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Basis for design of structures — Seismic actions on structures

Bases du calcul des constructions — Actions sismiques sur les structures

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Foreword

ISO (the International Organization for Standardization) is a worldwide federation of national standards bodies (ISO member bodies). The work of preparing International Standards is normally carried out through ISO technical committees. Each member body interested in a subject for which a technical committee has been established has the right to be represented on that committee. International organizations, governmental and non-governmental, in liaison with ISO, also take part in the work. ISO collaborates closely with the International Electrotechnical Commission (IEC) on all matters of electrotechnical standardization.

International Standards are drafted in accordance with the rules given in the ISO/IEC Directives, Part 3.

Draft International Standards adopted by the technical committees are circulated to the member bodies for voting. Publication as an International Standard requires approval by at least 75 % of the member bodies casting a vote.

Attention is drawn to the possibility that some of the elements of this International Standard may be the subject of patent rights. ISO shall not be held responsible for identifying any or all such patent rights.

International Standard ISO 3010 was prepared by Technical Committee ISO/TC 98, *Bases for design of structures*, Subcommittee SC 3, *Loads, forces and other actions*.

This second edition cancels and replaces the first edition (ISO 3010:1988), which has been technically revised.

Annexes A, B, C, D, E, F, G, H, I and J of this International Standard are for information only.

Introduction

This International Standard presents basic principles for the evaluation of seismic actions on structures. The seismic actions described are fundamentally compatible with ISO 2394.

It also includes principles of seismic design, since the evaluation of seismic actions on structures and the design of the structures are closely related.

Basis for design of structures — Seismic actions on structures

1 Scope

This International Standard specifies principles of evaluating seismic actions for the seismic design of buildings, towers, chimneys and similar structures. Some of the principles can be referred to for the seismic design of structures such as bridges, dams, harbour installations, tunnels, fuel storage tanks, chemical plants and conventional power plants.

The principles specified in this International Standard do not cover nuclear power plants, since these are dealt with separately in other International Standards.

In regions where the seismic hazard is low, methods of design for structural integrity may be used in lieu of methods based on a consideration of seismic actions.

This International Standard is not a legally binding and enforceable code. It can be viewed as a source document that is utilized in the development of codes of practice by the competent authority responsible for issuing structural design regulations.

NOTE 1 This International Standard has been prepared mainly for engineered structures. The principles are, however, applicable to non-engineered structures.

NOTE 2 The qualification of the level of seismic hazard that would be considered low depends on not only the seismicity of the region but other factors, including types of construction, traditional practices, etc. Methods of design for structural integrity include regional design horizontal forces which provide a measure of protection against seismic actions.

2 Normative reference

The following normative document contains provisions which, through reference in this text, constitute provisions of this International Standard. For dated references, subsequent amendments to, or revisions of, any of these publications do not apply. However, parties to agreements based on this International Standard are encouraged to investigate the possibility of applying the most recent edition of the normative document indicated below. For undated references, the latest edition of the normative document referred to applies. Members of ISO and IEC maintain registers of currently valid International Standards.

ISO 2394, *General principles on reliability for structures*

3 Terms and definitions

For the purposes of this International Standard, the following terms and definitions apply.

3.1
complete quadratic combination method
method to evaluate the maximum response of a structure by the quadratic combination of modal response values

3.2
ductility
ability to deform beyond the elastic limit under cyclic loadings without serious reduction in strength or energy absorption capacity

3.3
liquefaction
loss of shear strength and degradation of stiffness under cyclic loadings in saturated, loose, cohesionless soils

3.4
moderate earthquake ground motion
moderate ground motion caused by earthquakes which may be expected to occur at the site during the service life of the structure

3.5
normalized design response spectrum
spectrum to determine the base shear factor relative to the maximum ground acceleration as a function of the fundamental natural period of the structure

3.6
paraseismic influences
ground motion whose characteristics are similar to those of natural earthquake ground motions, but its sources are mainly due to human activities

3.7
P-delta effect
second-order effect which is caused by the additional moment due to the large displacement and the gravity load

3.8
restoring force
force exerted from the deformed structure or structural elements which tends to move the structure or structural elements to the original position

3.9
seismic force distribution factor of the ith level
$k_{\mathrm{F},i}$
factor to distribute the seismic shear force of the base to the ith level, which characterizes the distribution of seismic forces in elevation, where

$$\sum k_{\mathrm{F},i} = 1$$

3.10
seismic hazard zoning factor
k_{Z}
factor to express the relative seismic hazard of the region

NOTE This is usually unity at the region of the highest seismic hazard.

3.11
seismic shear distribution factor of the ith level
$k_{\mathrm{v},i}$
ratio of the seismic shear factor of the ith level to the seismic shear factor of the base, which characterizes the distribution of seismic shear forces in elevation

NOTE $k_{\mathrm{v},i} = 1$ at the base and usually becomes largest at the top.

3.12
severe earthquake ground motion
severe ground motion caused by an earthquake that could occur at the site

3.13
square root of sum of squares method
method to evaluate the maximum response of a structure by the square root of the sum of the squares of modal response values

3.14
structural factor
k_D
factor to reduce design seismic forces or shear forces taking into account ductility, acceptable deformation, restoring force characteristics and overstrength (or overcapacity) of the structure

4 Symbols and abbreviated terms

CQC	Complete quadratic combination
$F_{\mathrm{E,s},i}$	Design lateral seismic force of the ith level of a structure for SLS
$F_{\mathrm{E,u},i}$	Design lateral seismic force of the ith level of a structure for ULS
$F_{\mathrm{G},i}$	Gravity load at the ith level of the structure
k_D	Structural factor
$k_\mathrm{E,s}$	Representative value of earthquake ground motion intensity for SLS
$k_\mathrm{E,u}$	Representative value of earthquake ground motion intensity for ULS
$k_{\mathrm{F},i}$	Seismic force distribution factor of the ith level
k_R	Ordinate of the normalized design response spectrum
$k_{\mathrm{V},i}$	Seismic shear distribution factor of the ith level
k_Z	Seismic hazard zoning factor
n	Number of levels above the base
SLS	Serviceability limit state
SRSS	Square root of sum of squares
ULS	Ultimate limit state
$V_{\mathrm{E,s},i}$	Design lateral seismic shear force of the ith level of a structure for SLS
$V_{\mathrm{E,u},i}$	Design lateral seismic shear force of the ith level of a structure for ULS
$\gamma_\mathrm{E,s}$	Load factor as related to reliability of the structure for SLS
$\gamma_\mathrm{E,u}$	Load factor as related to reliability of the structure for ULS

5 Bases of seismic design

The basic philosophy of seismic design of structures is, in the event of earthquakes,

— to prevent human casualties,

— to ensure continuity of vital services, and

— to minimize damage to property.

It is recognized that to give complete protection against all earthquakes is not economically feasible for most types of structures. This International Standard states the following basic principles.

a) The structure should not collapse nor experience other similar forms of structural failure due to severe earthquake ground motions that could occur at the site (ultimate limit state: ULS).

b) The structure should withstand moderate earthquake ground motions which may be expected to occur at the site during the service life of the structure with damage within accepted limits (serviceability limit state: SLS).

In order to ensure safety and vital services, elements controlling services to buildings, such as cables, pipe lines, air-conditioning, fire-fighting system, elevator system and other similar systems, should be protected against seismic actions.

NOTE 1 In addition to the seismic design and construction of structures stated in this International Standard, it is useful to consider adequate countermeasures against secondary disasters such as fire, leakage of hazardous materials from industrial facilities or storage tanks, and large-scale landslides which may be triggered by the earthquake.

NOTE 2 Following an earthquake, earthquake-damaged buildings may need to be evaluated for safe occupation during a period of time when aftershocks occur. This International Standard, however, does not address actions that can be expected due to aftershocks. In this case a model of the damaged structure is required to evaluate seismic actions.

6 Principles of seismic design

6.1 Construction site

Characteristics of construction sites under seismic actions should be evaluated, taking into account microzonation criteria (vicinity to active faults, soil profile, soil behaviour under large strain, liquefaction potential, topography, subsurface irregularity, and other factors such as interactions between these).

6.2 Structural configuration

For better seismic resistance, it is recommended that structures have simple forms in both plan and elevation.

a) **Plan irregularities**

Structural elements to resist horizontal seismic actions should be arranged such that torsional effects become as small as possible. Irregular shapes in plan causing eccentric distribution of forces are not desirable, since they produce torsional effects which are difficult to assess accurately and which may amplify the dynamic response of the structure (see annex F).

b) **Vertical irregularities**

Changes in mass, stiffness and capacity along the height of the structure should be minimized to avoid damage concentration (see annex D).

When a structure with complex form is to be designed, an appropriate dynamic analysis is recommended in order to check the potential behaviour of the structure.

6.3 Influence of non-structural elements

The building, including non-structural as well as structural elements, should be clearly defined as a lateral load-resisting system which can be analysed. In computing the earthquake response of a building, the influence of not only the structural frames but also walls, floors, partitions, stairs, windows, etc., should be considered.

NOTE Non-structural elements neglected in seismic analysis can provide additional strength and stiffness to the structure, which may result in favourable behaviour during earthquakes. The non-structural elements, however, may cause unfavourable behaviour, e.g. spandrel walls may reduce clear height of reinforced concrete columns and cause the brittle shear failure to the columns, or unsymmetrical allocation of partition walls (which are considered to be non-structural elements) may cause large torsional moments to the structure. Therefore, all elements should be considered as they behave during earthquakes. If neglecting the non-structural elements does not cause any unfavourable behaviour, they need not be included in seismic analysis.

6.4 Strength and ductility

The structural system and its structural elements (both members and connections) should have both adequate strength and ductility for the applied seismic actions.

The structure should have adequate strength for the applied seismic actions and sufficient ductility to ensure adequate energy absorption (see annex B). Special attention should be given to suppressing the brittle behaviour of structural elements, such as buckling, bond failure, shear failure, and brittle fracture. The deterioration of the restoring force under cyclic loadings should be taken into account.

Local capacities of the structure may be higher than that assumed in the analysis. Such overcapacities should be taken into account in evaluating the behaviour of the structure, including the failure mode of structural elements, failure mechanism of the structure, and the behaviour of the foundations due to severe earthquake ground motions.

6.5 Deformation of the structure

The deformation of the structure under seismic actions should be limited, neither causing malfunction of the structure for moderate earthquake ground motions, nor causing collapse or other similar forms of structural failure for severe earthquake ground motions.

NOTE There are two kinds of deformations to be controlled: the interstorey drift which is the lateral displacement within a storey and the total lateral displacement at some level relative to the base. The interstorey drift should be limited to restrict damage to non-structural elements such as glass panels, curtain walls, plaster walls and other partitions for moderate earthquake ground motions and to control failure of structural elements and the instability of the structure in the case of severe earthquake ground motions. The control of the total displacement is concerned with sufficient separations of two adjoining structures to avoid damaging contact for severe earthquake ground motions. The control of the total displacement may also decrease the amplitude of vibration of the structure and reduce panic or discomfort for moderate earthquake ground motions. In the evaluation of deformations under severe earthquake ground motions, it is generally necessary to account for the second order effect (P-delta effect) of additional moments due to gravity plus vertical seismic forces acting on the displaced structure which occurs as a result of severe earthquake ground motions.

6.6 Response control systems

Response control systems for structures, e.g. seismic isolation, can be used to ensure continuous use of the structure for moderate earthquake ground motions and to prevent collapse during severe earthquake ground motions (see annex J).

6.7 Foundations

The type of foundation should be selected carefully in accordance with the type of structure and local soil conditions, e.g. soil profile, subsurface irregularity, groundwater level. Both forces and deformations transferred through the foundations should be evaluated properly considering the strains induced to soils during earthquake ground motions as well as kinematic and inertial interactions between soils and foundations.

7 Principles of evaluating seismic actions

7.1 Variable and accidental actions

Seismic actions shall be taken either as variable actions or accidental actions.

Structures should be verified against design values of seismic actions for ULS and SLS. The verification for the SLSs may be omitted provided that it is satisfied through the verification for the ULSs (see 8.1).

Accidental seismic actions can be considered for structures in regions where seismic activity is low to ensure structural integrity.

NOTE Verification of the SLS may be omitted in low seisimicity regions, where the SLS actions are low, and for stiff structures (e.g. shear wall buildings) which are designed to remain nearly elastic under ULS actions.

7.2 Dynamic and equivalent static analyses

The seismic analysis of structures shall be performed either by dynamic analysis or by equivalent static analysis. In both cases the dynamic properties of the structure shall be taken into consideration.

Appropriate post-elastic performance shall be provided by adequate choice of the structural system and ductile detailing. The sequence of behaviour of the structure, including the formation of the collapse mechanism, should be established.

NOTE 1 Usually the sequence of behaviour can be verified through non-linear static analysis under lateral loads.

a) **Dynamic analysis**

A dynamic analysis is highly recommended for specific structures such as slender high-rise buildings and structures with irregularities of geometry, mass distribution or stiffness distribution. A dynamic analysis is also recommended for structures with innovative structural systems (e.g. response control systems, see 6.6), structures made of new materials, structures built on special soil conditions, and structures of special importance.

b) **Equivalent static analysis**

Ordinary and regular structures may be designed by the equivalent static method using conventional linear elastic analysis.

NOTE 2 If it is essential that services (e.g. mechanical and electrical equipment and pipings) retain their functions during and after severe or moderate earthquake ground motions, then the design of these services should preferably be done by dynamic analysis procedures based on the earthquake response of the structure which supports them.

7.3 Criteria for determination of seismic actions

The design seismic actions shall be determined based on the following considerations.

a) **Seismicity of the region**

The seismicity of the region where a structure is to be constructed is usually indicated by a seismic zoning map, which may be based on either the seismic history or on seismotectonic data of the region, or on a combination of historical and seismotectonic data. In addition, the expected values of the maximum intensity of the earthquake ground motion in the region in a given future period of time should be determined on the basis of the regional seismicity.

NOTE 1 In addition to the consideration of the historical records of earthquakes, investigation of actual earthquake faults in the region could provide valuable guidance for estimating the future occurrence of earthquakes.

NOTE 2 There exist many kinds of parameters which can be used to characterize the intensity of earthquake ground motion. These are seismic intensity scales, peak ground acceleration and velocity, “effective” peak ground acceleration and velocity which is related to smoothed response spectra, input energy, etc. Recently a method has been proposed to determine the parameters from a probabilistic seismic hazard analysis to give uniform hazard for structures of different periods of vibration. The selection of the type of parameter depends mainly on available data and the type of structure.

b) **Soil conditions**

Dynamic properties of the supporting soil layers of the structure should be investigated and considered.

NOTE 3 The ground motion at a particular site during earthquakes has a predominant period of vibration which, in general, is shorter on firm ground and longer on soft ground. Attention should be paid to the possibility of local amplifications of earthquake ground motions, which may occur (*inter alia*) in the presence of soft soils and near the edge of alluvial basins. The possibility of liquefaction should also be considered, particularly in saturated, loose, cohesionless soils.

NOTE 4 The properties of earthquake ground motions such as predominant periods of vibration and duration of motion are also important features as far as the destructiveness of earthquakes is concerned. Furthermore, it should be recognized that structures constructed on soft ground often suffer damage due to uneven or large settlements during earthquakes.

c) **Dynamic properties of the structure**

Dynamic properties, such as periods and modes of vibration and damping properties, should be considered for the overall soil-structure system. The dynamic properties depend on the shape of the structure, mass distribution, stiffness distribution, soil properties, and the type of construction. Non-linear behaviour of the structural elements should also be taken into account (see 8.1a). A larger value of the seismic design force should be considered for a structure having less ductility capacity or for a structure where a structural element failure may lead to complete structural collapse.

d) **Importance of the structure in relation to its use**

A higher level of reliability is required for buildings where large numbers of people assemble, or structures which are essential for public well-being during and after the earthquakes, such as hospitals, power stations, fire stations, broadcasting stations and water supply facilities (see annex A).

NOTE 5 From the point of view of national and political economics, the load factors as related to reliability of the structure $\gamma_{\mathrm{E,u}}$ and $\gamma_{\mathrm{E,s}}$ (see 8.1) should generally be increased in urban areas with a high damage potential and a high concentration of capital investment.

e) **Spatial variation of earthquake ground motion**

Usually the relative motion between different points of the ground may be disregarded. However, in the case of long-span or widely spread structures, this action and the effect of a travelling wave which can come with phase delay should be taken into account.

8 Evaluation of seismic actions by equivalent static analysis

8.1 Equivalent static loadings

In the seismic analysis of structures based on a method using equivalent static loadings, the variable seismic actions for ULS and for SLS may be evaluated as follows.

a) **ULS**

The design lateral seismic force of the ith level of a structure for ULS, $F_{\mathrm{E,u},i}$, may be determined by

$$F_{\mathrm{E,u},i} = \gamma_{\mathrm{E,u}}\, k_{\mathrm{Z}}\, k_{\mathrm{E,u}}\, k_{\mathrm{D}}\, k_{\mathrm{R}}\, k_{\mathrm{F},i} \sum_{j=1}^{n} F_{\mathrm{G},j} \qquad (1)$$

or the design lateral seismic shear force for ULS, $V_{\mathrm{E,u},i}$, may be used instead of the above seismic force:

$$V_{\mathrm{E,u},i} = \gamma_{\mathrm{E,u}}\, k_{\mathrm{Z}}\, k_{\mathrm{E,u}}\, k_{\mathrm{D}}\, k_{\mathrm{R}}\, k_{\mathrm{V},i} \sum_{j=1}^{n} F_{\mathrm{G},j} \qquad (2)$$

where

$\gamma_{\mathrm{E,u}}$ is the load factor as related to reliability of the structure for ULS (see annex A);

k_{Z} is the seismic hazard zoning factor to be specified in the national code or other national documents (see annex A);

$k_{\text{E,u}}$ is the representative value of earthquake ground motion intensity for ULS to be specified in the national code or other national documents by considering the seismicity (see annex A);

k_{D} is the structural factor to be specified for various structural systems according to their ductility, acceptable deformation, restoring force characteristics and overstrength (see annex B);

k_{R} is the ordinate of the normalized design response spectrum, as a function of the fundamental natural period of the structure considering the effect of soil conditions (see annex C) and damping property of the structure (see annex H);

$k_{\text{F},i}$ is the seismic force distribution factor of the ith level to distribute the seismic shear force of the base to each level, which characterizes the distribution of seismic forces in elevation, where $k_{\text{F},i}$ satisfies the condition $\sum k_{\text{F},i} = 1$ (see annex D);

$k_{\text{v},i}$ is the seismic shear distribution factor of the ith level which is the ratio of the seismic shear factor of the ith level to the seismic shear factor of the base, and characterizes the distribution of seismic shear forces in elevation, where $k_{\text{v},i} = 1$ at the base and usually becomes largest at the top (see annex D);

$F_{\text{G},j}$ is the gravity load at the jth level of the structure;

n is the number of levels above the base.

b) **SLS**

The design lateral seismic force of the ith level of a structure for SLS, $F_{\text{E,s},i}$, may be determined by

$$F_{\text{E,s},i} = \gamma_{\text{E,s}}\, k_{\text{Z}}\, k_{\text{E,s}}\, k_{\text{R}}\, k_{\text{F},i} \sum_{j=1}^{n} F_{\text{G},j} \tag{3}$$

or the design lateral seismic shear force of the ith level for SLS, $V_{\text{E,s},i}$, can be used instead of the above seismic force:

$$V_{\text{E,s},i} = \gamma_{\text{E,s}}\, k_{\text{Z}}\, k_{\text{E,s}}\, k_{\text{R}}\, k_{\text{v},i} \sum_{j=1}^{n} F_{\text{G},j} \tag{4}$$

where

$\gamma_{\text{E,s}}$ is the load factor as related to reliability of the structure for SLS (see annex A);

$k_{\text{E,s}}$ is the representative value of earthquake ground motion intensity for SLS to be specified in the national code or other national documents by considering the seismicity (see annex A).

$k_{\text{E,u}}$ and $k_{\text{E,s}}$ may be replaced by a unique representative k_{E}, as specified in ISO 2394, in the verification procedure, by which the reliability of the structure and the consequences of failure, including the significance of the type of failure, are taken into account to specify the load factors $\gamma_{\text{E,u}}$ and $\gamma_{\text{E,s}}$ (see Table A.2 of annex A).

The values of the gravity load should be equal to the total permanent load plus a probable variable imposed load (see annex D). In snowy areas, a probable snow load is also to be considered.

NOTE Depending on the definition of the seismic actions as variable or accidental, the values for the combination of seismic actions and other actions may be different. For the combination of actions, see ISO 2394.

8.2 Seismic action components and torsion

The two horizontal and vertical components of the earthquake ground motion and their spatial variation, leading to torsional excitation of structures, should be considered (see annexes E and F).

The torsional effects of seismic actions should, in general, be taken into account with due regard to the following quantities: eccentricity between the centres of mass and stiffness; the dynamic magnification caused mainly by the coupling between translational and torsional vibrations; effects of eccentricities in other stories; inaccuracy of computed eccentricity; and rotational components of earthquake ground motions.

NOTE 1 The fact that the seismic actions in any direction do not always attain their maxima at the same time should be borne in mind.

NOTE 2 The vertical component of the earthquake ground motion is usually less intense than the horizontal components and is characterized by higher frequencies. In the vicinity of the epicentre, however, the vertical peak acceleration can be higher than the horizontal peak acceleration.

NOTE 3 In a number of structural forms, the magnitude of structural response from torsional vibration can be comparable to or greater than that from translational vibration. For highly irregular structures, two- or three-dimensional non-linear dynamic analyses are recommended.

NOTE 4 Corner columns of buildings are subjected to large seismic actions because of the combined effects of torsional vibrations plus translational vibrations in both directions.

8.3 Seismic actions on parts of structures

When the seismic actions for the parts of the structure are evaluated by the equivalent static analyses, appropriate factors for seismic forces or shear forces should be used taking into account higher mode effects of the structure including the parts (see annex D). Larger seismic actions than those given in 8.1 can act on parts of structures such as cantilever parapets, structures projecting from the roof, ornamentations and appendages. In addition, curtain walls, infill panels and partitions adjacent to exit ways or facing streets should be designed for safety using the appropriate values of seismic actions.

In the case of parapets, curtain walls, etc., the seismic actions should be considered to take place in both the normal and tangent directions to their surface. Vertical forces should also be considered for connections of such appendages.

9 Evaluation of seismic actions by dynamic analysis

9.1 General

When performing a dynamic analysis, it is important to consider the following items (see annex G).

a) A proper model should be set up, which can represent the dynamic properties of the real structure.

b) Appropriate earthquake ground motions or design response spectra should be chosen, taking into account the seismicity and local soil conditions.

9.2 Dynamic analysis procedures

The usual dynamic analysis procedures may be classified as

a) the response spectrum analysis for linear or equivalent linear systems, or

b) the time history analysis for linear or non-linear systems.

NOTE The time history analysis is preferable when large amounts of post-elastic deformation can be expected and in the case of structures such as described in 7.2 a).

9.3 Response spectrum analysis

A site-specific design response spectrum shall be established in the response spectrum analysis. The spectrum should be based on the proper damping ratio (see annex H). Due consideration should be given to the amount of expected post-elastic deformation and associated restoring force characteristics. The design response spectrum should be smoothed.

In the response spectrum analysis, the maximum dynamic response is usually obtained by the superposition method of SRSS, taking the predominant vibration modes into consideration (see annex G). Sufficient numbers of modes should be considered.

Attention should be given to the fact that the SRSS method does not always lead to conservative values, particularly when frequencies of two or more natural modes are closely spaced. This condition often arises in the vibration of buildings having large setbacks and in the torsional vibration (see 8.2). For these types of buildings, the CQC method is recommended (see annex G).

9.4 Earthquake ground motions for time history analysis

Time history analysis may require several earthquake ground motion records to ensure adequate coverage of expected seismic events. Simulated earthquake ground motions may be used as an alternative. In both cases, the stochastic nature of earthquake ground motions should be taken into account.

Appropriate earthquake ground motions should be determined for each limit state, taking into account the seismicity, local soil conditions, return period of historical earthquakes, distance to active faults, errors in the prediction and design service life of the structure.

a) **Recorded earthquake ground motions**

When recorded earthquake ground motions are considered in a dynamic analysis, the following records may be referred to:

— strong earthquake ground motions recorded at or near the site; or

— strong earthquake ground motions recorded at other sites with similar geological, topographic and seismotectonic characteristics.

Usually these earthquake ground motion records have to be scaled according to the corresponding limit state and seismicity of the site.

b) **Simulated earthquake ground motions**

Since it is impossible to predict exactly the earthquake ground motions expected at a site in the future, it may be appropriate to use simulated earthquake ground motions as design seismic inputs. The parameters of the simulated earthquake ground motions as well as the number of design inputs should reflect statistically the geological and seismological data available for the construction site.

NOTE The parameters of the simulated earthquake ground motions are predominant periods, spectral configuration, time duration (time envelope of the simulated motions), intensity, etc.

9.5 Model of the structure

When setting up a model of the structure, it should represent the dynamic properties of the real structure, such as the natural periods and modes of vibration, damping properties and restoring force characteristics, taking into account material ductility and structural ductility. The dynamic properties can be estimated through analytical procedures and/or experimental results. Consideration should be given to the following:

a) coupling effects of the structure with its foundation and supporting ground;

b) damping in fundamental and higher modes of vibration (see annex H);

c) restoring force characteristics of the structural elements in linear and non-linear ranges including ductility properties;

d) effects of non-structural elements on the behaviour of the structure;

e) torsional effects in linear and non-linear ranges;

f) effects of axial deformation of columns and other vertical elements, or overall bending deformation;

g) effects of irregular distribution of lateral stiffness in elevation (e.g. abrupt change of stiffness in particular stories);

h) effects of floor diaphragm stiffness.

When soil structure interaction is considered, it is recommended to establish the model which includes the structure, foundation, piles and soil.

9.6 Evaluation of analytical results

When dynamic analysis is carried out, the evaluation of seismic actions may be possible solely based on the results of dynamic analysis. However, the evaluation of seismic actions by equivalent static analysis also gives useful information.

When the dynamic analysis gives a lower base shear than the equivalent static analysis does, it is recommended that the design base shear should have some lower limit, e.g. 0,75 to 0,8 of the base shear determined by the equivalent static analysis.

10 Estimation of paraseismic influences

This standard may be used as an introductory approach for paraseismic influences whose characteristics are similar to natural earthquakes, e.g. underground explosions, traffic vibration, pile driving and other human activities. Some advisory remarks are presented in annex J.

Annex A
(informative)

Load factors as related to the reliability of the structure, seismic hazard zoning factor and representative values of earthquake ground motion intensity

A.1 Load factors as related to reliability of the structure, $\gamma_{E,u}$ and $\gamma_{E,s}$

A.1.1 General

$\gamma_{E,u}$ and $\gamma_{E,s}$ are the load factors for ULS and SLS, respectively. They are partial factors for action according to the partial factor format in ISO 2394 and can be determined by means of reliability theory. The factors are related to

a) the required degree of reliability,

b) the representative value of the earthquake ground motion intensity,

c) the variability of seismic actions, and

d) the uncertainty associated with idealization of seismic actions and structures, for the corresponding limit state.

A.1.2 Required degree of reliability

The required degree of reliability depends mainly on the importance and/or use of the structure. The importance of the structure should be determined from the viewpoint of possible consequences of failure during and/or after earthquakes, e.g. loss of lives, human injuries, potential economic losses and social inconveniences.

For ULS, where design requirements correspond to risk to life during and following severe earthquake ground motions, $\gamma_{E,u}$ should be determined according to the following categories of structures.

a) High degree of importance

— structures containing large quantities of hazardous materials whose release to the public may lead to serious consequences; e.g. storage tanks of chemical materials;

— structures closely related to the safety of lives of the public; e.g. hospitals, fire stations, police stations, communication centres, emergency control centres, major facilities in water supply systems, electric power supply systems and gas transmission lines, major roads and railroads;

— structures with high occupancy; e.g. schools, assembly halls, cultural institutions, theatres.

b) Normal degree of importance:

— ordinary structures; e.g. residential houses and apartments, office buildings;

c) Low degree of importance:

— structures with low risk to human lives and injuries; e.g. sheds for cattle or plants, warehouses for non-hazardous materials.

For SLS, where design requirements correspond to loss of normal use of the structure during and/or after moderate earthquake ground motions, $\gamma_{E,s}$ should be determined according to the loss of expected use, and the cost and disruption due to repair.

A.1.3 Variability of seismic actions and uncertainty associated with idealisation of seismic actions and structures

Because of variability of seismic actions, $\gamma_{E,u}$ and $\gamma_{E,s}$ should be determined taking into account the stochastic nature of seismic actions. The variability comes from various sources, e.g. seismic activity at the site, propagation path of seismic waves, local amplification of earthquake ground motion due to soils and structural response. The uncertainties associated with the idealization of seismic actions and calculation models of the structure should be taken into account.

A.1.4 Examples of load factors associated with representative values

$\gamma_{E,u}$ and $\gamma_{E,s}$ are, as examples, listed in Tables A.1 and A.2 for a region of relatively high seismic hazard, along with the representative values of earthquake ground motion intensity $k_{E,u}$ and $k_{E,s}$ (see A.3). Return periods for the corresponding representative values are also shown, where the return period is defined as the expected time interval between which events greater than a certain magnitude are predicted to occur.

An example using the unity load factor for a normal degree of importance is shown in Table A.1, where the return period for the corresponding limit state is taken into account by $k_{E,u}$ or $k_{E,s}$. In Table A.2, a common representative value k_E is used and the degree of importance is taken into account by $\gamma_{E,u}$ or $\gamma_{E,s}$ for the corresponding limit state.

Table A.1 — Example 1 for load factors $\gamma_{E,u}$ and $\gamma_{E,s}$, and representative values $k_{E,u}$ and $k_{E,s}$ (where $k_{E,u} \neq k_{E,s}$)

Limit state	Degree of importance	$\gamma_{E,u}$ or $\gamma_{E,s}$	$k_{E,u}$ or $k_{E,s}$	Return period for $k_{E,u}$ or $k_{E,s}$
Ultimate	a) High	1,5 to 2,0	0,4	500 years
	b) Normal	1,0		
	c) Low	0,4 to 0,8		
Serviceability	a) High	1,5 to 3,0	0,08	20 years
	b) Normal	1,0		
	c) Low	0,4 to 0,8		

Table A.2 — Example 2 for load factors $\gamma_{E,u}$ and $\gamma_{E,s}$, and representative values k_E

Limit state	Degree of importance	$\gamma_{E,u}$ or $\gamma_{E,s}$	$k_E = k_{E,u} = k_{E,s}$	Return period for k_E
Ultimate	a) High	3,0 to 4,0	0,2	100 years
	b) Normal	2,0		
	c) Low	0,8 to 1,6		
Serviceability	a) High	0,6 to 1,2		
	b) Normal	0,4		
	c) Low	0,16 to 0,32		

A.2 Seismic hazard zoning factor, k_Z

The seismic hazard zoning factor, k_Z, reflects the relative seismic hazard of the region. This factor is evaluated taking into account historical earthquake data, active fault data and other seismotectonic data in and around the construction site. Usually at the region of the highest seismic hazard, the factor is unity and the factor decreases according to the seismic hazard of the respective region. A zoning factor larger than unity can be used when the seismic hazard of the region is extremely high. A contour map for the representative value of earthquake ground motion intensity may be provided instead of specifying the zoning factors.

In practical applications, a set of discrete values may be specified based on the seismic hazard maps available. When the maps do not reflect the effects of soil and geology at the respective site, the influences of near-faults, etc., the factor values should be determined taking into account these effects and influences.

A.3 Representative values of earthquake ground motion intensity, $k_{E,u}$ and $k_{E,s}$

The representative values $k_{E,u}$ and $k_{E,s}$ are usually described in terms of horizontal peak ground acceleration as a ratio to the acceleration due to gravity. If the peak ground velocity or other spectral ordinates are given, those values should be transformed into the acceleration.

The representative values for the earthquake ground motion intensity at a region should be evaluated on a statistical basis (e.g. in terms of the return period) or on previous engineering practice and acquired experience. Currently, $k_{E,u}$ is approximately 0,4 at a region with the highest seismic hazard in the world for a return period of approximately 500 years.

A seismic hazard map which expresses the expected horizontal acceleration as a ratio to the acceleration due to gravity k_Z $k_{E,u}$ or k_Z $k_{E,s}$ of the respective region may also be used instead of giving k_Z and $k_{E,u}$ and $k_{E,s}$ separately.

Annex B
(informative)

Structural factor

The structural factor, k_D, is used to reduce design seismic forces or shear forces, taking into account the ductility, acceptable deformation, restoring force characteristics and overstrength (or overcapacity) of the structure.

The factor can be divided into two factors: namely $k_{\mathrm{D}\mu}$ and k_Ds and is expressed as the product of them as follows:

$$k_\mathrm{D} = k_{\mathrm{D}\mu}\, k_\mathrm{Ds} \tag{B.1}$$

where

$k_{\mathrm{D}\mu}$ is related to ductility, acceptable deformation and restoring force characteristics;

k_Ds is related to overstrength.

The factor can also be expressed as follows:

$$k_\mathrm{D} = \frac{1}{R} = \frac{1}{R_\mu\, R_\mathrm{s}} \tag{B.2}$$

where R_μ and R_s are the inverse of $k_{\mathrm{D}\mu}$ and k_Ds, respectively.

Recent studies indicate that $k_{\mathrm{D}\mu}$ also depends on the natural period of vibration of the structure and the possible reduction in strength remains minimal for structures having shorter fundamental natural periods. k_Ds is a function of the difference between the actual strength and calculated strength and varies according to the method of strength calculation. Quantification of these factors is a matter of debate, and one generic term k_D has been adopted in most codes. The structural factor, k_D, may be, for example,

— 1/5 to 1/3 for systems with excellent ductility,

— 1/3 to 1/2 for systems with medium ductility, and

— 1/2 to 1 for systems with poor ductility.

These ranges of k_D are under continuing investigation and may take other values in some circumstances.

The ductility is defined as the ability to deform beyond the elastic limit under cyclicloadings without serious reduction in strength or energy absorption capacity. The ductility factor (usually denoted by μ) is defined as the deformation divided by the elastic limit deformation.

The structural systems given below with different ductilities are only typical examples. It should be noted that detailing of members and joints to get appropriate ductility is important in the assessment of the structural factor. Therefore the structure in one category could be classified in another category depending on the detailing of structural elements (both members and joints).

a) A structural system with excellent ductility is a structural system where the lateral resistance is provided by steel or reinforced concrete moment-resisting frames with adequate connection details and ductility of structural elements.

b) A structural system with medium ductility is a structural system where the lateral resistance is provided by steel-braced frames or reinforced concrete shear walls.

c) A structural system with poor ductility is a structural system where the lateral resistance is provided by unreinforced or partially reinforced masonry shear walls.

The term k_D is affected significantly by the type of failure mechanism. The values shown above are adopted with the assumption that the structure would form the failure mechanism considered in design, and when the structure fails in a different mechanism, larger ductility would be demanded of some part of the structure. Care should be taken to ensure that the failure mechanism assigned in design occurs.

According to the results of nonlinear dynamic analyses of structures subjected to strong earthquake ground motions, k_D (or $1/R$) is $1/\mu$ if the displacement-constant rule is applied and $1/\sqrt{2\mu - 1}$ if the energy-constant rule is applied, where μ is the ductility factor. Therefore the maximum lateral deflection Δ_max expected in ULS may be estimated by simple formulae as follows (see Figure B.1):

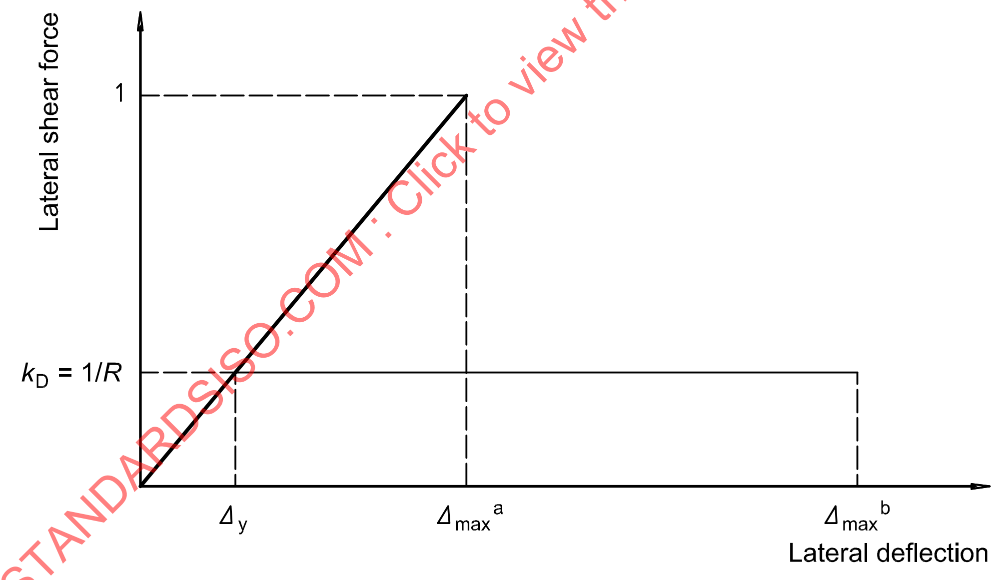
$$\Delta_\mathrm{max} = \frac{1}{k_\mathrm{D}} \Delta_\mathrm{y} = R\Delta_\mathrm{y} \tag{B.3}$$

$$\Delta_\mathrm{max} = \frac{1}{2}\left(\frac{1}{k_\mathrm{D}^2} + 1\right)\Delta_\mathrm{y} = \frac{1}{2}\left(R^2 + 1\right)\Delta_\mathrm{y} \tag{B.4}$$

where Δ_y is the lateral deflection calculated by elastic analysis for the design lateral seismic forces or shear forces defined in equation (1) or (2).

Generally, equation (B.3) is applicable to structures with a longer natural period and equation (B.4) is to structures with a shorter natural period.

The cumulative ductility (or equivalently energy dissipation) demanded of the structure is also a factor not to be overlooked in ULS design, because the structure tends to lose its strength under cyclic loadings (such behaviour is termed cumulative damage). Much research has been conducted to quantify the cumulative ductility demand, and design procedures to allow for this demand might be provided in the future.



[a] By displacement-constant rule

[b] By energy-constant rule

Figure B.1 — Relationship between lateral shear force and deflection

Annex C
(informative)

Normalized design response spectrum

The normalized design response spectrum can be interpreted as an acceleration response spectrum normalized by the maximum ground acceleration for design purpose.

It may be of the form

$$k_{\mathrm{R}} = 1 \text{ for } T = 0 \tag{C.1}$$

$$\text{Linear interpolation for } 0 < T \leqslant T_{\mathrm{c}}' \tag{C.2}$$

$$k_{\mathrm{R}} = k_{\mathrm{Ro}} \text{ for } T_{\mathrm{c}}' < T \leqslant T_{\mathrm{c}} \tag{C.3}$$

$$k_{\mathrm{R}} = k_{\mathrm{Ro}} \left(\frac{T_{\mathrm{c}}}{T} \right)^{\eta} \text{ for } T > T_{\mathrm{c}} \tag{C.4}$$

where

k_{R} is the ordinate of the normalized design response spectrum;

k_{Ro} is a factor dependent on the soil profile and the characteristics of the structure, e.g. the damping of the structure; for a structure with a damping ratio of 0,05 resting on the average quality soil, k_{Ro} may be taken as 2 to 3;

T is the fundamental natural period of the structure;

T_{c} and T_{c}' are the corner periods as related to the soil condition, as illustrated in Figure C.1;

η is an exponent that can vary between 1/3 and 1; when $\eta = 1$, the response velocity becomes constant as $\left(\frac{g}{2\pi} k_{\mathrm{Ro}} T_{\mathrm{c}} \right)$ for $T > T_{\mathrm{c}}$, therefore, T_{c} is closely related to the response velocity;

T_{c}, T_{c}' and η are dependent on tectonic and geological conditions; T_{c}' may be taken as (1/5) to (1/2) of T_{c}.

For example, for horizontal motions T_{c} can be taken as

— 0,3 s to 0,5 s for stiff and hard soil conditions,

— 0,5 s to 0,8 s for intermediate soil conditions, and

— 0,8 s to 1,2 s for loose and soft soil conditions.

For the classification of soil conditions, the thickness of the soil layers should be taken into account.

The fundamental natural period, T, can be calculated from calibrated empirical formulae, from Rayleigh's approximation, or from an eigenvalue formulation. For the estimation of T, the reduction of stiffness of concrete elements due to cracking should be taken into account.

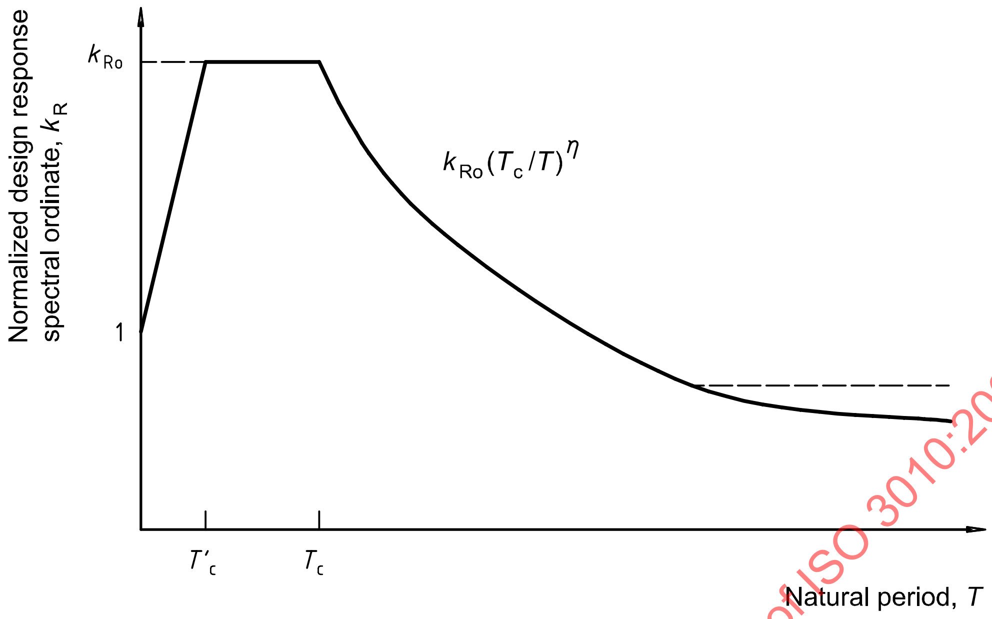


Figure C.1 — Normalized design response spectrum

Figure C.1 indicates that k_R is unity at $T = 0$ and linearly increases to k_{Ro} at $T = T'_c$. It is recommended, however, to use $k_R = k_{Ro}$ for $0 < T \leqslant T'_c$, as the dotted line of Figure C.1, because of the following reasons:

— uncertainty of ground motion characteristics in this range;

— low sensitivity of strong motion accelerometers in this range, and therefore a possibility of a higher value of k_R than the apparent one;

— possibility of an unconservative estimate of the structural factor k_D for short period structures.

For determination of forces at longer periods, it is recommended that a lower limit be considered as indicated by the dashed line in Figure C.1. The value of this level may be taken as 1/3 to 1/5 of k_{Ro}.

For determination of the displacements at longer periods, Figure C.1 becomes too conservative. For long periods, the response displacement becomes a function of the maximum displacement of earthquake ground motions. There is uncertainty about the ground displacement close to faults in very large magnitude earthquakes, therefore extrapolation of data from smaller earthquakes should be made with care.

An equivalent linearlization approach may also be used for estimating the maximum deformations of structural systems. In this approach, a system involving hysteretic behaviour is replaced with a linear system having an equivalent natural period and an equivalent viscous damping ratio. The maximum deformation of the hysteretic system is estimated as that of the equivalent linear system. A number of proposals are available for determining the equivalent natural period and viscous damping ratio, which are primarily specified as a function of the expected ductility factor. In recent years, design concepts based upon displacement analysis have been advanced, and the equivalent linearlization approach is often used for determining the required strength for a given maximum deformation.

Annex D
(informative)

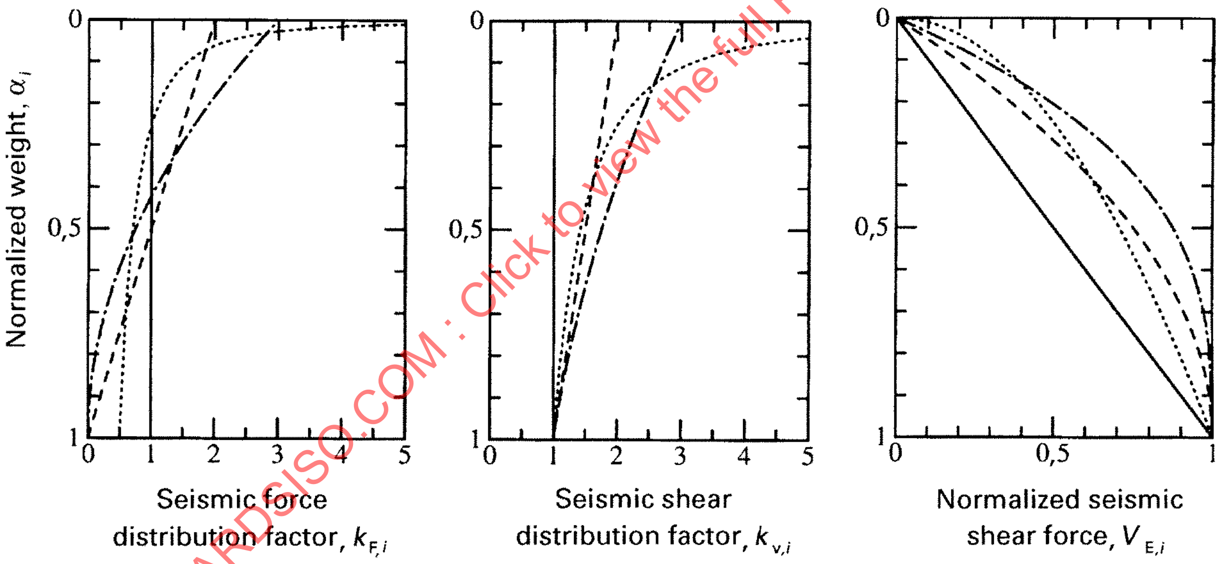
Seismic force distribution factor and seismic shear distribution factor

General characteristics of distributions of seismic force parameters above the base are as follows.

a) For very low and stiff buildings, whole parts from the top to the base move along with the ground motion. In this case, the distribution of seismic forces is uniform and the seismic shear forces increase linearly from the top to the base (uniform distribution of seismic forces, see solid lines of Figure D.1. In Figure D.1 the normalized shear force $V_{\mathrm{E},i}$ is the shear force of the ith level divided by the base shear).

b) For low-rise buildings, the distribution of seismic forces becomes similar to the inverted triangle and the distribution of seismic shear forces is assumed to be a parabola whose vertex locates at the base (inverted triangular distribution of seismic forces; see dashed curves of Figure D.1).

c) For high-rise buildings, seismic forces at the upper part become larger because of a higher mode effect. If the building is assumed to be a uniform shear type elastic body fixed at the base and to be subjected to white noise excitation, the distribution of seismic shear forces becomes a parabola whose vertex locates at the top (distribution for shear type structure subjected to white noise excitation; see dotted curves of Figure D.1).



Key

———— $\nu = 0$ in equation (D.1), or $k_1 = 0$, $k_2 = 0$ in equation (D.4)

- - - - $\nu = 1$ in equation (D.1), or $k_1 = 1$, $k_2 = 0$ in equation (D.4)

-·-·-·- $\nu = 2$ in equation (D.1)

·········· $k_1 = 0$, $k_2 = 1$ in equation (D.4)

Figure D.1 — Distribution of seismic force parameters

Taking into account the above mentioned characteristics of seismic force parameters, the seismic force distribution factor, $k_{\mathrm{F},i}$, can be determined by

$$k_{\mathrm{F},i} = \frac{F_{\mathrm{G},i} h_i^{\nu}}{\sum_{j=1}^{n} F_{\mathrm{G},j} h_j^{\nu}} \tag{D.1}$$

where

$F_{\mathrm{G},i}$ is the gravity load of the structure at the ith level, which includes the probable variable imposed load (0,2 to 0,3 of the total imposed load);

h_i is the height above the base to the ith level;

n is the number of levels above the base.

The exponent v may be taken as follows:

— $\nu = 0$ for very low buildings (up to two-storey buildings), or structures for which $T \leqslant 0{,}2$ s;

— $\nu = 0$ to 1 for low-rise buildings (three to five-storey buildings), or structures for which $0{,}2\ \text{s} < T \leqslant 0{,}5$ s;

— $\nu = 1$ to 2 for intermediate buildings, or structures for which $0{,}5\ \text{s} < T \leqslant 1{,}5$ s;

— $\nu = 2$ for high-rise buildings (higher than 50 m or more than fifteen-storey buildings), or structures for which $T > 1{,}5$ s.

Distributions of seismic force parameters given by equation (D.1) are shown as solid lines in Figure D.1 for $\nu = 0$, as dashed curves in Figure D.1 for $\nu = 1$, and as dash-dotted curves in Figure D.1 for $\nu = 2$.

Since equation (D.1) does not give an appropriate distribution for high-rise buildings, even if the exponent ν becomes 2 (see dash-dotted curves of Figure D.1), the seismic force distribution factor, $k_{\mathrm{F},i}$, for high-rise buildings can be determined by

$$k_{\mathrm{F},n} = \rho \tag{D.2}$$

$$k_{\mathrm{F},i} = (1 - \rho)\,\frac{F_{\mathrm{G},i} h_i}{\sum\limits_{j=1}^{n} F_{\mathrm{G},j} h_j} \tag{D.3}$$

where ρ is the factor to give a concentrated force at the top; approximately $\rho = 0{,}1$.

Since equations (D.2) and (D.3) do not always give an appropriate distribution and a concentrated force at the top is not practical for buildings with setbacks, it is preferable using the seismic shear distribution factor, $k_{\mathrm{V},i}$, instead of seismic force distribution factor $k_{\mathrm{F},i}$. The factor $k_{\mathrm{V},i}$ is interpreted as the shear factor of the ith level normalized by the base shear factor.

The seismic shear distribution factor, $k_{\mathrm{V},i}$, can be determined by

$$k_{\mathrm{V},i} = 1 + k_1(1 - \alpha) + k_2\left(\frac{1}{\sqrt{\alpha_i}} - 1\right) \tag{D.4}$$

where

k_1 and k_2 are factors from 0 to 1 and are determined mainly by the height or the fundamental natural period of the structure;

α_i is the normalized weight and is given by

$$\alpha_i = \frac{\sum\limits_{j=i}^{n} F_{\mathrm{G},j}}{\sum\limits_{j=1}^{n} F_{\mathrm{G},j}} \tag{D.5}$$

The normalized weight is used instead of the height of levels above the base, because the normalized weight is more convenient and rational to express distributions of seismic force parameters. The ordinate in Figure D.1 is the normalized weight. In the case of a structure with uniform mass distribution, the normalized weight may be approximated by the height as follows:

$$\alpha_i \approx \frac{h_n - h_{i-1}}{h_n} \tag{D.6}$$

Distributions of seismic force parameters given by equation (D.4) are shown as solid lines in Figure D.1 for $k_1 = 0$ and $k_2 = 0$ (uniform distribution of seismic forces), as dashed curves in Figure D.1 for $k_1 = 1$ and $k_2 = 0$ (inverted triangular distribution of seismic forces), and as dotted curves in Figure D.1 for $k_1 = 0$ and $k_2 = 1$ (distribution for shear type structure subjected to white noise excitation). Therefore, the factor k_1 and k_2 may be taken as follows:

— $k_1 \approx 0$ and $k_2 \approx 0$ for very low buildings;

— $k_1 \approx 1$ and $k_2 \approx 0$ for low-rise buildings;

— $k_1 \approx 0{,}5$ and $k_2 \approx 0{,}5$ for intermediate buildings;

— $k_1 \approx 0$ and $k_2 \approx 1$ for high-rise buildings.

When the seismic actions for the parts of the structure projecting from the roof are evaluated, the seismic shear factor can be calculated by equation (D.4) assuming $k_1 \approx 0$ and $k_2 \approx 1$, and substituting the normalized weight of the part.

Since the deformation caused by the earthquake ground motions concentrates at the level which has less stiffness, $k_{\mathrm{F},i}$ or $k_{\mathrm{v},j}$ should be adjusted to take account of such behaviour.

Annex E
(informative)

Components of seismic action

The two horizontal components of the earthquake ground motion influence the total seismic actions on the structure, for example

a) torsional moment of the structure with two-directional eccentricity, and

b) axial force of corner columns.

When the two horizontal components of the seismic action are designated as E_x and E_y according to the orthogonal axes x-y, the directions of which follow the layout of the structures, sometimes the SRSS (square root of sum of squares) method is applied to obtain the total design seismic action, E. The method, however, often underestimates the maximum response. To avoid this problem, it is recommended to use the following quadratic combination:

$$E = \sqrt{E_x^2 + 2\varepsilon E_x E_y + E_y^2} \tag{E.1}$$

While the factor ε can be from -1 to 1 ($\varepsilon = 0$ means the SRSS method), ε may empirically be taken as 0 to 0,3.

First-order approximation of equation (E.1) leads to the following formulae, which may be used instead:

$$E = E_x + \lambda E_y, \qquad E = \lambda E_x + E_y \tag{E.2}$$

The value of λ may be taken as 0,3 to 0,5.

The relationships E/E_x in terms of E_y/E_x by equations (E.1) and (E.2) are shown in Figure E.1.

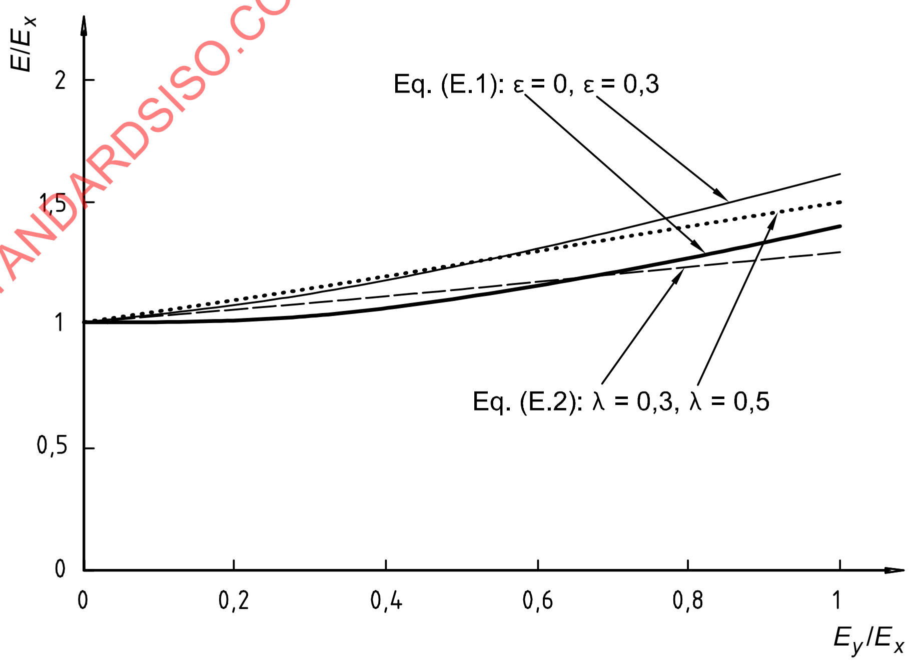


Figure E.1 — Relationships between E/E_x in terms of E_y/E_x according to equations (E.1) and (E.2)

The vertical component E_z is usually not considered explicitly. It is, however, taken into account with its most unfavourable value, for example in the following cases:

a) prestressed structures;

b) horizontal structural elements with clear spans of more than 20 m;

c) constructions with high arching forces;

d) cantilever elements;

e) concrete columns and shear walls subjected to high shear forces, especially at construction interfaces.

The vertical peak ground acceleration may be taken as 1/2 to 2/3 of the horizontal peak ground acceleration. However, it should be borne in mind that the vertical acceleration could be larger than the horizontal one near the epicentre over shallow focus earthquakes.

Annex F
(informative)

Torsional moments

The torsional moment of the ith level of the structure, M_i, which is usually calculated in each direction of the orthogonal axes x and y of the structure as schematically illustrated in Figure F.1, may be determined by

$$M_i = V_i e_i \tag{F.1}$$

where V_i is the seismic shear force of the ith level:

$$V_i = \sum_{j=i}^{n} F_j \tag{F.2}$$

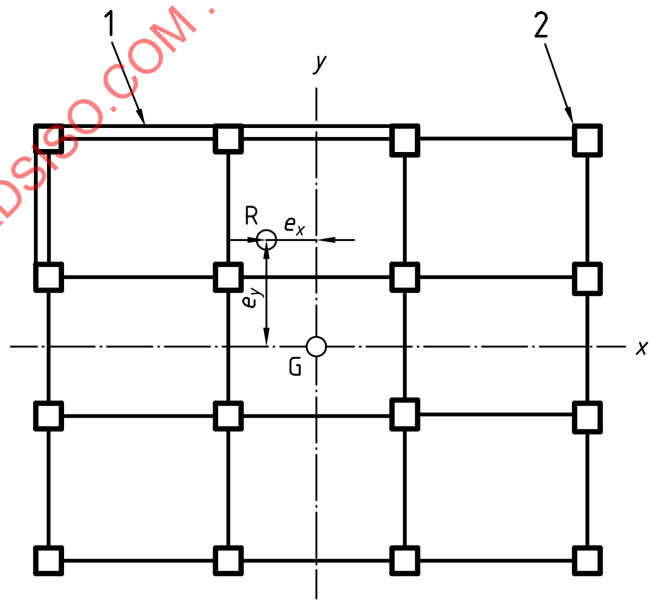
where

n is the number of levels above the base;

e_i is one of the following two values, whichever is the most unfavourable for the structural element under consideration:

— the eccentricity between the centres of mass and stiffness, multiplied by a dynamic magnification factor representing the coupling of transverse and torsional vibrations, plus the incidental eccentricity of the ith level;

— the eccentricity between the centres of mass and stiffness, minus the incidental eccentricity.



Key

1 Shear wall

2 Column

Figure F.1 — Centre of mass G, centre of stiffness R and eccentricity e_x, e_y

The dynamic magnification factor will be specified in the national code or other national documents. For example, this value may be taken as 1 to 2.

The incidental eccentricity which covers the inaccuracy of estimated eccentricity as well as rotational components of ground motion is assumed to be not less than 0,05 of the dimension of the structure perpendicular to the direction of the applied forces.

The strength and ductility of structural elements should be well arranged considering the torsional moment which gives additional seismic action effects to structural elements.

Annex G
(informative)

Dynamic response

G.1 Response spectrum analysis

When natural frequencies of different modes are not closely spaced to each other, the combination to estimate the maximum response quantity may generally be performed using the following formula (SRSS method):

$$S = \sqrt{\sum_{i=1}^{n} S_i^2} \tag{G.1}$$

where

S is the maximum response quantity under consideration;

S_i is the maximum response quantity in the ith mode of vibration.

When natural frequencies of two or more modes are closely spaced, the combination may be performed using the following formula (CQC method) which is derived from the random vibration theory:

$$S = \sqrt{\sum_{i=1}^{n} \sum_{k=1}^{n} S_i \rho_{i,k} S_k} \tag{G.2}$$

$$\rho_{i,k} = \frac{8\sqrt{\xi_i \xi_k}\,(\xi_i + \chi \xi_k)\,\chi^{3/2}}{(1-\chi^2)^2 + 4\xi_i \xi_k \chi\,(1+\chi^2) + 4\,(\xi_i^2 + \xi_k^2)\,\chi^2} \tag{G.3}$$

where

ξ_i, ξ_k are the damping ratios for the ith and kth mode, respectively;

χ is the ratio of the ith mode natural frequency to the kth mode natural frequency.

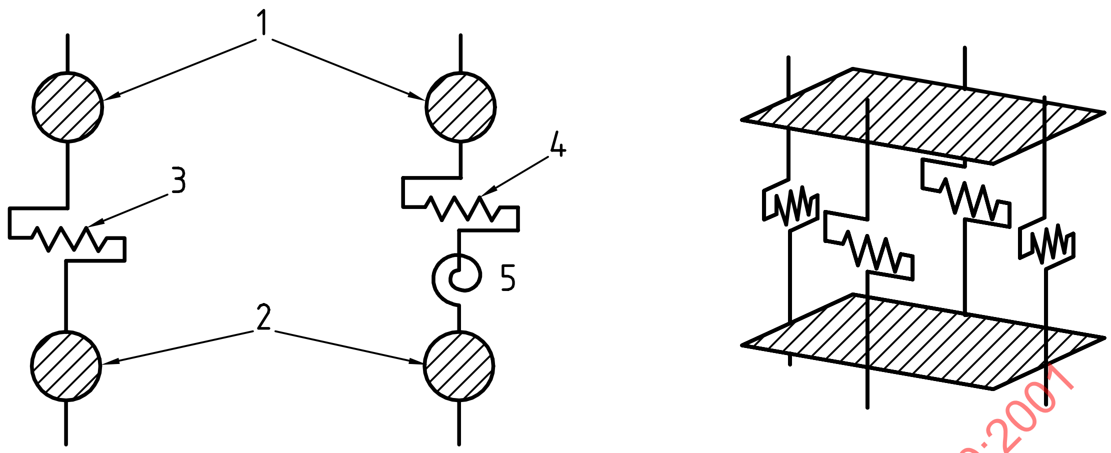
All modes with a significant contribution to the total structural response should be considered for equations (G.1) and (G.2).

The response quantity in each mode of vibration should be obtained by the site-specific response spectrum established for this purpose. In the absence of such a spectrum, the normalized design response spectrum indicated in annex C may be employed.

G.2 Time history analysis

G.2.1 Types of models of the structure

Models of the structure should be chosen based on the purpose of the analysis. Basically, the models used in the time history analysis are the same as those used in the response spectrum analysis. Some examples of the models are shown in Figures G.1 and G.2.



a) Shear model b) Shear-flexure model c) Simplified three-dimensional model

Key

1 Mass at level $(i+1)$
2 Mass at level i
3 Equivalent shear spring
4 Shear spring
5 Flexural spring

Figure G.1 — Examples of models of the structure

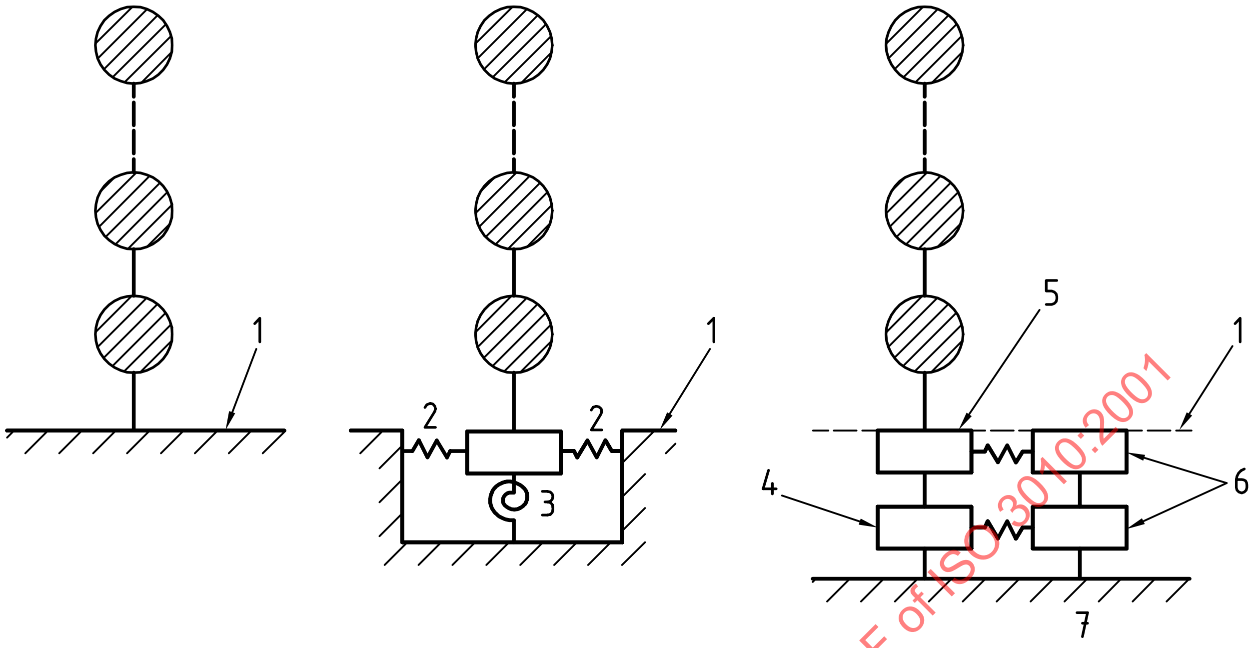
In many cases one-dimensional lumped mass shear models are used for low- to medium-rise buildings, where a lumped mass represents the mass of each storey and the lateral stiffness of storeys are independent [Figure G.1 a)]. For high-rise buildings and slender structures (in which the height-to-width ratio exceeds 3,0), shear-flexural models are recommended to be used, taking into account the axial deformation of columns or the flexural deformation of overall bending of the structure [Figure G.1 b)]. The flexural stiffness may be regarded as elastic even in the post-elastic range of shear stiffness. Simplified three-dimensional models [Figure G.1 c)] are employed to evaluate the torsional response of the structure.

Figure G.2 shows the classification of models from the view point of soil-structure interaction. In general, models fixed at the base may be employed [Figure G.2 a)]. When the effects of ground compliance are to be considered, sway-rocking models with sway springs, rocking springs or combination of them [Figure G.2 b)] may be employed. Soil-structure interaction models [Figure G.2 c)] may be used when earthquake ground motions are defined at the bedrock.

G.2.2 Restoring force characteristics

Although any rational restoring force characteristics are accepted, in principle, they should be elasto-plastic. Elastic models may be accepted where response of the post-elastic range is not expected or quite limited. In general, bilinear or trilinear [Figure G.3 a)] restoring force characteristic models are used for steel elements. For reinforced concrete elements, degrading trilinear models [Figure G.3 b)] are used, since the stiffness degradation of those elements can not be neglected. [Many other types of restoring force characteristics have been proposed, therefore Figures G.3 a) and G.3 b) are only examples.]

Elasto-plastic restoring force characteristics, which represent the relationship between the lateral shear force and inter-storey drift, are recommended to be established by a static load incremental frame analysis of the whole structure. In the analysis, the distribution of lateral forces or shear forces may be assumed to be in proportion to the distribution explained in annex D or obtained by SRSS or CQC method. Instead of assuming the distribution of lateral forces or shear forces, the distribution of lateral displacements may be assumed to be in proportion to those caused by the design lateral forces or shear forces.

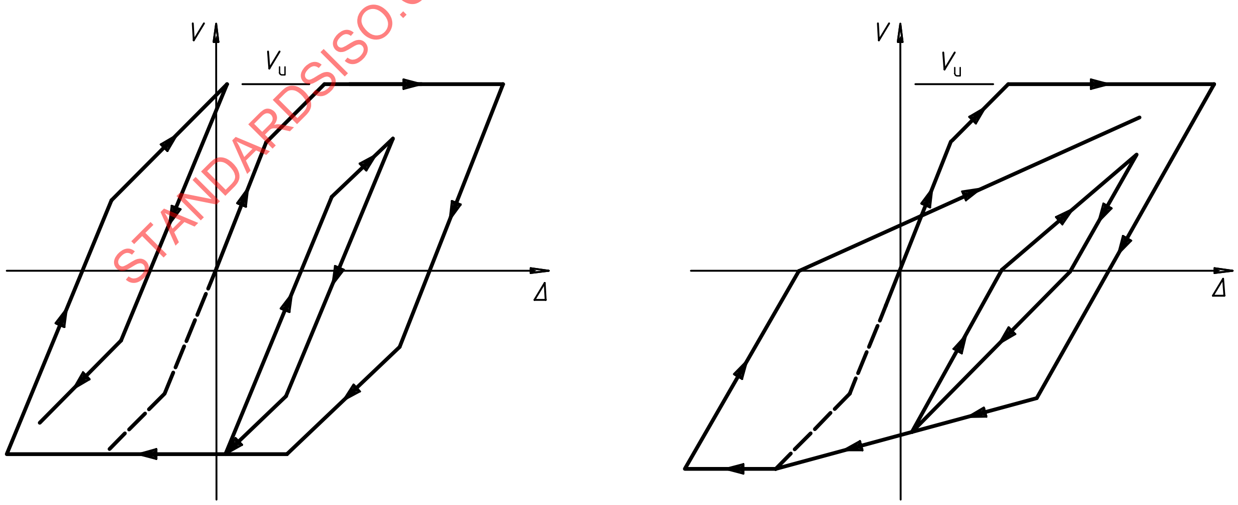


a) Fixed-base model b) Sway-rocking model c) Interaction model

Key

1 Ground level
2 Sway spring
3 Rocking spring
4 Pile
5 Foundation/basement
6 Soil
7 Bedrock

Figure G.2 — Examples of models from the view point of soil structure interaction



a) Trilinear model b) Degrading trilinear model

Figure G.3 — Examples of restoring force characteristic models

G.2.3 Input earthquake ground motions

G.2.3.1 Recorded earthquake ground motions

When recorded earthquake ground motions are used as input ground motions, they should be scaled appropriately. In general, the acceleration records are scaled to have the same maximum velocity in order to avoid the fluctuation in the response. This scaling for the maximum velocity is essential when evaluating seismic actions on the structure with a longer fundamental natural period of vibration. This is because, for structures with a longer natural period which exceeds 2,0 s, excessive fluctuation of the response is observed if the records are scaled by their maximum acceleration. Even if the records are scaled by the maximum velocity, the response of the structure is affected by peaks and troughs in the response spectra of the records. Therefore in the evaluation of the response, it should be borne in mind that the use of recorded earthquake ground motions sometimes leads to the results that are governed by the specific characteristics of the records and that these may not occur at the site.

G.2.3.2 Simulated earthquake ground motions

Simulated earthquake ground motions may be established either at the ground surface or at the bedrock. It is more rational to establish the simulated earthquake ground motions at the bedrock which can be used directly in the soil-structure interaction model analysis. When simulated earthquake ground motions are set up at the ground surface, they should reflect the dynamic characteristics of the soil.

Sometimes the earthquake ground motions established at the bedrock are used as direct input to the fixed base models, being multiplied by a factor of 2,0. This simplified procedure is not recommended unless the soil dynamic characteristics have been fully investigated and the validity of the scale factor confirmed.

The response spectrum of the simulated earthquake ground motions defined at the ground surface or at the bedrock should have a smooth shape and can have a simplified form as follows:

— constant acceleration response for natural periods not more than T_c; and

— constant velocity response for the natural periods more than T_c.

The corner period, T_c, should be determined considering the effect of the soil condition at the site.

G.2.4 Consideration for reliability of the structure

As in the equivalent static analysis, the reliability of the structure should also be considered in the dynamic analysis; e.g. scaling the input ground motions by an appropriate factor which is equivalent to the load factors described in annex A.