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Annexes

Basis for design of structures—Seismic actions on structures

1 Scope

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

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

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

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

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

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

5 Bases of seismic design

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

－to prevent human casualties,

－to ensure continuity of vital services, and

－to minimize damage to property.

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

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

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

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

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

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

6 Principles of seismic design

6.1 Construction site

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

6.2 Structural configuration

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

a) Plan irregularities

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

b) Vertical irregularities

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

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

6.3 Influence of non-structural elements

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

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

6.4 Strength and ductility

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

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

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

6.5 Deformation of the structure

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

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

6.6 Response control systems

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

6.7 Foundations

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

Annex A
(informative)

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

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

A.1.1 General

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

a) the required degree of reliability,

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

c) the variability of seismic actions, and

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

A.1.2 Required degree of reliability

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

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

a) High degree of importance

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

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

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

b) Normal degree of importance:

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

c) Low degree of importance:

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

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

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

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

A.1.4 Examples of load factors associated with representative values

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

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

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

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

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

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

A.2 Seismic hazard zoning factor, k_Z

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

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

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

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

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

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

Annex B
(informative)

Structural factor

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

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

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

where

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

k_{Ds} is related to overstrength.

The factor can also be expressed as follows:

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

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

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

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

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

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

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

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

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

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

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

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

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

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

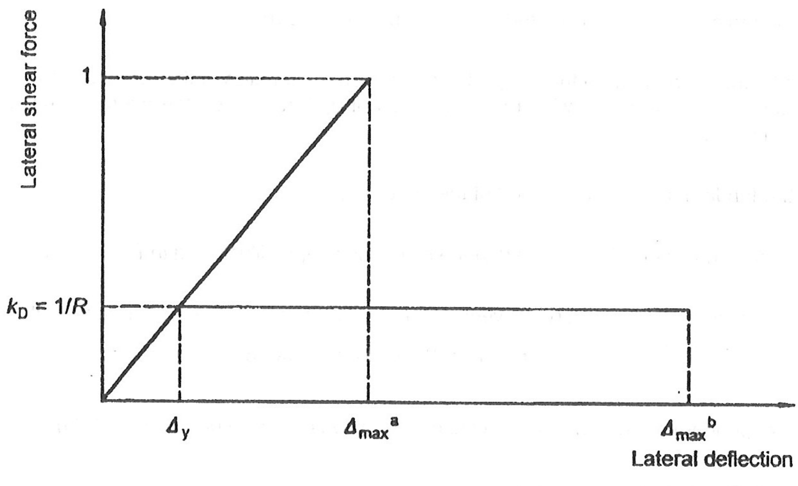
$$\Delta_{max} = \frac{1}{k_D} \Delta_y = R\Delta_y \tag{B.3}$$

$$\Delta_{max} = \frac{1}{2}\left(\frac{1}{k_D^2} + 1\right) \Delta_y = \frac{1}{2}\left(R^2 + 1\right) \Delta_y \tag{B.4}$$

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

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

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



[a] By displacement-constant rule

[b] By energy-constant rule

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

Annex C
(informative)

Normalized design response spectrum

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

It may be of the form

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

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

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

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

where

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

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

T is the fundamental natural period of the structure;

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

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

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

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

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

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

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

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

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

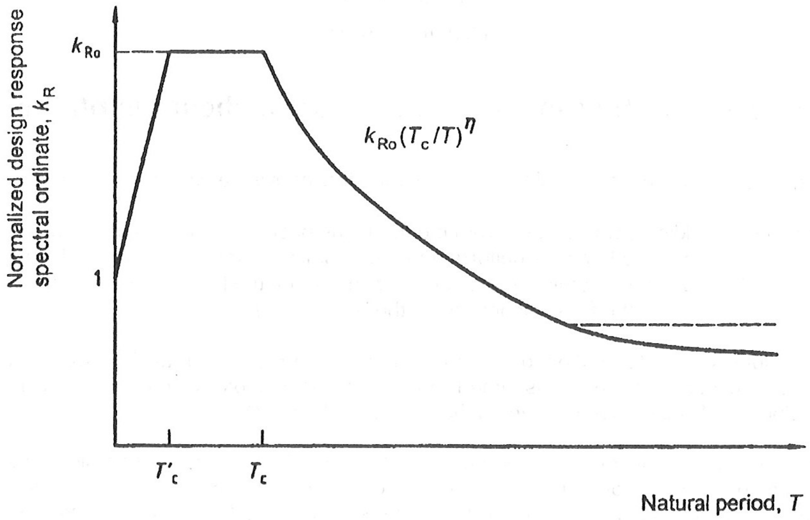


Figure C.1 — Normalized design response spectrum

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

— uncertainty of ground motion characteristics in this range;

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

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

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

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

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