) The width to thickness ratio can be reduced by welding steel plates to the flange and/or webs.

(2) The width to thickness ratio of hollow sections can be reduced by welding external steel plates.

(3) Lateral restraints should be provided on both flanges, by means of stiffeners of resistance not less than:

$$0,06 f_{yc} \cdot b_f \cdot t_f \qquad \text{(B.19)}$$

where

$b_f$ is the flange width;

$t_f$ is the flange thickness; and

$f_{yc}$ is the yield strength of the column steel; for existing steel one can take $f_{yc}$ equal to the average value obtained from *in situ* tests and from additional sources of information, multiplied by the confidence coefficient, CF, given in Table 3.1 for the appropriate level of knowledge (see section **3.5(2)**); for new steel, $f_{yc}$ can be taken to be equal to the nominal value multiplied by the reserve resistance (over-resistance) coefficient $\gamma_{ov}$ for the pier steel, determined in accordance with Annex 1, section **6.2(3)**, **(4)** and **(5)**.

#### B.5.4.2 Insufficient resistance

(1) To increase the bending capacity of the section, steel plates can be welded to the flanges and/or webs in H-sections and to the walls in hollow sections.

(2) To increase their bending capacity, structural steel columns can be embedded in reinforced concrete.

(3) Seismic adequacy should ensure that for all primary seismic piers the axial compression in the calculated seismic situation is not greater than 1/3 of the calculated value of the plastic resistance to normal forces of the gross transverse section of the column $N_{pl,Rd} = (A_a f_{yd} + A_c f_{cd} + A_s f_{sd})$ (see section **7.6.4(2)** of Annex 1) in the damage limit state and 1/2 of $N_{pl,Rd}$ in the significant damage and near collapse limit states.

#### B.5.4.3 Repair of buckled and fractured flanges and fractured joints

(1) Buckled and/or fractured flanges and fractured joints should be reinforced or replaced with new

plates.

(2) Buckled and fractured flanges should be repaired either by removing the buckled plate and replacing it with a similar plate or by direct flame stretching.

(3) Fractured joints should be repaired by adding external plates over the flanges of the column with continuous full penetration welds. The damaged portion should be removed and replaced with sound material. The thickness of the added plates should be equal to the thickness of the existing plates. The new material should be aligned so that the rolling direction is equal to that of the column.

(4) Small holes should be drilled at the edge of the cracks in the columns to prevent crack propagation.

(5) Magnetic particle or dyed liquid penetrant tests should be carried out to ensure that there are no subsequent defects and/or discontinuities up to at least 150 mm from a crack.

#### B.5.4.4 Requirements for joints in columns

(1) New joints should be placed in the middle third of the clear height of the column. They should be designed to develop a calculated value for the shear resistance not less than the lower of the shear resistance provided in the two connected members and a calculated value for the bending resistance less than 50 % of the lower of the bending resistance provided in the two connected sections. Therefore, the welded joints in columns should satisfy the following expression for each flange:

$$A_{\mathrm{spl}} \cdot f_{\mathrm{yd}} \geq 0{,}50 \cdot f_{\mathrm{yc}} \cdot A_{\mathrm{fl}} \tag{B.20}$$

where

$A_{\mathrm{spl}}$ is the area of each flange at the joint;

$f_{\mathrm{yd}}$ is the calculated value of the yield strength of the flange of the joint;

$A_{\mathrm{fl}}$ is the flange area of the smaller of the two connected columns; and

$f_{\mathrm{yc}}$ is the yield strength of the column material as defined in section **B.5.4.1(3)**.

#### B.5.4.5 Column panel zone

(1) In the column subject to seismic retrofitting, the panel zone at the beam-column connection should remain elastic in the damage-limiting limit state.

(2) The thickness, $t_{\mathrm{w}}$, of the column panel zone (including the reinforcing plate, if present, see **(3)**) should satisfy the following expression to avoid premature buckling under the action of significant inelastic shear deformations:

$$t_{\mathrm{w}} \leq \frac{d_{\mathrm{z}} + w_{\mathrm{z}}}{90} \tag{B.21}$$

where

$d_z$ is the depth of the panel zone between the continuity plates;

$w_z$ is the width of the panel area between the flanges of the column.

Plug welds should be used between the web and the added plate.

(3) Steel plates parallel to the web and welded to the flange edge (gusset plates) can be used to stiffen and reinforce the column web.

(4) Transverse stiffeners should be welded into the column web at the flanges of the beams.

(5) To ensure satisfactory performance in all limit states, continuity plates no less than the thickness of the flanges of the beam flanges should be symmetrically placed on both sides of the column web.

#### B.5.4.6 Mixed elements

(1) To improve the rigidity, resistance and ductility of steel columns, they can be embedded in reinforced concrete.

(2) To achieve effective composite action, shear stresses should be transferred between structural steel and reinforced concrete through shear connectors placed along the length of the column.

(3) To avoid shear bond failure, the ratio of the width of the steel flange to the width of the composite column, $b_f/B$, should not be greater than the critical value of the ratio, defined below:

$$\left(\frac{b_f}{B}\right)_{cr} = 1 - 0{,}35 \cdot \left[0{,}17 \cdot \left(1 + 0{,}073 \cdot \frac{N_{Ed}}{A_g}\right) \cdot \sqrt{f_{cd}} + 0{,}20 \cdot \rho_w \cdot f_{yw,d}\right] \tag{B.22}$$

where

$N_{Ed}$ is the axial force in the calculated seismic situation;

$A_g$ is the gross area of the mixed section;

$f_{cd}$ is the calculation value of the compressive strength of concrete;

$\rho_w$ is the amount of transverse reinforcement;

$f_{yw,d}$ is the calculated value of the yield strength of the transverse reinforcement;

$B$ is the width of the mixed section;

$b_f$ is the width of the steel flange.

### B.5.5 Triangulations (bracings)

#### B.5.5.1 Insufficient stability

(1) Section **B.5.4.1(1)** applies for triangulations formed by hollow sections.

(2) Section **B.5.4.2(1)** applies.

(3) Any envelope of steel bracing elements in cases of seismic retrofitting should comply with Annex 1.

(4) The lateral rigidity of diagonal triangulations can be improved by increasing the rigidity of end connections.

(5) For seismic adequacy, X-triangulations should be preferred to V-triangulations or inverted V-triangulations. K-triangulations cannot be used.

(6) Closely spaced clamps are effective in improving the post-buckling response of triangulation or bracing elements, particularly those in double angle or double channel. If the existing columns already have stiffeners or clamps, new plates can be welded and/or the connections of the existing ones should be reinforced.

#### B.5.5.2 Insufficient resistance

(1) In the damage-limiting limit state, the axial compression in the calculated seismic situation should not exceed 80 % of the calculated value of the plastic normal stress resistance of the triangulation transverse section, $N_{\mathrm{pl,Rd}}$.

(2) Unless only the near collapse limit state is verified, the compressive capacity of portal frame triangulations with centred triangulations should not be less than 50 % of the plastic resistance at normal stress of the transverse cross-section, $N_{\mathrm{pl,Rd}}$.

#### B.5.5.3 Mixed elements

(1) Embedding steel bracing elements in reinforced concrete increases their rigidity, resistance and ductility. Triangulation elements with H-section can be either partially or fully embedded.

(2) Fully embedded triangulation members should be provided with stiffeners and stirrups, and partially embedded ones with welded straight bars, in accordance with section **7.6.5** of Annex 1. The abutments should be evenly spaced along the length of the triangulation member and should comply with the requirements specified for the medium ductility class (MDC) in sections **7.6.4(3 and 4)** of Annex 1.

(3) Only the structural steel section should be taken into account in the calculation of the tensile capacity of mixed triangulations.

#### B.5.5.4 Non-adhesive triangulations

(1) Triangulations can be stiffened by non-bonded incorporation into both reinforced concrete walls and concrete-filled pipes.

(2) The triangulation should be covered with anti-adhesion material to reduce the adhesion between the steel element and the reinforced concrete panel or the concrete filling of the pipe.

(3) Low yield strength steels are suitable for steel triangulations; steel fibre reinforced concrete can be used as anti-adhesion material.

(4) Triangulations, stiffened by means of non-bonded incorporation into reinforced concrete walls, should comply with the following:

$$\left(1 - \frac{1}{n_E^B}\right) \cdot m_y^B > 1{,}30 \cdot \frac{a}{l} \tag{B.23}$$

where

$a$ is the initial imperfection of the steel triangulation;

$l$ is the length of the steel triangulation;

$m_y^B$ is the dimensionless resistance parameter of the reinforced concrete panel:

$$m_y^B = \frac{M_y^B}{N_{pl,R} \cdot l} \tag{B.24}$$

$n_E^B$ is the dimensionless parameter of rigidity of the reinforced concrete panel:

$$n_E^B = \frac{N_E^B}{N_{pl,R}} \tag{B.25}$$

where

$$M_y^B = \frac{5 \cdot B_S \cdot t_c^2 \cdot f_{ct}}{6} \tag{B.26}$$

$$N_E^B = \frac{5 \cdot \pi^2 \cdot B_S \cdot E_c \cdot t_c^3}{12 \cdot l^2} \tag{B.27}$$

where

$E_c$ is the modulus of elasticity of the concrete;

$B_s$ is the width of the steel triangulation in the form of a flat bar;

$t_c$ is the thickness of the reinforced concrete panel;

$f_{ct}$ is the tensile strength of the concrete;

$N_{pl,R}$ is the plastic capacity of the tensile steel triangulation, calculated on the basis of the average value of the yield strength of the steel obtained from *in situ* tests and additional sources of information, divided by the confidence coefficient, CF, given in Table 3.1 for the appropriate level of knowledge.

(6) The edge reinforcement of the reinforced concrete panel should be adequately anchored to prevent punching failure.

(7) Concrete filled tubes with anti-seize material should be adequate to prevent buckling of the steel triangulation.

## B.6 Seismic adequacy of the connection

### B.6.1 General considerations

(1) Connections of elements subject to seismic retrofitting should be tested taking into account the resistance of the elements after retrofitting, which may be higher than that of the original elements (before seismic retrofitting).

(2) The seismic retrofit strategies provided can be applied to steel or composite bending-resisting and triangulated frames.

### B.6.2 Beam-column connections

#### B.6.2.1 General considerations

(1) Seismic retrofitting should aim to move the plastic hinge in the beam away from the face of the column (see first row of Table B.6).

(2) The beam-column connections can be seismically retrofitted either by replacing the welds, by using a weakening strategy, or by using a strengthening strategy.

(3) To ensure the development of plastic hinges in the beams, rather than in the columns, the column-to-beam bending capacity ratio (*CBMR*) should satisfy the following condition:

$$CBMR = \frac{\sum M_{\mathrm{Rd,c}}}{\sum M_{\mathrm{pl,R,b}}} \geq 1{,}30 \tag{B.28}$$

where

(a) for steel columns:

$$\sum M_{\mathrm{Rd,c}} = \sum \left[ Z_{\mathrm{c}} \cdot \left( f_{\mathrm{yd,c}} - \frac{N_{\mathrm{Ed}}}{A_{\mathrm{c}}} \right) \right]_{\mathrm{i}} \tag{B.29}$$

where the summation extends to the column sections around the joint, and

$Z_{\mathrm{c}}$ is the plastic modulus of the column section, calculated on the basis of the actual geometric properties, if available, and taking into account gussets, if present;

$N_{\mathrm{Ed}}$ is the axial load of the column in the calculated seismic situation;

$A_{\mathrm{c}}$ is the cross-sectional area of the pier;

$f_{yd,c}$ is the calculated value of the yield strength of the column steel, calculated on the basis of the average value of the yield strength of the steel obtained from *in situ* tests and from additional sources of information, divided by the confidence coefficient, CF, given in Table 3.1 for the appropriate level of knowledge.

(b) $\sum M_{pl,R,b}$ is the sum of the bending resistances at the locations of the plastic hinges in beams connected to the connection in the horizontal direction considered, taking into account the eccentricity of the column axis:

$$\sum M_{pl,R,b} = \sum \left( Z_b \cdot f_{yb} + M_{cc,Ed} \right)_j \tag{B.30}$$

where

$Z_b$ is the plastic modulus of the beam section at the potential location of the plastic hinge, calculated on the basis of the actual geometry;

$f_{yb}$ is the yield strength of the beam steel, as defined in section **B.5.1(5)**;

$M_{cc,Ed}$ is the additional moment in the column axis due to the eccentricity of the shear force in the hinge of the beam plastic hinge.

**Table B.6 – Requirements for connections subject to seismic retrofitting and the resulting rotational capacities**

| | Connections | | | | |
|---|---|---|---|---|---|
| | welded enhanced connections without flange reinforcement | bottom gusset welded | welded with top and bottom gusset | welded to flanges with joint cover plate | on reduced beam sections |
| Hinge location (from the axis of the column) | $(d_c/2) + (d_b/2)$ | $(d_c/2) + l_h$ | $(d_c/2) + l_h$ | $(d_c/2) + l_{cp}$ | $(d_c/2)+(b/2)+a$ |
| Beam depth (mm) | ≤ 1 000 | ≤ 1 000 | ≤ 1 000 | ≤ 1 000 | ≤ 1 000 |
| Beam span-to-edge ratio | ≥ 7 | ≥ 7 | ≥ 7 | ≥ 7 | ≥ 7 |
| Thickness of the flange of the beam (mm) | ≤ 25 | ≤ 25 | ≤ 25 | ≤ 25 | ≤ 44 |
| Height of the column (mm) | No restriction | ≤ 570 | ≤ 570 | ≤ 570 | ≤ 570 |
| Rotation in the DL limit state (rad) | 0.013 | 0.018 | 0.018 | 0.018 | 0.020 |
| Rotation in the SD limit state (rad) | 0.030 | 0.038 | 0.038 | 0.040 | 0.030 |
| Rotation in the NC limit state (rad) | 0.050 | 0.054 | 0.052 | 0.060 | 0.045 |

Legend:

| | |
|---|---|
| $d_c$ | Column ridge |
| $d_b$ | Beam edge |
| $l_h$ | Gusset length |
| $l_{cp}$ | Length of cover plate |
| $a$ | Measurement of the cut edge from the edge of the rafter |
| $b$ | Cut length at beam flange |

(4) The requirements for beams and columns in seismically retrofitted connections are given in Table B.6. The same table gives the rotational capacity in the three limit states provided by the connection if the requirements are met.

### B.6.2.2 Replacement of welds

(1) Existing filler material should be removed and replaced with material in good condition.

(2) Shoring rebars should be removed after welding, as they may be the cause of crack initiation.

(3) Transverse stiffeners should be used at the top and bottom of the panel area used to reinforce and stiffen the column panel (see section **B.5.4.5(4)**). Their thickness should not be less than that of the flanges of the beam.

(4) The transverse and web stiffeners should be welded to the flanges of the column and to the web of the column by means of full penetration joint welds.

#### B.6.2.3 Weakening strategies

##### B.6.2.3.1 Connections with reduced beam cross-sections

(1) Reduced beam sections (RBS), designed in accordance with **(5)**, can force the development of plastic hinges within the reduced section, thus reducing the possibility of fracture at the flange welds of the beam and nearby heat-affected zones.

(2) The beam should be connected to the column flange either by web welding or by connecting plates welded to the face of the column flange and the beam web. The length of the plate should be equal to the distance between the weld access holes, with a margin of 5 mm. The minimum thickness of the plate should be 10 mm. The connector bars should be cut at right angles or with chamfered edges (chamfering the corner 15°) and should be placed on both sides of the web of the beam.

(3) Butt welds (full penetration) or fillet welds should be used for the flange of the beam and fillet welds for the web of the beam. Alternatively, the connecting plate may be bolted to the web of the beam.

(4) Shear connectors should not be placed within the RBS zones.

(5) The project procedure for RBS connections is summarised below:

i. Reduced section beams are used in accordance with the procedure in section **B.5.3.4**, but calculating the plastic moment of the beam, $M_{pl,Rd,b}$, as follows:

$$M_{pl,Rd,b} = Z_{RBS} \cdot f_{yb} \cdot \left( \frac{L - d_c}{L - d_c - 2 \cdot b} \right) \tag{B.31}$$

where

$f_{yb}$ is the yield strength of the beam steel, as defined in section **B.5.1(5)**;

$L$ is the distance between the centre lines of the columns;

$d_c$ is the column depth; and

$b$ is the reduced section length of the beam.

ii. The beam shear, $V_{pl,Rd,b}$, is calculated in accordance with step v in section **B.5.3.4(3)** for a distance $L'$ between plastic hinges:

$$L' = L - d_c - 2 \cdot b \tag{B.32}$$

iii. The web connection, e.g. the welded connecting plate, is checked for the shear stress $V_{pl,Rd,b}$ in step ii above.

iv. Testing that the column to beam bending capacity ratio, *CBMR*, satisfies the following condition:

$$CBMR = \frac{\sum Z_c \left( f_{yd,c} - \frac{N_{Ed}}{A_c} \right)}{\sum Z_b \cdot f_{yb} \cdot \left( \frac{L - d_c}{L - d_c - 2 \cdot b} \right)} \geq 1{,}20 \tag{B.33}$$

where

$Z_b$ and $Z_c$ are the plastic moduli of the beams and columns, respectively;

$N_{Ed}$ is the axial load on the column at the design seismic condition;

$A_c$ is the cross-sectional area of the pier;

$f_{yb}$ is the yield strength of the beam steel as defined in section **B.5.1(5)**,

$f_{yd,c}$ is the calculated value of the yield strength of the column steel, as defined in section **B.6.2.1(3)**.

v. The thickness of the continuity plates to stiffen the column web is determined at the level of the top and bottom flanges of the beam. This thickness should be at least equal to the flange thickness of the beam.

vi. Testing is carried out to ensure that the resistance and rigidity of the panel area is sufficient so that the panel remains elastic:

$$d_{wc} \cdot t_{wc} \cdot \frac{f_{yw,d}}{\sqrt{3}} \geq \frac{\sum Z_b \cdot f_{yb}}{d_b} \cdot \left( \frac{L - d_c}{L - d_c - 2 \cdot b} \right) \cdot \left( \frac{H - d_b}{H} \right) \tag{B.34}$$

where

$d_{wc}$ is the web edge of the column;

$t_{wc}$ is the thickness of the column web, including reinforcement bars, if any;

$f_{yw,d}$ is the calculated value of the yield strength of the panel area;

$Z_b$ is the plastic modulus of the beams;

$N_{Ed}$ is the axial load on the column in the calculated seismic situation;

$A_c$ is the cross-sectional area of the pier;

$f_{yb}$ is the yield strength of the beam steel as defined in section **B.5.1(5)**; and

$H$ is the storey height of the portal frame.

vii. The welds between the joined parts are calculated and detailed.

#### B.6.2.3.2 Semi-rigid connections

(1) Semi-rigid or partial resistance connections, either steel or mixed, can be used in order to achieve large plastic deformations without risk of fracture.

(2) Full interaction shear connectors should be welded to the top flange of the beam.

(3) Semi-rigid connections can be calculated assuming that the shear resistance is given by the web elements and the bending resistance by the flanges of the beam and the slab reinforcement, if any.

### B.6.2.4 Reinforcement strategies

#### B.6.2.4.1 Gusset connections

(1) Beam-column connections can be strengthened by adding gussets either at the bottom only or at the top and bottom flanges of the beam, forcing the dissipative zone to move towards the end of the gusset. It is more appropriate to add gussets only at the bottom flange since the lower flanges are much more accessible than the upper flanges; Moreover, the composite slab, if any, does not have to be removed.

(2) Triangular T-gussets are the most effective of the different types of gussets. If only bottom gussets are added, their height should be about a quarter of the beam edge. In connections with gussets at the top and bottom, the gusset height should be about one third of the beam edge.

(3) To strengthen the panel area of the column, transverse stiffeners should be used at the top and bottom flanges of the beam.

(4) Transverse stiffeners should also be used at the gusset edges to stiffen the column web and the beam web.

(5) The vertical stiffeners in the web of the beam web should cover the full height of the web and be welded on both sides of the web. Their thickness should be sufficient to resist the vertical component of the force due to the gusset flange at that location, and should not be less than the thickness of the beam flange. The local checks defined in section **6.2.6** of Annex 26 of the Structural Code should be satisfied.

(6) The gussets should be welded to the flanges of both the pier and the beam with full penetration welds.

(7) Plates used in bolted shear connections, if present, may be left in place. Such elements may be used in elements subject to seismic retrofitting, where this is necessary for reasons of resistance or performance.

(8) For gusset connections, a staged design procedure may be applied as follows:

i. Preliminary gusset dimensions are selected based on the web slenderness limitation of the gusset web. The following relationships can be used, as a first test, for the gusset length, $a$, and for the angle of the gusset flange to the element gusset, $\theta$.

$$a = 0,55 \cdot d_{\mathrm{b}} \qquad \text{(B.35)}$$

$$\theta = 30^{\circ} \tag{B.36}$$

where $d_b$ is the beam edge. The resulting gusset height, $b$, given by

$$b = a \cdot \tan \theta. \tag{B.37}$$

should respect architectural constraints, e.g. the presence of roofs and non-structural elements.

ii. The plastic moment of the beam at the gusset edge, $M_{pl,Rd,b,}$ is calculated from expression (B.17).

iii. The plastic shear of the beam ($V_{pl,Rd,b}$) is to be calculated according to step v. in section **B.5.3.4(3)** for the span $L'$ between the plastic hinges at the gusset ends.

iv. It is to be verified that the column-to-beam bending capacity ratio, *CBMR*, satisfies the condition:

$$CBMR = \frac{\sum Z_c \cdot \left( f_{yd,c} - \frac{N_{Ed}}{A_c} \right)}{\sum M_c} \geq 1,20 \tag{B.38}$$

where

$Z_c$ is the plastic modulus of the column section;

$f_{yd,c}$ is the calculated value of the yield strength of the column steel, as defined in section **B.6.2.1(3)**;

$N_{Ed}$ is the axial load on the column in the calculated seismic situation;

$A_c$ is the cross-sectional area of the pier;

$M_c$ is the sum of the column moments at the top and bottom ends of the enlarged panel area resulting from the moment development of the beam, $M_{pl,R,b}$, within each beam of the connection:

$$\sum M_c = \left[ 2M_{pl,R,b} + V_{pl,Rd,b} \cdot (L - L') \right] \cdot \left( \frac{H_c - \overline{d_b}}{H_c} \right) \tag{B.39}$$

$L$ is the distance between the centre lines of the columns;

$\overline{d_b}$ is the depth of the beam including the gusset; and

$H_c$ is the height of the portal frame plan.

v. Calculate the value of the dimensionless parameter $\beta$ given by:

$$\beta = \frac{b}{a} \cdot \left( \frac{3 \cdot L' \cdot d + 3 \cdot a \cdot d + 3 \cdot b \cdot L' + 4 \cdot a \cdot b}{3 \cdot d^2 + 6 \cdot b \cdot d + 4 \cdot b^2 + \frac{12 \cdot I_b}{A_b} + \frac{12 \cdot I_b}{A_{hf} \cos^3 \theta}} \right) \tag{B.40}$$

where $A_{hf}$ is the area of the flange of the gusset.

vi. Calculate the value of the dimensionless parameter $\beta_{min.}$ given by:

$$\beta_{min.} = \frac{\frac{\left(M_{pl,Rd,b} + V_{pl,Rd,b} \cdot a\right)}{S_x} - 0{,}80 \cdot f_{uw,d}}{\frac{V_{pl,Rd,b} \cdot a}{S_x} + \frac{V_{pl,Rd,b}}{I_b \cdot \tan\theta} \cdot \left(\frac{d^2}{4} - \frac{I_b}{A_b}\right)} \tag{B.41}$$

where

$f_{uw,d}$ is the calculated value of the tensile weld resistance;

$S_x$ is the modulus of elasticity of the beam (with respect to its strong axis);

$d$ is the span of the beam;

$A_b$ e $I_b$ are, respectively, the area and moment of inertia of the beam.

vii. The dimensionless values $\beta$, calculated above, are compared. If $\beta < \beta_{min.}$, the gusset dimensions are sufficient and further local checks should be made according to step viii below. If $\beta < \beta_{min.}$, the rigidity of the gusset flange should be increased, either by increasing the gusset flange area $A_{hf}$ or by modifying the gusset geometry.

viii. Testing is carried out on the resistance and stability of the gusset flange:

(of resistance) $$A_{hf} \geq \frac{\beta \cdot V_{pl,Rd,b}}{f_{yhf,d} \cdot \mathrm{sen}\,\theta} \tag{B.42}$$

(of stability) $$\frac{b_{hf}}{t_{hf}} \leq 10 \cdot \sqrt{\frac{235}{f_{yhf,d}}} \tag{B.43}$$

where

$f_{yhf,d}$ is the calculated value of the yield strength of the gusset flange;

$b_{hf}$ y $t_{hw}$ are the flange overhang and gusset flange thickness, respectively.

ix. Testing is performed on the resistance and stability of the gusset web:

(of resistance) $$\tau_{hw} = \frac{a \cdot V_{pl,Rd,b}}{2 \cdot (1+\upsilon) \cdot I_b} \left[ \frac{L'}{2} - \frac{\beta}{\tan\theta} \left( \frac{d}{2} \right) + \frac{(1-\beta) \cdot a}{3} \right] \leq \frac{f_{yhw,d}}{\sqrt{3}} \tag{B.44}$$

(of stability) $$\frac{2 \cdot a \cdot \text{sen}\,\theta}{t_{hw}} \leq 33 \cdot \sqrt{\frac{235}{f_{yhw,d}}} \tag{B.45}$$

where

$f_{yhw,d}$ is the calculated value of the gusset web yield strength;

$t_{hw}$ is the web thickness;

$\nu$ is the Poisson's ratio of the steel.

x. The shear capacity of the web of the beam is tested in accordance with section **6.2.6** of Annex 26 of the Structural Code for a shear force, to be resisted by the web of the beam, given by:

$$V_{pl,Rd,bw} = (1-\beta) \cdot V_{pl,Rd,b} \tag{B.46}$$

where $\beta$ is given by expression (B.40).

xi. The transverse and web stiffeners of the beam are calculated to resist the concentrated force $\beta V_{pl,Rd,b}$ / tan $\theta$. The web stiffeners should have sufficient rigidity to resist, together with the beam web, the concentrated load (1 - $\beta$) $V_{pl,Rd,b}$. To prevent buckling, the width-to-thickness ratios of the stiffeners should be limited to 15.

xii. Welds with full penetration welds are detailed to connect the stiffeners to the flange of the beam. A weld bead of 8 mm on both sides is sufficient to connect the stiffeners to the web of the beam.

#### B.6.2.4.2 Connections with cover plates

(1) Connections with cover plates can reduce the stress in the flange welds of the beam and force plasticisation to occur at the ends of the cover plates.

(2) Steel cover plates can be used either on the lower flange of the beam only, or on the upper and lower flanges of the beam.

(3) The steel flashing plates should be rectangular in shape and should be laid with the direction of rolling parallel to that of the beam.

(4) Welded web welded joints and relatively thin and short cover plates should be preferred to bolted web joints and long and heavy cover plates.

(5) Long cover plates should not be used for beams with short spans and high shear forces.

(6) For connections by means of cover plates, a staged calculation procedure can be used as follows:

i. The dimensions of the cover plate are selected based on the size of the beam:

$$b_{cp} = b_{bf} \tag{B.47}$$

$$t_{cp} = 1{,}20 \cdot t_{bf} \tag{B.48}$$

$$l_{cp} = \frac{d_b}{2} \tag{B.49}$$

where

$b_{cp}$ is the width of the cover plate;

$t_{cp}$ is the thickness of the cover plate;

$b_{cf}$ is the width of the flange of the beam;

$t_{cf}$ is the thickness of the flange of the beam;

$l_{cp}$ is the length of the cover plate; and

$d_b$ is the edge of the beam.

ii. The plastic moment ($M_{pl,Rd,b}$) of the beam at the edge of the cover plate is calculated as in expression (B.7).

iii. Calculate the plastic shear force ($V_{pl,Rd,b}$) of the beam, according to step v in section **B.5.3.4(3)** for the distance, $L'$, between the plastic hinges in the beam:

$$L' = L - d_c - 2 \cdot l_{cp} \tag{B.50}$$

iv. Calculate the moment in the flange of the column, $M_{cf,Ed}$:

$$M_{cf,Ed} = M_{pl,Rd,b} + V_{pl,Rd,b} \cdot l_{cp} \tag{B.51}$$

v. Verify that the area of the plates, $A_{cp}$, satisfies the requirement:

$$\left[ Z_b + A_{cp} \cdot \left( d_b + t_{cp} \right) \right] \cdot f_{yd} \geq M_{cf,Sd} \tag{B.52}$$

where $f_{yd}$ is the calculated value of the yield strength of the joint cover plates.

vi. It is to be verified that the column-to-beam bending capacity ratio, *CBMR*, satisfies the condition:

$$CBMR = \frac{\sum Z_c \left( f_{yd,c} - f_a \right)}{\sum Z_b \cdot f_{yb} \cdot \left( \frac{L - d_c}{L - d_c - 2 \cdot L_{cp}} \right)} \geq 1,20 \qquad (B.53)$$

where

$Z_b$ and $Z_c$ are the plastic moduli of the beams and columns, respectively;

$f_{yb}$ is the yield strength of the beam steel, as defined in section **B.5.1(5)**; and

$f_{yd,c}$ is the calculated value of the yield strength of the column steel, as defined in section **B.6.2.1(3)**.

vii. The thickness of the continuity plates placed at the top and bottom flanges of the beam to stiffen the column web is determined. This thickness should not be less than that of the flange of the beam.

viii. Testing is carried out to ensure that the resistance and rigidity of the panel area is sufficient for the panel to remain elastic:

$$d_c \cdot t_{wc} \cdot \frac{f_{yw,d}}{\sqrt{3}} \geq \frac{\sum M_f}{d_b} \cdot \left( \frac{L}{L - d_c} \right) \cdot \left( \frac{H - d_b}{H} \right) \qquad (B.54)$$

where

$d_c$ is the web edge of the column;

$t_{wc}$ is the thickness of the column web, including reinforcement bars, if any;

$f_{yw,d}$ is the calculated value of the yield strength of the panel area; and

$H$ is the storey height of the portal frame.

ix. The welds between the joined parts, i.e. between the beam and the cover plates, between the beam and the cover plates and between the beam and the column, are dimensioned and detailed. The weld overlays should always use the same electrodes as the original electrodes or at least electrodes with similar mechanical properties.

### B.6.3 Connections of triangulations and seismic couplings

(1) The connections of triangulations and seismic couplings should be dimensioned taking into account the effects of post-buckling cyclic behaviour.

(2) Rigid connections should be preferred to nominally hinged connections (see section **5.2.2** of Annex 26 of the Structural Code).

(3) In order to improve the out-of-plane stability of the triangulated connection, the continuity of

beams and columns should not be interrupted.

(4) The axes of the triangulation and the beam should not intersect outside the seismic coupling.

(5) At connections between diagonal triangles and beams, the axes of these elements should intersect within the coupling or at its end.

(6) For the connection of a seismic coupling to a column at the flange face of the column, support plates should be used between the flange plates of the beam.

(7) The seismic adequacy of the beam-column connections may change the length of the seismic coupling. Therefore, the coupling should be tested after the repair strategy is adopted.

(8) Seismic couplings connected to the column should be short.

(9) Welded connections of a seismic coupling to the weak axis of a column should be avoided.

# Appendix C

# Recommendations for factory buildings

## C.1 Purpose and scope

(1) This appendix contains recommendations for the assessment and design of seismic retrofitting of masonry buildings in seismic regions.

(2) The recommendations of this appendix are applicable to lateral force resisting masonry elements such as concrete blocks or bricks, within an unreinforced, confined or reinforced masonry building system.

## C.2 Identification of the geometry, construction details and materials

### C.2.1 General considerations

(1) The following aspects should be carefully examined:

i. Type of masonry (e.g. clay, concrete, hollow, solid, etc.).

ii. Physical condition of the masonry elements and the presence of any degradation.

iii. Configuration of masonry elements and their connections, and the continuity of load paths between lateral resistance elements.

iv. Properties of the materials of which the masonry members are composed and the quality of the joints.

v. The presence and attachment of cladding, the presence of non-structural components, the spacing between partitions.

vi. Information on adjacent buildings that may interact with the building under consideration.

### C.2.2 Geometry

(1) The data collected should include the following items:

i. Size and location of all shear walls, including their height, length and thickness.

ii. Dimensions of masonry elements.

iii. Location and size of openings in the walls (doors, windows).

iv. Distribution of gravity loads in load-bearing walls.

### C.2.3 Construction details

(1) The data collected should include the following items:

i. Classification of walls as unreinforced, confined or reinforced.

ii. Presence and quality of mortar.

iii. For reinforced masonry walls, the amount of horizontal and vertical passive reinforcement.

iv. For masonry walls of more than one layer (rubble-filled masonry walls), identification of the number of layers, respective distances, and location of tie keys, where present.

v. For cast masonry, assessment of the type, quality and location of concrete paste.

vi. Determination of the type and condition of mortar and mortar joints; examination of resistance, erosion and hardness of mortar; identification of defects such as cracks, internal voids, weak components and deterioration of mortar.

vii. Identification of type and condition of knots between perpendicular walls.

viii. Identification of the type and condition of knots between walls and slabs or decks.

ix. Identification and location of horizontal cracks in tendons, vertical cracks in flanges and masonry, and diagonal cracks near openings or apertures.

x. Examination of overhangs (deviations in verticality) of walls and separation of external leaves or other elements such as parapets or chimneys.

### C.2.4 Materials

(1) Non-destructive tests can be performed to quantify and confirm the uniformity of the quality of the construction and the presence and degree of deterioration. The following types of tests can be performed:

i. Mechanical or ultrasonic pulse velocity measurement, to detect variations in density and modulus of masonry materials, as well as to detect the presence of cracks and discontinuities.

ii. The impact echo, to confirm if the reinforced walls have concrete slurry.

iii. Radiographs and pachometers, where appropriate, to confirm the location of passive reinforcement.

(2) Additional tests may be carried out to improve the level of confidence in the material properties of the masonry, or to assess the condition of the masonry. The possible tests are:

i. Sclerometer tests, to assess the surface hardness of external masonry walls.

ii. Testing with a hydraulic flat jack, to measure the *in situ* shear resistance of the masonry. This test can be carried out in conjunction with flat jacks by applying a known vertical load to the masonry parts to be tested.

iii. Flat hydraulic jack tests, to measure the *in situ* vertical compressive stress resisted by the masonry. This test provides information such as gravity load distribution, bending stresses in walls, and stresses in masonry veneered walls compressed by the surrounding concrete frames.

iv. Diagonal compression tests, to estimate the shear resistance and shear modulus of the masonry.

v. Large scale destructive tests on specific areas or elements, to increase the level of confidence in the overall structural properties or to provide particular information such as out-of-plane resistance, joint and void behaviour, mid-plane resistance and deformation capacity.

## C.3 Methods of analysis

### C.3.1 General considerations

(1) In setting up the model for the analysis, the stiffness of the walls should be assessed taking into account both flexural and shear flexibility, using the cracked stiffness. In the absence of more accurate assessments, both stiffness contributions can be taken as equal to half of their respective uncracked values.

(2) The masonry spandrels forming lintels and parapets in the openings of a wall can be introduced into the model as coupling beams between two walls.

### C.3.2 Linear methods: Static and multi-modal

(1) These methods are applicable under the following conditions, which are in addition to the general conditions of section **4.4.2(1)**.

i. The walls resisting lateral loads are regularly arranged in both horizontal directions.

ii. Walls are continuous over their full height.

iii. The floors have sufficient rigidity along their median plane and are sufficiently connected to the perimeter walls to assume that they can act as rigid diaphragms distributing the inertia forces between the vertical elements.

iv. Floor slabs on both sides of a common wall are at the same height.

v. For each slab, the ratio of the in-plane lateral stiffnesses of the stiffest wall to the weakest primary seismic-resisting wall, evaluated taking into account the existence of voids, is not greater than 2.5.

vi. The masonry interspaces introduced into the model as coupling beams are formed by pieces or blocks suitably interconnected to those of the adjacent walls, or are provided with linking keys.

### C.3.3 Non-linear methods: Static and dynamic

(1) These methods should be applied when the conditions of section **C.3.2** are not met.

(2) The capacity is defined in terms of deck displacement. The ultimate displacement capacity is assimilated to the displacement of the deck for which the total lateral resistance (base shear) has

decreased to 80 % of the peak resistance of the structure, due to the progressive damage and failure of the resisting elements of the horizontal loads.

(3) The demand, to be compared with the capacity, is the displacement of the deck corresponding to the displacement predicted in section **4.4.4.4** of this Annex and in section **4.3.3.4.2.6(1)** of Annex 1 for the seismic action considered.

NOTE Appendix B of Annex 1 gives a procedure for the determination of the predicted displacement from the elastic response spectrum.

## C.4 Capacity models for assessment

### C.4.1 Models for overall assessment

#### C.4.1.1 Near Collapse (NC) limit state

(1) The assessment criteria given in terms of global response measures can be applied only when the analysis is non-linear.

(2) The overall capacity in the near collapse (NC) limit state may be taken equal to the ultimate displacement capacity defined in section **C.3.3(2)**.

#### C.4.1.2 Significant Damage (SD) limit state

(1) Section **C.4.1.1(1)** applies.

(2) The overall capacity in the significant damage (SD) limit state can be taken equal to 3/4 of the ultimate displacement capacity as defined in section **C.3.3(2)**.

#### C.4.1.3 Damage Limitation (DL) limit state

(1) If a linear analysis is performed, the criterion for the overall assessment is defined in terms of the shear stress at the base in the horizontal direction of seismic action. The capacity can be taken equal to the sum of the shear capacities of the individual walls, to the extent that these capacities are controlled by the bending (see section **C.4.2.1(1)**) or by the shear stress (see section **C.4.3.1(1)**) in the horizontal direction of seismic action. The demand is the estimate of the maximum base shear in that direction from the linear analysis.

(2) If a non-linear analysis is carried out, the capacity for the global assessment is defined as the yield stress (force and displacement at the elastic limit) of the idealised perfect elasto-plastic force/displacement relationship of the equivalent system with a single degree of freedom.

NOTE Appendix B of Annex 1 gives a procedure for the determination of the force and displacement at the yield stress of the idealised perfect elasto-plastic force/displacement relationship of the equivalent system with a single degree of freedom.

### C.4.2 Elements subjected to normal and bending stresses

#### C.4.2.1 Significant Damage (SD) limit state

(1) The capacity of an unreinforced masonry wall is controlled by bending if the value of its shear capacity given in section **C.4.2.1(3)** is less than the value given in section **C.4.3.1(3)**.

(2) The capacity of an unreinforced masonry wall controlled by bending can be expressed in terms of relative displacement and taken equal to $0.008 \cdot H_0/D$ for primary earthquake-resisting walls, and equal to $0.012 \cdot H_0/D$ for secondary earthquake-resisting walls, where:

$D$ is the horizontal dimension in the plane of the wall (length);

$H_0$ is the distance between the section where the bending capacity is obtained and the inflection point.

(3) The shear capacity of an unreinforced masonry wall controlled by bending under an axial load $N$, can be taken equal to:

$$V_f = \frac{D N}{2 H_0} \left( 1 - 1{,}15\, \nu_d \right) \tag{C.1}$$

where

$D$ y $H_0$ are defined in **(2)**;

$\nu_d = N/(Dtf_d)$ is the normalised axial load, with $f_d = f_m/CF_m$ (where $f_m$ is the average compressive strength obtained from *in situ* tests and additional sources of information, $CF_m$ is the confidence factor applicable to the factory given in Table 3.1 for the appropriate level of knowledge m), $t$ is the thickness of the wall.

#### C.4.2.2 Near Collapse (NC) limit state

(1) Section **C.4.2.1(1)** and **(3)** applies.

(2) The capacity of a masonry wall controlled by bending may be expressed in terms of relative displacement and taken equal to 4/3 of the values in section **C.4.2.1(2)**.

#### C.4.2.3 Damage Limitation (DL) limit state

(1) Section **C.4.2.1(1)** applies.

(2) The capacity of an unreinforced masonry wall controlled by bending may be taken to be equal to the shear capacity given in section **C.4.2.1(3)**.

### C.4.3 Elements under shear stress

#### C.4.3.1 Significant Damage (SD) limit state

(1) The capacity of an unreinforced masonry wall is controlled by shear if the value of its shear capacity given in section **C.4.3.1(3)** is less than or equal to the value given in section **C.4.2.1(3)**.

(2) The shear-controlled capacity of an unreinforced masonry wall may be expressed in terms of relative displacement and taken equal to 0.004 for primary earthquake-resisting walls and 0.006 for secondary earthquake-resisting walls.

(3) The shear capacity of an unreinforced masonry wall, controlled by the shear force under an axial load $N$, can be taken to be equal to:

$$V_f = f_{vd} D't \tag{C.2}$$

where

$D$ is the length of the compressed area of the wall;

$t$ is the thickness of the wall; and

$f_{vd}$ is the shear resistance of the masonry taking into account the presence of the vertical load: = $f_{vm0} + 0.4 \cdot N/D't \leq 0.065 f_m$, where $f_{vm0}$ is the average shear resistance in the absence of vertical load and $f_m$ is the average compressive resistance, both values obtained from *in situ* tests and from additional sources of information, and divided by the confidence coefficients, defined in section **3.5(1)** and Table 3.1, for the level of knowledge achieved. For primary earthquake-resisting walls, the resistances of both materials are further divided by the partial coefficient for the masonry according to section **9.6** of Annex 1.

#### C.4.3.2 Near Collapse (NC) limit state

(1) Section **C.4.3.1(1)** and **(3)** applies.

(2) The shear-controlled capacity of an unreinforced masonry wall may be expressed in terms of relative displacement and taken as 4/3 of the values given in section **C.4.3.1(2)**.

#### C.4.3.3 Damage Limitation (DL) limit state

(1) Section **C.4.3.1(1)** applies.

(2) The capacity of an unreinforced masonry wall controlled by shear stress may be taken as the shear capacity given in section **C.4.3.1(3)**.

## C.5 Structural interventions

### C.5.1 Repair and reinforcement techniques

#### C.5.1.1 Repair of cracks

(1) If the crack opening is relatively small (e.g. less than 10 mm) and the wall thickness is relatively small, the cracks can be sealed with mortar.

(2) If the crack width is small but the wall thickness is not, grout injections (flowable cement mortar) should be used. Where possible, a non-shrink mortar should be used. An epoxy mortar can be used instead for hairline cracks.

(3) If the cracks are relatively wide (e.g. more than 10 mm), the damaged area should be reconstructed using bricks or elongated (stitching) stones. Otherwise, dovetail staples, metal plates or polymeric mesh should be used to join the two sides of the crack. Gaps should be filled with cement mortar of appropriate flowability.

(4) When the tendons are reasonably level, the resistance of the walls to vertical cracking can be considerably improved by placing small diameter stranded steel cables or polymeric mesh strips in the tendons.

(5) For the repair of large diagonal cracks, vertical concrete ribs can be formed in the irregular grooves made in the masonry wall, usually on both sides. Such ribs should be reinforced with closed abutments and longitudinal reinforcement. The stranded steel cables specified in **(4)** should cross the concrete ribs. Alternatively, polymeric mesh may be used covering one or both sides of the masonry walls, combined with appropriate mortar and rendering.

### C.5.1.2 Repair and reinforcement of wall intersections

(1) To improve the connection between concurrent walls, use should be made of bricks or cross-faced stones. The connection can be made more effective in a number of ways:

i. By the construction of a reinforced concrete tie beam;

ii. By incorporating steel plates or mesh into the tendons;

iii. By inserting inclined steel reinforcement in holes drilled in the masonry and subsequently filled with a mortar of suitable fluidity;

iv. By post-tensioning.

### C.5.1.3 Reinforcement and stiffening of horizontal diaphragms

(1) Wooden floors can be reinforced and stiffened against in-plane distortions in several ways:

i. By nailing an additional layer (perpendicular or oblique) of timber decking over the existing decking.

ii. By forming a reinforced concrete overlay with welded steel mesh. The concrete overlay should have a flush connection with the timber floor and the reinforcement should be anchored to the walls.

iii. Placing a mesh in the two diagonal directions of steel tie spanners anchored to the beams and the perimeter walls.

(2) Roof trusses should be braced and anchored to the supporting walls. A horizontal diaphragm should be created at the bottom chords of the trusses (e.g. by bracing elements providing adequate resistance to prevent distortions in both directions).

### C.5.1.4 Tie beams

(1) If existing tie beams between walls and floors are damaged, they should be repaired or rebuilt. If there are no tie beams in the original building structure, they should be added.

### C.5.1.5 Reinforcement of buildings by steel tie rods

(1) The addition of steel tie rods, longitudinally or transverse to walls, external to or within wall penetrations, is an efficient way of connecting walls and improving the overall performance of masonry buildings.

(2) Post-tensioned tie rods can be used to improve the resistance of walls to tensile stresses.

### C.5.1.6 Reinforcement of rubble-filled masonry walls (multi-leaf walls)

(1) The rubble infill can be reinforced with flowable mortar if the penetration of this mortar is satisfactory. If the adhesion of the mortar to the rubble is poor, the addition of this mortar should be supplemented by steel reinforcement inserted through the backfill and anchored to the outer leaves of the wall.

### C.5.1.7 Reinforcement of walls by reinforced concrete liners or steel sections

(1) The concrete should be applied by the gunning method and the liners should be reinforced with welded steel mesh or steel bars.

(2) The liners may be applied to one side of the wall only, or preferably to both. The two layers of the liner applied to opposite sides of the wall should be connected by transverse ties through the masonry. Liners applied to only one face should be connected to the masonry by chases.

(3) Steel sections may be used in a similar manner, provided they are properly connected to both faces of the wall or to one face only.

### C.5.1.8 Reinforcement of walls by means of polymeric mesh liners

(1) Polymer mesh can be used to reinforce existing and new masonry elements. In the case of existing elements, the mesh should be connected to the masonry walls on one or both sides and anchored to the perpendicular walls. In the case of new elements, the intervention may require the additional insertion of meshes in the horizontal mortar layers (tendels) between bricks. The pastes to cover the polymer meshes should be ductile, preferably lime-cement mixtures with fibre-based reinforcement.

# ANNEX 4

# Design of seismically resistant structures

# Silos, tanks and pipes

## 1 General considerations

### 1.1 Field of application

(1) The scope of the Seismic Resistance Standard is defined in section **1.1.1** of Annex 1, and the scope of this Annex is defined in this section. The other parts of the Seismic Resistance Standard are listed in section **1.1.3** of Annex 1.

(2) This Annex specifies the principles and rules of application for the seismic design of the structural aspects of facilities consisting of above-ground or buried piping systems, storage tanks of different types and uses, as well as separate elements, such as for example elevated water tanks having a specific purpose or groups of silos containing granular materials, etc.

(3) This Annex includes the rules and supplementary criteria necessary for the seismic design of these structures, without restrictions due to size, structural type or other functional characteristics. It also provides detailed evaluation methods and testing rules for the seismic-resistant design of some types of tanks and silos.

(4) The provisions of this Annex may not be sufficient for installations posing a high risk to the public or to the environment, which may need to satisfy specific additional requirements. This Annex is also insufficient for construction works involving unusual structural elements which, in order to ensure seismic protection, require special measures and studies. For these two cases, general principles are provided in this Annex, but not detailed rules of application.

(5) Although large diameter pipelines are within the scope of this Annex, the corresponding calculation criteria do not apply to other apparently similar installations, such as tunnels and large underground excavations.

(6) The facilities covered by this Annex are often characterised by their nature as essential infrastructure and therefore require concepts, models and methods that may differ substantially from those in common use for more common structures. In addition, the response and stability of silos and tanks subjected to strong seismic actions may involve rather complex interaction phenomena between the soil-structure and the stored material (whether fluid or granular), not easily reducible to simplified calculation methods. A similar challenge can be to calculate a pipeline system crossing areas with poor and possibly unstable soils. For these reasons, the organisation of this Annex is somewhat different from other parts of the Seismic Resistance Standard. This Annex is generally limited to basic principles and methodological approaches.

NOTE Appendices A and B provide detailed analysis methods for certain typical situations that go beyond the basic principles and methodological approaches. These appendices are not of a regulatory nature.

(7) In the formulation and implementation of the general requirements, a distinction has been made between independent structures and redundant systems, through the choice of factors of importance and/or through the definition of specific testing criteria.

(8) If surface pipelines are provided with seismic protection by means of seismic isolation devices between the pipelines and their supports (especially in piles), then Annex 2 applies, as appropriate. For the design of tanks, silos or individual installations or components of piping systems with seismic isolation the relevant provisions of Annex 1 apply.

## 1.2 Standards for reference and consultation

(1) The provisions of section **1.2** of Annex 1 apply.

## 1.3 Assumptions

(1) , as specified in section **1.3** of Annex 1, applies.

## 1.4 International system of units (I.S.)

(1) The provisions of section **1.4** of Annex 1 apply.

## 1.5 Terms and definitions

### 1.5.1 General considerations

(1) For the purposes of this Annex, the following definitions apply.

### 1.5.2 Common terms

(1) The terms and definitions given in section **1.4** of Annex 18 of the Structural Code apply.

### 1.5.3 Other terms used in this Seismic Resistance Standard

(1) For the purposes of this Annex, the terms given in section **1.5.2** of Annex 1 apply.

### 1.5.4 Other terms used in this Annex

**Independent structure:**
Structure whose structural and functional behaviour during and after an earthquake is not influenced by that of other structures, and whose failure consequences are only related to its functional requirements.

**Equivalent surface:**
In a silo, a horizontal plane delimiting the same volume of stored solid as the actual surface.

## 1.6 Symbols

(1) For the purposes of this Annex, the following symbols apply:

$A_{Ed}$ calculated value of the seismic action ( = $\gamma_I A_{Ek}$)

$A_{Ek}$ seismic action characteristic value for the reference return period

$b$ horizontal dimension of a silo parallel to the horizontal component of the seismic action

$d_c$ inner diameter of a circular silo

$d_g$ design displacement of the soil, according to section **3.2.2.4(1)** of Annex 1, used in expression (4.1)

$g$ gravitational acceleration

$h_b$ total height of the silo, measured from a flat bottom or hopper outlet to the equivalent surface of the stored contents

$q$ behaviour coefficient

$r$ radius of the circular silo, silo compartment, bin or pipe

$r_s^*$ geometrical quantity defined for silos by the expression (3.5) as $r_s^* = \min(H, Br_s/2)$

$t$ thickness

$x$ vertical distance between a point on a silo wall and the flat bottom of the silo or the apex of a conical or pyramid hopper

$x$ distance between the anchorage point of the piping and the point of attachment to the tank

$z$ downward vertical co-ordinate in a silo, measured from the equivalent surface of the stored content

$\alpha(z)$ ratio of the acceleration response of the silo at the height of interest, z, to the acceleration of gravity

$\beta$ angle of inclination of the hopper wall of a silo, measured with the vertical, or the angle of inclination of steepest slope with the vertical of a pyramidal hopper wall

$\gamma$ apparent specific gravity of the granular material in a silo, see section **3.1(2)**.

$\gamma_I$ importance factor

$\gamma_p$ amplification factor for the forces transmitted by the pipes to the anchorage zone in the wall of a vessel, for the zone to be calculated to remain elastic, see section **4.5.1.3(3)**.

$\Delta$ minimum value of the imposed relative displacement between the first anchorage point of the pipe and the container, given by the expression (4.1)

$\Delta_{ph,s}$ additional normal pressure on the silo wall due to the response of the granular solid to the horizontal component of the seismic action

$\Delta_{ph,so}$ reference pressure on the silo walls given in section **3.3(8)**, expression (3.6)

$\theta$ angle ($0° \leq \theta < 360°$) between the radius passing through the point of interest of the wall of a circular silo and the direction of the horizontal component of the seismic action

$\lambda$ base shear correction factor, obtained by the lateral force analysis method of section **4.3.3.2.2(1)** of Annex 1

$\nu$ reduction factor for the effects of seismic action corresponding to the damage limitation condition

$\xi$ viscous damping ratio (in per cent)

$\psi_{2,i}$ combination coefficient for the quasi-permanent value of a variable action *i*

$\psi_{E,i}$ combination ratio for a variable action *i*, used to determine the effects of the calculated seismic action.

# 2 General principles and rules of application

## 2.1 Security requirements

### 2.1.1 General considerations

(1) This Annex deals with structures which may differ widely in such basic aspects as:

- the nature and quantity of the contents and their associated potential hazard;

- the functional requirements during and after an earthquake;

- environmental conditions.

(2) Depending on the specific combinations of the above characteristics, different formulations of the general requirements will be appropriate. For consistency with the general framework of the Structural Code, the principle of limit states is maintained, with a suitably adapted definition.

### 2.1.2 Ultimate limit state

(1) The ultimate limit state for which a system should be tested is defined as the state corresponding to structural failure. In some circumstances it is possible that, after an acceptable number of repairs, a system may partially recover the operational capability that was lost when the ultimate limit state was exceeded.

NOTE 1 Such circumstances are those defined by the competent authority or, in the absence of the competent authority, by the owner.

(2) For particular elements of a network, as well as for independent structures whose complete collapse would have serious consequences, the ultimate limit state is defined as the state prior to structural collapse which, despite its severity, would prevent brittle rupture and allow controlled release of its contents. When the failure of the aforementioned elements does not lead to serious consequences, the ultimate limit state can be defined as the state corresponding to the total collapse of the structure.

(3) The calculated seismic action for which the ultimate limit state may not be exceeded must be established on the basis of the direct and indirect consequences of structural failure.

(4) The calculated seismic action, $A_{Ed}$, should be expressed as a function of:

a) the reference seismic action, $A_{Ek}$, associated with a reference exceedance probability, $P_{NCR}$, of 10 % in 50 years or a reference return period, $T_{NCR}$ = 475 years; and

b) the significance factor $\gamma_I$, in order to take into account the different degrees of reliability (see Annex 18 of the Structural Code).

$$A_{Ed} = \gamma_I \, A_{Ek} \tag{2.1}$$

NOTE See section **2.1** and section **3.2.1(3)** of Annex 1.

(5) The ability of structural systems to resist in the non-linear range the seismic actions corresponding to the ultimate limit state generally allows them to be designed to resist seismic forces

lower than those corresponding to a linear elastic response.

(6) In order to avoid an explicit inelastic analysis in the design, the capacity of structural systems to dissipate energy, mainly through the ductile behaviour of their elements and/or other mechanisms, can be taken into account by performing an elastic-linear analysis based on a reduced response spectrum with respect to the elastic one, called the ‘design spectrum’. This reduction is carried out by introducing a behavioural coefficient $q$, which is an approximation of the ratio between the seismic forces that the structure would experience if its response were fully elastic with a viscous damping of 5 %, and the seismic forces that can be included in the calculation with a conventional elastic-linear analysis model, still ensuring a satisfactory performance of the structural system in the ultimate limit state.

(7) In the relevant chapters of this Annex, values for the behavioural coefficient $q$, which also take into account the influence of viscous damping other than 5 %, are given for the different types of constructions included in its scope of application.

### 2.1.3 Damage Limitation limit state

(1) Depending on the characteristics and purpose of the structure under consideration, it may be necessary to satisfy a state of damage limitation that meets one or both of the following levels of performance:

- 'integrity';

- ‘minimum operating level’.

(2) In order to satisfy the 'integrity' requirement, the system under consideration, including a given set of accessory elements integrated into it, must remain fully serviceable and watertight under the relevant seismic action.

(3) In order to satisfy the ‘minimum operating level’ requirement, the extent and amount of damage to the system under consideration, including some of its components, should be limited so that, after testing and damage control work has been carried out, the capability of the system can be restored to a predefined operating level.

(4) The seismic action for which this limit state cannot be exceeded must have an annual probability of exceedance whose value is established on the basis of the following:

- the consequences of loss of function and/or leakage of contents, and

- losses due to reduction of system capacity and necessary repairs.

(5) The seismic action for which the "damage limitation" state cannot be exceeded must have a probability of exceedance, $P_{DLR}$, of 10% in 10 years or a return period, $T_{DLR}$ = 95 years. To obtain the seismic action for testing the damage limitation state, and in the absence of more precise data, the reduction factor applied to the calculated seismic action according to section **2.2(3)** can be used.

### 2.1.4 Degrees of reliability

(1) The level of protection of piping networks and stand-alone structures, whether tanks or silos, must be proportionate to the number of persons exposed to the risk and the economic losses resulting

from failure to achieve their expected performance.

(2) Reliability levels should be achieved by appropriately adjusting the values of the annual probability of exceedance of the calculated seismic action.

(3) This adjustment shall be carried out by classifying the structures into different importance classes and applying to the reference seismic action an importance factor $\gamma_I$, as defined in section **2.1.2(4)** of this Annex and in section **2.1(3)** of Annex 1, the value of which depends on the importance class. The specific values of the factor $\gamma_I$, necessary to modify the seismic action so that it corresponds to an earthquake with a chosen return period, depend strictly on the hazard of each zone; However, for the purposes of this Seismic Resistance Standard, these $\gamma_I$ factors take a constant value for each importance class. The value of the importance factor $\gamma_I = 1.0$ is associated with the seismic action with the reference return period indicated in section **2.1.2(4)**.

NOTE For the dependence of the value of $\gamma_I$ see, in Annex 1, the note to section **2.1(4)** and section **3.2.1(3)**.

(4) For structures within the scope of this Annex it is appropriate to consider four different classes of significance, depending on the potential risk of loss of life due to the failure of a particular structure and depending on the economic and social consequences of a failure. Within each class of significance a further classification may be made, depending on the use and contents of the installation and its implications for public safety.

NOTE Importance classes I, II and III/IV correspond approximately to consequence classes CC1, CC2 and CC3, respectively, as defined in Appendix B of Annex 18 of the Structural Code.

(5) Class I corresponds to situations where the risk to human life is low and the economic and social consequences in case of failure are small or negligible.

(6) Class II corresponds to situations with a medium risk to human life and with local economic and social consequences in case of failure.

(7) Class III corresponds to situations with a high risk to life and with significant economic and social consequences in case of failure.

(8) Class IV corresponds to situations with an exceptional risk to human life and with extreme economic and social consequences in case of failure.

NOTE The values of $\gamma_I$ may be different for different seismic zones, depending on the seismic hazard conditions (see the note of section **2.1(4)** of Annex 1) and the public safety considerations detailed in section **2.1.4**. However, for the purposes of this Seismic Resistance Standard, the $\gamma_I$ factors take a constant value for each importance class. The value of $\gamma_I$ for importance class II is, by definition, equal to 1.0.

The $\gamma_I$ value for the different importance classes is:

- Importance class I: $\gamma_I = 0.8$
- Importance class II: $\gamma_I = 1$
- Importance class III: $\gamma_I = 1.3$
- Importance class IV: $\gamma_I = 1.6$

(9) A pipeline system crossing a large geographic area will typically encounter a wide variety of seismic and soil hazard conditions. In addition, along a pipeline system there may be sub-systems, which can be both appurtenant installations (tanks, storage tanks, etc.) and pipeline installations (valves, pumps, etc.). In these circumstances, critical sections of the pipeline (e.g. less redundant parts

of the system) and critical components (pumps, compressors, control equipment, etc.) must be sized to ensure higher reliability in relation to earthquakes. The other components, which are less essential and can withstand a certain acceptable level of damage, do not need to be sized with such severe criteria.

#### 2.1.5 Element reliability versus system reliability

(1) The reliability requirements specified in section **2.1.4** must be applied to the complete system under consideration, whether it consists of a single element or a group of elements connected in different ways to perform the required functions.

(2) Although it is beyond the scope of this annex to formally approach the analysis of the reliability of a system, the designer shall explicitly consider the role of the various elements in ensuring the continued operation of the system, especially where it is not redundant. In the case of very complex systems, the calculation should be based on sensitivity analysis.

(3) Elements of the network, or of a network structure, which are critical to the failure of the system, should be provided with an additional margin of protection, commensurate with the consequences of failure. In the absence of previous experience, these critical elements should be studied experimentally in order to test whether the design assumptions are acceptable.

(4) In the absence of more rigorous analysis, the margin of additional protection for critical elements can be obtained by assigning them a reliability class (expressed in terms of importance class) one level higher than is appropriate for the system as a whole. Capacity calculation rules can also be used for the calculation of critical elements of a network structure, taking into account the actual resistance of elements not considered as critical.

#### 2.1.6 Conceptual project

(1) Even when the overall seismic response is specified to be elastic, structural elements must be designed and detailed constructively for some local ductility and made of ductile materials.

(2) For the mitigation of seismic effects, the following general aspects should be taken into account in the design of a network or a freestanding structure:

- functional redundancy of systems;

- absence of interaction between mechanical and electrical components and structural elements;
- easy access for inspection, maintenance and repair of damage;

- quality control of components.

(3) In order to avoid the propagation of damage in functionally redundant systems due to the structural interdependence of components, the relevant parts must be functionally isolated.

(4) In the case of major installations which are vulnerable to earthquakes, and for which the repair of damage is difficult or time-consuming, spare parts or pre-assemblies must be available.

### 2.2 Seismic action

(1) The seismic action to be used in the calculation of silos, tanks and pipelines must be that defined

in section **3.2** of Annex 1 in the different equivalent forms of elastic response spectra, depending on the location (section **3.2.2** of Annex 1), and a representation as a function of time (section **3.2.3.1** of Annex 1). Additional provisions for spatial variation of terrain movement for buried pipelines are included in Chapter 6.

(2) Section **2.1.2(4)** specifies the seismic action for which the ultimate limit state is to be tested. If the determination of the effects of seismic action is based on a linear elastic analysis with a behavioural coefficient *q* greater than 1, according to section **3.2.2.5(2)** of Annex 1, then the elastic analysis design spectrum according to section **3.2.2.5** (see also section **2.1.2(6)** of this Annex) is to be used.

(3) A reduction factor $\nu$ may be applied to the calculated seismic action corresponding to the ultimate limit state to take into account the shorter return period of the seismic action associated with the damage limitation state, as mentioned in section **2.1(1)** of Annex 1. The value of the reduction factor $\nu$ may also depend on the importance class of the structure. Its use implicitly assumes that the elastic response spectrum of the seismic action under which the damage limitation state is to be tested has the same shape as the elastic response spectrum of the calculated seismic action corresponding to the ultimate limit state according to section **2.1(1)** and section **3.2.1(3)** of Annex 1 (see sections **3.2.2.1(2)** and **4.4.3.2** of Annex 1).

NOTE The recommended values for $\nu$ are: $\nu = 0.5$ for significance classes I and II and $\nu = 0.4$ for significance classes III and IV. Other values can be obtained from special zoning studies.

## 2.3 Analysis

### 2.3.1 Methods of analysis

(1) For structures within the scope of this Annex, the effects of seismic actions shall be determined on the basis of a linear behaviour of the structures and the surrounding soils.

(2) Non-linear methods of analysis may be used to obtain the effects of seismic actions, especially in those particular cases where, because of the nature of the problem, the non-linear behaviour of the structure or of the surrounding soils must be taken into account, or where an elastic solution is economically unfeasible.

(3) The analysis to assess the effects of seismic action corresponding to the damage limitation state must be elastic-linear, using the elastic spectra defined in sections **3.2.2.2** and **3.2.2.3** of Annex 1, multiplied by the reduction factor $\nu$ given in section **2.2(3)**. For the viscous damping of the elastic spectra, the weighted average value of the damping of the different materials/elements shall be taken into account, in accordance with section **2.3.5** and Annex 1, section **3.2.2.2(3)**.

(4) To evaluate the effects of the seismic action corresponding to the ultimate limit state, elastic-linear analysis can be applied, in accordance with section **2.1.2(6)** and section **3.2.2.5** of Annex 1, using the calculation spectra specified in section **3.2.2.5** of Annex 1 for a damping rate of 5 %. The behavioural coefficient *q* is used which takes into account the ability of the structure to dissipate energy, mainly through the ductile behaviour of its elements and/or other mechanisms, as well as the influence of viscous damping other than 5% (see also section **2.1.2(6)**).

(5) Unless otherwise indicated for particular types of structures in the relevant parts of this Annex, the types of analysis which may be applied are those indicated in section **4.3.3** of Annex 1, namely:

a) the "lateral force" (elastic-linear) method of analysis (see section **4.3.3.2** of Annex 1);

b) the "modal response spectra" (elastic-linear) method of analysis (see section **4.3.3.3** of Annex 1);

c) the non-linear static (incremental thrust) analysis (see section **4.3.3.4.2** of Annex 1);

d) non-linear (dynamic) analysis in the time domain (see section **4.3.3.4.3** of Annex 1).

(6) Section **4.3.1(1)**, **(2)**, **(6)**, **(7)** and **(9)**, and section **4.3.3.1(5)** and **(6)**, of Annex 1 must be applied for the modelling and analysis of the different types of structures covered by this Annex.

(7) The elastic-linear "lateral force" analysis shall be carried out in accordance with section **4.3.3.2.1(1)**, section **4.3.3.2.2(1)** and **(2)** (with $\lambda = 1.0$) and section **4.3.3.2.3(2)** of Annex 1. This method is appropriate for structures that respond to each of the components of the seismic action approximately as a single degree of freedom system: elevated and rigid (i.e. concrete) tanks or silos on relatively flexible and nearly massless supports.

(8) Modal analysis by response spectra shall be carried out in accordance with sections **4.3.3.3.1(2)**, **(3)** and **(4)** and section **4.3.3.3.2** of Annex 1. This method is appropriate for structures whose response is significantly affected by contributions from modes other than those of a system with only one degree of freedom in each principal direction.

(9) The non-linear analysis, whether static (incremental thrust) or dynamic (time domain), shall satisfy section **4.3.3.4.1** of Annex 1.

(10) The non-linear static (incremental thrust) analysis shall be carried out in accordance with section **4.3.3.4.2.2(1)** and sections **4.3.3.4.2.3** and **4.3.3.4.2.6** of Annex 1.

(11) The non-linear dynamic analysis (in the time domain) shall satisfy section **4.3.3.4.3** of Annex 1.

(12) For the analysis of tanks, silos and isolated installations or elements of piping systems with isolation at their base, the corresponding provisions of Annex 1 apply.

(13) For the analysis of above ground pipelines having seismic isolation devices between them and their supports, the corresponding provisions of Annex 2 apply.

### 2.3.2 Interaction with the soil

(1) The effects of soil-structure interaction should be dealt with in accordance with Chapter **6** of

Annex 5.

NOTE Further information on methods for considering soil-structure interaction is given in both Appendix A and Appendix C of Annex 6.

### 2.3.3 Damping

#### 2.3.3.1 Damping of the structure

(1) If damping values are not obtained from specific data, the following damping ratio values shall be used for the linear analysis:

a) damage limitation state: the values specified in section **4.1.3(1)** of Annex 2;

b) ultimate limit state: $\xi = 5$ %.

#### 2.3.3.2 Content buffering

(1) The value $\xi = 0.5$ % may be adopted as the damping ratio for water and other liquids, unless a different value is determined.

NOTE Reference is made to Appendix A for additional information concerning the damping ratios of liquids.

(2) For granular materials an appropriate value of the damping ratio should be used. In the absence of more specific information, the value $\xi = 10$ % may be adopted.

#### 2.3.3.3 Foundation damping

(1) The damping of materials varies with the nature of the soil and the intensity of the shock. Where more precise indications are not available, the values given in Table 4.1 of Annex 5 should be used.

(2) Radial damping depends on the direction of movement (horizontal translation, vertical translation, oscillation, etc.), the geometry of the foundation and the soil stratification and morphology. The values adopted in the analysis must be compatible with the actual site conditions and must be justified by reference to recognised theoretical and/or experimental results. The radiation damping values used in the analysis must not exceed a maximum value, $\xi_{max.}$ = 30 %.

NOTE Guidelines for the choice and use of damping values associated with different foundation movements are given in Annex 6.

#### 2.3.3.4 Weighted damping

(1) The overall mean damping of the system as a whole shall take into account the contributions of the various materials/elements to the damping.

NOTE Methods for taking into account the contribution of individual materials/elements to the overall mean damping of the system are given in the note to section **4.1.3(1)** of Annex 2, and in Appendix B to Annex 6.

### 2.4 Behavioural coefficients

(1) The behavioural coefficient $q$ for the damage limitation state must be equal to 1.0.

NOTE For structures covered by this Annex no significant energy dissipation is to be expected in the damage limitation state.

(2) In testing for the ultimate limit state, only coefficients $q$ greater than 1.5 are permitted, provided that the sources of energy dissipation are explicitly identified and quantified and the ability of the structure to take advantage of them is demonstrated by appropriate construction details.

(3) If seismic protection is provided by seismic isolation, the value of the behavioural coefficient for the ultimate limit state must not exceed $q$ = 1.5, except as provided in **(4)**.

(4) If seismic protection is provided by seismic isolation, a value equal to 1 must be taken for $q$ for the following cases:

a) for the design of the substructure (i.e. the elements below the plane of isolation).

b) for the part of the response of the superstructure of the tanks due to the convective contribution of the liquid response (convective wave).

c) for the calculation of the isolators.

### 2.5 Safety testing

#### 2.5.1 General considerations

(1) The safety testing must be carried out for the limit states defined in section **2.1**, following the specific provisions given in sections **3.5**, **4.5**, **5.6** and **6.5**.

(2) If the thickness of the plates is increased to take into account the effects of future corrosion, testing shall be carried out at both the unincreased and increased thickness. The analysis may be based on a single plate thickness value.

#### 2.5.2 Combinations of seismic action with other actions

(1) The calculated value $E_d$ of the effects of the actions in the calculated seismic situation must be determined in accordance with section **6.4.3.4** of Annex 18 of the Structural Code. In addition, the inertial effects of the calculated seismic action are to be evaluated in accordance with section **3.2.4(2)** of Annex 1.

(2) In fully or partially buried tanks, the permanent loads include, in addition to the weight of the structure, the weight of the overburden and possible permanent external pressures due to groundwater.

(3) The combination coefficients $\psi_{2,i}$ (for the quasi-permanent value of the variable action i) must be those given in the specific regulations in force, or, failing that, in the specific technical documents which the author of the project, under their responsibility, considers most appropriate. The combination coefficients $\psi_{Ei}$, introduced in section **3.2.4(2)** of Annex 1 for the calculation of the effects of seismic actions, must be taken equal to $\psi_{2,i}$ multiplied by a factor $\varphi$, whose value is $\varphi = 1$, when the silos, tanks or pipes are full, and $\varphi = 0$, when they are empty.

(4) The effects of content must be taken into account in variable loads for two filling levels: empty or full. In multi-cell silos or compartmentalised tanks, the different likely distributions of full and empty cells should be considered, according to the operating rules of the installation. As a minimum, calculation scenarios in which all cells are empty or full should be considered. In the calculated seismic situation only symmetrical filling loads of silos or silo cells are to be considered.

## 3 Specific principles and application rules for silos

### 3.1 Introduction

(1) A distinction has been made between:

- Silos supported directly on the terrain or on the foundation; and
- Elevated silos, supported on a bottom plate lining (skirt) extended to the ground or on a series of columns, braced or not.

The main effect of seismic action in silos supported on the terrain is the stresses induced in the wall sheeting due to the response of the silo contents (see section **3.3(3)** and **(5 to 12)** for additional normal pressures on the wall sheeting). Most important in the seismic design of elevated silos is the supporting structure, its ductility and energy dissipation capacity (see section **3.4(4 and 5)**).

(2) The determination of the characteristics of the granular material stored in the silo, including its specific weight, $\gamma$, must be carried out on the basis of tests, in accordance with the specific regulations in force or, failing that, with the specific technical documents that the author of the project, under his responsibility, considers most appropriate.

NOTE Alternatively, for the stored materials listed in table E.1 of Annex E of Standard UNE-EN 1991-4, the values of the said table may be adopted, considering the definitions of the wall surfaces given in table 4.1 of the said standard. In particular, for $\gamma$, the upper characteristic value of the specific gravity $\gamma_u$ specified in the above Table E.1 shall be adopted.

(3) Under seismic conditions, the pressure exerted by the granular material on the walls, hopper and bottom may exceed the value corresponding to the condition without seismic action. For calculation purposes, this increased pressure is considered to be due only to the inertia forces acting on the stored material due to seismic action (see section **3.3(5)**).

(4) The equivalent area of the stored contents considered in the calculation must be, in the calculated seismic situation, compatible with the value of the combination coefficients $\psi_{Ei}$ used in the calculation of the effects of seismic actions according to section **2.5.2(3)**.

## 3.2 Combination of the components of the terrain movement

(1) In axisymmetric silos, or parts thereof, a single horizontal component of the seismic action acting together with the vertical component can be considered. In all other cases, the silos must be calculated for the simultaneous action of the two horizontal components and the vertical component of the seismic action.

(2) When the structural response is evaluated for each component of the seismic action separately, section **4.3.3.5.2(4)** of Annex 1 may be applied to determine the worst effect resulting from the application of the simultaneous components.

(3) If expressions (4.20), (4.21), (4.22) of section **4.3.3.5.2(4)** of Annex 1 are applied to calculate the effects of the action of the simultaneous components, the sign of the effect of the action due to each of the components shall be taken as the most unfavourable for the particular action effect considered.

(4) If the analysis is carried out for all three components of the seismic action simultaneously, using a spatial model of the structure, the maximum values of the overall response for the combined action of the horizontal and vertical components obtained from the analysis should be used for the structural testing.

## 3.3 Silo analysis

(1) The silo analysis shall be carried out in accordance with sections **2.3** and **3.3**.

(2) The model to be used to determine the effects of seismic action must accurately reproduce the rigidity, mass and geometric properties of the containing structure. It must also take into account the response of the contained granular material and the effects of any interaction with the foundation soil. The modelling and analysis of the steel silos must be in accordance with the specific regulations in force or, failing that, with the specific technical documents that the author of the project, under his responsibility, considers most appropriate.

(3) Unless adequate justification is given for a non-linear calculation, a silo must be calculated considering the elastic behaviour of its sheet and its supporting structure, if any.

(4) Unless more precise assessments are made, the overall seismic response and the effects of seismic action on the supporting structure can be calculated assuming that the contained grains move along with the silo shell, modelled with their effective mass concentrated at their centre of gravity and with their rotational inertia relative to it. Unless a more precise assessment is made, the silo contents can be considered to have an effective mass equal to 80 % of its total mass.

(5) Unless the mechanical properties and dynamic response of the granular material are explicitly and precisely taken into account in the analysis (e.g. by using finite elements to model the mechanical properties and dynamic response of the granular solid), the effect on the sheet of the response of the granular material to the horizontal component of the seismic action can be represented by an additional normal pressure on the wall, $\Delta_{ph,s}$ (positive for compression) specified in **(6 to 10)**, with conditions **(11 and 12)**. This additional pressure shall be applied only on the part of the wall that is in contact with the stored contents, i.e. up to the equivalent surface of the stored contents, for the calculated seismic situation (see section **3.1(4)**).

(6) In circular silos (or silo compartments) the additional normal pressure exerted on the wall can be taken as:

$$\Delta_{ph,s} = \Delta_{ph,so} \cos\theta \tag{3.1}$$

where

$\Delta_{ph,so}$ is the reference pressure, see **(8)**;

$\theta$ is the angle ($0° \leq \theta < 360°$) between the radius passing through the point of interest in the wall and the direction of the horizontal component of the seismic action.

(7) In rectangular silos (or silo compartments) the additional normal pressure exerted on the wall by a horizontal component of the seismic action parallel or normal to the silo walls can be taken as:

On the wall normal to the horizontal component of the seismic action located at the furthest position according to the direction of arrival of the seismic action:

$$\Delta_{ph,s} = \Delta_{ph,so} \tag{3.2}$$

On the wall normal to the horizontal component of the seismic action at the nearest position in the direction of arrival of the seismic action:

$$\Delta_{ph,s} = -\Delta_{ph,so} \tag{3.3}$$

On the walls parallel to the horizontal component of the seismic action:

$$\Delta_{ph,s} = 0 \tag{3.4}$$

(8) At points on the silo wall at a vertical distance $x$ from the flat bottom or the apex of a conical or pyramidal hopper, the reference pressure $\Delta_{ph,so}$ can be taken to be:

$$\Delta_{ph,so} = \alpha(z)\gamma \min(r_s^*; 3x) \tag{3.5}$$

where

$\alpha(z)$ is the ratio of the acceleration response of the silo at a vertical distance z from the equivalent surface of the stored contents to the acceleration of gravity;

$\gamma$ is the apparent specific gravity of the granular material for the calculated seismic situation (see section **3.1(1)**) and

$r_s^*$ is defined as:

$$r_s^* = \min(h_b, d_c/2) \tag{3.6}$$

where

$h_b$ is the total height of the silo, measured between a flat bottom or hopper outlet and the equivalent surface of the stored contents, and

$d_c$ is the inside dimension of the silo parallel to the horizontal component of the seismic action (for circular silos or compartments it is the inside diameter, $d_c$, and for rectangular silos or compartments it is the inside horizontal dimension $b$ parallel to the horizontal component of the seismic action).

(9) Expression (3.6) applies to the vertical walls of the silo. Within the height of the hopper, the reference pressure can be taken as $\Delta_{ph,so}$:

$$\Delta_{ph,so} = \alpha(z)\gamma \min(r_s^*; 3x)/\cos\beta \tag{3.7}$$

where

$\beta$ is the angle of inclination of the hopper wall, measured with the vertical, or the angle of the steepest slope line with the vertical of the wall of a pyramidal hopper.

(10) If only the value of the acceleration response at the centre of gravity of the granular material is available (see, for example, section **2.3.1(4)** and **(7)**), the corresponding ratio of the acceleration response to the acceleration of gravity can be used as $\alpha(z)$ in expression (3.7).

(11) The sum of the static pressure of the granular material on the silo wall and the effect of seismic action, $\Delta_{ph,s}$, must not be less than zero at any point on the silo wall.

(12) If at any point on the silo wall the sum of

- $\Delta_{ph,s}$ given in **(6)** to **(10)** and expressions (3.1) to (3.3); and

- the static pressure of the granular material on the wall

is negative (implying a net suction on the wall), then neither **(6)** nor **(7)** can be applied. In this case, the additional normal pressures on the wall, $\Delta_{ph,s}$, must be redistributed to ensure that, when the static pressure of the granular material on the wall is added to them, the result is not negative at any point, and the same resultant force on the same horizontal plane as the values of $\Delta_{ph,s}$ given in **(6)** or **(7)** is retained.

## 3.4 Behavioural coefficients

(1) Silos without base isolation must be calculated according to one of the following concepts (see sections **5.2.1**, **6.1.2** and **7.1.2** of Annex 1):

a) low dissipative structural behaviour;

b) dissipative structural behaviour.

(2) For concept a), the effects of seismic action can be calculated on the basis of a global elastic analysis, without taking into account significant non-linear behaviour of the material. When using the design spectrum defined in section **3.2.2.5** of Annex **1**, the behavioural coefficient $q$ may take a maximum value of 1.5. The calculation according to concept a) is called the calculation for Low ductility class (DCL). For Low ductility class (DCL), the selection of materials, the evaluation of the resistance and the construction details of the elements and connections shall be carried out as specified in Chapters **5** to **7** of Annex 1.

(3) Silos supported directly on the terrain or on the foundation shall be calculated in accordance with a) and **(2)**.

(4) Concept b) can be applied to elevated silos. According to this concept, the ability of the parts of the supporting structure to resist seismic actions beyond their elastic range (their dissipation zones) is taken into account. Supporting structures calculated with this concept shall belong, depending on their structural material, to the Medium ductility class (DCM) or High ductility class (DCH), defined and described in Chapters **5** to **7** of Annex 1. They shall comply with the requirements specified therein concerning the structural type, the materials and the dimensioning and construction details of the parts or joints for ductility. When using the design spectrum for the linear elastic analysis defined in section **3.2.2.5** of Annex 1, a behavioural coefficient $q$ greater than 1.5 may be taken. The value of $q$ depends on the chosen ductility class (DCM or DCH).

(5) Given the low redundancy, the high axial forces due to the weight of the silo contents and the absence of non-structural elements contributing to seismic resistance and energy dissipation, the energy dissipation capacity of the structural types normally used to support overhead silos is, in general, lower than that of a similar type of structure when used in buildings. Therefore, for elevated silos designed according to concept b), the upper bounds of the $q$ coefficient values are defined according to the $q$ coefficients specified in Chapters **5** to **7** of Annex 1, for the chosen ductility class (DCM or DCH), as follows:

- For silos supported on a bottom plate lining (skirt), the upper limits of the $q$ coefficient values defined for inverted pendulum structures in Chapters **5** to **7** of Annex 1 may be used, provided that the skirt is designed and detailed to ensure dissipative behaviour. If the skirt is not structurally detailed for dissipative behaviour, it shall be calculated in accordance with concept (a) and **(2)**.

- For silos supported by moment resisting trusses or braced trusses, and for in situ concrete silos supported by continuous concrete walls up to the foundation, the upper bounds of the coefficients $q$ are those defined in Chapters **5** to **7** of Annex 1 for the corresponding structural system, multiplied by a factor equal to 0.7 to take into account the irregularity in elevation.

## 3.5 Testing

### 3.5.1 Damage Limitation limit state

(1) For the calculated seismic situation corresponding to the damage limitation limit state, it must be checked that the silo structure satisfies the appropriate testing of the serviceability limit state required in the Structural Code and in the specific regulations in force (or, failing this, in the specific technical documents that the author of the project, under their responsibility, considers most appropriate).

NOTE For steel silos, adequate reliability with regard to the occurrence of elastic or inelastic buckling phenomena in the calculated seismic situation corresponding to the damage limitation state is considered to be assured if the checks corresponding to these phenomena are satisfied for the ultimate limit state in the calculated seismic situation.

### 3.5.2 Ultimate limit state

#### 3.5.2.1 Overall stability

(1) In the calculated seismic situation there must be no overturning or failure of the bearing capacity of the soil. The shear resistance at the contact surface between the base of the structure and the foundation must be assessed taking into account the effects of the vertical component of the seismic action. A certain degree of slippage is permissible, provided that it is shown that the implications of slippage for the joints between parts of the structure and between the structure and pipes have been taken into account in the calculations and testing (see also section **5.4.1.1(7)** of Annex 5).

(2) In order for the silos to be allowed to rise out of the ground, this must be taken into account in the calculation and subsequent testing of the structure, piping and foundations (e.g. when assessing the overall stability).

#### 3.5.2.2 Sheets

(1) The maximum effects of the action (membrane forces and bending moments, circumferential or meridional, and membrane shear forces) produced for the calculated seismic situation must be less than or equal to the resistance of the wall sheets evaluated for the persistent or transient design situations. This includes all types of failure modes:

(a) For steel sheets:

- yielding (plastic collapse),

- shear buckling, or

- simultaneous vertical compression and transverse tension buckling ('elephant's foot' type of failure), etc.

(b) For concrete slabs:

- the ultimate limit state in bending with axial force,

- the ultimate limit state in shear for in-plane or radial shear, etc.

(2) The calculation of the resistances and Testing must be carried out in accordance with Annexes 19, 22 and 25 of the Structural Code and, taking into account that these structures are outside the scope of application of the Structural Code, additionally, with the specific regulations in force or, failing that, with the specific technical documents that the author of the project, under their responsibility, considers most appropriate.

#### 3.5.2.3 Anchors

(1) In general, anchorage systems must be designed to remain elastic in the calculated seismic situation. However, they must also be provided with sufficient ductility to avoid brittle failure. The connection of the anchorage elements to the structure and its foundation must have an over-resistance factor of not less than 1.25 with respect to the resistance of the anchorage elements.

(2) If the anchorage system is part of the dissipation mechanisms, testing shall be carried out to ensure that it has the necessary ductility capacity.

#### 3.5.2.4 Foundations

(1) The foundations must be checked in accordance with section **5.4** of Annex 5 and with the specific regulations in force or, failing the latter, with the specific technical documents which the author of the project, under their responsibility, considers most appropriate.

(2) The effects of seismic action for testing the foundations and foundation elements must be obtained in accordance with section **5.3.1** of Annex 5 and sections **4.4.2.6** and **5.8** of Annex 1.

# 4 Specific principles and application rules for deposits

## 4.1 Compliance criteria

### 4.1.1 General considerations

(1) The general requirements specified in section **2.1** are considered to be met if the containers comply, in addition to the tests specified in section **4.4**, with the additional measures specified in section **4.5**.

(2) The compliance criteria and application rules given in this chapter do not fully cover steel containers with a floating jacket.

NOTE Special care should be taken to avoid damage to the wall sheeting due to local effects of the impact of the floating jacket. Such effects may cause fires in tanks with combustible contents.

### 4.1.2 Damage Limitation limit state

(1) In order to satisfy the 'integrity' requirements under the seismic action corresponding to the damage limitation state:

– The overall system should be tested for leak tightness;

– tanks should be provided with an adequate safeguard or headroom for the maximum vertical displacement of the liquid surface to prevent damage to the jacket due to the pressure exerted by the wave due to the convective response of the liquid (convective wave), or to prevent, in tanks without a rigid jacket, undesirable effects of liquid spillage;

- testing should be carried out to ensure that hydraulic systems which are part of, or connected to, the container are capable of accommodating stresses and distortions due to relative displacements between containers or between containers and the ground without impairing their functions.

(2) In order to satisfy the 'minimum performance level' requirement under the seismic action corresponding to the damage limitation state, it should be tested that if local buckling occurs, it is reversible and does not trigger a collapse.

### 4.1.3 Ultimate limit state

(1) For the calculated seismic situation the following conditions have to be tested:

– The overall stability of the tank is to be tested according to section **4.4.2.4** of Annex 1. The overall stability refers to the behaviour as a rigid solid and can be lost by sliding or overturning. According to section **5.4.1.1(7)** of Annex 5, a certain degree of displacement is permissible, provided that it is tolerated by the piping system and that the tank is not anchored to the foundation.

– Inelastic behaviour is restricted to well-defined parts of the tanks in accordance with the provisions of this Seismic Resistance Standard,

– The ultimate deformations of the materials are not exceeded.

– The nature and extent of buckling phenomena in the wall sheeting are controlled according to the appropriate testing.

– Hydraulic systems which are part of, or connected to, a tank are designed to prevent loss of tank contents in the event of failure of any of their components.

## 4.2 Combination of the components of the terrain movement

(1) Deposits must comply with section **3.2(1)**.

(2) Deposits must comply with section **3.2(2)**.

(3) Deposits must comply with section **3.2(3)**.

## 4.3 Calculation methods

### 4.3.1 General considerations

(1) The model used to determine the seismic effects must adequately reproduce the stiffness, rigidity, strength, damping, mass and geometric properties of the containing structure, and must also take into account the hydrodynamic response of the contained liquid as well as, where necessary, the effects of interaction with the foundation soil.

NOTE Soil-liquid-structure interaction parameters can significantly influence the natural frequencies and radiation damping of the soil. As the shear wave velocity of the soil increases, the vibration behaviour changes from a horizontal vibration combined with soil-influenced rocking to the typical vibration mode of a tank on rigid soil. For heavily stressed tank structures or hazardous products, a global (three-dimensional) calculation may be necessary.

(2) In general, tanks should be calculated assuming a linear elastic response. In some cases a non-linear response may be justified by appropriate methods of analysis.

NOTE Information on seismic computational methods for commonly shaped tanks is given in Appendix A.

(3) Where appropriate, the possible interaction between different tanks due to connecting pipes should be considered.

### 4.3.2 Hydrodynamic effects

(1) A rational method based on the solution of the hydrodynamic equations, with appropriate boundary conditions, should be used to evaluate the seismic response of the storage system.

(2) In particular, the calculation should consider, in an appropriate manner and where appropriate, the following:

- the convective and impulsive components of the liquid motion;
- the deformation of the reservoir sheet due to hydrodynamic pressures and stresses from interaction with the impulsive component;
- the deformability of the foundation soil, with consequent modification of the response;
- the effects of a floating cover, if applicable.

(3) To evaluate the dynamic response under seismic actions, it can be generally assumed that the fluid is incompressible.

(4) The determination of the maximum hydrodynamic pressures induced by horizontal and vertical excitations requires, in principle, the use of a non-linear dynamic (time domain) calculation. Simplified methods allowing a straightforward application of the response spectrum analysis can be

used, provided that appropriate conservative rules are adopted for the combination of the maximum modal contributions.

NOTE Appendix A provides information on acceptable methods for combining the maximum modal contributions in response spectrum analysis. It also provides expressions for calculating the wave height due to the convective response of the liquid.

## 4.4 Behavioural coefficients

(1) Deposits of other types than those mentioned in **(4)** and **(5)** must be calculated either for an elastic response (considering for over-resistance a value of the behavioural coefficient $q$ not greater than 1.5) or, in duly justified cases, for an inelastic response (see section **2.3.1(2)**), provided that the inelastic response is shown to be acceptable.

(2) The energy dissipation corresponding to the chosen value of $q$, as well as the necessary ductility provided by the ductile design, must be adequately justified.

(3) The convective part of the fluid response (convective wave) should always be evaluated on the basis of an elastic response (i.e. with $q$ = 1.0) and associated spectra (see sections **3.2.2.2** and **3.2.2.3** of Annex 1).

(4) The behavioural coefficients specified in section **3.4** should also be applied to the part of the response of the elevated reservoirs not due to the convective component. For this part, the rules specified in section **3.4(4)** for silos supported on a bottom plate liner (skirt) also apply to tanks raised on a single support.

(5) Steel tanks (except those insulated at the base) having a vertical axis and resting directly on the terrain or on the foundation, may be calculated with a behavioural coefficient $q$ greater than 1.5, respecting the following:

- that $q$ = 1.0 is adopted for the convective response of the liquid;

- the tank or its foundation is designed to allow for some uplift and/or sliding;

- that the location of plastic deformations in the wall sheet, in the bottom plate or at their intersection is avoided.

Under these conditions, and in the absence of a more accurate assessment of the inelastic response, a behavioural coefficient $q$ less than or equal to the following values can be taken:

- 2.0 for non-anchored tanks, provided that they are constructively designed and detailed to ensure dissipative behaviour consistent with this coefficient, in accordance with the specific regulations in force or, failing this, with the specific technical documents that the author of the project, under their responsibility, considers most appropriate, especially with regard to the thickness of the bottom plate, which must be less than the thickness of the lower part of the sheet.

- 2.5 for tanks with ductile anchorages specially calculated to allow an increase in the anchorage length without breakage equal to $R/200$, where $R$ is the radius of the tank.

## 4.5 Testing

### 4.5.1 Damage Limitation limit state

#### 4.5.1.1 General considerations

(1) Under the seismic action corresponding to the damage limitation state, the tank structure must satisfy, as appropriate, the testing of the serviceability limit state specified in the specific regulations in force or, failing that, in the specific technical documents that the designer, under their responsibility, considers most appropriate.

#### 4.5.1.2 Sheets

##### 4.5.1.2.1 Reinforced and prestressed concrete slabs

(1) The crack width under seismic action corresponding to the damage limitation state shall be tested with the limit values specified in section **4.4.2** of Annex 19 of the Structural Code, taking into account the appropriate environmental exposure class and the sensitivity of the steel to corrosion.

(2) In the case of concrete tanks fitted with an inner lining, the width of transient cracks in the concrete shall not exceed a value which could induce a local deformation in the lining exceeding its ultimate uniform elongation by 50 %.

##### 4.5.1.2.2 Steel sheets

(1) Steel containers shall comply with section **3.5.1(2)**.

#### 4.5.1.3 Piping

(1) Unless special requirements are specified for active elements mounted in the lines, such as valves or pumps, pipelines need not be tested for damage limitation status.

(2) If pipelines and tank(s) are supported on different types of foundations, relative displacements due to differential seismic movements of the terrain should be taken into account.

(3) The area of the tank in which the pipes are embedded shall be calculated to remain elastic to the forces transmitted by the pipes, amplified by a factor $\gamma_{p1}$ = 1.3.

### 4.5.2 Ultimate limit state

#### 4.5.2.1 Stability

(1) Deposits must comply with section **3.5.2.1(1)**.

(2) Deposits must comply with section **3.5.2.1(2)**.

#### 4.5.2.2 Sheets

(1) Deposits must comply with section **3.5.2.2(1)**.

NOTE Appendix A provides information on the breaking strength capacity of the sheet, controlled by different failure modes.

#### 4.5.2.3 Piping

(1) If no reliable data are available, or no more precise calculations are made, a relative displacement between the first anchorage point of the pipe and the tank in the worst case direction shall be assumed, with a minimum value equal to:

$$\Delta' = \frac{X}{X_0} d_g \tag{4.1}$$

where

$X$ = distance between the anchorage point of the pipe and the junction point with the container (in metres);

$x_o$ = 500 m; and

$d_g$ = calculation displacement of the terrain given in section **3.2.2.4(1)** of Annex 1.

(2) It must be tested that, for the calculated seismic situation, including the relative displacement assumed in **(1)**, the plastification is restricted to the pipe and does not extend to its joints with the tank, even when an over-resistance factor $\gamma_{p2}$ = 1.3 is taken into account in the design resistance of the pipe.

(3) The design resistance of piping elements must be assessed for both persistent and transient design situations.

#### 4.5.2.4 Anchors

(1) Deposits must comply with section **3.5.2.3(1)**.

#### 4.5.2.5 Laying foundations

(1) Deposits must comply with section **3.5.2.4(1)**.

(2) Deposits must comply with section **3.5.2.4(2)**.

## 4.6 Accompanying measures.

### 4.6.1 Containment of spills

(1) Tanks, in isolation or in groups, which have been designed to control or prevent leakage in order to prevent fires, explosions and the release of toxic materials, must be provided with a spill containment system (i.e. surrounded by a trench and/or an embankment).

(2) If groups of tanks are constructed, spill containment can be provided either for each individual tank or for the entire group. If the consequences associated with the potential failure of the spill

containment system are considered to be severe, then spill containment should be used for each tank.

(3) Spill containment systems should be designed to maintain their integrity (absence of leakage) under the calculated seismic action corresponding to the ultimate limit state of the system they surround.

#### 4.6.2 Wave due to convective response of the liquid (convective wave)

(1) In the absence of explicit justifications (see section **4.1.2(1)**), a clearance space, not less than the height calculated for waves due to the convective part of the liquid response, must be provided.

NOTE Appendix A provides information for determining the height of the convective wave.

(2) If the contents are toxic or may cause damage to piping or undermine the foundation when discharged, then a clearance height equal to or greater than the calculated height of the convective wave must be provided.

(3) A lower clearance height than that calculated for the convective wave may be sufficient, provided that the deck is designed to withstand the associated pressure increase or that a spillway is provided to control spillage.

(4) Damping devices, such as gratings or vertical partitions, may be used to reduce the effect of the convective part of the liquid response.

#### 4.6.3 Interaction with pipelines

(1) Pipelines should be designed to minimise the unfavourable effects of interaction between tanks and between tanks and other structures.

## 5 Specific principles and application rules for surface pipelines

### 5.1 General considerations

(1) The objective of this chapter is to provide principles and rules of application for the structural seismic design of above ground piping systems. It can also be used as a basis for assessing the resistance of existing surface pipelines and for estimating any necessary reinforcement.

(2) The seismic design of a surface pipeline includes the determination of the location and characteristics of its supports, in order to limit the unit deformation of the pipeline components and the loads applied to the equipment located in the pipeline, such as valves, tanks, pumps or instrumentation. These limits are not defined in this Annex and shall be provided by the owner of the installation or by the manufacturer of the equipment.

(3) Piping systems typically include several associated facilities, such as pumping stations, operations centres, maintenance stations, etc., each housing different types of mechanical and electrical equipment. Since these facilities have a considerable influence on the continued operation of the system, they need to be adequately considered in the seismic design in order that the overall reliability requirements are met. However, the explicit treatment of these installations is outside the scope of this Annex. In particular, the seismic design of mechanical and electrical equipment requires additional specific criteria that are beyond the scope of this Seismic Resistance Standard.

NOTE See section **1.1(8)** for seismic protection of individual installations or piping system components by means of seismic isolation.

(4) For the formulation of the general requirements to be fulfilled, as well as for their application, the following piping systems are to be distinguished:

- independent pipelines;

- redundant networks.

(5) A pipeline is to be considered as independent if its behaviour during and after an earthquake is not influenced by that of other pipelines and if the consequences of its failure only affect its functional requirements.

## 5.2 Security requirements

### 5.2.1 Damage Limitation limit state

(1) Piping systems must be constructed in such a way that they continue to maintain their functional capability to supply as an overall service system after the seismic action corresponding to the 'minimum operating level' (see section **2.1.3**) even with considerable local damage.

(2) An overall pipe deformation less than or equal to 1.5 times the deformation corresponding to the yield stress is acceptable, provided that there is no risk of buckling and that the loads applied to the active equipment, such as valves, pumps, etc., are within their operating range.

### 5.2.2 Ultimate limit state

(1) The main safety hazards directly associated with the rupture of a pipeline during an earthquake are explosion and fire, especially in gas pipelines. In order to establish the seismic action level corresponding to the ultimate limit state, the distance from the site and the number of inhabitants exposed to the consequences of a rupture should be taken into account.

(2) In order to define the acceptable risk in pipeline systems located in environmentally sensitive areas, the damage caused to the environment by pipeline ruptures should also be taken into account.

## 5.3 Seismic action

### 5.3.1 General considerations

(1) The following types of direct and indirect seismic risks are relevant for the seismic design of surface piping systems:

– The movement due to the inertia of the pipelines induced by the seismic movement applied to their supports.

– differential movement of pipeline supports.

(2) For the differential movement of the supports, two different situations can exist:

– For supports that are located directly on the terrain, significant differential movement is only possible if there are terrain failures and/or permanent deformations.

– For supports that are located on different structures, the seismic response of the structure can create differential movements in the pipe.

### 5.3.2 Seismic action for inertial motions

(1) The quantification of the horizontal components of the seismic action must be done from the response spectrum (or a compatible time domain representation) as specified in section **3.2.2** of Annex 1.

(2) Only the three translational components of the seismic action should be taken into account (i.e. the rotational components may be ignored).

### 5.3.3 Differential motion

(1) Where the pipeline is supported directly on the terrain, differential movement may be ignored, except where ground failure or permanent deformation is likely to occur. In this case the extent of movement shall be assessed by appropriate techniques.

(2) Where the pipe is supported on different structures, their differential movement should be defined from the analysis of their seismic responses or by simplified envelope approximations.

## 5.4 Methods of analysis

### 5.4.1 Modelling

(1) The pipe model must be able to represent the rigidity, damping and mass properties, as well as the dynamic degrees of freedom of the system, explicitly considering the following aspects, as appropriate:

- the flexibility of the foundation soil and the foundation system;

- the mass of the fluid inside the pipe;

- the dynamic characteristics of the supporting structures;

- the type of joint between the pipe and the supporting structure;

- the joints along the pipe and between the supports.

### 5.4.2 Analysis

(1) The surface pipes can be calculated by means of modal analysis using response spectra, with the corresponding calculation spectrum given in section **3.2.2.5** of Annex 1, combining the modal responses in accordance with section **4.3.3.3.2** of Annex 1.

NOTE Additional rules concerning the combination of modal responses, in particular for the use of the Full Quadratic Combination, are given in section **4.2.1.3** of Annex 2.

(2) Time domain analysis with spectrum compatible accelerograms can also be applied according to section **3.2.3** of Annex 1.
(3) The calculation may also be made using the 'lateral force' (elastic-linear) method of analysis, provided that the acceleration value applied is justified. A value equal to 1.5 times the maximum of the spectrum applied to the support is acceptable. If appropriate, the principles and rules of application

specified in section **4.3.3.2** of Annex 1 may be applied.

(4) The seismic action shall be applied separately along two orthogonal directions (transverse and longitudinal, for rectilinear pipes). The maximum combined response shall be obtained in accordance with section **4.3.3.5.1(2)** and **(3)** of Annex 1.

(5) The spatial variation of the movement must be taken into account whenever the length of the pipe exceeds 600 m or when there are geological discontinuities or marked topographical changes.

(6) To take account of the spatial variation of movement, the principles and rules of application of section **3.3** of Annex 2 may be used.

NOTE Additional models to account for spatial variation of movement are presented in Appendix D of Annex 2.

### 5.5 Behavioural coefficients

(1) The dissipative capacity of a surface pipe is limited, if it exists, to that of its support structure, since it is difficult and undesirable for energy to be dissipated in the supported pipes, except for welded steel pipes. On the other hand, as the shape and materials used for the supports vary greatly, it is not feasible to establish behavioural coefficient values that are generally applicable.

(2) For pipe support structures without seismic isolation, appropriate values for $q$ can be taken from Annexes 1 and 2, based on their specific plan layout, materials and construction details.

(3) Welded steel pipes, provided their thickness is sufficient, exhibit significant deformation and dissipation capacity. For testing of pipes which are not seismically isolated and whose ratio of radius to thickness ($r/t$) is less than 50, a behavioural coefficient $q$ equal to 3.0 may be taken. If the ratio $r/t$ is less than 100, $q$ can be taken to be equal to 2.0. For the other cases, the $q$ value for the pipe calculation cannot be higher than 1.5.

(4) For testing of the supports, the effects of seismic action obtained from the analysis shall be multiplied by $(1+q)/2$, where $q$ is the behavioural coefficient of the pipe used in its calculation.

### 5.6 Testing

(1) The load-induced effect on the load-bearing elements (columns, portal frames, etc.) in the calculated seismic situation must be less than or equal to the design resistance assessed for persistent or transient design situations.

(2) For the worst-case combination of axial and rotational deflections, due to the application of the seismic action corresponding to the 'minimum operating level' requirement, testing must be carried out to ensure that the joints are not damaged to such an extent that they become leak-tight.

## 6 Specific principles and application rules for buried pipelines

### 6.1 General considerations

(1) The objective of this chapter is to provide principles and application rules for the seismic design and for the assessment of the seismic resistance of buried piping systems.

(2) Although distinctions can be made between different piping systems, such as independent pipelines or redundant systems, for practical reasons a pipeline is considered here to be independent

if its mechanical behaviour during and after an earthquake is not influenced by that of other pipelines, and if the consequences of its possible failure only affect its functional requirements.

(3) Networks are often too large and complex to be treated as a whole, so it is possible and appropriate to differentiate several separate networks within an overall network. This differentiation can be made by separating the larger scale part of the system (e.g. regional distribution) and the smaller scale part (e.g. urban distribution), or by distinguishing between the different functions performed by the same system.

(4) As an example of **(3)**, an urban water distribution system can be separated into a network to supply fire hydrants and a second network for private users. This separation would facilitate the allocation of different levels of reliability for the two systems. It should be emphasised that the separation is linked to functions and is therefore not necessarily physical: two different networks may have several elements in common.

(5) The design of pipe networks implies reliability requirements and design methods complementary to those set out in this Seismic Resistance Standard.

## 6.2 Security requirements

### 6.2.1 Damage Limitation limit state

(1) Buried pipelines must be designed and constructed in such a way that, even with considerable local damage, they maintain their integrity or part of their functional supply capacity after the earthquake corresponding to the damage limitation state (see section **2.1.3**).

### 6.2.2 Ultimate limit state

(1) Buried pipelines must comply with section **5.2.2(1)**.

(2) Buried pipelines must comply with section **5.2.2(2)**.

## 6.3 Seismic action

### 6.3.1 General considerations

(1) The seismic design of buried piping systems must take into account the following types of direct and indirect seismic hazards:

a) seismic waves propagating in solid ground producing different ground motions at different points on the surface and spatial deformation fields in the ground;

b) earthquake-induced permanent deformations, such as earthquake fault displacements, landslides and liquefaction-induced ground displacements.

(2) All general requirements for damage limitation state and ultimate limit state must be satisfied for all types of hazards specified in **(1)**.

(3) It can generally be assumed that for the types of hazard type (b) specified in **(1)**, the satisfaction of the ultimate limit state requirements implies also the fulfilment of the damage limitation requirements, so that a single testing can be carried out.

(4) The fact that piping systems cross or extend over large geographical areas and the need to connect certain sites does not always allow the best choice to be made in relation to the nature of the soil that supports them. Moreover, it may be impossible to avoid crossing potentially active faults or to avoid soils susceptible to liquefaction, as well as areas that could be affected by seismically induced landslides and large permanent differential deformations of the terrain.

(5) The situation described in **(4)** is clearly at odds with that of other structures, for which the very possibility of their construction requires that the probability of any kind of terrain failure is negligible. Consequently, in most cases, the occurrence of the type (b) risks specified in **(1)** cannot be excluded. In order to model this risk, assumptions based on experience and available data should be used.

#### 6.3.2 Seismic action for inertial motions

(1) The quantification of the seismic vibration components shall be in accordance with section **2.2**.

#### 6.3.3 Seismic wave modelling

(1) A model must be established for the seismic waves with which to obtain the unit deformations and the curvatures of the terrain that affect the pipe.

NOTE Methods for some cases of calculation of unit strains and pipe curvatures are given in Appendix B, with some simplifying assumptions.

(2) The vibrations of the terrain produced by earthquakes are due to the sum of shear waves, compression waves, Love waves and Rayleigh waves, the propagation velocities of which depend on their path through materials of higher or lower velocity. The different particle motions associated with these types of waves mean that the unit deformations and pipe curvatures also depend on the angle of incidence of the waves. As a rule of thumb, it can be assumed that sites close to the epicentre are more affected by shear and compression waves (internal waves), while for sites further away Love and Rayleigh waves (surface waves) tend to be more significant.

(3) The choice of the types of waves to be taken into account and their corresponding propagation velocities must be based on geophysical considerations.

#### 6.3.4 Permanent movements of the terrain

(1) The ground rupture modes associated with earthquake ground motions, whether due to fault surface movements or landslides, are generally complex, with considerable displacement variations depending on the geology and soil type, as well as the magnitude and duration of the earthquake. The possibility of occurrence of these phenomena at particular sites must be established and appropriate models defined (see Annex 5).

### 6.4 Calculation methods (wave passage)

(1) It is acceptable to take advantage of post-elastic deformation of pipes. The deformation capacity of a pipe must be evaluated.

NOTE A valid calculation method for pipes buried in stable soil, based on approximate assumptions of the characteristics of the terrain movement, is given in Appendix B.

### 6.5 Testing

### 6.5.1 General considerations

(1) Pipelines buried in stable and sufficiently homogeneous soils can be tested only for the effects of soil deformations due to the passage of waves.

(2) Buried pipelines passing through areas where terrain failure or local deformation, such as lateral expansion, liquefaction, landslides and fault movements may occur, must be designed to resist such phenomena.

### 6.5.2 Pipelines buried in stable soil

(1) The responses obtained by calculation must include the maximum value of the axial unit deflections and bending and, in the case of non-welded joints (reinforced or prestressed concrete pipes), that of the rotations and axial deflections at the joints.

(2) For welded steel pipes, the combination of axial unit deflection and bending due to calculated seismic action must be compatible with the available ductility of the material in tension and with the local and global buckling resistance in compression.

- admissible unitary deformation in tension: 3 %;

- admissible unit deformation in compression: min {1%; $20t/r$ (%)};

where $t$ and $r$ are, respectively, the thickness and radius of the pipe.

(3) In concrete pipes, under the most unfavourable combination of axial unit deflection and curvature due to calculated seismic action, the limit unit deflections specified in Annex 19 of the Structural Code for concrete and steel shall not be exceeded.

(4) In concrete pipes, under the most unfavourable combination of axial unit deflection and curvature due to calculated seismic action corresponding to the damage limitation state, the tensile unit deflection of the reinforcement steel must not exceed values that may result in residual crack widths incompatible with the sealing requirements.

(5) Under the effect of the most unfavourable combination of axial and rotational deformations, the joints of a pipe shall not be damaged in a manner incompatible with the specified damage limitation requirements.

### 6.5.3 Buried pipelines subjected to differential terrain movements (welded steel pipes)

(1) Testing shall be carried out to ensure that in the pipe segment deformed by terrain displacement, whether due to fault movement, landslide or lateral expansion, the available ductility of the material in tension is not exceeded and that there is no local or global buckling in compression. The limit unit deflections shall be in accordance with section **6.5.2**.

## 6.6 Provisions for the calculation of the failure crossover

(1) The decision to use special calculations for pipelines crossing a potentially active fault zone depends on cost, fault activity, consequences of a rupture, environmental impact and possible exposure to other risks during the lifetime of the pipeline.

(2) When designing a pipeline across a fault, the considerations in **(3)** to **(9)** will generally improve

the ability of the pipeline to resist differential movements along the fault.

(3) Whenever practicable, a pipe passing through a tearing fault shall be oriented so that the pipe is stressed in tension.

(4) The angle of intersection with vertically moving faults shall be as small as possible in order to minimise compressive unit deformations. If significant shear displacements are also foreseeable, the angle at which the pipe crosses the fault shall be chosen such that tensile elongation of the pipe is favoured.

(5) In fault zones, the depth to which a pipe is buried should be minimised in order to reduce the restraint exerted by the soil on the pipe during fault movement.

(6) An increase in the wall thickness of a pipe will increase the ability of the pipe to withstand the fault displacement for a given level of maximum unit tensile strain. Pipes with relatively thick walls should be used within 50 m on either side of a fault.

(7) Reducing the friction angle of the contact surface between the pipe and the terrain increases the ability of the pipe to withstand the fault displacement, for a given level of maximum unit strain. This reduction can be realised by means of a smooth and hard coating.

(8) A detailed check of the backfill surrounding the pipeline should be carried out within 50 m on each side of the fault. In general, a loose to medium density granular soil, free of cobbles and boulders, is a suitable backfill material. If the existing soil is significantly different from this, oversized trenches should be excavated for a distance of approximately 15 m on either side of the fault.

(9) In the case of welded steel pipes, the process of adapting them to fault movement is to utilise their tensile deformation capacity within the inelastic range in order to withstand the deformations of the terrain without rupturing. Whenever possible, the route of a pipe across a fault should be chosen so that the pipe is subjected to composite tension with moderate bending. Layouts that could subject pipes to compression should be avoided as far as possible, since the ability of pipes to withstand compressive unit strains without rupture is considerably less than the ability to withstand tensile unit strains. Possible compressive unit strains shall be limited to those that cause wrinkling or local buckling of the pipe.

(10) In all areas of potential terrain failure, pipes shall consist of relatively straight sections, avoiding abrupt changes in direction and level. As far as possible, pipes shall be laid without bends, elbows or flanges which tend to anchor the pipe to the ground.

# Appendix A

# Recommended methods of seismic analysis for reservoirs

## A.1 Introduction and scope

This Annex provides information on methods of seismic analysis for tanks subjected to horizontal or vertical seismic action, with the following characteristics:

a) cylindrical shape, with vertical axis and circular or rectangular transverse cross-section;

b) rigid or flexible foundation;

c) fully or partially anchored to the foundation.

The necessary additions for the treatment of elevated tanks or cylindrical tanks with horizontal axis are briefly discussed.

A rigorous analysis of the dynamic interaction phenomena between the movement of the contained fluid, the deformation of the tank walls and the deformation of the foundation soil, including possible uplift, is a problem of considerable analytical complexity, usually requiring unusual computational resources and stresses. Several methods of analysis, valid for specific calculation situations, have been proposed. Since their accuracy depends on the problem at hand, the appropriate choice of method requires a good level of expertise on the part of the estimator. Attention should be drawn to the need to maintain a uniform level of accuracy throughout the calculation process: for example, it would be inconsistent to choose an accurate solution for determining the hydrodynamic pressure, and then not use a mechanical model with a corresponding degree of refinement for the reservoir (e.g. a finite element model) for the assessment of stresses due to the pressures.

## A.2 Rigid vertical circular tanks supported on the ground, connected to the foundations

### A.2.1 Horizontal seismic action

#### A.2.1.1 General considerations

The motion of the fluid contained in a rigid cylinder can be expressed as the sum of two different contributions, called respectively ‘rigid impulsive’ and ‘convective’. The ‘rigid impulsive’ component satisfies exactly the boundary conditions at the walls and bottom of the vessel, but gives (incorrectly, due to the presence of waves in the dynamic response) a zero pressure at the initial position of the free surface of the fluid in a static situation. The term 'convective' does not modify the boundary conditions that have already been satisfied, while satisfying the correct equilibrium condition at the free surface. A cylindrical coordinate system is used: $r$, $z$, $\theta$, with origin at the centre of the tank bottom and with the $z$-axis vertical. The height of the reservoir to the initial free surface of the fluid and the radius of the reservoir are denoted $H$ and $R$, respectively, $\rho$ is the density of the fluid, while = $\xi = r/R$ and $\varsigma = z/H$ are dimensionless coordinates.

### A.2.1.2 Rigid impulsive pressure

The spatio-temporal variation of the ‘rigid impulsive’ pressure is given by the expression:

$$p_i(\xi,\varsigma,\theta,t)=C_i(\xi,\varsigma)\ \rho\ H\ \cos\theta\ A_g(t) \tag{A.1}$$

where

$$C_i(\xi,\varsigma)=2\sum_{n=o}^{\infty}\frac{(-1)^n}{I_1'(v_n/\gamma)\ v_n^2}\cos(v_n\varsigma)\ I_1\left(\frac{v_n}{\gamma}\xi\right) \tag{A.2}$$

where:

$v_n=\frac{2n+1}{2}\pi\ ;\quad \gamma=H/R$

$I_1(¿)$ and $I_1'(¿)$ denote the modified Bessel function of order 1 and its derivative[4].

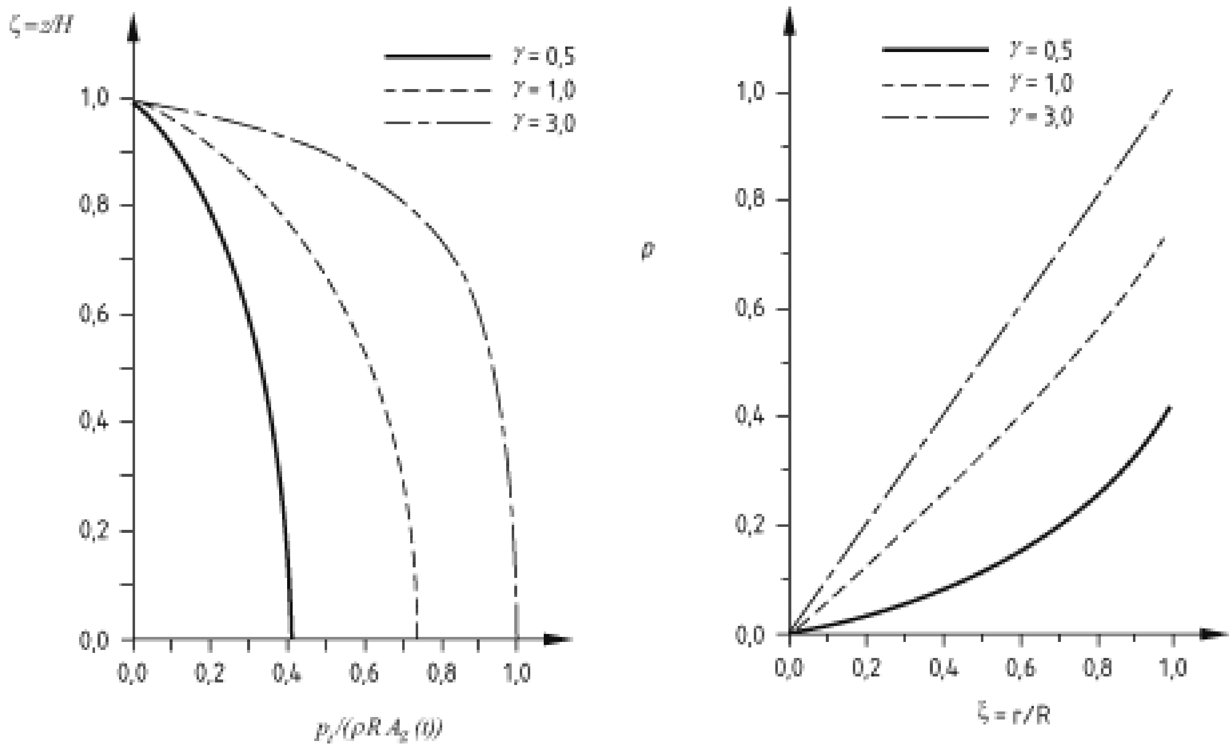



**Figure A.1 - Impulsive pressure variation (normalised to $\rho R\ a_g$) for three values of $\gamma = H/R$. a) variation along the height; b) radial variation at the bottom of the vessel**

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[4] )The derivative can be expressed in terms of the modified Bessel functions of order 0 and 1 as:

$$I_1'(x)=\frac{dI_1(x)}{dx}=I_0(x)-\frac{I_1(x)}{x}$$

$A_g(t)$ in expression (A.1) is the log of the free-field time domain ground acceleration (with a maximum value defined by $a_g$). The function $C_i$ gives the distribution of $p_i$ along the height. This function is shown in figure A.1a) for $\xi = 1$ (i.e. at the tank wall) and cos $\theta = 1$ (i.e. in the plane of horizontal seismic action), normalised to $\rho R a_g$, for three values of the slenderness parameter $\gamma = H/R$. Figure A.1b) shows the radial variation of $p_i$ at the bottom of the reservoir as a function of $\gamma$. For high values of $\gamma$, the pressure distribution at the bottom becomes linear.

Resultant pressures: The horizontal resultant of the 'rigid impulsive' pressure at the base of the wall $Q_i$, from expression (A.1) is:

Impulsive shear at the base:

$$Q_i(t) = m_i\, A_g(t) \tag{A.3}$$

$m_i$, called *impulsive mass*, denotes the mass of the contained fluid moving in conjunction with the walls and is given by the expression:

$$m_i = m 2\gamma \sum_{n=0}^{\infty} \frac{I_1(\nu_n/\gamma)}{\nu_n^3\, I_1'(\nu_n/\gamma)} \tag{A.4}$$

where

$m = \rho\pi R^2 H$ is the total mass of contained fluid.

The total moment about an axis orthogonal to the direction of motion of the seismic action, $M'_i$, just below the bottom of the container includes contributions from the pressures exerted on the walls, given by expression (A.1), and from those exerted on the bottom of the container. The total momentum $M'_i$, just above the bottom of the container includes only the contributions from the pressures exerted on the walls.

Impulsive moment at the base (just below the bottom of the tank):

$$M'_i(t) = m_i\, h'_i\, A_g(t) \tag{A.5a}$$

where

$$h'_i = H \frac{\frac{1}{2} + 2\gamma \sum_{n=0}^{\infty} \frac{\nu_n + 2(-1)^{n+1}}{\nu_n^4} \frac{I_1(\nu_n/\gamma)}{I_1'(\nu_n/\gamma)}}{2\gamma \sum_{n=0}^{\infty} \frac{I_1(\nu_n/\gamma)}{\nu_n^3\, I_1'(\nu_n/\gamma)}} \tag{A.6a}$$

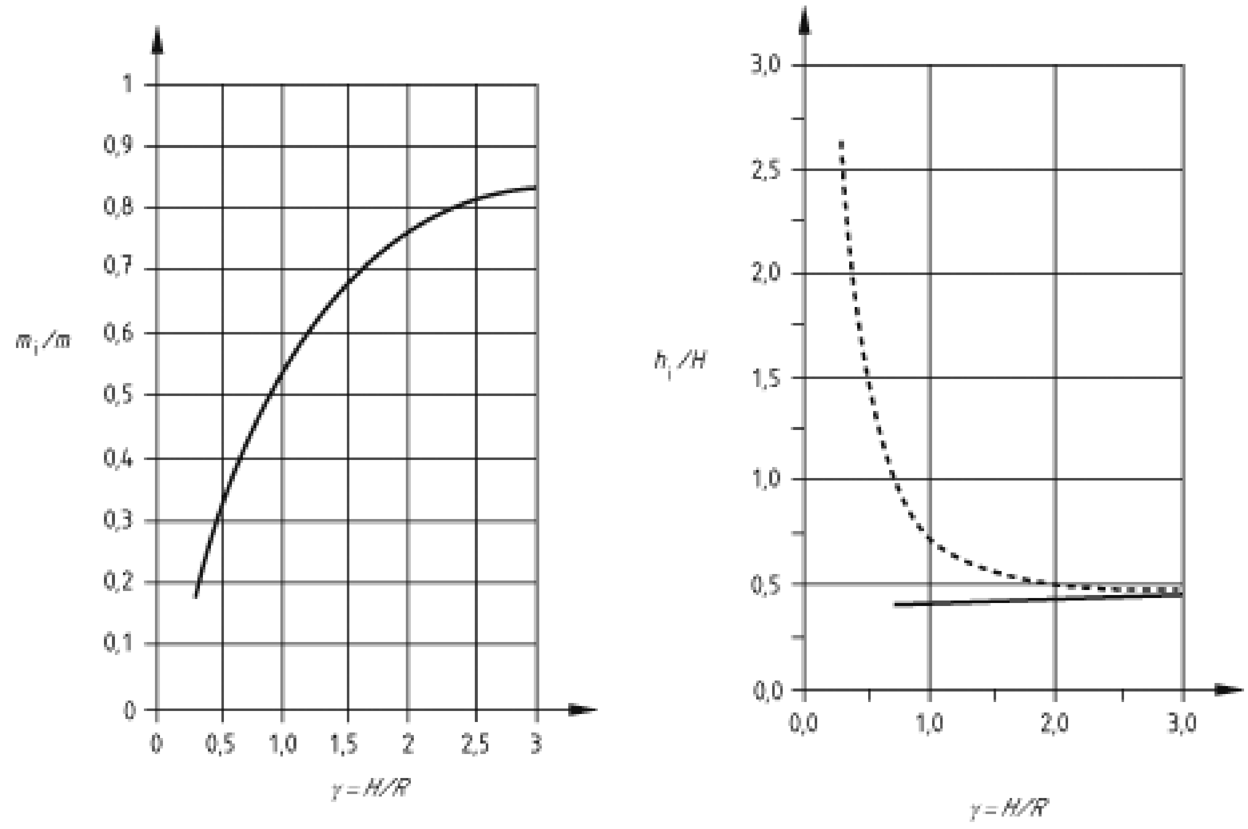
Impulsive moment at the base (just above the bottom of the container):

$$M_i(t) = m_i\, h_i A_g(t) \tag{A.5b}$$

with

$$h_i = H \frac{\sum_{n=0}^{\infty} \frac{(-1)^n}{v_n^4} \frac{I_1(v_n/\gamma)}{I_1'(v_n/\gamma)} \left(v_n(-1)^n - 1\right)}{\sum_{n=0}^{\infty} \frac{I_1(v_n/\gamma)}{v_n^3\, I_1'(v_n/\gamma)}} \tag{A.6b}$$

Figure A.2 shows the values of $m_i$, $h'_i$ and $h_i$ as a function of $\gamma = H/R$. The value of $m_i$ increases with $\gamma$, tending asymptotically to the total mass, while both $h_i$ and $h'_i$ tend to stabilise approximately for values at mid-height. For short reservoirs $h_i$ is slightly lower than mid-height, while the value of $h'_i$ is significantly higher than that of $H$ due to the predominant contribution to the value of $M'_i$ of the pressures exerted at the bottom.




a) b)

**Legend to figure A.2(b):** ——: On the bottom plate; - - - - - : Under the bottom plate

**Figure A.2 - Ratios $m_i/m$, $h_i/H$ and $h'_i/H$ as a function of tank slenderness (see also table A.2, columns 4, 6 and 8)**

### A.2.1.3 Convective pressure component

The spatio-temporal variation of the "convective" pressure component is given by:

$$p_c(\xi,\varsigma,\theta,t)=\rho\sum_{n=1}^{\infty}\psi_n\cosh(\lambda_n\gamma\varsigma)\,J_1(\lambda_n\xi)\,\cos\theta\;A_{\mathrm{cn}}(t) \tag{A.7}$$

where

$$\psi_n=\frac{2\,R}{(\lambda_n^2-1)\,J_1(\lambda_n)\cosh(\lambda_n\gamma)} \tag{A.8}$$

where

$J_1$ = first order Bessel function;

$\lambda_1$ = 1.841, $\lambda_2$ = 5.331, $\lambda_3$ = 8.536; and

$A_{\mathrm{cn}}(t)$ = time domain acceleration of the response of a single degree of freedom oscillator having a circular frequency $\omega_{\mathrm{cn}}$ equal to:

$$\omega_{\mathrm{cn}}=\sqrt{g\,\frac{\lambda_n}{R}\tanh(\lambda_n\gamma)} \tag{A.9}$$

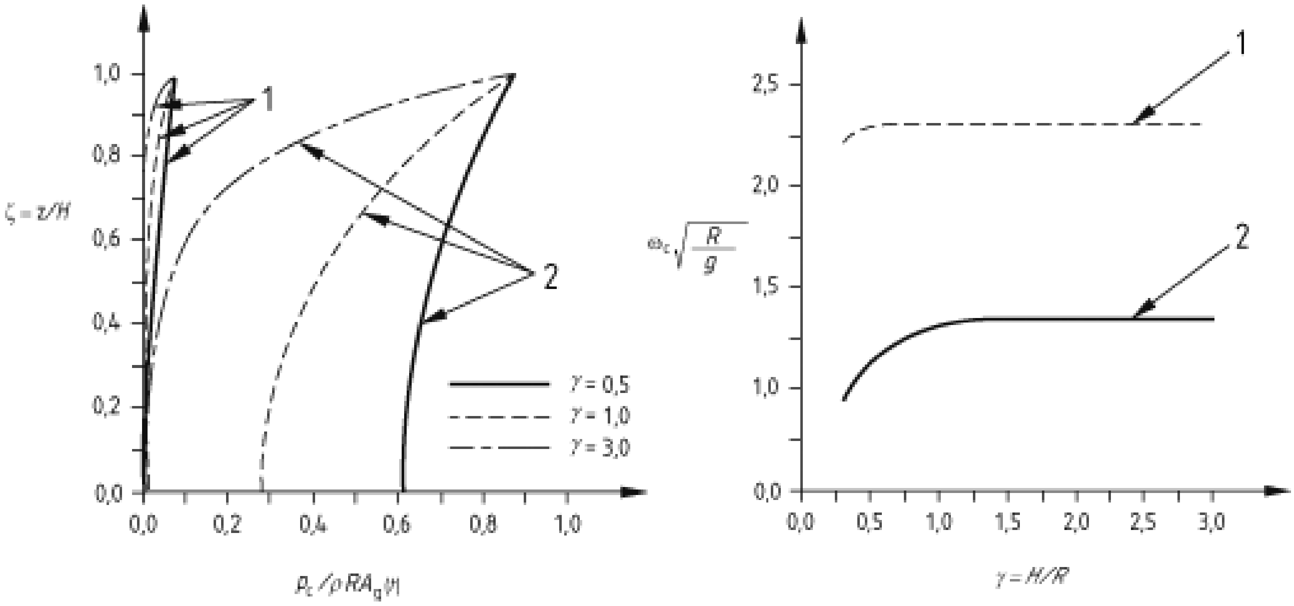
and an appropriate damping ratio for the convective response of the fluid.

For calculation purposes, in expression (A.7) it is only necessary to consider, for the convective component, the first mode and the first oscillation frequency of the fluid response ($n$ = 1).

Figure A.3a) shows the vertical distribution of the pressures due to the convective response of the fluid for the first two modes, while figure A.3b) gives the values of the first two frequencies, as a function of $H/R$. In low reservoirs (relative to their diameter), the pressures generated by the convective component maintain relatively high values near the bottom, whereas, in slender reservoirs, the effect is limited to the vicinity of the fluid surface. For $\gamma$ larger than a value around 1, the frequencies become almost independent of $\gamma$. For these values of $\gamma$, $\omega_{\mathrm{c1}}$ is approximately equal to :

$$\omega_{\mathrm{c1}}=4{,}2/\sqrt{R} \qquad (R\ \text{in metres}) \tag{A.10}$$

which, for the usual values of R, gives oscillation periods of the order of a few seconds.




**a)** **b)**

Legend:

1 2nd mode

2 1st mode

**Figure A.3 - a) Variation with height of the convective pressures in the first two oscillation modes and b) values of the frequencies of the convective response in the first two modes as a function of γ**

Resultant pressures:

Convective shear at the base:

$$Q_c(t)=\sum_{n=1}^{\infty} m_{\mathrm{cn}}\ A_{\mathrm{cn}}(t) \tag{A.11}$$

where the nth convective modal mass is:

$$m_{\mathrm{cn}}=m\frac{2\ \tanh\left(\lambda_n\ \gamma\right)}{\gamma\ \lambda_n\left(\lambda_n^2-1\right)} \tag{A.12}$$

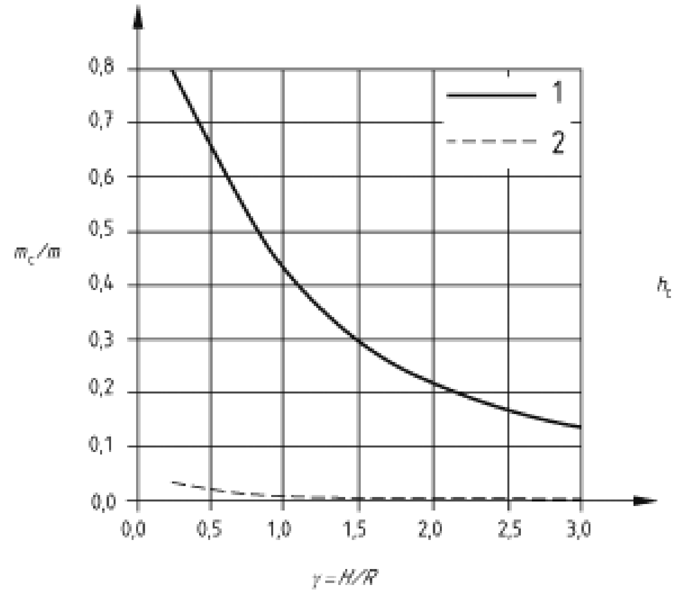
Moment just below the bottom plate of the reservoir:

$$M^{'c}(t)=\sum_{n=1}^{\infty}\left(m_{cn}\,A_{cn}(t)\right)h^{'cn}=\sum_{n=1}^{\infty}\ Q_{cn}(t)\ h^{'cn} \tag{A.13a}$$

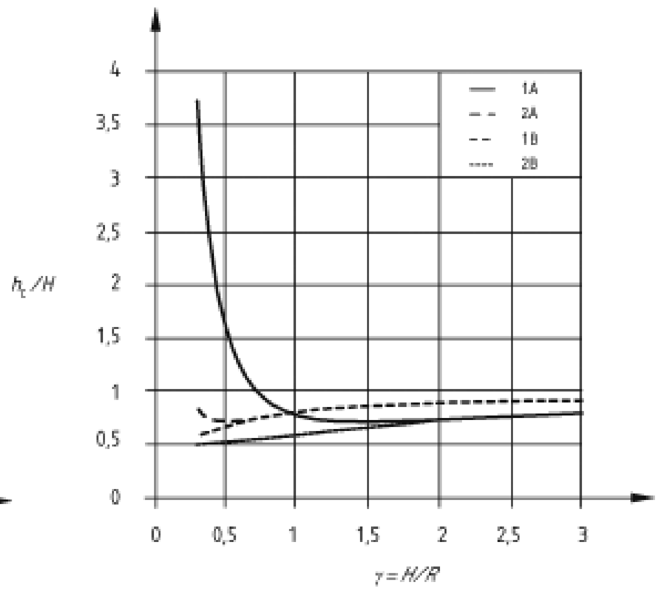
where

$$h'_{\mathrm{cn}}=H\left(1+\frac{2-\cosh\left(\lambda_{\mathrm{n}}\ \gamma\right)}{\lambda_{\mathrm{n}}\gamma\ \mathrm{senh}\left(\lambda_{\mathrm{n}}\ \gamma\right)}\right) \tag{A.14a}$$

Figure A.4 shows the values of $m_{c1}$ and $m_{c2}$ and the corresponding values of $h_{c1}$, $h_{c2}$, $h'_{c1}$ and $h'_{c2}$ as a function of $\gamma$.




**a)**




**b)**

Legend to figure A.4(a):

1 1st mode

2 2nd mode

Legend to figure A.4(b):

1A 1st mode, under bottom plate

2A 2nd mode, under bottom plate

1B 1st mode, above bottom plate

2B 2nd mode, above bottom plate

**Figure A.4 - a) Modal masses of the liquid convective response of the first two oscillation modes and b) corresponding heights $h_{c1}$, $h_{c2}$, $h'_{c1}$ and $h'_{c2}$ as a function of $\gamma$ (see also Table A.2, columns 5, 7 and 9)**

Moment on the tank wall just above the bottom plate:

$$M_c(t) = \sum_{n=1}^{\infty} \left( m_{cn} A_{cn}(t) \right) h_{cn} = \sum_{n=1}^{\infty} Q_{cn}(t)\; h_{cn} \qquad \text{(A.13b)}$$

where $h_{cn}$ is:

$$h_{cn} = H \left( 1 + \frac{1 - \cosh(\lambda_n \gamma)}{\lambda_n \gamma \, \mathrm{senh}(\lambda_n \gamma)} \right) \qquad \text{(A.14b)}$$

The convective component of the response can be obtained from that of oscillators with masses $m_{cn}$, attached to the rigid vessel by springs with rigidities: $K_n = \omega_n^2 m_{cn}$ . (a single oscillator for each mode considered as significant, usually only the first one). The tank is subjected to the time domain ground acceleration $A_g(t)$ and the masses respond with the accelerations $A_{cn}(t)$. $h'_{cn}$ or $h_{cn}$ are the levels at which the oscillator must be applied to give, respectively, the correct values of $M'_{cn}$ or $M_{cn}$.

#### A.2.1.4 Convective wave height

The wave height due to the convective response of the liquid is mainly given by the first mode. The expression for the maximum height at the edge is:

$$d_{\max} = 0,84\ R\ S_e\left(T_{c1}\right)/g \tag{A.15}$$

where $S_e(\cdot)$ is the elastic acceleration response spectrum corresponding to the 1st convective mode of the fluid for the damping corresponding to the convective response of the fluid, and $g$ is the acceleration of gravity.

#### A.2.1.5 Effect of wall inertia

For steel tanks, the inertia forces acting on the wall sheet due to its own mass are small compared to the hydrodynamic forces and can be neglected. For concrete tanks, on the other hand, they should not be neglected. The inertia forces are parallel to the horizontal seismic action and induce a pressure normal to the surface of the sheet given by:

$$p_w = \rho_s s(\varsigma) \cos\theta\ A_g(t) \tag{A.16}$$

where

$\rho_s$ = density of the wall material

$s(\varsigma)$ = thickness of the wall

The effects of the action of this pressure component, which follows the variation in wall thickness with height, must be added to those of the impulsive component given by expression (A.1).

The total shear effect at the base due to the inertia forces of the tank wall and shell can be taken equal to the total mass of the tank walls and shell multiplied by the ground acceleration. Similarly, the contribution to the overturning moment at the base is equal to the mass of the wall multiplied by the half-height of the wall (for a constant wall thickness), plus the mass of the shell multiplied by the average distance from the shell to the base and by the ground acceleration.

#### A.2.1.6 Combined effects of impulsive and convective pressure action

The time domain response of the total pressure is the sum of the following two time domain responses:

- the impulsive, controlled by $A_g(t)$ (including wall inertia);

- the convective, controlled by $A_{c1}(t)$ (neglecting the higher order components).

Just as the dynamic response associated with the two pressure components is characterised by different damping coefficients, it can also be associated with different hysteresis energy dissipation mechanisms. No energy dissipation can be associated with the convective response of the fluid, while the response due to the impulsive pressures and the inertia of the vessel walls may be accompanied by some hysteresis energy dissipation, which comes from the vessel itself and the way it is supported on (or anchored to) the terrain. If the energy dissipation is taken into account by modifying the elastic spectrum with the behavioural coefficient $q$, a different value of $q$ should be used for the deduction of the effects of the action of the two components: i.e. $q$ = 1.0 for the effects of the action of convective pressures and $q$ = 1.5 (or a higher value) for the effects of the action of impulsive pressures and the inertia of the tank walls.

If, as is common in calculation practice, the response spectrum method is used to calculate the maximum of the dynamic response, the maximum values of the effects of the seismic action of the two time domain responses given by the response spectrum should be appropriately combined. Since the predominant frequencies of the terrain motion and the convective fluid response are generally far apart, the "square root of the sum of the squares" rule may not be conservative, so it may be preferable to use the alternative (upper bound) rule of summing the absolute values of the two maxima for the calculation. Each of these two maxima shall be obtained from the values of $q$ and the damping ratio considered appropriate for the corresponding component.

For testing, the values of the moment and shear stress just above the base of the container should be used for the calculation of the resulting stresses and strains in the container walls and at the junction with the base. The value of the moment just below the base of the tank should be used for testing of the supporting structure of the tank, the base anchors or the foundation.

Given the long period of the convective component of the liquid response, only the moment below the tank bottom plate that is due to this pressure component is relevant for testing the static equilibrium of the tank (overturning). Given their high frequencies, impulsive pressures and the inertia of the vessel walls can be considered as not contributing to the destabilisation momentum in the testing of the vessel at overturning.

### A.2.2 Vertical component of seismic action

The hydrodynamic pressure on the walls of a rigid tank due to a vertical acceleration of the ground $A_v(t)$ is given by

$$p_{vr}(\varsigma,t)=\rho\,H\,(1-\varsigma)\,A_v(t) \tag{A.17}$$

Being axisymmetric, this hydrodynamic pressure produces no shear stress or resultant moment in any horizontal section of the tank, either just above or just below the base.

### A.2.3 Combined effects of the horizontal and vertical components of the seismic action, including the effects of other actions

The maximum combined pressure on the tank walls due to horizontal and vertical seismic action can be obtained by applying the rule in Section **3.2**. The combined pressure should be added to the hydrostatic pressure exerted on the wall on one side of the tank (the side where the wall is accelerated towards the liquid) and subtracted as suction from the opposite side. The dynamic pressures exerted by the earth and water in the terrain should be considered to act on the entire buried part of the reservoir on the side where the seismic pressure is considered as a suction. The earth pressures should be evaluated on the basis of the coefficient of buoyancy of the terrain at rest.

## A.3 Deformable circular vertical tanks supported on the ground, attached to the foundation

### A.3.1 Horizontal components of seismic action

It is usually not conservative to consider a tank as rigid (especially for steel tanks). In flexible tanks, the fluid pressure is usually expressed as the sum of three contributions, termed: 'impulsive rigid', 'convective' and 'flexible'. The third contribution satisfies the condition that the radial velocity of the fluid along the wall is equal to the deformation velocity of the vessel wall, as well as the conditions of zero vertical velocity at the bottom of the vessel and zero pressure at the free surface of the fluid. The dynamic coupling between the convective and flexible components is very small, due to the large differences between the oscillation frequencies of the fluid and the wall deformation, which allows the third component to be determined independently of the other two. For this reason, the rigid impulsive and convective components discussed in chapter **A.2** are not affected.

The flexible pressure distribution depends on the modes of vibration of the fluid-tank system, of which only those with a circumferential wave, of the following type, are of interest:

$$\phi(\varsigma,\theta) = f(\varsigma)\cos\theta \tag{A.18}$$

In the following, the term "fundamental frequency" or "first frequency" or "first mode" is not related to the true fundamental modes of the filled vessel, but only to eigenmodes of the type described by expression (A.18).

The radial distribution of the flexible impulsive pressure over the bottom of the vessel is qualitatively the same as that of the rigid impulsive pressure. Assuming that the modes are known, the distribution of the flexible pressure exerted on the walls has the form:

$$p_f(\varsigma, \theta, t) = \rho\, H\, \psi\, \cos\theta \sum_{n=0}^{\infty} d_n \cos(v_n\, \varsigma)\, A_{\mathrm{fn}}(t) \tag{A.19}$$

where

$$\psi=\frac{\int_0^1 f(\varsigma)\left[\frac{\rho_s}{\rho}\frac{s(\varsigma)}{H}+\sum_{n=0}^{\infty} b_n^{'}\cos(\nu_n\varsigma)\right]d\varsigma}{\int_0^1 f(\varsigma)\left[\frac{\rho_s}{\rho}\frac{s(\varsigma)}{H}f(\varsigma)+\sum_{n=0}^{\infty} d_n\cos(\nu_n\varsigma)\right]d\varsigma} \tag{A.20}$$

$$b_n^{'}=2\frac{(-1)^n}{\nu_n^2}\frac{I_1(\nu_n/\gamma)}{I_1^{'}(\nu_n/\gamma)} \tag{A.21}$$

$$d_n=2\frac{\int_0^1 f(\varsigma)\cos(\nu_n\varsigma)\,d\varsigma}{\nu_n}\frac{I_1(\nu_n/\gamma)}{I_1^{'}(\nu_n/\gamma)} \tag{A.22}$$

$\rho_s$ is the density of the wall sheet, $s(\varsigma)$ is its thickness and $A_{fn}(t)$ is the acceleration response (with respect to its base) of a simple oscillator having the period and damping rate of mode $n$. Normally the fundamental mode ($n$=1) is sufficient, so that in expressions (A.19), (A.21) and (A.22) the mode index, $n$, and the sum total of all modal contributions disappear.

In most cases of flexible reservoirs, the pressure $p_f(\cdot)$ in expression (A.19) provides the predominant contribution to the total pressure, due to the fact that, while the impulsive rigid term - expression (A.1) - varies with the terrain acceleration $A_g(t)$, the flexible term - expression (A.19) - varies with the acceleration response $A_{fn}(t)$, which, for the usual range of periods of fluid-reservoir systems, is considerably amplified with respect to $A_g(t)$.

To determine the first mode deformation of the reservoir, the following iterative method is proposed. A first mode deformation is chosen for $f(\varsigma)$, in the expressions (A.18)-(A.22) (a deformation proportional to $\varsigma$, especially for slender reservoirs, is usually a good approximation). Calling $f^i(\varsigma)$ the deformation used in the $i$-th iteration, the "effective" density of the wall sheet is evaluated as:

$$\rho^i(\varsigma)=\frac{p_f^i(\varsigma)}{2g\, s(\varsigma)\, f^i(\varsigma)}+\rho_s \tag{A.23}$$

where $p_f^i(\varsigma)$ is the value of the pressure evaluated from expression (A.19) at the $i$th step. The effective density obtained from expression (A.23) can then be used in the structural analysis of the reservoir to evaluate the mode deformation at the ($i$+1)-th iteration, and so on until it converges.

The fundamental angular frequency of the fluid-tank system can be evaluated by means of the following approximate expression, for steel tanks.

$$\omega_f=2\pi\frac{\sqrt{E\, s(\varsigma)/\rho\, H}}{R(0,157\gamma^2+\gamma+1,49)} \qquad \text{(for } \varsigma = 1/3\text{)} \tag{A.24}$$

where $E$ is the modulus of elasticity of the vessel wall material.

The shear stress at the base is:

$$Q_f(t)=m_f\ A_f(t) \tag{A.25}$$

where

$$m_f=m\psi\gamma\sum_{n=0}^{\infty}\frac{(-1)^n}{v_n}d_n \tag{A.26}$$

The moment just above the base of the container can be calculated as:

$$M_f(t)=m_f\ h_f\ A_f(t) \tag{A.27}$$

where

$$h_f=H\frac{\left[\gamma\sum_{n=0}^{\infty}d_n\frac{(-1)^n v_n-2}{v_n^2}+\sum_{n=0}^{\infty}\frac{d_n I_1'(v_n/\gamma)}{v_n}\right]}{\gamma\sum_{n=0}^{\infty}d_n\frac{(-1)^n}{v_n}} \tag{A.28}$$

### A.3.2 Combination of the pressure terms due to the horizontal components of the seismic action

#### A.3.2.1 General methods

The time domain response of the total pressure in flexible tanks is the sum of the time domain responses of the rigid impulsive pressure (expression (A.1)), the convective pressure (expression (A.7)) and the flexible pressure (expression (A.19)), each having a different distribution along the height and a different variation in time. The time variation of the shear stress at the base produced by these pressures (expressions (A.3), (A.11) and (A.25)) is:

$$Q(t)=m_i\ A_g(t)+\sum_{n=1}^{\infty}m_{\text{cn}}\ A_{\text{cn}}(t)+m_f\ A_f(t) \tag{A.29}$$

where Acn(t) is the absolute or total acceleration response of a simple oscillator with an angular frequency cn (expression (A.9)) and an appropriate damping rate for the convective response under an acceleration of the base $A_g(t)$, while $A_f(t)$ is the acceleration response (relative to the base) of a simple oscillator with angular frequency $\omega_f$ (expression (A.24)) and with an appropriate damping for the fluid-tank system, also subjected to $A_g(t)$.

If the individual maxima of the terms of expression (A.29) are known, e.g. by using a response spectrum of the absolute and relative accelerations, the corresponding pressures in the vessel, necessary for a detailed stress analysis, can be obtained by distributing the resultant of each of the three terms of expression (A.29) over the vessel walls and bottom according to the corresponding distribution of the pressures. In order to facilitate the calculation process, the masses $m_i$, $m_{cn}$ and $m_f$, the latter of which is based on the assumed deformations for the first modes, have been calculated as a function of the coefficient $\gamma$, and are available in the form of tables or diagrams in the specialised technical literature (see, for example, figures A.2(a), A.4(a) and columns 4 and 5 of Table A.2). However, the use of the expression (A.29) in combination with the response spectra raises the problem of the combination of maxima. Unless a relative acceleration response spectrum is calculated for $A_f(t)$, there is no exact way to combine the maximum of $A_g(t)$ with that of $A_f(t)$. In fact, since the seismic excitation and its response cannot be assumed to be independent in the relatively high frequency range considered, the "square root of the sum of squares" (SRSS) rule is not sufficiently accurate. On the other hand, the simple summation of the individual maxima may lead to overly conservative estimates.

Taking into account these difficulties, several authors have proposed different approximate methods based on the above theory. Some of these methods, by Veletsos and Yang, by Haroun and Housner, and by Scharf, are briefly presented here.

The method of Veletsos and Yang consists of replacing the expression (A.29) by the following:

$$Q(t)=m_i\ A_{\mathrm{fa}}(t)+\sum_{n=1}^{\infty} m_{\mathrm{cn}}\ A_{\mathrm{cn}}(t) \tag{A.30}$$

i.e. assuming that the entire impulsive mass responds with the amplified absolute acceleration of the flexible tank ($A_{\mathrm{fa}}(t)$ = $A_f(t)+A_g(t)$) with an angular frequency $\omega_f$ (expression (A.24)) and a damping appropriate for the fluid-tank system. The maximum value of $A_{\mathrm{fa}}(t)$ is obtained directly from the appropriate response spectrum. The total shear stress at the base can be evaluated approximately by the expression:

$$Q_{\mathrm{w}}(t) = (\varepsilon_{\mathrm{o}} \cdot m) \cdot A_{\mathrm{fa}}(t) \tag{A.31}$$

where $(\varepsilon_o \cdot m)$ is the mass of the tank wall actually participating in the first mode, $m$ represents the total mass of the fluid-tank system and the coefficient $\varepsilon_o$ can be determined from Table A.1:

**Table A.1 – Mass of the vessel wall actually participating in the first mode, as a fraction of the total mass, in the Veletsos and Yang procedure**

| $H/R$ | 0.5 | 1.0 | 3.0 |
|---|---|---|---|
| $\varepsilon_o$ | 0.5 | 0.7 | 0.9 |

Veletsos and Yang's procedure provides an upper bound estimate, acceptable for $H/R$ ratios not much greater than 1. Above this value it is suggested to adopt corrections that reduce conservatism. In view of the conservative nature of the method, the inertial effects of the reservoir can generally be ignored.

In the Haroun and Housner method, the expression (A.29) is written in a form appropriate for the use of the response spectrum:

$$Q(t)=(m_i-m_f)\ A_g(t)+\sum_{n=1}^{\infty} m_{\mathrm{cn}}\ A_{\mathrm{cn}}(t)+m_f\ A_{\mathrm{fa}}(t) \tag{A.32}$$

The masses $m_i$ and $m_f$ are given as plots, as a function of $H/R$ and $s/R$, as well as the heights at which these should be placed to obtain the correct value of the momentum at the base. The inertial effects of the tank wall are included in the values of the masses and their heights.

To combine the maximum values of the three components in the expression (A.32), the 'square root of the sum of the squares' rule is used.

Finally, based on the fact that the absolute and relative acceleration responses do not differ considerably over the frequency range of interest, in Scharf's method the expression (A.29) is written as:

$$Q(t)=m_i\, A_g(t)+\sum_{n=1}^{\infty} m_{\mathrm{cn}}\, A_n(t)+m_f\, A_{fa}(t) \tag{A.33}$$

To combine the maximum values of the three components in the expression (A.34), the 'square root of the sum of the squares' rule is used.

$$Q=\sqrt{(m_i a_g)^2+(m_f a_{fa})^2+(\sum_{n=1}^{\infty} m_{cn} a_{cn})^2} \tag{A.34}$$

#### A.3.2.2 Simplified procedure for cylindrical tanks with fixed base

In a similar vein to the method of Veletsos and Yang, Malhotra, P.K. et al. have proposed an even more simplified method, which is summarised below.

Here again, section **A.2.1.4** applies, with regard to the different hysteresis energy dissipation mechanisms (and the associated values of the behavioural coefficient $q$) characterising the different pressure components.

##### A.3.2.2.1 Model

The tank-liquid system is modelled by two one-degree-of-freedom systems, one corresponding to the impulsive component, which moves in phase with the flexible wall, and the other to the convective component. The impulsive and convective responses are combined by taking their numerical sum.

The natural periods of the impulsive and convective responses, in seconds, are taken as:

$$T_{\mathrm{imp}}=C_i \frac{\sqrt{\rho}H}{\sqrt{\mathrm{s/R}}\sqrt{E}} \tag{A.35}$$

$$T_{\mathrm{con}}=C_c\sqrt{R} \tag{A.36}$$

where

$H$ = height to the free surface of the liquid;

$R$ = radius of the tank;

$s$ = equivalent uniform thickness of the vessel wall (weighted average along the height of contact of the liquid with the vessel wall, the weighting coefficient can be taken proportional to the unit deformation in the wall; which is maximum at the base of the vessel);

$\rho$ = density of the liquid; and

$E$ = modulus of elasticity of the container material.

**Table A.2 – Coefficients $C_i$ and $C_c$ of the natural periods, masses $m_i$ and $m_c$ and heights $h_i$ and $h_c$ measured from the base of the point of application of the resultant of the pressures exerted on the wall, for the impulsive and convective components.**

| $H/R$ | $C_1$ | $C_c$ (s/m$^{1/2}$) | $m_i/m$ | $m_c/m$ | $h_i/H$ | $h_c/H$ | $h'_i/H$ | $h'_c/H$ |
|---|---|---|---|---|---|---|---|---|
| 0.3 | 9.28 | 2.09 | 0.176 | 0.824 | 0.400 | 0.521 | 2.640 | 3.414 |
| 0.5 | 7.74 | 1.74 | 0.300 | 0.700 | 0.400 | 0.543 | 1.460 | 1.517 |
| 0.7 | 6.97 | 1.60 | 0.414 | 0.586 | 0.401 | 0.571 | 1.009 | 1.011 |
| 1.0 | 6.36 | 1.52 | 0.548 | 0.452 | 0.419 | 0.616 | 0.721 | 0.785 |
| 1.5 | 6.06 | 1.48 | 0.686 | 0.314 | 0.439 | 0.690 | 0.555 | 0.734 |
| 2.0 | 6.21 | 1.48 | 0.763 | 0.237 | 0.448 | 0.751 | 0.500 | 0.764 |
| 2.5 | 6.56 | 1.48 | 0.810 | 0.190 | 0.452 | 0.794 | 0.480 | 0.796 |
| 3.0 | 7.03 | 1.48 | 0.842 | 0.158 | 0.453 | 0.825 | 0.472 | 0.825 |

The coefficients $C_i$ and $C_c$ are obtained from Table A.2. The coefficient $C_i$ is dimensionless, whereas if $R$ is expressed in metres, $C_c$ is expressed in s/m$^{1/2}$.

The impulsive and convective masses, $m_i$ and $m_c$,, are given in Table A.2 as fractions of the total mass of the liquid $m$, together with the heights measured from the base, $h_i$ and $h_c$, of the point of application of the resultant of the impulsive and convective hydrodynamic pressures exerted on the wall.

### A.3.2.2.2 Seismic response

The total shear stress at the base is:

$$Q=\left(m_i+m_w+m_r\right)\ S_e\left(T_{\text{imp}}\right)+m_c\ S_e\left(T_{\text{con}}\right) \tag{A.37}$$

where

$m_w$ = mass of the tank wall;

$m_r$ = mass of the tank shell;

$S_e(T_{imp})$ = impulsive spectral acceleration, obtained from an elastic response spectrum with a damping value compatible with the limit state considered according to section 2.3.3.1;

$S_e(T_{con})$ = convective spectral acceleration, obtained from an elastic response spectrum with 0.5% damping.

The overturning moment at a point just above the base plate is:

$$M=\left(m_i\ h_i+m_w h_w+m_r h_r\right)S_e\left(T_{\mathrm{imp}}\right)+m_c h_c S_e\left(T_{\mathrm{con}}\right) \tag{A.38}$$

$h_w$ y $h_r$ represent respectively the heights of the centres of gravity of the tank wall and the tank shell.

The overturning moment at a point just below the base plate is given by:

$$M'=\left(m_i h'^{i}+m_w h_w+m_r h_r\right)S_e\left(T_{\mathrm{imp}}\right)+m_c h'^{c} S_e\left(T_{\mathrm{con}}\right) \tag{A.39}$$

The vertical displacement of the liquid surface due to the convective response is given by the expression (A.15).

### A.3.3 Vertical component of seismic action

In addition to the pressure $p_{vr}(\varsigma,t)$ given by expression (A.17), due to the vessel moving rigidly in the vertical direction with acceleration $A_v(t)$, there is a contribution to the pressure, $p_{vf}(\varsigma,t)$, due to the deformations of the wall sheet resulting from the "radial breathing mode" of vibration. This additional term can be calculated as:

$$p_{vf}\left(\varsigma,t\right)=0{,}815\ f\left(\gamma\right)\ \rho\ H\ \cos\left(\frac{\pi}{2}\varsigma\right)\ A_{vf}\left(t\right) \tag{A.40}$$

where

$$f(\gamma) = 1.078 + 0.274 \ln\gamma \qquad \text{for } 0.8 \leq \gamma < 4 \tag{A.41a}$$

$$f(\gamma) = 1.0 \qquad \text{for } \gamma < 0.8 \tag{A.41b}$$

$A_{vf}(t)$ is the acceleration response of a simple oscillator having a frequency equal to the fundamental frequency of the axisymmetric mode of vibration of the container with the fluid.

The fundamental frequency can be estimated from the expression:

$$f_{vd}=\frac{1}{4R}\left[\frac{2\ E\ I_1\left(\gamma_1\right)\ s\left(\varsigma\right)}{\pi\ \rho\ H\left(1-v^2\right)\ I_o\left(\gamma_1\right)}\right]^{1/2} \qquad \text{(for } \varsigma = 1/3\text{)} \tag{A.42}$$

where

$\gamma_1 = \pi/(2\gamma)$;

$I_0(\cdot)$ and $I_1(\cdot)$ are respectively the modified Bessel functions of order 0 and 1;

$E$ and ν are respectively the Young's modulus and Poisson's ratio of the deposit material.

The maximum value of $p_{vf}(t)$ is obtained from the vertical acceleration response spectrum for the appropriate values of period and damping. If the flexibility of the soil is neglected (see Chapter **A.7**), the applicable damping values are those of the wall sheet material. The value of the behavioural coefficient, $q$, adopted for the response due to the impulsive component of the pressure and the inertia of the shell wall, can be used for the response to the vertical component of the seismic action. The maximum value of the pressure due to the combined effect of $p_{vr}(\cdot)$ and $p_{vf}(\cdot)$ can be obtained by applying the 'square root of the sum of the squares' rule to the individual maximum values.

### A.3.4 Combined effects of the horizontal and vertical components of the seismic action, including the effects of other actions

The pressure on the container walls should be determined according to section **A.2.3**.

## A.4 Rectangular tanks

### A.4.1 Rigid rectangular tanks supported on the ground, attached to the foundation

For tanks whose walls can be assumed to be rigid, the total pressure is again obtained by the sum of an impulsive contribution and a convective contribution:

$$p(z,t) = p_i(z,t) + p_c(z,t) \tag{A.43}$$

The impulsive component follows the expression

$$p_i(z,t) = q_0(z)\ \rho\ L\ A_g(t) \tag{A.44}$$

where

$L$ is the half-width of the deposit in the direction of seismic action;

$q_o(z)$ is a function giving the variation of $p_i(\cdot)$ with height, as plotted in Figure A.5 ($p_i(\cdot)$ is constant in the direction orthogonal to the seismic action). The trend and numerical values of $q_o(z)$ are very close to those of a cylindrical tank with radius $R = L$ (see Figure A.6).

The convective component of the pressure is given by a sum of modal terms (convective modes of the fluid). As for cylindrical tanks, the dominant contribution is that of the fundamental mode:

$$p_{c1}(z,t) = q_{c1}(z)\ \rho\ L\ A_1(t) \tag{A.45}$$

where

$q_{c1}(z)$ is a function shown in figure A.7 together with the second mode contribution $q_{c2}(z)$, and

$A_1(t)$ is the acceleration response of a simple oscillator with the first mode frequency and the appropriate damping value, when subjected to an accelerating excitation $A_g(t)$.

The period of oscillation of the first convective mode is:

$$T_1 = 2\,p\left(\frac{L/g}{\frac{p}{2}\tanh\left(\frac{p}{2}\frac{H}{L}\right)}\right)^{1/2} \tag{A.46}$$

The shear stress at the base and the moment at the foundation can be evaluated based on the expressions (A.44) and (A.45). The values of the masses $m_i$ and $m_{c1}$, as well as those of the corresponding heights above the foundation, $h'_i$ and $h_{c1}$, calculated for cylindrical tanks and given respectively by expressions (A.4), (A.12) and (A.6), (A.14), can also be adopted for the calculation of rectangular tanks (replacing $R$ by $L$), with an error of less than 15 %.

### A.4.2 Flexible rectangular tanks supported on the ground, attached to the foundation

As in the case of cylindrical tanks of circular cross-section, the flexibility of the walls generally results in a significant increase in impulsive pressures, while convective pressures remain practically unchanged. Studies on the seismic response of flexible rectangular tanks are few and their results cannot be expressed in a form suitable for direct application in the calculation. For the calculation, one approach is to use the same vertical pressure distribution as for rigid walls (see expression (A.44) and Figures A.5 and A.6), but replacing the soil acceleration $A_g(t)$ in expression (A.44) by the acceleration response of a simple oscillator having the frequency and damping rate of the first liquid-reservoir impulsive mode.

This vibration period can be approximated by:

$$T_f = 2\,\pi\left(d_f/g\right)^{1/2} \tag{A.47}$$

where

$d_f$ is the deflection of the wall about the vertical axis at the height of the driving mass, when the wall is stressed by a uniform load of magnitude $m_i g/4BH$ in the direction of ground motion; $m_i g/4BH$;

$2B$ is the width of the tank perpendicular to the direction of seismic action.

The impulsive mass $m_i$ can be obtained as the sum of that obtained from expression (A.4), from Figure A.2(a) or from column 4 of Table A.2, plus the mass of the wall.

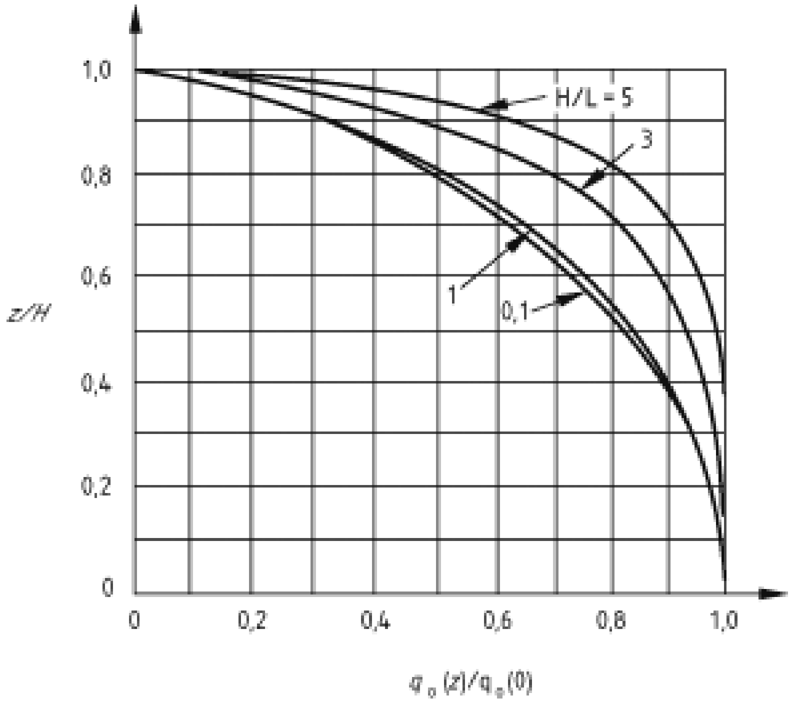



**Figure A.5 - Distribution in height of the dimensionless impulsive pressures acting on the wall perpendicular to the horizontal component of the seismic action of a rectangular tank**

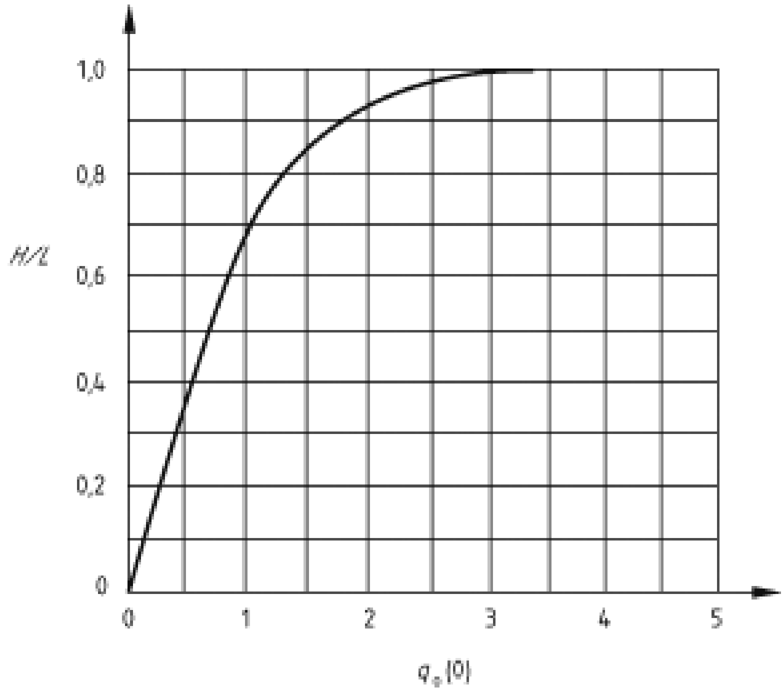



**Figure A.6 - Maximum value of dimensionless impulsive pressures acting on a rectangular wall perpendicular to the horizontal component of the seismic action of a rectangular tank**

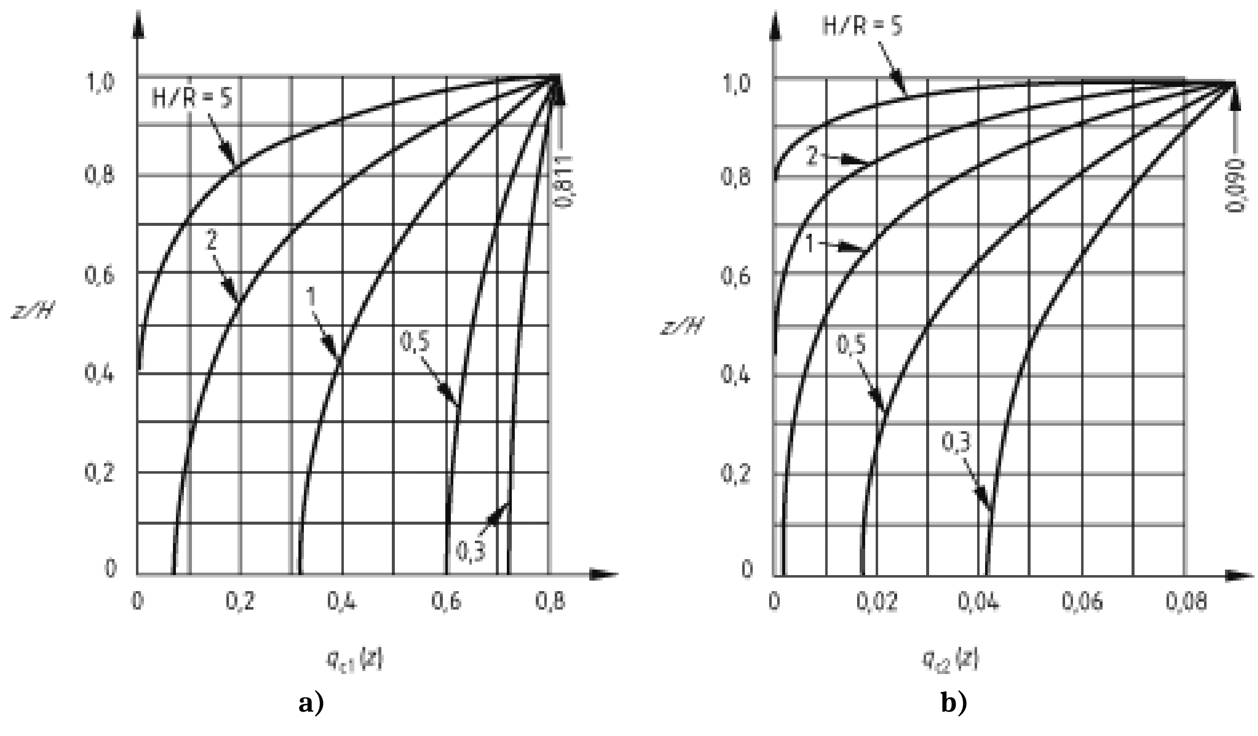
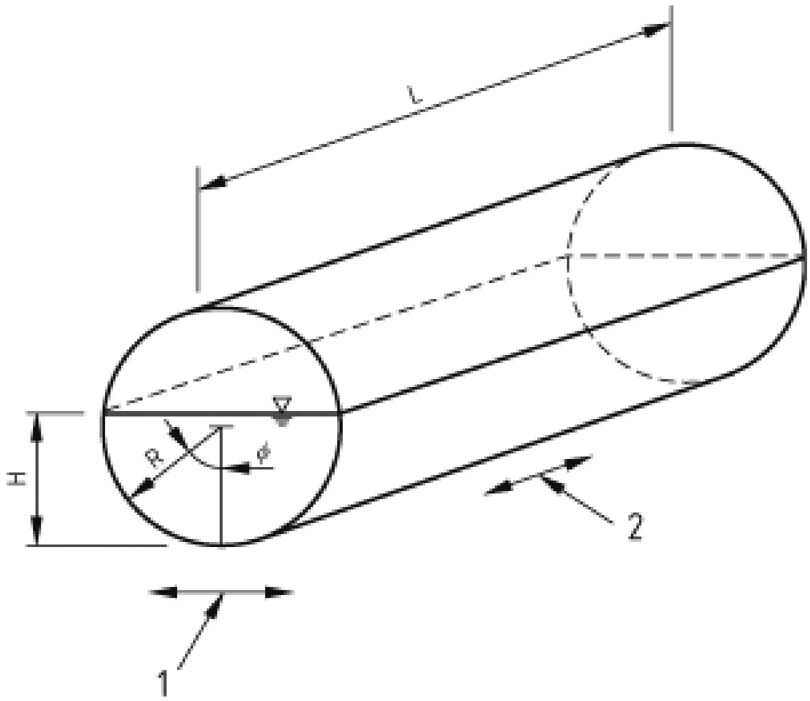



**Figure A.7 - Dimensionless convective pressures acting on the wall perpendicular to the horizontal component of the seismic action of a rectangular tank**

### A.4.3 Combination of the effects of the action due to the different components and actions

Section **A.2.1.6** is applied in relation to the different hysteresis energy dissipation mechanisms (and the associated values of the behavioural coefficient $q$) characterising the different pressure components. Section **A.2.2** can be applied to evaluate the effects of the vertical component of the seismic action and Section **A.2.3** for the combination of the effects of the horizontal and vertical components, including the effects of the other actions for the calculated seismic situation.




Legend:

1 Seismic action in transverse direction

2 Seismic action in longitudinal direction

**Figure A.8 - Notation for a cylindrical tank with horizontal axis**

## A.5 Horizontal cylindrical tanks supported on the ground

Horizontal cylindrical tanks should be calculated for seismic action along the longitudinal and transverse axes (see Figure A.8 for notations).

Approximate values of the hydrodynamic pressures induced by the seismic action, both in the longitudinal and transverse directions, can be obtained by considering a rectangular tank with the same depth at the liquid level, with the same dimension as the real one in the direction of the seismic action, and with a third dimension (width) such that the liquid volume is maintained. This approximation is sufficiently accurate for calculation purposes in the range of $H/R$ between 0.5 and 1.6. If $H/R$ exceeds 1.6, the tank should be assumed to behave as if it were full, i.e. with the total mass of the fluid acting in solidarity with the tank.

In the following, a more accurate solution for seismic action in the transverse direction (perpendicular to the axis) in partially filled reservoirs is described.

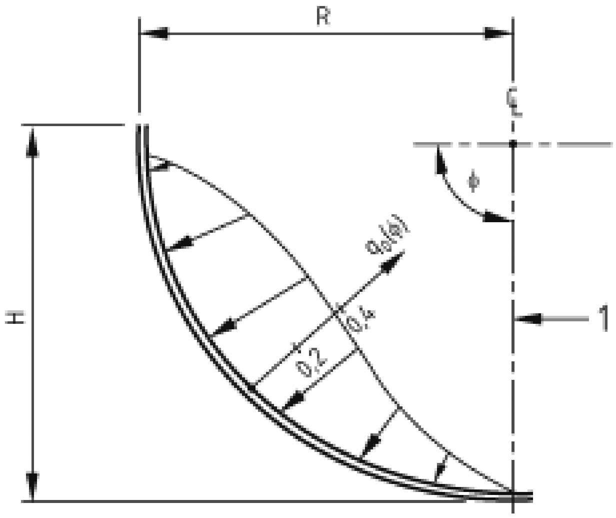
The impulsive pressure distribution is given by:

$$p_i(\varphi) = q_o(\varphi)\ \gamma\ R\ A_g(t) \tag{A.48}$$

For $H = R$ the pressure function $q_o(\cdot)$ takes the form:

$$q_o(\varphi) = \frac{4}{\pi} \sum_{1=n}^{\infty} \frac{(-1)^{n-1}}{(2n)^2 - 1} \operatorname{sen} 2n\,\varphi \tag{A.49}$$

which is represented graphically in Figure A.9.




Legend:

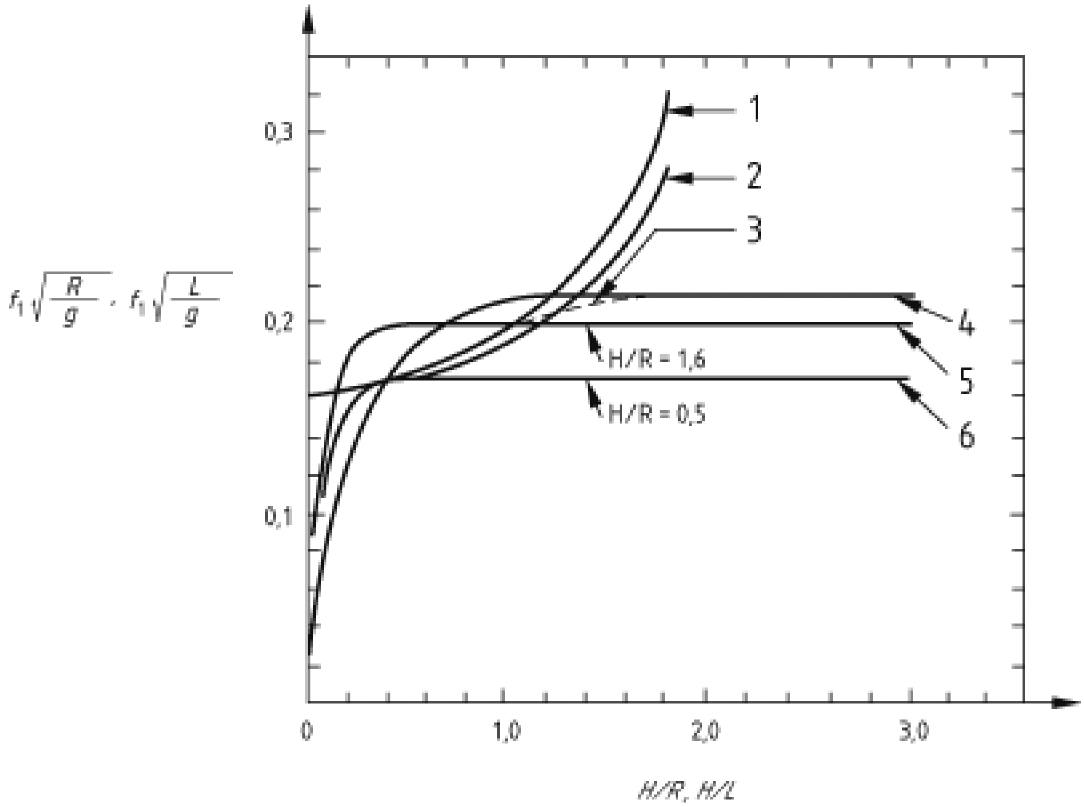
1 Anti-symmetric pressure relative to the cylinder axis

**Figure A.9 – Impulsive pressures acting on a horizontal cylinder with *H* = *R*. Transverse seismic action**

Integrating the pressure distribution, the impulsive mass for $H = R$ becomes:

$$m_i = 0.4\ m \tag{A.50}$$

As the pressures are exerted in the radial direction, the forces acting on the cylinder pass through the centre of the circular section and it should be assumed that both impulsive and convective masses act at this point.




Legend:

1 Sphere

2 Horizontal cylinder, transverse seismic action

3 Vertical cylinder, spherical base

4 Vertical cylinder

5 Rectangular tank (length: $2L$)

5 and 6 Horizontal cylinder, longitudinal seismic action (length: $2L$)

**Figure A.10 – Dimensionless frequency of the first convective mode for rigid vessels of different shapes**

There are no solutions available for the convective pressures of the shape that are suitable for the calculation. When the tank is approximately half-filled ($H \cong R$), the mass of the first convective mode can be evaluated as:

$$m_{c1} = 0.6\ m \qquad \text{(A.51)}$$

The expressions (A.50) and (A.51) are considered to be reasonable approximations for $H/R$ values between 0.8 and 1.2.

Figure A.10 plots the frequencies of the first convective mode for rigid reservoirs of different shapes, including horizontal cylinders with the seismic action acting in the transverse or longitudinal directions relative to their axis.

## A.6 Elevated tanks

In the structural model including also the supporting structure, the fluid in the tank can be taken into account by considering two masses:

- an impulsive mass $m_i$ rigidly attached to the tank walls, located at a height $h'_i$ or $h_i$ above the tank bottom (expressions (A.4) and, respectively, (A.6a), (A.6b));

- a mass $m_{c1}$, attached to the walls by a spring of rigidity $K_{c1} = \omega^2_{c1}m_{c1}$, where the value of $\omega_{c1}$ is given by expression (A.9), located at height $h'_{c1}$ or $h_{c1}$ (expressions (A.12) and respectively (A.14a), (A.14b)).

The system response can be evaluated by the usual methods of modal analysis and spectral response.

In the simplest case, the global model has only two degrees of freedom, corresponding to the masses $m_i$ and $m_{c1}$. A mass $\Delta m$ equal to the tank mass and an appropriate part of the support mass should be added to $m_i$. The mass ($m_i$ + $\Delta m$) should be attached to the terrain by means of a spring representing the rigidity of the support.

Normally the rotational inertia of the mass ($m_i$ + $\Delta m$) and the corresponding additional degree of freedom should also be included in the model.

An elevated tank in the shape of an inverted truncated cone can be considered in the model as an equivalent cylinder, with the same volume of liquid and a diameter equal to the diameter of the cone at the liquid level.

## A.7 Soil-structure interaction effects in ground-supported deposits

### A.7.1 General considerations

The base movement of cemented deposits in relatively deformable soils can be significantly different from free-field movement; In general, the translational component is modified and a rocking component appears. Furthermore, for the same excitation motion, as the flexibility of the terrain increases, the fundamental period of the reservoir-fluid system and the total damping also increase, reducing the maximum of the response. The increase in period is more pronounced in tall and slender reservoirs, as the contribution of the roll component is greater. However, the reduction in maximum response is generally smaller in tall tanks, as the damping associated with rocking is smaller than that associated with a horizontal translation.

In the specialised technical literature, a simple procedure initially proposed for buildings is extended for the impulsive (rigid and flexible) components of the response of tanks by increasing the fundamental period and the damping of the structure, which is considered to rest on a rigid soil and is subjected to free-field motion. It is assumed that the soil-structure interaction does not affect the periods and pressures of the convective mode. A good approximation can be obtained by using an equivalent simple oscillator, with the parameters adjusted to correspond to the frequency and maximum response of the real system. For such a method, different authors give the properties of this equivalent oscillator, in the form of diagrams as a function of the $H/R$ ratio, for fixed values of other parameters: wall thickness ratio $s/R$, initial damping, etc.

### A.7.2 Simple procedure

#### A.7.2.1 Introduction

An approximate procedure can be adopted and is summarised below. It consists of modifying separately the frequency and damping of the rigid impulsive and flexible impulsive pressure contributions considered in chapters **A.2** to **A.5**. In particular, for the impulsive rigid pressure components, whose time domain responses are given by the horizontal free-field accelerations $A_g(t)$, and vertical accelerations $A_v(t)$, the inclusion of soil-structure interaction effects is equivalent to replacing these time domain responses by the acceleration response functions of a one degree of freedom oscillator having a natural period and damping as specified below.

### A.7.2.2 Modified natural periods:

— ‘rigid tank’, horizontal impulsive effect

$$T_i^{¿}=2\pi\left(\frac{m_i+m_o}{k_x\,\alpha_x}+\frac{m_i\,h_i^{'2}}{k_\theta\,\alpha_\theta}\right)^{1/2} \tag{A.52}$$

— ‘deformable tank’, horizontal impulsive effect

$$T_f^{¿}=T_f\left(1+\frac{k_f}{k_x\,\alpha_x}\cdot\left[1+\frac{k_x\,h_f^2}{k_\theta\,\alpha_\theta}\right]\right)^{1/2} \tag{A.53}$$

— ‘rigid tank’, vertical

$$T_{vr}^{¿}=2\pi\left(\frac{m_{\text{tot}}}{k_v\,\alpha_v}\right)^{1/2} \tag{A.54}$$

— ‘deformable tank’, vertical

$$T_{vd}^{¿}=T_{vd}\left(1+\frac{k_l}{k_v\,\alpha_v}\right)^{1/2} \tag{A.55}$$

where

$m_i$, $h'_i$ are the mass and height of the impulsive component;

$m_o$ is the mass of the foundation;

$k_f$ is the rigidity of the ‘deformable tank’ = $4\pi^2\frac{m_f}{T_f^2}$ ;

$m_{tot}$ is the total mass of the filled tank, including the foundation;

$k_l$ = $4\pi^2 \frac{m_l}{T_{vd}^2}$ , with $m_l$ = mass of the liquid;

$k_x$, $k_\theta$, $k_v$ are the horizontal, rolling and vertical stiffnesses of the foundation; and

$\alpha_x$, $\alpha_\theta$, $\alpha_v$ are frequency-dependent coefficients that convert static rigidities into dynamic rigidities.

### A.7.2.3 Modified damping values:

The general expression for the effective damping ratio of the reservoir-citation system is:

$$\xi = \xi_s + \frac{\xi_m}{\left(T'/T\right)^3} \tag{A.56}$$

where

$\xi_s$ is the radiation damping in the soil; and

$\xi_m$ is the damping of the material in the tank.

Both $\xi_s$ and $\xi_m$ depend on the specific mode of vibration.

In particular for $\xi_s$:

— for the horizontal impulsive mode of the ‘rigid tank’:

$$\xi_s = \frac{2\pi^2 m_i}{k_x T_i'^2} a \left( \frac{\beta_x}{\alpha_x} + \frac{k_x h_i'^2 \beta_\theta}{k_\theta \alpha_\theta} \right) \tag{A.57}$$

— for the horizontal impulsive mode of the ‘deformable tank’:

$$\xi_s = \frac{2\pi^2 m_f}{k_x T_f'^2} a \left( \frac{\beta_x}{\alpha_x} + \frac{k_x h_f^2 \beta_\theta}{k_\theta \alpha_\theta} \right) \tag{A.58}$$

— for the vertical mode of the ‘rigid tank’:

$$\xi_s = \frac{2\pi^2 m_{tot}}{k_v T_{vr}^{2}}\, a \frac{\beta_v}{\alpha_v} \tag{A.59}$$

where

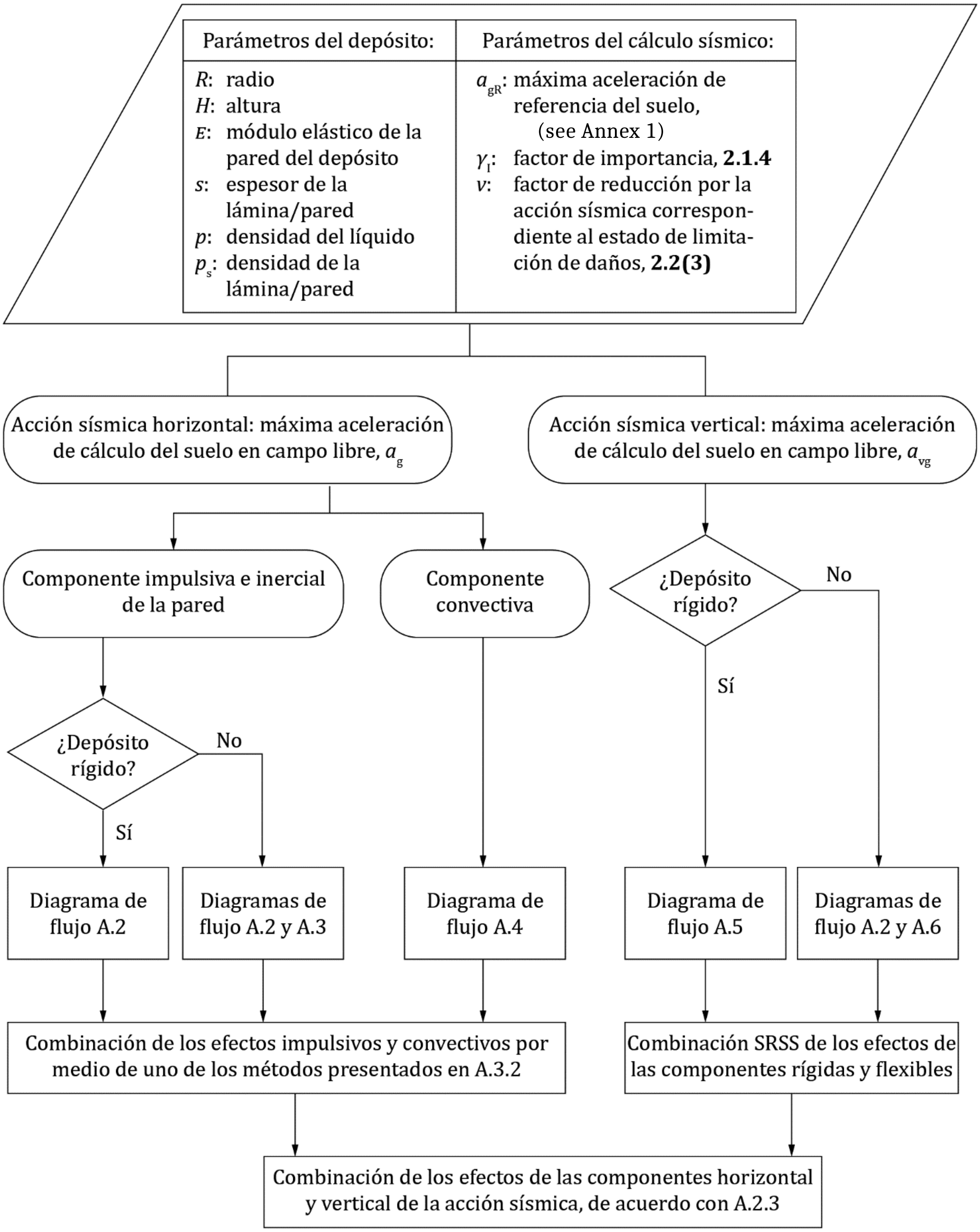
$a$ is the dimensionless frequency = $\frac{2\pi R}{V_s T}$ ($V_s$ = velocity of shear waves in the ground);

$\beta_x, \beta_\theta, \beta_v$ are the frequency-dependent coefficients that provide the radiation damping values for horizontal, vertical and roll motions.

## A.8 Flowcharts for the calculation of hydrodynamic effects in vertical cylindrical tanks

The following flow diagrams present an overview of the determination of hydrodynamic effects in vertical cylindrical tanks subjected to horizontal and vertical seismic actions. These diagrams essentially concern the application of the response spectra method.

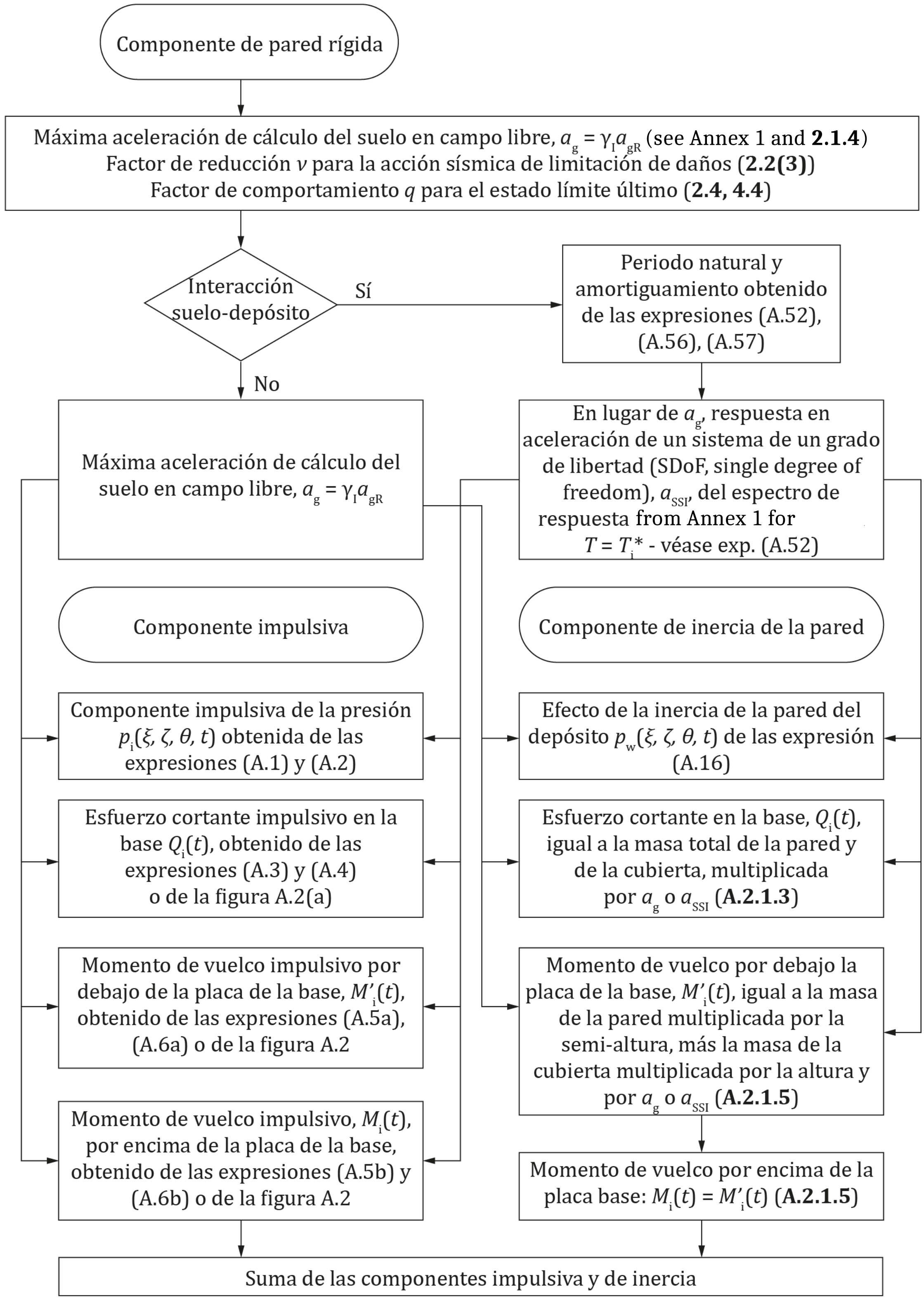
Flowchart A.1 provides an overview of the calculation process and the combination of the different response components. Flowcharts A.2 to A.6 deal with the different hydrodynamic components or components of the seismic action.




| Parámetros del depósito: | Tank parameters: |
|---|---|
| *R*: radio<br>*H:* altura<br>E: módulo elástico de la pared del depósito | *R*: radius<br>*H*: height<br>E: elastic modulus of tank wall |

| | |
|---|---|
| s: espesor de la lámina/pared<br>*p:* densidad del líquido<br>$p_s$: densidad de la lámina/pared | s: thickness of sheet/wall<br>*p*: liquid density<br>$p_s$: density of sheet/wall |
| Parámetros del cálculo sísmico: | Seismic calculation parameters: |
| $a_{gR}$: máxima aceleración de referencia del suelo, (véase Anejo 1)<br>$\gamma_1$: factor de importancia, **2.1.4**<br>v: factor de reducción por la acción sísmica correspondiente al estado de limitación de daños, **2.2(3)** | $a_{gR}$: maximum reference soil acceleration, (see Annex 1).<br>$_1$: importance factor, **2.1.4**<br>v: reduction factor for seismic action corresponding to the damage limitation state, **2.2(3)** |
| Acción sísmica horizontal: máxima aceleración de cálculo del suelo en campo libre, $a_g$ | Horizontal seismic action: maximum calculated acceleration of the ground in free field, $a_g$ |
| Acción sísmica vertical: máxima aceleración de cálculo del suelo en campo libre, $a_{vg}$ | Vertical seismic action: maximum free-field calculated ground acceleration, $a_{vg}$ |
| Componente impulsiva e inercial de la pared | Impulsive and inertial wall component |
| Componente convectiva | Convective component |
| ¿Depósito rígido? | Rigid tank? |
| No | No |
| Sí | Yes |
| Diagrama de flujo A.2 | Flowchart A.2 |
| Diagramas de flujoA.2 y A.3 | Flowcharts A.2 and A.3 |
| Diagrama de flujo A.4 | Flowchart A.4 |
| Diagrama de flujo A.5 | Flowchart A.5 |
| Diagramas de flujoA.2 y A.6 | Flowcharts A.2 and A.6 |
| Combinación de los efectos impulsivos y convectivos por medio de uno de los métodos presentados en A.3.2 | Combination of impulsive and convective effects by one of the methods presented in A.3.2. |
| Combinación SRSS de los efectos de las componentes rígidas y flexibles | SRSS combination of rigid and flexible component effects. |
| Combinación de los efectos de las componentes horizontal y vertical de la acción sísmica, de acuerdo con A.2.3 | Combination of the effects of the horizontal and vertical components of the seismic action, in accordance with A.2.3. |

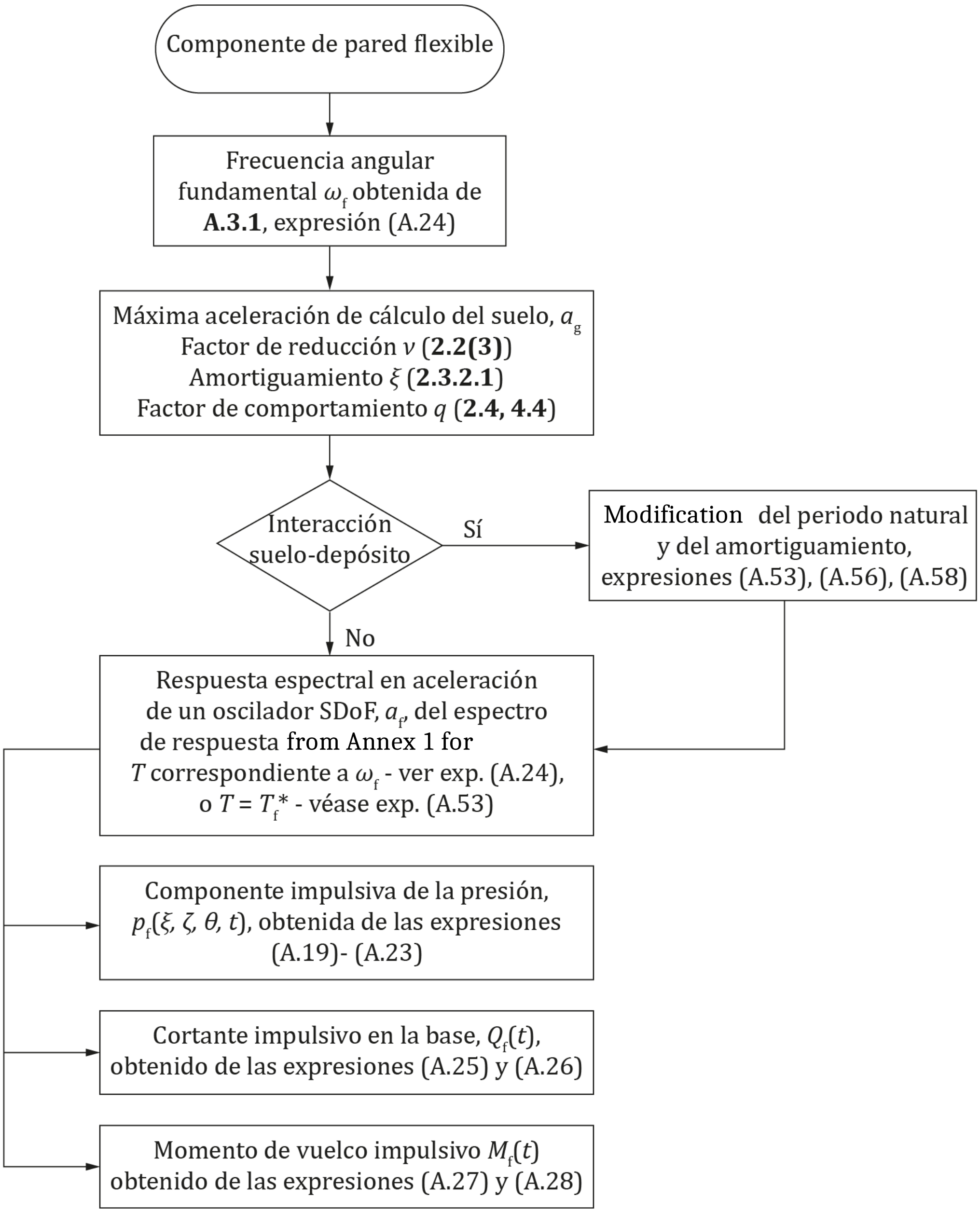
**Flowchart A.1: Overview/comprehensive view of the determination of hydrodynamic effects in vertical cylindrical tanks anchored on the terrain, taking into account soil-structure interaction**




| Componente de pared rígida | Rigid wall component |
|---|---|
| Máxima aceleración de cálculo del suelo en campo libre, $a_g = \gamma_1 a_{gR}$ (véase Anejo 1 y **2.1.4**)<br>Factor de reducción $v$ para la acción sísmica de | Maximum free-field design acceleration of the ground, $a_g = \gamma_1 a_{gR}$ (see Annex 1 and **2.1.4**).<br>Reduction factor $v$ for damage limiting seismic action |

| | |
|---|---|
| limitación de daños **(2.2(3))**<br>Factor de comportamiento *q* para el estado límite último **(2.4, 4.4)** | **(2.2(3))**<br>Behavioural factor *q* for the ultimate limit state **(2.4, 4.4)** |
| Interacción suelo-depósito | Soil-deposit interaction |
| Sí | Yes |
| No | No |
| Periodo natural y amortiguamiento obtenido de las expresiones (A.52), (A.56), (A.57) | Natural period and damping obtained from expressions (A.52), (A.56), (A.57) |
| Máxima aceleración de cálculo del suelo en campo libre, $a_g = \gamma_1 a_{gR}$ | Maximum free-field design acceleration of soil, $a_g = \gamma_1 a_{gR}$ |
| En lugar de $a_g$, respuesta en aceleración de un sistema de un grado de libertad (SDoF, single degree of freedom), $a_{SSI}$, del espectro de respuesta del Anejo 1 para $T=T_i^*$ - véase exp. (A.52) | Instead of $a_g$, acceleration response of a single degree of freedom (SDoF) system, $a_{SSI}$, from the response spectrum of Annex 1 for $T=T_i^*$ - see exp. (A.52) |
| Componente impulsiva | Impulsive component |
| Componente de inercia de la pared | Inertia component of the wall |
| Componente impulsiva de la presión $p_i(\xi, \zeta, \theta, t)$ obtenida de las expresiones (A.1) y (A.2) | Impulsive component of the pressure $p_i(\xi, \zeta, \theta, t)$ obtained from expressions (A.1) and (A.2) |
| Efecto de la inercia de la pared del depósito $p_w(\xi, \zeta, \theta, t)$ de la expresión (A.16) | Effect of tank wall inertia $p_w(\xi, \zeta, \theta, t)$ from expression (A.16) |
| Esfuerzo cortante impulsivo en la base $Q_i(t)$- obtenido de las expresiones (A.3) y (A.4) o de la figura A.2(a) | Impulsive shear stress in the base $Q_i(t)$ - obtained from expressions (A.3) and (A.4) or from Figure A.2(a) |
| Esfuerzo cortante en la base, $Q_i(t)$, igual a la masa total de la pared y de la cubierta, multiplicada por $a_g$ o $a_{SSI}$ **(A.2.1.3)** | Base shear stress, Qi(t), equal to the total mass of the wall and deck, multiplied by $a_g$ or $a_{SSI}$ **(A.2.1.3)** |
| Momento de vuelco impulsivo por debajo de la placa de la base, $M'_i(t)$, obtenido de las expresiones (A.5a), (A. 6a) o de la figura A.2 | Impulsive overturning moment under the base plate, $M'_i(t)$, obtained from expressions (A.5a), (A. 6a) or Figure A.2 |
| Momento de vuelco impulsivo, $M_i(t)$, por encima de la placa de la base, obtenido de las expresiones (A.5b), (A. 6b) o de la figura A.2 | Impulsive overturning moment, $M_i(t)$, above the base plate, obtained from expressions (A.5b), (A. 6b) or Figure A.2. |
| Momento de vuelco por debajo la placa de la base, $M'_i(t)$, igual a la masa de la pared multiplicada por la semi-altura, más la masa de la cubierta multiplicada por la altura y por $a_g$ o $a_{SSI}$ **(A.2.1.5)** | Overturning moment below the base plate, $M'_i(t)$, equal to the mass of the wall multiplied by the half-height, plus the mass of the deck multiplied by the height and by $a_g$ or $a_{SSI}$ **(A.2.1.5)** |
| Momento de vuelco por encima de la placa base: $M_i(t) = M'_i(t)$ **(A.2.1.5)** | Overturning moment above the base plate: $M_i(t) = M'_i(t)$ **(A.2.1.5)** |
| Suma de las componentes impulsiva y de inercia | Sum of the impulse and inertia components |

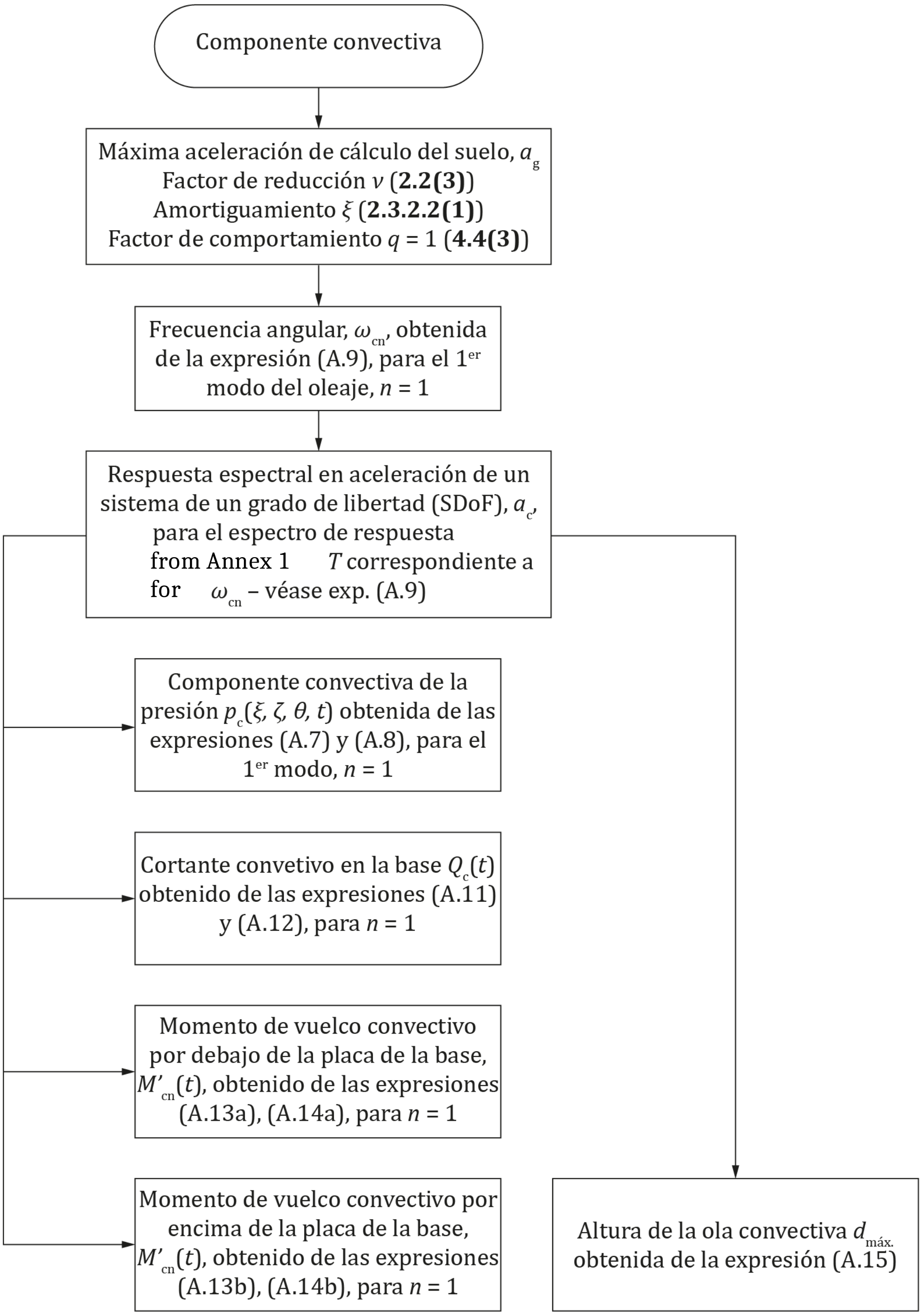
**Flowchart A.2: Horizontal seismic action, impulsive component for a rigid wall (see A.2.1, A.7.2)**




| | |
|---|---|
| Componente de pared flexible | Flexible wall component |
| Frecuencia angular fundamental $\omega_f$ obtenida de **A.3.1,** expresión (A.24) | Basic angular frequency $\omega_f$ obtained from **A.3.1**, expression (A.24) |
| Máxima aceleración de cálculo del suelo, $a_g$<br>Factor de reducción $v$ **(2.2(3))**<br>Amortiguamiento $\xi$ **(2.3.2.1)**<br>Factor de comportamiento $q$ **(2.4, 4.4)** | Maximum calculated ground acceleration, $a_g$<br>Reduction factor $v$ **(2.2(3))**<br>Damping $\gamma$ **(2.3.2.1)**<br>Behavioural factor $q$ **(2.4, 4.4)** |
| Interacción suelo-depósito | Soil-deposit interaction |
| Sí | Yes |
| No | No |
| Modificación del periodo natural y del amortiguamiento, expresiones (A.53), (A.56), (A.58) | Modification of natural period and damping, expressions (A.53), (A.56), (A.58) |
| Respuesta espectral en aceleración de un oscilador SDoF, $a_f$, del espectro de respuesta del Anejo 1 para $T$ correspondiente a $\omega_f$ - ver exp. (A.24), *o* $T$=$T_f$* - véase | Acceleration spectral response of an SDoF oscillator, $a_f$, from the response spectrum of Annex 1 for $T$ corresponding to $\omega_f$ - see exp. (A.24), *or* $T$=$T_f$ * - see |

| | |
|---|---|
| exp. (A.53) | exp. (A.53) |
| Componente impulsiva de la presión, $p_i(\xi, \zeta, \theta, t)$, obtenida de las expresiones (A.19) - (A.23) | Impulsive pressure component, $p_i(\xi, \zeta, \theta, t)$, obtained from expressions (A.19) - (A.23) |
| Cortante impulsivo en la base, $Q_f(t)$, obtenido de las expresiones (A.25) y (A.26) | Impulsive base overturning shear, $Q_f(t)$, obtained from expressions (A.25) and (A.26) |
| Momento de vuelco impulsivo $M_f(t)$ obtenido de las expresiones (A.27) y (A.28) | Impulsive overturning moment, $M_f(t)$, obtained from expressions (A.27) and (A.28) |

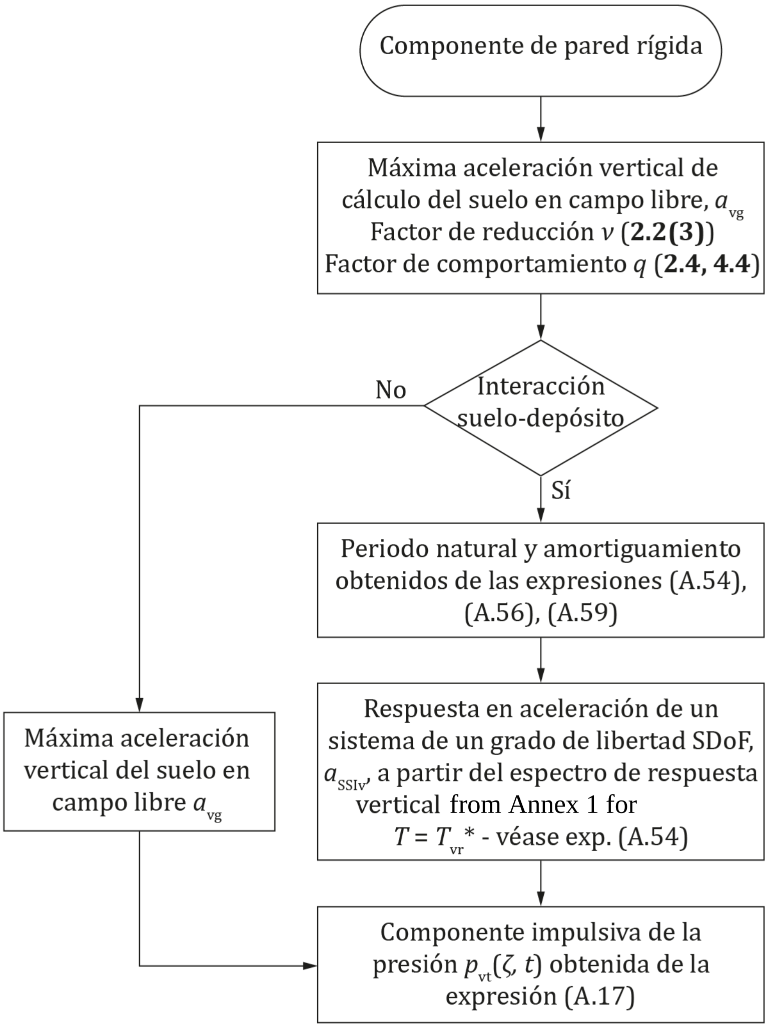
**Flowchart A.3: Horizontal seismic action, impulsive component for a flexible wall (see A.3.1, A.7.2)**




| Componente convectiva | Convective component |
|---|---|
| Máxima aceleración de cálculo del suelo, $a_g$<br>Factor de reducción $\nu$ (**2.2(3)**)<br>Amortiguamiento $\xi$ (**2.3.2.2(1)**)<br>Factor de comportamiento $q$ = 1 (**4.4(3)**) | Maximum calculated ground acceleration, $a_g$<br>Reduction factor $\nu$ (**2.2(3)**)<br>Damping $\xi$ (**2.3.2.2(1)**)<br>Behavioural factor $q$ = 1 (**4.4( 3)**) |
| Frecuencia angular, $\omega_{cn}$, obtenida de la expresión (A.9), para el 1er modo del oleaje, $n$ = 1 | Angular frequency, $\omega_{cn}$,, obtained from expression (A.9), for the 1st wave mode, $n$ = 1 |
| Respuesta espectral en aceleración de un sistema de un grado de libertad (SDoF), $a_c$, para el espectro de | Acceleration spectral response of a one degree of freedom (SDoF) system, $a_c$, for the response spectrum |

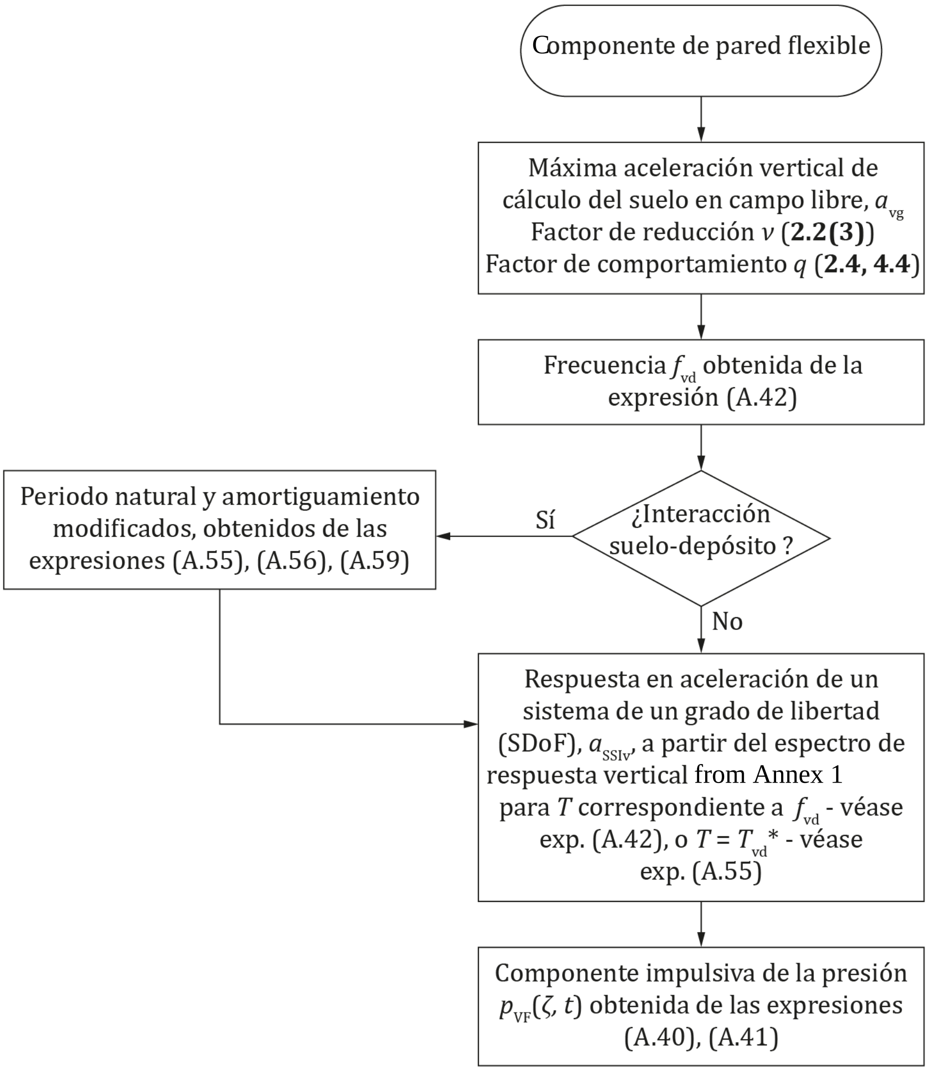
| | |
|---|---|
| respuesta del Anejo 1 para T correspondiente a $\omega_{cn}$- véase exp. (A.9) | of Annex 1 for T corresponding to $\omega_{cn}$- see exp. (A.9) |
| Componente convectiva de la presión, $p_c(\xi, \zeta, \theta, t)$, obtenida de las expresiones (A.7) y (A.8), para el 1$^{er}$ modo, $n$ = 1 | Convective pressure component, $p_c(\xi, \zeta, \theta, t)$, obtained from expressions (A.7) and (A.8), for the 1$^{st}$ mode, $n$ = 1 |
| Cortante convetivo en la base $Q_c$(t) obtenido de las expresiones (A.11) y (A.12), para $n$ = 1 | Convective base shear $Q_c$(t) obtained from expressions (A.11) and (A.12), for $n$ = 1 |
| Momento de vuelco convectivo por debajo de la placa de la base, $M'_{cn}$(t), obtenido de las expresiones (A.13a), (A.14a), para $n$ = 1 | Convective overturning momentum under the base plate, $M'_{cn}$(t), obtained from expressions (A.13a), (A.14a), for $n$ = 1 |
| Momento de vuelco convectivo por encima de la placa de la base, $M'_{cn}$(t), obtenido de las expresiones (A.13b), (A.14b), para $n$ = 1 | Convective overturning momentum above the base plate, $M'_{cn}$(t), obtained from expressions (A.13b), (A.14b), for $n$ = 1 |
| Altura de la ola convectiva $d_{max.}$ obtenida de la expresión (A.15) | Convective wave height $d_{max.}$, obtained from expression (A.15) |

**Flowchart A.4: Horizontal seismic action, convective component (see A.2.1)**




| | |
|---|---|
| Componente de pared rígida | Rigid wall component |
| Máxima aceleración vertical de cálculo del suelo en campo libre, $a_{vg}$<br>Factor de reducción $v$ **(2.2(3))**<br>Factor de comportamiento $q$ **(2.4, 4.4)** | Maximum calculated vertical acceleration of the ground in free field, $a_{vg}$<br>Reduction factor $v$ **(2.2(3))**<br>Behavioural factor $q$ **(2.4, 4.4)** |
| No | No |
| Sí | Yes |
| Interacción suelo-depósito | Soil-deposit interaction |
| Periodo natural y amortiguamiento obtenidos de las expresiones (A.54), (A.56), (A.59) | Natural period and damping obtained from expressions (A.54), (A.56), (A.59) |
| Máxima aceleración vertical del suelo en campo libre $a_{vg}$ | Maximum free-field vertical ground acceleration $a_{vg}$ |
| Respuesta en aceleración de un sistema de un grado de libertad SDoF, $a_{SSIv}$, a partir del espectro de respuesta vertical del Anejo 1 para $T = T_{vr}$* - véase exp. (A.54) | Acceleration response of a one degree of freedom system SDoF, $a_{SSIv}$, from the vertical response spectrum of Annex 1 for $T = T_{vr}$* - see exp. (A.54) |
| Componente impulsiva de la presión $p_{vt}(\zeta$,t) obtenida de laexpresión (A.17) | Impulsive pressure component $p_{vt}(\zeta$,t) obtained from expression (A.17) |

**Flowchart A.5: Vertical seismic action, component for a rigid wall (see A.2.2, A.7.2)**




| | |
|---|---|
| Componente de pared flexible | Flexible wall component |
| Máxima aceleración vertical de cálculo del suelo en campo libre, $a_{vg}$<br>Factor de reducción v **(2.2(3))**<br>Factor de comportamiento $q$ **(2.4, 4.4)** | Maximum calculated vertical acceleration of the ground in free field, $a_{vg}$<br>Reduction factor v **(2.2(3))**<br>Behavioural factor $q$ **(2.4, 4.4)** |
| Frecuencia $f_{vd}$ obtenida de la expresión (A.42) | Frequency $f_{vd}$ obtained from expression (A.42) |
| Periodo natural y amortiguamiento modificados, obtenidos de las expresiones (A.55), (A.56), (A.59) | Modified natural period and damping, obtained from expressions (A.55), (A.56), (A.59) |
| Sí | Yes |
| ¿Interacción suelo-depósito? | Soil-deposit interaction? |
| No | No |
| Respuesta en aceleración de un sistema de un grado de libertad (SDoF), $a_{SSIv}$, a partir del espectro de respuesta vertical del Anejo 1 para $T$ correspondiente a $f_{vd}$ - véase exp. (A.42), o $T = T_{vd}$* - véase exp. (A.55) | Acceleration response of a system of one degree of freedom (SDoF), $a_{SSIv}$, from the vertical response spectrum in Annex 1 for $T$ corresponding to $\mathbf{f_{vd}}$ - see exp. (A.42), or $T = \mathbf{T_{vd}}$* - see exp. (A.55) |
| Componente impulsiva de la presión $p_{vp}(\zeta,t)$ obtenida de las expresiones (A.40), (A.41) | Impulsive component of the pressure $\mathbf{p_{vp}(\zeta,t)}$ obtained from expressions (A.40), (A.41) |

**Flowchart A.6: Vertical seismic action, component for a flexible wall (see A.3.3, A.7.2)**

## A.9 Tanks supported on the ground, not anchored to the foundation

### A.9.1 General considerations

In ground-supported tanks that are not anchored to the foundation, uplift of the tank bottom will occur due to the seismic overturning moment. This uplift is more pronounced in uncovered tanks. Uplift can lead to plastic deformations in the tank, especially in the base plate. However, cracking and fluid leakage should be prevented by appropriate calculated calculations.

In most cases the effects of uplift, and the accompanying rocking motion, on the magnitude and distribution of pressures are not considered. For most cases this is conservative, as rocking increases the flexibility of the system and shifts the period to a range of lower dynamic amplification of the forces.

To deal with this case, an approximate iterative procedure for calculating vertical cylindrical tanks can be used, taking into account the uplift and dynamic nature of the problem. The flow diagrams obtained by this procedure are applied to tanks with a fixed shell and refer to specific values of parameters, such as the ratio of wall thickness to radius, soil rigidity, type of wall foundation, etc.

Once the maximum hydrodynamic pressures are known, whether or not they have been determined considering uplift, the calculation of the stresses in the tank is a static structural design problem, where the designer has a certain freedom of choice in the level of sophistication of the method they use. For a tank to be lifted, an accurate model should necessarily involve a non-linear finite element model for the tank, the soil and its contact surface. More refined experiments and calculations have shown that direct methods that do not require the use of computers are neither conservative nor adequate to take into account all the variables involved in the problem.

The main effect of uplift is to increase the vertical compressive stress in the wall lamina, which is critical for buckling rupture modes. On the side of the wall opposite to that which is lifted, the vertical compression is maximised and circumferential compressive stresses occur in the sheet due to the membrane action of the base plate.

Bending plasticisation of the base plate is accepted and testing of the maximum tensile stress is considered appropriate.

### A.9.2 Vertical compressive membrane forces and stresses in the wall due to uplift

For the cylindrical steel tanks with fixed deck supported on the ground that are common in the petrochemical industry, the increase of the vertical membrane force due to uplift ($N_u$) with respect to the stress in the anchored case ($N_a$) can be estimated from Figure A.11, as a function of the dimensionless overturning moment $M/WH$ ($W$ = total liquid weight). This increase is very significant for slender tanks. For fixed decks, the values given in figure A.11 are on the safe side, as they have been calculated (by static finite element analysis) assuming that the bearing soil is very stiff (the ballast modulus of the soil in the Winkler model being $k$ = 4000 MN/m$^3$),which is an unfavourable situation for vertical membrane forces.

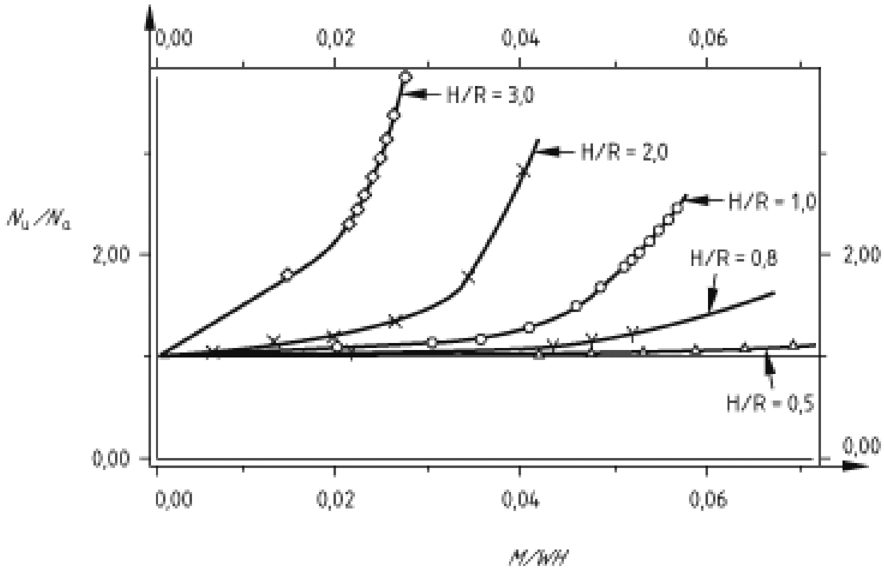



**Figure A.11 - Relationship between the maximum axial compressive diaphragm force for cylindrical tanks with fixed roof supported on the ground, not anchored, and that for anchored tanks, as a function of the overturning moment**

### A.9.3 Wall sheet uplift and base plate uplift length

Figure A.12 shows, as a function of the overturning moment $M/WH$ for different values of $H/R$, the value of the vertical uplift at the base edge, $w$, as obtained from a parametric study with finite element models for unanchored ground-supported cylindrical steel tanks of usual geometry and with a fairly heavily loaded fixed deck. The results in Figure A.12 would underestimate the uplift in uncovered or floating deck tanks.

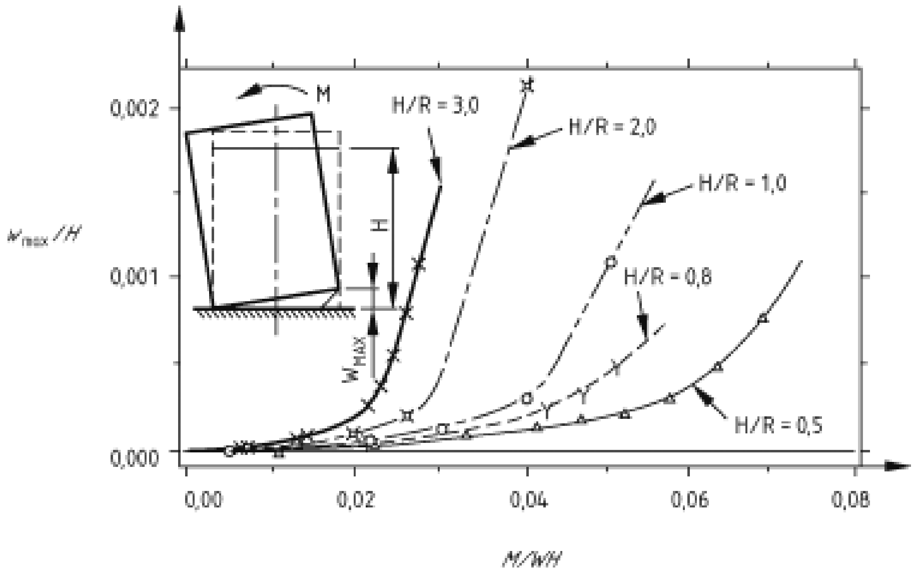
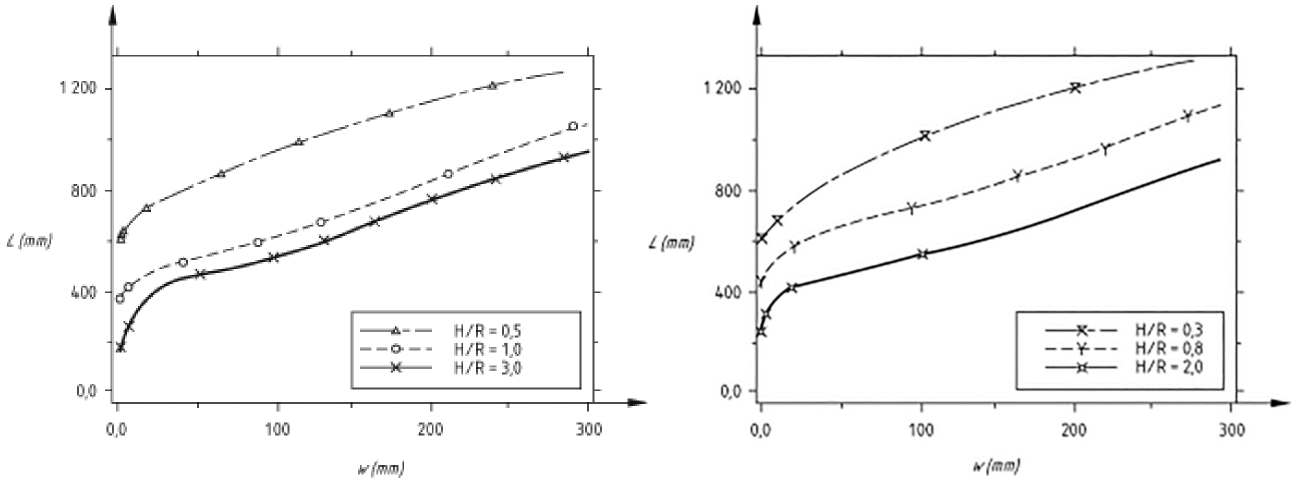



**Figure A.12 - Maximum uplift value of cylindrical tanks with fixed deck supported on the ground and not anchored as a function of the overturning moment *M*/*WH***

In order to evaluate the radial diaphragm stresses in the plate, it is necessary to know the length $L$ of the lifted part of the tank bottom. Figure A.13 shows the results obtained for tanks with a fixed roof. Once uplift occurs, the dependence of $L$ on the vertical uplift $w$ is almost linear.




**a)** **b)**

**Figure A.13 - Length of the raised part of the base in cylindrical tanks with fixed deck supported on the ground and not anchored, as a function of the vertical lift at the rim**

## A.9.4 Radial membrane stresses in the base plate

The following expression allows estimation of the membrane stress $\sigma_{rb}$ in the base plate due to uplift:

$$\sigma_{\rm rb}=\frac{1}{s}\left(\frac{2}{3}\frac{E}{1-\nu^2}sp^2R^2(1-\mu)^2\right)^{1/3} \tag{A.60}$$

where

$s$ is the thickness of the base plate;

*p)* is the pressure at the base;

$\mu$ $=1-L/(2R)$, with $L$ = uplifted part of the base.

When significant uplift occurs in large diameter tanks, the stress state in the uplifted part of the base plate at the ultimate limit state is dominated by the bending of the plate (including the effects of the pressure acting on the tank base) and not by the membrane stresses. In such cases the finite element method should be used for the calculated stress state.

### A.9.5 Plastic rotation of the base plate

It is recommended to calculate the circular bottom ring with a thickness less than the wall thickness, in order to avoid bending plasticisation at the base of the wall.

The rotation of the plastic hinge at the base of the tank should be compatible with the available ductility in bending. Assuming a maximum allowable unit steel deflection of 0.05 and a length of the plastic hinge equal to 2*s*, the maximum allowable rotation is 0.20 rad. According to figure A.14 the rotation associated with an uplift at the edge *w* and a base separation *L* is:

$$\theta=\left(\frac{2w}{L}-\frac{w}{2R}\right) \tag{A.61}$$

which should be less than the estimated rotation capacity of 0.20 rad.

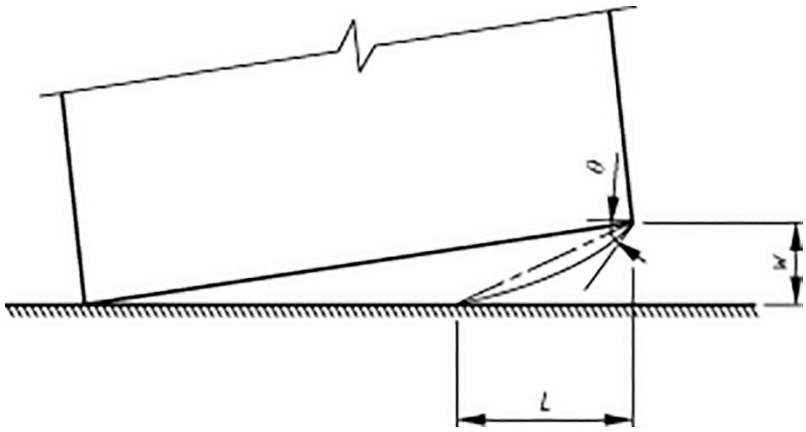


**Figure A.14 - Plastic rotation of the base plate of an uplifted tank**

## A.10 Testing for steel tanks

### A.10.1 Introduction

The integrity of the angle zone between the base plate and the wall of tanks, anchored or unanchored, should be checked under the stresses and unit strains predicted there by the calculated for the calculated seismic situation. In addition, the stability of the tank wall near the base and above the base should be tested for two possible failure modes.

### A.10.2 Testing for elastic buckling

This type of buckling has been observed in those parts of the wall sheet where the thickness is reduced with respect to the thickness of the base and/or the internal pressure (which has a stabilising effect) is also reduced with respect to the maximum value reached at the base. For tanks with constant or variable wall thickness, the elastic buckling testing should be carried out at the base as well as at the wall above the base. Due to the stabilising effect of the internal pressure, the testing should be based on the lowest possible value of the internal pressure in the calculated seismic situation.

The testing can be carried out in accordance with the specific regulations in force or, failing that, with the specific technical documents that the author of the project, under their responsibility, considers most appropriate.

As an alternative, the following inequality can be tested:

$$\frac{\sigma_m}{\sigma_{c1}} \leq 0,19 + 0,81 \frac{\sigma_p}{\sigma_{c1}} \tag{A.62}$$

where

$\sigma_m$ is the maximum vertical membrane stress.

$$\sigma_{c1} = 0,6 \cdot E \frac{s}{R} \tag{A.63}$$

is the ideal critical buckling stress for cylinders stressed in axial compression, and

$$\sigma_p = \sigma_{c1} \left[ 1 - \left( 1 - \frac{\overline{p}}{5} \right)^2 \left( 1 - \frac{\sigma_o}{\sigma_{c1}} \right)^2 \right]^{1/2} \leq \sigma_{c1} \tag{A.64}$$

where

$$\overline{p} = \frac{p\ R}{s\ \sigma_{c1}} < 5 \tag{A.65}$$

with $p$ representing the minimum possible value of the internal pressure for the calculated seismic situation,

$$\sigma_o = f_y \left( 1 - \frac{\lambda^2}{4} \right) \text{ if: } \lambda^2 = \frac{f_y}{\overline{\sigma}\ \sigma_{c1}} \leq 2 \tag{A.66a}$$

$$\sigma_o = \overline{\sigma}\ \sigma_{c1} \qquad \text{si :} \qquad \lambda^2 \geq 2 \tag{A.66b}$$

with

$$\overline{s} = 1 - 1,24 \left( \frac{d}{s} \right) \left[ \left( 1 + \frac{2}{1,24 \left( \frac{d}{s} \right)} \right)^{1/2} - 1 \right] \tag{A.67}$$

and $\delta/s$ denoting the ratio of the maximum amplitude of the imperfections to the wall thickness, which can be taken as:

$$\frac{0{,}06}{a}\sqrt{\frac{R}{s}} \tag{A.68}$$

where

$a$ = 1 for normal constructions;

$a$ = 1.5 for good quality constructions;

$a$ = 2.5 for very good quality constructions.

### A.10.3 Elasto-plastic collapse

This form of buckling (‘elephant’s foot’) normally occurs near the base of the tank, and is due to a combination of vertical compressive stresses and circumferential tensile stresses inducing an inelastic biaxial stress state. In tanks with variable wall thickness, testing for this buckling mode should not be limited to the section near the base of the tank, but should be extended to the bottom section of all parts of the wall having a constant thickness.

The empirical equation developed for testing this form of instability is:

$$\sigma_m = \sigma_{c1}\left[1-\left(\frac{pR}{s\,f_y}\right)^2\right]\left(1-\frac{1}{1{,}12+r^{1{,}15}}\right)\left[\frac{r+f_y/250}{r+1}\right] \tag{A.69}$$

where

$$r = \frac{R/s}{400}$$

$f_y$ is the yield strength of the tank wall material in MPa; and

$p$) is the maximum possible value of the internal pressure for the calculated seismic situation, in MPa.

# Appendix B

# Recommendations for buried pipelines

## B.1 General calculated considerations

(1) As a general rule, pipelines should rest in soils that have been tested for their ability to remain stable under calculated seismic action. Where this condition cannot be satisfied, the nature and extent of the unfavourable phenomena should be explicitly assessed and appropriate remedial measures implemented in the design.

(2) Two extreme cases are worth mentioning: soil liquefaction and fault movements, as they generally require case-specific calculated solutions.

(3) Whenever liquefaction has occurred in past earthquakes, it has been one of the main causes of damage to pipelines.

(4) Depending on the circumstances, the solution may require either increasing the burial depth (possibly by also encasing the pipes in larger rigid conduits) or placing the pipe above ground, supported on columns with a good foundation and located at rather wide intervals. In the latter case, flexible joints should also be considered to allow for relative displacements between supports.

(5) Calculating for fault movements requires estimating, and sometimes postulating, a number of parameters, including: the location, the size of the affected area, the type and the amplitude of the fault displacement. With these parameters, the simplest way to model the phenomenon is to consider a rigid displacement between the contacting soil masses on the fault.

(6) The general criterion for minimising the effect of an imposed displacement is to introduce a maximum of flexibility in the system subjected to the displacement.

(7) In the case under consideration, this can be done:

- by reducing the burial depth, in order to reduce the constraint exerted by the soil;

- by providing a large pit for the pipes, which is filled with a soft material;

- installing the pipe above the terrain and introducing flexible and extensible pipe elements.

## B.2 Seismic actions in buried pipelines

(1) The movement propagating below the ground surface is made up of a combination of internal (compressional, shear) and surface (Rayleigh, Love, etc.) waves. The actual combination of these waves depends particularly on the focal depth and the distance between the focus and the site.

(2) Different types of waves have different propagation velocities and different particle motions (i.e. parallel to the direction of wave propagation, orthogonal to it, elliptical, etc.). Although geophysical and seismological studies can provide some data, in general they cannot predict the actual shape of the waves, so conservative assumptions have to be made.

(3) A frequent assumption is to successively consider that the waveform consists entirely of a single type of waves, the most unfavourable for each particular effect on the pipeline.

(4) In this case it is easy to construct the wave trains based on the frequency content of the elastic response spectrum appropriate to the site, attributing to each frequency component an estimated value of the propagation velocity.

(5) Theoretical arguments and numerous numerical simulations indicate that the inertia forces due to the interaction between the pipe and the soil are much smaller than the forces induced by the deformation of the soil. This fact makes it possible to reduce the soil-pipe interaction problem to a static problem, i.e. a problem in which the pipe is deformed by the passage of a displacement wave, without considering dynamic effects.

(6) Consequently, the forces acting on the pipe can be obtained by a time domain analysis, where time is a parameter whose function is to move the wave along or through the structure, the latter being connected to the ground by radial and longitudinal springs.

(7) A much simpler method is often used, due to Newmark, which has been shown to be of comparable accuracy to the more rigorous procedure described above and which gives, in all cases, an upper bound estimate for the unit strains in the pipe, as it assumes it to be sufficiently flexible to follow, without slip or interaction, the deformation of the soil.

(8) According to this method the ground motion is represented by a single sine wave:

$$u(x,t) = d \operatorname{sen} \omega (t - \frac{x}{c}) \tag{B.1}$$

where *d* is the total amplitude of the displacement and *c* is the apparent velocity of the wave.

(9) The motion of the particle is assumed to occur alternately along the direction of propagation (compression waves) and in the direction normal to it (shear waves). For simplicity, and in order to take the worst case, the axis of the trough is taken to coincide with the direction of propagation.

(10) The longitudinal movement of the particles produces in the soil and in the pipe unit deformations given by the expression:

$$\varepsilon = \frac{\partial u}{\partial x} = -\frac{\omega d}{c} \cos \omega \left(t - \frac{x}{c}\right) \tag{B.2}$$

the maximum value of which is:

$$\varepsilon_{\text{máx.}} = \frac{v}{c} \tag{B.3}$$

where

$v = \omega d$, the maximum soil velocity.

(11) The transverse movement of the particles produces in the soil and in the pipe a curvature $\chi$ which is given by the expression:

$$\chi = \frac{\partial^2 u}{\partial x^2} = - \frac{\omega^2 d}{c^2} \operatorname{sen} \omega (t - \frac{x}{c}) \tag{B.4}$$

the maximum value of which is:

$$\chi_{\max} = \frac{a}{c^2} \tag{B.5}$$

where

$a = \omega^2 d$, the maximum ground acceleration.

(12) For the condition of perfect adhesion between the pipe and the ground to be satisfied, the available frictional force per unit length should balance the variation of the longitudinal force, which leads to:

$$\tau_{av} = s E \frac{a}{c^2} \tag{B.6}$$

where

$E$ modulus of elasticity of the pipe;

$s$ thickness of the pipe; and

$\tau_{av}$ average shear stress between the pipe and the soil, which depends on the coefficient of friction between the soil and the pipe and the burial depth.

# ANNEX 5

# Design of seismically resistant structures

# Foundations, retaining structures and geotechnical aspects

## 1 General considerations

### 1.1 Object and field of application

(1) The scope of the Seismic Resistance Standard is defined in section 1.1.1 of Annex 1. This Annex sets out the requirements, criteria and rules for the location and terrain of the foundation of earthquake-resistant structures. It covers the design and calculated design of different foundation systems, the design of earth retaining structures and soil-structure interaction under seismic actions.

The requirements contained in this Annex are considered to be complementary to the specific regulations and standards applicable to the geotechnical design in the absence of seismic loads.

(2) The specifications contained in this Seismic Resistance Standard are applicable to buildings (Annex 1), bridges (Annex 2), towers, masts and chimneys (Annex 6) and silos, tanks and pipes (Annex 4). The parts of the Seismic Resistance Standard are listed in section 1.1.3 of Annex 1.

(3) Special requirements for the design of the foundations of certain types of structures should be sought, where necessary, in the relevant Annexes of this Seismic Resistance Standard.

(4) Appendix B of this Annex provides empirical graphs for the assessment of liquefaction potential, while Appendix E provides a simplified procedure for the seismic analysis of earth retaining structures.

NOTE 1 Appendix A contains information on topographic amplification coefficients.

NOTE 2 Appendix C contains information on static pile rigidities.

NOTE 3 Appendix D contains information on dynamic soil-structure interaction.

NOTE 4 Appendix F contains information on the bearing capacity of shallow foundations subjected to seismic loading.

NOTE 5 Appendices A, C, D and F of this Annex are not of a regulatory nature.

### 1.2 Standards for reference and consultation

(1) The provisions of section **1.2** of Annex 1 apply.

### 1.3 Assumptions

(1) The provisions of section **1.3** of Annex 1 apply.

## 1.4 International system of units (I.S.)

(1) The provisions of section **1.4** of Annex 1 apply.

## 1.5 Terms and definitions

### 1.5.1 Common terms

(1) The terms and definitions of section **1.4** of Annex 18 of the Structural Code apply.

### 1.5.2 Other terms used in this Annex

(1) Geotechnical terms specifically related to earthquakes (such as liquefaction) are given in the text.

(2) The terms defined in section **1.5.2** of Annex 1 apply.

## 1.6 Symbols

For the purposes of this Annex the following symbols are used. All symbols used in this Annex are defined in the text when they first appear for ease of use. A list of the symbols used is given below. Some symbols that are used only in the appendices are defined in the appendices.

| | |
|---|---|
| $E_{d}$ | Calculated value of the effect of the actions |
| $E_{pd}$ | Lateral resistance of a footing due to passive earth pressure |
| $ER$ | Energy quotient measured in the Standard Penetration Test (SPT) |
| $F_{H}$ | Calculated value of the horizontal seismic inertial force |
| $F_{V}$ | Calculated value of the vertical seismic inertia force |
| $F_{Rd}$ | Calculated value of the frictional force between the horizontal base of the footing and the terrain |
| $G$ | Transverse modulus of elasticity |
| $G_{max.}$ | Transverse modulus of elasticity for small deflections |
| $L_{e}$ | Distance of the anchorages to the wall under dynamic conditions |
| $L_{s}$ | Distance of the anchorages to the wall under static conditions |
| $M_{Ed}$ | Calculated time |
| $N_{1}(60)$ | Standard penetration value normalised for total earth pressure and energy quotient effects |
| $N_{Ed}$ | Calculated axial force in the horizontal plane |
| $N_{SPT}$ | Number of blows measured in the Standard Penetration Test |
| $IP$ | Soil plasticity index |
| $R_{d}$ | Calculated soil resistance |
| $S$ | Soil coefficient as defined in Annex 1, section **3.2.2.2** |

| | |
|---|---|
| $S_T$ | Topographical amplification coefficient |
| $V_{Ed}$ | Calculated horizontal tangential force |
| $W$ | Weight of the sliding mass |
| $a_g$ | Calculated value of ground acceleration in terrain type A ($a_g = \gamma_I\, a_{gR}$) |
| $a_{gR}$ | Maximum reference acceleration in terrain type A |
| $a_{vg}$ | Calculated value of ground acceleration in the vertical direction |
| $c'$ | Soil cohesion in effective stresses |
| $c_u$ | Shear strength of soil in undrained test |
| $d$ | Pile diameter |
| $d_r$ | Displacement of retaining walls |
| $g$ | Acceleration of gravity |
| $k_h$ | Horizontal seismic coefficient |
| $k_v$ | Vertical seismic coefficient |
| $q_u$ | Unconfined compressive resistance |
| $r$ | Coefficient for calculating the horizontal seismic coefficient (Table 7.1) |
| $v_s$ | Shear wave propagation velocity |
| $v_{s,max.}$ | Mean value of the shear wave propagation velocity for small deformations ($< 10^{-5}$). |
| $\alpha$ | Ratio of the calculated value of the acceleration in terrain type A, $a_g$, and the acceleration of gravity, $g$ |
| $\gamma$ | Specific gravity of the soil |
| $\gamma_d$ | Specific gravity of dry soil |
| $\gamma_I$ | Importance factor |
| $\gamma_M$ | Partial coefficient of safety for material properties |
| $\gamma_{Rd}$ | Partial model coefficient |
| $\gamma_w$ | Specific gravity of water |
| $\delta$ | Angle of friction between terrain and footing or retaining wall |
| $\phi'$ | Angle of effective internal friction |
| $\rho$ | Density |
| $\sigma_{vo}$ | Total earth pressure (total vertical stress) |
| $\sigma'_{vo}$ | Effective earth pressure (effective vertical stress) |
| $\tau_{cy,u}$ | Cyclic shear resistance of the undrained soil |

$\tau_e$ Earthquake-induced tangential stress

# 2 Seismic action

## 2.1 Definition of seismic action

(1) The seismic action must be consistent with the fundamental concepts and definitions given in section **3.2** of Annex 1, taking into account the provisions given in section **4.2.2**.

(2) Combinations of seismic action with other actions must be carried out according to section **6.4.3.4** of Annex 18 of the Structural Code and section **3.2.4** of Annex 1.

(3) Simplifications in the choice of seismic action are introduced in this Annex where appropriate.

## 2.2 Temporal representation

(1) If the analysis is carried out in the time domain, both artificial accelerograms and real recorded motions can be used. Their maximum values and frequency content must be in accordance with the specifications in section **3.2.3.1** of Annex 1.

(2) In dynamic stability checks involving calculations of permanent ground deformations, the excitation should preferably be defined by means of accelerograms of real earthquakes recorded in soils, since these have a realistic low frequency content and an adequate time correlation between the horizontal and vertical components of the movement. The duration of the strong motion phase should be selected in a manner consistent with section **3.2.3.1** of Annex 1.

# 3 Properties of the terrain

## 3.1 Resistance parameters

(1) Generally the resistance parameters of the soil under undrained static conditions can be used. For cohesive materials the most appropriate resistance parameter is the undrained shear strength $c_u$ - adjusted for the (fast) rate of load application and the effects of cyclic degradation of the soil under seismic loading - where such an adjustment is necessary and justified by appropriate experimental evidence. For non-cohesive soils the most appropriate resistance parameter is the undrained cyclic shear resistance, $\tau_{cy,u}$, which shall take into account the possible growth of dynamic interstitial pressure in the loading process.

(2) Alternatively, the values of the effective resistance parameters can be used with an interstitial pressure equal to that generated during the cyclic loading process. For rocks, the unconfined compressive strength, $q_u$.

(3) The partial safety factors ($\gamma_M$) for the terrain parameters tan $\phi'$, c', $c_u$, $\tau_{cy,u}$, and $q_u$, (denoted respectively by: $\gamma_{\phi'}$, $\gamma_{c'}$, $\gamma_{cu}$, $\gamma_{\tau cy}$ and $\gamma_{qu}$) shall be adopted as specified in Tables 3.1 and 3.2 below.

NOTE In the field of building, in areas classified as low and very low seismicity, the safety approach of the Technical Building Code shall be applied, considering the values of the partial coefficients given in the Basic Document on Structural Safety Foundations (DB-SE-C). In the field of building, in areas other than those classified as low and very low seismicity, the verification shall be carried out using the different values or approaches prescribed: both in the Technical Building Code and in this section, with their corresponding coefficients.

**Table 3.1 - Partial coefficients $\gamma_M$ applicable to geotechnical parameters in the verification of structural (STR) and geotechnical (GEO) ultimate limit state (set M1), except in the verification of slope stability and global stability[(1)]**

| Soil parameter | Symbol | Value |
|---|---|---|
| Angle of internal friction at effective pressures $\phi$‘ (this coefficient applies to tan $\phi$') | $\gamma_{\phi'}$ | 1.0 |
| Effective cohesion ($c$’) | $\gamma_{c'}$ | 1.0 |
| Undrained shear strength ($c_u$) | $\gamma_{cu}$ | 1.0 |
| Undrained cyclic shear strength ($\tau_{cy,u}$) | $\gamma_{\tau cy}$ | 1.0 |
| Simple compressive strength ($q_u$) | $\gamma_{qu}$ | 1.0 |

(1) These coefficients are applied in the geotechnical testing to be carried out with Project Approach 2 (as defined in paragraph 2.4.7 of UNE-EN 1997-1).

**Table 3.2 - Partial coefficients $\gamma_M$ applicable to the geotechnical parameters in the verification of slope stability and overall stability (set M2)[(1)]**

| Limit state | Action | Project situation | $\gamma_M$ [(2)] | | | | |
|---|---|---|---|---|---|---|---|
| | | | tan$\phi$’ | c’ | $c_u$ | $\tau_{cy,u}$ | $q_u$ |
| Slope stability | a) Newly constructed cuttings slopes (without crowning structure or slope)<br>b) Compacted fills (embankment, fill and all-one type)<br>c) Minor hydraulic infrastructures (small dams and reservoirs classified as C because of the potential risk of rupture) | Accidental with earthquake | 1.1 | 1.1 | 1.1 | 1.1 | 1.1 |
| Overall stability of structures | Structures in road works | | 1.1 | 1.1 | 1.1 | 1.1 | 1.1 |
| | Structures in maritime or port works | | 1.1 | 1.1 | 1.1 | 1.1 | 1.1 |
| | Building structures | | 1.2 | 1.2 | 1.2 | 1.2 | 1.2 |

(1) These coefficients apply to the geotechnical testing to be carried out with Project Approach 3 (as defined in paragraph 2.4.7 of UNE-EN 1997-1).

(2) The values of the partial coefficients $\gamma_M$ corresponding to tan $\phi$’, $c$’, $c_u$, $\tau_{cy,u}$ and $q_u$ may be reduced by up to 7 % when the social, environmental and economic impacts of failure are reduced.

## 3.2 Parameters for rigidity and damping

(1) Due to its influence on the calculated seismic actions, the main stiffness parameter of the terrain under seismic loading is the transverse modulus of elasticity $G$, given by:

$$G = \rho v_s^2 \tag{3.1}$$

where $\rho$ is the density of the soil and $v_s$ is the shear wave propagation velocity in the terrain.

(2) Criteria for the determination of $v_s$, including its dependence on the level of deformation of the terrain, are given in sections **4.2.2** and **4.2.3**.

(3) Damping should be considered as an additional property of the terrain in cases where the soil-structure interaction effects specified in Chapter **6** are taken into account.

(4) Internal damping, caused by the inelastic behaviour of the soil under cyclic loading, and geometric damping, caused by the propagation of seismic waves beyond the foundation, must be considered separately.

# 4 Site and foundation terrain requirements

## 4.1 Site

### 4.1.1 General criteria

(1) An assessment of the chosen construction site must be carried out to ensure that the foundation terrain is suitable to minimise the hazards of failure, slope instability, liquefaction and susceptibility to high densification in the event of an earthquake.

(2) The potential for these adverse phenomena should be investigated as specified in the following sections.

### 4.1.2 Proximity to seismically active faults

(1) Buildings of Classes II, III and IV, as defined in section **4.2.5** of Annex 1, must not be constructed in the vicinity of tectonic faults classified as seismically active according to the state of existing knowledge. For these purposes, it shall be considered that a fault is close to the projected building if, in the event of an earthquake associated with said fault, the breakage of the fault or the permanent deformation of the ground around it may affect the construction. An additional distance shall be considered as a safety margin to allow for uncertainty in the mapping of the fault at the mapping scale used.

(2) The absence of movements in the Upper Pleistocene (129 000 years ago) can be used as a criteria for identifying non-active faults for most structures that are not critical to public safety.

NOTE A fault may be classified as seismically active by a competent public administration reporting the existence of specific substantiated investigations on the activity of this fault. The absence of such a report from a competent public administration cannot be taken to indicate that the fault is not seismically active.

(3) Special geological studies should be carried out for urbanisation plans and for important structures that are built near potentially active faults, in order to determine the dangerousness in terms of ground breakage and magnitude of seismic motion.

### 4.1.3 Slope stability

#### 4.1.3.1 General requirements

(1) The stability of the terrain must be tested for structures to be built on or near natural or artificial slopes to ensure that the safety and/or serviceability of structures subjected to the calculated earthquake is preserved.

(2) Under seismic loading conditions, the limit state for slopes corresponds to the occurrence of

unacceptably large displacements of the soil mass at depths that may affect the mechanical and functional behaviour of structures.

(3) Stability testing may be omitted for buildings of importance class I if it is known from comparative studies or proven experience that the soil at the chosen site is stable.

#### 4.1.3.2 Seismic action

(1) The calculated seismic action used for testing the stability shall be in accordance with the definitions given in section **2.1**.

(2) In the case of testing for stability of structures with a significance factor $\gamma_I$ greater than 1.0, located close to or on slopes, the calculated seismic action must be increased by a topographic amplification coefficient.

NOTE Guidelines for the determination of the topographic amplification coefficient are given in Appendix A.

(3) The definition of seismic action can be simplified as specified in section **4.1.3.3**.

#### 4.1.3.3 Methods of analysis

(1) The response of the soil slopes to the computed earthquake must be determined by recognised methods of dynamic analysis, such as finite element or rigid block models, or by simplified pseudo-static methods subject to the limitations set out in (3) and (8) of this section.

(2) When modelling the mechanical behaviour of the soil, the degradation of the material with increasing strain rate and the possible effects of increasing pore water pressure under dynamic loading must be taken into account.

(3) Stability testing can be carried out by simplified pseudo-static methods in cases where the topography of the terrain surface and the stratigraphy of the terrain are not very irregular.

(4) Pseudo-static methods of stability analysis introduce inertial forces (horizontal and vertical) applied to the soil mass and any gravity loads acting on the top of the slope.

(5) The seismic inertia forces, $F_H$ and $F_V$ acting on the terrain mass, in the horizontal and vertical directions respectively, are to be taken in the pseudo-static analysis as:

$$F_H = 0,5\alpha \cdot S \cdot W \tag{4.1}$$

$$F_V = \pm 0,5\, F_H \quad \text{if } a_{vg} / a_g \text{ is greater than 0.6} \tag{4.2}$$

$$F_V = \pm 0,33\, F_H \quad \text{if } a_{vg} / a_g \text{ is not greater than 0.6} \tag{4.3}$$

where

$\alpha$ the quotient of the calculated value of the acceleration for terrain type A, $a_g$, and the acceleration of gravity, $g$.

$a_{vg}$ the calculated value of the acceleration in the vertical direction.

$a_g$ the calculated value of the acceleration for a terrain type A.

$S$ the soil coefficient, given in section **3.2.2.2** of Annex 1.

$W$ the weight of the sliding mass.

A topographic amplification coefficient according to section **4.1.3.2(2)** must be applied to $a_g$.

(6) The limit state condition for the potential (least safe) slip surface must also be tested.

(7) The serviceability limit state condition can be tested by calculating the permanent displacement of the sliding mass using a simplified dynamic model consisting of a rigid block sliding frictionally on the slope. In this model, the seismic action shall consist of an accelerogram according to section **2.2**, and be based on the calculated value of the unreduced acceleration.

(8) Simplified methods, such as the pseudo-static methods mentioned in (3) to (6) of this section, should not be used in the case of soils capable of developing high interstitial pressures or of undergoing significant degradation in rigidity under cyclic loading.

(9) The increase in pore water pressure shall be assessed by appropriate tests. Where such tests are not available, and in preliminary studies, it may be estimated from empirical correlations.

#### 4.1.3.4 Safety check using the pseudo-static method

(1) For saturated soils in areas with $\alpha \cdot S > 0.15$, the possible degradation of their resistance and the increase of pore water pressure due to seismic loading must be considered, in accordance with the limitations set out in section **4.1.3.3(8)**.

(2) In the study of areas with frequent landslides with a high likelihood of reactivation under the effect of earthquakes, values of the resistance parameters of the terrain associated with large deformations must be used. For cohesionless materials susceptible to cyclic increases in pore water pressure within the limits set out in section **4.1.3.3**, the latter effect may be taken into account by decreasing the resistant frictional force by an appropriate interstitial pressure parameter, proportional to the maximum increase in pore water pressure. This increase can be estimated as indicated in section **4.1.3.3(9)**.

(3) It is not necessary to apply a reduction of the tangential resistance in the case of very dilatant cohesionless soils, as is the case for dense sands.

(4) The testing of the safety of soil slopes must be carried out in accordance with the principles laid down in the specific regulations in force or, failing that, with the specific technical documents which the author of the project, under their responsibility, considers most appropriate.

### 4.1.4 Potentially liquefiable soils

(1) The decrease in resistance and/or rigidity caused by increased pore water pressure occurring in cohesionless terrain subjected to seismic movement, leading to significant permanent deformations and even to a condition of near zero effective stresses in the soil, is hereinafter referred to as liquefaction.

(2) The susceptibility to liquefaction must be assessed when the foundation terrain consists of

extended layers or strong lenses of loose sand, located below the water table whether or not they contain fines - silts or clays - and when the water table is close to the ground surface. This assessment must be made for the free field conditions at the site (position of the terrain surface, position of the water table) that will prevail during the life of the structure.

(3) The terrain investigations required for this assessment should include, as a minimum, the performance of Standard Penetration Tests (SPT), according to UNE-EN ISO 22476-3, or Static Penetration Tests with the Mechanical Cone (CPT), according to UNE-EN ISO 22476-12, as well as the determination of the granulometric curves in the laboratory, according to UNE-EN ISO 17892-4.

(4) In the case of the SPT test, the measured values of the $N_{SPT}$ penetration rate, expressed in blows/30 cm, must be normalised for an effective terrain pressure of 100 kPa, as well as for an impact energy that is 0,6 of the theoretical free fall energy. For depths less than 3 m, the measured values of the SPT test shall be reduced by 25 %.

(5) Normalisation for the effects of earth surcharge can be made by multiplying the measured value of $N_{SPT}$ by the coefficient $(100/\sigma'_{vo})^{1/2}$ , where $\sigma'_{vo}$ (kPa) is the effective pressure acting at the depth at which the SPT test was performed, at the time of its execution. The normalisation coefficient $(100/\sigma'_{vo})^{1/2}$ shall be not less than 0.5 and not more than 2.

(6) Normalisation of the impact energy requires multiplication of the penetration rate value obtained in **(5)** above by the coefficient $ER/60$, where $ER$ is 100 times the specific energy quotient for the test equipment.

(7) For buildings with shallow foundations, the assessment of liquefaction susceptibility may be omitted when the saturated sandy soil layer is deeper than 15 m measured from the terrain surface.

(8) It is permissible to disregard the liquefaction hazard when $\alpha \cdot S < 0.15$ and at least one of the following conditions is met:

- the sand has a clay content greater than 20 % with a plasticity index $IP > 10$; according to UNE-EN ISO 17892-12.
- the sand has a silt content greater than 35 % and, at the same time, the SPT number of blows (normalised for total earth pressure effects and for the energy quotient), $N_1(60)$, is greater than 20;
- the sand is clean, with an SPT number of blows (normalised for total earth pressure effects and energy quotient), $N_1(60)$, greater than 30.

(9) Where none of the above conditions are met, the liquefaction hazard must be assessed at least by the usual methods in geotechnical engineering, based on correlations between *in situ* measurements and critical cyclic shear stresses that have caused liquefaction in previous earthquakes.

(10) Appendix B gives empirically derived graphs showing these correlations for different types of *in situ* measurements. From these graphs the seismic shear stress, $\tau_e$, can be estimated from the following simplified expression:

$$\tau_e = 0,65\ \alpha \cdot S \cdot \sigma_{vo} \tag{4.4}$$

where $\sigma_{vo}$ is the total vertical pressure exerted by the terrain and the other variables are the same as in expressions (4.1) to (4.3). This formula cannot be used for depths greater than 20 m.

(11) If correlations with *in situ* measurements are used, a terrain with a horizontal surface should be considered to be susceptible to liquefaction if the earthquake-induced shear stress exceeds a certain fraction $\lambda$ of the critical stress known to have caused liquefaction in previous earthquakes. The value $\lambda = 0.8$ shall be adopted, giving a partial safety factor of 1.25.

(12) If it is concluded that the soil is susceptible to liquefaction and its effects are capable of affecting the bearing capacity or stability of the foundations, measures must be taken to ensure that the safety of the foundation is adequate, either by ground improvement methods or by piling (to transmit the load to non-liquefiable soil strata).

(13) Ground improvement to prevent liquefaction should consist of either soil compaction to increase the resistance to penetration above critical values or the use of drainage to reduce the excess interstitial pressure generated by ground movement.

NOTE The possibility of compaction of the terrain depends mainly on its fines content and depth.

(14) The exclusive use of piles should be considered with caution because of the significant forces induced in the piles as the soil loses its bearing capacity in the liquefiable layers, as well as the inevitable uncertainties associated with the location and thickness of these layers.

### 4.1.5 Excessive settlement of floors under cyclic loading

(1) Consideration should be given to the susceptibility of foundation terrain to densification and excessive settlement caused by earthquake-induced cyclic stresses where there are extended layers or strong lenses of loose, cohesionless, unsaturated, shallow unsaturated materials.

(2) Excessive settlement may also occur in very soft clays due to cyclic degradation of their shear resistance when subjected to long-term seismic ground motions.

(3) The densification and potential settlement of the above soils should be assessed by traditional geotechnical engineering methods using, where necessary, appropriate static and dynamic laboratory tests on representative samples of the materials under investigation.

(4) If settlement caused by densification or cyclic degradation could potentially affect the stability of the foundations, appropriate methods of terrain improvement should be considered.

## 4.2 Prospection and terrain studies

### 4.2.1 General criteria

(1) The prospection and study of the foundation terrain in seismic zones must follow the same criteria adopted for non-seismic zones, complemented, as a minimum, by the specific surveys and studies necessary to comply with this Seismic Resistance Standard.

(2) Except for buildings of importance class I, static penetration tests (CPT) should be included in the terrain exploration, whenever feasible, preferably with interstitial pressure measurements, as they provide a continuous record of the mechanical characteristics of the soil as a function of depth.

(3) In the cases indicated in sections **4.1** and **4.2.2**, specific investigations and explorations of the

terrain, aimed at defining its dynamic properties, may be required.

### 4.2.2 Determination of the type of terrain for the definition of seismic actions

(1) Sufficient geological and geotechnical data must be collected at the site of the works to enable the determination of an average terrain type and/or the associated response spectrum as defined in sections **3.1** and **3.2** of Annex 1.

(2) For this purpose it is recommended to incorporate data from other nearby areas with similar geological characteristics to the data collected *in situ*.

(3) Existing seismic microzonation maps and criteria should be taken into account, provided that they comply with **(1)** and that they are corroborated by terrain investigations carried out at the site of the works.

(4) In stable terrain, the profile of ground-wave propagation speeds, $v_s$, should be considered as the most reliable indicator on which to base the determination of the characteristics of the seismic action depending on the type of location.

(5) In regions of high seismicity, especially in terrain type D, $S_1$, or $S_2$ (see section **3.1.2** of Annex 1), the $v_s$ profile should be obtained *in situ* by the application of geophysical methods within boreholes.

(6) In any other case, where natural periods of vibration of the terrain are to be determined, the $v_s$ profile may be estimated by empirical correlations with in situ penetration resistance or other geotechnical properties (taking into account the dispersion of such correlations), or determined by instrumental measurements *in situ*.

(7) The internal damping of the terrain shall be measured by appropriate laboratory or field tests. In the absence of direct measurements, if the $a_g \cdot S$ product is less than 0.1 $g$ (i.e. less than 0.98 m/s$^2$), a damping ratio of 0.03 should be used. Cemented soils and soft rocks may require special consideration.

### 4.2.3 Variation of soil stiffness and damping as a function of deformation level

(1) In all calculations involving the dynamic properties of the terrain under steady state conditions, the difference between the $v_s$ values for small deformations (such as values measured *in situ*), and the $v_s$ values compatible with the levels of deformation induced by the computed earthquake, must be taken into account.

(2) In the case of local terrain conditions of type C or D, with a shallow water table and in the absence of materials with plasticity index $IP > 40$, the $v_s$ reduction coefficients given in Table 4.1 can be used in the absence of specific data. For stiffer terrain and with a deeper water table the magnitude of the reduction should be proportionally smaller (and the range of variation should be reduced).

(3) In the absence of specific measurements, if the product $a_g \cdot S$ is greater than or equal to 0.1 $g$ (i.e. greater than or equal to 0.98 m/s$^2$) the internal damping coefficients of the terrain given in Table 4.1 should be used.

**Table 4.1 -Average terrain damping coefficients and average reduction coefficients (± a standard deviation) for the shear wave propagation velocity, $v_s$, and the transverse modulus of elasticity, $G$, in the first 20 m of depth**

| Acceleration ratio of the terrain, $\alpha \cdot S$ | Damping coefficient | $\frac{v_s}{v_{s,máx.}}$ | $\frac{G}{G_{máx.}}$ |
|---|---|---|---|
| 0.10 | 0.03 | 0.90 (± 0.07) | 0.80 (± 0.10) |
| 0.20 | 0.06 | 0.70 (± 0.15) | 0.50 (± 0.20) |
| 0.30 | 0.10 | 0.60 (± 0.15) | 0.36 (± 0.20) |

$v_{s,max.}$ is the average value of $v_s$ for small deformations (< $10^{-5}$); not exceeding 360 m/s;

$G_{max.}$ is the mean value of the transverse modulus of elasticity for small deformations.

NOTE Using the range ± a standard deviation, the designer can be more or less conservative, depending on factors such as the rigidity and layering of the terrain profile. For example, values of $v_s/v_{s,max.}$ and $G/G_{max.}$ above the mean value can be used for harder terrain and values of $v_s/v_{s,max.}$ and $G/G_{max.}$ below the mean value for softer terrain.

# 5 Foundation system

## 5.1 General requirements

(1) In addition to the provisions of the specific regulations in force and/or, where appropriate, the specific technical documents which the designer, under their responsibility, considers most appropriate, the foundation of a structure in a seismic zone must meet the following requirements:

a) The relevant loads of the superstructure must be transmitted to the terrain without significant permanent deformations, in accordance with the criteria of section **5.3.2**.

b) Seismically induced deformations of the terrain must be compatible with the essential functional requirements of the structure.

c) The foundation must be designed, engineered and constructed in compliance with the rules given in section **5.2** and the minimum measures of section **5.4**, so as to limit the risk associated with the uncertainty of the seismic response.

(2) The dependence of the dynamic properties of the soil on the level of deformations (see **4.2.3**) as well as the effects related to the cyclic nature of the seismic loads must be taken into account. The properties of *in situ* improved or even replaced soils must be considered, if the improvement or replacement of the original soil becomes necessary due to its susceptibility to liquefaction or densification.

(3) Where appropriate (or necessary), soil properties or resistance coefficients other than those mentioned in section **3.1(3)** may be used, provided that they correspond to the same level of safety.

NOTE Examples of the above are the resistance coefficients applied to the results of pile load tests.

## 5.2 Basic rules of conceptual design

(1) For structures other than bridges and pipelines, mixed foundations (e.g. piles and shallow foundations) should only be used if a specific study is carried out to demonstrate the validity of such a solution. Mixed type foundations may be used in dynamically independent units of the same structure.

(2) The following aspects should be taken into account when selecting the type of foundation:

a) The rigidity of the foundation must be adequate to transmit the localised actions received from the superstructure to the terrain as uniformly as possible.

b) The effects of relative horizontal displacements between vertical elements must be taken into account when selecting the foundation rigidity in the horizontal plane.

c) If the decrease in amplitude of seismic motion with depth is taken into account, this effect must be justified by an appropriate study and in no case shall the maximum acceleration be decreased below a fraction $p = 0.65$ of the $\alpha \cdot S$-value at the terrain surface.

## 5.3 Effects of calculated actions

### 5.3.1 Dependence on structural design

(1) *Dissipative structures*. The effects of actions on the foundation of dissipative structures must be based on capacity design considerations, taking into account the development of a possible reserve of resistance (over-resistance). The assessment of these effects should be carried out in accordance with the appropriate requirements of this Seismic Resistance Standard. In particular, for buildings, the limitation given in section **4.4.2.6(2)** of Annex 1 is to be applied.

(2) *Non-dissipative structures*. The effects of seismic actions on the foundation of non-dissipative structures must be obtained from the analysis in the calculated seismic situation without consideration of capacity sizing. See also section **4.4.2.6(3)** of Annex 1.

### 5.3.2 Transferring the effects of actions to the terrain

(1) In order to comply with the requirements of section **5.1(1)**(a) of the foundation system, the following criteria for the transfer of horizontal and normal forces and bending moment to the terrain shall be adopted. For piers and piles the additional criteria specified in section **5.4.2** must be taken into account.

(2) *Horizontal stress*. The transfer of the calculated horizontal shear force, $V_{Ed}$, must be carried out according to the following mechanisms:

a) by a calculated frictional force, $F_{Rd}$, between the horizontal base of a footing or slab and the terrain, as described in section **5.4.1.1**;

b) by a calculated shear resistance between the vertical foundation walls and the terrain;

c) by the calculated resistance thrusts of the terrain on the foundation, according to the limitations and conditions set out in sections **5.4.1.1**, **5.4.1.3** and **5.4.2**.

(3) The combination of the shear resistance with up to 30 % of the resistance obtained in the case of

full mobilisation of the passive earth thrust should be allowed.

(4) *Axial force and bending moment*. The calculated actions (axial force, $N_{Ed}$ and bending moment, $M_{Ed}$), must be transmitted to the terrain by one or more of the following mechanisms:

a) by the calculated value of the vertical resistance forces acting at the base of the foundation;

b) by the calculated value of the bending moments produced by the calculated resistance tangential forces between the terrain and the lateral surface of the deep foundation elements (piles, caissons, caissons) in accordance with the limitations set out in sections **5.4.1.3** and **5.4.2**;

c) by the calculated value of the vertical shear resistance between the terrain and the lateral surface of the buried deep foundations (caissons, piles, large diameter piles and caissons).

## 5.4 Testing and dimensioning criteria

### 5.4.1 Direct foundations (shallow or buried)

(1) The following criteria are to be applied for testing and dimensioning of shallow or buried footings supported directly on the underlying soil.

#### 5.4.1.1 Footings (calculated at ultimate limit state)

(1) In accordance with the design criteria for the ultimate limit state, the footings must be tested for sliding failure and bearing capacity.

(2) *Sliding failure*. In the case of foundations whose base is located above the water table, failure shall be prevented by frictional force and, under the conditions specified in **(5)** of this section, by lateral earth thrust.

(3) In the absence of more specific studies, the calculated value of the frictional force above the water table, $F_{Rd}$, can be obtained from the following expression:

$$F_{Rd} = N_{Ed} \frac{\tan \delta}{\gamma_M} \tag{5.1}$$

Where

$N_{Ed}$ is the calculated axial force at the horizontal base;

$\delta$ is the soil-structure friction angle at the base of the footing, which can be evaluated in accordance with the specific regulations in force or, failing that, with the specific technical documents that the author of the project, under their responsibility, considers most appropriate.

$\gamma_M$ is the partial safety factor applied to the material properties, taken with a value equal to that applicable to tan $\phi'$ (see section **3.1(3)**).

(4) In the case of foundations below the water table, the calculated value of the frictional force must be evaluated from the undrained resistance.

NOTE In this respect, the provisions of paragraph 6.5.3 of UNE-EN 1997-1 shall be considered.

(5) The calculated lateral resistance $E_{pd}$ as a consequence of passive thrust on the footing sides may be taken into account as specified in section **5.3.2**, provided that it is ensured that appropriate measures have been taken on site, such as proper compaction of the backfill material against the footing sides, construction of a vertical foundation wall in the soil, or execution of the footings by direct pouring of the concrete on a clean, vertical face of soil.

(6) To prevent sliding failure of the horizontal footing, the following inequality must be complied with:

$$V_{\mathrm{Ed}} \leq F_{\mathrm{Rd}} + E_{\mathrm{pd}} \tag{5.2}$$

(7) In the case of foundations above the water table, a small amount of foundation slippage is permissible if the following two conditions are satisfied:

- the soil properties remain unchanged during the earthquake;

- the foundation slippage does not affect the performance of utility networks connected to the structure (e.g. water, gas, access or telecommunications);

The magnitude of the earthquake must be reasonable when considering the overall behaviour of the structure.

(8) *Bearing capacity failure*. To satisfy requirement (a) in section **5.1(1)**, the bearing capacity of the foundation subjected to a combination of the effects of the calculated actions $N_{\mathrm{Ed}}$, $V_{\mathrm{Ed}}$ and $M_{\mathrm{Ed}}$.

NOTE For testing the bearing capacity of the foundation subjected to seismic actions, the general expression and design criteria given in Appendix F may be used, which allow for taking into account the inclination and eccentricity of the load due to the inertia forces of the structure, as well as considering the possible effects of the inertia forces on the soil itself.

(9) It is noted that, under dynamic loading, some thixotropic clays may suffer degradation of their shear strengths, and that some non-cohesive materials are susceptible to develop an increase in interstitial pressure and subsequent dissipation of interstitial pressure after the earthquake as water moves towards the terrain surface due to the effect of excess interstitial pressure built up in underlying layers.

(10) The assessment of the bearing capacity of a soil under seismic loading should adequately consider the possible degradation mechanisms of its resistance and rigidity that can occur even at relatively low levels of deformation. If these effects are considered, the partial safety coefficients of the material can be reduced. Otherwise, the values specified in section **3.1(3)** should be used.

(11) The increase in interstitial pressure under cyclic loading shall be considered, either by taking into account its effect on the undrained resistance (in an analysis in total pressures) or on the initial interstitial pressure (analysis in effective pressures). For structures where the significance factor $\gamma_I$ is greater than 1.0, a non-linear behaviour of the soil must be considered in order to determine the possible permanent deformations in earthquakes.

#### 5.4.1.2 Horizontal joints in the foundation

(1) In accordance with section **5.2**, the additional effects of actions induced in the structure by relative horizontal displacements in the foundation must be assessed and appropriate measures taken in the design.

(2) For buildings, the requirements set out in **(1)** of this section are considered to be satisfied if the entire foundation is in the same horizontal plane and tie beams or a foundation slab are provided at the level of the footings or pile heads. These measures are not compulsory in the following cases: a) for terrain of type A; and b) for Type B terrain in areas of low seismicity.

(3) The beams of the lower floor of a building may be considered as tie beams provided they are less than 1.0 m from the bottom face of footings or pile heads. A foundation slab may replace the tie beams if it is also less than 1.0 m from the bottom face of the footings or pile heads.

(4) The required tensile resistance of these connection elements can be estimated by simplified methods.

(5) In the absence of more precise rules or methods of calculation, foundation connections complying with the conditions set out in (6) and (7) of this section shall be considered adequate.

(6) *Tie beams*

The following measures shall be taken:

a) the tie beams shall be sized to withstand an axial force, both tensile and compressive, equal to:

$\pm 0.3\, \alpha \cdot S \cdot N_{\mathrm{Ed}}$ for terrain type B

$\pm 0.4\, \alpha \cdot S \cdot N_{\mathrm{Ed}}$ for terrain type C

$\pm 0.6\, \alpha \cdot S \cdot N_{\mathrm{Ed}}$ for terrain type D

where $N_{\mathrm{Ed}}$ is the average value, in the calculated seismic situation, of the calculated axial forces in the connected vertical elements.

b) the longitudinal reinforcement shall be fully anchored in the body of the footing or in the other tie beams concurrent with the beam under consideration.

(7) *Foundation slab*

The following measures shall be taken:

a) The joint zones shall be sized to withstand axial forces equal to those given in **(6)**(a) of this section.

b) The longitudinal reinforcement of the connection zones shall be fully anchored in the footing body or in the continuous slab.

#### 5.4.1.3 Spread footings and slabs

(1) All specifications given in section **5.4.1.1** apply to spread footings and/or slabs taking into

account the following considerations:

a) In the case of a single foundation slab, the overall frictional resistance may be considered. For single slab on grade footings, the area of an equivalent footing at each of the crossings may be considered.

b) Spread footings and/or foundation slabs may be considered as connecting elements; the rule for their sizing is applicable to an effective width equal to that of the spread footing or to a slab width equal to ten times its thickness.

(2) It may also be necessary to carry out testing of the set of spread footings and/or slabs on their own plane (diaphragm), subjected to their own lateral inertia forces and to the horizontal forces induced by the superstructure.

#### 5.4.1.4 Caisson foundations

(1) All specifications described in section **5.4.1.3** apply to caisson foundations. In addition, the lateral soil resistance specified in sections **5.3.2(2)** and **5.4.1.1(5)** may be considered for all soil categories and with the limitations indicated above.

### 5.4.2 Piers and piles

(1) Piers and piles shall be sized to resist the following two types of effects due to seismic actions:

a) *Inertia forces of the superstructure*. These forces, combined with the static loads, give the calculated values $N_{Ed}$, $V_{Ed}$ and $M_{Ed}$ specified in section **5.3.2**.

b) *Kinematic forces* generated by the deformation of the soil adjacent to the passage of seismic waves.

(2) The ultimate transverse resistance of the piles must be tested in accordance with the specific regulations in force or, failing this, with the specific technical documents that the author of the project, under their responsibility, considers most appropriate.

(3) The analyses to determine the internal forces along the length of the pile, as well as the displacement and head rotation of the pile, must be based on discrete or continuous models that realistically reproduce (even if only approximately):

- the bending rigidity of the pile;
- the soil reactions along the pile, considering cyclic loading effects and the magnitude of soil deformations;
- the dynamic interaction effects between piles (also called "dynamic group effects");
- the degree of freedom of rotation at/of the pile head, or of the pile-structure connection.

NOTE The expressions given in Appendix C can be used as a guide for calculating the pile rigidity.

(4) The lateral resistance of soil layers susceptible to liquefaction or substantial degradation in resistance under cyclic loading must be ignored.

(5) If inclined piles are used, they must be dimensioned to safely support both axial forces and bending moments.

NOTE Inclined piles are not recommended for transmitting lateral loads to the ground.

(6) Bending moments due to kinematic interaction should only be calculated when the following conditions are simultaneously fulfilled:

- the soil profile is of type D, $S_1$ or $S_2$, and contains consecutive layers with rigidities that vary greatly with respect to each other;
- the site is of moderate or high seismicity, i.e. the $a_g \cdot S$ product is greater than 0.10 g (greater than 0.98 m/s$^2$) and the structure is of class III or IV.

(7) In principle, piles should be sized to remain elastic, although under certain conditions plastic hinges may occur at the pile heads. Areas of potential plastic hinge formation shall be sized in accordance with Section **5.8.4** of Annex 1.

# 6 Soil-structure interaction

(1) Dynamic soil-structure interaction effects must be taken into account in the following cases:

a) structures where P-δ (second order) effects play a relevant role;

b) structures with massive and deep foundations, such as bridge piers, offshore caissons and silos;

c) tall and slender structures, such as towers and chimneys, covered by Annex 6;

d) structures founded on very soft soils, with an average shear wave propagation velocity, $v_{s,max.}$. (as defined in table 4.1) of less than 100 m/s, as is the case for soils type $S_1$.

NOTE Information on the general effects and the importance of dynamic soil-structure interaction is given in Appendix D.

(2) The effects of soil-structure interaction in piles must be assessed in accordance with section **5.4.2** for all structures.

# 7 Earth retaining structures

## 7.1 General requirements

(1) Earth retaining structures must be designed to fulfil their function during and after the earthquake without significant structural damage.

(2) Permanent displacements, in the form of combinations of sliding and twisting (the latter due to irreversible deformations in the foundation soil) may be acceptable if they can be shown to be compatible with the functional and/or aesthetic requirements of the structure.

## 7.2 Selection of the type of structure and general project considerations

(1) The selection of the type of structure must be based on normal service conditions, in accordance with the general principles laid down in the specific regulations in force or, failing this, in the specific technical documents which the author of the project, under their responsibility, considers most appropriate.

(2) Particular attention should be paid to the fact that it may be necessary to adjust and, occasionally, to choose a more suitable type of structure to meet all the additional seismic requirements.

(3) The granulometry of the backfill material of the structure should be carefully chosen and compacted *in situ*, in order to achieve the greatest possible homogeneity with the pre-existing soil mass.

(4) The drainage systems at the rear of the structure must have sufficient capacity to absorb transient and permanent movements without affecting its functioning.

(5) In particular, in the case of non-cohesive soils containing water, drainage must be effective to a depth significantly greater than the potential failure surface behind the retaining structure.

(6) It must be ensured that the subgrade soil has a sufficient margin of resistance to liquefaction when subjected to the calculated earthquake.

## 7.3 Methods of analysis

### 7.3.1 General methods

(1) Any method based on the procedures of structural dynamics and soil dynamics, and tested by experience and observations, is in principle acceptable for assessing the safety of an earth retaining structure.

(2) The following aspects shall be taken into account:

a) the generally non-linear behaviour of the soil during its dynamic interaction with the retaining structure;

b) the effects of inertial forces associated with the soil masses and the structure, as well as other mass forces that may contribute to the interaction process;

c) hydrodynamic effects generated by the presence of water in the soil at the back of the wall and/or by water on the outer face of the wall;

d) the compatibility between the deformations of the soil, the wall and the anchorages, where present.

### 7.3.2 Simplified methods: pseudo-static analysis

#### 7.3.2.1 Basic models

(1) The basic model for pseudo-static analysis must include the retaining structure and its foundation, the soil wedge behind the structure (assumed to be in a state of active limit equilibrium, if the structure is sufficiently flexible), the overloads acting on the soil wedge and, where appropriate, a soil mass at the foot of the wall assumed to be in a state of passive equilibrium.

(2) For an active ground state to be generated there must be movement of the wall of sufficient magnitude during the calculated earthquake, either due to bending in flexible retaining structures, or due to sliding and/or twisting in rigid retaining structures. The wall movement necessary for an active limit state to develop can be assessed in accordance with the specific regulations in force or, failing

that, with the specific technical documents that the author of the project, under their responsibility, considers most appropriate.

(3) In the case of rigid structures, such as basement walls or gravity walls founded on rock or piles, thrusts greater than the active ones develop and, in this case, it is more appropriate to assume a state of soil at rest as indicated in section **E.9**. The same assumption should be made for anchored retaining walls where no movement is permitted.

### 7.3.2.2 Seismic action

(1) When a pseudo-static analysis is to be performed, the seismic action should be represented as a set of static horizontal and vertical forces, obtained as the product of the gravity forces by a seismic coefficient.

(2) The vertical seismic action should be considered to act upwards or downwards so as to produce the most unfavourable effect.

(3) The intensity of these equivalent seismic forces depends, for a given seismic zone, on the magnitude of the permanent displacement which is both acceptable and permitted in practice for the structural solution adopted.

(4) In the absence of specific studies, the horizontal ($k_h$) and vertical ($k_v$) seismic coefficients affecting all masses should be taken as:

$$k_h = \alpha \frac{S}{r} \tag{7.1}$$

$$k_v = \pm 0,5\, k_h \quad \text{if } a_{vg} / a_g \text{ is greater than 0.6} \tag{7.2}$$

$$k_v = \pm 0,33\, k_h \quad \text{in any other case} \tag{7.3}$$

where the values of the coefficient $r$ are shown in Table 7.1, depending on the type of retaining structure. For walls less than 10 m high, the seismic coefficient must be considered constant over their entire height.

**Table 7.1 - *r*-values for the calculated horizontal seismic coefficient**

| Type of retaining structure | $r$ |
|---|---|
| Unconstrained gravity walls with maximum displacement $d_r = 300\, \alpha \cdot S$ (mm) | 2 |
| Unconstrained gravity walls with maximum displacement $d_r = 200\, \alpha \cdot S$ (mm) | 1.5 |
| Reinforced concrete flexible walls, anchored or propped walls, reinforced concrete walls founded on vertical piles, basement walls with restricted movements and bridge abutments | 1 |

(5) In the presence of saturated non-cohesive soils susceptible to develop high interstitial

pressures:

a) the coefficient *r* in Table 7.1 shall not be taken to be greater than 1.0;

b) the coefficient of safety against liquefaction shall not be taken to be less than 2.

NOTE The value of 2 for the safety factor is derived from the application of section **7.2(6)** in the context of the simplified method proposed in section **7.3.2**.

(6) In the case of retaining structures higher than 10 m, and for more information on the coefficient *r*, see section **E.2**.

(7) For walls other than gravity walls, the effects of vertical acceleration on the retaining structures can be neglected.

#### 7.3.2.3 Calculated values for earth thrust and water pressure

(1) The total design force acting on a wall under seismic conditions must be calculated considering the boundary equilibrium condition of the model described in section **7.3.2.1**.

(2) This force can be determined as given in Appendix E.

(3) The calculated force described in **(1)** must be considered to be the resultant of the static and dynamic earth pressures.

(4) The point of application of the force due to dynamic earth pressure is to be taken as the mid-height of the wall, provided that there is no more detailed study considering the relative rigidity, type of movements and relative mass of the retaining structure.

(5) In the case of walls that are free to rotate about their footing, it may be assumed that the dynamic force is applied at the same point as the static force.

(6) The inclination of the total distribution of pressures, static and dynamic, with respect to the normal to the wall must be assumed to be no greater than $(2/3)\phi'$ for the active state and zero for the passive state.

(7) For soil below the water table, a distinction must be made between dynamically permeable conditions, where water can move freely through the solid skeleton, and dynamically impermeable conditions, where essentially no drainage can occur under seismic action.

(8) In most situations and for soils with a permeability coefficient of less than $5 \times 10^{-4}$ m/s, interstitial water cannot move freely between the pores; seismic action therefore occurs essentially in the absence of drainage and the soil can be treated as a single-phase medium.

(9) For dynamic impermeability conditions, all the above considerations must be applied, with appropriate modification of the soil specific gravity and the horizontal seismic coefficient.

(10) Necessary modifications to the dynamic sealing conditions can be made in accordance with sections **E.6** and **E.7**.

(11) For the case of fills under dynamic permeability conditions, it should be assumed that the effects induced by seismic action on soil and water are decoupled.

(12) Therefore, a hydrodynamic pressure must be added to the hydrostatic interstitial pressure in

accordance with section **E.7**. The point of application of the force resulting from the hydrodynamic pressure may be taken at a depth equal to 60 % of the thickness of the saturated layer, measured from the water table.

#### 7.3.2.4 Hydrodynamic pressure on the outer face of the wall

(1) The maximum pressure variation (positive or negative) with respect to the existing hydrostatic pressure, due to the oscillation of the water on the outer face of the wall, must be taken into account.

(2) This pressure can be evaluated according to section **E.8**.

## 7.4 Stability and resistance testing

### 7.4.1 Stability of the foundation soil

(1) The following testing is required:

- general stability;

- local soil failure.

(2) Testing for general stability must be carried out in accordance with the rules given in section **4.1.3.4**.

(3) The ultimate capacity of the foundation must be tested for sliding failure and bearing capacity failure (see **5.4.1.1**).

### 7.4.2 Anchors

(1) Anchorages (including tie rods, anchorage mechanisms, heads and fasteners) must have sufficient resistance and length to ensure equilibrium of the critical soil wedge against seismic conditions (see **7.3.2.1**), as well as the necessary capacity to accommodate seismic ground deformations.

(2) The anchorage resistance must be determined, for permanent and transient ultimate limit state calculated situations, in accordance with the specific regulations in force or, failing this, with the specific technical documents which the designer, under their responsibility, considers most appropriate.

(3) It must be ensured that the anchored soil maintains the resistance required for anchorage operation during the calculated earthquake, and in particular, that it has a sufficient safety margin against liquefaction.

(4) The distance between the anchorage point and the wall, $L_e$, must be greater than the distance $L_s$, required for static loads.

(5) The distance $L_e$ for anchorages in a soil tank with a horizontal surface and similar characteristics to those of the soil behind the wall can be evaluated according to the following expression:

$$L_e = L_s \ (1 + 1{,}5\ \alpha \cdot S) \tag{7.4}$$

### 7.4.3 Structural strength

(1) It must be demonstrated that, under the combination of seismic action with other possible loads, equilibrium is achieved without exceeding the calculated resistances of the wall and the supporting structural elements.

(2) To this end, the relevant limit state situations that may lead to the failure of the structure must be considered, in accordance with the specific regulations in force or, failing this, with the specific technical documents that the author of the project, under their responsibility, considers most appropriate.

(3) Testing must be carried out to ensure that all structural elements satisfy the following condition:

$$R_d > E_d \tag{7.5}$$

Where

$R_d$ is the calculated resistance of the element, determined in the same way as in the non-seismic situation;

$E_d$ is the calculated value of the effect of the actions, obtained according to the analysis described in section **7.3**.

# Appendix A

## Recommended topographical amplification coefficients

**A.1** This Appendix gives some simplified seismic amplification coefficients for testing slope stability. These coefficients, designated $S_T$, are a first approximation and are considered to be independent of the fundamental period of vibration and therefore multiply by a constant factor the ordinates of the calculated elastic response spectrum given in Annex 1. These amplification coefficients should preferably be applied in the case of slopes forming part of two-dimensional topographical irregularities, such as ridges and cliffs of great length and height greater than 30 m.

**A.2** For slopes with an average gradient of less than 15°, the effects of topography can be neglected, but a specific study is recommended in case of locally strong topographical irregularities. For steeper slopes, the following guidelines apply:

a) *Isolated slopes and cliffs*. A value of $S_T \geq 1.2$ should be used for sites close to the upper edge.

b) *Crags with width at the top significantly less than at the base*. A value of $S_T \geq 1.4$ should be used in the vicinity of the top of the slope if the average slope is greater than 30° and $S_T \geq 1.2$ for lesser angles.

c) *Presence of a loose surface layer*. If a loose topsoil is present, the lowest $S_T$ value given in a) and b) should be increased by at least 20 %.

d) *Spatial variation of the amplification coefficient*. The $S_T$ coefficient can be assumed to decrease linearly with height above the base of the crest or cliff, to a value equal to unity at its base.

**A.3** In general, seismic amplification also decreases rapidly with depth within the crest. Therefore, the topographic effects to be considered in stability analyses are larger and mainly of a superficial type at the crest of the ridges, and much smaller for deep-seated landslides with rupture surfaces passing near the base of the slopes. In the latter case, if a pseudo-static calculated method is used, topographic effects can be neglected.

# Appendix B

# Empirical curves for simplified liquefaction analysis

## B.1 General considerations

The empirical curves used for simplified liquefaction analysis represent correlations between *in situ* measurements and cyclic shear stresses known to have produced liquefaction in previous earthquakes. The horizontal axis of these plots represents a soil property measured in situ, such as the normalised penetration resistance or shear wave propagation velocity, $v_s$, while the vertical axis represents the seismically induced cyclic shear stress, $\tau_e$, usually normalised by the initial effective pressure, $\sigma'_{vo}$. All graphs show a cyclic resistance limit curve separating the zone of no liquefaction (on the right) from the zone where liquefaction is possible (on the left and above the curve). In some cases there is more than one curve; e.g. for soils with different fines content or for earthquakes of different magnitudes.

Except when using the CPT resistance, it is preferable not to apply the empirical liquefaction criteria in cases where potentially liquefiable soils occur in layers or lenses not more than a few tens of centimetres thick.

In soils with a significant gravel content, liquefaction susceptibility cannot be ruled out, although the available observational data are insufficient at this stage to construct a reliable liquefaction graph.

## B.2 Curves based on the SPT index

Among the most commonly used graphs are the curves shown in Figure B.1 for clean sands and silty sands. The normalised SPT index, corrected for the effects of pressure and energy quotient, $N_1(60)$, is obtained as indicated in section **4.1.4**.

Liquefaction is not likely to occur below a certain threshold of $\tau_e$ because the soil behaves elastically and no dynamic interstitial pressure increases are generated. Therefore, the limit curve cannot be extrapolated to the origin. To apply this criterion to earthquakes of magnitude other than $M_w$ = 7.5 (where $M_w$ is the moment magnitude), the values of the ordinates of the curves in Figure B.1 should be multiplied by the CM coefficient given in Table B.1.

**Table B.1 – Values of the CM coefficient**

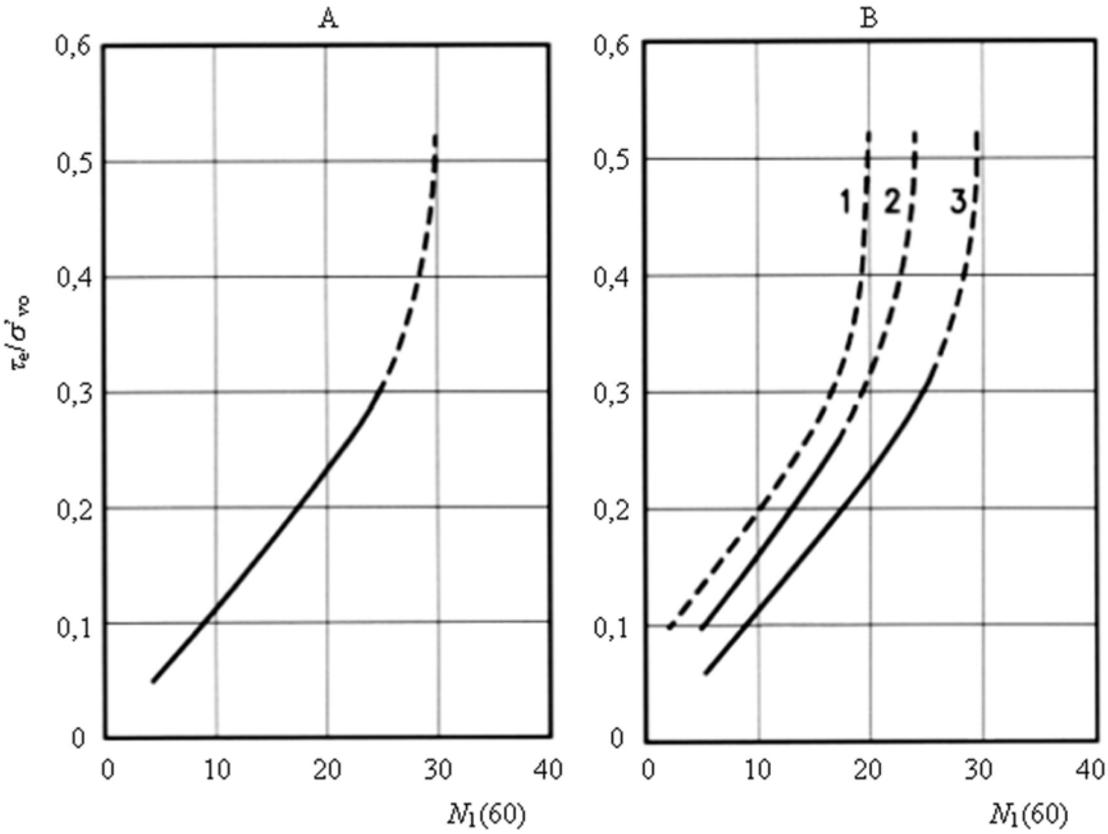
| $M_w$ | CM |
|---|---|
| 5.5 | 2.86 |
| 6.0 | 2.20 |
| 6.5 | 1.69 |
| 7.0 | 1.30 |
| 8.0 | 0.67 |

## B.3 Curves based on the resistance obtained from the CPT test

Design curves similar to those in Figure B.1 have been established on the basis of numerous studies on the correlation between the static penetration resistance, CPT, and the liquefaction resistance of the soil. These direct correlations are to be preferred to indirect correlations using the relationship between the SPT index and the CPT parameter (penetration rate with the mechanical cone).

## B.4 Shear wave velocity based curves, $v_s$

This property has a promising future as a field index to assess the liquefaction susceptibility of soils which are difficult to sample (such as silts and sands) or difficult to penetrate (gravels). Moreover, in recent years there have been significant advances in the in situ measurement of the $v_s$ parameter. However, correlations between $v_s$ and soil resistance to liquefaction are still under development and should not be used without the technical assistance of a specialist in this field.




Legend:

| | | | |
|---|---|---|---|
| $\tau_e$ / $\sigma'_{vo}$ | Normalised cyclic shear stress | | |
| A | Clean sands | B | Silty sands |
| | | | Curve 1: 35 % fineness |
| | | | Curve 2: 15 % fineness |
| | | | Curve 3: < 5 % fineness |

**Figure B.1 - Relationship between the ratio of liquefying stresses to the values of $N_1$ (60) values for clean sands and silty sands subjected to earthquakes of magnitude $M_w$ = 7.5**

## Appendix C

## Recommended expressions for the determination of the static stiffness of piles

**C.1** The pile rigidity is defined as the force (moment) to be applied to the pile head to produce a unit displacement (rotation) in the same direction (displacements/rotations being zero in the other directions); is designated as $K_{HH}$, (horizontal stiffness), $K_{MM}$, (bending stiffness) and $K_{HM} = K_{MH}$ (mixed rigidity).

The following notation is used in Table C.1:

$E$ is the Young's Modulus of the soil, equal to 3G;

$E_p$ is the Young's Modulus of the pile material;

$E_s$ is the Young's Modulus of the soil at a depth equal to the pile diameter;

$d$ is the diameter of the pile;

$z$ is the depth.

**Table C.1 - Expressions of the static rigidity of flexible piles founded on three different soil models**

| **Soil model** | $\frac{K_{HH}}{dE_s}$ | $\frac{K_{MM}}{d^3E_s}$ | $\frac{K_{HM}}{d^2E_s}$ |
|---|---|---|---|
| $E = E_s \cdot z/d$ | $0,60\left(\frac{E_p}{E_s}\right)^{0,35}$ | $0,14\left(\frac{E_p}{E_s}\right)^{0,80}$ | $-0,17\left(\frac{E_p}{E_s}\right)^{0,60}$ |
| $E = E_s\sqrt{z/d}$ | $0,79\left(\frac{E_p}{E_s}\right)^{0,28}$ | $0,15\left(\frac{E_p}{E_s}\right)^{0,77}$ | $-0,24\left(\frac{E_p}{E_s}\right)^{0,53}$ |
| $E = E_s$ | $1,08\left(\frac{E_p}{E_s}\right)^{0,21}$ | $0,16\left(\frac{E_p}{E_s}\right)^{0,75}$ | $-0,22\left(\frac{E_p}{E_s}\right)^{0,50}$ |

# Appendix D

## Recommendations on soil-structure dynamic interaction: general effects and significance

**D.1** As a result of dynamic soil-structure interaction, the seismic response of a structure with a flexible foundation, i.e. a structure founded on deformable terrain, will differ in several respects from the response of the same structure founded on rigid terrain (fixed foundation) and subjected to the same free-field excitation for the following reasons:

a) the foundation movement of a structure supported on a flexible foundation will differ from the free-field movement and may include a significant twisting component which will not occur in the rigid foundation case;

b) the fundamental period of vibration of a structure with a flexible foundation will be greater than that of a structure on a fixed foundation;

c) the natural periods, modes of vibration and modal distribution coefficients of a structure founded on a flexible foundation shall be different from those of a structure on a fixed foundation;

d) the overall damping of a flexibly founded structure shall include both geometric damping and internal soil damping, in addition to the damping associated with the superstructure.

**D.2** For most of the usual building structures, the effects of dynamic soil-structure interaction tend to be beneficial as they reduce the bending moments and shear forces in the various elements of the superstructure. However, for the structures listed in Chapter 6, such effects can be detrimental.

# Appendix E

## Simplified analysis of earth retaining structures

**E.1** Conceptually, the coefficient *r* is defined as the quotient between the value of the acceleration that produces the maximum permanent displacement compatible with the existing restraints and the value corresponding to the limit equilibrium state (occurrence of displacements). Therefore, *r* is higher for walls that can tolerate larger displacements.

**E.2** For retaining structures over 10 m in height, a one-dimensional free-field analysis of the vertical wave propagation can be made, which improves the estimation $\alpha$ of the effects of expression (7.1) by considering an average value of the maximum horizontal accelerations of the tiled floor over the height of the structure.

**E.3** The total calculated force, $E_d$, acting on the rear face of a retaining structure is given by

$$E_{\mathrm{d}} = \frac{1}{2}\gamma^{*}(1 \pm k_{\mathrm{v}})\,K \cdot H^{2} + E_{\mathrm{ws}} + E_{\mathrm{wd}} \qquad \text{(E.1)}$$

Where

*S* is the height of the wall;

$E_{ws}$ is the static water force;

$E_{wd}$ is the dynamic water force (defined below);

$\gamma^*$ is the specific gravity of the soil (defined in formulae (E.5) to (E.7) below);

*K* is the earth buoyancy coefficient (static + dynamic);

$k_v$ is the vertical seismic coefficient (see expressions (7.2) and (7.3)).

**E.4** The earth thrust coefficient can be calculated from the Mononobe-Okabe expression:

For active states:

if $\beta \leq \phi'_{\mathrm{d}} - \theta$

$$K = \frac{\operatorname{sen}^{2}(\psi + \phi'_{\mathrm{d}} - \theta)}{\cos\theta \operatorname{sen}^{2}\psi \operatorname{sen}(\psi - \theta - \delta_{\mathrm{d}})\left[1 + \sqrt{\dfrac{\operatorname{sen}(\phi'_{\mathrm{d}} + \delta_{\mathrm{d}})\operatorname{sen}(\phi'_{\mathrm{d}} - \beta - \theta)}{\operatorname{sen}(\psi - \theta - \delta_{\mathrm{d}})\operatorname{sen}(\psi + \beta)}}\right]^{2}} \qquad \text{(E.2)}$$

if $\beta > \phi'_d - \theta$

$$K = \frac{\mathrm{sen}^2(\psi + \phi - \theta)}{\cos\theta \ \mathrm{sen}^2 \psi \ \mathrm{sen}(\psi - \theta - \delta_d)} \tag{E.3}$$

For passive states (no friction between floor and wall):

$$K = \frac{\mathrm{sen}^2(\psi + \phi'_d - \theta)}{\cos\theta \ \mathrm{sen}^2 \psi \ \mathrm{sen}(\psi + \theta)\left[1 - \sqrt{\dfrac{\mathrm{sen}\,\phi'_d \ \mathrm{sen}(\phi'_d + \beta - \theta)}{\mathrm{sen}(\psi + \beta)\ \mathrm{sen}(\psi + \theta)}}\right]^2} \tag{E.4}$$

In the above expressions the following notation has been used:

$\phi'_d$ is the calculated value of the calculated angle of shear resistance of the soil, i.e.

$$\phi'_d = \tan^{-1}\left(\frac{\tan\phi'}{\gamma_{\phi'}}\right);$$

$\psi$ and $\beta$ are the angles of inclination of the back of the wall and the backfill surface measured from the horizontal, as shown in Figure E.1;

$\delta_d$ is the calculated value of the angle of friction between the wall and the soil, i.e.

$$\delta_d = \tan^{-1}\left(\frac{\tan\delta}{\gamma_{\phi'}}\right);$$

$\theta$ is the angle defined in sections **E.5** to **E.7** below.

The passive state expressions should preferably be used for walls with a vertical wall ($\psi$ = 90°).

## E.5 Water table below the base of the retaining wall. Earth thrust coefficient

The following parameters apply:

$\gamma^*$ is the specific gravity $\gamma$ of the soil (E.5)

$$\tan\theta = \frac{k_h}{1 \mp k_v} \tag{E.6}$$

$$E_{wd} = 0 \tag{E.7}$$

Where

$k_h$ is the horizontal seismic coefficient (see expression (7.1)).

Alternatively, the tables and graphs applicable to the static case (with gravity loads only) can be used, with the following modifications:

where

$$\tan \theta_A = \frac{k_h}{1 + k_v} \tag{E.8}$$

and

$$\tan \theta_B = \frac{k_h}{1 - k_v} \tag{E.9}$$

the complete wall-floor system is rotated, as appropriate, by an additional angle $\theta_A$ or $\theta_B$. The acceleration of gravity is replaced by the following value:

$$g_A = \frac{g(1 + k_v)}{\cos \theta_A} \tag{E.10}$$

or

$$g_B = \frac{g(1 - k_v)}{\cos \theta_B} \tag{E.11}$$

## E.6 Soil below the water table under dynamically impermeable conditions. Earth thrust coefficient

The following parameters are used:

$$\gamma^* = \gamma - \gamma_w \tag{E.12}$$

$$\tan \theta = \frac{\gamma}{\gamma - \gamma_w} \frac{k_h}{1 \mp k_v} \tag{E.13}$$

$$E_{wd} = 0 \tag{E.14}$$

Where

$\gamma$ is the specific (total) weight of the saturated soil;

$\gamma_w$ is the specific gravity of water.

## E.7 Soil below the water table under conditions of (high) dynamic permeability. Earth thrust coefficient

The following parameters are used:

$$\gamma^* = \gamma - \gamma_W \tag{E.15}$$

$$\tan \theta = \frac{\gamma_d}{\gamma - \gamma_W} \frac{k_h}{1 \mp k_v} \tag{E.16}$$

$$E_{wd} = \frac{7}{12} k_h \gamma_W \cdot H'^2 \tag{E.17}$$

Where

$\gamma_d$ is the specific gravity of the dry soil;

$H'$ is the height of the water table measured from the base of the wall.

## E.8 Hydrodynamic pressure on the outer face of the wall

This pressure, $q(z)$, can be estimated as follows:

$$q(z) = \pm \frac{7}{8} k_h \cdot \gamma_w \cdot \sqrt{h \cdot z} \tag{E.18}$$

Where

$k_h$ is the horizontal seismic coefficient with $r$ = 1 (see expression (7.1));

$h$ is the height of the water surface;

$z$ is the vertical coordinate, measured downward, with the origin at the water surface.

## E.9 Earth pressure in rigid structures

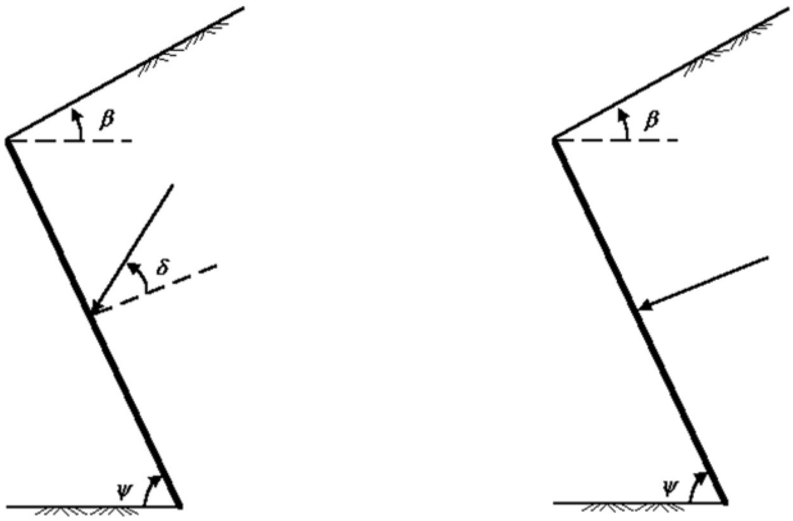
In rigid structures that are completely fixed, and therefore no active states can develop in the soil, and in vertical walls with horizontal backfill, the dynamic increment of the earth thrust can be taken equal to:

$$\Delta P_d = \alpha \cdot S \cdot \gamma \cdot H^2 \tag{E.19}$$

Where

$S$ is the height of the wall.

The point of application can be taken at the mid-height of the active wall.




active passive

**Figure E.1 - Convention for the measurement of the angles used in the formulae for calculating the earth thrust factor**

# Appendix F

# Recommendations for the determination of the bearing capacity of shallow foundations subjected to seismic loading

## F.1 General expression

The bearing capacity failure stability of a shallow footing supported on the surface of a homogeneous soil and subjected to seismic loading can be tested using the following expression, which relates the resistance of the soil to the effects of the calculated actions at the foundation level ($N_{Ed}$, $V_{Ed}$, $M_{Ed}$) and to the inertia forces in the soil:

$$\frac{\left(1 - e\bar{F}\right)^{c_T}\left(\beta\bar{V}\right)^{c_T}}{\left(\bar{N}\right)^{a}\left[\left(1 - m\bar{F}^{k}\right)^{k'} - \bar{N}\right]^{b}} + \frac{\left(1 - f\bar{F}\right)^{c'_M}\left(\gamma\bar{M}\right)^{c_M}}{\left(\bar{N}\right)^{c}\left[\left(1 - m\bar{F}^{k}\right)^{k'} - \bar{N}\right]^{d}} - 1 \leq 0 \tag{F.1}$$

where

$$\bar{N} = \frac{\gamma_{Rd}\, N_{Ed}}{N_{máx.}}, \qquad \bar{V} = \frac{\gamma_{Rd}\, V_{Ed}}{N_{máx.}}, \qquad \bar{M} = \frac{\gamma_{Rd}\, M_{Ed}}{B\, N_{máx.}} \tag{F.2}$$

$\bar{N}$ is the ultimate bearing capacity of the foundation for a centred vertical load (defined in sections **F.2** and **F.3**);

$N_{max.}$ is the width of the foundation;

$B$ is the dimensionless soil inertia force (defined in sections **F.2** and **F.3**);

$\bar{F}$ is the partial model coefficient (values for this parameter are given in section **F.6**).

$a$, $b$, $c$, $d$, $e$, $f$, $m$, $k$, $k'$, $c_T$, $c_M$, $c'_M$, $\beta$, $\gamma$ are numerical parameters depending on the soil type, defined in section **F.4**.

## F.2 Purely cohesive soil

For purely cohesive soils or saturated cohesionless soils the ultimate bearing capacity for a centred vertical load, $N_{max}$, is given by:

$$N_{máx.} = (\pi + 2)\,\frac{\bar{c}}{\gamma_M}\,B \tag{F.3}$$

Where

$\bar{c}$ is the undrained soil shear resistance, $c_u$, for a cohesive soil, or undrained cyclic shear resistance, $\tau_{cy,u}$, for a cohesionless soil;

$\gamma_M$ is the partial safety coefficient for the material properties of the soil (see **3.1(3)**).

The dimensionless inertia force $\bar{F}$ of the soil is given by:

$$\overline{F} = \frac{\rho \cdot a_g \cdot S \cdot B}{\overline{c}} \tag{F.4}$$

Where

$\rho$ is the density of the soil

$a_g$ is the calculated value of the ground acceleration in a terrain type A ($a_g = \gamma_I\, a_{gR}$);

$a_{gR}$ is the maximum reference ground acceleration in a terrain type A

$\gamma_I$ is the coefficient of importance

$S$ is the soil coefficient as defined in section **3.2.2.2** of Annex 1.

The following limitations apply to the general expression of bearing capacity:

$$0 < \overline{N} \leq 1, \quad \left|\overline{V}\right| \leq 1 \tag{F.5}$$

## F.3 Cohesionless soil

For purely granular, cohesionless, either dry or saturated soils with no appreciable growth of dynamic interstitial pressure, the ultimate bearing capacity of the foundation for a centred vertical load, $N_{max.}$, is given by:

$$N_{máx.} = \frac{1}{2}\rho\, g \left(1 \pm \frac{a_v}{g}\right) B^2 N_\gamma \tag{F.6}$$

Where

$g$ is the acceleration of gravity

$a_v$ is the vertical acceleration of the terrain (can be taken equal to 0,5 $a_g \cdot S$

$N_\gamma$ s the bearing capacity coefficient, which is a function of the calculated value of the internal friction angle of the soil, $\phi'_d$ (which includes the partial safety coefficient, $\gamma_M$, from section **3.1(3)**) for the corresponding material property (see **E.4**).

The dimensionless inertia force, $\overline{F}$, of the soil is given by:

$$\overline{F} = \frac{a_g}{g \tan \phi'_d} \tag{F.7}$$

The following limitation applies to the general expression of the bearing capacity in this case:

$$0 < \overline{N} \leq \left(1 - m\overline{F}\right)^{k'} \tag{F.8}$$

## F.4 Numerical parameters

The values of the numerical parameters in the general bearing capacity expression are given in Table F.1, depending on the soil types identified in sections **F.2** and **F.3**.

**Table F.1 – Values of numerical parameters used in the expression (F.1)**

| | Purely cohesive soil | Cohesionless soil |
|---|---|---|
| $A$ | 0.70 | 0.92 |
| $B$ | 1.29 | 1.25 |
| $C$ | 2.14 | 0.92 |
| $D$ | 1.81 | 1.25 |
| $E$ | 0.21 | 0.41 |
| $F$ | 0.44 | 0.32 |
| $M$ | 0.21 | 0.96 |
| $K$ | 1.22 | 1.00 |
| $k'$ | 1.00 | 0.39 |
| $c_{\mathrm{T}}$ | 2.00 | 1.14 |
| $c_{\mathrm{M}}$ | 2.00 | 1.01 |
| $c'_{\mathrm{M}}$ | 1.00 | 1.01 |
| $\beta$ | 2.57 | 2.90 |
| $\gamma$ | 1.85 | 2.80 |

**F.5** In most cases $\overline{F}$ can be taken equal to 0 for cohesive soils. For cohesionless soils $\overline{F}$ it can be neglected if $a_g \cdot S < 0.1\, g$ (i.e. if $a_g \cdot S < 0.98$ m/s$^2$).

**F.6** The values of the partial model coefficient, $\gamma_{\mathrm{Rd}}$, are given in Table F.2 as a function of soil type.

**Table F.2 – Values of the partial model coefficient, $\gamma_{\mathrm{Rd}}$**

| Medium or dense sand | Dry loose sand | Saturated loose sand | Non-sensitive clay | Sensitive (thixotropic) clay |
|---|---|---|---|---|
| 1.00 | 1.15 | 1.50 | 1.00 | 1.15 |

# ANNEX 6

# Design of seismically resistant structures

# Towers, masts and chimneys

## 1 General considerations

### 1.1 Object and field of application

(1) The scope of the Seismic Resistance Standard is defined in section **1.1.1** of Annex 1, and the scope of this Annex is defined in items **(2)** to **(4)** of this section. The other parts of the Seismic Resistance Standard are listed in section **1.1.3** of Annex 1.

(2) This Annex 6 sets out the requirements, criteria and rules for the design of tall and slender structures: towers, including bell towers, tapping towers, radio and television towers, masts, chimneys (including self-supporting industrial chimneys) and lighthouses. Chapters **5** and **6** set out, respectively, specific additional provisions for reinforced concrete and steel chimneys. Chapters **7** and **8** provide, respectively, specific additional provisions for steel towers and for guyed steel masts. Requirements for non-structural elements, such as antennae, internal lining material of chimneys and other equipment, are also laid down.

NOTE 1 Appendix A provides guidance and information for linear dynamic analysis, taking into account the rotational components of ground motion.

NOTE 2 Appendix B provides information and guidance on modal damping in response spectrum modal analysis.

NOTE 3 Appendix C provides information on soil-structure interaction and guidance for taking it into account in the linear dynamic analysis.

NOTE 4 Appendix D provides additional information and guidance on the number of degrees of freedom and the number of modes of vibration to be considered in the analysis.

NOTE 5 Appendix E provides information and guidance for the seismic resistance calculation of factory chimneys.

NOTE 6 Appendix F provides additional information on the seismic behaviour and design of power line transmission towers.

NOTE 7 Appendices A, B, C, D, E and F of this Annex are not regulatory.

(3) These provisions do not apply to cooling towers and offshore structures.

(4) For towers supporting tanks, Annex 4 applies.

### 1.2 Standards for reference and consultation

(1) The provisions of section **1.2** of Annex 1 apply.

## 1.3 Assumptions

(1) The provisions of section **1.3** of Annex 1 apply.

## 1.4 International system of units (I.S.)

(1) The provisions of section **1.4** of Annex 1 apply.

## 1.5 Terms and definitions

### 1.5.1 Common Terms and Other Terms Used in this Seismic Resistance Standard

(1) The specifications in sections **1.5.1** and **1.5.2** of Annex 1 apply.

### 1.5.2 Other terms used in this Annex

**tower:**
A self-supporting, cantilevered lattice steel structure, triangular, square or rectangular in plan, or circular and polygonal monopoles.

**transmission tower:**
A tower used to support low or high voltage electrical transmission cables.

**angular tower:**
Transmission tower used where the cable line changes direction forming an angle in plan of more than 3°. It supports the same types of actions as a tangent tower.

**dead end tower** (also called anchor towers)**:**
Transmission towers capable of supporting the dead end stresses of all cables located on the same side, in addition to vertical and transverse loads.

**tangent tower:**
Transmission tower used where the cable line is straight or forms an angle in plan not exceeding 3°. It supports vertical loads, a transverse load due to the angular tension of the cables, a longitudinal load due to the difference in length of the supports and the forces resulting from cable tensioning or cable breakage.

**telescopic joint:**
Joint between tubular elements without flange with the inside diameter of one element being equal to the outside diameter of the other.

**main elements:**
Elements that constitute the main load-bearing resistance system in the truss structure.

**primary bracing elements:**
Elements other than the main elements that transmit the forces caused by the loads imposed on the structure.

**secondary bracing elements:**
Elements used to reduce the buckling lengths of other elements.

**cable-stayed mast:**
A steel lattice structure, triangular, square or rectangular in plan, or a cylindrical steel structure, stabilised at intervals in its height by braces that are anchored to the ground or to a permanent structure.

**mast shaft:**
The vertical steel structure of a mast.

**chimney:**
Vertical construction work or building components that conduct waste or other combustion, supply or exhaust gases into the atmosphere.

**self-supporting chimney:**
Chimney whose supporting shaft is not connected to any other construction above the level of its base.

**cable-stayed chimney:**
Chimney whose load-bearing shaft is supported by braces at one or more levels of its height.

**structural chimney shaft:**
The main steel supporting structure of the chimney, excluding any flanges.

**inner flue or liner:**
Structural element (membrane) of the inner liner system, contained within the structural shaft.

**inner liner system:**
Complete system, if any, that separates the flue gases from the structural shaft. This includes the ducts and their supports, the space between the inner liner or duct and the structural shaft, and the isolation, if any.

**stiffening rings:**
Horizontal elements to prevent ovalisation and keep the chimney shaft circular during manufacture and transportation. Horizontal elements to provide stiffening at cutouts and openings, and at possible changes in slope of the structural shaft.

**tie rod:**
An element subject only to tension, connected at the ends to terminals to form an assembly that provides the structure with horizontal support at discrete levels. The lower end of the assembly is anchored to the ground or to another structure and usually includes devices for adjusting the tension in the tie rod.

## 1.6 Symbols

### 1.6.1 General considerations

(1) The specifications in sections **1.6.1** and **1.6.2** of Annex 1 apply.

(2) For ease of use, other symbols used in connection with the seismic calculated calculation of towers, masts and chimneys are defined in the text where they appear. However, additionally, the symbols that appear most frequently in this Annex are listed and defined in section **1.6.2**.

### 1.6.2 Other symbols used in this Annex

$E_{eq}$ equivalent modulus of elasticity;
$M_i$ effective modal mass for the ith mode of vibration;
$R^\theta$ ratio of the maximum moment on the spring of an oscillator with one rotational degree of freedom to the moment of inertia about the axis of rotation. The representation of $R^\theta$ as a function of the natural period is the rotation response spectrum;
$R_x^\theta$, $R_y^\theta$, $R_z^\theta$, rotation response spectra with respect to the x, y, and z axes, in rad/s$^2$;
$\gamma$ unit weight of the cable;
$\sigma$ tensile stress in the cable;
$\bar{\xi}_j$ equivalent modal damping ratio or index (with respect to the critical) for the jth mode.

## 2 Performance requirements and compliance criteria

### 2.1 Key requirements

(1) For the types of structures covered in this Annex, the requirement of non-collapse, or non-collapse, specified in section **2.1(1)** of Annex 1 applies in order to protect the safety of persons, nearby buildings and adjacent installations.

(2) For the types of structures covered in this Annex, the damage minimisation requirement specified in section **2.1(1)** of Annex 1 applies in order to maintain continuity of operation of facilities, industries and communications systems in the event of earthquakes.

(3) The damage minimisation requirement refers to a seismic action that has a probability of exceedance greater than that of the calculated seismic action. The structure must be designed and built to withstand this action without damage or limitation of use, the cost of this damage being assessed taking into account the effects on the equipment supported by the structure and the limitation of use due to the interruption of the operation of the installation.

(4) As defined in section **2.2.1(3)** and section **3.2.1(4)** of Annex 1, in cases of low seismicity, the fundamental requirements may be satisfied by calculating the structure for the calculated seismic situation and for non-dissipative behaviour, disregarding energy dissipation by hysteresis and disregarding the rules of this Annex which specifically refer to energy dissipation capacity. In this case, the behavioural coefficient shall not be taken to be greater than 1,5, this value being taken into account in order to take account of over-resistance (see section **2.2.2(2)** of Annex 1).

### 2.2 Compliance criteria

#### 2.2.1 Laying foundations

(1) The calculated foundation must be in accordance with Annex 5.

#### 2.2.2 Ultimate limit status

(1) The provisions of section **2.2.2** of Annex 1 apply.

### 2.2.3 Damage limitation status

(1) In the absence of any specific owner requirements, the provisions specified in section **4.9** apply to ensure that the structure itself, non-structural elements and installed equipment do not suffer any damage considered unacceptable for this limit state. Deformation limits are set with respect to a seismic action having a probability of occurrence greater than that of the calculated seismic action, as specified in section **2.1(1)** of Annex 1.

(2) Unless special precautions are taken, the provisions of this Annex do not specifically provide any protection against damage to equipment and non-structural members from calculated seismic action as defined in section **2.1(1)** of Annex 1.

# 3 Seismic action

## 3.1 Definition of seismic excitation

(1) In addition to the translational components of seismic motion, as defined in sections **3.2.2** and **3.2.3** of Annex 1, the rotational component of the terrain motion shall be taken into account for high-rise structures in areas of high seismicity. Structures higher than 80 m in areas where the $a_g \cdot S$ product exceeds 0.25g shall be considered.

NOTE Appendix A provides a possible method for defining the rotational components of the motion and provides recommendations for taking them into account in the analysis.

## 3.2 Elastic response spectrum

(1) Section **3.2.2.2** of Annex 1 defines the elastic response spectrum in terms of acceleration for the horizontal translational components, and section **3.2.2.3** of Annex 1 for the vertical translational component.

## 3.3 Calculated response spectrum

(1) The calculated response spectrum is defined in section **3.2.2.5** of Annex 1. The value of the behavioural coefficient, $q$, reflects, in addition to the hysteresis dissipation capacity of the structure, the influence of viscous damping other than 5 %, including damping due to soil-structure interaction (see **2.2.2(2)** and **3.2.2.5(2)** and **(3)** of Annex 1).

(2) A calculated elastic behaviour up to the ultimate limit state may be appropriate for towers, masts and chimneys, depending on the transverse section of the elements. In this case, the factor $q$ shall not exceed $q$ = 1.5.

(3) As an alternative to **(2)**, the calculated elastic behaviour may be based on the elastic response spectrum with $q$ = 1.0 and appropriate damping values for the particular situation considered, in accordance with section **4.2.4**.

## 3.4 Representation as a function of time

(1) Section **3.2.2.5** of Annex 1 applies to the representation of the seismic action in terms of

acceleration as a function of time. In the case of rotational components of the terrain motion, rotational accelerations are simply used instead of translational accelerations.

(2) Separate time-dependent representations shall be used for any two different components of the terrain motion (including translational and rotational components).

### 3.5 Long-period components of single-point motion

(1) Towers, masts and chimneys are often sensitive to long period components of terrain movement. Soft soils or peculiar topographical conditions could abnormally amplify the long period content of ground motion. This amplification shall be taken into account appropriately.

NOTE Sections **4.2.2** of Annex 5 and **3.1.2** of Annex 1 provide the basis for the evaluation of soil type to determine appropriate terrain spectra. Appendix A of Annex 5 also gives recommendations for cases where topographic amplification of movement may be significant.

(2) Where site-specific studies have been carried out, with particular reference to the long-period content of the motion, it is appropriate to consider lower values of the factor $\beta$ in expression (3.16) in Annex 1.

NOTE For masts and chimneys the value of $\beta$ defined in Annex 1 for buildings applies. It may be taken $\beta$ = 0.1 only when specific local seismicity studies have been carried out with a special emphasis on the low frequency content of the seismic action.

### 3.6 Components of the terrain movement

(1) The two horizontal components and the vertical component of the seismic action shall be considered to act simultaneously.

(2) Where taken into account, the rotational components of the ground motion shall be considered as acting simultaneously with the translational components.

## 4 Seismic calculations for towers, masts and chimneys

### 4.1 Importance classes and importance factors

(1) Towers, masts and chimneys are classified into four classes of significance, depending on the consequences of their damage or collapse, their importance for public safety and civil protection in the immediate aftermath of the earthquake, and the social and economic consequences of collapse or damage.

(2) The definitions of the significance classes are given in Table 4.1.

**Table 4.1 – Importance classes for towers, masts and chimneys**

| Importance classes | |
|---|---|
| I | Towers, masts or chimneys of minimal importance to public safety |
| II | Towers, masts or chimneys which do not fall into classes I, III or IV |
| III | Towers, masts or chimneys whose collapse may affect surrounding buildings or areas with frequent crowding of persons |
| IV | Towers, masts or chimneys whose integrity is of vital importance to keep civil |

| | |
|---|---|
| | protection services operational (water supply systems, power generation plants, telecommunications, hospitals). |

(3) The importance factor $\gamma_I$ = 1.0 is associated with an earthquake with the reference return period indicated in section **3.2.1(3)** of Annex 1.

(4) The value of $\gamma_I$ for importance class II is, by definition, equal to 1.0.

(5) The importance classes are characterised by different importance factors $\gamma_I$, as described in section **2.1(3)** of Annex 1.

NOTE The values of $\gamma_I$ may be different for different seismic zones, depending on the seismic hazard conditions (see note in section **2.1(4)** of Annex 1) and public safety considerations. However, for the purposes of this Seismic Resistance Standard, the $\gamma_I$-factors take a constant value for each class of importance.

The $\gamma_I$ value for the different importance classes is:

- Importance class I (minor importance): $\gamma_I$ = 0.8
- Importance class II (normal importance): $\gamma_I$ = 1.0
- Importance class III (major importance): $\gamma_I$ = 1.3
- Importance class IV (special importance): $\gamma_I$ = 1.4

## 4.2 Modelling rules and assumptions

### 4.2.1 Number of degrees of freedom

(1) The mathematical model shall:

- take into account the rigidity of the foundation against rotation and translation;
- include sufficient degrees of freedom (and associated masses) to determine the response of any significant structural elements, equipment or appurtenances;
- include the rigidities of cables and braces;
- take into account relative displacements of equipment or machinery supports (e.g. interaction between an insulation layer and the outer pipe of a chimney);
- take into account duct-to-duct interactions, external structural constraints, hydrodynamic loads (both mass and rigidity effects as appropriate).

(2) Models of power transmission lines shall be representative of the entire line. As a minimum, at least three consecutive towers should be included in the model so that the mass and stiffness of the cables are representative of the conditions applicable to the central tower.

(3) Dynamic models of bell towers shall take into account the oscillation of the bells, if their mass is significant with respect to that of the highest part of the bell tower.

### 4.2.2 Masses

(1) The discretisation of the masses in the model must be representative of the distribution of the inertial effects of the seismic action. When a coarse discretisation of the translational masses is used, the rotational inertias of the masses must be assigned to the corresponding rotational degrees of freedom.

(2) The masses must include all permanent constructions, fittings, ducts, isolations, any dust or ash deposits adhering to the surface, existing or future linings, interior linings (including any relevant short or long term effects of fluids or moisture on their density) and equipment. The permanent value of the mass of structures or their permanent elements, etc., the quasi-permanent value of the mass of equipment and of the ice or snow load, and the quasi-permanent value of the load applied to platforms (arranged for maintenance and temporary equipment) must be taken into account.

(3) The combination coefficients $\Psi_{Ei}$ introduced in the expression (3.17) of section **3.2.4(2)** of Annex 1 for the calculation of the inertia effects of the seismic action, must be taken equal to the combination coefficients $\Psi_{2i}$ for the quasi-permanent value of the variable action $q_i$, in accordance with the specific regulations in force or, failing this, with the specific technical documents that the author of the project, under their responsibility, considers most appropriate.

(4) The mass of the cables and stays must be included in the model.

(5) If the mass of the cable or stay is significant compared to that of the tower or mast, the cable or stay should be modelled as a system of concentrated masses.

(6) The total effective mass of the submerged part of the intake towers should be taken equal to the sum of:

- the actual mass of the tower shaft (without reduction for hydrostatic thrust);

- the mass of any water contained inside the tower (hollow towers);

- the added mass of external water moving in phase with the tower;

NOTE In the absence of a rigorous analysis, the added mass of external water can be estimated according to Appendix F of Annex 2.

### 4.2.3 Rigidity

(1) For concrete members, the rigidity properties shall be evaluated taking into account the cracking effect. If the calculated value is based on a value of $q$ greater than 1, with the corresponding calculated spectrum, these rigidity properties shall correspond to the onset of plastification and can be determined in accordance with section **4.3.1(6)** and **(7)** of Annex 1. If the calculation is based on a value of $q$ = 1 and on the elastic response spectrum or on a time-dependent representation of the terrain movement, the stiffness of the concrete elements shall be calculated from the properties of the cracked transverse section that are consistent with the stress level during seismic action.

(2) The effect of high temperatures on the rigidity and resistance of steel or reinforced concrete, in steel or concrete chimneys respectively, shall be taken into account.

(3) If a cable is modelled as a single spring for its entire length, rather than as a series of concentrated masses connected by springs, the rigidity of the single spring shall take into account the deflection of the cable. This can be achieved by using the following equivalent modulus of elasticity:

$$E_{eq} = \frac{E_c}{1 + \frac{(\gamma \ell)^2}{12\sigma^3} E_c} \tag{4.1}$$

where

$E_{eq}$ is the equivalent modulus of elasticity;

$\gamma$ is the weight per unit length of the cable, including the weight of any ice load on the cable in the calculated seismic situation;

$\sigma$ is the tensile stress in the rope;

$l$ is the length of the cable;

$E_c$ is the modulus of elasticity of the cable material.

(4) For cable wires consisting of stranded wires or stranded wires, $E_c$ is generally less than the modulus of elasticity $E$ of only one of their strands. In the absence of specific data provided by the manufacturer, the following reduction may be applied:

$$\frac{E_c}{E} = \cos^3 \beta \tag{4.2}$$

where $\beta$ is the strand angle of a single strand.

(5) If the preload of the rope is such that the sag is negligible, or if the tower height is less than 40 m, the rope may then be modelled as a linear spring.

NOTE As specified in section **4.2.2(4)**, the mass of the wire rope shall be fully taken into account.

### 4.2.4 Damping

(1) If the analysis is carried out according to section **3.3(3)** on the basis of the elastic response spectrum of section **3.2.2.2** of Annex 1, a viscous damping different from 5 % may be adopted. In this case, a modal analysis by response spectrum with a different damping to critical ratio for each mode of vibration can be applied.

NOTE A modal analysis procedure by response spectrum taking into account modal damping is developed in Appendix B.

### 4.2.5 Soil-structure interaction

(1) For structures founded on soft soil tanks, the specifications in section **4.3.1(9)** of Annex 1 are applied for the calculated soil-structure interaction effects.

NOTE 1 Appendix C provides criteria for taking into account soil-structure interaction in the analysis.

NOTE 2 For tall structures, e.g. those with a height greater than five times the maximum base dimension, the resistance to soil heave is of importance and can significantly increase second order effects.

## 4.3 Methods of analysis

### 4.3.1 Applicable methods

(1) The effects of the seismic action and the effects of the other actions considered in the calculated seismic situation can be determined on the basis of a linear elastic behaviour of the structure.

(2) The specifications in section **4.3.3.1(2)**, **(3)**, **(4)** and **(5)** of Annex 1 apply.

NOTE The note to section **4.3.3.1(4)** of Annex 1 applies.

(3) For the 'rigid diaphragm' scenario to be applicable to steel towers, the towers must be fitted with a horizontal bracing system capable of providing the required rigid diaphragm action.

(4) For the "rigid diaphragm" scenario to be applicable to steel chimneys, they must be fitted with horizontal stiffening rings at short intervals.

(5) If the conditions for the application of the "rigid diaphragm" hypothesis are not satisfied, a three-dimensional dynamic analysis, capable of representing the deformation of the structure in horizontal planes, must be carried out.

### 4.3.2 Lateral force method

#### 4.3.2.1 General considerations

(1) This type of analysis is applicable to structures satisfying the following two conditions:

a) The lateral rigidity and mass distribution are approximately symmetrical in plan about two orthogonal horizontal axes, so that a separate model can be used along each of these orthogonal axes.

b) The response is not significantly affected by the contribution of the higher modes of vibration.

(2) For the condition of **(1)(b)** to be satisfied, the fundamental period in each of the two horizontal directions of **(1)(a)** shall satisfy section **4.3.3.2.1(2)(a)** of Annex 1. In addition, the lateral rigidity, mass and horizontal dimensions of the structure shall remain constant or be reduced gradually, without abrupt change, from the base to the crown.

The supplementary or detailing conditions for the lateral force analysis method to be applicable are: $H \leq 40$ metres and importance class I or II.

(3) If the relative displacement between the supports of piping and equipment supported at different points is of importance for the testing of such piping or equipment, a modal analysis by response spectrum shall be used to take into account the contribution of higher modes to the magnitude of this relative displacement.

NOTE The lateral force analysis method may underestimate the magnitude of the differential displacement between different points of the structure.

#### 4.3.2.2 Seismic forces

(1) The analysis for the determination of the effects of seismic action consists of the application of horizontal forces $F_i$, $i$ = 1, 2, ... $n$, to the $n$ concentrated masses in which the structure has been discretised, including the foundation masses. The sum of these forces is equal to the shear force at the base, which is taken to be equal to:

$$F_t = S_d(T) \sum_1^n m_j \tag{4.3}$$

where

$S_d(T)$ is the ordinate of the calculated response spectrum, as defined in section **3.2.2.5** of Annex 1, for the fundamental period of vibration, $T$, in the horizontal direction of the lateral forces. If the period $T$ is not evaluated as in section **4.3.3.2.2(2)** of Annex 1, the spectral value $S_d(T_c)$ shall be taken in expression (4.3).

(2) The distribution of the horizontal forces $F_i$ between the $n$ concentrated masses must be carried out as specified in section **4.3.3.2.3** of Annex 1.

NOTE The lateral force method normally overestimates the effects of seismic action for conical towers where the distribution of masses decreases appreciably with height.

### 4.3.3 Modal analysis by response spectrum

#### 4.3.3.1 General considerations

(1) This method of analysis can be applied to all structures, defining the seismic action by means of a response spectrum.

#### 4.3.3.2 Number of modes

(1) The specifications in section **4.3.3.3.1(2)** of Annex 1 apply.

(2) The requirements specified in **(1)** may be considered to be satisfied if the sum of the effective modal masses for the modes taken into account is at least 90 % of the total mass of the structure.

NOTE 1 Appendix D provides additional information and recommendations for the application of **(2)**.

NOTE 2 The number of modes required to calculate the seismic actions at the top of the structure is generally greater than is sufficient to evaluate the overturning moment or the total shear stress at the base of the structure.

NOTE 3 Quasi-asymmetric structures generally have modes of vibration in close proximity that require special consideration.

#### 4.3.3.3 Combination of modes

(1) The specifications in section **4.3.3.3.2(1)**, **(2)** and **(3)** apply for the combination of the maximum modal responses.

## 4.4 Combination of the effects of the seismic action components

(1) The effects of any rotation component of the ground motion around a horizontal direction can be combined with those of the translation component in the orthogonal horizontal direction by the rule of the square root of the sum of the squares (SRSS combination).

(2) The combination of the effects of the seismic action components shall be taken into account by one of the two alternative procedures specified in section **4.3.3.5.2(4)** of Annex 1. According to **(1)**, for the application of the procedure in section **4.3.3.5.2(4)** of Annex 1, based on expressions (4.20) to (4.22), the rotation components around a horizontal direction shall first be combined with those of the translation component in the orthogonal horizontal direction.

## 4.5 Combination of seismic and other actions

(1) For the combination of the seismic action with other actions in the calculated seismic situation, the specifications of section **6.4.3.4** of Annex 18 of the Structural Code and section **3.2.4(1)** and **(4)** of Annex 1 applies.

## 4.6 Displacements

(1) For the calculation of the displacements induced by the calculated seismic action, the specifications in sections **4.3.4(1)** and **(3)** apply.

## 4.7 Safety testing

### 4.7.1 Ultimate limit state

(1) The requirement of non-collapse (ultimate limit state) for the calculated seismic situation is considered to be satisfied if the conditions detailed in the following sections relating to element and joint resistance, ductility and stability are fulfilled.

### 4.7.2 Resistance condition of structural elements

(1) The following ratio must be satisfied for all structural members, including joints:

$$R_{\mathrm{d}} \geq E_{\mathrm{d}} \tag{4.4}$$

where

$R_{\mathrm{d}}$ is the calculated design resistance of the element, calculated according to the mechanical models and material specific rules (in terms of characteristic values of the material property, $f_{\mathrm{k}}$, and the partial safety factor, $\gamma_{\mathrm{M}}$);

$E_{\mathrm{d}}$ is the calculated value of the effect of the action for the calculated seismic situation (see **6.4.3.4** of Annex 18 of the Structural Code), including, if necessary, second order effects (see **4.7.3**) and thermal effects (see **4.8**). Redistribution of bending moments is permitted in accordance with Annexes 19, 22 and 30 of the Structural Code.

NOTE The same values of partial factors of safety for steel, concrete, structural steel, masonry and other materials are to be used for the partial factors of safety as defined in sections **5.2.4(3)**, **6.1.3(1)**, **7.1.3(1)** and **9.6(3)** of Annex 1.

### 4.7.3 Second order effects

(1) Second order effects must be taken into account, unless the condition in **(2)** is satisfied.

(2) Second order effects need not be taken into account if the following condition is satisfied:

$$\frac{\delta M}{M_0} < 0,10 \tag{4.5}$$

where

$\delta M$ is the overturning moment due to the second order effect, effect (P-Δ);

$M_0$ is the first order overturning moment.

### 4.7.4 Resistance of joints

(1) For non-dissipative welded or bolted joints, the resistance must be determined in accordance with the provisions of Annex 22 of the Structural Code.

(2) The resistance provided by welded or bolted dissipative connections must be greater than the plastic resistance of the connected dissipative element, calculated on the basis of the design yield strength of the material as defined in Annex 22 of the Structural Code, taking into account also the coefficient of over-resistance (see sections **6.1.3(2)** and **6.2** of Annex 1).

(3) Annex 26 of the Structural Code is applied to determine the requirements and properties to be met by bolts and welding consumables.

(4) Non-dissipative joints of dissipative members, executed by means of full penetration butt welds, are considered to satisfy the over-strength criterion.

### 4.7.5 Stability

(1) The overall stability of the structure must be tested for the calculated seismic situation, taking into account, where relevant to this situation, the effect of pipe interaction and the effect of hydrodynamic loads.

(2) The overall stability may be considered as tested if the rules for testing of stability in Annexes 19, 22 and 25 of the Structural Code are satisfied for structures within the scope of the Structural Code. For structures outside this scope, the designer must justify the design in accordance with the specific regulations in force or, failing this, with the specific technical documents which, under their responsibility, he considers most appropriate.

(3) In structural steel elements, the use of class 4 sections is permitted, provided that all of the following conditions are met:

(a) the specific provisions of section **5.5** of Annex 22 of the Structural Code are satisfied;

(b) the value of the behavioural coefficient, $q$, is limited to 1.5 (see also the special provisions defined in Chapters **6** and **7** for structures with Class 4 sections); and

(c) the slenderness $\lambda$ is not greater than:

- 120 for main members,

- 180 for primary seismic bracing members,

- 250 in secondary seismic bracing elements,

NOTE See terms and definitions in section **1.5.2**.

### 4.7.6 Ductility and energy dissipation condition

(1) The structural elements and the structure as a whole must have sufficient ductility and energy dissipation capacity for the demands caused by the calculated seismic action. The value of the behavioural coefficient considered in the calculated shall be related to the ductility and energy dissipation capacity of the structure.

(2) The requirement specified in **(1)** is considered to be satisfied by using one of the following calculation methods:

(a) calculating the structure for dissipative behaviour, using a value of the behavioural coefficient greater than 1.5 and applying the special rules given in Chapters **5**, **6**, **7** and **8** for the energy dissipation capacity of the different types of structures dealt with in those chapters.

(b) calculation of the structure for non-dissipative (or weakly dissipative) behaviour, using a value of the behavioural coefficient not greater than 1.5 and applying that specified in section **2.1(4)**.

### 4.7.7 Foundations

(1) The provisions of section **2.2.2(4)** of Annex 1 apply.

(2) The calculated and testing of the foundation shall be carried out as specified in section **4.4.2.6** of Annex 1. Where the effect of the action obtained from the analysis for the calculated seismic action, $E_{F,E}$, using expression (4.30) of Annex 1, is the vertical force, $N_{Ed}$, due to the earthquake, the contribution of the vertical component of the seismic action for $N_{Ed}$ may be neglected if it causes push-off of the foundation.

### 4.7.8 Braces and accessories

(1) To determine the requirements and properties of cables, stranded cables, wires and accessories, the specific regulations in force shall be applied or, failing this, the specific technical documents which the author of the project, under their responsibility, considers most appropriate.

## 4.8 Thermal effects

(1) In accordance with Annexes 20, 23 and 31 of the Structural Code, account shall be taken of the

thermal effects associated with the normal operating temperature on the mechanical properties of the structural members, such as the modulus of elasticity and yield strength. Thermal effects due to temperatures of the structural members below 100°C may be neglected. For self-supporting steel chimneys see UNE-EN 13084-7.

## 4.9 Damage Limitation limit state

(1) The damage minimisation requirement sets limits applicable to displacements for damage limitation seismic action. Chapters **5**, **6**, **7** and **8** provide limits that depend on the type of structure.

(2) Reduced displacement limits may apply if the operation of the structure is sensitive to deformation (e.g. in telecommunication towers where deformation could lead to permanent damage to equipment or loss of signal).

(3) The displacements for the damage limitation requirement may be calculated as the displacements obtained in accordance with section **4.6(1)** for the calculated seismic action corresponding to the 'ultimate limit state requirement', multiplied by a reduction coefficient, $\upsilon$, which takes into account the shortest return period of the seismic action associated with the damage limitation requirement (see **4.4.3.1** of Annex 1).

(4) The value of the reduction factor, $\upsilon$, also depends on the importance class of the structure, where: $\upsilon = 0.4$ for importance classes III and IV; and $\upsilon = 0.5$ for importance classes I and II.

## 4.10 Behaviour coefficient

### 4.10.1 General considerations

(1) The value of the behavioural coefficient $q$ is to be calculated as:

$$q = q_0 k_r \geq 1,5 \tag{4.6}$$

Where

$q_0$ is the base value of the behavioural coefficient, reflecting the ductility of the system resisting lateral loads, the values of which are defined in chapters **5**, **6**, **7** and **8** for each different type of structure;

$k_r$ is the modification coefficient reflecting the output with respect to a regular distribution of mass, stiffness or resistance, the values of which are defined in section **4.10.2**.

### 4.10.2 Values of the modification coefficient $k_r$

(1) The value $k_r$ should be taken to be equal to 1.0, unless it is modified due to the existence of any of the following irregularities in the structure:

a) Horizontal eccentricity of the mass corresponding to a given horizontal level with respect to the centre of rigidity of the elements of that level, exceeding the parallel dimension of the structure by 5 %:

$$k_r = 0.8$$

b) Openings in a shaft or structural frame resulting in a reduction of 30 % or more of the moment of inertia of the transverse section:

$$k_r = 0.8$$

c) Mass concentrated in the upper third in height of the structure, which contributes 50 % or more to the overturning moment at the base:

$$k_r = 0.7$$

(2) Where more than one of the above irregularities are present, $k_r$ should be assumed to be equal to the product of 0.9 and the lowest value of $k_r$.

# 5 Specific rules for reinforced concrete chimneys

## 5.1 Field of application

(1) This chapter applies to concrete chimneys with a ring-shaped transverse cross-section (circular hollow section).

(2) Concrete chimneys calculated in accordance with this Annex must comply with the provisions of Annexes 19 and 20 of the Structural Code and also with the supplementary rules described in this chapter. For self-supporting concrete chimneys, the provisions of Standard UNE-EN 13084-2 that are complementary and not contradictory to the rules of the specific regulations in force shall also apply.

(3) The concrete must have a characteristic resistance, $f_{ck}$, of not less than 25 N/mm$^2$, as defined in Annex 19 of the Structural Code.

## 5.2 Calculations for dissipative behaviour

(1) Concrete chimneys can be calculated for dissipative behaviour with a basic value of the behavioural coefficient $q_0$ = 2.5, by applying the critical sections defined in **(2)** of the provisions of this section **5.2**.

(2) The critical region shall be taken as follows:

- from the base of the stack to a height $D$ above the base;

- from an abrupt change of cross-section to a height $D$ above the abrupt change of cross-section;

- a height $D$ above and below those sections of the chimney where more than one opening exists;

where $D$ is the outside diameter of the stack at the centre of the critical region.

(3) In the calculated calculation for dissipative behaviour, a minimum value of the local ductility coefficient in curvature, $\mu_\varphi$, shall be stipulated at the critical sections defined in **(2)**. This value of the

local ductility coefficient in curvature shall be ensured by placing confining reinforcement, in accordance with the provisions of **(4)** and section **5.4.3.2.2(10)** and **(11)** of Annex 1.

(4) The volumetric mechanical strength of the confining reinforcement, $\omega_{wd}$, defined in accordance with section **5.4.3.2.2(8)** of Annex 1, shall be associated with the coefficient of local ductility in curvature, $\mu_{\varphi}$,after spalling of the overlying concrete, by the general method based on:

a) the definition of the ductility coefficient in curvature from the curvatures for the ultimate limit state and for the yield stress, as $\mu_{\Phi} = \phi_u / \phi_y$;

b) the calculated $\phi_u$ as $\phi_u = \varepsilon_{cu2,c} / \chi_u$ and $\phi_y$ as $\phi_y = 1.5 f_y / (E_s D)$, where $D$ is the diameter as defined in **(2)**;

c) the depth of the neutral axis, $x_u$, estimated from the section equilibrium for the ultimate limit state conditions;

d) the stress-strain models defined in section **3.1.9** of Annex 19 to the Structural Code, and the ultimate strength and deflection of the confined concrete, $f_{ck,c}$ and $\varepsilon_{cu2,c}$, as a function of the effective lateral confining stress in accordance with section **3.1.9** of Annex 19 to the Structural Code; and

e) the expression of the effective lateral confining stress as $0,5\alpha w_{wd}$, taking the value of the confining efficiency coefficient, $\alpha$, from section **5.4.3.2.2(8)(b)** or **(c)** of Annex 1.

(5) The value of the curvature ductility coefficient, $\mu_{\Phi}$, to be considered in **(3)** and **(4)** can be determined from the displacement ductility coefficient, $\mu_{\delta}$, using the expression:

$$\mu_{\varphi} = \frac{\phi_u}{\phi_y} = 1 + \frac{\mu_{\delta} - 1}{4 \frac{L_{pl}}{L_V} \left( 1 - 0,5 \frac{L_{pl}}{L_V} \right)} \qquad (5.1)$$

where

$L_{pl}$ is the length of the plastic hinge;

$L_V = M_{Ed}/V_{Ed}$ is the shear stress span of the chimney in the lower section of the critical zone, calculated on the basis of the moment and shear stress determined in the analysis.

(6) The value of the displacement ductility coefficient, $\mu_{\delta}$, to be considered in expression (5.1) can be derived from the following relationship between $\mu_{\delta}$ and $q_0$:

$$\mu_{\delta} = q_0 \text{ if } T_1 \geq T_c \qquad (5.2)$$

$$\mu_{\delta} = 1 + (q_0 - 1)\, T_c / T_1 \text{ if } T_1 < T_c \qquad (5.3)$$

where $T_1$is the fundamental period of vibration of the stack, and $T_c$ is the period corresponding to the upper bound of the constant acceleration section of the spectrum, according to section **3.2.2.2(2)** of

Annex 1.

(7) The value of the length of the plastic hinge, $L_{pl}$, to be entered in the expression (5.1), can be taken as equal to:

$$L_{pl} = 0.5\ D \tag{5.4}$$

where $D$ is the outside diameter of the stack as defined in **(2)**.

(8) In order to avoid sudden spalling of the concrete of the inner surface of critical sections as defined in **(2)**, the value of the quotient of the outside diameter as defined in **(2)** and the wall thickness shall not exceed 20.

(9) Horizontal construction joints shall be avoided in the critical sections defined in **(2)**.

(10) In the critical regions defined in (2), the provisions of section **6.2.3** of Annex 2 apply.

## 5.3 Construction detail of the reinforcement

### 5.3.1 Minimum reinforcement (vertical and horizontal)

(1) In chimneys whose external diameter, $D$, is 4 m or more, the vertical and horizontal reinforcement shall be arranged in two layers (curtains) each: one layer per direction close to the inner face, and another layer, per direction, close to the outer face, with not less than half of the total vertical reinforcement placed in the layer closest to the outer face.

(2) In chimneys with an external diameter of 4 m or more, the minimum ratio of vertical reinforcement to transverse area shall not be less than 0.003.

(3) For chimneys with an external diameter of 4 m or more, the minimum ratio of horizontal reinforcement to the transverse area shall not be less than 0.0025. For self-supporting concrete chimneys, the corresponding rule of Standard UNE-EN 13084-2 also applies.

(4) In chimneys with an external diameter of less than 4 m, all the vertical or horizontal reinforcement may be placed in a single layer (curtain) per direction, in the vicinity of the external face. In this case, the ratio between the reinforcement of the outermost layer and the gross transverse section area shall not be less than 0.002, for each direction.

(5) In the vicinity of the top of the chimney, where the stresses due to permanent loads are small, the minimum amount of vertical reinforcement may be taken to be equal to that of the horizontal reinforcement.

(6) The spacing of the vertical bars shall not exceed 250 mm, and that of the horizontal bars shall not exceed 200 mm.

(7) Horizontal reinforcement bars shall be placed between the vertical bars and the surface of the concrete. Clevises shall be placed between the inner and outer layers of reinforcement at horizontal and vertical intervals of not more than 600 mm.

### 5.3.2 Minimum reinforcement around openings

(1) Complementary reinforcement shall be provided around the perimeter and corners of openings in addition to the reinforcement provided at a certain distance from the openings. The supplementary reinforcement shall include diagonal bars as well as vertical and horizontal bars at the corners; In addition, they shall be placed as close to the outer face of the opening as ordinary construction conditions permit. The bars shall extend beyond the edge of the opening for a full anchorage length.

(2) The area of the complementary horizontal and vertical reinforcement in each direction shall not be less than that of the bars interrupted by the presence of the opening. The amount of vertical reinforcement shall not be less than 0.0075 along a horizontal distance from any of the vertical edges of the opening equal to half the width of the opening.

## 5.4 Special rules for analysis and calculation

(1) It is only necessary to take into account a horizontal component of the terrain movement, except for the case specified in **(2)**.

(2) It is necessary to take into account both horizontal components of ground motion in the case of chimneys with openings of horizontal dimensions greater than the thickness of the chimney wall, in the critical zones defined in section **5.2(2)**.

(3) The vertical component of the terrain movement may be neglected.

(4) Where the liner (composed of brick, steel, or other materials) is laterally supported by the structural shaft of the chimney at close points, so that the relative movement of the liner relative to the shaft is considered negligible, the mass of the liner may be incorporated into the mass of the structural shaft, without including separate degrees of freedom for the liner.

(5) Where the liner supports at the top of the chimney, and possibly at intermediate points, allow movement of the inner liner relative to the structural shaft, the inner liner shall be included in the dynamic analysis model separately from the concrete shaft. In this case, if, according to section **3.3(2)** and section **4.2.4**, the analysis is carried out using the elastic response spectrum, the value of the damping ratio or index to be used for the lining shall depend on its construction.

NOTE Appendix B proposes damping ratio values for typical materials used in the inner lining.

## 5.5 Damage Limitation limit state

(1) The flue gas discharge ducts of chimneys shall be tested against the imposed deformations between the support points and against the gaps between the inner elements, so that under the action of the displacements calculated in accordance with section **4.9(3)** the gas tightness is not lost, and a sufficient safety margin against rupture of the gas duct is maintained.

(2) The damage minimisation requirement is considered to be satisfied if the lateral displacement of the crown of the structure, calculated in accordance with section **4.9(3)**, does not exceed 0.5 % of the height of the structure.

(3) For the minimisation of damage to the inner skin, the relative deflection between the different bearing points of the inner skin, calculated in accordance with section **4.9(3)**, shall be dimensioned.

Unless stricter limits are specified for a particular project, the following limits shall be observed in relation to the relative lateral displacements of adjacent bearing points of the inner lining:

a) if arrangements are made to allow relative movement between separate parts of the inner lining, (e.g. by the construction of an inner lining consisting of tubes independent of each other, with appropriate spacing):

$$d_r \leq 0.020\ \Delta S \tag{5.5}$$

b) in all other cases:

$$d_r \leq 0.020\ \Delta S \tag{5.6}$$

where $\Delta H$ is the vertical distance between adjacent platforms supporting the inner liner.

# 6 Specific rules for metal chimneys

## 6.1 Calculations for dissipative behaviour

(1) Steel portal frames or triangulated structures providing lateral support for flues in chimneys may be calculated for dissipative behaviour in accordance with the corresponding rules of Chapter **6** of Annex 1. In that case, their calculated calculation shall be based on values of the basic behavioural coefficient, $q_0$, not exceeding the following:

(a) bending resistance portal frames or frames with off-centre bracing $q_0 = 5$;

(b) frames with centred bracing: $q_0$ taken from Figure 7.1.

(2) Steel chimneys consisting of a structural shaft calculated for dissipative behaviour shall satisfy the requirements of sections **5.4.3** and **5.6** of Annex 22 of the Structural Code for the global plastic analysis. In this case, their calculated value may be based on a value of the basic behavioural coefficient:

$$q_0 = 2.5$$

(3) Depending on the chosen transverse sections, the basic value of the behavioural coefficient is limited by the values given in Table 6.1.

NOTE Steel cable-stayed chimneys are generally lightweight. Therefore, their calculated lateral action is usually governed by the wind, unless they have large flare openings or other masses near the crown.

**Table 6.1 – Restrictions on the basic value of behavioural coefficient depending on the cross-section class of the steel elements**

| Basic values of the behavioural coefficient $q_0$ | Permissible transverse cross-section classes |
|---|---|
| $q_0 \leq 1.5$ | Classes 1, 2, 3 or 4 (according to section **4.7.5(3)**) |
| $1.5 < q_0 \leq 2$ | Classes 1, 2 or 3 |
| $2 < q_0 \leq 4$ | Classes 1 or 2 |

| | |
|---|---|
| $q_0 > 4$ | Class 1 |

## 6.2 Materials

### 6.2.1 General considerations

(1) The structural steel must comply with the Structural Code and the specific regulations in force or, failing this, with the specific technical documents that the author of the project, under their responsibility, considers most appropriate.

(2) The structural steel must comply with the provisions of section **3.2** of Annex 22 of the Structural Code.

(3) The thickness of the steel elements must comply with the requirements of Table 2.1 of Annex 28 of the Structural Code, depending on the energy of the Charpy V-notch test (CVN) and other relevant parameters, as well as with the requirements of the specific regulations in force or, failing this, with the specific technical documents that the designer, under their responsibility, considers more appropriate.

(4) When stainless steel or alloy steel components are joined to carbon steel components, bolted connections are preferable. In order to avoid accelerated corrosion due to galvanic action, such joints shall include insulating sealing gaskets. Welding is permitted, provided that special metallurgical control is exercised over the welding process and the selection of electrodes.

### 6.2.2 Mechanical properties of carbon structural steels

(1) The mechanical properties of S 235, S 275, S 355, S 420 and S 460 carbon structural steels shall be as defined in Annex 22 of the Structural Code, and, for properties at higher temperatures, in UNE-EN 13084-7.

### 6.2.3 Mechanical properties of stainless steels

(1) The mechanical properties for stainless steels shall be as specified in Annex 24 of the Structural Code for temperatures up to 400°C and, for higher temperatures, in UNE-EN 13084-7.

### 6.2.4 Joints

(1) For joint materials, welding consumables, etc., the provisions of Annex 26 of the Structural Code and the relevant product requirements specified in Article 85 of the Structural Code shall apply.

## 6.3 Damage Limitation limit state

(1) The provisions of section **5.5(1)** apply.

(2) The provisions of section **5.5(2)** apply.

## 6.4 Ultimate limit state

(1) Calculation in accordance with this standard, including the values of the specified performance factors for dissipative or non-dissipative behaviour, ensures that low cycle fatigue of structural details (especially joints) will not contribute to the ultimate limit state.

(2) In the calculated calculation of constructional details, such as flanges, the plastic stress distribution must be taken into account.

(3) When testing a chimney for the calculated seismic situation, a corrosion allowance shall be made, unless special measures are taken for corrosion protection as specified in Annex 22 of the Structural Code. This over-thickness shall be adopted, where appropriate, in accordance with the specific regulations in force or, failing this, with the specific technical documents which the designer, under their responsibility, considers most appropriate.

(4) Any weakening of a transverse section due to notches or openings (manholes, emission inlets) must be compensated by local reinforcement of the structural shaft (e.g. through stiffeners around the edges of the openings), also taking into account local stability considerations.

# 7 Specific rules for steel towers

## 7.1 Field of application

(1) Steel towers designed in accordance with this Annex must comply with the relevant parts of the Structural Code (Annexes 22 to 29) and with the specific regulations in force or, failing this, with the specific technical documents which the author of the project, under their responsibility, considers most appropriate, as well as with the additional rules specified in this chapter.

## 7.2 Calculations for dissipative behaviour

(1) The calculated calculation of steel towers for dissipative behaviour shall be in accordance with the relevant provisions of Chapter **6** of Annex 1. In that case, its calculated calculation shall be based on values of the basic behavioural coefficient, $q_0$, not exceeding the following:

(a) bending-resistant frames or frames with off-centre bracing: $q_0 = 5$;

(b) frames with concentric bracing: $q_0$ obtained from Figure 7.1.

(2) The specifications of section **6.1(3)** apply.

(3) If the main diagonals of the triangulated tower are tubular, the basic value of the behavioural coefficient shall be limited to 2.

## 7.3 Materials

(1) The structural steel must comply with the provisions of Annex 22 of the Structural Code. In addition, it must also comply with the specific regulations in force or, failing that, with the specific technical documents that the author of the project, under their responsibility, considers most

appropriate.

(2) The provisions of section **6.2.1(2)** apply.

(3) The provisions of section **6.2.1(3)** apply.

(4) The requirements specified in section **6.2** of Annex 1 apply.

(5) The thickness of cold-formed tower elements shall be at least 3 mm.

NOTE Steel towers are sometimes designed to remain in service without maintenance for 30 years or 40 years or even longer. A weathering steel cannot then be used unless corrosion protection, such as hot-dip galvanisation, is applied.

## 7.4 Calculation of concentrically bracketed towers

(1) Figure 7.1 indicates the values of $q_0$ to be used for the calculated typical configurations of concentrically braced steel towers for dissipative behaviour.

(2) For the frames in Figures 7.1 (a) to (e) and (h), both tensile and compressive diagonals must be taken into account in an elastic analysis of the structure under seismic action.

(3) The frames in Figures 7.1 (a) to (c) belong to the 'K' bracing types and are not allowed for dissipative behaviour. The value of $q$ for such frames is limited to 1.5.

(4) The frames in Figures 7.1 (d) and (h) can be assimilated to V-braced frames with diagonals intersecting at the level of a continuous horizontal element. The calculated dissipative behaviour shall be in accordance with the provisions given in section **6.7** of Annex 1 applicable to V-braced frames.

(5) For the portal frame in Figure 7.1 (e), the calculated dissipative behaviour shall be in accordance with the rules in Section **6.7** of Annex 1 applicable to frames with diagonal bracing where the diagonals are not configured as X-bracing.

(6) The X-braced frames in Figures 7.1 (f) and (g) may be considered as frames with diagonal X-bracing. In the calculated dissipative behaviour, only the tensile diagonals shall be taken into account in an elastic analysis of the structure against seismic action. This calculation shall be in accordance with the rules given in Section **6.7** of Annex 1 applicable to frames with X-braced diagonal bracing.

(7) If the basic value of the behavioural coefficient used in the calculated design is equal to or greater than 3.5, horizontal bracing with full triangulation, such as that shown in Figure 7.2, shall be provided.

## 7.5 Special rules for the calculated calculation of electricity transmission towers

(1) The calculated calculation shall take into account the unfavourable effects of cables between adjacent towers on the tower to be designed.

(2) The requirements of **(1)** may be satisfied if the effects of seismic action on the tower structure are calculated by a simple sum of the following summands (no combination rules such as square root of the sum of squares, or similar, shall be used):

- The effects of seismic action due to the forces exerted by the cables on the tower, assuming that the tower is statically displaced relative to adjacent towers in the most unfavourable direction. The assumed relative displacement shall be equal to twice the calculated displacement of the terrain as specified in section **3.2.2.4** of Annex 1. The set of all physically possible relative displacements between towers shall be analysed considering the assumption that the towers are embedded at their base;

- The effects of seismic action due to inertia forces resulting from a dynamic analysis as specified in section **4.2.1(2)**. If a three-tower model is used, a simplified assumption may be adopted for the two adjacent towers if they are tangent towers. In this case, the inertia forces can be calculated assuming that the adjacent tower has an elastic support at the height of the cables in the direction of the cables.

## 7.6 Damage Limitation limit state

(1) For the damage limitation limit state, depending on the tower function and for each particular project, displacement limits, calculated according to section **4.9(3)**, shall be specified.

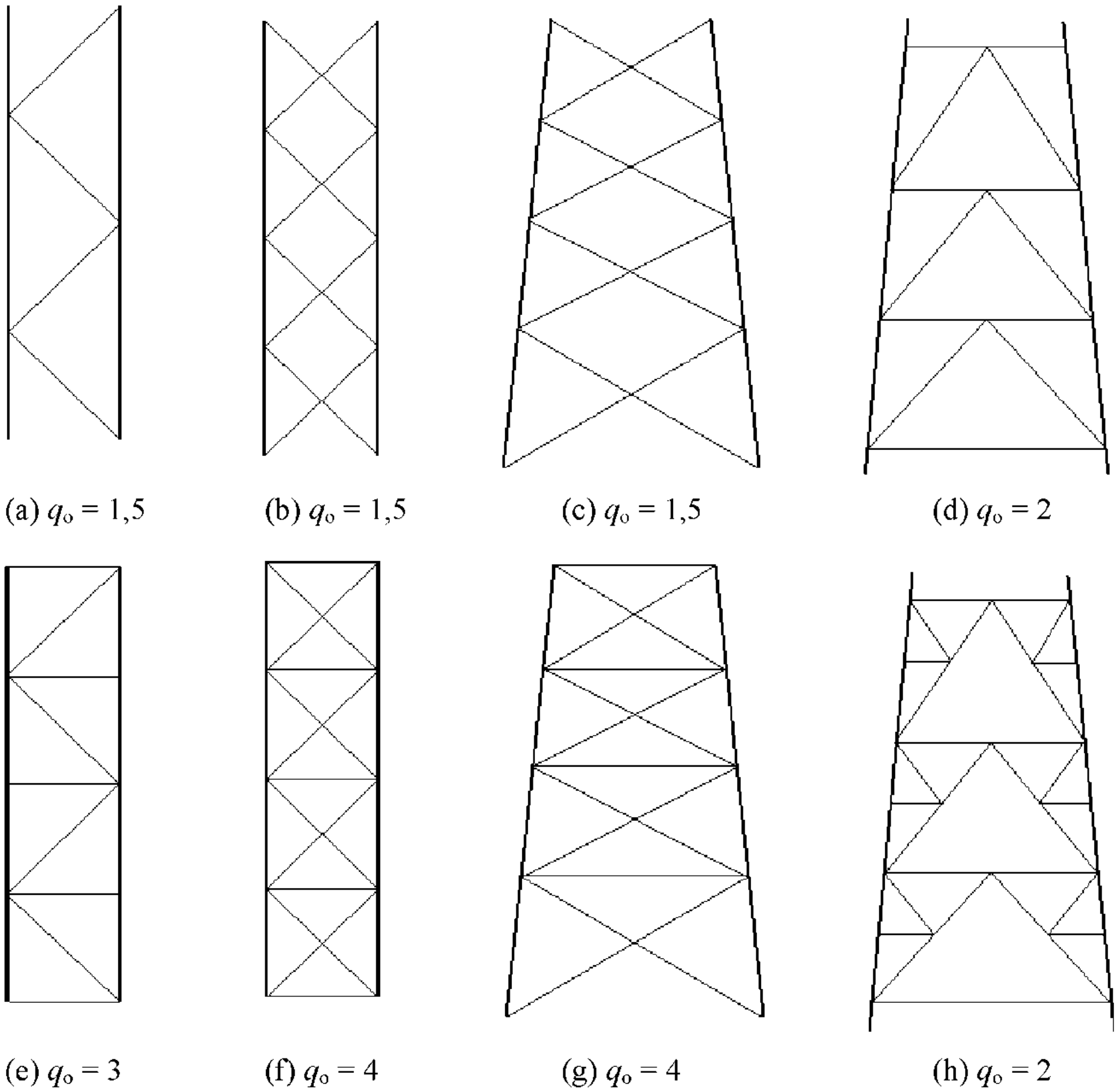



**Figure 7.1 - Basic values of the behavioural coefficient for steel portal frame configurations with bracing**

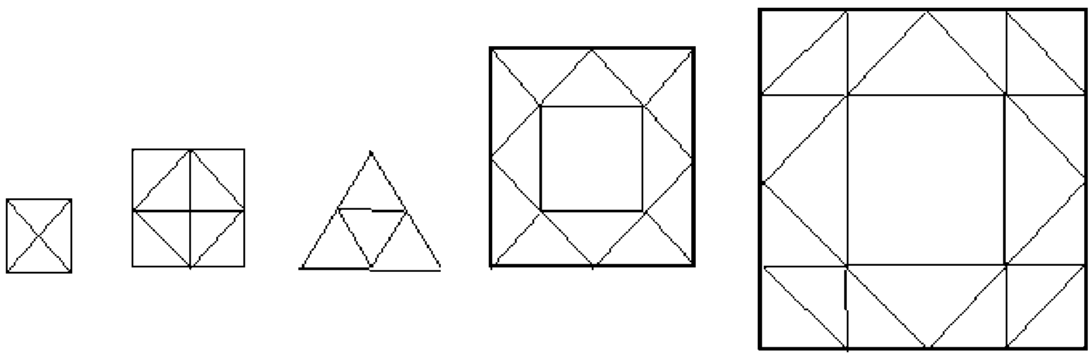


**Figure 7.2 - Examples of horizontal bracings with full triangulation, to be used in towers with $q_0$ ≥ 3.5**

### 7.7 Other special calculation rules

(1) 'Telescopic joints' may only be used in tubular steel towers if they are experimentally qualified.

(2) The anchorage to the foundation shall be designed at the base of the columns to resist a tensile force equal to the greater of the following two forces, if these are tensile:

(a) the force calculated in accordance with the provisions of section **4.2.1(2)**;

(b) the force calculated from the analysis for the calculated seismic situation, using a value of the behavioural coefficient not greater than $q = 2$.

(3) Tower connections shall be calculated and constructively detailed to meet the relevant requirements of Annex 1, Chapter **6**, for connections of structural systems of similar type and configuration, calculated for the same basic value of the behavioural coefficient, $q_0$, as the tower.

## 8 Specific rules for stayed masts

### 8.1 Field of application

(1) This chapter relates to steel masts.

(2) Steel masts calculated in accordance with this Annex must comply with the relevant parts of the Structural Code (Annexes 22 to 29). In addition, they must comply with the provisions of the specific regulations in force or, failing that, with the specific technical documents that the author of the project, under their responsibility, considers most appropriate, as well as with the complementary rules specified in this chapter.

### 8.2 Special requirements for analysis and calculation

(1) Calculation for dissipative behaviour is not permitted for stayed masts. They shall be calculated for low dissipative behaviour with $q = 1.5$.

(2) The tension in the stay cables due to the calculated seismic action must be lower than the pre-existing tension in the cables.

(3) The elastic restraint induced in the mast by the stayed mast guy wires shall be taken into account as follows:

- on relatively short masts (up to 30 m or 40 m) the guy wires can be considered to act as simple tensile stay cables, the rigidity of which remains constant when the mast is deflected;
- on higher towers, the deflection of the stay cables is large and should be taken into account by considering a rigidity for the cables that depends on the deflections, as specified in sections **4.2.3(2)** and **(3)**.

(4) The deflection of the stay cables due to the ice load considered in the calculated seismic situation shall be taken into account.

(5) For both stay cables with significant deflections and straight cables, the horizontal component of the cable stiffness shall be taken to be equal to:

$$K_{\mathrm{eff,h}} \cos^2 \alpha \frac{A_{\mathrm{c}} E_{\mathrm{eq}}}{l} \tag{8.1}$$

where

$A_{\mathrm{c}}$ is the transverse area of the cable-stayed cable transverse section;

$E_{\mathrm{eq}}$ is the effective modulus of elasticity of the stay cable (considering the deflection as specified in sections **4.2.3(3)** and **(4)**, if required, in accordance with **(3)** and **(4)**;

$l$ is the length of the cable;

$\alpha$ is the angle that the stay cable makes with the horizontal.

(6) If both the sag and the mass of the stay cable are significant, the possibility of an impulsive action of the cable on the mast in the calculated seismic situation shall be considered.

## 8.3 Materials

(1) The provisions of section **7.3(1)** apply.

(2) The provisions of section **6.2.1(2)** apply.

(3) The provisions of section **6.2.1(3)** apply.

(4) The requirements of section **6.2** of Annex 1 apply.

## 8.4 Damage Limitation limit state

(1) The specifications of section **5.5(2)** apply.

(2) For the damage limitation condition, depending on the function of the mast and for each particular design, a limit for the relative displacements between horizontal stiffening elements, calculated in accordance with section **4.9(3)**, shall be specified.

## Appendix A

## Recommendations for linear dynamic analysis taking into account the rotational components of the terrain motion

(1) When taking into account the rotational components of the ground motion observed during the earthquake, the seismic action can be represented by three elastic response spectra for the translational components and three elastic response spectra for the rotational components.

(2) The elastic response spectra for the two horizontal translational components (*x* and *y* axes) and for the vertical component (*z* axis) are those described in sections **3.2.2.2** and **3.2.2.3** of Annex 1.

(3) The rotation response spectrum is defined analogously to the response spectrum of the translational components, i.e. considering the maximum response of the rotational motion of an oscillator of a single rotational degree of freedom, of natural period *T* and critical damping ratio $\xi$.

(4) $R^{\theta}$ represents the ratio of the maximum moment on the oscillator spring to the rotational moment of inertia about its axis of rotation. For given values of $\xi$, the plot of the values of $R^{\theta}$ with respect to the natural period, *T*, is the rotational response spectrum.

(5) In the absence of specific research results or well-documented field measurements, the rotational response spectra can be determined as follows:

$$R_x^{\theta}(T) = 1,7\,\pi S_e(T)/\,v_s T \tag{A.1}$$

$$R_y^{\theta}(T) = 1,7\,\pi S_e(T)/\,v_s T \tag{A.2}$$

$$R_z^{\theta}(T) = 2,0\,\pi S_e(T)/\,v_s T \tag{A.3}$$

where

$R_x^{\theta}$, $R_y^{\theta}$, $R_z^{\theta}$ are the rotational response spectra around the *x*, *y* and *z* axes, in rad/s$^2$;

$S_e\,(T)$ is the elastic response spectrum for the horizontal components at the location, in m/s$^2$;

$T$ is the period, in seconds;

$v_s$ is the average S-wave velocity, in m/s, corresponding to the upper 30 m of the soil profile. The value corresponding to low amplitude vibrations, i.e. shear strains of the order of $10^{-6}$, may be used.

(6) The value of $v_s$ is obtained directly from field measurements, or by determining by laboratory tests the shear modulus of elasticity, $G$, for small deformations, and the density of the soil $\rho$, and applying the expression (3.1) inverted from section **3.2(1)** of Annex 5:

$$v_s = \sqrt{G / \rho}$$

(7) In cases where $v_s$ is not assessed by experimental measurements in accordance with **(6)**, the value in Table A.1 may be used, in so far as it is representative of the soil type at the site:

**Table A.1 - Default shear wave velocity values for the five standard soil types for the five standard soil types**

| Soil type | Shear wave velocity, $v_s$ m/s |
|---|---|
| A | 800 |
| B | 580 |
| C | 270 |
| D | 150 |

(8) When considering a translational acceleration of the terrain $\ddot{x}(t)$ in the horizontal x-direction associated with a rotational acceleration $\ddot{\theta}(t)$ in the vertical x-z plane, then, if $[M]$ is the inertia matrix, $[K]$ is the rigidity matrix, and $[C]$ is the damping matrix, the equations of motion for the resulting multi-degree-of-freedom system are given by:

$$[M]\{\ddot{u}\} + [C]\{\dot{u}\} + [K]\{u\} = -\left(\{m\}\ddot{x} + \{m\,h\}\ddot{\theta}\right) \tag{A.4}$$

where

$\{\ddot{u}\}$ is the vector whose components are the accelerations relative to the base, of the degrees of freedom of the structure;

$\{\dot{u}\}$ is the vector whose components are the velocities of the degrees of freedom of the structure;

$\{u\}$ is the vector whose components are the displacements relative to the base of the degrees of freedom;

$\{m\}$ is the vector whose components are the translational masses, in the horizontal direction of the translational excitation. This vector coincides with the main diagonal of the mass matrix $[M]$, if the vector $\{u\}$ includes only the translations in the horizontal direction of the excitation;

$\ddot{x}(t)$ is the translational acceleration of the terrain, represented by $S_e$;

$\ddot{\theta}(t)$ is the rotational acceleration of the base, represented by $R^{\theta}$.

(9) To account for the $\{m\}$ term, the $k$-mode participation factor in the modal analysis is:

$$a_{\mathrm{ku}} = \frac{\{\Phi\}^{\mathrm{T}}\{m\}}{\{\Phi\}^{\mathrm{T}}[M]\{\Phi\}} \tag{A.5}$$

while, for the $\{mh\}\ddot{\theta}$ term, the participation factor is:

$$a_{\mathrm{k}\theta} = \frac{\left\{(\Phi h)^{T}\right\}\{m\}}{\{\Phi\}^{\mathrm{T}}[M]\{\Phi\}} \tag{A.6}$$

where

$\{\Phi\}$ is the k-th modal vector;

$\{\Phi h\}$ is the vector whose components are the products of the modal amplitude $\Phi_{\mathrm{i}}$ for the i-th degree of freedom by its elevation $h_i$.

(10) Normally, the effects of the two stresses should be superimposed. They are generally not in phase and, consequently, the effects of the rotational excitation of the terrain can be combined with those of the translational excitation by means of the square root of the sum of squares rule (SRSS).

## Appendix B

## Recommendations for the consideration of modal damping in modal analysis using response spectra

(1) When the calculated response spectrum is applied, the value of the behavioural coefficient $q$ incorporates the energy dissipation in the elastic zone of the structural response, the energy dissipation due to the soil-structure interaction, and the energy dissipation due to the hysteretic behaviour of the structure. When using the elastic spectrum in the analysis, the damping factor (relative to the critical) needs to be explicitly defined. When a modal analysis is carried out, the damping factors need to be defined for each mode of vibration. If a given mode involves essentially a single material of the structure, the damping factor should be in accordance with the dissipation characteristics of the material and be consistent with the amplitude of the deformation.

(2) For most common structural materials, the damping values given in section **4.1.3** of Annex 2 can be used.

(3) If non-structural elements are considered to contribute to energy dissipation, higher values of damping may be considered. Because of its dependence on the strain amplitude, damping factor values between the lower limits are generally appropriate for damage limiting seismic action, whereas values between the higher limits are appropriate for calculated seismic action. These limits can be taken as:

- for a ceramic covering: 0.015 – 0.05;

- for an internal brickwork lining: 0.03 – 0.10;

- for a steel internal lining: 0.01 – 0.04;

- for fibre-reinforced polymer internal lining: 0.015– 0.03.

(4) The representative ranges of the damping ratio for energy dissipation in a terrain modelled with dampers are as follows:

- for the horizontal degree of freedom (associated with the back-and-forth motion of the ground): 0.10 – 0.20;

- for the rotational degree of freedom (associated with the rolling motion of the ground): 0.07 – 0.15;

- for the vertical degree of freedom (associated with the vertical translation of the floor): 0.15– 0.20.

(5) Low damping factors should be assigned to dampers associated with foundations built on a shallow sedimentary soil tank underlain by a rocky substrate or similar rigidity.

(6) In general, for the type of structures discussed in this Annex, any mode of vibration involves the deformation of more than one material. In this case, for each mode, it is appropriate to consider an average modal damping based on the elastic strain energy stored in that mode.

(7) The formulation leads to:

$$\overline{\xi}_j = \frac{\{\Phi\}^T \left[\overline{K}\right] \{\Phi\}}{\{\Phi\}^T [K] \{\Phi\}} \tag{B.1}$$

where

$\overline{\xi}_j$ is the equivalent modal damping factor corresponding to the *j*th mode;

$[K]$ is the rigidity matrix;

$\left[\overline{K}\right]$ is the modified stiffness matrix, with terms equal to the product of the corresponding term of the stiffness matrix, $[K]$, times the damping factor for that element;

$\{\Phi\}$ is the *j*th modal vector.

(8) Other techniques may also be used if more detailed data on the damping characteristics of structural subsystems are available.

(9) It is recommended that the value $\overline{\xi}_j$ should not exceed 0.15, unless experimentally justified.

## Appendix C

## Recommendations for consideration of soil-structure interaction

(1) This appendix contains complementary information to Appendix D of Annex 5.

(2) The design seismic motion is defined at the surface of the terrain under free-field conditions, i.e. unaffected by inertial forces due to the presence of structures. When the structure is founded on soil tanks or soft terrain, the resulting movement at the base of the structure will differ from that at the same elevation in free field, due to the deformability of the soil. For tall structures, ground sway can be important and can significantly increase second order effects.

(3) Modelling methods for soil-structure interaction should take into account:

(a) the degree of soil embedment;

(b) the depth of the possible rock layer;

(c) the stratification of the soil layers;

(d) the variability of soil properties in each stratigraphic layer; and

(e) the dependence of the soil properties (shear modulus and damping) on the level of deformation.

(4) Generally, the horizontal stratification assumption can be considered applicable.

(5) Unless the soil investigation suggests an adequate range of variability for the dynamic soil modulus, an upper bound for the soil rigidity can be obtained by multiplying the set of best estimate values of the moduli by 2, and the lower bound by multiplying this set by 0.5.

(6) Being strain dependent, the damping and shear modulus of each soil layer should be consistent with the intensity of the effective shear deformation expected during the seismic action under consideration. An equivalent linear method is acceptable. In this case, the analysis should be carried out iteratively. The analysis is linear in each iteration, but the soil properties are adjusted from iteration to iteration, until the calculated deformations are compatible with the soil properties considered in the analysis. The iterative procedure can be developed for the free-field soil tank, suppressing the presence of the structure.

(7) The amplitudes of the effective shear deformation of any layer, to be considered for the evaluation of dynamic moduli and damping in the equivalent linear methods, can be taken as:

$$\gamma_{eff} = 0,65\ \gamma_{máx,t} \tag{C.1}$$

where $\gamma_{max,t}$ is the maximum value of the shear deformation in the free-field soil layer during the considered seismic action.

(8) If the soil is modelled by the finite element method, the criteria for determining the location of the lower and lateral boundary of the modelled zone should be justified. In general, excitation functions simulating seismic motion are applied at these boundaries. In such cases, it is required to generate an excitation system acting on these boundaries, such that the response motion at the ground surface, in free field, is identical to the terrain motion due to the considered seismic action. The methods and theories followed for the generation of such excitation systems should be presented.

(9) If the half-space modelling method (concentrated parameters) is used, the parameters used in the analysis to consider ground deformability should not take into account stratification. The variability of soil properties and deformation-dependent properties should also be taken into account.

(10) Any other modelling methods used for soil-structure interaction analysis should be clearly explained.

(11) The decision not to take into account soil-structure interaction in the analysis should be justified.

## Appendix D

## Recommendations for the consideration of the number of degrees of freedom and modes of vibration

(1) A dynamic analysis (e.g. a time-dependent or response spectrum analysis method) is used when it is considered that the use of the lateral force method is not justified.

(2) The analysis should:

- take into account the rocking and translation response of the foundation;

- include a sufficient number of masses and degrees of freedom to determine the response of any structural elements and equipment;

- include a sufficient number of modes of oscillation to ensure the participation of all significant modes;

- provide for the maximum relative displacement between equipment or machine supports (for a chimney, the interaction between the inner and outer tubes);

- take into account significant effects such as interactions between pipes, external constraints applied to the structure, hydrodynamic loads (both mass and rigidity effects), as well as possible non-linear behaviour;

- provide 'storey or slab response spectra', where the structure supports important light-weight equipment or appurtenances.

(3) The effective modal mass, $M_i$, for mode *i*, referred to in section **4.3.3.2(2)**, is defined as

$$M_i = \left[ \{\varphi\}^T [M] \{i\} \right]^2 / \{\varphi\}^T [M] \{\varphi\} \qquad \text{(D.1)}$$

where

$\{\varphi\}$ is the *i*-th modal vector;

$\{i\}$ is a column vector whose terms are equal to 0 or 1, which represents the displacement induced in the associated degree of freedom when its base is subjected to a unit displacement in the direction of the component of the seismic action considered.

(4) The criterion given in section **4.3.3.2(2)** does not ensure adequate discretisation of the mass, if it is a lightweight equipment or a structural annex. In that case, the above condition could be fulfilled, but the mathematical model of the structure may instead be inadequate to describe the response of the equipment or attachment. Where analysis of the equipment or attachment is required, a 'floor or slab response spectrum' should be defined, applicable to the elevation of the slab on which the equipment/attachment rests. This method is also recommended when a part of the structure needs to be analysed independently, e.g. an interior lining of a chimney constructed of masonry, attached by insulated supports to the shaft of the chimney.

# Appendix E

## Recommendations for masonry chimneys

### E.1 Introduction

(1) A masonry chimney is a chimney constructed of masonry or masonry blocks and mortar, hereafter referred to as ‘masonry’. Masonry chimneys should be constructed, anchored, supported and reinforced as indicated in this appendix.

### E.2 Footings and foundations

(1) Foundations for masonry chimneys should be constructed of concrete or solid masonry at least 300 mm thick, and should extend on all sides at least 150 mm beyond the chimney face or supporting wall. Footings should be founded on undisturbed natural terrain or compacted backfill at a level below the frost heave at the site. In areas not subject to frost, footings should be at least 300 mm below the surface of the terrain.

### E.3 Behaviour coefficient

(1) The behavioural coefficient $q$ should be taken as 1.5, corresponding to low dissipative behaviour.

### E.4 Minimum vertical reinforcement

(1) In chimneys up to 1 m horizontal dimension, a total of four continuous vertical bars of 12 mm diameter should be placed in the foundation, either in the concrete between the masonry blocks or inside the cavities of the hollow masonry blocks filled with mortar grout. Adhesion of the grout to the inner lining of the inner chimney flue should be avoided in order to prevent restriction of its thermal expansion. In chimneys with a horizontal dimension greater than 1 m, two additional continuous vertical bars of 12 mm diameter should be added for each additional metre of horizontal dimension, or fraction thereof.

### E.5 Minimum horizontal reinforcement

(1) Vertical reinforcement should be surrounded by 6 mm diameter chain links or other reinforcement of equivalent transverse section, spaced not more than 400 mm apart.

## E.6 Minimum seismic anchorage

(1) A masonry chimney through the floors and roof of a building should be anchored to each roof and floor slab more than 2 m above the terrain, except when constructed entirely on the inside of the exterior walls of the building. Two 5 mm by 25 mm steel flanges, with a minimum length of 300 mm, should be embedded in the chimney. The flanges should be anchored by hooks placed around the outer bars and should extend 150 mm beyond the bend of the pins. Each flange should be fixed to a minimum number of four floor joists by two 12 mm bolts.

## E.7 Cantilever

(1) A masonry chimney should not be designed so that, in plan, it protrudes from a wall or foundation by more than half the thickness of the chimney wall. A masonry chimney should not be designed so that it protrudes in plan from a wall or foundation that is less than 300 mm thick, unless the projection is the same on each side of the wall. By way of exception, on the second storey of a two-storey building, the chimney stack projection outside the exterior wall may be equal to the thickness of the wall. The height in plan of a course of brickwork should not exceed the lesser of half the height of the brickwork or one third of the depth of its bedding.

## E.8 Dimensional variations

(1) The wall of a chimney, or the inner flue of a chimney, should not change in shape or size for a length of 150 mm above or below the level at which the chimney passes through a slab, the roof, or its components.

## E.9 Offset

(1) Where a masonry chimney is constructed with a fireclay lining surrounded by a sheet of masonry, the maximum offset should be such that the axis of the flue above the offset does not lie beyond the centre of the chimney wall below the offset. The limits of the maximum offset do not apply where the chimney is designed so that its offset is supported by the masonry below it.

## E.10 Additional vertical loads

(1) Chimneys should not support vertical loads in addition to their own weight unless designed to do so. Masonry chimneys may be constructed as an integral part of the masonry or concrete walls of the building.

## E.11 Wall thickness

(1) The walls of a masonry chimney should be constructed of solid masonry, or hollow masonry completely filled with mortar grout, with a minimum nominal thickness of 100 mm.

## Appendix F

## Recommendations for electricity transmission towers

(1) Wind loads, often combined with ice loads or asymmetric longitudinal cable actions, normally control the calculated design of power transmission and distribution structures as well as substation conductor cable supports. The calculated seismic situation does not generally control the calculated design of these structures, except when the calculated design includes important ice loads. The behaviour of these structures in earthquakes has shown that seismic loads can be resisted by designing them for the traditional actions of electrical transmission and distribution, substation cable support structures. Heavy equipment, such as transformers in distribution structures, can give rise to significant seismic actions and problems.

(2) Damage caused by earthquakes to power transmission and distribution structures or substation conductor supports is often due to large displacements of foundations as a result of landslides, terrain failure or liquefaction. These phenomena usually involve local structural damage or failure, without total loss of function and integrity of the structure.

(3) The fundamental frequency of these types of structure typically ranges from 0.5 Hz to 6 Hz. The fundamental mode frequencies of monopod structures are in the range of 0.5 Hz to 1.5 Hz. The fundamental mode of H-frame structures has frequencies in the range of 1 Hz to 3 Hz, with the lowest frequencies in the direction normal to the plane of the structure, and with the highest frequencies in the plane of the structure. The fundamental mode of four-legged structures has frequencies in the range 2 Hz to 6 Hz. Typical tangent towers normally have lower frequencies within this range; angle towers and dead-end or anchor towers have the highest frequencies within the range. These frequency ranges can be used to determine whether the actions caused by the earthquake can be expected to control the structural calculation of the tower. If so, a more detailed evaluation of the vibration frequencies and modal shapes of the structure should then be carried out.