INTERNATIONAL STANDARD

ISO 3010

Third edition
2017-03

Bases for design of structures — Seismic actions on structures

Bases du calcul des constructions — Actions sismiques sur les structures

Reference number
ISO 3010:2017(E)

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Foreword

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This document was prepared by ISO/TC 98, *Bases for design of structures*, Subcommittee SC 3, *Loads, forces and other actions*.

This third edition cancels and replaces the second edition (ISO 3010:2001), which has been technically revised.

Introduction

This document 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.

Annexes A to P of this document are for information only.

NOTE 1 ISO 23469 and ISO 13033 are companion documents to this document. They provide basic design criteria for geotechnical works and for nonstructural components and systems, respectively.

NOTE 2 ISO 23469 specifies the procedure to determine the design ground motion for the dynamic analysis of geotechnical works. The procedure in ISO 23469 is applicable to the generation of design ground motion for the structures that exhibit interaction with the ground or the geotechnical works.

NOTE 3 ISO 13033 and its annexes use the same terms and definitions that are used in this document. The ground motion criteria specified in ISO 13033 are the same criteria that are used in this document. The demand on nonstructural components and systems is directly related to the response of the building in which they are located. Therefore, the procedures used to determine the design ground motion and building seismic response are directly referenced to this document.

Bases for design of structures — Seismic actions on structures

1 Scope

This document specifies principles of evaluating seismic actions for the seismic design of buildings (including both the super structure and foundation) and other structures.

This document is not applicable to certain structures, such as bridges, dams, geotechnical works and tunnels, although some of the principles can be referred to for the seismic design of those structures.

This document is not applicable to 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 can be used in lieu of methods based on a consideration of seismic actions.

This document 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 document has been prepared mainly for new engineered structures. The principles are, however, applicable to developing appropriate prescriptive rules for non-engineered structures (see Annex N). The principles could also be applied to evaluating seismic actions on existing structures.

NOTE 2 Other structures include self-supporting structures other than buildings that carry gravity loads and are required to resist seismic actions. These structures include seismic force-resisting systems similar to those in buildings, such as a trussed tower or a pipe rack, or systems very different from those in buildings, such as a liquid storage tank or a chimney. Additional examples include structures found at chemical plants, mines, power plants, harbours, amusement parks and civil infrastructure facilities.

NOTE 3 The level of seismic hazard that would be considered low depends not only on the seismicity of the region but also on other factors, including types of construction, traditional practices, etc. Methods of design for structural integrity include nominal design horizontal forces (such as an equivalent static loading determined from a simplified equivalent static analysis) which provide a measure of protection against seismic actions.

2 Normative references

The following documents are referred to in the text in such a way that some or all of their content constitutes requirements of this document. For dated references, only the edition cited applies. For undated references, the latest edition of the referenced document (including any amendments) applies.

ISO 13033, *Bases for design of structures — Loads, forces and other actions — Seismic actions on nonstructural components for building applications*

3 Terms and definitions

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

ISO and IEC maintain terminological databases for use in standardization at the following addresses:

— IEC Electropedia: available at http://www.electropedia.org/

— ISO Online browsing platform: available at http://www.iso.org/obp

3.1
base shear
design horizontal force acting at the base of the structure

3.2
complete quadratic combination method
CQC
method to evaluate the maximum response of a structure by the quadratic combination of modal response values

3.3
ductility
ability to deform beyond the elastic limit under cyclic loadings without significant reduction in strength or energy absorption capacity

3.4
liquefaction
loss or significant reduction of shear strength and stiffness under cyclic loadings in saturated, loose, cohesionless soils

3.5
moderate earthquake ground motion
ground motion used for SLS caused by earthquakes which may be expected to occur at the site during the service life of the structure

Note 1 to entry: See Annex A.

3.6
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

3.7
paraseismic influences
ground motion whose characteristics are similar to those of earthquake ground motions, but its sources are mainly due to industrial, explosive, traffic, and other human activities

3.8
P-delta effect
second-order effect which is caused by the action of gravity on the displaced mass

3.9
restoring force
force exerted by the deformed structure or structural elements which tends to move the structure or structural elements to the original position

3.10
seismic force distribution factor of the *i*th level
$k_{F,i}$
factor to distribute the seismic base shear to the ith level, which characterizes the distribution of seismic forces in elevation, where $\sum k_{F,i} = 1$

Note 1 to entry: See Annex C.

3.11
seismic hazard zoning factor
k_Z
factor to express the relative seismic hazard of the region

3.12
seismic shear factor
factor to give seismic shear of one level, that is defined as the seismic shear of the level divided by the weight of the structure above the level

3.13
seismic shear distribution factor of the *i*th level
$k_{V,i}$
ratio of the seismic shear factor of the *i*th level to the seismic shear factor of the base, which characterizes the distribution of seismic shears in elevation where $k_{V,i}$ = 1 at the base and usually becomes largest at the top

Note 1 to entry: See Annex C.

3.14
severe earthquake ground motion
ground motion used for ULS caused by an earthquake that could occur at the site

Note 1 to entry: See Annex A.

3.15
soil-structure interaction
effect by which structure and surrounding soil mutually affect their overall response

3.16
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

3.17
structural design factor
k_D
factor to reduce seismic forces or shears to levels to be used for design, taking into account ductility, acceptable deformation, restoring force characteristics, and overstrength of the structure

4 Symbols and abbreviated terms

$F_{E,s,i}$	design lateral seismic force of the *i*th level of a structure for SLS
$F_{E,u,i}$	design lateral seismic force of the *i*th level of a structure for ULS
$F_{G,i}$	gravity load at the *i*th level of the structure
$k_{E,s}$	representative value of earthquake ground motion intensity for SLS
$k_{E,u}$	representative value of earthquake ground motion intensity for ULS
k_R	ordinate of the normalized design response spectrum
k_S	soil factor
n	number of levels above the base
SLS	serviceability limit state
SRSS	square root of sum of squares
SSI	soil-structure interaction
ULS	ultimate limit state

$V_{E,s,i}$ design lateral seismic shear of the ith level of a structure for SLS

$V_{E,u,i}$ design lateral seismic shear of the ith level of a structure for ULS

$\gamma_{E,s}$ load factor as related to reliability of the structure for SLS

$\gamma_{E,u}$ 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 reduce damage to property.

In addition to these, societal goals for the environment should be considered.

It is recognized that to give complete protection against all earthquakes is not economically feasible for most types of structures. This document 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)]. Higher reliability for this limit state should be provided for structures with high consequence of failure.

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)].

Structural integrity should also be examined by considering the behaviour of the structure after exceeding each of the limit states (SLS and ULS). If it is essential that services (e.g. mechanical and electrical equipment including their distribution systems) retain their functions after severe or moderate earthquake ground motions, then the seismic actions should be evaluated in accordance with the requirements of ISO 13033. The structure itself should also be verified that essential functions remain operational under the same level of the motions.

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

NOTE 2 Following an earthquake, earthquake-damaged structures might need to be evaluated for safe occupation during a period of time when aftershocks occur. This document, 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 Site conditions

Conditions of the site 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).

In the case of liquefaction prone sites, appropriate foundations and/or ground improvement should be introduced to accommodate or control such phenomena (see ISO 23469).

In areas prone to tsunami hazard, certain important structures (vertical evacuation refuges, hospitals, emergency communication facilities, etc.) are required to resist tsunami actions (see Annex O).

6.2 Structural configuration

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

6.2.1 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).

6.2.2 Vertical irregularities

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

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 nonstructural elements

The structure, including nonstructural as well as structural elements, should be clearly defined as a seismic force-resisting system which can be analysed. In computing the earthquake response of a structure, the influence of not only the structural system elements but also nonstructural walls, partitions, stairs, windows, etc. should be considered when they are significant to the structural response.

NOTE Nonstructural elements are often neglected in seismic analysis. In many cases, the nonstructural elements can provide additional strength and stiffness to the structure, which may result in favourable behaviour during earthquakes which justifies their being neglected. However, in some cases, the nonstructural elements can cause unfavourable behaviour. Examples are: spandrel walls that reduce clear height of reinforced concrete columns and cause the brittle shear failure to the columns, and unsymmetrical arrangement of partition walls (which are considered to be nonstructural elements) that causes large torsional moments to the structure. Therefore, it is important to consider all elements as they behave during earthquakes. If neglecting the stiffness and strength of nonstructural elements does not cause any unfavourable behaviour, they need not be included in seismic analysis. ISO 13033 provides additional criteria regarding when nonstructural components should be included in the building seismic analysis model.

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. Adequate post-elastic performance should be provided by appropriate choice of the structural system and/or ductile detailing. The structure should have adequate strength for the applied seismic actions and sufficient ductility to ensure adequate energy absorption (see Annex D). Special attention should be given to suppressing the low ductile 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, in order to restrict damage for moderate earthquake ground motions and to avoid collapse or other similar forms of structural failure for severe earthquake ground motions.

For long period structures such as high-rise buildings and seismically isolated buildings, effects of repeated large displacement response should be evaluated for severe ground motions with long period and long duration and limited to be within the deformation capacity.

NOTE There are two kinds of deformation to control: (1) inter-storey drift to restrict damage to nonstructural elements and (2) total lateral displacement to avoid damaging contact with adjoining structures (see Annex L).

6.6 Response control systems

Response control systems for structures, e.g. seismic isolation or energy dissipating devices, can be used to ensure continuous use of the structure for moderate and, in some cases, severe earthquake ground motions and also to prevent collapse during severe earthquake ground motions (see Annex M).

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 should be taken either as variable actions or accidental actions.

Structures should be verified against design values of seismic actions for ULS and SLS.

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

NOTE The verification for the SLS can be omitted provided that it is satisfied through the verification for the ULS. The verification of the SLS can also be omitted in low seismicity 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 should be performed either by dynamic analysis or by equivalent static analysis. In both cases, the dynamic properties of the structure should be taken into consideration.

When performing nonlinear analysis, the sequence of nonlinear behaviours of the structure, including the formation of the collapse mechanism, should be determined when nonlinear behaviour is anticipated for severe earthquake ground motions.

NOTE Nonlinear static analysis can be used to determine collapse mechanisms (see Annex H and Annex I).

7.2.1 Equivalent static analysis

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

7.2.2 Dynamic analysis

A dynamic analysis should be performed for structures whose seismic response may not be predicted accurately by an equivalent static analysis. Examples include those structures with irregularities of geometry, mass distribution or stiffness distribution, or very tall structures at sites with high seismic hazard (see Annex K). 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. Dynamic analysis is classified as either a) the response spectrum analysis, b) linear response history analysis or c) nonlinear response history analysis (see Annex H).

7.2.3 Nonlinear static analysis

Structures where nonlinear sequence of behaviour is difficult to predict should utilize nonlinear static analysis to determine the sequence (see Annex I).

7.3 Criteria for determination of seismic actions

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

7.3.1 Seismicity of the region

The seismicity of the region where a structure is to be constructed is usually indicated by mapping a seismic zoning parameter [peak ground motion value(s) or design ground motion spectral response value(s)], which should be based on either the seismic history or on seismological data of the region (including active faults), or on a combination of historical and seismological 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 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, spectral response parameters that are related to smoothed response spectra, input energy, etc. Often, these parameters are determined by a probabilistic seismic hazard analysis to give uniform hazard for a range of natural periods of vibration. In some cases, the hazard analysis is extended to encompass the variation in hazard level with probability level and to integrate that variation with structural fragility to reach a consistent reliability against collapse.

7.3.2 Site conditions

Dynamic properties of the supporting soil layers of the structure should be investigated and the effect on the ground motion at the site should be considered. Geographical and geological conditions and influence of deep subsurface structure (basin effects) should also be taken into consideration.

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 The properties of earthquake ground motions including intensity, frequency content and duration of motion are important features as far as the destructiveness of earthquakes is concerned. Furthermore, structures constructed on soft ground often suffer damage due to uneven or large settlements during earthquakes if not constructed on deep foundations.

7.3.3 Dynamic properties of the structure

Dynamic properties, such as periods and modes of vibration and damping, 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. Nonlinear behaviour of the structural elements should also be taken into account (see 8.1.1). 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.

7.3.4 Consequence of failure of the structure

Consequence of possible failures as well as expense and effort required to reduce the risk of those failures should be taken into account. By considering them and minimizing risk, design with a higher reliability level is appropriate 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). For high-rise buildings, also see Annex K. For national and political economic reasons, a higher level of reliability may be required in urban areas with a high damage potential and a high concentration of capital investment.

NOTE The load factors as related to reliability of the structure $\gamma_{E,u}$ and $\gamma_{E,s}$ (see 8.1) are generally increased when consequence class is high (see Annex A). For response history analysis, the input ground motions are either amplified or more stringent acceptance criteria are used, consistent with the increase of the desired reliability.

7.3.5 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. Spatial variation of wave due to the differences of the ground condition and subsurface geological structure should also be considered.

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.

8.1.1 ULS

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

$$F_{E,u,i} = \gamma_{E,u}\ k_Z\ k_{E,u}\ k_S\ k_D\ k_R\ k_{F,i} \sum_{j=1}^{n} F_{G,j} \tag{1}$$

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

$$V_{E,u,i} = \gamma_{E,u}\ k_Z\ k_{E,u}\ k_S\ k_D\ k_R\ k_{V,i} \sum_{j=i}^{n} F_{G,j} \tag{2}$$

where

$\gamma_{E,u}$ 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);

$k_{E,u}$ 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_S is the ratio of the earthquake ground motion intensity considering the effect of soil conditions to the earthquake ground motion intensity for the reference site condition (see Annex A);

k_D is the structural design factor to be specified for various structural systems according to their ductility, acceptable deformation, restoring force characteristics, and overstrength (see Annex D);

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 B) and damping property of the structure (see Annex G);

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

$k_{V,i}$ 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 shears in elevation, where $k_{V,i} = 1$ at the base and usually becomes largest at the top (see Annex C);

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

n is the number of levels above the base.

8.1.2 SLS

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

$$F_{E,s,i} = \gamma_{E,s}\ k_Z\ k_{E,s}\ k_S\ k_R\ k_{F,i} \sum_{j=1}^{n} F_{G,j} \tag{3}$$

or the design lateral seismic shear for SLS, $V_{E,s,i}$, may be used instead of the above seismic force:

$$V_{E,s,i} = \gamma_{E,s}\ k_Z\ k_{E,s}\ k_S\ k_R\ k_{V,i} \sum_{j=i}^{n} F_{G,j} \tag{4}$$

where

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

$k_{E,s}$ 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);

$k_{E,u}$ and $k_{E,s}$ 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 $\gamma_{E,u}$ and $\gamma_{E,s}$ (see Table A.3).

The values of the gravity load should be equal to the total permanent load plus a probable variable imposed load (see Annex C). 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 might be different. For the combination of actions, see ISO 2394.

8.2 Seismic action effects within the seismic force-resisting system

The two horizontal and the vertical components of the earthquake ground motion and their spatial variation, leading to torsional excitation of structures, should be considered (see Annex 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 storeys; inaccuracy of computed eccentricity; and rotational components of earthquake ground motions.

Modelling of the structure should include realistic stiffness of structural elements (including cracking where pertinent, especially at ULS). Where the stiffness of horizontal diaphragm system(s) connecting the frames resisting horizontal seismic forces is very low and transfer of horizontal forces between horizontal lines of seismic resistance is negligible, each line of resistance may be analysed independently with effective mass in its tributary area instead of constituting and analysing a three-dimensional model of the total structure (flexible diaphragm assumption).

NOTE 1 Seismic actions in any direction do not always attain their maxima at the same time.

NOTE 2 The vertical component of the earthquake ground motion is characterized by higher frequencies than the horizontal component. The peak vertical acceleration is usually less than the peak horizontal acceleration; however, in the vicinity of the fault, the vertical peak can be higher than the horizontal.

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 nonlinear dynamic analyses are recommended.

NOTE 3 Corner columns of structures are subjected to large seismic actions because of the combined effects of torsional response combined with translational response from the two horizontal components of motions.

8.3 Seismic actions on parts of structures

When the seismic actions for the parts of the structure are evaluated by an equivalent static analysis, appropriate factors for seismic forces or shears should be used, taking into account higher mode effects of the structure including the parts (see Annex C). Seismic actions larger than those given in 8.1 may 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 accordance with the requirements of ISO 13033.

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 H).

a) An appropriate 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 established, taking into account the seismicity and site 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 response history analysis for linear or nonlinear systems.

9.3 Response spectrum analysis

A design response spectrum should be defined as the input to perform a response spectrum analysis. This spectrum may either be a) a code specified response spectrum for the site or b) a site-specific design response spectrum developed considering the proper damping ratio (see Annex G). The design response spectrum should be smoothed.

In the response spectrum analysis, the maximum dynamic response is usually obtained by statistical combinations, taking the predominant vibration modes into consideration (see Annex H). Sufficient numbers of modes should be considered.

NOTE 1 Usually, large amount of post-elastic deformation and effects of duration of seismic actions cannot be considered in response spectrum analysis.

NOTE 2 Higher mode effects on equivalent linear system can be evaluated by CQC or SRSS (see Annex H).

9.4 Response history analysis and earthquake ground motions

A set of earthquake ground motions is required as input in order to perform a response history analysis. These motions are either recorded and/or simulated earthquake ground motions that are selected and scaled to generally match the design response spectrum for the site. For both types of ground motions, 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, recurrence period of past earthquakes, distance to active faults, source characteristics of possible earthquakes, uncertainty in the prediction, design service life of the structure, and consequence of failure of the structure.

For that purpose, reference ground motion, which is independent of the characteristics of the structures, should be evaluated using simulated or recorded ground motions as the ground motion at the free surface of the ground, at bedrock, or at an equivalent bedrock depth. Then seismic action should be evaluated from the reference ground motion, considering the effect of various factors such as dynamic behaviour of structures and soil-structure interaction.

9.4.1 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;

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

Recorded earthquake ground motions should be scaled according to the corresponding limit state, seismicity of the site, and dynamic linear and nonlinear characteristics of the structure.

9.4.2 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. Simulated ground motions may be obtained by techniques as follows:

— spectral matching techniques;

— fault-rupture simulations based on earthquake scenarios;

— stochastic representations, e.g. white noise.

Simulated earthquake ground motions should be established according to the corresponding limit state, seismicity of the site, and dynamic linear and nonlinear characteristics of the structure.

NOTE 1 The parameters of the simulated earthquake ground motions are predominant periods, spectral configuration, time duration (time envelope of the simulated motions), intensity, amount of energy input to the structure, etc.

NOTE 2 Earthquake scenarios are based on the information of tectonic plates, seismic fault parameters, slip distribution, etc.

NOTE 3 Components of simulated ground motions include effects of coherence.

NOTE 4 In terms of classification of ground motion as design seismic inputs, simulated earthquake ground motions that are generated to match the elastic response spectra can be called artificial earthquake ground motion.

9.5 Model of the structure

The analytical model of the structure should represent the dynamic characteristics 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 characteristics can be estimated through analytical procedures and/or experimental results. Consideration should be given to the following:

a) the mass should include the mass of permanent construction and an appropriate portion of the imposed loads;

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

c) damping in fundamental and higher modes of vibration (see Annex G);

d) restoring force characteristics of the structural elements in linear and nonlinear ranges including ductility properties and the effect of cracking in concrete and masonry construction;

e) effects of nonstructural elements on the behaviour of the structure;

f) effects of torsion in linear and nonlinear ranges;

g) effects of axial deformation of columns and other vertical elements, or overall bending deformation of the structure;

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

i) effects of floor diaphragm stiffness, including cracking where appropriate.

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

9.6 Evaluation of analytical results

When dynamic analysis is carried out, the evaluation of seismic actions and/or action effects 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.

Therefore, it is recommended that the base shear obtained from the dynamic analysis should be compared with the base shear of the equivalent static analysis, and the design base shear should have some limits as a percentage of the base shear of the equivalent static analysis in case the dynamic analysis gives lower base share.

10 Nonlinear static analysis

In nonlinear static analysis, a structure is subjected to lateral forces that are increased until the structure may collapse. The forces represent seismic forces induced by earthquake ground motions, where the configuration of the forces may be proportional to the design seismic forces or forces caused by the fundamental mode of the structure. The seismic forces are applied incrementally as static loads until a nonlinear state is encountered in a modelled member or connection. The member/connection properties are then adjusted to account for the encountered nonlinearity and then additional incremental forces applied. The process continues until the structural model reaches analytical instability (i.e. collapse) or a target global structural displacement is achieved. The analysis is known as “pushover analysis” and gives information on the nonlinear capacity, deformation, sequence of plastic hinge formation, failure mechanism of the structure, etc.

The obtained shear vs. deflection curves can be converted to a single curve for the equivalent single-degree-of-freedom (SDOF) of the structure. The curve that plots the shear against deflection for the equivalent SDOF is called the capacity spectrum and can be compared with the demand spectrum (S_a – S_d spectrum) to verify the seismic performance of the structure (see Annex I).

11 Estimation of paraseismic influences

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

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 (sometimes called importance 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 depend on the representative value of the earthquake ground motion intensity and are determined for corresponding limit state by considering:

a) the required degree of reliability,

b) the variability of seismic actions,

c) the uncertainty associated with idealization of seismic actions and structures.

A.1.2 Required degree of reliability

The required degree of reliability depends mainly on the consequence of possible failures. The consequence class should be determined from the viewpoint of possible consequences of failure during and/or after earthquakes in terms of, e.g. loss of lives, human injuries, potential economic losses, social inconveniences and environmental impact. The extent and magnitude of the consequence can depend on the context of projects and differ from different perspectives. Thus, these should be carefully determined by considering consequences for all relevant stakeholders such as owners, suppliers and users.

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 consequence class:

— 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 consequence class:

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

c) Low consequence class:

— structures with low risk to human lives and injuries, e.g. sheds for cattle or plants.

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 idealization 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.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. The factor, k_Z, is typically determined for a rock site condition.

In practical applications, a set of discrete values may be specified based on the seismic hazard maps available. In general, these maps do not reflect modifications caused by the effects of the soil profile at a specific site or the influences of near-faults. Therefore, for a specific site, k_Z should be multiplied by an additional factor k_R, which is determined as a function of the soil profile, mapped value of k_Z, earthquake magnitude of the dominant earthquake source and distance to nearby active faults (see Annex B).

NOTE From the perspective of code making, there is a freedom of choice regarding the way the relevant influences on seismic action effect are considered in utilizing k_Z and k_R. For example, a single factor (instead of two factors k_R and k_Z in the formulation above) can be adopted to represent all the relevant 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.

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.

A.4 Reference information for determination of factors $\gamma_{E,u}$, $\gamma_{E,s}$, k_Z, $k_{E,u}$, $k_{E,s}$ and k_S

The results obtained by seismic hazard analysis are used as reference information for determination of the factors $\gamma_{E,u}$, $\gamma_{E,s}$, k_Z, $k_{E,u}$ and $k_{E,s}$ (see A.1, A.2, A.3) as well as for determination of design ground motions. The seismic hazard analysis should be conducted, taking into account the latest findings in seismology as follows:

— regional seismicity (active faults, diffuse seismicity, etc.);

— propagation path characteristics from the source to the site;

— amplification due to deep subsurface structure;

— amplification due to shallow soil;

— epistemic uncertainty (model uncertainty) in predicted seismicity and in ground motion.

NOTE Effects of amplification of ground motion due to subsurface structure and shallow soil are usually considered in factor k_R (see Annex B).

The factor k_S is usually described as the ratio of the peak acceleration (usually at the basement of structure) considering the effect of soil conditions to the peak ground acceleration for the reference site condition. It can be modelled as the function of $k_Z\ k_{E,u}$ or $k_Z\ k_{E,s}$ as well as that of the soil condition (e.g. 30 m average shear wave velocity). Example values of k_S are tabulated in Table A.1, considering the nonlinear characteristics of ground motion amplification. k_S is usually assumed to be constant and to be unity for high seismicity regions.

Table A.1 — Example values of k_S

Soil condition	$k_Z\ k_{E,u}$ or $k_Z\ k_{E,s}$				
	<0,1	0,2	0,3	0,4	>0,5
Rock	1,0	1,0	1,0	1,0	1,0
Stiff soil	1,6	1,4	1,2	1,1	1,0
Soft soil	2,5	1,7	1,2	0,9	0,9

A.5 Examples of load factors associated with representative values

The load factors $\gamma_{E,u}$ and $\gamma_{E,s}$ and earthquake ground motion intensities $k_{E,u}$ and $k_{E,s}$ are determined as a function of the reference period and the probability of exceedance in the reference period. For a given probability of exceedance in a reference period, a larger value of the earthquake ground motion intensity results in a smaller value of load factor and vice versa. $\gamma_{E,u}$ and $\gamma_{E,s}$ are, as examples, listed in Tables A.2 and A.3 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.

It is common to select a return period of approximately 500 years for the ULS, although some nations have defined longer intervals. In areas where damaging earthquakes occur frequently, the return interval selected for the SLS is generally no more than the service life of the facility, although in some nations, this return interval varies with the consequence class of the facility. SLS may be implicitly treated by appropriate selection of the ULS criteria. In areas where damaging earthquakes are uncommon, the SLS may be ignored. It is also common practice to place judgmental limits on the ground motion values computed from probabilistic seismic hazard analysis. In many nations, these limits begin to be applied where the ULS ground motion parameter exceeds a peak ground acceleration of 0,4 g. Another way to view is to have same return period with different load factors for SLS and ULS.

An example using the unity load factor for the normal consequence class of structures is shown in Table A.2, where the return period for the corresponding limit state is taken into account by $k_{E,u}$ or $k_{E,s}$. In Table A.3, a common representative value k_E is used and the level of consequence is taken into account by $\gamma_{E,u}$ or $\gamma_{E,s}$ for the corresponding limit state. In Table A.2, the return period of 500 years is used for the ultimate limit state. Longer return period (e.g. 2 500 years) may be appropriate for the return period instead of 500 years, if the ground motion for the ultimate limit state is considered as a collapse ground motion. By adopting a longer return period for design, rare earthquake events caused by such as active faults are more likely to be included in seismic demand modelling, especially in low- or medium-seismicity regions. The appropriate return period is evaluated based on the examination of the safety margin of conventionally designed structures.

Table A.2 — Example 1 for load factors $\gamma_{E,u}$ and $\gamma_{E,s}$ and representative values $k_{E,u}$ and $k_{E,s}$ (where $k_{E,u} \neq k_{E,s}$, for normal soils in high seismic area)

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

Table A.3 — Example 2 for load factors $\gamma_{E,u}$ and $\gamma_{E,s}$ and representative value k_E (for normal soils in high seismic area)

Limit state	Consequence class	Load factors $\gamma_{E,u}$ or $\gamma_{E,s}$	k_Z	$k_E = k_{E,u} = k_{E,s}$	Return period for k_E
Ultimate	a) High	3,0 to 4,0	1,0	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			

Annex B
(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 of Formulae (B.1) to (B.4):

$$k_R = 1 + \left(k_{R0} - 1\right)\frac{T}{T_a} \quad \text{for } 0 \le T < T_a \tag{B.1}$$

$$k_R = k_{R0} \quad \text{for } T_a \le T < T_v \tag{B.2}$$

$$k_R = k_{R0}\frac{T_v}{T} \quad \text{for } T_v \le T < T_d \tag{B.3}$$

$$k_R = k_{R0}\frac{T_v\, T_d}{T^2} \quad \text{for } T_d \le T \tag{B.4}$$

where

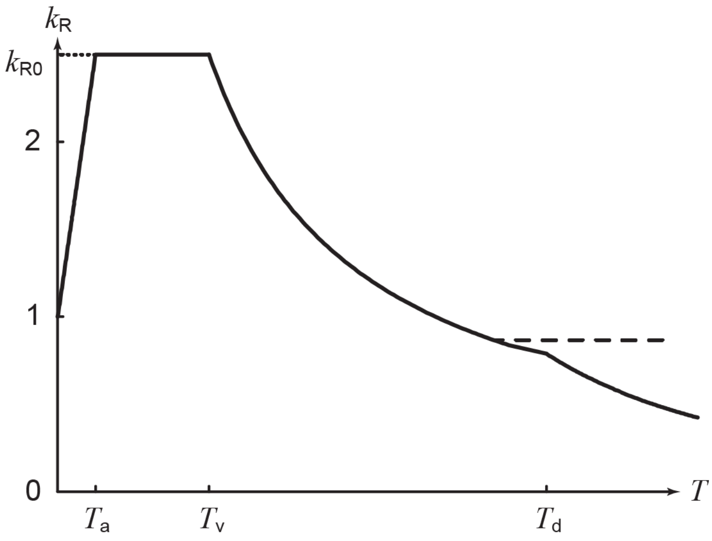
k_R is the ordinate of the acceleration response spectrum normalized by the representative value of earthquake ground motion acceleration;

k_{R0} is the ratio of the maximum acceleration response over a short period range to the representative value of earthquake ground motion acceleration;

T is the fundamental natural period of the structure;

T_a, T_v and T_d are the corner periods of the spectrum, as illustrated in Figure B.1.

The quantities of k_{R0}, T_a, T_v and T_d are dependent on the soil profile, nonlinear characteristics of soil, earthquake magnitude of the dominant earthquake source and distance to nearby active faults as well as 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_{R0} may be taken as 2 to 3.



Key

k_R	acceleration response spectrum normalized by the peak ground acceleration for design purpose
k_{R0}	ratio of the maximum acceleration response to the peak ground acceleration
T	fundamental natural period of the structure
T_a, T_v, T_d	corner periods of the spectrum
....	k_R of short period structures for design
– – – –	lower limit of k_R for design at long periods

Figure B.1 — Normalized design response spectrum

Formula (B.2) shows that k_R is constant for $T_a \leq T < T_v$ (acceleration constant range). For a sinusoidal motion, the velocity amplitude is calculated as the acceleration amplitude divided by the circular frequency $\omega = \frac{2\pi}{T}$. Then, Formula (B.3) implies that the velocity amplitude is constant for $T_v \leq T < T_d$ (velocity constant range). Similarly, Formula (B.4) implies that the displacement amplitude is constant for $T_d \leq T$ (displacement constant range). Therefore, T_a, T_v and T_d are closely related to the response of acceleration, velocity and displacement, respectively.

T_a may be taken as $\frac{1}{5}$ to $\frac{1}{2}$ of T_v and T_v for horizontal motions can be taken as follows:

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

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

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

When considering the soil profile effect, deep subsurface structure around the site as well as shallow soil structure at the site should be considered.

Figure B.1 indicates that k_R is unity at $T = 0$ and linearly increases to k_{R0} at $T = T_a$. It is recommended, however, to use $k_R = k_{R0}$ for $0 < T \leq T_a$, as the horizontal dotted line of Figure B.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 design factor k_D for short period structures.

For determination of seismic forces at longer periods, it is recommended that a lower limit be considered as indicated by the horizontal dashed line in Figure B.1. The value of this level may be taken as $\frac{1}{3}$ to $\frac{1}{5}$ of k_{R0}. 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.

The fundamental natural period T of the structure 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.

Annex C
(informative)

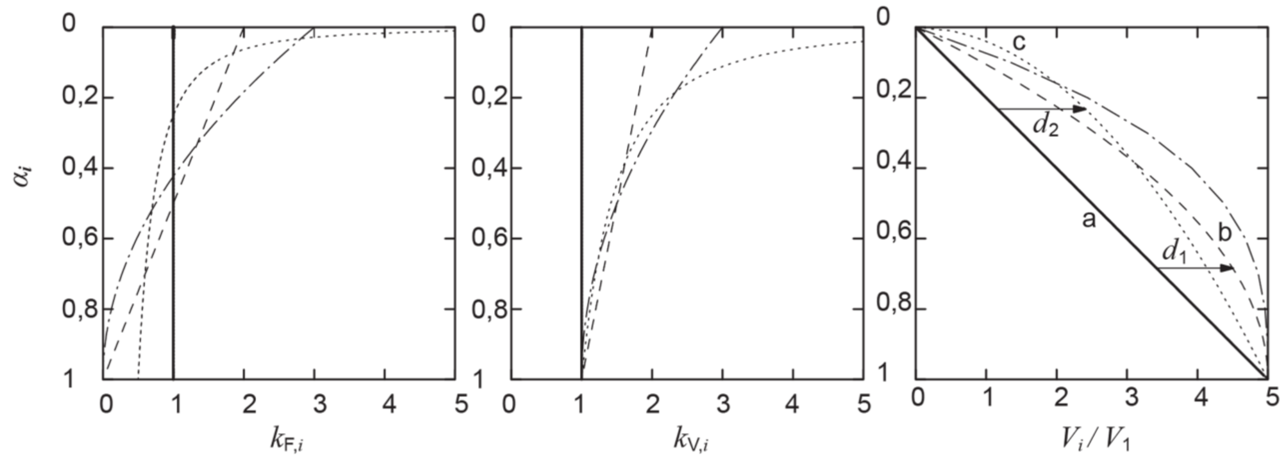
Seismic force distribution parameters for equivalent static analysis

General characteristics of seismic force distribution parameters above the base for equivalent static analysis are as follows.

a) For extremely stiff structures, for example, with period less than 0,05 s, whole parts from the top to the base move along with the ground motion. Then the distribution of seismic forces is uniform and the seismic shears increase linearly from the top to the base. This is called the uniform distribution of seismic forces (see the solid lines of Figure C.1). In Figure C.1, the normalized weight α_i [see Formula (C.5)] is used as the ordinate, instead of height.

b) For low-rise buildings, the distribution of seismic forces becomes similar to the inverted triangle. Then the distribution of seismic shears is assumed to be a parabola whose vertex locates at the base. This is called the inverted triangular distribution of seismic forces (see the dashed lines of Figure C.1).

c) For high-rise buildings, seismic forces at the upper part become larger because of a higher mode effect. If the structure 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 shears becomes a parabola whose vertex locates at the top (see the dotted lines of Figure C.1). This may be called the distribution for shear type structure subjected to white noise excitation or simply "$\sqrt{\alpha}$ distribution", because the shear distribution is proportional to $\sqrt{\alpha_i}$.



Key

α_i	normalized weight
$k_{F,i}$	seismic force distribution factor
$k_{V,i}$	seismic shear distribution factor
V_i / V_1	normalized seismic shear
————	ν = 0 in Formula (C.1), or k_1 = 0, k_2 = 0 in Formula (C.4)
- - - - - -	ν = 1 in Formula (C.1), or k_1 = 1, k_2 = 0 in Formula (C.4)
·········	k_1 = 0, k_2 = 1 in Formula (C.4)
-·-·-·-	ν = 2 in Formula (C.1)

Figure C.1 — Seismic force distribution parameters

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

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

where

$F_{G,i}$ 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 ν may be taken as follows (where T is the fundamental period of the structure):

— ν = 0 to 1 for low-rise buildings (up to five-storey buildings), or structures for which $T \leq 0{,}5$ s;

— ν = 1 to 2 for mid-rise buildings, or structures for which 0,5 s < $T \leq 1{,}5$ s;

— ν = 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 Formula (C.1) are shown in Figure C.1 as solid lines for ν = 0, as dashed lines for ν = 1, and as dash-dotted lines for ν = 2.

Formula (C.1) does not give an appropriate distribution for high-rise buildings, even if $\nu = 2$ (see dash-dotted lines in Figure C.1). Then the seismic force distribution factor, $k_{F,i}$, for high-rise buildings may be determined by

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

$$k_{F,i} = \left(1 - \rho\right) \frac{F_{G,i}\, h_i}{\sum_{j=1}^{n} F_{G,j} h_j} \tag{C.3}$$

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

Since Formulae (C.2) and (C.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 other types of distribution that can be derived as follows.

Three of four types of the normalized seismic shear in the right of Figure C.1 are denoted as "a", "b" and "c" that correspond to the above items a), b) and c), respectively. The normalized seismic shear $V_i \,/\, V_1$ (seismic shear of the ith level divided by the base shear) is given as follows:

a) For the uniform distribution of seismic forces (see the solid line "a" of Figure C.1),

$$V_i \,/\, V_1 = \alpha_i$$

b) For the inverted triangular distribution of seismic forces (see the dashed line "b" of Figure C.1),

$$V_i \,/\, V_1 = 1 - (1 - \alpha_i)^2 = 2\alpha_i - \alpha_1{}^2$$

c) For the $\sqrt{\alpha}$ distribution (see the dotted line "c" of Figure C.1),

$$V_i \,/\, V_1 = \sqrt{\alpha_i}$$

The difference d_1 between "b" and "a" is given by $d_1 = \alpha_i - \alpha_i{}^2$, and the difference d_2 between "c" and "a" is $d_2 = \sqrt{\alpha_i} - \alpha_i$. Therefore, adjusting the factors k_1 and k_2, various types of the normalized shear distribution can be expressed as follows:

$$V_i \,/\, V_1 = \alpha_i + k_1 d_1 + k_2 d_2$$

$$= \alpha_i + k_1(\alpha_i - \alpha_1{}^2) + k_2(\sqrt{\alpha_i} - \alpha_i)$$

Dividing the above formula by α_i gives the seismic shear distribution factor $k_{V,i}$, that is the seismic shear factor of the ith level normalized by the base shear factor, as follows.

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

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, and α_i is the normalized weight that is given by

$$\alpha_i = \frac{\sum_{j=i}^{n} F_{G,j}}{\sum_{j=1}^{n} F_{G,j}} \tag{C.5}$$

The normalized weight α_i is used instead of the height h_i above the base, because the normalized weight is more convenient and rational to express distributions of seismic force parameters. Because of using α_i, various types of seismic force parameter can be compared as Figure C.1.

In the case of a structure with uniform mass distribution, the normalized weight α_i may be approximated by the height h_i using Formula (C.6):

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

Distributions of seismic force parameters given by Formula (C.4) are shown as solid lines in Figure C.1 for $k_1 = 0$ and $k_2 = 0$ (uniform distribution of seismic forces), as dashed lines in Figure C.1 for $k_1 = 1$ and $k_2 = 0$ (inverted triangular distribution of seismic forces), and as dotted lines in Figure C.1 for $k_1 = 0$ and $k_2 = 1$ ($\sqrt{\alpha}$ distribution).

Therefore, the factors k_1 and k_2 may be taken as follows:

— $k_1 \approx 1$ and $k_2 \approx 0$ for low-rise buildings, or structures for which $T \leq 0{,}5$ s;

— $k_1 \approx 0{,}5$ and $k_2 \approx 0{,}5$ for mid-rise buildings, or structures for which $0{,}5 \text{ s} < T \leq 1{,}5$ s;

— $k_1 \approx 0$ and $k_2 \approx 1$ for high-rise buildings, or structures for which $T > 1{,}5$ s.

Incidentally, substituting $k_1 = k_2 = 2T / (1 + 3T)$, Formula (C.4) becomes as shown in Formula (C.7):

$$k_{V,i} = 1 + \left(\frac{1}{\sqrt{\alpha_i}} - \alpha_i\right) \frac{2T}{1 + 3T} \tag{C.7}$$

This is denoted as A_i in the Japanese seismic code that has been used since 1981.

When the seismic actions for the parts of the structure projecting from the roof are evaluated, the seismic shear factor can be calculated by Formula (C.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, $k_{F,i}$ or $k_{V,j}$ should be adjusted to take account of such behaviour.

Annex D
(informative)

Structural design factor for linear analysis

The structural design factor k_D is used to reduce seismic forces computed for fixed-base linear elastic models (equivalent static and modal response spectrum analysis) to account for the beneficial effects of anticipated inelastic behaviour and foundation structure interaction, considering the structure's restoring force characteristics, ductility, damping, and overstrength.

The factor can be divided into two factors: namely $k_{D\mu}$ and k_{Ds} and is expressed as the product of them as given in Formula (D.1):

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

where

$k_{D\mu}$ is related to ductility, foundation structure interaction, restoring force characteristics, including damping, and the amount of damage considered acceptable at the ultimate limit state;

k_{Ds} is related to overstrength.

The factor can also be expressed as given in Formula (D.2):

$$k_D = k_{D\mu} \, k_{Ds} = \frac{1}{R} = \frac{1}{R_\mu \, R_s} \tag{D.2}$$

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

Recent studies indicate that $k_{D\mu}$ depends on the structure's natural period of vibration with the possible reduction in strength required remaining minimal for structures having shorter fundamental natural periods. k_{Ds} is a function of the difference between the actual strength and calculated design strength and varies according to the inherent characteristics of the structural system, the unique aspects of a structure's design and 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 design factor k_D with $k_{D\mu}$ may be, for example, as per the values in Table D.1.

Table D.1 — Example of structural design factor k_D and $k_{D\mu}$

Structural system with	$k_{D\mu}$	k_D
Excellent ductility	1/5 to 1/3	1/12 to 1/6
Medium ductility	1/3 to 1/2	1/6 to 1/3
Poor ductility	1/2 to 1	1/3 to 1
The difference between $k_{D\mu}$ and k_D is mainly caused by the overstrength. Calibration from the values in this table shows that k_{Ds} is 1 to 2.		

k_D will be larger where the limit state aims for limited damage rather than near collapse. These ranges of k_D are under continuing investigation (as are the values of $k_{D\mu}$ and k_{Ds}) and may take other values in some circumstances.

Ductility is defined as the ability to deform beyond the elastic limit under cyclic loadings without serious reduction in strength. The ductility factor (usually denoted by μ) is defined as the total deformation divided by the elastic limit deformation.

To achieve levels of ductility $k_{D\mu}$ stated in Table D.1, the configuration of the structure and all the details used are important. The ductility factor chosen should be consistent with the expected inelastic performance of the actual materials, details and configuration of the structural system. Levels of inelastic material strain implied by the chosen ductility factor and structural configuration should be able to be reliably achieved at the ULS. Suitably appropriate detailing requirements may be prescribed in the material design standard being used in conjunction with this document.

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 design 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 detailing of structural elements to assure reliable nonlinear response.

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.

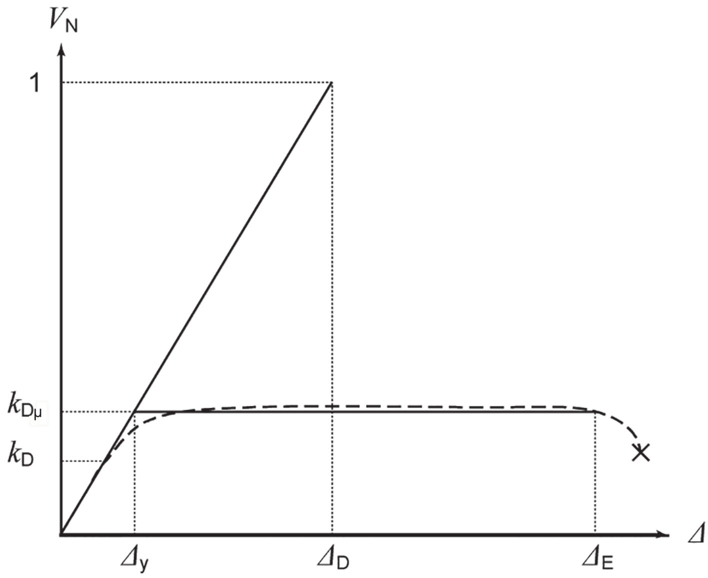
Results of nonlinear dynamic analyses of structures subjected to strong earthquake ground motions indicate that $k_{D\mu}$ (or $1 / R_\mu$) is proportional to $1/\mu$ for longer period structures and $1/\sqrt{2\mu - 1}$ for short period structures, where μ is the ductility factor. Therefore, the maximum lateral displacement Δ_{max} expected in ULS may be estimated by simple formulae as follows (see Figure D.1):

$$\Delta_{max} = \Delta_D = \frac{1}{k_{D\mu}} \Delta_y = R_\mu \Delta_y \tag{D.3}$$

$$\Delta_{max} = \Delta_E = \frac{1}{2}\left(\frac{1}{k_{D\mu}{}^2} + 1\right)\Delta_y = \frac{1}{2}\left(R_\mu{}^2 + 1\right)\Delta_y \tag{D.4}$$

where Δ_y is the lateral displacement calculated by linear analysis for the design lateral seismic forces or shear forces defined in Formula (1) or (2) in the main text.

Generally, Formula (D.3) is applicable to structures with a longer natural period (displacement-constant rule) and Formula (D.4) is to structures with a shorter natural period (energy-constant rule). 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.



Key

V_N	normalized shear
k_D	structural design factor
$k_{D\mu}$	structural design factor related to ductility
Δ	lateral displacement
Δ_y	lateral displacement calculated by linear analysis for the design lateral shear
Δ_D	maximum displacement by displacement-constant rule
Δ_E	maximum displacement by energy-constant rule
– – –	actual shear and displacement curve
×	collapse of the structure

Figure D.1 — Relationship between lateral shear and displacement for idealized elasto-plastic system

Annex E
(informative)

Combination of components of seismic action

E.1 Combination of horizontal components

Among the three components of ground motion, combination of the two horizontal components strongly influences the total seismic actions on the structure, for example:

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

b) axial force of corner columns.

Unless orthogonal pairs of ground motion are applied simultaneously in response history analysis, combination of the two horizontal components of seismic actions should be considered. 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 (ε = 0 means the SRSS method), ε may empirically be taken as 0 to 0,3.

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

$$E = E_x + \lambda E_y$$

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

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 Formulae (E.1) and (E.2) are shown in Figure E.1.

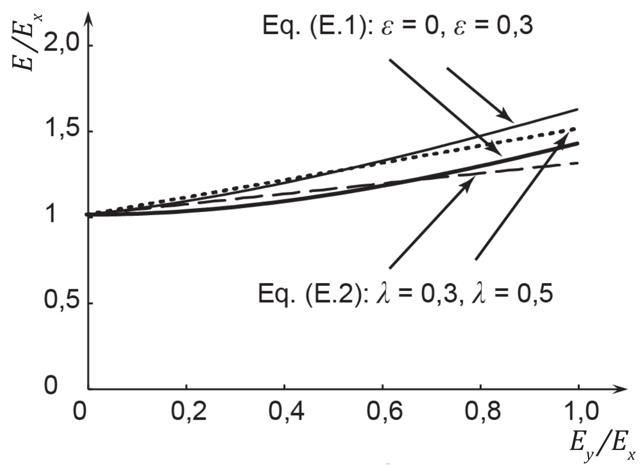


Figure E.1 — Relationships between E / E_x in terms of E_y / E_x according to Formulae (E.1) and (E.2)

E.2 Vertical component

The vertical component, E_z may be evaluated by using Formula (E.3):

$$E_z = k_{\mathrm{E,v}}\, k_{\mathrm{R,z}}\, F_{\mathrm{Ge}} \tag{E.3}$$

where

$k_{\mathrm{E,v}}$ is the vertical peak ground acceleration expressed by the ratio to gravity acceleration, which may be taken as 1/2 to 2/3 of the horizontal peak ground acceleration. However, it is recommended to take 1,0 of the horizontal peak ground acceleration in case that the motion is caused by faults close to the site;

$k_{\mathrm{R,z}}$ is the response amplification, which may be taken as 2,5;

F_{Ge} is the effective gravity loads.

The vertical component, if combined with horizontal components, may be multiplied by the factor, λ, which empirically may be taken as 0,2 to 0,4.

It is recommended to evaluate the vertical component, employing more precise dynamic analysis in cases where effects of the vertical component are critical. Such cases include but not limited to:

a) horizontal structural elements with very long clear spans and long cantilever elements;

b) constructions with high arching forces;

c) concrete columns and shear walls subjected to high shear forces, especially at construction interfaces;

d) isolators of seismic isolating systems.

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 Formula (F.1):

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

where V_i is the seismic shear of the ith level [see Formula (F.2)]:

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

where

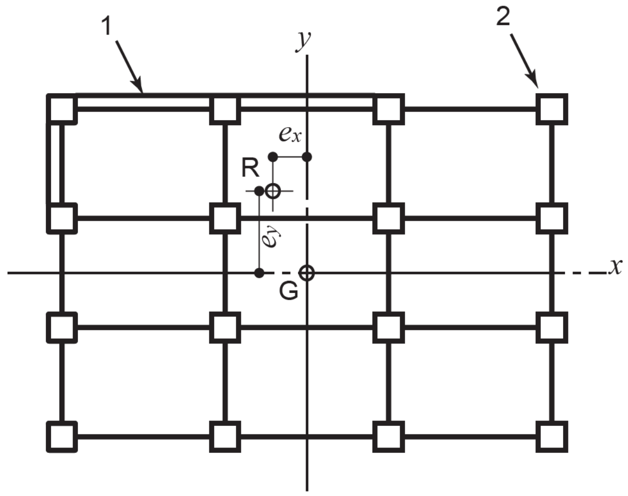
F_j is the seismic force of the jth level;

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
G	centre of mass
R	centre of stiffness
e_x, e_y	eccentricity

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

For equivalent static analysis, the torsion will require amplification to account for dynamic response effects. 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)

Damping ratio

Damping in the structure is classified as follows:

— internal damping of structural elements (both members and joints);

— hysteretic damping derived from hysteresis-based restoring force-deformation relations;

— damping due to nonstructural elements;

— damping due to energy dissipation into the ground derived from superstructure vibration.

In general, these types of damping, except for treating the hysteretic one as it is, are represented by viscous damping in dynamic analysis. The hysteretic damping may be taken into account as a part of the viscous damping in equivalent linear models; otherwise, it should be incorporated in the hysteresis-based restoring force-deformation relations. The latter option leads to more refined results in response history analysis, but involves more calculation effort.

The magnitude of the design seismic force is greatly affected by the value of damping ratio. Unfortunately, there are many unknowns in the nature of damping, thus resulting in large uncertainty about the damping ratio.

In principle, values of damping should be evaluated on the basis of vibration tests, shaking table tests, and earthquake observations of actual structures or full-scale structure models. The range of member deformations in the experiments is recommended near to expected deformations by calculations. If these data are not available, the results of similar structures in the similar conditions may be utilized.

This evaluation method of damping is appropriate for evaluating directly total damping of the structures. In the case of evaluating total damping by summing up damping values derived from the experiments of parts of the structures, careful examinations are required.

Recommended values of damping may be listed in some codes or similar regulations. In such cases, the above-mentioned principle should be taken into consideration.

Structures that have few sources for frictional energy dissipation, such as bare welded steel structures, may require lower values of damping, whereas those with more sources of friction, such as buildings with wood sheathing for example, may increase damping. It should be noted that the damping ratio is affected by the configuration of the structure as well as the type of construction.

The value of the fraction of critical damping (damping ratio) is adopted very often between 0,01 and 0,10, depending on the material, type of structure, their connections and the relative magnitude of the deformations produced. The value leads to increase as the frequency increases but with large fluctuations.

A damping ratio of 0,01 is often employed in wind design, and a similar value is found out at assessing floors and pedestrian bridges subjected to passage of persons.

In evaluating seismic actions, where a larger amount of deformation is considered, a higher value of the damping ratio may be employed. For design purposes, the damping ratio for the fundamental mode of regular structures of steel, concrete or masonry construction is in the range of 0,02 to 0,05 depending on the type of construction and the intensity of the ground motion considered that implies the stress level suffered by the structure.

On modelling of the structures, one of the classical damping matrices is the Rayleigh damping, for which the damping matrix $[C]$ is given as shown in Formula (G.1):

$$[C] = \alpha_0\,[M] + \alpha_1\,[K] \tag{G.1}$$

where

$[M]$	is the mass matrix;
$[K]$	is the stiffness matrix;
α_0 and α_1	are the coefficients to be determined depending upon the damping ratios of two different modes.

The above damping matrix may not provide appropriate damping ratios for modes other than the two modes considered for determining the coefficients α_0 and α_1. In such cases, other damping matrices in which modal damping ratios can be specified individually for multiple modes may be applied.

Energy dissipation due to inelastic behaviour of the structure and structural design factor are described in conjunction of some parameters in Annex D. Normalized design response spectrum in classified soil conditions is mentioned in detail, and principle of capacity spectrum method is also demonstrated in Annex I. Some parts of them are closely related to damping of the structure or damping ratio in both annexes. If needed, related portions are preferable to be referred.

Effects of viscous damping on the overall response become less significant with the increase in hysteretic damping. There are several formulae to obtain some reduction or increment of the acceleration peak for damping ratio different from 0,05. Then the ordinate k_{R0} may be multiplied, for instance, by:

$$k_\zeta = \frac{1,5}{1 + 10\zeta} \tag{G.2}$$

or

$$k_\zeta = \sqrt{\frac{0,1}{0,05 + \zeta}} \tag{G.3}$$

where ζ is the damping ratio of the structure in linear systems. It is recommended not to reduce k_ζ less than 0,55.

Although most seismic codes utilize a constant damping ratio of 0,05, it varies according to the structural material, construction system and behaviour during earthquakes. Some examples of the damping ratio for SLS are as follows.

Reinforced concrete	0,04
Reinforced masonry	0,04
Prestressed concrete	0,03
Welded or bolted (preloaded) steel	0,03
Bolted (non-preloaded) steel	0,05

For ULS, since the inelastic behaviour of structures is significant and hysteretic damping becomes larger, those effects could be included in Formula (G.2) or Formula (G.3), or should be considered, choosing appropriate value of k_D (see Annex D).

In case the inelastic behaviour of structures is not very significant, the damping ratio may be as follows.

Reinforced concrete	0,07
Reinforced masonry	0,07
Prestressed concrete	0,05
Welded or bolted (preloaded) steel	0,04
Bolted (non-preloaded) steel	0,07

Annex H
(informative)

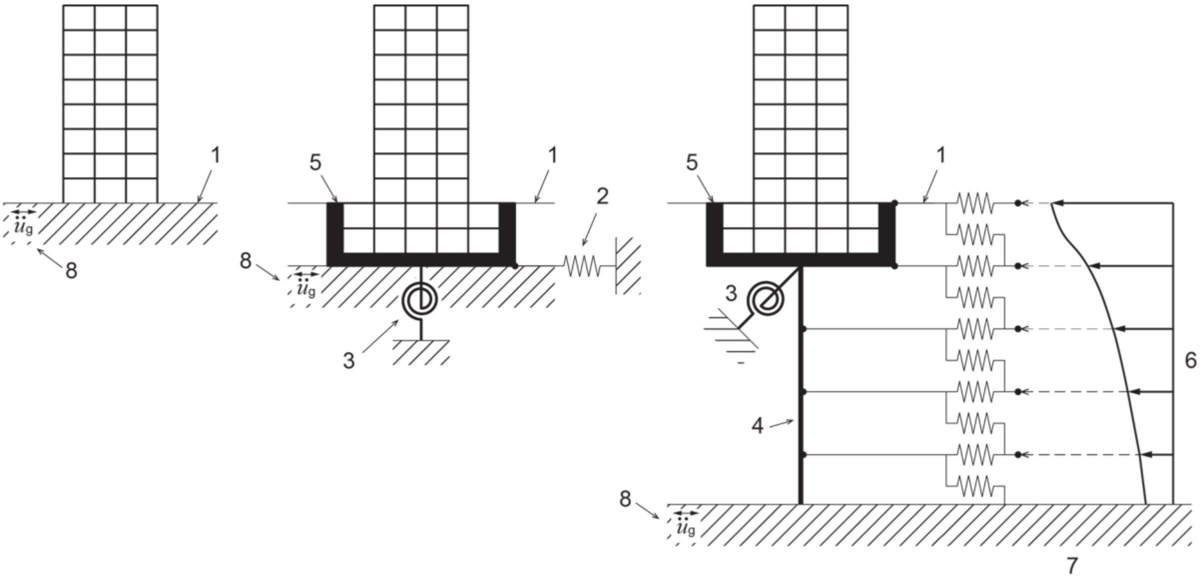
Dynamic analysis

H.1 Model of structure for dynamic analysis

Models of structure for dynamic analysis should include spatial representation of the mass as well as the dynamic characteristics of all elements intended to participate in resistance of earthquake forces. In general, sufficient degrees of freedom to capture significant response characteristics in three dimensions should be included. Planar models may be permitted only when torsional response can be demonstrated to be insignificant. In addition, if horizontal stiffness of a storey can be appropriately represented by a series of translational and rotational springs, one-dimensional lumped mass and spring models may be useful for simple but practical evaluation of seismic action.

Advanced numerical methods that can deal with the continuum mechanics should be utilized if it is necessary to consider the detail of material behaviour of structure and the effect of soil behaviour, etc. These methods are also useful to consider the spatial variation and propagation effect of ground motion.

Models may either be fixed at the base [see Figure H.1 a)] or represent the compliance of supporting soils with appropriate translational and/or rotational springs as illustrated in Figure H.1 b). More detailed soil-foundation-structure interaction models illustrated in Figure H.1 c) are often used when earthquake motion is defined at the bedrock.



a) Fixed model b) Sway-rocking (SR) model c) Interaction model of structure with piles

Key

1 ground level
2 sway spring
3 rocking spring
4 piles
5 foundation/basement
6 forces caused by soil
7 bedrock
8 ground motion acceleration

Figure H.1 — Examples of soil-structure interaction models

H.2 Response spectrum analysis

H.2.1 Method of analysis

Response spectrum analysis is conducted for the site-specific response spectrum established for the purpose of the analysis. In the absence of such a spectrum, the normalized design response spectrum indicated in Annex B multiplied by the maximum ground acceleration for the earthquake ground motion intensity may be employed. Elastic models of structures with same stiffness assumptions for linear response history analysis indicated in H.3.2.1 should be employed in response spectrum analysis. Seismic actions and/or action effects should be evaluated by combining elastic modal response.

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):

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

where

S is the maximum response quantity under consideration;

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

Regardless whether natural frequencies of different modes are closely spaced or not, the combination may be performed using Formulae (H.2) and (H.3) (CQC method) which is derived from the random vibration theory:

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

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

where

ζ_i, ζ_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 Formulae (H.1) and (H.2).

H.2.2 Seismic action and action effect

The response from the combination of modes should be multiplied by an appropriate scaling factor to relate the dynamic analysis base shear to the equivalent static base shear (described in 8.1). For checking ULS, the response should be additionally multiplied by the appropriate structural design factor described in Annex D.

H.3 Response history analysis

H.3.1 Method of analysis

Response history analysis may be classified into linear analysis and nonlinear analysis. Appropriate method should be chosen based on the purpose of the analysis.

H.3.1.1 Linear response history analysis

The purpose of linear response history analysis is to predict the values of element force and global structural deformation response values assuming linear response.

Linear response history analysis is often employed in evaluating seismic action effects for SLS where behaviour of structural elements within elastic limit is assumed. For ULS, however, nonlinear behaviour of structural elements is basically of importance and the element force obtained by the analysis should be multiplied by appropriate structural design factor described in Annex D as in the prescribed response spectrum analysis. The global structural deformation should also be multiplied by the structural design factor and, in addition, be multiplied by the appropriate deflection amplification factor, which has to be established for various types of structural systems.

H.3.1.2 Nonlinear response history analysis

The purpose of nonlinear response history analysis is to predict the values of global structural deformation and individual element strength and deformation demands directly at response levels beyond the elastic limit and to demonstrate either implicitly or explicitly that the structure has sufficient strength, stiffness, damping and deformation capacity to meet the performance goals.

Nonlinear response history analysis is normally employed in evaluating seismic action effects for ULS as nonlinear global structural deformation can be obtained without relying on prescriptive parameters such as structural design factors and deflection amplification factors. In addition, cyclic plastic deformation in each element can be evaluated directly. It should be noted that results of nonlinear response history analysis are to verify structural performances rather than to determine seismic demands derived from combination of factored loads. Appropriate acceptance criteria of the response should be established and applied in the verification.

H.3.2 Restoring force characteristics

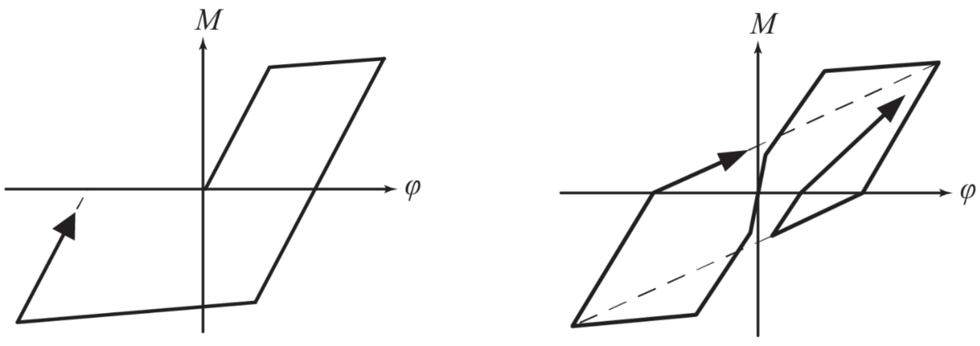
H.3.2.1 Stiffness assumptions for linear analysis

Force-deformation characteristics of structural steel elements should be based on gross section properties and should account for the effects of panel zone stiffness and other sources of deformation in structural joints. Effects of composite action of concrete may also be considered. Force-deformation characteristics of masonry and concrete elements should account for the effective cracked section stiffness.

H.3.2.2 Force-deformation assumptions for nonlinear analysis

Stiffness assumptions of structural elements before effective yield should be basically same as in linear analysis. In some cases, however, initial stiffness of concrete structures without cracks is taken and nonlinear behaviour after cracks and before yielding is accounted to incorporate effects of hysteresis damping in small deformation range. Force-deformation characteristics should be based on existing laboratory testing of similar elements and should account for strength and stiffness degradation in concrete elements due to cyclic loadings within the anticipated range of response. In steel elements, Bauschinger effects are sometimes taken into account. Figure H.2 illustrates examples of hysteretic models.

Forces in elements of structures due to dead and live loads should be taken into account as initial conditions for nonlinear analysis unless such effects are not significant.



a) Normal bilinear model **b) Degrading trilinear model**

Key

M bending moment

φ deflection angle

Figure H.2 — Examples of restoring force characteristic models

Elements which are expected to behave within or nearly within elastic limit may be modelled as linear elements on the condition that such behaviour of the elements is confirmed by the nonlinear analysis.

H.3.3 Input earthquake ground motions

Basically, input earthquake ground motions should be provided for two orthogonal horizontal and a vertical directions. In spatial model analysis, simultaneous input of the ground motion in the two directions may be conducted instead of conducting analyses in the two directions independently and combining the results. Normally, vertical motions are considered separately by more simplified procedures as described in Annex E. The following input motions are often employed:

a) recorded earthquake ground motions;

b) artificial motions of which the response spectrum is compatible with the design spectrum;

c) simulated motions based on characteristics of source and of the site.

H.3.3.1 Recorded earthquake ground motions

When recorded earthquake ground motions are used as input ground motions, they should be appropriately selected to represent the magnitude range, fault distance and site conditions associated with the structure and its design earthquake. The records should be scaled or modified in amplitude so that their linear response spectra match to the site-specific response spectrum established for considered limit state (e.g. SLS or ULS) within a period range that captures the structure's primary response modes, considering potential period lengthening. In the absence of site-specific response spectrum, the normalized design response spectrum indicated in Annex B may be employed instead. 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 or in every future earthquake. Therefore, it is recommended to consider a sufficiently large set of motions to capture a best estimate of mean response and also to provide information on potential variability in response.

H.3.3.2 Artificial earthquake ground motions consistent with response spectrum

Artificial earthquake ground motions are often developed adopting random phases, phase characteristics of recorded ground motions or phase difference models so that their spectra fit the site-specific or normalized design response spectrum prescribed in H.3.3.1. Durations of the accelerograms should be sufficient in the light of the magnitude and other relevant features of considered earthquakes as well as dynamic characteristics of objective structures.

Artificial ground motions may be established either at the ground surface or at the bedrock but it is more rational to establish them at the bedrock which can be used directly in the soil-structure interaction model analysis. When artificial earthquake ground motions are set up at the ground surface, they should reflect the dynamic characteristics of the soil in the deformation range corresponding the intensity of considered earthquakes.

H.3.3.3 Simulated earthquake ground motions

Simulated earthquake ground motions developed based on the design earthquake parameters including the magnitude, fault location, slip distribution, direction of rupture, etc., and also on the travel path mechanism and surface soil characteristics may be employed as input earthquake ground motions. Various simulation methods, some of which are introduced in ISO 23469, have been developed. As the simulated motions can produce considerably intense actions, it is recommended to evaluate their hazard level, such as return period, etc.

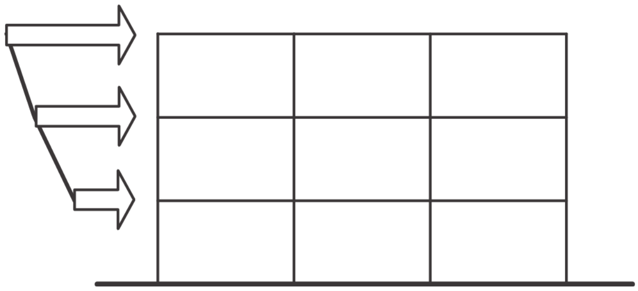
The simulated ground motions are effective especially in demonstrating peculiar characteristics of certain types of earthquakes that are critical in some structures. On the contrary, common demands of ordinary seismic actions may not be incorporated. Consequently, when response history analysis is conducted for simulated earthquake ground motions, analysis for artificial or recorded ground motions should also be conducted.

Annex I
(informative)

Nonlinear static analysis and capacity spectrum method

I.1 Nonlinear static analysis

Nonlinear static analysis (pushover analysis; see Figure I.1) gives nonlinear response of the structural model under the constant lateral load distribution shape. Generally, the lateral force distribution shape (the ratio of the amount of lateral force at each floor) is defined prior to conducting the analysis by considering predominant vibration modes. The amount of the lateral force is then gradually increased.

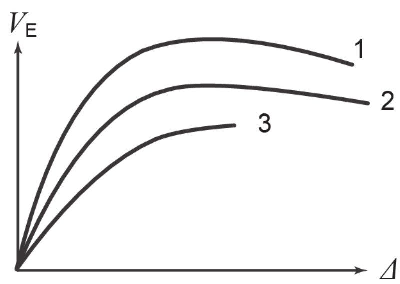


Key

 lateral forces

Figure I.1 — Nonlinear static analysis

From the nonlinear static analysis, the relationship between storey shear and inter-storey drift of each storey can be obtained as shown in Figure I.2. From this relationship, issues such as the amount of base shear, most vulnerable storey, and failure mechanism can be discussed. Moreover, the yield hinge developments of the structure at step by step can be checked as shown in Figure I.3, and deformation and restoring force of each member can be traced.



Key

V_E	storey shear
Δ	inter-storey drift
1, 2, 3	storey number

Figure I.2 — Example of the relationship between storey shear and inter-storey drift

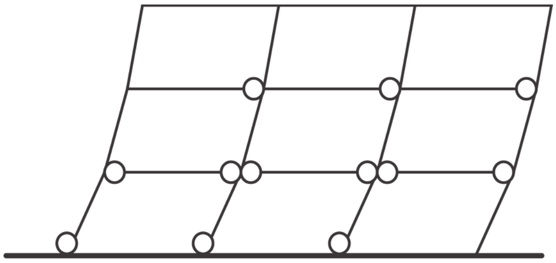


Figure I.3 — Example of yield hinge development

I.2 Capacity spectrum method

By considering the predominant vibration mode, multi-degree-of-freedom (MDOF) system can be simplified down to the response of SDOF system as shown in Figure I.4. The simplified shear divided by the equivalent mass, $\ddot{\Delta}$, is called "representative acceleration" and calculated from Formula (I.1). The simplified displacement, Δ, is called "representative displacement" and calculated from Formula (I.2).

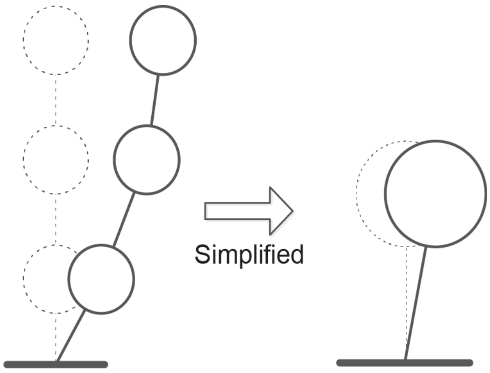


Figure I.4 — Simplification of MDOF system into equivalent SDOF system

$$\ddot{\Delta} = \frac{\sum m_i \cdot x_i^2}{\left(\sum m_i \cdot x_i\right)^2} \cdot \sum P_i \tag{I.1}$$

$$\Delta = \frac{\sum m_i \cdot x_i^2}{\sum m_i \cdot x_i} \tag{I.2}$$

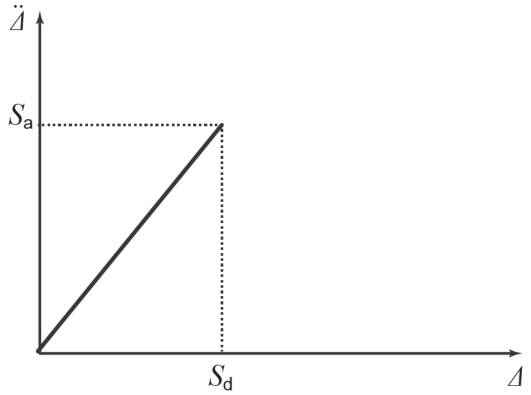
where

m_i is the mass at ith storey;

x_i is the relative displacement at ith floor to the base of the structure;

P_i is the amount of lateral force acting at ith floor.

If the system is linear, the maximum value of $\ddot{\Delta}$ and Δ under an earthquake are equal to the values of the acceleration response spectrum S_a and the displacement response spectrum S_d at the predominant period of the structure as shown in Figure I.5. The curve, of which horizontal axis is Δ and vertical axis is $\ddot{\Delta}$, is called performance curve, and the curve of which horizontal axis is S_d and vertical axis is S_a is called demand curve. The maximum response point is the intersection between the performance curve and the demand curve. The demand curve is usually defined from the design spectrum.



Key

$\ddot{\Delta}$ representative acceleration

S_a acceleration response

Δ representative displacement

S_d displacement response

Figure I.5 — Maximum response, S_a and S_d

If the performance curve shows nonlinearity, the damping increases due to additional energy dissipation during the nonlinear response. The equivalent damping, ζ_{eq}, should be defined, taking into account the shape of the hysteresis cycles of the structural systems and dissipating components. When specific values are not available, Formula (I.3) can be used to compute the equivalent viscous damping, where a linear viscous damping of 0,05 is considered.

$$\zeta_{eq} = \gamma \left(1 - \frac{1}{\sqrt{\mu}} \right) + 0{,}05 \tag{I.3}$$

where

γ is the coefficient that depends on the structural characteristics. Some recommended values are shown in Table I.1;

μ is the ductility factor.

Table I.1 — Example of γ value

Structural system	γ
Reinforced concrete walls and reinforced masonry walls	0,2
Ductile reinforced concrete frames	0,25
Dual wall-frame systems	See Formula (I.4)
Moment resisting steel frames Braced steel frames avoiding buckling of braces	0,25
Braced steel frames not avoiding buckling of braces	Specific studies needed
Unreinforced masonry	0,09
Timber structures with ductile connections	0,09
Timber structures with ordinary connections	Specific studies needed

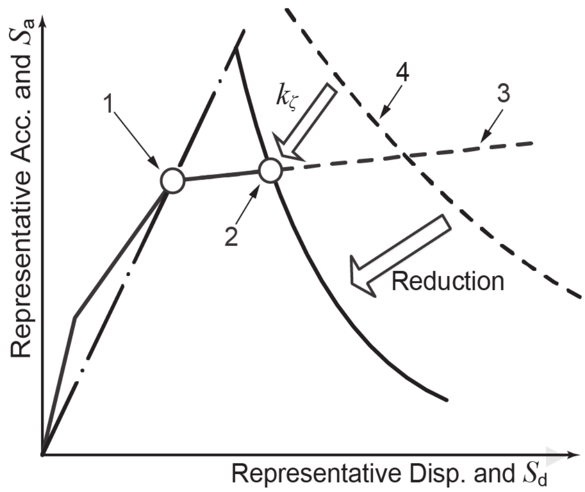
$$\zeta_{eq} = \frac{\zeta_{eq,W} V_W + \zeta_{eq,F} V_F}{V_W + V_F} \quad (I.4)$$

where $\zeta_{eq,W}$ and $\zeta_{eq,F}$ are the equivalent viscous damping ratios computed for wall and frame sub-systems, respectively, and V_W and V_F are the sum of shear force at the base of wall and frames elements, respectively. Careful evaluation of the floor diagram rigidity is required to apply Formula (I.4).

The demand reduction factor due to the nonlinearity, k_ζ, is calculated according to the equivalent damping ζ_{eq}. Some formulae such as Formula (I.5) may be informative.

$$k_\zeta = \frac{1,5}{1 + 10\zeta_{eq}} \quad (I.5)$$

As shown in Figure I.6, the maximum response can be estimated at the intersection between the performance curve and reduced demand curve by k_ζ. If no structural member reaches the safety limit state such as shear failure, bonding failure, or compression failure, the structure is evaluated safe.



Key

S_a acceleration response
S_d displacement response
k_ζ demand reduction factor due to the nonlinearity
1 yielding
2 maximum response
3 performance curve
4 damped curve (5 % damping)

Figure I.6 — Capacity spectrum method

Annex J
(informative)

Soil-structure interaction

J.1 Phenomena of soil-structure interaction (SSI)

For most structures, SSI effects are not considered when determining seismic design forces. For these structures, the design ground motions are input at their base, assuming a rigid foundation stiffness (fixed base assumption). However, for some structures such as low-rise buildings or mid-rise buildings sited on soft soil deposits, SSI effects can significantly change the seismic response of the structures by modifying the dynamic response characteristics (fundamental period and damping) of the soil structure system. The phenomena of change of period and damping ratio induced by soil conditions are called the dynamic SSI.

Due to embedment of foundation and piles, the input earthquake motions to the superstructure are changed compared with the earthquake motion defined on a ground surface. The input earthquake motions, which are less with frequency, are dependent on the depth of embedment and rigidity of piles. The phenomena of change of input motion are called the kinematic interaction. On the other hand, the phenomena of change of period and damping ratio by seismic force of structure are the inertial interaction.

The effects of SSI on the structures are summarized as follows:

a) elongation of natural period compared to the base-fixed condition;

b) change in damping ratio from the base-fixed condition;

c) decrease of input earthquake motion from the motion on ground surface.

Figure J.1 presents a model of superstructure, foundation and soil springs under the SSI, so-called sway-rocking (SR) model, and effects of sway and rocking springs on displacement of the model. For simplicity, the superstructure is set to be one mass. Due to inertial force of superstructure and foundation, three kinds of displacement are combined. There are displacements of superstructure itself, sway spring (horizontal mode of foundation) and rocking spring (rotation mode of foundation). The period of superstructure is estimated based on the displacement of superstructure (u_b). The period with SSI is estimated based on the total displacement of superstructure, sway (u_s) and rocking (u_r). The period of structure with SSI is always larger than with base-fixed condition. With soil deposit softer, the effects of sway and rocking displacements are more significant.

J.2 Simplified estimation of period and damping ratio

Under Figure J.1 b), the displacement of the SSI (u_e) is defined as three springs connected serially [see Formula (J.1)]:

$$u_e = u_b + u_s + u_r = \frac{F}{K_b} + \frac{F}{K_s} + \frac{FH^2}{K_r} \tag{J.1}$$

where F and H are the equivalent static horizontal force and the equivalent height of superstructure under fundamental vibration mode, respectively. K_b, K_s and K_r are spring constants of superstructure, sway and rocking, respectively.

The spring of SSI (K_e) is expressed as Formula (J.2):

$$\frac{1}{K_e} = \frac{1}{K_b} + \frac{1}{K_s} + \frac{H^2}{K_r} \tag{J.2}$$

The period of SSI system is as Formula (J.3):

$$T_e = \sqrt{T_b{}^2 + T_s{}^2 + T_r{}^2} \tag{J.3}$$

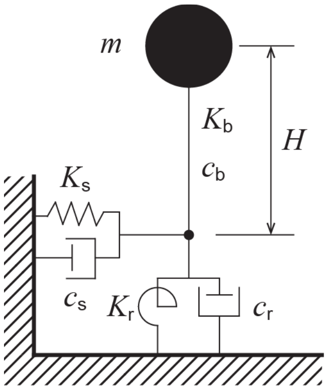
where T_b, T_s and T_r are natural periods of superstructure, sway and rocking, respectively.

In the same way, the damping ratio of SSI system is obtained. See Formulae (J.4) and (J.5).

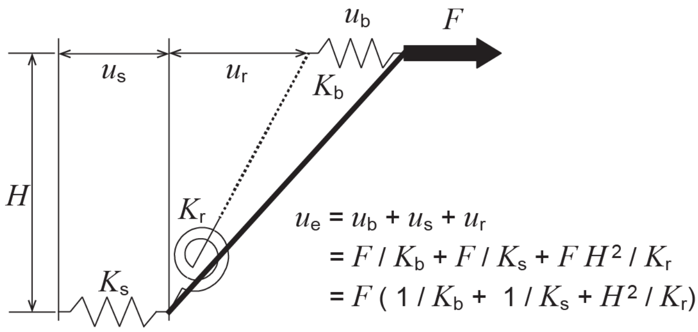
$$\zeta_e = \zeta_b \left(\frac{T_b}{T_e}\right)^3 + \zeta_s \left(\frac{T_s}{T_e}\right)^3 + \zeta_r \left(\frac{T_r}{T_e}\right)^3 \tag{J.4}$$

$$\zeta_b = \frac{1}{2\omega_b}\frac{c_b}{m},\ \zeta_s = \frac{1}{2\omega_s}\frac{c_s}{m},\ \zeta_r = \frac{1}{2\omega_r}\frac{c_r}{mH^2} \tag{J.5}$$

where ζ_b, ζ_s and ζ_r are damping ratios of superstructure, sway and rocking, ω_b, ω_s and ω_r are circular frequencies, c_b, c_s, and c_r are damping coefficients, respectively. m is equivalent mass of superstructure with fundamental vibration mode.



a) SR model



b) Displacement of SR model

Key

m	equivalent mass of superstructure with fundamental vibration mode
H	equivalent height of superstructure with fundamental vibration mode
F	inertia force by mass
K_b, c_b and u_b	spring constant, damping coefficient and displacement of superstructure
K_s, c_s and u_s	spring constant, damping coefficient and displacement by sway
K_r, c_r and u_r	spring constant, damping coefficient and displacement by rocking
u_e	total displacement

Figure J.1 — SR model and displacement distribution

Annex K
(informative)

Seismic design of high-rise buildings

K.1 General

Normally, large numbers of people assemble in multi-storey high-rise buildings and failure of a high-rise building usually causes more serious impacts in surrounding facilities than those caused by failures of low- or mid-rise buildings. In this context, high-rise buildings call for enhanced reliability in ULS. In addition, as their sizes are usually quite large, damage in high-rise buildings is serious in terms of loss or repair cost and long down-time. Therefore, enhanced reliability may also be required in SLS.

However, most of the current seismic design codes do not explicitly require increase in the importance factor, which is similar to the load factor as related to reliability of the structure, just because the building is a high-rise one. Instead, due regards are commonly paid to the following issues in the seismic design of multi-storey high-rise buildings.

a) Employ the most advanced methods and models of structures in evaluating seismic action effects.

b) Select appropriate design input ground motions including those that are most critical in the light of dynamic characteristics that are distinctive in high-rise buildings.

c) Enforce normal design considerations or introduce more stringent acceptance criteria including but not limited to:

- minimize eccentricity between the centres of mass and stiffness;
- minimize abrupt variation in horizontal storey stiffness;
- introduce additional damping or response control system;
- assign special margins to critical elements and portions of the structure to maintain ductile behaviour.

NOTE Typically high-rise buildings are defined as those greater than 50 m in height with significant mass participation and lateral response in higher modes of vibration.

K.2 Method of evaluation and model of structure

In principle, bases of evaluating seismic actions and action effects are common in all buildings including high-rise ones. Usually, dynamic analysis procedures are employed in the seismic design of high-rise buildings as the presumptions implicit in equivalent static analysis may not be appropriate for high-rise buildings.

As it is common in all types of dynamic analysis, spatial or three-dimensional representation of models of structures is recommended. This principle applies to high-rise buildings because space frames or other three-dimensional structural systems are commonly employed in them and effects of frames in the direction orthogonal to the seismic actions are not negligible in evaluating the action effects. In addition, effects of combination of the two horizontal components indicated in Annex D, which are more important in high-rise buildings, can be evaluated without introducing empirical factors, ε or λ, as it is sufficient just to conduct analysis for simultaneous application of two orthogonal ground motions to spatial or three-dimensional models.

In designing large scale structures including high-rise buildings, the influence of soil-structure interaction should be included in evaluating seismic actions for buildings on soft soil and supported by deep foundations.

Response history analyses for ULS seismic actions should be conducted with nonlinear models as the precise information of nonlinear behaviours during earthquakes of each element of the structure is essential especially in high-rise buildings. Such behaviours include not only element force, but also maximum nonlinear deformation, number of stress reversals, etc. It should be noted that structural design factors to assess nonlinear response from elastic response are established for prototype of mainly low- or mid-rise buildings and may have to be reviewed in applying to various types of innovative structural systems of high-rise buildings.

K.3 Input ground motion

In addition to the considerations for uniform hazard representation of design ground motions, those with critical components in the light of dynamic characteristics of high-rise buildings should be employed. High-rise buildings are usually structures with long periods and call for particular attention in selecting design ground motion history to include ones containing high levels of long-period component. In nonlinear response history analysis, durations and/or numbers of large amplitude motion may also be important. Sometimes, considerations for the ground motions due to mega earthquakes occurring along boundaries of crustal plates, even if they are far from the site, result in unexpectedly large and long lasting response in high-rise buildings. Due consideration should be made to these phenomena in providing simulated earthquake ground motions based on deterministic scenario.

K.4 Introducing response control system

Research and development of response control systems for structures are advancing rapidly and various types of systems described in Annex M, particularly passive control or damping systems, have reached the stage of practical application. Consequently, response control is becoming a standard equipment of high-rise buildings in high seismic risk regions to reduce maximum floor response and duration time of vibration due to seismic actions as well as to improve habitability during frequent wind actions.

In employing response control systems, their characteristics should be fully considered and the system that is most effective to control the effects of expected type and intensity of seismic actions should be selected. Appropriate analytical models of devices including their specific characteristics should be established. For example, dependency of their damping properties, if any, on temperature, amplitude of vibration, etc. should be properly incorporated to avoid overestimation of the response control effects. In addition, the influence of fatigue under cyclic deformations should also be considered for steel or other metal dampers.

K.5 Soil-structure interaction (SSI)

Inertia force in superstructures due to seismic actions is transmitted through foundations to and resisted by the ground causing displacements of foundations and/or basements. As a result, dynamic properties including natural periods and damping ratios change. If the site soil is soft, the effects are outstanding also in high-rise buildings. In addition, seismic motion input to the superstructure is not same as the ground surface motions, which are usually used as earthquake input to fixed-base models of structures, due to the effects of basements and/or piles. A more detailed description for this issue, the soil-structural interaction, is given in Annex J.

While the SSI and its influence are seldom considered in the cases of low- or mid-rise buildings, it is often considered for high-rise buildings constructed on soft soil and supported by deep foundations. Where the influence of the interaction on seismic response is significant, it should be properly taken into account by employing the structural models as indicated in H.1 and Annex J.

Annex L
(informative)

Deformation limits

There are two kinds of deformations to be controlled: the storey drift which is the lateral displacement within a storey and the total lateral displacement at some height relative to the base. The storey drift should be limited to restrict damage to nonstructural 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. Limits are frequently expressed in terms of the storey drift ratio, which is the storey drift divided by the storey height. 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.

For control of life threatening damage in occupied buildings at the ULS, the storey drift ratio should be limited to values between 0,005 (1/200) to 0,025 (1/40), depending on the materials of construction, the height of the building, and the use of the building. An example tabulation of such effects is shown in Table L.1. In other kinds of structures, limitations on storey drift may be governed by the drift capacity of nonstructural elements and systems. In critical facilities, the limits on storey drift ratio should be smaller as necessary to preserve function of the essential systems.

Table L.1 — Example limiting storey drift ratios for buildings

	Normal consequence class	**High consequence class**
Low rise, without masonry	0,010 to 0,025 (1/100 to 1/40)	0,004 to 0,015 (1/250 to 1/67)
High rise, without masonry	0,005 to 0,020 (1/200 to 1/50)	0,002 to 0,010 (1/500 to 1/100)
With structural masonry	0,005 to 0,010 (1/200 to 1/100)	0,002 to 0,010 (1/500 to 1/100)

The control of the total displacement is concerned with sufficient separations of two adjoining structures to avoid damaging contact for severe earthquake ground motions. There are two common methods for quantifying the necessary building separation based on the deformations of the two structures, depending upon the degree of assurance and vulnerability to damaging contact: 1) use the absolute sum, or 2) use the square root of the sum of the squares. Also, for members spanning between two structures, the bearings should have sufficient displacement capacity to maintain support.

Annex M
(informative)

Response control systems

Recently, response control systems including seismic isolation have been gradually applied to various structures, e.g. buildings, highway bridges and power plants and LNG tanks. The response control systems are utilized not only for new structures but also for existing structures to retrofit them. There are some response control systems to protect contents of structures, isolating the floors which support those contents, etc.

The response control systems are classified as shown in Figure M.1, and some examples for the response control systems are illustrated in Figure M.2. All systems except active (including partially active that is semi-active) control systems can be classified into passive control systems. The seismic isolation is to reduce the response of the structure by the isolators and dampers which are usually installed between the foundation and the structure. Since the isolators elongate the natural period of the structure and dampers increase damping, the acceleration response is reduced as shown in Figure M.3, but a large relative displacement occurs at the isolator installed storey.

Energy absorption devices and the addition of masses to structures are also used to control the response. As shown in Figure M.4 a), for the structure without response control, the input energy to the structure during earthquake is distributed to viscous damping of structure, hysteretic energy of structure and radiated energy into ground. Figure M.4 b) indicates that, for the structure with response control, seismic dampers absorb large amount of energy, and the hysteretic energy caused by the damage of structural elements can be reduced effectively.

The energy absorption devices increase the damping of the structures by plastic deformation or viscous resistance of the passive control devices. The response of structure is also reduced by vibration of additional masses or liquid materials. The active response control systems reduce the response of structure caused by earthquakes and winds using computer controlled systems.

The response control systems are used to reduce floor response and inter-storey drift. The reduction of floor response can ensure seismic safety, improve habitability, ease mental anxiety, protect furniture from overturning, etc. The reduction of inter-storey drift can decrease the amount of construction materials, reduce damage to nonstructural elements, increase design freedom, etc.

The design of the systems should take into account the mechanical characteristics of isolators or additional devices, e.g. hysteretic, frictional and hydraulic dampers. Dynamic analysis is preferable for these systems, since restoring force characteristics of devices have much influence on the characteristics of structures. Analytical models for newly developed materials should be verified through experiments. In addition to seismic loading, for seismic response control systems (especially for seismic isolation system), it is also necessary to consider wind loads for structural design to ensure that the threshold before the onset of nonlinear behaviour of the response control system is greater than the design wind loading.

Since the systems can be influenced by the environment, it is necessary to take into account the effects of ageing, creep, fatigue, temperature, exposure to moisture, etc.

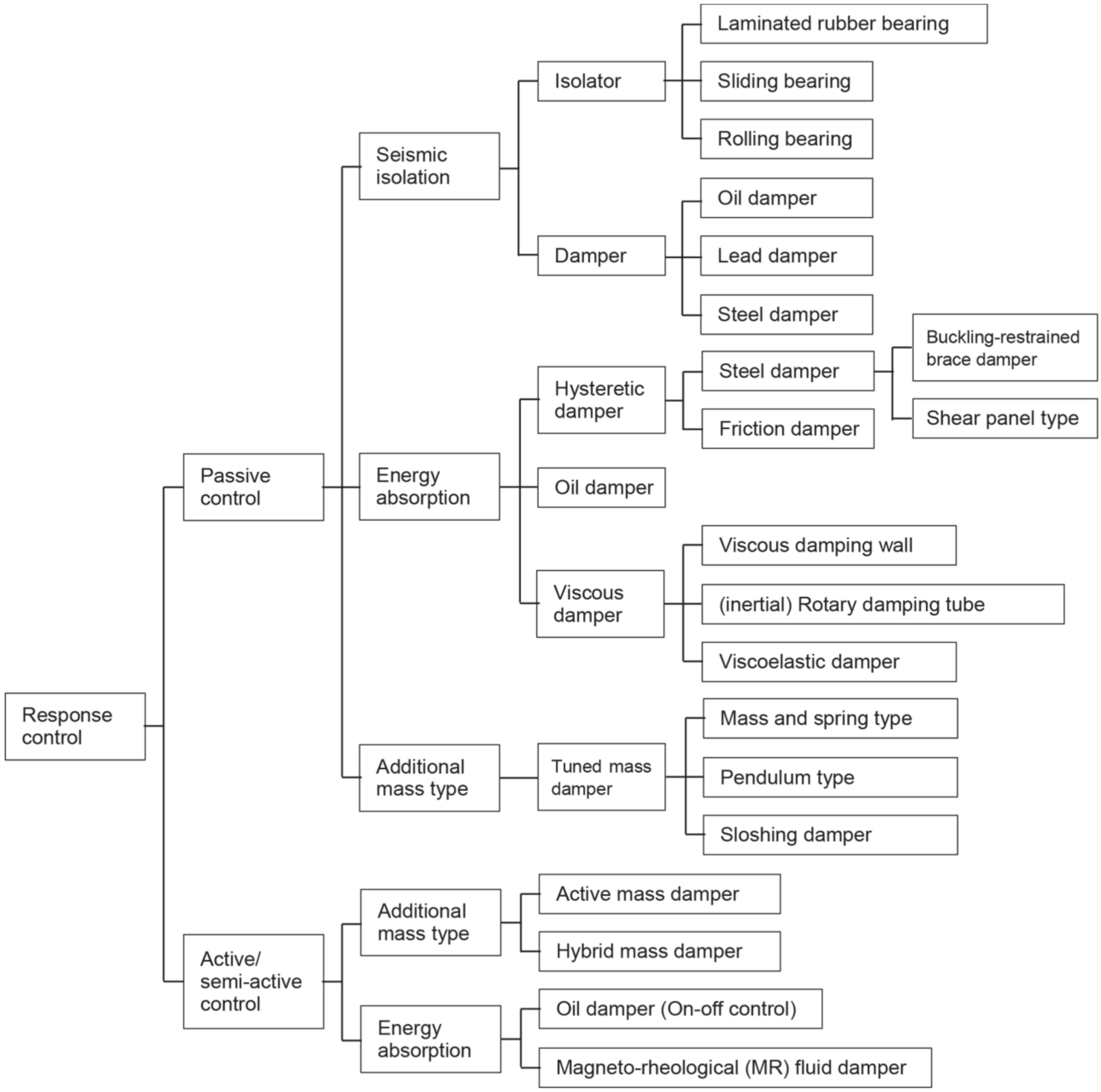
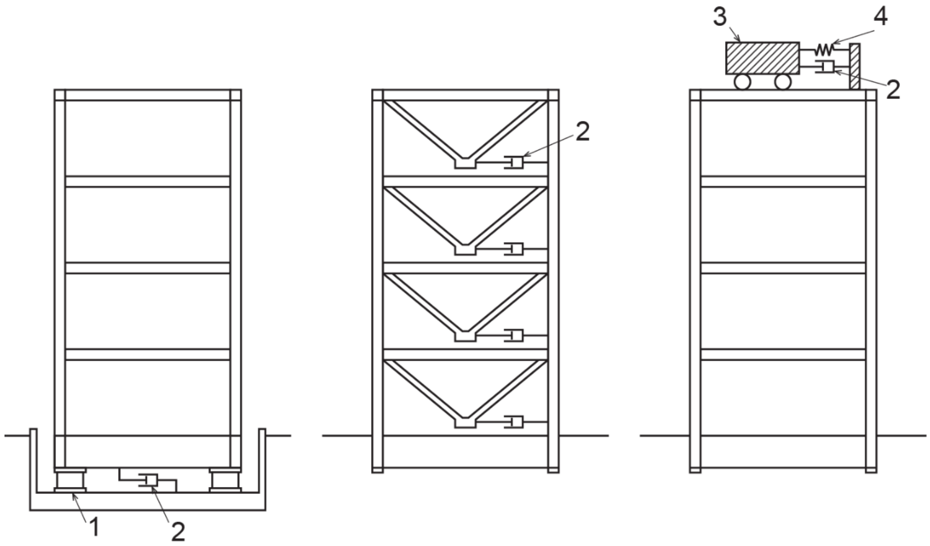


Figure M.1 — Classification of response control systems with dampers

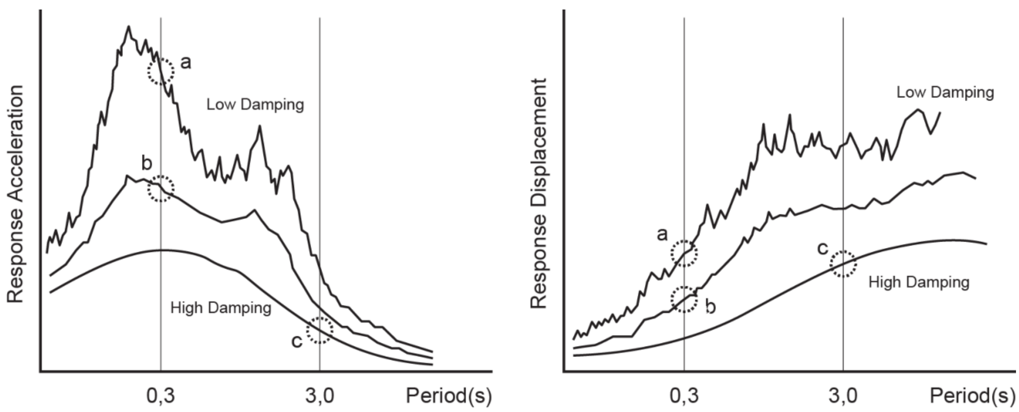


a) Seismic isolation   b) Energy absorption   c) Additional mass type

Key

1 isolator
2 damper
3 mass
4 spring

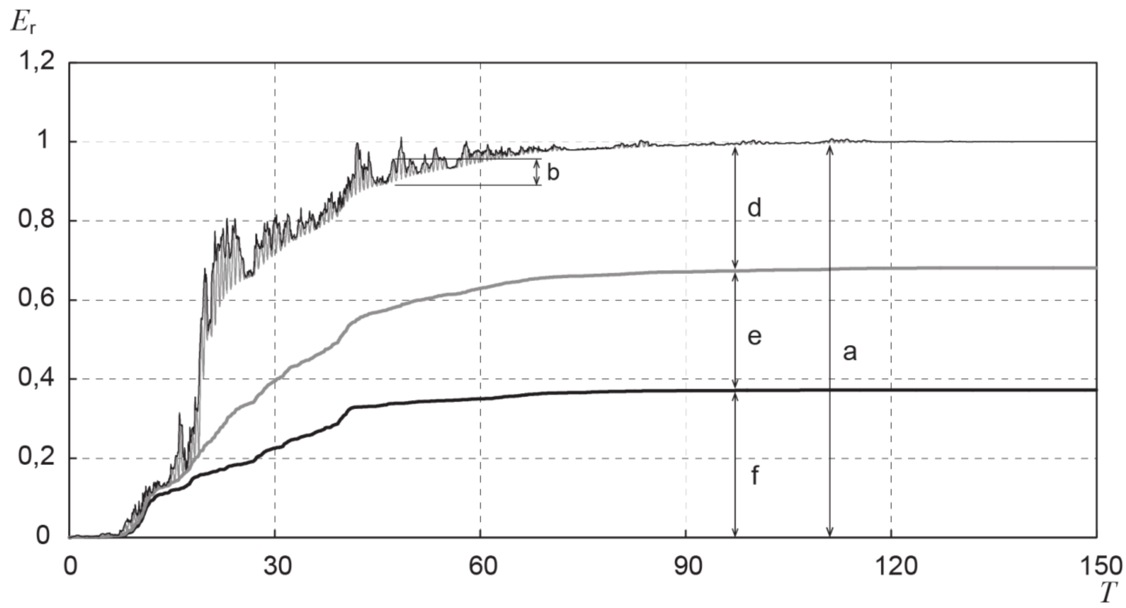
Figure M.2 — Example of passive control system



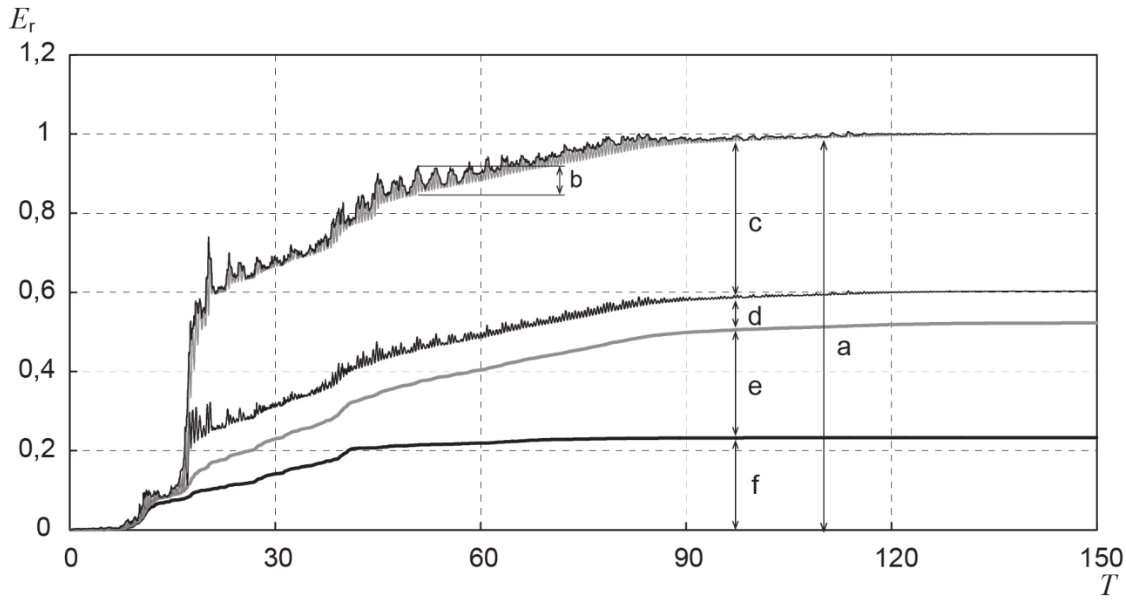
Key

a response of ordinary structures
b response of response controlled structures [see Figure M.2 b) and c)]
c response of seismic isolated structures (isolator + damper) [see Figure M.2 a)]

Figure M.3 — Effects of response control systems on the response of structures



a) Without response control by dampers



b) With response control by dampers

Key

E_r normalized absorbed energy

T time (s)

a total input energy during earthquake

b vibration energy of structure

c energy absorbed by seismic dampers

d hysteretic energy of structure

e energy absorbed by viscous damping of structure

f radiated energy into ground

Figure M.4 — Example of energy absorption of SDOF structure with and without response control

Annex N
(informative)

Non-engineered construction

N.1 Various types of non-engineered construction

Many structures are spontaneously and informally constructed in various countries in the traditional manner with little or no intervention by qualified architects and/or engineers, and are often called “non-engineered”. Some types of non-engineered construction are 1) unreinforced masonry (stone, brick or concrete block masonry), 2) confined masonry, 3) wooden construction, 4) earthen construction (adobe or tapial, i.e. rammed earth), etc. Many of these types of construction are unsatisfactory for use in seismically hazardous regions. Some of these types of construction can deliver satisfactory seismic performance given simple rules on basic layout, materials, and connections. Proper limitation on the size, height, and use (consequence class) of such empirically designed structures is essential.

N.2 Characteristics and vulnerability specific to non-engineered structure

N.2.1 Unreinforced masonry

Masonry walls of this type consist of fired bricks, solid concrete blocks, hollow concrete or mortar blocks, etc. The main weaknesses in unreinforced masonry construction are a) heavy and stiff structures, attracting large seismic inertia forces, b) very low tensile and shear strength, particularly with poor quality mortars, c) brittle behaviour in tension as well as compression, d) weak connections between walls, etc. Therefore, use of mud or very lean mortars is unsuitable.

N.2.2 Confined masonry

This type consists of masonry wall of clay brick or concrete block units and horizontal and vertical reinforced concrete members that confine the masonry wall panels at four sides. Vertical members are called “tie-columns”, and though they resemble columns in reinforced concrete (RC) frame construction, they are of much smaller cross-section. Horizontal elements, called “tie-beams”, resemble beams in RC frame construction, but also of much smaller section. It should be understood that the confining elements are not beams and columns in the way these are used in RC frames. Rather, they function as horizontal and vertical ties or bands for resisting tensile stresses.

N.2.3 Wooden construction

Wood has a high strength per unit weight. Wood structures are often connected with dowel type of steel connectors (nails, screws and bolts) which offer some ductility. This combination of low density and ductile connections make wood very suitable for earthquake resistant structures. However, heavy claddings (including walls and roofs) impose high lateral loads when placed on a wooden post and beam frame and can load the frame beyond its structural capacity. Where small wood framing members are combined with nailed sheathing of various materials for floors, roofs and walls, seismic performance has been very successful, and simple rules for providing adequate amounts of shear walls/braces have proven successful in non-engineered construction. Therefore, non-engineered wooden construction is suitable in those areas where wood is still abundantly available as a renewable resource.

N.2.4 Earthen construction

Walls are the basic structural elements and can be classified as a) adobe or blocks, b) tapial or rammed earth, and c) wood or cane mesh frameworks with mud. This material has clear advantage of costs, aesthetics, acoustics, heat insulation and low energy consumption, but it has some disadvantages such as being weak under earthquake forces and water action. However, technology developed to date has allowed some reduction of its disadvantages. Earthen construction is, in general, spontaneous and a great difficulty is experienced in the dissemination of knowledge about its adequate use.

N.3 Possible approach to enhance structural integrity (structural robustness)

Examples of possible approach to prevent vulnerable failures specific to non-engineered construction are as follows. Minimizing cost additional to current practice is essential to all the approach.

N.3.1 Improvement on materials and components

Use of stabilizers (cement, lime, asphalt, etc.) to improve the strength and durability for earthen construction, enrichment of cement mixture ratio and improvement curing treatment for concrete blocks, improvement of kilns to burn bricks with higher temperature are the several practical ways.

N.3.2 Connections between components

Separation of masonry walls at corners, failures at joints of confining RC members (columns and beams) and wood frames (between post, beams, braces, etc.) are typical examples of critical structural weaknesses. Addition of connections to prevent these failures is necessary.

N.3.3 Addition of reinforcements

For some very vulnerable parts/components, addition of reinforcement is effective. Examples include reinforcement/supports for masonry gables, lintel or sill beams in masonry walls, reinforcement inside brick/adobe walls, mortar plastering on walls with mesh, etc.

Annex O
(informative)

Tsunami actions

O.1 General

Damaging tsunamis are generally caused by large offshore earthquakes with moment magnitudes greater than M_W7.5 that induce significant vertical offsets in the sea floor. Tsunamis may inundate coastal regions several times during an event. Because tsunami waves have longer wavelength and have very low damping, they can travel great distances across oceans and still have considerable damaging energy particularly for coasts with unfavourable site configurations. Tsunamis may be also generated by a landslide in sea or lake, a mountain collapse, etc. Structures, that are located on land in tsunami hazard areas and required to withstand tsunamis, should be designed against tsunami actions.

O.2 Principles of calculating tsunami actions

Tsunami actions on structures are tsunami wave forces and debris impacts.

Tsunami wave forces on structures (see Figure O.1) can be calculated from a design tsunami inundation depth h and a design current velocity v on a site based on the stochastic method as hydrostatic forces F_S or hydrodynamic forces F_D in both horizontal and vertical directions. For example, hydrostatic pressure q_z in horizontal direction is evaluated as shown in Formula (O.1):

$$q_z = \rho \ g \ (ah - z) \tag{O.1}$$

where

g is the acceleration due to gravity (m/s²);

ρ is the density of sea water (kg/m³);

a is the water depth factor;

h is the design inundation height (m);

z is the height of the building at the level concerned (m).

The water depth factor a depends on the distance from the costal line and may be from 1,5 to 3,0. Hydrostatic forces F_S (N) is evaluated by integral of q_z by height multiplied by width of the structure.

A hydrodynamic force F_D (N) in horizontal direction is evaluated as Formula (O.2):

$$F_D = \frac{1}{2} \rho \ C_D \ v^2 \ h \ B \tag{O.2}$$

where

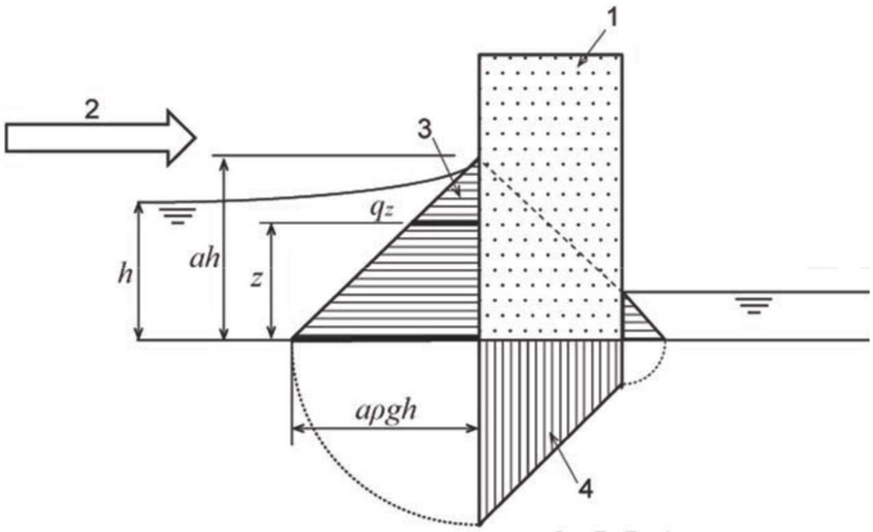
C_D is the drag factor;

ρ is the density of sea water (kg/m^3);

v is the design current velocity (m/s);

h is the design inundation depth (m);

B is the width of a structure (m).



Key

1 structure
2 direction of tsunami
3 hydrostatic force in horizontal direction
4 hydrostatic force in vertical direction (buoyancy)
a water depth factor
h design inundation depth (m)
z height of the building at the level concerned (m)
q_z hydrostatic pressure
ρ density of sea water (kg/m^3)
g acceleration due to gravity (m/s^2)

Figure O.1 — Tsunami wave force on a structure

A tsunami wave force is evaluated as a drag or a difference of tsunami wave pressures acting on both sides of walls in a structure or the structure itself.

Tsunami flood water conveys various debris: trees, containers, vehicles, trains, ships, houses, timbers, furniture, etc. Structures should be designed to avoid progressive collapse owing to debris impacts.

Tsunami wave forces on structures can be reduced based on the concept that tsunami wave pressures may be regarded not to act on openings (windows, doors, including pilotis, etc.) of structures owing to failure of openings. However, tsunami forces on glasses of openings equivalent to the strength of the glass are considered. Tsunami forces on inner walls and rear walls of structures are also considered.

All expected incident directions of tsunamis should be considered. Backwash (a backward flow of water) of tsunamis also should be considered as well as anaseism (opposite of backwash, a forward flow of water) of tsunamis.

Sea water is regarded as non-compressible fluid. The density ρ of the sea water can be regarded $1{,}0 \times 10^3$ kg/m^3. When sea water contains mud, sands and other debris, the density of the sea water should be appropriately determined.

Damage by earthquakes, liquefaction, scoring around foundations, damming of debris are also considered.

Annex P
(informative)

Paraseismic influences

The techniques of seismic design and construction are useful where structures are subject to ground motions caused by sources other than earthquakes. Such actions are called paraseismic influences in this document. Sources of paraseismic influences are classified as follows:

— underground explosions;

— shocks from mine, induced seismicity (rockbursts);

— above ground explosions (e.g. quarries);

— above ground impacts and shocks (e.g. pile driving);

— traffic vibrations transmitted through ground to structures (from surface motorways, streets, railway lines, underground railways);

— other sources such as industry activities, machines.

Some guidelines on the use of Formulae (1), (2) or (3), (4) for estimating paraseismic influences are as follows:

— k_Z, the paraseismic hazard zoning factor can be taken from paraseismic hazard zoning maps, individually obtained from case monitoring or direct measurements;

— $k_{E,u}$, $k_{E,s}$, representative values of ground motion intensity, can also be obtained from case monitoring or direct measurements; consideration should be given to the fact that in general, the return period is very short in comparison to earthquakes;

— k_D, the structural design factor to reduce design forces is acceptable only in exceptional cases and the value should not be less than 0,5;

— k_R, the normalized design response spectrum will generally have to be adjusted to a shape somewhat different than that used for seismic design.

Respective response spectrum should be constructed based on a collection of the strongest surface records of paraseismic events, e.g. mine tremors. Due to possible high frequency shifts of paraseismic effects (often from 10 Hz to 40 Hz), the intensity measures should avoid direct acceleration peak values which give overestimations of the intensities. Peak values of the horizontal particle velocity are the best parameter to quantify intensity of paraseismic effects. Multi-components of horizontal and vertical directions should be simultaneously considered for many cases, particularly for closely situated sources of shocks. In case exact data are not available, Formula (E.2) can be used.

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ICS 91.080.01; 91.120.25
Price based on 60 pages