DRAFT FOR DEVELOPMENT

DD ENV 1998-1-2 : 1996

Eurocode 8:

Design provisions for earthquake resistance of structures

Part 1.2 General rules — General rules for buildings

ICS 91.120.20

Committees responsible for this Draft for Development

The preparation of this Draft for Development was entrusted by Technical Committee B/525, Building and civil engineering structures, to Subcommittee B/525/8, Structures in seismic regions, upon which the following bodies were represented:

Association of Consulting Engineers
British Geological Survey
Department of the Environment (Building Research Establishment)
Department of the Environment (Property and Building Directorate)
Federation of Civil Engineering Contractors
Institution of Civil Engineers
Steel Construction Institute

This Draft for Development, having been prepared under the direction of the Sector Board for Building and Civil Engineering, was published under the authority of the Standards Board and comes into effect on 15 May 1996

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Contents

National foreword

This Draft for Development has been prepared by Subcommittee B/525/8 and is the English language version of ENV 1998-1-2 : 1994 *Eurocode 8: Design provisions for earthquake resistance of structures Part 1-2: General rules — General rules for buildings* published by the European Committee for Standardization (CEN).

ENV 1998-1-2: 1994 results from a programme of work sponsored by the European Commission to make available a common set of rules for the structural and geotechnical design of buildings and civil engineering works. The full range of codes covers the basis of design and actions, the design of structures in concrete, steel, composite construction, timber, masonry and aluminium alloy, and also geotechnical and seismic design.

This publication is not to be regarded as a British Standard.

An ENV or European Prestandard is made available for provisional application, but does not have the status of a European Standard. The aim is to use the experience gained to modify the ENV so that it can be adopted as a European Standard (EN).

There is no existing British Standard equivalent to ENV 1998-1-2 and there is no requirement in the Building Regulations to consider seismic actions on buildings and civil engineering works in the UK.

ENV 1998-1-1 states that the provisions of Eurocode 8 may be simplified or not observed in regions of low seismicity. Within the UK, the application of Eurocode 8 should not be necessary, unless the client or user of the works assesses that the associated seismic risk is such that it needs to be addressed.

Users of this document are invited to comment on its technical content, ease of use and any ambiguities or anomalies, both with respect to use in the UK and also in other more actively seismic areas. These comments will be taken into account when preparing the UK national response to CEN on the question of whether the ENV can be converted into an EN.

Comments should be sent in writing to the Secretary of B/525/8 at BSI, 389 Chiswick High Road, London W4 4AL, quoting the document reference, the relevant clause and, where possible, a proposed revision, by 30 April 1997. Comments after this time will still be possible through corporate bodies, such as the engineering institutions.

EUROPEAN PRESTANDARD
PRÉNORME EUROPÉENNE
EUROPÄISCHE VORNORM

ENV 1998-1-2

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ICS 91.120.20

Descriptors: Civil engineering, buildings, structures, earthquake resistant structures, earthquake-resistant design, computation

English version

Eurocode 8 — Design provisions for earthquake resistance of structures — Part 1-2: General rules — General rules for buildings

Eurocode 8 — Conception et dimensionnement des structures pour la résistance aux séismes — Partie 1-2: Règles générales — Règles générales pour les bâtiments

Eurocode 8 —Auslegung von Bauwerken gegen Erdbeben — Teil 1-2: Grundlagen — Allgemeine Regeln für Hochbauten

This European Prestandard (ENV) was approved by CEN on 1993-12-17 as a prospective standard for provisional application. The period of validity of this ENV is limited initially to three years. After two years the members of CEN will be requested to submit their comments, particularly on the question whether the ENV can be converted into an European Standard (EN).

CEN members are required to announce the existence of this ENV in the same way as for an EN and to make the ENV available promptly at national level in an appropriate form. It is permissible to keep conflicting national standards in force (in parallel to the ENV) until the final decision about the possible conversion of the ENV into an EN is reached.

CEN members are the national standards bodies of Austria, Belgium, Denmark, Finland, France, Germany, Greece, Iceland, Ireland, Italy, Luxembourg, Netherlands, Norway, Portugal, Spain, Sweden, Switzerland and United Kingdom.

CEN

European Committee for Standardization
Comité Européen de Normalisation
Europäisches Komitee für Normung

Central Secretariat: rue de Stassart 36, B-1050 Brussels

Ref. No. ENV 1998-1-2 : 1994 E

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FOREWORD

Objectives of the Eurocodes

(1) The "Structural Eurocodes" comprise a group of standards for the structural and geotechnical design of buildings and civil engineering works.

(2) They cover execution and control only to the extent that is necessary to indicate the quality of the construction products, and the standard of the workmanship needed to comply with the assumptions of the design rules.

(3) Until the necessary set of harmonized technical specifications for products and for the methods of testing their performance are available, some of the Structural Eurocodes cover some of these aspects in informative Annexes.

Background of the Eurocode Programme

(4) The Commission of the European Communities (CEC) initiated the work of establishing a set of harmonized technical rules for the design of building and civil engineering works which would initially serve as an alternative to the different rules in force in the various Member States and would ultimately replace them. These technical rules became known as the "Structural Eurocodes".

(5) In 1990, after consulting their respective Member States, the CEC transferred the work of further development, issue and updating of the Structural Eurocodes to CEN, and the EFTA Secretariat agreed to support the CEN work.

(6) CEN Technical Committee CEN/TC250 is responsible for all Structural Eurocodes.

Eurocode Programme

(7) Work is in hand on the following Structural Eurocodes, each generally consisting of a number of parts:

EN 1991 Eurocode 1 Basis of design and actions on structures
EN 1992 Eurocode 2 Design of concrete structures
EN 1993 Eurocode 3 Design of steel structures
EN 1994 Eurocode 4 Design of composite steel and concrete structures
EN 1995 Eurocode 5 Design of timber structures
EN 1996 Eurocode 6 Design of masonry structures
EN 1997 Eurocode 7 Geotechnical design structures
EN 1998 Eurocode 8 Design provisions for earthquake resistance of
EN 1999 Eurocode 9 Design of aluminium alloy structures

(8) Separate sub-committees have been formed by CEN/TC250 for the various Eurocodes listed above.

(9) This Part 1-2 of Eurocode 8 is being published as a European Prestandard (ENV) with an initial life of three years.

(10) This Prestandard is intended for experimental application and for the submission of comments.

(11) After approximately two years CEN members will be invited to submit formal comments to be taken into account in determining future actions.

(12) Meanwhile feedback and comments on this Prestandard should be sent to the Secretariat of CEN/TC250/SC8 at the following adress:

IPQ c/o LNEC
Avenida do Brasil 101
P - 1799 LISBOA Codex
PORTUGAL

or to your national standards organization.

National Application Documents (NAD's)

(13) In view of the responsibilities of authorities in member countries for safety, health and other matters covered by the essential requirements of the Construction Products Directive (CPD), certain safety elements in this ENV have been assigned indicative values which are identified by [] ("boxed values"). The authorities in each member country are expected to review the "boxed values" and may substitute alternative definitive values for these safety elements for use in national application.

(14) Some of the supporting European or International standards may not be available by the time this Prestandard is issued. It is therefore anticipated that a National Application Document (NAD) giving any substitute definitive values for safety elements, referencing compatible supporting standards and providing guidance on the national application of this Prestandard, will be issued by each member country or its Standards Organization.

(15) It is intended that this Prestandard is used in conjunction with the NAD valid in the country where the building or civil engineering works is located.

Matters specific to this Prestandard

(16) The scope of Eurocode 8 is defined in ENV 1998-1-1 clause 1.1.1; the scope of this Part of Eurocode 8 is defined in clause 1.1.1. Additional Parts of Eurocode 8 which are planned are indicated in ENV 1998-1-1 clause 1.1.3.

(17) This Prestandard was developed from one of the Parts that was included in the Draft of Eurocode 8 dated May 1988 published by the CEC and subjected to public enquiry. Such Draft contained also Parts 1-2 and 1-3 which are now presented as separate Prestandards.

(18) As mentioned in clause 1.1.1, attention must be paid to the fact that for the design of structures in seismic regions the provisions

of Eurocode 8 are to be applied in addition to the provisions of the other relevant Eurocodes.

(19) In using this Prestandard in practice, particular regard should be paid to the underlying assumptions given in clause 1.3 of Part 1-1.

(20) This Prestandard includes three Appendices which develop some aspects of clauses presented in the main part of the text. They are useful for the conceptual design of buildings and for the analysis of specific cases which allow some simplifications.

1 GENERAL

1.1 Scope

(1)P Part 1-2 is concerned with buildings. It contains general rules for the earthquake-resistant design of buildings and shall be used in conjunction with Parts 1-1 and 1-3.

(2)P Whereas guidance on base-isolated buildings is not given in the code, the use of base-isolation is not precluded, provided special studies are undertaken.

1.2 Symbols

In addition to the symbols listed in Part 1-1, the following symbols are used in Part 1-2 with the following meanings:

E_E	effect of the seismic action
E_{Edx}, E_{Edy}	design values of the action effects due to the horizontal components of the seismic action;
E_{Edz}	design value of the action effects due to the vertical component of the seismic action;
F	horizontal seismic force;
F_a	horizontal seismic force acting on a non-structural element (appendage);
H	building height;
R_d	design resistance;
T_1	fundamental vibration-period of a building;
T_a	fundamental vibration-period of a non-structural element (appendage);
W	weight;
W_a	weight of a non-structural element (appendage);
d	displacement;
d_r	design interstorey drift;
e_1	accidental eccentricity of a storey mass from its nominal location;
h	interstorey height;

m	mass;
q_a	behaviour factor of a non-structural element;
q_d	displacement behaviour factor;
s	displacement of a mass m in the fundamental mode shape of a building;
z	height of the mass m above the level of application of the seismic action;
γ_a	importance factor of a non-structural element;
θ	interstorey drift sensitivity coefficient.

2 CHARACTERISTICS OF EARTHQUAKE RESISTANT BUILDINGS

2.1 Basic principles of conceptual design

(1)P The aspect of seismic hazard shall be taken into consideration in the early stages of the conceptual design of the building.

(2) The guiding principles governing this conceptual design against seismic hazard are

- structural simplicity,
- uniformity and symmetry,
- redundancy,
- bidirectional resistance and stiffness,
- torsional resistance and stiffness,
- diaphragmatic action at storey level,
- adequate foundation.

(3) Commentaries to these principles are given in Annex B.

2.2 Structural regularity

2.2.1 General

(1)P For the purpose of seismic design, building structures are distinguished as regular and non-regular.

(2) This distinction has implications on the following aspects of the seismic design:

- the structural model, which can be either a simplified planar or a spatial one,

- the method of analysis, which can be either a simplified modal or a multi-modal one,

- the value of the behaviour factor q, which can be decreased depending on the type of non-regularity in elevation, i.e.

  - geometric non-regularity exceeding the limits given in 2.2.3.(4),

  - non-regular distribution of overstrength in elevation exceeding the limits given in 2.2.3.(3).

(3)P With regard to the implications of structural regularity on the design, separate consideration is given to the regularity characteristics of the building in plan and in elevation, according to table 2.1.

Table 2.1: Consequences of structural regularity on seismic design

REGULARITY		ALLOWED SIMPLIFICATION		BEHAVIOUR
PLAN	ELEVATION	MODEL	ANALYSIS	FACTOR
YES	YES	PLANAR	SIMPLIFIED*	REFERENCE
YES	NO	PLANAR	MULTIMODAL	DECREASED
NO	YES	SPATIAL**	MULTIMODAL**	REFERENCE
NO	NO	SPATIAL	MULTIMODAL	DECREASED

* If the condition of 3.3.2.1.(2)b) is also met.

**Under the specific conditions given in clause A1 of Annex A simpler models and methods of analysis, described in Annex A, may be used.

(4) Criteria describing regularity in plan and in elevation are given in 2.2.2 and 2.2.3; rules concerning modelling and analysis are given in 3; the relevant behaviour factors are given in Part 1-3.

(5)P The regularity criteria given in 2.2.2 and 2.2.3 are to be considered as necessary conditions. The designer shall verify that the assumed regularity of the building structure is not impaired by other characteristics, not included in these criteria.

2.2.2 Criteria for regularity in plan

(1) The building structure is approximately symmetrical in plan with respect to two orthogonal directions, in what concerns lateral stiffness and mass distribution.

(2) The plan configuration is compact, i.e. it does not present divided shapes as H, I, X, etc. The total dimension of re-entrant corners or recesses in one direction does not exceed 25 % of the overall external plan dimension of the building in the corresponding direction.

(3) The in-plane stiffness of the floors is sufficiently large in comparison with the lateral stiffness of the vertical structural elements, so that the deformation of the floor has a small effect on the distribution of the forces among the vertical structural elements.

(4) Under the seismic force distribution given in 3.3.2.3, applied with the accidental eccentricity given in 3.2, at any storey the maximum displacement in the direction of the seismic forces does not exceed the average storey displacement more than 20 %.

2.2.3 Criteria for regularity in elevation

(1) All lateral load resisting systems, like cores, structural walls or frames, run without interruption from their foundations to the top of the building or, when setbacks at different heights are present, to the top of the relevant zone of the building.

(2) Both the lateral stiffness and the mass of the individual storeys remain constant or reduce gradually, without abrupt changes, from the base to the top.

(3) In framed buildings the ratio of the actual storey resistance to the resistance required by the analysis should not vary disproportionately between adjacent storeys. Within this context the special aspects of masonry infilled concrete frames are treated in clause 2.9 of Part 1-3.

(4) When setbacks are present, the following additional provisions apply:

a) in case of gradual setbacks preserving axial symmetry, the setback at any floor is not greater than 20 % of the previous plan dimension in the direction of the setback (see fig. 2.1.a and 2.1.b),

b) in case of a single setback within the lower 15 % of the total height of the main structural system, the setback is not greater than 50 % of the previous plan dimension (see fig. 2.1.c). In that case the structure of the base zone within the vertically projected perimeter of the upper stories shall be designed to resist at least 75 % of the horizontal shear forces that would develop in that zone in a similar building without the base enlargement.

c) in case the setbacks do not preserve symmetry, in each face the sum of the setbacks at all storeys is not greater than 30 % of the plan dimension at the first storey, and the individual setbacks are not greater than 10 % of the previous plan dimension (see fig. 2.1.d).

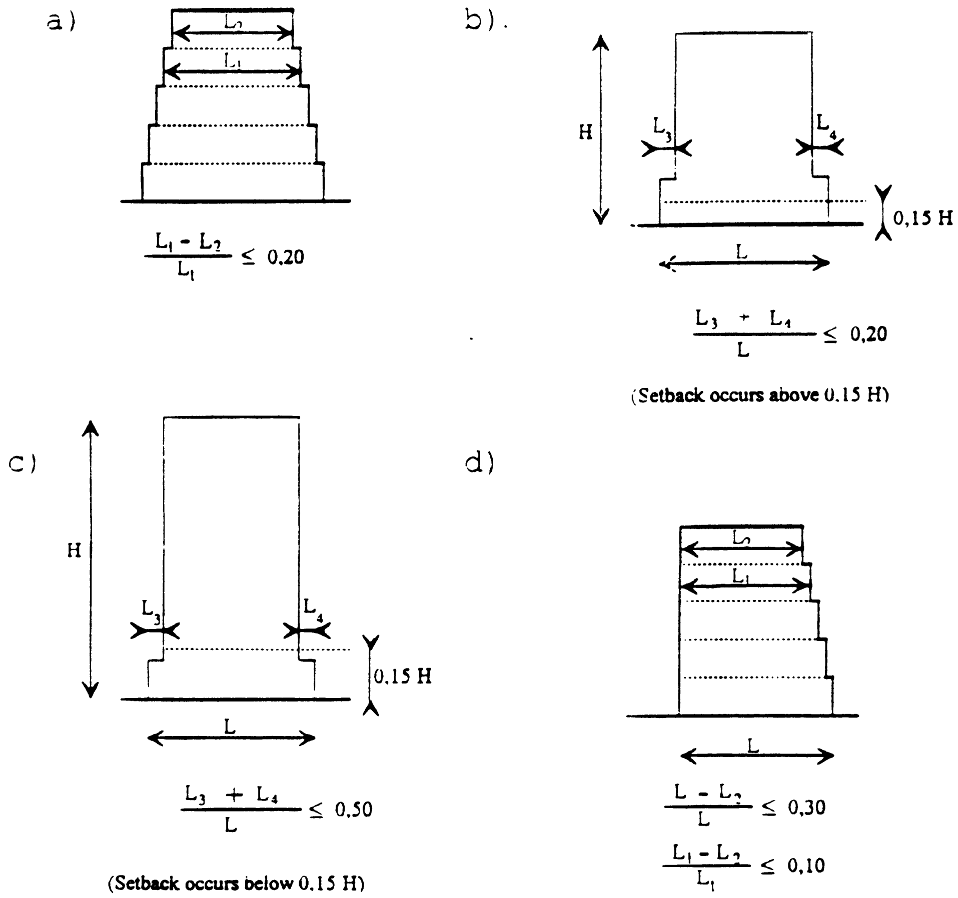

Figure 2.1: Criteria for regularity of setbacks

3 STRUCTURAL ANALYSIS

3.1 Modelling

(1)P The model of the building shall adequately represent the distribution of stiffness and mass so that all significant deformation shapes and inertia forces are properly accounted for under the seismic action considered[1].

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[1]The model should also account for the contribution of joint regions to the deformability of the building, e.g. the end zones in beams or columns of frame type structures. Non-structural elements, which may influence the response of the main resisting structural system, should also be accounted for.

(2) In general the structure may be considered to consist of a number of vertical and lateral load resisting systems, connected by horizontal diaphragms.

(3) When the floor diaphragms of the building are sufficiently rigid in their plane, the masses and the moments of inertia of each floor may be lumped at the center of gravity, thus reducing the degrees of freedom to three per floor (two horizontal displacements and a rotation about the vertical axis).

(4) For buildings complying with the criteria for regularity in plan (see 2.2.2) or with the regularity criteria given in clause A1 of Annex A, the analysis can be performed using two planar models, one for each main direction.

(5) In reinforced concrete buildings and in masonry buildings the stiffness of the load bearing elements should, in general, be evaluated assuming uncracked sections[2].

(6) Infill walls which increase significantly the lateral stiffness of the building should be taken into account, see clause 2.9 of Part 1-3 for masonry infills of concrete frames.

(7)P The deformability of the foundation soil shall be considered in the model whenever it may have an adverse influence on the structural response.

(8)P The masses shall be calculated from the gravity loads appearing in the combination of actions given in 4.4.(2) of Part 1-1. The combination coefficients ψ_{Ei} are given in 3.6.(2).

3.2 Accidental torsional effects

(1)P In addition to the actual eccentricity, in order to cover uncertainties in the location of masses and in the spatial variation of the seismic motion, the calculated center of mass at each floor i shall be considered displaced from its nominal location in each direction by an additional accidental eccentricity:

$$e_{1i} = \pm 0{,}05 \cdot L_i \tag{3.1}$$

where

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[2] In reinforced concrete buildings this assumption may lead to unconservative estimates of the displacements, especially when high values of the behaviour factor q are assumed. In such cases and if displacements are critical, a more accurate estimation of the stiffness of the elements under the seismic action may be necessary with regard to the displacement analysis according to 3.4..

e_{1i} accidental eccentricity of storey mass i from its nominal location, applied in the same direction at all floors,

L_i floor-dimension perpendicular to the direction of the seismic action.

3.3 Methods of analysis

3.3.1 General

(1)P Within the scope of Part 1-2, the seismic effects and the other action effects to be considered according to the combination rules given in clause 4.4 of Part 1-1 may be determined on the basis of a linear-elastic behaviour of the structure.

(2)P The reference method for determining the seismic effects is the modal response analysis, using a linear-elastic model of the structure and the design spectrum given in clause 4.2.4 of Part 1-1.

(3) Depending on the structural characteristics of the building one of the following two types of analysis may be used:

- the "simplified modal response spectrum analysis" for buildings meeting the conditions given in 3.3.2,
- the "multi-modal response spectrum analysis", which is applicable to all types of buildings (see 3.3.3).

(4) As alternatives to these basic methods other methods of structural analysis, such as

- power spectrum analysis,
- (non-linear) time history analysis,
- frequency domain analysis,

are allowed under the conditions specified in paragraphs (5) and (6)P and in 3.3.4.

(5) Non-linear analyses can be used, provided 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.

(6)P If a non-linear analysis is used, the amplitudes of the accelerograms derived for the reference return period (see clause 4.3.2 of Part 1-1) shall be multiplied by the importance factor γ_I of the building (see 3.7).

3.3.2 Simplified modal response spectrum analysis

3.3.2.1 General

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

(2) These requirements are deemed to be satisfied by buildings which

a1) meet the criteria for regularity in plan and in elevation given in 2.2.2 and 2.2.3

or

a2) meet the criteria for regularity in elevation given in 2.2.3 and the regularity criteria given in clause A1 of Annex A

and

b) have fundamental periods of vibration T_1 in the two main directions less than the following values

$$T_1 \leq \begin{cases} 4 \cdot T_C \\ 2{,}0 \text{ s} \end{cases} \tag{3.2}$$

where T_C is given in table 4.1 of Part 1-1.

3.3.2.2 Base shear force

(1)P The seismic base shear force F_b for each main direction is determined as follows:

$$F_b = S_d(T_1) \cdot W \tag{3.3}$$

where

$S_d(T_1)$ ordinate of the design spectrum (see clause 4.2.4 of Part 1-1) at period T_1,

T_1 fundamental period of vibration of the building for translational motion in the direction considered,

W total weight of the building computed in accordance with 3.1.(8).

(2) For the purpose of determining the fundamental vibration periods T_1 of both planar models of the building, approximate expressions based on methods of structural dynamics (e.g. by Rayleigh method) may be used[3].

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[3]For preliminary design purposes the approximate expressions for T_1 given in Annex C may be used.

3.3.2.3 Distribution of the horizontal seismic forces

(1)P The fundamental mode shapes of both planar models of the building may be calculated using methods of structural dynamics or may be approximated by horizontal displacements increasing linearly along the height of the building.

(2)P The seismic action effects shall be determined by applying, to the two planar models, horizontal forces F_i to all storey masses m_i.

(3)P The forces shall be determined by assuming the entire mass of the structure as a substitute mass of the fundamental mode of vibration, hence:

$$F_i = F_b \cdot \frac{s_i \cdot W_i}{\Sigma s_j \cdot W_j} \qquad (3.4)$$

where

F_i horizontal force acting on storey i,

F_b seismic base shear according to exp.(3.3);

s_i, s_j displacements of masses m_i, m_j in the fundamental mode shape,

W_i, W_j weights of masses m_i, m_j computed according to 3.1.(8).

(4) When the fundamental mode shape is approximated by horizontal displacements increasing linearly along the height, the horizontal forces F_i are given by:

$$F_i = F_b \cdot \frac{z_i \cdot W_i}{\Sigma z_j \cdot W_j} \qquad (3.5)$$

where

z_i, z_j heights of the masses m_i, m_j above the level of application of the seismic action (foundation).

(5)P The horizontal forces F_i determined in the above manners shall be distributed to the lateral load resisting system assuming rigid floors.

3.3.2.4 Torsional effects

(1) In case of symmetric distribution of lateral stiffness and mass and if no more exact method is applied regarding 3.2, the accidental torsional effects may be accounted for by amplifying the action

effects in the individual load resisting elements - evaluated according to 3.3.2.3.(5) - with a factor δ given by:

$$\delta = 1 + 0{,}6 \cdot \frac{x}{L_e} \tag{3.6}$$

where

x distance of the element under consideration from the centre of the building measured perpendicularly to the direction of the seismic action considered,

L_e distance between the two outermost lateral load resisting elements measured as previously.

(2) Whenever the conditions given in clause A1 of Annex A are met, the approximate analysis of torsional effects as described in Annex A can be applied.

3.3.3 Multi-modal response spectrum analysis

3.3.3.1 General

(1)P This type of analysis shall be applied to buildings which do not satisfy the conditions given in 3.3.2.1.(2) for applying the simplified modal response spectrum analysis.

(2) For buildings complying with the criteria for regularity in plan (see 2.2.2) or with the regularity criteria given in Clause A1 of Annex A, the analysis can be performed using two planar models, one for each main direction.

(3)P Buildings not complying with these criteria shall be analysed using a spatial model.

(4)P Whenever a spatial model is used, the design seismic action shall be applied along all relevant horizontal directions (with regard to the structural layout of the building) and their orthogonal horizontal axes. For buildings with resisting elements in two perpendicular directions these two directions are considered as the relevant ones.

(5)P The response of all modes of vibration contributing significantly to the global response shall be taken into account.

(6) Paragraph (5) may be satisfied by either of the following:

- By demonstrating that the sum of the effective modal masses for the modes considered amounts to at least 90% of the total mass of the structure.

- By demonstrating that all modes with effective modal masses greater than 5 % of the total mass are considered.

NOTE: The effective modal mass m_k, corresponding to a mode k, is determined so that the base shear force F_{bk}, acting in the direction of application of the seismic action, may be expressed as $F_{bk} = S_d(T_k) \cdot m_k \cdot g$. It can be shown that the sum of the effective modal masses (for all modes and a given direction) is equal to the mass of the structure.

(7) When using a spatial model, the above conditions have to be verified for each relevant direction.

(8) If paragraph (6) cannot be satisfied (e.g. in buildings with a significant contribution from torsional modes), the minimum number k of modes to be considered in a spatial analysis should satisfy the following conditions:

$$k \geq 3 \cdot \sqrt{n} \tag{3.7}$$

and

$$T_k \leq 0,20 \text{ s} \tag{3.8}$$

where

k number of modes considered,

n number of storeys above ground,

T_k period of vibration of mode k.

3.3.3.2 Combination of modal responses

(1)P The response in two vibration modes i and j (including both translational and torsional modes) may be considered as independent of each other when their periods T_i and T_j satisfy the following condition:

$$T_j \leq 0,9 \cdot T_i \tag{3.9}$$

(2) Whenever all relevant modal responses (see 3.3.3.1.(5)-(8)) can be regarded as independent of each other, the maximum value E_E of a seismic action effect may be taken as

$$E_E = \sqrt{\Sigma E_{Ei}^2} \tag{3.10}$$

where

E_E seismic action effect under consideration (force, displacement, etc.),

E_{Ei} value of this seismic action effect due to the vibration mode i.

(3)P If paragraph (1)P is not satisfied, more accurate procedures for the combination of the modal maxima (e.g. the "Complete Quadratic Combination") shall be adopted.

3.3.3.3 Torsional effects

(1) Whenever a spatial model is used for the analysis, the accidental torsional effects referred in 3.2 may be determined as the envelope of the effects resulting from an analysis for static loadings, consisting of torsional moments M_{1i} about the vertical axis of each storey i:

$$M_{1i} = e_{1i} \cdot F_i \tag{3.11}$$

where

- M_{1i} torsional moment of storey i about its vertical axis,
- e_{1i} accidental eccentricity of storey mass i according to eq.(3.1) for all relevant directions, see 3.3.3.1.(4),
- F_i horizontal force acting on storey i, as derived in 3.3.2.3 for all relevant directions.

(2) The effects of the loading according to paragraph (1) should be considered with alternate signs (the same for all storeys).

(3) Whenever two separate planar models are used for the analysis, the torsional effects may be accounted for by applying the rules of 3.3.2.4.(1) or of Annex A to the action effects computed according to 3.3.3.2.

3.3.4 Alternative methods of analysis

3.3.4.1 General

(1)P If the alternative methods of analysis described below are used, it shall be demonstrated, that the fundamental requirements according to clause 2.1 of Part 1-1 are met with a level of reliability commensurate with the use of the reference method described in 3.3.3.

(2) Paragraph (1)P may be satisfied by either of the following:

a) By demonstrating that the sum of the computed horizontal shear forces at all supports in each of two orthogonal directions is not less than 80 % of the corresponding sums obtained by multimodal analysis according to 3.3.3.

b) Where the sum in either direction is less than 80 % of the value from multi-modal analysis, the computed values of all response variables shall be scaled proportionately by the scale factor required to bring the base shear force to the value needed for satisfying the condition a).

3.3.4.2 Power spectrum analysis

(1) A linear stochastic analysis of the structure can be performed, either by using modal analysis or frequency dependent response matrices, using as input the acceleration power spectrum, defined in clause 4.3.1 of Part 1-1.

(2)P The elastic action effects shall be defined as the 50%-fractile of the probability distribution of the peak response in a time interval equal to the assumed duration of the motion.

(3)P The design values shall be determined by dividing these elastic effects by the ratio of the ordinate of the elastic response spectrum to the ordinate of the design spectrum corresponding to the fundamental period of the building, multiplied by g.

3.3.4.3 Time-history analysis

(1) The time dependent response of the structure can be obtained through direct numerical integration of its differential equations of motion, using the accelerograms, defined in clause 4.3.2 of Part 1-1 to represent the ground motions.

(2) When the structure is considered to behave non-linearly, the provisions of 3.3.1.(5)-(6)P apply.

3.3.4.4 Frequency domain analysis

(1)P The seismic action input is the same as in 3.3.4.3, but with each accelerogram cast in the form of a Fourier summation. The response is obtained by convolving over the frequency domain the harmonic components of the input with their respective frequency response matrices or functions.

(2)P The elastic action effects shall be defined as the mean values of the peak responses calculated for the various accelerograms.

(3)P The design values shall be determined by dividing the elastic effects by the ratio of the ordinate of the elastic response spectrum to the ordinate of the design spectrum corresponding to the fundamental period of the building, multiplied by g.

3.3.5 Combination of the components of the seismic action

3.3.5.1 Horizontal components of the seismic action

(1)P In general the horizontal components of the seismic action (see clause 4.2.1.(2) of Part 1-1) shall be considered as acting simultaneously.

(2) The combination of the horizontal components of the seismic action may be accounted for as follows:

- The structural response to each horizontal component shall be evaluated separately, using the combination rules for modal responses as given in 3.3.3.2.
- The maximum value of each action effect on the structure due to the two horizontal components of the seismic action may then be estimated by the square root of the sum of the squared responses to each horizontal component.

(3) As an alternative to paragraph (2) the action effects due to the combination of the horizontal components of the seismic action may be computed using the two following combinations:

a) E_{Edx} "+" $0{,}30 \cdot E_{Edy}$

b) $0{,}30 \cdot E_{Edx}$ "+" E_{Edy}

where

"+" implies "to be combined with",

E_{Edx} action effects due to the application of the seismic action along the chosen horizontal axis x of the structure,

E_{Edy} action effects due to the application of the same seismic action along the orthogonal horizontal axis y of the structure.

(4) The sign of each component in the above combinations shall be taken as the most unfavourable for the effect under consideration.

(5)P For buildings satisfying the regularity criteria in plan and in which walls are the only horizontal load resisting components, the seismic action may be assumed to act separately along the two main orthogonal horizontal axes of the structure.

(6)P When using time-history analysis according to 3.3.4.3 and employing a spatial model of the structure, simultaneously acting accelerograms shall be considered for both horizontal components.

3.3.5.2 Vertical component of the seismic action

(1)P The vertical component of the seismic action, as defined in clause 4.2.1.(3) of Part 1-1, shall be taken into account in the following cases:

- Horizontal or nearly horizontal structural members spanning 20 meters or more;
- Horizontal or nearly horizontal cantilever components;
- Horizontal or nearly horizontal prestressed components;
- Beams supporting columns.

(2) In general, the analysis for determining the effects of the vertical component of the seismic action can be made based on a partial model of the structure which includes the elements under consideration and takes into account the stiffness of the adjacent elements.

(3) The effects of the vertical component need only be considered for the elements under consideration and their directly associated supporting elements or substructures.

(4) In case the horizontal components of the seismic action are also relevant for these elements, the following three combinations may be used for the computation of the action effects:

a) $0{,}30 \cdot E_{Edx}$ "+" $0{,}30 \cdot E_{Edy}$ "+" E_{Edz}

b) E_{Edx} "+" $0{,}30 \cdot E_{Edy}$ "+" $0{,}30 \cdot E_{Edz}$

c) $0{,}30 \cdot E_{Edx}$ "+" E_{Edy} "+" $0{,}30 \cdot E_{Edz}$

where

E_{Edx} see 3.3.5.1.(3),

E_{Edy} see 3.3.5.1.(3),

E_{Edz} action effects due to the application of the vertical component of the design seismic action as defined in clause 4.2.1.(3) of Part 1-1.

3.4 Displacement analysis

(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:

$$d_s = q_d \cdot d_e \cdot \gamma_I \qquad (3.12)$$

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 unless otherwise specified in Part 1-3,

d_e displacement of the same point of the structural system, as determined by a linear analysis based on the design response spectrum according to clause 4.2.4 of Part 1-1,

γ_I importance factor (see 3.7).

(2)P When determining the displacements d_e, the torsional effects of the seismic action shall be taken into account.

3.5 Non-structural elements

3.5.1 General

(1)P Non-structural elements (appendages) of buildings (e.g. parapets, gables antennae, mechanical appendages and equipment, curtain walls, partitions, railings) that might, in case of failure, cause risks to persons or affect the building main structure or services of critical facilities, shall - together with their supports - be verified, to resist the design seismic action.

(2)P In the case of non-structural elements of great importance or of a particularly dangerous nature, the seismic analysis shall be based on a realistic modelling of the relevant structures and on the use of appropriate response spectra derived from the response of the supporting structural elements of the main seismic resisting system.

(3)P In all other cases properly justified simplifications of this procedure (e.g. as given in 3.5.2.(2)) are allowed.

3.5.2 Analysis

(1)P The non-structural elements as well as their connections and attachments or anchorages shall be verified to withstand the combination of the relevant permanent, variable and seismic actions (see clause 4.4 of Part 1-1).

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

$$F_a = (S_a \cdot W_a \cdot \gamma_a)/q_a \qquad (3.13)$$

where

F_a horizontal seismic force, acting at the center mass of the non-structural element in the most unfavourable direction,

W_a weight of the element,

S_a seismic coefficient pertinent to non-structural elements, see paragraph (3),

γ_a importance factor of the element, see 3.5.3,

q_a behaviour factor of the element, see table 3.1.

(3) The seismic coefficient S_a may be calculated as follows:

$$S_a = \alpha \cdot 3 \cdot (1 + Z/H)/(1 + (1 - T_a/T_1)^2) \qquad (3.14)$$

where

α ratio of the design ground acceleration a_g to the acceleration of gravity g,

T_a fundamental vibration period of the non-structural element,

T_1 fundamental vibration period of the building in the relevant direction,

Z height of the non-structural element above the base of the building,

H total height of the building.

3.5.3 Importance factors and behaviour factors

(1)P For the following non-structural elements the importance factor γ_a shall not be chosen less than 1,5:

- Anchorage of machinery and equipment required for life safety systems.
- Tanks and vessels containing toxic or explosive substances considered to be hazardous to the safety of the general public.

(2) In all other cases the importance factor γ_a of a non-structural element can be assumed to have the same value as the importance factor γ_I of the building concerned.

(3) Values of the behaviour factor q_a for non-structural elements are given in table 3.1.

Table 3.1: Values of q_a for non-structural elements

Type of non-structural element	q_a
- Cantilevering parapets or ornamentations - Signs and billboards - Chimneys, masts and tanks on legs acting as unbraced cantilevers along more than one half of their total height	1,0
- Exterior and interior walls - Chimneys, masts and tanks on legs acting as unbraced cantilevers along less than one half of their total height, or braced or guyed to the structure at or above their center of mass - Anchorage for permanent floor supported cabinets and bookstacks - Anchorage for false (suspended) ceilings and light fixtures	2,0

3.6 Combination coefficients for variable actions

(1)P The combination coefficients ψ_{2i} appearing in clause 4.4 of Part 1-1 are given in Part 1 of Eurocode 1.

(2)P The combination coefficients ψ_{Ei} introduced in clause 4.4 of Part 1-1 for calculating the effects of the seismic actions shall be calculated from the following expression:

$$\psi_{Ei} = \varphi \cdot \psi_{2i} \qquad (3.15)$$

where the values of φ shall be obtained from table 3.2.

Table 3.2: Values of φ for calculating ψ_{Ei}

Type of variable action	Occupation of storeys		φ
<u>Categories A-C</u>*	storeys independently occupied	top storey other storeys	[1,0] [0,5]
<u>Categories A-C</u>*	some storeys having correlated occupancies	top storey storeys with correlated occupancies other storeys	[1,0] [0,8] [0,5]
<u>Categories D-F</u>* <u>Archives</u>			[1,0]

* Categories as defined in Part 1 of Eurocode 1

3.7 Importance categories and importance factors

(1)P Buildings are generally classified into 4 importance categories which depend on the size of the building, on its value and importance for the public safety and on the possibility of human losses in case of a collapse.

(2)P The importance categories are characterized by different importance factors γ_I as described in clause 2.1 of Part 1-1.

(3) The importance factor γ_I = 1,0 is associated with a design seismic event having a reference return period as indicated in clause 4.1.(3) of Part 1-1.

(4) The definitions of the importance categories and the related importance factors are given in table 3.3.

Table 3.3: Importance categories and importance factors for buildings

Importance category	Buildings	Importance factor γ_I
I	Buildings whose integrity during earthquakes is of vital importance for civil protection, e.g. hospitals, fire stations power plants, etc.	[1,4]
II	Buildings whose seismic resistance is of importance in view of the consequences associated with a collapse, e.g. schools, assembly halls, cult. institutions etc.	[1,2]
III	Ordinary buildings, not belonging to the other categories	[1,0]
IV	Buildings of minor importance for public safety, e.g. agricultural buildings, etc.	[0,8]

(5) Different values of γ_I may be required for the various seismic zones of a country.

4 SAFETY VERIFICATIONS

4.1 General

(1)P For the safety verifications the relevant limit states (see 4.2 and 4.3) and specific measures (see clause 2.2.4 of Part 1-1) shall be considered.

(2) For buildings of importance categories II - IV (see table 3.3) the verifications prescribed in 4.2 and 4.3 may be considered satisfied, if the following two conditions are met:

a) The total base shear due to the seismic design combination (see clause 4.4 of Part 1-1), calculated with a behaviour factor q = [1,0], is less than that due to the other relevant action combinations for which the building is designed on the basis of a linear elastic analysis.

b) The specific measures described in clause 2.2.4 of Part 1-1 are taken, with the exception, that the provisions contained in clause 2.2.4.1.(2)-(3) of Part 1-1 need not be demonstrated as having been met.

4.2 Ultimate limit state

4.2.1 General

(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, equilibrium, foundation stability and seismic joints are met.

4.2.2 Resistance condition

(1)P The following relation shall be satisfied for all structural elements - including connections - and the relevant non-structural elements (see 3.5.1.(1)):

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

where

$E_d = E\{\Sigma G_{kj},\ \gamma_I \cdot A_{Ed},\ P_k,\ \Sigma\psi_{2i} \cdot Q_{ki}\}$

design value of the action effect, due to the seismic design situation (see Clause 4.4 of Part 1-1), including - if necessary - second order effects (see (2)),

$R_d = R\{f_k/\gamma_M\}$

corresponding design resistance of the element, calculated according to the rules specific to the pertinent material (characteristic value of property f_k and partial safety factor γ_M) and according to the mechanical models which relate to the specific type of structural system, as given in Part 1-3 and in the relevant Eurocodes.

(2) Second order effects (P-Δ-effects) need not be considered when the following condition is fulfilled in all storeys:

$$\theta = \frac{P_{tot} \cdot d_r}{V_{tot} \cdot h} \leq 0{,}10 \qquad (4.2)$$

where

θ interstorey drift sensitivity coefficient,

P_{tot} total gravity load at and above the storey considered, in accordance with the assumptions made for the computation of the seismic action effects,

d_r design interstorey drift, evaluated as the difference of the average lateral displacements at the top and bottom of the storey under consideration and calculated according to 3.4,

V_{tot} total seismic storey shear,

h interstorey height.

(3) In cases when $0{,}1 < \theta \leq 0{,}2$, the second order effects can approximately be taken into account by increasing the relevant seismic action effects by a factor equal to $1/(1 - \theta)$.

(4)P The value of the coefficient θ shall not exceed 0,3.

4.2.3 Ductility condition

(1)P It shall be verified that both the structural elements and the structure as a whole possess adequate ductility taking into account the expected exploitation of ductility, which depends on the selected system and the behaviour factor.

(2)P Specific material related requirements as defined in Part 1-3 shall be satisfied, including - when indicated - capacity design provisions in order to obtain the hierarchy of resistance of the various structural components necessary for ensuring the intended configuration of plastic hinges and for avoiding brittle failure modes.

(3) Capacity design rules are presented in detail in Part 1-3.

4.2.4 Equilibrium condition

(1)P The building structure shall be stable under the set of actions given by the combination rules of clause 4.4 of Part 1-1. Herein are included such effects as overturning and sliding.

(2) In special cases the equilibrium may be verified by means of energy balance methods or by geometrically non-linear methods with the seismic action defined as described in clause 4.3.2 of Part 1-1 (see also 3.3.1.(5)-(6)).

4.2.5 Resistance of horizontal diaphragms

(1)P Diaphragms and bracings in horizontal planes shall be able to transmit with sufficient overstrength the effects of the design seismic action to the various lateral load resisting systems to which they are connected.

(2) Paragraph (1) is considered satisfied if for the relevant resistance verifications the forces obtained from the analysis are multiplied by a factor equal to 1,3.

4.2.6 Resistance of foundations

(1)P The foundation system shall be verified according clause 5.4 of Part 5 and to Eurocode 7.

(2)P The action effects for the foundations shall be derived on the basis of capacity design considerations accounting for the development of possible overstrength, but they need not exceed the action effects corresponding to the response of the structure under the seismic design situation inherent to the assumption of an elastic behaviour ($q = 1,0$).

(3) If the action effects for the foundation have been determined using a behaviour factor $q \leq [1,5]$, no capacity design considerations according to (2)P are required.

4.2.7 Seismic joint condition

(1)P Buildings shall be protected from collisions with adjacent structures induced by earthquakes.

(2) Paragraph (1) is deemed to be satisfied if the distance from the boundary line to the potential points of impact is not less than the maximum horizontal displacement according to eq.(3.12).

(3) If the floor elevations of a building under design are the same as those of the adjacent building, the above referred distance may be reduced by a factor of [0,7].

(4) Alternatively, this separation distance is not required, if appropriate shear walls are provided on the perimeter of the building to act as collision walls ("bumpers"). At least two such walls must be placed at each side subject to pounding and must extend over the total height of the building. They must be perpendicular to the side subject to collisions and they can end on the boundary line. Then the separation distance for the rest of the building can be reduced to [4,0] cm.

4.3 Serviceability limit state

4.3.1 General

(1)P The requirement for limiting damage (serviceability limit state) is considered satisfied, if - under a seismic action having a larger probability of occurrence than the design seismic action - the inter-storey drifts are limited according to 4.3.2.

(2) Additional verifications for the serviceability limit state may be required in the case of buildings important for civil protection or containing sensitive equipment.

4.3.2 Limitation of interstorey drift

(1)P Unless otherwise specified in Part 1-3, the following limits shall be observed:

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

$$d_r/\nu \leq [0,004] \cdot h \qquad (4.3)$$

b) for buildings having non-structural elements fixed in a way as not to interfere with structural deformations:

$$d_r/\nu \leq [0,006] \cdot h \qquad (4.4)$$

where

d_r design interstorey drift as defined in 4.2.2.(2),

h storey height,

ν reduction factor to take into account the lower return period of the seismic event associated with the serviceability limit state.

(2) The reduction factor can also depend on the importance category of the building. Values of ν are given in table 4.1.

Table 4.1: Values of the reduction factor ν

Importance category	I	II	III	IV
Reduction Factor ν	[2,5]	[2,5]	[2,0]	[2,0]

(3) Different values of ν may be required for the various seismic zones of a country.

ANNEX A (NORMATIVE)

APPROXIMATE ANALYSIS OF TORSIONAL EFFECTS

A1 General

(1) For buildings not satisfying the criteria for regularity in plan given in 2.2.2 but fulfilling one of the sets of conditions given as criterion 1 in A2 and as criterion 2 in A3, the approximate analysis of torsional effects described in A4 can be used.

A2 Criterion 1

(1) The building has well distributed and relatively rigid cladding and partitions.

(2) The building height does not exceed [10] m.

(3) The building aspect ratio (height/length) in both main directions does not exceed [0,4].

A3 Criterion 2

(1) The in-plane stiffness of the floors is large enough in comparison with the lateral stiffness of the vertical structural elements, so that a rigid floor diaphragm behaviour may be assumed.

(2) The centers of lateral stiffness and of mass are each approximately located on a vertical line.

(3) Usually (2) may be regarded as met, if the following conditions are satisfied:

a) All lateral load resisting systems, like cores, structural walls or frames, run without interruption from their foundations to the top of the building.

b) The deflected shapes of the individual systems under horizontal loads do not differ too much. (This condition may be satisfied in case of frame systems and wall systems; generally is it not satisfied in case of dual systems.)

(4) If both conditions a) and b) of (3) are met, the common position of the centers of stiffness of all storeys may be calculated as the center of some quantities, proportional to a system of forces, having the distribution specified in 3.3.2.3 and producing an unit displacement at the top of the individual lateral load resisting system.

(5) In case of slender walls with prevailing flexural deformations those quantities may be the moments of inertia of the wall cross sections. If, in addition to flexural deformations, shear deformations are also relevant, this may be accounted for by equivalent moments of inertia of the cross sections.

A4 Approximate analysis

(1) The analysis can be performed using two planar models, one for each main direction. The torsional effects are determined separately by these two directions.

(2) The horizontal forces F_i shall be determined according to 3.3.2.3 or 3.3.3.2.

(3) The horizontal force F_i at storey i is, with regard to the considered direction of the seismic action, displaced from its nominal location in relation to the mass centre M by an additional eccentricity e_2 (see fig. A1), which can be be approximated as the lower of the following two values:

$$e_2 = 0,1 \cdot (L+B) \cdot \sqrt{10 \cdot e_o / L} \leq 0,1 \cdot (L+B) \qquad \text{(A.1)}$$

and

$$e_2 = \frac{1}{2 \cdot e_o} \left[l_s^2 - e_o^2 - r^2 + \sqrt{(l_s^2 + e_o^2 - r^2)^2 + 4 \cdot e_o^2 \cdot r^2} \right] \qquad \text{(A.2)}$$

where

e_2 additional eccentricity taking account of the dynamic effect of simultaneous translational and torsional vibrations,

e_o actual eccentricity between the stiffness center S and the nominal mass center M (see fig. A1),

l_s^2 $= (L^2 + B^2)/12$ (square of "radius of gyration"),

r^2 ratio of the storey torsional and lateral stiffness (square of "torsional radius"),

(4) The additional eccentricity e_2 may be neglected, if the ratio r^2 of the storey torsional and lateral stiffness exceeds the value of $5 \cdot (l_s^2 + e_o^2)$.

(5) The torsional effects may be determined as the envelope of the effects resulting from an analysis for two static loadings, consisting of torsional moments M_i due to the two eccentricities (see fig. A1):

$$M_i = F_i \cdot e_{max} = F_i \cdot (e_o + e_1 + e_2) \tag{A.3}$$

and

$$M_i = F_i \cdot e_{min} = F_i \cdot (e_o - e_1) \tag{A.4}$$

where

e_1 accidental eccentricity of storey mass according to exp.(3.1)

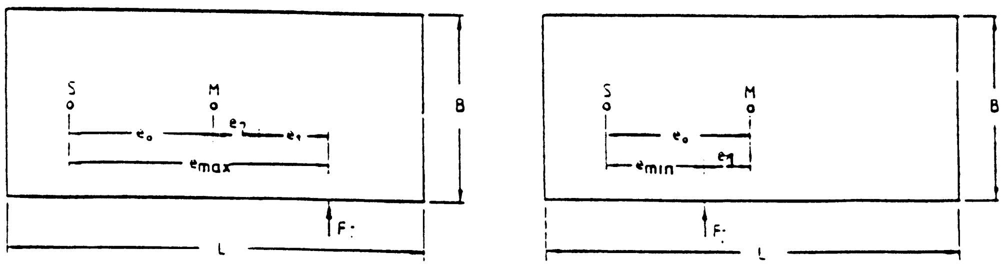

Figure A1: Determination of the eccentricities of the horizontal force F_i

ANNEX B (INFORMATIVE)

BASIC PRINCIPLES OF CONCEPTUAL DESIGN

B1 General

(1) The possible occurrence of earthquakes must be an important aspect to be accounted for in the conceptual design of a building in a seismic region.

(2) Such aspect has to be taken in consideration in the early stages of development of the building design, thus enabling the achievement of a structural system which , within acceptable costs, satisfies the fundamental requirements according to clause 2.1 of Part 1-1.

(3) To this end, the conceptual design of buildings in seismic regions should, as much as possible, reflect the considerations described in B2 - B7.

B2 Structural simplicity

(1) Structural simplicity, characterized by the existence of clear and direct paths for the transmission of the seismic forces, is an important objective to be pursued since the modelling, analysis, dimensioning, detailing and construction of simple structures are subject to much less uncertainty and thus the prediction of its seismic behaviour is much more reliable.

B3 Uniformity and symmetry

(1) Uniformity, which is somehow related to simplicity, is characterized by an even distribution of the structural elements which, when fulfilled in-plan, allows short and direct transmissions of the inertia forces created in the distributed masses of the building. If necessary, uniformity may be realized by subdividing the entire building by seismic joints into dynamically independent units.

(2) Uniformity in the development of the structure along the height of the building is also relevant, since it tends to eliminate the occurrence of sensitive zones where concentrations of stresses or large ductility demands might prematurely cause collapse.

(3) A close relationship between the distribution of masses and the distribution of resistance and stiffness naturally eliminates large eccentricities between mass and stiffness.

(4) In symmetrical or quasi-symmetrical building configurations, symmetrical structural layouts, well distributed in-plan, are thus obvious solutions for the achievement of uniformity.

(5) Finally, the use of evenly distributed structural elements increases redundancy and allows for a more favourable redistribution of action effects and widespread energy dissipation across the entire structure.

B4 Bidirectional resistance and stiffness

(1) Horizontal seismic motion is a bidirectional phenomenon and thus the building structure must be able to resist horizontal actions in any direction. Accordingly, the structural elements should be arranged in such a way as to provide such resistance which, usually, is achieved by organizing them within an orthogonal in-plan structural mesh and ensuring similar resistance and stiffness characteristics in both main directions.

(2) Furthermore, the choice of the stiffness characteristics of the structure, while attempting to minimize the effects of the seismic action (taking into account its specific features at the site) should also limit the development of excessive displacements that might lead to instabilities due to second order effects or large damages.

B5 Torsional resistance and stiffness

(1) Besides lateral resistance and stiffness, building structures must possess adequate torsional resistance and stiffness in order to limit the development of torsional motions which tend to stress, in a non-uniform way, the different structural elements. In this respect, arrangements in which the main resisting elements are distributed close to the periphery of the building present clear advantages.

B6 Diaphragm action at storey level

(1) In buildings, floors play a very important role in the overall seismic behaviour of the structure. In fact, they act as horizontal diaphragms that, not only collect and transmit the inertia forces to the vertical structural systems but also ensure that those systems act together in resisting the horizontal action.

(2) Consequently, floors are an essential part of the whole building structure and naturally its diaphragm action is especially relevant in cases of complex and non-uniform layouts of the vertical structural systems or when systems with different horizontal deformability characteristics are used together (e.g. dual systems).

(3) It is thus of the utmost importance that the floor systems be provided with adequate in-plan stiffness and resistance and with efficient connections to the vertical structural systems. In this respect, particular care should be taken in the cases of non-compact or very elongated in-plan shapes and in the case of existence of large floor openings, especially if the latter are located in the vicinity of the main vertical structural elements thus hindering such efficient connection.

B7 Adequate foundation

(1) With regard to the seismic action the design and construction of the foundations and of the connection to the superstructure shall ensure that the whole building is excited in a uniform way by the seismic motion.

(2) Thus, for structures composed of a discrete number of structural walls, likely to differ in width and stiffness, a rigid, box-type or cellular foundation, containing a foundation slab and a cover slab, should be chosen.

(3) For buildings with individual foundation elements (footings or piles) the use of a foundation slab or tie-beams between these elements in both main directions should be considered, subject to the criteria of clause 5.4.1.2 of Part 5.

ANNEX C (INFORMATIVE)

APPROXIMATE FORMULAE FOR THE FUNDAMENTAL PERIOD OF BUILDINGS

C1 General

(1) The approximate formulae for the fundamental period T_1 of buildings, given in C2 and C3, may be used for preliminary design purposes.

C2 Formula 1

(1) For buildings with heights up to [80] m the value of T_1 may be approximated from the following formula:

$$T_1 = C_t \cdot H^{3/4} \tag{C.1}$$

where

T_1 fundamental period of building, in s,

$$C_t = \begin{cases} 0{,}085 & \text{for moment resistant space steel frames,} \\ 0{,}075 & \text{for moment resistant space concrete frames and for eccentric braced steel frames,} \\ 0{,}050 & \text{for all other structures,} \end{cases}$$

H height of the building, in m.

(2) Alternatively, the value C_t for structures with concrete or masonry shear walls may be taken as

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

with

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

where

A_c combined effective area of the shear walls in the first storey of the building, in m²,

A_i effective cross-sectional area of the shear wall i in the first storey of the building, in m²,

l_{wi} length of the shear wall i in the first storey in the direction parallel to the applied forces, in m,

with the restriction that l_{wi}/H shall not exceed 0,9.

C3 Formula 2

(1) Alternatively, the estimation of T_1 can be made by the following expression:

$$T_1 = 2 \cdot \sqrt{d} \qquad (C.2)$$

where

T_1 fundamental period of building, in s,

d lateral displacement of the top of the building, in m, due to the gravity loads applied horizontally.

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