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Cranes — Principles for seismically resistant design

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National foreword

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Cranes — Principles for seismically resistant design

Appareils de levage à charge suspendue — Principes pour une conception résistante à la sismicité

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Foreword

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The committee responsible for this document is ISO/TC 96, *Cranes*, Subcommittee SC 10, *Design principles and requirements*.

Introduction

An economically acceptable protection against the effects of earthquake is usually based on two design limit states which specify the required crane response to a moderate and a severe earthquake and which are expressed in terms of serviceability and ultimate limit states.

— Serviceability limit state (SLS) imposes that the crane should withstand moderate earthquake ground motions which may occur at the site during its service life. The resulting stresses would remain within the accepted limits.

— Ultimate limit state (ULS) imposes that the crane structure should not collapse nor experience similar forms of structural failure due to severe earthquake ground motions, the suspended load, or any part of the crane should not fall and the safety of the public, operators and workers should be safe guarded. The crane is not expected to remain operational after the earthquake. However, in the case of a failure in the main load path, it is still possible to lower the load to the ground after the earthquake.

Cranes — Principles for seismically resistant design

1 Scope

This International Standard establishes general methods for calculating seismic loads to be used as defined in the ISO 8686 series and for proof of competence as defined in ISO 20332, for the structure and mechanical components of cranes as defined in ISO 4306.

This International Standard evaluates dynamic response behaviour of a crane subjected to seismic excitation as a function of the dynamic characteristics of the crane and of its supporting structure.

The evaluation takes into account dynamic effects both of regional seismic conditions and of the local conditions on the surface of the ground at the crane location.

The operational conditions of the crane and the risks resulting from seismic damage to the crane are also taken into account.

This International Standard is restricted to the serviceability limit state (SLS), maintaining stresses within the elastic range in accordance with ISO 20332.

The present edition does not extend to proofs of competence which include plastic deformations. When these are permitted by agreement between crane supplier and customer, other standards or relevant literature taking them into account can be used.

2 Normative references

The following documents, in whole or in part, are normatively referenced in this document and are indispensable for its application. For dated references, only the edition cited applies. For undated references, the latest edition of the referenced document (including any amendments) applies.

ISO 4306 (all parts), *Lifting appliances — Vocabulary*

ISO 8686 (all parts), *Cranes — Design principles for loads and load combinations*

ISO 20332, *Cranes — Proof of competence of steel structures*

3 Symbols

The main symbols used in this International Standard are given in Table 1.

Table 1 — Main symbols

Symbol	Description
K_H	Horizontal seismic design coefficient
K_V	Vertical seismic design coefficient
A_{bg}	Normalized basic acceleration
A_{sg}	Normalized acceleration at ground surface
f_{con}	Conversion factor
f_{rec}	Recurrence factor
β_2	Subsoil amplification factor
β_3	Acceleration response factor

Table 1 *(continued)*

Symbol	Description
β_3^*	Basic acceleration response factor; β_3 of the crane whose damping ratio is 0,025 and given by Figure 2
γ_n	Risk factor
η	Damping correction factor
δ	Response amplification factor
ζ	Damping ratio
c	Vertical influence factor
F_H	Horizontal seismic design force
F_V	Vertical seismic design force
F_{RH}, F_{RV}	Seismic forces (horizontal and vertical) on suspended load

4 Seismic design methods

There are three main methods of seismic response analysis used in seismic design:

— Modified Seismic Coefficient Method;

— Maximum Response Spectrum Method;

— Time History Response Method.

In the *Modified Seismic Coefficient Method*, the applied quasi-static seismic forces are calculated as a product of seismic coefficients and crane weights. The evaluation of seismic coefficients takes into account crane location, its seismic characteristics, basic dynamic characteristics of the crane, i.e. natural frequency or period and damping characteristics, in three principal orthogonal directions of the crane (one vertical and two horizontal).

The method is the basis of this International Standard on account of its simplicity (see Clause 5) and its procedure is executed as part of the design iterative process indicated in the flow chart in Annex A.

The *Maximum Response Spectrum Method* (see Clause 6) is an alternative method of seismic response analysis used where:

— more accurate seismic response of the crane is required than that produced by the Modified Seismic Coefficient Method;

— demand on significant computational resources is economically acceptable.

Its application is limited only to linear systems and to system where nonlinearities if present can be neglected.

In the Maximum Response Spectrum Method, natural frequencies or periods and associated mode shapes of the crane are calculated first. Seismic forces and the crane response are then calculated for the selected vibration modes of the crane structure, using the maximum response accelerations (selected from the maximum response spectra which again take into account seismic characteristics at crane location and the damping characteristics of crane structure) together with the calculated mode shapes, frequencies and mass distribution of the crane.

The *Time History Response Method* is the third method of seismic response analysis available. It is employed when:

— only an accurate seismic response of crane is acceptable (see Annex D);

— nonlinearities (due to material behaviour, such as plastic deformations and stresses or dynamic behaviour nonlinearities, such as gaps, friction, wheels lifting off the rails, or slack in ropes, etc.), if present, need to be taken into account;

— the associated cost of high computational requirements is acceptable.

In the Time History Method, the seismic response is evaluated by using numerical step-by-step integration in time to solve the formula of motions for crane structure and ground excitation under consideration, selected to represent seismic condition at crane site.

5 Seismic design by Modified Seismic Coefficient Method

5.1 General

In this method, seismic forces and accelerations acting on the crane are calculated using horizontal and vertical seismic coefficients, K_H and K_V. For cranes with an enhanced risk, the risk coefficient, γ_n, with a value greater than unity shall be applied, in accordance with Clause 7.

5.2 Calculation of horizontal seismic design coefficient, K_H

5.2.1 General

The horizontal seismic design coefficient, K_H, shall be calculated as follows:

$$K_H = A_{bg} \times \beta_2 \times \beta_3 \times f_{con} = A_{sg} \times \beta_3 \times f_{con} \quad (1)$$

where

A_{bg} is the normalized basic acceleration (see 5.2.2);

A_{sg} is the normalized surface ground acceleration

β_2 is the subsoil amplification factor (see 5.2.3);

β_3 is the acceleration response factor (see 5.2.4);

f_{con} is the conversion factor f_{con} = 0,16 for a return period of 475 years (see 5.2.2) converted to 72 years appropriate for serviceability limit state (SLS) of a seismically resistant crane.

The direction of the normalized accelerations, A_{bg} and A_{sg}, are considered to be arbitrary unless seismological considerations dictate otherwise. When the direction is arbitrary, it shall be applied to produce the maximum effect.

5.2.2 Determination of normalized basic acceleration, A_{bg}

Normalized basic acceleration, A_{bg}, is calculated from the Formula (2):

$$A_{bg} = a_g / g \times f_{rec} \quad (2)$$

where

a_g is the maximum horizontal basic acceleration, in m/s^2 (see Annex B);

g is the gravity acceleration, in m/s^2;

f_{rec} is a factor depending on the recurrence interval R; for crane design in general a design earthquake, which may recur once in intervals of 100 years to 475 years (R = 100 to R = 475) may be selected:

f_{rec} = 1,0 for R = 475; this is the default value,

f_{rec} = 0,5 for R = 100; used only for cranes intended for temporary use at different sites.

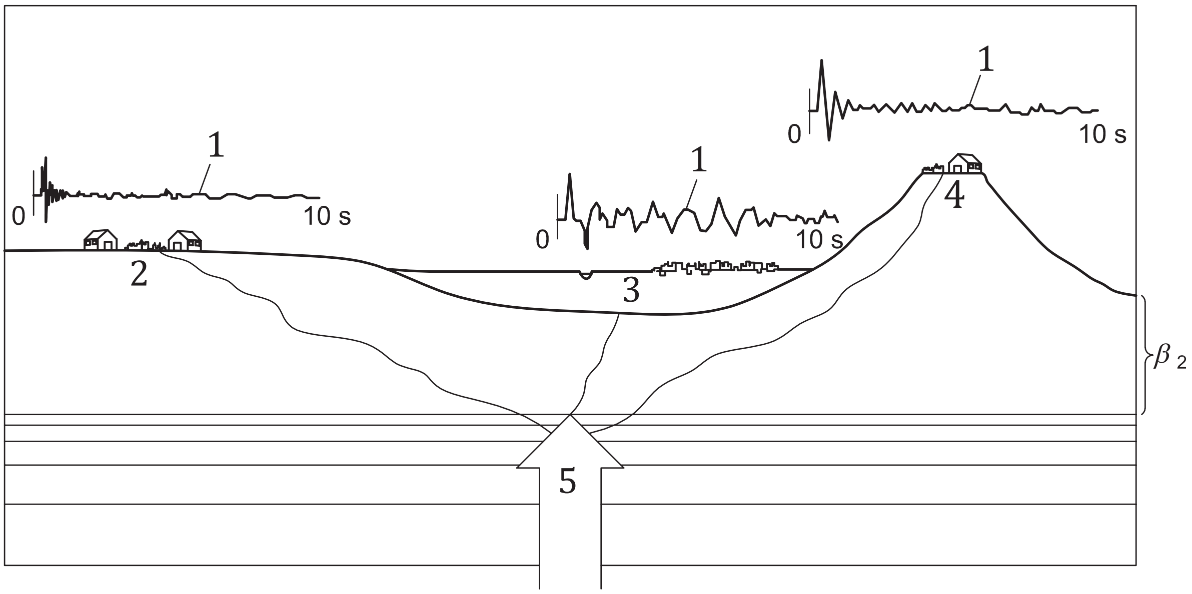
See Annex B for suggested values of A_{bg} and A_{sg}, for different countries, taking into account regional seismic damage experiences and regional seismicity.

In B.1, the accelerations, A_{bg} and A_{sg}, are based on the return period of 475 years (f_{rec} = 1,0).

NOTE 475 years is the most accepted return period used within the seismic data available.

5.2.3 Determination of subsoil amplification factor, β_2

The subsoil amplification factor expresses the influence of the soil surface on the intensity and the frequencies of the seismic excitation. The principle of this influence is illustrated in Figure 1.



Key

1 seismic effects on the surface (recorded seismograms), represented by A_{sg} in this International Standard

2 rock

3 soft to medium stiff ground

4 stiff ground

5 normalized basic accelerations A_{bg} (related to seismic bedrock)

Figure 1 — Illustration of the subsoil amplification factor (β_2)

In Table 2, subsoil categories are classified as a function of $v_{s,30}$, the average shear-velocity through the upper 30 m of soil. The values of β_2 shall be selected from this table, for subsoil category at crane location.

Table 2 — Determination and values of β_2

Category	Subsoil	Shear-wave velocity $v_{s,30}$ m/s	β_2
Category 0	Rock	$v_{s,30} > 800$	1,0
Category 1	Stiff ground composed of hard sandy soil strata where soil types overlying rock are stable deposits of sands, gravels, or stiff clays.	$360 < v_{s,30} \le 800$	1,4
Category 2	Medium ground excluding categories 1 and 3.	$180 < v_{s,30} \le 360$	1,6
Category 3	Soft-to-medium-stiff ground composed of alluvial soil strata or muddy soil strata characterized by about 30 m or more soft-to-medium-stiff clay.	$v_{s,30} \le 180$	2,0

5.2.4 Determination of acceleration response factor, β_3

5.2.4.1 General

The value of acceleration response factor, β_3, shall be determined as a function of

— dynamic characteristics of crane support structure where applicable,

— frequency or period of the most significant mode of the crane in the direction under consideration,

— damping ratio of the same mode, and

— subsoil category at the location of the crane.

The most significant modes of the crane are selected from natural periods or frequencies determined by measurement or by calculation, using recognized computational techniques.

β_3 shall be defined as

$$\beta_3 = \beta_3^* \times \eta \times \delta \tag{3}$$

where

β_3^* is the basic acceleration response factor (see 5.2.4.2);

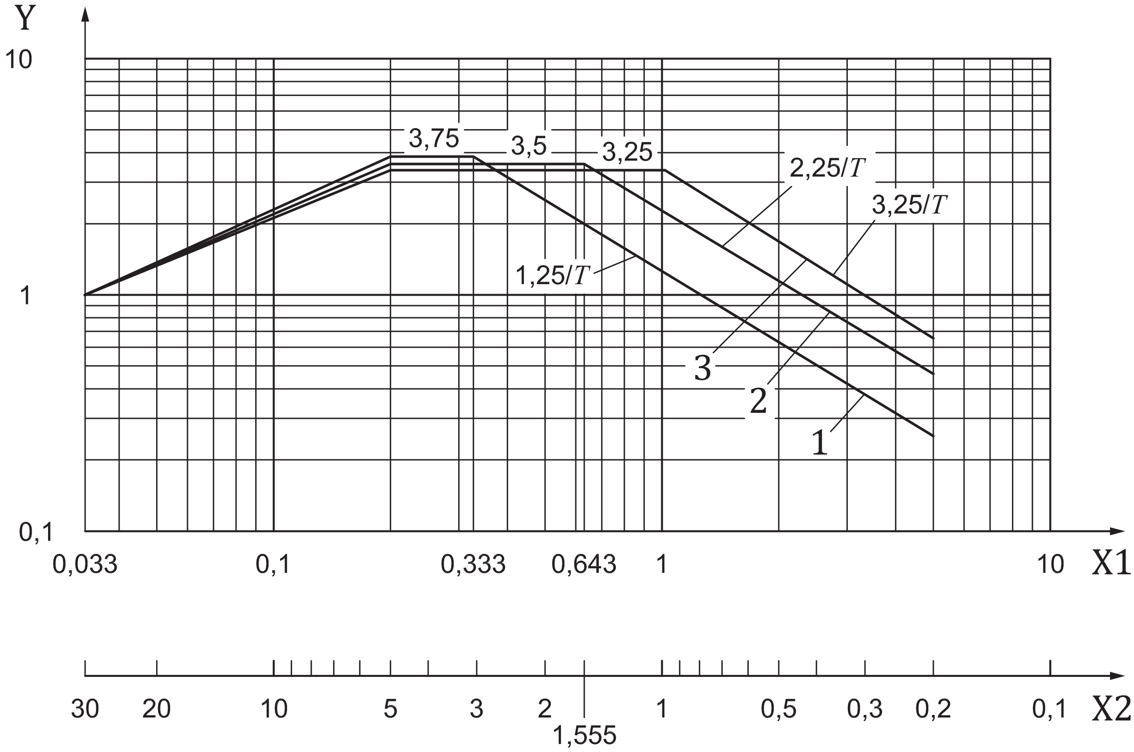
η is the damping correction factor (see 5.2.4.3);

δ is the response amplification factor (see 5.2.4.4).

5.2.4.2 Basic acceleration response factor, β_3^*

β_3^* is the basic acceleration response factor of a crane structure with damping ratio of 0,025.

Its values as a function of the natural period or frequency of the crane and of subsoil category at crane location are shown in Figure 2.



Key

1 subsoil categories 0 and 1
2 subsoil category 2
3 subsoil category 3
X1 axis for natural period T_c [s] of crane structure
X2 axis for natural frequency f_c [Hz] of crane structure
Y axis for the basic acceleration response factor, β_3^*

Figure 2 — Factor β_3^* (as a function of crane natural period or frequency and of subsoil category at crane location)

5.2.4.3 Damping correction factor, η

Damping correction factor, η, in Formula (3) shall be defined according to the value of damping ratio, ζ, of the crane structure as shown in Table 3.

Table 3 — Damping correction factor, η

Damping ratio, ζ	**0,01**	**0,015**	**0,02**	**0,025**	**0,03**	**0,04**	**0,05**	**0,1**
η	1,24	1,15	1,06	1,0	0,94	0,87	0,80	0,62

Typical values of damping ratios for structures, with the members generally stressed below 50 % of the elastic limit, are ζ = 0,025 for welded construction, ζ = 0,04 for bolted construction and ζ = 0,03 for welded and bolted construction combined. Higher values of damping ratios may be used for the same types of construction stressed above 50 % of the elastic limit of the material.

Where a buckling failure mode controls the design higher levels of damping shall not be used.

Alternatively, damping ratios can be obtained by accepted methods, such as the following:

— measurement;

— an evaluation of the hysteresis of a force-displacement diagram of nonlinear items such as structural members with nonlinear behaviour or joints with dry friction.

5.2.4.4 Response amplification factor, δ

For cranes operating on rails laid directly on the ground δ shall be defined as unity, $\delta = 1$.

For cranes operating on rails laid on a supporting structure (e.g. building, pier, jetty) the value of δ can be determined from the Formula (4):

$$\delta = 0{,}71 \cdot \sqrt{\frac{1+\lambda^2}{\lambda^2 + \left(1-\lambda^2\right)\cdot \kappa^2}} \geq 1 \tag{4}$$

where

λ is a factor related to the degree of coupling between crane structure and supporting structure as given in Table 4;

κ is a factor related to the equivalent damping of the coupled structure between crane structure and supporting structure as given in Figure 3, where ζ is the damping ratio of the crane structure (see 5.2.4.3).

Table 4 — Factor λ

Natural period ratio	λ
$T_C/T_P \leq 0{,}9$	$\sqrt{1-\left(1-\theta\right)\cdot\left(\frac{1{,}8\cdot T_c\cdot T_p}{T_c^2+0{,}81\cdot T_P^2}\right)^2}$
$0{,}9 < T_C/T_P \leq 1{,}1$	$\sqrt{\theta}$
$T_C/T_P > 1{,}1$	$\sqrt{1-\left(1-\theta\right)\cdot\left(\frac{2{,}2\cdot T_c\cdot T_p}{T_c^2+1{,}21\cdot T_P^2}\right)^2}$
where	
$\theta = \frac{m_c}{m_c+m_s}$	is the mass ratio of the crane structure and the supporting structure;
m_C	is the mass of the crane as a whole;
m_S	is the mass of the supporting structure as a whole;
T_C	is the largest natural period of the crane structure with the supporting structure assumed rigid;
T_P	is the largest natural period of the supporting structure with the crane structure assumed rigid.

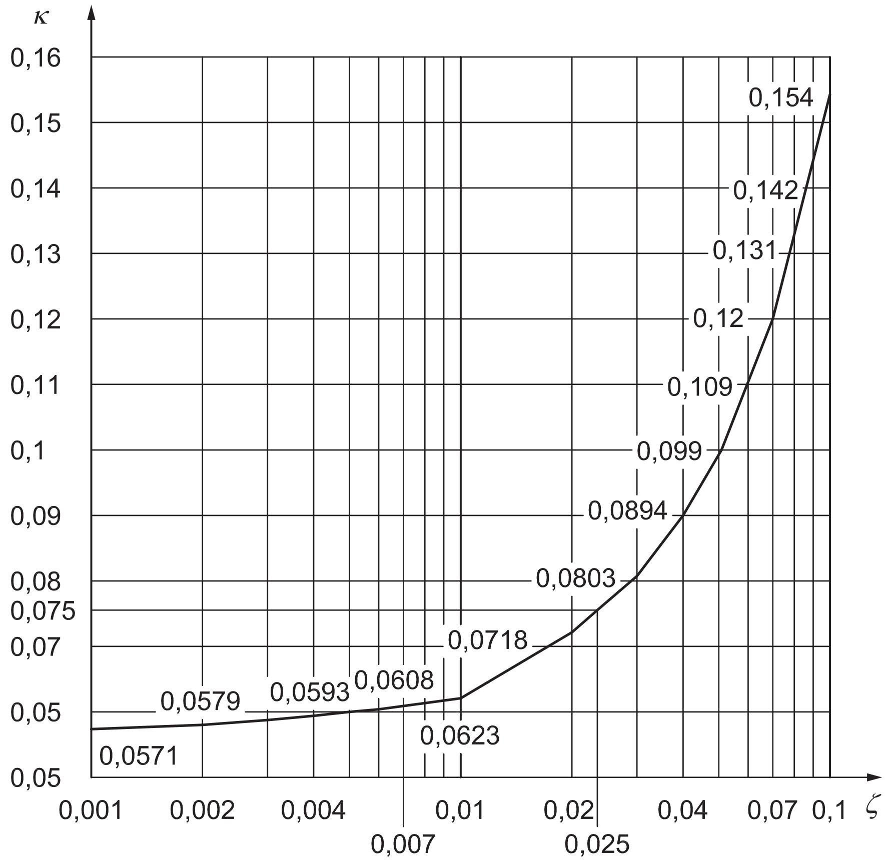


Figure 3 — Factor *κ*

5.3 Calculation of vertical seismic design coefficient, K_V

The vertical seismic design coefficient, K_V, shall be calculated by Formula (5):

$$K_v = c \times K_H \tag{5}$$

where

c is the vertical influence factor which in this International Standard shall be set to 0,5 (see Annex F for further information);

K_H is the horizontal seismic design coefficient as calculated using Formula (1) in 5.2.1.

5.4 Calculation of seismic design loads

5.4.1 Calculation of seismic accelerations

Maximum horizontal and vertical seismic accelerations, a_H and a_v, shall be calculated from horizontal and vertical seismic coefficients, K_H and K_V, using Formulae (6) and (7):

$$a_H = K_H \times g \tag{6}$$

$$a_v = K_V \times g \tag{7}$$

5.4.2 Calculation of seismic forces

Horizontal seismic design force, F_H, and vertical design force, F_V, applied to each component or member of crane structure shall be calculated using Formulae (8) and (9):

$$F_H = K_H \times W_c \quad \text{or} \quad F_H = a_H \times m_c \tag{8}$$

$$F_V = K_V \times W_c \quad \text{or} \quad F_V = a_H \times m_c \tag{9}$$

where

W_c is the dead weight of a member or component of the crane under consideration;

m_c is the mass of a member or component of the crane under consideration.

Seismic forces on suspended load(s) are given by, F_{RV} and F_{RH}, for the vertical and horizontal direction respectively. When the horizontal seismic force(s) can be shown to be negligible, only the vertical seismic force needs to be considered.

$$F_{RH} = K_H \times \chi \times W_R \quad \text{or} \quad F_{RH} = a_H \times \chi \times m_R \tag{10}$$

$$F_{RV} = K_V \times \chi \times W_R \quad \text{or} \quad F_{RV} = a_V \times \chi \times m_R \tag{11}$$

where

χ is the coefficient of seismic effect on suspended load;

W_R is the gross load of the crane;

m_R is the mass of the gross load.

NOTE The selection of χ in the range of 0,0 to 1,0 could be chosen according to crane classes of ISO 4301-1, as in Table 5.

Table 5 — Coefficient of seismic effect on suspended load, χ

Crane class (ISO 4301-1)	A1	A2	A3	A4	A5	A6	A7	A8
χ	0,0	0,14	0,28	0,43	0,57	0,71	0,86	1,0

6 Seismic design based on Maximum Response Spectrum Method

6.1 General

This approach calculates the seismic response of a crane in frequency domain and with contributions from multiple modes of vibration taken into account. Response calculation is usually carried out by calculating separately the responses in three orthogonal directions, two horizontal and one vertical. The response in each direction is obtained as a combination of the responses for the selected vibrational modes. The response for each of these modes is calculated using the maximum response acceleration or displacement obtained from the maximum response spectrum, for the frequency/period and damping value of the mode under consideration and the effective mass of the mode.

In this International Standard, the vertical response spectrum is calculated as 50 % of the horizontal spectrum. In the instance that the two horizontal spectra differ, the vertical response shall be calculated using the larger of the two.

An estimate of the total response in each of the three directions is calculated as a combination of the contributions of the individual significant modes using one of the recognized methods, including the following:

— sum of absolute values of all contributions;

— square root of the sum of the squares of all contributions (SRSS);

— complete quadratic combination of all contributions (CQC).

The total responses in the three directions are combined to yield the total seismic response of the crane, the effect of which can then be considered in conjunction with conventional crane service loads.

Crane structure (with an infinite number of degrees of freedom in reality), shall be reduced to a finite number multi-degree-of-freedom dynamic system using a lumped mass-spring modelling approach, by employing e.g. finite element analysis (FEA) or any other recognized tools and while ensuring that all salient vibrational characteristics of the crane have been retained.

The resulting model shall be used to calculate natural periods/frequencies, vibrational mode shapes and modal participation factors.

Salient steps of seismic response analysis based on the maximum response spectrum method are shown in Table C.1 (where the example shown refers only to seismic excitation in the x direction).

The method assumes elastic and linear behaviour of crane structure and its accuracy increases with the increasing number of modes included in the analysis.

6.2 Calculation procedure for total seismic response (TSR)

From the different possibilities mentioned in 6.1, the present International Standard employs the SRSS method as the default method for combining modal contributions and directional contributions. Employing the various parameters from the modified seismic coefficient method, the procedure is as follows.

Using a dynamic model of the crane, together with the finite element analysis (FEA), the maximum response spectrum method has the following steps.

— Calculation of all natural modes and their frequencies below the rigid body limit, set at 30 Hz.

— Selection of the appropriate basic design acceleration response factor, β_3^*, from the three options shown in Figure 2, depending on the type of subsoil at the crane site, each curve representing the relative spectrum acceleration values [g].

— Selection of the significant modes, so that the total sum of participating modal mass exceeds an agreed limit. A value of 90 % of total mass is the accepted target. However, this value may not be achievable in some instances, such as the following: an overhead crane with stiff end carriages, a crane with base ballast, or a crane with a large suspended load included in the model,

— Inclusion of all non-participating mass responding at the zero period acceleration in the analytical model.

— Calculation of the values of final design spectrum accelerations for the selected significant modes by multiplying the values of the acceleration response factor β_3^* (from the basic design spectrum) by the

  — conversion factor value equal to 0,16 [see Formula (1)],

  — normalized basic acceleration A_{bg} of the crane location (see 5.2.2),

  — subsoil amplification factor β_2 (see 5.2.3), and

  — damping correction factor η (see 5.2.4.3).

— Use of the final design spectrum accelerations and participation factors as inputs for calculating the responses for the selected modes (the response being internal forces, stresses, components or combined and/or displacements, at nodes of interest), for each of the three principal directions (X, Y and Z axis directions). Combine the responses from all selected modes using the SRSS method to calculate the total seismic response [resp_t(X), resp_t(Y) and resp_t(Z), for each of the three directions of excitation.

— Calculation of the total seismic response (TSR) by combining the total responses for the three principal directions, $\text{resp}_t\,(X)$, $\text{resp}_t\,(Y)$ and $\text{resp}_t\,(Z)$, using SRSS method which can be written as follows:

$$\text{TSR}_{\text{SRSS}} = \sqrt{\text{resp}_t(X)^2 + \text{resp}_t(Y)^2 + \text{resp}_t(Z)^2} \tag{12}$$

An alternative to the SRSS Method is a combination method based on a sum of factored total seismic responses in the three principal directions, such as either of the following:

— The 100-40-40 Method (see Reference [5]) where the combinations can be written as follows:

$$\begin{aligned} \text{TSR}_{100-40-40} &= 1,0 \cdot |\text{resp}_t(X)| + 0,4 \cdot |\text{resp}_t(Y)| + 0,4 \cdot |\text{resp}_t(Z)| \text{ or} \\ \text{TSR}_{100-40-40} &= 0,4 \cdot |\text{resp}_t(X)| + 1,0 \cdot |\text{resp}_t(Y)| + 0,4 \cdot |\text{resp}_t(Z)| \text{ or} \\ \text{TSR}_{100-40-40} &= 0,4 \cdot |\text{resp}_t(X)| + 0,4 \cdot |\text{resp}_t(Y)| + 1,0 \cdot |\text{resp}_t(Z)| \end{aligned} \tag{13}$$

— The sum of absolute values of all contributions where the combinations can be written as follows:

$$\begin{aligned} \text{TSR}_{\text{abs}} &= 1,0 \cdot |\text{resp}_t(X)| + 1,0 \cdot |\text{resp}_t(Y)| \text{ or} \\ \text{TSR}_{\text{abs}} &= 1,0 \cdot |\text{resp}_t(Z)| + 1,0 \cdot |\text{resp}_t(Y)| \end{aligned} \tag{14}$$

7 Combinations of seismic and non-seismic effects

7.1 General

Regarding the proof of static strength and the proof of elastic stability, two methods of combining the effects of seismic loads and other non-seismic loads are shown in 7.2 and 7.3, with the method shown in 7.2 being the preferred method of this International Standard.

7.4 deals with the proof of global stability of the crane.

7.2 Proof of static strength: load combinations in accordance with ISO 8686-1

The seismic design load actions calculated in Clause 5 shall be combined with other load actions according to the principles of ISO 8686-1 using the following Table 6.

Table 6 — Seismic load combinations

Loads	Load combinations	
	C1	C2
Mass of the crane	1	1
Mass of the gross load	1	1
Excitation of the crane foundation in vertical direction: F_V, F_{RV}	1	0,4
Excitation of the crane foundation in horizontal direction[a] F_H, F_{RH}	0,4	1

[a] Horizontal forces F_H and F_{RH} can act in any horizontal direction and the direction for a specific crane component under consideration shall be selected so that the most unfavourable effect for that component is obtained.

NOTE 1 ISO 8686-1 defines a risk factor, γ_n, for special cases, where the human or economic consequences of failure are exceptionally severe, in order to obtain an increased reliability. The risk coefficient value is within the range from 1,0 to 2,0. It may be specified individually for a structural member or a mechanism of the crane, or with different values for the proof of global stability of the crane (see 7.4). If a risk factor γ_n greater than 1,0 is selected, then only seismic design load actions in the Table 6 (F_V, F_{RV}, F_H, F_{RH}) are multiplied by γ_n.

NOTE 2 Other similar factors, such as importance factors could be found in national documents.

Different load magnitudes, as well as varying positions of suspended load and mobile parts (e.g. trolley, jib, moving counterweights) shall be considered to determine the maximum effect.

7.3 Proof of static strength: load combination according to SRSS Method

The load case total response resulting from the calculations in 6.2, using the Square Root of the Sum of the Squares Method (SRSS), shall be combined with other load actions using the following Table 7.

Table 7 — Seismic load combinations in SRSS Method

Loads	Load combinations			
	C3	C4	C5	C6
Mass of the crane	1	1	1	1
Mass of the gross load	1	0	1	0
Total seismic response (SRSS) — Loaded crane	1	0	−1	0
Total seismic response (SRSS) — Un laden crane	0	1	0	−1

Different load magnitudes, as well as varying positions of suspended load and mobile parts (e.g. trolley, jib, moving counterweights) shall be considered to determine the maximum effect.

7.4 Proof of global stability

For each orthogonal horizontal direction, the seismic design load actions shall be combined with other load actions according to the principles of ISO 8686-1 using the following Table 8.

Table 8 — Load combinations for global stability including seismic loads

Loads	Load combination C1
Mass of the crane, unfavourable[a]	1,05
Mass of the crane, favourable[a]	1,0
Mass of the gross load	1,0
Total seismic load	1,0
[a] When calculating the loads from gravitation for a given load combination and crane configuration, the masses of the different parts of the crane either increase ("unfavourable") or decrease the resulting load effect ("favourable") in the critical point under consideration.	

The risk factor, γ_n, whose value is within the range 1,0 to 2,0 shall be specified only for the total seismic load. It may be specified also for unfavourable masses of the crane.

Horizontal forces, F_H and F_{RH}, can act in any horizontal direction. Therefore, the direction shall be selected in a manner that the most unfavourable effect for the crane stability is obtained.

All suspended load magnitudes and crane configurations shall be considered when checking crane stability under seismic conditions unless governing load combination(s) for the stability case under consideration can be clearly identified.

7.5 Proof of competence for crane structures

The proof of competence for crane structures shall be made according to ISO 20332, where the value of the general resistance factor, γ_m, shall be set to 1,0, and with second order effects taken into consideration when appropriate.

Annex A
(informative)

Flow chart of seismic design

A typical flow chart of seismic design procedure of cranes based on the modified seismic coefficient method is shown in Figure A.1.

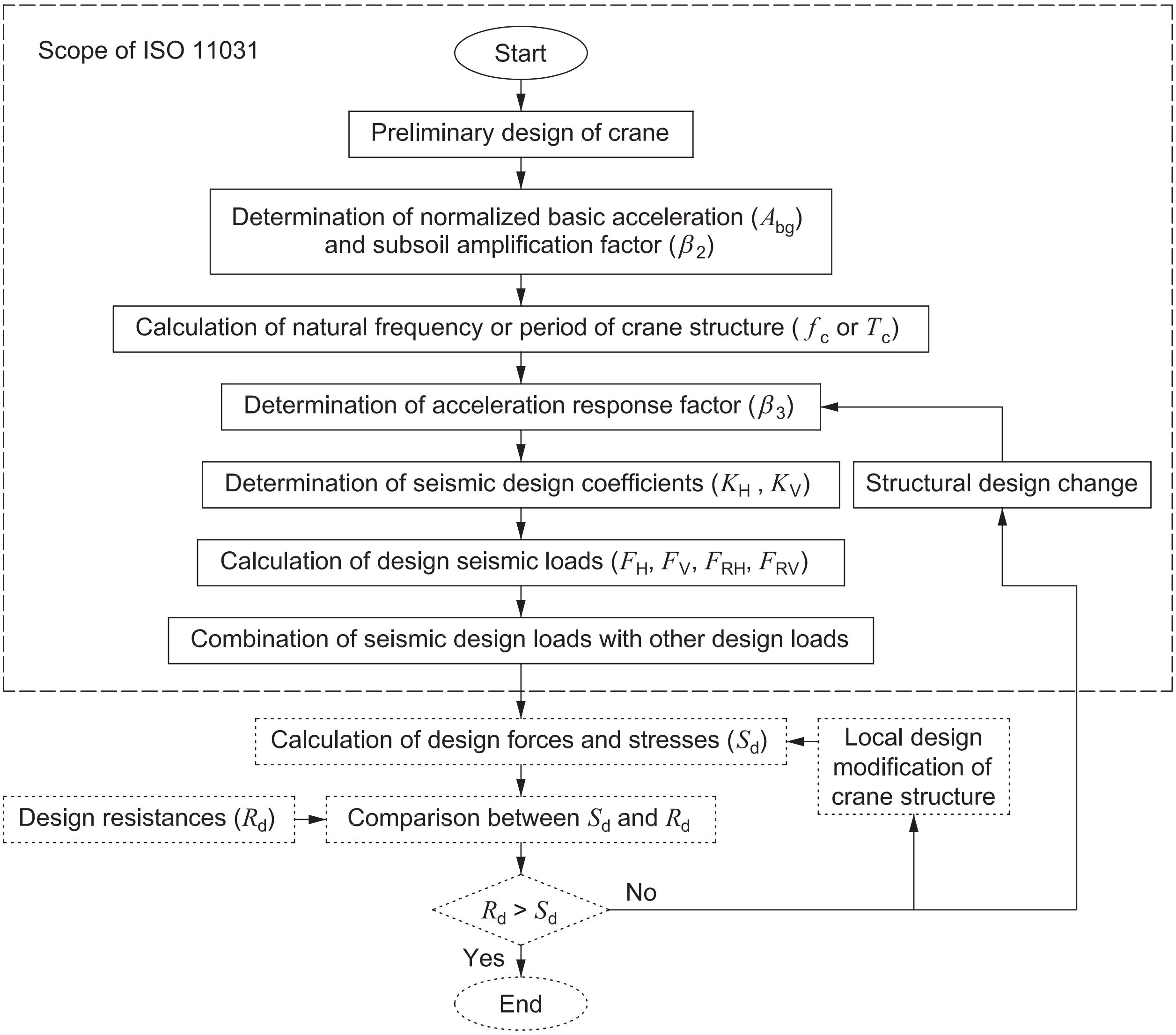


Figure A.1 — Seismic design flow chart of cranes based on modified seismic coefficient method

Annex B
(informative)

Design accelerations and seismic zones

B.1 General

The normalized basic acceleration, A_{bg}, depends on the earthquake hazard assessment at the zone and the country where the crane is located. The normalized surface ground acceleration, A_{sg}, takes into account the characteristics of the subsoil.

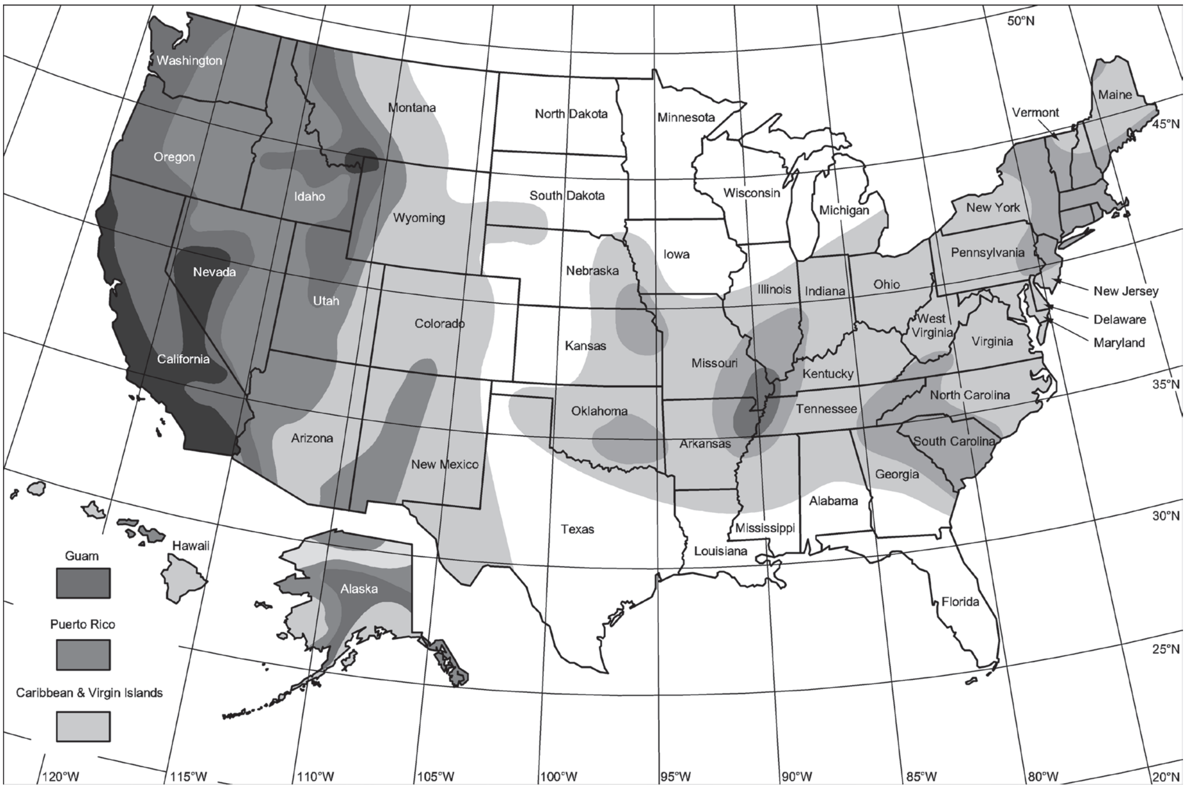
The normalized basic accelerations, A_{bg}, given in tables for different countries are based on a return period of 475 years. They correspond to a 10 % probability of exceedance in 50 years (0,2 % probability of exceedance in one year).

Data specific for the country and zone under consideration should be used. In the absence of such specific data, examples of national data included in this Annex may be used.

As the latest data becomes available, seismic zones for other regions or countries not included in the present Annex may be included in future editions of this International Standard.

B.2 Seismic zones in the USA

Figure B.1 illustrates the seismic zone map for areas in United States of America (USA) by Uniform Building Code (UBC). USA land is divided into five zones, namely, 1, 2A, 2B, 3 and 4.



Key

Zone 0	Zone 2B
Zone 1	Zone 3
Zone 2A	Zone 4

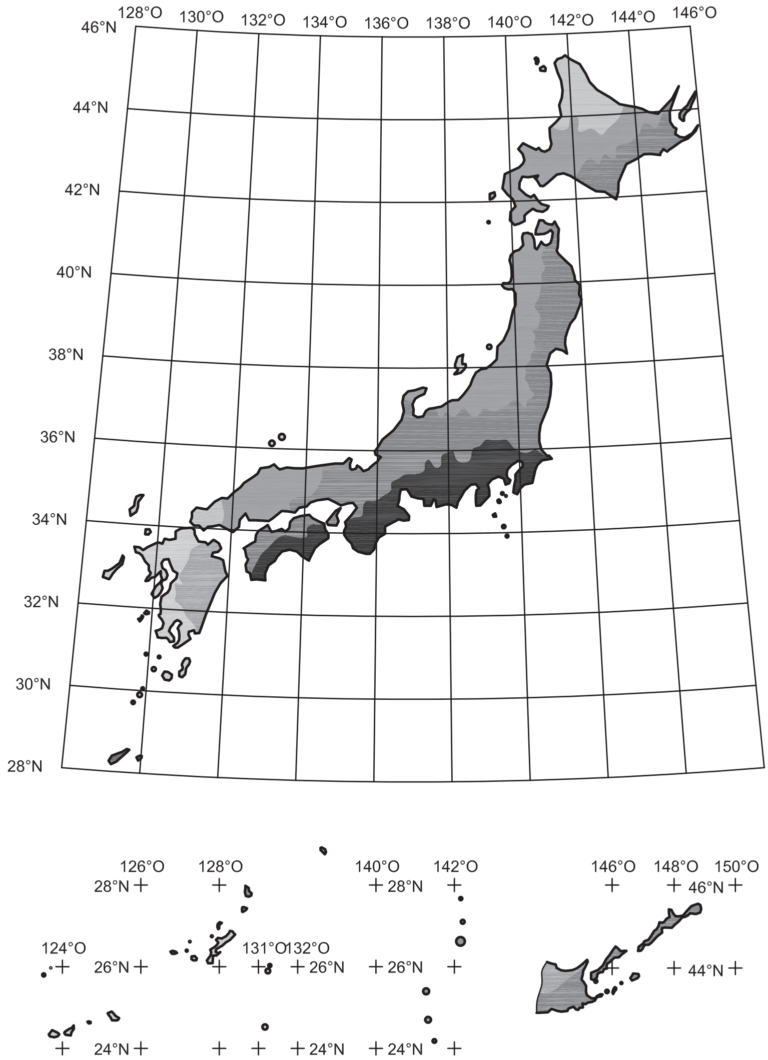
Figure B.1 — Seismic zones in USA

Table B.1 — Normalized basic accelerations in the USA

Zone	Normalized basic accelerations, A_{bg} g
0	0
1	0,075
2A	0,15
2B	0,20
3	0,30
4	0,40

B.3 Seismic zones in Asia

B.3.1 Japan



Key

Zone C

Zone B

Zone A

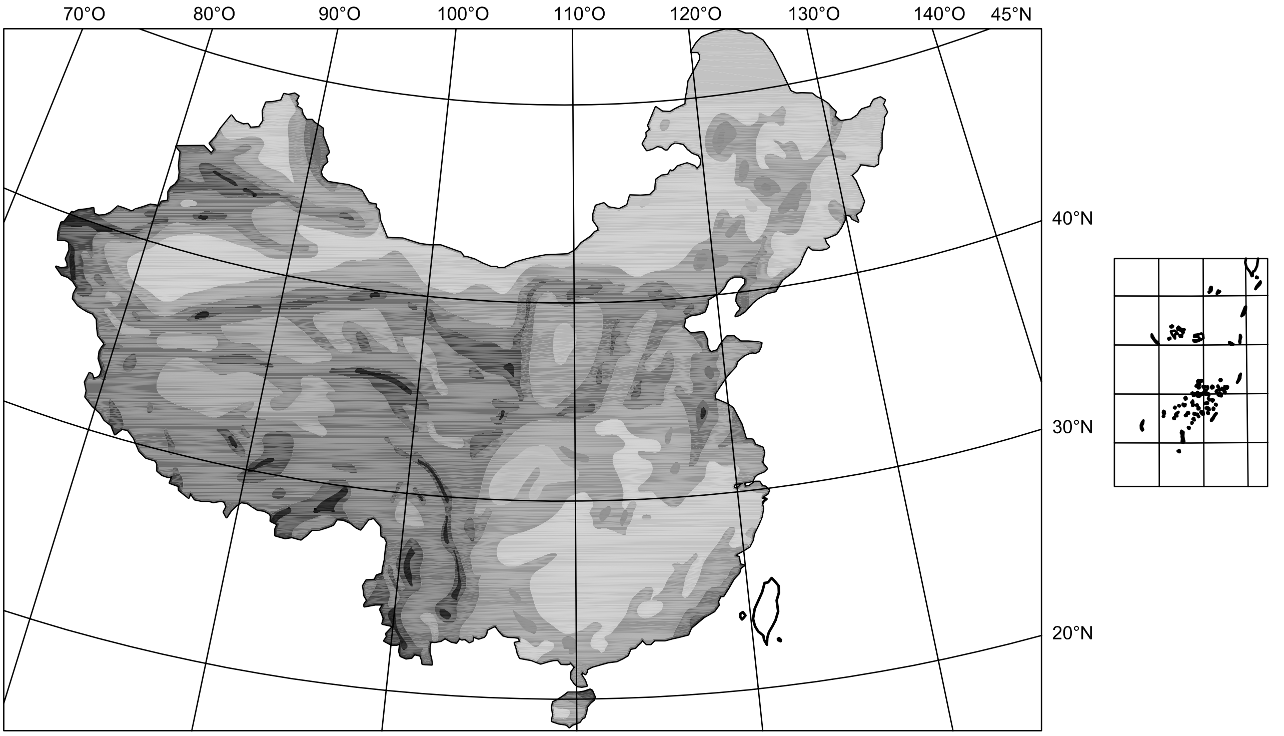
Zone SA

Figure B.2 — Seismic zones in Japan

Table B.2 — Normalized basic accelerations in Japan

Zone	Normalized basic accelerations, A_{bg} g
SA	0,45
A	0,36
B	0,27
C	0,18

B.3.2 China



Key

Zone 1

Zone 2

Zone 3

Zone 4

Zone 5

Figure B.3 — Seismic zones in China

Table B.3 — Normalized basic accelerations in China

Zone	Normalized basic accelerations, A_{bg} g
1	0,05
2	0,1
3	0,25
4	0,35
5	0,45

B.3.3 India

Key

Zone II

Zone III

Zone IV

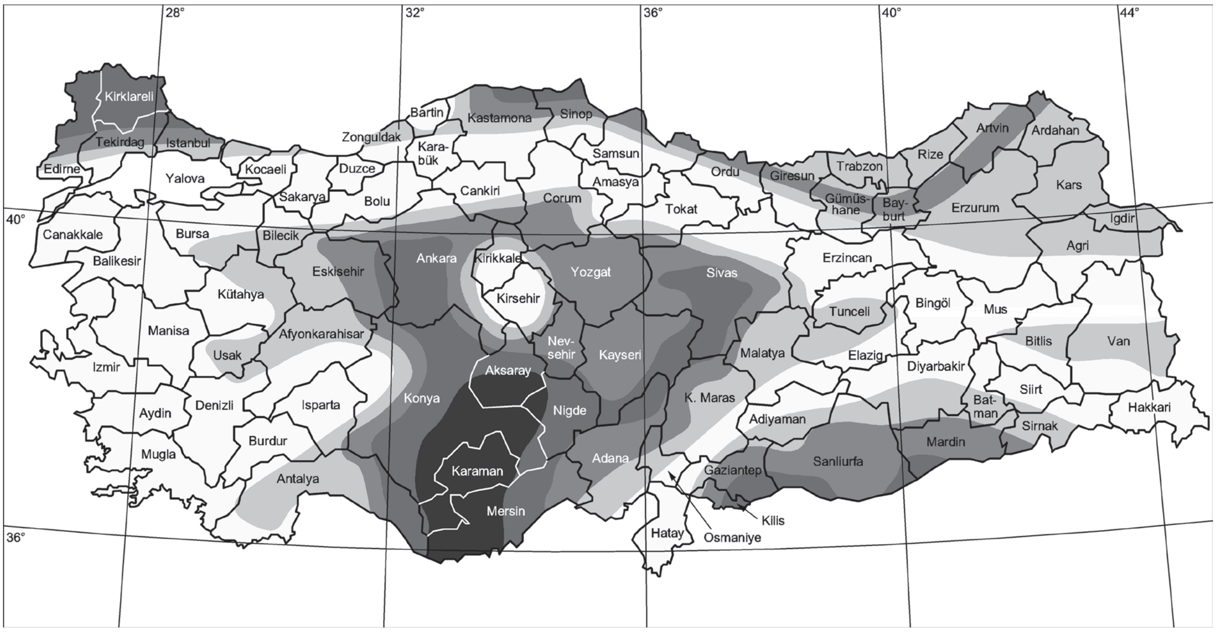
Zone V

Figure B.4 — Seismic zones in India

Table B.4 — Normalized basic accelerations in India

Zone	Normalized basic accelerations, A_{bg} g
II	0,05
III	0,1
IV	0,25
V	0,35

B.3.4 Turkey



Key

Zone I

Zone II

Zone III

Zone IV

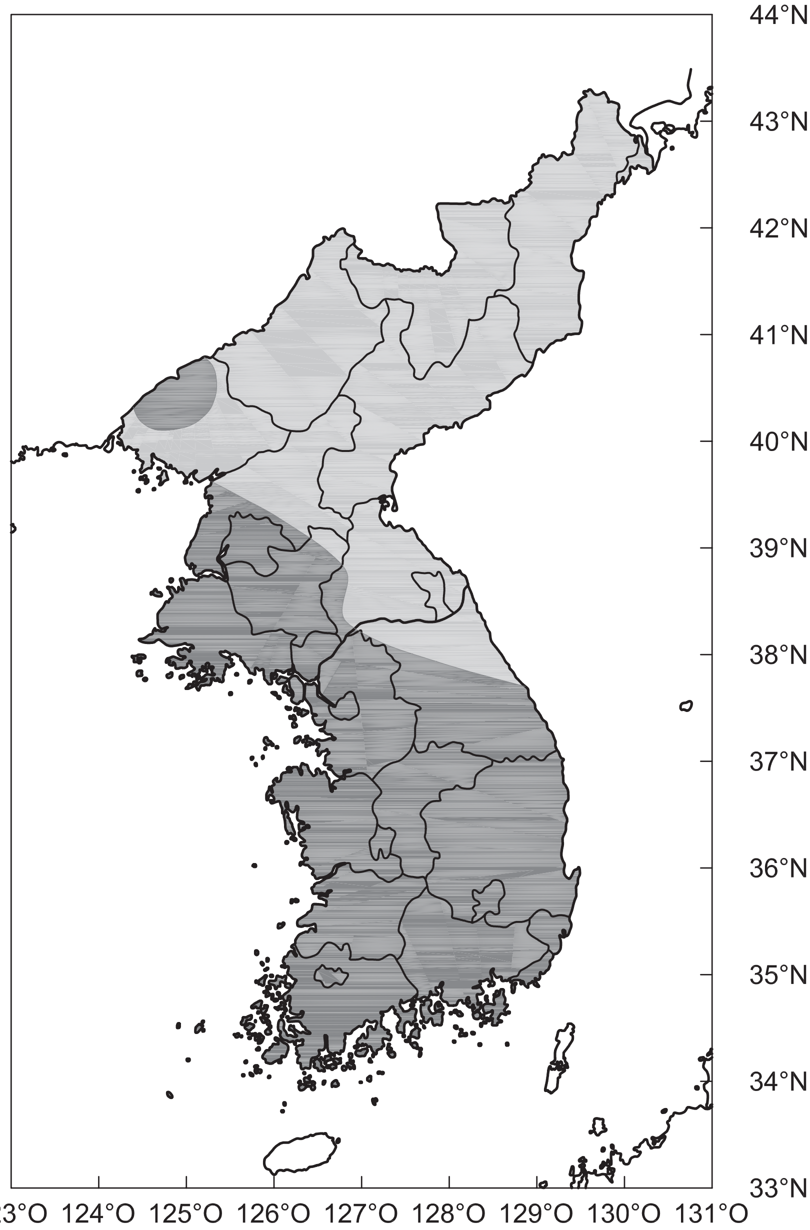
Zone V

Figure B.5 — Seismic zones in Turkey

Table B.5 — Normalized basic accelerations in Turkey

Zone	Normalized basic accelerations, A_{bg} g
I	>0,40
II	0,40
III	0,30
IV	0,20
V	0,10

B.3.5 Korea



Key

Zone I

Zone II

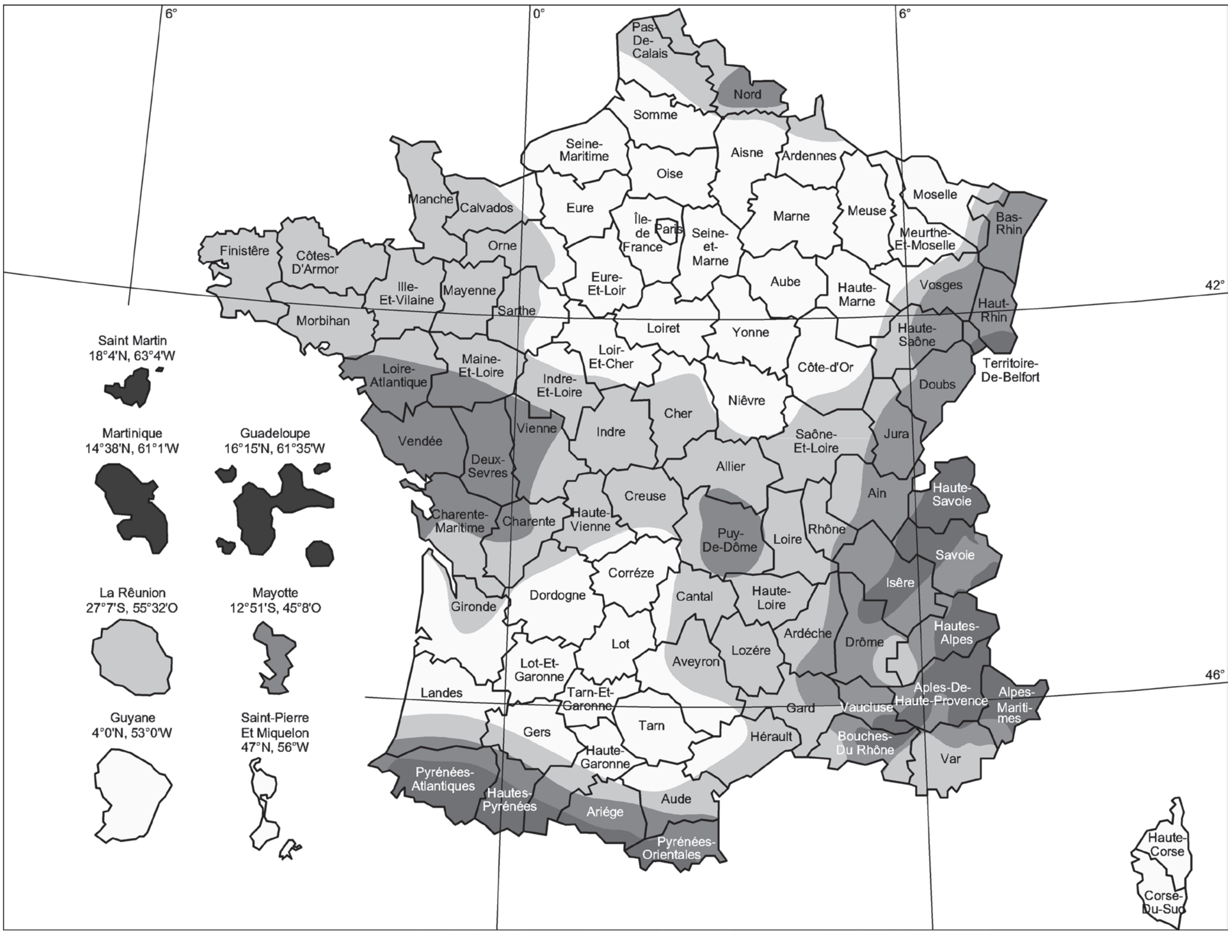
Figure B.6 — Seismic zones in Korea

Table B.6 — Normalized basic accelerations in Korea

Zone	Normalized basic accelerations, A_{bg} g
I	0,11
II	0,07

B.4 Seismic zones in Europe

B.4.1 France



Key

Zone1 | Zone 4

Zone 2 | Zone 5

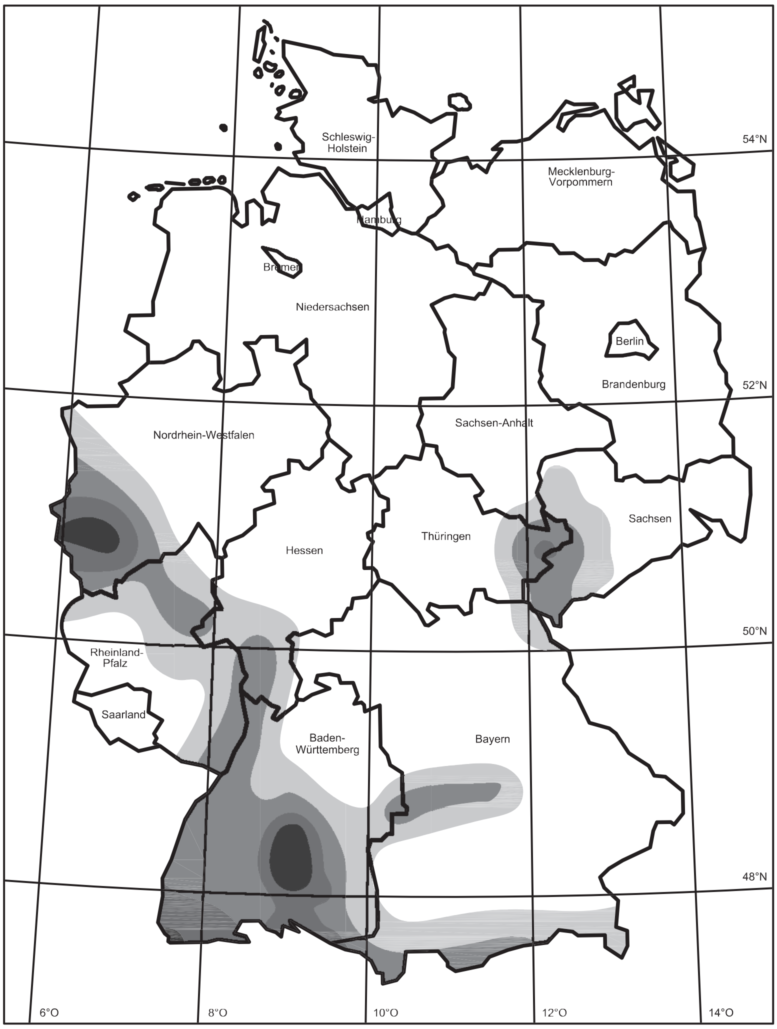
Zone 3

Figure B.7 — Seismic zones in France

Table B.7 — Normalized basic accelerations in France

Zone	Normalized basic accelerations, A_{bg} g
1	0,04
2	0,07
3	0,11
4	0,16
5	0,30

B.4.2 Germany



Key

Zone 0

Zone 1

Zone 2

Zone 3

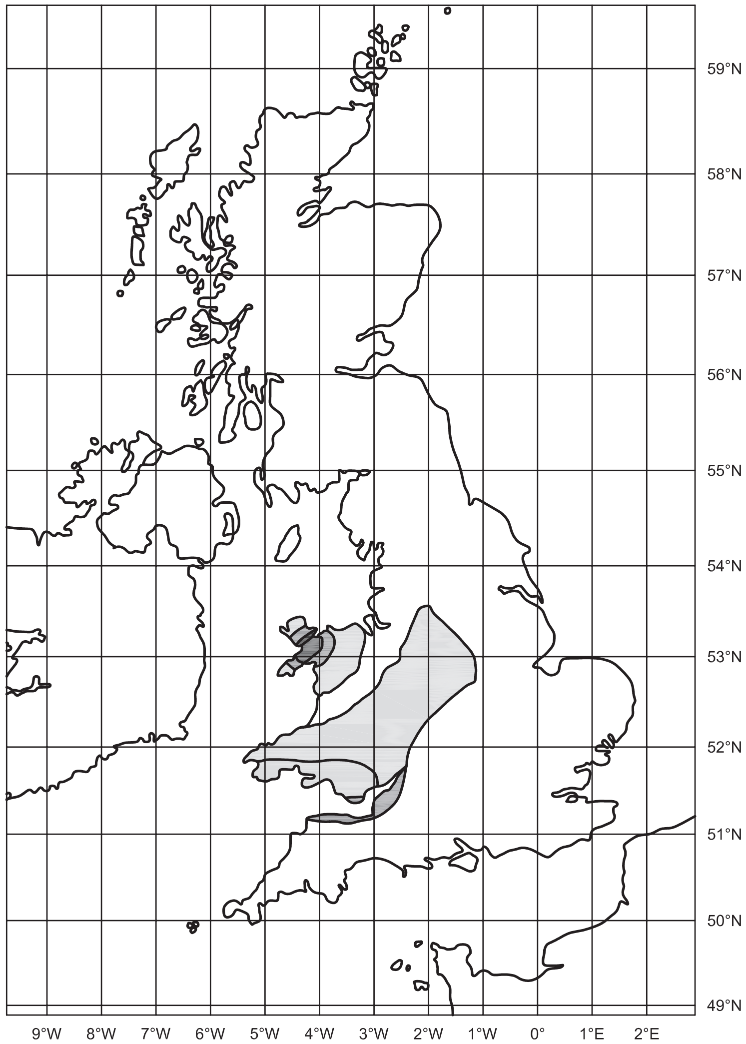
Figure B.8 — Seismic zones in Germany

Table B.8 — Normalized basic accelerations in Germany

Zone	Normalized basic accelerations, A_{bg} g
0	0,02
1	0,04
2	0,06
3	0,08

NOTE The proof of competence for seismic loads is not necessary for cranes located in zone 0.

B.4.3 United Kingdom



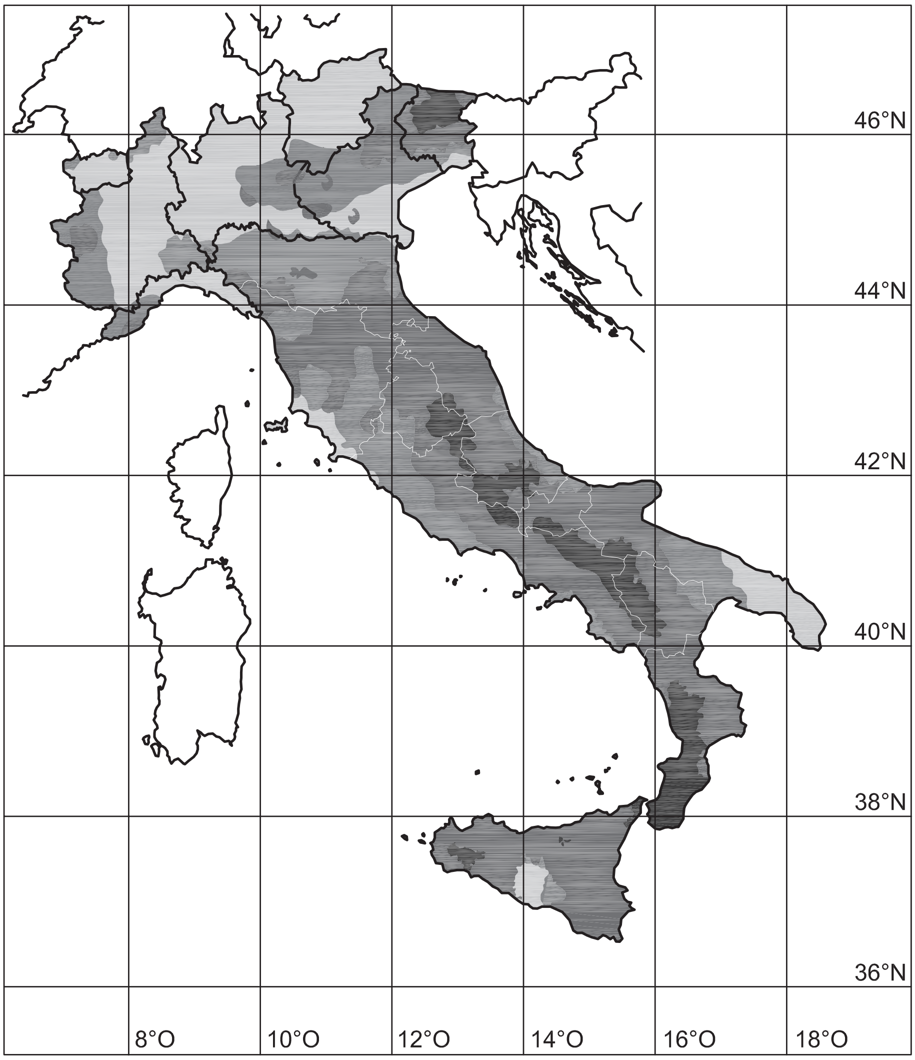
Key

Figure B.9 — Seismic zones in United Kingdom

Table B.9 — Normalized basic accelerations in United Kingdom

Zone	Normalized basic accelerations, A_{bg} g
0	0,04
1	0,06
2	0,10
3	0,12

B.4.4 Italy



Key

Zone1

Zone 2

Zone 3

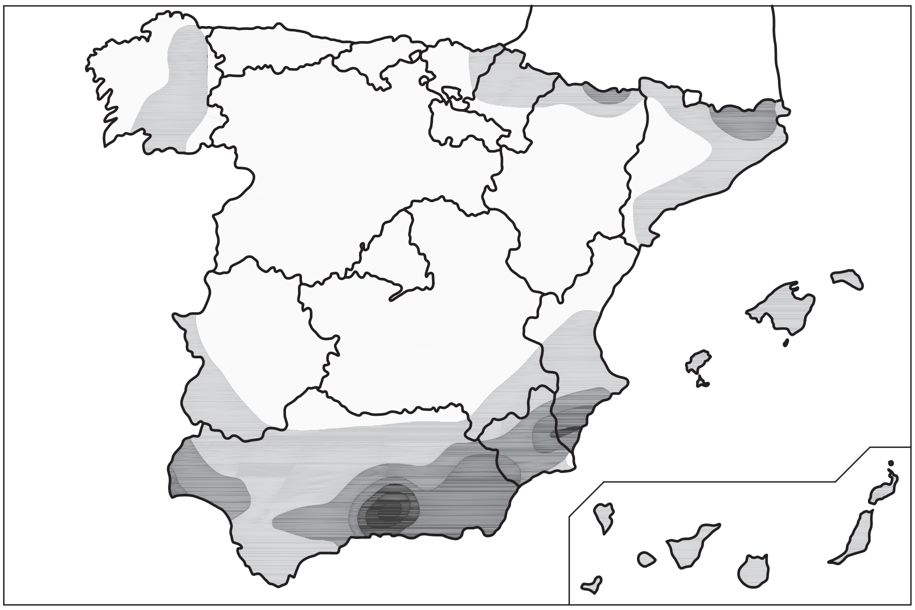
Zone 4

Figure B.10 — Seismic zones in Italy

Table B.10 — Normalized basic accelerations in Italy

Zone	Normalized basic accelerations, A_{bg} g
1	0,05
2	0,15
3	0,25
4	0,35

B.4.5 Spain



Key

Zone 1

Zone 2

Zone 3

Zone 4

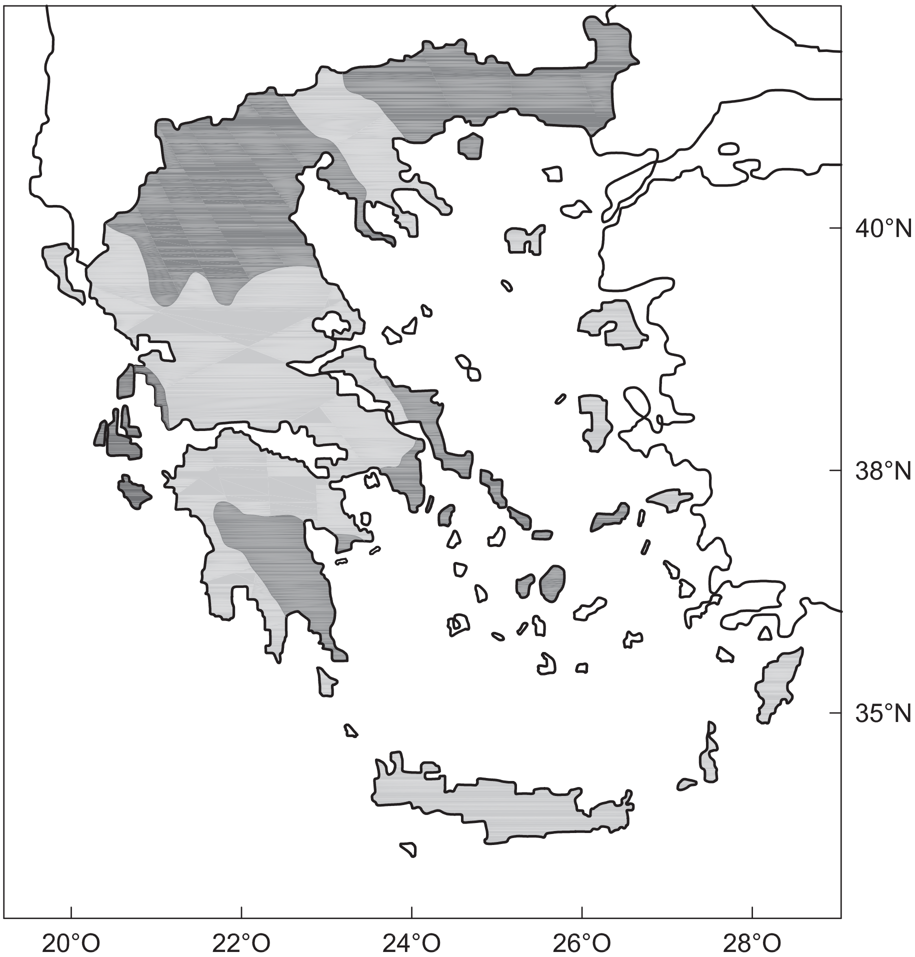
Zone 5

Figure B.11 — Seismic zones in Spain

Table B.11 — Normalized basic accelerations in Spain

Zone	Normalized basic accelerations, A_{bg} g
1	0,04
2	0,08
3	0,12
4	0,16
5	≥0,16

B.4.6 Greece



Key

Zone I

Zone II

Zone III

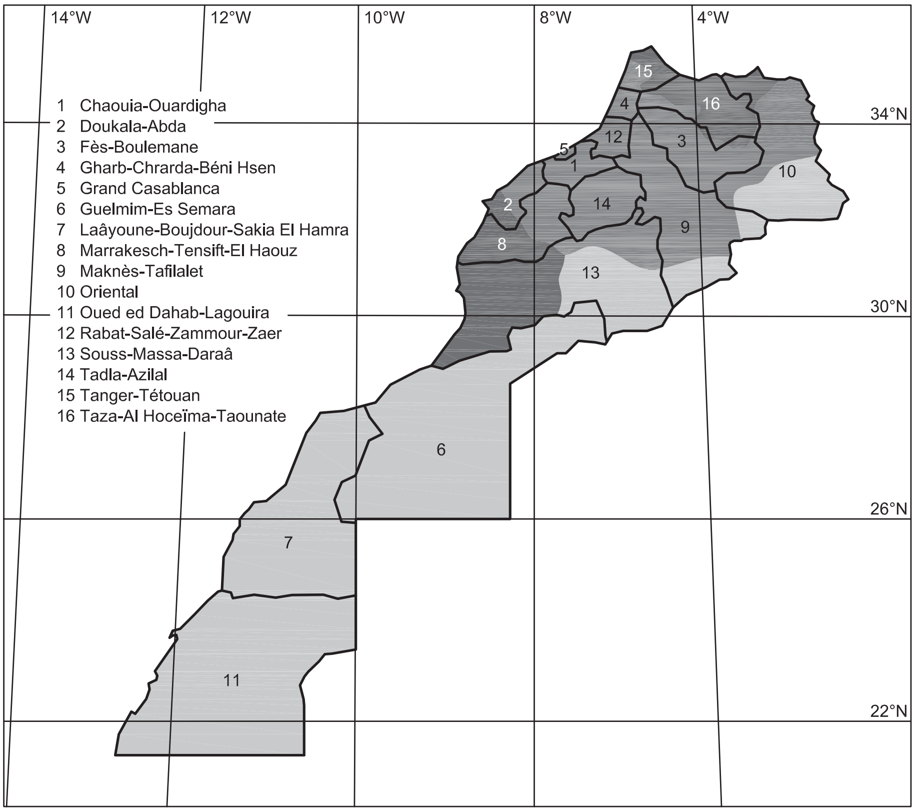
Figure B.12 — Seismic zones in Greece

Table B.12 — Normalized basic accelerations in Greece

Zone	**Normalized basic accelerations,** A_{bg} g
I	0,16
II	0,24
III	0,36

B.5 Seismic zones in Africa

B.5.1 Morocco



Key

Zone 1

Zone 2

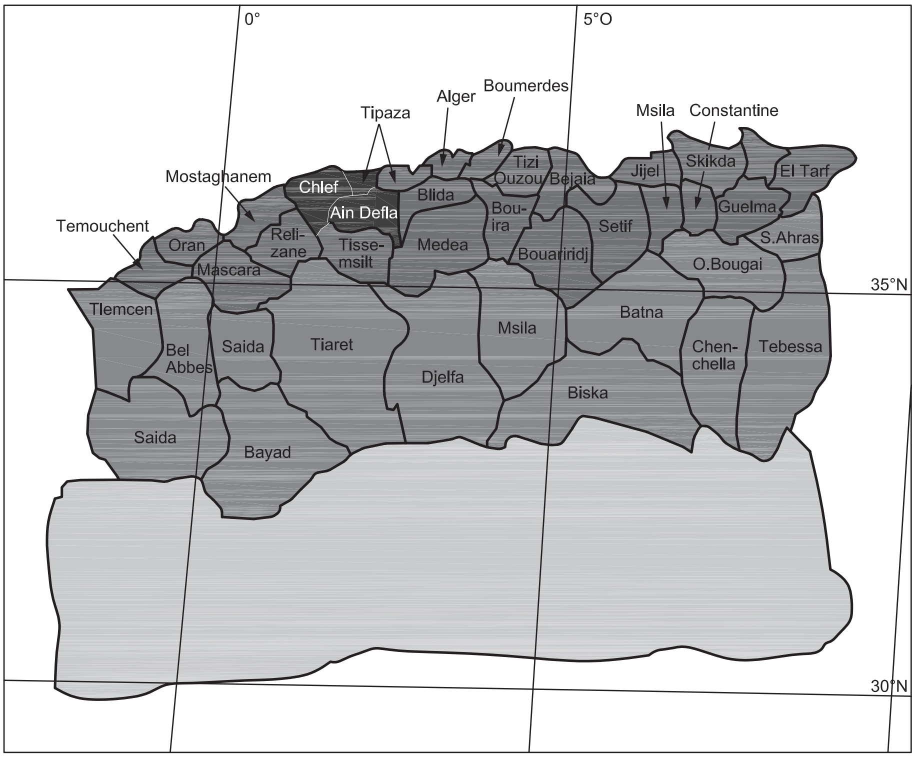
Zone 3

Figure B.13 — Seismic zones in Morocco

Table B.13 — Normalized basic accelerations in Morocco

Zone	Normalized basic accelerations, A_{bg} g
1	0,10
2	0,25
3	0,35

B.5.2 Algeria



Key

Zone 0

Zone 1

Zone 2

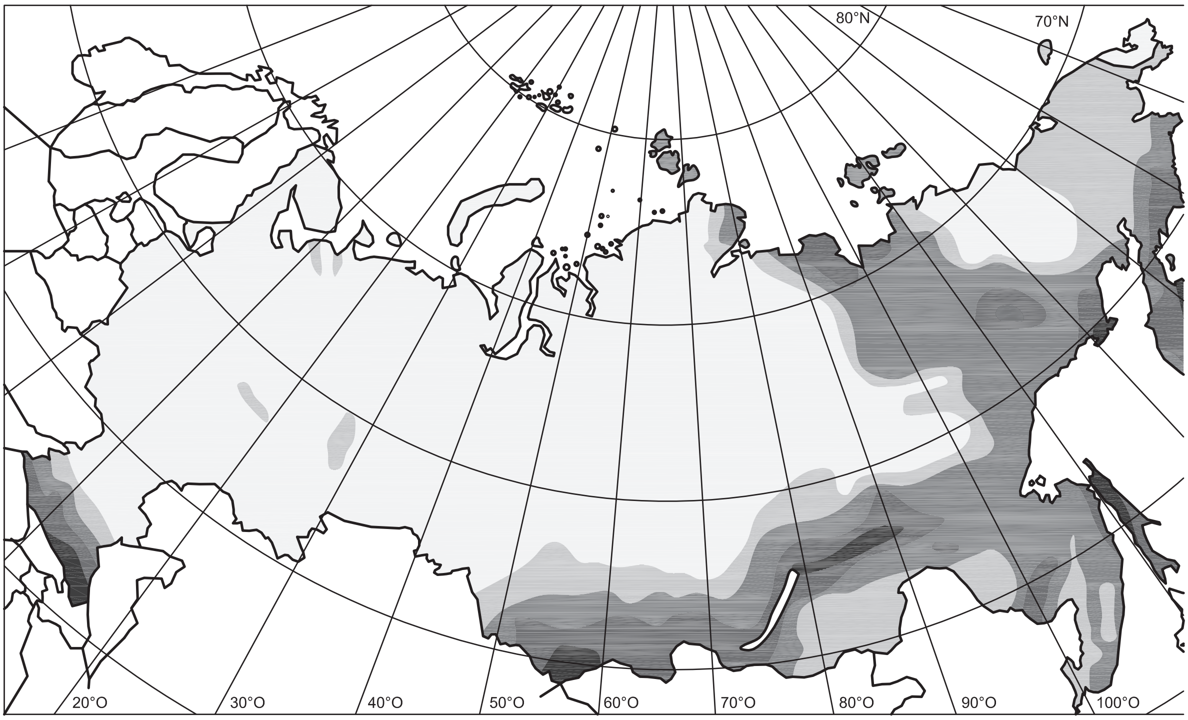
Zone 3

Figure B.14 — Seismic zones in Algeria

Table B.14 — Normalized basic accelerations in Algeria

Zone	Normalized basic accelerations, A_{bg} g
0	0,05
1	0,10
2	0,25
3	0,35

B.6 Seismic zones in Russia



Key

Zone 5

Zone 6

Zone 7

Zone 8

Zone 9

Figure B.15 — Seismic zones in Russia

Table B.15 — Normalized basic accelerations in Russia

Zone	Normalized basic accelerations, A_{bg} g
5	0,025
6	0,05
7	0,1
8	0,2
9	0,4

B.7 Seismic zones in New Zealand

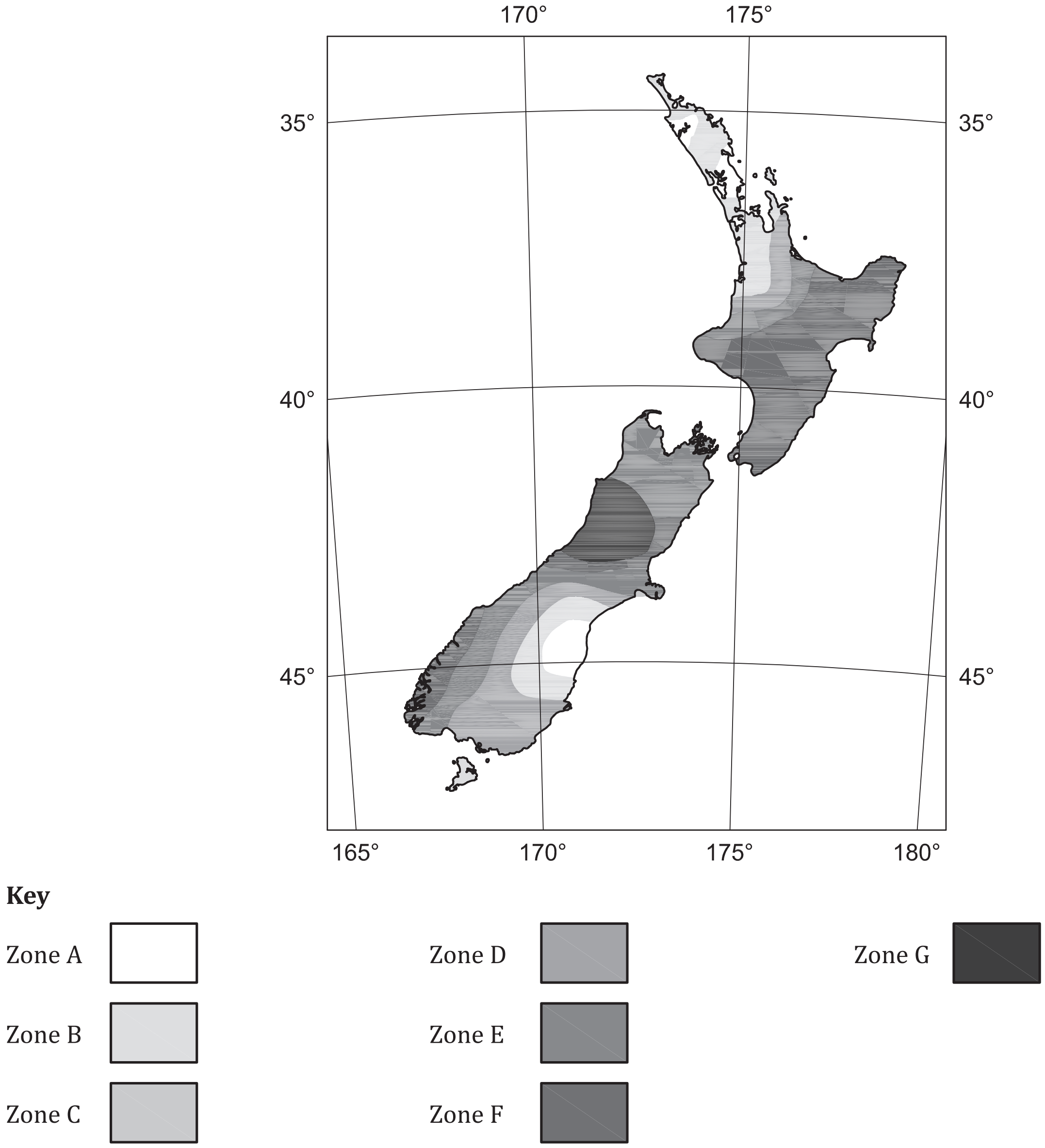


Figure B.16 — Seismic zones in New Zealand

Table B.16 — Normalized basic accelerations in New Zealand

Zone	Normalized basic accelerations, A_{bg} g
A	0,18
B	0,23
C	0,28
D	0,33
E	0,38
F	0,42
G	>0,45

Annex C
(informative)

Information about Maximum Response Method

The steps of the Response Spectrum Analysis Method are described in Table C.1.

Table C.1 — Steps for calculating seismic response in one direction of excitation [designated as “resp (dir)”] using Response Spectrum Analysis Method

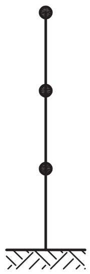
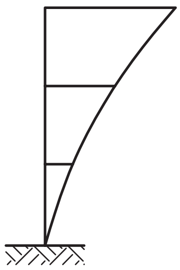
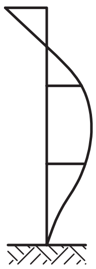
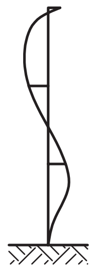
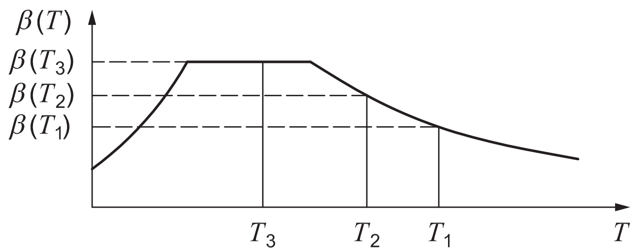
Step 1 — Calculation of mode shapes and natural periods/frequencies			
			
Lumped-mass idealization of a cantilever beam	Mode shape 1	Mode shape 2	Mode shape 3
Step 2 — Determination of seismic accelerations from response spectra, for the direction under consideration and for the selected modes			
			

Table C.1 — (*continued*)

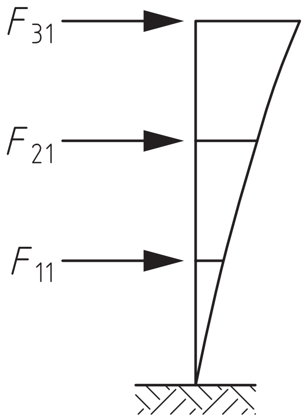
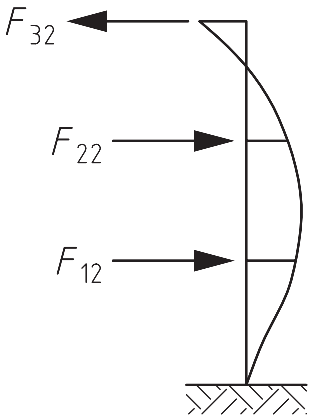
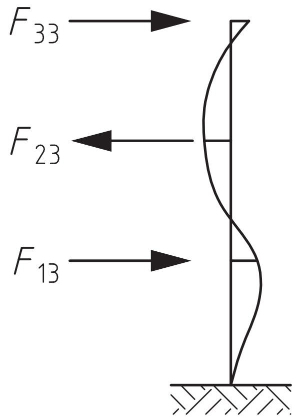
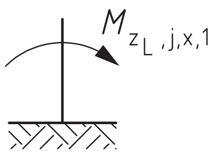
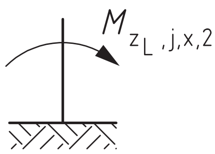
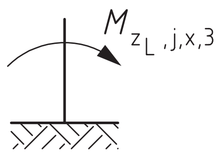
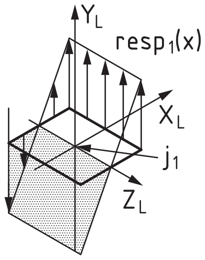
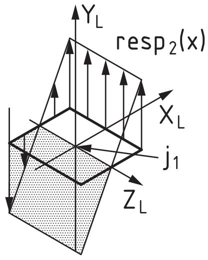
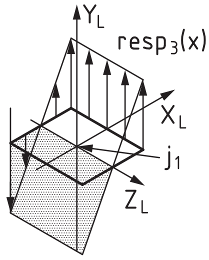
Step 3 — Calculation of seismic forces, F_{ji}, at all nodes and for all selected modes		
		
F_{j1} seismic force at node *j* for mode 1	F_{j2} seismic force at node *j* for mode 2	F_{j3} seismic force at node *j* for mode 3
NOTE For the selected modes, seismic loads at all nodes are calculated using participation factors and seismic response spectrum accelerations for the modes.		
Step 4 — Calculation of modal components of internal forces N_{ji} (axial forces), V_{ji} (shear forces) and M_{ji} (bending moments) at all nodes *j* and for all selected modes *i*		
EXAMPLE Modal components $M_{zL,j,x}$ of bending moment at node *j*, for modes 1, 2 and 3 that are selected as significant, all due to seismic excitation in the direction *x*.		
		
$M_{zL,j,x,1}$ bending moment at the section at node *j*, at tower base, about local axis z_L and for mode 1 (seismic excitation in direction *x*)	$M_{zL,j,x,2}$ bending moment at the section at node *j*, at tower base, about local axis z_L and for mode 2 (seismic excitation in direction *x*)	$M_{zL,j,x,3}$ bending moment at the section at node *j*, at tower base, about local axis z_L and for mode 3 (seismic excitation in direction *x*)

Table C.1 — (*continued*)

Step 5 — Calculation of modal components of seismic response, resp (dir) (viz. stresses and/or displacements of interest) due to the internal forces *N*, *V* and *M*, calculated at Step 4, for all selected modes, at all nodes of interest

EXAMPLE Response required, the modal components of normal bending stress $\sigma_{b,zL,j1,x,i}$, i.e. normal stress due to a bending moment about the local axis z_L, at point 1of section at node *j*, at the base of tower, for modes 1, 2 and 3, selected as significant (all due to seismic excitation in the direction *x*)

		
$resp_1$ (*x*) = $\sigma_{b,zL,j1,x,1}$ normal stress at point 1 of section at node *j*, due to bending about the local axis z_L, from mode 1 (seismic excitation in direction *x*)	$resp_2$ (*x*) = $\sigma_{b,zL,j1,x,2}$ normal stress at point 1 of section at node *j*, due to bending about the local axis z_L, from mode 2 (seismic excitation in direction *x*)	$resp_3$ (*x*) = $\sigma_{b,zL,j1,x,3}$ normal stress at point 1 of section at node *j*, due to bending about the local axis z_L, from mode 3 (seismic excitation in direction *x*)

Step 6 — Calculation of the total seismic response, TSR, for the direction of seismic excitation under consideration, made up of the contributions from all selected modes, using the SRSS method (total stresses or displacements of interest, at all nodes of interest):

$$\mathrm{resp}_t\left(\mathrm{dir}\right)=\sqrt{\mathrm{resp}_1\left(\mathrm{dir}\right)^2+\mathrm{resp}_2\left(\mathrm{dir}\right)^2+\mathrm{resp}_3\left(\mathrm{dir}\right)^2+\cdots}$$

EXAMPLE Total normal bending stress $\sigma_{b,zL,j1,x}$ due to seismic excitation in the direction *x*, at point 1 of section at node *j* (at the base of tower), is made up from the normal stress components, due to bending moments about the local axis z_L, calculated in Step 5 for modes, 1, 2 and 3 (selected as significant). It is given by

$$\sigma_{b,z_L,j1,x}=\sqrt{\sigma_{b,z_L,j1,x,1}{}^2+\sigma_{b,z_L,j1,x,2}{}^2+\sigma_{b,z_L,j1,x,3}{}^2}$$

Annex D
(informative)

Time History Response Method and a comparison of different seismic methods available

D.1 General

The Time History Analysis Method is an alternative to the modified seismic coefficient method and maximum response spectrum method for the calculation of seismic loads, particularly when a very precise assessment of seismic design loads is required and/or nonlinear behaviour is to be allowed for.

The time history method is a very precise method, but only for the particular time history used as input. Since a number of time histories can be generated to represent a single response spectrum with a greater or lesser effect on the crane under consideration, it is important that at least two, preferably three statistically independent time histories that generate the same response spectrum curve are used, otherwise results may be misleading.

The features of each method are compared in Table D.1.

Table D.1 — Features of three methods of seismic response analysis

	Complexity and difficulty level	**Type of structural and dynamic behaviour covered**	**Accuracy of calculated seismic accelerations**	**Accuracy of seismic load estimation**	**Response characteristics**
Modified Seismic Coefficient Method	Simple and easy, computer resources advisable but not essential	Elastic and linear	Approximation	Approximation, aimed to be conservative	Depend on the response spectrum used and estimate of natural period/frequency of the crane
Response Spectrum Analysis Method (see Clause 6 and Annex C)	More complex and computer resources required	Elastic and linear	Higher precision, but within the limitation of the method using only the maxima of seismic accelerations	Higher precision, but yielding only upper limit estimates of seismic forces and crane response	Depend on the response spectrum used
Time History Response Analysis Method (see D.2)	Complex and with high demands on computer resources	Elastic, plastic, linear and nonlinear	Precise simulation of actual seismic accelerations for the input used	Precise simulation of crane response to seismic excitation and of resulting seismic forces	Depend on the design seismic wave

D.2 Method seismic response by Time History Analysis Method

D.2.1 General

Seismic accelerograms are used for inputs of seismic excitation and these can be recorded or synthetic (artificial or simulated).

Recorded accelerograms are records of actual earthquakes from the past.

Artificial accelerograms are computer generated. They are designed to match response spectrum required using the relationship between the design value of the peak ground acceleration and the duration of the stationary part of the accelerogram.

Accelerograms are generated using a physical simulation of the source of the seismic disturbance.

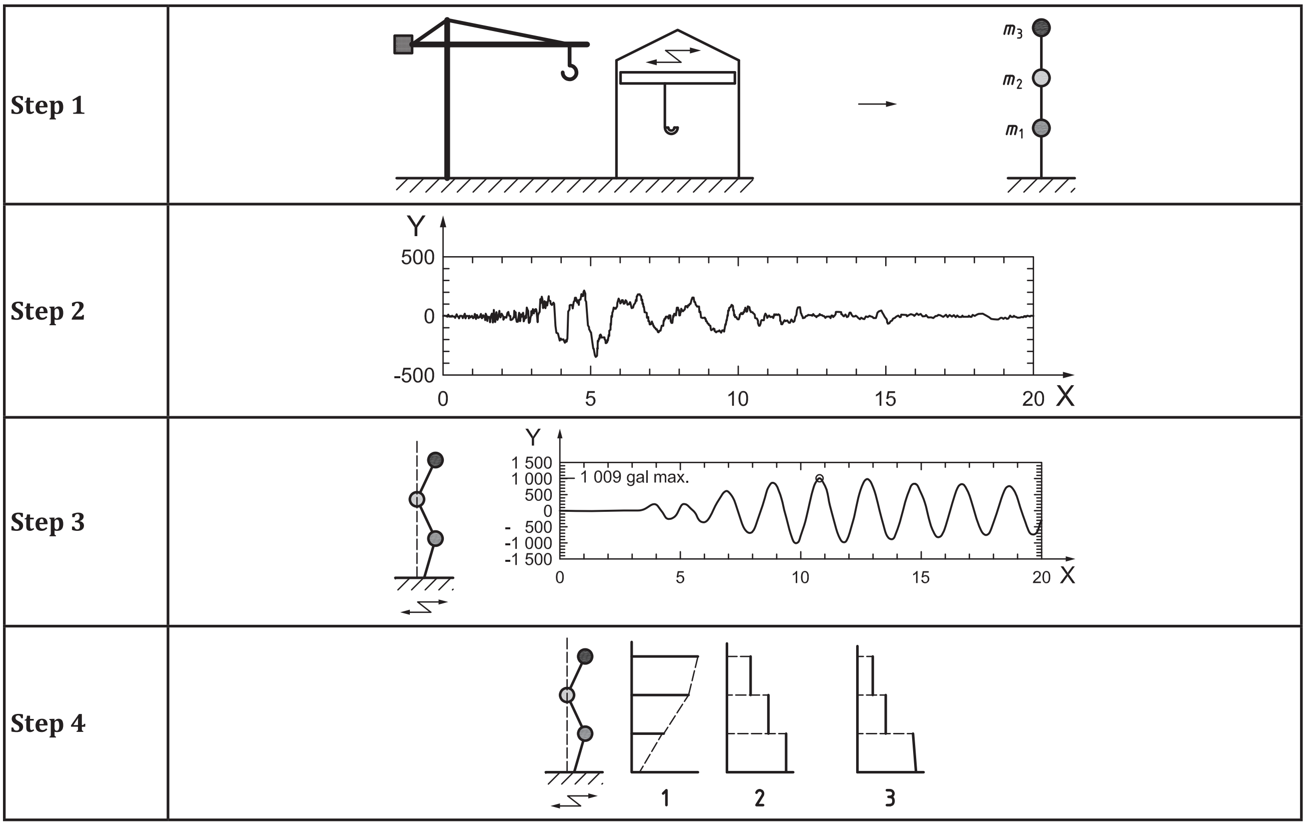
A different accelerogram may have to be employed for each direction of seismic excitation.

Two options of time history analysis are available.

— The full direct integration method applies numerical step-by-step integration to the full set of system formulae. It is the more powerful of the two methods and can deal with all types of nonlinearities but at a cost of being significantly more complex and demanding (both on computer and engineering resources).

— The mode superposition method calculates the time history of seismic response from a sum of factored selected mode shapes, obtained previously from a modal analysis. It is less demanding than the full direct integration method but its ability of handling nonlinearities is limited.

Salient steps of seismic response analysis in one direction and based on the Time History Method are shown in Figure D.1.



Key

Step 1: Modelling of crane structure using spring-lumped mass technique, e.g. as performed by finite elements analysis packages.

Step 2: Selection of earthquake excitation to be used as input, i.e. seismic accelerogram (artificial, recorded or simulated.

Step 3: Calculation of crane response, in terms of time variable nodal accelerations, calculated using step-wise integration of system equations in time domain, with the earthquake excitation and other non-seismic loads, e.g. self-weights, as applied as input loads.

Step 4: Calculation of time-variable displacements and internal forces at nodes and in structural members and determination of maximum values

1 maximal acceleration

2 maximum shear force

3 maximum bending moment

X time

Y acceleration

Figure D.1 — Steps of time history seismic response analysis (in one direction)

Annex E
(informative)

Relation between basic acceleration, Mercalli and Richter scales

Table E.1 — Relation between basic acceleration, Mercalli and Richter scales

Richter scale	Approximate basic acceleration a_g m/s^2	Approximate Mercalli equivalent
<3,5	<0,01	I
3,5	0,025	II
4,2	0,025	III
4,5	0,10	IV
4,8	0,25	V
5,4	0,50	VI
6,1	1	VII
6,5	2,5	VIII
6,9	2,5	IX
7,3	5	X
8,1	7,50	XI
>8,1	9,80	XII

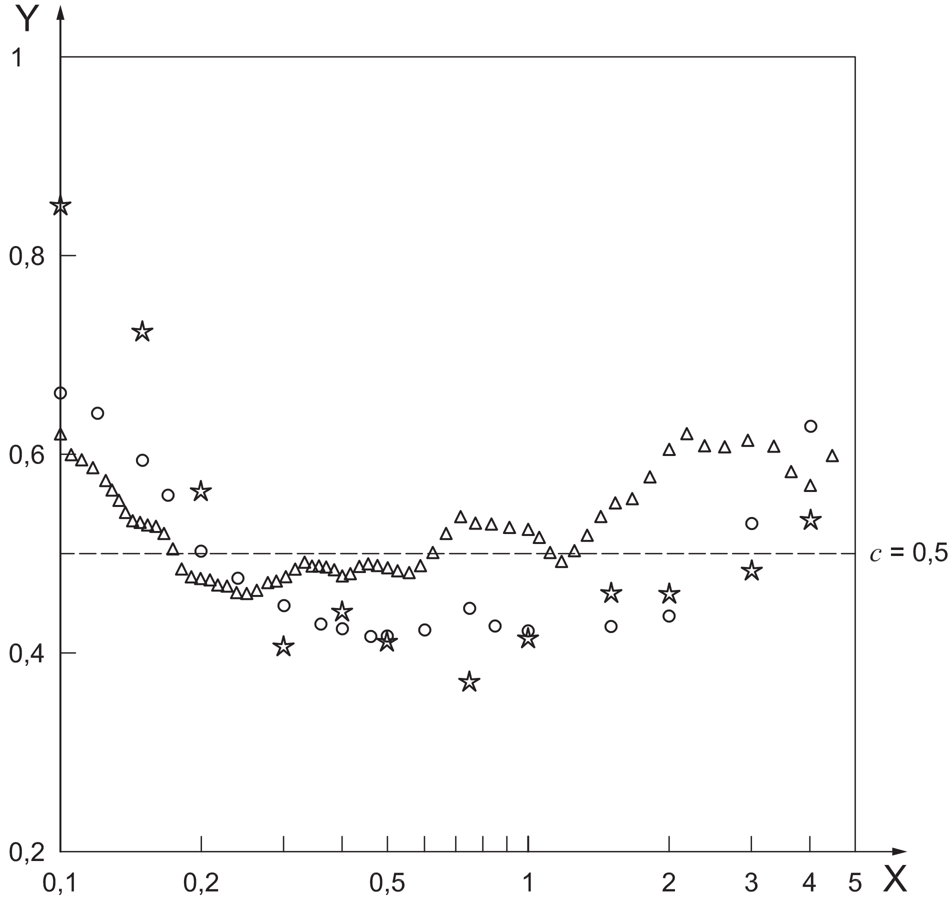
Annex F
(informative)

Vertical seismic intensity

Vertical influence factor, *c*, relates vertical and horizontal seismic design coefficients, K_V and K_H.

The ratio of the vertical and horizontal acceleration of response spectra obtained from the empirical attenuation formula range from approximately 0,4 to 0,7 in the period range of 0,1 to 5 s (see Figure F.1 and References [1], [2] and [3]).

It is recommended to set the vertical influence factor, *c* to 0,5.



Key

△	Ohno et al. (2001)	X	period(s)
☆	Campbell and Bozorgnia (2003)	Y	*V*/*H*
○	Abrahamson and Silva (1997)	*c*	vertical seismic coefficient

Figure F.1 — Vertical to horizontal ratio (*V*/*H*) of response spectra from empirical attenuation formula versus periodic component of earthquake

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