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Title: Draft BS EN ISO 19901-2 Oil and gas industries including lower carbon energy - Specific requirements for offshore structures.  
Part 2: Seismic design procedures and criteria.

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## Introduction

This draft standard is based on national and international discussions. Your comments on this draft are invited and will assist in the preparation of the consequent standard.

For international standards, comments will be reviewed by the relevant UK national committee before sending the consensus UK vote and comments to the international committee, which will then decide appropriate action. If the international standard is approved, it is usual for the text to be published as a British Standard.

For national standards, comments will be reviewed by the relevant UK national committee and the resulting standards published as a British Standard.

## UK Vote

Please indicate whether you consider the UK should submit a negative (with supporting technical reasons) or positive vote on this draft. Please indicate if you are aware of any reason why this draft standard should not be published as a British Standard.

## Submission of Comments

- **Annotated drafts are not acceptable and will be rejected.**
- All comments should be submitted online at <https://standardsdevelopment.bsigroup.com/>. You will need to register in order to comment.

## Template for comments and secretariat observations

|                  |                        |
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| 1  | 2                                          | (3)                          | 4               | 5                                                                                                                                  | (6)                                                                      | 7                                                  |
|----|--------------------------------------------|------------------------------|-----------------|------------------------------------------------------------------------------------------------------------------------------------|--------------------------------------------------------------------------|----------------------------------------------------|
| MB | Clause No./ Subclause No./Annex (e.g. 3.1) | Paragraph/ Figure/Table/Note | Type of comment | Comment (justification for change) by the MB                                                                                       | Proposed change by the MB                                                | Secretariat observations on each comment submitted |
|    | 3.1                                        | Definition 1                 | ed              | Definition is ambiguous and needs clarifying.                                                                                      | Amend to read '...so that the mains connector to which no connection...' |                                                    |
|    | 6.4                                        | Paragraph 2                  | te              | The use of the UV photometer as an alternative cannot be supported as serious problems have been encountered in its use in the UK. | Delete reference to UV photometer.                                       |                                                    |



# DRAFT International Standard

## ISO/DIS 19901-2

### Oil and gas industries including lower carbon energy — Specific requirements for offshore structures —

#### Part 2: Seismic design procedures and criteria

ICS: 75.180.10

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## Foreword

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

The procedures used to develop this document and those intended for its further maintenance are described in the ISO/IEC Directives, Part 1. In particular, the different approval criteria needed for the different types of ISO documents **should** be noted. This document was drafted in accordance with the editorial rules of the ISO/IEC Directives, Part 2 (see [www.iso.org/directives](http://www.iso.org/directives)).

Attention is drawn to the possibility that some of the elements of this document **may** be the subject of patent rights. ISO **shall** not be held responsible for identifying any or all such patent rights. Details of any patent rights identified during the development of the document will be in the Introduction and/or on the ISO list of patent declarations received (see [www.iso.org/patents](http://www.iso.org/patents)).

Any trade name used in this document is information given for the convenience of users and does not constitute an endorsement.

For an explanation of the voluntary nature of standards, the meaning of ISO specific terms and expressions related to conformity assessment, as well as information about ISO's adherence to the World Trade Organization (WTO) principles in the Technical Barriers to Trade (TBT), see [www.iso.org/iso/foreword.html](http://www.iso.org/iso/foreword.html).

This document was prepared by Technical Committee ISO/TC 67, *Oil and gas industries including lower carbon energy*, Subcommittee SC 7, *Offshore structures*, in collaboration with the European Committee for Standardization (CEN) Technical Committee CEN/TC 12, *Oil and gas industries including lower carbon energy*, in accordance with the Agreement on technical cooperation between ISO and CEN (Vienna Agreement).

This fourth edition cancels and replaces the second edition (ISO 19901-2:2022), which has been technically revised.

The main changes are as follows:

- scope expanded to cover offshore wind and other renewable energy offshore structures;
- incorporates requirements from common industry specifications (IOGP JIP 35);
- the seismic hazard maps have been updated;
- written with clear and concise requirements.

A list of all parts in the ISO 19901 series can be found on the ISO website.

Any feedback or questions on this document **should** be directed to the user's national standards body. A complete listing of these bodies can be found at [www.iso.org/members.html](http://www.iso.org/members.html).

## Introduction

The International Standards on offshore structures prepared by TC 67 address design requirements and assessments of all offshore structures used in the energy sector worldwide. Through their application, the intention is to achieve reliability levels appropriate for normally occupied and normally unoccupied offshore structures, whatever the type of structure and the nature or combination of the materials used.

Structural integrity is an overall concept comprising models for describing actions, structural analyses, design or assessment rules, safety elements, workmanship, quality control procedures and national requirements, all of which are mutually dependent. The modification of one aspect of design or assessment in isolation can disturb the balance of reliability inherent in the overall concept or structural system. The implications involved in modifications, therefore, need to be considered in relation to the overall reliability of all offshore structural systems.

The International Standards on offshore structures prepared by TC 67 are intended to provide a wide latitude in the choice of structural configurations, materials, and techniques without hindering innovation. Sound engineering judgement is, therefore, necessary in the use of these International Standards.

The overall concept of structural integrity is described above. Some additional considerations apply for seismic design. These include the magnitude and probability of seismic events, the use and importance of the offshore structure, the robustness of the structure under consideration and the allowable damage due to seismic actions with different probabilities. All of these, and any other relevant information, need to be considered in relation to the overall reliability of the structure.

Seismic conditions vary widely around the world, and the design criteria depend primarily on observations of historical seismic events together with consideration of seismotectonics and local soil conditions. In many cases, site-specific seismic hazard assessments will be required to complete the design or assessment of a structure.

This document is intended to provide general seismic design procedures for different types of offshore structures, and a framework for the derivation of seismic design criteria. Further requirements are contained within the general requirements International Standard, ISO 19900, and within the structure-specific International Standards, ISO 19902, ISO 19903, ISO 19904, and ISO 19906. The consideration of seismic events in connection with mobile offshore units is addressed in ISO 19905.

# Oil and gas industries including lower carbon energy — Specific requirements for offshore structures —

## Part 2: Seismic design procedures and criteria

### 1 Scope

This document contains provisions for seismic design and assessment of offshore structures.

Recommendations for the effects of seismic events on floating structures are introduced.

Design and assessment of earthquake-induced ground motions are specifically addressed. Other geologically induced hazards such as liquefaction, slope instability, faults, tsunamis, mud volcanoes and shock waves are briefly covered.

Provisions for site-specific probabilistic seismic hazard analysis are provided for offshore structures in high seismic areas and for offshore structures with high consequence levels.

NOTE Guidance and background information relating to the requirements is included in [Annex A](#).

### 2 Normative references

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

ISO 19900, *Oil and gas industries including lower carbon energy — General requirements for offshore structures*

### 3 Terms and definitions (Pending)

For the purposes of this document, the terms and definitions given in ISO 19900 and the following apply.

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

- ISO Online browsing platform: available at <https://www.iso.org/obp>
- IEC Electropedia: available at <https://www.electropedia.org/>

#### 3.1 abnormal level earthquake ALE

intense earthquake of abnormal severity with a very low probability of occurring during the life of the structure, generally of return period in the 1 000's of years.

Note 1 to entry: The ALE event is comparable to the abnormal event in the design of fixed structures that are described in ISO 19902<sup>[6]</sup> and ISO 19903<sup>[7]</sup>.

#### 3.2 attenuation

decay of seismic waves as they travel from the earthquake source to the site under consideration.

### 3.3

#### **deaggregation**

separation of seismic hazard contribution from different faults and seismic source zones

### 3.4

#### **escape and evacuation system**

system provided on the offshore structure to facilitate escape and evacuation in an emergency.

EXAMPLE Passageways, chutes, ladders, life rafts and helidecks.

### 3.5

#### **extreme level earthquake**

##### **ELE**

strong earthquake with a reasonable probability of occurring during the life of the structure, generally of return period in the 100's of years.

### 3.6

#### **fault movement**

movement occurring on a fault during an earthquake.

### 3.7

#### **ground motion**

accelerations, velocities or displacements of the ground produced by seismic waves radiating away from earthquake sources.

Note 1 to entry: A fixed offshore structure is founded in or on the *seabed* (3.17) and consequently only seabed motions are of significance. The expression "ground motions" is used rather than seabed motions for consistency of terminology with seismic design for onshore structures.

Note 2 to entry: Ground motions can be at a specific depth or over a specific region within the seabed.

### 3.8

#### **liquefaction**

fluidity of soil due to the increase in pore pressures caused by earthquake action under undrained conditions.

### 3.9

#### **modal combination**

combination of response values associated with each dynamic mode of a structure.

### 3.10

#### **mud volcano**

diapiric intrusion of plastic clay causing high pressure gas-water seepages which carry mud, fragments of rock (and occasionally oil) to the surface.

Note 1 to entry: The surface expression of a mud volcano is a cone of mud with continuous or intermittent gas escaping through the mud.

### 3.11

#### **probabilistic seismic hazard analysis**

##### **PSHA**

framework permitting the identification, quantification, and rational combination of uncertainties in earthquakes' intensity, location, rate of recurrence and variations in *ground motion* (3.7) characteristics.

### 3.12

#### **probability of exceedance**

probability that a variable (or that an event) exceeds a specified reference level given exposure time.

EXAMPLE The annual probability of exceedance of a specified magnitude of ground acceleration, ground velocity or ground displacement.

**3.13****response spectrum**

maximum responses of a series of single-degree-of freedom systems subjected to a given base motion, plotted as a function of natural frequencies for specific values of damping.

**3.14****safety system**

systems provided on the offshore structure to detect, control and mitigate hazardous situations.

**EXAMPLE**

Gas detection, emergency shutdown, fire protection, and their control systems.

**3.15****sea floor**

interface between the sea and the *seabed* (3.17)

**3.16****seabed slide**

failure of *seabed* (3.17) slopes

**3.17****seabed**

soil material below the sea in which a structure is founded.

**3.18****seismic risk category****SRC**

category defined from the consequence level and the intensity of seismic motions (i.e., the spectral acceleration having an annual probability of exceedance of 1/1 000).

**3.19****seismic hazard curve**

curve showing the annual *probability of exceedance* (3.12) against a measure of seismic intensity.

Note 1 to entry: The seismic intensity measures can include parameters such as peak ground acceleration, *spectral acceleration* (3.22), or *spectral velocity* (3.23).

**3.20****seismic reserve capacity factor**

factor indicating the structure's ability to sustain ground motions due to earthquakes beyond the level of the *extreme level earthquake* (3.5)

Note 1 to entry: Refer to A.9.3.6 for more detail.

**3.21****site response analysis**

wave propagation analysis permitting the evaluation of the effect of local geological and soil conditions on the *ground motions* (3.7) as they propagate up from depth to the surface at the site.

**3.22****spectral acceleration**

maximum absolute acceleration response of a single degree of freedom oscillator subjected to *ground motions* (3.7) due to an earthquake.

**3.23****spectral velocity**

maximum pseudo velocity response of a single degree of freedom oscillator subjected to *ground motions* (3.7) due to an earthquake.

Note 1 to entry: The pseudo velocity spectrum is computed by factoring the displacement or acceleration spectra by the oscillator's circular frequency or the inverse of its frequency, respectively.

**3.24****spectral displacement**

maximum relative displacement response of a single degree of freedom oscillator subjected to *ground motions* (3.7) due to an earthquake.

**3.25****static pushover analysis**

application and incremental increase of a global static pattern of actions on a structure, including equivalent dynamic inertial actions, until a global failure mechanism occurs.

**3.26****tsunami**

long period sea waves caused by rapid vertical movements of the *sea floor* (3.15)

Note 1 to entry: The vertical movement of the sea floor is often associated with fault rupture during earthquakes or with *seabed slides* (3.16).

**4 Symbols and abbreviated terms (Pending)****4.1 Symbols**

|                  |                                                                                                                            |
|------------------|----------------------------------------------------------------------------------------------------------------------------|
| $a_R$            | tail slope of the seismic hazard curve                                                                                     |
| $C_a$            | site coefficient, a correction factor applied to the acceleration part (shorter periods) of a response spectrum            |
| $C_c$            | correction factor applied to the spectral acceleration to account for uncertainties not captured in a seismic hazard curve |
| $C_r$            | seismic reserve capacity factor; see <a href="#">Formulae (7)</a> and <a href="#">(10)</a>                                 |
| $C_v$            | site coefficient, a correction factor applied to the velocity part (longer periods) of a response spectrum                 |
| $D$              | scaling factor for damping                                                                                                 |
| $G_{\max}$       | initial (small strain) shear modulus of the soil                                                                           |
| $g$              | acceleration due to gravity                                                                                                |
| $M$              | magnitude of an earthquake measured by the energy released at its source                                                   |
| $N_{\text{ALE}}$ | scale factor for conversion of the site 1 000-year acceleration spectrum to the site ALE acceleration spectrum             |
| $p_a$            | atmospheric pressure                                                                                                       |
| $P_{\text{ALE}}$ | annual probability of exceedance for the ALE event                                                                         |
| $P_e$            | probability of exceedance                                                                                                  |
| $P_{\text{ELE}}$ | annual probability of exceedance for the ELE event                                                                         |
| $P_f$            | target annual probability of failure                                                                                       |
| $q_c$            | cone penetration resistance of soil                                                                                        |
| $q_{cl}$         | normalized cone penetration resistance of soil                                                                             |
| $\bar{q}_{cl}$   | average normalized cone penetration resistance of sand in the effective seabed                                             |

|                   |                                                                                                                                                                                                                       |
|-------------------|-----------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|
| $S_a(T)$          | spectral acceleration associated with a single degree of freedom oscillator period, $T$                                                                                                                               |
| $T$               | mean spectral acceleration associated with a single degree of freedom oscillator period, $T$ ; obtained from a PSHA                                                                                                   |
| $S_{a,ALE}(T)$    | is the spectral acceleration for a single degree of freedom oscillator period $T$ resulting from a suite of time histories that, when the structure is exposed to, would result in a probability of collapse of 50 %. |
| $S_{a,ELE}(T)$    | ELE spectral acceleration associated with a single degree of freedom oscillator period, $T$                                                                                                                           |
| $S_{a,map}(T)$    | 1 000-year rock outcrop spectral acceleration obtained from maps associated with a single degree of freedom oscillator period, $T$                                                                                    |
| $S_{a,P_e}(T)$    | mean spectral acceleration associated with a probability of exceedance, $P_e$ , and a single degree of freedom oscillator period, $T$ , obtained from a PSHA                                                          |
| $S_{a,P_{NC}}(T)$ | spectral acceleration having an annual probability of associated with a target annual probability of failure, $P_f$ , and a single degree of freedom oscillator period, $T$ , obtained from a PSHA                    |
| $S_{a,site}(T)$   | site spectral acceleration corresponding to a return period of 1 000 years and a single degree of freedom oscillator period, $T$                                                                                      |
| $s_u$             | undrained shear strength of the soil                                                                                                                                                                                  |
| $\bar{s}_u$       | average undrained shear strength of the soil in the effective seabed                                                                                                                                                  |
| $T$               | natural period of a simple, single degree of freedom oscillator                                                                                                                                                       |
| $T_{dom}$         | dominant modal period of the structure                                                                                                                                                                                |
| $T_{return}$      | return period                                                                                                                                                                                                         |
| $v_s$             | representative shear wave velocity                                                                                                                                                                                    |
|                   | average of representative shear wave velocity in the effective seabed                                                                                                                                                 |
| $\rho$            | mass density of soil                                                                                                                                                                                                  |
| $\eta$            | per cent of critical damping                                                                                                                                                                                          |
| $\sigma_{LR}$     | logarithmic standard deviation of uncertainties not captured in a seismic hazard curve                                                                                                                                |
| $\sigma'_{v0}$    | in situ vertical effective stress of soil                                                                                                                                                                             |

## 4.2 Abbreviated terms

|            |                                        |
|------------|----------------------------------------|
| DL         | damage limitation limit state          |
| DSHA       | deterministic seismic hazard analysis  |
| DSRA       | dynamic site response analysis         |
| L1, L2, L3 | consequence level derived in ISO 19900 |
| MOU        | mobile offshore unit                   |
| NC         | near collapse limit state              |
| NP         | nonlinear pushover                     |

|      |                                 |
|------|---------------------------------|
| NTHA | nonlinear time history analysis |
| PGA  | peak ground acceleration        |
| RP   | return period                   |
| THA  | time history analysis           |
| TLP  | tension leg platform            |
| ULS  | ultimate limit state            |

## 5 Seismic hazards

**5.1** Seismic design and assessment of offshore structures **shall** include the effect of ground motions due to earthquakes.

**5.2** The design and assessment of offshore structures **shall** also include the effects of the following seismic hazardous events, as developed by specialists in geologic site hazards:

- a) soil liquefaction.
- b) seabed slide.
- c) fault movement.
- d) tsunamis.
- e) mud volcanoes.
- f) velocity pulse from directivity effects.

## 6 Performance objectives, limit states and damage states

### 6.1 General

**6.1.1** The risks, due to the structure being exposed to seismic hazardous events, **shall** be demonstrated to be tolerable in conformance with [6.1.2](#) to [6.1.5](#).

NOTE 19901-2 requirements will result in the structure having sufficient strength and ductility such that the risks (life-safety, environmental-pollution, and business-disruption) are tolerable when the structure is exposed to seismic hazardous events as defined by the hazard curve for the site.

**6.1.2** The structure **shall** meet the performance objectives (as defined in [6.3](#)).

**6.1.3** Demonstration that the structure meets the performance objectives **shall** be by limit state verification in conformance with ISO 19900.

**6.1.4** The performance objective for life-safety risk, typically associated with the ALE, **shall** specify the maximum tolerable annual probability that the (damage) state of structure can exceed the Near Collapse (NC) limit state.

NOTE 1 the NC limit state is described in [6.2.1](#). When the state of the structure is at the NC limit state, the structural system remains stable, but one or more components can have failed. Since the structure remains stable and does not collapse, fatalities are limited or avoided following the seismic event, and thus the life- life-safety consequence is negligible

**6.1.5** The performance objective for business-disruption risk, typically associated with the ELE, **shall** specify the maximum tolerable annual probability that the (damage) state of structure can exceed the Damage Limitation (DL) limit state.

NOTE 1 the DL limit state is described in [6.2.2](#). When the state of the structure is at the DL limit state, the structural system remains stable, but lateral system of the structure (braces, joints, and piles) can have strains approaching the yield strain. Any damage is localised and readily repairable and thus the functionality of the facility is restored in a short time scale following the seismic event, and thus the business-disruption consequence is tolerable.

Limit state verification for the DL and NC limit states shall be in conformance with [clauses 6](#) to [10](#), as illustrated in [Figure 1](#).

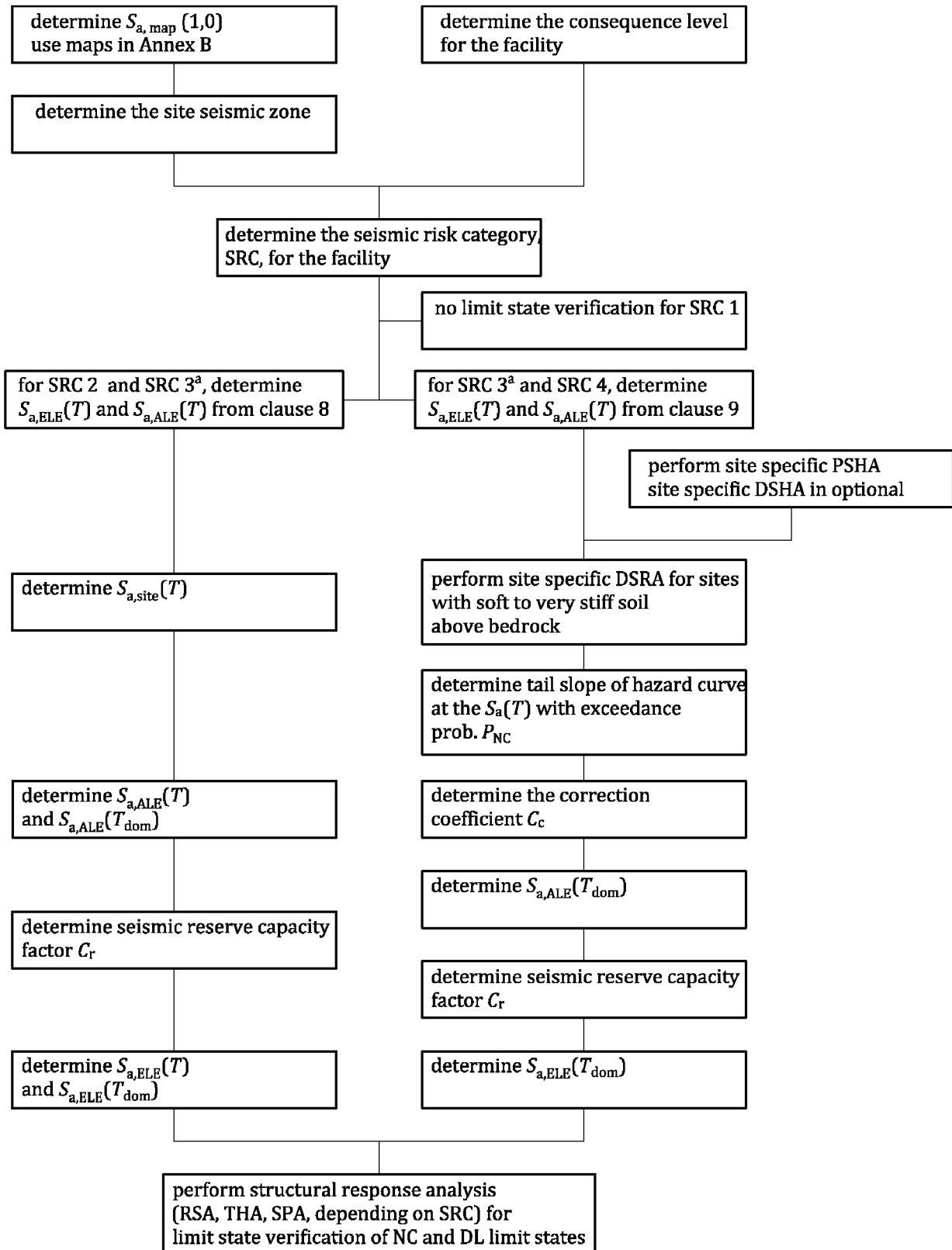


Figure 1 — Steps for limit state verification

NOTE limit state verification of SRC 3 facilities can be by [clause 8](#) or [clause 9](#)

## 6.2 Limit states and damage states

**6.2.1** The state of the structure at the Near Collapse (NC) limit state associated with the ALE:

- a) **shall not** include collapse of the structure's gravity system (i.e. legs, soil axial capacity and pile to leg connection) or collapse of the sub-system that supports the living quarters.
- b) **may** include damage to members (e.g., plastic strains, plastic local buckling in steel or spalling in concrete).
- c) **may** include severed braces (by fracture or low-cycle fatigue).
- d) **may** include plasticity at joints.
- e) **may** include collapse or fracture of joints.
- f) **may** include plastic tensile strain less than the fracture strain in piles.

NOTE DNV-RP-C208 provides guidance on fracture limits for local strain.

- g) **shall not** include damage to safety systems, escape routes, and evacuation systems that prevent their functionality.
- h) **shall not** include loss of supports for critical hydrocarbon equipment that could lead to escalation by loss of process containment (LOPC).
- i) **shall not** include collapse of the living quarters, masts, derricks, flare structures and other safety critical structures.

**6.2.2** The state of the structure at the Damage Limitation (DL) limit state associated with the ELE:

- a) **shall not** include damage to the gravity system components of the structure (i.e. legs, soil axial capacity and pile to leg connection).
- b) **shall not** include damage due to brittle mechanisms (e.g., flexural buckling, local buckling in steel or spalling in concrete).

NOTE ISO 19902 requires the seismic action is increased by a factor of  $C_r$  for code checks of leg members when the structure is subjected to seismic actions of intensity  $S_{a,ELE}$

- c) **should** limit strains at the outer fibre of braces, joints, and piles (i.e. bending plus axial strain) to the yield strain.

NOTE ISO 19902 requires the lateral system (braces, joints, and piles) are dimensioned for ductility and thus can plastically strain without local buckling during seismic actions of intensity  $S_{a,ALE}$

- d) **shall not** include damage to safety systems, escape routes, and evacuation systems that prevent their functionality.
- e) **shall not** include damage to pipelines, conductors, risers, and other safety-critical components due to displacements at mudline elevation of the structure.
- f) **shall not** include damage due to toppling of topsides equipment and cable trays.
- g) **shall not** include damage to masts, derricks, and flare structures.
- h) **shall not** include damage to sliding supports that prevent their functionality.
- i) **shall not** include damage to piping systems that prevents their functionality due to differential displacement of supports.
- j) **should not** include damage that will result in dropped objects.

### 6.3 Performance objectives

**6.3.1** The NC limit state verification of the structure **shall** be demonstrated for the ALE event in conformance with [6.3.2](#).

**6.3.2** The annual probability of the structure exceeding the NC limit state, as defined in [6.2.1](#), **shall not** be greater than the value of  $P_{NC}$  listed in [Table 1](#).

**Table 1 — Maximum annual probability of exceeding NC limit state.**

| Consequence level | $P_{NC}$                      |
|-------------------|-------------------------------|
| L1                | $4 \times 10^{-4} = 1/2\ 500$ |
| L2                | $1 \times 10^{-3} = 1/1\ 000$ |
| L3                | $2,5 \times 10^{-3} = 1/400$  |

NOTE 1 The annual probability in [Table 1](#) for the NC accounts for the epistemic uncertainty in the annual probability and is determined by use of the mean hazard curve and the mean fragility curve.

NOTE 2 the simplified procedure in [clause 8](#) has been calibrated to the probabilities listed in [Table 1](#).

**6.3.3** The maximum annual probability of exceeding the NC limit state **may** be less than listed in [Table 1](#) if specified by the operator or regulator.

**6.3.4** The DL limit state verification of the structure **shall** be demonstrated in conformance with the ELE events defined in [6.3.5](#).

**6.3.5** The annual probability of the as-designed structure exceeding the DL limit state, as defined in [6.2.2](#), **shall not** be greater than  $P_{ELE}$  or the value of  $P_{DL}$  listed in [Table 2](#).

**Table 2 — Maximum annual probability of exceeding the DL limit state.**

| Consequence level | $P_{DL}$ |
|-------------------|----------|
| L1                | 1 / 200  |
| L2                | 1 / 100  |
| L3                | 1 / 50   |

where  $P_{ELE}$  is defined in [Figure 7](#).

### 6.4 Seismic risk category

**6.4.1** The facility **shall** be classed by seismic risk category (SRC) in conformance with [Table 3](#).

NOTE SRC is a function of consequence level and site seismic zone.

**6.4.2** Site seismic zone **shall** be determined from the 1,0 s horizontal spectral acceleration maps in [Annex B](#) or from a site specific seismic hazard study.

**Table 3 — Seismic risk category (SRC)**

| $S_{a,map}(1,0)$<br>(see <a href="#">Annex B</a> ) | Site seismic zone | Consequence level<br>(see ISO 19900: 2019, Cl 7.3) |       |       |
|----------------------------------------------------|-------------------|----------------------------------------------------|-------|-------|
|                                                    |                   | L1                                                 | L2    | L3    |
| <0,03 g                                            | 0                 | SRC 1                                              | SRC 1 | SRC 1 |
| 0,03 g to 0,10 g                                   | 1                 | SRC 3                                              | SRC 2 | SRC 2 |
| 0,11 g to 0,25 g                                   | 2                 | SRC 4                                              | SRC 2 | SRC 2 |
| 0,26 g to 0,45 g                                   | 3                 | SRC 4                                              | SRC 3 | SRC 2 |
| >0,45 g                                            | 4                 | SRC 4                                              | SRC 4 | SRC 3 |

**6.4.3** For seismic design situations, a structure **shall not** be classified as consequence level L2 unless it is normally unoccupied (except for brief periods of time such as inspection and maintenance), but with environmental-pollution consequence level greater than L3.

NOTE It is assumed that no reliable forewarning of seismic actions is feasible and, consequently, it is not possible to evacuate prior to an earthquake.

## **6.5 DL limit state verification (using hazardous events with intensity = $S_{a,ELE}$ )**

**6.5.1**  $S_{a,ELE}$  **shall** be determined in conformance with [clause 8](#) or [clause 9](#), depending on the SRC (as described in [6.8](#)).

**6.5.2** DL limit state verification **shall** be performed using:

- actions arising from ground motion time histories where each ground motion has a spectral acceleration, at the sway period of the structure, defined by  $S_{a,ELE}$ .
- representative capacities of members, joints, piles, and soil, as defined in ISO 19900.

NOTE [clause 7](#) provides analysis types and requirements to determine the above actions.

**6.5.3** DL limit state verification of the structure **shall** be deemed to have been demonstrated provided ISO 19902:2020 formula 11.5-1 and 11.5-2 are satisfied.

## **6.6 NC limit state verification (using hazardous events with intensity = $S_{a,ALE}$ )**

**6.5.1**  $S_{a,ALE}$  **shall** be determined in conformance with [clause 8](#) or [clause 9](#), depending on the SRC (as described in [6.8](#)).

**6.6.1** NC limit state verification **shall** be performed using:

- actions arising from ground motion time histories where each ground motion has a spectral acceleration, at the sway period of the structure, defined by  $S_{a,ALE}$ .
- mean capacities of members, joints, piles, and soil (by use of mean yield stress, mean component capacities and expected soil capacity)

NOTE 1 representative capacities,  $R_k$ , as defined in ISO 19900, can be used to avoid determining the mean capacities, however, this introduces conservatism into the NC limit state verification.

NOTE 2 [clause 7](#) provides analysis types and requirements to determine the above actions.

**6.6.2** NC limit state verification of the structure **shall** be deemed to have been demonstrated provided that no more than 3 of 7 ground motion records (each having a spectral acceleration at the sway period of the structure defined by  $S_{a,ALE}$ ) cause collapse of the structure in a nonlinear time-history analysis.

## 6.7 Design procedure

**6.7.1** The design procedure **shall** follow either:

- the two-stage procedure described in [6.7.2](#)
- the three-stage procedure described in [6.7.3](#)

NOTE the basis of the three-stage procedure is similar to that for the two-stage procedure, but the structure is more likely to achieve the NC limit state verification without re-sizing members or joints.

### 6.7.2 Two-stage seismic design procedure

The two-stage design procedure consists of two steps as explained in the following and shown in [Figure 2](#):



**Figure 2 — Two-stage seismic design procedure**

Stage 1. ELE Linear Design Checks

Perform linear ELE Response spectra analyses:

- DL limit state verification **shall** be satisfied for jacket braces and foundation pile stresses (mainly due to bending).

Stage 2. ALE Nonlinear Verification

Perform verification nonlinear ALE analyses:

- Member checks – NC Limit state verification **shall** be satisfied for all the elements
- System checks – confirm the NC limit state is not reached before the ALE event
- Joint 100 % strength requirement – ensure braces fail before joints

- Joint 100 % strength requirement **may** be relaxed when using an ALE nonlinear time history analysis or nonlinear pushover analysis

NOTE nonlinear time history analysis or pushover analysis provide confidence in the joints meeting the NC limit state. When performed correctly, nonlinear pushover analysis is a more rigorous check of joint strength than time-history analysis.

### 6.7.3 Three-stage seismic design procedure

The three-stage design procedure consists of three steps as explained in the following and shown in [Figure 3](#):

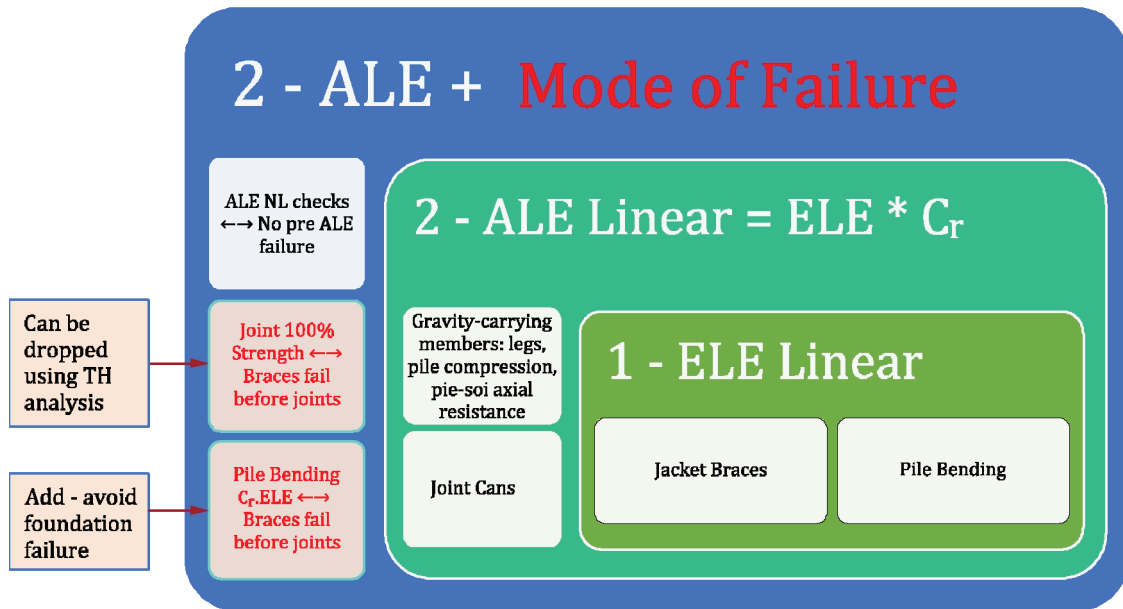


Figure 3 — Three-stage seismic design procedure

#### Stage 1. ELE Linear Design Checks

Perform linear ELE Response spectra analyses:

- DL limit state verification **shall** be satisfied for jacket braces and foundation pile stresses (mainly due to bending).
- Designer **may** run linear ELE analyses using a lower  $C_r$  (see 9.5) for piles relative to the jacket structure.

NOTE using a linear ELE design check twice, once for jacket and the second for piles, with a lower  $C_r$  value for the piles, limits failure away from the piles. This provides a check of the jacket separate from the foundation.

#### Stage 2. ALE Linear Design Checks

Perform linear ALE (ELE x  $C_r$ ) response spectra analyses:

- DL limit state verification **shall** be satisfied for all the gravity-carrying members:
  - Legs;
  - Pile in compression;
  - Pile-soil axial resistance;
  - Joint cans

NOTE by ensuring linear performance of gravity-carrying members, nonlinearity for the ALE analysis is limited to bending elements (e.g. bracing) which are ductile elements.

## Stage 3. ALE Nonlinear Verification

Perform verification nonlinear ALE analyses:

- Member checks – NC limit state verification **shall** be satisfied for all the elements
- System checks – confirm that NC limit state is not reached before the ALE event
- Joint 100 % strength requirement **may** be relaxed when using an ALE nonlinear time history analysis or nonlinear pushover analysis

NOTE nonlinear time history analysis or pushover analysis provide confidence in the joints meeting the NC limit state. When performed correctly, nonlinear pushover analysis is a more rigorous check of joint strength than time-history analysis.

## 6.8 Methods for limit state verification

### 6.8.1 General

**6.8.1.1** The limit state verification methods **shall** depend on the Seismic Risk Category as described in the following clauses.

### 6.8.2 Limit state verification methods

NOTE the requirements in this clause are summarised in [Table 4](#)

**Table 4 — Summary of requirements for NC limit state verification**

| SRC | procedure for hazard analysis                  | NTHA or NP  | procedure for determining $S_{a,ELE}$ and $S_{a,ALE}$ |
|-----|------------------------------------------------|-------------|-------------------------------------------------------|
| 1   | None                                           | None        | None                                                  |
| 2   | ISO maps or regional maps                      | Permitted   | Simplified ( <a href="#">clause 8</a> )               |
| 3   | Site specific PSHA, ISO maps, or regional maps | Recommended | Simplified ( <a href="#">clause 8</a> )               |
|     | Site specific PSHA                             | Recommended | Detailed ( <a href="#">clause 9</a> )                 |
| 4   | Site specific PSHA                             | Required    | Detailed ( <a href="#">clause 9</a> )                 |

**6.8.2.1** Limit state verification of facilities classed as SRC 4:

- a) **shall** perform the hazard analysis using PSHA in conformance with [9.1](#).
- b) **may** perform a complementary hazard analysis using DSHA in conformance with [9.2](#).
- c) **shall** determine  $S_{a,ALE}$  for NC limit state verification using the  $C_c$  method in conformance with [9.5](#).
- d) **should** perform the structural response analysis for the NC limit state by nonlinear time history analysis (NTHA) in conformance with [7.2](#).
- e) **may** perform structural response analysis for the NC limit state by nonlinear pushover (NP) in conformance with [7.3](#).

NOTE NP

- f) **shall** determine the structural response for limit state verification of the DL limit state by either:
  1. linear time history analysis in conformance with [7.2](#) or
  2. response spectrum analysis in conformance with [7.1](#).

**6.8.2.3** Limit state verification of facilities classed as SRC 3:

- a) **shall** perform the hazard analysis using PSHA in conformance with [9.1](#).
- b) **may** perform a complementary hazard analysis using DSHA in conformance with [9.2](#).
- c) **shall** determine  $S_{a,ALE}$  for limit state verification of the NC limit state by either:
  - 1. using the  $C_c$  method in conformance with [9.5](#), or
  - 2. the  $N_{ALE}$  scale factor method in conformance with [8.8](#).

NOTE a 1 000-year uniform hazard spectrum can be produced in conformance with [9.1](#) rather than the 1 000-year design response spectrum produced in conformance with [8.5](#) to [8.8](#).

- d) **shall** determine the structural response for limit state verification of the NC limit state by either:
  - 1. nonlinear time history analysis in conformance with [7.2](#), or
  - 2. nonlinear pushover analysis in conformance with [7.3](#)

NOTE for structures in soft soils with potential for collapse due to pile double hinging, nonlinear time history analysis will better capture these effects for the NC

- e) **shall** determine the structural response for limit state verification of the DL limit state by either:
  - 1. linear time history analysis in conformance with [7.2](#) or
  - 2. response spectrum analysis in conformance with [7.1](#).

**6.8.2.5** Limit state verification of facilities classed as SRC 2:

- a) **shall** perform the hazard analysis by either:
  - 1. a PSHA in conformance with [9.1](#) or
  - 2. use of the seismic maps and site correction factors in conformance with [8.2](#) to [8.4](#)
- b) **may** perform a complementary hazard analysis using DSHA in conformance with [9.2](#)
- c) **shall** determine  $S_{a,ALE}$  for limit state verification (for both NC and DL limit states) by either:
  - 1. using the  $C_c$  method in conformance with [9.5](#) or
  - 2. the  $N_{NC}$  scale factor method in conformance with [8.5](#) to [8.8](#)

NOTE a 1 000-year uniform hazard spectrum can be produced in conformance with [9.1](#) rather than the 1 000-year design response spectrum produced in conformance with [8.5](#) to [8.8](#).

- d) **shall** determine the structural response for limit state verification of the DL limit state by response spectrum analysis in conformance with [7.1](#).
- e) **may** determine the structural response for limit state verification of the NC limit state by either:
  - 1. nonlinear time history analysis in conformance with [7.2](#) or
  - 2. nonlinear pushover analysis in conformance with [7.3](#).

NOTE The simplified seismic action procedure is typically more conservative than the equivalent detailed seismic action procedure.

**6.8.2.6** Limit state verification of facilities classed as SRC 1 **shall not** be performed.

**6.8.2.7** For facilities classed as SRC >1, the risks due to collapse of the facility in seismic hazardous events **shall** be minimised by adopting the provisions in ISO 19902 for ductile design of the primary structure and the connections.

## 7 Analysis types for structural response

### 7.1 Response spectrum analysis

**7.1.1** Combination of modal responses **shall** be performed by the complete quadratic combination (CQC).

**7.1.2** Directional responses for both horizontal directions and the vertical direction **shall** be determined.

**7.1.3** Combination of directional responses:

- a) **should** be performed by the square root of the sum of the squares (SRSS).
- b) **may** be performed by linear combination.

NOTE linear combination can be applied by taking one component at its maximum and the other two components at 40 % of their respective maximum values (with the sign of each response parameter selected to maximise the response combination).

**7.1.4** The number of modes used in a response spectrum analysis **shall** be the number of modes to achieve at least 90 % mass participation in each horizontal direction.

NOTE ASCE 7-22 provides guidance on achieving 90 % mass participation.

**7.1.5** Response spectrum analysis **shall not** be used if seismic isolation or passive energy dissipation devices are employed.

**7.1.6** Linearised foundation stiffness values **shall** be compatible with the magnitude of soil deformations resulting from ground motions with spectral accelerations of  $S_{a,ELE}$  and  $S_{a,ALE}$  respectively.

**7.4.7** Damping values **shall** be compatible with deformation magnitudes resulting from ground motions with spectral accelerations of  $S_{a,ELE}$  and  $S_{a,ALE}$  respectively.

**7.4.8** Damping values **should** be based on recommendations in the International Standard applicable to the type of offshore structure, i.e. ISO 19901-3,<sup>[4]</sup> ISO 19902,<sup>[6]</sup> ISO 19903,<sup>[7]</sup> ISO 19904,<sup>[8]</sup> ISO 19905<sup>[9]</sup> and ISO 19906<sup>[10]</sup>.

**7.4.9** A damping ratio of 5 % **may** be used to represent structural and soil damping.

**7.4.10** Larger values of damping due to hydrodynamics or soil deformation (hysteretic and radiation) **may** be used provided the damping values used are substantiated.

**7.4.11** The foundation **may** be modelled with equivalent elastic springs, mass, and damping elements.

NOTE off-diagonal stiffness and damping values as well as frequency dependence can be significant.

### 7.2 Time history analysis

**7.2.1** Probability of response directionality **shall** be incorporated by time history analysis for a minimum of 7 ground motion records applied to the structure in a random orientation.

NOTE 1 7 ground motion records applied in a random orientation provide variability in the directional response of the structure.

NOTE 2 if only 4 ground motion records are available for the site, then 4 additional records can be created by exchanging the horizontal acceleration values.

**7.2.2** The ground motion records for input to the time history analysis **should** be selected based on deaggregation of the seismic hazard curve.

NOTE The structure's response can be very different for two ground motion records having the same spectral acceleration at the dominant period, typically the 1st sway modes i.e.  $S_a(T_1)$ , if the frequency content for the two records is sufficiently different. A large magnitude earthquake with a focus far from the site can have the same  $S_a(T_1)$  as a smaller magnitude earthquake with a focus close to the site, however the ground motion records for the earthquake far from the site typically has less high frequency motions as they are attenuated by the larger site to source distance.

**7.2.3** Down-pile displacement time histories (derived from a DSRA) **should** be applied (in conformance with [47](#)), rather than applying near-surface displacement time histories uniformly from pile tip to pile head.

**7.2.4** Soil radiation damping **should** be represented in a nonlinear time history analysis in conformance with IOGP S-631-11 A.11.6.4.

**7.2.5** Ground motions **may** be scaled based on the conditional mean spectrum rather than the uniform hazard spectrum.

**7.2.6** Limit state verification for the DL limit state by a linear time history analysis **shall** use a nonlinear soil model.

**7.2.7** A free vibration analysis following an initial prescribed lateral deck deflection **should** be performed, prior to the time history response, to verify the time-domain model, sway periods and damping.

**7.2.8** Limit state verification for the DL limit state **shall** be demonstrated when using the time history analysis, if URs for all members and all connections are less than unity at each time step.

NOTE For the ELE the maximum member or joint utilization is taken during each time history record to assess the component performance.

**7.2.9** Time history analysis and the structural nonlinear model for the hazardous event actions for the NC limit state **shall** be in conformance with the criteria specified in ISO 19902 and IOGP S-631-11.

**7.2.10** NC limit state verification of the as-designed structure **shall** be deemed to have been demonstrated provided that no more than 3 of 7 ground motion records (each having a spectral acceleration at the sway period of the structure defined by  $S_{a,ALE}$  cause collapse of the structure in a nonlinear time-history analysis.

NOTE for limit state verification, if no more than 3 of the 7 time histories analysis result in collapse of the structure then the annual probability of collapse is no greater than  $1/RP$  where  $RP$  is the return period for the  $S_a(T_{dom})$  of the 7-time history events.

**7.2.11** Time history analysis **shall** be used to obtain deck motions and deck motion response spectra.

NOTE Typically deck displacement is relative to the ground motion displacement while deck acceleration spectra are based on the total deck acceleration (i.e., the relative acceleration plus the ground motion acceleration).

**7.2.12** Time history analysis **shall** be performed for a minimum of an additional 15 seconds after the end of the seismic motions [18](#).

NOTE An additional 15 seconds allows the platform to come to rest and allows the residual tilt or permanent displacement to be determined.

### 7.3 Nonlinear pushover analysis

**7.3.1** Limit state verification for the NC limit state using the nonlinear pushover analysis **shall** be performed in conformance with [A.7.3](#)

**7.3.2** Limit state verification for the NC limit state for facilities with compliant or soft-link systems, (e.g., tethers on a tension leg platform (TLP), or portal action of TLP foundations subjected to lateral actions) **shall** be performed using a pushover analysis with displacement-control.

NOTE the response of the compliant or soft-link systems are simulated by nonlinear geometry and nonlinear material while the hull is represented using small strain and linear material.

**7.3.3** Nonlinear pushover analysis **shall not** be used if seismic isolation or passive energy dissipation devices are employed.

**7.3.4** NC verification of the as-designed structure **shall** be deemed to have been demonstrated provided that no more than 3 of 7 of the deck displacements (from a SDOF time-history analysis of 7 ground motion records, each ground motion having a spectral acceleration at the sway period of the structure defined by  $S_{a,ALE}$ ) causes collapse of the structure in a nonlinear pushover analysis.

NOTE for further information on nonlinear pushover analysis see ISO 19902:2020 (11.6.3).

## 8 Simplified procedure for determining $S_{a,ELE}$ and $S_{a,ALE}$

### 8.1 General

**8.1.1** The following steps **shall** be followed in sequence to determine  $S_{a,ELE}$  and  $S_{a,ALE}$  based on accelerations obtained from seismic maps.

### 8.2 Spectral accelerations

**8.2.1** Rock outcrop spectral accelerations at oscillator periods of 0,2 s and 1,0 s,  $S_{a,map}(0,2)$  and  $S_{a,map}(1,0)$  **shall** be obtained from seismic maps in [Annex B](#) (ISO maps) or [Annex C](#) (Regional maps), or from regional seismic maps supplied by the operator.

### 8.3 Site class

**8.3.1** The designer **shall** determine site class (A to F) from the shear wave velocities in [Table 5](#) based on the seabed soils and the average properties of the effective seabed.

**8.3.2** The effective seabed **should** be taken as the top 30 m of the seabed. Alternatively, rational analysis **may** be used to define the effective seabed. Site specific study **should** be used if soft soils are present below 30 m.

**8.3.3** Shear wave velocity **shall** be determined through field measurements.

NOTE Measured soil shear wave velocity can depend on the type of measurement used in situ or in the laboratory; see ISO 19901-8 for additional guidance.

**8.3.4** The average of the representative shear wave velocity in a 30 m deep effective seabed ( $\bar{v}_s$ ) **shall** be determined from [Formula \(1\)](#):

$$\bar{v}_s = 30 / \sum_{i=1}^n \frac{d_i}{v_{s,i}} \quad (1)$$

where:

$n$  is the number of distinct soil layers in the effective seabed;

$d_i$  is the thickness of layer,  $i$ ;

$\bar{v}_{s,i}$  is the representative shear wave velocity of layer,  $i$ .

NOTE For sites with less than 30 m to bedrock or high stiffness contrast within top 30 m, a site response analysis typically produces more reliable results than [formula \(1\)](#).

**8.3.5** Similarly, the average of the representative values of normalized cone penetration resistance ( $\bar{q}_{cl}$ ) or soil undrained shear strength ( $\bar{s}_u$ ) **shall** be determined according to [Formula \(1\)](#), where  $\bar{v}_s$  is replaced by  $q_{cl}$  or  $s_u$ , respectively.

## 8.4 Site correction factor coefficients

**8.4.1** For shallow foundations, the site coefficient,  $C_a$  **shall** be determined from [Table 6](#) based on site class and the mapped 0,2 s spectral accelerations,  $S_{a,map}(0,2)$  and

**8.4.2** For shallow foundations, the site coefficient,  $C_v$  **shall** be determined from [Table 7](#) based on site class and the mapped 1,0 s spectral accelerations,  $S_{a,map}(1,0)$  [46 Seyhan].

**8.4.3** For structures in SRC 4,  $C_v$  **shall** be based on site specific study.

Table 5 — Determination of site class

| Site class                                                                                                                                                                                                               | Soil profile name                                | Average properties in the effective seabed                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                  |                                                                                        |                                                                      |
|--------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|--------------------------------------------------|-------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|----------------------------------------------------------------------------------------|----------------------------------------------------------------------|
|                                                                                                                                                                                                                          |                                                  | Soil shear wave velocity<br>$\bar{v}_s$<br><br>m/s                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                          | Cohesionless:<br>normalized cone penetration resistance<br>$\bar{q}_{cl}$ <sup>a</sup> | Cohesive:<br>soil undrained shear strength<br>$\bar{s}_u$<br><br>kPa |
| A/B                                                                                                                                                                                                                      | Hard rock/rock, thickness of soft sediments <5 m | $\bar{v}_s > 750$                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                           | Not applicable                                                                         | Not applicable                                                       |
| C                                                                                                                                                                                                                        | Very dense hard soil and soft rock               | $350 < \bar{v}_s \leq 750$                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                  | $\bar{q}_{cl} \geq 200$                                                                | $\bar{s}_u \geq 200$                                                 |
| D                                                                                                                                                                                                                        | Stiff to very stiff soil                         | $180 < \bar{v}_s \leq 350$                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                  | $80 \leq \bar{q}_{cl} < 200$                                                           | $80 \leq \bar{s}_u < 200$                                            |
| E                                                                                                                                                                                                                        | Soft to firm soil                                | $120 < \bar{v}_s \leq 180$                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                  | $\bar{q}_{cl} < 80$                                                                    | $\bar{s}_u < 80$                                                     |
| F                                                                                                                                                                                                                        | —                                                | Any profile, including those otherwise classified as A to E, containing soils having one or more of the following characteristics:<br>a) $\bar{v}_s \leq 120$<br>b) soils vulnerable to potential failure or collapse under seismic actions such as liquefiable soils, highly sensitive clays, collapsible weakly cemented soils<br>c) ooze <sup>b</sup> with a thickness of more than 10 m<br>d) soil layers with high gas content or ambient excess pore pressure greater than 30 % of in situ effective overburden<br>e) layers greater than 2 m thick with sharp contrast in shear wave velocity (greater than ±30 %) and/or undrained shear strength (greater than ±50 %) compared to adjacent layers. |                                                                                        |                                                                      |
| <sup>a</sup> $q_{cl} = (q_c/p_a) \times (p_a/\sigma'_{v0})^{0,5}$<br>where:<br>$q_c$ is the cone penetration resistance.<br>$p_a$ is atmospheric pressure = 100 kPa.<br>$\sigma'_{v0}$ is the vertical effective stress. |                                                  |                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                             |                                                                                        |                                                                      |
| <sup>b</sup> Clay containing more than 30 % calcareous or siliceous material of biogenic origin.                                                                                                                         |                                                  |                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                             |                                                                                        |                                                                      |

**8.4.4** For deep pile foundations, the site coefficients  $C_a$  and  $C_v$  **shall** be determined from [Table 8](#) based on the site class alone.

**8.4.5** For SRC 4 structures,  $C_v$  **shall** be based on site specific study.

**8.4.6** For structures with other SRCs where site specific studies are performed, those  $C_a$  and  $C_v$  values **shall** supersede the values in [Table 8](#).

**8.4.7** For all foundations with site class F, a site-specific marine soil investigation and dynamic site response **shall** be performed.

**Table 6 — Values of  $C_a$  for shallow foundations and 0,2 s period spectral acceleration**

| Site class | $S_{a,map}(0,2)$ |        |        |       |        |              |
|------------|------------------|--------|--------|-------|--------|--------------|
|            | $\leq 0,25$ g    | 0,50 g | 0,75 g | 1,0 g | 1,25 g | $\geq 1,5$ g |
| A/B        | 0,9              | 0,9    | 0,9    | 0,9   | 0,9    | 0,9          |
| C          | 1,3              | 1,3    | 1,2    | 1,2   | 1,2    | 1,2          |
| D          | 1,6              | 1,4    | 1,2    | 1,1   | 1,0    | 1,0          |
| E          | 2,4              | 1,7    | 1,3    | 1,1   | 1,0    | 0,8          |
| F          | —                | —      | —      | —     | —      | —            |

**Table 7 — Values of  $C_v$  for shallow foundations and 1,0 s period spectral acceleration**

| Site class | $S_{a,map}(1,0)$ |       |       |       |       |              |
|------------|------------------|-------|-------|-------|-------|--------------|
|            | $\leq 0,1$ g     | 0,2 g | 0,3 g | 0,4 g | 0,5 g | $\geq 0,6$ g |
| A/B        | 0,8              | 0,8   | 0,8   | 0,8   | 0,8   | 0,8          |
| C          | 1,5              | 1,5   | 1,5   | 1,5   | 1,5   | 1,4          |
| D          | 2,4              | 2,2   | 2,0   | 1,9   | 1,8   | 1,7          |
| E          | 4,2              | 3,3   | 2,8   | 2,4   | 2,2   | 2,0          |
| F          | —                | —     | —     | —     | —     | —            |

**Table 8 — Values of  $C_a$  and  $C_v$  for deep pile foundations**

| Site class | $C_a$ | $C_v$ |
|------------|-------|-------|
| A/B        | 1,0   | 0,8   |
| C          | 1,0   | 1,0   |
| D          | 1,0   | 1,2   |
| E          | 1,0   | 1,8   |
| F          | —     | —     |

## 8.5 1 000-year horizontal acceleration spectrum

**8.5.1** A seismic acceleration spectrum,  $S_{a,site}(T)$ , **shall** be created for different oscillator periods,  $T$ , as shown in [Figure 4](#).

**8.5.2** For periods,  $T$ , less than or equal to 0,2 s, the site spectral acceleration,  $S_{a,site}(T)$ , **shall** be determined from [Formula \(2\)](#):

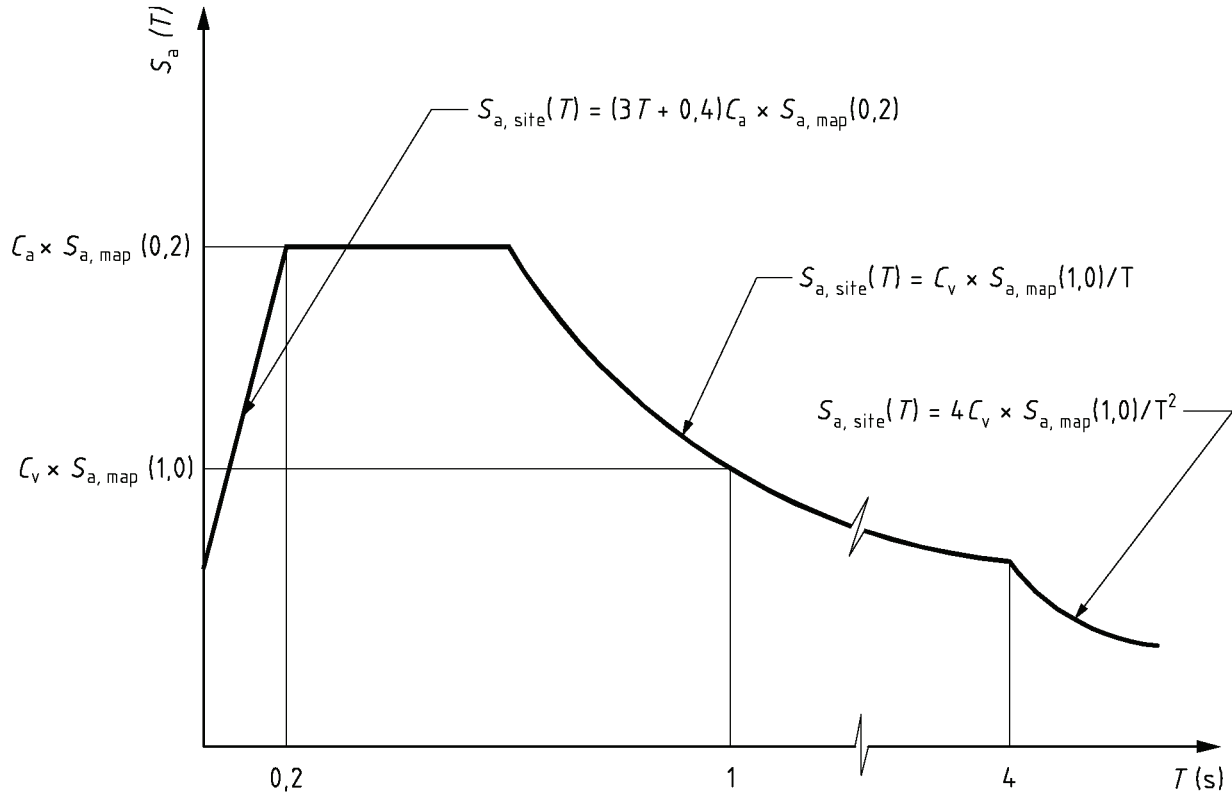
$$S_{a,site}(T) = (3T + 0,4) C_a \times S_{a,map}(0,2) \quad (2)$$

**8.5.3** For periods greater than 0,2 s and less than or equal to 4,0 s, the site spectral acceleration,  $S_{a,site}(T)$ , decays in proportion to  $1/T$ , and **shall** be determined from [Formula \(3\)](#):

$$S_{a,site}(T) = \frac{C_v \times S_{a,map}(1,0)}{T} \text{ except } S_{a,site}(T) \leq C_a \times S_{a,map}(0,2) \quad (3)$$

**8.5.4** For periods greater than 4,0 s, the site spectral acceleration,  $S_{a,site}(T)$ , decays in proportion to  $1/T^2$  and **shall** be determined from [Formula \(4\)](#):

$$S_{a,site}(T) = \frac{4C_v \times S_{a,map}(1,0)}{T^2} \quad (4)$$



#### Key

|                  |                                                                                                                                                                 |
|------------------|-----------------------------------------------------------------------------------------------------------------------------------------------------------------|
| $T$              | natural period of a simple, single degree of freedom oscillator                                                                                                 |
| $C_a, C_v$       | site coefficients                                                                                                                                               |
| $S_a(T)$         | spectral acceleration                                                                                                                                           |
| $S_{a,site}(T)$  | site spectral acceleration corresponding to a return period of 1 000 years and a single degree of freedom oscillator period, $T$                                |
| $S_{a,map}(0,2)$ | 1 000-year rock outcrop spectral acceleration obtained from maps in <a href="#">Annex B</a> associated with a single degree of freedom oscillator period, 0,2 s |
| $S_{a,map}(1,0)$ | 1 000-year rock outcrop spectral acceleration obtained from maps in <a href="#">Annex B</a> associated with a single degree of freedom oscillator period, 1,0 s |

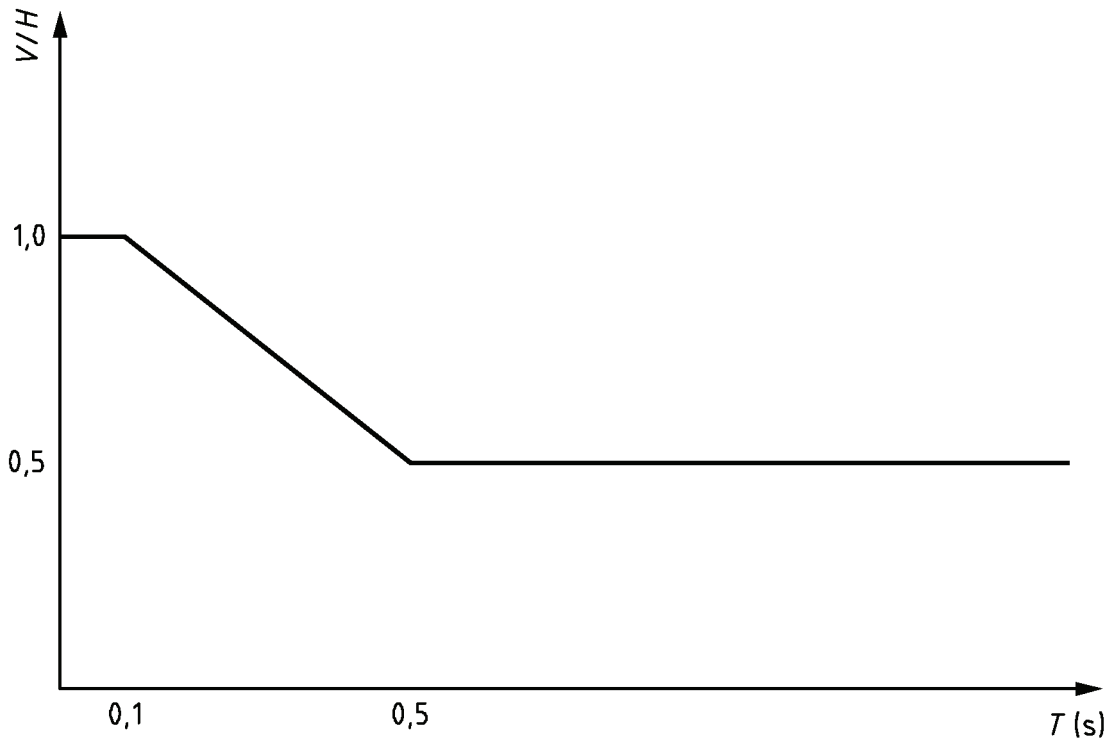
**Figure 4 — Seismic acceleration design response spectrum for 5 % damping**

## 8.6 1 000-year vertical acceleration spectrum

**8.6.1** For seismic zones 0, 1 and 2, the vertical spectral acceleration at a period,  $T$ , **shall** be taken to be equal to half the corresponding horizontal spectral acceleration.

**8.6.2** For seismic zones 3 and 4, the vertical spectral acceleration at a period,  $T$ , **shall** be taken to be equal to the product of the amplification function in [Figure 5](#) multiplied by the corresponding horizontal acceleration spectral value (see Reference [37 Boz & Camp]).

**8.6.3** The vertical spectrum **shall** not be reduced further due to water depth effects.



**Key**

$T$  natural period of a simple, single degree of freedom oscillator  
 $V/H$  amplification function applied to horizontal spectral accelerations to derive vertical spectral accelerations

**Figure 5 — Vertical to horizontal spectral amplification function for seismic zones 3 and 4**

## 8.7 Damping adjustment

To obtain acceleration spectra corresponding to damping values other than 5 %, the acceleration spectra **may** be multiplied by a scale correction factor,  $D$ , as shown in [Formula \(5\)](#):

$$D = \frac{\ln(100/\eta)}{\ln(20)} \quad (5)$$

where  $\eta$  is the per cent of critical damping.

NOTE The acceleration spectra obtained in [8.5](#) and [8.6](#) correspond to 5 % damping.

## 8.8 Determining $S_{a,ELE}$ and $S_{a,ALE}$

**8.8.1** The hazardous event actions (horizontal and vertical) for limit state verification of the NC limit state **shall** be calculated by applying [Formula \(6\)](#):

$$S_{a,ALE}(T) = N_{ALE} \times S_{a,site}(T) \quad (6)$$

where:

$N_{ALE}$  is the scale factor from [Table 9](#), based on the consequence level.  
 $S_{a,site}(T)$  is the site 1 000-year spectral acceleration ([8.5](#), [8.6](#)), potentially adjusted for damping ([8.7](#)).

**Table 9 — Scale factors from  $S_{a,site}(T)$  to  $S_{a,ALE}(T)$** 

| Consequence level | scale factor $N_{ALE}$ |
|-------------------|------------------------|
| L1                | 1,60                   |
| L2                | 1,15                   |
| L3                | 0,85                   |

**8.8.2** The ELE horizontal and vertical spectral accelerations at oscillator period,  $T$ , **shall** be obtained from Formula (7):

$$S_{a,ELE}(T) = \frac{S_{a,ALE}(T)}{C_r} \quad (7)$$

where  $C_r$  is a factor representing the ratio of  $S_{a,ALE}(T_{dom})$  to  $S_{a,ELE}(T_{dom})$ . Refer to [A.9.5](#) for more details.

NOTE Values of  $C_r$  for fixed steel and concrete structures are presented in ISO 19902<sup>[6]</sup> and ISO 19903<sup>[7]</sup> respectively. See also requirements for  $C_r$  for steel fixed structures in IOGP S-631-11.

**8.8.3** To avoid return periods for  $S_{a,ELE}$  that are too short,  $C_r$  values **shall** not exceed the values given in ISO 19902 for steel structures and ISO 19903 for concrete structures.

NOTE Estimating the value of  $C_r$  prior to the structural design can help achieve an economic design that will resist damage due to an ELE and also likely meet the ALE performance requirements.

**8.8.4** Values of  $C_r$  **may** be determined by either:

- a) a nonlinear pushover analysis, or
- b) using  $C_r$  determined from prior detailed assessment of a similar structure.

**8.8.4** Values of  $C_r$  **may** be justified by prior detailed assessment of similar structures.

**8.8.5** ALE analysis **may** be used to verify  $C_r$  values other than those specified.

## 9 Detailed procedure for determining $S_{a,ELE}$ and $S_{a,ALE}$

### 9.1 Probabilistic seismic hazard analysis (PSHA)

**9.1.1** Probabilistic ground motions at a site **shall** be based on the probability of earthquakes of different sizes, on all potential sources (faults or areas), that can affect the site, see [Figure 6 a](#)).

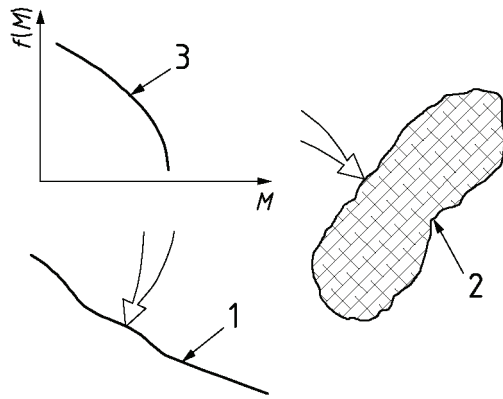
**9.1.2** A PSHA **shall** account for the randomness in attenuation of seismic waves travelling from a source to the site, see [Figure 6 b](#)).

**9.1.3** A PSHA **shall** include uncertainty in the definition of input parameters.

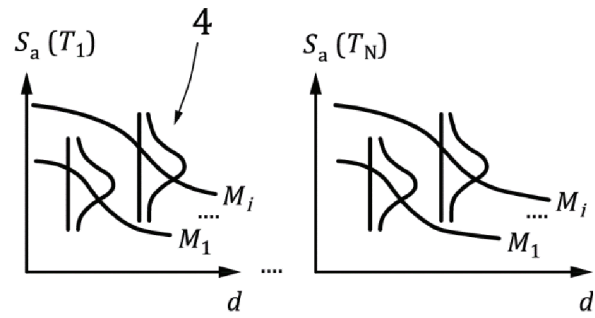
NOTE A probability-based approach considers the maximum magnitude for a given source, the magnitude recurrence relationship, the attenuation equation, and geographical boundaries defining the location of a source zone.

**9.1.4** Individual probabilities **shall** be summated from different sources to provide total annual probability of exceedance for a given level of peak ground acceleration (PGA) or spectral acceleration, see [Figure 6 c](#)).

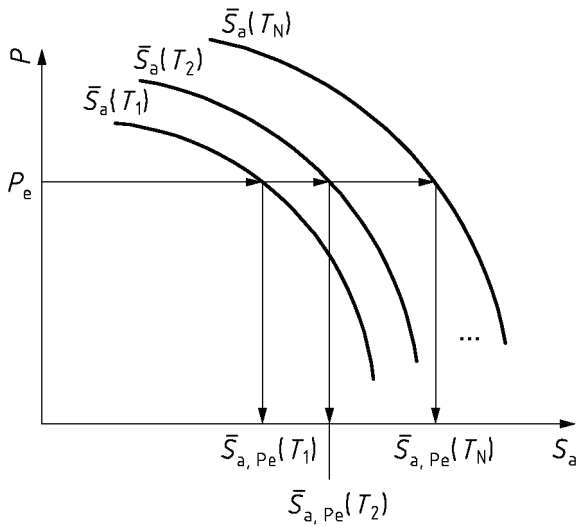
NOTE The curve of probability of exceedance versus ground motion or response of the single degree of freedom oscillator (e.g., spectral acceleration, spectral velocity, or spectral displacement) is often referred to as a “hazard curve”.



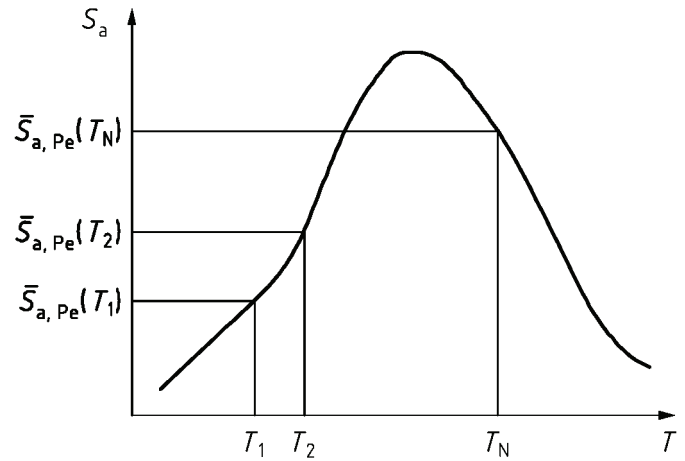
a) Earthquake source seismicity and geometry



b) Definition of attenuation curves for spectral accelerations at periods,  $T_1 \dots T_N$



c) Mean hazard curves for spectral accelerations at each period and spectral accelerations with selected annual probability of exceedance:  
 $S_{a,Pe}(T_1) \dots S_{a,Pe}(T_N)$



d) Uniform hazard spectrum of spectral accelerations at the selected annual probability of exceedance, developed from c)

#### Key

- 1 line source (fault)
- 2 area source
- 3 cumulative annual frequency of magnitude,  $M$
- 4 attenuation uncertainty
- $M$  magnitude
- $M_i$  magnitude,  $i$

- $P_e$  annual probability of exceedance
- $f(M)$  frequency
- $T_i$  single degree of freedom oscillator periods
- $D$  distance from source
- $P_e$  annual probability of exceedance
- $S_a(T_i)$  spectral acceleration associated with a single degree of freedom oscillator period,  $T_i$
- $S_{a,Pe}(T_i)$  spectral acceleration for oscillator period,  $T_i$ , at selected annual probability of exceedance

Figure 6 — Probabilistic seismic hazard analysis overview

**9.1.5** A series of hazard curves for different periods,  $T$ , **shall** be derived based on spectral response variation with the natural period of the oscillator [see [Figure 6 c](#)].

NOTE The results from a PSHA are a series of hazard curves each for a spectral acceleration corresponding to a structure natural period, e.g.,  $T_1, T_2, \dots, T_N$  [see [Figure 6 c](#)]. Because of uncertainties in PSHA input parameters, each of these hazard curves has an uncertainty band.

**9.1.6** The results from a PSHA **shall** be used to derive a uniform hazard spectrum [see [Figure 6 d](#)], where all points on the spectrum correspond to the same annual probability of exceedance.

**9.1.7** The mean (i.e., expected value) of each hazard curve **should** be used to construct a uniform hazard spectrum corresponding to a given exceedance probability [see [Figure 6 d](#)].

**9.1.8** The return period of a uniform hazard spectrum is the inverse of the annual probability of exceedance, as shown in [Formula \(8\)](#):

$$T_{\text{return}} = 1 / P_e \quad (8)$$

where  $T_{\text{return}}$  is the return period in years, and  $P_e$  is the annual probability of exceedance.

**9.1.9** Spectral accelerations for natural periods other than  $T_{\text{dom}}$  (i.e.  $S_{a,ELE}$  and  $S_{a,ALE}$ ) **may** be determined from the PSHA results.

**9.1.10** Baseline corrections to acceleration time histories **shall** be applied.

NOTE baseline correction removes trends in velocity and displacement time histories when an acceleration record is integrated. Baseline corrected time histories are essential if the ground motions are applied to the structural model using prescribed displacement time history.

**9.1.11** The signal processing for baseline corrections:

- a) **should** add zero pads of sufficient length to the beginning and end of the time series to prevent filter transients (wrap-around effects) from being introduced into the record during filtering.
- b) **should** use a bandpass Butterworth filter.

NOTE bandpass Butterworth filters are flat at the maximum response in the passband while decaying rapidly in the transition zone.

- c) **should** use a bidirectional filter to result in zero-phase shift.

## 9.2 Deterministic seismic hazard analysis (DSHA)

**9.2.1** Deterministic ground motion extremes at a site **shall** be based on a single hazardous event with specified magnitude and distance from the site.

NOTE Deterministic ground motion estimates are not associated with a specific return period, such as 1 000 years, although the single hazardous event used can have a return period associated with it. The return period for the hazardous event with the maximum magnitude on a given fault can vary from several hundred to several thousand years, depending on the activity rate of the fault.

**9.2.2** Deterministic analysis **shall** be based on:

- a) definition of an earthquake source (e.g., a known fault) and its location relative to the site, and
- b) definition of an earthquake magnitude that the source can produce, and
- c) a relationship which describes the attenuation of ground motion with distance.