International Standard ISO 6258

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Nuclear power plants — Design against seismic hazards

Centrales nucléaires — Conception antisismique

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Foreword

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Draft International Standards adopted by the technical committees are circulated to the member bodies for approval before their acceptance as International Standards by the ISO Council. They are approved in accordance with ISO procedures requiring at least 75 % approval by the member bodies voting.

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0 Introduction

Earthquakes have the potential to cause serious damage to nuclear power plants and to jeopardize their safety. Therefore, at any nuclear power plant site the acceptability of ground motions shall be determined and the plant designed to withstand these motions.

It should be understood that the parameters of the design ground motions by themselves do not represent the overall antiseismic protection of the plants; the whole set of assumptions taken into account in the structural analysis, including soil, is equally important. Therefore, an unrealistic increase of the design ground motion parameters, or unrealistic conservative assumptions during the structural analysis process, do not necessarily represent a better protection of the plant and do not guarantee a more conservative design. The conservatism of the antiseismic protection of a nuclear power plant should be achieved by harmonizing the safety margins during the whole process — from the definition of the design ground motion parameters to the assessment of the allowable stresses in the structural elements.

The amplitude and nature of the earth motions selected depend on the position of the site and on the specified risk level that the plant is designed to achieve. The specification of the risk level and of the design basis earthquake (DBE, see 2.2.18) to be used is the responsibility of national regulatory authorities and is not dealt with in this International Standard.

In preparing this International Standard, account has been taken of existing national standards and codes of practice. The requirements of this International Standard are compatible with IAEA 50-C-S [1] and for some subjects1) in line with the approaches presented in IAEA 50-SG-S1 [2] and IAEA 50-SG-S2 [3]. However, for some other subjects2), the approaches are different from those followed in IAEA 50-SG-S1 [2] and IAEA 50-SG-S2 [3].

1 Scope and field of application

This International Standard specifies the requirements to be taken into consideration when designing a nuclear power plant in relation to seismic hazards.

It specifies the data required and the way in which these data should be used in order to determine the earth motions to be taken as the design basis ground motion for design purposes.

It does not specify, on the other hand, earthquake levels for other purposes, for example inspection level earthquakes or operating basis earthquakes, which in reality have no simple relationship to the DBE (see 2.2.18) and other seismic motions; thus the specification and use of these other motions shall be agreed with national regulatory authorities.

This International Standard specifies the way in which the proof of seismic design adequacy should be established and documented for the various parts of the plant (foundation material, buildings, systems and components); the instrumentation is also specified.

The methods to be used for both probabilistic and deterministic methods for the determination of the DBE are covered in this International Standard and either method is acceptable. The detailed requirements in this International Standard are applicable to sites in which the maximum potential ground motion is predicted to be equivalent to intensity MSK VII or above.

This International Standard is entirely applicable when the DBE is greater than or equal to intensity VII on the MSK scale. When the DBE is less than intensity VII on the MSK scale, the structural analysis (see clause 5 *et seq.*) could also be performed by using simpler rules which are outside the scope of this International Standard.

1) For example the seismotectonic approach.

2) For example, probabilistic approach, seismic analysis and absence of a lower level design basis earthquake (S1).

2 Definitions

2.1 Terms concerning geology and seismology

2.1.1 magnitude: The Briggsian logarithm of the maximum amplitude of motion, in micrometres, recorded by a standard seismograph at 100 km from the epicentre of the earthquake. Magnitude gives an approximate measure of the energy released as seismic waves.

NOTE — As several methods for determining magnitude exist, it is necessary to specify the way in which magnitude was determined.

The Richter magnitude originally defined by Richter (M_L) was the Briggsian logarithm of the maximum deviation, in micrometres, recorded by a standard seismograph at 100 km from the epicentre in California.

Symbol: M, with a subscript indicating the method of determination (M_b, body wave; M_L, Richter magnitude; M_S, surface wave; M_D, magnitude determined over a period).

2.1.2 seismic intensity: An expression, according to a conventionalized scale, of the extent of the felt or observed effects of an earthquake at a particular point.

A maximum degree of intensity corresponding to a given earthquake can be defined by evaluating the maximum effects ascribable to it, these latter generally being observed in the epicentral area. The symbol for this maximum intensity is I_0.

2.1.3 isoseismal (line): Line, characterized by a value of seismic intensity, which divides a zone where the intensity is equal or superior to this value from a zone where it is inferior.

2.1.4 zone of damage: Region where damage is observed, generally bounded by the isoseismal VII (MSK) (see the annex).

2.1.5 macroseismic epicentre: The centre of the region where the maximum seismic intensity is observed.

2.1.6 amplitude of seismic motion: A general expression used to specify the ground motion at a given point. It may correspond to the maximum value of one of the parameters characterizing the seismic motion: acceleration, velocity, displacement.

2.1.7 seismic source: The region within the Earth where the energy of an earthquake is released.

2.1.8 focus; hypocentre:

2.1.8.1 instrumental focus: The point that can be computed using the arrival times of seismic waves in different localities and which represents the first point of rupture of the rocks.

2.1.8.2 energetic focus: The centre of gravity of the volume within which the energy of the earthquake is released; for some applications, this volume is taken as a sphere.

2.1.9 instrumental epicentre: The point on the Earth's surface situated directly above the instrumental focus.

2.1.10 inactive fault: A fault showing no signs of recent geologic movement or of significant seismic activity.

2.1.11 active fault: A fault presenting any proper significant seismic activity or any potential of proper significant seismic activity, whether or not the existence of recent geologic movement related to it can be proven.

2.1.12 capable fault; fault capable of seismic activity: A fault which has significant potential for relative displacement at or near the ground surface.

2.1.13 surface faulting: The cracks or offsets on the ground surface caused by the movement of a fault at or beneath the ground surface.

2.1.14 free field ground motion: The ground motion resulting from an earthquake in the absence of discontinuities caused by construction features.

2.1.15 macroearthquake: An earthquake sufficiently intense for it to be felt by man.

2.1.16 microearthquake: An earthquake detected by instrumental means only.

2.1.17 microtremor: The virtually continuous, extremely low amplitude ground vibrations from natural or man-made sources such as wind, waves and industrial activity.

2.1.18 earthquake prone structure: Geologic structures likely to bring about earthquakes.

2.1.19 seismotectonic province: A geographic region typified by a similarity in geologic structures and in the characteristics of its earthquakes.

2.2 Terms concerning testing and interpretation methods for seismic evaluation of structures, systems and components

2.2.1 time history motion: A representation as a function of time of the vibratory motion of supports expressed in terms of acceleration, velocity or displacement.

2.2.2 ground time history motion: A representation as a function of time of one or more earthquake motions in the free field of the ground at the foundation level of the buildings or structures or at other defined free field ground locations.

2.2.3 design ground time history motion: A representation as a function of time in which time-scales or amplitudes have been suitably modified to take into account the variability and uncertainty of input earthquake motions.

2.2.4 floor time history motion: A representation as a function of time of one or more earthquake motions at a particular building or structure elevation.

2.2.5 design floor time history motion: A representation as a function of time in which time-scales or amplitudes have been suitably modified to take into account the variability and uncertainty in input earthquake motion and in building and foundation characteristics.

2.2.6 response spectrum: A plot of the maximum response of a family of oscillators each having a single degree of freedom with fixed viscous damping, as a function of natural frequencies of these oscillators when subjected to vibratory motion input at their supports.

2.2.7 ground response spectrum: A response spectrum determined from vibratory ground motion input in the free field of the ground at the foundation level of buildings or structures or at other defined free field ground locations.

2.2.8 design ground response spectrum: A response spectrum obtained by modifying one or more ground response spectra in order to take into account the variability and uncertainty of input earthquake motions.

2.2.9 floor response spectrum: The response spectrum of the motion at a particular level of a structure for a given earthquake.

2.2.10 design floor response spectrum: The response spectrum defined at a particular building elevation and obtained by modifying one or more floor response spectra in order to take into account the variability and uncertainty of input earthquake motion and of building and foundation characteristics.

2.2.11 static analysis: An analysis carried out using a static force or displacement which represents the earthquake motion acting on an item without explicit consideration of the dynamic characteristics or the dynamic nature of the input motion.

2.2.12 dynamic analysis: An analysis carried out using either a static or dynamic force or displacement which represents the earthquake motion acting on an item with explicit consideration of the dynamic characteristics and the dynamic nature of the input motion.

2.2.13 damping: A progressive diminution of the response motion amplitude resulting from energy loss in structural elements due to friction and hysteretic losses within the material as well as small non-linearities such as cracking, joint slippage and other changes in structural element stiffness during the response to earthquake input motions. Structural damping as used in modal analysis is normally expressed as a percentage of the viscous critical damping.

2.2.14 liquefaction: The significant loss of strength and rigidity of saturated, cohesionless soils due to vibratory ground motion.

2.2.15 soil-structure interaction: Change of the foundation motion in relation to the free field motion, due to the presence of foundations and the building.

2.2.16 rigid range: The high frequency range of a response spectrum for which the amplified response acceleration does not exceed the input maximum acceleration (zero period acceleration) by more than 10 %.

2.2.17 rigid item: An item which has all its natural frequencies in the rigid range of the applicable support point response spectra considering all three input directions.

2.2.18 design basis earthquake; DBE: The set of free field ground motions derived for design purpose.

2.3 Terms concerning seismic instrumentation

2.3.1 acceleration pick-up: An instrument which measures acceleration and transforms it into a signal which can be transmitted.

2.3.2 acceleration measuring device: A device which measures and records the absolute acceleration as a function of time. The device consists essentially of an acceleration pick-up, recorder and seismic trigger.

2.3.3 triaxial measuring instrument: An instrument which measures linear acceleration in three orthogonal directions, one of which is vertical.

2.3.4 seismic trigger: A seismic detector which causes the detection and recording of the measured values of acceleration to commence and cease.

2.3.5 seismic detector: A measuring device which emits signals when a measured value of acceleration exceeds a preset level.

2.3.6 recorder: An instrument capable of making a permanent record of absolute acceleration as a function of time after actuation by a seismic trigger.

3 Collection and presentation of geologic and seismic information

3.1 Information and investigation of earthquakes

Earthquake data of two types can be collected:

a) historical data;

b) instrumental data.

3.1.1 Historical data

A major part of the information base for determining DBEs is a complete set of historical relevant earthquake data. Therefore it is necessary to collect all available historical records, extending as far back in time as possible. Most such records will naturally be of a descriptive nature, for example the number of houses damaged or destroyed, or the behaviour of the population. But from such information a measure of the intensity scale value of each earthquake in seismic intensity scale values may be determined.

A comparison between various intensity scales is given in the annex. It shall be clearly stated what intensity scale is used for the description of the earthquake.

To evaluate effective ground motions of the site area, information has to be collected for all historical earthquakes within a region which includes the seismotectonic province of the site. This requires consideration of an area which depends upon the characteristics of the region. This area shall be big enough to allow the collection and consideration of all the geologic or geophysical data in relation to the seismicity of the site.

The information to be obtained, subject to availability, is as follows:

- the intensity scale value at the epicentre or maximum intensity scale value, as appropriate;
- the intensity at the site area;
- isoseismal maps related also to the local geologic conditions;
- the magnitude;
- the locations of the epicentre and the hypocentre.

In the absence of instrumental data, intensity scale values, building damage and ground effects data, in conjunction with a knowledge of local faults, are used to the maximum extent possible to determine the epicentre and magnitude of each historical earthquake.

3.1.2 Instrumental and reported data

It is necessary to collect all available earthquake information derived from instrumental recordings in the region. The following information to be obtained, subject to availability, is

- the location of the epicentre and the hypocentre;
- the origin time;
- the magnitude;
- the aftershock zone;
- the maximum reported intensity;
- isoseismal maps;
- the ground motions and intensity in the site area;
- the earthquake mechanisms and other available information that may be helpful in evaluating seismotectonics.

When the determination of earthquake level at the site relies upon the definition and localization of faults, it may prove necessary in some seismotectonic areas of unusual geologic complexity, such as areas of complex neotectonics or where unreliable seismicity data exist, to supplement the available historical and instrumental data on earthquakes by establishing a network of sensitive seismographs having microearthquake recording capability within a few tens of kilometres of the site. Earthquakes recorded within and near the network shall be carefully located for use in seismotectonic studies of the region and to determine the appropriate design basis ground motions.

NOTE — Microearthquakes cannot be used as such to determine design basis ground motion.

Strong motion recordings are available for some parts of the world. These recordings shall be collected and used in developing seismic wave attenuation functions appropriate for use in the region and in developing the design response spectra for the proposed nuclear power plant. Where there is a reasonable expectation of obtaining recordings not otherwise available, strong motion accelerographs may be installed within the site area.

3.2 Geologic information and investigations

3.2.1 Regional geologic data

The main purpose of the regional geologic information and investigations is to provide knowledge of the general geologic setting and tectonic framework of the region needed to interpret the earthquake data to define seismotectonic provinces. The information also serves to identify the types of geologic hazards that exist in the region and to study them in relation to the seismic and geologic investigations of the site area and site vicinity, described in 3.2.1.1 to 3.2.1.5. The following regional scale information shall be obtained.

3.2.1.1 Characteristics of the ground

Where geologic maps exist, special attention shall be given to identifying lithologic units — crystalline, volcanic, sedimentary, alluvium, etc.

3.2.1.2 Stratigraphy

Superposition and age of strata, their lateral extent, and possibly their depth, thickness and relationship to one another shall be investigated.

3.2.1.3 Regional tectonics

Faults are given special attention. Topography and geomorphology may be useful for showing possible recent ground displacements. Consideration shall also be given to the tectonic style of the region, i.e. horizontal continuity of strata, folding and faulting as well as tectonic history, particularly the age of folding and faulting.

3.2.1.4 Characteristics of tectonic features

Style and type of faulting in the region and large faults associated with seismotectonic provinces shall be described. The length, depth, strike, and dip of faults, structural relationships among faults, and their age and history of movements shall be studied for information on the possible presence of seismically active or capable faults. Particular attention shall be given to the evaluation of Quaternary deposits and detailed neotectonic studies shall be carried out.

3.2.1.5 Subsurface characteristics

Where there is no surface manifestation of base rock and where appropriate data exist, a structural map of the base rock surface (hypogeologic map) is prepared, by using information available from regional geophysical investigations, such as seismic, gravimetric, and magnetic prospecting, to obtain the necessary subsurface details. This map may permit a determination of possible relationships between historical earthquake activity and deep tectonic structures which may lack direct expression at the surface.

Regional geologic data are usually obtained from published sources. An extensive use of remote sensing data, such as satellite photographs, sidescan radar, aerial photographs, aeromagnetics and gravimetrics, is necessary. Where published information is insufficient, it is permissible to make a demonstrably conservative assessment of the deep structure characteristics or it is necessary to perform field investigations, such as boring, trenching and the use of seismic reflection and refraction methods particularly in some areas of unusual geologic complexity, to supplement the published information on regional geology and to aid interpretation of the remote sensing data.

3.2.2 Site area and site vicinity geologic data

It is necessary to conduct a special detailed investigation of the geology of the site area and the site vicinity to identify tectonic structures which might localize earthquakes in the site area, to establish a basis for determining the age of movement of faults that may be present, to identify geologic hazards such as solutioning or subsidence that may affect the safety of the nuclear power plant, and to determine seismic energy transmission characteristics of the site area.

Local geologic and physical conditions, such as properties of soils, can influence ground motion spectra and can consequently modify the observed effects. The following investigations are necessary.

3.2.2.1 A determination of the geologic and physical characteristics, such as thickness, depth, and properties, of strata in the site area.

3.2.2.2 An assessment of the local tectonics, including the presence of faults on or beneath the surface of the site vicinity and their geometry, such as length, inclination and where possible, depth. The structural relationship of local faults to regional faults, particularly to active or capable faults and correlation with historical earthquakes shall be assessed.

It is also necessary to carry out field and laboratory investigations as described in 3.2.2.2.1 to 3.2.2.2.4. Regarding the site area, the investigations shall be conducted with due consideration to the proposed design of the nuclear power plant.

3.2.2.2.1 Bearing strata studies

Local bearing strata investigations and laboratory tests shall be conducted to determine the depth and properties of the different layers namely, Poisson ratio, Young modulus, shear modulus and density.

3.2.2.2.2 Borings

For moderately shallow strata the configuration of the bearing strata and of the bedrock[1] and in some cases of the base rock[2] can be determined by boring. As borings are made, samples shall be taken at different depths for soil and rock property tests.

3.2.2.2.3 Test excavations

When bearing strata and/or bedrock properties and structure cannot be clearly determined by either seismic methods or boring, a test excavation, trench, shaft or tunnel shall be made. The practicability of such excavations is dependent upon the characteristics of the bearing strata and the depth of the bedrock.

3.2.2.2.4 Vibration testing of models

The natural vibration frequencies of buildings, structures and equipment are influenced by the properties of the bearing strata under the foundation of the facility. Therefore a knowledge of the range of effective values of the stiffness properties of the bearing strata in evaluating structural response during earthquake is required.

4 Methods for deriving design basis ground motions

Design basis earthquakes (DBEs) may be established either by a deterministic analysis including seismotectonic and geologic techniques or by a probabilistic analysis.

The use of probabilistic analysis in general is practical only in regions where there exists a reliable data base of relatively long duration (several hundred years). In most cases, this corresponds to DBEs equal to or less than VII MSK intensity.

4.1 Deterministic analysis

A deterministic approach consists of the consideration of a logical sequence of events starting from an assumed initiating event; to each element of the sequence is assigned a unique description which includes the conservatism factors, i.e. pessimism or prudence factors, which can "reasonably" be associated to the elements.

1) **bedrock :** The first hard geologic formation to be encountered from the Earth's surface downwards, the mechanical properties of which contrast considerably with those of the overlying deposits.

2) **base rock :** The well consolidated geologic formation that may be considered on a regional scale as being homogeneous as far as the transmission of seismic waves is concerned.

Seismotectonic techniques consist of:

a) identifying the region, the seismically active structures and their maximum earthquake potential, the seismotectonic provinces and their maximum earthquake potential;

b) evaluating the design basis ground motion produced at the site by the occurrence of this maximum earthquake potential at the nearest point to the site on the seismically active structure or at the borders of the seismotectonic provinces. If the seismically active structure is close to the site the physical dimension of the source may, if possible, be taken into account.

4.1.1 Identification of seismotectonic provinces

The purpose of seismotectonic studies is to define geographic regions which have similar earthquake potential.

The seismotectonic provinces will be the areas identified by similarity of geologic structures and of the characteristics of the seismicity.

The seismic and geologic data previously discussed shall be developed into a coherent well-documented description of the regional tectonic characteristics, listing details of tectonic structure, tectonic history and present-day earthquake activity which distinguish various seismotectonic provinces. For example a seismotectonic province boundary may separate areas which show strongly contrasting tectonic framework, areas which have greatly different late Tertiary and Holocene tectonic histories.

A number of precautions shall be observed in defining the boundaries of seismotectonic provinces. All the structures in a contiguous area which have the same seismotectonic or geologic style shall be included in the same province. Each tectonic structure relevant to the seismicity shall in its entirety lie within the same seismotectonic province. When there is doubt that one structure is a continuation of another, then both shall be considered as one structure and consequently included in the same province.

In some areas of the world, the boundaries of lithospheric plates present a special problem. For example where the mode of plate interaction is subduction, the lower (subducting) plate is generally considered a separate seismotectonic province from the upper (crustal) plate. It has been demonstrated that different sectors of plates have different potential for maximum earthquakes. These sectors may also be considered seismotectonic provinces.

Significant differences in rates of seismicity may suggest different tectonic conditions that can be used in defining seismotectonic provinces. The length of time during which historical data are available should be long enough to demonstrate that conclusions based on these data are reasonable. However, significant differences in hypocentre depth (for example 10 to 30 km versus 200 to 400 km) may alone justify the differentiation.

Alternative interpretations of the seismotectonics of the region given in available literature sources shall be discussed. When alternative interpretations are judged to explain the observed seismic and geologic data equally well the interpretation which results in the more conservative assessment of the potential ground motion at the site area shall be used.

4.1.2 Association of earthquakes with seismically active structures and seismotectonic provinces

The fundamental data needed for associating earthquakes with tectonic structure and seismotectonic province shall be collected and properly prepared.

4.1.2.1 Association of earthquakes with seismically active structures

Whenever an earthquake epicentre or a group of earthquake epicentres can be reasonably associated with a tectonic structure, the rationale for the association shall be given together with consideration of the characteristics of the structure, its geographic extent and its structural relationship to the regional tectonic framework. This assessment shall include consideration of methods used to determine the earthquake epicentres and an estimate of the errors in their locations. A detailed comparison of these tectonic structures with others in the same seismotectonic province with regard to factors such as age of origin, sense of movement and history of movement shall be made. Other available seismologic information, such as source mechanisms, stress environments and aftershock distributions, shall also be evaluated. Tectonic structures with which significant seismicity is correlated shall be considered seismically active.

4.1.2.2 Identification of maximum earthquake potential associated with seismically active structures

For seismically active structures, which are pertinent to determining the earthquake potential for the site, the maximum earthquake potential which can reasonably be expected in association with these structures shall be determined.

The geologic and seismologic data previously discussed and relating to the dimensions of the structure, amount and direction of displacement, maximum historical earthquake and earthquake frequency shall be used in this determination. The dimensions of fault rupture in an earthquake can often be determined from the distribution of aftershocks. In the absence of suitable local data, the maximum earthquake potential for a tectonic structure can be estimated based on methods[4] which relate the dimensions (length and vertical depth, displacement) of the fault rupture with the magnitude. However, to use these relationships the fraction of the total length of a structure which can move in a single earthquake has to be known. A value of about one half of the total fault length has been assumed in certain regions of the world.

In applying this methodology, it has to be remembered that earthquake magnitude is a function of both the source dimensions and the stress drop. Stress drop usually will not be known, but reasonable upper bound values based on available published studies may be used.

An alternative method[5] for estimating the maximum earthquake potential for a seismically active structure or for a seismotectonic province has been described. This method is based mainly on the statistical analysis of earthquake data associated with the structure or province.

When sufficient information about the seismicity and geologic history of the movement of a fault or fault zone is available, a method exists for evaluating the maximum magnitude potential from total area and maximum slip of the fault in the Quarternary period[6]. In this method the usual hypothesis is made for the statistical distribution of the number of earthquakes as a function of their magnitude. The slip is then correlated with seismic moment and therefore with the magnitude. Finally, from the total slip the maximum magnitude may be estimated.

4.1.3 Earthquake not associated with seismically active structures

The maximum earthquake potential not associated with tectonic structures which can be reasonably expected with a very low probability in the tectonic province shall be evaluated on the basis of historical data and on the seismotectonic characteristics of the region. Comparison with similar regions where very extensive historical data exist may be useful, but considerable judgement is needed for this evaluation.

4.2 Probabilistic analysis

The objective of the probabilistic approach is to determine the level and form of ground motion that has an acceptably low probability of being exceeded during the operating life of the plant. This requires a basic data sample of the intensity of ground motion experienced in the region from historical earthquakes and also an acceptable probabilistic model. A number of mathematical models for determining earthquake probabilities have been proposed. In general, all of the models require conditions on the data sample that cannot be fully demonstrated for most areas of the world. If, however, the sample of earthquakes experienced in a region is accepted as being adequate (i.e. meeting the requirements of the model), the calculations are relatively simple.

In calculations of earthquake probability, the confidence level on the estimates depends strongly on the time span of the available data sample and to a lesser degree on the completeness of the data set. Thus, to make estimates of ground motion intensities which have an acceptably high confidence level, the data set has to cover the longest possible time span. This usually requires the use of pre-instrumental data which are known to be incompletely and inaccurately reported. An assessment of the completeness and accuracy of the available data set shall be made and taken into account in making the probability calculations. One simple but useful method[7] for assessing the completeness of a data set is described in the literature.

When this method is used to make the probability calculations, it may be desirable to augment the available data set with low ground motion intensities measured in the region. This may be important in areas with limited historical records. Part of the required data can be obtained by operating an earthquake recording network such as described in 3.1.2. However, it shall be stressed that a probabilistic law cannot be extrapolated on the basis of these data alone. In using earthquake data, caution has to be exercised in taking into account poor data on epicentre locations and the tendency of earthquakes to cluster in time.

4.3 Determination of site ground motion

The evaluation may be performed as follows.

a) For each seismically active structure, the maximum earthquake potential is considered to be moved to the appropriate location on the structure closest to the site area. For earthquakes near to the site the physical dimension of the source may be taken into account.

b) The maximum earthquake potential in the seismotectonic province of the site that cannot be associated with seismically active structures is assumed to occur at a certain distance from the site. In certain countries this distance may be accepted by the regulatory body on the basis of studies and investigations which ensure that within this distance there are no seismically active structures and, therefore, that the related probability of earthquakes occurring therein is very low. This distance may be in the range of a few to tens of kilometres and depends on the focal depth of the earthquakes of the province. In evaluating it the physical dimension of the source shall also be considered[8].

c) Maximum earthquake potential in seismotectonic provinces adjacent to the province of the site is assumed to occur at the locations on the province boundaries nearest to the site.

d) An appropriate attenuation function is used to determine the ground motion which these earthquakes would cause at the site.

For the probabilistic method, on the basis also of seismic considerations, the evaluation may be performed as follows.

a) For each seismic source that can affect the site, a mathematical model called "earthquake-generating source" is selected (models of point sources, linear sources and sources distributed superficially are available)[9 to 11].

b) A probability density function is derived for a range of earthquake sizes (usually in terms of epicentral intensity or magnitude) for each earthquake-generating source.

c) The probability of not exceeding a selected level of ground motion at a site (in terms of acceleration, velocity, displacement or intensity) is determined for each earthquake-generating source using an attenuation law.

d) The level of ground motion which will not be exceeded at the site with a selected total probability is determined by combining the contributions from all earthquake-generating sources.

4.4 Induced seismicity

Special attention shall be given to potential induced seismicity particularly that from large dams or reservoirs, and extensive fluid injection into or extraction from the ground. Modification of the stress conditions in rocks and geologic structures due to these conditions may produce seismic activity. Earthquakes resulting from such phenomena usually have shallow foci and are located in the vicinity of the reservoir or of the injection or extraction area. In the case of reservoir loading, some earthquakes have had relatively high magnitude (close to magnitude 6). Usually the larger earthquakes have occurred in association with deep reservoirs, but no simple rule regarding a fixed reservoir depth above which induced seismicity occurs can be applied everywhere. Earthquakes associated with fluid injection and extraction activity are generally smaller in magnitude than those caused by reservoir loading. Seismic networks in areas where the potential for such a problem exists may provide useful information for assessing the significance of induced seismicity.

4.5 Free field ground motion characteristics [12]

The ground motion characteristics may be characterized either

a) by free field response spectra for various damping coefficients, or

b) by time history or histories,

both characteristics preferably being specified at free field surface ground level; however, other locations are acceptable as long as these are precisely described.

4.5.1 Spectral shape appropriate to the site area

The spectral shape of the ground motion is determined according to the relative influences of the source spectral characteristics for earthquakes in the region and the attenuation characteristics of geologic materials which transmit the seismic waves from the hypocentres to the site area. In strata above the base rock the seismic waves in the free field are amplified or attenuated according to the frequency transfer characteristics of the strata and the strain level of the vibration. Therefore, the response spectra of accelerograms of several different earthquakes obtained at the same site area at the surface and on base rock have different frequency characteristics.

The following alternative methods are acceptable for obtaining the design ground response spectra.

4.5.1.1 Site specific response spectrum

Wherever possible, response spectra shall be developed from strong motion time histories recorded at the site. However, for the majority of sites an adequate sample of strong motion time histories cannot be obtained in a reasonable number of years. Therefore, response spectra obtained at places having similar seismic, geologic and soil characteristics may be useful in establishing the response spectrum specific to the site. An evaluation shall be performed to determine whether response spectra obtained at these other places appropriately reflect the site area response energy absorption characteristics, as well as the source mechanisms generating earthquakes affecting the site. Due consideration shall be paid to the different frequency characteristics associated with different strain levels of the ground motions.

In conclusion the method may be summarized as follows:

a) collection of strong motion accelerograms from the site or more probably from similar sites;

b) normalization of these accelerograms to the zero period ground acceleration defined for the site;

c) evaluation of the response spectrum for each accelerogram using various damping factors;

d) modification of the shape of the normalized spectrum taking into account the strain level in the subsurface by the ground motion and other uncertainties.

4.5.1.2 Standard response spectrum

An alternative method is to use a standard response spectrum with a relatively smooth shape which is generalized for application and which has been obtained from many response spectra derived from records of past earthquakes. However, in certain parts of the world higher values in some frequency fields have been observed so that some modifications of this spectrum may be required. This standard response spectrum is scaled up to the value of ground acceleration, velocity displacement, etc. If this approach is followed, a specific study or a justification should be carried out in order to verify the applicability of the standard response spectrum to the specific site.

4.5.2 Radiation damping

It is important to select realistic values of soil radiation damping since an overly conservative selection of soil damping values can distort the frequency response of a free field system.

4.5.3 Time histories of earthquake ground motion

Time histories can be developed for the site area. The time histories take account of the maximum velocity (or alternatively, maximum acceleration or spectral intensity) and the duration of "deterministic intensity function" which represents the envelope of the ground motion intensity time history.

For design, the time histories of vibratory ground motion may be based on:

a) strong motion records obtained in the site vicinity from past earthquakes, or adequate modifications thereto, such as adjusting the peak acceleration, applying an appropriate frequency filter, combinations of records, etc.;

b) strong motion records obtained at places which have similar seismic, geologic, and soil characteristics. In some cases these records may need appropriate modifications, such as applying wave propagation theory to modify frequency characteristics;

c) calculation models simulating earthquake ground motion, such as generating random time series by computer and filtering out to obtain the specific frequency characteristics.

Regardless of the procedure used, the design time histories and the design response spectra shall be compatible. This implies that it is necessary to choose a sufficient number of time histories with pertinent characteristics so that the envelope of their response spectra does not lie significantly below the smoothed design response spectrum throughout the entire frequency range of interest.

4.5.4 Ratio of motion in vertical and horizontal directions

The design response ground spectra and design time histories for the vertical direction shall be evaluated using the same procedure as for the horizontal direction. Appropriate vertical time histories may be the basis for this evaluation. If no specific information is available on the peak acceleration of vertical ground motion at the site, the ratio of peak acceleration in the vertical direction to the horizontal should not be assumed to be less than 1/2.

5 Classification of plant equipment and building

Seismic classification of plant items depends on the specific safety requirements of the plant and shall be in agreement with the national regulatory authority requirements, since such seismic classification lies beyond the scope of this International Standard.

6 Design methods

6.1 General

The verification of the adequacy of the seismic design of nuclear safety class structures and components is divided into a number of categories and subcategories as shown in the figure. In general, either test or experimental methods or analytical techniques employing mathematical models of the system to be evaluated are used. Combined methods can also be used.

Verification by test is more typically used when the potential failure modes either in terms of structural or functional adequacy cannot be easily identified in terms of stress-strain or deformation outputs from an analytical solution. Seismic verification test procedures are discussed in general in IEEE Std 344[13] and will not be discussed in detail in this International Standard. However, since the referenced procedures are written specifically for electrical components, care should be taken in applying the requirements of this recommended practice directly to mechanical components and structures.

The analytical methods used to verify seismic design adequacy fall into two main categories: dynamic analysis and static analysis, as shown in the figure. The dynamic analysis category is further divided into four subcategories, consisting of seismic input, damping, model and analytical techniques. Static analysis may be used extensively in seismic design verification, particularly for relatively low seismic intensity sites.

In static analysis, dynamic characteristics of components and structures are not determined explicitly. In such cases, an inertia acceleration taken as some coefficient times gravity is selected as representative of seismic acceleration and applied statically to the mass distribution of the component to determine seismic inertia forces in the component. This method is used on conventional building structures and is well-documented in national building codes. In nuclear power plant design, the maximum value of the seismic static coefficient is taken as 1,5 times the peak of the applicable response spectrum. Values less than 1,5 times the peak of the applicable response spectrum may be used, provided valid analytical justifications are presented.

In dynamic analysis the dynamic characteristics of the components are normally expressed in terms of frequency and mode shape and are used to determine the characteristic response of the component. The dynamic characteristics of frequency and mode shape can be used with the applicable response spectra or time histories to define seismic forces. For some systems, particularly non-linear system components, direct integration of the equations of motion is utilized to determine seismic response.

6.2 Civil engineering structures

6.2.1 Safety related buildings

6.2.1.1 Seismic input

Two horizontal components or a resultant ground motion in the form of response spectra or time histories of motion constitute inputs for the dynamic analysis. The vertical component may be considered in the same manner as the horizontal motions or a constant static value for the vertical component may be assumed if no foundation-structure interaction is present and the structure is sufficiently rigid vertically.

6.2.1.2 Modelling of structures

The model of the structure shall take foundation-structure interaction into account when the foundation media has a shear wave velocity less than 1 200 m/s. To this end, consideration is given to rigid body translational, rocking and torsional modes, either individually or coupled as appropriate.

Subsystems (which may include inertia effects of contained liquids) located in the building should also be taken into account in the model of the building structures, depending on the nature of the subsystem. The following procedures may be used:

a) for rigid items rigidly connected to the structure, or subsystems that meet the decoupling criteria presented in references [14 and 15] (see 6.4), it is necessary to include the mass of the subsystem in the mass of the structure;

b) for parts connected to the structure the fundamental frequency of which is less than one quarter of that of the structure, the subsystem may be neglected in the analysis of the structure;

c) for other subsystems, it is necessary to include the subsystem in the analysis of the main structural model.

6.2.1.3 Analytical methods

A dynamic analysis is performed in order to obtain seismic response motions or loads. Both time history and response spectrum inputs are acceptable.

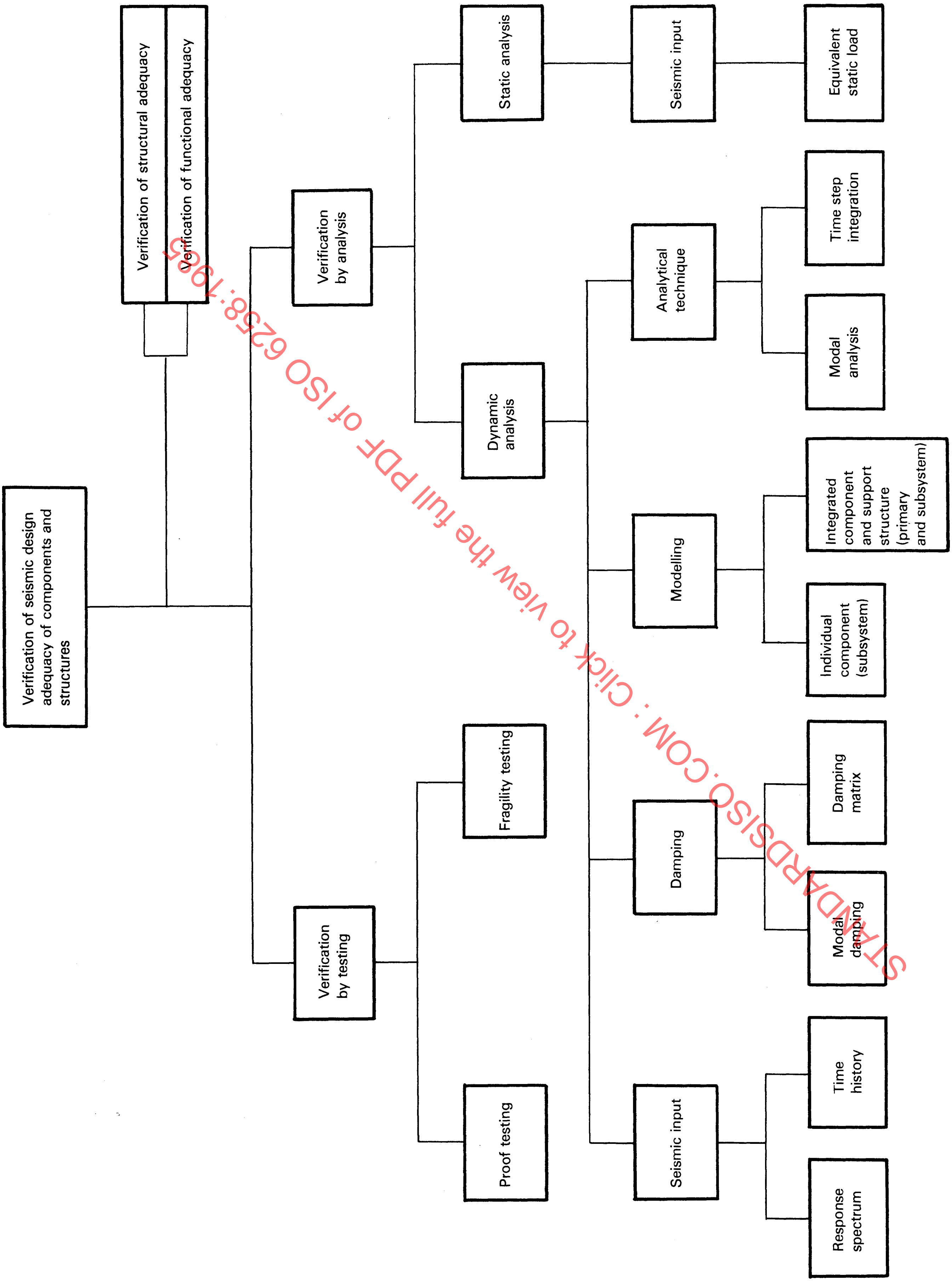

Figure — Summary of seismic design verification techniques and input parameters

6.2.1.4 Lateral earth pressures

It is necessary to evaluate lateral earth pressures induced on underground portions of structures by ground motion. This evaluation shall be made by using applicable procedures such as those outlined in clause 10. Usually these procedures provide values of the dynamic pressure increment induced by the earthquake; this increment is then added to the applicable static lateral earth pressure. Simplified procedures[16, 17] may also be used to calculate this dynamic pressure increment. These simplified procedures, however, do not take into account the effect of adjacent structures on this pressure increment; this effect may be significant in some cases depending on the plant layout.

6.2.1.5 Other considerations

A gap in structural joints between adjacent structural parts or between adjacent buildings shall be provided to accommodate the maximum out-of-phase displacement between parts or adjacent buildings without interaction.

For the layout and connections of the various structural elements, it is advisable to take account of the rules of the art and of the derived construction arrangements acknowledged as imparting optimum resistance capability to the structures in the case of high dynamic stresses[18].

6.2.2 Buried long structures

The following earthquake effects on buried pipes and ducts, well casings, etc., should be considered:

a) deformations imposed by surrounding soil during the earthquake;

b) differential displacements or loads at end connections to buildings or other structures[19, 20].

6.2.3 Foundation and earth structures

6.2.3.1 General recommendation

The seismic performance and stability of all soil and rock supporting the foundations of the nuclear power plant (NPP) and all earth structures (such as slopes, both natural and man-made, embankments, dams) as they affect the safety of the NPP should be evaluated.

6.2.3.2 Liquefaction

Liquefaction potential of saturated granular soil layers during the design basis vibratory ground motion should be evaluated for the DBE (see clause 10).

6.2.3.3 Settlement and soil-bearing capacity

Potential settlement (especially differential settlement) caused by seismic shaking may be of significance particularly in the case of non-uniform soil conditions; in such cases these settlements should be evaluated. The potential for a bearing capacity failure during or following the earthquake shall be investigated.

6.3 Mechanical and electrical components

6.3.1 Seismic input

For components not included in the model for the supporting structure, the horizontal input for analysis is the floor response motion, expressed either as design floor time histories or as design floor response spectra. A floor motion may be used in the vertical direction.

6.3.1.1 Design floor response spectrum

A floor response spectrum can be obtained on the basis of structure response to design basis ground motion in the form of an input ground time history. When developing a design floor response, spectrum uncertainty in the calculation of foundation and building mass and stiffness properties should be taken into account; when the design floor response spectrum is developed from a time history or histories, this can be achieved by broadening the peak by at least ± 10 %.

Simplified methods may also be used to calculate design floor response spectra, such as the decayed sinusoidal ground motion methods[21, 22], the continuous beating sine wave method[23, 24] or stochastic methods[25, 26].

6.3.1.2 Design floor time histories

Design floor time histories may be obtained based on the structural response to design basis time history ground motion when the response spectra developed from the design basis time history ground motion essentially envelops the design ground response spectra.

The analysis of mechanical and electrical components using design floor response spectrum should take into account uncertainties in input, materials and structural characteristics by broadening the peak of the response spectrum. The analysis using floor motion time history rather than response spectrum should take into account similar uncertainties by altering the time scale used to define the time history motion[14, 27].

Alternatively, design floor time histories may be obtained by developing time history motions which will develop response spectra that essentially envelop the design floor response spectrum. In this instance altering the time scale is not required.

6.3.2 Modelling piping systems

Methods commonly used for the modelling and analysis of piping systems include[14]:

a) the equivalent static load method;

b) the finite element-response spectrum method.

Evaluation of the flexibility or stiffness of bends, tees, variable spring hangers, hydraulic and mechanical snubbers is usually necessary. Constant spring hangers have usually no stiffness effect, but they behave as a constant external load on the system through their travel range. The stiffness of other support elements should also be considered unless it can be demonstrated that consideration of their stiffness characteristics does not significantly alter the dynamic response characteristics of the piping system. All the additional masses, such as valves, pumps, snubbers, liquid inside the pipe, thermal insulation and effective mass of mechanical snubbers, including any effect of eccentricity, should be considered. Specific modelling techniques are presented in 6.4.

6.3.3 Mechanical equipment and electrical distribution systems

Modelling and methods of analysis such as those discussed in 6.3.2 and 6.4, as applicable, can be used.

6.3.4 Instrumentation and electrical equipment

The seismic qualification of such items should be accomplished by testing, analysis, or a combination of both. The functional capability of these items should be checked by shaking tests (see clause 9).

It should be verified that the acceleration or velocity at the point where the item is installed is lower than the input used for testing with an appropriate margin (see clause 9).

6.4 General modelling techniques

6.4.1 General

A nuclear power plant is a complex system and a single complete model of the entire plant would be too cumbersome. Thus the first step in the analysis of a NPP is to divide up the structure into main systems and subsystems.

Major structures that are considered in conjunction with foundation media in forming a soil-structure interaction model constitute the main systems. Other structures, systems, and components constitute the subsystems.

6.4.2 Decoupling criteria

Decoupling criteria are obtained by putting some limits on the relative mass ratio between the subsystems and the supporting main system, limits which are more severe when there is a possibility of resonance between the subsystem and the main system[14].

If the decoupling criteria are not satisfied, a suitable model of the subsystem should be included in the main system model. For a subsystem having all its resonant frequencies in the rigid range (the flexibility of the support being taken into account), only its mass needs to be included in the main system model.

6.4.3 Modelling of systems or subsystems

The stiffness, mass and damping characteristics of the structural systems should be adequately incorporated in the analytical models.

The commonly used modelling methods are:

a) the lumped mass method;

b) the finite element method;

c) the transfer matrix method.

6.4.3.1 Lumped mass method

The lumped mass method is the most commonly used for the main systems. It consists of lumping the masses (and inertia moments, if necessary) at some adequately chosen points (floor levels in a building, for example) and determining the stiffness (or flexibility) coefficients by a static study of all the single movements corresponding to the selected degrees of freedom. This method is described in various publications[19, 28 to 30].

6.4.3.2 Finite element method

This method is described in detail in many publications[31, 32]. Various types of elements can be used: beam elements (for frames, pipes, or long slender cylindrical structures); axisymmetrical shell elements, which can account for non-axisymmetrical loadings or displacements by a Fourier representation on the azimuthal variable (for containment buildings, pressure vessels or other axisymmetrical structures); shell and plate elements (for complex or unsymmetric shell structures); three-dimensional elements (for thick wall structures).

6.4.3.3 Transfer matrix method

The basis of this method is that for some structures (pipes, axisymmetrical shells and, more generally, monodimensionally modelled simple structures), it is possible to calculate analytically for each frequency the transfer matrix between a vector state (forces, moments, displacements, and rotations) at a point and the vector state at the next point. Then, by successive matrix multiplication, it is possible to obtain the frequency dependent matrix of the whole system, including boundary conditions. This allows the determination of the natural frequencies, mode shapes or response to sinusoidal inputs, or, by a Fourier transform, the response to more general inputs[25, 33].

6.4.3.4 Application to particular components

Modelling of structures and components is typically divided into five categories, defined in 6.4.3.4.1 to 6.4.3.4.5.

6.4.3.4.1 Rigid body model

For those items where the item itself is in the rigid range, the model is typically represented by springs or stiffness or flexibility matrices. Response of the item then would be by rocking or translational modes of vibration at support points. Typical valve, pumps, motors, fans and some heat exchangers fall into this category.

6.4.3.4.2 Single mass model

For these items the total mass is assumed to be lumped at a single point with the composite stiffness restraining the mass represented as a single element. More than one degree of freedom may be permitted. In general, this type of modelling is considered as an alternative to the previous one and is applicable to the same type of items.

6.4.3.4.3 Beam model or one-dimensional finite element

This type of modelling is typically applied to beams, columns, frames, piping, ducts, cable trays, conduit, symmetric tanks, cabinets, storage racks, pressure vessels and heat exchangers and may be formulated as continuous or one-dimensional finite elements in two- or three-dimensional space. Representation of masses may be made by lumped parameter which develops a diagonalized elemental mass matrix or by means of consistent mass matrices which have the same off-diagonal form as the elemental stiffness matrices.

6.4.3.4.4 Plate or shell or two-dimensional finite elements

This type of modelling is typically performed on items whose primary mode of failure is by biaxial bending, plane stress or plane strain. Included in this category are typically, foundation media, cabinets, slabs and tanks, pressure vessels and heat exchangers the shells of which support significant eccentric loads which would tend to excite shell or lobar modes of vibrations.

6.4.3.4.5 Three-dimensional finite element

This type of modelling has not been used extensively to date but would be applicable to thick walled vessels and soil-structure interaction.

6.5 Non-linear analysis

When the stresses in components are higher than the elastic limit of the material or for other reasons (for example fretting, gaps), the stiffness matrix can be dependent on the amplitude of response.

For sufficiently weak non-linearities, the main effect is an increase of the energy losses which is taken into account by larger damping coefficients[19, 34].

For greater non-linearities, it may be necessary to consider the change in stiffness which can significantly change the response of the system, and thus the amplitude of displacements.

7 Damping

The internal material damping of usual structural materials is generally very low. The effect of apparent damping in structure to limit response is due primarily to a combination of energy losses (frictional, hysteretic and other possible non-linearities associated with cracking, local yielding, joint slippage and support gaps). This effective damping generally increases with the amplitude of motion. This results in the use of an overall modal structural damping coefficient, expressed in terms of a ratio to critical damping, for the various items of an NPP. Such coefficients can be specified for two or more ranges of amplitudes of motion.

Values of percentage critical damping coefficients applicable to various structures and components are contained in references [19, 34 and 35]. Results of appropriate tests to evaluate damping coefficients for specific items can also be used for analysis. Modal damping factors for composite structures can be obtained by energy weighting techniques[36] or by other techniques[32].

7.1 Damping matrix

In the time history direct integration method, it is necessary to construct a damping matrix. An acceptable procedure is to express the damping matrix as a combination of the mass and stiffness matrices of the model as follows:

$$C = \alpha M + \beta K$$

where C, M and K are the damping, mass and stiffness matrices, respectively, of the model. The parameters α and β are determined by the requirement that the modal damping in the frequency range of interest is equal to, or less than, the prescribed modal damping. Other methods for obtaining the damping matrix are available[32].

7.2 Foundation-structure damping

7.2.1 Lumped parameter models

The most important contribution to foundation-structure damping for lumped parameter models of soil-structure interaction is energy dissipation within the ground in the form of internal material and radiation damping. Both damping mechanisms are used to define energy loss in the foundation-structure analysis. Material damping in the foundation media typically ranges from 5 to 15 % of critical damping. Radiation damping for nuclear power plants typically ranges from 5 to 40 % of critical damping. It is recognized that the value of damping that is used in seismic analysis of structures will require conservatively applied engineering judgement; conservative practices should be followed in all cases. However, care should be taken not to limit radiation damping coefficients so conservatively that the response dominant frequencies are erroneously shifted. If soil radiation damping is evaluated with reference to a rigid structure on an elastic half-space model, it is suggested that the weighted modal damping be limited to some fixed figure[20, 35]. Variation of damping factors with frequency may be taken into account if warranted by experimental data.

7.2.2 Finite element models

For finite element models of soil-structure interaction damping is considered as material damping. Such damping may be expressed either in the form of a damping matrix predominantly as a function of stiffness as shown in 7.1 or as percentage critical damping. The equivalent of radiation damping in the finite element formulation is normally represented by transmitting boundaries[37 to 39].

8 Load combinations and allowable behaviour

Load combination, including seismic loads, shall be defined. These load combinations are strongly dependent on the risk level associated with the seismic design adequacy. These are therefore topics that are the concern of the national regulatory authority in a country.

9 Verification of seismic design adequacy

Two techniques are available to verify seismic design adequacy: analysis and test. Analysis is usually performed on those components which can be represented with reasonable accuracy in mathematical models and whose behaviour can be determined as a function of calculated stresses or deformations. Tests are performed on those components which typically have to function or respond actively (i.e. move or change state) and whose behaviour cannot be determined simply as a function of stress or deformation.

9.1 Design adequacy demonstrated by analysis

A detailed set of analytical calculations shall be prepared which presents the following information:

a) the seismic input;

b) the way in which seismic input was used to determine the behaviour of the equipment;

c) the analytical method and model used;

d) the basis of the acceptance criteria in terms of applicable codes or standards, allowable stress, deformation or other behaviour criteria;

e) the limiting behaviour, for active components;

f) a demonstration that the stresses, deformations, etc., determined from postulated limiting behaviour analysis, do not violate the acceptance criteria.

Also required is an administrative procedure which ensures that the calculations have been adequately checked for numerical accuracy, that the analytical method is valid, that the boundary conditions are correct and that a valid computational algorithm or computer program has been used.

9.2 Design adequacy demonstrated by test

9.2.1 General

When the analysis of the integrity of functional capability of an item cannot be performed with a reasonable degree of confidence, an experimental test shall be performed to prove the seismic design adequacy of the item.

Seismic tests may be performed on the item itself or on full-scale models or where appropriate, reduced scale models may be used. However, for qualification purposes, testing should be of the equipment itself or of a full-scale model. If there is no other practical alternative, the careful use of a reduced scale model may be permitted. The tests include functional tests intended to ensure the continued adequate functioning of the equipment during and after an earthquake and integrity tests aimed to prove the mechanical strength of the equipment.

9.2.2 Fundamental concepts

For the purpose of assessing integrity and functional capability, a meaningful test requires that the conditions existing for the systems on the plant during the earthquake are correctly simulated or that any departure from these will not significantly influence the result. Among these conditions the most important are:

a) mechanical boundary conditions;

b) environmental conditions (pressure, temperature, etc.);

c) operational conditions (if functional capability has to be assessed);

d) input motion.

The functional and integrity testing of complex structures, such as control panels, may initially require the functional testing of individual components and then the testing of the full assembly or of a suitable model of it to establish that the forces exerted on components during an earthquake do not exceed limits derived from the initial tests on the components.

In establishing a qualification test, consideration should be given to such effects as ageing or other conditions which may cause deterioration or otherwise alter characteristics of the item during its in-service life. In addition, consideration should be given to the need for and possibility of in-service inspection.

Tests may also be used to check a calculation or to validate an analytical model.

9.2.3 Full-scale testing

If a tested piece of equipment is to be installed in the plant, its ability to operate reliably and safely after seismic testing shall be assessed by either one or a combination of the following techniques, as practicable:

a) instrumental assessment during seismic testing to evaluate stresses and deformations;

b) thorough inspection;

c) *in situ* operational testing.

Mechanical damage (fatigue, incremental distortion, wear) induced by testing shall be kept within acceptable limits as set for normal and occasional operating conditions; if this cannot be done, the item shall be repaired or replaced.

9.2.3.1 Laboratory testing

9.2.3.1.1 Preliminary testing

Natural frequencies and other vibration characteristics of the components may generally be assessed by a preliminary test (for which a sine wave input can be used) during which any response parameter of interest is monitored (for example displacements, electrical signals).

9.2.3.1.2 Qualification testing

For systems in the rigid range, a sinusoidal motion, at a frequency significantly lower than the first natural frequency of the system and having the required maximum acceleration and number of cycles, may be used.

When, on the contrary, the system has one or more mechanical resonances in the frequency range of interest, the test input motion used should have a response spectrum not less than the required design response spectrum. This can be achieved by use of a natural or artificial time history and, in particular, by a suitable combination of sine waves.

When the natural frequencies are well separated, separate tests may be made, for example, with suitable scaled sinusoidal movements at the given natural frequency enveloped by a half-sine[13] or other appropriate time envelope. "Suitably scaled" means that the corresponding amplitude of the spectrum at the natural frequency is adequately higher than the amplitude of the required spectrum.

However, it is recommended that tests be conducted with one or more natural or artificial time histories the spectrum of which is not less than the required design response spectrum. Several different time histories may be used to overcome simulation deficiencies which could arise from the peculiarities of a single time history.

If random vibration input motion is used, detailed recommendations[40] should be followed. Input motion duration should be decided on the basis of anticipated earthquake duration and number of possible aftershocks. In practice, however, usually no problem arises from this aspect of testing performance because of the feasibility of conservatively overprolonged tests.

For components the functional capability of which has to be demonstrated by test under earthquake conditions, excitation in one direction at a time can be considered adequate when one of the following conditions applies

— if component design and review, and visual inspection or exploratory tests clearly demonstrate that the effects of excitation in the three directions on the component are sufficiently independent of each other.

— if the severity of shaking-table tests can be increased in such a way as to take into account the interaction effects due to simultaneous excitation in the three directions; for example the amplitude excitation can be increased and the excitation directions varied.

9.2.4 Reduced scale model testing

For structures or equipment which are too big to be tested on a full scale, model tests can be made to check the analytical models or, in those cases where there is no practical alternative, to directly qualify the system.

Great care shall be taken in designing and performing model tests and in interpreting their results. The scale of models should not be less than a value dependent on the adequacy of the modelling technique to simulate correctly at that reduced scale the material, structural and geometric properties of the original. For example ratios of 1/5 for concrete structures and 1/10 for steel may be used as reasonable limiting values at this time. The similarity law used should be clearly expressed with special attention given to the method used to correct the necessary discrepancy between the gravity and stiffness effects. The amplitude and frequency content of the input motion should be adequately scaled. Where possible, the representativeness of the model should be verified by vibration testing of the component or structure, preferably after installation.

9.3 Maintenance and in-service inspection

It shall be verified, by means of periodic inspections, that all devices or instruments the operation of which can only be initiated by the occurrence of a noticeably strong earthquake are able to function correctly. Among these items are, for example, devices which limit the rapid motions of piping (snubbers, either hydraulic or mechanical) or the components of the plant seismic instrumentation system.

10 Effect of ground motion on the site

10.1 Seismic response of soil deposits and earth structures

10.1.1 General

Soils exhibit a strong non-linear behaviour under cyclic loading conditions. This basic material characteristic shall, therefore, be taken into account when evaluating seismic response of soil deposits or earth structures. Acceptable methods for measuring modulus (including field and laboratory procedures) and damping (laboratory procedures) of soils with ranges of values of these characteristics for various types of soils, are presented in reference [41].

At each specific NPP site, an adequate amount of field measurements (geophysical shear wave measurements in particular) should be performed and cyclic laboratory tests (particularly resonant column tests and controlled-strain or stress triaxial tests) should be conducted on representative samples of soils to develop the variations of moduli and damping as a function of strain level for these soils.

The extent of field measurements and the number of laboratory tests shall be sufficient to develop an adequate relationship between these properties and strain for all the major soil layers to a depth of approximately 50 to 100 m at the site. The actual details of field and laboratory programmes, however, should be decided on a case by case basis. Alternatively, limited field and laboratory programmes may be acceptable if adequate variations in moduli and damping are incorporated in a parametric study of seismic analysis and evaluations.

For seismic waves arriving at a site, two basic solution techniques are currently available:

a) body wave (shear and compression) techniques;

b) surface wave (Love and Rayleigh) techniques.

10.1.2 Body wave solutions

These solutions have been employed from the early 1900s, but in the past 15 or so years, they have been modified to incorporate more correctly the stress-strain behaviour of soils under cyclic loading conditions. Such solutions are available for the one-dimensional condition and for two- or three-dimensional conditions.

Deposits that consist of essentially extensive horizontal soil layers and that are at locations away from plant structures may be considered as semi-infinite layers and can be treated as one-dimensional shear beams. Methods of solution for a shear beam representation are summarized in 10.1.2.1.

Deposits that have significant geometric variations from that considered above (for example significantly sloping underlying rock boundaries or other irregular topographies) cannot normally be treated as semi-infinite layers. The two- or three-dimensional effects should be taken into consideration. Similarly, for deposits adjacent to plant structures and for earth structures, the two- or three-dimensional effects should also be taken into consideration. Method of solution for these cases are summarized in 10.1.2.2.

10.1.2.1 One-dimensional solution

The seismic response of a shear beam representation of a soil deposit may be calculated by several procedures. Each procedure shall be capable of taking into account the non-linear behaviour of soils under cyclic loading conditions. The equivalent linear approach for a nuclear power plant site[42] is considered acceptable to represent this non-linear behaviour.

a) **Lumped mass method.** This method has been employed using either a bilinear representation[43, 44] of the soil stress-strain characteristics or an equivalent linear representation[44]. The solution is carried out in the time domain.

b) **Method of characteristics.** This method[45] uses a non-linear representation of the soil stress-strain characteristics. The solution is carried out in the time domain.

c) **Complex modulus method.** This method[46], which is also referred to as the wave propagation method, is carried out in the frequency domain, but the final output may be transformed into the time domain. The non-linear soil behaviour is accounted for by the equivalent linear approach. This method has been most widely used in practice at NPP sites.

d) **Finite element method.** This method has also been used for the evaluation of the seismic response of a semi-infinite layer. The solution can be carried out either in the time domain[47] or in the frequency domain[48]. The non-linear soil behaviour is accounted for by the equivalent linear approach.

e) **Special features.** Normally, the input motion to the shear beam representative of the semi-infinite layer is specified at the base of the layer (in what is considered to be rock or rock-like material; for this purpose rock-like material is defined as rock or soil having a free field shear wave velocity greater than approximately 1 200 m/s). Solutions carried out in the frequency domain provide also for specifying the input motion at any point in the soil deposit. When the input motion is, for example, specified at the free field surface ground level, the response of the deposit can then be computed utilizing this feature of these solutions. Motion at the base of the deposit which has been correlated with motion at other levels can then be computed and used as input in analyses.

10.1.2.2 Two- or three-dimensional solutions

The finite element method has been most widely used for the analysis of the seismic response of soil deposits and earth structures requiring two- or three-dimensional solutions. In most situations, a two-dimensional solution is adequate. The solution can be carried out in the time domain[47] or in the frequency domain[48]. The non-linear soil behaviour is accounted for by the equivalent linear approach. Other methods (for example method of characteristics and lumped mass procedures) have also been used for two-dimensional analyses[45, 49].

The solution algorithm outlined in reference [47] has been most commonly used for analysis of earth dams and slopes. Evaluation of the behaviour of the soils adjacent to and beneath structures has been most commonly done using the solution algorithm outlined in reference [48].

10.1.3 Surface wave solutions

Surface wave solutions have been used in seismologic evaluations for many years. These solutions have been used for evaluating the nature of ground motions in soil deposits in a number of studies[50 to 53] but are not commonly used in practice. Applications using these solutions are currently in progress.

10.2 Liquefaction and ground failure

10.2.1 General

Liquefaction and consequent ground failure resulting from earthquakes have been the cause of significant damage or catastrophic failures. Examples of damage or ground failure caused by liquefaction during earthquakes include:

a) settling and tilting of buildings[54 to 56];

b) floating of buried structures[57];

c) major landslides[58 to 61];

d) lateral movement of bridge supports[61];

e) failure or significant lateral movement of waterfront retaining structures[62];

f) failure or significant lateral deformations of dams and embankments[58, 59].

Therefore, it is necessary that potentially liquefiable soil layers at a nuclear power plant site be identified, their characteristics be evaluated and the potential and consequences of liquefaction in these layers be assessed during the postulated seismic event. Appropriate field and laboratory investigations together with stratigraphic correlations and soil parameter variability are required to identify and evaluate the static and dynamic characteristics of these soil layers.

10.2.2 Methods of evaluation

Evaluation of the liquefaction potential at a site can be carried out using either of two approaches; namely, empirical approaches or an analytical approach coupled with appropriate laboratory tests[63]. Either approach requires that appropriate field tests be conducted.

One empirical approach is based on correlation with observations from past earthquakes for a wide range of conditions[63] and use of a simplified procedure[64] to calculate the level of stresses induced by the postulated design earthquake. Other empirical approaches[54] also based on observation from past earthquakes are available, but are valid for the more limited range of site conditions and seismic excitation upon which they are based.

Analytical evaluation of liquefaction potential consists of the following steps[64, 65]:

a) calculation of the stresses induced by the design earthquakes. The procedures summarized in 10.1 may be used for this calculation. These stresses have non-uniform amplitudes as a function of time. These stresses are then converted to an equivalent number of uniform cycles of shear stress [66];

b) establishment of the cyclic strength characteristics of the soil in this layer. Field data (such as penetrometer results, standard penetration blow count) may be used as a basis for estimating cyclic strength. Where appropriate, a cyclic testing [67] programme on representative samples should also be conducted to establish the cyclic strength. Details regarding appropriate utilization of these field and/or laboratory tests are discussed in reference [63];

c) comparison of the available cyclic strength with the stresses induced by the earthquake. A sufficient margin of safety should be available against the possibility of development of liquefaction in the critical soil layers and fill.

It is recommended that, for sites in high seismic regions, the analytical approach be used to the maximum extent possible; the empirical approach being used as a check. For sites of relatively low seismicity, the use of the empirical approach is generally sufficient.

10.2.3 Safety factors

The safety factor against the occurrence of liquefaction is computed as the ratio of the applicable available cyclic strength (based on the stress required to cause a specified level of strain) divided by the induced stress.

This ratio shall be greater than unity in all critical soil layers and fill. The minimum acceptable value of this safety factor shall be decided on a case by case basis. Generally, a minimum safety factor of the order of 1,5 should be used. However, a substantially lower value can be accepted if warranted by the details of the geologic, seismologic and soil evaluations.

In deciding a minimum acceptable value for a specific site, the following aspects shall be taken into account: the selection of the failure criterion used in making the liquefaction evaluation, degree of conservatism incorporated in the determination of the pertinent seismologic and site characteristics and the consequences of liquefaction on the safety of the plant structures and components and relative density of poorly graded cohesiveless foundation media.

10.3 Slope stability

10.3.1 General

Slope failures have occurred in several historic earthquakes. Therefore, it is necessary that earth and rock slopes, both natural and man-made (cuts, fills, embankments, dams, etc.) be adequately evaluated. Appropriate field and laboratory investigations are required to determine:

a) the extent and distribution of soil layers (or rock formations for rock slopes) within, adjacent to and beneath the slope;

b) the geometry of the slope;

c) the static and dynamic characteristics of the soils (rock);

d) water levels and fluctuations of the water table.

For rock slopes, the formation details and aspects related to zones of fracture (orientation, localized weathering, etc.) should also be gathered.

10.3.2 Methods of evaluation

Seismic stability of slopes has been evaluated by either pseudo-static methods or by dynamic methods of analysis.

10.3.2.1 Pseudo-static method

In this method, the stability of the slope is assessed by computing a safety factor against sliding. A conventional method of slices [68] is used for this computation. The driving forces consist of gravity, surcharge and earthquake loads. The earthquake load is represented by a seismic coefficient and is assumed to act at the centroid of the potential sliding mass. The resisting forces are based on the static strength of the soils computed along the potential sliding surfaces.

10.3.2.2 Dynamic method

This method is based on the use of dynamic response analysis incorporating soil strength characteristics determined by laboratory cyclic tests [58, 59, 69]. The dynamic response analysis is conducted using the two-dimensional method of analysis outlined in 10.1.2.2. A model of the slope and its foundation soils is constructed using the finite element method. The input to this model is a time history motion, either the horizontal or the horizontal and vertical components being simultaneously applied at the base of the model and compatible with the specified ground motion at the plant site. The results of the response evaluation provide the time histories of induced stresses throughout the model.

These stresses shall be compared with the cyclic test results. As cyclic strength depends on the initial stresses, the static stresses within the slope and its foundation soils (i.e. stresses existing before the earthquake) shall be computed; this computation is done using static finite element procedures [69].

The ratio of the cyclic strength stress limit to induced stress is defined as the local safety factor and is computed at various locations throughout the model. The stability of the slope is then assessed by examining the range and variations of the values of this local safety factor.

Procedures are also available to compute the amount of potential deformations of the slope using the results of these dynamic analyses.

10.3.2.3 Other methods

Procedures based on the concept of yielding acceleration [70] can also be used to calculate the potential movement of slopes.

11 Seismic instrumentation

The specific definitions applying to this clause are given in 2.3.

11.1 General

The basic aim of seismic instrumentation is to provide data on a real earthquake affecting the site and the plant in order to make it possible:

a) to determine the physical characteristics of the ground movements with respect to this earthquake in order to compare them with reference seismic movements taken into account in the design;

b) to determine the effects of the earthquake on the various reactor systems and components;

c) to deploy alarms or appropriate safety devices.

The proposed instrumentation corresponds to the minimum instrumentation for low seismicity areas, intensity ⩽ VII MSK. For high seismicity areas, the following minimum requirements should be improved and additional acceleration measuring devices may be provided consistent with individual national standards. As a guideline, the instrumentation suggested in 6.2 of IAEA 50-SG-S2[3] could be considered.

11.2 Number and composition

The requirements set out in the following, regarding number and composition, apply to sites having a DBE corresponding to an intensity ⩽ VII MSK.

11.2.1 One triaxial acceleration pick-up shall be installed in the free field of the ground surface. The pick-up shall be placed at a distance from the reactor building which corresponds to at least twice the length of the greatest foundation dimension and at a distance from other buildings which corresponds to the greatest plan view dimension of these buildings. A seismic trigger and a seismic detector shall be locally connected to this acceleration pick-up.

11.2.2 One triaxial acceleration pick-up shall be installed in the foundations of the reactor building.

11.2.3 The acceleration measuring devices and seismic detectors shall be accessible for the necessary servicing and maintenance. The acceleration measuring devices shall be designed and installed in an earthquake-proof manner such that the degree to which the recordings can be evaluated is not impaired.

11.2.4 The acceleration pick-ups, seismic detectors and seismic triggers shall be set in such a manner that their axes lie parallel to the system of co-ordinates assumed in the seismic calculation.

11.2.5 The acceleration pick-ups, seismic detectors and triggers shall be secured in such a manner that no movement relative to the support can occur.

11.3 Multi-unit plants

The provision of seismic instrumentation for a single-unit plant is sufficient in the case of multi-unit plants if essentially the same vibration behaviour of the reactor building of the other plant can be reckoned with. If the vibration behaviour of the other plant is not essentially the same, then both the reactor building of the other plant and the reactor building of the single-unit plant shall be provided with a triaxial acceleration pick-up.

An essentially similar vibration behaviour exists if the response deviation between reactor buildings in a frequency range of 0,1 to 35 Hz does not amount to more than ± 10 % for any frequency. In this case, the response spectra of the open ground and storeys of the reactor buildings shall be compared for the purpose of assessing the oscillation behaviour.

Annex

Earthquake intensity scales

(This annex does not form part of the standard.)

A.1 General

The intensity of an earthquake at a given place is defined by the effects of the earthquake, for example effects on buildings (cracks, destruction), effects of a geologic nature (ground cracks, landslides, etc.) and psychological effects (fright, etc.). These effects are dependent on many factors, such as the types of structures, the nature of the soil and the distance from the earthquake focus, so that the concept of intensity cannot aim to characterize any particular earthquake in contrast with the concept of magnitude.

It is, however, customary to try to characterize an earthquake, whenever possible, by allotting to it a maximum intensity, and also sometimes, by the law of intensity decrease with distance. In fact many historical earthquakes were known only by these methods until the advent of direct recording of ground motion a few decades ago. The concept of intensity thus remains essential in seismic hazard evaluation.

Intensity scales consequently meet an essential requirement, i.e. to qualify the various effects of earthquakes according to an appropriate grading system. Effects can thus be evaluated at different locations for one and the same earthquake, or for different earthquakes, allowing mutual comparison and subsequent correlation.

In the past, several scales have been proposed and used to this end. At present, several are in use in different countries. However, the establishment of correlations between earthquakes which have affected a given country in the past, as well as those which have affected different countries requires that a comparison be made between these various scales, which all convey different information. It is therefore useful to have a notion of what the various scales cover to know to what extent a correspondence can be established between them. In practice, it is often difficult, if not imposssible, owing to the lack of detailed data, to reassess all the required intensity values on a single scale.

The various earthquake intensity scales developed and used throughout the world at various times are[1]:

— in 1627, Pocardi (Italy) applied a four gradation scale for describing the effects of an earthquake at different points;

— in 1757, M.V. Lomonosov in *Slovo o rozhdenii metallov ot tryasenyia zemli* divided seismic movements into four categories and noted at this early stage that horizontal oscillations were particularly destructive;

— the 19th century saw a proliferation of descriptive scales such as those of D. Brooks (1811), P. Egan (1828), P. Macfarlane (1839), A. Petermann (1856), R. Mallet (1858), R. Williamson (1870);

— in 1873, M.S. De Rossi (Italy) and F.A. Forel (Switzerland) joined forces to compile a ten point scale;

— in 1900, F. Omori (Japan) set up a seven point scale in which a certain value of maximum acceleration was assigned to each intensity;

— in 1902, Mercalli proposed a ten point scale;

— in 1917, the International Seismological Association adopted a revision of the Mercalli scale known as Mercalli-Cancani-Sieberg scale (MCS) — this scale, which includes 12 intensity levels, is used in Europe;

— in 1931, H.O. Wood and Frank Neumann put forward a 12 point scale known as the modified Mercalli scale (MM), which is widely used especially in the USA — the modified Mercalli scale was amended again in 1966 to include building or construction types (New Zealand version 1965);

— in 1920, F. Omori (Japan) improved his scale, the final version of which, including eight ratings, was published in 1950 as the Japanese scale;

— in 1931, the OST VKS 4537 scale, very similar to the Mercalli-Cancani-Sieberg scale, was adopted in the USSR. It includes 12 ratings. Another scale, also including 12 ratings, was established in 1952 by the Institute of Physics of the Earth of the Academy of Sciences of the USSR;

1) MEDVEDEV, S.V. *Engineering Seismology.* Moscow, Akademija Nauk SSSR, 1962.

— in 1956, the Chinese Academy of Sciences adopted a 12 point scale analogous to the USSR scale, but allowing for peculiarities in Chinese buildings and structures;

— in 1964, a new scale, very similar to the scale of the Institute of Physics of the Earth of the USSR Academy of Sciences, was put forward by Medvedev, Sponheuer and Karnik (MSK). It includes 12 ratings and takes account of the type and percentage of affected structures and of the amount of damage.

NOTE — This scale was proposed as the international scale, in particular to UNESCO, and it is the scale predominantly used today in Europe and apparently in the USSR.

A detailed description of the main scales listed below follows:

— MM Neumann scale (see A.5.1) and abridged version (see A.5.2);

— MM New Zealand scale (see A.5.3);

— Japanese scale (see A.5.4);

— scale of the Institute of Physics of the Earth, USSR (see A.5.5);

— MSK 1964 scale (see A.5.6).

Except for the Japanese scale, all modern scales are similar and the various ratings do more or less correspond. S.V. Medvedev, however, points out that accurate correlation of the various scales is not possible and that the accuracy of comparison decreases as the description of the elements involved in the assessment of the intensity levels becomes less detailed, particularly when the intensity criteria retained in the definition of successive ratings are not the same. No wonder then that comparisons between certain scales are hardly possible and that the correlations drawn by otherwise eminent specialists may differ somewhat from one another.

It does not appear desirable under such conditions to propose the standardization of a special correlation. It may, on the contrary, be very useful to put forward several correlation tables, selected from the work of highly qualified authors on the subject.

Such tables are believed to offer both essential elements for the assessment of the possible correspondences between the various scales and an order of magnitude for the expected accuracy of such correspondences.

Table 5 allows the comparison of 40 different scales with reference to the scale of the Institute of Physics of the Earth of the Academy of Sciences of the USSR. It was worked out by G.P. Gorshkov and G.A. Shenkarev.

Table 6 compares eight scales, it was established by Medvedev.

Table 7 is taken from *Use of seismic intensity data to predict the effects of earthquakes and underground nuclear explosion in various geologic settings*, by P.J. Barosh. It compares the MM 1931 scale, the Russian 1952 scale and the Japanese 1950 scale.

Table 8 was drawn up by N.V. Shebalin[1] and correlates eight intensity scales. The document cited is based on a detailed comparative study of the intensity criteria of a large number of earthquakes in the Balkans and by virtue of this constitutes a most interesting reference document. Attention is drawn to the rigorous correspondence between the MCS 1917 scale and the MSK 1964 scale. Table 8 also gives a comparison of the modified Mercalli 1931 scale and of the Rossi-Forel (RF) 1873 scale with the former Mercalli (FM) scale. The comparison with the qualified scales FR-M, FM-M and MCS-M concerns the various scales used by Professor Mikailovic in the Balkans at different times. It bears witness to the personal appreciation of an author who used his own versions of conventional scales.

Generally speaking it has to be noted that the accuracy in the definition of the various intensity degrees of a scale is, at best, about 1/2 degree. It is understood under these conditions that correspondences between the various scales may be difficult to establish.

A.2 Standardization of an intensity scale

In a critical study of the concept of intensity by Eily in 1966, reported by P.J. Barosh in the above mentioned document, the author emphasizes that a single international intensity scale is not advisable, because, for practical application, intensity criteria should be adapted to the characteristics of local constructions, which vary considerably throughout the world. The same reason also motivated IAEA not to recommend the selection of any particular intensity scale.[2] This viewpoint is widely shared by specialists, and no specially set scale should therefore be contemplated for standardization. The use of modern scales (especially MSK 1964, MM New Zealand scale for "Western" countries) should, however, be promoted.

1) Shebalin, N.V. *UNDP/UNESCO Survey of the Seismicity of the Balkan Region. Catalogue of Earthquakes. Part I: 1901-1970; Part II: Prior to 1901.* Skopje, UNESCO, 1974.

2) IAEA Safety guide 50-SG-S1, *Earthquakes and associated topics in relation to nuclear power plant siting.* Vienna, International Atomic Energy Agency, 1979.

A.3 Standardization of a law of correspondence between earthquake intensity and magnitude

Many authors have put forward relationships between the epicentral maximum intensity and the magnitude of earthquakes. Others also have assumed a decrease of earthquake intensity with the epicentral distance as a function of magnitude, focus depth, etc. Such relationships may be of interest for the specific region in which they have been proven. The intensity in one given place depends in fact on many factors: type and natural period of buildings and structures, transfer function of geologic formations, duration and amplitude of seismic movements, frequencies of seismic waves, etc. Such relationships should consequently be handled with great care by qualified persons. Under no circumstances could this topic form the subject of any draft standard in the present state of the art.

A.4 Standardization of a correspondence between intensity and the physical parameters of ground movements (acceleration, velocity, displacement)

The literature abounds in articles on this subject, which preoccupies engineers in paraseismic engineering in particular, since such a correspondence would enable the quantification of the qualitative concept of intensity. This is a field where information is in rapid evolution, thanks to the development throughout the world of networks of recording stations and especially of accelerographs for strong movements. Such devices have already made it possible to obtain a number of direct records of the time history of earthquakes at a given place and to make correlations with the effects observed at the same place (felt intensity). Several relationships of the intensity to the maximum acceleration or velocity of the ground have been proposed. They vary actually from one year to another. However, their usefulness is obvious for experienced specialists who are able to judge their limits of accuracy and credibility and who shall be their sole users.

Today it is acknowledged that the effects of an earthquake are not related in a simple manner to one parameter of the ground motion; the spectral distribution of the seismic motion energy, its duration and other factors (such as the types of the buildings and structures, etc.) also play essential parts. The same intensity can correspond to motions of different spectral shapes; for a single intensity one can observe a very large dispersion of peak values. It therefore seems premature to carry out studies on this question within the framework of standardization activities.

A.5 Description of the main scales

A.5.1 Modified Mercalli 1931 intensity scale (MM, Neumann)

[adapted from the Mercalli-Cancani-Sieberg 1923 scale, modified and condensed. Source: Wood and Neumann (1931)]

I Not felt or rarely felt under especially favourable circumstances, under certain conditions at and outside the boundary of the area in which a great shock is felt
Sometimes birds, animals, reported uneasy or disturbed
Sometimes dizziness or nausea experienced
Sometimes trees, structures, liquids, bodies of water, may sway — doors may swing very slowly

II Felt indoors by few, especially on upper floors, or by sensitive or nervous persons
Phenomena similar to those listed in grade I, but often more noticeably
Sometimes hanging objects may swing, especially when delicately suspended
Sometimes trees, structures, liquids, bodies of water, may sway — doors may swing very slowly
Sometimes birds, animals, reported uneasy or disturbed
Sometimes dizziness or nausea experienced

III Felt indoors by several, motion usually rapid vibration
Sometimes not recognized to be an earthquake at first
Duration estimated in some cases
Vibration like that due to passing of light or lightly loaded trucks, or heavy trucks some distance away
Hanging objects may swing slightly
Movements may be appreciable on upper levels of tall structures
Standing motor cars rocked slightly

IV Felt indoors by many, outdoors by few
Awakened few, especially light sleepers
Frightened no one, unless apprehensive from previous experience
Vibration like that due to passing of heavy or heavily loaded trucks
Sensation like heavy body striking building or falling of heavy objects inside
Rattling of dishes, windows, doors, glassware and crockery clink and clash

	Creaking of walls, frame, especially in the upper range of this grade Hanging objects swung Liquids in open vessels disturbed slightly Standing motor cars rocked noticeably
V	Felt indoors by practically all, outdoors by many or most Outdoors direction estimated Awakened many or most Frightened few — slight excitement, a few run outdoors Buildings trembled throughout Some dishes, glassware broken Windows cracked in some cases, but not generally Vases, small or unstable objects overturned in many instances, with the occasional fall Hanging objects swing, doors bang a little or considerably Pictures knocked against walls or swung out of place Doors, shutters abruptly opened or closed Pendulum clocks stopped, started or caused to run fast or slow Small objects, furniture moved, the latter to slight extent Liquids spilled in small amounts from well-filled open containers Trees, bushes, shaken slightly
VI	Felt by all, indoors and outdoors Frightened many, general excitement, some alarm, many run outdoors Awakened all Walking becomes difficult Trees, bushes, shaken slightly to moderately Liquids set in strong motion Small bells (church, chapel, school, etc.) caused to ring Slight damage to poorly built buildings Fall of plaster in small amount Plaster cracked somewhat, especially on chimneys in some instances Dishes, glassware, broken in considerable quantity, also some windows Knick-knacks, books, pictures caused to fall Furniture overturned in many instances Moderately heavy furniture moved
VII	Frightened all, general alarm, all run outdoors Some, or many, find it difficult to stand Noticed by persons driving motor cars Trees and bushes shaken moderately to strongly Waves formed on ponds, lakes and running water Water made turbid from mud stirred up Small cavities formed in sand or gravel stream banks Large bells (church, etc.) caused to ring Suspended objects made to quiver Damage negligible in buildings of good design and construction, slight to moderate in ordinary buildings, considerable in poorly built or badly designed buildings, adobe houses, old walls (especially where laid up without mortar), spires, etc. Chimneys cracked to a considerable extent, walls to some extent Fall of plaster in considerable to large amounts, also some stucco Numerous windows, furniture broken Loose brick work and tiles shaken down Weak chimneys broken off at the roofline (sometimes damaging roofs) Fall of cornices from towers and high buildings Bricks and stones dislodged Heavy furniture overturned and damaged Considerable damage to concrete irrigation ditches
VIII	General fright, alarm approaches panic Persons driving motor cars disturbed Trees shaken strongly: branches, trunks break, especially palm trees Sand and mud ejected in small amounts Temporary or permanent changes in flow of springs and wells: wells dry up, dried-up sources restart, change in temperature of spring and well waters Slight damage to structures (brick) built especially to withstand earthquakes

Considerable damage to ordinary substantial buildings, partial collapse, collapse of wooden houses in some cases, panel walls in frame structures thrown out, decayed piling broken
Walls collapse
Solid stone walls cracked and broken
Crevices form in wet ground and ground on steep slopes
Chimneys, columns, monuments, also factory stacks, towers, etc., twisted and caused to fall
Very heavy furniture visibly moved and overturned

IX General panic
Ground obviously cracked
Considerable damage to (masonry) structures built especially to withstand earthquakes
Some wood-frame houses built especially to withstand earthquakes thrown out of plumb
Great damage to large (masonry) buildings, some partially collapse or are shifted off their foundations
Frames twisted
Serious damage to reservoirs
Underground pipes sometimes broken

X Ground cracked, especially when loose and wet, up to widths of several millimetres
Fissures up to a metre in width run parallel to canal and stream banks
Landslides considerable from river banks and steep escarpments
Sand and mud shifted horizontally on beaches and flat land
Level of water changed in wells
Water thrown on to banks of canals, lakes, rivers, etc.
Serious damage to dams, dikes, embankments
Severe damage to well-built wooden structures and bridges, some destroyed
Dangerous cracks develop in excellent brick walls
Most masonry and frame structures destroyed, also their foundations
Railway tracks bent slightly
Underground pipelines ruptured or crushed endwise
Open cracks and broad wavy folds in cement pavements and asphalt road surfaces

XI Ground disturbances many and widespread, varying with ground material
Broad fissures, earth slumps, and land slips in soft, wet ground
Water ejected in large amount charged with sand and mud
Formation of tidal waves of significant magnitude
Severe damage to wood frame structures, especially near shock centres
Great damage to dams, dikes, embankments, often over long distances
Few (masonry) structures left standing
Large well-built bridges destroyed by the wrecking of supporting piers or pillars, wooden bridges less affected
Railway tracks bent greatly and come apart
Underground pipelines rendered completely out of service

XII Total destruction, practically all constructions greatly damaged or destroyed
Ground disturbances great and varied, numerous shearing cracks
Landslides, significant falls of rock, collapse of river banks, etc., numerous and extensive
Large rock masses wrenched loose and torn off
Fault slips in firm rock, with notable horizontal and vertical offset displacements
Water channels, surface and underground, disturbed and diverted
Lakes dammed, producing waterfalls, deflected rivers, etc.
Waves seen on ground surfaces (seen in formation in some cases)
Lines of horizon distorted
Objects thrown up in the air

A.5.2 Modified Mercalli 1931 intensity scale (MM abridged)

I Not felt except by a very few under especially favourable circumstances

II Felt only by a few persons at rest, especially on upper floors of buildings; delicately suspended objects may swing

III Felt quite noticeably indoors, especially on upper floors of buildings, but many people do not recognize it as an earthquake; standing motor cars may rock slightly; vibration like passing of truck; duration estimated

IV During the day felt indoors by many, outdoors by few; at night some awakened; dishes, windows, doors disturbed; walls made cracking sound; sensation like heavy truck striking building; standing motor cars rocked noticeably

V	Felt by nearly everyone; many awakened; some dishes, windows, etc. broken; a few instances of cracked plaster; unstable objects overturned; disturbance of trees, poles and other tall objects sometimes noticed; pendulum clocks may stop
VI	Felt by all; many frightened and run outdoors; some heavy furniture moved; a few instances of fallen plaster or damaged chimneys; damage slight
VII	Everybody runs outdoors; negligible damage to buildings of good design and construction, slight to moderate to well-built ordinary structures, considerable to poorly built or badly designed structures; some chimneys broken; noticed by persons driving motor cars
VIII	Slight damage to specially designed structures, considerable to ordinary substantial buildings with partial collapse, great to poorly built structures; panel walls thrown out of frame structures; fall of chimneys, factory stacks, columns, monuments, walls; heavy furniture overturned; sand and mud ejected in small amounts; changes in well water; persons disturbed driving motor cars
IX	Considerable damage to specially designed structures; well-designed frame structures thrown out of plumb; great damage to substantial buildings, with partial collapse, buildings shifted off foundations; ground obviously cracked; underground pipes ruptured
X	Serious damage to some well-built wooden structures; most masonry and frame structures destroyed as well as their foundations; ground badly cracked; rails bent; considerable landslides from river banks and steep slopes; sand and mud shifted; water splashed (slopped) over banks
XI	Few, if any, (masonry) structures remain standing; bridges destroyed; broad fissures in ground; underground pipelines completely out of service; earth slumps and land slips in soft ground; rails bent greatly
XII	Total destruction; waves seen on ground surfaces; lines of horizon distorted; objects thrown up in the air

A.5.3 Modified Mercalli 1965 intensity scale (MM New Zealand)

(Source: ELBY. *New Zealand Journal of Geology and Geophysics* 1966: 124-128.)

MM1	Not felt by humans, except in especially favourable circumstances, but birds and animals may be disturbed Reported mainly from the upper floors of buildings more than 10 storeys high Dizziness or nausea may be experienced Branches of trees, chandeliers, doors and other suspended systems of long natural period may be seen to move slowly Ripples on water in ponds, lakes, reservoirs, etc.
MM2	Felt by a few persons at rest indoors, especially by those on upper floors or otherwise favourably placed The long period effects listed under MM1 may be more noticeable
MM3	Felt indoors, but not identified as an earthquake by everyone Vibration may be likened to the passing of light traffic It may be possible to estimate the duration, but not the direction Hanging objects may swing slightly Standing motor cars may rock slightly
MM4	Generally noticed indoors, but not outside Very light sleepers may be awakened Vibration may be likened to the passing of heavy traffic or to the jolt of a heavy object falling or striking the building Walls and frames of buildings are heard to creak Doors and windows rattle Glassware and crockery rattles Liquids in open vessels may be slightly disturbed Standing motor cars may rock, and the shock can be felt by their occupants
MM5	Generally felt outside and by almost everyone indoors Most sleepers awakened A few people frightened Direction of motion can be estimated Small unstable objects are displaced or upset Some glassware and crockery broken Some windows cracked A few earthenware toilet fixtures cracked

Hanging pictures move
Doors and shutters may bang
Pendulum clocks stop, start or change rate

MM6 Felt by all
People and animals alarmed
Many run outside
Diffficulty experienced in walking steadily
Slight damage to masonry D
Some cracks or falls of plaster
Isolated cases of chimney damage
Windows, glassware and crockery broken
Objects fall from shelves, and pictures from walls
Heavy furniture moved; unstable furniture overturned
Small church and school bells ring
Trees and bushes shake or are heard to rustle
Loose material may be dislodged from existing slips, talus slopes or shingle slides

MM7 General alarm
Difficulty experienced in standing
Noticed by drivers of motor cars
Trees and bushes strongly shaken
Large bells ring
Masonry D cracked and damaged
A few instances of damage to masonry C
Loose brickwork and tiles dislodged
Unbraced parapets and architectural ornaments may fall
Stone walls cracked
Weak chimneys broken, usually at the roof-line
Domestic water tanks burst
Concrete irrigation ditches damaged
Waves seen on ponds and lakes
Water made turbid by stirred-up mud
Small slips and caving-in of sand and gravel banks

MM8 Alarm may approach panic
Steering of motor cars affected
Masonry C damaged, with partial collapse
Masonry B damaged in some cases
Masonry A undamaged
Chimneys, factory stacks, monuments, towers and elevated tanks twisted or brought down
Panel walls thrown out of frame structures
Some brick facings damaged
Decayed wooden piles broken
Frame houses not secured to the foundation may move
Cracks appear on steep slopes and in wet ground
Landslips in roadside cuttings and unsupported excavations
Some tree branches may be broken off
Changes in the flow or temperature of springs and wells may occur
Small earthquake fountains

MM9 General panic
Masonry D destroyed
Masonry C heavily damaged, sometimes collapsing completely
Masonry B seriously damaged
Frame structures twisted
Damage to foundations in general
Frame houses not secured to the foundations shifted off
Brick facings fall and expose frames
Obvious cracking of the ground
Minor damage to paths and roadways
Sand and mud ejected in alluviated areas, with the formation of earthquake fountains and sand craters
Underground pipes ruptured
Serious damage to reservoirs

MM10 Most masonry structures destroyed, with their foundations
Some well-built wooden buildings and bridges seriously damaged
Dams, dykes and embankments seriously damaged
Railway lines slightly bent
Cement and asphalt roads and pavements badly cracked and thrown into waves
Large landslides on river banks and steep coasts
Sand and mud on beaches and flat land moved horizontally
Large and spectacular sand and mud fountains
Water from rivers, lakes, and canals thrown up on banks

MM11 Wooden frame structures destroyed
Great damage to railway lines
Great damage to underground pipes

MM12 Damage virtually total; practically all constructions destroyed or greatly damaged
Large rock masses displaced
Lines of horizon distorted
Visible wave-motion of the ground surface reported
Objects thrown up in the air

Categories of non-wooden construction

Masonry A Structures designed to resist lateral forces of about 9,1 % g, such as those satisfying the New Zealand Model Building Bylaw, 1955. Typical buildings of this kind are well-reinforced by means of steel or ferro-concrete bands, or are wholly of ferro-concrete construction. All mortar is of good quality and the design and workmanship is good. Few buildings erected prior to 1935 can be regarded as in category A

Masonry B Reinforced buildings (good workmanship and with sound mortar), but not designed in detail to resist lateral forces

Masonry C Buildings of ordinary workmanship, with mortar of average quality. No extreme weakness, such as inadequate bonding of the corners but neither designed nor reinforced to resist lateral forces

Masonry D Buildings with low standards of workmanship, poor mortar, or constructed of weak materials like mud brick and rammed earth. Weak horizontally

Windows

Window breakage depends greatly upon the nature of the frame and its orientation with respect to the earthquake source. Windows cracked at MM5 are usually either large display windows or windows tightly fitted to metal frames.

Chimneys

The "weak chimneys" listed under MM7 are unreinforced domestic chimneys of brick, concrete block or poured concrete.

Water tanks

The "domestic water tanks" listed under MM7 are of the cylindrical corrugated-iron type common in New Zealand rural areas. If these are only partly full, movement of the water may burst soldered and riveted seams. Hot water cylinders constrained only by supply and delivery pipes may move sufficiently to break the pipes at about the same intensity.

A.5.4 Japanese intensity scale

[Source: Kawasumi (1951, p. 481) with minor additions from the unpublished version used by the Central Meteorological Observatory]

0 Not felt: too weak to be felt by humans, registered only by seismographs

I Slight: felt only feebly by persons at rest or by those who are especially observant of earthquakes

II Weak: felt by most persons, slight shaking of windows and Japanese latticed sliding doors (*shōji*)

III Moderately strong: shaking of houses and buildings, heavy rattling of windows and *shōji*, swinging of hanging objects; stopping of some pendulum clocks; moving of liquids in vessels; some people are so frightened that they run out of doors

IV Strong: strong shaking of houses and buildings; overturning of unstable objects; spilling of liquids out of vessels

V Very strong: cracking brick and plaster walls, overturning stone lanterns and gravestones, and similar objects; damage to chimneys and mud and plaster warehouses; landslides in steep mountains

VI Disastrous: destruction of 1 to 30 % of Japanese wooden houses; large landslides, fissures in flat ground and some in low fields accompanied by mud and waterspouts

VII Ruinous: destruction of more than 30 % of houses; large landslides, fissures and faults

A.5.5 Geofian scale

(Source: MEDVEDEV, S.V. *Engineering Seismology.* Moscow, Akademija Nauk SSSR, 1962, pp. 129-134.)

Slightly modified seismic scale of the Earth Physics Institute of the USSR Academy of Sciences, and description of aftereffects of earthquakes.

The intensity representing the earthquake force is determined by the quantity X_e, which represents the largest displacement of the spherical pendulum of a seismometer with a natural period of 0,25 s, a logarithmic damping decrement of 0,50 and a static magnification of unity (see table 1).

Table 1 — Geofian scale

Intensity	X_e mm	Brief description of earthquake
1		Oscillations of the ground detected with instruments
2		In individual cases felt by very sensitive persons at rest
3		Oscillations felt by few persons
4	< 0,5	Noted by many persons, windows or doors may rattle
5	0,5 to 1,0	Objects swing, floors squeak, glasses jar, outer plaster crumbles
6	1,1 to 2,0	Light damage to buildings, thin cracks in plaster, cracks in tile furnaces, etc.
7	2,1 to 4,0	Considerable damage to buildings, thin cracks in plaster and stripping of individual pieces, thin cracks in walls
8	4,1 to 8,0	Destruction in buildings, large cracks in walls, falling of cornices or chimneys
9	8,1 to 16,0	Collapse of some buildings, destruction of walls, roofs, floors
10	16,1 to 32,0	Collapse of many buildings, fissures in ground about 1 m wide
11	> 32,0	Numerous fissures on the surface of the earth, large landslides in mountains
12		Large-scale change in relief

The force of the earthquake at points where there are no seismographs is determined from the aftereffects of the earthquake, such as:

a) buildings and structures;

b) residual phenomena in ground and change in the state of the ground and surface water;

c) other indications.

The degree of damage and destruction resulting from an earthquake in buildings constructed without the necessary earthquake countermeasures is established in accordance with the following subdivisions —

a) **by group of buildings:**

group A single storey buildings with walls of unfinished stone, raw brick, adobe, etc.;

group B brick and stone houses;

group C wooden frame houses

b) **by degree of damage:**

light damage	thin cracks in plaster and in tile furnaces, crumbling of outer plaster, etc.;
considerable damage	cracks in plaster, falling of pieces of plaster, thin cracks in the walls, cracks in partitions, damage to chimneys, furnaces, etc.;
destruction	large cracks in walls, splitting of masonry, destruction of individual parts of walls, falling of cornices and parapets, collapse of plaster, falling of chimneys, furnaces, etc.;
collapses	destruction of walls, roofs and floors of an entire building or of considerable parts of a building and large deformation of the walls

c) **by the number of buildings:**

majority;

many;

individual.

A.5.5.1 Buildings and structures: intensity

I No damage

II No damage

III No damage

IV No damage

V Light creaking of floors and partitions
Jarring of glasses
Crumbling of outer plaster
Movement of unclosed doors and windows
Slight damage in individual buildings

VI Light damage in many buildings
In individual buildings of groups A and B, considerable damage
In rare cases (in the case of wet ground) thin cracks on the roads

VII In most buildings of group A, considerable damage and in individual cases destruction
In most buildings of group B, light damage and in many cases considerable damage
In many buildings of group C, light damage, with considerable damage in individual buildings
In some cases, landslides on steep slopes of road embankments, cracks in roads and dislocations in joints of pipelines
Stone walls damaged

VIII In many buildings of group A, there is destruction and individual buildings collapse
In most buildings of group B, there is considerable damage, and destruction in individual ones
In most buildings of group C, light damage and in many of them considerable damage
Small slides on steep banks, cuts or embankments of roads
In individual cases piping joints break
Statues and tombstones shift
Stone walls destroyed

IX In many buildings of group A — collapse
In many buildings of group B — destruction and individual ones collapse
Many buildings of group C are considerably damaged and some are destroyed
In individual cases, railway lines are twisted and embankments damaged
Many cracks in roads
Breaking and damaging of pipelines
Monuments and statues overturned
Most stacks and towers destroyed

X In many buildings of group B — collapse
In many buildings of group C — destruction and in some cases collapse
Considerable damage to embankments and dams
Local bending of rails
Smokestacks, towers, monuments and stone walls collapse

XI Total destruction of buildings
Destruction of embankments over great lengths
Pipelines become completely useless
Railway lines bent over great lengths

XII Total destruction of buildings and structures

A.5.5.2 Residual phenomena in ground with change in status of ground and surface waters: intensity

I No damage

II No damage

III No damage

IV No damage

V Small waves in unstable water reservoirs; in some cases the spring flow is changed

VI Cracks in wet ground with widths up to 1 cm
In mountainous regions there are sporadic cases of slides and crumbling of ground
Small changes in the spring flow and the water level in wells

VII Thin cracks in dry ground
Large numbers of cracks in wet ground
Individual cases of slides on river banks
Small slides in mountainous regions and crumbling of ground
Possible landslides in the mountains
In individual cases the water becomes muddy in reservoirs and in rivers
Spring flow and the water level are changed
In some cases new springs appear or existing ones dry up

VIII Cracks in ground reach several centimetres
Many cracks on slopes of mountains and in wet ground
Extensive crumbling of ground, slides and mountain landslides
Water in reservoirs becomes turbid
New water reservoirs are formed
New springs of water appear and existing ones dry up
In many cases, spring flow and the water level in wells change

IX Fissures in ground reach widths of 10 cm; and more than 10 cm on slopes and river banks
Large number of thin fissures in ground
Mountain landslides
Many slides and crumbling of ground
Small mud eruptions
Pronounced waves on water reservoirs
New water springs frequently arise or old ones dry up

X Fissures in ground with widths of several decimeters and in individual cases reaching 1 m
Rock slides in mountainous regions and at the seashore
Large mud flows of sand and clay
Surf and splashing of water in reservoirs and rivers
New lakes are produced

XI Numerous fissures are produced on the surface of the earth
Vertical displacement of strata
Large landslides and earth slips
Water saturated friable sediments come out of the fissures
The conditions in the springs and water reservoirs change strongly, as well as the ground water level

XII Large-scale change in the relief
Tremendous landslides and earthslides
Considerable vertical and horizontal faulting and displacement
Large changes in the state of the ground and surface waters
Waterfalls and lakes are produced
River beds change

A.5.5.3 Other indications: intensity

I Earthquakes not felt by persons
The oscillations of the earth are registered with instruments

II Noticed by individual persons who are very sensitive and who are perfectly at rest

III Oscillations noted by a few persons who are at rest inside buildings
Careful observers note only a slight swinging of hanging objects

IV Light swaying of hanging objects and of standing automobiles
Slight vibration of liquids in vessels
Slight rattling of densely stacked unstable dishes
Earthquake perceived by most people indoors
In rare cases, sleepers are awakened
Felt by individual people outdoors

V Hanging objects swing noticeably
In rare cases, pendulums of wall clocks stop
Water splashes sometimes from filled vessels
Unstable dishes and ornaments on shelves sometimes topple over
Felt by all persons inside buildings and by a majority of persons outdoors
All wake up
Animals are restless

VI Hanging objects swing
Sometimes books fall off shelves and pictures shift
Many pendulums of wall clocks stop
Light furniture shifts
Dishes fall
Many persons run outdoors
Movement of persons unstable
Animals run out of their shelters

VII Chandeliers swing strongly
Light furniture shifts
Books, vessels, and vases fall down
All persons run out of buildings and in individual cases jump out of windows
It is difficult to move without support

VIII Some hanging lamps are damaged
Furniture shifts and frequently tilts over
Light objects jump and tilt over
It is difficult to remain standing
All run outdoors

IX Furniture topples over and breaks
Animals very panicky

X Considerable damage to household goods
Animals cry and howl

XI Loss of life, animals and property caused by fragments from buildings

XII Great catastrophe
A considerable part of the population is killed by collapse of buildings
Vegetation and animals destroyed by avalanches and landslides in mountainous regions

A.5.6 Medvedev-Sponheuer-Karnik 1964 intensity scale (MSK)

(Source: SPONHEUER, W. Institut für Geodynamik, Jena, 1965.)

A.5.6.1 Classification

A.5.6.1.1 Types of structures (non-antiseismic)

type A	buildings in field-stone, rural structures, adobe houses, clay houses;
type B	ordinary brick buildings, buildings of the large block and prefabricated type, half-timbered structures, buildings in natural hewn stone;
type C	reinforced buildings, well-built wooden structures.

A.5.6.1.2 Definition of quantity

single, few	about 5 %;
many	about 50 %;
most	about 75 %

A.5.6.1.3 Classification of damage to buildings

grade 1	slight damage	fine cracks in plaster, fall of small pieces of plaster
grade 2	moderate damage	small cracks in walls, fall of fairly larger pieces of plaster, tiles slip off, cracks in chimneys, parts of chimneys fall down
grade 3	heavy damage	large and deep cracks in walls, fall of chimneys
grade 4	destruction	gaps in walls, parts of buildings may collapse, separate parts of the building lose their cohesion, inner walls and filled-in walls of the frame collapse
grade 5	total damage	total collapse of buildings

A.5.6.1.4 Arrangement of the scale

a) persons and surroundings;

b) structures of all kinds;

c) the natural environment.

A.5.6.2 Intensity scale

I Not noticeable

a)[1] The intensity of the vibration is below the limit of sensibility
The tremor is detected and recorded by seismographs only

II Scarcely noticeable (very slight)

a) Vibration is felt only by individual people at rest in houses, especially on upper floors of buildings

1) See A.5.6.1.4.

III Weak, partially observed only

a) The earthquake is felt indoors by a few people, outdoors only in favourable circumstances
The vibration is like that due to the passing of a light truck
Attentive observers notice a slight swinging of hanging objects, somewhat more heavily on upper floors

IV Widely observed

a) The earthquake is felt indoors by many people, outdoors by few
Here and there people awaken, but no one is frightened
The vibration is like that due to the passing of a heavily loaded truck
Windows, doors and dishes rattle
Floors and walls crack
Furniture begins to shake
Hanging objects swing slightly
Liquids in open vessels are slightly disturbed
In standing motor cars the shock is noticeable

V Awakening

a) The earthquake is felt indoors by all, outdoors by many
Many sleeping people awake, a few run outdoors
Animals become uneasy
Buildings tremble throughout
Hanging objects swing considerably
Pictures knock against walls or swing out of place
Occasionally pendulum clocks stop
Unstable objects may be overturned or shifted
Open doors and windows are thrust open and slam back again
Liquids spill in small amounts from well-filled open containers
The sensation of vibration is like that due to a heavy object falling inside the building

b) Slight damage of grade 1 in buildings of type A is possible

c) Sometimes change in flow of springs

VI Frightening

a) Felt by most indoors and outdoors
Many people in buildings are frightened and run outdoors
A few people lose their balance
Domestic animals run out of their stalls
In a few instances dishes and glassware may break, books fall down
Heavy furniture may possibly move and small steeple bells may ring

b) Damage of grade 1 is sustained in single buildings of type B and in many of type A
Damage in a few buildings of type A is of grade 2

c) In a few cases, cracks up to widths of 1 cm possible in wet ground
In mountains occasional landslips
Changes in flow of springs and in level of well water are observed

VII Damage to buildings

a) Most people are frightened and run outdoors
Many find it difficult to remain standing
The vibration is noticed by persons driving motor cars
Large bells ring

b) In many buildings of type C damage of grade 1 is caused
In many buildings of type B damage is of grade 2
Many buildings of type A suffer damage of grade 3, a few of grade 4
In single instances, landslips of roadway on steep slopes, cracks in roads, seams of pipelines damaged, cracks in stone walls