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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

ISO/CEN PARALLEL PROCESSING

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Contents

Foreword

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

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

This document 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,* in collaboration with the European Committee for Standardization (CEN) Technical Committee CEN/TC 12, *Materials, equipment and offshore structures for petroleum, petrochemical and natural gas industries,* in accordance with the Agreement on technical cooperation between ISO and CEN (Vienna Agreement).

This third edition cancels and replaces the second edition (ISO 19901-2:2017), which has been technically revised.

The main changes are as follows:

— the seismic hazard maps have been updated;

— Clause 3 has been updated.

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

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

Introduction

The International Standards on offshore structures prepared by TC 67 (i.e. ISO 19900, ISO 19902, ISO 19903, ISO 19904 and ISO 19906) 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 type of structure and the nature or combination of the materials used.

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

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

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

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

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

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

Part 2: Seismic design procedures and criteria

1 Scope

This document contains requirements for defining the seismic design procedures and criteria for offshore structures; guidance on the requirements is included in Annex A. The requirements focus on fixed steel offshore structures and fixed concrete offshore structures. The effects of seismic events on floating structures and partially buoyant structures are briefly discussed. The site-specific assessment of jack-ups in elevated condition is only covered in this document 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 exposure level of the offshore structure 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 abnormal limit state (ALS) to ensure that it meets reserve strength and energy dissipation requirements.

Procedures and requirements for a site-specific probabilistic seismic hazard analysis (PSHA) are addressed for offshore structures in high seismic areas and/or with high exposure levels. However, a thorough explanation of PSHA procedures is not included.

Where a simplified design approach is allowed, worldwide offshore maps, which are included in Annex B, show the intensity of ground shaking corresponding to a return period of 1 000 years. In such cases, these maps can be used with corresponding scale factors to determine appropriate seismic actions for the design of a structure, unless more detailed information is available from local code or site-specific study.

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. Seismic requirements for floating structures are contained in ISO 19904, for site-specific assessment of jack-ups and other MOUs in the ISO 19905 series, for arctic structures in ISO 19906 and for topsides structures in ISO 19901-3.

2 Normative references

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

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

ISO 19901-8, *Petroleum and natural gas industries — Specific requirements for offshore structures – Part 8: Marine soils Investigation*

ISO 19902, *Petroleum and natural gas industries — Fixed steel offshore structures*

ISO 19903, *Petroleum and natural gas industries — 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.

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

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

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

3.1
abnormal level earthquake
ALE
intense earthquake of abnormal severity with a very low probability of occurring during the life of the structure

Note 1 to entry: The ALE event is comparable to the abnormal event in the design of fixed structures that are described in ISO 19902 and ISO 19903.

3.2
attenuation
decay of seismic waves as they travel from the earthquake source to the site under consideration

3.3
deaggregation
separation of seismic hazard contribution from different faults and seismic source zones

3.4
escape and evacuation system
system provided on the offshore structure to facilitate escape and evacuation in an emergency

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

3.5
extreme level earthquake
ELE
strong earthquake with a reasonable probability of occurring during the life of the structure

Note 1 to entry: The ELE event is comparable to the extreme environmental event in the design of fixed structures that are described in ISO 19902 and ISO 19903.

3.6
fault movement
movement occurring on a fault during an earthquake

3.7
ground motion
accelerations, velocities or displacements of the ground produced by seismic waves radiating away from earthquake sources

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

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

3.8
liquefaction
fluidity of soil due to the increase in pore pressures caused by earthquake action under undrained conditions

3.9
modal combination
combination of response values associated with each dynamic mode of a structure

3.10
mud volcano
diapiric intrusion of plastic clay causing high pressure gas-water seepages which carry mud, fragments of rock (and occasionally oil) to the surface

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

3.11
probabilistic seismic hazard analysis
PSHA
framework permitting the identification, quantification and rational combination of uncertainties in earthquakes' intensity, location, rate of recurrence and variations in *ground motion* (3.7) characteristics

3.12
probability of exceedance
probability that a variable (or that an event) exceeds a specified reference level given exposure time

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

3.13
response spectrum
function representing the peak elastic response for single degree of freedom oscillators with a specific damping ratios in terms of absolute acceleration, pseudo velocity, or relative displacement values against natural frequency or period of the oscillators

3.14
safety system
systems provided on the offshore structure to detect, control and mitigate hazardous situations

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

3.15
sea floor
interface between the sea and the *seabed* (3.17)

3.16
seabed slide
failure of *seabed* (3.17) slopes

3.17
seabed
soil material below the sea in which a structure is founded

3.18
seismic risk category
SRC
category defined from the exposure level and the expected intensity of seismic motions

3.19
seismic hazard curve
curve showing the annual *probability of exceedance* (3.12) against a measure of seismic intensity

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

3.20
seismic reserve capacity factor
factor indicating the structure's ability to sustain ground motions due to earthquakes beyond the level of the *extreme level earthquake* (3.5)

Note 1 to entry: The seismic reserve capacity factor is a structure specific property that is used to determine the extreme level earthquake acceleration from the *abnormal level earthquake* (3.1) acceleration.

3.21
site response analysis
wave propagation analysis permitting the evaluation of the effect of local geological and soil conditions on the *ground motions* (3.7) as they propagate up from depth to the surface at the site

3.22
spectral acceleration
maximum absolute acceleration response of a single degree of freedom oscillator subjected to *ground motions* (3.7) due to an earthquake

3.23
spectral velocity
maximum pseudo velocity response of a single degree of freedom oscillator subjected to *ground motions* (3.7) due to an earthquake

Note 1 to entry: The pseudo velocity spectrum is computed by factoring the displacement or acceleration spectra by the oscillator's circular frequency or the inverse of its frequency, respectively. The pseudo spectrum is either relative or absolute, depending on the type of response spectra that is factored.

3.24
spectral displacement
maximum relative displacement response of a single degree of freedom oscillator subjected to *ground motions* (3.7) due to an earthquake

3.25
static pushover analysis
application and incremental increase of a global static pattern of actions on a structure, including equivalent dynamic inertial actions, until a global failure mechanism occurs

3.26
tsunami
long period sea waves caused by rapid vertical movements of the *sea floor* (3.15)

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

4 Symbols and abbreviated terms

4.1 Symbols

a_R slope of the seismic hazard curve

C_a site coefficient, a correction factor applied to the acceleration part (shorter periods) of a response spectrum

C_c	correction factor applied to the spectral acceleration to account for uncertainties not captured in a seismic hazard curve
C_r	seismic reserve capacity factor; see Formulae (7) and (10)
C_v	site coefficient, a correction factor applied to the velocity part (longer periods) of a response spectrum
D	scaling factor for damping
G_{max}	initial (small strain) shear modulus of the soil
g	acceleration due to gravity
M	magnitude of an earthquake measured by the energy released at its source
N_{ALE}	scale factor for conversion of the site 1 000-year acceleration spectrum to the site ALE acceleration spectrum
p_a	atmospheric pressure
P_{ALE}	annual probability of exceedance for the ALE event
P_e	probability of exceedance
P_{ELE}	annual probability of exceedance for the ELE event
P_f	target annual probability of failure
q_c	cone penetration resistance of soil
q_{cl}	normalized cone penetration resistance of soil
$\bar{q}_{cl}$	average normalized cone penetration resistance of sand in the effective seabed
$S_a(T)$	spectral acceleration associated with a single degree of freedom oscillator period, T
$\bar{S}_a(T)$	mean spectral acceleration associated with a single degree of freedom oscillator period, T; obtained from a PSHA
$S_{a,ALE}(T)$	ALE spectral acceleration associated with a single degree of freedom oscillator period, T
$\bar{S}_{a,ALE}(T)$	mean ALE spectral acceleration associated with a single degree of freedom oscillator period, T; obtained from a PSHA
$S_{a,ELE}(T)$	ELE spectral acceleration associated with a single degree of freedom oscillator period, T
$\bar{S}_{a,ELE}(T)$	mean ELE spectral acceleration associated with a single degree of freedom oscillator period, T; obtained from a PSHA
$S_{a,map}(T)$	1 000-year rock outcrop spectral acceleration obtained from maps associated with a single degree of freedom oscillator period, T
$\bar{S}_{a,Pe}(T)$	mean spectral acceleration associated with a probability of exceedance, P_e, and a single degree of freedom oscillator period, T, obtained from a PSHA
$\bar{S}_{a,Pf}(T)$	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

s_u undrained shear strength of the soil

$\bar{s}_u$ average undrained shear strength of the soil in the effective seabed

T natural period of a simple, single degree of freedom oscillator

T_{dom} dominant modal period of the structure

T_{return} return period

v_s representative shear wave velocity

$\bar{v}_s$ average of representative shear wave velocity in the effective seabed

ρ mass density of soil

η per cent of critical damping

σ_{LR} logarithmic standard deviation of uncertainties not captured in a seismic hazard curve

σ'_{v0} in situ vertical effective stress of soil

4.2 Abbreviated terms

L1, L2, L3 exposure level derived in accordance with the International Standard applicable to the type of offshore structure

MOU mobile offshore unit

PGA peak ground acceleration

TLP tension leg platform

ULS ultimate limit state

5 Earthquake hazards

Actions and action effects due to seismic events shall be evaluated 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 of indicative seismic accelerations; however, for many areas, depending on indicative accelerations and exposure levels, seismicity shall be determined on the basis of detailed seismic hazard investigations (see Clause 8).

Evaluation of seismic events for seismically active regions shall include investigation of the characteristics of ground motions and of 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, when warranted, should be addressed by special studies (e.g. mudflow loading, seabed deformation). The following hazards can be caused by a seismic event:

— soil liquefaction;

— seabed 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 (e.g. simultaneous seabed and structure excitation, spatially varying motions).

6 Seismic design principles and methodology

6.1 Design principles

This clause addresses the design of structures to the ultimate limit state (ULS) for frequent earthquakes (ELE) and to the abnormal limit state (ALS) for rare earthquakes (ALE).

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. It is recommended that the structure 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 can be modified to meet targets set by owners in consultation with regulators, or to meet regional requirements where they exist. Regional requirements for select regions are found in Annex C.

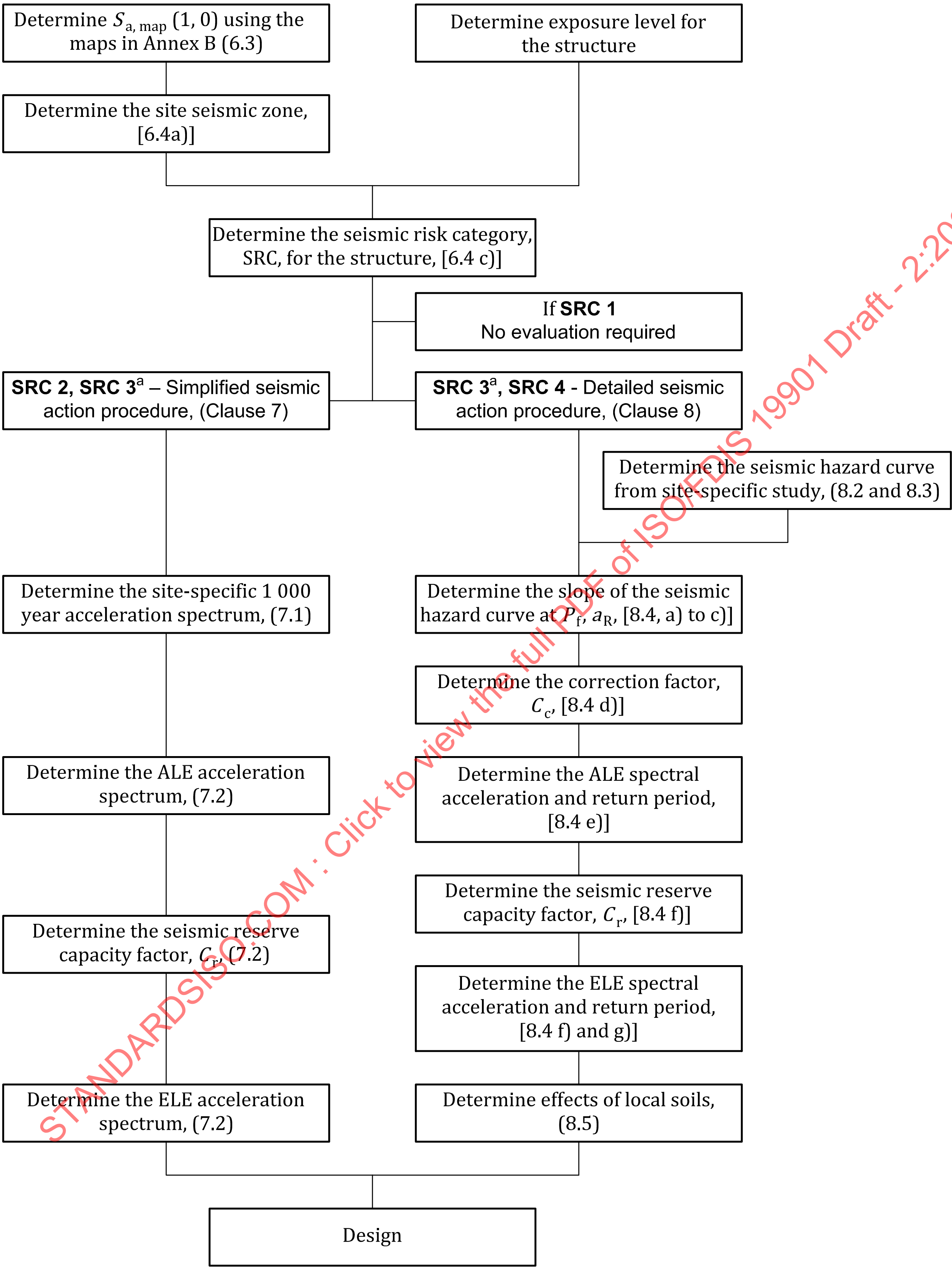
6.2 Seismic design procedures

6.2.1 General

Two procedures for seismic design are provided: a simplified method and a detailed method. The simplified method may be used where seismic considerations are unlikely to govern the design of a structure. 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 (see Clause 7) allows the use of generic seismic maps provided in Annex B; while the detailed procedure (see 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.

When a structural design is asymmetric in geometric configuration or directional capacity, additional analyses shall be included to demonstrate suitable performance in weaker directions. For time history analyses, this can require different orientations of the earthquake horizontal records to demonstrate performance requirements (see Clause 9).

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



[a] SRC 3 structures may be designed using either the simplified or the detailed seismic action procedure (see Table 4).

Figure 1 — Seismic design 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 methods of analysis may be used for the ELE design check:

a) the response spectrum analysis method;

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. Damping compatible with the ELE deformation levels should be used in the ELE design, as guided by the recommendations in the relevant International Standards on offshore structures prepared by TC 67 (see Introduction) Higher values of damping due to hydrodynamics or soil deformation (hysteretic and radiation) may be used; however, the damping used 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. The complete quadratic combination (CQC) method can be used to capture the correlation between closely spaced modes. Sufficient modes should be included in the modal combination to obtain at least 90 % structural mass participation in each horizontal direction. 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 square 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 four 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. Satisfactory performance shall be achieved for either the greater of four or half the total sets of time history records. Satisfactory performance of a given time history record constitutes all code utilizations being less than or equal to 1,0.

Equipment on the deck shall be designed to withstand motions that account for the transmission of ground motions through the structure. The structure can amplify the ground motion such that the deck accelerations are much higher than the earthquake excitation. The time history analysis method shall be used for obtaining deck motions (especially relative motions) and deck motion response spectra (typically absolute acceleration spectra).

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

6.2.3 Abnormal level earthquake design

In high seismic areas, it is uneconomic to design a structure such that the ALE event would be resisted without 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 representative 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 methods of may be used for the ALE design check:

a) the static pushover or extreme displacement method;

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 and is recommended for ALE analysis. A minimum of four 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 seven or more time history records are used, global structure survival shall be demonstrated in half or more of the time history analyses (see 9.2). If fewer than seven time history records are used, global survival shall be demonstrated in at least four 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 structural 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 (see Clause 7) and to determine the seismic risk category (see 6.4). For each area, two maps are presented in Annex B:

— 0,2 s period;

— 1,0 s period.

The acceleration values are expressed in g and correspond to 5 % damped spectral accelerations on rock outcrop, defined as site class A/B in Table 5. 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) To determine the site seismic zone (see 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
Seismic zone	0	1	2	3	4

b) To determine the structure's exposure level [see the relevant International Standards on offshore structures prepared by TC 67 (see Introduction)]. 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. Since it is not possible to evacuate prior to an earthquake, the manned-evacuated L2 condition is not allowed. All manned offshore structures shall be categorized as L1 for seismic actions. For unmanned medium consequence offshore structures, exposure level, L2, should be used only when the offshore structure is manned at the minimum amount of time possible. Offshore structures that are manned on an interim basis, for example, daylight hours only, should be considered manned.

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) To 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		
	L1	**L2**	**L3**
0	SRC 1	SRC 1	SRC 1
1	SRC 3	SRC 2	SRC 2
2	SRC 4	SRC 2	SRC 2
3	SRC 4	SRC 3	SRC 2
4	SRC 4	SRC 4	SRC 3

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 recategorized as SRC 2.

6.5 Seismic design requirements

Table 4 gives the seismic design requirements for each SRC. Figure 1 presents an overview of the seismic design process and expands the steps associated with the development of seismic ALE and ELE spectra for the simplified and detailed procedures.

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, the primary structure configuration and joint detailing should follow the requirements and recommendations for ductile design for all SRC except SRC 1 (good practice, but not required for SRC 1). See the relevant International Standards on offshore structures prepared by TC 67 (see Introduction).

For floating structures, only the ALE should be considered.

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[a]	Simplified	Site-specific, ISO maps or regional maps	Recommended
	Detailed	Site-specific	Recommended
4	Detailed	Site-specific	Required

[a] 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).

6.6 Site investigation

Soil data used in seismic analyses shall be acquired in accordance with ISO 19901-8.

7 Simplified seismic action procedure

7.1 Soil classification and spectral shape

Having obtained the rock outcrop spectral accelerations at oscillator periods of 0,2 s and 1,0 s, $S_{a,map}(0,2)$ and $S_{a,map}(1,0)$ (see Annex B), the site response spectrum corresponding to a return period of 1 000 years shall be determined as follows.

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 effective seabed (see Table 5). The effective seabed can normally be taken as the top 30 m of the seabed. Alternatively, rational analysis can be used to define the effective seabed. For structures supported by embedded foundations, this can be taken as the average of the top 30 m below the vertical location of the footing (e.g. spudcan of a jack-up). For structures supported on driven piles or suction caissons, this can be taken as the top 30 m below the sea floor.

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

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

where

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

d_i is the thickness of layer, i;

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

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

Table 5 — Determination of site class

Site class	Soil profile name	Average properties in the effective seabed		
		Soil shear wave velocity $\bar{v}_s$ m/s	Cohesionless: normalized cone penetration resistance $\bar{q}_{cl}$ [a]	Cohesive: soil undrained shear strength $\bar{s}_u$ 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	$350 < \bar{v}_s \leq 750$	$\bar{q}_{cl} \geq 200$	$\bar{s}_u \geq 200$
D	Stiff to very stiff soil	$180 < \bar{v}_s \leq 350$	$80 \leq \bar{q}_{cl} < 200$	$80 \leq \bar{s}_u < 200$
E	Soft to firm soil	$120 < \bar{v}_s \leq 180$	$\bar{q}_{cl} < 80$	$\bar{s}_u < 80$
F	—	Any profile, including those otherwise classified as A to E, containing soils having one or more of the following characteristics: $\bar{v}_s \leq 120$; soils vulnerable to potential failure or collapse under seismic actions such as liquefiable soils, highly sensitive clays, collapsible weakly cemented soils; ooze[b] with a thickness of more than 10 m; soil layers with high gas content or ambient excess pore pressure greater than 30 % of in situ effective overburden; layers greater than 2 m thick with sharp contrast in shear wave velocity (greater than ±30 %) and/or undrained shear strength (greater than ±50 %) compared to adjacent layers.		

[a] $q_{cl} = (q_c/p_a) \times (p_a / \sigma'_{v0})^{0,5}$

where

q_c is the cone penetration resistance;

p_a is atmospheric pressure = 100 kPa;

σ'_{v0} is the vertical effective stress.

[b] 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, respectively[46]. 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	$S_{a,map}(0,2)$					
	≤0,25 *g*	0,50 *g*	0,75 *g*	1,0 *g*	1,25 *g*	≥1,5 *g*
A/B	0,9	0,9	0,9	0,9	0,9	0,9
C	1,3	1,3	1,2	1,2	1,2	1,2
D	1,6	1,4	1,2	1,1	1,0	1,0
E	2,4	1,7	1,3	1,1	1,0	0,8
F	[a]	[a]	[a]	[a]	[a]	[a]

[a] A site-specific marine soil 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	$S_{a,map}(1,0)$					
	≤0,1 *g*	0,2 *g*	0,3 *g*	0,4 *g*	0,5 *g*	≥0,6 *g*
A/B	0,8	0,8	0,8	0,8	0,8	0,8
C	1,5	1,5	1,5	1,5	1,5	1,4
D	2,4	2,2	2,0	1,9	1,8	1,7
E	4,2	3,3	2,8	2,4	2,2	2,0
F	[a]	[a]	[a]	[a]	[a]	[a]

[a] A site-specific marine soil 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]

[a] A site-specific marine soil 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 using Formula (2):

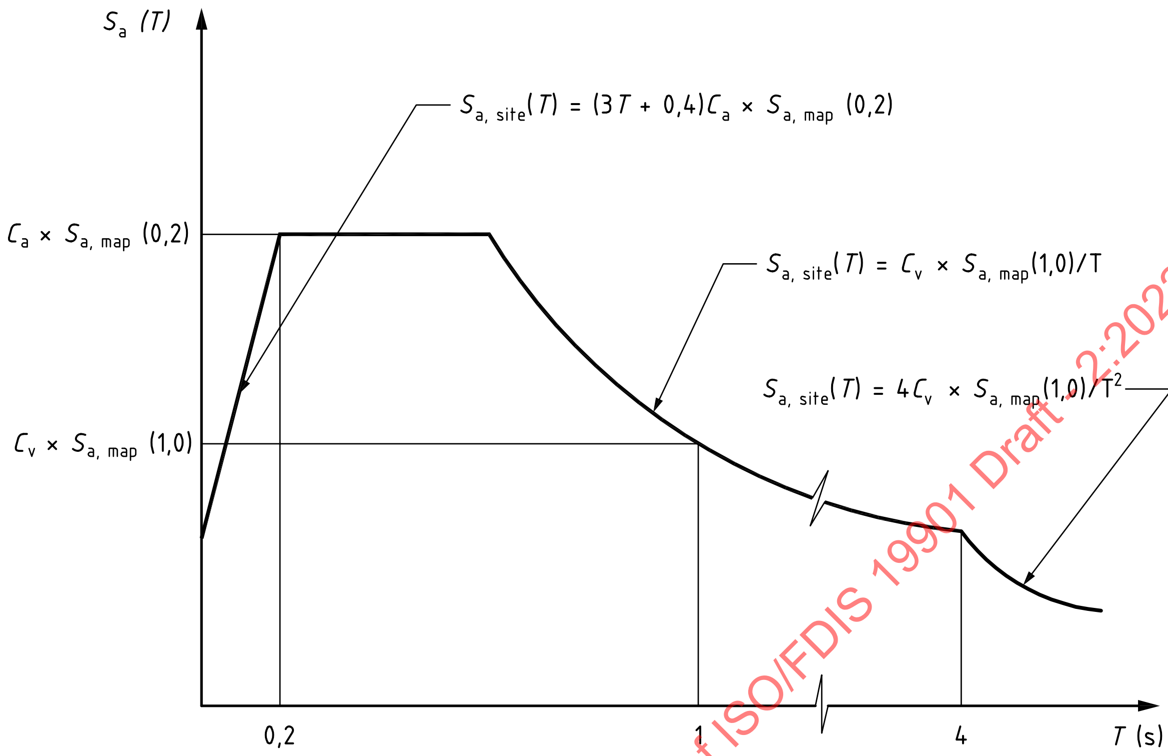
$$S_{a,site}(T) = (3T + 0{,}4) C_a \times S_{a,map}(0{,}2) \quad (2)$$

3) For periods greater than 0,2 s and less than or equal to 4,0 s, the site spectral acceleration, $S_{a,site}(T)$, shall be taken using Formula (3):

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

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

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

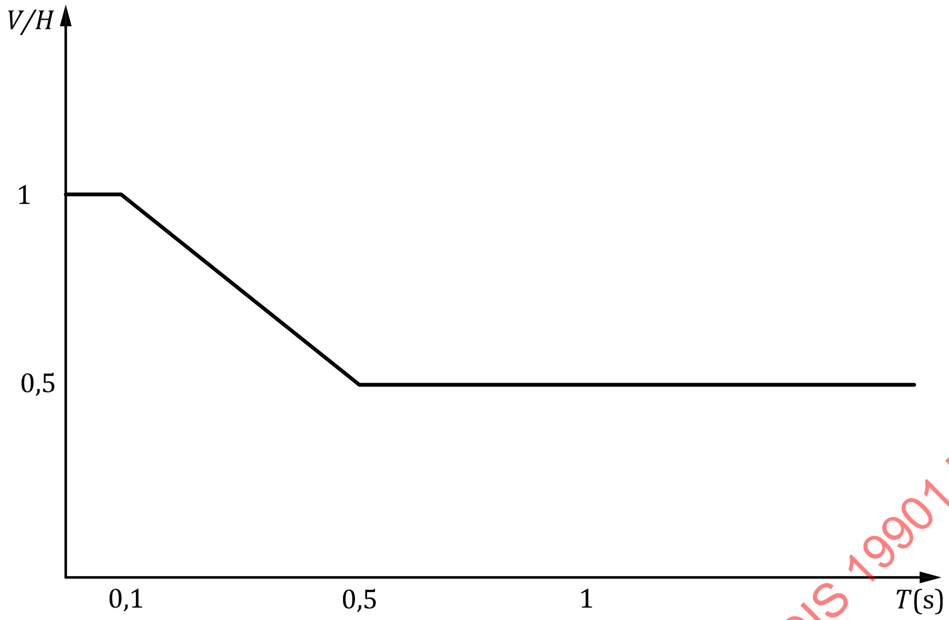


Key

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 design response spectrum for 5 % damping

d) For seismic zones 0, 1 and 2 (see Table 1), the site ELE and ALE vertical spectral acceleration at a period, T, shall be taken as half the corresponding horizontal spectral acceleration. For seismic zones 3 and 4, the site ELE and ALE vertical spectral acceleration at a period, T, shall be taken as the product of the amplification function in Figure 3 times the corresponding horizontal acceleration spectral value (see Reference [37]). The vertical spectrum shall not be reduced further due to water depth effects.



Key

T natural period of a simple, single degree of freedom oscillator

V/H amplification function applied to horizontal spectral accelerations to derive vertical spectral accelerations

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

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, as shown in Formula (5):

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

— where η is the per cent 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)], as shown in Formula (6):

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

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 using Formula (7):

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

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 and concrete structures shall be determined in accordance with ISO 19902 and ISO 19903, respectively. Values of C_r other than those recommended in the relevant International Standards on offshore structures prepared by TC 67 (see Introduction) 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}
L1	1,60
L2	1,15
L3	0,85

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.

8.2 Probabilistic seismic hazard analysis

The different elements of a PSHA are shown graphically in Figure 4. 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 [see Figure 4 a)]. A PSHA also accounts for the randomness in attenuation of seismic waves travelling from a source to the site [see Figure 4 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 [see Figure 4 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 4 c)].

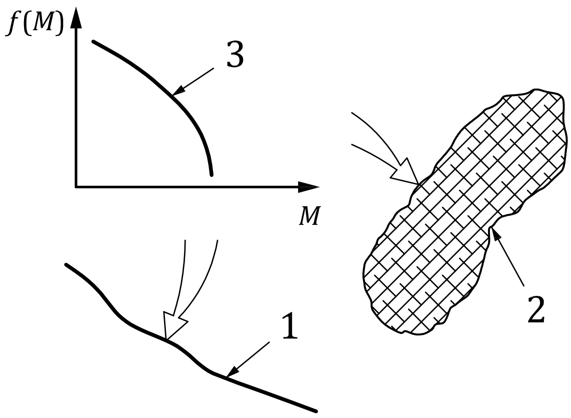
The results from a PSHA are used to derive a uniform hazard spectrum [see Figure 4 d)], where all points on the spectrum correspond to the same annual probability of exceedance. The relationship between the return period of a uniform hazard spectrum and the target annual probability of exceedance (P_e) may be taken as shown in Formula (8):

$$T_{return} = 1/P_e \tag{8}$$

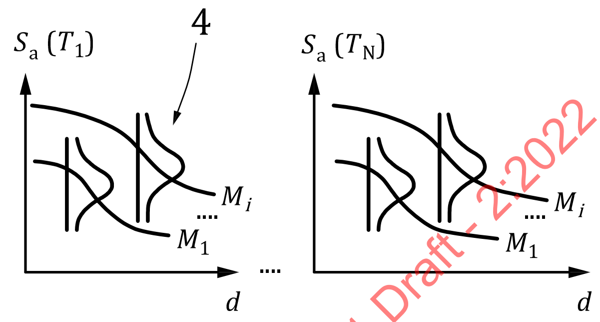
where T_{return} is the return period in years.

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.

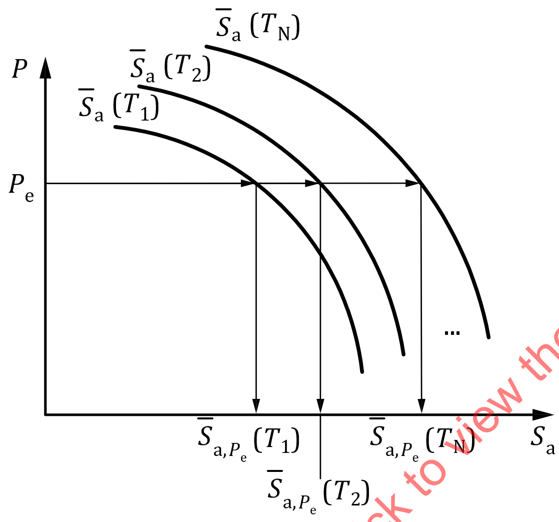
The results from a PSHA are a series of hazard curves each for a spectral acceleration corresponding to a structure natural period, e.g. T_1, T_2,...T_N [see Figure 4 c)]. Because of uncertainties in PSHA input parameters, each of these hazard curves has an uncertainty band. The mean (or expected value) of each hazard curve should be used to construct a uniform hazard spectrum corresponding to a given exceedance probability, P_e [see Figure 4 d)]. All references to hazard curves in 8.4 refer to the mean of the hazard curve. The PSHA enables an understanding of the different contributing faults through a magnitude- distance deaggregation, which can inform additional analyses.



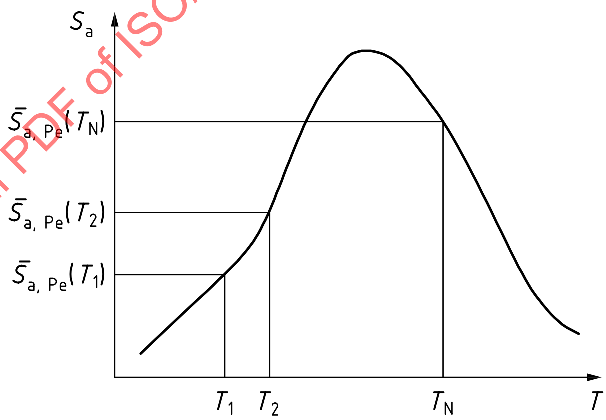
a) Earthquake source seismicity and geometry



b) Definition of attenuation curves for spectral accelerations at periods, T_1 ... T_N



c) Seismic hazard curves [from a) and b)] for spectral accelerations at each period and the selected target annual probability of exceedance and mean uniform hazard spectral accelerations, $\bar{S}_a\ (T_1)$ ... $\bar{S}_a\ (T_N)$



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

Key

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

Figure 4 — Probabilistic seismic hazard analysis overview

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 that 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 dimension (length, width, area, etc.) 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.

8.4 Seismic action procedure

This procedure is based on the results of a PSHA (see 8.2 and Figure 4). 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, $\bar{S}_{\mathrm{a}}\left(T_{\mathrm{dom}}\right)$; such curves are illustrated in Figure 4 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, $\bar{S}_{\mathrm{a}}(1{,}0)$.

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 5 and are described in the following steps:

a) Plot the site-specific hazard curve for $T = T_{\mathrm{dom}}$ [a curve such as those shown in Figure 4 c)] on a $\log_{10}$-$\log_{10}$ basis, i.e. showing the probability distribution of the parameter $\bar{S}_{\mathrm{a}}\left(T_{\mathrm{dom}}\right)$ [see Figure 5 a)].

b) 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}, $\bar{S}_{\mathrm{a},P_{\mathrm{f}}}\left(T_{\mathrm{dom}}\right)$ from Figure 5 a).

c) 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 5 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 5 a); P_2 should preferably be close to P_{f}].

d) 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

e) Determine the ALE spectral acceleration by applying the correction factor C_c to $\bar{S}_{a,P_f}(T_{dom})$, the site-specific spectral acceleration at the required P_f and the structural dominant period, T_{dom}, as shown in Formula (9):

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

The annual probability of exceedance for the ALE event (P_{ALE}) can then be directly read from the seismic hazard curve [see Figure 5 b)]. The ALE return period is determined from the annual probability of exceedance using Formula (8). P_{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).

f) For certain structure types whose reserve strength and ductility characteristics are known, the ELE spectral acceleration is next determined from Formula (10):

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

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 relevant International Standards on offshore structures prepared by TC 67 (see Introduction) may be used in design; however, such values shall be verified by an ALE analysis; see also Annex A.

g) The annual probability of exceedance for the ELE event, P_{ELE}, can now be read from the seismic hazard curve [see Figure 5 b)]. The ELE return period is determined from the annual probability of exceedance using Formula (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). Soil-structure interaction analyses can be performed.

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. For high seismic areas, extreme displacements and shock waves are often of primary interest in the design of a taught mooring system. The hull structure is designed elastically for the corresponding actions. Some consideration should be given to the effects of shock waves (see References [38] to [40]) on the facilities critical mechanical and electrical systems (e.g. power generation, emergency shutdown, etc.).

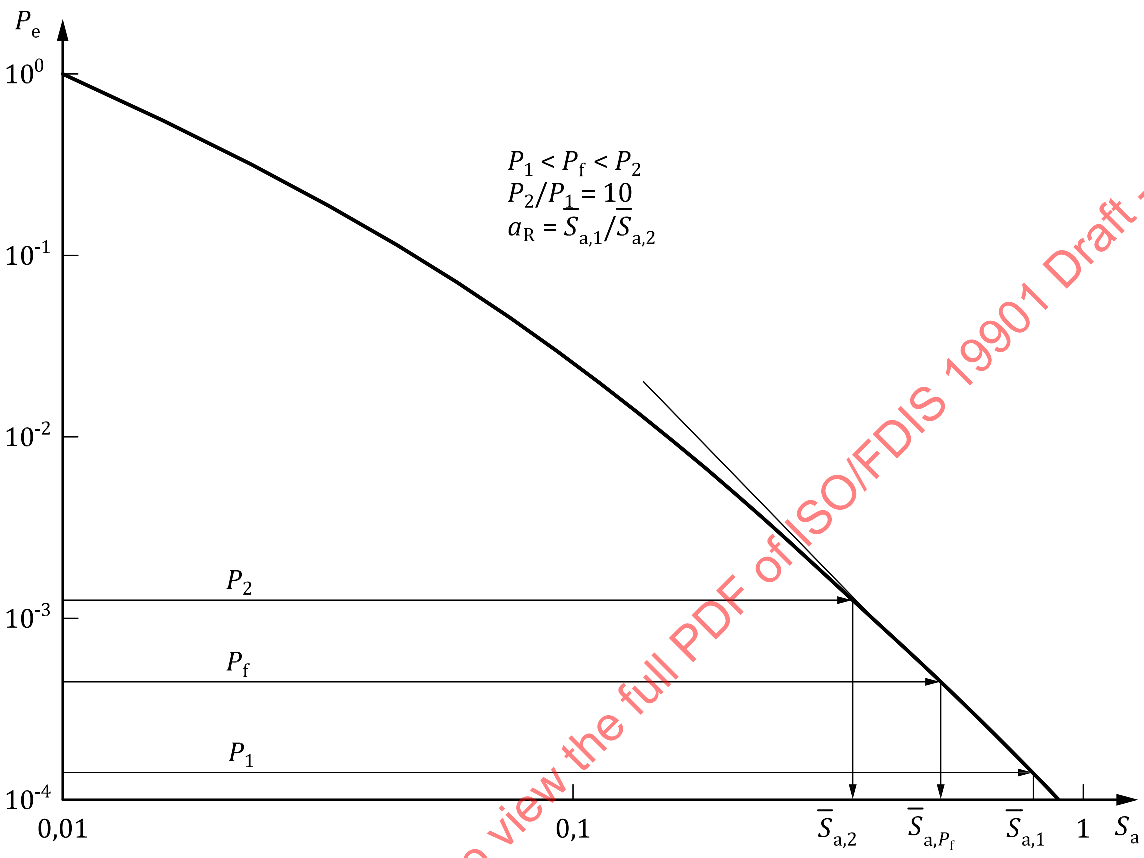
Minimum ELE return periods are given in Table 11 to ensure economic viability of a design, as a function of exposure level. If the ELE return period that is obtained from the procedure in this subclause is lower than the corresponding return period listed in Table 11, the return period in Table 11 shall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

Table 11 — Minimum ELE return periods

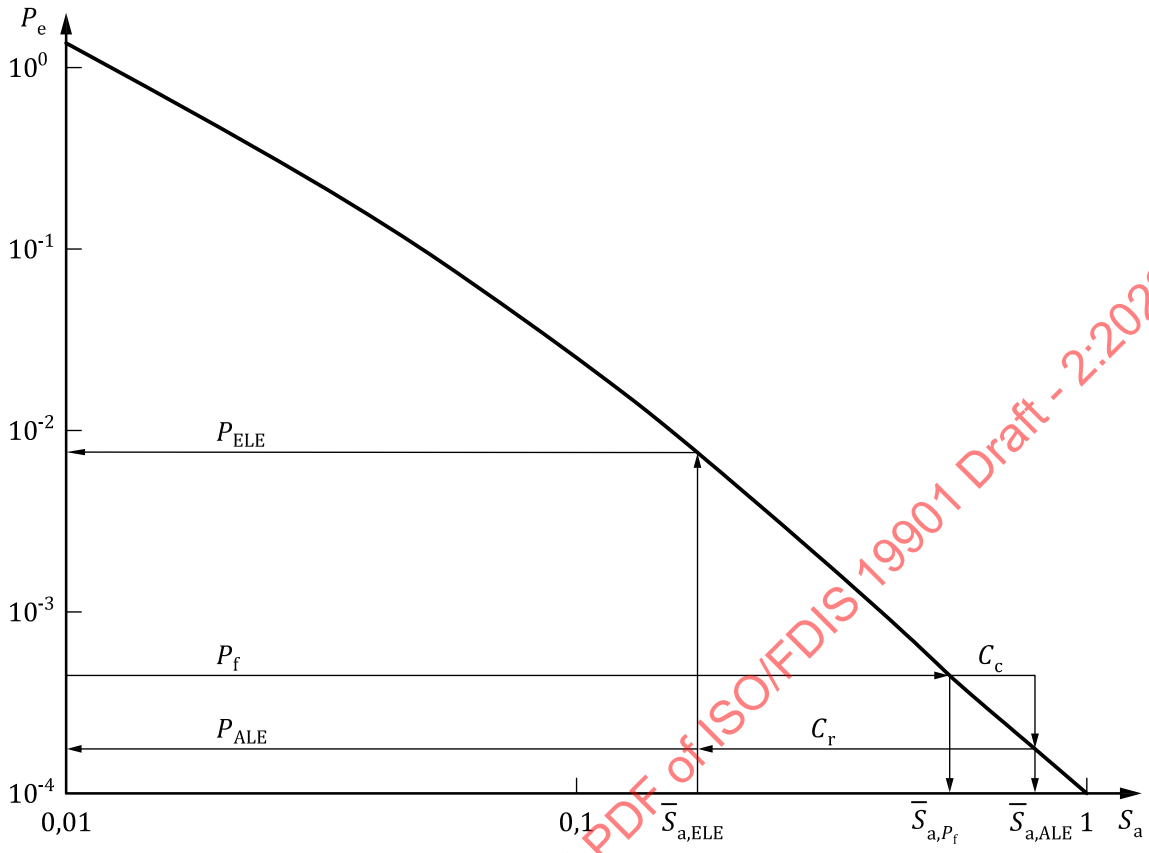
Exposure level	Minimum ELE return periods
L1	200
L2	100

Table 11 *(continued)*

Exposure level	Minimum ELE return periods
L3	50



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



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 5 — Typical seismic hazard curve

8.5 Local site response analyses

In the detailed seismic action procedure (see 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 rock 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 (equivalent 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.

— 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.

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 can 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 relevant International Standards on offshore structures prepared by TC 67 (see Introduction) apply.

Annex A
(informative)

Additional information and guidance

Scope (see Clause 1)

The background to and the development of the philosophy for this document are presented in Reference [5].

Earthquake hazards (see Clause 5)

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 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. Additional information on the impact of soil liquefaction on the structural design of offshore structures can be found in References [41], [44] and [45].

Earthquakes can initiate failure of sea floor slopes that are stable under normal self-weight and wave conditions, resulting in seabed 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 seabed 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, by large seabed or subaerial slides that can be triggered by earthquakes or volcanic caldera collapse. 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[42] 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.

Further information on the effect of earthquakes on floating offshore structures can be found in References [42] and [43].

Design principles (see 6.1)

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 uneconomic.

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 document address the balance between the ALE and ELE design criteria.

Extreme level earthquake design (see 6.2.2)

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 relevant International Standards on offshore structures prepared by TC 67 (see Introduction). 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.

The first objective can 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. The best practice is to develop a minimum of seven sets of time history records. Following a PSHA (see 8.2), the dominating ELE events may be identified through a procedure that is referred to as deaggregation (see References [6] to [10]). 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.

Both amplitude scaling and spectral matching procedures can be used to develop suitable time history records. The first option is a simple amplitude scaling of the record such that the geometric mean of two horizontal component spectral ordinates defined as $(H1*H2)^{1/2}$ produce a “best fit” to the horizontal ELE response spectrum over a period band that exceeds the expected significant structural vibration modes. The period band should extend above the expected fundamental period (to accommodate inelastic behaviour softening) and below higher modes expected to contribute to the seismic response. The second option, spectral matching, uses Fourier amplitude modulation to achieve a close match to the target spectrum without significantly disturbing the velocity and displacement histories. Spectral matching techniques can provide a better estimate of mean response values; however, amplitude scaling gives a better understanding of the potential variability of responses, thereby potentially identifying brittle behaviour.

Abnormal level earthquake design (see 6.2.3)

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.

Care is required to develop pushover load patterns that correctly place demands on critical components of a structure and that are representative of a seismic excitation.

Selection of earthquake records for ALE time history analysis and scaling of those records follow the same procedures as those outlined in the additional information on ELE design above.

Spectral acceleration data (see 6.3)

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 maps were developed[11] by combining published seismic hazard studies which included a combination of broad regional maps and local studies. Among the 94 studies selected, the most authoritative and recent reference was chosen for each region. The authority of a reference was given priority over the publication date. Maps published by national research institutes such as United States Geological Survey (USGS, e.g. Reference [12]) were given preference over publications by individual researchers. Maps developed by multi-national and multi-institution research organizations, such as the ongoing work by Global Earthquake Model (GEM, e.g. Reference [13]) and Seismic Hazard Harmonization in Europe (SHARE, see Reference [14]), were given preference over national research institute maps. For regions where recent and reliable data was not available, the 1999 Global Seismic Hazard Program maps (GSHAP, see Reference [54]) were used. 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 document. 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.

Seismic risk category (see 6.4)

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 some regions, the 1 000-year spectral acceleration at 1,0 s and 1 000-year PGA values will be of comparable intensity, which should help users who are more familiar with PGA.

This document 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 that 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.

Seismic design requirements (see 6.5)

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 Formula (8) or, alternatively, using Formula (A.2).

Soil classification and spectral shape (see 7.1)

The site soil class represents the properties of the local soils where the seismic excitation acts on the foundation. These local soils effect the attenuation or amplification of seismic waves as they propagate through the soil profile into the foundation. The effective seabed can be taken nominally as the top 30 m around the location where most of the seismic energy is transmitted into the foundation. For surface foundations, such as mat foundations for subsea structures, the effective seabed should be taken as the top 30 m of the seabed. For embedded foundations, such as spudcan foundations for jack-up rigs, the effective seabed can be taken as the top 30 m below the vertical location of the foundation. For driven piles or suction caissons, the seismic energy is transmitted along the length of the piles or caissons. Analyses can be performed to determine the location of maximum soil-pile interaction, where most of the seismic energy is transmitted. This location depends on soil conditions as well as the diameter and stiffness of the piles or caissons. For typical offshore piled foundations, the location of maximum soil-pile interaction is usually at a depth of 6 m to 15 m below the sea floor.

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[15]. 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 can be placed on the sea floor. If a sea floor seismic source were used with these on-bottom arrays, shear wave velocity can, with suitable conditions, be interpreted from either seismic reflection, seismic refraction or spectral analysis of surface waves (SASW) methods[16].

If direct field measurements are not available, then the shear wave velocity can be inferred from other data acquired in a marine soil investigation. The shear wave velocity can be determined based on information from specific geotechnical laboratory testing, i.e. from the initial shear modulus, G_{max} and the mass density of the soil, ρ, as shown in Formula (A.1):

$$V_s = \sqrt{\frac{G_{max}}{\rho}} \tag{A.1}$$

Formula (A.1) 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 initial shear 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 obtained from a marine soil 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, s_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} can 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[12][46]. 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 significantly differ from 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[21].

Seismic action procedure (see 7.2)

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_{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, and

— 2,0 for L3 structures.

Probabilistic seismic hazard analysis (see 8.2)

The background to the PSHA procedure and the different elements have been developed in Reference [22]. The basic approach to 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, and 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 [see 7.1 d)].

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 shown in Formula (A.2):

$$T_{\text{return}} = \frac{-1}{\ln\left(1 - P_{\text{e}}\right)} \qquad \text{(A.2)}$$

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

Seismic action procedure (see 8.4)

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, σ_{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 σ_{LR} and the seismic hazard slope, a_R. A value of σ_{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 σ_{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 σ_{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**[a] 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

[a] 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, must be established prior to performing the seismic design, the above margins of safety must 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 Annex A). 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 5).

Local site response analyses (see 8.5)

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 [27]. 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 (see References [29] to [36]).

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 (see 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.

The use of constant ground motions can lead to under prediction of bending moments along the piles. Depth-varying input ground motions along the piles should be used to size the pile wall thickness as the multiple support excitation can produce a better match of bending moments along the piles, when compared to results of centrifuge tests[47].

Performance requirements (see Clause 9)

For some offshore structures, a seismic risk lower than that implied in the design procedure defined in this document can be required. The accuracy with which low probability seismic events can be predicted tends to limit the validity of extrapolating beyond the ALE seismic accelerations defined in this document. Enhanced seismic performance can be best attained by enhancing the resistance to the ALE event by designing to maximize the robustness and ductility of the structure.

For L1 structures, in addition to any consideration of an extremely rare seismic event ($P_f << 4 \times 10^{-4}$), demonstration of survivability to the 4×10^{-4} event listed in Table 2 is required, even if performance to a lower probability seismic event is shown to be acceptable. When site response analyses are performed, different non-linear soil response can result with a higher probability event developing higher seismic demands on the structure.

Annex B
(informative)

Simplified action procedure spectral accelerations

The world regions shown in Figure B.1 are expanded with further detail in Figures B.2 to B.15 to provide spectral acceleration values for developing seismic response spectra of offshore installations. These more detailed maps[11] give generic 5 % damped spectral accelerations, expressed in *g* of a rock outcrop for 0,2 s and 1,0 s oscillator periods 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 this annex 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 the lack of complete understanding or knowledge (epistemic or Type II uncertainties). However, a site-specific assessment is required by this document for any structure in which failure would have significant consequences and in which seismic considerations can affect the design.

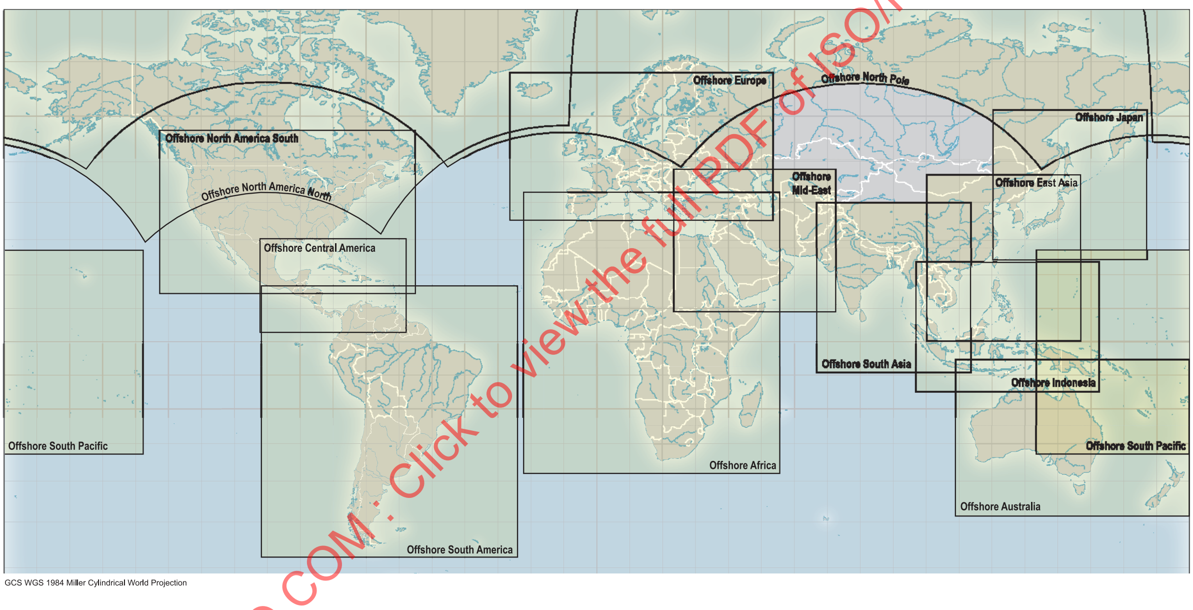


Figure B.1 — Worldwide map showing regions for spectral response accelerations

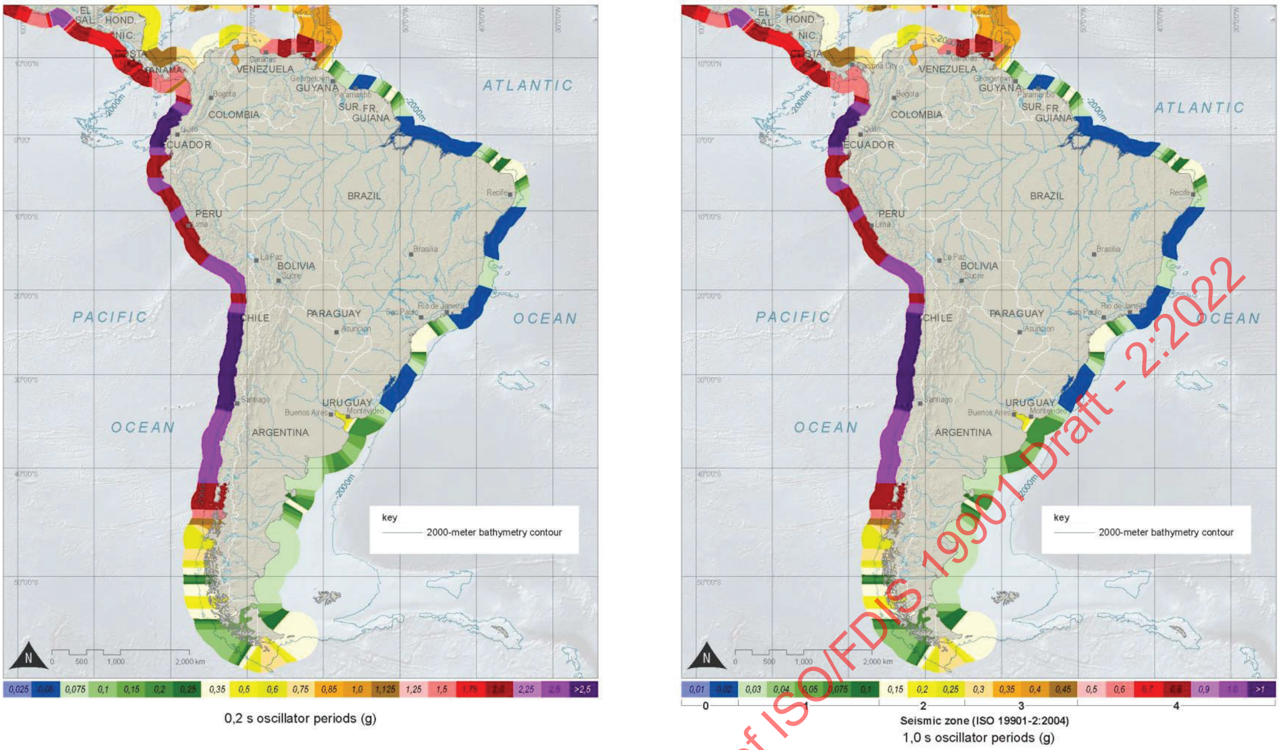


Figure B.2 — 5 % damped spectral response accelerations for offshore South America

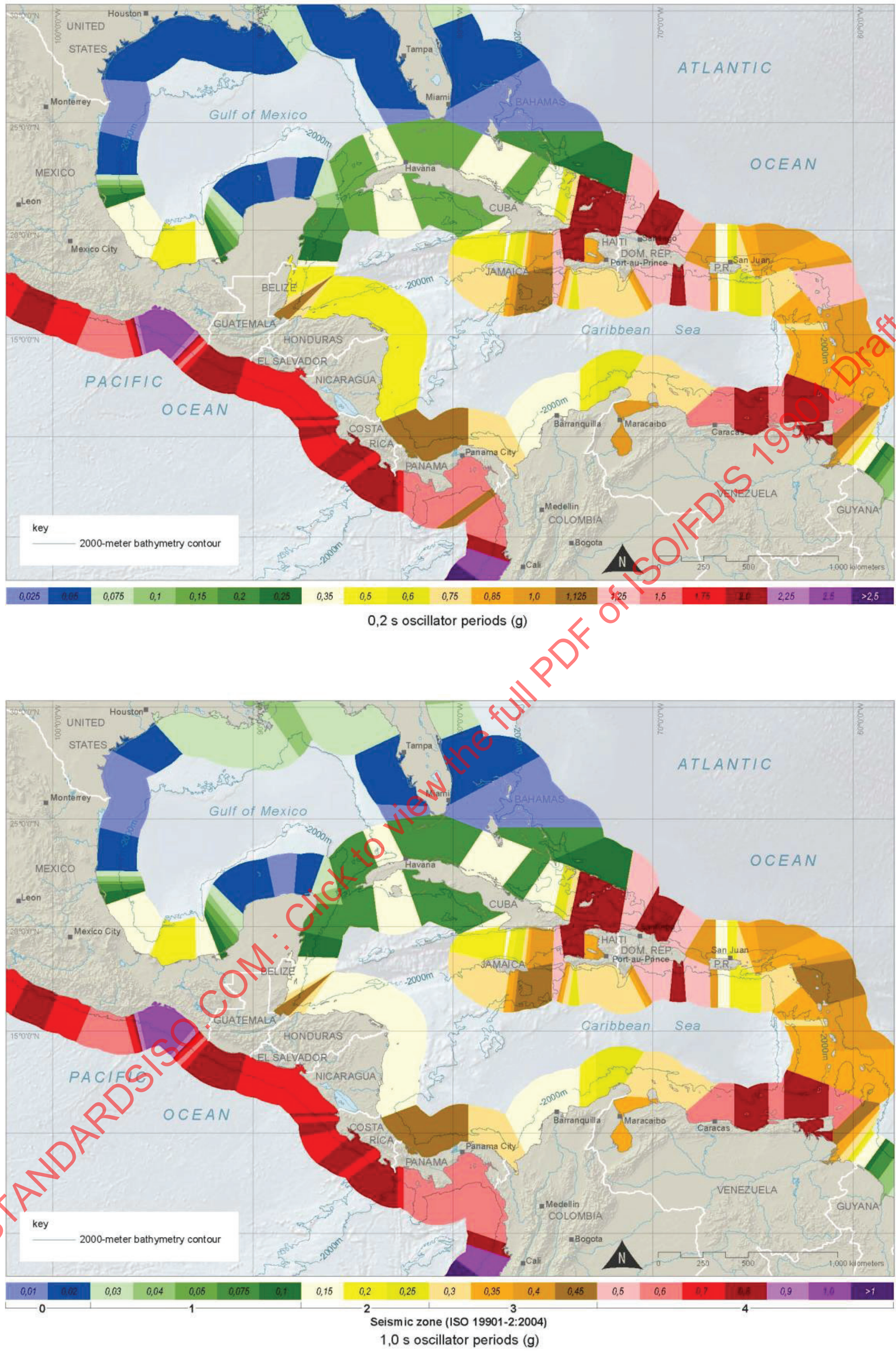


Figure B.3 — 5 % damped spectral response accelerations for offshore Central America

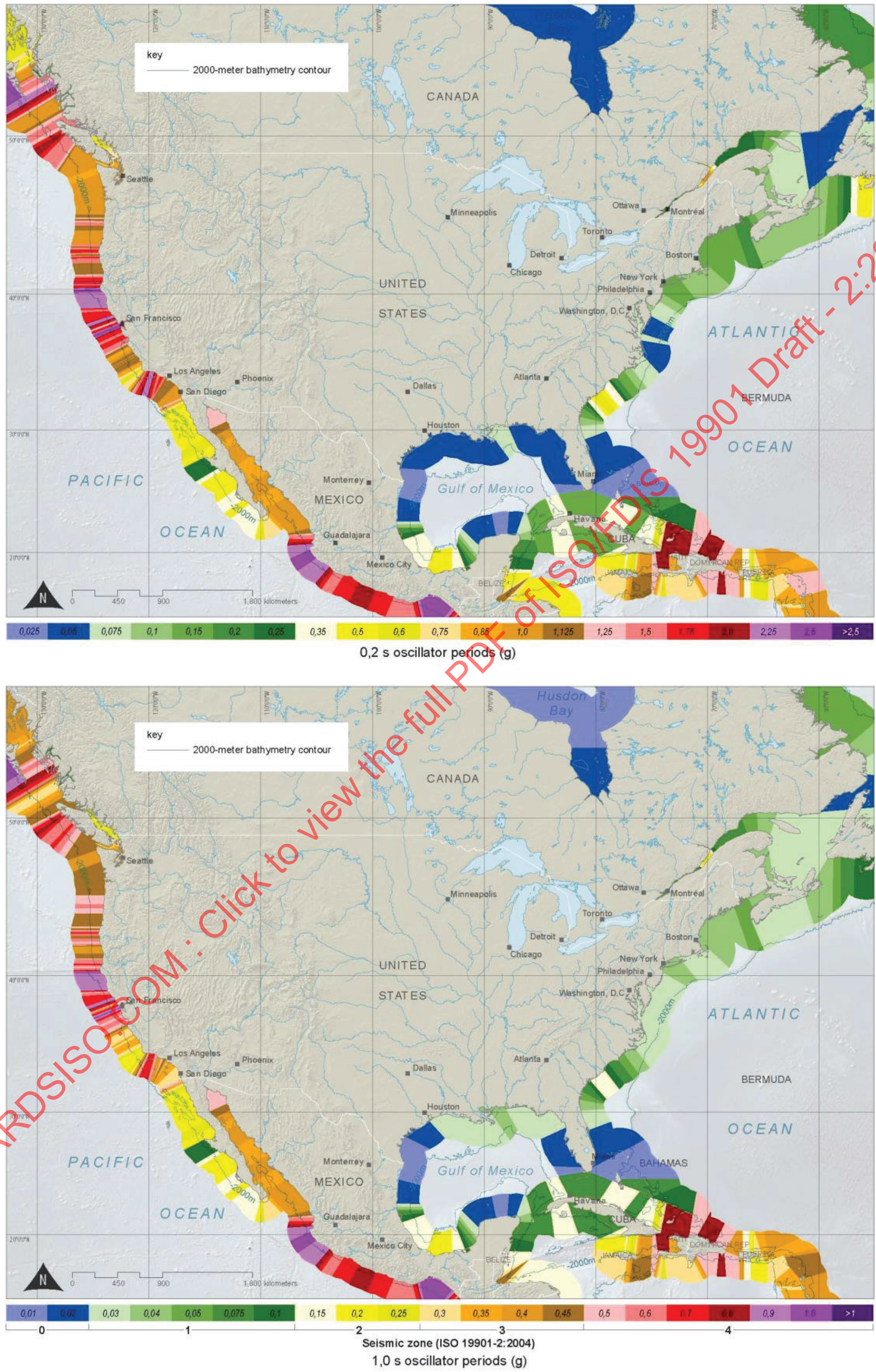


Figure B.4 — 5 % damped spectral response accelerations for offshore North America - Southern

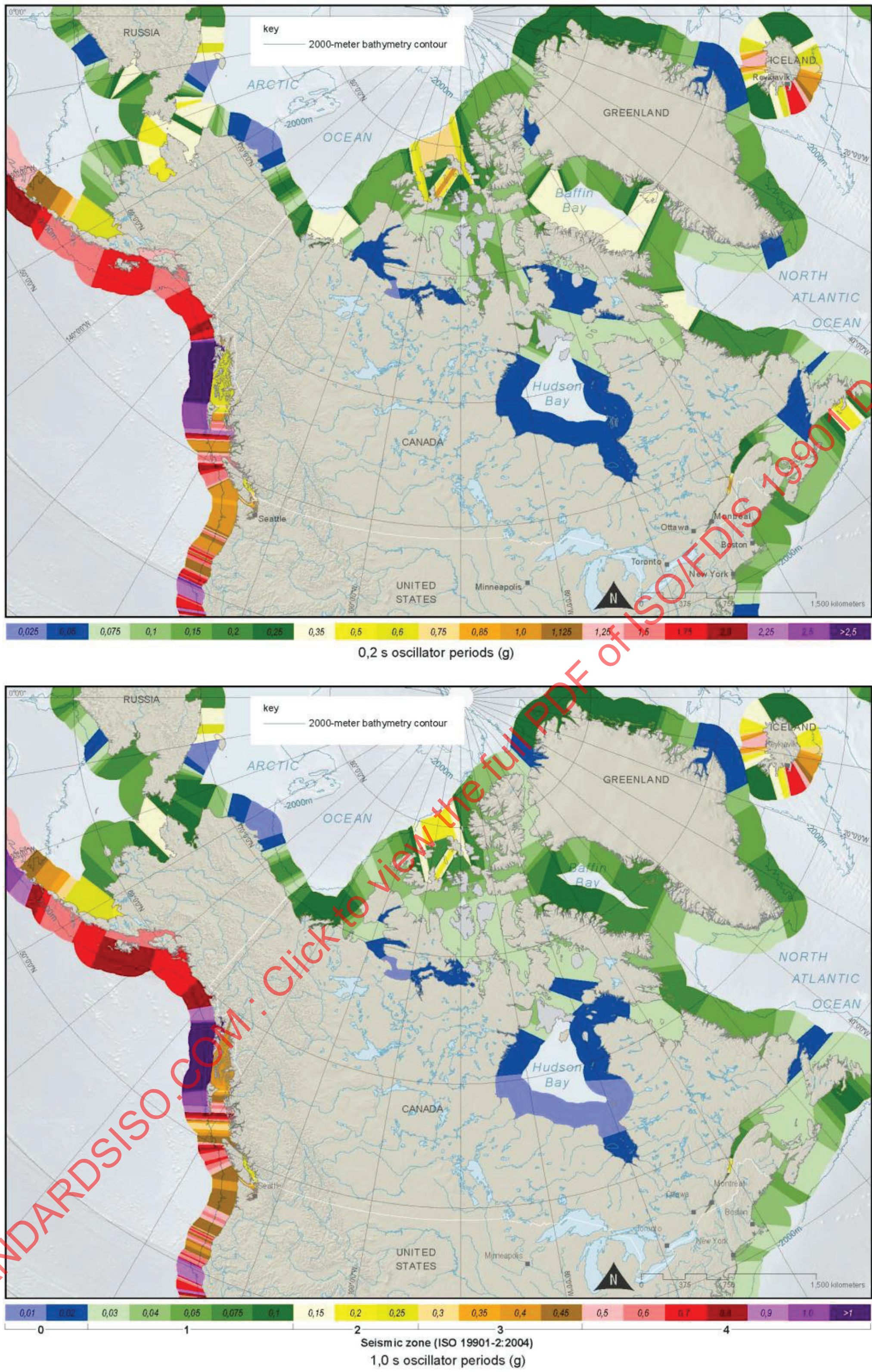


Figure B.5 — 5 % damped spectral response accelerations for offshore North America - Northern

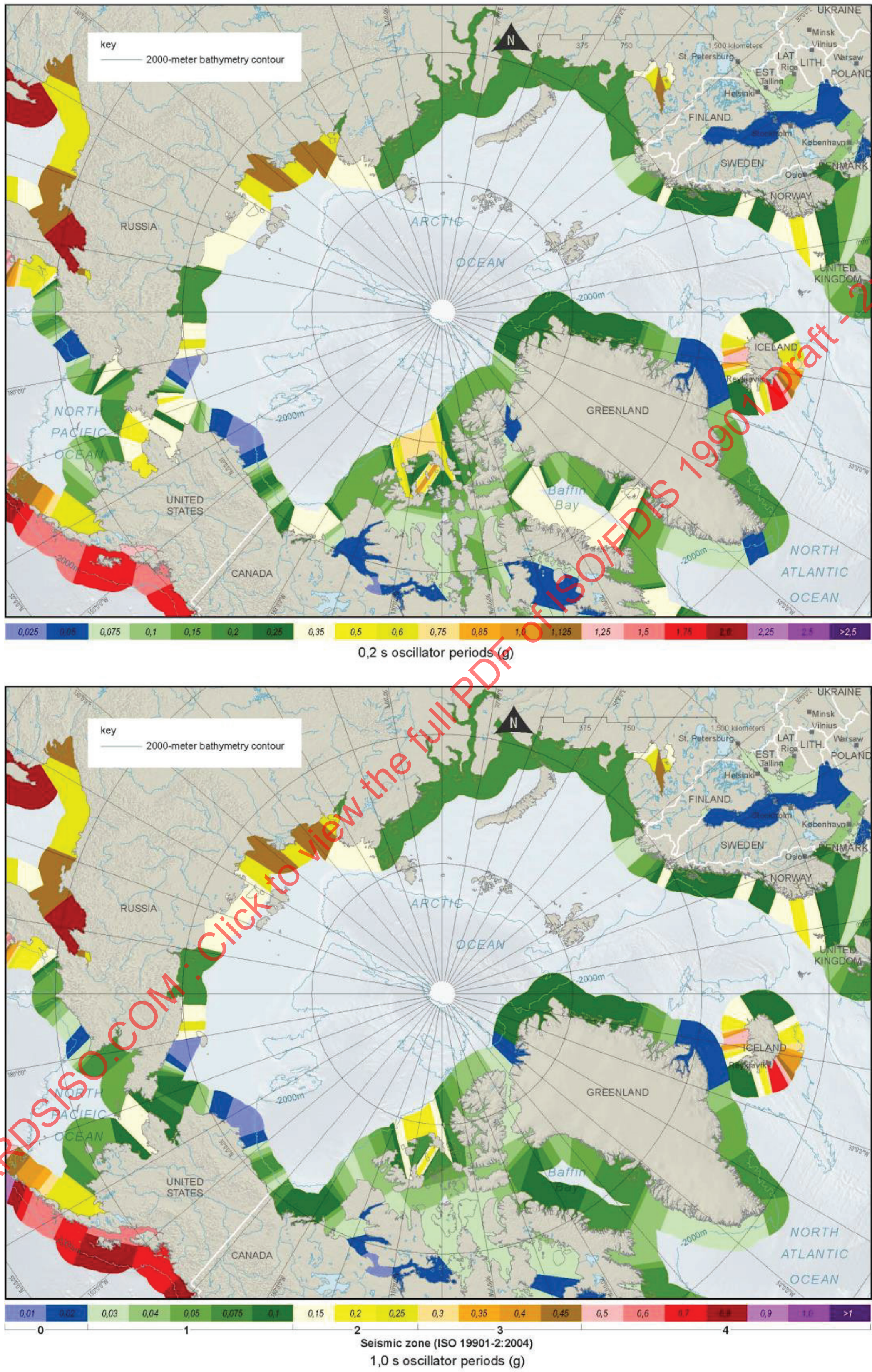


Figure B.6 — 5 % damped spectral response accelerations for offshore North Pole - Arctic

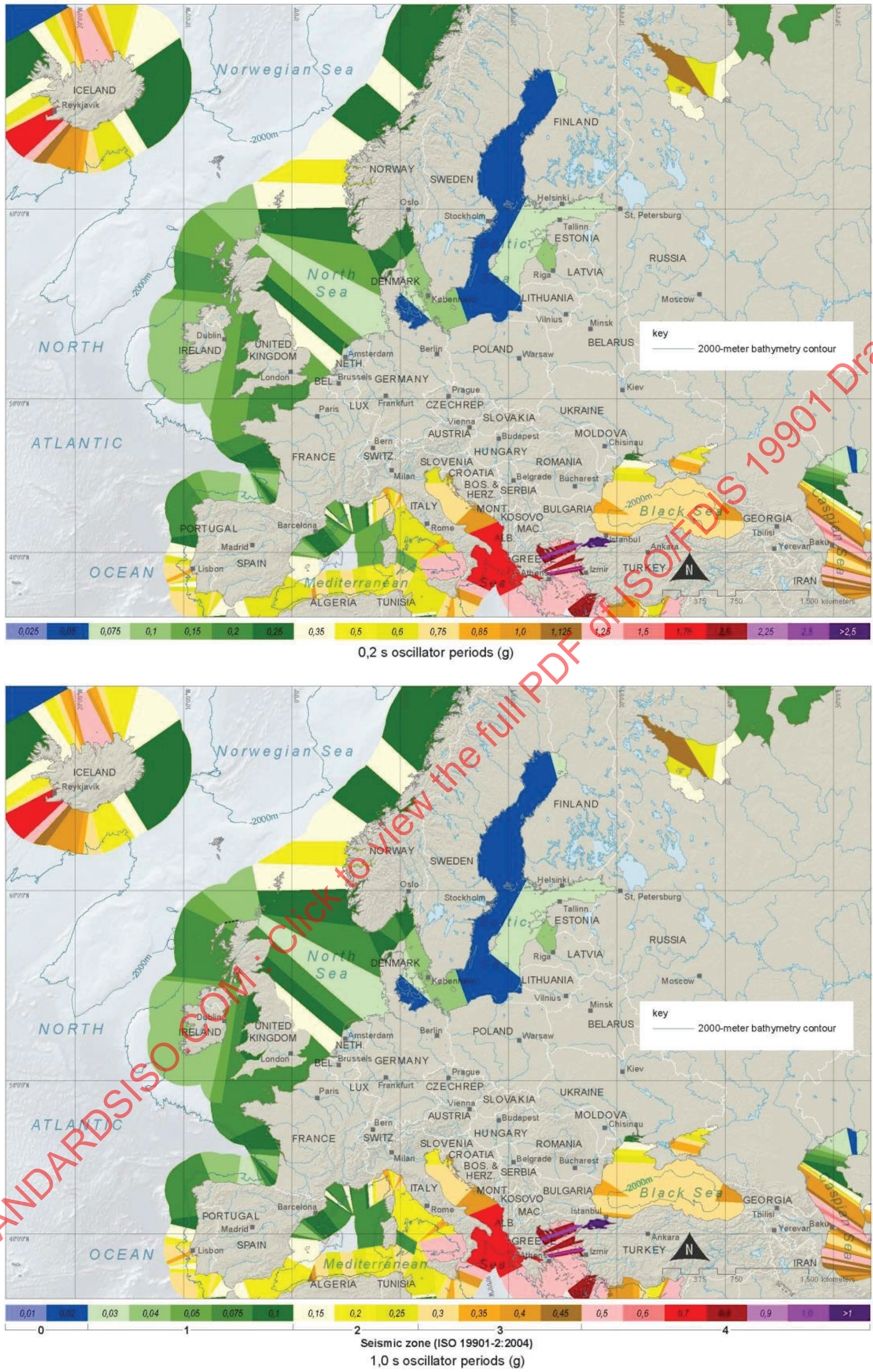


Figure B.7 — 5 % damped spectral response accelerations for offshore Europe

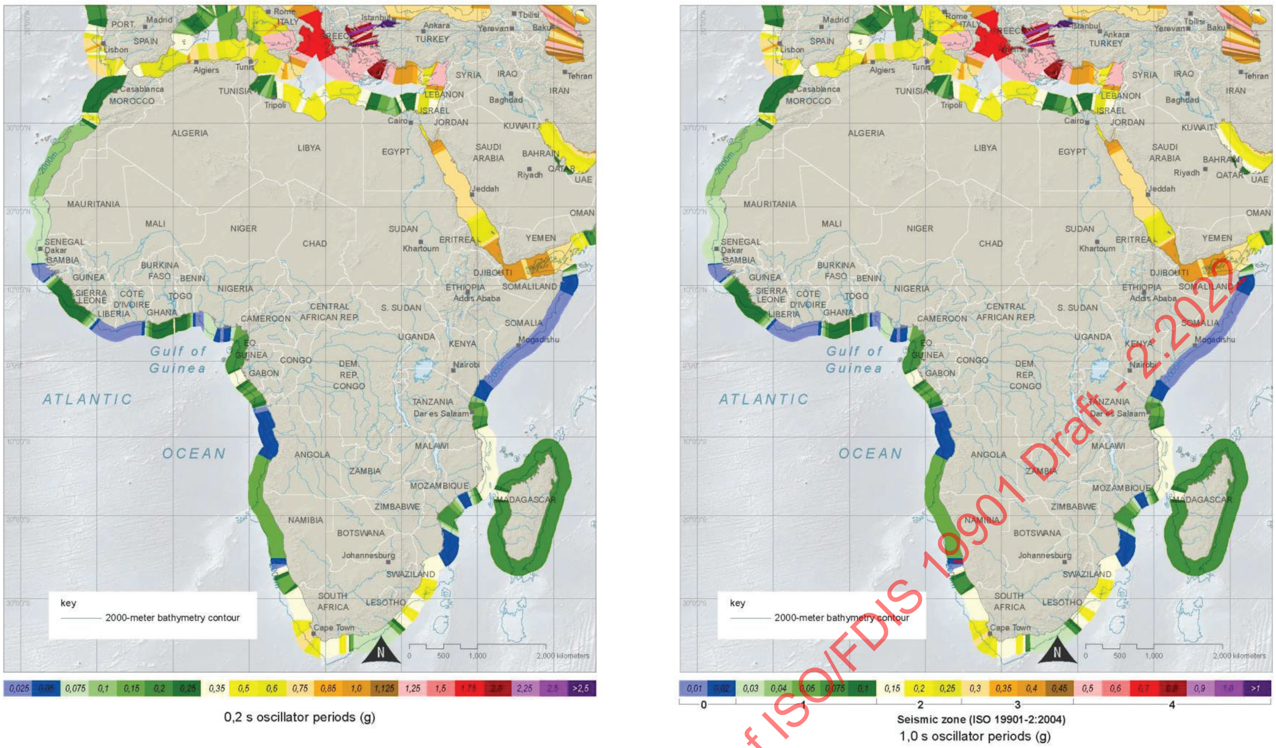


Figure B.8 — 5 % damped spectral response accelerations for offshore Africa

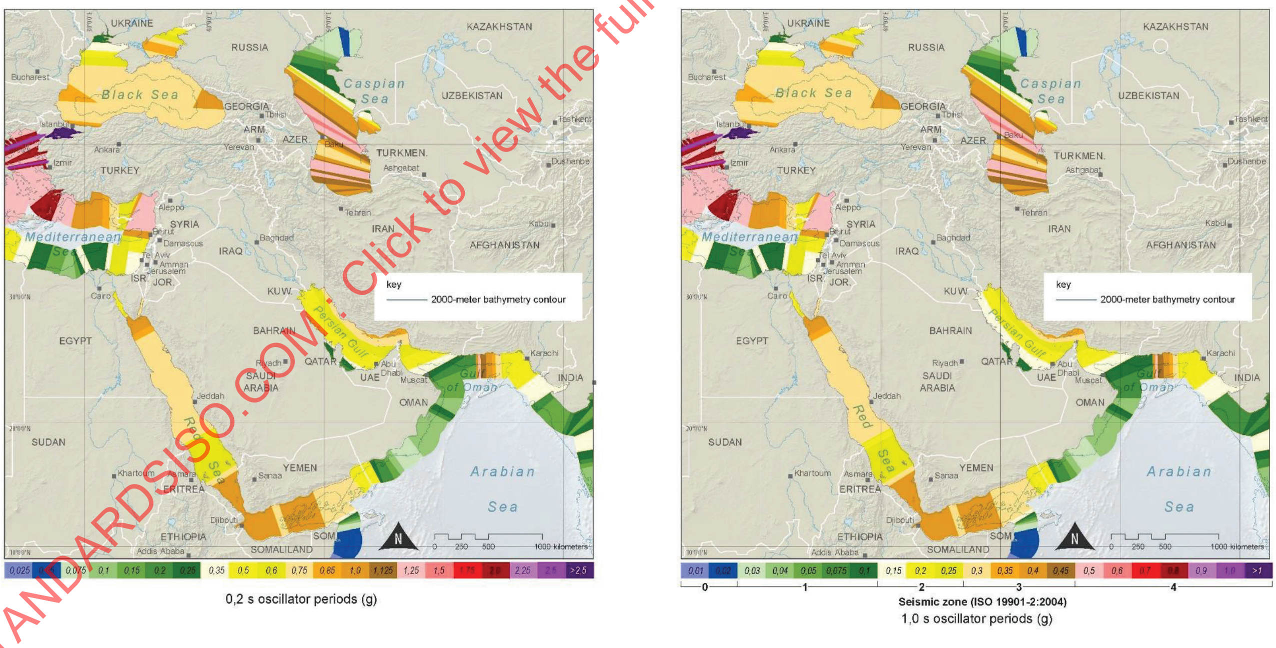


Figure B.9 — 5 % damped spectral response accelerations for offshore Middle East

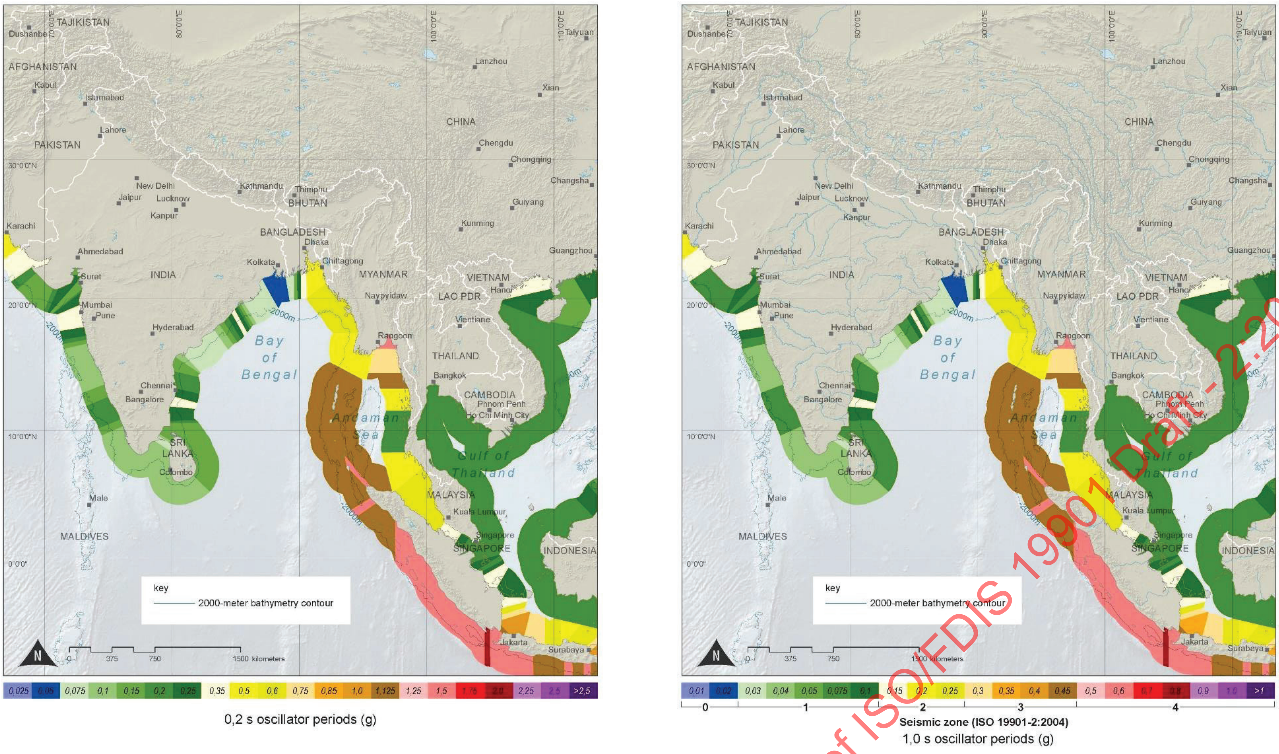


Figure B.10 — 5 % damped spectral response accelerations for offshore Southern Asia

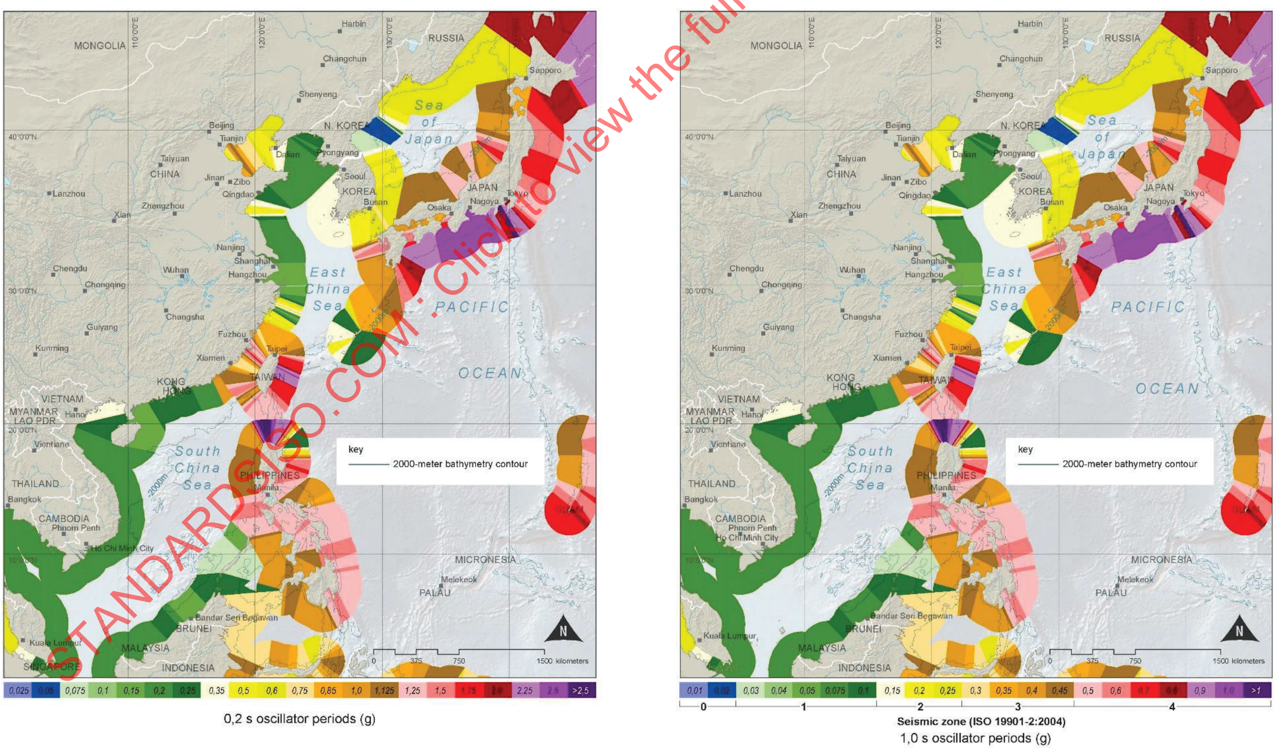


Figure B.11 — 5 % damped spectral response accelerations for offshore Southeast Asia

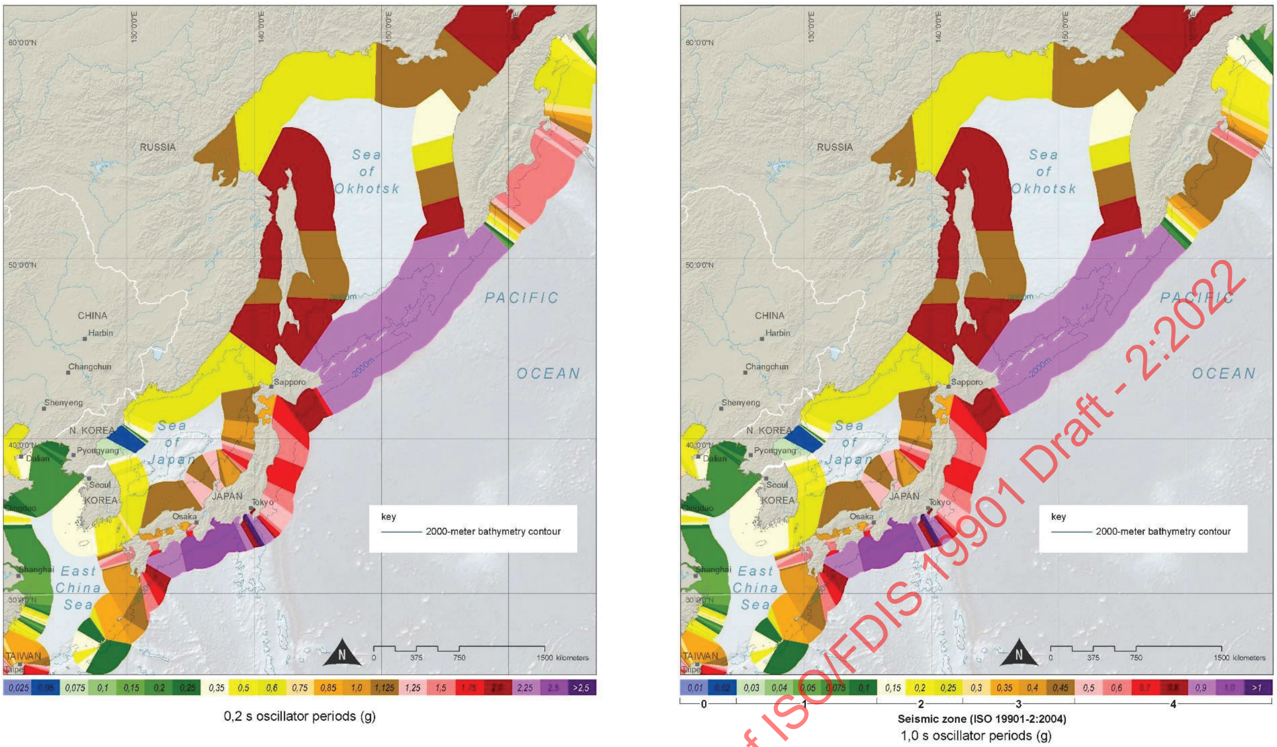


Figure B.12 — 5 % damped spectral response accelerations for offshore Japan and North China