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Seismic Evaluation of Existing Nuclear Power Plants

International Atomic Energy Agency, Vienna, 2003

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SEISMIC EVALUATION OF EXISTING NUCLEAR POWER PLANTS

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FOREWORD

One of the statutory functions of the IAEA is to establish or adopt standards of safety for the protection of health, life and property in the development and application of nuclear energy for peaceful purposes. The IAEA is also required to provide for the application of these standards to its own operations as well as to assisted operations and, at the request of the parties, to operations under any bilateral or multilateral arrangement, or, at the request of a State, to any of that State's activities in the field of nuclear energy.

Requirements pertaining to the seismic hazard assessment of a site and the seismic design of a nuclear power plant (NPP) are established and recommendations on how to meet them are provided in IAEA safety standards. However, the extant safety standards do not cover the issues specific to the seismic evaluation of existing NPPs. This Safety Report provides guidance on good practices in relation to the seismic evaluation of existing NPPs, in support of the relevant safety standards. It covers the seismic evaluation of sites and installations. The content of this report was reviewed at an IAEA Technical Committee Meeting held in Vienna in December 2001 and finalized in accordance with the recommendations of this meeting, particularly regarding the selected values of parameters presented in the tables.

The features of seismic evaluation of an existing NPP that differ from the practices applicable to the design of a new NPP are particularly developed in the Safety Report. Among them the most prominent are:

(a) *The role of the feedback of seismic experience:* The seismic evaluation of existing plants depends, much more than does the qualification of new plants, on the feedback of expert experience of the effects of actual earthquakes on industrial facilities. The role played by the feedback of such experience, the associated practice of plant walkdowns and the qualification by experts are discussed in this Safety Report.

(b) *Non-linear analyses:* As opposed to the design process, the evaluation process includes dealing with post-elastic behaviour. In accordance with recent advances, the purpose of seismic evaluation should be to analyse the strains induced by the postulated input motion and to compare them with the ultimate admissible strains. Unfortunately, this type of approach is not compatible with classical engineering education and practices (including standards, criteria and computer codes), which are orientated towards stress analysis. For this reason, in order to provide convenient guidance, this Safety Report has been prepared in the general framework of stress analysis.

Seismic re-evaluation programmes have been performed for several NPPs in Eastern Europe in the past ten years. These re-evaluations were carried out on the basis of guidelines that were reviewed by the IAEA and that are now incorporated into this Safety Report. The guidance provided in this report has therefore to this extent already been extensively used in the seismic re-evaluations of existing plants.

The text includes numerous 'should' statements, which represent guidance on international good practices and not recommendations based on an international consensus as provided in a Safety Guide.

The IAEA staff member responsible for this publication was P. Labbé of the Division of Nuclear Installation Safety.

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1. INTRODUCTION

1.1. BACKGROUND

The IAEA nuclear safety standards publications address the site evaluation and the design of new nuclear power plants (NPPs), including seismic hazard assessment and safe seismic design, at the level of the Safety Requirements as well as at the level of dedicated Safety Guides. It rapidly became apparent that the existing nuclear safety standards documents were not adequate for handling specific issues in the seismic evaluation of existing NPPs, and that a dedicated document was necessary. This is the purpose of this Safety Report, which is written in the spirit of the nuclear safety standards and can be regarded as guidance for the interpretation of their intent.

Worldwide experience shows that an assessment of the seismic capacity of an existing operating facility can be prompted for the following:

(a) Evidence of a greater seismic hazard at the site than expected before, owing to new or additional data and/or to new methods;
(b) Regulatory requirements, such as periodic safety reviews, to ensure that the plant has adequate margins for seismic loads;
(c) Lack of anti-seismic design or poor anti-seismic design;
(d) New technical finding such as vulnerability of some structures (masonry walls) or equipment (relays), other feedback and new experience from real earthquakes.

Post-construction evaluation programmes evaluate the current capability of the plant (i.e. the plant '*as is*') to withstand the seismic concern and identify any necessary upgrades or changes in operating procedures. Seismic qualification is distinguished from seismic evaluation primarily in that seismic *qualification* is intended to be performed at the design stage of a plant, whereas seismic *evaluation* is intended to be applied after a plant has been constructed.

Although some guidelines do exist for the evaluation of existing NPPs, these are not established at the level of a regulatory guide or its equivalent. Nevertheless, a number of existing NPPs throughout the world have been and are being subjected to review of their seismic safety. Rational feasible criteria for resolving the main issues have been developed in some Member States.[1]

[1] Particularly in the USA; these criteria have in some instances been adapted for the specific conditions in western and eastern European countries.

1.2. OBJECTIVE

The main purpose of this report is to provide guidance for conducting seismic safety evaluation programmes for existing NPPs in a manner consistent with internationally recognized practice.

This report may be used as a tool for regulatory organizations and other organizations responsible for the execution of seismic safety evaluation programmes, giving a clear definition to different parties, organizations and specialists involved in their implementation of:

(a) The objectives of the seismic evaluation programme;
(b) The phases, tasks and priorities in accordance with specific plant conditions;
(c) A common and integrated technical framework for acceptance criteria and capacity evaluation.

1.3. SCOPE

The scope of this report covers the seismic safety evaluation programmes to be performed on NPPs so as to ensure that the required basic safety functions are available, with particular application to the safe shutdown of reactors.

Seismic safety evaluation programmes should contain three important parts, which are discussed in this Safety Report:

(1) The assessment of the seismic hazard as an external event, specific to the seismotectonic and soil conditions of the site, and of the associated input motion;
(2) The safety analysis of the NPP resulting in an identification of the selected structures, systems and components (SSSCs) appropriate for dealing with a seismic event with the objective of a safe shutdown;
(3) The evaluation of the plant specific seismic capacity to withstand the loads generated by such an event, possibly resulting in upgrading.

Seismic evaluation of existing NPPs relies much more on feedback experience than qualification of new NPPs does; and the feedback experience is mainly revealed through the practice referred to as walkdowns. Both outlines of feedback experience and conducting of walkdowns are also discussed in this Safety Report.

Evaluation programmes at existing operating plants are plant specific or regulatory specific. This means that this report is meant to define the minimum generic requirements and may need to be supplemented on a plant specific basis to consider particular aspects of the original design basis.

Among the options available, two methods are particularly appropriate for assessing the seismic safety of facilities, the seismic margin assessment (SMA) method and the seismic probabilistic safety assessment (SPSA) method. Both SMA and SPSA are discussed in this report.

Current NPP design criteria and comprehensive seismic design procedures (e.g. Ref. [1]), as applied to the design of new facilities but using a re-evaluated seismic input, may be applied in the seismic evaluation programme. It is noted that these would be a conservative and usually expensive approach for evaluation of an existing operating facility and they are not discussed further in this report.

Evaluation of existing NPPs may result in the identification of items of the SSSC list which have to be upgraded. Upgrading itself is not covered by this Safety Report; however, some general principles are presented in order to preserve consistency between evaluation and upgrading processes. (It should be pointed out that when an upgrading programme has to be carried out, it necessitates more engineering resources than the evaluation process does; similarly upgrading is too large and complex a matter to be covered by this Safety Report.)

1.4. STRUCTURE

Section 2 presents the general philosophy of seismic evaluation; Section 3 discusses data collection and investigations; Section 4 is devoted to seismic hazard assessment; Section 5 discusses the safety analysis of the NPP; Section 6 discusses the practice of walkdown; Section 7 covers the criteria and methods used for seismic capacity assessment of SSSCs; Section 8 discusses the principle of the design of a possible seismic upgrading; Section 9 specifies some rules of quality assurance and organization.

2. GENERAL PHILOSOPHY

2.1. MAIN LINES OF SEISMIC EVALUATION

2.1.1. Purpose of seismic evaluation

It is fundamental to the successful completion of any seismic evaluation that the purpose of the evaluation is established before the evaluation process is initiated. There are significant differences and choices in the available evaluation procedures depending on the purpose. If the evaluation is being conducted as a periodic or general review of all NPPs in a group, the selection of the review level earthquake (RLE), for instance, would be done one way and not another[2]. If the evaluation were to address new information on the seismic hazard to a plant, the RLE might be selected to closely capture the new hazard. Again for a general review, the use of generic fragility tables may be appropriate, but if there is a specific challenge or if the RLE is about equal to the safe shutdown earthquake (SSE), the use of plant specific fragility information may be required.

2.1.2. Philosophy of the present Safety Report

It is recognized that the final judgement on the safety of an existing NPP should integrate the information about the level of input motion, the analysis methodology and the capacity assessment criteria. A lower level of input motion may be compensated for by more severe capacity assessment criteria. The philosophy of this Safety Report is to retain a rather high level of input motion associated with adapted capacity assessment criteria. It is expected that this approach will lead to examination of the possible non-linear behaviour of some SSSCs and therefore result in a deeper investigation of the features of the NPP under consideration and in a better understanding of failure modes and available margins.

The guidance provided in this Safety Report is intended to be used for the assessment of the functionality of the NPP under consideration. For instance, the approach presented in this Safety Report is intended to be applicable to structures that have to support equipment during an earthquake or to piping systems that have to retain their flow capacity.

2.1.3. Seismic input

The assessment of the seismic hazard of the site should be divided into two tasks:

(1) Evaluation of the geological stability of the site, for example the absence of any capable fault that could produce differential ground displacement phenomena underneath or in the close vicinity of buildings and structures important to nuclear safety.

(2) Determination of the severity of the seismic ground motion at the site. The underlying principle is that the severity is similar to the one that would be

[2] For example, for the seismic portion of the US Individual Plant Examination for External Events of all US NPPs, the choice of the RLEs was based on a broad class of seismic hazard estimates.

calculated for a new NPP on the same site. This point is further developed in Section 4.

2.1.4. Safety analysis

The purpose of the safety analysis is to determine the SSSCs required for ensuring the safety objectives in the case of a seismic event, and to specify for each of them the required functions they have to assure or the failure modes that have to be prevented during or after an earthquake.

Once the seismic demand has been established and this demand exceeds the original design basis, the seismic safety assessment of the facility can be conducted by using one of the two following methods:

(a) The SMA method, in which the earthquake level is designated to be the RLE.
(b) The SPSA method, in which the earthquake hazard to be evaluated consists of a continuous range of earthquakes which tend to bracket the design basis earthquake so that realistic earthquake induced core damage frequencies can be assessed.

As a minimum, it should be demonstrated that the SSSCs have adequate capacity to ensure the required function under a RLE if the SMA method is used or to ensure acceptable consequences if the SPSA method is used.

2.1.5. Feedback experience and walkdowns

Evaluation of the seismic capacity or fragility of SSSCs relies to a large extent on feedback experience (real earthquakes or experimentation) gathered in databases. This feedback experience

- Should be taken into account in the safety analysis process,
- Supports the capacity evaluation of the SSSCs proposed in this report,
- Implies the practice of walkdowns,
- May come from diverse sources so that it must be validated as to its applicability to the specific evaluation.

The main objectives of walkdowns are:

(a) To review the SSSCs: to confirm the list of the SSSCs, their required functions, their possible failure modes; to screen out the SSSCs which feature a seismically robust construction and to identify the ‘easy-fixes’ that have to be carried out regardless of any analysis; to confirm that the database is appropriate to the SSSCs under consideration.

(b) To check the extent to which the as-built conditions correspond to the design drawings when the evaluation is based on analysis.

(c) To define representative configurations for further evaluations.

2.1.6. Capacity assessment

In principle, the criteria for the assessment of the seismic margin capacity should be more conservative than those which would be permitted in conventional seismic evaluation of industrial facilities but less conservative than those currently required for qualification of new NPPs.

The evaluation process basically deals with post-elastic behaviour; the model should be representative of the physical phenomena involved. Nevertheless, to the extent possible, it is recommended to keep the model as simple as possible and to avoid unreasonable sophisticated non-linear models and controversial results. In order to make a judgement, it is highly preferable to document the 'as-is' facilities relevant data that permit credit for ductile performance rather than to provide a large number of non-linear analyses. However, static non-linear analyses (such as the 'pushover' method) may be of interest to assess the margins of a structure or of a mechanical system and/or to obtain a better understanding of its behaviour.

2.2. TECHNICAL FINDINGS

The main technical findings relevant to seismic evaluation of existing NPPs are summarized in this section. These technical findings should be considered when establishing the seismic evaluation programme of the plant.

It is a known technical finding that well designed industrial facilities, especially NPPs, have an inherent capability to resist earthquakes larger than the earthquake used in their original design. This inherent capability is a direct consequence of the conservatism that exists in seismic design procedures and is usually described in terms of '*seismic design margin*'.

Although the peak ground acceleration (PGA) is a parameter widely used to scale the seismic input, it is also a known technical finding that the capacity of seismic input motion to cause damage is poorly correlated to the PGA level; even the elastic response spectrum is a poor tool for that. It is now recognized that other parameters (such as velocity, displacement and duration of the strong motion) play a significant role in a judicious evaluation of the effects of an input motion. In this regard, it is known that near field earthquakes with small magnitudes can produce significant PGA levels but do not produce significant damage to structures and mechanical equipment; nevertheless, they may produce spurious behaviour of electrical and/or instrumentation and control (I&C) systems. On the other hand, it is suspected that remote earthquakes with long duration and significant low frequency energy may pose a liquefaction or sloshing hazard.

Regarding the structures and the mechanical components, a result of R&D in the past decade shows that, due to dynamic aspects of the phenomena, a safe anti-seismic design relies more on ductile capacities in accommodating large strains than on capacities in balancing large forces (such as the forces which are usually estimated on the basis of elastic behaviour and of a static equivalent approach).

Typically, seismic design criteria applicable to NPPs are specified in such a way that, although it is known that they introduce very large seismic design margins, their size is not usually quantified. At this stage, an adequate seismic design margin is ensured through the use of design criteria in industry norms, standards and guidelines. Because of the ways that seismic design margins are introduced by design criteria, the seismic margin typically varies greatly from one location in the plant to another, from one structure, system and component to another, and from one location to another in the same structure.

After the plant has been constructed it may be very costly to add the same seismic design margin. At the post-construction stage, an adequate seismic margin can be identified through the use of special post-construction safety evaluation procedures, such as plant walkdowns. These plant walkdowns should be conducted by highly qualified engineers, with knowledge of the specific details of the seismic induced damage or failure that could occur for each of the SSSCs to be re-evaluated. In examining only the lower seismic design margins important to safety, these procedures are considered more efficient than traditional seismic design criteria and methods. The facts that the plant is already constructed and operating, and the details of its construction and 'as-is' conditions can be inspected, are also important considerations in deciding on the level of effort and methods that can be used in its seismic evaluation.

3. DATA COLLECTION AND INVESTIGATIONS

3.1. SITE AND PLANT DATA

3.1.1. Soil data

For site seismic response analysis, both static and dynamic material properties of soil and rock are required. For rock layers, documentation of rock properties at low seismic strain is adequate. For soil layers, the density and low strain properties (normally in situ measurements of P and S wave velocities and in-laboratory measurement of material damping) have to be provided. The variation of dynamic shear modulus and damping values with increasing strain levels is needed. Generic soil property variation with strain level may be used if soil types are properly correlated with the generic classifications. Appropriately conservative ranges of static and dynamic values, which account for all the elements of the site geotechnical specificity, should be investigated and documented. Information on the mean level and variation of the water table should also be obtained.

The strain compatible shear moduli and damping values are the basis for the derivation of the mathematical model of the layered soil; they should be provided.

Other soil data may also be necessary under certain circumstances. To the extent possible, the collection of these data should be carried out in compliance with a forthcoming IAEA Safety Guide [2].

3.1.2. Collection of original design basis data

Emphasis should be given to the collection and compilation of original design basis data and documentation in order to minimize the effort required for the seismic evaluation programme.

In that regard the following aspects should be covered:

(1) Seismic input used in the original analysis and design

(a) Seismic parameters, such as the magnitude or intensity used to define the original input motion.
(b) Free field ground motion parameters by means of either acceleration time histories or elastic ground response spectra.
(c) If some structures were designed in accordance with design codes whose design spectra have implicit reductions for inelastic behaviour, the corresponding elastic ground response spectra should be derived in order to provide a basis for comparison with the requirements of the newly defined RLE and of the seismic evaluation programme.

(2) Assumptions and methods of analysis

Assumptions and structural analysis methods used to apply the free field ground motions for the original design of SSSCs including:

(a) Soil–structure interaction effects.

(b) Modelling techniques and analytical methods used to calculate the seismic response of structures and the in-structure response spectra.
(c) Material and system damping.
(d) Allowance for inelastic behaviour.

(3) Standards and procedures

(a) Procedures and standards adopted to specify the properties of the materials and their mechanical characteristics.
(b) Procedures and standards used to define load combinations and to calculate seismic capacities.
(c) National procedures and standards for conventional buildings should be considered as a minimum requirement.

(4) Plant documentation

The plant specific information applicable to SSSCs should include the following:

(a) Design and as-built drawings of safety related structures and supports for systems and components.
(b) Design calculations.
(c) Reports of tests performed for seismic qualification of equipment.
(d) Field installation and erection criteria.
(e) Quality assurance documentation.

3.1.3. Additional important data

Other relevant data have to be collected such as:

(a) As-built conditions for materials, geometry and configuration. It is important to establish the accuracy of the data. As discussed later in Section 6 the preliminary screening walkdown should confirm documented data and acquire new information.
(b) Internal PSA results if any.
(c) Reports of tests (if any) performed for dynamic identification of structures and elements.
(d) Data about any significant modification or/and upgrading measures.
(e) Data about service life remaining and end of life properties when relevant.

3.2. EARTHQUAKE EXPERIENCE AND SEISMIC TEST DATA

3.2.1. Framework for the use of feedback experience

The estimate of the seismic capacity of systems and components is often accomplished by the use of experience gained from seismic events causing very strong motion. Data from strong motion earthquakes have generally been collected to provide the information required to directly verify the seismic adequacy of individual items in existing operating plants.

Such qualification requires that the seismic excitation of an item installed in an industrial facility subjected to a strong motion, at its point of installation in the building structure, envelops the seismic input motion defined for similar items at the given NPP. It also requires that the item being evaluated and the one that underwent the strong motion have similar physical characteristics and have similar support or anchorage characteristics. Alternatively, the support or anchorage capacities can be evaluated by additional analysis. In the case of active items, it is in general also necessary to show that the item subjected to the strong motion earthquake performed similar functions during or following that earthquake, including potential aftershock effects, as would be required for the safety related item being evaluated.

3.2.2. Databases

In response to the US Nuclear Regulatory Commission (NRC) unresolved safety issue A-46, the Seismic Qualifications Utility Group (SQUG) developed jointly with the NRC through the Senior Seismic Review and Advisory Panel (SSRAP) an earthquake experience and test based judgemental procedure (i.e. the generic implementation procedure (GIP) [3]). GIP uses seismic empirical methods to verify the seismic adequacy of the specified safety related equipment in operating NPPs.

This procedure is primarily based upon the performance of installed mechanical and electrical equipment which has been subjected to actual strong motion earthquakes as well as upon the behaviour of equipment components during simulated seismic tests.

Except for limitations such as the ones mentioned hereunder, the SSRAP/GIP [3] approach of using real earthquake experience and generic test data is an alternative to formal seismic qualification of systems and components in operating NPPs for those systems and components included in the available databases. Before the SSRAP/GIP data are used for a specific evaluation, the applicability of the data should be verified.

It should be noted that most building structures and some systems and components are so specialized that they are not included in the earthquake experience database. For those SSSCs, the seismic qualification should be carried out usually by analysis in the case of structures, systems and mechanical components, and by tests or a combination of tests and analysis for electrical and I&C equipment.

In particular, building structures, systems and components forming part of the reactor coolant systems pressure boundary and main heat transport systems are excluded from the uses of the ‘earthquake experience procedures’.

The procedure was adapted to other types of NPPs outside the USA, particularly to water cooled, water moderated (WWER) NPPs in eastern Europe. This type of adaptation requires that the adequacy of the available database be carefully assessed and possibly that a new database be set up, because components used in one country may be of significantly different design from those used in another and therefore may not be represented in the available database.

3.3. SEISMIC INSTRUMENTATION

The main objectives of seismic instrumentation are:

(a) To provide data on the seismic motion parameters at selected locations to confirm or validate the design and evaluation bases,
(b) To help in the decision making process for the appropriate response in the case of earthquake occurrence.

The current situation of seismic instrumentation and scram systems at the plant, along with their operation and functions, should be reviewed.

The review of the existing instrumentation should consider: (a) the local seismological network at the near region around the site; (b) the seismic instrumentation at the plant itself.

4. SEISMIC HAZARDS

The assessment of the seismic hazards specific to the seismotectonic conditions at a site is performed on the following bases:

(a) IAEA Safety Guides [2, 4],
(b) Use of current internationally recognized methods and criteria,
(c) New data.

The seismic level 2 (SL-2) (as defined in IAEA Safety Guide NS-G-3.3 [4]) should be updated in accordance with the above bases in the event that a reason for this has appeared since the evaluation of the SL-2 design level and should be used in the evaluation. In particular, the PGA of the RLE should not be less than 0.1*g*.

If the original seismic design basis is equivalent to the new SL-2, a seismic evaluation may not be needed for the facility provided that the criteria used in the plant design reflect the degree of conservatism embodied in the current criteria, such as those outlined in IAEA Safety Guide NS-G-1.6 [1].

The use of the above mentioned bases should also determine that there is no capable fault at or near the site vicinity.

4.1. SMA METHODOLOGY: BASIS FOR RLE DETERMINATION

The RLE has a level of extreme ground motion which has a very low probability of being exceeded during the lifetime of the plant and represents the maximum level of ground motion to be used for evaluation purposes. In general terms, the RLE should not be lower than the SL-2.

Other considerations that may lead to the choice of a higher or lower RLE level may be addressed. An RLE higher than the design level may be used to ensure that no unsafe condition appears just above the design level. On the other hand, in some Member States, a short expected residual life of the installation may lead to a lower RLE. The RLE should be established in consultation with the regulator.

The RLE is generally specified by the PGA level; however, other parameters such as velocity and displacement can be used.

Regardless of the parameter used to specify the RLE, the description of the input motion should encompass relevant parameters (such as peak values, time histories, response spectra and/or other types of spectra for acceleration, velocity and/or displacement, as well as duration of the strong motion and/or classical indicators, for example Arias intensity and cumulative absolute velovity (CAV)) appropriate to a realistic estimate of the capacity to damage of this input motion and relevant to the selected methodology of capacity assessment.

Special considerations can be made for defining the spectral shape for evaluation purposes in relation to the spectra used for original design purposes, but in all cases the spectral shape should correspond to the elastic response. It is recommended to determine a median response spectral shape appropriate to the site conditions. In case a standard spectrum is used, it is recommended to choose a smooth average spectrum so that realistic compatible accelerograms can be generated. The principles of a forthcoming IAEA Safety Guide [2] apply for identification of soft sites and for determination of the associated site specific response spectra.

Far and near field events should be taken into consideration. If desired, it is possible to address separately the different types of events.

The vertical ground motion can be determined on a similar way to the horizontal motion. It is recommended that the vertical and the horizontal ground accelerations be combined in an appropriate way considering the source characteristics and soil conditions. As a default, the vertical ground acceleration can be determined inclusively as being equal to or greater than two thirds of the horizontal ground acceleration throughout the entire frequency range. Special care has to be taken of the possible high vertical PGA from a near field earthquake.

Regarding accelerograms, either natural or artificial ones may be used; natural accelerograms are preferred. Natural accelerograms should be selected with respect to the magnitude, distance and other relevant parameters that describe the seismic source. Artificial time histories should be generated so that their mean response spectrum fits the target response spectrum.

The assumption of non-linearity generally requires time history analyses; the results of such analyses are known to be very sensitive to the choice of the input motion. When such analyses cannot be avoided (in some geotechnical issues for instance), several accelerograms should be used and they should be selected carefully.

4.2. SPSA METHODOLOGY

If it is decided to perform an SPSA for evaluation purposes, it is necessary to conduct a site specific probabilistic seismic hazard analysis (PSHA). General guidelines on conducting site seismic hazard assessments can be found in IAEA Safety Guide NS-G-3.3 [4]. In addition to this, state of the art methodologies for conducting PSHA are available in IAEA-TECDOC-724 [5], NRC Regulatory Guide 1.165 [6] and NUREG-1407 [7]. Further detailed guidelines for conducting a PSHA are given in NUREG/CR-6372 [8], NUREG/CR-5250 [9] and EPRI-NP-6395-D [10]. The applicability of these references to a specific country or a specific NPP has to be assessed.

In order to identify potential vulnerabilities and identify important areas for upgrading or modifications, a full seismic hazard uncertainty analysis may not be necessary. A mean hazard estimate may be adequate for evaluation purposes. (It should be noted that a mean hazard estimate convolved with a mean fragility will result in a mean failure probability.)

Most SPSAs in the past have used PGA as the hazard parameter. However, in some cases, use of spectral acceleration or average spectral acceleration over the frequency range of interest has been found to be more desirable.

Spectral shapes to be used in an SPSA study should be broadband and site specific. (For instance, NUREG/CR-0098 [11] median spectral shapes may be used for relatively low to moderate seismicity sites.) However, even in low to moderate seismicity sites, modifications to this shape may be needed to account for site specific effects, for instance in the case of soft sites [2]. Other spectral shapes can be used with the agreement of the regulator.

5. SAFETY ASPECTS

5.1. PROPOSED METHODOLOGIES

The decision should be made early on whether either SMA or SPSA seismic safety evaluation methods are to be used. Factors to be considered include: (i) possible already existing probabilistic safety assessment (PSA) results for internal initiating events other than seismic events, (ii) the seismic hazard level of the site and (iii) the overall objectives of the study (whether a risk estimate is required by the regulator).

The SMA and/or SPSA methods have an advantage in that the entire plant may be evaluated as an integrated unit, including system and spatial interactions, common cause failure, human actions, non-seismic failures and operating procedures. Thus, these methods identify vulnerabilities which affect the overall plant safety, and resulting improvements may include hardware improvements as well as procedural ones.

5.1.1. SMA methodology

The SMA method (success path or fault tree/event tree based), described in NUREG-1407 [7], NUREG/CR-4334 [12] and EPRI-NP-6041 [13], has typically been used for the seismic safety evaluation of existing operating facilities at a level beyond design basis earthquake events, also referred to as RLEs. The methodology is deterministic and follows the same pattern as design procedures, but is more liberal than criteria for new designs. Still, it has a probabilistic basis, which ensures a high reliability of the plant to shut down safely in the event of an RLE. This method permits a determination of whether the capacity of the as-built plant meets or exceeds the RLE.

It was not necessary to verify the seismic adequacy of all the plant equipment defined as being in Seismic Category 1 for the design basis of new facilities in IAEA Safety Guide NS-G-1.6 [1]. In the SMA method it is common to focus the evaluation only on those structures, systems and components (SSCs) (e.g. mechanical and electrical items, I&C and distribution systems) essential to bring the plant from a normal operation condition to a safe shutdown condition and to ensure safety during and following the occurrence of an RLE. The objectives are to identify seismic vulnerabilities, if any, which, if remedied, will result in the plant being able to be shut down safely in the event of such an earthquake. These SSSCs are a subset of the structures, systems and components important to safety.

The SSSCs may be expanded to include additional components as requested or required by the owner, operator, licensee or regulator. Typical examples of expanded scope are (a) cooling of the spent fuel pool, (b) mitigation and containment systems required in the event of a design basis accident, (c) integrity of the radioactive waste system and (d) additional instrumentation. The actual decision about the scope depends on the objectives, purpose and regulatory requirements of the programme, which should have been defined at the start.

For determining the SSSCs, the following criteria and assumptions can be applied:

(a) The plant must be capable of being brought to and maintained in a safe shutdown condition for as long as the recovery actions require[3] following the occurrence of the RLE.
(b) Simultaneous off-site and plant generated power (other than the seismically qualified emergency power) loss occurs for up to 72 h.
(c) The required safe shutdown systems should to the extent practical include one main path and one diverse alternate path.
(d) Loss of make-up water capacity from off-site sources occurs for up to 72 h.
(e) Other external events such as fires, flooding, tornadoes and sabotage are not postulated to occur simultaneously.
(f) A loss of coolant accident (LOCA) and high energy line breaks (HELBs) are not postulated concurrent with the RLE.

The time needed has to be assessed taking into account the reactor type, any off-site network system, and any other plant and site specific conditions. The 72 h guideline indicated in (a), (b) and (d) of the previous paragraph is based on experience from post-earthquake observations. These observations indicate that, within this period, plant operators can typically perform repairs needed to damaged plant items and line up alternate power sources for I&C, cooling water and lubrication.

The actual plant conditions corresponding to 'safe shutdown' will vary from one plant to another. The intent is that the plant be brought to the point where the long term decay heat removal system would start. The conditions for that should be verified for the specific plant being evaluated.

In arriving at the SSSC list, the alternative possibilities for a non-seismically qualified component of being or not being out of order have to be considered.

[3] In some Member States, it is 72 h.

(The fact that a device is not seismically rugged does not imply that it will be unavailable because of a seismic event.)

The minimum critical functions to be assured during and after the RLE occurrence are:

— reactivity control
— reactor coolant system pressure control
— reactor coolant system inventory control
— decay heat removal.

Methods to achieve these four functions are exemplified in Appendix I for the case of a WWER.

Because there is redundancy and diversity in the design of NPPs, there may be several paths or trains which could be used to accomplish the four safety functions mentioned above. As a minimum condition, only the SSSCs in a primary path and a backup path need to be identified for seismic evaluation purposes for a newly defined RLE. The preferred safe shutdown path should be selected and clearly indicated. In selecting the primary path, a single active failure should be considered.

Fluid system flow diagrams, as well as electrical power and I&C diagrams, of the selected systems should be marked up to show the systems and components subject to seismic evaluation. Then a detailed equipment list can be produced. The systems can be categorized as exemplified in Appendix II.

5.1.2. SPSA methodology

This method models the plant response to initiating events using fault trees and event trees. The conditional probability of failure of essential structures and components is represented by fragility curves. Using the event tree/fault tree models, fragility curves and probabilistic seismic hazard curves, the frequency of core damage can be computed. Seismic PSA is generally performed by building on and modifying internal event PSA models. The internal event and the fault trees are modified to include spatial interactions, failure of passive components such as structures and supports, and common cause effects of seismic excitation. A detailed discussion of SPSA methodology can be found in IAEA-TECDOC-724 [5].

Most of the criteria and assumptions developed for the margin method are equally applicable to the SPSA method. The primary difference is that in the SPSA method the list of SSSCs to be reviewed is based on the results of the PSA plant systems analysis. Using the PSA methodology the list of SSSCs to be evaluated may be further limited to those SSSCs which make a significant contribution to core damage frequency. A detailed discussion of the interpretation of SPSA results can be found in Ref. [5].

The basic elements of an SPSA are the following:

— seismic hazard analysis
— response analysis of SSSCs
— evaluation of component fragilities and failure modes
— sequence analysis.

Though the objective of SPSAs is to assess core damage frequency, the last element is not referred to in this report.

To maintain a scope of equipment and structures similar to that resulting from the margin methodology, it is suggested that, as a minimum, SPSA be Level 1 PSA and include all event trees associated with seismic induced transients. It should be noted that the seismic transient trees may branch out to secondary LOCAs, and these scenarios must be retained in the seismic safety evaluation analysis. Concurrently with the PSA Level 1, the performance of the containment should be assessed.

Most of the paragraphs developed for the seismic safety margin method also provide guidelines for the parallel SPSA activities. Discussions related to relay review, building response analysis and failure modes are also pertinent to SPSA methodology. In the following some other general considerations are described.

Details of and methods for fragility and high confidence of low probability of failure (HCLPF) calculations are discussed in a number of references, for example NUREG/CR-4334 [12], EPRI-NP-6041 [13], NUREG/CR-2300 [14], NUREG/CR-4659 Vols 1–3 [15], NUREG/CR-5076 [16] and NUREG/CR-5270 [17]. It is recognized that large uncertainties exist in the estimation of fragilities [17]. A perspective on how this uncertainty affects the results of analysis (numerical and other insights, e.g. dominant sequences and components) should be maintained.

Consistent with the use of a mean hazard, one can use a single mean component fragility curve for each component and hence for sequence level and plant level assessments. This mean curve is defined by the median capacity and the composite uncertainty, $ß_c$. $ß_c$ is such that $ß_c^2 = ß_r^2 + ß_u^2$, when $ß_r$ and $ß_u$ are estimated separately ($ß_r$ and $ß_u$ represent random uncertainty and modelling uncertainty, respectively). It is also acceptable to use a family of fragility curves instead of a single curve.

When a single mean fragility curve is available, the HCLPF capacity for a component (sequence or plant) can be approximated by $-2.3ß_c$ below the median (i.e. a 1% composite probability of failure is essentially equivalent to a 95% confidence of less than a 5% probability of failure).

5.2. SSSC REQUIRED FUNCTIONS, FAILURE MODES

For each SSSC, the required functions that it has to ensure in the case of a seismic event have to be specified. For instance:

(a) For a structure it should be specified whether stability and/or functionality (supporting of equipment) is required. Due consideration should be given to structural elements required for fulfilling leaktightness requirements.
(b) For mechanical components, those which should keep their integrity and those which should remain operable should be listed.
(c) For HVAC, pressure retention may be required.
(d) For cables the functionality, if required, is the signal/power delivery.

At this stage, it is necessary to develop a clear definition of what constitutes failure for each of the SSSCs being evaluated. Several modes of seismic failure (each with a different consequence) may have to be considered.

It may be possible to identify the failure mode which is most dominant or most likely to be caused by the seismic event by reviewing the SSSC design and then to consider only that mode. Identification of credible failure modes is based largely on the feedback experience and judgement of the reviewers. In this task, a review of the performance of similar structures, systems and components and of reported failures in industrial facilities subjected to strong motion earthquakes will provide useful information. Likewise, consideration of (i) design criteria, (ii) qualification test results, (iii) calculated stress levels in relation to allowable limits, and (iv) seismic fragility evaluation studies done on other plants will prove helpful.

Losses of functionality of electrical, mechanical and electromechanical equipment are considered to be the dominant failure modes.

For any type of component, rupture of anchorages has to be regarded as the dominant initiating event of a possible failure mode.

It is a well known technical finding that inertial loads do not result in failure of piping systems; rupture resulting from excessive differential displacement is considered to be the dominant failure mode [18]. However, other limiting conditions may also occur for a pipe when, for instance, the pipe seismic displacements are limited to avoid seismic interactions and excessive impacts with other piping, equipment or structures in the vicinity of the pipe.

The main causes of tank failures (i.e. loss of contents) during earthquakes are breaks or tears of pipe connections to the tank as a result of large relative displacements that occur with or without ‘elephant’s foot’ buckling. On a priority basis, all tank connections should be made flexible and capable of displacements of the order of tens of centimetres without losing their functional integrity. Otherwise their possible functional failure has to be examined.

Structures may be considered to fail functionally when inelastic deformations of the structure under seismic loads are estimated to be sufficient to potentially interfere with the operability of safety related equipment attached to or in close proximity to the structure or when they deform sufficiently that equipment attachments fail. These failure modes represent a conservative lower bound of the structure seismic capacity since a larger margin of safety against building collapse exists for nuclear structures.

Except in cases where special appropriate steps have been taken, masonry walls should be regarded as having poor seismic ruggedness and their failure has to be taken into account in the safety analysis.

A structure failure is generally assumed to result in failure of all safety related systems housed within the portion of the structure which has been judged to fail, i.e. the structural failure mode results in a common cause failure of multiple SSSCs if they are housed in or supported by that structure. An important example consists of safety related panels and electrical conduits mounted on unreinforced masonry walls.

In the past it has also been common practice to assume that all non-seismically qualified or designed structures, systems or components will fail in the event of an RLE. Such an assumption should be based on specific evaluation (walkdown, analysis, etc), because many components have an inherent seismic resistance and would not fail in such an event even if they are not seismically qualified.

It should also be understood that in many instances generic seismic capacity estimates have been developed and that they are available in the literature for systems and components. If the seismic capacity is not controlled by anchorage, support or interaction and if the component is able to meet any specified caveats, these estimates may be used to evaluate system and component seismic capacity for the plant under consideration. In general, careful scrutiny is needed to ensure the applicability of generic seismic capacity.

6. PLANT SEISMIC WALKDOWN

Plant seismic walkdown is one of the most critical components of the seismic evaluation of existing facilities, for both SMA and SPSA methods, regarding the collection of as-built data and the assessment of the seismic capacity of equipment. Detailed guidelines on how to organize, conduct and document walkdowns are provided in GIP [3] and EPRI-NP-6041 [13]. It is crucial that the recommendations of these documents are followed. The objectives of walkdowns are introduced in Section 2.1.5.

The main focus of walkdowns will be on:

(a) Equipment characteristics and inherent seismic capabilities,
(b) Anchorage of equipment,
(c) Load path from the anchorage through the equipment,
(d) Spatial and any other types of interaction.

6.1. ORGANIZATION

6.1.1. Walkdown teams

Each walkdown team should contain two experienced seismic capability engineers, plus systems engineers and plant personnel, as appropriate. The seismic capability engineers should be degree level engineers with an adequate number of years of experience in the design and analysis of systems, structures and components for resisting earthquakes and other loads arising from operation, accidents and external events.

The other members of the walkdown team are there to provide support to the seismic capability engineers and these may be plant personnel. At least one team member must be familiar with the design and operation of the system, structure and component being walked down. Support from several technical disciplines such as mechanical, electrical and I&C departments may be required.

Prior to the walkdown, the team should be provided with the appropriate documentation and it should conduct a systematic review of this documentation. A description and schedule of the tasks to be carried out during the walkdown should be made available. The route of the team should be carefully planed. The as low as reasonably achievable (ALARA) principle should be applied.

6.1.2. Scope of the walkdown

As the basis for plant seismic walkdowns, the SSSC list should be prepared in advance, indicating the functions required to be ensured.

Structures, large vertical tanks in general and main heat transport systems (reactor coolant system including branch lines up to the isolation valves; main steam, normal and emergency feedwater systems up to quick acting isolation valves) should be evaluated by analysis (and verified by tests where appropriate) to determine whether modifications are required. They should nevertheless be examined during the walkdown.

Depending on the level of seismic performance of the original design, other piping systems may be evaluated by limited analysis, using sampling and walkdown procedures. Lines of less than 5 cm nominal diameter and low energy piping systems may be adequately evaluated using walkdown procedures.

Plant seismic walkdowns for distribution systems may be ‘line specific’ for piping/tubing systems or ‘area specific’ for cable tray/conduit and heating, ventilation and air conditioning (HVAC) duct evaluations.

After a first estimate based on the available documentation, the walkdown procedure has to be organized so as to check whether the components are represented by the earthquake experience feedback database and, therefore, whether the procedure can be applied to the component in question. Anchorage and spatial interaction are usually evaluated separately.

Screening approaches based on earthquake experience feedback data, supplemented by generic and specific analysis and test data, may be applied to identify representative cases for analysis and necessary upgrades.

General experience with some plant walkdowns indicates that many electrical and instrument cabinets require modifications to increase the anchorage capacity and that unreinforced masonry walls require upgrades. Electrical and mechanical distribution system supports require selective upgrading. Some mechanical equipment usually requires an upgrading to increase the anchorage capacity.

The seismic walkdowns may be conducted in two stages as indicated in the following subsections:

(1) A preliminary screening walkdown,
(2) A detailed screening walkdown.

6.1.3. Preliminary screening walkdown

This preliminary walkdown should accomplish the following specific objectives:

(a) Determine the location in the plant of each SSSC,
(b) Identify any other SSSC needed for safe shutdown which should be included in the list,
(c) Group all the components located within or on larger items of equipment (rules of the box),
(d) Evaluate whether the seismic capacity is adequate for the specified RLE.

Each SSSC should be visually examined in the walkdown. After the preliminary screening walkdown, there will be three alternative disposition categories for each SSSC being evaluated during the walkdown, as follows:

(1) Disposition 1: A modification is required.
(2) Disposition 2: The seismic capacity is uncertain and further evaluation is needed to determine whether a modification is required.
(3) Disposition 3: The seismic capacity is adequate for the specified RLE.

The three alternative dispositions are primarily based on judgement and the walkdown teams must be experienced in both seismic analysis and earthquake experience databases in order to make these judgements. If modifications are required, it must be further decided whether the modification falls within the high priority category (Section 8).

The main result of a preliminary screening walkdown is the identification of those obvious seismically robust SSSCs which can be considered as being in disposition category 3 and, therefore, are screened out of further evaluations because they are seismically robust. The preliminary walkdown should be properly documented. In this regard, an example of a screening verification data sheet is provided in Annex I.

The other disposition categories, 1 and 2, require a more detailed walkdown which is performed in a second stage, where a specific evaluation form is completed for each item.

6.1.4. Detailed screening walkdown

After conducting the preliminary screening walkdown, a more detailed walkdown should proceed in a second stage. In this regard it should be pointed out that experience from conducting seismic evaluation has shown that weak links in plants that are ultimately upgraded are often found in a plant detailed screening walkdown performed by qualified engineers according to established procedures and forms.

After such a walkdown, the walkdown engineers will typically have broken the SSSCs into the following two sets:

(1) In the first set, the walkdown engineers evaluate in more detail the system or component not screened out during the first preliminary walkdown. This detailed evaluation usually includes an anchorage calculation and determines whether or not the component needs further analysis or modification.
(2) In the second set, plant modifications are clearly warranted. An example would be an unanchored electrical cabinet. In these cases, the walkdown engineers suggest the modification to be implemented.

There are other plant specific conditions that affect the parameters of the economic decision that the plant will face in such circumstances.

For helping in the detailed walkdown documentation, a specific form referred to as a ‘seismic evaluation work sheet’ (SEWS) is provided in Annex II as an example for a specific class of component. It is also advisable to supplement the documentation by pictures and/or video records.

6.2. INTERACTIONS

6.2.1. Spatial interactions

The walkdown of the plant is the key tool to identify the spatial interactions that can potentially affect the performance of the SSSC during the occurrence of an earthquake and could render this equipment inoperable. These interactions include falling, proximity which could result in the physical impact of the item in question, spray and flood. A major concern in these areas is seismic 'housekeeping'.

The identification and assessment of potential interactions require good judgement from the walkdown team. Only those conditions which truly represent a serious interaction hazard should be identified or will require modification.

6.2.1.1. Falling

Falling is the structural integrity failure of a non-safety related item that can impact with and damage a safety related item. In order for the interaction to be a threat to an SSSC, the impact must release considerable energy and the target must be vulnerable.

A light fixture falling on a 10 cm diameter pipe may not be a credible damaging interaction with a pipe. However, the same light fixture falling on an open relay panel is an interaction which should be remedied. Unreinforced masonry walls will be the most common source of falling interactions. Masonry walls are generally in close enough proximity that their failure could damage the safety related equipment within the enclosure bounded by them. Those cases where failures of these walls have a reasonable probability of damaging safety related equipment and blocking access to critical areas should be identified for upgrading. If the wall is close to electrical, instrumentation or control cabinets, failure of the wall could result in damage to the cabinets and their contents. Conversely, failure of masonry walls that results in an impact on large diameter pipes is not considered to result in a piping failure and does not require an upgrade.

6.2.1.2. Proximity

Proximity interactions are defined as a condition where two items are close enough that their seismic displacements will result in impact. According to feedback experience, the impact of pipes with other pipes is generally not a proximity issue. It is impacts on soft targets such as relay panels, instruments or valve operators that are of the most concern.

6.2.1.3. Spray and flood

Spray and flood can result from failure of piping, systems or vessels which are not properly supported or anchored. Most sources of spray arise from wet fire protection piping systems. Impact and fracture or leakage of sprinkler heads is the most common source of spray. Seismic anchor motion may also fracture small pipes. If spray sources can spray equipment sensitive to water spray, then the source itself should be appropriately backfitted, usually by adding supports to reduce deflections, impacts or stresses. Large tanks which are not properly anchored may be potential flood sources. If such a flood source can fail, the walkdown team, with the assistance of plant personnel, should assess the potential consequences and the capability of the floor drainage system to mitigate the consequences of the source failure.

7. EVALUATION OF SEISMIC MARGIN CAPACITY

The evaluation of the seismic margin capacities should be carried out to:

(a) Screen out from further consideration those SSSCs having capacities generically higher than the RLE,
(b) Identify the SSSCs required for safe shutdown which may require some modification to withstand the RLE.

The final objective of this task is to identify SSSCs which do not have the required seismic capacity. A list of such elements and their degree of non-compliance should be produced as a result of this task.

7.1. PRINCIPLE OF THE EVALUATION OF SEISMIC MARGIN CAPACITY

In practice, studies of seismic evaluation of existing structures that necessitate guidance are the ones for which it is not possible to avoid taking into account some post-elastic behaviour. Then, according to the state of the art, the purpose of the evaluation of seismic margin capacity should be to analyse the strains induced by the postulated RLE in the structure and to compare them with the ultimate strains. Basically this means that approaches orientated towards strain evaluation (displacements approach) are more relevant than those based on stress evaluation (forces approach).

Consistent strain analysis is generally difficult to achieve because engineering practices and engineering tools (e.g. education, standards, criteria and computer codes) are orientated towards stress analysis. For this reason, in order to provide convenient guidance, the rules that follow are expressed in the general framework of stress analysis; in this framework the inelastic energy absorption factor F_μ is introduced (see Annex III for notation and terminology). Nevertheless should a strain based approach be proposed, it should be regarded with interest and carefully examined.

The general criteria for the assessment of the seismic margin capacity are contained in EPRI-NP-6041 [13]. In accordance with the principle of Section 2.1.6, they are more conservative than those which would be permitted in conventional seismic design but more liberal than those in the original NPP design. These values should be reviewed for applicability to the plant under consideration. Criteria for different types of components can be found in DOE/EH-0545 [19].

Estimating the seismic capacity of a system, structure or component requires

— An estimation of the seismic response, conditional on the occurrence of the RLE;
— An assessment of the capacity of the SSSC under consideration, including seismic effects.

These two steps are addressed in this section. A subsection is devoted to inelastic energy absorption factors which are involved in both steps 1 and 2. Seismic capacity and other factors relevant to relays and anchorages are discussed in particular subsections.

The approach recommended may be summed up by the main following steps:

Step 1: Calculate the elastic seismic demand in members and connections by elastic seismic response analysis, using the elastic response spectrum.

Step 2: Calculate the inelastic seismic demand in specific members, taking into account the inelastic energy absorption according the method given in Section 7.3.1.

Step 3: Combine the inelastic seismic demand with the best estimate of concurrent non-seismic demand using unity load factors to determine the total demand according to Section 7.3.1.

Step 4: Estimate seismic capacity by ultimate strength or limit strength provisions according to the method given in Section 7.3.2.

Step 5: Evaluate total demand to capacity ratios for members and connections based on the results of steps 3 and 4. When the ratio values exceed unity, strengthening measures should be considered and properly implemented.

This procedure is valid also for equipment, as specified in Section 7.3.1. The input consists then of floor motions instead of ground motion. It is recommended that the F_{μ} factors of the supporting structure and of the equipment are selected in a consistent way (it would not make sense to upgrade equipment so that it sustains an RLE that would not be sustained by the supporting structure).

The narrative of this section is consistent with the usual earthquake engineering approach. For specific issues, the intent of this section should be met and appropriate analyses should be developed accordingly. Examples of specific issues are: the analysis of ground displacements on buried structures, soil liquefaction assessments and the effects of near field earthquakes.

7.2. RESPONSE ANALYSIS

In computing the response of the structure, the following principles should apply:

(a) A reference model of the structure, including soil–structure interaction effects, should be derived from a best estimate approach, without intentional conservative bias, however.
(b) Parametric studies have to be carried out in order to cover the uncertainties of the model.

The same principles apply in the computation of the response of the equipment.

In order to make possible a comprehensive assessment of the response analysis, details should be provided such as:

- Conditions regarding the use of time history and response spectrum analysis methods;
- Non-linear time history analysis conditions;
- Details of the response spectrum analysis such as response combination rules, directions of excitation and accumulative total modal effective response, and any missing mass corrections;
- Description of simplified methods including equivalent static techniques.

7.2.1. Soil–structure interaction modelling

Simple models (lumped parameters) can be used to assess the potential importance of soil–structure interaction (SSI), but such models should preserve the basic characteristics of the SSI phenomena and any non-symmetric response.

The RLE seismic response analysis, including SSI effects, may be best estimated or median centred. The SSI evaluation and structural modelling may both be median centred with no intentional conservative bias. Median estimates of parameters such as damping may be used (see parametric studies hereunder). If SSI effects are important, careful consideration should be given to induced structure stress levels before increased structure damping or non-linear behaviour is taken into account.

Consistently with a forthcoming IAEA Safety Guide [2], if the shear wave velocity in the foundation soil is higher than 1100 m/s the SSI effects may be neglected.

7.2.2. Structural modelling

The seismic response of building structures should be evaluated on the basis of dynamic analysis of appropriate structural models, taking into account, if needed, SSI effects. In order to develop appropriate structural models, special attention should be given to:

(a) Structural configuration and construction details (joints, gaps, restraints and supports).
(b) Appropriate representation of the geometrical size and arrangement of the foundations of coupled vibrating structures (concrete base mat, strip foundation and individual foundations).
(c) Static and dynamic load paths.
(d) Non-structural elements, such as masonry or precast reinforced concrete panels that may modify the structure response. The stiffness and strength of such panels, and those of their attachments to the structure, should be considered and possibly accounted for in the formulation of the models.
(e) As-built material properties and the dimensions of structural members.
(f) Geotechnical data of foundation materials and their potential implications for the necessity to perform an SSI analysis.
(g) Decoupling criteria for structure and major subsystems and between or within subsystems.

Models for structural analysis should provide sufficient detail commensurate with the complexity in mass and stiffness distribution, including non-symmetric geometry effects and load carrying mechanisms of the structure. The models should be appropriate to the objectives of the analysis, i.e. either:

(1) To determine the seismic response of the structure and the corresponding internal forces in the structural members, or
(2) To calculate the floor response spectra and the seismic displacements in selected locations of the structure for evaluation of the seismic capacity of the components and systems.

Dynamic analysis techniques for determination of seismic response of structures and in-structure response spectra should comply with accepted international practice and standards.

Response analysis should be conducted on the basis of best estimate damping values, as exemplified in Appendix III. The applicability of these values to the plant under consideration should be examined. The basis for the use of higher values should be documented.

7.2.3. Parametric studies and floor response spectra

The variability of soil, structure and component mechanical characteristics has to be taken into account in the analysis by way of parametric studies. In cases in which these characteristics were obtained from tests, the range of parametric studies should be derived accordingly. The number of configurations to be analysed has to be reasonably limited; Sections 7.2.3.1–7.2.3.3 give guidance for an acceptable practice.

7.2.3.1. Soil properties

Variability of soil properties should be taken into account according to the recommendations of a forthcoming IAEA Safety Guide [2]. According to this guide, values of foundation material properties used in the analysis are based on a best estimate, and 0.5 and 2 times the best estimate values, unless site specific soil data indicate that a reduced variability is justified. However, taking into account that varying the foundation material properties is a way to account for uncertainties in the modelling of soil and structures, under no circumstances should the variation in foundation material properties for soil founded structures encompass less than the three following cases: best estimate, and 0.67 and 1.5 times best estimate.

7.2.3.2. Structure properties

Variability of structure dynamic characteristics and the effects on the structure itself have to be reflected by a variation in the natural frequency of the structure. The effects on equipment have to be taken into account through a broadening or a shifting of the floor response spectra consistent with the variation of the natural frequency of the structure. Several cases are possible:

(a) If the analysis of the structure is carried out with $F_{\mu p} = 1$, then the variation of the natural frequency has to be at least from –15 to +15% around the best estimate; the same applies for the broadening or shifting of the floor response spectra.

(b) If the analysis of the structure is carried out with $F_{\mu p} > 1$, then the variation of the natural frequency has to be at least from –30 to +15% around the best estimate; the same applies for the broadening or shifting of the floor response spectra.
(c) In the case of an extensive use of $F_{\mu p}$ factors, the lateral stiffness of the overall structure may be significantly affected. In such cases the generation of in-structure response spectra should consider the reduced best estimated lateral stiffness. The variability should be taken into account by a variation at least from –15 to +15% around the new best estimate.

For the computation of floor response spectra the effects of variation of soil properties and of structural properties should not be cumulated. The parametric study of structural variability associated with the best estimate value of soil stiffness only should be carried out. The consequences of the structural variability may thus be enveloped by the consequences of soil variability, or conversely, on very stiff sites.

The consequences of soil and structure variabilities for floor response spectra can be taken into account either

(a) By enveloping the family of the possible spectra at a given floor or
(b) By conducting a parametric study that covers the entire range of possible spectra.

It should be noted that (a) minimizes the number of configurations to be analysed but may lead to excessively pessimistic results for the equipment while (b) is more complicated but less conservative. Thus (b) is often preferable in an evaluation process.

The seismic capacity of masonry walls is difficult to assess and exhibits some randomness. In the case that a structure contains a significant amount of masonry walls, the necessity of a parametric study, covering different situations, should be examined.

7.2.3.3. Position of cranes

If it is demonstrated that a particular crane would be parked at a particular location more than 98% of the time and if such a requirement would be enforced by a written plant operation procedure, it would be acceptable to perform the seismic evaluation of the building with the crane assumed to be unloaded in its parked position (i.e. without any further parametric study on its position). These criteria may be applicable for evaluation purposes as established in this report.

7.3. CAPACITY EVALUATION

The capacity evaluation of the SSSCs relies on the comparison of the seismic demand with the seismic capacity. For each item of the SSSC list, the seismic demand and the seismic capacity should be evaluated. In cases in which the seismic demand exceeds the seismic capacity some upgrading actions should be implemented.

7.3.1. Seismic demand

The stresses[4] and displacements induced by the RLE can be computed as follows.

For a primary structure (generally the primary structure is a civil structure):

S_p Inertial stresses computed assuming an elastic behaviour of the structure, with ground motion RLE as input.

S'_p S_p reduced by the appropriate $F_{\mu p}$ value of this primary structure, as follows:

$$S'_p = S_p / F_{\mu p}$$

D_p Displacements computed assuming an elastic behaviour of the structure, with ground motion RLE as input.

D'_p D_p amplified by the appropriate $F_{\mu p}$ value of this primary structure (if several $F_{\mu p}$ values are used, the largest value is retained) as follows:

$$D'_p = D_p F_{\mu p}$$

For a secondary structure (e.g. piping systems, components and ducts), two types of input motion have to be provided: floor response spectra and seismic anchor motion; they have to be consistent, i.e. computed with the same model of the primary structure. In most cases, seismic anchor motion has to be taken into account to the extent that it results in non-zero differential displacements.

S_s Inertial stresses computed assuming an elastic behaviour of the secondary structure.

S'_s S_s reduced by the appropriate $F_{\mu s}$ value of this secondary structure, as follows:

$$S'_s = S_s / F_{\mu s}$$

[4] Depending on the circumstances, ‘stresses’ may also mean ‘forces’, ‘moments’ or any similar relevant concept.

S_m Stresses induced by seismic anchor motion computed assuming an elastic behaviour of the secondary structure, with the D'_p motion of the primary structure as the seismic anchor motion.

S'_m S_m modified as follows (primary part of a displacement controlled load):

$$S'_m = S_m / \mu_s$$

In cases in which the displacements in this secondary structure have to be estimated (for instance the displacements of a run pipe are the imposed seismic anchor motions for a connected branch pipe), the following formulas can be used:

D_s Displacements computed assuming an elastic behaviour of the secondary structure, with D'_p as seismic anchor motion.

D'_s D_s amplified by the appropriate $F_{\mu s}$ value of this secondary structure, as follows:

$$D'_s = D_s F_{\mu s}$$

As developed in Annex III, it is also possible to split the F_μ factor into two parts, the global one that is assumed to take into account the non-linear effect on the response of the structure (primary or secondary) as a whole, and the local part that takes into account the non-linear behaviour of individual structural elements.

Alternatively, the S' and D' values could have been obtained by the application of inelastic displacement spectra as a function of allowable μ values, or by any more sophisticated approach such as non-linear transient analysis, provided that the constitutive model is validated and that its use is covered by some sensitivity studies.

Regarding the stresses, the above mentioned rule basically means that the stresses induced by differential displacements may be divided by a factor μ which represents the ductile capacity of the component, while the inertial stresses may be divided by a smaller factor F_μ. The μ and F_μ factors are used to estimate the primary part of the stresses considered, in view of comparison with the allowable stresses.

As opposed to common design practice, and according to feedback experience, the proposed evaluation practice emphasizes the effects of differential displacements rather than the effects of inertial stresses.

(a) According to common design practice, stresses induced in a component by differential displacements are secondary in nature since they are limited by the response of the supporting structure. They are generally not evaluated because the relatively low number of seismically induced cycles (typically less than 50 cycles) is not supposed to result in a failure.

(b) According to the feedback experience, differential displacements are a more frequent cause of failure than inertial stresses. Furthermore, in the framework of the use of inelastic energy absorption factors, differential anchor motions are increased.

In the computation of the seismic demand, the following typical load combination can be used:

$$1.0\mathrm{DL} + 1.0\mathrm{LL}' + 1.0P + 1.0T_0 + 1.0\mathrm{RLE}$$

The following definitions, if relevant for the component under consideration, are valid for the above items:

DL Dead load, including permanently installed equipment.

LL′ Live load applicable on primary structures at the time of occurrence of the RLE, and which is typically assumed to be the 0.25LL used in normal design.

P Pressure, or any other primary load that has to be regarded as acting at the time of occurrence of the RLE.

T_0 Operating temperature and imposed displacement loads (the restraint of free end displacement). The relevance of this load case should be discussed on the basis that only the primary part of the computed stresses has to be considered.

RLE Stresses due to the RLE are the S' stresses introduced at the beginning of this section (7.3.1).

The coefficient 1.0 means that the margins usually regarded as mandatory for design purposes are not required for the purpose of evaluation of an existing facility.

7.3.2. Seismic capacity

The adequacy and the capacity of the foundation material for structures, retaining walls, embankments (such as cooling water channels) and buried components (such as piping and cables) should be evaluated.

Material strengths should be sufficiently conservative that there is only a very low probability that the actual strengths are less than those used in the SMA review. When test data are available, about 95% exceedance probability strengths should be used to achieve this goal. Otherwise, code or design specified strengths should be used. The reviewer should verify that the in situ material properties and components (such as embedments) meet these minimum criteria. In most cases, the use of 95% exceedance probability of actual test data strengths will result in a 5–10% increase over code or design specified minimum strengths.

The allowable stresses (and the load combinations) for structures should be in general the ones associated with extreme environmental abnormal loading (US terminology), the ultimate limit state (European terminology) or an equivalent. More severe allowable stresses or other limits should be required in cases in which the required function resulting from safety analysis is more restrictive than that from structure stability (see the criteria on storey drift).

In general, unreinforced masonry walls are not assumed to provide lateral load resistance (their best estimate stiffness has still to be modelled). When reinforced, they can provide such a lateral resistance which has to be evaluated according to appropriate standards.

For equipment components the allowable stresses (and the load combinations) of ASME Level D or any equivalent national standard may be used for integrity and also for the functionality of piping systems. Operational failure modes may require lesser limits, particularly if they require movement or change of state. If the leak-before-break concept is applicable, its application should be re-evaluated to demonstrate the influence of modifications introduced by the seismic strengthening.

Critical large bore and elevated temperature piping systems should be evaluated by analysis procedures. Alternatively, simplified evaluation procedures particularly applicable to small bore and cold piping systems, including walkdowns, may be used provided they can correlate with acceptable results obtained by analysis.

Methods to demonstrate the operable capacity (covering functional capacity) of systems and equipment include:

(a) Component specific tests or analyses to demonstrate operability during and/or after the RLE and structural integrity. Most commonly, these are design qualification tests.
(b) Earthquake experience data to demonstrate operability after the RLE and structural integrity.
(c) Generic seismic qualification proof or fragility tests to demonstrate operability during and/or after the RLE and structural integrity.

For all methods, an anchorage and spatial interactions evaluation should be performed. For the last two methods, similarity of plant equipment with earthquake experience database equipment should be demonstrated.

For the purpose of this report, it should be assumed that, due to the effects on anchoring systems (Section 7.6), a significant interstorey drift might have consequences for the capacity of a supported system. The drift limit depends on the structure configuration, for example operational or functional failures of systems and components attached to a shear wall should be assumed where the seismically induced lateral interstorey deflection exceeds 0.5% of the element height between floor diaphragms. Structural or integrity failure of these attached systems and components should be assumed where seismically induced lateral deflection exceeds 0.8% in this example.

7.4. INELASTIC ENERGY ABSORPTION FACTOR AND DUCTILE CAPACITY

Nearly all structures and components exhibit at least some ductility (i.e. ability to strain beyond the elastic limit) before failure or even significant damage. Because of the oscillatory nature of earthquake ground motion, this energy absorption is highly beneficial in increasing the seismic margin against failure of structures and components. Ignoring this effect will usually lead to unrealistically low estimations of the seismic safety margin. Limited inelastic behaviour is usually permissible for those facilities with adequate design details such that ductile response is possible or for those facilities with redundant lateral load paths.

This inelastic energy absorption capacity is accounted for in the evaluation approach by specifying the so-called 'inelastic energy absorption factor', F_μ, for each system, structure member or component. These factors express the amount by which the elastically computed seismic demand for the specific system, structure member or component is reduced to determine the inelastic seismic demand. The inelastic seismic demand should be combined with other concurrent loads to determine the total demand on all the elements of the facility. The total demand is then compared with the capacities given by the ultimate strength code or special type provisions including strength reduction factors.

The inelastic energy absorption factor F_μ is defined as a function of the ductility μ (i.e. the ratio of permissible inelastic to yield deformation), as developed in Annex III. This factor is associated with a permissible level of inelastic distortions specified at a failure probability level of approximately 5%. This type of analysis is often expensive and controversial and, therefore, a set of standard values is provided for the most common structural systems.

It should be pointed out that structural safety under seismic loads relies principally on the actual ductile capacity of the structures. This technical finding is emphasized in the case of an evaluation that necessitates the use of inelastic energy absorption factors. The F_μ values proposed in this Safety Report are low and reliable. However, the use of these values should be documented so as to provide evidence that an actual minimal ductile capacity exists and that brittle failure modes are excluded. The following, for instance, should be documented:

- The details of reinforcement in concrete structures;
- The ductile capacity of junctions of steel frames;
- The basic properties of the weldments (i.e. yield strength, ultimate tensile strength and ductility), which have to be at least as good as those for the basic materials;
- The possible radiation embrittlement of vessels or pipes;
- The possible brittle anchorages of components;
- In general all the crucial items in relation to ductile capacity.

As far as possible, elimination of possible non-ductile features should be included in the 'easy-fixes' programme, whatever the results of the analysis and the possible use of a inelastic energy absorption factor may be.

In conjunction with the other guidance of this Safety Report, F_μ values are presented in Appendix III. Larger values may be used, provided they are supported by an appropriate documentation including experimental evidence and analytical background (such as the displacement orientated approach introduced in Section 7.1) and a consistent analysis process is used for estimating the response of the structure or components considered.

As developed in Annex III, F_μ factors in nature depend on the frequency content of both the structure and the input motion. This dependence can be taken into account on the basis of an appropriate documentation such as the one referred in Annex III.

7.5. RELAYS REVIEW

The relays to be evaluated using a two step process should be identified. First, the systems to be examined will be those identified pursuant to Section 5 of this report. Using this approach, the SSSC list should include:

(1) Electrically controlled or powered safe shutdown equipment whose function could be affected by relay malfunction,
(2) Safe shutdown equipment which is not required to change state but for which relay malfunction could cause spurious operation.

Second, drawings of the electrical circuits of the plant associated with the above safe shutdown equipment will be used to identify relays to be evaluated. (For some facilities, a test programme may be needed to test the various types of safety related relays to determine the seismic adequacy.) Certain additional assumptions will be used to establish the scope of the relay review:

(a) Relays will not be physically damaged by an earthquake.

(b) Unqualified relays are assumed to malfunction during the short period of strong motion during an earthquake.
(c) Relay types to be reviewed include auxiliary relays, protective relays, contactors, control switches and other similar contact devices occurring in circuits controlling the systems identified.
(d) Solid state relays and mechanically actuated switches are considered to be seismically rugged and need not be evaluated for contact chatter.

The relays as set forth in the previous section should be evaluated for the consequences of relay malfunction for safe shutdown functions. The relays whose malfunction will not prevent achievement of any safe shutdown function and will not otherwise cause unacceptable spurious actuation of equipment will not be further evaluated. The seismic adequacy of the remaining essential relays will be verified to ensure that safe shutdown can be achieved and maintained in the event of an RLE.

The seismic adequacy of the essential relays identified pursuant to the above requirements should be verified by comparing the relay seismic capacity with the seismic demand imposed upon the relay. Three types of data can be used to establish the seismic capacity of essential relays:

(1) Generic equipment ruggedness spectra (GERS),
(2) Earthquake experience data,
(3) Plant-specific or relay-specific seismic test data.

One or more walkdowns should be conducted, as required, to accomplish the following four objectives:

(1) Obtain information as required to determine in-cabinet amplification, including identification of cabinets, panels and/or racks which house or support essential relays;
(2) Verify the seismic adequacy of the cabinets or enclosures which support essential relays;
(3) Spot check relay mountings;
(4) Spot check relay types and locations.

The relay walkdowns can be accomplished together with, or separate from, the main walkdown.

7.6. ANCHORAGE, SUPPORTS AND NOZZLES

The presence of adequate anchorage is perhaps the most important single item which affects the seismic performance of distribution systems and components.

Strong motion earthquakes have demonstrated that components will slide, overturn, or move excessively when not properly anchored. This is true for large as well as for heavy components. Anchored component failures in the earthquake experience database include expansion, cast in place and grouted in place anchor bolt failures and friction clip failures. It is recommended that equipment anchorage be verified for adequate strength as well as for adequate base stiffness.

The load or demand on the anchorage system can be determined from the floor response spectral acceleration for the prescribed damping value and at the estimated fundamental or dominant frequency of the system or component. A conservative estimate of the spectral acceleration may be taken as the peak of the applicable spectra. This acceleration is then applied to the mass of the component or system at its centre of gravity.

There are various combinations of inspections, limited analyses, tests and engineering judgements which can be used to verify the adequacy of component anchorage. In general, the main process for evaluating the seismic adequacy of equipment anchorage includes the following four steps:

First step:	anchorage installation inspection,
Second step:	anchorage capacity determination,
Third step:	seismic demand determination,
Fourth step:	comparison of capacity with demand.

This process is discussed in detail in Section 4.4 and Appendix C of Ref. [3].

The expansion anchor bolt strength acceptance criteria are governed by the manufacturer's average test failure loads divided by a safety factor depending on the failure mode. This factor should be at least 3.0 for anchorage systems which exhibit ductile failure modes (i.e. tensile failure of the bolt). For anchors which can exhibit non-ductile failure modes (i.e. concrete cone failure) the factor should be increased to at least 4.0.

For anchorage systems, the capacities of which are sensitive to cracking of concrete, the possible cracking under seismic load should be assessed, as well as the resulting capacity of the anchorage systems.

For supports of components and nozzle attachments it is common practice to evaluate seismic anchor motions as primary because the required ductility or flexibility is not self-limiting in the support. The relative or differential motion of the building structure or the main distribution system motion to branch lines at the different points of attachment should be input to a model of the multiply supported component or system. Resultant forces, moments and stresses in the support system determined from the seismic anchor motion effects acting alone should meet the same limits contained in this report for inertia stresses (Section 7.3.1).

Loads on nozzles should be determined in the same manner as loads on supports. In general it is conservative to assume nozzles to be rigid and to provide restraint in all six degrees of freedom (three translations and three rotations) when making such evaluations. In cases where the seismic capacity of the support or nozzle is not established it is possible to introduce their documented flexibility into the analysis.

8. UPGRADING PRINCIPLES

8.1. ITEMS TO BE UPGRADED

A result of the evaluation process is a list of the SSSCs which do not have the required seismic capacity, together with their degree of non-compliance. This information, together with safety and economic considerations, will provide the basis for decision making on the necessity of upgrades. These upgrades should be classified as higher priority and lower priority for implementation.

An important consideration for implementing upgrades is that the structural elements to be upgraded with higher priority should be those which contribute most to the enhancement of the seismic reliability of the safe shutdown path.

Items to be upgraded tend to be a small subset of either the SSSCs as defined by the SMA method or significant contributors to core damage frequency as identified by the SPSA method. How small a subset, is very much a function of the design basis earthquake criteria. Obviously for the plants with little or no anti-seismic original design the list would be larger than one where the original anti-seismic design was more robust.

It should be noted that most SSCs in industrial facilities have significant ability to withstand seismic loads without malfunction or failure even if they were not explicitly anti-seismically designed. This phenomenon is the basis for much of the screening that is performed on individual systems and components which permits them to be effectively screened out from any further consideration for seismic upgrades. It should be understood that because of their uniqueness in design and potential to behave as an inverted pendulum, in response to earthquake motion, structures are often candidates for significant seismic upgrading.

For plants which were not originally anti-seismically designed or for which seismic design considerations played a relatively low part, an easy-fix programme is recommended. In such a programme, plant-wide upgrades are instituted, such as simple positive anchorage of all safety related equipment or minimum lateral bracing being provided for safety related distribution systems, independently of a formal SMA or SPSA programme.

8.2. DESIGN OF MODIFICATIONS

Modifications design should be in accordance with the recognized norms, codes and standards used in the nuclear industry. As a minimum the design of upgrades should satisfy the original design standard.

Upgrading design necessitates as input an earthquake level associated with a set of standards. As a principle, the upgraded design should provide reasonable margins against an evaluation procedure for an existing installation. A possible approach is to adopt the RLE level associated with an evaluation methodology such as that presented in this Safety Report. In any case, according to good engineering practices, the choice of the input level and of the set of standards should be made so as to lead to as homogeneous as possible margins in the installation.

9. QUALITY ASSURANCE AND ORGANIZATION

9.1. ORGANIZATION AND RESPONSIBILITIES

A work plan should be drawn up for the implementation of the seismic safety evaluation programme of a plant, keeping in mind the long term characteristics of such a programme.

It is important for the successful completion of the seismic evaluation programme that for its development there is an organization with a clear responsibility and with the required technical capabilities. It is recommended that nuclear plants establish an engineering group outside the normal operational duties, supervised by a project manager who reports directly to the plant director.

A prioritization scheme, based on an optimal risk reduction principle, may be used to address problems created by limited resources. Owing to funding constraints, the programme may be broken into smaller basic tasks, maintaining the logical technical sequence.

The timing for the execution of the programme is not given in this report. This important aspect should be defined by the regulator in accordance with a general ‘milestone’ schedule for safety upgrades and available resources. If additional non-seismic upgrades must be performed, compatibility between seismic and non-seismic assessments and analyses is recommended.

9.2. DOCUMENTATION

Implementation of the seismic evaluation and upgrading programme has to be carried out under a quality assurance programme (QAP). This should comprise all those planned and systematic actions necessary to provide adequate confidence that SSSCs will perform satisfactorily. Such QAPs should meet the requirements and intent of IAEA Safety Code and Guides Q1–Q14 [20]. The QAP requirements should be defined

— at the start of the seismic evaluation,
— at the start of the upgrading programme.

For convenience, the evaluation process can be split into major tasks, each of which covers several actions; for instance, the following tasks can be identified:

— compilation of the available seismic related information;
— identification of missing information and obtaining it;
— determination of the seismic hazard;
— identification of the SSSCs;
— walkdowns;
— computation of the seismic response of building and structures, including floor response spectra;
— computation of the seismic response of equipment;
— evaluation of the seismic capacity of buildings and structures;
— evaluation of the seismic capacity of equipment;
— identification of possible lack of seismic capacity and of SSSCs to be upgraded.

For each task, a detailed work plan should be prepared, identifying all the relevant actions; each action should be fully documented according to QA procedures. Documentation of the walkdown is required to the level of detail described in Refs [3,13].

Appendix I

EXAMPLE PWR WITH APPLICATION TO WWERs

The methods to achieve the safe shutdown functions are as follows:

(a) Reactivity control
- (i) In the short term: by insertion of the control rods (reactor trip),
- (ii) In the long term: by chemical injection.

(b) Reactor coolant system pressure control
- (i) The pressure is decreased by:
  - — opening the pressurizer relief valves,
  - — shrinkage due to cooling of primary system.
- (ii) The pressure is increased by:
  - — safety injection pump operation,
  - — operation of the pressurizer heaters (normally not required),
  - — accumulator injection (if present).

(c) Reactor coolant system inventory control
- (i) The inventory is decreased by:
  - — opening the pressurizer relief valves,
  - — shrinkage due to the cooling of the primary system.
- (ii) The inventory is increased by:
  - — safety injection pump operation.

(d) Decay heat removal
- (i) By secondary side ‘feed and bleed’ (emergency feedwater plus main steam relief valves discharging to the atmosphere),
- (ii) By the closed loop emergency cooling system of the secondary system,
- (iii) By the alternate method of primary side feed and bleed.

Appendix II

EXAMPLE OF SYSTEM CATEGORIZATION

As an example, in the case of a WWER reactor, the selected fluid systems, electrical power systems and I&C systems can be categorized as follows.

(a) Main process systems:
- (i) Primary coolant system;
- (ii) Make-up water system;
- (iii) Main steam and feedwater systems, including atmospheric relief valves;
- (iv) Feedwater system;
- (v) Primary heat removal system;
- (vi) Control rod drive system;
- (vii) Safety injection system.

(b) Support process systems:
- (i) High pressure sampling system;
- (ii) Essential water system (portions);
- (iii) HVAC system (portions).

(c) Electrical systems:
- (i) Emergency AC power supply, including diesel generators, auxiliaries and distribution systems;
- (ii) Emergency DC power supply, including the distribution system.

Off-site power and power generated by the plant turbine generators are assumed to be unavailable for the time defined by the safety analysis. Therefore, all the equipment required for safe shutdown after an earthquake should be identified and must be fed by an emergency power supply, seismically qualified, from the diesel generators to the components.

(d) Instrumentation and control systems:
- (i) Reactor protection and automatic diesel loading printer, I&C systems required for safe shutdown functions.
- (ii) Monitoring instrumentation — The instrumentation required to measure important parameters of the safety functions and the proper operation of the main and support systems should be identified and listed for seismic evaluation.

(iii) Control rooms — Main control room integrity and operability are required for safe shutdown.

(e) Structures and buildings which house or support safe shutdown main and support process and power systems, and the I&C systems.

Appendix III

DAMPING VALUES AND F_μ VALUES

TABLE III.1. TYPICAL DAMPING VALUES TO BE USED FOR SEISMIC EVALUATION OF EXISTING NPPs
(percentage of critical damping)

ITEMS	With stress levels < yield	With stress levels > yield
Structures		
Reinforced concrete structures	7.0	10.0
Welded steel structures	5.0	7.0
Bolted or riveted steel structures	7.0	10.0
Reinforced masonry walls	7.0	10.0
Unreinforced masonry walls	5.0	7.0
Steel structures with precast panels	7.0	7.0
Systems and components	5.0	5.0
except the following:		
Tank, liquid sloshing modes	0.5	0.5
Cable raceway	10.0	15.0
HVAC duct	7.0	7.0
Vertical pumps	3.0	3.0
Instrument racks	3.0	3.0

Note: Values in the left column apply for SSCs that are not permitted to undergo stress levels beyond the elastic limit under seismic loads.

TABLE III.2. TYPICAL F_μ VALUES FOR THE SEISMIC EVALUATION OF EXISTING NPPs

Concrete columns where flexure dominates	1.25–1.50
Concrete columns where shear dominates	1.00–1.25
Concrete beams where flexure dominates	1.50–1.75
Concrete beams where shear dominates	1.25–1.50
Concrete connections	1.00–1.25
Concrete shear walls	1.50–1.75
Steel columns where flexure dominates	1.25–1.50
Steel columns where shear dominates	1.00–1.25
Steel beams where flexure dominates	1.50–2.00
Steel beams where shear dominates	1.25–1.50
Steel connections	1.00–1.25
Welded steel pipes	1.50–2.00

Note: A range of values is proposed because the choice of the appropriate value should be consistent with national practices (e.g. design practices, quality of construction and severity of control).

REFERENCES

[1] INTERNATIONAL ATOMIC ENERGY AGENCY, Seismic Design and Qualification for Nuclear Power Plants, Safety Standards Series, IAEA, Vienna (in preparation).

[2] INTERNATIONAL ATOMIC ENERGY AGENCY, Geotechnical Aspects of Power Plants Site Evaluation and Foundations, Safety Standards Series, IAEA, Vienna (in preparation).

[3] SEISMIC QUALIFICATION UTILITIES GROUP, Generic Implementation Procedure for Seismic Verification of Nuclear Power Plant Equipment, Rev. 2, Office of Standards Development, Washington, DC (1992).

[4] INTERNATIONAL ATOMIC ENERGY AGENCY, Evaluation of Seismic Hazards for Nuclear Power Plants, Safety Standards Series No. NS-G-3.3, IAEA, Vienna (2002).

[5] INTERNATIONAL ATOMIC ENERGY AGENCY, Probabilistic Safety Assessment for Seismic Events, IAEA-TECDOC-724, Vienna (1993).

[6] US NUCLEAR REGULATORY COMMISSION, Identification and Characterization of Seismic Sources and Determination of Safe Shutdown Earthquake Ground Motion, Regulatory Guide 1.165, Office of Standards Development, Washingon, DC (1997).

[7] US NUCLEAR REGULATORY COMMISSION, Procedural and Submittal Guidance for the Individual Plant Examination of External Events (IPEEE) for Severe Accident Vulnerabilities, Rep. NUREG-1407, Office of Standards Development, Washington, DC (1991).

[8] US NUCLEAR REGULATORY COMMISSION, Probabilistic Seismic Hazard Analysis: Guidance on Uncertainty and Use of Experts, Vols 1 and 2, Rep. NUREG/CR-6372, Office of Standards Development, Washington, DC (1994).

[9] BERNREUTER, D.L., SAVY, J.B., MENSING, R.W., CHEN, J.C., Seismic Hazard Characterization of 69 Nuclear Plant Sites East of the Rocky Mountains — Results and Discussion for the Batch 2 Sites, Rep. NUREG/CR-5250, UCID-21517, Vol. 3, Office of Standards Development, Washington, DC (1989).

[10] ELECTRIC POWER RESEARCH INSTITUTE, Probabilistic Seismic Hazard Evaluation at Nuclear Plant Sites in the Central and Eastern United States: Resolution of the Charleston Issue, Rep. EPRI-NP-6395-D, Palo Alto, CA (1989).

[11] US NUCLEAR REGULATORY COMMISSION, Development of Criteria for Seismic Review of Selected Nuclear Power Plants, Rep. NUREG/CR-0098, Office of Standards Development, Washington, DC (1978).

[12] US NUCLEAR REGULATORY COMMISSION, An Approach to the Quantification of Seismic Margins in NPPs, Rep. NUREG/CR-4334, Office of Standards Development, Washington, DC (1985).

[13] ELECTRIC POWER RESEARCH INSTITUTE, A Methodology for Assessment of Nuclear Power Plant Seismic Margin, Rep. EPRI-NP-6041, Palo Alto, CA (1988).

[14] US NUCLEAR REGULATORY COMMISSION, PRA Procedures Guide, Rep. NUREG/CR-2300, Office of Standards Development, Washington, DC (1983) Ch. 11.

[15] US NUCLEAR REGULATORY COMMISSION, Seismic Fragility of Nuclear Power Plant Components, Rep. NUREG/CR-4659, Vols 1, 2 and 3, Office of Standards Development, Washington, DC (1986/1987/1990).

[16] US NUCLEAR REGULATORY COMMISSION, An Approach to the Quantification of Seismic Margins in NPPs: The Importance of BWR Plant Systems and Functions to Seismic Margins, Rep. NUREG/CR-5076, Office of Standards Development, Washington, DC (1988).
[17] US NUCLEAR REGULATORY COMMISSION, Assessment of the Seismic Margin Calculation Methods, Rep. NUREG/CR-5270, Office of Standards Development, Washington, DC (1989).
[18] US NUCLEAR REGULATORY COMMISSION, Report of the US NRC Piping Review Committee, Rep. NUREG-1061, Office of Standards Development, Washington, DC (1985).
[19] UNITED STATES DEPARTMENT OF ENERGY, Seismic Evaluation Procedure for Equipment in US Department of Energy Facilities, Rep. DOE/EH-0545, Washington, DC (1997).
[20] INTERNATIONAL ATOMIC ENERGY AGENCY, Quality Assurance for Safety in Nuclear Power Plants and other Nuclear Installations, Code and Safety Guides Q1–Q14, Safety Standards Series No. 50-C/SG-Q, IAEA, Vienna (1996).

ABBREVIATIONS AND ACRONYMS

ALARA	as low as reasonably achievable
CAV	cumulative absolute velocity
EPRI	Electric Power Research Institute
GERS	generic equipment ruggedness spectra
GIP	generic implementation procedure
HCLPF	high confidence of low probability of failure
HELB	high energy line breaks
HVAC	heat, ventilation and air conditioning
I&C	instrumentation and control
LOCA	loss of coolant accident
PGA	peak ground acceleration
PSHA	probabilistic seismic hazard analysis
QAP	quality assurance programme
RLE	review level earthquake
SEWS	seismic evaluation work sheet
SMA	seismic margin assessment
SPSA	seismic probabilistic safety assessment
SQUG	seismic qualifications utility group
SSC	structures, systems and components
SSI	soil–structure interaction
SSRAP	Senior Seismic Review and Advisory Panel
SSSC	selected structures, systems and components

Annex I

EXAMPLE OF SCREENING VERIFICATION DATA SHEET (from Ref. [19])

Sheet 1 of _____

Equip. class	Equip. ID No.	System/equipment description	Bldg	Floor elev.	Room or row/col.	Base elev.	HCLPF G	Demand G	Capacity > demand?	Caveats OK?	Anchorage OK?	Interactions OK?	Equipment status	Notes

Notes: Enter applicable notation: Y = Yes; N = No; U = Unknown; NA = Not applicable

Status categories:
1 = A physical modification is required.
2 = The seismic capacity is uncertain and further evaluation is required.
3 = Structure, system and component seismic capacity are adequate.

SIGNATURES:

All the information contained on this Screening Verification Data Sheet is, to the best of our knowledge and belief, correct and accurate. 'All information' includes each entry and conclusion (whether evaluated to be seismically adequate or not).

Approved: All Seismic Capability Engineers on the Seismic Review Team should sign.

____________________ Print or Type Name

____________________ Signature

____________________ Date

Annex II

EXAMPLE OF SEISMIC EVALUATION WORK SHEET (FOR HORIZONTAL PUMPS) (from Ref. [3])

SEISMIC EVALUATION WORK SHEET (SEWS) Sheet 1 of 2

Equip. ID No. ______ Equip. Class 5 — Horizontal pumps

Equipment description ____

Location: Bldg ________ Floor El. ________ Room, Row/col. ___________

Manufacturer, model, etc. (if known) ____________________________________

Horsepower/motor rating (if known) _____ rev./min _____ Head _____ Flow rate ____

A. Seismic capacity versus demand

1.	Elevation where equipment receives seismic input _____		
2.	Elevation of seismic input below about 13.0 metres from ground		Y N U N/A
3.	Equipment has fundamental frequency above about 8 Hz		Y N U N/A
4.	Capacity based on:	existing documentation	DOC
		bounding spectrum	BS
		1.5 × bounding spectrum	ABS
5.	Demand based on:	ground response spectrum	GRS
		1.5 × ground response spectrum	AGS
		in-structure response spectrum	ISRS
	Does capacity exceed demand?		Y N U

B. Caveats — bounding spectrum (Identify with a numbered note in the margin those caveats which are met by intent without meeting the specific wording of the caveat rule and explain the reason for this conclusion in the comments section.)

1.	Equipment is included in earthquake experience equipment class	Y N U N/A
2.	Driver and pump connected by rigid base or skid	Y N U N/A
3.	No indication that shaft does not have thrust restraint in both axial directions	Y N U N/A
4.	No risk of excessive nozzle loads such as gross pipe motions or differential displacement	Y N U N/A
5.	Base vibration isolators adequate for seismic loads	Y N U N/A
6.	Attached lines (cooling, air, electrical) have adequate flexibility	Y N U N/A
7.	Relays mounted on equipment evaluated	Y N U N/A
8.	Have you looked for and found no other adverse concerns?	Y N U N/A
	Is the intent of all the caveats met for the bounding spectrum?	Y N U N/A

C. Anchorage

1. Appropriate equipment characteristics determined (mass, centre of gravity, natural freq., damping and centre of rotation)	Y N U N/A
2. Type of anchorage covered by experience	Y N U N/A
3. Sizes and locations of anchors determined	Y N U N/A
4. Visual inspection that the anchorage installation in place is adequate (e.g. weld quality and length, nuts and washers, and anchor bolt installation), no significant erosion or corrosion	Y N U N/A
5. Factors affecting anchor bolt capacity or margin of safety: embedment length, anchor spacing, free edge distance, concrete strength/condition, concrete cracking and gap under base less than 6.0 mm	Y N U N/A
6. Factors affecting motion sensitive devices (relays, switches, etc.) considered: gap under base, capacity reduction for expansion anchors	Y N U N/A
7. Base has adequate stiffness or effect of prying action on anchors considered	Y N U N/A
8. Strength of equipment base and load path to centre of gravity of component	Y N U N/A
9. Embedded steel, grout pad or large concrete pad adequacy evaluated	Y N U N/A
Are anchorage requirements met?	Y N U N/A

D. Interaction effects

1. Soft targets free from impact by nearby equipment or structures	Y N U N/A
2. If equipment contains motion sensitive devices, equipment is free from all impact by nearby equipment or structures	Y N U N/A
3. Attached lines have adequate flexibility	Y N U N/A
4. Overhead equipment or distribution systems are not likely to collapse	Y N U N/A
5. Have you looked for and found no other adverse concerns?	Y N U N/A
Is equipment free of interaction effects?	Y N U
IS EQUIPMENT SEISMICALLY ADEQUATE?	Y N U

COMMENTS

Attach any applicable photos, sketches, drawings and calculations. If there are any suggested improvements they can be described on the back of this sheet.

Evaluated by: ______________________________ Date: ____________________

Annex III

SCIENTIFIC BACKGROUND, NOTATION AND TERMINOLOGY FOR THE F_μ FACTORS

For the sake of clarity, the μ and F_μ factors are introduced here for the example of an elastic, perfectly plastic single degree of freedom (SDOF) oscillator.

III–1. DUCTILE CAPACITY AND DUCTILE DEMAND

III–1.1. Ductility or ductile capacity

The elastic, perfectly plastic constitutive relationship is shown in Fig. III–1. Here ε_e is the elastic yield strain of the material while ε_u is its ultimate strain (or rupture limit); ε_{ad} is the admissible strain for the purpose of safety assessment. In practice, ε_u is a random parameter. For the purpose of this Safety Report, it is recommended to choose ε_{ad} so that

$$\text{Probability}(\varepsilon_u < \varepsilon_{ad}) < 5\%$$

According to classical definitions, the ductile capacity or ductility μ is defined by $\mu = \varepsilon_u/\varepsilon_e$. For the purpose of the present Safety Report, the ductility μ is defined as

$$\mu = \varepsilon_{ad}/\varepsilon_e$$

III–1.2. Ductile demand and margin under seismic input

An oscillator made of the above described material experiences a strain history $\varepsilon(t)$ when subjected to the accelerogram $\gamma(t)$ as seismic input. The maximum of

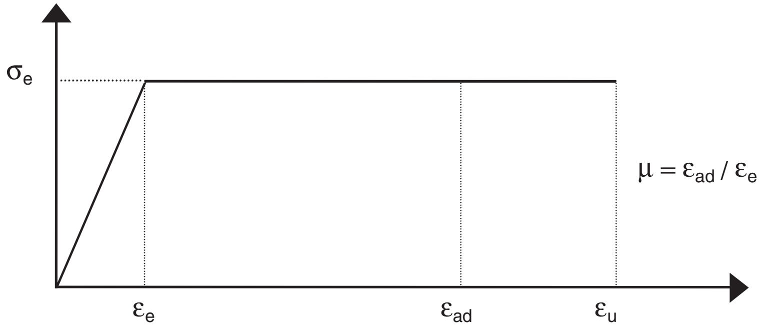

FIG. III–1. The elastic, perfectly plastic relationship.

the absolute value of $\varepsilon(t)$ is ε_{max}. The ratio $\varepsilon_{max}/\varepsilon_e$ is the ductile demand. Let us denote by $\gamma_e(t)$ a $\gamma(t)$ calibrated so that:

$\gamma_e(t)$ as an input results in $\varepsilon_{max}(t) = \varepsilon_e$

The available margin for the oscillator under consideration is λ, so that the ductile demand equals the ductile capacity:

$\gamma_{ad}(t) = \lambda\gamma_e(t)$ as an input results in $\varepsilon_{max}(t) = \varepsilon_{ad}$

λ is a function of μ and the dynamic features of the oscillator. In addition, λ depends on the accelerogram, so there is no mathematical form that covers all possible cases. The computation of λ values was the basis for the development of the inelastic response spectrum [III–1].

III–2. STRESS CLASSIFICATION AND DESIGN CRITERIA

III–2.1. Primary and secondary stresses or loads

The classical framework for the engineering approach is (i) to assume structures with an elastic behaviour, (ii) on this basis to compute stresses in the structures and (iii) to compare these stresses with the admissible stresses. In this approach, the stresses induced by force controlled loads are addressed in a different way than stresses induced by displacement controlled loads, as exemplified below.

We consider two identical straight rods (Fig. III–2) made of the material introduced in Section III–1.1. Rod 1 is subject to a force controlled load, while rod 2 is subject to a displacement controlled load. *L* and *S* are the length and section of the rods; *E* is the elastic modulus of the material.

We denote by $\tilde{\sigma}_1$ and $\tilde{\sigma}_2$ the stresses calculated in each rod under the elastic assumption:

$\tilde{\sigma}_1 = F/S, \tilde{\sigma}_2 = ED/L$

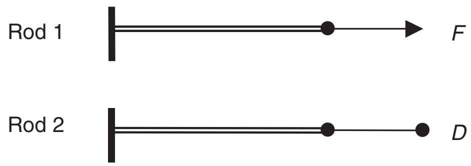

FIG. III–2.

With a safety factor k, F and D are admissible to the extent the corresponding stresses are such that:

$$\tilde{\sigma}_1 < \sigma_e/k \tag{III–1a}$$
$$\tilde{\sigma}_2 < \mu\sigma_e/k \tag{III–1b}$$

In the classical approach, $\tilde{\sigma}_1$ is a primary stress while $\tilde{\sigma}_2$ is a secondary stress or, in other words, F is a primary load while D is a secondary load. It appears clear that, owing to ductile capacity, the criteria on secondary stresses may be far less restricting. Even codes dealing with very ductile materials (design rules for mechanical equipment) do not use this type of criteria for displacement controlled loads.

III–2.2. Primary ratio of a seismic load

A third identical rod (Fig. III–3) is subjected to a seismic type input. We denote by $\tilde{\sigma}_3$ the maximum of the stresses calculated assuming an elastic constitutive law.

According to Section III–1.2, a seismic input is acceptable to the extent the following criterion is satisfied:

$$\tilde{\sigma}_3 < \lambda\sigma_e/k \tag{III–1c}$$

The three above mentioned criteria may be rewritten in the following form:

$$\tilde{\sigma}_1 < \sigma_e/k \tag{III–2a}$$
$$\tilde{\sigma}_2/\mu < \sigma_e/k \tag{III–2b}$$
$$\tilde{\sigma}_3/\lambda < \sigma_e/k \tag{III–2c}$$

This means that in any case the calculated stress is compared with the admissible stress under a primary load. The μ and λ factors account for the fact that D and $\gamma(t)$ are not primary loads. It may be said that $\tilde{\sigma}_2/\mu$ is the primary part of $\tilde{\sigma}_2$ and that $\tilde{\sigma}_3/\lambda$ is the primary part of $\tilde{\sigma}_3$, i.e. $1/\mu$ and $1/\lambda$ are the primary ratios of $\tilde{\sigma}_2$ and $\tilde{\sigma}_3$, or of D and $\gamma(t)$.

These primary parts of the displacement induced stresses, thermally induced stresses, seismically induced stresses and of any other stresses are relevant to Section 7.3.1 of this Safety Report.

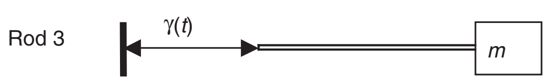

FIG. III–3. A rod subjected to a seismic type input.

III–3. F_μ FACTORS

III–3.1. Relations between μ and F_μ

The purpose of the F_μ factors is to avoid time history analyses and to provide inclusive values of λ.

In the early developments of the inelastic response spectrum [III–1], the following values were proposed:

$F_\mu = \mu$	if the dominant natural frequency is less than 2 Hz
$F_\mu = (2\mu - 1)^{1/2}$	if the dominant natural frequency is between 2 and 8 Hz
$F_\mu = 1$	if the dominant frequency is above 33 Hz

and F_μ has a linear transition between 8 and 33 Hz.

The following points about the limit values of F_μ should be noted:

(a) $F_\mu = \mu$ corresponds to the fact that the seismic input is similar to a displacement controlled load for flexible structures.

(b) $F_\mu = 1$ corresponds to the fact that the seismic input is similar to a force controlled load for stiff structures.

A further step is to simplify the approach by introducing a non-frequency-dependent F_μ factor. This is what is proposed in this Safety Report. However, frequency dependent F_μ factors are permitted.

III–3.2. Further developments

III–3.2.1. Frequency dependence

Further developments [III–2] have shown that, instead of the dominant natural frequency of the structure, the frequency dependence is better controlled by the following *r* factor:

$$r = \frac{\text{fundamental frequency of the structure or component to be analysed}}{\text{central frequency of the input motion}}$$

The definition of the central frequency of the input motion can be found in Ref. [III–3].

III–3.2.2. Case of piping systems

Design criteria for piping systems have been under discussion for more than a decade. Focusing on seismically induced stresses, a typical criterion is

$$\tilde{\sigma}_p + \tau_i \tilde{\sigma}_i + \tau_d \tilde{\sigma}_d < \tilde{\sigma}_{ad}$$

where

- $\tilde{\sigma}_p$ Stress due to pressure and other permanent loads to be considered,
- $\tilde{\sigma}_i$ Stress due to seismic inertial effects,
- $\tilde{\sigma}_d$ Stress induced by seismic differential displacements,
- $\tilde{\sigma}_{ad}$ Admissible stress,
- τ_i Primary ratio of inertial stresses,
- τ_d Primary ratio of stresses induced by differential displacements.

For the current criteria $\tau_i = 1$ and $\tau_d = 0$. The approach proposed in the present Safety Report corresponds to $\tau_i = 1/(2\mu - 1)^{1/2}$ (possibly also frequency dependent) and $\tau_d = 1/\mu$.

When new design criteria for piping systems are adopted, and if they are relevant for the design of the piping system under consideration, they should be used in the context of the evaluation of an existing installation in the spirit of Section 2.1.6 and the F_μ factors should be selected accordingly.

III–4. GLOBAL EFFECTS OF PLASTIC DRIFTS ON A STRUCTURE

III–4.1. Global and local F_μ factors

In an ideal transient non-linear analysis of the response of a structure that is subject to strains beyond the elastic limit, the effects of local plastic drifts are automatically accounted for in computing the dynamic response of the structure.

In the framework of the engineering approach proposed in this Safety Report, such analyses of transients should generally not be carried out. The analysis of a structure that undergoes strains beyond the elastic limit could be carried out in a two step procedure:

(1) Globally, the non-linear effects on the response of the structure as a whole should be assessed and the corresponding displacement field in the structure computed.

(2) Locally, it should be verified for each element of the structure that the imposed displacements on it are acceptable according to its ductility.

The assumptions of step 1 should be consistent with the extension and the magnitude of plastic strains calculated in step 2.

Such an approach is consistent with the principles introduced in Section 7.1. However, as also mentioned in the first paragraph of that section, step 2 of this approach is difficult because engineering tools (design criteria) generally do not exist for the assessment of displacement controlled loads.

A possible way to cope with this situation in the framework of the existing engineering criteria is the following:

(1) In the first step the global effect on the structure is reflected by the use of a global factor, denoted by $F_{\mu g}$. In particular, $F_{\mu g}$ is intended to be used in the evaluation of the displacements in the structure.

(2) In the second step the reduction factor on the elements of a structure is then the product of this global factor $F_{\mu g}$ by a local factor $F_{\mu 1}$ so that:

$$F_{\mu} = F_{\mu g} F_{\mu l}$$

$F_{\mu g}$ should be larger than 1 (except when all the F_{μ} are equal to 1). Its value should be consistent with the magnitude and the extension of the estimated post-elastic deformations in the structure and selected with a reasonable margin so as to avoid any underestimate of the displacements.

III–4.2. Displacements

As an application of the global factor introduced above, the formula proposed in Section 7.3.1 for the assessment of the displacements in a primary structure becomes:

$$D'_{p} = D_{p} F_{\mu pg}$$

If a global factor is not identified, the displacements are assessed with the following formula introduced in the same paragraph:

$$D'_{p} = D_{p} F_{\mu p}$$

For reasons of kinematical continuity, it is not possible to use several $F_{\mu p}$ values in the assessment of D'. Therefore, if several $F_{\mu p}$ values are available, the largest value should be retained for the assessment of D', as specified in Section 7.3.1.

It is recognized that, according to the classical assumption in earthquake engineering, the displacements associated with a non-linear behaviour are very similar to those associated with an elastic behaviour. Consequently, as compared with the classical assumption, the above formulas may lead to an overestimate of the displacements. This is intentional and consistent with the philosophy of the present Safety Report, which seeks to learn lessons from experience feedback. According to the seismic experience feedback, damage or failure is often the consequence of an underestimate of displacements.

REFERENCES TO ANNEX III

[III–1] US NUCLEAR REGULATORY COMMISSION, Development of Criteria for Seismic Review of Selected Nuclear Power Plants, Rep. NUREG/CR-0098, Office of Standards Development, Washington, DC (1978).

[III–2] LABBÉ, P., NOÉ, H., “Ductility and seismic design criteria”, Earthquake Engineering (Proc. 10th World Conf. Madrid, 1992), Brookfield, Rotterdam (1992).

[III–3] PREUMONT, A., Random Vibrations and Spectral Analysis, Presses Polytechniques et Universitaires Romandes, Lausanne (1991) (in French) [English translation: Kluwer Academic Publishers, Dordrecht (1994)].

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Technical Committee Meeting

Vienna, Austria: 3–7 December 2001