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Bases for design of structures — Seismic actions for designing geotechnical works

Bases du calcul des constructions — Actions sismiques pour le calcul des ouvrages géotechniques

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Foreword

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ISO 23469 was prepared by Technical Committee ISO/TC 98, *Bases for design of structures*, Subcommittee SC 3, *Loads, forces and other actions* in collaboration with ISSMGE/TC4 and CEN/TC205/SC8.

Introduction

This International Standard provides guidelines to be observed by experienced practising engineers and code writers when specifying seismic actions in the design of geotechnical works. Geotechnical works are those comprised of soil or rock, including buried structures (e.g. buried tunnels, box culverts, pipelines and underground storage facilities), foundations (e.g. shallow and deep foundations, and underground diaphragm walls), retaining walls (e.g. soil retaining and quay walls), pile-supported wharves and piers, earth structures (e.g. earth and rockfill dams and embankments), gravity dams, landfill and waste sites. The seismic actions described are compatible with ISO 2394.

The seismic performance of geotechnical works is significantly affected by ground displacement. In particular, soil-structure interaction and effects of liquefaction play major roles and pose difficult problems for engineers. This International Standard addresses these issues in a systematic manner within a consistent framework.

The seismic performance criteria for geotechnical works cover a wide range. If the consequences of failure are minor and the geotechnical works are easily repairable, their failure or collapse may be acceptable and explicit seismic design may not be required. However, geotechnical works that are an essential part of a facility handling hazardous materials or a post-earthquake emergency facility shall maintain full operational capacity during and after an earthquake. This International Standard presents a full range of methods for the analysis of geotechnical works, ranging from simple to sophisticated, from which experienced practising engineers can choose the most appropriate one for evaluating the performance of a geotechnical work.

Bases for design of structures — Seismic actions for designing geotechnical works

1 Scope

This International Standard provides guidelines for specifying seismic actions for designing geotechnical works, including buried structures (e.g. buried tunnels, box culverts, pipelines and underground storage facilities), foundations (e.g. shallow and deep foundations, and underground diaphragm walls), retaining walls (e.g. soil retaining and quay walls), pile-supported wharves and piers, earth structures (e.g. earth and rockfill dams and embankments), gravity dams, landfill and waste sites.

NOTE The guidelines provided in this International Standard are general enough to be applicable for both new and existing geotechnical works. However, for use in practice, procedures more specific to existing geotechnical works can be needed, such as those described for existing structures in ISO 13822.

2 Normative references

The following referenced documents are indispensable for the application 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 2394:1998, *General principles on reliability for structures*

ISO 3010:2001, *Bases for design of structures — Seismic actions on structures*

ISO 13822:2001, *Bases for design of structures — Assessment of existing structures*

3 Terms and definitions

For the purposes of this document, the terms and definitions given in ISO 2394, ISO 3010 and ISO 13822 and the following apply.

3.1
array observation
simultaneous recording of earthquake ground motions and/or microtremors by an array of seismometers

3.2
basin effects
effects on earthquake ground motions caused by the presence of a basin-like geometrical boundary beneath the site

NOTE Deep basin effects are defined as effects due to the geometry of the interface between the upper crustal rock and the overlying firm ground or soil deposits. Shallow basin effects are defined as effects due to the geometry of the interface between the firm ground (or shallow upper crustal rock) and the local soil deposits and may be treated as part of the local site response.

3.3
coherency function
function describing a degree of correlation between two time histories

3.4
crest
top of a geotechnical structure, typically defined for embankments and dams

3.5
culvert
tunnel-like structure constructed typically in embankments or ground forming a passage or allowing drainage under a road or railroad

3.6
damping
mechanism that dissipates energy of motion

3.7
deep foundation
foundation having a large depth to width ratio, which transfers applied loads to deep soil deposits

EXAMPLES Pile foundation, sheet pile foundation, cofferdam foundation, caisson foundation.

3.8
design working life
duration of the period for which a structure or a structural element is designed to perform as intended with expected maintenance, but without major repair being necessary

3.9
deterministic seismic hazard analysis
seismic hazard analysis based on the selection of individual earthquake scenarios

3.10
dynamic analysis
analysis for computing the dynamic response of a system based on the equations of motion

3.11
earth pressure
pressure from soil on a wall or an embedded portion of a structure

3.12
earth structure
geotechnical work consisting primarily of soil or rock

EXAMPLES Earth and rockfill dams, and embankments.

3.13
earthquake ground motions
transient motions of the ground caused by earthquakes, including those at the ground surface, within the local soil deposit, and at the interface between the firm ground and the local soil deposit

3.14
effective stress analysis
analysis with consideration of pore pressure changes

3.15
equivalent linear model
linear model incorporating elastic shear moduli and damping factors that are compatible, at various strain amplitudes, with the non-linear stress-strain relationship under cyclic loading

3.16
equivalent static analysis
static analysis that approximates the dynamic response of the system

3.17
excess pore water pressure
change of water pressure in the soil pores with respect to those at a reference state

3.18
failure mode
pattern of failure defined by distinctive features of the deformed shape after failure

3.19
fault displacement
permanent tectonic ground displacement associated with fault dislocation

3.20
firm ground
soft rock or stiff soil layer

3.21
free field
ground not subject to the effect of geotechnical works or structures

3.22
geotechnical characterization
specification of material and geometrical parameters of soil or rock

3.23
geotechnical hazard
hazard associated with geotechnical phenomena, including ground failure and subsidence

3.24
geotechnical work
work that includes soil or rock as primary components with or without structural parts made of concrete, steel, or other materials

EXAMPLES Buried structures (e.g. buried tunnels, box culverts, pipelines and underground storage facilities), foundations (e.g. shallow and deep foundations, and underground diaphragm walls), retaining walls (e.g. soil retaining and quay walls), pile-supported wharves and piers, earth structures (e.g. earth and rockfill dams and embankments) gravity dams, landfill and waste sites.

3.25
ground failure
mass movement of soil including liquefaction-induced ground deformations (settlement, lateral spreading, flow failure) and non-liquefaction-induced ground deformations (seismic compaction, permanent deformations and landslides)

3.26
horizontal wave propagation effect
effect causing spatial variation of ground motion in the horizontal direction due to the finite speed of wave propagation

3.27
hydro-dynamic pressure
transient pressure exerted by a fluid on a structure in a system subject to dynamic motion

3.28
importance of a structure or facility
degree of possible consequences of failure of a structure or facility caused by a reference earthquake motion

3.29
inertial interaction
part of soil-structure interaction arising from the inertia forces acting on the structure

3.30
kinematic interaction
part of soil-structure interaction arising from the deformation of the soil relative to that of the structure

3.31
liquefaction
large drop in soil shear strength and/or stiffness caused by an increase in pore water pressure that may cause significant reduction in the shear resistance of geotechnical works and ground or may induce large ground displacement

3.32
liquefaction potential
susceptibility of the soil to the onset of liquefaction under a reference earthquake motion

3.33
local site effect
effect of the local geological configuration on earthquake ground motions

3.34
lumped mass
mass assigned at discrete points of a model representing a continuum

3.35
microtremors
small amplitude vibration of the ground generated by either human activities or natural phenomena

3.36
overstrength
strength of a structure or structural element, typically specified by the ratio of actual strength to nominal design strength

3.37
performance criteria
set of conditions for specifying the response of a geotechnical work to meet the expected state defined by engineering parameters, such as acceptable displacements, strains or stresses, that characterize the performance objectives of design

3.38
performance objective
expression of the expected performance of a facility in order to fulfil its purposes and functions

3.39
phase velocity
velocity at which a monochromatic seismic wave travels along a surface

3.40
pipeline
long tube or a network of tubing used for the transportation of fluid, gas, or solid mixed with fluid or gas

3.41
probabilistic seismic hazard analysis
seismic hazard analysis considering the probability of occurrence of different levels of ground shaking at a site during the reference period

3.42
reference earthquake motions
earthquake motions specified for evaluating seismic performance of a geotechnical work (seismic actions are specified, in a subsequent stage, based on the reference earthquake motions)

3.43
residual displacement
displacement present after the earthquake, typically due to non-reversible deformation or sliding

3.44
residual response
response of a system remaining after the earthquake

3.45
residual strength
shear strength of the soil after failure including liquefaction

3.46
retaining wall
wall supporting backfill soil, embankment soil or a cut slope

3.47
scenario earthquake
earthquake that is specified for determining earthquake ground motions typically by deterministic seismic hazard analysis

3.48
seismic actions
loads, deformations, or other actions imposed upon models of structures and geotechnical works during and after an earthquake

3.49
seismic coefficient
coefficient that represents the dynamic forces on the structure by static forces as a fraction of the weight of the structure

3.50
seismic coefficient approach
static approach in which the dynamic response of soil-structure system is evaluated by an inertia force distributed over the system

3.51
seismic hazard analysis
analysis for determining earthquake ground motions on the basis of the regional seismic activity and characteristics of source and wave propagation

3.52
seismic performance
response of a structure or geotechnical work during and after an earthquake compared to specified performance criteria

3.53
shallow foundation
foundation having a small depth to width ratio, which is supported directly by soil at or near the ground surface without using piles or other structural elements

EXAMPLES Spread foundation, footing foundation.

3.54
site amplification factor
factor describing the increase in amplitude of earthquake motions in local soil deposit, defined as the ratio of the peak ground surface motion to the peak earthquake motion input to the local soil deposit

3.55
site classification
differentiation of sites based on soil profile and other parameters

3.56
site response analysis
analysis of the response of a site to earthquake ground motion taking into account the local soil deposits

3.57
site-specific
characterization of conditions specific to a site

3.58
sliding soil mass
portion of a geotechnical work, typically defined as that part of the soil or rock expected to slide along a failure surface

3.59
soil-structure interaction
effect by which soil and adjacent structures mutually affect their overall response

3.60
spatial variation of ground motion
lateral variations of ground motion over a given area

3.61
stress resultants
bending moments, shear forces and axial forces in a structure

3.62
subgrade reaction
resulting stresses on a surface in the ground (typically a surface of a foundation or retaining wall) due to external loading

3.63
superstructure
that part of a structure constructed above the ground surface

NOTE This definition is adopted for the purpose of this International Standard (for further discussion, see H.2).

3.64
surface wave
seismic wave that travels along the ground surface and whose amplitude decreases exponentially in the half space with depth

3.65
threshold limit
limit beyond which a structure exhibits an irreversible response

EXAMPLES Sliding limit, elastic limit.

3.66
total stress analysis
analysis without explicit consideration of pore pressure changes

EXAMPLES Linear analysis, equivalent linear analysis, non-linear total stress analysis.

4 Symbols and abbreviated terms

CPT cone penetration test

FE finite element

LDPT large diameter penetration test; detailed specifications are available for Becker penetration test

PSHA probabilistic seismic hazard analysis

SPT standard penetration test

1-D one-dimensional

2-D two-dimensional

3-D three-dimensional

5 Principles and procedure

5.1 Principles

5.1.1 Purposes and functions

In designing geotechnical works, the purposes and functions shall be defined in accordance with broad categories of use such as commercial, public and emergency use.

5.1.2 Performance objectives for seismic design

Performance objectives for seismic design of geotechnical works should generally be specified on the following basis, depending on the expected functions during and after an earthquake:

— serviceability during and after an earthquake: minor impact to social and industrial activities, the geotechnical works may experience acceptable residual displacement, with function unimpaired and operations maintained or economically recoverable after temporary disruption;

— safety during and after an earthquake: human casualties and damage to property shall be minimized, geotechnical works that are an essential part of a facility handling hazardous materials or a post-earthquake emergency facility shall maintain full operational capacity, and geotechnical works shall not collapse.

The performance objectives should also reflect the possible consequences of failure.

Seismic actions on geotechnical works shall be specified, which are compatible with the performance objectives.

NOTE The collapse of a certain type of geotechnical works such as pipelines might not necessarily cause human casualties if fail-safe measures such as shutdown valves are provided. In this design situation, the collapse can be allowed.

5.1.3 Reference earthquake motions

For each performance objective described in 5.1.2, reference earthquake motions shall be specified for evaluating seismic performance of the geotechnical works as follows:

- for serviceability during or after an earthquake: earthquake ground motions that have a reasonable probability of occurrence during the design working life;
- for safety during or after an earthquake: earthquake ground motions associated with rare events that may involve very strong ground shaking at the site.

NOTE Annex D describes in more detail the concepts of reference earthquake motions and their applicability in different circumstances.

5.1.4 Performance criteria and limit states

Performance criteria shall generally be specified by engineering parameters that characterize the response of geotechnical works to the reference earthquake motions. These engineering parameters shall be specified considering the design working life.

The engineering parameters depend on the process for verifying that the performance criteria have been met. The importance of the facility differentiates the level of performance objectives. These issues shall be taken into account in the formulation of the performance criteria.

The seismic performance of geotechnical works can be described with reference to a specified set of limit states. These limit states are

- serviceability limit state during or after an earthquake: a limit state for satisfying serviceability during and after an earthquake, and defined by an acceptable state of displacement, deformation, or stress, and
- ultimate limit state during or after an earthquake: a limit state for satisfying safety requirements during and after an earthquake, and defined by a state with appropriate margin against collapse.

More than one serviceability limit state may be introduced. For example, if one serviceability limit state is defined as the state with no residual displacements, another serviceability limit state may be defined as the state with an acceptable residual displacement and operation of the facility recoverable after minimum disruption with reasonable cost for repair.

One may evaluate only one limit state, provided that the seismic performance objectives specified by other limit states can be satisfied through the evaluation of the one limit state.

NOTE 1 In conventional seismic design of geotechnical works based on the equivalent static method, a seismic coefficient has been used to achieve both serviceability and safety during and after an earthquake. However, as a result of case histories of seismic damage during the 1990s, limitations of conventional seismic design have been recognized widely. The approach described in this International Standard can be used to overcome these limitations.

NOTE 2 The conventional approach in which margin to a specified limit state is specified in terms of the load factor is described in ISO 3010.

5.1.5 Specific issues related to geotechnical works

Seismic actions on geotechnical works shall be specified taking the following factors into account:

- seismic response that involves non-linear behaviour of soil and structural materials;
- appropriate mode of and path to failure so that damage can be readily repaired and local failure of a geotechnical work does not immediately lead to global failure;
- performance criteria in terms of residual displacements, deformations, strains and stability;

— soil-structure interaction, including fluid-structure interaction, that is often simplified as actions on a local system within a global system.

These factors can be sensitive to the details of earthquake ground motions. Improved knowledge shall be used through the procedures described in Clause 6 for evaluating earthquake ground motions in designing geotechnical works.

5.2 Procedure for determining seismic actions

Seismic actions on geotechnical works shall be determined as follows:

1st stage: characterize

— the firm ground (or bedrock) motion at the site through seismic hazard analysis;

— the fault displacements if applicable;

— the free field earthquake motions by site response analysis; and

— the potential for earthquake-induced phenomena such as ground failure and other geotechnical hazards, including liquefaction;

2nd stage: specify, based on the results of the 1st stage, the seismic actions due to

— the earthquake ground motions;

— the ground displacements due to fault movement; and

— ground failure and other geotechnical hazards, taking due account of the methods of analysis to be used for modelling the geotechnical works.

Clauses 7 to 9 describe seismic actions on various models of analysis.

NOTE Annex A presents the primary issues for specifying seismic actions. Seismic actions depend on the model of analysis.

6 Evaluation of earthquake ground motions, ground failure, and fault displacements

6.1 General

6.1.1 Earthquake ground motions and fault displacements

In the 1st stage described in 5.2, earthquake ground motions defined in 5.1.3 and fault displacements shall be evaluated for use as basic variables in subsequent analyses (i.e. in the 2nd stage described in 5.2) for specifying seismic actions on geotechnical works.

6.1.2 Ground failure and other geotechnical hazards

Liquefaction potential shall also be evaluated in the 1st stage described in 5.2 (see Annex G). If liquefaction is judged to occur, the effects of liquefaction shall be incorporated in the 2nd stage described in 5.2 either as seismic actions or effects on the model of the soil-structure system, depending on the models and methods of analysis used. Ground displacements due to liquefaction, including induced ground displacement, shall be evaluated in the 1st stage described in 5.2 as basic variables to be used in the subsequent analysis for specifying seismic actions.

The potential for ground failure in the form of landslides or deformations shall be evaluated.

The potential for flooding or inundation due to subsidence or ground failure may be considered.

6.2 Seismic hazard analysis

6.2.1 Probabilistic and deterministic analyses

The earthquake ground motions, liquefaction potential, ground failure, and fault displacements shall be determined by either probabilistic or deterministic analyses.

The earthquake ground motion for evaluating serviceability during or after an earthquake shall be determined by probabilistic analysis.

The earthquake ground motion for evaluating safety during or after an earthquake shall be determined by either probabilistic or deterministic analysis. This earthquake ground motion can be determined by deterministic analysis when an active seismic fault is assumed to be located nearby. As in ISO 3010, the earthquake ground motion for evaluating safety during or after an earthquake in a region of low seismicity may be determined by deterministic analysis (see Annexes C and D).

NOTE 1 A deterministic analysis evaluates earthquake ground motion by selecting individual earthquake scenarios, including earthquake magnitude, fault location, fault dimension, and source mechanism. Deterministic analysis does not explicitly consider the probability of occurrence of earthquakes, but it does consider uncertainties involved in the evaluation of ground motion from a scenario earthquake. An approximate range of probability of occurrence of the earthquake ground motion from the scenario earthquake can be assessed taking into account the regional seismic activity.

NOTE 2 At the current state of practice, earthquake ground motions are often determined on an empirical or historical basis.

NOTE 3 An active seismic fault is a fault that is capable of generating earthquakes and has moved during the recent geological period. There are wide variations in the time periods since the last fault movement that are used to define an active seismic fault.

6.2.2 Analysis for evaluation of earthquake ground motion

Both probabilistic and deterministic seismic hazard analyses should capture the characteristics of the ground motions based on earthquake magnitude, fault type and distance with or without site parameters. More detailed seismic hazard analyses should capture the near source effects and directivity effects and should be based on seismic source parameters (e.g. the geometry of the active fault and propagation of fault rupture), attenuation of earthquake motions from the fault, and deep basin effects. The uncertainties in the model parameters of the seismic source, attenuation relations, and deep basin effects should be considered (see Annex D).

Seismic hazard analysis methods including empirical, semi-empirical, and theoretical methods, or a combination of these methods, shall be chosen, consistent with the degree of refinement required for analysis of the geotechnical works, based on

— the importance of a structure, and

— the available information on seismic faults and deep basin structures in the vicinity of a site.

Results of seismic hazard analysis may be available over a country or region from the relevant authorities giving the representative values of earthquake ground motions for use in the subsequent analyses.

6.2.3 Outputs of seismic hazard analysis

Earthquake ground motions at the interface between firm ground and local soil deposits shall be developed through seismic hazard analysis for use in design of geotechnical works.

The earthquake ground motions can be specified in terms of simple scalar values (e.g. peak acceleration, peak velocity, peak displacement, Fourier and response spectral values) or time histories of acceleration, velocity, and displacement. The earthquake ground motions can include spatial variation. An appropriate set of variables shall be evaluated for specifying the seismic actions depending on the models and the methods of analysis.

The ground motions at the interface between the firm ground and local soil deposit can be used in a subsequent analysis for site response analysis, assessment of liquefaction potential and for dynamic analysis of soil-structure systems.

NOTE Even if the earthquake ground motions at the ground surface are not directly used for seismic design of the geotechnical works, it is advisable to compute these motions in order to confirm the consistency of the seismic design with the design of buildings and other structures constructed above the ground surface.

6.3 Site response analysis and assessment of liquefaction potential

6.3.1 General

The earthquake motions at the ground surface and within the subsoil shall be obtained for use in determining seismic actions on geotechnical works. The assessment of liquefaction potential shall also be performed for evaluating the effects on performance of geotechnical works.

To this end, the methods may be broadly categorized as follows (see Figure E.7):

a) empirical analysis: based on a site category using prescribed site amplification factors;

b) site-specific simplified analysis: based on the assumed response mode on a site-specific basis [one-dimensional (1-D) lumped mass model];

c) site-specific simplified dynamic analysis: based on total or effective stress analysis on a site-specific basis, typically by 1-D analysis;

d) site-specific detailed dynamic analysis: based on coupled soil-structure interaction analysis of geotechnical works on a site-specific basis. For foundations, 2-D or 3-D analysis should be performed.

From the methods a) through c), the computed earthquake motion variables at the ground surface, ground displacement in the subsoil, and liquefaction potential, can be used as input for subsequent simplified analysis of geotechnical works, as described in Clauses 8 and 9. The method d) directly computes the seismic response of geotechnical works.

The methods used for site response analysis and assessment of liquefaction potential shall be selected on the basis of

— required seismic behaviour specified by performance criteria, and

— quality of geotechnical data from the site.

Effects of topography and irregular stratigraphy should be considered where applicable.

6.3.2 Empirical analysis

In empirical analysis, local site effects may be evaluated using prescribed site amplification factors that are based on statistical analysis of field data and associated with specific site categories. *In-situ* site parameters such as shear wave velocity, standard penetration test (SPT) *N*-value, and cone penetration test (CPT) data over a specified or total depth and the thickness of the local soil deposits above the firm ground can be used to establish the site classification. Geological data can also be used to establish the site classification. This site classification leads to the use of specified site amplification factors or site dependent response spectra.

Geotechnical characterization through microtremor measurements and any available earthquake motion records obtained nearby would improve the reliability of the evaluation of local site effects.

From the results of the site response analysis, liquefaction potential can be evaluated based on appropriate *in-situ* geotechnical tests, including SPT and CPT for sandy or non-plastic silty soil, or large diameter penetration tests (LDPT) for gravelly soil, using generally accepted empirical correlations.

6.3.3 Site-specific simplified analysis

In the simplified analysis of site response, local site effects may be evaluated with the aid of modal analysis on a site-specific basis. This type of analysis can determine maximum site responses, including peak ground displacements, at specified depths in the ground.

From the results of the site response analysis, the liquefaction potential of sandy or non-plastic silty soils can be evaluated using generally accepted liquefaction assessment charts based on results of *in-situ* geotechnical tests, including SPT, CPT, LDPT or shear wave velocity measurement.

6.3.4 Site-specific simplified dynamic analysis

Dynamic site response may be evaluated numerically either through total stress analysis of the free field using equivalent linear or non-linear models or through effective stress analysis. This analysis is typically done by 1-D analysis. The site response can be represented by time histories of acceleration, shear stress, and shear strain at specified locations in the ground.

Shear stress ratios derived from time histories from a total stress analysis are used for evaluating liquefaction potential based on a comparison with the cyclic resistance evaluated by cyclic laboratory tests, and/or from empirical procedures based on appropriate *in-situ* geotechnical test data, including SPT, CPT, LDPT data, or shear wave velocity.

If a geotechnical work is designed for a site susceptible to liquefaction, effective stress analysis of the free field response may be carried out to determine the relevant site response parameters. Total stress analysis may also be used by incorporating the effects of liquefaction through appropriate reduction of shear moduli.

6.3.5 Site-specific detailed dynamic analysis

In detailed dynamic analysis of a soil-structure system, site response is often not evaluated independently but can be evaluated as a part of the soil-structure interaction analysis of geotechnical works on a site-specific basis. The analysis can be done through appropriate numerical procedures such as finite element, finite difference, or boundary element methods. The analysis can be carried out on 2-D or 3-D models of the soil profile (see 9.2).

6.4 Spatial variation

6.4.1 General

The spatial variation of earthquake ground motions shall be evaluated for the design of a long or a large structure when the lateral dimension of the structure is large enough compared with the representative seismic wavelength. Annex F describes this issue in more detail.

Earthquake ground motions can vary spatially due to signifizant variation in topography, soil properties, and stratigraphy in the lateral direction. Appropriate characterization for lateral variation in these geotechnical conditions shall be performed. The characterization effort can involve additional field tests, evaluation of uncertainty through different models of the site profile, or a combination of both.

When the lateral variation in the geotechnical conditions is negligible, the horizontal wave propagation effect can be the major cause of the spatial variation. Among the seismic waves that cause the horizontal wave propagation effect are surface waves and inclined shear waves. Parameters such as phase velocity, wavelength and direction of propagation shall be appropriately defined for evaluating spatial variation. In addition to the horizontal wave propagation effect, spatial incoherency, which can be predominant at high frequencies, may be defined using a coherency function.

Analysis of spatial variation may be broadly categorized as follows:

a) empirical analysis: based on an assumed distribution of harmonic static ground displacement in the horizontal direction, or based on the coherency function. Such analysis can be combined with the site response analysis described in 6.3.2 or 6.3.3;

b) site-specific simplified analysis: based on the surface wave propagation effect, which is evaluated on a site-specific basis. Such analysis may be combined with the site response analysis described in 6.3.4;

c) site-specific simplified dynamic analysis: based on the site response analysis (6.3.4) performed at multiple locations;

d) site-specific detailed dynamic analysis: based on the evaluation of the effects of lateral variation in geotechnical conditions.

The analysis described in a) and b) evaluates the spatial variation due to horizontal wave propagation effect and/or spatial incoherency and is applied where the lateral variation in the geotechnical conditions is not significant. The analysis described in c) and d) evaluates the spatial variation due to lateral variation in geotechnical conditions.

Spatial variations in soil conditions or properties can affect the local displacements of a geotechnical work even if the earthquake ground motions are not considered to vary spatially across the structure. Allowance shall be made for variations in local displacements based on knowledge of the site heterogeneity, the characteristics of the structure, and the simplifications of the site conditions made for analysis (see Annex L).

6.4.2 Empirical analysis

In empirical analysis, spatial variation of earthquake ground motions may be evaluated based on an assumed distribution of harmonic static ground displacement in the horizontal direction at the depth of a buried structure or at the ground surface for a long or a large structures constructed above ground. The wavelength can be determined based on the apparent seismic wave velocity in the horizontal direction and the period of the seismic wave that causes the spatial variation. This analysis may be combined with the site response analysis described in 6.3.2 or 6.3.3 for the subsequent evaluation of seismic actions on geotechnical works.

In empirical analysis, spatial variation can be evaluated based on the combination of the apparent velocity of seismic waves and coherency function appropriately calibrated, using recorded earthquake ground motions.

6.4.3 Site-specific simplified analysis

Horizontal wave propagation effects, including the frequency-dependent nature of the phase velocity, may be evaluated either by elastic-wave velocity structures above and below the interface between the firm ground and the local soil deposit, or with *in-situ* array observations of microtremors and/or earthquake ground motions. This analysis may be combined with the site response analysis described in 6.3.4 for the subsequent evaluation of seismic actions on geotechnical works.

6.4.4 Site-specific simplified dynamic analysis

In site-specific simplified dynamic analysis, spatial variation of earthquake ground motions may be evaluated based on the site response analysis described in 6.3.4 applied at multiple locations. In this analysis, the spatial variation such as that due to phase difference at the firm ground should be considered.

6.4.5 Site-specific detailed dynamic analysis

In site-specific detailed dynamic analysis, earthquake ground motions with the effects of spatial variation shall be evaluated based on the effects of lateral geotechnical heterogeneity. The deep basin effects that depend on the heterogeneity below the local soil deposit shall also be considered. The analysis can be done through appropriate numerical methods, including finite element, finite difference, or boundary element method. A complete analysis of seismic wave propagation from the source to the site can be performed by incorporating a fault rupture model in the analysis. The site-specific detailed dynamic analysis may be integrated as a part of the site response analysis described in 6.3.5.

6.5 Fault displacements, ground failure, and other geotechnical hazards

Construction of geotechnical works at a site on or in the vicinity of a known active fault, or at a site with potential ground failure or other geotechnical hazards, should be avoided if possible.

Otherwise, the effects of fault displacements, ground failure and other geotechnical hazards shall be considered in the design of geotechnical works (see Annex L). The basic design procedure can be described as follows:

a) evaluate the location and associated displacements of fault, ground failure or other geotechnical hazards;

b) incorporate means to allow for fault displacements or to mitigate damage from ground failure or other geotechnical hazards, through structural or geotechnical design or other options.

6.6 Paraseismic influences

This standard may be used as a preliminary standard for paraseismic influences such as those due to underground explosions and shocks. In the context of this International Standard, sources of paraseismic influences include various sources of vibration, e.g.

a) activities related to quarrying and mining, and

b) collapse of abandoned mines.

7 Procedure for specifying seismic actions

7.1 Types and models of analysis

7.1.1 General procedure

Following assessment of the free field earthquake motions, fault displacements, ground failure, and other geotechnical hazards, the seismic actions on the geotechnical works shall be appropriately specified. The following procedure shall be used:

— 1st step: select type of analysis (7.1.2);

— 2nd step: select model and method of analysis (7.1.3);

— 3rd step: specify performance criteria parameters (7.1.4);

— 4th step: perform geotechnical characterization (7.1.5).

The definition of seismic actions varies according to the nature of the geotechnical works. Provided in 7.2 and 7.3 is guidance on the types of seismic actions for equivalent static and dynamic analysis.

— 5th step: evaluate the seismic actions based on an equivalent static (Clause 8) or dynamic (Clause 9) approach.

7.1.2 Types of analysis

The type of analysis to be adopted for evaluating the seismic performance of the geotechnical works shall be chosen based on

— the available data,

— the importance of the structure and performance objectives,

— the performance criteria parameters, and

— the level of complexity and non-linearity.

The types of analysis may be broadly categorized as follows:

1) equivalent static: the peak values or a fraction of the peak values of the dynamic responses, such as inertia forces and ground displacements, are idealized as static actions on models for analysis of geotechnical works. In cases where a fraction of the peak value is adopted, this fraction may be evaluated on the basis of relevant case history data, 1 g or centrifuge model tests, and/or numerical analysis and should be determined in accordance with specified limit states;

2) dynamic: the dynamic responses of analytical or computational models are directly computed for the evaluation of the seismic performance of geotechnical works.

   Each of these categories may be further subdivided based on the modelling of the geotechnical works as follows:

   A) simplified: soil-structure interaction is modelled as an action on a structure model defined in a global soil-structure system;

   B) detailed: the interaction of a soil-structure system is included in a global computational model.

NOTE Soil-structure interaction can include fluid-structure interaction.

7.1.3 Models for analysis

Models for analysis of geotechnical works classified by the type of analysis may be further characterized by the modelling of

— non-linearity,

— inertial and kinematic soil-structure interaction, and

— interaction with a superstructure (if applicable),

and by

— the number of dimensions considered in the analysis, 1-D, 2-D or 3-D, boundary conditions and size/scale of the model, and

— the numerical algorithms and procedures.

The most appropriate model and method of analysis shall be used for evaluating the seismic performance of the geotechnical work (see Annex H).

NOTE Verification of models for analysis can include comparison with case histories, 1g or centrifuge model tests depending on the complexity of the soil-structure system.

7.1.4 Performance criteria parameters

Depending on the type of analysis, the following performance criteria parameters can be evaluated:

1A) simplified equivalent static analysis:

— margins to threshold limit for overall stability,

— margins to structural elastic limits,

— acceptable residual responses based on assumed failure modes;

1B) detailed equivalent static analysis:

— acceptable peak and/or residual response,

— failure modes (see Annex H.1);

2A) simplified dynamic analysis:

— acceptable peak and/or residual responses based on assumed failure modes,

— margins to structural elastic limits where applicable;

2B) detailed dynamic analysis:

— response modes and acceptable peak response values from equivalent linear analysis,

— failure modes and acceptable peak and residual response values from non-linear analysis.

7.1.5 Geotechnical characterisation

Geotechnical and material studies shall be carried out to determine appropriate input parameters for the selected models and types of analyses. Effects of response control and ground improvement shall also be considered where applicable (see Annex M).

7.2 Seismic actions for equivalent static analysis

7.2.1 Simplified equivalent static analysis (1A)

Seismic actions for simplified equivalent static analysis of geotechnical works may include the following:

a) seismic actions from a superstructure: actions such as inertia forces on a superstructure, typically defined for a shallow or deep foundation;

b) seismic actions without spatial variation: actions such as those due to ground displacement at a site, acting on the buried portion of a structure, typically defined for a deep foundation or a transverse section of a buried structure;

c) seismic actions with spatial variation: actions due to transverse and longitudinal components of seismic ground displacements with spatial variation resulting from horizontal wave propagation effect or lateral variation in geotechnical properties, typically defined for a long or large structures;

d) seismic earth and hydro-dynamic pressures: actions from the ground retained by a structure and/or from the fluid in front or behind it, typically defined for a retaining wall;

e) seismic actions on soil and structure masses: actions due to the inertia forces on a selected soil block, typically defined for earth structures.

7.2.2 Detailed equivalent static analysis (1B)

Seismic actions for detailed equivalent static analysis of geotechnical works may be specified in terms of inertia forces distributed over the domain of analysis of the global system.

7.3 Seismic actions for dynamic analysis

7.3.1 Simplified dynamic analysis (2A)

Seismic actions for simplified dynamic analysis of geotechnical works may be specified through the same procedure as the equivalent static analysis, except that the seismic actions are considered to be a function of time and therefore the simplified dynamic analysis is performed in the time domain.

NOTE Annex H discusses the differences between simplified equivalent static and simplified dynamic analyses in more detail.

7.3.2 Detailed dynamic analysis (2B)

Seismic actions for detailed dynamic analysis of geotechnical works are time histories of earthquake ground motions or forces defined at the bottom and side boundaries of the global computational model.

The effects of liquefaction and induced ground displacements may be determined directly by a detailed dynamic analysis (see Annex G).

8 Seismic actions for equivalent static analysis

8.1 Seismic actions for simplified equivalent static analysis

8.1.1 Seismic actions from a superstructure

In simplified equivalent static analysis of a shallow or deep foundation, the interaction between a superstructure and a foundation can be idealized in terms of actions from the superstructure.

Seismic actions from a superstructure may be specified in terms of the horizontal and vertical forces with or without a moment at the instance of the peak shear force acting at the base of the superstructure. These seismic actions may be specified as described in ISO 3010 in terms of loading on the superstructure, on the basis of the acceleration response spectrum defined at the ground surface. The effect of overstrength in a superstructure shall be considered when evaluating the seismic actions from the structure through the structural factor defined in ISO 3010.

Non-linear behaviour of foundations and soils shall be incorporated in the model of analysis either through linear modelling with reduced stiffness or non-linear modelling. Further non-linear effects, such as P-delta effects, should be taken into account where appropriate.

Performance criteria for shallow or deep foundations may be specified in terms of the following parameters:

a) for shallow foundations:

— margins to the threshold limits of sliding and bearing capacity,

— margins to the structural elastic limits specified in terms of stress resultants (i.e. bending moments, shear forces, and axial forces) or stresses,

— acceptable residual responses specified in terms of displacements, tilting, and uplift based on assumed failure modes;

b) for deep foundations:

— margins to the threshold limits of bearing capacity, pull-out resistance, and lateral resistance;

— margins to the structural elastic limits specified in terms of stress resultants or stresses;

— acceptable response beyond structural elastic limit characterized by displacements and strains of piles based on an assumed failure mode.

For deep foundations, actions due to ground displacements, described in 8.1.2, may be combined with the actions from a superstructure (see Annex H). Consideration should be given to the possibility that the maximum ground displacements and the maximum acceleration of superstructure may not occur at the same time (see Annex I).

NOTE When part of the structure is below ground level, the actions from that part of the structure on the foundation should also be considered (see H.2).

8.1.2 Seismic actions without spatial variation

In simplified equivalent static analysis of geotechnical works, soil-structure interaction can be idealized as actions on a structural model by the surrounding ground.

For foundations, the actions of the ground may be specified in terms of the displacement distribution of the subsoil relative to the base of the foundation at the instant when the maximum relative displacement occurs between the top and bottom of the buried portion of the structure (see Annex I).

Non-linear behaviour of foundations and soils shall be incorporated in the model for analysis either through linear modelling with a reduced stiffness or non-linear modelling. Further non-linear effects, such as P-delta effects, should be taken into account where appropriate.

Performance criteria for buried portions of a structure may be specified in terms of the following parameters:

a) for deep foundations:

  — the same as 8.1.1 b);

b) for transverse sections of buried structures:

  — margins to the structural elastic limits specified in terms of stress resultants or stresses.

8.1.3 Seismic actions with spatial variation

Spatial variation in the ground motion commonly affects long or large structures such as large dams, tunnels or tube-like buried structures. Spatial variation in the ground motions can also affect long bridge structures and certain forms of embankment.

In simplified equivalent static analysis of a tunnel or long tube-like buried structure or a large geotechnical work, soil-structure interaction may be represented as actions from the surrounding or adjacent ground on a structural model.

Seismic actions from the ground may be specified in terms of displacement distribution of the subsoil at the depth of the buried structure and may be evaluated through the empirical (6.4.2) or site-specific simplified (6.4.3) analysis. For a large geotechnical work constructed above the ground surface, seismic actions can be specified in terms of deformation of the ground surface or at the top of the foundation. These actions may be taken as the same as the free field displacement with spatial variation or the displacement distribution at the instant when the maximum deformation occurs across the structure.

Non-linear behaviour of soils shall be considered when large relative displacements are expected. In such cases, the non-linearity can be modelled approximately by reducing the moduli of the soils based on the expected magnitude of the induced soil displacements or strains.

Performance criteria for long or large structures may be specified in terms of the following parameters:

— the margins to the structural elastic limits with respect to the stress resultants or stresses;

— acceptable cyclic strains with or without consideration of fatigue.

8.1.4 Seismic earth and hydro-dynamic pressures

In simplified equivalent static analysis of a retaining wall or a transverse section of a shallow or semi-buried structure such as a rigid culvert or semi-buried road, soil-structure interaction can be simplified to earth pressures acting on a wall or idealized buried structural model. Soil-structure interaction for a flexible culvert may be modelled as described in 8.2. If a part of the wall or structure is exposed to fluid, fluid-structure interaction may be simplified to hydro-dynamic pressures.

The earth pressures and hydro-dynamic pressures may be specified based on the peak acceleration response of the free field or a fraction thereof at the ground surface (see Annexes H and J).

Non-linear behaviour of soils should be carefully considered as this can lead to amplification or attenuation of the peak acceleration.

Performance criteria may be specified in terms of the following parameters:

a) for a gravity wall:

— margins to the threshold limits for sliding, overturning, and bearing capacity,

— acceptable residual response specified in terms of displacements based on an assumed failure mode;

b) for an embedded wall:

— margins to the threshold limits for overall stability,

— margins to the structural elastic limits of the embedded portion of a wall;

c) for a rigid buried structure:

— margins to the structural elastic limits of a buried structure.

NOTE Simplified equivalent static analysis using the earth and hydro-dynamic pressures has been shown by case histories to be effective for moderate earthquake ground motions. However, case histories of seismic damage during the 1990s have demonstrated limitations of this method for intense earthquake ground motions and an improved method of analysis has been emerging as shown in Annex J or other methods of analysis, including the dynamic analysis as described in Clause 9, has been used.

8.1.5 Seismic actions on soil and structure masses

For simplified equivalent static analysis, a simplified model of the earth structure may be used in which an inertia force acts on a sliding wall or a sliding soil mass model bounded by an assumed failure surface.

The inertia force should be specified based on the peak acceleration response at the centre of gravity of the assumed mass of soil or structure. This inertia force may be specified in terms of the peak or a fraction of the peak acceleration response of the free field at the ground surface if the earth structure is relatively small. If an earth structure is relatively large, this inertia force may be specified based on the response accelerations evaluated by the total stress analysis of the earth structure using an equivalent linear model.

Performance criteria may be specified in terms of the parameters such as margin with respect to the threshold limit for sliding.

NOTE Simplified equivalent static analysis has been shown by case histories to be effective for moderate earthquake ground motions. However, case histories of seismic damage during the 1980s and 1990s have demonstrated limitations of this method for intense earthquake ground motions and an improved method of analysis has been emerging or other methods of analysis, including the dynamic analysis as described in Clause 9, has been used.

8.1.6 Effects of soil liquefaction and induced ground displacement

In simplified equivalent static analysis, the effects of liquefaction and induced ground displacement may be considered as follows (see Annexes G and K):

a) immediately after the triggering of liquefaction: geotechnical works in the liquefied soil may be designed against the inertia forces. The effects of liquefaction may be considered through a reduction factor for the subgrade reaction or the soil stiffness resisting the actions from the superstructure. The effects of ground displacements following liquefaction may also be included, if significant;

b) in the later phase of liquefaction: geotechnical works may be designed taking into account the displacement of liquefied ground with a reduced subgrade reaction, or earth pressures from the liquefied ground. The effects of appropriate inertia forces may also be taken into account, if significant;

c) after reconsolidation of the soil: geotechnical works may be designed for total and differential settlements that may occur across the structure or geotechnical works.

When uplift of buried structures or stability of walls is evaluated, liquefaction effects may be incorporated in the analysis as buoyancy forces, hydro-dynamic pressures, and/or reduction in shear resistance.

NOTE For some types of geotechnical works such as retaining walls, simplified equivalent static analysis can be used where remedial measures such as ground improvement are being implemented. Alternatively, a more sophisticated total or effective stress analysis of liquefaction effects may be performed.

8.1.7 Effects of fault displacements

When a structure crosses an active fault, displacement of the fault may be applied to the structure as a forced displacement (see Annex L).

8.2 Seismic actions for detailed equivalent static analysis

8.2.1 Detailed equivalent static analysis

In detailed equivalent static analysis, the seismic performance of geotechnical works shall be analysed using a global computational model of the soil-structure system. An appropriate model of the superstructure, if applicable, shall also be incorporated in the global model to account for the interaction between the superstructure and the foundation. The effect of overstrength in the superstructure shall be considered either in the modelling or in evaluating seismic actions from the structure.

Appropriate types of numerical models, such as finite element or lumped mass models, should be used for the analysis.

Approximate failure modes and paths may be evaluated by detailed equivalent static analysis.

8.2.2 Seismic actions for a seismic coefficient approach

In detailed equivalent static analysis, seismic actions can be simplified as inertia forces distributed over the analysis domain of the global soil-structure system.

The inertia forces applied over the analysis domain may be specified in terms of the acceleration distribution of free field at the instance when the maximum relative displacement occurs between the top and bottom of the buried structure. In order to take full advantage of a detailed equivalent static analysis, site-specific analysis shall be performed to evaluate the free field acceleration.

Alternative procedures may also be used in which the nodal forces over the finite element domain of analysis are specified based on the strain distribution of free field response. Complimentary shear stresses should also be applied along the interface between a buried structure and the surrounding ground.

Non-linear behaviour of soils and structures shall be incorporated in the model of analysis either through linear modelling with reduced moduli or non-linear modelling.

Performance criteria may be specified in terms of appropriate failure modes and acceptable peak or residual responses of geotechnical works.

8.2.3 Effects of soil liquefaction and induced ground displacement

Effects of soil liquefaction and induced ground displacement may be analysed by incorporating the stiffness and strength reduction in the liquefied soils either through the use of equivalent shear modulus in the linear analysis or through the use of reduced stiffness and residual strength in the non-linear analysis (see G.9 and Annex K).

Allowance shall be made for differential settlements caused by site heterogeneity, taking into account the characteristics of the structure and the simplifications in the analysis.

9 Seismic actions for dynamic analysis

9.1 Seismic actions for simplified dynamic analysis

9.1.1 Seismic actions from a superstructure

In simplified dynamic analysis of a shallow or deep foundation, seismic actions from the superstructure shall be specified in terms of the dynamic response of the superstructure. The effect of overstrength in the superstructure shall be considered in evaluating the seismic actions from the superstructure.

Non-linear behaviour of foundations and soils shall be incorporated in the analysis either through linear modelling with reduced stiffness and increased damping or non-linear modelling. Further non-linear effects, such as P-delta effects, should be taken into account where appropriate.

Performance criteria may be specified in terms of the following parameters:

a) for shallow foundations:

- acceptable residual response specified in terms of displacements due to sliding, soil yielding, and uplift;
- margins to the structural elastic limits of the footing;

b) for deep foundations:

- margins to the threshold limits of bearing capacity, pull-out resistance, and lateral resistance;
- acceptable response beyond structural elastic limit specified in terms strains or ductility factors of piles.

For shallow and deep foundations, actions due to ground displacements, described in 9.1.2, may be combined with the actions from a superstructure.

When part of the structure is below ground level, the actions from that part of the structure on the foundation should also be considered (see Annex H.2).

9.1.2 Seismic actions without spatial variation

In simplified dynamic analysis, seismic actions may be specified in the manner described in 8.1.2, except that the seismic actions are described in terms of time histories of free field motions. These time histories of ground motions shall be based on the free field response to a reference earthquake motion, input at the appropriate level in the free field.

Non-linear behaviour of foundations and soils shall be incorporated in the analysis either through linear modelling with reduced stiffness or non-linear modelling. Further non-linear effects, such as P-delta effects, should be taken into account where appropriate.

Performance criteria may be specified in terms of the following parameters:

a) for deep foundations:

— the same as those shown in 9.1.1 b);

b) for transverse sections of buried structures:

— margins to the structural elastic limits specified in terms of stress resultants or stresses.

9.1.3 Seismic actions with spatial variation

Spatial variation in the ground motion commonly affects long or large structures such as large dams, tunnels or tube-like buried structures. Spatial variation in the ground motions can also affect long bridge structures and certain forms of embankment.

In simplified dynamic analysis using a (massless) beam model of a tunnel or long tube-like buried structure, seismic actions may be specified in the same manner as described in 8.1.3, except that the seismic actions are specified in terms of time histories of the free field dynamic displacement including any spatial variations.

Non-linear behaviour of soils shall be considered when large displacements or strains are expected. In such cases, the behaviour is usually modelled by reducing the stiffness of the soils and increasing the damping factors to be compatible with the expected magnitude of soil displacements or strains. Non-linear behaviour of the buried structure can also be considered.

Performance criteria may be specified in terms of the parameters such as residual displacements and strains beyond the elastic limit states. The effects of fatigue may also be considered when specifying the performance criteria.

9.1.4 Seismic actions on soil and structure masses

In simplified dynamic analysis of a gravity wall or an earth structure, seismic actions may be specified on the basis of the acceleration time histories acting on the sliding wall or sliding soil mass, bounded by an assumed failure surface. Hydro-dynamic pressures may also be considered. The acceleration time history at the ground surface in the free field is appropriate if the wall or earth structure is relatively small. If the wall or earth structure is relatively large, the spatial variation of the inertia forces arising from the variation of accelerations over the structure should be accounted for.

Performance criteria may be specified in terms of the following parameters:

a) for a gravity wall:

— acceptable residual displacement of wall;

b) for an earth structure:

— acceptable residual response defined in terms of crest settlement or displacements at other critical locations relative to the original geometry.

9.1.5 Effects of soil liquefaction and induced ground displacement

In simplified dynamic analysis, the effects of liquefaction and induced ground displacement may be considered as described in 8.1.6 (see G.9 and Annex K).

9.2 Seismic actions for detailed dynamic analysis

9.2.1 Seismic actions for a soil-structure system

Appropriate types of computational models, such as finite element, finite difference or lumped mass models shall be used for analysis. In these types of analyses, site response and liquefaction potential are often not evaluated independently, but are evaluated as a part of the soil-structure or fluid-structure interaction analysis. Firm ground motions or motions at the base of the analysis domain shall be given as seismic actions for the global computational model. Input motions for analysis shall be determined by a site-specific study.

Non-linear stress-strain behaviour of soils and structures, including damping, shall be appropriately modelled for the analysis of the soil-structure systems.

Failure modes and paths can be determined directly by appropriate dynamic analysis. Performance criteria may be specified in terms of appropriate failure modes and acceptable peak or residual responses of geotechnical works.

9.2.2 Effects of soil liquefaction and induced ground displacement

In detailed dynamic analysis, the effects of soil liquefaction and induced ground displacement may be obtained directly from the analysis. Appropriate formulations, constitutive models and numerical procedure shall be used. Detailed geotechnical characterization should be used for determining the model parameters.

NOTE Inertial and kinematic soil-structure interactions are automatically included in a detailed dynamic analysis.

Annex A
(informative)

Primary issues for specifying seismic actions

The primary issues for specifying seismic actions described in this International Standard are shown in Figure A.1. As shown in this figure, seismic actions are determined through two stages (Clause 5). The first stage determines basic seismic action variables, including the earthquake ground motion at the site, the potential for earthquake-associated phenomena such as liquefaction and induced ground displacement, fault displacements and landslide (Clause 6). These variables are used, in the second stage, for specifying the seismic actions for designing geotechnical works (Clauses 7 through 9).

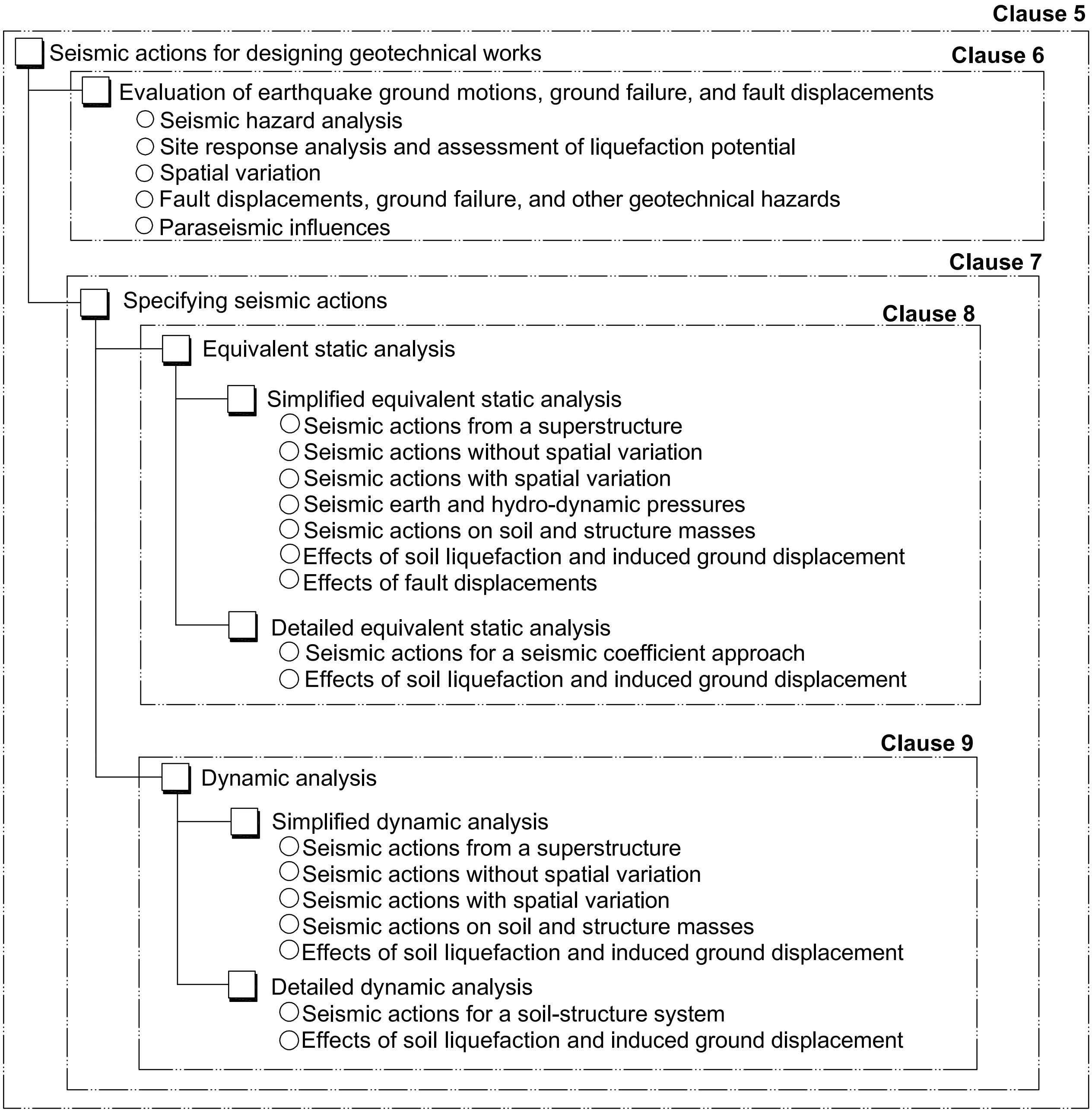


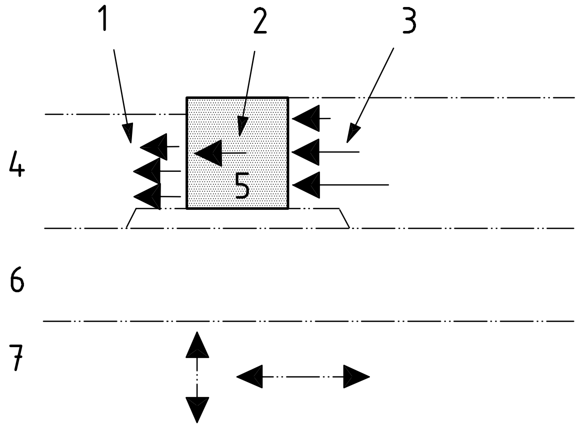
Figure A.1 — Primary issues for specifying seismic actions

In the second stage, the soil-structure interaction (an effect of ground motion by which soil and adjacent structures mutually affect their overall response) plays a major role. Types of analyses are classified based on a combination of static/dynamic analyses (Clauses 8 and 9) and the procedure for soil-structure, or fluid-structure, interaction classified as follows:

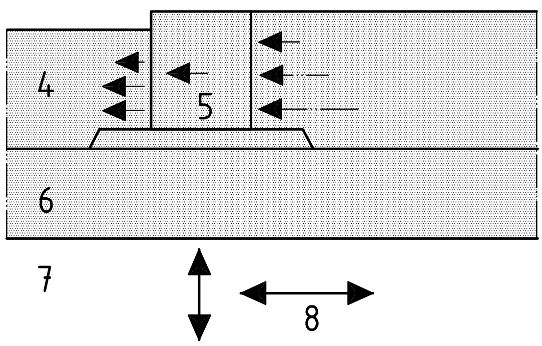
— simplified: the interaction of a soil-structure system is modelled as an action on a substructure (8.1 and 9.1);

— detailed: the interaction of a soil-structure system is modelled as a coupled system (8.2 and 9.2).

For example, in the simplified equivalent static analysis of a caisson quay wall, the model for analysis is defined for the wall as indicated by the shaded area in Figure A.2(a). Actions on this model are inertia force, seismic earth and hydro-dynamic pressures. Action effects of this model are the margin with respect to the threshold levels beyond which the wall begins to slide, overturn, or lose bearing capacity.



a) Simplified model



b) Detailed model

Key

1 hydrodynamic pressure
2 inertia force
3 earth pressure
4 sea
5 caisson
6 seabed
7 firm ground
8 earthquake motions

Figure A.2 — Examples of models of analysis of a caisson quay wall

In detailed dynamic analysis of a caisson quay wall, a model for analysis is defined for an entire earth structure system, including caisson, backfill soil, sea water, and foundation soil below the caisson as indicated by the shaded area in Figure A.2(b). Actions on this soil-structure model are input earthquake motions at the boundary of the domain of analysis. Action effects of this model for dynamic analysis are responses of the soil-structure system, including accelerations, velocities, displacements, stresses and strains in various parts of the soil-structure system. In particular, seismic earth pressures and hydro-dynamic pressures acting on the caisson wall are action effects and computed from, rather than specified for, the response analysis.

These examples show how actions specified for designing a geotechnical work depend on the model of analysis. This principle is adopted in the general procedure for evaluating seismic actions described in Clauses 6 and 7 of this standard, as shown in Figure A.3.

Examples for other geotechnical works are summarized in Annex H. Effects of liquefaction for various models of analysis for geotechnical works are described in Annex K.

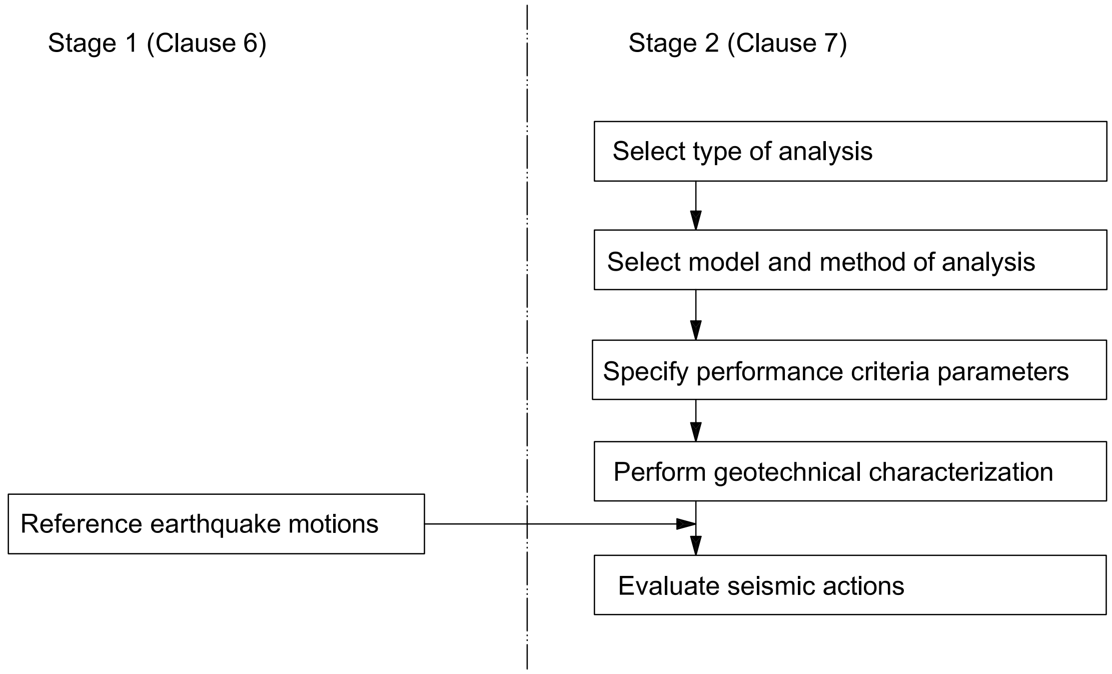


Figure A.3 — Flowchart for specifying seismic actions

Annex B
(informative)

Upper crustal rock, firm ground, and local soil deposit

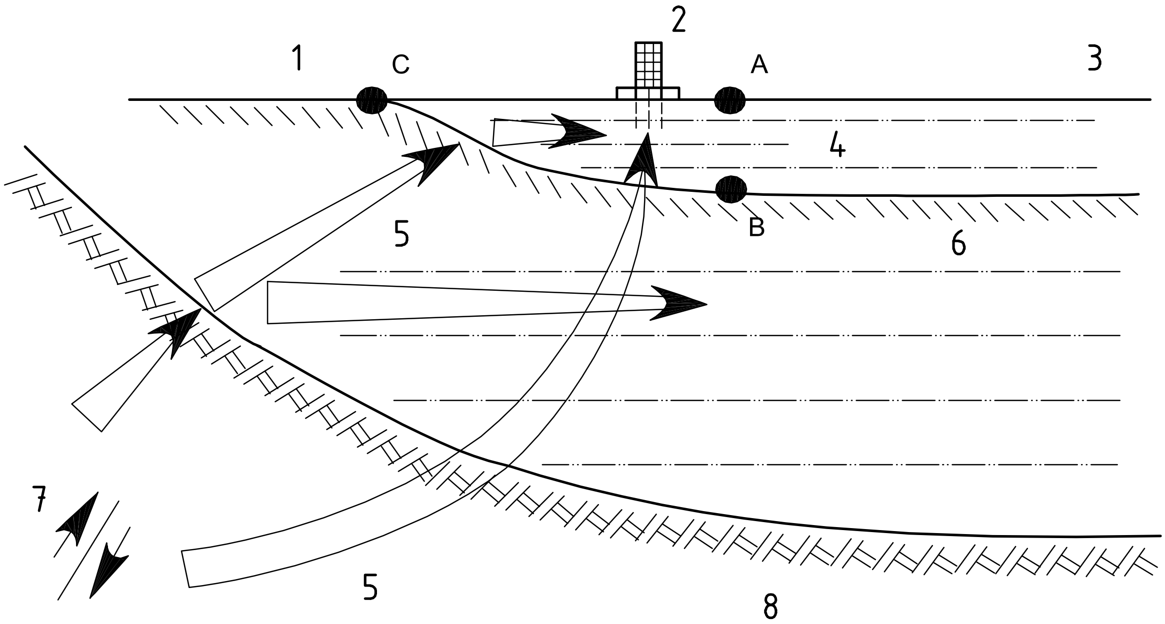
As shown in Figure B.1, seismic waves are generated at a fault rupture and they propagate through upper crustal rock and bedrock to the site of interest. The seismic waves then propagate through the local soil deposits, reaching the ground surface and affecting structures.

The bedrock is idealized, in the analysis of seismic wave propagation, as a uniform material filling the half space. The upper crustal rock having shear wave (S-wave) velocity of 3,5 km/s generally satisfies the definition of the bedrock. In engineering practice, however, the bedrock or its equivalent is defined at the upper boundary of rock or soil layer much softer than the upper crustal rock.

The upper boundary of the bedrock or its equivalent, called “engineering base layer,” may be defined at the interface between the firm ground and local soil deposit. Although the bedrock should be the idealized uniform material filling the half space, there is a geological structure below the engineering base layer. Site response analysis is performed by assuming that the effects of the geological structure below the engineering base layer are negligibly small. This assumption is justified when there is a high contrast in the impedance (i.e. a product of density and wave propagation velocity) between the local soil deposits and the soil or rock below the engineering base layer. S-wave velocity of the soil or rock below the engineering base layer can often be in the range from 300 m/s to 700 m/s.

The effects of the geological structure below the firm ground become significant when the geological structure is characterized as a basin-like geometry, in which seismic waves are trapped and often amplified at the basin edge, and surface waves are generated at the basin edges. These effects are called deep basin effects and are often taken into account in the evaluation of earthquake ground motions in detailed site-specific analysis. In particular, the effects at the basin edge are called basin edge effects.

The site response analysis requires the motion at point B (see Figure B.1) at the interface between the firm ground and local soil deposit. This motion is used as input for the site response analysis. Conventionally, the outcrop motion at point C is first defined (see E.1) and then its incident wave component is calculated at point B for use in the analysis. Alternatively, if the motion at point A is defined, deconvolution can be used for determining the motion at point B (See D.2.2).



Key

1 firm ground outcrop
2 structure
3 ground surface
4 local soil deposit
5 seismic wave propagation
6 firm ground
7 fault rupture
8 upper crustal rock

Figure B.1 — Schematic figure of propagation of seismic waves through the upper crustal rock, firm ground, and local soil deposit

Annex C
(informative)

Design situations for combination of actions

Addressed in 6.2.1 are the circumstances in which it is considered appropriate to compute the earthquake ground motions either by probabilistic or deterministic analysis. Because this International Standard describes guidelines for specifying seismic actions only, 6.2.1 addresses these circumstances in a straightforward manner without classifying actions into variable and accidental actions as described in ISO 2394.

NOTE Variable and accidental actions are defined in ISO 2394 are as follows:

— variable action: action for which the variation in magnitude with time is neither negligible in relation to the mean value nor monotonic;

— accidental action: action that is unlikely to occur with a significant value on a given structure over a given reference period. Note: accidental actions are in most cases of short duration.

However, when a combination of actions, including the seismic actions, is considered in a general design procedure, it may be necessary to specify the design situation as either variable or accidental. This specification can be done by classifying the earthquake ground motion as follows:

— an earthquake ground motion for evaluating serviceability during or after an earthquake: a basic variable for specifying variable action, and

— an earthquake ground motion for evaluating safety during or after an earthquake: a basic variable for specifying either a variable or accidental action.

In accordance with these design situations, an appropriate method of analysis is chosen as follows:

— a basic variable for specifying variable action: determined through a probabilistic analysis;

— a basic variable for specifying accidental action: determined through either a deterministic or probabilistic analysis.

Annex D
(informative)

Seismic hazard analysis and earthquake ground motions

D.1 Seismic sources

D.1.1 Types of seismic sources

Seismic sources can be modelled as areal and as fault sources. Areal sources are used to represent distributed seismicity that cannot be associated with known faults. Fault sources are characterized by clearly identified active faults or large inter- or intra-plate earthquakes occurring periodically in the past. All faults are potential seismic sources. If a fault has no historical record of seismicity, paleoseismic studies are necessary to decide whether the fault should be considered active or not. Trenching across the fault will reveal the dislocations of strata by past earthquakes that can be dated. When this information is combined with geological or geodetic data on slip rates, estimates of the magnitude and frequency of past events outside the historical record can be made. Another method of evaluating paleoseismicity involves dating features of paleoliquefaction such as sand blows and sand intrusions or sand dikes.

In both probabilistic and deterministic seismic hazard analyses, the first step is to identify earthquake sources around the site of interest at distances within which they contribute at least specified minimum hazard values to the site.

D.1.2 Parameters characterizing fault sources

Fault sources are characterized by

— location and geometry of the fault,

— tectonic regime and type of faulting,

— history of seismic activities based on the historical record and/or paleoseismic data from trenching,

— earthquake magnitude,

— average and deviation of recurrence intervals, and

— location of asperities, from which stronger seismic waves are generated.

These parameters are evaluated based on geologic, geodetic, and geophysical information such as seismic reflection tests and GPS (Global Positioning System) monitoring.

D.1.3 Parameters characterizing areal sources

Areal sources are characterized by

— location and geometry of areal seismic source,

— earthquake recurrence rates, and

— maximum earthquake magnitude.

These parameters are evaluated based on past earthquake data, geological structure, and seismotectonic features.

D.2 Earthquake ground motions from scenario earthquakes

D.2.1 Modelling scenario earthquakes and parameters of earthquake ground motions

In both probabilistic and deterministic seismic hazard analyses, earthquake ground motions are generated by scenario earthquakes. Scenario earthquake models vary, from a point source model with a specified location and magnitude, to a seismic fault model with a detailed rupture process specified. Earthquake ground motions may be specified in various ways depending on the design procedure: by simple scalar data such as peak acceleration and velocity, or in more detail by time histories of acceleration, velocity or displacement, response spectra and Fourier spectra.

D.2.2 Methods of evaluating earthquake ground motions

D.2.2.1 General

Earthquake ground motions from scenario earthquakes may be determined by empirical, semi-empirical, theoretical, and hybrid methods. All these methods, even the most sophisticated, are limited in their ability to model the complex mechanism of seismic wave generation and propagation.

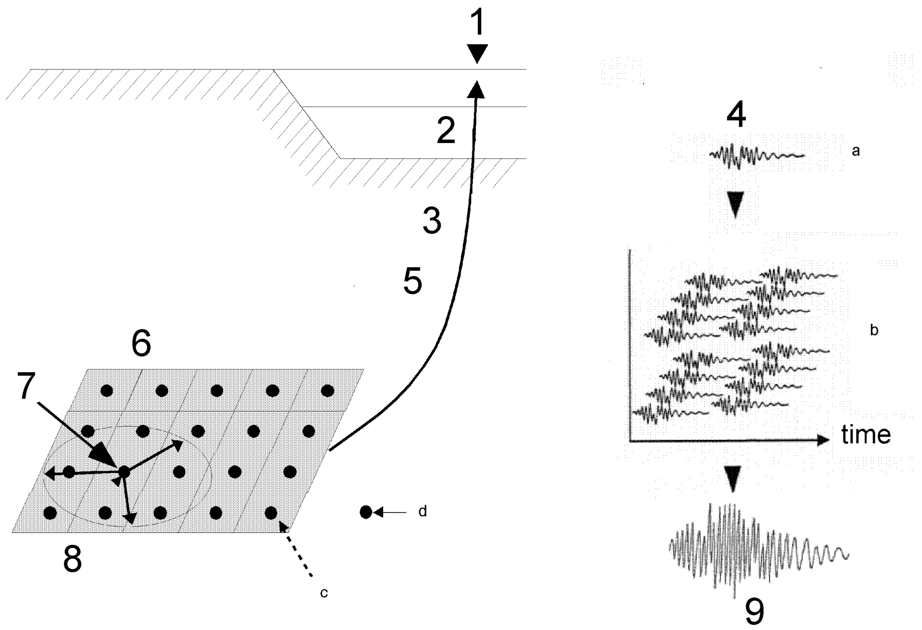
D.2.2.2 Empirical method

Ground motions for design are frequently estimated using attenuation relations that predict peak ground motion parameters such as acceleration or spectral acceleration, in terms of earthquake magnitude, hypocentral depth, and distance from the seismic event for firm ground. There are many different definitions of distance to the event. When these relations or printed plots of these relations are used, it is imperative to determine which distance definition applies. Some attenuation relations include terms that take the type of faulting into account and the effects of surface soils overlying the firm ground. In more recent attenuation relations, the firm ground is defined by specifying its shear wave velocity (see Annex B).

D.2.2.3 Semi-empirical method

The semi-empirical method computes time histories of earthquake ground motions caused by a large scenario earthquake by combining the recorded or simulated earthquake ground motions from smaller earthquakes. The empirical Green's function method uses the small to moderate earthquake ground motions as element motions (i.e. Green's functions). The statistical Green's function method uses simulated ground motions based on the statistical average of the recorded earthquake ground motions as element motions. Figure D.1 illustrates the computation procedure. These methods take into account the detailed fault rupture process and the effects of asperities.

If the empirical or statistical Green’s function at the ground surface (A in Figure B.1) is used, the ground is idealized as linear. Then this motion is used as input motion for deconvolution analysis to obtain the motion at



Key

1 site
2 local soil deposit
3 firm ground
4 recordings of small earthquake motions
5 seismic wave propagation
6 fault rupture process
7 initiation of fault rupture
8 fault plane for large earthquake
9 large earthquake motion

[a] Assuming that a small earthquake motion is generated from each fault element.

[b] Superposition of small earthquake motions taking into account rupture process and distance.

[c] Scaling relation between the large and small earthquakes is used for determining size and number of fault elements.

[d] Small earthquake.

Figure D.1 — Procedure for determining earthquake ground motions by the semi-empirical method

the interface between the firm ground and local soil deposit (B in Figure B.1). The resulting motion at the interface is then used as input for non-linear site response analysis.

D.2.2.4 Theoretical method

The theoretical method computes time histories of earthquake ground motions using both a theoretical seismic fault model and a computational seismic wave propagation model. This method takes into account the detailed fault rupture process and the effects of asperities. This method is used to estimate ground motions in a period range that is long enough not to be significantly affected by the randomness of the fault rupture process. This method is not appropriate for motions in the shorter period range that are significantly affected by the randomness of the fault rupture process.

D.2.2.5 Hybrid method

The hybrid method computes time histories of earthquake ground motions by combining a longer period component determined by the theoretical method and a shorter period component determined by the semi-empirical method. This method takes into account the detailed fault rupture process and the effects of asperities. This method is applicable over a wide period range.

D.2.3 Local Site effects

Earthquake ground motions are affected significantly by both the local soil deposit above the firm ground and the geological structure below the interface between the firm ground and the local soil deposit. Site response analysis evaluates the effect of the local soil deposit as defined in 3.56 whereas the effects of both the local soil deposit and the geological structure below the interface between the firm ground and the local soil deposit are called "local site effects", as defined in 3.33.

There are three empirical methods used to estimate local site effects.

a) Ground motion amplification with respect to a reference site condition may be estimated using empirical amplification factors based on a site classification system. Many building codes use such a method. Some codes uses site categories classified based on the average S-wave velocity over the top 30 m (V_{s30}). These codes also include amplification factors for short and long period motions that are dependent on ground motion intensity in recognition of the non-linear response of soil.

b) If ground motions of the required intensity are available for the site, these should be used directly since they incorporate local site effects.

c) Peak ground motion parameters or spectral values may be estimated using attenuation relations that include a term dependent on site conditions.

In the semi-empirical method, local site effects are automatically taken into account when the earthquake motion records are used as element motions.

In the theoretical method, local site effects are taken into account through a computational model. In particular, a 2-D or 3-D finite difference method can idealize non-uniformity in the horizontal direction, including a basin type geological structure.

D.3 Reference earthquake motions

D.3.1 Introduction

Two general approaches are used to determine ground motions for design; one is probabilistic and the other deterministic. Probabilistic seismic hazard analysis (PSHA) is used to determine ground motions for design that have a specified probability of exceedance. In deterministic analysis, a design earthquake is selected and then the motions are calculated by any of the methods described earlier. In the current practice, it is recognized that both methods have a lot in common and both have a role to play in establishing ground motions, especially for critical structures.

D.3.2 Probabilistic analysis

There are four steps in the process of PSHA: 1) identify active seismic sources that affect the site; 2) characterize the recurrence rates of different magnitudes for each source; 3) select an appropriate attenuation relation for the ground motion parameter of interest, and 4) using all these data, derive the hazard curves for the ground motion parameters of interest. These steps are used to determine the ground motion parameters, including response spectrum, for the specified probability over the reference period.

The probabilistic parameters, such as acceleration, may be used directly in design procedures or in the scaling of appropriate ground motions. They are also used to scale existing normalized design spectra. The probabilistic spectral values can be specified as uniform hazard spectra, which have the same probability of exceedance at all periods. Such spectra may be used as design spectra, although it is more common to use a simplified bi-linear spectrum based on one short period and one long period (NEHRP, 2001). The spectra may be also used in the generation of spectrum compatible ground motions. However, one must not forget that no earthquake can generate such motions. There have not been sufficient studies to ensure that the effects of spectrum compatible motions are consistent with those of real motions.

The motion parameters determined by PSHA result from the contributions of the multiple earthquakes considered in the analysis. To obtain scenario earthquakes for projects that require it, the hazard must be disaggregated into its component magnitude-distance pairs to determine the significant contributors to hazard. The scenario earthquake may be used to determine the normalized response spectral shape or the phase characteristics of the spectrum compatible ground motion.

D.3.3 Deterministic approach

The deterministic approach is applicable and adequate for determining earthquake ground motions for design especially when uncertainty is small in evaluating the parameters of the scenario earthquake. In this case, the deterministic approach can be an adequate method for evaluating a low probability event as an alternative to the probability approach that typically involves a large uncertainty in the tail end of the probability function.

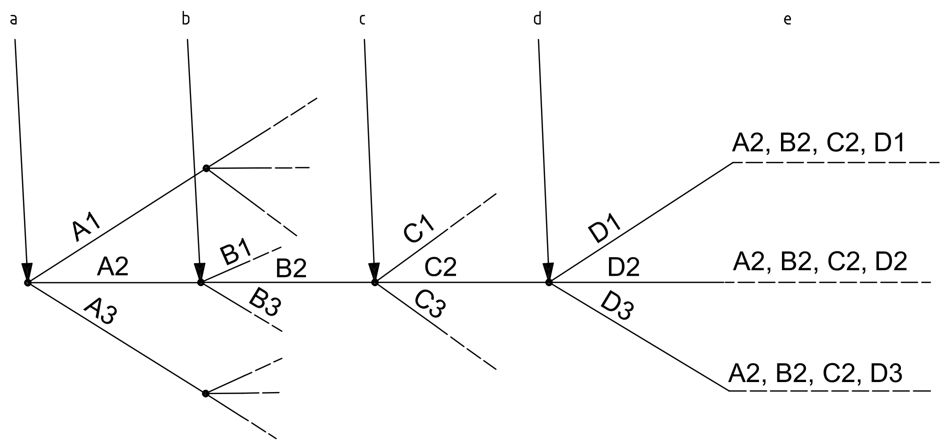
There seems to be a mistaken impression that the deterministic approach does not consider uncertainty. The deterministic approach does include evaluation of uncertainty. It uses the same logic tree approach but does not explicitly include the analytical apparatus of the PSHA.

D.3.4 Evaluation of uncertainty

The distinguishing characteristic of the modern practice is the formal treatment of the uncertainty, associated with almost every aspect of seismic hazard analysis. There are two kinds of uncertainty; aleatory and epistemic. Aleatory uncertainty is due to the random nature of seismic events. No matter how many data are accumulated on ground motions, for example, the standard deviation about the median in attenuation relations remains significant. Epistemic uncertainty is considered to be due to a lack of scientific knowledge and is evaluated by processing opinions from a number of experts. The manner in which such opinions are elicited and interpreted is considered crucial to the final assessment of hazard. Appropriate guidelines should be established for it.

An example of aleatory uncertainty usually considered in seismic hazard analysis is the distribution of values about the median attenuation which is assumed to be a log-normal distribution. This is effectively incorporated in a PSHA.

Coping with epistemic uncertainty depends entirely on expert opinions. Source definition is one of the major contributors to epistemic uncertainty as it involves location and geometry, maximum magnitude, recurrence rates and choice of attenuation relations. It has been demonstrated frequently that expert opinions differ widely and a weighting of the final hazards is necessary to arrive at a decision on a design motion. A logic tree is often used to process the differing expert opinions in the march to a final hazard estimate. A typical logic tree is shown in Figure D.2.



a Combination of active sources.

b Seismicity parameters.

c Maximum magnitudes.

d Ground motion functions.

e Hazard analysis cases.

Figure D.2 — Logic tree representation of uncertain parameters in PSHA

D.4 Probability level

D.4.1 Probability specified for reference earthquake motions

The overriding concern in establishing the reference level of probability for codes or norms is life safety. Next is maintaining a desired serviceability level for more frequent earthquakes that do not threaten life safety. Different probabilities may be developed for different industries and different types of structures. Examples of structures are high consequence dams, nuclear power stations and structures for offshore oil and gas development. The probability levels in building codes are based on the recommendations of leading scientists and engineers and are given authorized status by the adopting authority. Other probability levels may be recommended for other industries and types of structures such as for nuclear power and offshore oil and gas development.

When an organization adopts criteria for acceptable probability levels, they typically take into account factors such as

- life safety or serviceability,
- types of geotechnical work, and
- importance of the structure or consequence of failure.

These probabilities must receive approval from regulating agencies before they are used in practice.

Examples of the probability level for the specific types of structures may be found in the existing regional and national codes or recommended practice such as CEN/TC250/SC8 (2003) for Europe and NEHRP (2001) for U.S.A.

D.4.2 Possible future direction

A method based on the principle of minimum life-cycle cost may be developed in future years. In this method, the probability level is used as a parameter for computing the total cost but not used as a fixed value specified for design. If the required performance level becomes high, the construction cost increases but potential repair cost after the earthquake may decrease. Provided that the life safety requirement is satisfied, the procedure consists of the following steps:

a) specify the required performance level;

b) calculate the construction cost, the intended maintenance cost and the cost for demolishing or de-commissioning when the working life of the structure ends;

c) calculate the potential repair cost after the earthquake, the socio-economic impact due to the earthquake induced damage, taking into account the probability of occurrence of earthquake ground motion;

d) return to a) and repeat the procedure through c), until the minimum life-cycle cost, which is the summation of the costs listed above, is obtained.

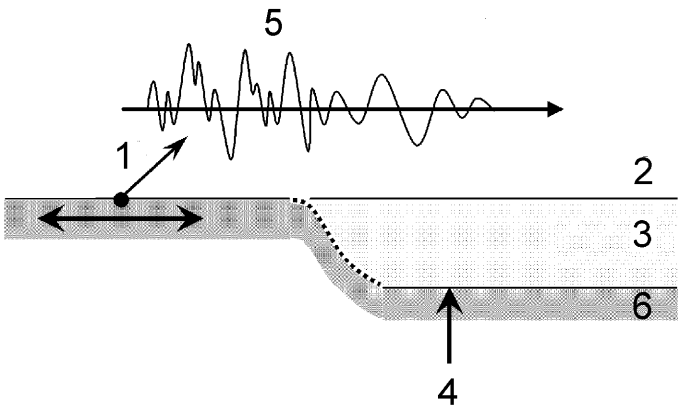
Annex E
(informative)

Site response analysis

E.1 Introduction

Site response analyses are performed in order to evaluate the effects of local soil deposits above the firm ground or upper crustal rock. The input for site response analyses is typically specified based on the firm ground outcrop motion by assuming that this motion is the result of superposition of the seismic waves incident to and reflected at the firm ground outcrop surface as shown in Figure E.1. Time histories of outcrop motion input to the interface between the firm ground and local soil deposit should be chosen from sites with properties consistent with those of the firm ground (see Annex B for further discussion).

This annex describes theoretical bases of site response analyses, dimension of models adopted, data oriented approaches, and application of this knowledge to seismic design.



Key

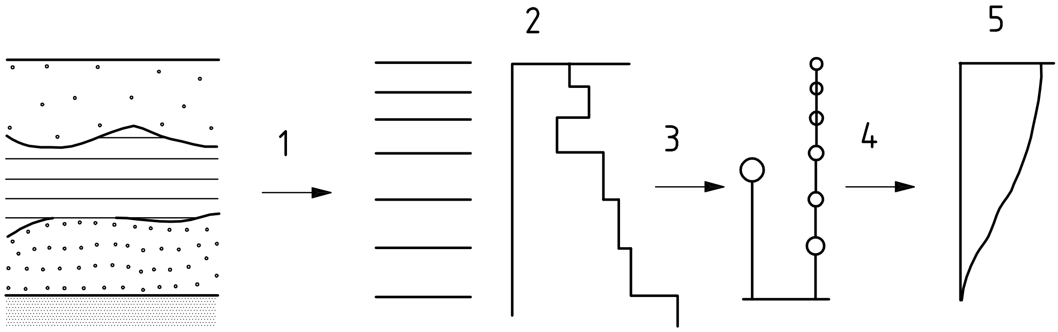
1 firm ground outcrop
2 ground surface
3 local soil deposits
4 incident wave
5 outcrop motion
6 firm ground

Figure E.1 — Schematic figure of site response analysis

E.2 Theoretical background

E.2.1 Single- or multi-degree of freedom system

It is assumed that the movement of the free surface of a soil deposit is mainly due to the vertical propagation of shear waves from the bedrock towards the surface. Generally, a soil deposit consists of multiple layers with different mechanical properties. When both the bedrock surface and the soil layers are essentially horizontal, each layer can be modelled by a lumped mass and a linear or non-linear spring. When the linear spring model is used, modal analysis can be performed as shown Figure E.2 and described in E.2.2. If the rock profile or the soil layers are inclined, finite element (FE) techniques have to be employed.



Key

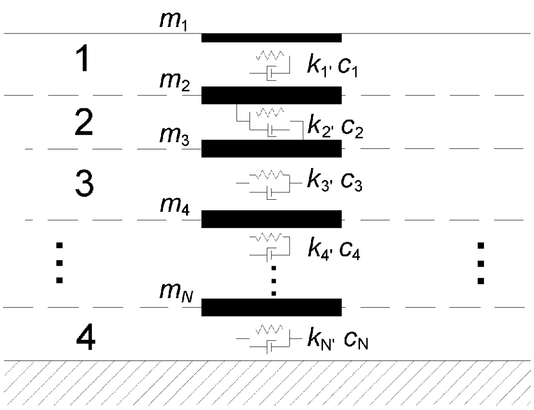
1 assumed horizontally layered ground
2 shear wave velocity
3 modelling single- or multidegree of freedom system
4 analysis
5 displacement

Figure E.2 — Single- or multidegree of freedom analysis of local soil deposit

A simplified mechanical model of the soil deposit in Figure E.2 is shown in Figure E.3. The lumped masses, m_i, are computed, using the following equation assuming that the soil profile has a unit thickness perpendicular to the paper:

$$m_1 = \frac{\rho_1 h_1}{2} \text{ and } m_i = \frac{\left(\rho_{i-1} h_{i-1} + \rho_i h_i\right)}{2}, \quad i = 2,3,...,N \tag{E.1}$$

where m_i is the lumped mass assigned for the ith layer having a density ρ_i with a thickness h_i.



Key

1 layer 1
2 layer 2
3 layer 3
4 layer N

Figure E.3 — Model with lumped masses for the seismic analysis of a soil deposit

The lumped masses are interconnected by springs and viscous damper elements which model the stiffness and damping of the soil deposit during horizontal displacement. The spring stiffness k_i of layer i can be obtained by considering the shear deformation of this soil layer. With a soil column of unit section area, a height of h_i and G as the shear modulus, the spring stiffness, linear or non-linear, is equal to

$$k_i = \frac{G_i}{h_i} \tag{E.2}$$

The equation of motion for all N soil layers in matrix form is given by

$$[m]\{\ddot{u}\} + [c]\{\dot{u}\} + [k]\{u\} = -\mathrm{diag}[m]\ddot{u}_\mathrm{r} \tag{E.3}$$

with $\ddot{u}_\mathrm{r}$: acceleration at the bedrock level, u: relative displacement to the base, and

$$[m] = \begin{bmatrix} m_1 & 0 & \cdots & 0 \\ 0 & m_2 & \cdots & 0 \\ \cdots & \cdots & \cdots & 0 \\ 0 & 0 & 0 & m_N \end{bmatrix}, \quad [c] = \begin{bmatrix} c_1 & -c_1 & 0 & \cdots & 0 \\ -c_1 & c_1 + c_2 & -c_2 & \cdots & 0 \\ 0 & -c_2 & c_2 + c_3 & \cdots & 0 \\ \cdots & \cdots & \cdots & \cdots & -c_{N-1} \\ 0 & 0 & 0 & -c_{N-1} & c_{N-1} + c_N \end{bmatrix},$$

$$[k] = \begin{bmatrix} k_1 & -k_1 & 0 & \cdots & 0 \\ -k_1 & k_1 + k_2 & -k_2 & \cdots & 0 \\ 0 & -k_2 & k_2 + k_3 & \cdots & 0 \\ \cdots & \cdots & \cdots & \cdots & -k_{N-1} \\ 0 & 0 & 0 & -k_{N-1} & k_{N-1} + k_N \end{bmatrix}. \tag{E.4}$$

E.2.2 Modal analysis of single- or multi-degree of freedom systems

When the linear spring model is used, modal analysis can be performed in order to solve the equations of single- or multidegree of freedom systems. By taking the fundamental mode of the solutions of eigenvalue problem corresponding to Equation (E.3), the peak relative displacement $u_\mathrm{max}(z)$ and peak acceleration $a_\mathrm{max}(z)$ may be obtained using the acceleration response spectrum by

$$u_\mathrm{max}(z) = f_1(z) \cdot \frac{1}{\left(2\pi / T_\mathrm{g}\right)^2} \cdot S_\mathrm{a}\left(T_\mathrm{g}\right) \tag{E.5}$$

$$a_\mathrm{max}(z) = f_1(z) \cdot S_\mathrm{a}\left(T_\mathrm{g}\right) \tag{E.6}$$

where

$S_\mathrm{a}(T_\mathrm{g})$ is acceleration response spectrum ordinate at the natural period T_g of the subsoil for a reference earthquake motion at the firm ground outcrop,

$f_1(z)$ is the participation function of the fundamental mode of subsoil at depth z.

As shown in Equations (E.5) and (E.6), the fundamental mode is typically adopted for computing ground displacements and strains. If response accelerations are required, one may use superposition of multiple modes.

The natural period of the local soil deposit often plays a significant role in local site response. This period may be obtained from the eigenvalue solution of Equation (E.3).

Several simplified equations have also been adopted in practice, such as the basic expression

$$T_g = \frac{4h}{v_s} \tag{E.7}$$

where h and v_s are the thickness and the shear wave velocity of the local soil deposit, respectively. This formula is based on the assumption that the soil layers are on a rigid base. An approximation for an elastic base layer is given as follows (Sawada, 2004):

$$T_g = \frac{3\sum_{i=1}^{n-1} s_i t_i^3 + \sqrt{9\left(\sum_{i=1}^{n-1} s_i t_i^3\right)^2 - 8\left(\sum_{i=1}^{n-1} s_i t_i^2\right)\left(\sum_{i=1}^{n-1} s_i t_i^4\right)}}{4\sum_{i=1}^{n-1} s_i t_i^2} \tag{E.8}$$

where h_i is the thickness of the ith layer, v_{s0i} is the shear wave velocity of the ith layer, t_i and s_i are coefficients given by

$$t_i = \sum_{k=1}^{i} \frac{4h_k}{v_{s0k}} \tag{E.9}$$

$$s_i = -\frac{v_{s0i} - v_{s0i+1}}{v_{s0i} + v_{s0i+1}} \tag{E.10}$$

If T_g is an imaginary value, the deepest layer is removed.

E.2.3 Wave propagation theory in frequency domain

Transfer functions may also be used to perform site response analysis of linear elastic systems. A known time history at bedrock (input) is transformed in the frequency domain (usually using FFT algorithms), where it is multiplied by the soil deposit transfer function to produce the Fourier series of the output motion at the surface. The ground surface motion can then be transferred to the time domain by inverse FFT algorithms. The transfer function determines how each frequency component of the bedrock motion is modified by the soil deposit. Some special cases are discussed below.

a) Uniform, undamped soil on rigid rock

The simplest model deals with a uniform layer of isotropic, linear elastic soil overlying rigid bedrock. The harmonic horizontal motion of the latter produces vertically propagating shear waves in the overlying linear elastic soil deposit, leading to the transfer function:

$$F_1(\omega) = \frac{U(0,\omega)}{U(h,\omega)} = \frac{1}{\cos(\omega h / v_s)} \tag{E.11}$$

with ω as the circular frequency of ground shaking, h the thickness of the soil layer and v_s the shear-wave velocity. A and B in Figure E.4 are the amplitudes of waves traveling along the z-axis.

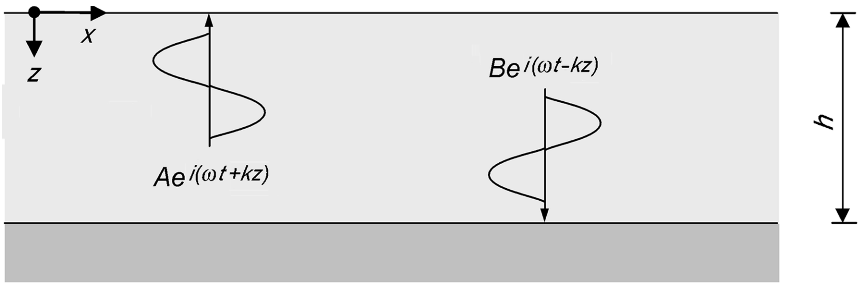


Figure E.4 — Linear elastic soil deposit underlain by rigid bedrock

b) Uniform, damped soil on rigid rock

In order to include energy dissipation or damping effects, a second model introduces soil shearing behaviour according to a Kelvin-Voigt model, leading to the transfer function:

$$F_2(\omega) = \frac{1}{\cos \dfrac{\omega h}{v_s(1+i\xi)}} \tag{E.12}$$

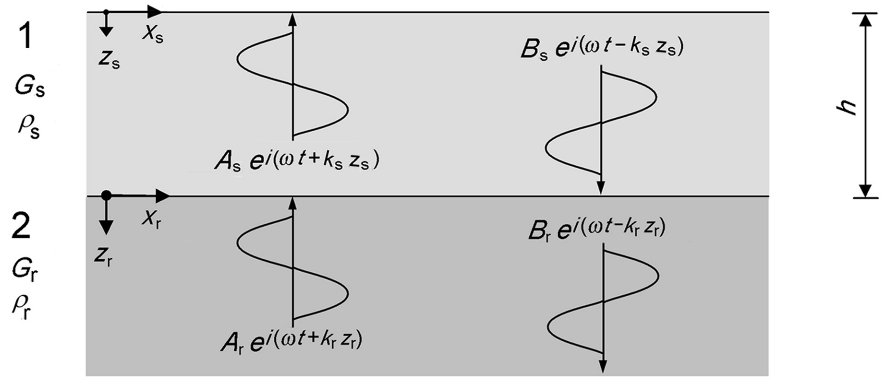
with ξ as damping ratio and the imaginary unit i.

c) Uniform, damped soil on elastic rock

If the bedrock is rigid, its motion will be unaffected by motions in the overlying soil. Any downward-traveling wave in the soil will be reflected back towards the surface by the rigid layer, thereby trapping all of the elastic wave energy within the soil layer. The two previous functions can be extended to consider the case where the underlying layer is an elastic medium. This will introduce a form of radiation damping, and it leads to smaller free surface motion amplitudes than in the case of the rigid bedrock. The transfer function now reads

$$F_3 = \frac{1}{\cos(\omega h / \overset{*}{v}_{ss}) + i\overset{*}{\alpha}_z \sin(\omega h / \overset{*}{v}_{ss})} \tag{E.13}$$

with $\alpha_z^* = \dfrac{\rho_s \overset{*}{v}_{ss}}{\rho_r \overset{*}{v}_{sr}}$ and $\overset{*}{v}_{ss}, \overset{*}{v}_{sr}$ as complex shear wave velocities of soil and rock, respectively.



Key

1 soil

2 rock

Figure E.5 — Homogeneous elastic soil layer on top of a half-space of elastic rock

d) Layered, damped soil on elastic rock

Finally, real ground response problems usually involve soil deposits with layers of different stiffness and damping characteristics and boundaries at which the elastic wave energy will be reflected and/or transmitted. All the layers are usually assumed to be horizontal, with different thickness, so that transfer functions of multi-layered soil deposits have to be used.

E.3 Dimensions of model

E.3.1 One-dimensional (1-D) models

The various models available for site response analysis may be grouped in three categories; one-dimensional (1-D), two-dimensional (2-D), and three-dimensional (3-D) models. This clause describes 1-D model.

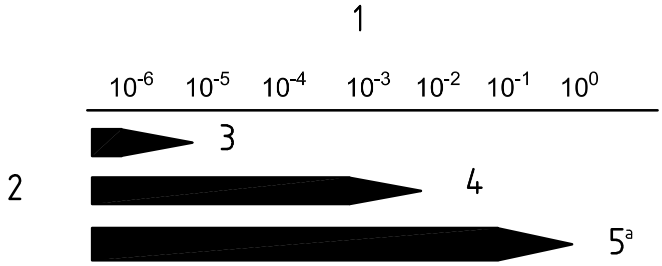
a) Linear approach

It is well known that soil behaviour can be essentially nonlinear, but even in this case linear methods can provide useful results. Generally, equivalent linear approximations of the non-linear response are made by setting the shear modulus G equal to the secant shear modulus at the relevant strain level and the viscous damping ratio ξ as the damping ratio that involves the same energy loss per cycle as the actual hysteretic loop. Since the linear approach requires G and ξ to be constant for each soil layer, the problem is to determine values consistent with the strain level in each layer. Even though an iterative process using strain-compatible soil properties may serve well for approximating nonlinear soil behaviour, it is important to remember that if the strain-compatible soil properties are kept constant throughout the duration of an earthquake, regardless of whether the strains are large or small at a particular time, the method is still a linear method.

b) Nonlinear approach

For solving problems involving some non-linear response of a soil deposit, numerical integration in the time domain is the method of choice. The system parameters of the non-linear constitutive model are updated at small time intervals. Most of the currently available nonlinear 1-D ground response analysis computer programs characterize the stress-strain relationship of the soil by cyclic stress-strain models.

A constitutive model may be selected on the basis of the expected strain level in the soil deposit. The approximate ranges of strains compatible with different models are shown in Figure E.6. The limit in applicability of the equivalent linear model is 1 % strain at the maximum; non-linear hysteresis model is appropriate over the largest strain range.



Key

1 shear strain amplitude

2 method of analysis

3 linear

4 equivalent linear

5 non-linear

[a] Strain range for non-linear analysis depends on the models.

Figure E.6 — Compatible strain ranges for linear, equivalent linear and non-linear models

E.3.2 Two-dimensional (2-D) models

The basic assumption on which the validity of 2-D analysis rests is that the problem being analysed can be adequately characterized by either plane strain or plane stress conditions. Many problems of practical interest, such as those dealing with sloping or irregular ground surfaces (e.g. sedimentary basins and valley floors) and/or the presence of man-made structures, walls and tunnels are solved in practice by 2-D analyses. Solutions are available both in the time and frequency domains, using finite element (FE), finite difference (FD), or boundary element (BE) methods. Both equivalent linear and non-linear approaches are possible.

E.3.3 Three-dimensional (3-D) models

3-D FE approach is required when the assumptions of 2-D analysis are not met. 3-D FEs have more nodal points with more degrees of freedom than corresponding 2-D elements, but the basic process of element mass, damping, and stiffness formulation, and their assembly into the general system of global equations of motion is identical.

E.4 Data oriented approach

E.4.1 Traditional parametric studies

Where possible, observed site response data should be used to check analytical results. Such studies are conducted mainly by the following methods:

a) Function approximation

By gathering enough data and performing a fitting process by standard statistical means, one can determine the effects of different input parameters on the response. This method has, for example, proven useful for the investigation of topographic effects.

b) Semi-theoretical approach

These methods are not purely data based but also have a theoretical background. Starting point is generally an appropriate model for describing the system behaviour, the parameters of which are then identified using the observed data.

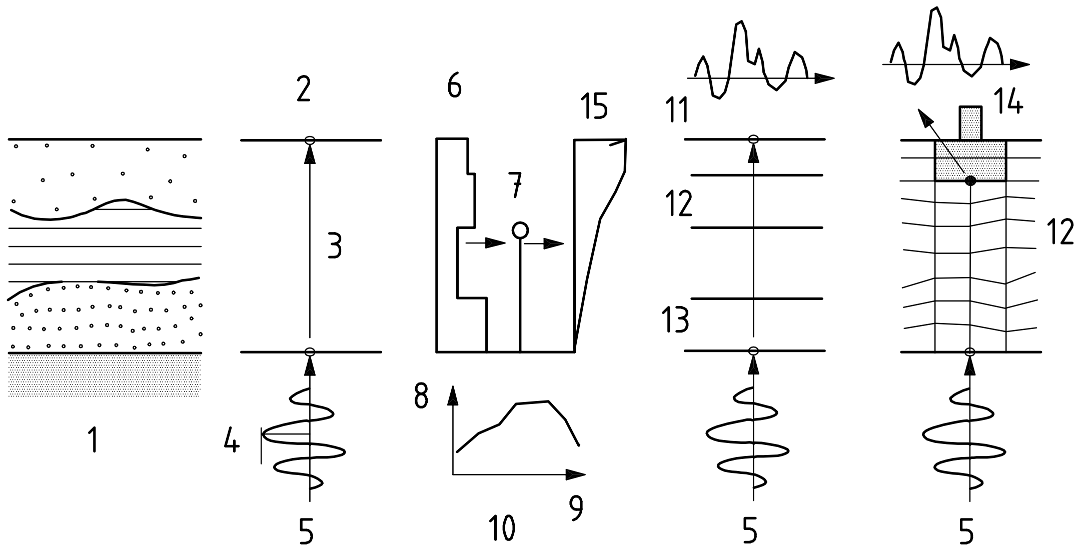
E.4.2 Data mining by soft computing methods

Soft Computing is a general term for methods combining different aspects of such relatively new computational techniques as Fuzzy Logic, Artificial Neural Networks and Evolutionary Algorithms. Two examples suitable in site response applications are Artificial Neural Networks and Adaptive Fuzzy Systems; the former can be trained with an adequate amount of sample information to numerically predict complex system behaviour albeit in a rather "black box" way, while the latter also yields insights into system causality.

E.5 Application to seismic design

E.5.1 General

The fundamental theories reviewed in E.2 through E.4 can be regarded as background for the analytical tools used for seismic design. The various methods for site response analysis are broadly classified into four categories as described in 6.3, shown in Figure E.7. Empirical, site-specific simplified, and site-specific simplified dynamic analyses are based on 1-D model, whereas site-specific detailed dynamic analysis is based on 2-D or 3-D model.



a) Empirical analysis **b) Site-specific simplified analysis** **c) Site-specific simplified dynamic analysis** **d) Site-specific detailed dynamic analysis**

Key

1 base
2 PGA$_{\text{ground surface}}$
3 amplification a of basic variables PGA$_{\text{ground surface}}$ = a·PGA$_{\text{base}}$; index for ground properties: predominant period T_g, V_{S30}, etc.
4 PGA$_{\text{base}}$
5 input ground motion
6 structure of shear wave velocity
7 modelling
8 response
9 period
10 response spectrum of input ground motion
11 response time history
12 dynamic response analysis
13 horizontally layered ground
14 effective input motion to structure
15 modal response of ground

Figure E.7 — Schematic figure of methods for site response analysis

E.5.2 Empirical analysis

The empirical analysis according to 6.3.2 yields peak response values and response spectra at the ground surface by using an amplification factor, which is correlated with parameters such as the natural period T_g of the local soil deposit and/or the average shear wave velocity over 30 m depth (V_{s30}). The correlation is typically derived from statistical analyses for recorded earthquake ground motions, leading to a set of site categories. The effect of nonlinearity of the local soil deposits may be evaluated through the statistical analysis by using parameters such as the level of earthquake ground motions.

In the empirical analysis, the site response within the subsoil may also be obtained by a simplified procedure from the response values at the ground surface, such as those used for the simplified analysis of liquefaction phenomena (see Annex G).

Various methods are available for obtaining the natural period; *in-situ* measurements, simplified equations and dynamic analyses.

Methods based on *in-situ* measurement mostly use microtremor recordings. Fourier amplitude spectra of the horizontal and vertical components of microtremors are used for determining the spectral ratio of $H(\omega)/V(\omega)$. The period for the peak spectral ratio typically identifies the fundamental natural period of the local soil deposit. The applicability of this method has been generally confirmed, although at some particular sites (e.g. with very soft topsoil and firm ground outcrops), the peak spectral ratio may become less distinctive. This method is not recommended for the determination of amplification factors.

The natural period can also be obtained through the theories reviewed in E.2.2.

E.5.3 Site-specific simplified analysis

The site-specific simplified analysis (6.3.3) is also used to obtain the peak response values such as acceleration, velocity, displacement, strain and spectrum ordinates. These values are obtained not only at the ground surface but also within the local soil deposit. Modal analysis is performed as shown in Figure E.7(b) by idealizing the local soil deposit as horizontally layered. Either a single degree of freedom system or a multi-degree of freedom system may be used for computation depending on the complexity of the shear wave velocity structure as shown in Figure E.2 and reviewed in E.2.2. Non-linearity is typically taken into account by using reduced shear moduli compatible with the expected strain level.

E.5.4 Site-specific simplified dynamic analysis

The site-specific simplified dynamic analysis (6.3.4) computes time history response of local soil deposit both at the ground surface and within the local soil deposit. Response parameters such as time histories of acceleration, velocity, displacement, stresses and strains can be computed.

The site-specific simplified dynamic analysis is accomplished either through total stress analysis using equivalent linear or non-linear model or through effective stress analysis. The total stress analysis using equivalent linear models is typically carried out through frequency domain approach. The total stress analysis using non-linear models is typically carried out through time history analysis using lumped mass model. Effective stress analysis is typically performed in order to obtain the free field motion of liquefiable local soil deposit.

Local soil deposit used for the analysis are idealized as horizontally layered ground as shown in Figure E.7(c).

While site-specific simplified analyses only yield peak response values, simplified dynamic analyses also yield corresponding time histories and/or Fourier spectra.

E.5.5 Site-specific detailed dynamic analysis

The site-specific detailed dynamic analysis (6.3.5) computes the entire response of a soil-structure system as shown in Figure E.7(d). 2-D or 3-D FE method may be used for the analysis. Nonlinear characteristics of soil and structure including those at the interfaces can be represented by appropriate constitutive models. Total stress analysis and effective stress analysis are available for the analysis. Effective stress analysis may be performed for liquefiable soil deposit (see Annex G).

The actual configurations of local soil deposits may be considered, including curving interfaces between layers and variation of soil and other parameters. Firm ground outcrop motions are used as input for the rigid base layer, whereas the in-layer motion is used for the elastic base layer. Differences in arrival times of incident waves along the base layer may be specified if the horizontal length of the local soil model is large relative to the wavelength considered. Side boundaries of the model should be modelled with appropriate energy absorption mechanisms to eliminate or reduce spurious reflected waves from the model boundaries.

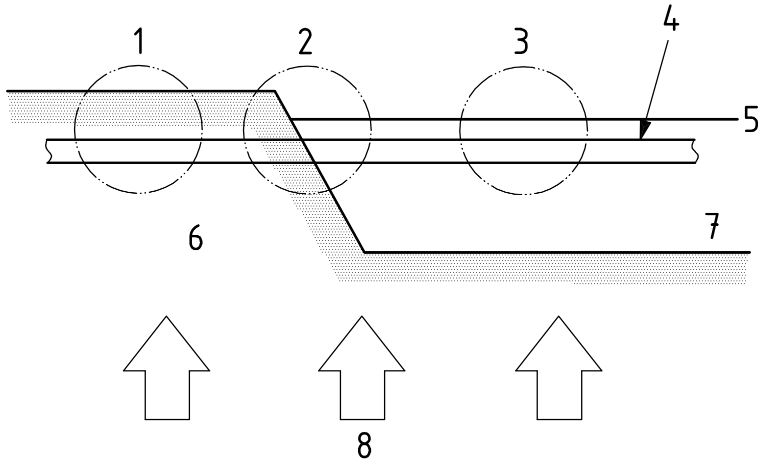
Annex F
(informative)

Spatial variation of earthquake ground motion

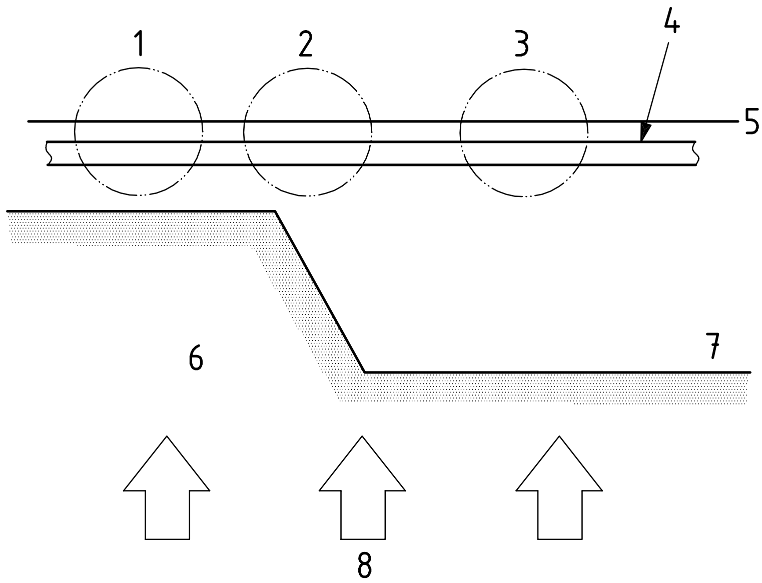
F.1 Spatial variation due to lateral variation in geotechnical conditions

As described in 6.4.1, it is necessary to evaluate the spatial variation of earthquake ground motions for the design of a long or large structure.

Lateral variation in the geotechnical conditions within the dimensions of a long or large structure along the direction of the axis can be a major cause of spatial variations of earthquake ground motions as illustrated in Figure F.1(a). The amplitude of motion can vary significantly across the boundary of the firm ground outcrop and the local soil deposit, thereby inducing significant strains in the ground and in the buried structure that extends across the boundary. Similar phenomena can occur when the geotechnical work is placed above the buried topography as shown in Figure F.1(b). Complicated effects of horizontal heterogeneity of the ground can be evaluated by two or three-dimensional response analysis of the ground as described in 6.4.5.



a) Structure extending across boundary between the firm ground and local soil deposit



b) Structure above buried topography

Key

1 small amplitude
2 concentration of strain
3 large amplitude
4 long buried structure
5 ground surface
6 firm ground
7 local soil deposit
8 incident wave

Figure F.1 — Examples of the spatial variation of earthquake ground motion induced by lateral variation in the geotechnical conditions

F.2 Spatial variation due to horizontal wave propagation effect

Other major causes of the spatial variation in ground motions are the horizontal wave propagation effect and spatial incoherency. The horizontal wave propagation effect is due to the finite speed and angle of propagation of the wave front, which ensures that all points in a long or large structure are not simultaneously exposed to the same level of motion. The slower the wave speed, the more significant is the horizontal wave propagation effect. The incoherency in the motions between two locations in a long or large structure is due to scattering and complex 3-D wave propagation. It is a stochastic phenomenon and is represented by empirical coherency functions derived from array data. The horizontal wave propagation effect often predominates at low frequencies.

The strain in the soil due to the horizontal wave propagation effect $\varepsilon(\omega)$ is a function of the amplitude of the particle velocity $v(\omega)$ and the apparent wave propagation velocity of seismic waves $c(\omega)$:

$$\varepsilon(\omega) = v(\omega)/c(\omega) \tag{F.1}$$

The particle velocity $v(\omega)$ is typically determined following the procedure described in 6.2 and 6.3. As can be seen in Equation (F.1), $\varepsilon(\omega)$ becomes larger for smaller $c(\omega)$.

F.3 Surface waves affected by the geological structure below the interface between the firm ground and the local soil deposit

F.3.1 Phase velocity of surface waves

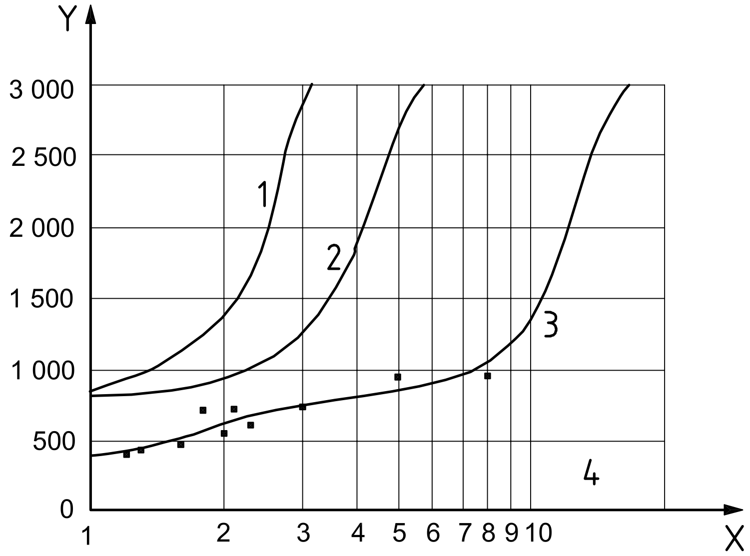
The apparent wave propagation velocity $c(\omega)$ in the previous section becomes the phase velocity in the case of surface waves. As noted in the previous section, the strains in the soil depend on the phase velocity of surface wave $c(\omega)$. The phase velocity of surface waves is, in general, smaller than that of shear waves for any angular frequency ω. Among surface waves, either the fundamental-mode Love wave or the fundamental-mode Rayleigh wave corresponds to the smallest value of $c(\omega)$ for any angular frequency ω.

The phase velocity of surface waves is frequency-dependent. The phase velocity $c(\omega)$ decreases with increasing angular frequency ω when the S-wave velocity of ground increases with increasing depth. Therefore, if one assumes a value of $c(\omega)$ that does not depend on the angular frequency ω, then it will lead to either underestimation of the effects of high-frequency waves or overestimation of the effects of low-frequency waves on the strain in the soil. Therefore, in the Clause 6.4.3, it is recommended that the phase velocity $c(\omega)$ which is site-specific and frequency-dependent be evaluated.

For example, Figure F.2 shows an example of dependence of the phase velocity of Love waves on the frequency in the waterfront area of Tokyo, Japan. The theoretical phase velocity (solid lines) is computed from the S-wave velocity structure model shown in Table F.1. The S-wave velocity structure model shown in Table F.1 includes the geological structure below the firm ground surface down to the upper crustal rock surface. Exclusion of the deeper part of the ground from the model will result in underestimation of the phase velocity at relatively long periods. The solid rectangles in Figure F.2 indicate the phase velocity obtained by array observations at this particular site. The phase velocity of the fundamental-mode Love wave is approximately 400 m/s at the period of one second and approximately 750 m/s at the period of 3 s. Therefore, using a constant value of 400 m/s will lead to an overestimation of the effects of the components around 3 s. Use of a constant value of 750 m/s will lead to an underestimation of the effects of the components around one second.

F.3.2 Passage effect of surface waves

The simplest way to incorporate the horizontal wave propagation effect in the evaluation of the deformation of the soil is to assume a harmonic ground displacement distribution in the horizontal direction as addressed in 6.4.2, with the assumption that seismic waves with one single frequency almost dominate the deformation of the ground and the contribution of seismic waves with other frequencies is negligible. Unfortunately, this assumption is often an oversimplification of the actual situation. In fact, earthquake ground motions specified through the process described in 6.2 and 6.3 include, in general, multiple frequencies, each of which can give rise to the horizontal wave propagation effect. In this case, the incorporation of the frequency-dependent phase velocity in the evaluation of the strain of the soil can be easily achieved as follows.



Key

X period(s)

Y phase velocity, m/s

1 2nd higher mode (third mode)

2 1st higher mode (second mode)

3 fundamental mode (first mode)

4 Love wave

Figure F.2 — An example of dependence of the phase velocity of Love waves on the frequency in the waterfront area of the city of Tokyo, Japan

Table F.1 — S-wave velocity structure model corresponding to the phase velocity in Figure F.2

Layer	Depth to layer from the ground surface (m)	Thickness of layer (m)	S-wave velocity (m/s)	Density (t/m^3)
1	0 to 50	50	250	1,8
2	50 to 170	120	410	1,9
3	170 to 1750	1 580	800	1,9
4	1 750 to 3 000	1 250	1 200	2,1
5	3 000 to 6 100	3 100	2 600	2,6
6	6 100	—	3 400	2,6

Let $a_0(t)$ denote the time history of ground motion evaluated through the process described in 6.2 and 6.3 at one representative point $(x = 0,\ y = 0)$ at the depth of interest on the horizontally layered ground. Let $c(\omega)$ denote the site-specific frequency-dependent phase velocity. Then, the ground motion $a(t)$ at an arbitrary point (x,y) at the same depth can be obtained as follows:

1) Compute Fourier transform of $a_0(t)$.

2) Compute Fourier transform of $a(t)$ as follows:

$$A(\omega) = A_0(\omega)\ \exp[-i(k_x x + k_y y)] \tag{F.2}$$

$$k_x = [\omega/c(\omega)]\cos\theta \tag{F.3}$$

$$k_y = [\omega/c(\omega)]\sin\theta \tag{F.4}$$

where $A_0(\omega)$ and $A(\omega)$ are the Fourier transform of $a_0(t)$ and $a(t)$, respectively, and θ is the angle between the positive-x direction and the direction of the seismic wave propagation.

3) Compute inverse Fourier transform of $A(\omega)$ to obtain $a(t)$.

Ideally, $c(\omega)$ should be determined based on the knowledge of the type of waves involved in the ground motion $a_0(t)$ evaluated at one point through the process in 6.2 and 6.3. In practice, however, the ground motion is often a mixture of many types of waves including surface waves and shear waves and it is not necessarily easy to extract surface wave components from the evaluated ground motion. One may adopt as $c(\omega)$ in the equations (F.3) and (F.4) the smaller of the phase velocities of fundamental-mode Love wave and fundamental-mode Rayleigh wave because either of these two waves corresponds to the smallest value of $c(\omega)$ for any angular frequency ω.

The angle θ can be determined based on the information of direction of seismic wave propagation when such information is available. For example, at some locations, one might expect significant arrival of surface waves from one particular direction. A limitation in this method is the great uncertainty in the direction of seismic wave propagation. Alternatively, in practice the most critical angle may be used for designing a long or large structure.

Main advantages of the procedure described above are

a) frequency-dependent phase velocity can be incorporated, and

b) it is not necessary to assume a harmonic static ground displacement distribution in the horizontal direction at the depth of interest.

Annex G
(informative)

Assessment of liquefaction

G.1 Introduction

Seismic liquefaction refers to a sudden loss in stiffness and strength of soil due to cyclic loading effects of an earthquake. The loss arises from a tendency for soil to contract under cyclic loading, and if such contraction is prevented or curtailed by the presence of pore water that cannot escape, the result is a rise in excess pore water pressure and a subsequent drop in effective stress. If the effective stress drops to zero (100 % rise in pore water pressure), the strength and stiffness can decrease significantly and, in the limit, the soil behaves like a heavy liquid. However, unless the soil is very loose, it will exhibit dilative tendency and, as it strains, it will regain some stiffness and strength. If this strength is sufficiently large, it will prevent a flow slide from occurring, but may still result in excessive displacements commonly referred to as lateral spreading. In addition, even for level ground conditions where there is no possibility of a flow slide, very significant settlements may occur due to dissipation of excess pore water pressures during and after the period of strong ground shaking.

G.2 Assessment of liquefaction

Assessment of liquefaction involves addressing the following concerns:

a) For the design earthquake, will liquefaction be triggered in significant zones of the earth structure, and if so,

b) What ground displacement will be induced by liquefaction?

These effects can be assessed by simplified (G.3) or detailed (G.4) procedures. Depending on the design situations, the assessment of liquefaction may be improved by combining additional information obtained from a geomorphological approach (G.5) or laboratory and *in-situ* tests (G.6).

Screening procedures are often adopted prior to the evaluation procedures for a) and b). For example, zoning by a geomorphological approach may precede the evaluation of liquefaction potential for a). A screening procedure based on the geometric data such as distance from face line of bulkheads, or other geotechnical data, may precede the evaluation of liquefaction-induced ground deformation for b).

G.3 Simplified procedure

G.3.1 Assessment of triggering of liquefaction

The simplified procedure involves the steps described in this and the following subclauses.

Compute a factor that represents a margin against triggering of liquefaction by comparing the cyclic resistance ratio (λ_{crr}) of the soil with the cyclic stress ratio (λ_{csr}) caused by the design earthquake, i.e.,

$$f_{\mathrm{trig}} = \frac{\lambda_{\mathrm{crr}}}{\lambda_{\mathrm{csr}}} \quad \text{(G.1)}$$

The cyclic resistance ratio λ_{crr} is generally obtained indirectly from penetration tests, or other index properties including shear wave velocity, and experience with similar soils during past earthquakes. Youd *et al.* [16] describes North American practice, Eurocode 8 [2] gives European practice, PIANC [12] describes procedures for ports, and Japanese Geotechnical Society [7] describes Japanese practice, among others. Penetration tests may involve the Standard Penetration Test (SPT), the Cone Penetration Test (CPT), and the Large Diameter Penetration Test (LDPT) such as Large Penetration Test (LPT) in Japan and Becker Penetration Test (BPT) in USA. All these methods are based on data from past earthquakes which relate liquefaction potential to a measure of ground shaking and an index of ground conditions. The methods differ in how they specify intensity of shaking and ground conditions.

When λ_{crr} is obtained from standard penetration test values (SPT-*N* values) corrected for energy and overburden stress, additional corrections are applied for confining stress, static bias, multi-directional shear, and a specified number of shear stress cycles through an aggregate correction factor k as

$$\lambda_{\text{crr}} = \lambda_{\text{crr0}} \cdot k \tag{G.2}$$

where λ_{crr0} is the cyclic resistance ratio for the reference state, typically corresponding to a confining stress of 100 kPa, at level ground conditions, and the specified number of load cycles (Youd *et al.* [16]).

The correction for equivalent number of shear stress cycles or one correlated with an earthquake magnitude is included in the correction factor for cyclic resistance ratio in Equation (G.2). Alternatively, this correction can be incorporated as a correction factor for cyclic stress ratio to be defined in Equation (G.3).

When the equivalent number of shear stress cycles is correlated with an earthquake magnitude as in North American design practice, the earthquake magnitude may be specified based on the de-aggregation. In the current Japanese design practice, an earthquake magnitude is often not explicitly considered as a parameter that is specified for design. Conservative estimate is adopted on the equivalent number of shear stress cycles based on the case histories during earthquakes of a relatively large magnitude (e.g. larger than 7.5). The equivalent number of cycles is significantly affected by the source mechanism, path and site effects. These effects may be evaluated through the seismic hazard analysis based on the semi-empirical, theoretical, or hybrid methods (see Annex D).

The cyclic stress ratio λ_{csr} is usually obtained from a site response analysis described in Clause 6.3. When the site response is obtained through either site-specific simplified analysis (6.3.3) or site-specific simplified dynamic analysis (6.3.4), the design earthquake motion is generally applied at the firm ground, and the maximum dynamic shear stress, τ_{max}, obtained at points within the soil mass from an iteration process so as to achieve strain compatible moduli and damping. The cyclic stress ratio λ_{csr} is evaluated as follows:

$$\lambda_{\text{csr}} = k_{\text{eqv}} \cdot \frac{\tau_{\text{max}}}{\sigma'_{\text{o}}} \tag{G.3}$$

where

k_{eqv} is a correction factor for equivalent number of seismic loading cycles;

σ'_{o} is the effective vertical stress at the depth.

Maximum shear moduli are generally based on site-specific shear wave velocity and/or penetration testing. Appropriate strain dependent shear moduli and damping values are generally based on cyclic test data on similar soils.

When the site response is obtained from empirical analysis (6.3.2), τ_{max} in Equation (G.3) at any depth for level ground conditions can be estimated from the peak horizontal ground surface acceleration a_{surf}, as

$$\tau_{\text{max}} = \sigma_0 \cdot \frac{a_{\text{surf}}}{g} \cdot k_{\text{rd}} \tag{G.4}$$

where

σ_{o} is the total vertical stress at the depth;

g is the acceleration of gravity;

k_{rd} is a reduction factor to account for flexibility of the soil.

A margin against triggering of liquefaction is typically specified by a factor $f_{trig} = \lambda_{crr} / \lambda_{csr}$. $f_{trig} \geqslant 1{,}0$ typically implies no liquefaction. The required minimum factor f_{trig} reflects the importance of the structure and the knowledge of the earthquake loading and soil parameters. Sites with a factor f_{trig} between 1,0 and 1,3 can still involve increase in pore-water pressure although the pore water pressure increase may not reach 100 % of initial confining pressure. The effects of these increases in pore water pressure should be accounted for during evaluations of stability and settlement.

Additional information on geomorphological and case history data of liquefaction occurrence can be used to improve the results of assessment of liquefaction potential based on the simplified procedure described above (see G.5).

G.3.2 Liquefaction-induced ground deformation

G.3.2.1 Evaluation of liquefaction-induced displacements

Liquefaction-induced displacements, both lateral and vertical, may be estimated as follows:

a) Lateral displacement

The lateral displacement may be estimated by various simplified analyses, including

— an empirical method based on case histories,

— a sliding block analysis with reduced strain compatible strength in zones predicted to liquefy,

— an equivalent static analysis with reduced stiffness in zones predicted to liquefy, and

— a simplified dynamic analysis using either reduced stiffness or viscosity in zones predicted to liquefy.

NOTE Large lateral displacements induced on nearly level ground can be estimated by Newtonian or Bingham fluid models.

b) Vertical displacement

The vertical displacement arises from consolidation (densification) following liquefaction and is estimated from field experience. The vertical strain resulting from reconsolidation in zones triggered to liquefy can be estimated from charts based on past case histories and laboratory test data. This strain in turn can be integrated to give the settlement profile.

Vertical displacement may also arise from deformation of soils associated with lateral movement, for example, behind a retaining wall, below an embankment, or a slope. Vertical displacement at a particular location can occur due to structure distortion.

G.3.2.2 Evaluation of flow slide based on residual strength

If residual strength of a liquefied soil (i.e. shear strength of the soil after failure due to liquefaction) can be evaluated for the liquefiable zones within the earth structure, the possibility of a flow slide can be estimated from a static limit equilibrium analysis as follows:

a) Identify the zones of liquefaction.

b) Assign residual strength to those zones predicted to liquefy, and conventional strengths to those not predicted to liquefy. In North American practice, some reduction in strength may also be considered for zones having factors of safety less than 1,5.

c) Apply the residual strength to the zones of liquefaction, and modifications to other non-liquefiable zones as noted above and perform a static stability analysis.

In North American practice, if the factor of safety for static slope stability is less than about 1,1, flow failure is considered probable. If the factor of safety is greater than about 1,1, then deformations can be estimated as described in G.3.2.1.

This residual strength approach is often adopted in North America for evaluating the seismic safety of dams with liquefaction potential and in a screening procedure before the evaluation of displacement.

Residual strength is usually determined from correlations between penetration test data such as the SPT-N values and residual strength based on case histories during past earthquakes. Residual strength can be measured by laboratory tests on relevant samples. However, the residual strength as determined from case histories can be the end result of a much more complicated process than is simulated in undrained testing. The presence of barrier layers with low permeability in the field may prevent drainage and cause a water injection effect leading to the formation of a water film at the base of such layers. The actions of these layers would be expected to give much lower strength than laboratory tests.

The correction factor for a large static bias applied in Equation (G.2) might exceed unity when this factor is specified based on the rise of excess pore water pressure. For loose sand where there is a possibility of flow slide, the correction factor for static bias should be kept less than or equal to unity based on engineering judgement. Cautious engineering judgement may be required in order to avoid a problem in the simplified procedure described here when an excess pore water pressure ratio of 100 % does not develop due to the large static bias. Even when the excess pore water pressure ratio of 100 % does not develop, cyclic shear stress pushes the stress path close to the failure line in stress space, inducing a cumulative shear strain beyond a limit, from which the stress path will be directed toward the flow failure state.

G.4 Detailed procedure

G.4.1 General

In detailed procedures, the liquefaction process is simulated by taking into account the increases in pore water pressures, the triggering of liquefaction, the subsequent losses in strength and stiffness, and dilation at large displacements that may be occurring during the applied time history of base motion. Such changes will affect the dynamic characteristics of the structure and lead to a more realistic prediction of response. This can be carried out in total or effective stress models (6.3.5).

G.4.2 Total stress analysis

In the total stress analysis, the cyclic shear stresses are tracked in each element as they occur. Each stress pulse is weighted according to its size and when or if sufficient cycles occur, the element liquefies and is given post-liquefaction stiffness and strength. In this way the weaker or more heavily loaded elements liquefy first, leading to softening of the structure and increasing lateral movement as more and more elements liquefy. If sufficient elements liquefy and their residual strength is not adequate for static stability, a flow slide is predicted. This procedure uses the same triggering and residual strength as the simplified procedures. However, it does so in a more realistic manner by combining the two analyses described in G.2 and G.3 into a single synthesized approach that allows both the magnitude and pattern of seismic displacements to be predicted.

G.4.3 Effective stress analysis

In effective stress dynamic analysis, pore water pressures are generated in response to the applied earthquake ground motion, and the stiffness and strength of the soil are modified accordingly. Such an approach allows coupled dynamic stress-flow analyses to be carried out in which both generation and dissipation of pore water pressures and their effects are considered for a specific base motion. The calibration and verification of such models is important and generally involve a 2-step process:

a) simulate and capture the element behaviour as observed in laboratory cyclic tests such as simple shear, triaxial, and hollow cylinder;

b) simulate and compare predicted and observed dynamic response for an earth structure.

Ideally, an actual earth structure should be selected. However, even for the best field case histories, such as the San Fernando dams during the 1971 San Fernando earthquake, neither the input motion nor the soil conditions are adequately known. For this reason, verification is currently based on centrifuge tests. Fully coupled effective stress approaches have been developed but further investigations are ongoing. Provided that appropriate calibrations of the model are performed, effective stress analyses can predict the magnitude and pattern of seismic displacements and excess pore water pressures.

G.5 Geomorphological approach

Certain geomorphological categories are associated with the occurrence of liquefaction in past earthquakes. Typical categories are valley plain, alluvial fan, natural levee, back marsh, abandoned river channel, former pond, marsh and swamp, dry river bed, delta, bar, sand dune, beach, interlevee lowland, reclaimed land by drainage, reclaimed land, a site with water spring, and dredged fill. Geomophological classification can be used to zone for liquefaction. Susceptibility gives a preliminary indication of areas where more quantitative measures for assessment of liquefaction potential may be required (ISSMGE/TC4, 1999).

G.6 Laboratory and *in-situ* tests

In projects where additional data are needed for liquefaction assessment, a direct measure of cyclic resistance can be obtained from cyclic loading tests of undisturbed samples in the laboratory. For sandy soils, this may involve freezing techniques to preserve sample volume and fabric. *In-situ* liquefaction tests can also be performed by using either a shaking device or blasting technique.

The liquefaction process can also be investigated by scaled model tests in the laboratory. Such tests are carried out on a shaking table in either $1g$ or centrifugal field.

The limitations in these tests should be taken into account in interpreting the results from the laboratory and *in-situ* tests.

G.7 Liquefaction resistance of coarse and fine soils

Most liquefaction failures have occurred in sand and silty sand materials, and most research on liquefaction has been conducted on these materials. However, coarse material such as gravel and cobble size particles can also liquefy, as can silts and clays. Liquefaction of all soils including cobble to clay size is caused by their tendency to compact under cyclic loading. If such compaction is curtailed by the presence of pore water that cannot escape during the period of shaking, liquefaction can result. Coarse gravels have a similar tendency to compact as sand, and will liquefy if pore water cannot escape. Such deposits can be assessed using the techniques described above for sandy material. However, because of much higher permeability, significant amounts of water may escape and thus curtail or prevent liquefaction. The presence of a low permeability layer overlying gravel may prevent drainage and lead to liquefaction of gravel and cobble material.

Plastic silts and clays have much lower permeability than sands and will respond in an undrained manner during shaking. However, they are less likely to compact than sand and generate 100 % rise of pore pressure. Pore pressure rise is commonly limited to about 60-80 %, depending on the stress history and sensitivity of the soil, and thus these soils are not considered susceptible to liquefaction. As a simple screening tool for detecting liquefaction of plastic silts and clays, a criterion commonly referred to as the Chinese criterion (Chinese Academy of Building Research [3]; Youd *et al.* [16]) has been developed based on their plastic properties and on field experience during past earthquakes. For important works, direct testing of undisturbed samples should be considered.

G.8 Deformations due to reduced stiffness and strength of silts and clays

In addition to failure associated with liquefaction or conventional type of shear failure due to earthquake-induced forces, soft deposits of silts and clays may develop significant shear strains and deformations during earthquake loading if the level and duration of shaking are significant. These deformations are associated with pore pressure rise in silts and clays due to cyclic loading. Presence of initial shear stresses due to gravity significantly affects the deformations due to cyclic loading. These deformations can govern the seismic performance of geotechnical works constructed on soft deposits of silts and clays. A methodology was developed for evaluating these deformations (Andersen *et al.* [1]; Nadim and Kalsnes [9]).

G.9 Reduced stiffness and strength of soils at post liquefaction state

Upon liquefaction, the shear stiffness may drop to essentially zero, but on straining the soil may regain significant stiffness and strength due to dilation. The reduced post liquefaction strength has been obtained from monotonic undrained tests (steady state strength), but as described earlier field experience indicate that the strength available may be much lower than the undrained strength. There are a number of possible causes for this including the following:

a) deformation due to the cyclic nature of seismic loading under static shear;

b) void redistribution and water film effects caused by a restriction to migrating pore water due to permeability contrasts arising from the layered nature of soil (stratigraphy);

c) mixing of soil layers producing a layer with soil weaker than its original components.

As mentioned earlier, post liquefaction strength derived from back analysis of field case histories should account for the various causes of strength reduction and are recommended for use (Seed and Harder [14]; Olson and Stark [11]). Post liquefaction stiffness, defined as secant modulus of the stress-strain relationship mobilized at the strain level used for design, is also evaluated based on the back analysis of field case histories.

Annex H
(informative)

Seismic actions defined for various models of geotechnical works

H.1 General

In Clauses 7 through 9, seismic actions are described based on the types and models of analysis rather than the type of geotechnical works. In design, however, the type of geotechnical works is first identified, and then the seismic actions are defined for a specific model for analysis.

Types of analysis are classified based on the modelling of soil-structure interaction and are classified as simplified if soil-structure interaction is modelled as an action on a structure model defined in a global soil-structure system, whereas classified as detailed if soil-structure interaction of a global system is included in a global computational model (7.1.2).

In the simplified equivalent static analysis, inertia forces are often specified by horizontal and/or vertical seismic coefficients. For some applications, the effect of vertical component may not be significant. In such situations, it may be neglected. The overall applicability of the method of analysis should be validated based on case history data with appropriate calibration of model parameters.

When more than two geotechnical works are constructed adjacent to each other, effects between these geotechnical works should be considered in the analysis and design.

Evaluation of a failure mode is an important part of the performance evaluation. For example, bending failure of piles is typically preferred to shear failure, and failure at the pile cap is preferred to the failure at the deeper portion of piles in the ground. Another example is sliding of a wall; sliding of a wall is typically preferred to overturning. In the simplified analysis, a failure mode is assumed for analysis whereas, in the detailed analysis, the failure mode is evaluated through the analysis and thus considered as an important element of performance criteria.

According to the performance objectives for the project, reference earthquake motions are specified, the relevant performance criteria are established in terms of engineering parameters, and seismic actions are specified for various models of geotechnical works (Clause 5). Seismic performance is then evaluated with respect to the specified performance criteria parameters. For various models of geotechnical works, this annex describes the parameters of reference earthquake motions and structural response, seismic actions specified based on these parameters, and the performance criteria parameters.

H.2 Spread foundations

H.2.1 Simplified equivalent static analysis

A typical model used for simplified equivalent static analysis of spread foundations is a rigid structure model defined by a sliding plane along the base or by a potential slip surface in the subsoil. For evaluation of densification of subsoil due to cyclic shear, an alternative model representing the ground below the spread foundation may be used.

Depending on the models of analysis, the parameters of reference earthquake motions and response of a superstructure, the actions on model for analysis, and the performance criteria parameters may be specified as follows:

1) for a rigid or deformable structure model defined by sliding plane along the base (see Figure H.1) or by a potential slip surface in the subsoil below the spread foundation:

   a) parameters of reference earthquake motions and structural response: peak acceleration at the ground surface or the bottom of the foundation, or of superstructure response, and total or differential settlements of subsoil below the spread foundation;

   b) actions: inertia forces and overturning moment from a superstructure, inertia force of a foundation, earth pressure and pressure due to permanent soil movement, total or differential settlements of subsoil below the spread foundation;

   c) performance criteria parameters:

   — margin with respect to the threshold limit for sliding; displacement due to sliding;

   — margin with respect to the elastic limit of the spread foundation;

   — acceptable residual displacement due to sliding where applicable;

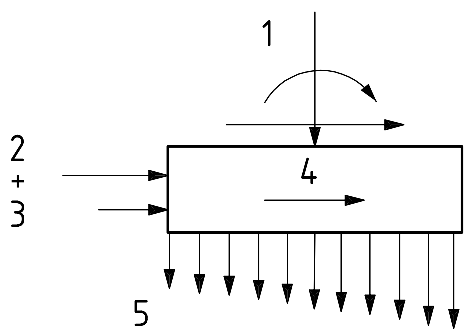
   — acceptable response beyond structural elastic limit specified in terms of strains or ductility factor of a footing;

2) for evaluating densification of subsoil below the spread foundation due to cyclic shear:

   a) parameters of reference earthquake motions and structural response: time history of shear stress in the subsoil below the spread foundation due to response of a superstructure and ground;

   b) actions: amplitude and number of cycles of equivalent uniform cyclic stress;

   c) performance criteria parameters: acceptable total and differential settlements of foundation, and tilting.



Key

1 actions from superstructure

2 seismic earth pressure

3 pressure due to permanent soil movement

4 spread foundation

5 uniform or differential settlement

Figure H.1 — Seismic actions on spread foundations for simplified equivalent static analysis

Depending on the embedment depth of the foundation, actions from the superstructure for simplified analysis may also include the effects of ground displacement. Take an example of a ten storey building with four underground levels for parking built on either spread or pile foundations. The effects of the superstructure in these cases should include those from the underground levels. Actions from the superstructure include the effects of ground displacement on the underground levels of the structure built on top of foundation. Alternatively the model of the geotechnical work can be defined for the part of the structure constructed below the ground surface, including the foundation. This approach simplifies the actions from the superstructure but the model of the geotechnical work becomes more complicated. In those complicated design situations, detailed analysis may be required.

H.2.2 Simplified dynamic analysis

Simplified dynamic analysis of spread foundations is typically based on elastic spring model using the following parameters and actions:

a) parameters of reference earthquake motions and structural response: acceleration time history at the ground surface or of superstructure response;

b) actions: time history of inertia force on a superstructure;

c) performance criteria parameters: peak response acceleration, velocity, or displacement of a superstructure based on assumed response mode.

H.2.3 Detailed dynamic analysis

Detailed dynamic analysis is typically employed for the analysis of coupled soil-structure interaction using appropriate numerical methods. The reference earthquake motions are used directly as action input to the soil-structure model and are acceleration or velocity time history of ground motion at the bottom boundary of the analysis domain and distribution of displacement or velocity time histories at the side boundaries.

Response mode, peak stresses and strains can be computed if an equivalent linear model is used whereas failure modes and residual displacements can be computed if a non-linear model is used.

Performance criteria can be specified in terms of

— appropriate response/failure mode of soil-structure system;

— acceptable peak or residual response specified in terms of the stress resultants, stress, strain, and ductility factor, and

— acceptable displacement of superstructure.

H.3 Pile foundations

H.3.1 Simplified equivalent static analysis

For simplified equivalent static analysis of pile foundations, a beam/frame model is typically used with the seismic actions and gravity as shown in Figure H.2.

This whole model can be idealized by a single spring called "spring foundation model".

Depending on the models of analysis, relevant parameters and actions may be specified as follows:

a) for a spring foundation model:

   1) parameters of reference earthquake motions and structural response: peak acceleration at the ground surface or the top of the piles, or of superstructure response;

   2) actions: inertia forces from a superstructure on pile cap;

   3) performance criteria parameters:

— acceptable displacement of pile cap;

— margins to the elastic limits specified in terms of shear force and overturning moment at the head of pile;

— acceptable response beyond the structural elastic limit of piles;

a) for a beam/frame model (see Figure H.2):

1) parameters of reference earthquake motions and structural response: peak response acceleration of a superstructure (for pile-supported wharves and piers, the superstructure is defined for the portion of a structure built on the top of the piles), peak displacement distribution of free field response;

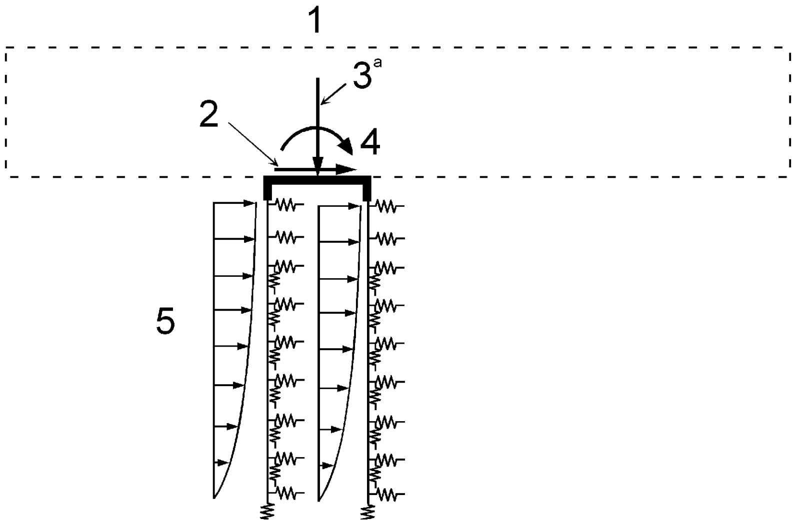
2) actions: inertia forces and overturning moment from a superstructure on pile cap, deformation of free field imposed on piles;

3) performance criteria parameters:

— margin with respect to the threshold limits of bearing capacity, pull-out resistance, and lateral resistance;

— margins to the elastic limits of piles specified in terms of resultants or stress;

— acceptable residual response beyond the elastic limit of piles.



Key

1 actions from superstructure

2 horizontal component of inertia force

3 gravity + vertical component of inertia force, a

4 overturning moment

5 ground displacement relative to the toe of the pile

[a] See H.1.

Figure H.2 — Seismic actions on pile foundations for simplified equivalent static analysis

The remarks on the embedment depth of foundation described in H.2.1 also apply this section. For phase difference between the inertial and kinematic interactions, see Annex I.

H.3.2 Detailed equivalent static analysis

In detailed equivalent static analysis of pile foundations, a FE or lumped mass model is used based on the seismic coefficient approach (see H.4.2). Relevant parameters and actions are as follows:

a) parameters of reference earthquake motions and structural response: peak acceleration of superstructure response; peak acceleration distribution of free field response;

b) actions: inertia force of a superstructure on pile cap; distributed inertia force on soil mass of soil-structure system;

c) performance criteria parameters:

 — acceptable residual response beyond the structural elastic limit specified in terms of strain or ductility factor, and displacement of piles;

 — failure modes.

H.3.3 Simplified dynamic analysis

Depending on the models of analysis, relevant parameters and actions may be specified as follows:

a) for a superstructure-spring foundation model:

 1) parameters of reference earthquake motions: acceleration time history at the ground surface;

 2) actions: time history of inertia force of a superstructure on pile cap;

 3) performance criteria parameters:

  — acceptable residual response specified in terms of displacement of pile cap;

  — acceptable residual response specified in terms of shear force and moment at the head of piles;

b) for a beam/frame model:

 1) parameters of reference earthquake motions: acceleration or velocity time history of ground motion at the bottom boundary of the analysis domain and distribution of displacement or velocity time history at the side boundaries;

 2) actions: the same as 1);

 3) performance criteria parameters:

  — acceptable residual response specified in terms of displacements associated with failures with respect to bearing capacity, pull-out resistance, and lateral resistance;

  — acceptable residual response beyond the structural elastic limit specified in terms of strain and displacement of the piles based on an assumed failure mode.

H.3.4 Detailed dynamic analysis

The model for detailed dynamic analysis is typically based on a FE or lumped mass model. Relevant parameters and actions are the same as described in H.2.3.

Response mode, peak stresses and strains can be computed if equivalent linear model is used whereas failure modes and residual displacements can be computed if non-linear model is used.

Performance criteria can be specified in terms of

— appropriate response/failure mode of soil-structure system,

— acceptable peak or residual response specified in terms of the stress resultants, stress, strain, ductility factor of piles,

— acceptable displacement of superstructure, and

— acceptable axial force of pile (peak values are evaluated through the equivalent linear model whereas peak and residual values are evaluated through the non-linear model).

H.4 Buried structures (transverse section)

H.4.1 Simplified equivalent static analysis

Buried structures such as culverts and buried tunnels in transverse section may be modelled by a frame-spring model. In more simplified modelling, the both sides of the structure may be modelled as walls resisting the surrounding ground.

Depending on the models of analysis, relevant parameters and actions may be specified as follows:

a) for a wall model:

   1) parameters of reference earthquake motions: peak acceleration at the ground surface;

   2) actions: seismic earth pressures;

   3) performance criteria parameters:

      — margins to structural elastic limits specified in terms of stress resultants or stress;

b) for a frame-spring model (see Figure H.3):

   1) parameters of reference earthquake motions: peak displacement distribution of the free field response or the displacement distribution at the instant when the maximum relative displacement or deformation occurs between the top and bottom of the buried portion of the structure;

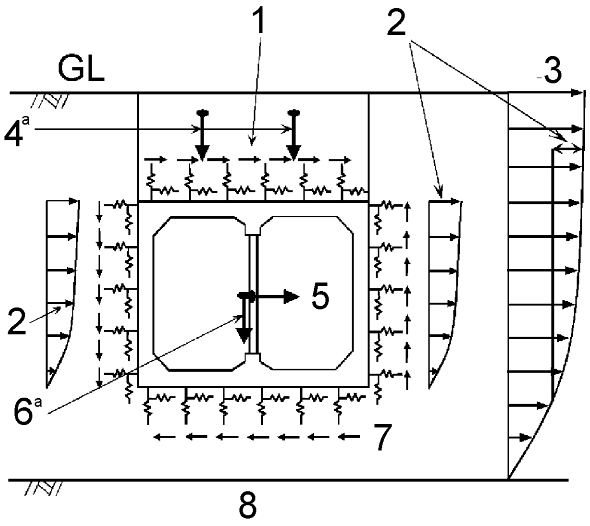
   2) actions: deformation of free field on walls, shear forces at soil-structure interface, and inertia forces on structures;

   3) performance criteria parameters:

      — margins to structural elastic limits specified in terms of stress resultants or stress of buried structure;

      — acceptable residual response beyond elastic limit specified in terms of strain or ductility factor of buried structure.

The overall applicability of the method of analysis should be validated based on case history data with appropriate calibration of model parameters.



Key

1 interface shear stresses + ground displacement relative to the bottom of the structure
2 ground displacement relative to the bottom of the structure
3 ground displacement relative to the firm ground
4 gravity + vertical inertia force of soil on buried structure
5 inertia force of structure
6 net gravity (> 0) + vertical inertia force of structure
7 interface shear stress
8 firm ground

a See H.1.

Figure H.3 — Seismic actions on buried structures (transverse section) for simplified equivalent static analysis

H.4.2 Detailed equivalent static analysis

In detailed equivalent static analysis of buried structure in horizontally layered ground, a FE model is used based on the seismic coefficient approach as shown in Figure H.4. As shown in this figure, acceleration of free field is at first computed through site response analysis such as site-specific simplified dynamic analysis, and then the distributed inertia force in ground is applied for each node of FE analysis domain of soil-structure system.

Relevant parameters and actions are as follows:

a) parameters of reference earthquake motions: the acceleration distribution at the instant when the maximum relative displacement or deformation occurs between the top and bottom of the buried portion of the structure;

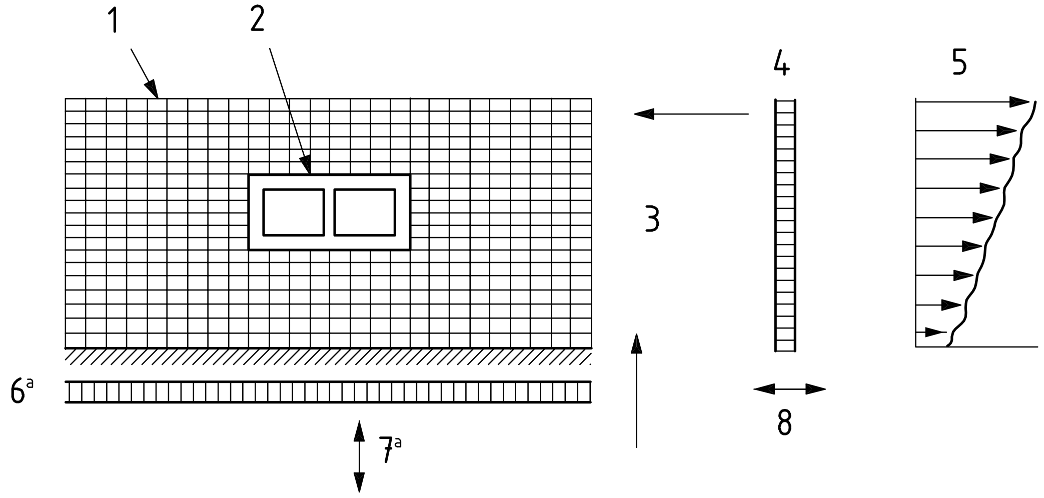
b) actions: distributed inertia force on soil mass of soil-structure system;

c) performance criteria parameters:

— appropriate response/failure mode;

— acceptable peak response specified in terms of the stress resultants or stresses;

— acceptable residual response specified in terms of strain or ductility factor.



Key

1 domain of finite element analysis

2 buried structure

3 distributed inertia force in ground applied for each node of finite element analysis domain

4 horizontal acceleration of free field

5 acceleration distribution

6 vertical acceleration of base

7 vertical earthquake motions

8 horizontal earthquake motions

a See H.1.

Figure H.4 — Seismic actions for detailed equivalent static analysis

As mentioned in 8.2.2, alternative procedures may also be used in which the nodal forces over the FE domain of analysis are specified based on the strain distribution of free field response (Tateishi, [15]).

H.4.3 Detailed dynamic analysis

The model for detailed dynamic analysis is typically based on a FE model. Relevant parameters and actions are the same as described in H.2.3.

Response mode, peak values of the stress resultants, stresses and strains in a structure, and peak deformation of structure can be computed if an equivalent linear model is used. Failure mode, peak and residual values of the stress resultants, stresses and strains in the structure, peak and residual deformation of the structure can be computed if a non-linear model is used.

Performance criteria can be specified in terms of

— appropriate response/failure mode of soil-structure system,

— acceptable peak response specified in terms of the stress resultants or stress, and

— acceptable residual response specified in terms of strain or ductility factor.

H.5 Tube-like buried structures (along axis)

H.5.1 Simplified equivalent static analysis

Seismic actions along axis of tube-like buried structures typically used in simplified equivalent static analysis are shown in Figure H.5.

A beam model is typically used with the following:

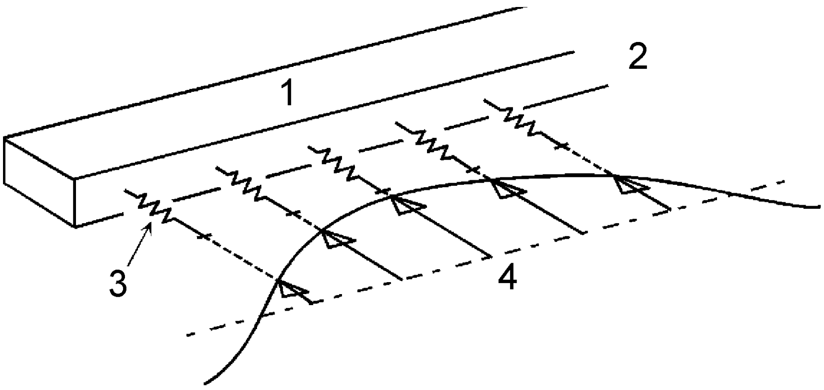
a) parameters of reference earthquake motions: displacements in transverse and longitudinal directions with spatial variation at the elevation of buried structures (see 6.4.2);

b) actions: deformation of subsoil with spatial variation on buried structure;

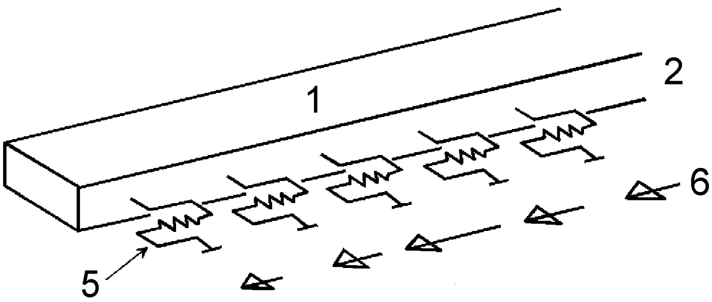
c) performance criteria parameters:

— margin with respect to the structural elastic limits specified in terms of stress resultants or stresses;

— acceptable residual response beyond elastic limit.



a) Transverse actions



b) Longitudinal actions

Key

1 tunnel- or tube-like underground structure
2 simplified equivalent static analysis
3 subgrade reaction spring
4 transverse seismic ground displacement
5 interface shear spring
6 longitudinal seismic ground displacement

Figure H.5 — Seismic actions with spatial variation on tunnel- or tube-like structures (along axis) for simplified equivalent static analysis

H.5.2 Simplified dynamic analysis

Relevant parameters and actions may be specified as follows:

a) parameters of reference earthquake motions: displacement time history of ground in transverse and longitudinal directions with spatial variation at the elevation of buried structures;

b) actions: the same as a);

c) performance criteria parameters

   — the stress resultants, stress and strain beyond the elastic limit of buried structure based on the assumed failure mode.

H.5.3 Detailed dynamic analysis

Detailed dynamic analysis is typically based on a FE or lumped mass model. An example of a FE model is shown in Figure H.6. Actions input to the FE or lumped mass model are ground motion with spatial variation at the bottom boundary of the analysis domain and free field motion at the side boundaries.

Response mode, peak displacement, the stress resultants, stress and strain of the buried structure can be computed if an equivalent linear model is used. Failure mode, residual displacement, peak and residual values of the stress resultants, stress and strain of buried structure can be computed if a non-linear model is used.

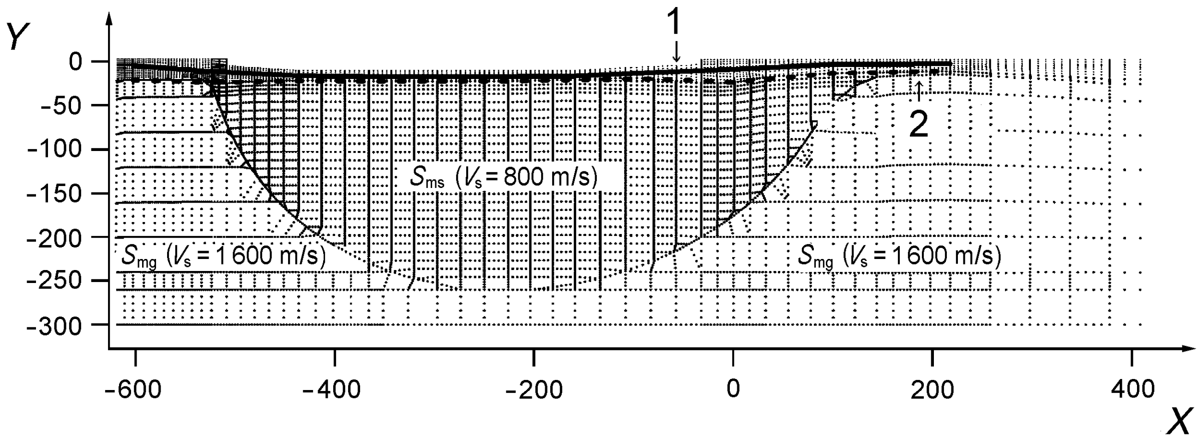
Performance criteria can be specified in terms of

— appropriate response/failure mode of soil-structure system,

— acceptable peak structural response specified in terms of the stress resultants or stresses, and

— acceptable residual structural response specified in terms of strains or ductility factor.

Dimensions in metres



Key

X in metres

Y in metres

1 buried tunnel

2 firm ground

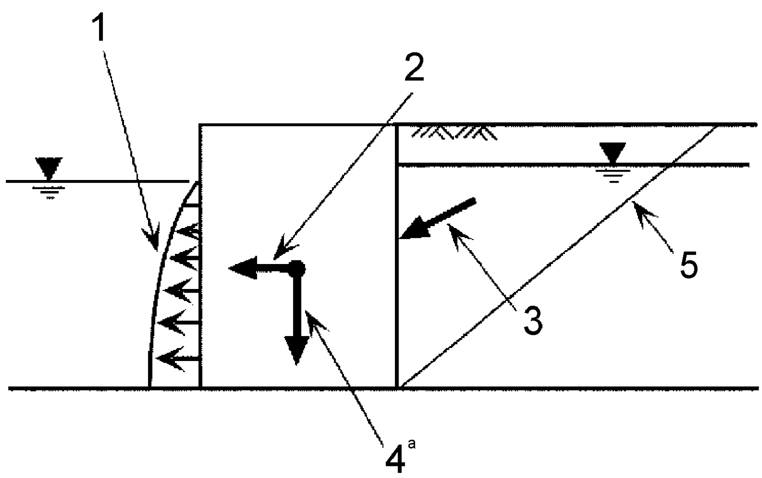
Figure H.6 — FE model for tunnel- or tube-like structures (along axis) for detailed dynamic analysis

H.6 Retaining walls

H.6.1 Simplified equivalent static analysis

Seismic actions and gravity acting on retaining walls typically used in simplified equivalent static analysis are shown in Figure H.7.

NOTE Static water pressures below sea surface or ground water table are typically treated as buoyant force to soil and structure, and the effect of a remaining difference between the sea surface level and the ground water table is typically treated as static residual water pressure acting on the wall.



Key

1 hydrodynamic pressure

2 horizontal component of inertia force

3 seismic earth pressure

4 gravity + vertical component of inertia force

5 failure plane

[a] See H.1.

Figure H.7 — Seismic actions on retaining walls for simplified equivalent static analysis

A rigid or flexible wall model is typically used with the following:

a) parameters of reference earthquake motions: peak acceleration at the ground surface. Depending on the analysis method used, peak accelerations at the elevation other than the ground surface may be used; for example 1) at the centres of gravity for wall and backfill, or 2) at the toe of the wall but at the centre of gravity for backfill;

b) actions: seismic earth pressures; hydro-dynamic pressures; inertia force on the wall;

c) performance criteria parameters:

— margin with respect to the threshold limit with respect to assumed failure mode;

— acceptable displacement based on assumed failure mode;

— margin with respect to the structural elastic limits specified in terms of stress resultants or stresses.

H.6.2 Simplified dynamic analysis

Depending on the analysis methods used, relevant parameters and actions are as follows:

a) for a rigid sliding block model:

   1) parameters of reference earthquake motions: acceleration time history of ground at the base of the sliding mass;

   2) actions: time history of seismic earth pressures; time history of hydro-dynamic pressures; time history of inertia force on the wall;

   3) performance criteria parameters:

      — acceptable residual wall displacement based on assumed mode;

      — acceptable residual response specified in terms of strains or ductility factor;

b) for a simplified chart based on parametric study (e.g. PIANC [12]):

   1) parameters of reference earthquake motions: acceleration or other appropriate ground motion parameters at the interface between the firm ground and the local soil deposit;

   2) actions: the same as a);

   3) performance criteria parameters:

      — acceptable residual wall displacement based on assumed mode;

      — acceptable residual response specified in terms of strains or ductility factor.

H.6.3 Detailed dynamic analysis

Detailed dynamic analysis is typically carried out by means of a FE model. Actions input to the FE model are acceleration or velocity time history of ground motion with or without spatial variation at the bottom boundary of the analysis domain and displacement or velocity distribution time histories at the side boundaries.

Response mode, peak displacement, the stress resultants, stress and strain of a structure can be computed if an equivalent linear model is used. Response and failure modes, peak and residual displacements, the stress resultants, stresses and strains of a structure can be computed if a non-linear model is used.

Performance criteria can be specified in terms of

— appropriate response/failure mode of soil-structure system,

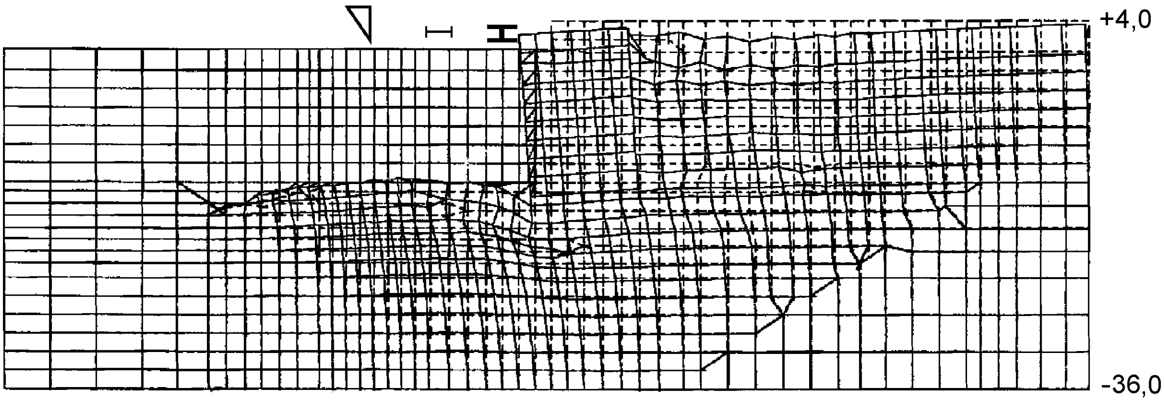
— acceptable structural peak response specified in terms of the stress resultants or stresses,

— acceptable structural residual response specified in terms of strains or ductility factor, and

— acceptable peak or residual response specified in terms of displacements.

Figure H.8 shows an example of residual displacement obtained through the detailed dynamic analysis of a gravity quay wall.

Dimensions in metres



Key

- inclination, 4,1°
- lateral displacement, 3,5 m
- vertical displacement, 1,5 m

Figure H.8 — Example of residual deformation of a gravity quay wall computed by detailed dynamic analysis using non-linear constitutive model

H.7 Earth structures

H.7.1 Simplified equivalent static analysis

Seismic actions and gravity acting on earth structures typically used in simplified equivalent static analysis are shown in Figure H.9(a).

Depending on the models of analysis, relevant parameters and actions may be specified as follows:

a) for a rigid block of soil model defined by a potential slip surface [see Figure H.9(a)]:

   1) parameters of reference earthquake motions: peak acceleration at the elevation of sliding block of soil;

   2) actions: inertia force on assumed sliding block of soil;

   3) performance criteria parameters:

      — margin to the threshold limit for sliding;

      — acceptable residual response specified in terms of displacement;

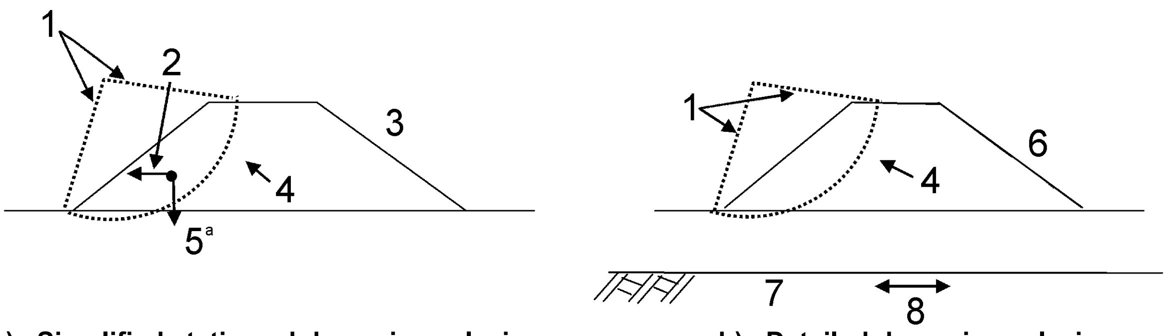
b) for evaluating densification of earth structure and foundation soil due to cyclic shear:

   1) parameters of reference earthquake motions: time history of shear stress in the earth structure and foundation soil;

   2) actions: amplitude and number of cycles of equivalent uniform cyclic stress;

   3) performance criteria parameters:

— acceptable residual response specified in terms of total and differential settlements of earth structure.



a) Simplified static and dynamic analysis **b) Detailed dynamic analysis**

Key

1 radius for failure circle
2 horizontal component of inertia force
3 simplified analysis
4 failure plane
5 gravity + vertical component of inertia force
6 detailed analysis
7 firm ground
8 earthquake motions

[a] See H.1.

Figure H.9 — Seismic actions on earth structures

H.7.2 Simplified dynamic analysis

Simplified dynamic analysis of earth structures are typically based on models of rigid sliding block of soil (Newmark method) with the following:

a) parameters of reference earthquake motions: acceleration time history of ground at the base of sliding mass;

b) actions: time history of inertia force on assumed sliding block of soil;

c) performance criteria parameters:

— acceptable residual response specified in terms of displacement (crest settlement) based on assumed failure mode;

— acceptable discontinuity in lateral and vertical displacements.

H.7.3 Detailed dynamic analysis

The model for detailed dynamic analysis is typically based on a FE model. Relevant parameters and actions are the same as described in H.2.3.

Response mode, peak and time histories of stresses and strains can be computed if an equivalent linear model is used. Failure mode and residual displacements, including crest settlement, can be computed if a non-linear model is used.

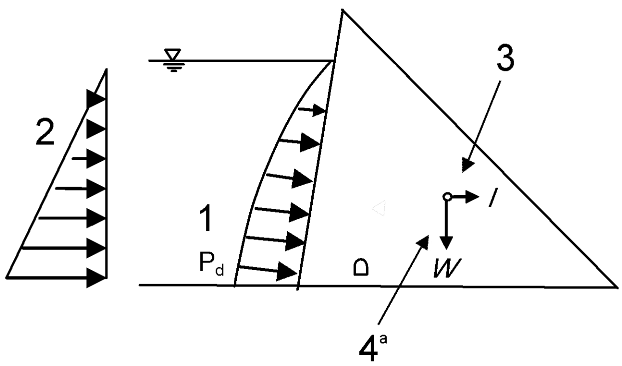
Performance criteria can be specified in terms of

- appropriate response/failure mode, and
- acceptable residual response specified in terms of residual displacements, typically defined in terms of crest settlement or other displacements and deformations with respect to the original geometry.

H.8 Gravity dams

H.8.1 Simplified equivalent static analysis

Seismic actions and gravity acting on gravity dams typically used in simplified equivalent static analysis are shown in Figure H.10.



Key

1 hydrodynamic pressure, P_d
2 static water pressure
3 horizontal component of inertia force
4 gravity + vertical component of inertia force

[a] See H.1.

Figure H.10 — Actions on gravity dams for simplified equivalent static analysis

Relevant parameters and actions are as follows:

a) parameters of reference earthquake motions: peak acceleration at ground surface, or acceleration or velocity response spectrum;

b) actions: inertia force, hydro-dynamic pressure;

c) performance criteria parameters:

- margin with respect to the overturning and sliding at any horizontal plane within the dam;
- margins to structural elastic limits.

H.8.2 Detailed equivalent static analysis

In detailed equivalent static analysis of gravity dams, a FE model is used based on the seismic coefficient approach. Relevant parameters and actions are as follows:

a) parameters of reference earthquake motions: acceleration or velocity spectrum;

b) actions: distributed inertia force, hydro-dynamic pressure, and water pressure at the bottom of the structure body;

c) performance criteria parameters:

  — acceptable peak structural response specified in terms of the stress resultants or stresses;

  — appropriate failure mode.

H.8.3 Detailed dynamic analysis

The model for detailed dynamic analysis is typically based on a FE model. Relevant parameters and actions are the same as described in H.2.3.

Response and failure mode of foundation-dam system; the stress resultants, stresses and strains can be computed by means of a linear or non-linear model.

Performance criteria can be specified in terms of

— appropriate response/failure mode,

— margin to the threshold limit for sliding, and

— margins to the structural elastic limits of the gravity dams.

Annex I
(informative)

Soil-structure interaction for designing deep foundations: phase for inertial and kinematic interactions

An earthquake response of a foundation embedded in soft ground is affected by both kinematic and inertial interactions. In order to evaluate stress resultants such as bending moment and shear force of deep foundations, effects of these two types of interactions should be taken into account.

In the simplified equivalent static analysis of the foundations, seismic actions induced by the inertial interaction can be idealized as pseudo-static inertia forces from a superstructure. These actions are specified based on the response acceleration distribution of the superstructure at the instance of the peak base shear.

The actions induced by kinematic interaction can be idealized as soil deformation relative to the bottom of the foundation. These actions can be specified based on the peak displacement distribution of the free field response.

Combination of the inertia force and the soil deformation requires careful evaluation. For some cases the response acceleration distribution of the superstructure at the instance of the peak base shear does not occur at the same time as the peak displacement distribution in the free field, making simple superposition of the two mechanisms inappropriate (see Annex H.3).

The characteristics of the soil-foundation-structure interaction, generally, are strongly controlled by the relationship between the periods of the structure, T_s, and the soil deposit, T_g. A recommended relationship for use in design may be found in a reference by Murono and Nishimura [8].

NOTE Period of the structure, T_s, is defined as the period that includes interaction effects between structure and soil.

Annex J
(informative)

Limitations in the conventional method and emerging trend for evaluating active earth pressure

The well-known Mononobe-Okabe method, based on a pseudo-static and limit-equilibrium approach, is widely used to calculate seismic earth pressure. It was suggested by previous investigations that, in general, the Mononobe-Okabe method can reasonably predict a total active earth pressure during moderate earthquake ground motions. However, the Mononobe-Okabe method provides unrealistic high active earth pressure for intense earthquake ground motions. An emerging trend is directed toward modifying the Mononobe-Okabe method for accommodating the situations during an intense earthquake ground motion. This Annex describes the limitations in the Mononobe-Okabe method and the emerging trend toward modifying it.

The Mononobe-Okabe method considers effects of an inertia force acting uniformly in the backfill soil having a Coulomb type soil wedge, with its horizontal and vertical components $k_\mathrm{h} \cdot W$ and $k_\mathrm{v} \cdot W$, respectively (see Figure J.1), where W is the self weight of the soil wedge, k_h and k_v are the horizontal and vertical seismic coefficients that are specified based on the peak acceleration ratio a/g at the centre of gravity of the assumed sliding soil mass. The total active earth pressure P_a can be evaluated as

$$P_\mathrm{a} = 1/2 \cdot \gamma h^2 (1 - k_\mathrm{v}) \cdot K_\mathrm{a} \tag{J.1}$$

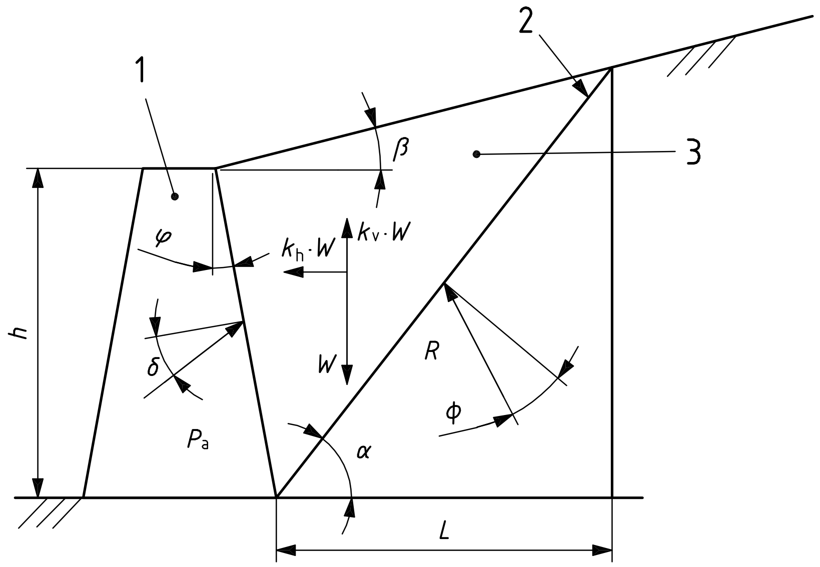
where γ is the unit weight of the backfill soil, h is the total height of the retaining wall, and K_a is the active earth pressure coefficient calculated as

$$K_\mathrm{a} = \frac{\cos^2(\phi - \varphi - \theta)}{\cos\theta \cdot \cos^2\varphi \cdot \cos(\delta + \varphi + \theta)\left[1 + \sqrt{\dfrac{\sin(\phi + \delta) \cdot \sin(\phi - \beta - \theta)}{\cos(\delta + \varphi + \theta) \cdot \cos(\varphi - \beta)}}\right]^2} \tag{J.2}$$

Here, ϕ is a soil internal friction angle which is uniform and isotropic in the backfill, δ is a friction angle at the interface between the retaining wall and the backfill soil (with the sign of δ defined positive for the case shown in Figure J.1), φ is a inclination of the back face of the retaining wall measured from the vertical direction, β is an angle of the surface slope of the backfill soil measured from the horizontal direction, and θ denotes the direction of the total of the inertia force and the self weight of the soil wedge measured from the vertical direction, which is given by

$$\theta = \tan^{-1}\left(\frac{k_\mathrm{h}}{1 - k_\mathrm{v}}\right) \tag{J.3}$$

With a constant ϕ, the value of K_a gradually increases with an increase in seismic coefficient k_h. There is an upper limit of k_h in the Mononobe-Okabe method, beyond which K_a cannot be evaluated where the term of "$\phi - \beta - \theta$" in the square-root term in Equation (J.2) becomes negative. Beyond this limit equilibrium state, the pseudo-static equilibrium of forces acting on the soil wedges shown in Figure J.1 cannot be maintained and the seismic earth pressures cannot be evaluated through the Mononobe-Okabe method.



Key

1 retaining wall

2 failure plane

3 backfill soil

Figure J.1 — Schematic figure of forces acting on soil wedge assumed by Mononobe-Okabe method

In the Mononobe-Okabe method, a shear resistance of the backfill soil is assumed to be uniform, isotropic and constant. It has been shown, however, that the behaviour of a sliding soil mass is affected by such factors as strength anisotropy, progressive failure and strain localization (a shear resistance angle mobilized along a failure plane reduces from a peak value ϕ_{peak} to a residual value ϕ_{res}). Among these, effects of strain localization into a shear band and associated strain-softening in the shear band are considered in the "Modified Mononobe-Okabe method" (Koseki *et al*. [6]).

The modified Mononobe-Okabe method may be compared with the Mononobe-Okabe method as follows:

a) The modified Mononobe-Okabe method evaluates an active earth pressure coefficient K_a which is larger than that predicted by the Mononobe-Okabe method with $\phi = \phi_{\text{peak}}$; the latter method underestimates the actual pressure because the post-peak reduction of the shear resistance in the backfill soil is not considered. On the other hand, the modified Mononobe-Okabe method evaluates a K_a value which is smaller than that predicted by the Mononobe-Okabe method with $\phi = \phi_{\text{res}}$.

b) The modified Mononobe-Okabe method evaluates the active earth pressure coefficient at large k_h values where the Mononobe-Okabe method with $\phi = \phi_{\text{res}}$ is not applicable.

c) The modified Mononobe-Okabe method provides a reduced zone of active failure in the backfill soil compared to that predicted by the Mononobe-Okabe method.

Annex K
(informative)

Effects of liquefaction considered in various models of geotechnical works

K.1 Introduction

In Clauses 7 through 9, effects of liquefaction and induced ground displacement are described based on the categories characterized by the general mechanisms of these effects rather than the type of geotechnical works. In design, the type of geotechnical works is first identified, and then these effects will be defined for a specified model for analysis. This annex describes how these effects can be considered in the analysis of various geotechnical works.

Among the four types of analysis described in Clause 7, the detailed dynamic analysis considers the effects of liquefaction and induced ground displacement in a manner not dependent on the type of geotechnical work analysed. The effects of soil liquefaction and induced ground displacement can be obtained directly from the FE or lumped mass analysis, provided appropriate formulations and constitutive models are used. In particular, the effective stress analysis can evaluate all the effects of liquefaction described in a) through c) listed in the next paragraph and will be the primary design tool in the decade to come (see Annex G.4.3). This description is common to all the types of geotechnical works, and will not be repeated for each type.

Studies are on-going on the effects of liquefaction on geotechnical works; new design methodologies are being developed but have not fully been established. In what follows, the effects of liquefaction considered in the current and emerging design practice are summarized, categorized into the following phases (see 8.1.6).

a) immediately after the triggering of liquefaction;

b) in the later phase of liquefaction;

c) after reconsolidation of the soil.

K.2 Spread foundations

K.2.1 Simplified equivalent static analysis

In the simplified equivalent static analysis of a spread foundation, the effects of liquefaction are evaluated as follows:

a) for a rigid structure model defined by a sliding plane along the base or by a circular slip surface in the foundation ground,

   1) immediately after the triggering of liquefaction: shear strength reduction along sliding or slip surface. The effects of ground displacements following liquefaction may also be included, if significant,

   2) in the later phase of liquefaction: displacement from the liquefied ground and settlements due to cyclic shear. The effects of appropriate inertia forces may also be taken into account, if significant, and

   3) after reconsolidation of the soil: as total and differential settlements;

b) for evaluating densification of foundation soil due to cyclic shear,

   1) as total and differential settlements as well as flooding due to ejected water.

Combined effects of a) undrained cyclic shear and b) dissipation of pore water pressure may be evaluated using an empirical chart of case history data based on foundation width and settlements.

K.2.2 Detailed equivalent static analysis

Detailed equivalent static analysis can be performed using a FE model. The effects of liquefaction are simulated in the analysis by the use of reduced shear modulus in the equivalent linear analysis or the use of residual strength in the non-linear analysis (see Annex G.9).

K.2.3 Simplified dynamic analysis

Simplified dynamic analysis of a spread foundation typically employs an elastic spring model. The effects of liquefaction are evaluated as follows:

a) immediately after the triggering of liquefaction: through a reduction factor for subgrade reaction;

b) in the later phase of liquefaction: displacement from the liquefied ground and reduced spring stiffness;

c) after reconsolidation of the soil: as total and differential settlements.

K.3 Pile foundations

K.3.1 Simplified equivalent static analysis

In the simplified equivalent static analysis of pile foundations, the effects of liquefaction are evaluated as follows:

For a beam/frame model,

a) immediately after the triggering of liquefaction: through a reduction factor for subgrade reaction. The effects of ground displacement may be included if significant,

b) in the later phase of liquefaction: a set of displacement distribution and reduced spring stiffness, or earth pressures from the liquefied ground. The effects of appropriate inertia forces may also be taken into account, if significant, and

c) after reconsolidation of the soil: as total and differential settlements.

K.3.2 Detailed equivalent static analysis

Detailed equivalent static analysis is performed using a FE or lumped mass model. The effects of liquefaction are simulated in the analysis by the use of reduced shear modulus in the equivalent linear analysis or the use of residual strength in the non-linear analysis (see Annex G.9).

K.3.3 Simplified dynamic analysis

Simplified dynamic analysis of pile foundation is typically done using either a superstructure-spring foundation model or a beam/frame model. The effects of liquefaction are evaluated as follows:

a) immediately after the triggering of liquefaction: through a reduction factor for subgrade reaction;

b) in the later phase of liquefaction: displacement from the liquefied ground and reduced spring stiffness;

c) after reconsolidation of the soil: as total and differential settlements.

K.4 Buried structures (transverse section)

K.4.1 Simplified equivalent static analysis

In the simplified equivalent static analysis of buried structures in transverse section, the effects of liquefaction are evaluated depending on the analysis model used as follows:

a) for a rigid or flexible wall model:

   1) immediately after the triggering of liquefaction: earth pressures; uplift force due to buoyancy; reduction in shear resistance through a reduction factor for subgrade reaction;

   2) in the later phase of liquefaction: displacement from the liquefied ground and reduced spring stiffness;

   3) after reconsolidation of the soil: as total and differential settlements;

b) for a frame-spring model:

   1) immediately after the triggering of liquefaction: displacement from the liquefied ground and springs with reduced rigidity; uplift force due to buoyancy, provided that the buried structure is lighter than the soil mass per unit volume;

   2) in the later phase of liquefaction: displacement from the liquefied ground and reduced spring stiffness;

   3) after reconsolidation of the soil: as total and differential settlements.

K.4.2 Detailed equivalent static analysis

Detailed equivalent static analysis using FE method can be performed for evaluating deformation of buried structures (transverse section), including extent of uplift due to liquefaction. The effects of liquefaction are simulated in the analysis by the use of reduced shear modulus in the equivalent linear analysis or the use of residual strength in the non-linear analysis (see Annex G.9).

K.5 Tube-like buried structures (along axis)

Simplified equivalent static analysis of tube-like buried structures is typically done using a beam model. The effects of liquefaction are evaluated as follows:

a) immediately after the triggering of liquefaction: uplift force due to buoyancy

b) in the later phase of liquefaction: displacement distribution from the liquefied ground and reduced stiffness of springs shown in Figure H.5;

c) after reconsolidation of the soil: as total and differential settlements.

Simplified dynamic analysis can also be used.

K.6 Retaining walls

As described in 8.1.6, for some types of geotechnical works such as retaining walls, detailed dynamic analysis of liquefaction effects are performed, or simplified equivalent static analysis may be used where remedial measures such as ground improvement are being implemented.

K.7 Earth structures

K.7.1 Simplified equivalent static analysis

In the simplified equivalent static analysis of earth structures, the effects of liquefaction are evaluated taking account of strength and stiffness reduction, and subsequent total and differential settlements due to pore pressure dissipation.

K.7.2 Detailed equivalent static analysis

Detailed equivalent static analysis is performed using a FE or lumped mass model. The effects of liquefaction are simulated in the analysis by the use of reduced shear modulus in the equivalent linear analysis or the use of residual strength in the non-linear analysis (see Annex G.9).

Annex L
(informative)

Evaluation of other induced effects

L.1 Introduction

This annex describes evaluation of other induced effects, including effects of fault displacements, landslides, permanent ground deformation, differential settlements, flooding, and inundation. The ability of a structure to accommodate differential movements between foundation elements depends on the configuration of a structure (e.g. bridges versus buildings, spacing between foundation elements or columns, ratio of building height to width, type of load-carrying frame, architectural finish, building contents) and foundation type (e.g. deep foundation, mat foundation, shallow footings, inclusion of connecting grade beams). Allowable foundation deformations for a given structure depend on the specified performance objectives for serviceability and safety of the structure and its contents.

Guidelines for acceptable foundation deformations have been widely established for settlement under gravity loading alone. Such guidelines are available from numerous references and organizations throughout the world and in foundation engineering textbooks. Some guidelines account for most of the various factors discussed above while others are simplified to a shorter set of factors and performance criteria.

Guidelines for acceptable foundation deformations induced by seismic loading are not well established for general conditions but rather are often modified from guidelines established for gravity loading based on consideration of the particular structure type and expected magnitude and distribution of foundation deformations. Consultation with the owner and structural designer is necessary for establishing the performance objectives and allowable settlements for specific structures.

The hazard level associated with these other induced effects should be determined consistent with the reference earthquake motions.

L.2 Fault displacements

Assessing the hazard from potential fault displacements requires a geologic site evaluation that considers the fault type, the fault’s age or activity, the potential magnitude of fault displacements, the possible distribution of deformations around the surface rupture zone, and the positioning of those deformations relative to the structure of interest.

Potential fault displacements can directly and indirectly affect a given structure. Direct effects come from the structure being located within the rupture zone and hence being subject to imposed foundation deformations. Indirect effects may occur where the fault displacement damages nearby facilities that in turn pose a hazard to the structure of interest. For example, fault displacements may damage utility lines (e.g. water, gas, communications), breach water retention structures (e.g. dams, levees), or result in subsidence.

L.3 Landslides

Earthquake-induced landslides can result from transient inertial loads during shaking and from strength loss in soil and rock masses. Inertial loads can cause a slope to yield and progressively deform during each cycle of shaking. In addition, cyclic loading and slope deformation can cause substantial strength loss in brittle materials, leading to dramatic slope failures.

Landslides can occur on many different scales, from small slopes to entire mountainsides, and can occur in a broad range of geologic materials. An assessment of landslide potential requires a geologic site evaluation that includes both geologic and historic evidence of past landslide activity in the area.

Landslides can directly or indirectly pose a hazard to a given structures. Direct hazards may come from the structure being located on or adjacent to the failure mass or being located in the potential path of the slide debris. Indirect hazards include, for example, the possibility that a slide renders the structure inaccessible, disrupts utilities, or blocks adjacent rivers.

L.4 Permanent ground deformations

Permanent ground deformations, both vertically and horizontally, may develop in nonliquefiable soils during seismic shaking.

Seismic compression refers to the volumetric contraction of soils under the cyclic loading induced by earthquakes, which then results in settlement of the ground surface. The potential for seismic compression is greatest in loose natural deposits and poorly compacted fills. Similarly, permanent shear deformations may also develop in otherwise-stable slopes and earthen structures due to shear strains induced in soils during earthquake loading. Together, the consequence of such strains can be differential settlements that depend on the spatial variations in soil properties, system geometry (e.g., fill thickness, stratigraphy), and proximity to sloping boundaries.

L.5 Differential settlements

Differential settlements for a structure on shallow foundations due to earthquake loading depend on the spatial variation in soil properties beneath the structure, the distribution of static and seismic loads among the foundation elements, the range of footing dimensions and geometries, and the characteristics of the structural connections between individual footings.

Differential settlements may be evaluated based on the spatial variability of the subsurface soils and the manner in which the consequent variations in soil strains will affect distribution of surface settlements. Soil strains due to interaction with the structural foundations will develop within the zone of influence of the foundations, whereas soil strains due to earthquake shaking alone may occur at all depths. The effects of liquefaction can be significant, including a situation where liquefaction develops in isolated pockets under individual footings or the potential for lateral spreading to produce variable amounts of deformation across the structure’s footprint. In extreme cases, the differential settlement between adjacent footings could almost equal the maximum surface settlement. In other scenarios, differential settlements may remain small despite large surface settlements, such as has been observed for relatively thick, stiff strata overlying a liquefiable layer of relatively uniform properties and thickness in an area not prone to lateral spreading.

Foundation footings of very different sizes, loads, and embedment may also be expected to experience increased differential settlements as a result of increases in the range of stresses and strains produced in the supporting soil. Footings without grade beam connections may settle relatively independent of one another, while the presence of grade beams or similar structural connecting members may reduce differential settlements by restraints of movement of individual columns and redistribution of vertical and lateral loads.

There may be limitations in most analysis methods to accurately predict the distribution of differential settlements for structures on shallow foundations. In any analysis, allowance is made for the conditions leading to the maximum and minimum foundation settlements and their potential proximity to each other. This includes allowing for the range of possible soil properties (weakest to strongest) and foundation loading conditions.

Case histories indicate that differential settlements due to earthquake ground motions can also occur at silt or clayey soil sites.

L.6 Flooding and inundation

There is potential for flooding as a consequence of earthquakes and the impact that it may have on a structure's performance.

Flooding can occur when an earthquake damages some water retention system that then releases sufficient water to flood or inundate the structure of interest. Flooding may result from earthquake-induced failure of upstream dams, levees, pipelines, aqueducts, or storage tanks.

Flooding can also result from tsunamis or seiche waves for structures located in susceptible waterfront areas.

Inundation of structures can also occur when earthquake-induced subsidence lowers the ground surface below adjacent water bodies, lowers the ground surface below the regional water table, or causes ponding of surface water due to disruption of surface runoff patterns.

The damaging effects of flooding or inundation include the hydraulic forces of rapidly moving water, the impact of suspended debris in moving water, the drowning hazard to people, and scour.

Annex M
(informative)

Concepts of response control and protection

The concept of response control similar to described for superstructures in ISO 3010 may be applied also to geotechnical works. These works are affected significantly by ground displacements and the effects of liquefaction. The effects of ground displacements may be reduced by introducing flexible or expansion joints as a structural inclusion in a geotechnical work that tends to attract high stress concentration. For example, the flexible joints may be used at the connections between buried pipe segments or at the pipe-structure connection.

Other examples are

— placing energy absorbing materials around a geotechnical work, and

— base-isolation or response control of superstructures for reducing seismic actions on foundations.

Protection of geotechnical works against possible seismic damage may include implementing controlling measures against triggering of liquefaction. Examples are

— densification,

— drainage,

— lowering of water table,

— removal and replacement of liquefiable soil, and

— grouting/soil mixing.

Other measures for controling the effects by liquefaction-induced displacements include

— flattening of slopes,

— addition of stabilizing berms, and

— insertion of structural piles or a continuous underground wall.

Annex N
(informative)

Interdependence of geotechnical and structure designs

The performance criteria for foundation components should be established with a full understanding of the behaviour of the overall system including both the superstructure and the foundation. This often requires a comprehensive understanding of the interdependence of the superstructure and foundation responses.

In general, it is difficult to determine prior to completing an analysis whether a stiffer or softer foundation will result in greater or smaller inelastic displacement and ductility demands on a superstructure during an earthquake. Furthermore, there are cases where the relative stiffness of different foundation components (e.g. shear wall versus individual column footings) can affect the distribution of inelastic yielding and local ductility demands within the superstructure. For these reasons, a geotechnical designer must avoid presuming that softer or weaker soil properties are conservative for design of the superstructure, as may be the case for many static design problems, and instead communicate with a structural designer regarding the implications of the characteristics of the foundation on the performance of the superstructure.

For example, consider the inelastic response of the foundation soils for a building. Inelastic force-deformation response of the soils (e.g. rocking, uplifting, lateral sliding, or vertical compression) has two important effects that must be considered: (1) the energy dissipated by the inelastic response of the soils can contribute to damping in the overall system and hence result in a reduction of the structural response, while (2) it can result in permanent foundation settlements, lateral displacements, or rotations which can compromise the long-term performance of the overall structure. In addition, changes in the foundation stiffness can lead to behaviour that results in significant period shift of the overall system. Understanding the trade-offs between these different consequences often is the key to delivering a well balanced overall design, with the foundation designed to sufficient overall global (geotechnical mode factor of safety) capacity to minimize potential permanent foundation displacement, while the designed system is not so overly stiff that it induces undesirable demands on the superstructure (e.g. larger force demands or unexpected distributions of inelastic yielding). Thus, the structural and geotechnical designers should work closely together, as part of establishing the final performance criteria, to understand how the foundation response affects the structural response, including the basis for the estimated foundation loads and the effect that varying those estimated foundation loads has on the final foundation design.

Similar situations develop for a wide variety of geotechnical works, whereby the establishment of reasonable performance criteria can only be reached through the close communication of the structural and geotechnical designers. Close communication is particularly important when the designs of the structural and geotechnical components of a project are conducted separately (in parts or in whole). A clear understanding of the design objectives and the interaction between the superstructure and foundation are needed by both the structural and geotechnical designers to ensure that the effects of design decisions and recommendations are recognized by all participants in the overall design of the system.

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