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**Petroleum and natural gas industries —  
Specific requirements for offshore  
structures —**

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

*Industries du pétrole et du gaz naturel — Exigences spécifiques  
relatives aux structures en mer —*

*Partie 2: Procédures de conception et critères sismiques*

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## Foreword

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

International Standards are drafted in accordance with the rules given in the ISO/IEC Directives, Part 2.

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

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

ISO 19901-2 was prepared by Technical Committee ISO/TC 67, *Materials, equipment and offshore structures for petroleum, petrochemical and natural gas industries*, Subcommittee SC 7, *Offshore structures*.

ISO 19901 consists of the following parts, under the general title *Petroleum and natural gas industries — Specific requirements for offshore structures*:

- *Part 1: Metocean design and operating considerations*
- *Part 2: Seismic design procedures and criteria*
- *Part 4: Geotechnical and foundation design considerations*
- *Part 5: Weight control during engineering and construction*
- *Part 7: Stationkeeping systems for floating offshore structures and mobile offshore units*

The following parts of ISO 19901 are under preparation:

- *Part 3: Topsides structure*
- *Part 6: Marine operations*

ISO 19901 is one of a series of standards for offshore structures. The full series consists of the following International Standards.

- ISO 19900, *Petroleum and natural gas industries — General requirements for offshore structures*
- ISO 19901 (all parts), *Petroleum and natural gas industries — Specific requirements for offshore structures*
- ISO 19902, *Petroleum and natural gas industries — Fixed steel offshore structures*
- ISO 19903, *Petroleum and natural gas industries — Fixed concrete offshore structures*
- ISO 19904-1, *Petroleum and natural gas industries — Floating offshore structures — Part 1: Monohulls, semi-submersibles and spars*

- ISO 19904-2, *Petroleum and natural gas industries — Floating offshore structures — Part 2: Tension leg platforms*
- ISO 19905-1, *Petroleum and natural gas industries — Site-specific assessment of mobile offshore units — Part 1: Jack-ups*
- ISO/TR 19905-2, *Petroleum and natural gas industries — Site-specific assessment of mobile offshore units — Part 2: Jack-ups commentary*
- ISO 19906, *Petroleum and natural gas industries — Arctic offshore structures*

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## Introduction

The series of International Standards applicable to types of offshore structure, ISO 19900 to ISO 19906, constitutes a common basis covering those aspects that address design requirements and assessments of all offshore structures used by the petroleum and natural gas industries worldwide. Through their application, the intention is to achieve reliability levels appropriate for manned and unmanned offshore structures, whatever the nature or combination of the materials used.

It is important to recognize that structural integrity is an overall concept comprising models for describing actions, structural analyses, design rules, safety elements, workmanship, quality control procedures and national requirements, all of which are mutually dependent. The modification of one aspect of design 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 series of International Standards applicable to types of offshore structure is 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 platform, 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. In many cases, site-specific seismic hazard assessments will be required to complete the design or assessment of a structure.

This part of ISO 19901 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 standard ISO 19900 and within the structure-specific standards, ISO 19902, ISO 19903, ISO 19904 and ISO 19905. The consideration of seismic events in connection with mobile offshore units is addressed in ISO 19905.

Some background to and guidance on the use of this part of ISO 19901 is provided in informative Annex A. The clause numbering in Annex A is the same as in the normative text to facilitate cross-referencing.

Regional information on expected seismic accelerations for offshore areas is provided in informative Annex B.

# Petroleum and natural gas industries — Specific requirements for offshore structures —

## Part 2: Seismic design procedures and criteria

### 1 Scope

This part of ISO 19901 contains requirements for defining the seismic design procedures and criteria for offshore structures; guidance on the requirements is included in Annex A. The requirements are applicable to fixed steel structures and fixed concrete structures. The effects of seismic events on floating structures and partially buoyant structures are also briefly discussed. The site-specific assessment of jack-ups in elevated condition is only covered in this part of ISO 19901 to the extent that the requirements are applicable.

Only earthquake-induced ground motions are addressed in detail. Other geologically induced hazards such as liquefaction, slope instability, faults, tsunamis, mud volcanoes and shock waves are mentioned and briefly discussed.

The requirements are intended to reduce risks to persons, the environment, and assets to the lowest levels that are reasonably practicable. This intent is achieved by using:

- a) seismic design procedures which are dependent on the platform's exposure level and the expected intensity of seismic events;
- b) a two-level seismic design check in which the structure is designed to the ultimate limit state (ULS) for strength and stiffness and then checked to abnormal environmental events or the accidental limit state (ALS) to ensure that it meets reserve strength and energy dissipation requirements.

For high seismic areas and/or high exposure level fixed structures, a site-specific seismic hazard assessment is required; for such cases, the procedures and requirements for a site-specific probabilistic seismic hazard analysis (PSHA) are addressed. However, a thorough explanation of PSHA procedures is not included.

Where a simplified design approach is allowed, worldwide offshore maps are included in Annex B that show the intensity of ground shaking corresponding to a return period of 1 000 years. In such cases, these maps may be used with corresponding scale factors to determine appropriate seismic actions for the design of a structure.

**NOTE** For design of fixed steel offshore structures, further specific requirements and recommended values of design parameters (e.g. partial action and resistance factors) are included in ISO 19902, while those for fixed concrete offshore structures are contained in ISO 19903. Specific seismic requirements for floating structures are to be contained in ISO 19904<sup>[2]</sup>, for site-specific assessment of jack-ups and other MOUs in ISO 19905<sup>[3]</sup>, for arctic structures in ISO 19906<sup>[4]</sup> and for topsides structures in ISO 19901-3<sup>[1]</sup>.

### 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 19900, *Petroleum and natural gas industries — General requirements for offshore structures*

ISO 19902<sup>1)</sup>, *Petroleum and natural gas industries — Fixed steel offshore structures*

ISO 19903<sup>1)</sup>, *Petroleum and natural gas industries — Fixed concrete offshore structures*

### 3 Terms and definitions

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

#### 3.1

##### **abnormal level earthquake**

###### **ALE**

intense earthquake of abnormal severity under the action of which the structure should not suffer complete loss of integrity

NOTE The ALE event is comparable to the abnormal event in the design of fixed structures which are described in ISO 19902 and ISO 19903. When exposed to the ALE, a manned structure is supposed to maintain structural and/or floatation integrity for a sufficient period of time to enable evacuation to take place.

#### 3.2

##### **attenuation**

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

#### 3.3

##### **directional combination**

combination of response values due to each of the three orthogonal components of an earthquake motion

#### 3.4

##### **escape and evacuation systems**

systems provided on a platform to facilitate escape and evacuation in an emergency

NOTE Escape and evacuation systems include passageways, chutes, ladders, life rafts and helidecks.

#### 3.5

##### **extreme level earthquake**

###### **ELE**

earthquake with a severity which the structure should sustain without major damage

NOTE The ELE event is comparable to the extreme environmental event in the design of fixed structures which are described in ISO 19902 and ISO 19903. When exposed to an ELE, a structure is supposed to retain its full capacity for all subsequent conditions.

#### 3.6

##### **fault movement**

movement occurring on a fault during an earthquake

#### 3.7

##### **ground motions**

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

NOTE A fixed offshore structure is founded in or on the seabed and consequently only seabed motions are of significance. The term ground motions is used rather than seabed motions for consistency of terminology with seismic design for onshore structures.

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1) To be published.

**3.8****liquefaction**

fluidity of cohesionless 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 volcanoes**

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

NOTE 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 characteristics

**3.12****probability of exceedance**

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

EXAMPLES Examples of probabilities of exceedance during a given exposure time are the annual probability of exceedance of a specified magnitude of ground acceleration, ground velocity or ground displacement.

**3.13****response spectrum**

plot representing structural response in terms of absolute acceleration, pseudo velocity, or relative displacement values against natural frequency or period

**3.14****safety systems**

systems provided on a platform to detect, control and mitigate hazardous situations

NOTE Safety systems include gas detection, emergency shutdown, fire protection, and their control systems.

**3.15****sea floor**

interface between the sea and the seabed

**3.16****sea floor slide**

failure of sea floor slopes

**3.17****seabed**

materials below the sea in which a structure is founded

NOTE The seabed can be considered as the half-space below the sea floor.

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

category defined from the exposure level and the expected intensity of seismic motions

**3.19**

**seismic hazard curve**

curve showing the probability of exceedance against a measure of seismic intensity

NOTE The seismic intensity measures can include parameters such as peak ground acceleration, spectral acceleration, or spectral velocity.

**3.20**

**seismic reserve capacity factor**

ratio of spectral acceleration which causes structural collapse or catastrophic system failure to the ELE spectral acceleration

**3.21**

**site response analysis**

wave propagation analysis permitting the evaluation of the effect of local geological and soil conditions on the design ground motions at a given site

NOTE The site response analysis results can include amplitude, frequency content and duration.

**3.22**

**spectral acceleration**

maximum absolute acceleration response of a single degree of freedom oscillator subjected to ground motions due to an earthquake

**3.23**

**spectral velocity**

maximum pseudo velocity response of a single degree of freedom oscillator subjected to ground motions due to an earthquake

**3.24**

**spectral displacement**

maximum relative displacement response of a single degree of freedom oscillator subjected to ground motions due to an earthquake

**3.25**

**static pushover method**

**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

NOTE The vertical movement of the sea floor is often associated with fault rupture during earthquakes or with seabed mud slides.

**4 Symbols and abbreviated terms**

**4.1 Symbols**

$a_R$  slope of the seismic hazard curve

$C_a$  site coefficient, a correction factor applied to the acceleration part 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 Equation (7)
$C_v$	site coefficient, a correction factor applied to the velocity part of a response spectrum
$c_u$	undrained shear strength of the soil
$\bar{c}_u$	average undrained shear strength of the soil of the top 30 m of the seabed
$D$	scaling factor for damping
$G_{\max}$	low amplitude shear modulus of the soil
$g$	acceleration due to gravity (9,81 m/s <sup>2</sup> )
$M$	magnitude of a given seismic 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 sand
$q_{cl}$	normalized cone penetration resistance of sand
$\bar{q}_{cl}$	average normalized cone penetration resistance of sand of the top 30 m of the seabed
$S_a(T)$	spectral acceleration associated with a single degree of freedom oscillator period $T$
$\bar{S}_a(T)$	mean spectral acceleration associated with a single degree of freedom oscillator period $T$ ; obtained from a PSHA
$S_{a,\text{ALE}}(T)$	ALE spectral acceleration associated with a single degree of freedom oscillator period $T$
$\bar{S}_{a,\text{ALE}}(T)$	mean ALE spectral acceleration associated with a single degree of freedom oscillator period $T$ ; obtained from a PSHA
$S_{a,\text{ELE}}(T)$	ELE spectral acceleration associated with a single degree of freedom oscillator period $T$
$\bar{S}_{a,\text{ELE}}(T)$	mean ELE spectral acceleration associated with a single degree of freedom oscillator period $T$ ; obtained from a PSHA
$S_{a,\text{map}}(T)$	1 000 year rock outcrop spectral acceleration obtained from maps associated with a single degree of freedom oscillator period $T$
	NOTE The maps included in Annex B are for oscillator periods of 0,2 s and 1,0 s.
$\bar{S}_{a,P_e}(T)$	mean spectral acceleration associated with a probability of exceedance $P_e$ and a single degree of freedom oscillator period $T$ ; obtained from a PSHA
$\bar{S}_{a,P_f}(T)$	mean spectral acceleration 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 $T$
$T$	natural period of a simple, single degree of freedom oscillator
$T_{dom}$	dominant modal period of the structure
$T_{return}$	return period
$u_i$	code utilization in time history analysis $i$
$\hat{u}$	median code utilization
$v_s$	shear wave velocity
$\bar{v}_s$	average shear wave velocity of the top 30 m of the seabed
$\rho$	mass density of soil
$\eta$	percent of critical damping
$\sigma_{LR}$	logarithmic standard deviation of uncertainties not captured in a seismic hazard curve
$\sigma_{v0}$	vertical effective stress of soil

#### 4.2 Abbreviated terms

ALE	abnormal level earthquake
ALS	accidental limit state
ELE	extreme level earthquake
L1, L2, L3	exposure level derived in accordance with the International Standard applicable to the type of offshore structure <sup>2)</sup>
MOU	mobile offshore unit
PGA	peak ground acceleration
PSHA	probabilistic seismic hazard analysis
SRC	seismic risk category
TLP	tension leg platform
ULS	ultimate limit state

### 5 Earthquake hazards

Actions and action effects due to seismic events shall be considered in the structural design of offshore structures in seismically active areas. Areas are considered seismically active on the basis of previous records of earthquake activity, both in frequency of occurrence and in magnitude. Annex B provides maps indicative of seismic accelerations, however for many areas, depending on indicative accelerations and exposure levels, seismicity shall be determined on the basis of detailed investigations, see 6.5.

2) International Standards applicable to types of offshore structure, include ISO 19902 and ISO 19903, and when available, ISO 19904 (all parts), ISO 19905 (all parts) and ISO 19906. See the Bibliography.

Consideration of seismic events for seismically active regions shall include investigation of the characteristics of ground motions and the acceptable seismic risk for structures. Structures in seismically active regions shall be designed for ground motions due to earthquakes. However, other seismic hazards shall also be considered in the design and should be addressed by special studies. The following hazards can be caused by a seismic event:

- soil liquefaction;
- sea floor slide;
- fault movement;
- tsunamis;
- mud volcanoes;
- shock waves.

Effects of seismic events on subsea equipment, pipelines and in-field flowlines shall be addressed by special studies.

## 6 Seismic design principles and methodology

### 6.1 Design principles

Clause 6 addresses the design of structures against base excitations, i.e. accelerations, velocities and displacements caused by ground motions.

Structures located in seismically active areas shall be designed for the ultimate limit state (ULS), abnormal environmental events and the accidental limit state (ALS) using different levels of earthquake.

The ULS requirements are intended to provide a structure which is adequately sized for strength and stiffness to ensure that no significant structural damage occurs for a level of earthquake ground motion with an adequately low likelihood of being exceeded during the design service life of the structure. The seismic ULS design event is the extreme level earthquake (ELE). The structure shall be designed such that an ELE event will cause little or no damage. Shutdown of production operations is tolerable and the structure should be inspected subsequent to an ELE occurrence.

The ALS requirements are intended to ensure that the structure and foundation have sufficient reserve strength, displacement and/or energy dissipation capacity to sustain large inelastic displacement reversals without complete loss of integrity, although structural damage can occur. The seismic ALS design event is the abnormal level earthquake (ALE). The ALE is an intense earthquake of abnormal severity with a very low probability of occurring during the structure's design service life. The ALE can cause considerable damage to the structure, however, the structure shall be designed such that overall structural integrity is maintained to avoid structural collapse causing loss of life and/or major environmental damage.

Both ELE and ALE return periods depend on the exposure level and the expected intensity of seismic events. The target annual failure probabilities given in 6.4 may be modified to meet targets set by owners in consultation with regulators, or to meet regional requirements where they exist.

### 6.2 Seismic design procedures

#### 6.2.1 General

Two alternative procedures for seismic design are provided. A simplified method may be used where seismic considerations are unlikely to govern the design of a structure, while the detailed method shall be used where seismic considerations have a significant impact on the design. The selection of the appropriate procedure depends on the exposure level of the structure and the expected intensity and characteristics of seismic events. The simplified procedure (Clause 7) allows the use of generic seismic maps provided in Annex B;

while the detailed procedure (Clause 8) requires a site-specific seismic hazard study. In all cases, the simplified procedure may be used to perform appraisal and concept screening for a new offshore development.

Figure 1 presents a flowchart of the selection process and the steps associated with both procedures.

### 6.2.2 Extreme level earthquake design

During the ELE event, structural members and foundation components are permitted to sustain localized and limited non-linear behaviour (e.g. yielding in steel, tensile cracking in concrete). As such, ELE design procedures are primarily based on linear elastic methods of structural analysis with, for example, non-linear soil-structure interaction effects being linearized. However, if seismic isolation or passive energy dissipation devices are employed, non-linear time history procedures shall be used.

For structures subjected to base excitations from seismic events, either of the following two methods of analysis are allowed for the ELE design check:

- a) the response spectrum analysis method, or
- b) the time history analysis method.

In both methods, the base excitations shall be composed of three motions, i.e. two orthogonal horizontal motions and the vertical motion. Reasonable amounts of damping compatible with the ELE deformation levels are used in the ELE design. The International Standard applicable to the type of offshore structure<sup>3)</sup> shall be consulted when available. Higher values of damping due to hydrodynamics or soil deformation shall be substantiated with special studies. The foundation may be modelled with equivalent elastic springs and, if necessary, mass and damping elements; off-diagonal and frequency dependence can be significant. The foundation stiffness and damping values shall be compatible with the ELE level of soil deformations.

In a response spectrum analysis, the methods for combining the responses in the three orthogonal directions shall consider correlation between the modes of vibration. When responses due to each directional component of an earthquake are calculated separately, the responses due to the three earthquake directions may be combined using the root of the sum of the squares method. Alternatively, the three directional responses may be combined linearly assuming that one component is at its maximum while the other two components are at 40 % of their respective maximum values. In this method, the sign of each response parameter shall be selected such that the response combination is maximized.

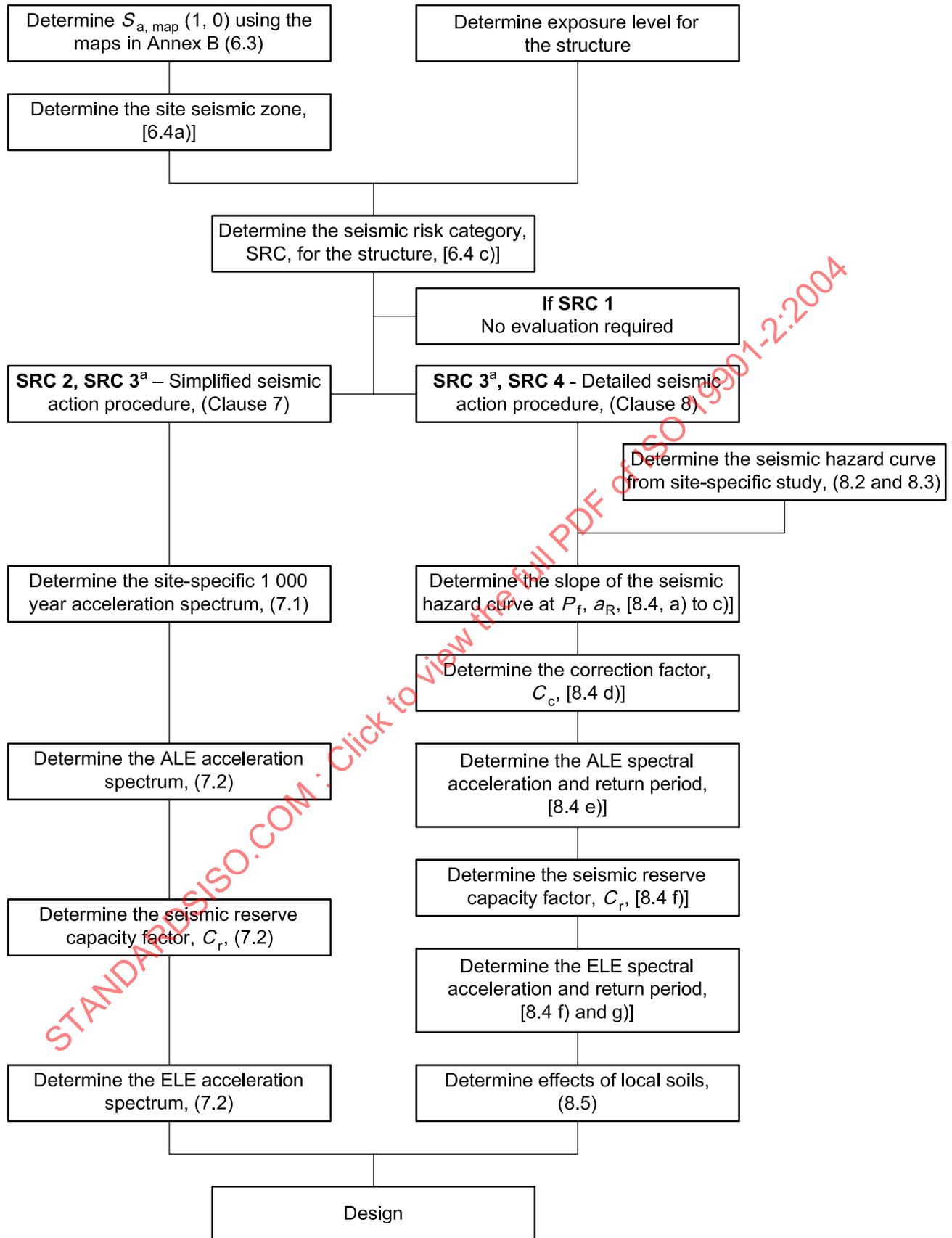
If the time history analysis method is used, a minimum of 4 sets of time history records shall be used to capture the randomness in seismic motions. The earthquake time history records shall be selected such that they represent the dominating ELE events. Component code checks are calculated at each time step and the maximum code utilization during each time history record shall be used to assess the component performance. The ELE design is satisfactory if the code utilization maxima are less than 1,0 for half or more of the records; a scale factor of 1,05 shall be applied to the records if less than 7 sets of records are used.

Equipment on the deck shall be designed to withstand motions that account for the transmission of ground motions through the structure. Deck motions can be much higher than those experienced at the sea floor. The time history analysis method is recommended for obtaining deck motions (especially relative motions) and deck motion response spectra.

The effects of ELE-induced motions on pipelines, conductors, risers and other safety-critical components shall be considered.

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3) International Standards applicable to types of offshore structure, include ISO 19902 and ISO 19903, and when available, ISO 19904 (all parts), ISO 19905 (all parts) and ISO 19906. See the Bibliography.



<sup>a</sup> SRC 3 structures may be designed using either a simplified or detailed seismic action procedure, see Table 4.

Figure 1 — Seismic design procedures

### 6.2.3 Abnormal level earthquake design

In most cases, it is not economical to design a structure such that the ALE event would be resisted without major non-linear behaviour. Therefore, the ALE design check allows non-linear methods of analysis, e.g. structural elements are allowed to behave plastically, foundation piles are allowed to reach axial capacity or develop plastic behaviour, and skirt foundations are allowed to slide. In effect, the design depends on a combination of static reserve strength, ductility, and energy dissipation to resist the ALE actions.

Structural and foundation models used in an ALE analysis shall include possible stiffness and strength degradation of components under cyclic action reversals. The ALE analysis shall be based on best estimate values of modelling parameters such as material strength, soil strength and soil stiffness. This can require reconsideration of the conservatism that is typically present in the ELE design procedure.

For structures subjected to base excitations from seismic events, either of the following two methods of analysis are allowed for the ALE design check:

- a) the static pushover or extreme displacement method, or
- b) the non-linear time history analysis method.

The two methods can complement each other in most cases. The requirements in 6.2.2 for the composition of base excitations from three orthogonal components of motion and for damping also apply to the ALE design procedure.

The static pushover analysis method may be used to determine possible and controlling global mechanisms of failure, or the global displacement of the structure (i.e. beyond the ELE). The latter may be achieved by performing a displacement controlled structural analysis. The non-linear time history analysis method is the most accurate method of ALE analysis. A minimum of 4 time history analyses shall be used to capture the randomness in a seismic event. The earthquake time history records shall be selected such that they represent the dominating ALE events. If 7 or more time history records are used, global structure survival shall be demonstrated in half or more of the time history analyses. If fewer than 7 time history records are used, global survival shall be demonstrated in at least 4 time history analyses.

Extreme displacement methods may be used to assess survival of compliant or soft-link systems, e.g. tethers on a tension leg platform (TLP), or portal action of TLP foundations subjected to lateral actions. In these methods, the system is evaluated at the maximum ALE displacement, including the associated action effects for the structure. The hull structure of the TLP is designed elastically for the corresponding actions. The effect of large structure displacements on pipelines, conductors, risers and other safety-critical components shall be considered separately.

### 6.3 Spectral acceleration data

Generic seismic maps of spectral accelerations for the offshore areas of the world are presented in Annex B. These maps should be used in conjunction with the simplified seismic action procedure (Clause 7). For each area, two maps are presented in Annex B:

- one for a 0,2 s oscillator period;
- the other for a 1,0 s oscillator period.

The acceleration values are expressed in  $g$  and correspond to 5 % damped spectral accelerations on bedrock outcrop, defined as site class A/B in 7.1. These accelerations have an average return period of 1 000 years and are designated as  $S_{a,map}(0,2)$  or  $S_{a,map}(1,0)$ .

Results from a site-specific seismic hazard assessment may be used in lieu of the maps in a simplified seismic action procedure.

## 6.4 Seismic risk category

The complexity of a seismic action evaluation and the associated design procedure depends on the structure's seismic risk category, SRC, as determined below. Acceleration levels taken from Annex B define the seismic zones, which are then used to determine the appropriate seismic design procedure. The selection of the procedure depends on the structure's exposure level as well as the severity of ground motion. The following steps shall be followed to determine the SRC:

- a) Determine the site seismic zone: from the worldwide seismic maps in Annex B, read the value for the 1,0 s horizontal spectral acceleration,  $S_{a,map}(1,0)$ ; using this value, determine the site seismic zone from Table 1.

**Table 1 — Site seismic zone**

$S_{a,map}(1,0)$	< 0,03 g	0,03 g to 0,10 g	0,11 g to 0,25 g	0,26 g to 0,45 g	> 0,45 g
<b>Seismic zone</b>	0	1	2	3	4

- b) Determine the structure's exposure level [consult the International Standard, when available, applicable to the type of offshore structure<sup>4)</sup>]. The target annual probabilities of failure associated with each exposure level are given in Table 2; these are required in the detailed procedure to determine seismic actions. Other target probabilities may be used in the detailed seismic action procedure if recommended or approved by local regulatory authorities. The simplified seismic action procedure has been calibrated to the target probabilities given in Table 2.

**Table 2 — Target annual probability of failure,  $P_f$**

Exposure level	$P_f$
L1	$4 \times 10^{-4} = 1/2\ 500$
L2	$1 \times 10^{-3} = 1/1\ 000$
L3	$2,5 \times 10^{-3} = 1/400$

- c) Determine the structure's seismic risk category, SRC, based on the exposure level and the site seismic zone the SRC is determined from Table 3.

**Table 3 — Seismic risk category, SRC**

Site seismic zone	Exposure level		
	L3	L2	L1
0	SRC 1	SRC 1	SRC 1
1	SRC 2	SRC 2	SRC 3
2	SRC 2	SRC 2	SRC 4
3	SRC 2	SRC 3	SRC 4
4	SRC 3	SRC 4	SRC 4

If the design lateral seismic action is smaller than 5 % of the total vertical action comprising the sum of permanent actions plus variable actions minus buoyancy actions, SRC 4 and SRC 3 structures may be reclassified as SRC 2.

4) International Standards applicable to types of offshore structure, include ISO 19902 and ISO 19903, and when available, ISO 19904 (all parts), ISO 19905 (all parts) and ISO 19906. See the Bibliography.

**6.5 Seismic design requirements**

Table 4 gives the seismic design requirements for each SRC; these requirements are also shown in Figure 1.

In seismically active areas, the designer shall strive to produce a robust and ductile structure, capable of withstanding extreme displacements in excess of normal design values. Where available for a given structure type, architectural and detailing requirements and recommendations for ductile design should be followed for all cases (except SRC 1). Consult the International Standard, when available, applicable to the type of offshore structure<sup>5)</sup>.

For floating structures, consideration of riser stroke, tether rotation angle, and similar geometric allowances shall be sufficient to address the ALE requirements.

**Table 4 — Seismic design requirements**

SRC	Seismic action procedure	Evaluation of seismic activity	Non-linear ALE analysis
1	None	None	None
2	Simplified	ISO maps or regional maps	Permitted
3 <sup>a</sup>	Simplified	Site-specific, ISO maps or regional maps	Recommended
	Detailed	Site-specific	Recommended
4	Detailed	Site-specific	Required

<sup>a</sup> For an SRC 3 structure, a simplified seismic action procedure is in most cases more conservative than a detailed seismic action procedure. For evaluation of seismic activity, results from a site-specific probabilistic seismic hazard analysis (PSHA), see 8.2, are preferred and should be used, if possible. Otherwise regional or ISO seismic maps may be used. A detailed seismic action procedure requires results from a PSHA whereas a simplified seismic action procedure may be used in conjunction with either PSHA results or seismic maps (regional or ISO maps).

**7 Simplified seismic action procedure**

**7.1 Soil classification and spectral shape**

Having obtained the bedrock spectral accelerations at oscillator periods of 0,2 s and 1,0 s,  $S_{a,map}(0,2)$  and  $S_{a,map}(1,0)$ , from Annex B, the following steps shall be followed to define the site response spectrum corresponding to a return period of 1 000 years:

- a) Determine the site class as follows.

The site class depends on the seabed soils on which a structure is founded and is a function of the average properties of the top 30 m of the effective seabed (see Table 5).

The average shear wave velocity in the top 30 m of effective seabed ( $\bar{v}_s$ ) shall be determined from Equation (1):

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

where

$n$  is the number of distinct soil layers in the top 30 m of effective seabed;

5) International Standards applicable to types of offshore structure, include ISO 19902 and ISO 19903, and when available, ISO 19904 (all parts), ISO 19905 (all parts) and ISO 19906. See the Bibliography.

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

$v_{s,i}$  is the shear wave velocity of layer  $i$ .

Similarly, the average of normalized cone penetration resistance ( $\bar{q}_{cl}$ ) or soil undrained shear strength ( $\bar{c}_u$ ) shall be determined according to Equation (1) where  $v_s$  is replaced by  $q_{cl}$  or  $c_u$ .

**Table 5 — Determination of site class**

Site class	Soil profile name	Average properties in top 30 m of effective seabed		
		Soil shear wave velocity $\bar{v}_s$ m/s	Sand: normalized cone penetration resistance $\bar{q}_{cl}$ <sup>a</sup>	Clay: soil undrained shear strength $\bar{c}_u$ 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	$350 < \bar{v}_s \leq 750$	$\bar{q}_{cl} \geq 200$	$\bar{c}_u \geq 200$
D	Stiff to very stiff soil	$180 < \bar{v}_s \leq 350$	$80 \leq \bar{q}_{cl} < 200$	$80 \leq \bar{c}_u < 200$
E	Soft to firm soil	$120 < \bar{v}_s \leq 180$	$\bar{q}_{cl} < 80$	$\bar{c}_u < 80$
F	—	Any profile, including those otherwise classified as A to E, containing soils having one or more of the following characteristics: $\bar{v}_s \leq 120$ ; soils vulnerable to potential failure or collapse under seismic actions such as liquefiable soils, highly sensitive clays, collapsible weakly cemented soils; poze <sup>b</sup> with a thickness of more than 10 m; soil layers with high gas content or ambient excess pore pressure greater than 30 % of <i>in situ</i> effective overburden; layers greater than 2 m thick with sharp contrast in shear wave velocity (greater than $\pm 30$ %) and/or undrained shear strength (greater than $\pm 50$ %) compared to adjacent layers.		
<sup>a</sup> $q_{cl} = (q_c / p_a) \times (p_a / \sigma'_{v0})^{0.5}$ where $q_c$ is the cone penetration resistance; $p_a$ is atmospheric pressure = 100 kPa; $\sigma'_{v0}$ is the vertical effective stress.				
<sup>b</sup> Clay containing more than 30 % calcareous or siliceous material of biogenic origin.				

b) Determine  $C_a$  and  $C_v$  as follows.

- 1) For shallow foundations, determine the site coefficients,  $C_a$  and  $C_v$ , from Table 6 and Table 7. The values of  $C_a$  and  $C_v$  are dependent on the site class and either the mapped 0,2 s or 1,0 s spectral accelerations,  $S_{a,map}(0,2)$  and  $S_{a,map}(1,0)$ .
- 2) For deep pile foundations, the site coefficients  $C_a$  and  $C_v$  are listed in Table 8.

**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	$\geq 1,25 g$
A/B	1,0	1,0	1,0	1,0	1,0
C	1,2	1,2	1,1	1,0	1,0
D	1,6	1,4	1,2	1,1	1,0
E	2,5	1,7	1,2	0,9	0,9
F	a	a	a	a	a

<sup>a</sup> A site-specific geotechnical investigation and dynamic site response analyses shall be performed.

**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	$\geq 0,5 g$
A/B	1,0	1,0	1,0	1,0	1,0
C	1,7	1,6	1,5	1,4	1,3
D	2,4	2,0	1,8	1,6	1,5
E	3,5	3,2	2,8	2,4	2,4
F	a	a	a	a	a

<sup>a</sup> A site-specific geotechnical investigation and dynamic site response analyses shall be performed.

**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	a	a

<sup>a</sup> A site-specific geotechnical investigation and dynamic site response analyses shall be performed.

c) Determine the site 1 000 year horizontal acceleration spectrum as follows.

- 1) A seismic acceleration spectrum shall be prepared for different oscillator periods ( $T$ ), as shown in Figure 2.
- 2) For periods,  $T$ , less than or equal to 0,2 s, the site spectral acceleration,  $S_{a,site}(T)$ , shall be taken as:

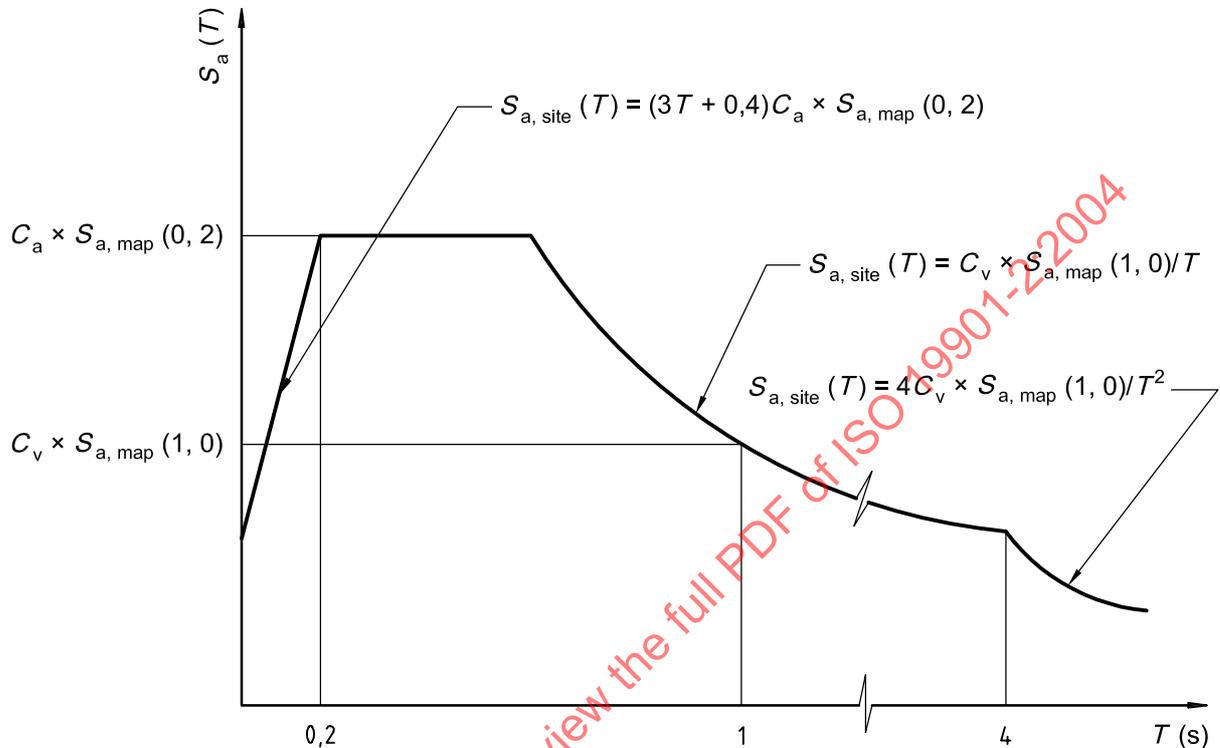
$$S_{a,site}(T) = (3T + 0,4) C_a \times S_{a,map}(0,2) \tag{2}$$

- 3) For periods greater than 0,2 s, the site spectral acceleration,  $S_{a,site}(T)$ , shall be taken as:

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

- 4) For periods greater than 4,0 s, the site spectral acceleration may be taken as decaying in proportion to  $1/T^2$  instead of  $1/T$  as given by Equation (4):

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



**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 Annex B 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 Annex B associated with a single degree of freedom oscillator period 1,0 s

**Figure 2 — Seismic acceleration spectrum for 5 % damping**

- d) The site vertical spectral acceleration at a period  $T$  shall be taken as half the corresponding horizontal spectral acceleration. The vertical spectrum shall not be reduced further due to water depth effects.
- e) The acceleration spectra obtained using the preceding steps correspond to 5 % damping. To obtain acceleration spectra corresponding to other damping values, the ordinates may be scaled by applying a correction factor  $D$ :

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

where  $\eta$  is the percent of critical damping.

As an alternative to the procedure given in a) to e), uniform hazard spectra obtained from PSHA may be modified by a detailed dynamic site-response analysis to obtain 1 000 year site-specific design response spectra.

**7.2 Seismic action procedure**

The design seismic acceleration spectra to be applied to the structure shall be determined as follows.

For each oscillator period  $T$ , the ALE horizontal and vertical spectral accelerations are obtained from the corresponding values of the site 1 000 year spectral acceleration [see 7.1 c) and 7.1 d)]:

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

where the scale factor  $N_{ALE}$  is dependent on the structure exposure level and shall be obtained from Table 9.

The ELE horizontal and vertical spectral accelerations at oscillator period  $T$  may be obtained from:

$$S_{a,ELE}(T) = S_{a,ALE}(T) / C_r \tag{7}$$

where  $C_r$  is a seismic reserve capacity factor for the structural system that considers the static reserve strength and the ability to sustain large non-linear deformations of each structure type (e.g. steel versus reinforced concrete). The  $C_r$  factor represents the ratio of spectral acceleration causing catastrophic system failure of the structure, to the ELE spectral acceleration. The value of  $C_r$  should be estimated prior to the design of the structure in order to achieve an economic design that will resist damage due to an ELE and is at the same time likely to meet the ALE performance requirements. Values of  $C_r$  may be justified by prior detailed assessment of similar structures. Values of  $C_r$  for fixed steel structures are specified in ISO 19902. Values of  $C_r$  other than those recommended in the International Standard, when available, applicable to the type of offshore structure<sup>6)</sup> may be used in design, however such values shall be verified by an ALE analysis.

To avoid return periods for the ELE that are too short,  $C_r$  values shall not exceed 2,8 for L1 structures; 2,4 for L2 structures; and 2,0 for L3 structures.

**Table 9 — Scale factors for ALE spectra**

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

**8 Detailed seismic action procedure**

**8.1 Site-specific seismic hazard assessment**

The most widely used seismic input parameter for the seismic design and analysis of offshore structures is the design acceleration spectrum. In site-specific studies, the design acceleration spectrum is usually derived from an acceleration spectrum computed from a probabilistic seismic hazard analysis (PSHA) with possible modifications based on local soil conditions. Deterministic seismic hazard analysis may be used to complement the PSHA results. These analyses are described in 8.2 to 8.5.

6) International Standards applicable to types of offshore structure, include ISO 19902 and ISO 19903, and when available, ISO 19904 (all parts), ISO 19905 (all parts) and ISO 19906. See the Bibliography.

## 8.2 Probabilistic seismic hazard analysis

The different elements of a PSHA are shown graphically in Figure 3. In a probabilistic approach, ground motions at a site are estimated by considering the probability of earthquakes of different sizes on all potential sources (faults or areas) that can affect the site [Figure 3 a)]. A PSHA also accounts for the randomness in attenuation of seismic waves travelling from a source to the site [Figure 3 b)]. Summation over individual probabilities from different sources provides total annual probability of exceedance for a given level of peak ground acceleration (PGA) or spectral acceleration [Figure 3 c)]. 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”. Spectral response varies with the natural period of the oscillator, therefore a family of hazard curves for different periods  $T$  is obtained [see Figure 3 c)].

The results from a PSHA are used to derive a uniform hazard spectrum [Figure 3 d)], where all points on the spectrum correspond to the same annual probability of exceedance. The relationship between the return period of a uniform hazard spectrum and the target annual probability of exceedance ( $P_e$ ) may be taken as:

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

where  $T_{\text{return}}$  is the return period in years.

Since a PSHA is a probability-based approach, it is important that uncertainty be considered in the definition of input parameters such as the maximum magnitude for a given source, the magnitude recurrence relationship, the attenuation equation, and geographical boundaries defining the location of a source zone.

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 3 c)]. Because of uncertainties in PSHA input parameters, each of these hazard curves has an uncertainty band. The mean (or expected value) of each hazard curve should be used to construct a uniform hazard spectrum corresponding to a given exceedance probability  $P_e$  [see Figure 3 d)]. All references to hazard curves in 8.4 refer to the mean of the hazard curve.

## 8.3 Deterministic seismic hazard analysis

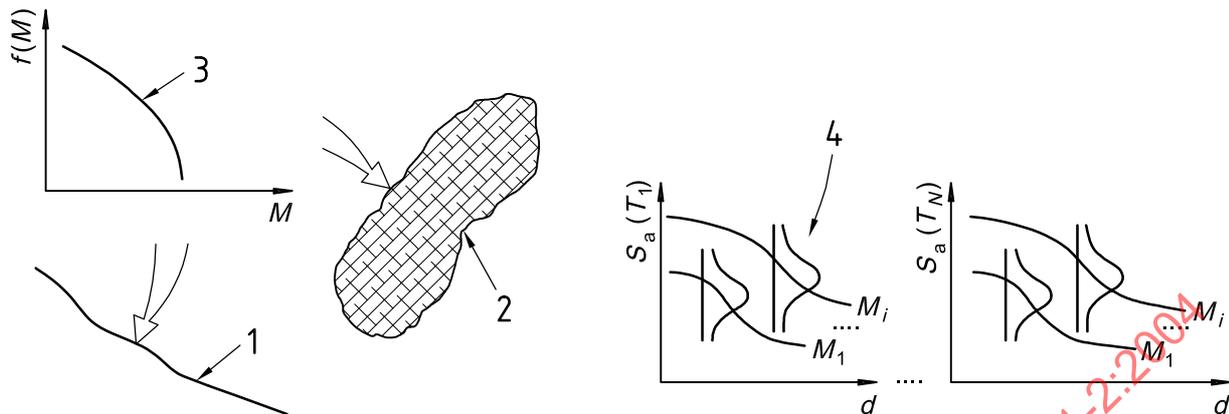
Deterministic estimates of ground motion extremes at a site are obtained by considering a single event of a specified magnitude and distance from the site. To perform a deterministic analysis, the following information is needed:

- definition of an earthquake source (e.g. a known fault) and its location relative to the site;
- definition of a design earthquake magnitude that the source is capable of producing;
- a relationship which describes the attenuation of ground motion with distance.

A site can have several known active faults in its proximity. A maximum magnitude is defined for each fault. The maximum magnitude is a function of the fault length and historical knowledge of past earthquakes on that particular source.

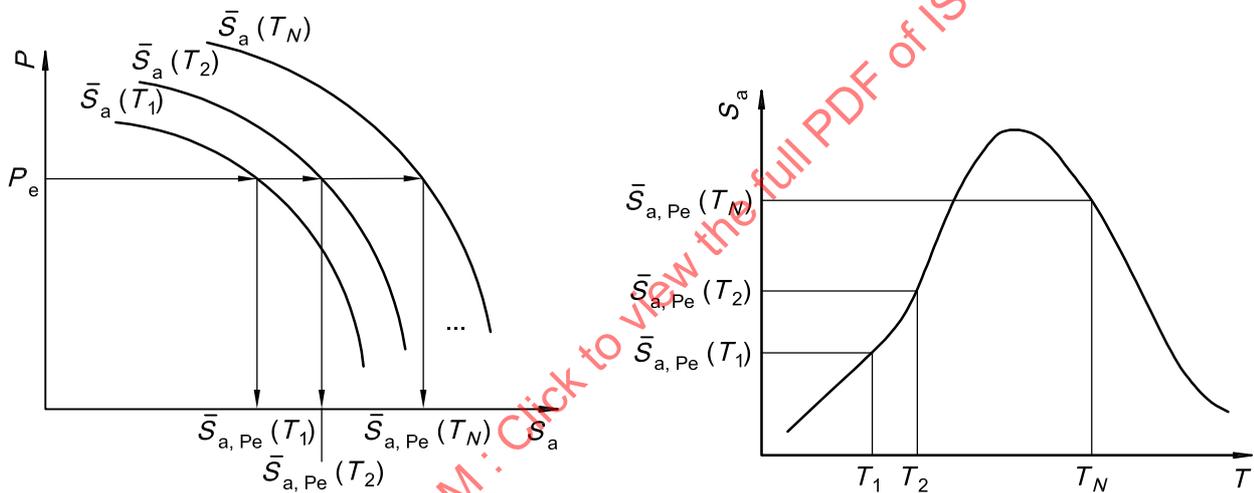
Deterministic ground motion estimates are not associated with a specific return period, such as 1 000 years, although the particular earthquake event used can have a return period associated with it. The return period for the maximum event on a given fault can vary from several hundred to several thousand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

A deterministic seismic hazard analysis may be performed to complement the PSHA results.



a) Define earthquake source seismicity and geometry

b) Define attenuation curves for spectral accelerations at periods  $T_1 \dots T_N$



c) From a) and b), develop seismic hazard curves for spectral accelerations at each period and the selected target annual probability of exceedance and obtain mean uniform hazard spectral accelerations  $\bar{S}_a(T_1) \dots \bar{S}_a(T_N)$

d) From c) construct uniform hazard spectrum of mean spectral accelerations at the selected target annual probability of exceedance

**Key**

1	line source (fault)	$f(M)$	frequency
2	area source	$T_i$	single degree of freedom oscillator periods
3	cumulative annual frequency of magnitude $M$	$S_a(T_i)$	spectral acceleration associated with a single degree of freedom oscillator period $T_i$
4	attenuation uncertainty	$d$	distance from source
$M$	magnitude	$P$	annual probability of exceedance
$P_e$	target level of annual probability of exceedance	$\bar{S}_{a,Pe}(T_i)$	mean spectral acceleration for oscillator period $T_i$ at selected target annual probability of exceedance

**Figure 3 — Probabilistic seismic hazard analysis procedure**

## 8.4 Seismic action procedure

This procedure is based on the results of a PSHA (see 8.2 and Figure 3). The site-specific seismic hazard curve shall have been determined in terms of the annual exceedance probability of a spectral acceleration corresponding to a period that is equal to the dominant modal period of the structure,  $\bar{S}_a(T_{\text{dom}})$ ; such curves are illustrated in Figure 3 c). In lieu of more specific information about the dominant modal period of the structure, the seismic hazard curve may be determined for the spectral acceleration at a period of 1,0 s,  $\bar{S}_a(1,0)$ .

The ALE spectral accelerations are determined from the site-specific hazard curve and the target annual probability of failure,  $P_f$ , listed in Table 2. The specific steps to define the ALE and ELE events are illustrated in Figure 4 and are described in the following steps.

- Plot the site-specific hazard curve for  $T = T_{\text{dom}}$  [a curve such as those shown in Figure 3 c)] on a  $\log_{10}$ - $\log_{10}$  basis, i.e. showing the probability distribution of the parameter  $\bar{S}_a(T_{\text{dom}})$  [see Figure 4 a)].
- Choose the target annual probability of failure,  $P_f$ , as a function of the exposure level as indicated in Table 2, and determine the site-specific spectral acceleration at  $P_f$ ,  $\bar{S}_{a,P_f}(T_{\text{dom}})$  from Figure 4 a).
- Determine the slope of the seismic hazard curve ( $a_R$ ) in the region close to  $P_f$  by drawing a tangent line to the seismic hazard curve at  $P_f$ . The slope  $a_R$  is defined [see Figure 4 a)] as the ratio of the spectral accelerations corresponding to two probability values, one at either side of  $P_f$ , that are one order of magnitude apart [ $P_1$  and  $P_2$  in Figure 4 a)];  $P_1$  should preferably be close to  $P_f$ .
- From Table 10 below determine the correction factor,  $C_c$ , corresponding to  $a_R$ . This correction factor captures the uncertainties not reflected in the seismic hazard curve.

Table 10 — Correction factor,  $C_c$

$a_R$	1,75	2,0	2,5	3,0	3,5
Correction factor, $C_c$	1,20	1,15	1,12	1,10	1,10

- Determine the ALE spectral acceleration by applying the correction factor  $C_c$  to  $\bar{S}_{a,P_f}(T_{\text{dom}})$ , the site-specific spectral acceleration at the required  $P_f$  and the structural dominant period  $T_{\text{dom}}$ :

$$\bar{S}_{a,ALE}(T_{\text{dom}}) = C_c \times \bar{S}_{a,P_f}(T_{\text{dom}}) \quad (9)$$

The annual probability of exceedance for the ALE event ( $P_{\text{ALE}}$ ) can then be directly read from the seismic hazard curve, see Figure 4 b). The ALE return period is determined from the annual probability of exceedance using Equation (8).  $P_{\text{ALE}}$  is smaller than  $P_f$  to accommodate uncertainties in action and resistance evaluations not represented in the seismic hazard curve (as captured in the correction factor  $C_c$ ).

- For certain structure types whose reserve strength and ductility characteristics are known, the ELE spectral acceleration is next determined from:

$$\bar{S}_{a,ELE}(T_{\text{dom}}) = \bar{S}_{a,ALE}(T_{\text{dom}}) / C_r \quad (10)$$

where  $C_r$  is the seismic reserve capacity factor for the structural system that considers static reserve strength and the ability to sustain large non-linear deformations of each structure type (e.g. steel versus reinforced concrete). The  $C_r$  factor represents the ratio of spectral acceleration causing catastrophic system failure of the structure to the ELE spectral acceleration. The value of  $C_r$  should be estimated prior to the design in order to achieve an economic design that will resist damage due to the ELE and is at the same time likely to meet the ALE performance requirements. Values of  $C_r$  may be justified by prior detailed assessment of similar structures. Values of  $C_r$  for fixed steel structures are specified in ISO 19902. Values of  $C_r$  other than those recommended in the International Standard applicable to the type of offshore structure<sup>7)</sup> may be used in design, however such values shall be verified by an ALE analysis; see also A.8.4.

7) International Standards applicable to types of offshore structure, include ISO 19902 and ISO 19903, and when available, ISO 19904 (all parts), ISO 19905 (all parts) and ISO 19906. See the Bibliography.

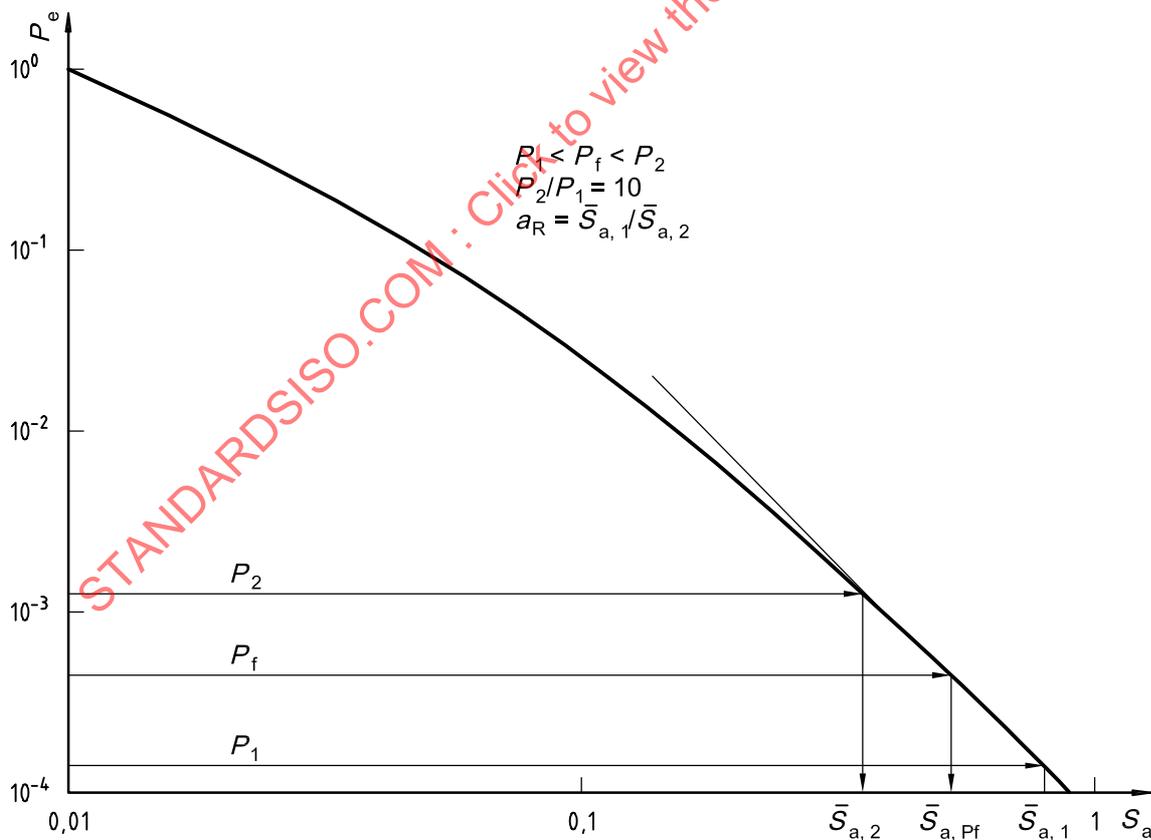
- g) The annual probability of exceedance for the ELE event ( $P_{ELE}$ ) can now be read from the seismic hazard curve, Figure 4 b). The ELE return period is determined from the annual probability of exceedance using Equation (8). Having determined ALE and ELE return periods, obtain ALE spectral accelerations and ELE spectral accelerations for other natural periods from the PHSA results, i.e.  $\bar{S}_{a,ALE}(T)$  and  $\bar{S}_{a,ELE}(T)$ .
- h) Modifications of ALE and ELE acceleration spectra for local geology and soil conditions shall be addressed by a site response analysis (see 8.5).

For floating structures (such as TLPs) and other structure types for which  $C_r$  is either not well defined or unknown, a design process which goes directly to avoiding catastrophic system failure in the ALE is recommended. Extreme displacements and shock waves are often of primary interest here, in order to design the mooring system. The hull structure is designed elastically for the corresponding actions.

Minimum ELE return periods are given in Table 11 to ensure economic viability of a design, as a function of exposure level. If the ELE return period that is obtained from the procedure in this subclause is lower than the corresponding return period listed in Table 11, the return period in Table 11 shall be used for  $S_{a,ELE}(T)$ .

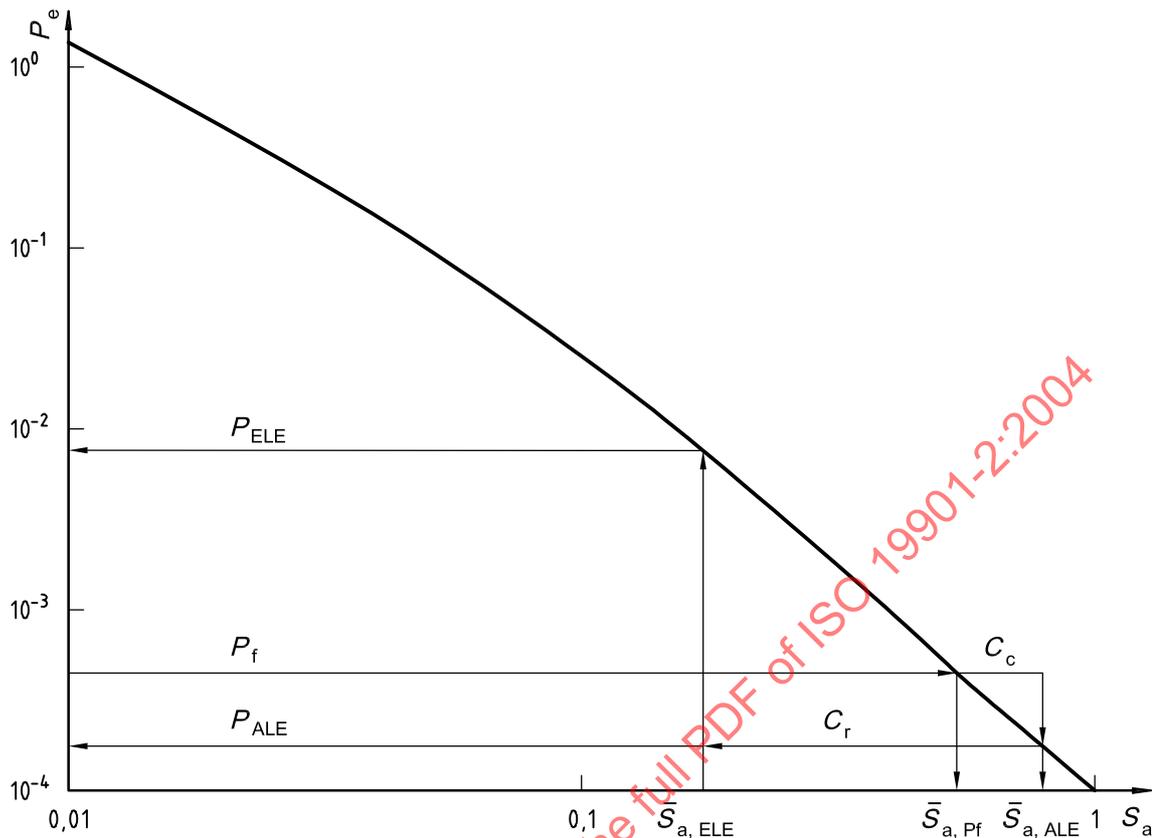
**Table 11 — Minimum ELE return periods**

Exposure level	Minimum ELE return periods
L3	50
L2	100
L1	200



a) Derivation of the slope  $a_R$  of the seismic hazard curve for  $T = T_{dom}$

**Figure 4 — Typical seismic hazard curve**



b) Derivation of spectral accelerations and probabilities for ALE and ELE events

**Key**

$P_e$  annual probability of exceedance  
 $S_a$  spectral acceleration ( $g$ )

Figure 4 (continued)

### 8.5 Local site response analyses

In the detailed seismic action procedure (8.4), the ALE and ELE design spectral accelerations  $\bar{S}_{a,ALE}(T)$  and  $\bar{S}_{a,ELE}(T)$  are based on uniform hazard curves where all points on the curves have the same return period. The return periods for ALE and ELE events are determined according to the procedure specified in 8.4. The probabilistic and deterministic seismic hazard analyses described in 8.2 and 8.3 produce ground motions applicable to moderately stiff, stiff, or bedrock sites. However, many offshore sites consist of a surface layer of soft soils overlying the stiffer materials. The ALE and ELE spectral accelerations shall be further modified to account for local soil conditions at the site. A dynamic site response analysis, using linear or non-linear models of the underlying soil, may be used to modify the ALE and ELE spectral accelerations and obtain site-specific spectral accelerations for design.

As an alternative to a dynamic site response analysis, the procedure in 7.1 may be used to modify the acceleration spectra. Following 7.1, an amplification spectrum is obtained from the ratio of acceleration spectrum corresponding to the local site class to that corresponding to stiff soil or rock site class. The amplification spectrum can then be used to modify the acceleration spectra from a PSHA corresponding to a stiff soil or rock site.

## 9 Performance requirements

### 9.1 ELE performance

The objectives of ELE design are to ensure that there is little or no damage to the structure during the ELE event and that there is an adequate margin of safety against major failures during larger events. The following ELE performance requirements shall be verified.

- All primary structural and foundation components shall sustain little or no damage due to the ELE. Limited non-linear behaviour (e.g. yielding in steel or tensile cracking in concrete) is permitted, however, brittle degradation (e.g. local buckling in steel or spalling in concrete) shall be avoided.
- Secondary structural components, such as conductor guide panels, shall follow the same ELE design rigour as that of primary components.
- The internal forces in joints shall stay below the joint strengths, using the calculated (elastic) forces and moments.
- Foundation checks shall be performed at either the component level or at the system level. At the component level an adequate margin shall exist with respect to axial and lateral failure of piles or vertical and sliding failure of other foundation elements. At the system level, an adequate margin shall exist with respect to large-deflection mechanisms which would damage or degrade, and require repairs to, the structure or its ancillary systems (e.g. pipelines or conductors).
- There shall not be any loss of functionality in safety systems or in escape and evacuation systems due to the ELE.
- Masts, derricks and flare structures shall be capable of sustaining the motions transmitted via the structure with little or no damage. The design shall include restraints to prevent toppling of topsides equipment and cable trays. Piping shall be designed for differential displacements due to support movements and sliding supports shall be maintained such that they act as intended in the design. The design should minimize the potential for equipment and appurtenances to become falling hazards during the ELE.

### 9.2 ALE performance

The objective of an ALE design check is to ensure that the global failure modes which can lead to high consequences such as loss of life or major environmental damage will be avoided. The following ALE performance requirements shall be verified.

- Structural elements are allowed to exhibit plastic degrading behaviour (e.g. local buckling in steel or spalling in concrete), but catastrophic failures such as global collapse or failure of a cantilevered section of the deck should be avoided.
- Stable plastic mechanisms in foundations are allowed, but catastrophic failure modes such as instability and collapse should be avoided.
- Joints are allowed to exhibit limited plastic behaviour but should stay within their ultimate strengths. Alternatively, where large deformations in the joints are anticipated, they shall be designed to demonstrate ductility and residual strength at anticipated deformation levels.
- The safety systems and escape and evacuation systems shall remain functional during and after the ALE.
- Topsides equipment failures shall not compromise the performance of safety-critical systems. Collapse of the living quarters, masts, derricks, flare structures and other significant topsides equipment should be avoided.
- Any post-ALE event strength requirements given in the International Standard, when available, applicable to the type of offshore structure<sup>8)</sup> apply.

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8) International Standards applicable to types of offshore structure, include ISO 19902 and ISO 19903, and when available, ISO 19904 (all parts), ISO 19905 (all parts) and ISO 19906. See the Bibliography.

## Annex A (informative)

### Additional information and guidance

NOTE The clauses in this annex provide additional information and guidance on clauses in the body of this part of ISO 19901. The same numbering system and heading titles have been used for the ease in identifying the subclause in the body of this part of ISO 19901 to which it relates.

#### A.1 Scope

The background to and the development of the philosophy for this part of ISO 19901 were presented at OMAE 2001<sup>[7]</sup>.

#### A.2 Normative references

No guidance is offered.

#### A.3 Terms and definitions

No guidance is offered.

#### A.4 Symbols and abbreviated terms

No guidance is offered.

#### A.5 Earthquake hazards

In addition to seismically induced motions, the planning and design of offshore structures should also consider other hazards that can be initiated by earthquakes. Most geologically induced hazards that are initiated by earthquakes can be avoided by proper site selection studies.

Liquefaction of soils can occur as a result of repeated cyclic motions of saturated loose cohesionless soils. The potential for liquefaction decreases as soil density increases. Poorly graded sands are more susceptible to liquefaction than well graded sands. Both gravity based and pile founded structures located in these types of soil will experience a decrease in capacity during a strong earthquake because the strength of the soil will degrade significantly.

Earthquakes can initiate failure of sea floor slopes that are stable under normal self weight and wave conditions, resulting in sea floor slides. The scope of site investigations in areas of potential instability should focus on identification of metastable geological features surrounding the site and definition of the soil engineering properties required for modelling and estimating sea floor movements. Analytical estimates of soil movement as a function of depth below the sea floor can be used with coupled soil engineering properties, to establish expected actions on structural members. The best mitigation of this hazard is to locate offshore structures away from such regions, although design of structures for sea floor slides has been used in the Gulf of Mexico.

Fault movement can occur as a result of seismic activity. Siting of facilities close to fault planes intersecting the sea floor should be avoided, if possible. If circumstances dictate siting structures nearby potentially active features, the magnitude and time scale of expected movement should be estimated on the basis of a geological study for use in the structure's design.

Tsunamis are generated by large (and sometimes distant) earthquakes and undersea fault movements, and by large sea floor slides that can be triggered by earthquakes. When travelling through deep water, these waves are long with low height and pose little hazard to floating or fixed structures. When they reach shallow water, the wave form pushes upward from the bottom creating a swell that can break in shallow water and can wash inland with great power. The greatest hazard to shallow water offshore structures from tsunamis results from inflow and outflow of water in the form of waves and currents. These waves can cause substantial actions on the structures and the currents can cause excessive scour problems.

Mud volcanoes are often found at pre-existing faults. These features are not directly caused by earthquakes, rather they use the fault zone as a conduit to bring gas, water and the associated muds to the sea floor, thereby creating surface features resembling a volcano cone. The best mitigation of this hazard is to locate offshore structures away from such regions.

Earthquake-induced shock waves in the water column, generated by motions of the sea floor, can have an impact on floating structures and certain appurtenances. The shock wave can radiate upward through the water column causing a possible impulsive action on buoyant or partially buoyant structures and therefore an increase in hull pressures and tendon or mooring line forces. This phenomenon is only likely to be an issue for the most severe earthquakes.

## A.6 Seismic design principles and methodology

### A.6.1 Design principles

The requirement for a two-level design check stems from the high degree of randomness in seismic events, uncertainties in seismic action calculations, and the fact that design for seismic events of abnormal severity on the basis of strength alone and without consideration of a structure's capacity to dissipate energy and sustain large inelastic displacements would be uneconomical.

A structure designed to the ELE has a margin of safety for more severe events due to explicit and implicit safety margins in design equations and due to its capacity for large non-linear deformations. In order to avoid repeating parts of the design process and to ensure that the ALE check demonstrates an acceptable design, the ratio of ALE to ELE spectral accelerations is set such that there is a high likelihood of meeting both ELE and ALE performance objectives. The seismic design procedures in this part of ISO 19901 address the balance between the ALE and ELE design criteria.

### A.6.2 Seismic design procedures

#### A.6.2.1 General

No guidance is offered.

#### A.6.2.2 Extreme level earthquake design

The seismic design of an offshore structure is primarily performed during an ELE evaluation where structural component dimensions are determined according to the design equations in the International Standard applicable to the type of offshore structure<sup>9)</sup>. In developing the ELE design procedure, two objectives are considered:

- a) The ELE design procedure and associated design criteria should ensure that the structure will be able to withstand seismic events of this severity with little or no damage.
- b) The ELE design procedure and associated design criteria leads to the design of a structure that is likely to meet the ALE performance criteria (see 9.2) with a minimum of design changes.

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9) International Standards applicable to types of offshore structure, include ISO 19902 and ISO 19903, and when available, ISO 19904 (all parts), ISO 19905 (all parts) and ISO 19906. See the Bibliography.

The first objective may be seen as an economic goal in that it avoids the need for frequent repairs, while the second objective is a safety goal.

In most cases, spectral acceleration is the controlling parameter in design of offshore structures. In these cases, the ELE design procedure may be specified in terms of seismic design spectra or acceleration (time history) records.

The earthquake records for time history analysis are selected such that they represent the ELE ground motion hazard at the site. Following a PSHA (see 8.2), the dominating ELE events may be identified through a procedure that is referred to as deaggregation<sup>[6] to [10]</sup>. In the deaggregation procedure, the contributions of various faults and seismic source zones to the probability of exceeding a given spectral acceleration are identified. The highest contributors represent the dominating ELE events.

Given the magnitude and distance of events dominating ELE ground motions, the earthquake records for time history analysis can be selected from a catalogue of historical events. Each earthquake record consists of three sets of tri-axial time histories representing two orthogonal horizontal components and one vertical component of motion. In selecting earthquake records, the tectonic setting (e.g. faulting style) and the site conditions (e.g. hardness of underlying rock) of the historical records should be matched with those of the structure's site. Although, if feasible, the records will match the target event's magnitude and distance, further scaling of the records will be required to match the level of ELE response spectrum. One option is a simple scaling of the record such that the average response spectrum due to the two horizontal components matches the horizontal ELE response spectrum at the dominant period of the structure/foundation system.

#### **A.6.2.3 Abnormal level earthquake design**

The ALE design check is performed to ensure that the safety goals are met and that the structure can sustain intense earthquakes of abnormal severity without loss of life or major environmental damage. The safety goal is defined in terms of an upper limit on the annual probability of failure due to a seismic event.

In order to ensure that the ALE design check is consistent with the safety goal, the design procedure and associated design criteria take into consideration randomness (Type I uncertainties) in seismic events and seismic wave attenuation, seismic action effects, and the resistance of the structure. Additionally, systematic uncertainties (Type II uncertainties) associated with seismotectonic modelling are considered. For example, these Type II uncertainties are typically included in a PSHA model.

Selection of earthquake records for ALE time history analysis and scaling of those records follow the same procedures as those outlined for ELE design in A.6.2.2.

#### **A.6.3 Spectral acceleration data**

In the maps included in Annex B the boundaries separating offshore zones of different spectral accelerations are generally the same for the 0,2 s and 1,0 s maps. The notable exception is North America where the boundaries on the two maps are different in the south-eastern and south-western portions of the U.S. These differences were judged to be necessary based on the comprehensive spectral acceleration mapping project completed by the U.S. Geological Survey<sup>[11]</sup> for the 1997 NEHRP Seismic Provisions<sup>[12]</sup>. The spectral acceleration values for the North Sea were based on the results in Reference [13]. Information on seismic hazard parameters in the offshore areas of Canada can be found in Reference [14].

The largest values of 1,25 g on the 0,2 s maps and 0,50 g on the 1,0 s maps are generally considered a sufficient representation of the ground motion hazard in areas of high seismic activity for the purpose of this part of ISO 19901. However, it is understood that, in certain locations, site-specific studies can produce estimates of the 1 000 year spectral accelerations that are significantly greater than these values. If the map spectral accelerations are in doubt in a given area, a site-specific PSHA should be undertaken.

#### A.6.4 Seismic risk category

The 1 000 year return period spectral acceleration at 1,0 s is used to gauge the exposure of an offshore structure to seismic events. Table 1 shows the site seismic zone as a function of this spectral acceleration. Because the spectral acceleration is a response property of a single degree of freedom oscillator, it is more representative of seismic exposure than other parameters such as the peak ground acceleration (PGA) or the peak ground velocity. The period of 1,0 s was selected as a compromise. In many regions the 1 000 year spectral acceleration at 1,0 s and 1 000 year PGA values will be of comparable magnitude, which should help users who are more familiar with PGA.

This part of ISO 19901 differs from the historical practice of directly recommending specific return periods for the design events. Instead, a procedure is outlined where the return period of the ALE event is determined indirectly from the target probability of failure and the results of a site-specific PSHA (if available). The ELE return period is, in turn, determined from that of the ALE event by considering the capacity for large deformations that is inherent in a structure.

The procedure recommended for seismic design uses the target annual probability of system failure ( $P_f$ ) as the starting point. This approach is different from load and resistance factor design (LRFD) codes where the target probability of failure is assigned to the component level. Both the simplified and detailed seismic action procedures are based on the concept that the ALE design should meet the target annual probability of failure of the structural system. The recommended target annual probabilities are listed in Table 2 and reflect the industry's experience in design of offshore structures for seismically active regions. Probabilities different from those in Table 2 may be recommended for specific types of offshore structure in specific regions.

In a detailed seismic action procedure, the designer may use  $P_f$  values which are different than those listed in Table 2. In a simplified seismic action procedure, the designer does not explicitly use  $P_f$ , however the procedure has been calibrated to meet the target annual probabilities listed in Table 2. Therefore, the simplified seismic action procedure is applicable only if the designer accepts the target probabilities listed in Table 2.

#### A.6.5 Seismic design requirements

The intensity and characteristics of seismic ground motions used for the design of an offshore structure may be determined either by a simplified seismic action procedure or from a detailed seismic action procedure. The simplified seismic action procedure may make use of the generic seismic maps presented in Annex B, regional maps, or site-specific PSHA results; the detailed seismic action procedure requires a site-specific seismic hazard study as described in 8.2. In both procedures the return period of the ELE or ALE events may be estimated from the annual probability of exceedance using Equation (8) or alternatively using Equation (A.2) (see A.8.2).

### A.7 Simplified seismic action procedure

#### A.7.1 Soil classification and spectral shape

The preferred method for determining the shear wave velocity is through field measurements. Field shear wave velocity measurements can be obtained by a variety of methods<sup>[15]</sup>. Usually shear wave velocities are obtained offshore from down-hole measurements in a single borehole. The seismic source is often located at the sea floor, while the geophones are positioned at varying depths down the borehole. A common offshore practice is to install geophones within a cone penetrometer system (seismic cone). Down-hole core logging techniques can also be used where both the seismic source and receivers are placed down-hole. If multiple boreholes are available, shear wave velocities can also be obtained from cross-hole techniques.

Some other techniques are also available which could be used to determine field shear wave velocities. Hydrophone arrays are now being placed on the sea floor to help determine reservoir changes with time (4D-seismic). If a sea floor seismic source were used with these on-bottom arrays, shear wave velocity measurements can be made with either seismic reflection, seismic refraction or spectral analysis of surface waves (SASW) methods<sup>[16]</sup>.

If direct field measurements are not available, then the shear wave velocity can be inferred from data collected in the soil boring investigation. The shear wave velocity can be determined, based on information from the soil boring, i.e. from the low amplitude shear modulus ( $G_{\max}$ ) and the mass density of the soil ( $\rho$ ) by:

$$v_s = \sqrt{\frac{G_{\max}}{\rho}} \quad (\text{A.1})$$

The above equation is approximate for a saturated soil because of coupling effects between the pore fluid and the soil skeleton. However, in most cases using the total mass density of the soil and water will give shear wave velocities within a few per cent of values determined when considering the coupling effects.

The low amplitude shear wave modulus ( $G_{\max}$ ) can be determined experimentally from dynamic laboratory tests such as the resonant column test, or it can be estimated from other soil properties determined from the soil boring investigation. It should be noted, however, that estimating  $G_{\max}$  from other soil properties will have the greatest degree of uncertainty.

For uncemented sands, Reference [17] provides empirical relationships for  $G_{\max}$  for both angular and rounded particle shapes. This relationship depends on the void ratio and the average effective confining stress applied to the soil sample. A more recent expression is provided in Reference [18] which is dependent on the overconsolidation ratio, the void ratio, Poisson's ratio, the average effective confining stress, and an empirical stiffness coefficient that can vary by as much as 50 %.

For clays, Reference [19] provides an empirical relationship which depends on the overconsolidation ratio, the void ratio, the average effective confining stress, and an empirical constant that depends on the plasticity index. Results presented in Reference [20] for onshore sites show that the value of  $G_{\max}$  ranges from about 1 000 times to 3 000 times the undrained shear strength ( $c_u$ ) of the soil for cases where the undrained shear strength is based on *in-situ* field tests, consolidated undrained laboratory tests, or unconsolidated laboratory tests corrected for sample disturbance. Experience with offshore clays indicates that  $G_{\max}$  could range from 600 times to 1 500 times the undrained shear strength.

The values presented in Table 6 and Table 7 are representative of the motion close to the sea floor<sup>[12]</sup>. For deep pile foundations, the effective horizontal and vertical input motions for dynamic analysis would occur at a lower depth. Therefore, the effective motions can be significantly lower than those listed in Table 6 and Table 7. For deep pile foundations, the soil amplification factors,  $C_v$  and  $C_a$ , are as recommended in Table 8. The values in Table 8 are independent of the intensity of the motion<sup>[21]</sup>.

### A.7.2 Seismic action procedure

The detailed seismic action procedure is described in Clause 8. This procedure involves a number of steps and associated checks to ensure that the objectives of the procedures are met. The simplified seismic action procedure is derived from the detailed procedure by simulations, using a range of input parameters and appropriately averaging the results. The main points of this derivation are briefly summarized below.

In the simplified seismic action procedure, the design is based on seismic maps depicting spectral accelerations with a return period of 1 000 years instead of on a probabilistic seismic hazard analysis (PSHA). In order to generate the ALE spectral acceleration from these maps, two steps are required:

- a) the spectral acceleration is changed from a return period of 1 000 years to a return period of  $1/P_f$  to match the target probability of failure;
- b) an ALE correction factor,  $C_c$ , is applied to the spectral acceleration corresponding to a return period of  $1/P_f$ , (see Clause 8 for details).

The factor  $C_c$  accounts for uncertainties not captured in a seismic hazard curve which can affect the reliability of an offshore structure, e.g. the uncertainty in structural resistance to earthquake actions. In developing the simplified seismic action procedure, these two steps were simulated using the target probabilities in Table 2 and a wide range of seismic hazard slopes. From these results, average scale factors,  $N_{\text{ALE}}$ , were calculated that combined the effects of the two steps; these scale factors are listed in Table 9. Therefore, the designer should be aware that the scale factors listed in Table 9 are consistent with the target probabilities listed in Table 2.

In the simplified seismic action procedure, the designer does not explicitly check against the minimum recommended ELE return periods in Table 11 (see Clause 8). In developing the simplified seismic action procedure, the ELE return period was simulated for target probabilities listed in Table 2, a range of seismic hazard slopes, and a range of  $C_r$  values. The resultant ELE return periods were then checked against the minimum values listed in Table 11 to ensure that they are higher than the minimum return periods listed in Table 11. Based on these results, maximum values of  $C_r$  allowed are:

- 2,8 for L1 structures;
- 2,4 for L2 structures;
- 2,0 for L3 structures.

## A.8 Detailed seismic action procedure

### A.8.1 Site-specific seismic hazard assessment

No guidance is offered.

### A.8.2 Probabilistic seismic hazard analysis

The background to the PSHA procedure and the different elements have been developed in Reference [22]. The basic approach to probabilistic seismic hazard assessment, PSHA, is described in References [23] to [27]. The PSHA is typically undertaken using special computer programs with input parameters that include the following.

- Definition of earthquake sources, either as faults or as area sources of diffused seismicity not directly attributable to a known fault. Also a maximum magnitude is assigned to each source.
- An annual frequency of earthquake occurrence as a function of magnitude, for each source.
- A definition of earthquake ground motion attenuation, including a probability distribution (typically log-normal) representing the uncertainty of the predicted ground motion at a site. The attenuation relationships are developed based on statistical analyses of historical ground motion records from earthquakes occurring in similar geological and tectonic conditions.

In a PSHA the probabilities associated with ground motion values are calculated by combining the probabilities of ground motion from many sources. Therefore the ground motion probabilities are not associated with a specific fault or event. In fact, while it sounds conservative to use the expected ground motion from the largest possible earthquake occurring at the closest location on the nearest fault, those values can be significantly smaller than ground motions calculated from a probabilistic method. This possible outcome is particularly true if the largest earthquake on the nearest fault is associated with a shorter return period than being considered in a probabilistic method, or if the site is affected by several faults, each contributing to the overall probability of exceedance. The opposite outcome is possible when the return period of the largest earthquake on the nearest fault is much greater than the desired return period of the ground motion.

The PSHA procedure can be applied for the prediction of both horizontal and vertical components of ground motion. As an alternative, the vertical component of the ground motion may be estimated based on established relationships for the ratio of vertical to horizontal spectral accelerations.

The relationship between the average return period (or inverse of the average recurrence rate) and the target annual probability of exceedance for a Poisson process is:

$$T_{\text{return}} = \frac{-1}{\ln(1 - P_e)} \quad (\text{A.2})$$

At the probabilities of failure being considered for seismic design, the difference between Equation (8) and Equation (A.2) is negligible.

### A.8.3 Deterministic seismic hazard analysis

No guidance is offered.

### A.8.4 Seismic action procedure

Given a target annual probability of failure equal to  $P_f$ , the annual probability of the ALE event should be lower than  $P_f$  and the corresponding return period of the ALE event should be greater than  $1/P_f$ . Such an increase in the ALE return period is needed to cover the randomness and uncertainties in seismic actions and structure resistance; these uncertainties are not captured in the seismic hazard curve and invariably increase the probability of failure. The associated increase in the ALE return period will primarily depend on two factors:

- the relative importance of these additional uncertainties (expressed by the logarithmic standard deviation,  $\sigma_{LR}$ );
- the slope of the seismic hazard curve at  $P_f$ , ( $a_R$ ).

The procedure developed in Reference [28] has been used to calculate a spectral acceleration correction factor ( $C_c$ ) which would guarantee a failure probability of  $P_f$  for the design of a structure meeting the ALE requirements. In the detailed seismic action procedure, the correction factor is applied on the mean spectral acceleration for  $T = T_{dom}$  with an exceedance probability equal to  $P_f$ . Table A.1 shows the correction factor as a function of both  $\sigma_{LR}$  and the seismic hazard slope ( $a_R$ ). A value of  $\sigma_{LR} = 0,3$  is judged to be representative of the uncertainties that are not captured in the seismic hazard curve, e.g. the uncertainty in displacement capacity of a non-linear system. These values of the correction factor  $C_c$  are the basis for the rounded values in Table 10. It should be noted that uncertainties can vary between traditionally framed fixed steel offshore structures, gravity based fixed concrete offshore structures and other offshore structure concepts. In certain cases where the calculation of seismic actions or the structure's resistance are more uncertain, higher values of the correction factor  $C_c$  should be considered. Alternatively appropriate adjustment factors (e.g. amplifying accelerations or displacement demands) can be derived for and applied to those structural components with greater uncertainties.

**Table A.1 — Correction factor  $C_c$  for ALE spectral acceleration**

Value of $\sigma_{LR}$	Correction factor for $a_R$ equal to:				
	1,75	2,0	2,5	3,0	3,5
0	1,00	1,00	1,00	1,00	1,00
0,2	1,08	1,07	1,05	1,04	1,04
0,3	1,20	1,16	1,12	1,10	1,09
0,4	1,35	1,28	1,20	1,18	1,16

Using the spectral acceleration correction factors recommended in Table 10 or Table A.1, one calculates the appropriate ALE spectral acceleration. The method in Reference [28] also allows one to calculate correction factors that are applied the other way round, i.e. on the annual probabilities of failure  $P_f$  instead of correction factors applied on spectral acceleration. Table A.2 lists the calculated correction factors on  $P_f$  as a function of the seismic hazard slope for  $\sigma_{LR} = 0,3$ . Also shown in Table A.2 (last column) are the required ALE return periods for L1 structures assuming an acceptable annual system probability of failure of  $1/2\ 500$ .

**Table A.2 — Correction factor on  $P_f$**

$a_R$	$P_f$ correction	ALE return period <sup>a</sup> $P_f = 1/2\ 500$
1,75	2,12	5 300
2,0	1,59	4 000
2,5	1,33	3 300
3,0	1,22	3 100
3,5	1,19	3 000

<sup>a</sup> The resultant ALE return period assumes an L1 structure with  $P_f = 4 \times 10^{-4}$ .

In both simplified and detailed seismic action procedures, the ELE return period is determined such that a balance exists between the ELE and ALE designs. Having this balance, a structure designed to the ELE should have a high likelihood of meeting the ALE design demand. This criterion reduces costly design cycles and meets the safety objective of the ALE.

In order to determine the ELE design event, the appropriate ALE spectral acceleration is reduced by the seismic reserve capacity factor ( $C_r$ ) that represents the available margin of safety for events beyond the ELE. The ELE safety margin is due to the following:

- the explicit safety factors in design equations used in the design of a structure's components;
- the implicit safety margins in the design of a structure's components, e.g. the difference between nominal and best estimate material strength;
- the robustness and redundancy of the structural system;
- the ability of the structural system to sustain large non-linear deformations.

Because the seismic reserve capacity factor,  $C_r$ , has to be established prior to performing the seismic design, the above margins of safety have to be estimated from the general knowledge of the material used, the design process, and the structure's configuration. For fixed steel structures, the margin of safety between the ALE and ELE can range from approximately 1,1 to 2,8. The lower values of  $C_r$  correspond to minimum structures with no redundancy and little or no ductility, while the higher values correspond to highly redundant and ductile designs.

In the detailed seismic action procedure, the designer may assume any value of  $C_r$  as long as an ALE analysis is performed to ensure that the design meets or exceeds the ALE requirements. A high estimate of  $C_r$  can lead to major modifications as a result of the ALE design check and thus costly design cycles. On the other hand, a low estimate of  $C_r$  can lead to a conservative design (more costly to build) that would easily meet the ALE design check.

The requirement of minimum ELE return periods in Table 11 should ensure that the design meets the economic objective of the ELE and that the structure is not susceptible to damage during more frequently occurring seismic events (see A.6.2.2). The minimum requirements in Table 11 also implicitly address the safety objective of a design meeting the ALE requirements. These requirements can control in regions where the slope of the seismic hazard curve, as defined by  $a_R$ , is low (see Figure 4).

### A.8.5 Local site response analyses

Numerical methods using linear or non-linear models of the underlying soil are available to estimate site-specific acceleration spectra. The site response analysis involves an evaluation of the propagation of seismic shear waves through a stack of soil layers of specified soil type, shear-wave velocity or shear modulus, total unit weight, and cyclic strain-softening characteristics<sup>[27]</sup>. The analysis requires the solution of equations of motion using strain-dependent dynamic properties of the layered soil column. Conventional analyses assess the effect of soil column on normally incident bedrock time histories in order to determine site-specific soil amplification spectra, time histories, and acceleration spectra at specified depths within the soil profile. There are several computer software applications that are commercially available and can be used for this purpose<sup>[29] to [36]</sup>.

The results of generic site response analyses, as well as analyses of historical motions recorded at soft soil sites, were used with judgement to select the amplification factors for different types of sites in the simplified procedure (7.1). However, it should be noted that amplification factors are desired at the point of action input into the structural system and not necessarily close to the sea floor. For deep pile foundations, the effective horizontal and vertical input motions would occur at lower depths. For example, the horizontal input motion may be assumed to be that at 1/3 of the pile length below the sea floor and the vertical input motion may be assumed to be that at the pile tip.

### A.9 Performance requirements

No guidance is offered.

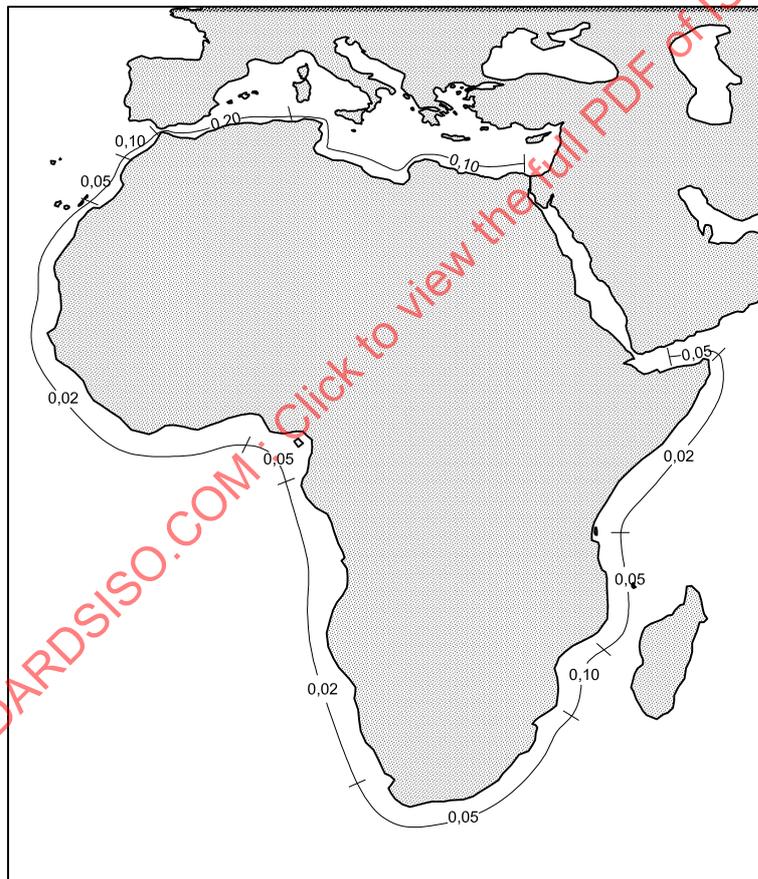
## Annex B (informative)

### Regional information

The maps shown in Figures B.1 to B.11 of this annex give generic 5 % damped spectral accelerations, expressed in  $g$ , for bedrock outcrop for a 1,0 s oscillator period and for a 0,2 s oscillator period, respectively, for determining the site seismic zone (see 6.4) of an area and for use in the simplified seismic action procedure (see Clause 7).

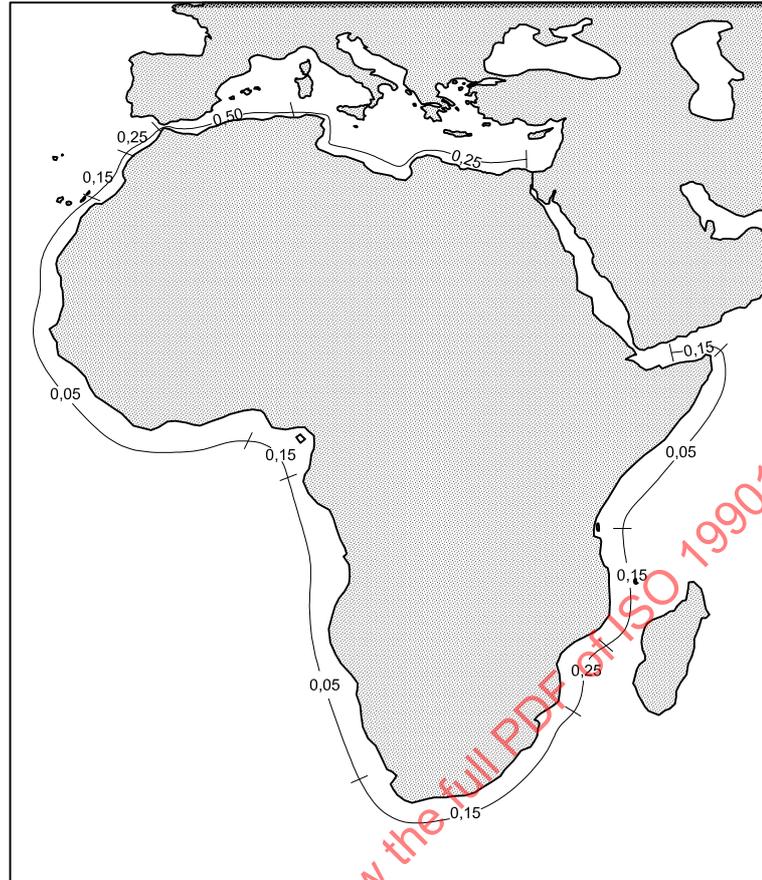
NOTE 1 The return period selected for the development of the ground motion maps in Annex B is 1 000 years.

NOTE 2 It is recognized that there is some uncertainty in the values given in this annex. This is due to lack of complete understanding or knowledge (epistemic or Type II uncertainties). The requirements of the standard are such that a site-specific assessment of the accelerations is required for any structure in which failure would have significant consequences and in which seismic considerations can affect the design.



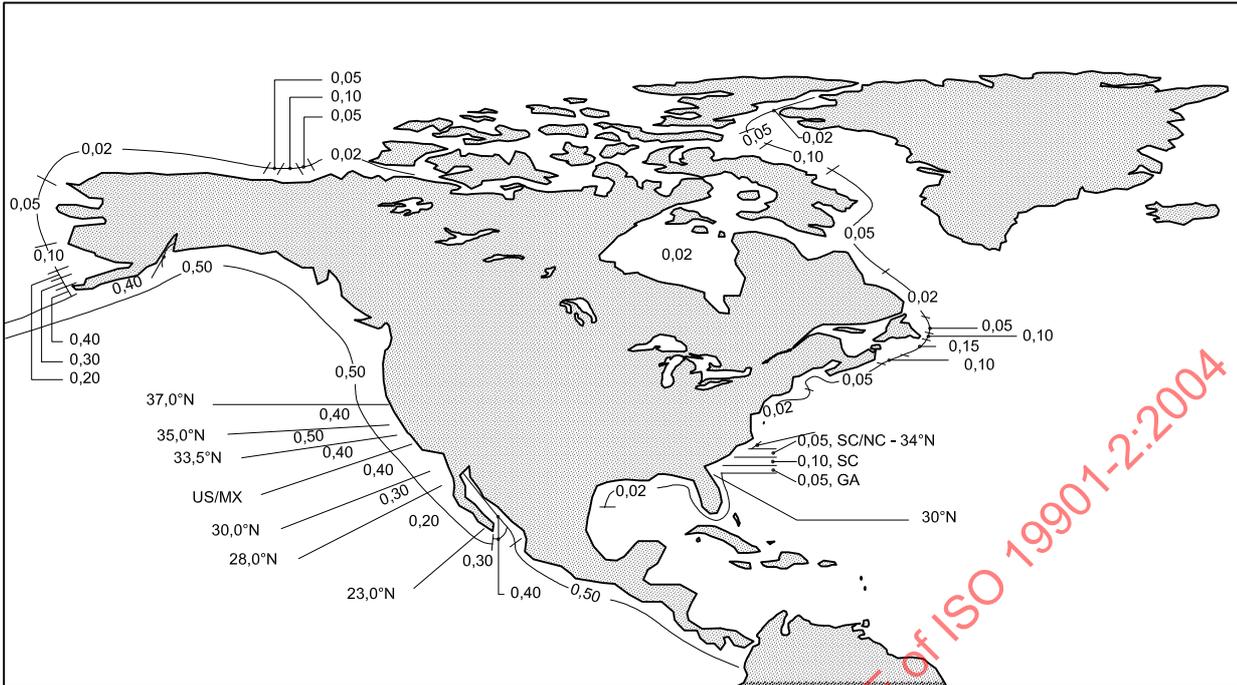
a) 1,0 s oscillator periods

Figure B.1 — 5 % damped spectral response accelerations for offshore Africa

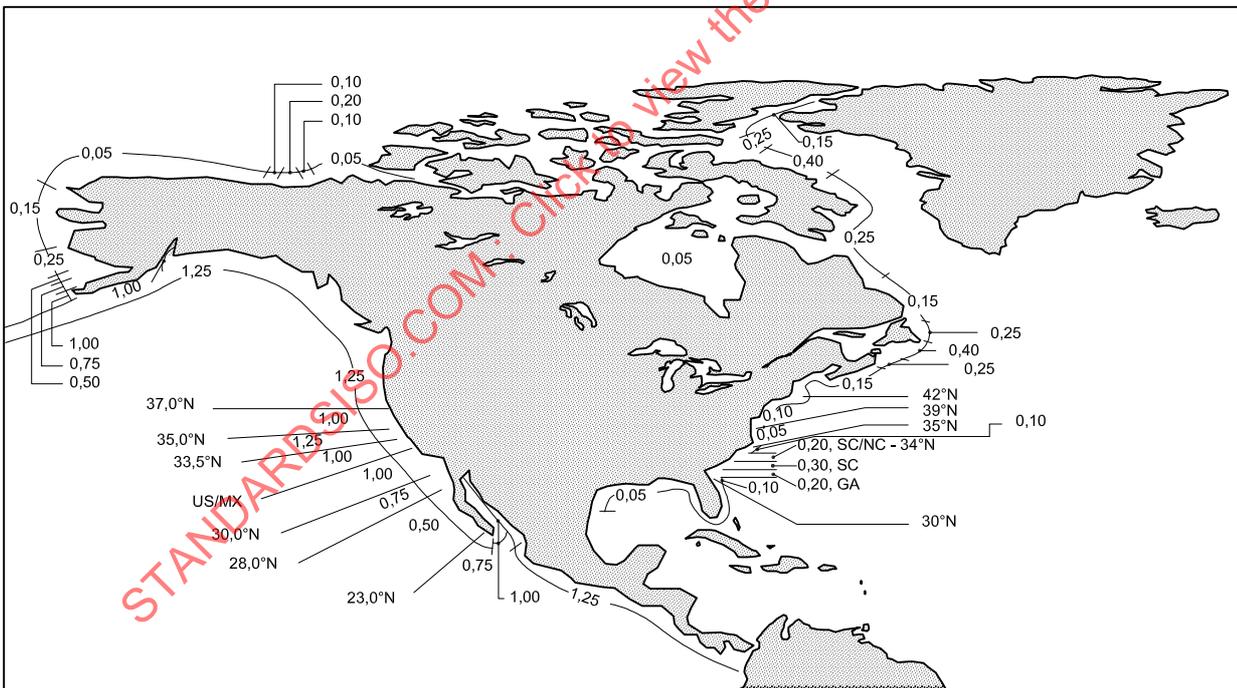


b) 0,2 s oscillator periods

Figure B.1 (continued)



a) 1,0 s oscillator periods



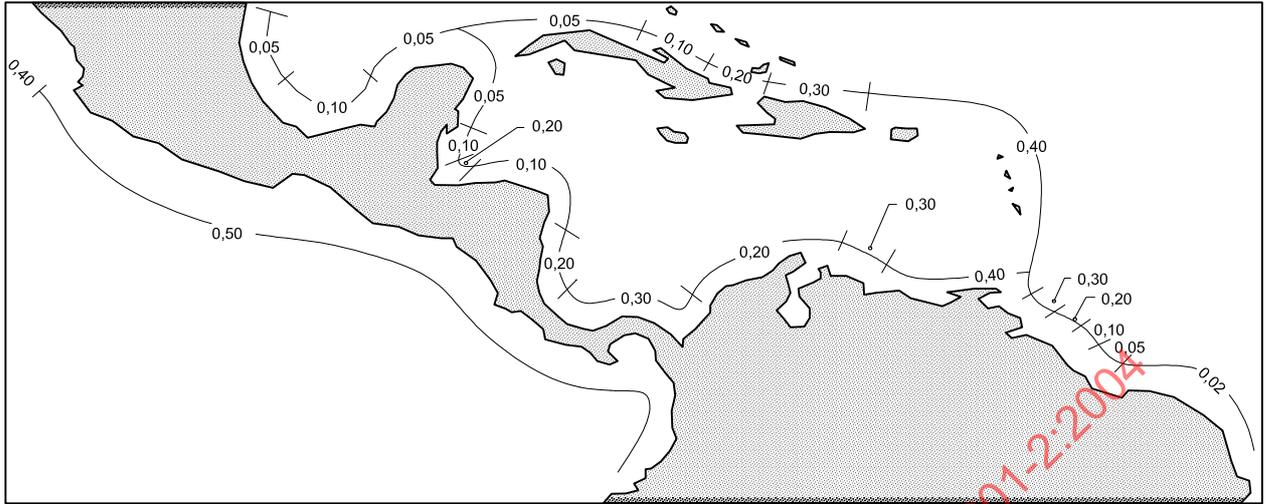
b) 0,2 s oscillator periods

**Key**

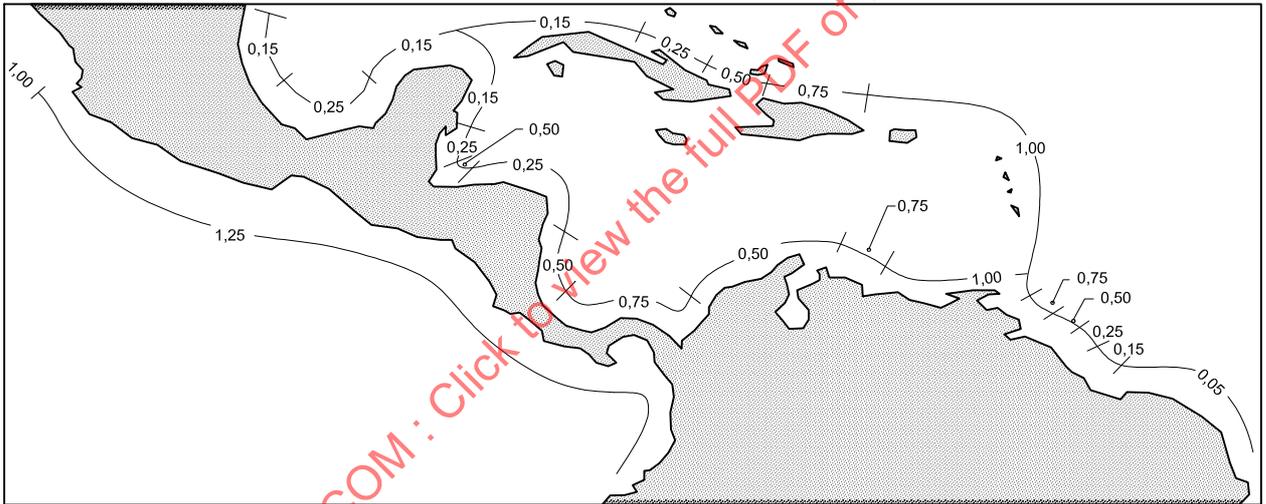
- |       |                                      |    |                |
|-------|--------------------------------------|----|----------------|
| US/MX | US — Mexico border                   | SC | South Carolina |
| SC/NC | South Carolina/North Carolina border | GA | Georgia        |

NOTE See also Reference [14] for offshore Canada.

**Figure B.2 — 5 % damped spectral response accelerations for offshore North America**



a) 1,0 s oscillator periods



b) 0,2 s oscillator periods

Figure B.3 — 5 % damped spectral response accelerations for offshore Central America



a) 1,0 s oscillator periods

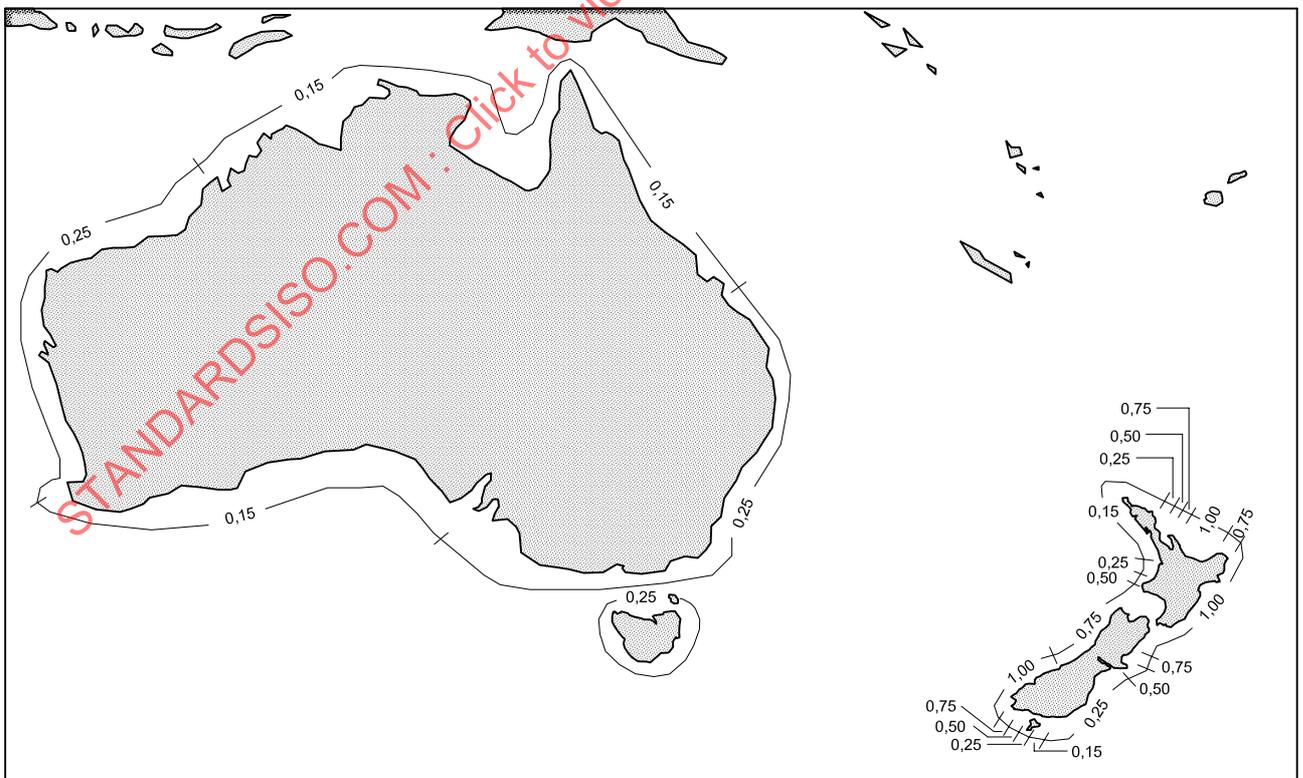


b) 0,2 s oscillator periods

Figure B.4 — 5 % damped spectral response accelerations for offshore South America



a) 1,0 s oscillator periods



b) 0,2 s oscillator periods

Figure B.5 — 5 % damped spectral response accelerations for offshore Australia and New Zealand