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English version

Eurocode 8: Design provisions for earthquake resistance of structures

Part 6: Towers, masts and chimneys

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Contents

FOREWORD

(1) For the design of structures in seismic regions the provisions of this Prestandard are to be applied in addition to the provisions of the other parts of Eurocode 8 and the other relevant Eurocodes. In particular, the provisions of the present Prestandard complement those of Eurocode 3, Part 3-1 " Towers and Masts ", and Part 3-2 " Chimneys", which do not cover the special requirements of seismic design.

NATIONAL ANNEX FOR EN 1998-6

Notes indicate where national choices have to be made. The National Standard implementing EN 1998-6 shall have a National annex containing all Nationally Determined Parameters to be used for the design in the country. National choice is required in the following sections.

Reference section	Item
2.1	Rules for low seismicity region. Value of the soil peak acceleration for a site being in this category.
4	Importance factors for musts towers and chimneys.
4.11	Temperature of structural elements above which the thermal effect on the mechanical properties shall be accounted for.
4.14	Values of the reduction factor ν that takes into account the shorter return period of the seismic action associated with the damage limitation requirement.
4.7.2.1	Height of the structure below which simplified dynamic analysis is allowed.
7.7	Behaviour factors for towers made of trussed tubes.
8.3	Drift ratio for masts.

1 GENERAL

1.1 Scope of Part 6 of Eurocode 8

(1)P EN 1998-6 establishes requirements, criteria, and rules for design of tall slender structures: towers, including bell-towers, intake towers, radio and tv-towers, masts, industrial chimneys and lighthouses. Different provisions apply to reinforced concrete and to steel structures. Requirements are set up for non-structural elements, such as the lining material of an industrial chimney, antennae and other technological equipment.

(2)P The present provisions do not apply to cooling towers and offshore structures. For towers supporting tanks, see EN 1998-4.

1.2 Normative References

(1)P The following normative documents contain provisions, which through references in this text, constitute provisions of this European standard. For dated references, subsequent amendments to or revisions of any of these publications do not apply. However, parties to agreements based on this European standard are encouraged to investigate the possibility of applying the most recent editions of the normative documents indicated below. For undated references the latest edition of the normative document referred to applies.

1.2.1 General reference standards

EN 1990:2002 Eurocode - Basis of structural design

EN 1992-1-1:200X Eurocode 2 – Design of concrete structures – Part 1-1: General – Common rules for building and civil engineering structures

EN 1993-1-1:200X Eurocode 3 – Design of steel structures – Part 1-1: General – General rules

EN 1994-1-1:200X Eurocode 4 – Design of composite steel and concrete structures – Part 1-1: General – Common rules and rules for buildings

EN 1995-1-1:200X Eurocode 5 – Design of timber structures – Part 1-1: General – Common rules and rules for buildings

EN 1996-1-1:200X Eurocode 6 – Design of masonry structures – Part 1-1: General – Rules for reinforced and unreinforced masonry

EN 1997-1:200X Eurocode 7 - Geotechnical design – Part 1: General rules

EN 1999-1-1:200X Eurocode 9 – Design of aluminium structures – Part 1: General rules

1.2.2 Reference Codes and Standards

(1)P EN 1998-6:200X incorporates other normative references cited at the appropriate places in the text. They are listed below:

ISO 1000 S I Units and recommendations for the use of their multiples and of certain other units.

ISO 8930 General principles on reliability for structures - List of equivalent terms.

EN 1090-1 Execution of steel structures - General rules and rules for buildings.

EN 10025 Hot rolled products of non-alloy structural steels - Technical delivery conditions.

prEN 1337-1 Structural bearings - General requirements.

prEN 10080-1 Steel for reinforcing of concrete - Weldable reinforcing steel- Part 1: General requirements, of March 1999.

prEN 10080-2 Steel for reinforcing of concrete - Weldable reinforcing steel-Part 2: Technical delivery conditions for class A, of March 1999.

prEN 10080-3 Steel for reinforcing of concrete - Weldable reinforcing steel-Part 6: Technical delivery conditions for class B, of March 1999.

prEN 10080-4 Steel for reinforcing of concrete - Weldable reinforcing steel-Part 4: Technical delivery conditions for class C, of March 1999.

prEN 10080-5 Steel for reinforcing of concrete - Weldable reinforcing steel-Part 5: Technical delivery conditions for welded fabric, of March 1999.

prEN 10080-6 Steel for reinforcing of concrete - Weldable reinforcing steel-Part 6:Technical delivery conditions for lattice girders, of March 1999.

prEN 206:2000 Concrete. – Part 1: Specification, performance, production and conformity, January 2000.

ISO Structural steel - Cold formed, welded, hollo sections -Dimensions and sectional properties.” Draft International Standard, ISI/DIS 4019, edited by ISO/TC 5/SC1, 1999.

prEN 10138 Prestressing steel. Part 1: General requirements. Part 2: Stress relieved cold drawn wire. Part 6: Strand. Part 4: Hot rolled and processed bars. Part 5: Quenced and tempered wire, November 1991.

1.3 Assumptions

(1)P The following assumptions apply:

- The design of structures is accomplished by qualified and experienced personnel.
- Adequate supervision and quality systems are provided in design offices, factories, plants and on site.
- Personnel having the appropriate skill and experience carry out the construction.
- The construction materials and products are used as specified in the Eurocodes or in the relevant material or product specifications.

- The structure will be adequately maintained.
- The structure will be used in accordance with the design brief.
- No change of the structure will be made during the construction phase or during the subsequent life of the structure, unless proper justification and verification is provided. Due to the specific nature of the seismic response, this applies even in the case of changes that lead to an increase of the structural resistance.

(2) In this code numerical values identified by [ ] are given as indications. The National Authorities may specify different values.

1.4 Distinction between principles and application rules

(1) The rules of clause 1.4 of EN 1990:2002 apply.

1.5 Definitions

(1) Unless otherwise stated in the following, the terminology used in International Standard ISO 8930 applies.

1.5.1 Special terms used in EN 1998-6

Stack: Stacks, flues, chimneys are construction works or building components that conduct waste gases, other flue gases, supply or exhaust air.

Supporting shaft or shell: The supporting shaft is the structural component, which supports the waste gas flues.

Waste gas flue: The flue that conducts waste gases is a component that carries waste gases from fireplaces through the stack outlet into atmosphere.

Internal flue: The internal flue is a waste gas conducting flue that is installed inside of the supporting shaft which protects all other stack components against thermal and chemical strains and aggressions.

Transmission tower: a tower used to support electric transmission cables, either at low or high voltage.

Tangent towers: Electric transmission towers used where the cable line is straight or has an angle not exceeding 3 degrees in plane. They support vertical loads, a transverse load from the angular pull of the wires, a longitudinal load due to unequal spans, and forces resulting from the wire-stringing operation, or a broken wire.

Angle towers: Towers used where the line changes direction by more than 3 degrees in plane. They support the same kinds of load as the tangent tower.

Dead-end towers (also called anchor towers): Towers able to support dead-end pulls from all the wires on one side, in addition to the vertical and transverse loads.

Other special, earthquake-related terms of structural significance used in Part 6 are defined in 1.4.2 of Part 1-1.

1.6 Symbols

1.6.1 General

(1) For the material-dependent symbols as well as for symbols not specifically related to earthquakes the provisions of the relevant Eurocodes apply.

(2) Further symbols, used in connection with seismic actions, are defined in the text where they occur, for ease of use. However, in addition, the most frequently occurring symbols used in EN 1998-6 are listed and defined in 1.6.2.

1.6.2 Further symbols used in Part 6

E_{eq} equivalent modulus of elasticity;

M_i effective modal mass for the i-th mode of vibration.

R^{θ} (given a one degree of fredom oscillator), the ratio between the maximum moment on the oscillator spring and the rotational moment of inertia about the axis of rotation. The diagram of R^{θ} versus the natural period is the rotation response spectrum;

R_x^{θ} , R_y^{θ} , R_z^{θ} the rotation response spectra around the axis x, y and z, in rad/sec^2

γ specific weight of the cable per unit volume;

σ tensile stress in the cable;

$\bar{\xi}_j$ equivalent modal damping ratio of the j-th mode,

1.7 S.I. Units

(1)P S.I. Units shall be used in accordance with ISO 1000. Forces are expressed in Newton’s or kiloNewtons, masses in kg or tons, and geometric dimensions in meters or mm.

2 PERFORMANCE REQUIREMENTS AND COMPLIANCE CRITERIA

2.1 Fundamental requirements

(1)P The design philosophy of EN 1998-6, is based on the general requirement that, under earthquake conditions, 1) danger to people, nearby buildings and adjacent facilities shall be prevented, and 2), the continuity of the function of plants, industries, and communication systems has to be maintained. The first condition identifies for the present structures with the non-collapse requirement defined in 2.1 of EN 1998-1-1:200X and the second condition with the damage limitation requirement defined in 2.1 of EN 1998-1-1:200X.

(2)P The damage limitation requirement refers to a seismic action having a probability of occurrence higher than that of the design seismic action. The structure shall be designed and constructed to withstand this action without damage and limitation of use, the cost of damage being measured with regards to the cost of involved equipment, and cost of limitation of use with regards to the cost of the interruption of activity of the plant. To this requirement importance classes are defined in 4.2.5.

(3) In regions of low seismicity, the rule 2.2.1 and the application of earthquake forces given in 4.6.2 adequately satisfy the fundamental requirements. It is recommended to consider as low seismicity region those in which the design ground acceleration a_g, on type A soil, is not higher than [0,08 g].

2.2 Compliance criteria

2.2.1 General

(1)P With the only exceptions explicitly mentioned in the present document, concrete structures shall conform to EN 1992, steel structures to EN 1993, and composite structures to EN 1994. Wind snow, and ice loads are defined in EN 1991.

(2)P For foundation design, see EN 1998-5:200X.

2.2.2 Ultimate limit state

(1) Most of the present structures are classified as non-dissipative, thus no account is taken of hysteretic energy dissipation and a behaviour factor not higher than 1,5 is selected. For dissipative structures a behaviour factor higher than 1,5 is adopted. It accounts for hysteretic energy dissipation occurring in specifically designed zones, called dissipative zones or critical regions.

(2)P The structure shall be designed so that after the occurrence of the design seismic event, it shall retain its structural integrity, with appropriate reliability, with respect to both vertical and horizontal loads. For each structural element, the amount of inelastic deformation shall be confined within the limits of the ductile behaviour, without substantial deterioration of the ultimate resistance of the element.

(3) Unless special precautions are taken, provisions of the Code do not specifically provide protection against damage to equipment and non-structural elements during the design seismic event.

2.2.3 Damage limitation state

(1) In the absence of a well precise requirement of the Owner, satisfying the deformation limits defined in 6.2.5 will ensure that damage would be prevented to the structure itself, to non-structural elements and to the installed equipment.

2.2.3.1 Foundations

(1)P The stiffness of the foundations shall be adequate for transmitting to the ground, as uniformly as possible, the actions received from the superstructure assuming a behaviour factor not greater than 1,5.

3 SEISMIC ACTION

3.1 Definition of the seismic input

(1) The free-field seismic excitation is specified through the definition of the translation motion at a point. For particular structures, the spatial variability of the translation motion at a point is important. The rotation motion at the point defines it.

(2) The translation motion is defined as in EN 1998-1-1:200X and the rotation motion is defined in Annex A.

3.2 Elastic response spectrum

(1)P The elastic response spectrum for acceleration is defined in clause 3.2.2 of EN 1998-1:200X. The influence of local ground conditions on the seismic action shall generally be accounted for by considering the five ground types A, B, C, D and E described in clause 3.1.1 of EN 1998-1:200X, according to the stratigraphic profiles. The transmission level is the elevation of the lower-most level of the foundation, or the top of the piles, if present.

3.3 Design response spectrum

(1) The design response spectrum is the q-reduced response spectrum, defined in 3.2.2.5 of EN 1998-1-1:200X. The behaviour factor q incorporates the elastic dissipation in the structure and that due the soil-to structure interaction and to the inelastic hysteretic behaviour of the structure.

3.4 Time-history representation

(1) If time-domain analyses are performed, both artificial accelerograms and records of historic strong motion can be used. Time-histories are generally used for non-linear step by step analyses. The relevant peak value and frequency content shall be consistent with the elastic response spectrum, (not with the q- reduced design response spectrum).

(2) In case artificial accelerograms are used, independent time history can be generated for translation and rotation acceleration.

(3) The strong motion duration should be selected in a way consistent with clause 3.2.3.1.2 of EN 1998-1:200X.

3.5 Long period components of the motion at a point

(1) Towers, masts, and chimneys are sensitive to the long period components of the seismic excitation. Soft soils or peculiar topographic conditions might provide abnormal amplifications to these components.

(2) A suitable geological and geotechnical survey should be developed, to identify the soil properties. It should be extended at least until the depth at which the static effects of the structure, due to dead load, are significant

(3) Lacking the geotechnical survey, the design spectrum corresponding to a soil profile more unfavourable for the structure shall be assumed, (see clause 3.2.2.2 of EN 1998-1:200X), with a soil factor $S = 1,5$

(4) Where site-specific studies of the ground motion have been carried out, with particular reference to the long period motions, the limitation of clause 3.2.2.5 of EN 1998-1:200X, $S_d \geq 0,2\ \alpha$, may be relaxed to $S_d \geq 0,1\ \alpha$.

3.6 Spatial variability of the seismic motion

(1) Structures taller than 80 meters, in regions of high seismic activity, $\alpha > [0,25]$, should be analysed with proper consideration to a spatial model of the seismic excitation.

(2) In general, tall structures may be sensitive to a spatially varying vertical excitation: a vertical ground motion propagating in any horizontal direction is expected to cause rocking of the structure, concurrent with the rocking caused by the horizontal excitation along that direction.

(3) A possible model to describe the rotation motion is given in Annex A.

4 DESIGN OF EARTHQUAKE RESISTANT TOWERS, MASTS AND CHIMNEYS

4.1 Importance factors

(1)P The following factors are applicable, in the absence of a more detailed risk analysis:

γ_I = [1,4] for structures whose operation is of strategic importance, in particular if vital component of a water supply system, an electric power plant or a communication facility.

γ_I = [1,2] for structures the height of which is greater than the distance from the surrounding buildings, for structures built in an area likely to be crowded, or for structures whose collapse may cause the shutdown of industries.

γ_I = [1,1] for all structures taller than 80 m, not pertaining to the above category.

γ_I = [1,0] for the remaining cases.

4.2 Number of degrees of freedom

(1) The mathematical model should consider:

- Rocking and translation stiffness of foundations;
- An adequate number of masses and degrees of freedom to determine the response of any significant structural element, equipment, and appendages;
- The mass and stiffness of cables and guys;
- The relative displacement among supports of equipment or machinery (for a chimney, the interaction between internal and external tubes);
- Significant effects such as piping interactions, externally applied structural restraints, hydrodynamics loads (both mass and stiffness effects);

(2) The torsion stiffness of the foundation shall be included if significant.

(3) For electric transmission towers, unless a complete dynamic model is made for a representative portion of the entire line, a group of at least three towers has to be modelled, so that an acceptable evaluation of the cable mass and stiffness can be accounted for the central tower.

4.3 Masses

(1)P The model shall include a discretization of masses so that a suitable representation of the inertia effects is ensured. As appropriate, translation and/or rotational mass shall be considered.

(2)P The masses shall include all permanent constructions, fittings, insulation, dust loads, clinging ash, present and future coatings, liners and the effect of fluids or moisture on density of liners, if relevant, and equipment. Permanent masses of structures and quasi-permanent equipment masses shall be considered.

(3) Applicable ψ_{2i} values are:

- imposed loads on platforms $\psi_{2i} = 0,2$;
- maintenance loads on platforms $\psi_{2i} = 0,2$;
- temporary equipment on transmission towers and masts $\psi_{2i} = 0,8$;
- For towers and masts in cold regions a suitable proportion of ice load shall be included.

(4)P If cables are present, a correct representation of the relevant masses shall be included in the model.

(5) Idealising a cable as a single spring does not allow for its inertia and consequent dynamic response. When the mass of the cable is significant in relation to that of the tower, the cable should be represented as chain of elements connecting lumped masses.

(6) The total effective mass of the immersed part of intake towers shall be assumed equal to the sum of:

- the actual mass of the tower shaft (without allowance for buoyancy);
- the mass of the water possibly enclosed within the tower (hollow towers);
- the added mass of externally entrained water.

(7) In absence of rigorous analysis, the added mass of entrained water may be estimated according to Annex F of EN 1998-2:200X.

4.4 Stiffness

(1) In concrete structures, if the analysis is made on the basis of a suitable q factor greater than 1, and the corresponding design response spectrum, the section properties according to 4.3.1 of EN 1998-1-1:200X. If q = 1, and the analysis is based on the elastic response spectrum or a corresponding time-history of the ground motion, the element stiffness should take into account the cracked cross-section properties, in agreement with the expected level of stress.

2 Due regards shall be given to the temperature effect on the stiffness and strength of the steel in steel chimneys structures.

(3) In case cables are integral part of a structure, a careful modelling of their stiffness should be done.

(4) If the sag of the cable is significant, the spring value should account for it. An iterative solution may be generally required. It can be based on the use of 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} equivalent modulus of elasticity;

γ specific weight of the cable, weight per unit volume;

σ tensile stress in the cable;

ℓ cable length;

E_c modulus of elasticity of cable material.

(5) For wrapped up ropes, E_c is generally lower than the single cord modulus of elasticity E. An applicable reduction is

$$\frac{E_c}{E} = \cos^3 \beta \qquad (4.2)$$

where β is the wrapping angle of the single cords.

(6) In cases where the sag of the cable is meaningful, the likelihood of impulsive loading between tower and the cable ends should be analysed.

(7) If the preload of the cable is such that the sag is meaningless, or if the tower is short, (less than 40 meters), then the presence of the cable can be represented in the dynamic model by a linear spring.

4.5 Damping

(1) If the analysis is performed without resorting to the reduced design spectrum, it is allowed to consider damping values different from 5%. In this case, the damping ratio of each mode of vibration may be defined according to Annex B and the corresponding elastic spectral ordinates as prescribed in 3.2.2.2(3) of EN 1998-1-1:200X.

4.6 Soil-structure interaction

(1) The design earthquake motion is defined at the soil surface, in free-field conditions, i.e. where it is not affected by the inertial forces due to the presence of structures. When the structure is founded on soil deposits or soft media, the resulting motion at the base of the structure will differ from that at the same elevation in the free-field, due to the soil deformability. Annex C provides suitable rules to account for soil compliance during earthquakes.

(2) For tall structures, (the height being over two times the maximum base dimension), the rocking compliance of the soil is important and may significantly increase the second order effects.

4.7 Methods of analysis

4.7.1 Applicable methods

(1) The standard methods of analysis is the linear analysis using the q-reduced design spectrum either as a simplified dynamic analysis or a multimodal analysis.

(2) Non-linear methods of analysis may be applied provided that they are properly substantiated with respect to the seismic input, the constitutive model used, the method of interpreting the results of the analysis, and the requirements to be met (see clause 4.3.3.1 of EN 1998-1-1:200X).

4.7.2 Simplified dynamic analysis

4.7.2.1 General

(1) This type of analysis are generally applied to structures that can be analysed by two planar models and whose response is not significantly affected by contributions of higher modes of vibration.

(2)P For regular structures, the method set forth in the literature based on the "rigid diaphragm" assumption, can be applied. For steel masts, an horizontal bracing system, capable of providing the required rigid diaphragm action, is to be present. Lacking this, a three dimensional dynamic analysis is necessary.

(3)P For steel chimneys, horizontal stiffening rings shall be present in the design, for the "rigid diaphragm" assumption being applicable. Otherwise, a suitable dynamic analysis, capable of identifying the hoop stresses, is required.

(4) Piping and equipment supported at different points should be analysed taking into account the relative motion between supports. This motion may be larger that that conceived by the simplified analysis.

(5) For reinforced concrete towers and chimneys, hoop reinforcement should take into account the ovalling of the horizontal cross section due to lateral forces. A dynamic analysis capable of identifying the hoop stresses, is suitable.

(6) Simplified dynamic analysis is allowed only if the importance factor is $\gamma_I \leq 1$, and the height is $H < [80]$ m.

4.7.2.2 Seismic forces

(1) The effects induced by the seismic action are determined by subdividing the structure into n distinct concentrated masses, including the masses of the foundations, to which the horizontal forces F_i, i = 1, 2...n, are applied, given by the expression:

$$F_i = \frac{h_i w_i}{\sum_{1}^{n} h_j w_j} F_t \qquad (4.3)$$

where

$$F_t = S_d(T) \sum_{1}^{n} w_j \qquad (4.4)$$

w_i weight of the i-th mass including permanent load and variable loads multiplied by the pertinent combination factor specified in 4.3;

h_i is the elevation of the i-th mass from the level of application of the seismic excitation;

$S_d (T)$ is the ordinate of the design spectrum as defined in EN 1998-1:200X, for the fundamental period of vibration T. In case the period T is not evaluated through a valuable structural model, the spectral value $S_d (T_c)$ shall be accounted for.

(2) The above method may provide a substantial overvaluation of the seismic action in the case of tapered towers where around the base the mass distribution sharply decreases with elevation.

4.7.3 Modal analysis

4.7.3.1 General

(1) This method of analysis can be applied to any structure, with an input motion defined by a response spectrum or by the corresponding time history.

4.7.3.2 Number of modes

(1) For a continuously distributed mass structure, cantilevering from the soil, the minimum number of modes, necessary to assure participation of all significant modes, is higher than the number suitable for a "shear type" building, with lumped masses.

(2) The minimum number of modes which is necessary to evaluate internal actions at the top of the structure is generally higher than that which is sufficient for evaluating the overturning moment or the total shear at the base of the structure.

(3) A practical rule to establish the sufficient number of modes is the following. For each mode i, and for each direction of the excitation, the "equivalent modal mass" M_i is evaluated. Then, for each direction, the sum of M_i is performed and is compared to the total mass of the structure M. If

$$\sum_{i}^{N} M_i \geq 0{,}9\, M \tag{4.5}$$

then the considered number of modes is adequate. An exception to the above rule may occur in case when a light equipment or a light structural appendix is concerned. Appendix D provides hints for the practical application of expression (4.5)

4.7.3.3 Combination of modes

(1)P For each quantity, (force, displacement, stress), the probable maximum value S of the earthquake effect in general shall be obtained as the square root of the sum of the squares of the contributions of individual modes, (SRSS combination):

$$S = \pm \sqrt{(s_1^2 + s_2^2 + s_3^2 + ...)} \tag{4.6}$$

where:

s_1, s_2, s_3 are the contributions to the selected quantity of modes 1, 2, 3... This action effect assumes both the positive and the negative sign.

(2)P For any one direction of the seismic excitation, when two significant modes *i* and *j* show closely spaced periods, with the ratio T_j/T_i exceeding 0,9 with $T_j < T_i$, the above rules becomes unconservative and more accurate rules must be applied.

4.7.3.4 Combination of different ground motion components

(1)P Effects of different components shall be combined according to clause 4.3.3.5.1 and 4.3.3.5.2 of EN 1998-1-1:200X. The effects of the translation and the rotation components of the ground excitation can be combined each to the other assuming as global effect the square root of the sum of the squares of the single effects, (SRSS combination).

4.7.3.5 Combination of internal actions

(1)P When combining different internal actions, for instance bending moment and axial forces, each internal action is to be computed according to the above rule. All physically possible combinations shall be considered.

4.8 Combinations of the seismic action with other actions

(1)P The design value E_d of the effects of actions in the seismic design situation shall be determined by combining the values of the relevant actions as follows:

$$\sum G_{kj} "+" \gamma_I A_{E_d} "+" P_k "+" \sum \psi_{2i} Q_{ki} \tag{4.7}$$

where

"+" implies "to be combined with";

$\sum$ implies "the combined effect of",

G_{kj} characteristic value of permanent action *j*;

γ_I importance factor (see 2.3);

A_{Ed} design value of the seismic action for the reference return period;

P_k characteristic value of prestressing action;

ψ_{2i} combination coefficient for quasi permanent value of variable action *i*;

Q_{ki} characteristic value of variable action *i*.

(2)P The effects of the seismic action shall be evaluated by taking into account the presence of all gravity loads appearing in the following combination of actions:

$$\sum G_{kj} "+" \sum \psi_{2i} \cdot Q_{ki} \tag{4.8}$$

where

ψ_{Ei} combination coefficient for variable action *i*.

(3) The combination coefficients ψ_{Ei} take into account the likelihood of the loads ψ_{2i} Q_{ki} being not present over the entire structure during the occurrence of the

earthquake. These coefficients may also account for a reduced participation of masses in the motion of the structure due to the nonrigid connection between them.

(4) Values of coefficient ψ_{2i} are given in 4.3, and values of ψ_{Ei} are given in 4.2.4 of EN 1998-1-1:200X. Laking a precise information, for the present structures $\psi_{Ei} = \psi_{2i}$ should be assumed.

4.9 Displacements

(1)P The displacements induced by the design seismic action shall be calculated on the basis of the elastic deformation of the structural system by means of the following simplified expression:

$$d_s = q_d \, d_e \gamma_i \qquad (4.9)$$

where:

d_s displacement of a point of the structural system induced by the design seismic action;

q_d displacement behaviour factor, assumed equal to q;

d_e displacement of the same point of the structural system, as determined by a linear analysis based on the design response spectrum.

4.10 Safety verifications

4.10.1 Ultimate limit state

(1)P The safety against collapse (ultimate limit state) under the seismic design situation is considered to be ensured if the following conditions regarding resistance, ductility and stability are met.

4.10.2 Resistance capacity of the structural elements

(1) The following relation must be satisfied for all structural element:

$$R_d \geq E_d \; (\gamma_i \, E, G, P, \ldots\ldots\ldots\ldots\ldots\ldots) \qquad (4.10)$$

where

R_d design resistance capacity of the element, calculated according to the mechanical models and the rules specific to the material, (characteristic value of property f_k, and partial safety factor γ_m).

E_d design value of the effect in the combination of actions including, if necessary, P-Δ effects and thermal effects.

4.10.3 Second order effects

(1)P Second order, P-Δ, effects shall be evaluated considering the displacements computed as indicated in 4.7, unless the condition (2) is respected.

(2) Second order, P-Δ, effects need not be considered when the following condition is fulfilled.

$$\delta M / M_o < 0{,}10 \qquad (4.10)$$

where

δM overturning moment due to P-Δ effect

M_o first-order overturning moment

4.11 Thermal effects

(1)P If the operating temperature of a structural elements is above [100º C], then the thermal effects on the mechanical properties of the structural element such as elastic modulus, yield stress and thermal expansion coefficient shall be taken into account. The relevant Eurocodes shall be applied to estimate such effects.

4.12 Ductility condition

(1)P It shall be verified that the structural elements and the structure as a whole possess adequate ductility to its expected exploitation, which depends on the selected system and the adopted behaviour factor.

4.13 Stability

(1) The stability of the structure shall be verified under the set of forces induced by the combination rules including piping interaction and hydrodynamic loads, if present.

(2) Special criteria of stability verification are reported for steel chimneys and steel towers and masts in EN 1993-3:200X.

4.14 Serviceability limit state

(1) Deflections for the serviceability limit state shall be calculated by reducing of the factor ν, defined in (2), the displacements given in the expression (4.9).

(2) The reduction factor ν takes into account the shorter return period of the seismic action associated with the damage limitation requirement. Suggested values are: v = [0,4] for structures to which $\gamma_I > 1$ is assigned, and v = [0,5] for other structures.

(3) If certain use of the structure is significantly affected by deflections, (for example in telecomunication towers), the deflection should be limited to appropriate values. Peak transient deflections need to be calculated if they lead to permanent damage.

4.15 Behaviour factor

4.15.1 General

(1)P The behaviour factor q is given by the product:

$$q = q_o\, k_r \geq 1{,}5 \qquad (4.11)$$

where:

q_o basic behaviour factor, reflecting the ductility of the lateral load resisting system, with values defined in 5, 6 and 7 for each different type of structure.

k_r modification factor reflecting departure from a regular distribution of mass, stiffness or strength, with values defined in 4.15.2.

4.15.2 Values of factor k_r

(1)P The value of k_r shall be taken as follows, depending of the existence of the following irregularities on the structure.

a) Horizontal mass eccentricity at a section exceeding 5% of the relevant structure dimension

$k_r = 0,8$

b) Opening in shaft causing a 30% or larger reduction of the moment of inertia of the cross section

$k_r = 0,8$

c) Concentrated mass within the top third elevation contributing 50% or more to the base overturning moment,

$k_r = 0,7$

(2)P When more than one irregularity are present, k_r shall be assumed equal to the product of the two lowest values of k_r.

5 SPECIFIC RULES FOR REINFORCED CONCRETE CHIMNEYS

5.1 Basic Behaviour factor

(1)P Critical regions shall be considered the concrete wall up to a distance d (where d denotes the outer diameter of the wall) above the bottom cross-section, and above sections where an abrupt change of thickness is made. Critical regions shall also be considered the concrete wall where more than one opening exist, and up to a distance d above and below those openings.

(2)P When a local curvature ductility of the critical sections of at least $\mu_{1/r} = 9$ is secured, if necessary by providing confining reinforcement,

$$q_o = 3$$

In all other cases

$$q_o = 1,5$$

(3) The design of chimneys is generally governed by wind considerations, with the exception of locations with medium to high seismicity, chimneys with large elevated masses, and chimneys with unusual geometries.

5.2 Materials

(1) All materials and material tests should conform to Eurocode 2.

(2) The same brand and type of cement should be used throughout the construction of the chimney wall. The maximum size of coarse aggregate should not be larger than 1/8 of the narrower dimension between forms nor larger than 1/2 the minimum clear distance between parallel reinforcing bars.

(3) The specified concrete should be of a class not lower than C20/25 as defined in Eurocode 2.

5.3 General

5.3.1 Minimum reinforcement (vertical and horizontal)

(1)P For a chimney with an outside diameter of 4 m or more, the minimum ratio of the vertical reinforcement to the gross-sectional area shall be not less than 0,003. The reinforcement shall be distributed in layers towards the inner and the outer face, with not less than half the reinforcement in the layers towards the outer face.

(2)P Close to the chimney top, where stresses due to permanent load are rather small, a minimum vertical reinforcement equivalent to that for the horizontal direction must be provided.

(3)P A chimney with an outside diameter of 4 m or more must be provided with layers of horizontal reinforcement in the proximity of both surfaces and the ratio to the gross-sectional area shall not be less than 0,001.

(4)P In chimneys with an outside diameter of less than 4 m, the inner layer of reinforcement may be omitted, but in that case the ratio of outer layer reinforcement to gross-sectional area shall not be less than 0,002.

(5) Circumferential bars should be placed around the exterior of, and secured to the vertical bars. All reinforcing bars should be tied at intervals of not more than 60 cm.

(6) Particular attention should be paid to placing and securing the circumferential reinforcing so that it cannot bulge or be displaced during the pouring and working of the concrete, so as to result in less than the required concrete cover of the circumferential reinforcement. Circumferential bars should be closed preferably by welding. No closure by splicing should be permitted.

5.3.2 Distance between reinforcement bars

(1) The distance between vertical bars should be not more than 250 mm and the distance between horizontal bars should not exceed 200 mm.

5.3.3 Minimum reinforcement around openings

(1) In addition to the reinforcement determined by the stability and temperature, extra reinforcement should be provided at the sides, bottom, top and corners of openings as hereinafter specified. This extra reinforcement should be placed near the outside surface of the chimney shell as close to the opening as proper spacing of bars will permit. Unless otherwise specified, all extra reinforcement should extend past the opening a sufficient distance to develop the bars in bond.

(2) The minimum vertical reinforcement ratio should be 0,0075, in a distance of half the width of the opening. Both sides of the opening should be reinforced.

5.3.4 Minimum cover to the reinforcement

(1) The concrete cover to the circumferential reinforcement should be 30 mm minimum with a tolerance of + 20 mm and - 10 mm.

5.3.5 Reinforcement splicing

(1) Not more than 50 % of the bars should be spliced along any plane unless specifically permitted on drawings or approved by the responsible engineering.

5.3.6 Concrete placement

(1) In the concrete shell no vertical construction joint should be used. Horizontal construction joint for jump form construction should be maintained at approximately uniform spacing throughout the height of the chimney.

(2) Concrete should be deposited in approximately level layers no greater than 40 cm deep. Particular care should be exercised when placing concrete in thin wall sections where two layers of reinforcing are present.

5.3.7 Construction tolerances

(1) The vertical alignment of centerpoint should not vary along the vertical axis by more than 1/1000 of the height of the shell at the time of measurement, or 2 cm, whichever is greater.

(2) The measured outside shell diameter at any section should not vary from the specified diameter by more than 1/100 of the specified or theoretical diameter.

(3) The measured wall thickness should not vary from the specified or theoretical wall thickness by more than -1 +2 cm. A single wall thickness is defined as the average of at least four measurements taken over a 60-degree arc.

(4) Tolerances on the size and location of opening and embedments should be established depending on the nature of their use. Lacking further requirements, tolerances for opening and embedment sizes and locations are 1/100 of the shell outside diameter.

5.4 Design loads

5.4.1 Construction loading

(1) In the design of a chimney for horizontal earthquake forces, only one horizontal direction need be considered. Unlike building structures, chimneys are generally axisymmetric, and the orthogonal effects from two horizontal earthquakes acting simultaneously in the two principal directions are negligible.

(2) The effect of the vertical component of the earthquake on the chimney is generally of no design significance, and can be disregarded.

(3) In cases the lining (brick, steel, or other materials) is laterally supported by the chimney shell at discrete points close one to the other so that a meaningful relative movement is not expected during a seismic shaking, the lining can be taken into account by incorporating its mass into that of the shell.

(4) For cases in which the chimney lining is supported at the top of the chimney shell or at intermediate points distant one to the other so that a meaningful relative movement is expected, a dynamic analysis including both concrete shell and liner should be used.

(5) When using the elastic response spectrum, appropriate damping values should be used for the liner depending on its construction (e.g., 1,5 percent for steel liners, 4,0 percent for brick liners, and 2,0 percent for fiber reinforced plastic liners).

(6) Consideration should be given to the construction loading, during the construction phase. In particular, if required during construction, temporary access openings may be provided in the construction shell. However, for the design of the shell, these openings should be designed as permanent openings.

5.5 Serviceability limit states

(1)P Waste gas flues in chimneys shall be checked for imposed deformations between support points, and imposed clearances between internal elements, so that gas tighness is not lost and sufficient reserve in maintained against the flue gas tube collapse.

(2) The requirement for limiting damage is considered satisfied if the maximum lateral deflection of the top of the structure, prior to the application of load factors, does not exceed the limits set forth by the following equation:

$$d_{max}\ \nu = 0{,}005 \cdot H \tag{5.1}$$

where

d_{max} is the lateral deflection at the top of the chimney;

H is the height of the structure;

ν the reduction factor to take into account the lower return period of the seismic action associated with the damage limitation requirement. Suggested values are are: ν = [0,4] for chimneys of $\gamma_I > 1$, and ν = [0,5] for chimneys of $\gamma_I = 1$.

(3) The relative deflection between shell and lining as well as the deflection of the supporting platform should be limited to ensure the serviceability of the lining. Unless otherwise specified by the Owner, the following limits on the relative deflection of adjacent supporting platforms shall be observed:

a) if provisions are taken to allow relative movement between separate parts of liner, such as if the liner is made by tubes independent one from the other, with suitable clearance,

$$d_r \leq 0{,}025\ \Delta H$$

b) In other case

$$d_r \leq 0{,}0125\ \Delta H$$

where ΔH is the distance, along the vertical axis, between liner supporting platforms.

(3) The deflection limit can be compared against the deflection calculated using uncracked concrete sections and a fixed base.

(4) Limiting deflections also serves to reduce the effects of secondary bending moments.

5.6 Ultimate limit state

(1) In the calculation of limit-state bending moments, allowance needs to be made for the moment caused by the weight of the chimney in its deflected shape.

6 SPECIAL RULES FOR STEEL CHIMNEYS

6.1 Basic behaviour factor

(1)P Steel frames or trusses structures lateral supporting flue gas ducts of chimneys:

a) Structures designed for dissipative behaviour according to the specific rules for steel buildings of 6 of EN 1998-1:200X:

Moment resisting frames or with eccentric bracing's $q_o = 5$

Frames with concentric diagonal bracing's $q_o = 4$

Frames with V-bracing's, see also figure 1, $q_0 = 2$

b) Structures not designed for dissipative behaviour, frames with K-bracing's, see also Figure 1 $q_o = 1,5$

(2)P Steel shell-type structures:

a) Structures with cross sections satisfying the requirements of 5.3.3 of EN 1993-1-1:200X for plastic global analysis $q_o = 2$

b) all other structures $q_o = 1,5$

6.2 General

(1) Guyed steel stacks and chimneys are generally lightweight. As such the design loads due to natural hazards are generally governed by wind. On occasion, large flares or other elevated masses located near the top may require an in-depth seismic analysis.

6.3 Materials

(1)P The mechanical properties and the chemical composition of structural steel shall comply with the European Standards requirements, in the series EN 10000.

(2) All structural requirement should be met both at the operating temperature, and at the ambient temperature. For the most commonly used grades of steel, Table 6.1 gives the mechanical properties versus temperature.

Table 6.1: Yield stress in N/mm², and Young modulus in kN/mm², versus temperature, in ºC

Temperature	yield stress			Young modulus
- ºC-	S235	S275	S355	kN/mm^2
20	235	275	355	210
200	207	242	312	201
250	196	229	295	197
300	183	214	276	192
350	169	197	255	185
400	152	178	230	173

(3) The most frequent qualification grades are B and C. In severe environmental conditions, mainly in case of low temperature, grade D should be used.

(4) As a result of the qualification tests for materials, a tensile strength up to 20 N/mm^2 less than the prescribed value is allowable for all steels. The upper limit of the tensile strength may be exceeded by:

- 20 N/mm^2 for all steels of class C, D and DD;
- 30 N/mm^2 for all flat products of a thickness less than 3 mm, made from steel of class A, B, C, D and DD.

(5) Other qualified steels can be used, provided that the minimum notch toughness of 28 joules is respected, and the minimum elongation of 3 %, on a standard specimen, with a gauge length L = 5 D, is granted.

(6) The use of special steel which does not respect the above limit, is dissuaded, unless it can be demonstrated that the thickness required for earthquake or wind loading is conveniently less than the provided thickness.

(7) Where stainless steel or alloy steel components are connected to carbon steel, bolted connections are preferred. In order to avoid accelerated corrosion due to galvanic action, such connections should include insulating gaskets. Welded procedure is permitted, provided specialized metallurgical control is exercised with regard to welding procedure, and electrode selection.

6.4 Design loads

(1) The permanent load should include the weight of all permanent constructions, fittings, linings, flues, insulation, present and future loadings, including corrosion allowances. For process plants in which a carry over of ash or dust burden is present, which can adhere to the interior surface of the structural shell or liner, an additional dead load should be added to the permanent load.

6.5 Serviceability limit state

(1)P Waste gas flues in chimneys shall be checked for imposed deformations between support points, and imposed clearances between internal elements, so that gas tighness is not lost and sufficient reserve in maintained against the flue gas tube collapse.

(2) The requirement for limiting damage is considered satisfied if the maximum lateral deflection of the top of the structure, prior to the application of load factors, does not exceed the limits set forth by the following equation:

$$d_{\max}\,\nu = 0{,}005 \cdot H \tag{6.1}$$

where

$d_{\max}$ is the lateral deflection at the top of the chimney;

H is the height of the structure;

ν the reduction factor to take into account the lower return period of the seismic action associated with the damage limitation requirement. Suggested values are: ν = [0,4] for chimneys of $\gamma_I > 1$, and ν = [0,5] for chimneys of $\gamma_I = 1$.

6.6 Ultimate limit state

(1) The use of the present procedure, combined with the partial safety factors will ensure that low cycle fatigue will not contribute to the failure of the chimney.

(2) In the design of details such as flanges, ultimate limit state may take into account of plastic stress distribution.

(3) At the time of construction, in the stress verifications, the minimum thickness allowance for corrosion is 2 mm, unless special care is exercised to minimize corrosion.

7 SPECIAL RULES FOR TOWERS

7.1 General and basic behaviour factor

(1)P Behaviour factors are defined according to the most appropriate identification of the structural arrangement with respect to those represented in Figure 1.

(2) The design of electrical transmission, substation wire support, and distribution structures is typically controlled by high wind, ice-wind combinations, and unbalance longitudinal wire loads. Seismic loads generally do not control their design. Earthquake performance of these structures has demonstrated that seismic loads can be resisted based on traditional electrical transmission, substation, and distribution wire support structure loading. However heavy equipment, such as transformer in distribution structures, may result in significant seismic load.

(3) Besides, earthquake-related damage to electrical transmission, substation wire support, and distribution structures is typically caused by large displacements of the foundations due to landslides, ground failure, and liquefaction. These situations have resulted in structural failure or damaged structural members without complete loss of structure function.

(4) The fundamental frequency of these structure types typically ranges from 0.5 to 6 Hz. Single pole type structures have fundamental mode frequencies in the 0.5 to 1.5 Hz range. H-frame structures have fundamental mode frequencies in the 1 to 3 Hz ranges, with the lower frequencies in the direction normal to the plane of the structure and the higher frequencies in plane. Four legged lattice structures have fundamental mode frequencies in the range of 2 to 6 Hz. Lattice tangent structures typically have lower frequencies with the higher frequencies being representative of angle and dead end structures. These frequency ranges can be used to determine if earthquake loading should be a design consideration. If it is determined that earthquake loads are significant then a more detailed evaluation of the structure vibration frequencies and mode shapes should be performed. This can be accomplished using available commercial finite element computer programs.

(5) For the use of the elastic response spectrum, the default viscous damping value to be used in such an analysis should be 2 percent. A higher damping value can be used if determined using sound engineering data.

(6) A minimum importance factor γ_I = 1,0 is required to minimize the loss of function after an earthquake event even though these systems are normally redundant.

(7) The behaviour factor values shown in Figure 1 reflect the inelastic reserve strength of the structural systems during an earthquake event. The values presented for these types of structures were determined based on a review of published values established for building structures and nonbuilding structures. In general, the q_o values shown reflect the earthquake performance of these structural systems and engineering judgement. Other values may be appropriate if determined using sound engineering data.

7.2 Materials

(1) Welding and bolts should conform to the requirements prescribed in clause 3 of ENV 1993-1-1:200X.

(2) When hot rolled angles are used for lattice towers, the mechanical properties and the composition of the steel should comply with EN 10025 or other equivalent standards.

(3) Hot rolled angles in high tensile steel should comply with Euronorm 10049. Low alloy, cold formed steel, are acceptable. When high strength, their deformability should comply to EN 10049.

(4) Thickness of cold-formed members for towers should be at least 3 mm.

(5) In bolted connections preferably high strength bolts in category 8.8, 10.9 should be used. Bolts of category 12.9 are allowed in shear connections, but are not recommended in general.

(6) Steel towers are normally designed to be in service, without any maintenance, for 30-40 years or more. Such criteria normally demand a satisfactory protection against corrosion, like hot dip galvanising which can be efficiently used for lattice towers made from open sections. Painting is sometimes still requested after galvanizing. Weathering steel may also be used

(7) The value of the yield strength $f_{y\ act}$ which cannot be exceeded by the actual material used in the fabrication of the structure should be specified and noted on the drawings; $f_{y\ act}$ should not be more than 10% higher than the design yield stress f_{yd} used in the design of dissipative zones.

7.3 Design loads

(1) In relation to the regional climate, ice loads may be included among the design loads, both on the structure and on the conductors, when they are present. In this case the loading combination for earthquake includes the ice loading with a factor for ice equal to 1.

7.4 Structural types

(1) In general, structural types and the relevant q factor should be assigned according to Section 6 of EN 1998-1-1:200X. Typical configurations are reported in Figure 1, with the applicable q factors. All of them pertain to the category of frames with concentric bracings, in which the horizontal forces are mainly resisted by members subjected to axial forces.

(2) The bracings may belong to one of the following categories:

- Active tension diagonal bracings, in which the horizontal forces can be resisted by the tension diagonals only, neglecting the compression diagonals. Dissipative zones may be located in the tensile diagonals.
- V bracings, in which the horizontal forces can be resisted by considering both tension and compression diagonals. The intersection point of these diagonals lies on

a horizontal member which must be continuous. Mechanism of dissipation in this configuration are not dependable.

- K bracings, in which the diagonals intersection lies on a column. This last configuration is not recommended.

7.5 Electric transmission towers

(1) In the present section a minimum requirement for accounting the effects of cables between tower and tower, is assessed.

(2) The structure should be analysed under the effect of two concurrent sets of seismic loads:

- A set of horizontal forces at the top of the tower, provided by the cables under the assumption that each tower moves statically with respect to the adjacent towers, in the most adverse direction. The assumed displacement should be equal to twice the maximum ground displacement specified in clause 3.2.2.4 of EN 1998-1-1:200X. A set of relative displacements between tower and tower should be analysed.
- Inertia loads resulting from the dynamic analysis. Unless a dynamic model is made for a representative portion of the entire line, a group of at least three towers should be modelled, so that an acceptable evaluation of the cable mass and stiffness can be accounted for the central tower.

(3) For tangent towers inertia loads are computed assuming the tower as a cantilever beam, elastically supported at the cable elevation along the direction of the cables.

(4) For anchor towers, inertia loads are computed in the most adverse condition resulting from modelling the tower as a cantilever beam standing alone, or a cantilever beam elastically supported at the top along the direction of the cables.

7.6 Serviceability limit state

(1) Unlike other structures, for steel transmission towers serviceability limit state for deflection are not critical. Steel towers can tolerate relatively large elastic and residual displacements.

7.7 Rules of practice

(1) Trussed tubes, involving major diagonals, suffer from inadequate ductility, and therefore are generally not recommended under severe earthquake conditions. A behaviour factor not higher than [2] should be adopted.

(2) When tension is likely to occur at the base of the columns, the corresponding anchorages to foundation should be able to transmit the full tension evaluated under the assumption of a behaviour factor equal to [2].

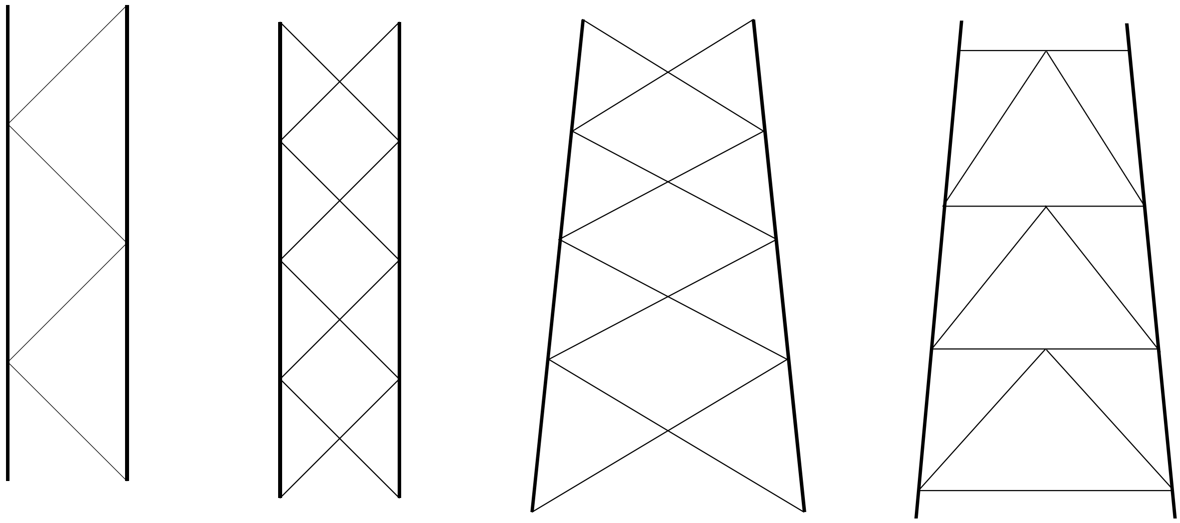
(3) Further critical items in relation to the seismic loading are:

- angles under alternate compression and tension;
- bolted connections, especially single bolt connections;

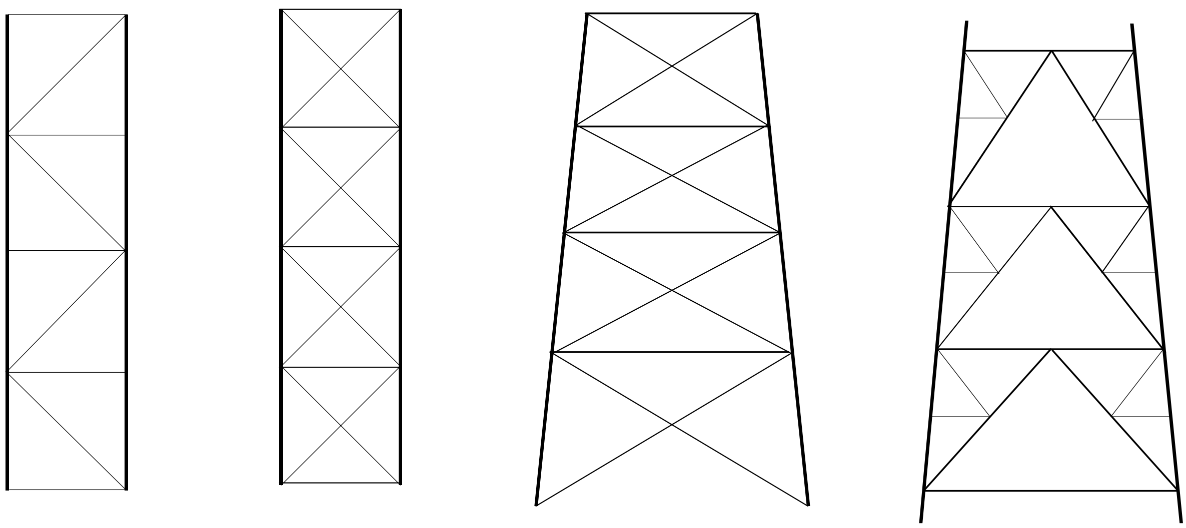
– joints in tubular steel towers.

(4) Members and connections should be experimentally qualified, to withstand a suitable number of cycles of alternate actions, up to their design intensity, without deteriorating the stiffness.

(5) For tubular steel towers, a particular care should be devoted to joints. "Telescope joints" can be used only if experimentally qualified.



$q_o = 2$ $q_o = 2$ $q_o = 2$ $q_o = 2$



$q_o = 3,5$ $q_o = 4$ $q_o = 4$ $q_o = 2$

Figure 1: Basic value of behaviour factors

Ductility class $1,5 < q < 4$ corresponds to a target global drift of the structure equal to 25 mrad.

Ductility class $q > 4$ corresponds to a target global drift of the structure equal to 35 mrad.

8 SPECIAL RULES FOR MASTS

8.1 Basic behaviour factor

(1)P Behaviour factors are defined according to the most appropriate identification of the structural arrangement with respect to those represented in Figure 1.

8.2 Materials

(1) Masts are generally built from laminated open profiles or tubes. Steels normally used are S235, S275, and S355. The most frequent qualification grade is B. When welding is envisaged, grade C is mandatory. However, in very severe environmental conditions, mainly in case of very low temperatures, grade D should be used.

(2) Standards do not put obstacles to the evolution process of production and usage of other steel types with enhanced properties. However under severe cycles of load reversal, the use of high strength steel should be dissuaded, unless an appropriate experimental evidence is provided both on members and on connections.

(3) Hot rolled sections, mainly angles are the most widely used. They can be connected with bolts or welding. Tubes are also used, because of their advantage mainly relating to triangular towers and masts. See Annex H.

8.3 Serviceability limit state

(1) The requirement for limiting damage is considered satisfied if the maximum lateral deflection of the top of the structure, prior to the application of load factors, does not exceed the limits set forth by the following equation:

$$d_{\max}\,\nu = 0{,}005 \cdot H \qquad (8.1)$$

where:

$d_{\max}$ is the lateral deflection at the top of the mast;

H is the height of the mast;

ν the reduction factor to take into account the lower return period of the seismic action associated with the damage limitation requirement. Suggested values are: ν = [0,4] for masts to which $\gamma_I > 1$ is assigned, and ν = [0,5] for other masts.

(2) A limiting drift ratio between horizontal stiffening elements, should be allocated, depending on the masts exercise.

8.4 Guyed masts

(1) A guyed mast (or a guyed tower) is essentially a slender column that is either fixed or hinged at the base and elastically restrained by the cables.

(2) As to the stiffness of the elastic restraint provided by the cables to the tower, they can be subdivided into two broad categories:

- Relatively short towers, (in the neighbourhood of 30÷40m), for which the cables are usually to be to be assumed as straight;
- Tall towers, for which the sag of the cables is large and must be accounted for.

(3) The main difference between the two cases is that the stiffness of a straight cable remains constant as the tower bends, whereas the stiffness of a sagging cable varies with tower deformations (see 4.3).

(4) Cable icing is likely to induce significant sagging, even in relatively short cables (icing loads are often in major importance in region of severe winter conditions, and may be of long duration).

(5) For both sagging and straigth cables, the horizontal component of the cable stiffness is

$$\cos^2(\alpha) = \frac{A_c E_c}{\ell} \tag{8.2}$$

in which

A_c is the cross section area of the cable;

E_c is the effective modulus of elasticity, (accounting for the sag, if the case);

ℓ is the length;

α is the angle of the cable with respect to the horizontal axis. In cases in which the sag of the cable is large, the spring value should account for it. In this case the likelihood of impulsive loading both on the tower and on the cable end should be analysed.

ANNEX A (Informative)

Linear dynamic analysis accounting for a rotational seismic spectrum

(1) The design ground motion during the earthquake is represented by three translation and three rotation response spectra.

(2) The translation ones are the elastic response spectra for the two horizontal components, (axis *x* and *y*), and the vertical component, (axis *z*), referred to in EN 1998-1-1:200X.

(3) The rotation response spectrum is defined in an analogous way as translation response spectrum, i.e. by consideration at a single degree of freedom oscillator, of rotational nature acted upon by the rotation motion. The natural period is denoted by T and damping with respect to the critical damping is denoted by ξ.

(4) Let R^{θ} be the ratio between the maximum moment on the oscillator spring and the rotational moment of inertia about its axis of rotation. The diagram of R^{θ} versus the natural period T, for given values of ξ, is the rotation response spectrum.

(5) Unless results of a specific investigation are available, the rotational response spectra are defined by:

$$R_{\mathrm{x}}^{\theta}(T) = 1{,}7\pi\, S_{\mathrm{e}}(T) / V_{\mathrm{s}} T \qquad \text{(A.1)}$$

$$R_{\mathrm{y}}^{\theta}(T) = 1{,}7\pi\, S_{\mathrm{e}}(T) / V_{\mathrm{s}} T \qquad \text{(A.2)}$$

$$R_{\mathrm{z}}^{\theta}(T) = 2{,}0\pi\, S_{\mathrm{e}}(T) / V_{\mathrm{s}} T \qquad \text{(A.3)}$$

where:

R_{x}^{θ}, R_{y}^{θ} and R_{z}^{θ} are the rotation response spectra around axis x, y and z, in rad/sec^2;

$S_{\mathrm{e}}(T)$ is the site dependent response spectra for the horizontal components, in m/sec^2;

T is the period in seconds.

V_{s} is the S-wave velocity, in m/sec, of the upper layer of the soil profile, or the average S-wave velocity of the first 50 m. The value corresponding to low amplitude vibrations, i.e., to shear deformations of the order of 10^{-6}, can be selected.

(6) The quantity V_{s} is directly evaluated by field measurements, or through the laboratory measurement of the shear modulus of elasticity G, at low strain, and the soil density ρ, being:

$$V_{\mathrm{s}} = \sqrt{\mathrm{G} / \rho}$$

(7) In cases V_s is not evaluated by an apposite experimental measurement, the following values are consistent with the subsoil classification:

Subsoil class	shear wave velocity V_s m/sec
A	800
B	580
C	270
D	150

(8) Consider a ground acceleration $\ddot{u}(t)$ along the horizontal direction, and a rotation acceleration $\omega(t)$ in he plane *u-z*. If the inertia matrix is [*M*], the stiffness matrix [*K*], and the damping matrix [*C*], the equations of motion for the resulting multi-degree-of-freedom system are given by:

$$[M]\{\ddot{u}\}+[C]\{\dot{u}\}+[K]\{u\}=\{m\}\,\ddot{x}+\{m\;h\}\,\ddot{\theta} \tag{A.4}$$

where:

$\{\ddot{u}\}$ vector comprising the system's displacements relative to the base;

$\{m\}$ vector comprising the translational masses in the direction of the u excitation. This vector coincides with the main diagonal of the mass [*M*] when the vector {*X*} includes only the translational displacements in u direction;

$\ddot{x}(t)$ translational ground acceleration, represented by S_e;

$\ddot{\theta}$ rotational acceleration, represented by R^{θ}.

(9) To account for the term $\{m\}\,\ddot{u}$ the participation factor in the modal analysis of mode n is:

$$a_{\mathrm{ku}}\,\frac{\sum_{\mathrm{i}} m_{\mathrm{i}}{}^{\mathrm{k}}\phi_{\mathrm{i}}}{\sum_{\mathrm{i}} m_{\mathrm{i}}{}^{\mathrm{k}}\phi_{\mathrm{i}}^{2}} \tag{A.5}$$

while, for the term $\{m\;h\}\,\ddot{\theta}$, the participation factor is:

$$a_{\mathrm{k}\theta}\,\frac{\sum_{\mathrm{i}} m_{\mathrm{i}} h_{\mathrm{i}}{}^{\mathrm{k}}\phi_{\mathrm{i}}}{\sum_{\mathrm{i}} m_{\mathrm{i}}{}^{\mathrm{k}}\phi_{\mathrm{i}}^{2}} \tag{A.6}$$

where:

$k_{\phi\mathrm{i}}$ i-th component of k-th modal vector

m_{i} i-th component of $\{m\}$

$m_i\ h_i$ i-th component of $\{m\ h\}$

(10) The effects of the two forcing functions are to be superimposed instant by instant. They are generally not in phase, and according the effects of the rotational ground excitation can be combined with the effects of the translational excitation as a square root of the sum of the squares.

ANNEX B (Informative)

Analysis procedure for damping

(1) When the design response spectrum is applied the behaviour factor q incorporates the elastic dissipation in the structure and that due the soil-to structure interaction and to the inelastic hysteretic behaviour of the structure. In those instances when the elastic spectrum is applied, the damping factor (or damping ratio relative to the critical damping), need be explicitly defined, and when the modal analysis is being performed, the damping factors need be defined for each mode of vibration. If a mode involves essentially a single structural material, than the damping ratio should conform to the material dissipation property and to the amplitude of deformation. Suggested ranges of values of damping ratios are:

	damping ratios		
steel elements	0,01	÷	0,04
concrete elements	0,02	÷	0,07
ceramic cladding	0,015	÷	0,05
brickwork lining	0,03	÷	0,1

(2) In case evidence is brought that non-structural elements contribute to energy dissipation, higher values of damping can be assumed. Due to the dependency on the amplitude of deformation, in general lower bounds of the ratios are suitable for the serviceability analysis, while upper bounds of the ratios are suitable for the ultimate state analysis.

(3) As to the energy dissipation in the soil, representative numbers for the dashpot associated with stiffness are:

swaying soil compliance	0,10	÷	0,20
Rocking soil compliance	0,07	÷	0,15
Vertical soil compliance	0,15	÷	0,20

(4) For linear footings, consistent compliance coefficients should be applied.

(5) Low dashpot values are assigned to foundations on a shallow soil deposit, over a stiff bedrock.

(6) In general, for the present structures any mode of vibration involves the deformation of more than one material. In this case, for each mode, an average modal damping based on the elastic energy of deformation stored in that mode of vibration is appropriate.

(7) The formulation leads to

$$\bar{\xi}_{\mathrm{j}} = \frac{\{\phi\}^{\mathrm{T}}[\bar{K}]\{\phi\}}{\{\phi\}^{\mathrm{T}}[K]\{\phi\}} \tag{B.1}$$

where:

$[K]$ stiffness matrix;

$\bar{\xi}_{\mathrm{j}}$ equivalent modal damping ratio of the j-th mode;

$[\bar{K}]$ modified stiffness matrix constructed by the product of the damping ratio appropriate for the element and the stiffness,

$\{\phi\}$ j-th modal vector.

(8) Other techniques can be used when more detailed data on the damping characteristics of structural subsystems are available.

(9) For each mode of vibration, the upper bound $\bar{\xi}_{\mathrm{j}} \leq 0{,}15$ is advisable, unless a suitable set of damping data are available on an experimental basis.

ANNEX C (Informative)

Soil-structure interaction

(1) The design earthquake motion is defined at the soil surface, in free-field conditions, i.e. where it is not affected by the inertial forces due to the presence of structure. When the structure is founded on soil deposits or soft media, the resulting motion at the base of the structure will differ from that at the same elevation in the free-field, due to the soil deformability. For elevated structures, the rocking compliance of the soil may be important and may significantly increase the second order effects.

(2) The modelling methods of soil-structure interaction should consider, 1) the extent of embedment, 2) the depth of the possible bedrock, 3) the layering of the soil strata, 4) the intrinsic variability of the soil moduli in any single stratum, and 5), the strain-dependence of soil properties, (shear modulus and damping)

(3) The assumption of horizontal layering is generally acceptable.

(4) Unless the soil investigation suggests a suitable range of variability for the dynamic soil moduli, the upper bound of the soil stiffness may be obtained by multiplying by 2 the entire set of the best estimate moduli, and the lower bound by multiplying the entire set by 0,5.

(5) Being strain-dependent, damping and shear moduli for each soil layer should be consistent to the effective shear strain intensity expected during the excitation. An equivalent linear method is acceptable. In this case the analysis should be performed iteratively. In each iteration the analysis is linear but the soil properties are adjusted from iteration to iteration until the computed strain are compatible with the soil properties used in the analysis. The iterative procedure can be developed on the free-field soil deposit, disregarding the presence of the structure.

(6) The effective shear strain amplitudes in any one layer, to be used to evaluate the dynamic moduli and damping in equivalent linear methods, can be taken as

$$\gamma_{eff} = 0,65\ \gamma_{max,t} \tag{C.1}$$

where $\gamma_{max,t}$ is the maximum value of the shear deformation in the soil layer, during the free-field excitation.

(7) If the finite elements modelling method for soil media is used, the criteria for determining the location of the bottom boundary and the side boundary should be justified. In general, the forcing functions to simulate the earthquake motion are applied at these boundaries. In such cases, it is required to generate an excitation system acting at boundaries such that the response motion of the soil media at the surface free field is identical to the design ground motion. The procedures and theories for generation of such excitation system should be discussed.

(8) If the half-space (lumped parameters) modelling method is used, the parameters used in the analysis for the soil deformability should account for the layering. Besides, it should consider the intrinsic variability of soil moduli, and strain-dependent properties.

(9) Any other modelling methods used for soil-structure interaction analysis is to be clearly explained, as is any basis for not including soil-structure interaction analysis.

ANNEX D (Informative)

Number of degrees of freedom and number of modes of vibration

(1) A dynamic analysis (e.g., response spectrum, power spectrum, or time history method) should be used when the use of the equivalent static load cannot be justified.

(2) The analysis should include:

- Consideration of the torsional, rocking and translational response of the foundations.
- An adequate number of masses and degrees of freedom to determine the response of any structural element and plant equipment.
- A sufficient number of modes to assure participation of all significant modes.
- Consideration of the maximum relative displacement among supports of equipment or machinery (for a chimney, the interaction between internal and external tubes).
- Significant effects such as piping interactions, externally applied structural restraints, hydrodynamic loads (both mass and stiffness effects), and possible nonlinear behaviour.
- Development of "floor response spectra", in the case of presence of important light equipment or appendices.

(3) The effective modal mass M_{i}, mentioned in 4.7.3.2, can be computed as

$$M_{\mathrm{i}} = [\ \{\phi\}^{\mathrm{T}}\ \{m\}\ \{i\}]^2\ /\ \{\phi\}^{\mathrm{T}}\ [M]\ \{\phi\} \qquad \text{(D.1)}$$

where:

$\{\phi\}$ i-th modal vector;

$\{i\}$ column vector, usually with 1 or 0 nondimensional components, which represents the displacement induced in the structure when its base is subjected to a unit staic displacement in the relevant direction.

(4) The criterion indicated in (3) does not assure the adequacy of the mass discretization, in the particular case where a light equipment or a structural appendix is concerned. In this case the above condition might be fulfilled but the mathematical model of the structure could be inadequate to represent the equipment or appendix motion.

(5) When the analysis of the equipment or appendix is necessary, a "floor response spectrum", applicable for the floor elevation where the equipment/appendix is located, can be developed. This approach is also advisable when a portion of the structure need to be analysed independently, for instance, an internal masonry tube of a chimney, supported at individual brackets inserted in the main shaft.

ANNEX E (Informative)

MASONRY CHIMNEYS

E.1 General

(1) A masonry chimney is a chimney constructed of concrete blocks, or masonry, hereinafter referred to as masonry. Masonry chimneys should be constructed, anchored, supported and reinforced as required in this chapter.

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 at least 150 mm beyond the face of the foundation or support wall on all sides. Footings should be founded on natural undisturbed earth or engineered fill below frost depth. In areas not subjected to freezing, footings should be at least 300 mm below finished grade.

E.3 Behaviour factor

(1) Masonry chimneys should be constructed, anchored, supported and reinforced as required in order to fulfil the requirement of the present code, by assuming a behaviour factor q_0 = 1,5.

E.4 Minimum vertical reinforcing

(1) For chimneys up to one meter wide, four Φ 12 continuous vertical bars anchored in the foundation should be placed in the concrete, between wythes of solid masonry or within the cells of hollow unit masonry and grouted. Grout should be prevented from bonding with the flue liner so that the flue liner is free to move with thermal expansion. For chimneys greater than one meter wide, two additional Φ 12 vertical bars should be provided for each additional meter in width or fraction thereof.

E.5 Minimum horizontal reinforcing

(1) Vertical reinforcement should be enclosed within 6 mm ties, or other reinforcing of equivalent net cross-sectional area, spaced not to exceed 400 mm on centre, or placed in the bed joints of unit masonry, at a minimum of every 400 mm of vertical height. Two such ties should be provided at each bend in the vertical bars.

E.6 Minimum seismic anchorage

(1) Masonry chimneys and foundations should be anchored at each floor, ceiling or roof line more than two meters above grade, except where constructed completely within the exterior walls. Two 5 mm × 25 mm straps should be embedded a minimum of 300 mm into the chimney. Straps should be hooked around the outer bars and extend 150 mm beyond the bend. Each strap should be fastened to a minimum of four floor joists with two 12-mm bolts.

E.7 Corbeling

(1) Masonry chimneys should not be corbeled more than half of the chimney's wall thickness from a wall or foundation, nor should a chimney be corbeled from a wall or foundation that is less than 300 mm in thickness unless it projects equally on each side of the wall, except that on the second story of a two-story dwelling, corbeling of chimneys on the exterior of the enclosing walls is permitted to equal the wall thickness. The projection of a single course should not exceed one-half the unit height or one-third of the unit bed depth, whichever is less.

E.8 Changes in dimension

(1) The chimney wall or chimney flue lining should not change in size or shape within 150 mm above or below where the chimney passes through floor components, ceiling components or roof components.

E.9 Offsets

(1) Where a masonry chimney is constructed with a fireclay flue liner surrounded by one wythe of masonry, the maximum offset should be such that the centerline of the flue above the offset does not extend beyond the centre of the chimney wall below the offset. Where the chimney offset is supported by masonry below the offset in an approved manner, the maximum offset limitations should not apply.

E.10 Additional load

(1) Chimneys should not support loads other than their own weight unless they are designed and constructed to support the additional load. Masonry chimneys are permitted to be constructed as part of the masonry walls or concrete walls of the building.

E.11 Wall thickness

(1) Masonry chimney walls should be constructed of concrete blocks, solid masonry units, or hollow masonry units grouted solid with not less than 100 mm nominal thickness.