Safety Reports Series

No. 103

Methodologies for Seismic Safety Evaluation of Existing Nuclear Installations

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SAFETY REPORTS SERIES No. 103

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INTERNATIONAL ATOMIC ENERGY AGENCY
VIENNA, 2020

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tel.: +43 1 2600 22417
email: sales.publications@iaea.org
www.iaea.org/publications

Printed by the IAEA in Austria
August 2020
STI/PUB/1893

IAEA Library Cataloguing in Publication Data

Names: International Atomic Energy Agency.
Title: Methodologies for seismic safety evaluation of existing nuclear installations / International Atomic Energy Agency.
Description: Vienna : International Atomic Energy Agency, 2020. | Series: IAEA safety reports series, ISSN 1020–6450 ; no. 103 | Includes bibliographical references.
Identifiers: IAEAL 20-01334 | ISBN 978–92–0–107219–1 (paperback : alk. paper) | ISBN 978–92–0–105320–6 (pdf)
Subjects: LCSH: Nuclear power plants — Safety measures. | Nuclear power plants — Earthquake effects. | Earthquake hazard analysis | Nuclear facilities.
Classification: UDC 621.039.58: 550.34 | STI/PUB/1893

FOREWORD

This publication provides guidance on the implementation of IAEA Safety Standard Series No. NS-G-2.13, Evaluation of Seismic Safety for Existing Nuclear Installations, and explores methodologies validated by international practices. Seismic evaluation programmes have been conducted for a number of nuclear installations worldwide. One such example is the seismic re-evaluation of nuclear power plants in Eastern Europe. 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. This publication also includes lessons identified based on the IAEA Action Plan on Nuclear Safety, in response to the accident at the Fukushima Daiichi nuclear power plant triggered by the Great East Japan Earthquake, which occurred on 11 March 2011.

The IAEA is grateful to all those who contributed to the drafting and review of this publication, in particular the contributions of J.J. Johnson and J.D. Stevenson (United States of America). The IAEA officer responsible for this publication was O. Coman of the Division of Nuclear Installation Safety.

CONTENTS

1. INTRODUCTION

1.1. BACKGROUND

This Safety Report complements the guidance in IAEA Safety Standards Series No. NS-G-2.13, Evaluation of Seismic Safety for Existing Nuclear Installations [1], based on the IAEA experience feedback and involvement in seismic re-evaluations of nuclear power plants in Eastern Europe and lessons identified from the 2011 Fukushima Daiichi nuclear power plant accident.

Worldwide experience shows that an assessment of the seismic capacity of an existing operating facility can be necessary for a number of reasons, including the evidence of a greater seismic hazard at the site than expected before or a poor anti-seismic design in all or in part of the facility.

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 and is operating or at the end of the design stage.

Seismic evaluation of existing nuclear installations differs from the practices applicable to the design of nuclear installations. The most prominent among these differences are:

(a) Plant condition: Paragraph 1.6 of NS-G-2.13 [1] states that "the seismic safety evaluation of existing installations strongly depends on the actual condition of the installation at the time the assessment is performed. This key condition is denoted the 'as-is' condition, indicating that an earthquake, when it occurs, affects the installation in its actual condition, and the response and capacity of the installation will depend on its actual physical and operating configuration. The as-is condition of the installation is the baseline for any seismic safety evaluation programme. The as-is condition includes the 'as-built', 'as-operated' and 'as-maintained' conditions of the installation, and its condition of ageing at the time of the assessment."
(b) Evaluation criteria: The criteria used in the evaluation are different from those used in the design. Design tends to use the applicable loads to size the SSC to meet the limits set in the design code, while in evaluation the aim is to establish the capacity of the SSC in the 'as-is' condition and use it in the overall seismic evaluation of the installation. In doing this, experience from exposure to past seismic events, testing and analytical estimates of

capacity are all utilized as sources of information. Thus, the process uses a significant level of expert judgement. The role played by the feedback of such experience, the associated practice of plant walkdowns and the qualification by experts are part of the evaluation methodologies discussed in this publication.

(c) Safety evaluation applicability: Seismic safety evaluations are used to assess the capacities of an installation when subject to beyond design basis seismic events. Seismic safety evaluation methodologies include probabilistic, deterministic and a combination of deterministic and probabilistic approaches.

A significant number of existing nuclear installations worldwide have undergone a seismic safety evaluation since the 1990s. Consequently, there is sufficient background experience supporting the seismic safety assessment methodologies presented in NS-G-2.13 [1] and detailed here. Seismic evaluation in the context of identification of vulnerabilities of nuclear power plants against external hazards is also addressed in Ref. [2].

1.2. OBJECTIVE

This publication provides detailed guidance on conducting seismic safety evaluation programmes for existing nuclear installations in a manner consistent with NS-G-2.13 [1]. This publication can be used as a tool by regulatory organizations or other organizations responsible for the conduct of a seismic safety evaluation programme and provides a clear definition of the following:

— The objectives of the seismic evaluation programme;
— The phases, tasks and priorities in accordance with specific plant conditions;
— The common and integrated technical framework for establishing the acceptance criteria and its use in the seismic safety evaluation process.

1.3. SCOPE

The scope of this publication covers the seismic safety evaluation programmes to be performed on existing nuclear installations in order to ensure that the required fundamental safety functions are available, with particular application to the safe shutdown of reactors. Nuclear installations include: (i) land based, stationary nuclear power plants and research reactors; and (ii) nuclear fuel cycle facilities, including enrichment plants, processing plants, independent spent fuel storage facilities and reprocessing plants.

Seismic safety evaluation programmes need to be developed as described in NS-G-2.13 [1] and in accordance with applicable regulatory requirements, such as the requirement for periodic safety review (see IAEA Safety Standards Series No. NS-G-1.6, Seismic Design and Qualification for Nuclear Power Plants [3]). Evaluation programmes at existing operating plants are plant specific or regulatory specific. Although this publication defines the minimum generic specifications for such a programme, they may need to be supplemented on a plant specific basis to consider particular aspects of the original design basis.

The purpose of a seismic evaluation is to identify margins encompassed in the original design of an installation, to take advantage of them in an updated seismic safety assessment of the installation, and if and where necessary to determine upgrading actions to obtain the desired safety level. Original design margins may result from enveloping design assumptions from the design acceptance criteria and from good practices such as diversity and redundancy of components, or it may have other origins.

Among the options available for assessing the seismic safety of existing facilities, the two methods particularly appropriate are the seismic margin assessment (SMA) method and the seismic probabilistic safety assessment (SPSA) method. Both methods are discussed in addition to a combination of SMA and SPSA called PSA based SMA, which can identify safety significant accident sequences and provide input for the improvement of severe accident mitigation measures with respect to seismic shaking motion hazard. PSA based SMA uses PSA techniques to generate accident sequences and, on this basis, it quantifies the seismic margin at the system level and at the plant level. It can be used to quantify the safety improvement following the implemented emergency measures (e.g. additional mobile power, cooling supply) and is less resource intensive than SPSA.

This publication also explores the use of a graded approach for nuclear installations other than nuclear power plants. Evaluation of existing nuclear installations may result in the identification of the structures, systems and components (SSCs) that have to be upgraded; the aspects of which are discussed here. Guidance provided here, describing good practices, represents expert opinion but does not constitute recommendations made on the basis of a consensus of Member States.

1.4. STRUCTURE

Section 2 presents the general philosophy of seismic evaluation, including the need for seismic safety evaluation, the formulation of the programme and considerations concerning selection of the appropriate seismic safety evaluation methodology. Section 3 explores in detail the data collection process and the investigations needed to support the seismic safety evaluation programme. This task is resource intensive and the quality of the collected data has a direct impact on the quality of the outcome of the seismic safety evaluation process. Section 4 outlines the seismic hazard assessment and the development of the review level earthquake (RLE), which constitute the seismic input for the seismic safety evaluation. Section 5 discusses the safety analysis of the nuclear installation and presents SMA, SPSA and PSA based SMA, and Section 6 discusses the application of seismic safety evaluation methodologies for medium and low hazard nuclear facilities (e.g. nuclear installations other than reactor, spent fuel facilities at nuclear power plants). Section 7 addresses the consideration of upgrading those SSCs found to be seismic vulnerabilities using the methodologies described and Section 8 describes a quality management system applicable to the evaluations.

Appendix I includes examples of seismic design categories and site foundation classifications. Appendix II provides suggested damping values, inelastic energy absorption factors and story drift limits for use in the seismic safety evaluation of existing nuclear installations. Appendix III provides the scientific background, notation and terminology for the inelastic energy absorption factors presented in this publication. Appendix IV describes the hybrid method for simplified fragility analysis and Appendix V provides the typical forms used to document seismic capability walkdowns.

2. FORMULATION OF THE PROGRAMME FOR SEISMIC SAFETY EVALUATION OF EXISTING NUCLEAR INSTALLATIONS

2.1. PERSPECTIVE

2.1.1. Existing nuclear installations

Paragraph 1.10 of NS-G-2.13 [1] applies to the seismic safety evaluation of existing installations and states:

"...existing nuclear installations are those installations that are either (a) at the operational stage (including long term operation and extended temporary shutdown periods) or (b) at a pre-operational stage for which the construction of structures, manufacturing, installation and/or assembly of components and systems, and commissioning activities are significantly advanced or fully completed."

2.1.2. Reasons to perform seismic safety evaluations

There are many reasons to perform seismic safety evaluations of existing nuclear installations. As stated in para. 2.10 of NS-G-2.13 [1], States may require an evaluation of the seismic safety in the event of any one of the following:

"(a) Evidence of a seismic hazard at the site that is greater than the design basis earthquake, arising from new or additional data (e.g. newly discovered seismogenic structures, newly installed seismological networks or new palaeoseismological evidence), new methods of seismic hazard assessment, and/or the occurrence of actual earthquakes that affect the installation;
(b) Regulatory requirements, such as the requirement for periodic safety reviews, that take into account the 'state of knowledge' and the actual condition of the installation;
(c) Inadequate seismic design[1],generally due to the vintage of the facility;
(d) New technical findings, such as vulnerability of selected structures and/or non-structural elements (e.g. masonry walls), and/or of systems or components (e.g. relays);
(e) New experience from the occurrence of actual earthquakes (e.g. better recorded ground motion data and the observed performance of SSCs);
(f) The need to address the performance of the installation for beyond design basis earthquake ground motions in order to provide confidence that there is no 'cliff edge effect'[2], that is, to demonstrate that no significant failures would occur in the installation if an earthquake were to occur that was slightly greater than the design basis earthquake…;
(g) A programme of long term operation [or plant life extension] of which such [a seismic safety] evaluation is a part."

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[1] Due to the perception of the seismic hazard being significantly higher than the original design level, or due to evolution of the seismic design methodologies.

[2] A cliff edge effect refers to the phenomenon of significant SSC failures occurring when earthquake ground motions slightly greater than the DBE occur at the site.

2.1.3. Margins beyond the design basis for new nuclear installations

NS-G-1.6 [3] applies to the seismic analysis and design of new nuclear power plants. Paragraphs 2.39–2.41 of NS-G-1.6 [3] address the need to provide margins for events beyond the design basis. When necessary, the methodologies for evaluation of existing nuclear installations are being implemented in the design stage for new nuclear power plants to ensure that seismic margins are present in the design. For standard designs of Generation III (or III+) nuclear installations, a beyond design basis earthquake (DBE) event is often considered. Such consideration is in the design phase for the standard design and for site specific SSCs. In addition to being designed for a DBE ground motion, there is a need to demonstrate capacity to withstand a beyond DBE ground motion with high confidence that a nuclear safety related failure will not occur. The level of the beyond DBE ground motion to be considered is established through applicable requirements. The seismic margin capacity is established during the design process and later verified for site specific seismic hazards and for SSCs designed for site specific conditions. In performing these evaluations of beyond DBEs, acceptance criteria different from those used in the design process may be used. For example, more realistic material properties rather than specified minimum values and an inelastic energy absorption and ductility factor may provide failure margins to some types of SSC.

In the United States of America, certified designs exist for standard nuclear power plants, which are currently designed to broadbanded ground response spectra anchored to 0.3g peak ground acceleration (PGA) in the horizontal direction and companion design ground response spectra in the vertical direction. These design basis ground motions, known as certified seismic design response spectra, are compatible with the specifications of the utility requirements document in the United States of America. For US certified designs, a plant high confidence of low probability of failure (HCLPF) capacity of at least 1.67 times an SL-2 earthquake[3] needs to be demonstrated. The term HCLPF is defined as 95% confidence of less than 5% probability of failure or a mean confidence of less than 1% probability of failure. During the certified design stage, a specific site has not been identified. Therefore, site specific seismic hazards are not known and, consequently, a site specific SPSA cannot be performed. In order to address the issue of unknown sites, SMA procedures are used [4–6]. Specifically, a PSA based SMA has to be performed for certified designs in the United States of America.

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[3] A seismic level 2 (SL-2) earthquake is defined in NS-G-1.6 [3] and "is associated with the most stringent safety requirements" for which the plant's design needs to ensure the fundamental safety functions.

The European Utility Requirements Document[4] (EUR), developed by a group of major European electricity producers specifies, similar requirements for standard designs in Europe: DBEs comprising broadbanded ground response spectra anchored to a PGA of 0.25g and consideration of beyond DBE events in the design phase. The EUR specifies that it needs to be demonstrated that a standard design achieves a plant seismic margin of 1.4 times SL-2. The EUR specifically excludes the performance of an SPSA, which means that the SMA approach will be implemented.

The former nuclear regulatory authority of Japan provided requirements for the seismic analysis and design of nuclear power plants. It recognized the possibility of beyond DBE occurring, denoting it residual risk [7]. This residual risk has to be considered for existing nuclear installations and for new installations.

2.1.4. Margins beyond the design basis for existing nuclear installations

Although engineering practice for existing installations was to include design margins, it did not make them as explicit as is now the case for new facilities. To the extent possible, the purpose of a seismic evaluation is to make these margins explicit, quantify them and assess the safety of the installation accordingly. This approach may lead to the conclusion that margins are not sufficient for some SSCs. In such cases, the purpose of seismic evaluation is also to propose engineered upgrading actions to achieve the desired safety level.

2.2. SELECTION OF METHODOLOGY

Two methodologies discussed in detail in this publication are the SPSA and the deterministic SMA (see Refs [8–12]). The PSA based SMA, a hybrid alternative to the SPSA approach, also known as the Nuclear Regulatory Commission (NRC) seismic margin method based SMA [13] is briefly discussed. The approach of design basis re-constitution is also an alternative, if required by the regulatory body, but it is not discussed in this publication.

The SPSA and SMA methodologies have matured after 30 years of development and applications, and industry standards have been developed in the United States of America. In addition, these evaluation procedures, adapted to individual nuclear installations, have also been selectively applied to existing nuclear power plants in Europe and Asia since 2008 (see Ref. [14] for an example).

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[4] See www.europeanutilityrequirements.org for further information.

The American Nuclear Society (ANS) developed a standard for the application of probabilistic risk assessment methodologies to external events for nuclear power plants. First published in 2003 and updated in 2007, and the standard became in 2008, together with the American Society of Mechanical Engineers, part of ASME/ANS RA-S–2008, Standard for Level 1/Large Early Release Frequency Probabilistic Risk Assessment for Nuclear Power Plant Applications [15]. Updated in 2009, ASME/ANS RA-Sa–2009 provides technical requirements both for SPSA and SMA assessments [16]. In Japan, the Atomic Energy Society published the standard AESJ-SC-P006 in 2007, which covers the SPSA methodology [17]. A short introductory description of all the available methodologies is given in the following subsections.

2.2.1. Seismic probabilistic safety assessment

As reported in Ref. [2], the SPSA is an integrated process to provide an estimate of the overall frequency of failure of a predetermined plant level damage state, such as reactor core damage frequency (CDF) or large early release frequency (LERF). It is a natural extension of the PSA techniques introduced for the assessment of risk derived from internal events and it constitutes the most comprehensive approach to seismic safety evaluation. It includes consideration of the following:

(a) Uncertainty and randomness of the seismic hazard;
(b) Uncertainty and randomness of structures, equipment, components and distribution systems failure rates conditional upon earthquake ground motion;
(c) Logic tree to calculate plant level damage states from the seismically induced failure rate of SSCs, random failures and operator errors.

2.2.2. Methodology for seismic margin assessment

The two SMA methods developed in the 1980s and 1990s are the deterministic SMA method developed by the Electric Power Research Institute (EPRI) [11] (also known as the EPRI seismic margin method) and the PSA based SMA was developed by the NRC (also known as the NRC seismic margin method [10, 13]). These SMA methods were developed as simplified alternatives to the SPSA and used in the evaluation of the seismic safety of nuclear power plants within the severe accident examination programme of the NRC, Individual Plant Examination of External Events (IPEEE) [18, 19]. The SMA methods differ from the SPSA in that they were specifically developed just to assess the seismic safety margin of nuclear power plants above the DBE. This margin is often expressed as a ground motion parameter that represents a HCLPF for the overall plant and individual SSCs. The SMA methods were designed to avoid frequency of occurrence arguments associated with the seismic hazard, which have often proved highly contentious and difficult to reconcile. In contrast to the SPSA, they do not provide estimates of seismic risks such as core damage or adverse public health effects.

As described in para. A–2 of NS-G-2.13 [1], the two seismic margin methods developed along different paths: the success path methodology (deterministic SMA), and the event tree/fault tree methodology (PSA based SMA):

> "The differences lie in the systems modelling approach and in the capacity evaluation. Systems modelling in the former method is done by success paths, and in the latter, by event trees or fault trees. Capacity evaluations of SSCs are made in terms of HCLPF values in the former; in the latter, capacity evaluations are made by probabilistically defined fragility functions."

One end result in each case is the HCLPF capacity of the plant expressed in terms of a ground motion parameter (e.g. PGA). Many other end products may be developed with applications in decision making, including importance ranking of SSCs.

The deterministic SMA approach has been the most dominant for worldwide applications to existing installations. In recent years, however, the PSA based SMA has been the most dominant SMA approach for evaluating new designs during the design process.

In the context of design [20]:

> "A PRA-based SMA should provide a clear understanding of significant seismic vulnerabilities and other seismic insights to demonstrate the seismic robustness of a standard design in lieu of a full-fledged seismic PRA. Accordingly, the level of detail of a PRA-based SMA needs to be sufficient to gain risk insights to be used to identify and support requirements important to the design and plant operation. To this end, DC/COL-ISG-020 [13] provides a three-tiered process for the PRA-based SMA implementation in licensing applications for new reactors. This three-tiered process includes (1) a PRA-based seismic margin analysis method and its implementation for DC applications, (2) site- and plant-specific updates of the DC PRA-based seismic margin evaluation for COL applications, and (3) post-COL licensee verification of as-designed and as-built plant seismic margin capacity preceding initial fuel load.

> "Each element of this process should be performed based on the technical information consistent with the application. Clear interface requirements between elements should be provided which are essential to ensure consistency and, therefore, the quality of the PRA-based SMA process."

2.2.3. Design basis re-constitution

Re-constitution of the seismic design basis and re-design and re-qualification of SSCs is one option to address the seismic evaluation of existing nuclear installations. This option is most often applied when the perception of the seismic hazard has increased significantly. In Japan, for example, re-constitution of the seismic design basis (seismic backcheck) has been necessary for numerous nuclear power plants. In a similar approach, the practice in France is to re-evaluate the design basis periodically. When the seismic design basis is modified, all the seismically classified SSCs are verified, the design margins are credited (e.g. using inelastic energy absorption factors, see Appendix II) and the analysis detail is graduated according to the importance of the SSCs in the seismic scenarios.

In other countries, the design basis re-constitution has typically been judged to be unnecessary to address issues such as those identified in Section 2.1.2. Paragraph 2.1 of NS-G-2.13 [1] states:

> "It is usually recognized that well designed industrial facilities, especially nuclear power plants, have an inherent capability to resist earthquakes larger than the earthquake considered in their original design. Conservatisms are compounded through the seismic analysis and the design chain. This inherent capability or robustness — usually described in terms of the 'seismic design margin'".

This existing seismic margin is one basis for the development and implementation of the SMA and SPSA methodologies, which successfully address many seismic issues that arise (e.g. restarting a nuclear power plant after experiencing on-site earthquake ground motion exceeding design basis ground motion).

2.2.4. Selection considerations

The approaches discussed in this publication are mainly the SPSA and the deterministic SMA and may be addressed by either the SPSA, SMA or both [as noted in brackets] in para. 2.11 of NS-G-2.13 [1]:

"If, for the reasons above [(see Section 2.1.2)] or for other reasons, a seismic safety evaluation of an existing nuclear installation is required, the purposes of the evaluation should be clearly established before the evaluation process is initiated. This is because there are significant differences among the available evaluation procedures and acceptance criteria, depending on the purpose of the evaluation. In this regard, the objectives of the seismic safety evaluation may include one or more of the following:

(a) To demonstrate the seismic safety margin beyond the original design basis earthquake and to confirm that there are no cliff edge effects. [SPSA/SMA[5]]
(b) To identify weak links in the installation and its operations with respect to seismic events. [SPSA/SMA]
(c) ...
(d) To provide input for risk informed decision making. [SPSA]
(e) To identify and prioritize possible upgrades. [SPSA/SMA]
(f) To assess risk metrics (e.g. core damage frequency and large early release frequency) against regulatory requirements, if any. [SPSA]
(g) To assess plant capacity metrics (e.g. systems level and plant level fragilities or high confidence of low probability of failure (HCLPF) values) against regulatory expectations." [SPSA/SMA]

Two examples of risk goals are those established in the United States of America and in Canada. In the United States of America, NRC staff identified probabilistic performance goals relative to CDF and large release frequency (LRF) in a 1990 staff requirements memorandum [21], in response to SECY-90-016 [22]. The NRC goals are less than 10^{-4} for mean annual CDF and less than 10^{-6} for mean annual LRF. The Department of Energy similarly established probabilistic performance goals to be used as a measure of acceptance of the design of nuclear facilities. The performance goals for Department of Energy nuclear facilities are that confinement needs to be ensured at an annual frequency of failure of between 10^{-5} and 10^{-4} [6].

In Canada, Regulatory Document RD-337, Design of New Nuclear Power Plants [23], specifies three probabilistic performance goals: small release frequencies less than 10^{-5} per reactor-year; LRFs less than 10^{-6} per reactor-year; and CDFs less than 10^{-5} per reactor-year.

[5] Consistency of seismic margin with the safety goals could also be required by the regulatory body.

Paragraph 2.12 of of NS-G-2.13 [1] states:

> "The objectives of the seismic safety evaluation of an existing installation should be established in line with the regulatory requirements, and in consultation and agreement with the regulatory body. Consequently, and in accordance with such objectives, the level of seismic input motion, the methodology for capacity assessment and the acceptance criteria to be applied, including the required end products, should be defined. In particular, for evaluating seismic safety for seismic events more severe than the event specified in the original design basis, the safety objectives should include the functions required to be ensured and the failure modes to be prevented during or after the earthquake's occurrence."

In conjunction with the objectives of the evaluation, potential end products of the seismic safety evaluation include ranking of the seismic capacity or risk of a group of installations in a region or Member State to aid in the prioritization of actions to be taken plant by plant, for instance, extensive upgrades to be implemented and as stated in para. 2.13 of NS-G-2.13 [1]:

"(a) Measures of the seismic capacity of the nuclear installation in deterministic or probabilistic terms;
(b) Identification of SSCs with low seismic capacity, and the associated consequences for plant safety, for decision making on upgrading programmes;
(c) Identification of operational modifications to improve seismic capacity;
(d) Identification of improvements to housekeeping practices (e.g. storage of maintenance equipment);
(e) Identification of interactions with fire prevention and protection systems, etc.;
(f) Identification of actions to be taken before, during and after the occurrence of an earthquake that affects the installation, including arrangements for and actions in operational and management response, analysis of the obtained instrumental seismic records and performed inspections, and the integrity evaluations to be performed as a consequence;
(g) A framework to provide input to risk informed decision making."

Establishing the short term and long term objectives of the programme plays a significant role in determining the methodology to be applied. The selection of the approach is based on all of the above mentioned considerations.

Finally, agreement with the regulatory body on the objectives, methodology, end products and acceptance criteria is as important as the approach to be employed.

Once the decision has been made on the approach, the licensee needs to commit the necessary expertise and resources to the execution of the programme. Ideally, this includes assigning licensee personnel to support the programme with expertise in systems analysis and design, operations of the installation and engineering (civil, mechanical, electrical, instrumentation and control, fire, flood). In addition, the licensee needs to commit resources to support walkdowns of the installation, such as escorts and plant personnel to provide access to equipment and components. In many cases, consultants with specialized expertise in various aspects of programme execution, such as fragility analysis for the SPSA, will be needed. The licensee will need to contract out where necessary, and the resources needed external to the licensee's staff will be greater for SPSAs than SMAs.

3. DATA COLLECTION AND INVESTIGATIONS

3.1. GENERAL

The activities associated with the development and collection of data to be used as input for seismic safety evaluation include the following:

(a) Seismic design basis documentation developed in the original design of the nuclear installation.
(b) Characteristics of site and building foundations that affect the seismic demands applicable to the building structure and specified safety related mechanical, electrical and instrumentation and control components and distribution systems and supports.
(c) Installation design documents, such as the final safety analysis report, piping and instrumentation diagrams, electrical one-line drawings, plant arrangement drawings, lists of safety functions, frontline systems that perform the functions, support systems and components, and dependency matrices between frontline and support systems.
(d) Previous systems analysis documentation. Even though it is not strictly necessary, it has become common practice, both for SPSA and SMA, to have an internal event level 1 PSA as the starting point for the systems analysis part of the evaluation.
(e) General configuration drawings of equipment and distribution subsystems.
(f) Anchorage detail drawings for equipment and supports.

(g) Seismic qualification documentation for SSCs. The data mainly include test results or stress, force and moment analyses of individual safety related SSCs and their supports, which demonstrate that the SSCs can withstand a specified earthquake intensity without loss of its safety functions.

Seismic input for the seismic safety evaluation depends on the selected methodology: SPSA uses the seismic hazard curves for the site; whereas SMA uses the definition of RLE (see Section 4).

Generic seismic experience and test experience data constitute a very important piece of information. Since the early 1980s, efforts have been initiated in States to collect and evaluate data on the behaviour of active industrial type mechanical and electrical components and distribution systems (e.g. those which potentially move or change state, such as pumps, valves, motor control centres, switchgears), as well as passive SSCs (e.g. buildings, vessels, tanks, heat exchangers, piping, duct, conduit, cable trays) in major, potentially damaging earthquakes (see Refs [10, 14, 24–28]). The data include operation characteristics, including malfunction, damage and failure (i.e. loss of leak tightness or structural integrity) of pressure retaining component (piping, ducts) and electrical components and distribution systems (i.e. wire, tubing raceway, conduit, distribution systems and components). Also contained in this data collection is the observed behaviour of mechanical and electrical components, instrumentation and devices that were shake table tested to determine their seismic qualification or fragility tested. From both the earthquake experience and test data, caveats (or restrictions) were developed for some of the components and systems. These caveats ensure that conditions that appear to have a significant effect on the ability of components and systems to resist seismic loading are met.

It is important to note that building structures and some other special components, in contrast to mechanical and electrical systems and components, are unique to the extent that they cannot be qualified and evaluated by experience data and need to be qualified and evaluated by review of their design and analysis calculations and/or reanalysis. Typically, reanalysis of building structures and other structural components is necessary to evaluate these structures for the RLE or to generate fragility functions for use in an SPSA.

3.2. HAZARD CLASSIFICATION OF INSTALLATIONS AND STRUCTURES, SYSTEMS AND COMPONENTS

To some extent, the collection of data for the evaluation of existing installations will depend on the hazard categorization of the installation and the SSCs contained within the installation. Existing nuclear installations are typically divided into high, moderate and low hazard categories, as a function of their potential unmitigated releases of radioactive material or waste as it affects the health of installation and site workers, the public and the environment. SSCs contained within the installation that are designed to prevent or mitigate such releases are also categorized [3, 29, 30].

High hazard installations are those where unmitigated release of radioactivity would likely have unacceptable consequences for workers, for the public or for the environment. High hazard SSCs are those intended to prevent or mitigate potential releases in high hazard installations. In general, high hazard installations have the potential for reactor core melt or hydrogen generation, either inside a power reactor or outside the reactor or containment structure. The latter is associated with spent fuel less than three to five years old, which needs active cooling to avoid fuel cladding or melt failure and the potential for hydrogen generation. Also included in the high hazard category, as a function of quantity and form, are large inventories of high energy radioactive waste or material storage and processing installations as defined by national requirements [29]. Data collection as described here refers to high hazard installations. Moderate and low hazard installations and moderate and low hazard SSCs are discussed in Section 6.

3.3. SEISMIC DESIGN BASIS DATA

Collection of data with regard to seismic design basis extends to the following categories:

(a) Seismic parameters used to define the original seismic hazard input motion.
(b) Evaluation of original design basis free field ground motion parameters which defined either the design basis acceleration time histories or site free field ground response spectra.
(c) High hazard SSCs that are designed in accordance with design standards with acceptance criteria limited to elastic stress levels[6]. The design basis will need to be investigated to find out:
    (i) Whether SSCs were originally designed in accordance with design codes whose acceptance criteria permit reduction factors defined based on implicit inelastic behaviour (i.e. define any reduction factor associated with inelastic energy absorption factors or limit states which permit other than elastic response, as in the Structural Eurocodes).

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[6] Some pressure retaining systems and components acceptance criteria may allow pseudo elastic stress levels greater than yield stress [31].

(ii) Whether SSCs were originally designed in accordance with design codes that permit pseudo elastic stress levels above yield (e.g. ASME BPVC design codes).

3.4. DATA COLLECTION AND DOCUMENTS

Initially, emphasis is placed on the collection and compilation of the original design basis and of the drawings, calculations and other seismic qualification documents in order to minimize any new effort needed to develop such data for an existing installation as part of a seismic re-evaluation program.

3.4.1. Site data

3.4.1.1. Foundation media

For site seismic response evaluations, both static and dynamic material properties as a function of depth in the foundation and adjacent soil and rock are needed. For high hazard installations, foundation soil and rock layers in situ and in laboratory data need to be available. In situ data include density of the material and low strain properties. The variation of dynamic shear modulus and damping values is also needed as well as soil property variation with foundation depth and strain levels in the foundation medium. At a minimum, the following foundation parameters need to be determined for each layer:

— Dynamic shear modulus versus shear strain curve;
— Damping ratio versus shear strain curve;
— Poisson's ratio;
— Density.

Conservative ranges of static and dynamic material shear wave velocity values that account for all the elements of a site's geotechnical properties need to be determined, evaluated and documented [1]. Information on the mean level and variation of the water table also needs to be obtained and documented.

Seismic evaluations of existing high hazard installations, regardless of how foundation effects were considered in the original design of the installation, need to rely on a mathematical model of the layered rock or soil foundation medium that includes strain compatible shear moduli and foundation media damping values. These may need to be developed in order that adequate in-structure response spectra (IRS) can be developed for evaluation of the existing nuclear installation SSCs. To the maximum extent possible, the collection of these data needs to be carried out in compliance with the IAEA Safety Standards Series No. NS-G-3.6, Geotechnical Aspects of Site Evaluation and Foundations for Nuclear Power Plants [32].

For design, the variability in the foundation media response to the earthquake input motion to the site is currently needed. This is typically done by modifying the effective stiffness response of a foundation media by using the best estimate and the upper and lower bound estimates, with the limit for the lower and upper foundation media stiffness changed by a factor of two as compared to the best estimate stiffness and enveloping the results in the generation of an applicable design earthquake response spectra. For development of the response spectra for re-evaluation, only the best estimate foundation media response spectra are usually applied.

An important aspect of the behaviour of foundations, which results in stress and strains other than those caused by inertia, is the potential for differential displacements caused by liquefaction during the active shaking phase of the earthquake or foundation media consolidation following the earthquake. Poorly consolidated saturated cohesionless soils (sands and gravels) below the water table have the potential to liquefy or consolidate as the result of applied earthquake motions. The parameters that are evaluated to determine the potential for liquefaction or consolidation are discussed in NS-G-3.6 [32] and need to be considered in the development and evaluation of geotechnical and foundation site data for high hazard installations.

3.4.1.2. Off-site seismic induced hazard

In addition to seismic motion effects at the site, off-site seismic induced hazard effects need to be considered. These mainly include flooding as the result of:

— Upstream dam failure;
— Tsunami;
— Seiche;
— Landslides or avalanche affecting the ultimate heat sink or other bodies of water affecting the plant site.

The possibility of seismic induced failure of nearby industrial installations, which could cause explosions, release of toxic substances or external missile generation, also needs to be considered. Consequently, data about these potential off-site hazards need to be gathered during this phase. IAEA Safety Standards Series Nos SSG-18, Meteorological and Hydrological Hazards in Site Evaluation for Nuclear Installations [33], and NS-G-3.1, External Human Induced

Events in Site Evaluation for Nuclear Power Plants [34], provide guidance on the data needed.

3.4.2. Assumptions and methods of analysis of existing structures, systems and components

Design computational models and analyses of building structures could be re-used in the seismic safety evaluation if some conditions are met. In order to assess their suitability, assumptions and structural analysis methods used in the design need to be identified as follows:

(a) Consideration of soil–structure interaction effects;
(b) Modelling techniques and analytical methods used to determine the seismic response of structures and the IRS;
(c) System damping values;
(d) Allowance for ductile detailing, if any.

3.4.3. Standards and procedures

Standards and procedures used for the design will need to be identified. Knowledge of these standards and procedures will be used during the seismic evaluation in order to compute the seismic capacities or fragilities, particularly:

(a) Procedures and standards adopted to specify the mechanical properties (definition of specified minimum and best estimate properties of the materials);
(b) Procedures and standards used to define load combinations, including applicable load factors or allowable stresses and to calculate seismic resultant;
(c) Procedures and standards used to define original acceptance criteria.

As a practical matter, the acceptance criteria used to determine the seismic capacities of SSCs for seismic safety evaluation of existing facilities are generally less conservative than those used in the original design. For example, a way to identify and quantify design margins in structures is to evaluate strains induced in the structure or components by the postulated RLE and to compare them with acceptable strains. For instance, in the case of concrete structures, interstory drifts can be computed and compared with acceptable drift values, such those presented in Ref. [30]. In the case of metallic components, design criteria are established to prevent different failure modes with some safety margins. Where design criteria are exceeded, substantiation of margin against the specific failure modes can be credited. Another option to identify and quantify design margins is to use seismic margin evaluation methods and criteria (allowing ductility, higher damping, higher displacement limits).

3.4.4. Nuclear installation documentation

The nuclear installation specific information applicable to evaluation of SSCs needs to include the following:

(a) Specifications for design and as-built or as-is drawings of safety related SSCs and their supports;
(b) Design calculations;
(c) Structural foundation design data: the original design basis for structures needs to be reviewed for such data;
(d) Reports of tests performed for seismic qualification of SSCs;
(e) Field installation and erection criteria;
(f) Design change notice and non-conformance reports and other quality assurance documentation.

3.4.5. Data for structures, systems and components

3.4.5.1. Installation specific data

Specific data on SSCs need to be collected, such as:

(a) ‘As-is’ conditions for materials, geometry and configuration. It is important to establish the accuracy of the data. A preliminary screening walkdown needs to confirm the correctness of the existing documentation data and to acquire missing or new information particularly associated with erosion or corrosion or other ageing effects on SSC.
(b) Reports of tests, if any, performed for identification of SSC dynamic characteristics.
(c) Data indicating any significant modification and/or upgrading measures performed since the original design of the installation’s SSCs.
(d) Data about service life remaining and end-of-life properties when relevant (i.e. SSCs subjected to high cycle fatigue loading).

In general, when the remaining service life is significantly less than the design life of the structure, this needs to be considered in developing the seismic re-evaluation.

3.4.5.2. Earthquake experience and seismic test data

The seismic capacity of structures in existing nuclear installations is usually obtained by modifying the existing design basis analysis seismic capacities by the use of modification parameters. In general, because of the unique nature of the foundation condition and the uniqueness of the building structural layout and associated seismic load paths and resultant structural design, it is generally not possible to use earthquake experience or test data as a basis to evaluate existing building structures.

On the other hand, the estimate of the seismic capacity of systems and components is often accomplished by the use of experience gained from historical strong motion seismic events potentially causing damage [1]. Data from strong motion earthquakes have generally been collected to provide the information needed to verify directly the seismic adequacy of individual systems and components in existing operating industrial facilities and power plants.

Such verification necessitates that the seismic excitation of an item installed in an industrial facility or power plant 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 nuclear installation. It is also necessary that the item being evaluated and the item that underwent the strong motion have similar physical characteristics and have similar support and anchorage characteristics. Different support or anchorage capacities can be evaluated by additional analysis. In the case of active items (i.e. that move or change state during or immediately following an earthquake), it is generally 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 necessary for the nuclear safety related item being evaluated.

A number of seismic databases based on strong motion earthquake experience has been developed. These include the data developed in an original class of 20 components by the EQE Engineering for the Seismic Qualification Utility Group (SQUG) [24]:

— Motor control centres;
— Low voltage switchgear;
— Medium voltage switchgear;
— Transformers;
— Horizontal pumps;
— Vertical pumps;
— Fluid operated valves;
— Motor operated valves;
— Fans;

— Air handlers;
— Chillers;
— Air compressors;
— Motor generators;
— Distribution panels;
— Battery racks;
— Battery chargers and inverters;
— Engine generators;
— Instrument racks;
— Temperature sensors;
— Control and instrumentation cabinets.

Supplemental information was prepared for the EPRI [25] and other database material was developed for the NRC [10, 26]. In addition, a seismic evaluation database was developed for the United States Department of Energy [27], with data collected for the following supplemental component classes:

— Above ground vertical and horizontal tanks;
— Underground tanks;
— Heat exchangers;
— Raceway systems including cable and conduits;
— Above ground piping;
— Underground piping;
— Heating, ventilation, and air-conditioning ducts and dampers;
— High efficiency particulate air filters;
— Glove boxes;
— Canisters and gas cylinders;
— Unreinforced masonry walls;
— Raised floors;
— Storage racks;
— Relays;
— Cranes.

Seismic experience data for mechanical and electrical components installed in water cooled, water moderated power reactors were developed for the IAEA (see Ref. [28]) and were employed in a number of nuclear power plants in Eastern European, originally of Soviet Union design, that have undergone existing plant seismic evaluation.

A great deal more recent earthquake experience data are being developed as a result of strong motion earthquake effects in existing nuclear power plants in Japan as well as associated shake table testing of safety related components. It is anticipated that new references will soon be available which summarize the new experience data developed. However, it needs to be emphasized that there has so far been no failure of a high hazard resistant designed nuclear power plant SSC for actual earthquake shaking motions up to twice the DBE PGA.

Most building structures and some systems and components are so specialized that they are not included in existing earthquake experience database. For those SSCs for which experienced based seismic qualification is not available, the seismic assessment or qualification of an existing installation needs to be carried out, usually by analysis in the case of structures and passive mechanical systems and components and by tests or a combination of tests and analysis for active mechanical, electrical and instrumentation and control systems and components.

3.4.6. Upgrades and nuclear installation modifications

To the extent the installation has undergone upgrades and modifications since initial operation, the design and installation basis for these needs to be determined.

3.4.7. Results of other safety evaluations

The results of previous safety evaluations, including any Level 1, 2 or 3 PSA considering external as well as internal events (if they have been performed), need to be made available, together with any operating experience and random failure rates data.

4. ASSESSMENT OF SEISMIC HAZARDS

NS-G-2.13 [1] provides overall recommendations for the seismic hazard as it pertains to the evaluation of nuclear installations. IAEA safety standards on assessing the seismic hazard and associated geotechnical issues include IAEA Safety Standards Series No. SSG-9, Seismic Hazards in Site Evaluation for Nuclear Installations [35], and NS-G-3.6 [32]. Paragraph 4.1 of NS-G-2.13 [1] states:

> "...the seismic hazard specific to the site should be assessed in relation to three main elements:

(a) Evaluation of the geological stability of the site..., with two main objectives:
   (i) To verify 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 safety. If new evidence indicates the possibility of a capable fault in the site area or site vicinity, the fault displacement hazard should first be assessed in accordance with the guidance provided in [SSG-9 [35]]. If a clear resolution of the matter is still not possible, the fault displacement hazard should be evaluated using probabilistic methods.
   (ii) To verify the absence of permanent ground displacement phenomena (i.e. liquefaction, slope instability, subsidence or collapse, etc.).
(b) Determination of the severity of the seismic ground motion at the site, that is, assessment of the vibratory ground motion parameters, taking into consideration the full scope of the seismotectonic effects at the four scales of investigation and as recommended in [SSG-9 [35]).
(c) Evaluation of other concomitant phenomena such as earthquake induced river flooding due to dam failure, coastal flooding due to tsunami, and landslides."

Items (a) and (c) are needed independently of the evaluation method of the seismic safety of existing nuclear installations (i.e. SPSA or SMA) and are dependent on the site. Items (a)(ii) and (c) need to be based on realistic ground motion specifications. Excessively conservative definitions of the ground motion need to be avoided for these site specific evaluations. Two methods for definition of the seismic ground motion at the nuclear installation site for the seismic safety evaluation are probabilistic seismic hazard analysis (PSHA) and RLE.

4.1. PROBABILISTIC SEISMIC HAZARD ANALYSIS

The PSHA is a necessary element in the calculation of all quantities that need convolution of the seismic hazard with the consequences of the earthquake (e.g. risk metrics, probability of failure of SSCs). It is a necessary input for developing an SPSA. SSG-9 [35] provides overall guidelines and states:

“6.3. The conduct of a probabilistic seismic hazard analysis should include the following steps:

(1) Evaluation of the seismotectonic model for the site region in terms of the defined seismic sources, including uncertainty in their boundaries and dimensions.
(2) For each seismic source, evaluation of the maximum potential magnitude, the rate of earthquake occurrence and the type of magnitude–frequency relationship, together with the uncertainty associated with each evaluation.
(3) Selection of the attenuation relationships for the site region, and assessment of the uncertainty in both the mean and the variability of the ground motion as a function of earthquake magnitude and seismic source to site distance.
(4) Performance of the hazard calculation....
(5) Taking account of the site response....”

Table 1 contains a listing of the typical output from a PSHA [35]. For a complete SPSA, all of these output elements may be needed depending on the types of analysis to be performed.

TABLE 1. TYPICAL OUTPUT OF PROBABILISTIC SEISMIC HAZARD ANALYSES *(table A–1 of SSG-9 [35])*

Output	Description	Format
Mean hazard curves	Mean annual frequency of exceedance for each ground motion level of interest associated with the suite of epistemic hazard curves generated in the probabilistic seismic hazard analysis.	Mean hazard curves should be reported for each ground motion parameter of interest in tabular as well as graphic format.
Fractile hazard curves	Fractile annual frequency of exceedance for each ground motion level of interest associated with the suite of epistemic hazard curves generated in the probabilistic seismic hazard analysis.	Fractile hazard curves should be reported for each ground motion parameter of interest in tabular as well as graphic format. Unless otherwise specified in the work plan, fractile levels of 0.05, 0.16, 0.50, 0.84 and 0.95 should be reported.

TABLE 1. TYPICAL OUTPUT OF PROBABILISTIC SEISMIC HAZARD ANALYSES *(table A–1 of SSG-9 [35]) (cont.)*

Output	Description	Format
Uniform hazard response spectra	Response spectra whose ordinates have an equal probability of being exceeded, as derived from seismic hazard curves.	Mean and fractile uniform hazard response spectra should be reported in tabular as well as graphic format. Unless otherwise specified in the work plan, the uniform hazard response spectra should be reported for annual frequencies of exceedance of 10^{-2}, 10^{-3}, 10^{-4}, 10^{-5} and 10^{-6} and for fractile levels of 0.05, 0.16, 0.50, 0.84 and 0.95.
Magnitude–distance deaggregation	A magnitude–distance (M–D) deaggregation quantifies the relative contribution to the total mean hazard of earthquakes that occur in specified magnitude–distance ranges (i.e. bins).	The M–D deaggregation should be presented for ground motion levels corresponding to selected annual frequencies of exceedance for each ground motion parameter considered in the probabilistic seismic hazard analysis. The deaggregation should be performed for the mean hazard and for the annual frequencies of exceedance to be used in the evaluation or design.
Mean and modal magnitude and distance	The M–D deaggregation results provide the relative contribution to the site hazard of earthquakes of different sizes and at different distances. From these distributions, the mean and/or modal magnitudes and the mean and/or modal distances of earthquakes that contribute to the hazard can be determined.	The mean and modal magnitudes and distances should be reported for each ground motion parameter and level for which the M–D deaggregated hazard results are given. Unless otherwise specified in the work plan, these results should be reported for response spectral frequencies of 1, 2.5, 5 and 10 Hz.

TABLE 1. TYPICAL OUTPUT OF PROBABILISTIC SEISMIC HAZARD ANALYSES *(table A–1 of SSG-9 [35]) (cont.)*

Output	Description	Format
Seismic source deaggregation	The seismic hazard at a site is a combination of the hazard from individual seismic sources modelled in the probabilistic seismic hazard analysis. A deaggregation on the basis of seismic sources provides an insight into the possible location and type of future earthquake occurrences.	The seismic source deaggregation should be reported for ground motion levels corresponding to each ground motion parameter considered in the probabilistic seismic hazard analysis. The deaggregation should be performed for the mean hazard and presented as a series of seismic hazard curves.
Aggregated hazard curves	In a probabilistic seismic hazard analysis, often thousands to millions of hazard curves are generated to account for epistemic uncertainty. For use in certain applications (e.g. a seismic probabilistic safety assessment), a smaller, more manageable set of curves is required. Aggregation methods are used to combine like curves that preserve the diversity in shape of the original curves as well as the essential properties of the original set (e.g. the mean hazard).	A group of aggregated discrete hazard curves, each with an assigned probability weight, should be reported in tabular as well as graphic format.
Earthquake time histories	For the purposes of engineering analysis, time histories may be required that are consistent with the results of the probabilistic seismic hazard analysis. The criteria for selecting and/or generating a time history may be specified in the work plan. Example criteria include the selection of time histories that are consistent with the mean and modal magnitudes and distances for a specified ground motion or annual frequency of exceedance.	The format for presenting earthquake time histories will generally be defined in the work plan.

4.2. REVIEW LEVEL EARTHQUAKE[7]

The RLE defines the earthquake ground motion for which the SMA is to be conducted. The RLE specified for the evaluation of the existing installation may be, and often is, different from the DBE. The RLE is not a new design earthquake. It is usually set higher than and different from the earthquake that was specified in the original design of the nuclear installation. Therefore, the RLE is defined by an earthquake ground motion ‘generally larger’ than the plant SL-2. It is ‘generally larger’ because there may be cases where the RLE defines the DBE; specifically, when a nuclear installation has no original seismic design or the original seismic design basis ground motion is well below the minimum value necessary in the current state of practice. The RLE needs to be sufficiently larger than the SL-2 to ensure that the SMA challenges the capacity of the plant SSCs so that a plant HCLPF capacity can be determined and weak links (if any) can be identified.

The RLE serves two purposes: (i) to define the ground motion for which the HCLPF capacity of SSCs are evaluated; and (ii) to define an initial screening level whereby SSCs may be screened out of the evaluation because their HCLPF capacity has been established to be greater than the RLE. During the walkdown, the initially screened out SSCs need at least a minimum level of verification that their as-is state in the installation is not degraded and that there are no issues in terms of the interaction of seismic spatial systems.

4.2.1. General characteristics of the review level earthquake

The general characteristics of the RLE are the following:

(a) The RLE is defined by three components of ground motion: two horizontal components and a vertical component.
(b) Typically, the RLE for high and medium hazard installations is defined in terms of ground motion response spectra anchored to PGA. The important aspect of the response spectra is the shape. It is assumed that the response spectra shape can be scaled up or down to meet the SMA criteria. Often, it is scaled such that the spectral acceleration amplitude conforms to one of the typical screening levels, 0.8g or 1.2g 5% damped spectral acceleration, across the frequency range of interest.
(c) In addition to the PGA and associated response spectra, the RLE may be defined by other important parameters, such as peak ground velocity, peak ground displacement, time histories and duration of strong motion shaking.

[7] This section is based on Ref. [2].

(d) The response spectral shape of the RLE may be selected to be site independent (i.e. only dependent on general site categorization, such as rock or soil). Median value site independent ground response spectra, published in Ref. [36], were specified as a preferred shape for the IPEEE programme in the United States of America [37].
(e) If acceleration time histories are to be used in the SMA, either recorded or artificially generated time histories may be used. Recorded time histories need to match the relevant source conditions (magnitude, distance) and the site conditions at the installation location. A series of recorded ground motions may be necessary to represent the RLE satisfactorily. The average of the response spectra generated from artificial time histories may be targeted to match the RLE. The variability of this series needs to match the variability of motions tied to the relevant source conditions. If non-linear time history analyses are to be performed to demonstrate the capacity of SSCs, recorded ground motions are preferred. Multiple analyses for different ground motions may be necessary to account properly for uncertainties in the ground motion and system characteristics.

4.3. BASES FOR DEVELOPMENT OF THE REVIEW LEVEL EARTHQUAKE

4.3.1. Probabilistic seismic hazard analysis

Currently for existing nuclear installations, the results of a site specific PSHA often form the basis for the definition of the RLE. For a given annual probability of exceedance level (e.g. 10^{-4}), uniform hazard spectra (UHS) at various confidence levels are generated (i.e. median, mean, 84% non-exceedance probability). Hence, any of these normalized shapes may be selected for the RLE. The selected shape may be smoothed and adjusted as judged appropriate by the regulatory body, licensee and/or the seismic review team (SRT). For high hazard installations the amplitude needs to be selected consistent with a probability of exceedance of about 10^{-5} to 10^{-4} per reactor year and, if possible, consistent with screening information established in publications such as Refs [11, 15].

In addition to selecting a UHS based shape, the principal contributors to the seismic hazard at the probability of exceedance level of interest need to be taken into account. The deaggregated results of the PSHA provide information to do so. Contributions to the site seismic hazard (UHS) arising from distinctly different source regions and magnitude levels may be treated as an envelope, but it may also be treated separately, especially for the evaluation of critical SSCs. Treating each separately is justified, since the likelihood of them occurring simultaneously is extremely small. An example of this situation is near field and far field events. Typically, far field events contribute to the low frequency (<10 Hz) portion of the ground response spectra; whereas near field events contribute to the high frequency portion (>10 Hz) of the ground response spectra. A realistic treatment of these two types of source is to treat them separately in the evaluation. The RLE could be partitioned if desired.

4.3.2. Absence of probabilistic seismic hazard analysis

In the absence of a site specific PSHA, an RLE may be selected based on site independent or broadly site dependent (rock, medium soil, soft soil) response spectral shapes, such as those in Ref. [36]. In practice, the RLE needs to be selected consistent with the screening levels established in Ref. [11], for satisfying one of the purposes of the RLE.

4.3.3. Regulatory specified

The regulatory body may specify the RLE for which the SMA is to be performed. One reason to do so is to provide a common basis for comparison of the seismic margin between installations of similar type (e.g. all nuclear power plants in a given Member State or region). This was the case for the IPEEE programme in the United States of America [19].

4.3.4. New nuclear power plants

For new nuclear power plants, Section 2.1.3 describes regulatory requirements for demonstrating a margin beyond the DBE. In the United States of America, the requirement is that the HCLPF capacity at the plant level be at least 1.67 times the design ground motion (PGA and associated design ground response spectra). This is to ensure that adequate seismic margin is explicitly incorporated into the design. Hence during the design phase, the plant designer usually performs a PSA based SMA to ensure that appropriate design and qualification criteria are implemented to achieve this margin. There are two definitions of the RLE for this case:

(a) For certified designs, this requirement applies to the nuclear island (i.e. the portion of the nuclear power plant design that has been licensed under the requirements of 10 CFR 52, Licenses, Certifications, and Approvals for Nuclear Power Plants [38], and has been designed to the certified seismic design response spectra, CSDRS). The requirement is that the certified design be demonstrated to have a plant level HCLPF capacity at least

1.67 times the CSDRS. For this portion of the design, the RLE is 1.67 times the CSDRS.

(b) For SSCs of the nuclear power plant design that are designed to site specific ground motion response spectra (GMRS), this requirement demands these SSCs satisfy the requirement of HCLPF capacities at least 1.67 times the GMRS.

The EUR[8] has specified similar design requirements for standard designs in Europe: DBEs comprise broadbanded ground response spectra anchored to a PGA of 0.25g and consideration of beyond DBE earthquake events in the design phase. The EUR has specified that it needs to be demonstrated that a standard design achieves a plant seismic margin of 1.4 times the SL-2. It is assumed that the European requirements with regard to the RLE will mirror that of the United States of America (i.e. for the standard design, the RLE is 1.4 times the standardized DBE). For SSCs that are designed to a site specific DBE, the RLE is 1.4 times the site specific DBE. In these two cases, the RLE is the earthquake ground motion for which the SMA is performed and it also defines the acceptance criteria.

5. SEISMIC SAFETY ASSESSMENT

NS-G-2.13 [1] specifies two methodologies for the evaluation of seismic safety of existing nuclear installations: SMA and SPSA. This section describes these two methods in detail. The PSA based SMA, also known as the NRC seismic margin method, is briefly discussed in a separate subsection due to its current application to new nuclear power plant designs.

5.1. SEISMIC MARGIN ASSESSMENT[9]

5.1.1. General

As already mentioned, the objective of a SMA review of an installation is to determine whether the installation can safely withstand an earthquake larger than the DBE. The SMA methodology was developed by the NRC [10] and the

[8] See www.europeanutilityrequirements.org for further information.

[9] This section is based on Ref. [2].

EPRI [11] as an alternative to the SPSA. There were two strong motivations to develop the SMA methods: (i) to avoid arguments associated with the seismic hazard, which are often highly contentious and difficult to reconcile; and (ii) to be more easily implemented by licensee engineers compared to the SPSA approach. The SMA method was designed to demonstrate margin over the design earthquake level to quantify plant safety, not to provide estimates of seismic risk metrics, as is one of the objectives of the SPSA. Elements of the SMA method comprise the approach that addressed NRC Unresolved Safety Issue A-46 (see Ref. [39]).

The SMA procedure in total or in part has been applied to evaluations of nuclear power plant seismic capacity in many States (e.g. Belgium, Bulgaria, Canada, France, Hungary, Republic of Korea, Slovakia, Spain, Sweden, United Kingdom, United States of America). Therefore, there is a large body of accumulated experience worldwide. The SMA approach described in NS-G-2.13 [1] is the EPRI or deterministic SMA approach [11], which is the most widely used and is therefore described here. The event tree/fault tree SMA methodology (NRC method [10, 13]), also known as the PSA based SMA method, is described in Section 5.3.

The deterministic SMA methodology is based on a 'success path' approach. One or more success paths need to be identified. Each success path consists of a selected group of safety functions capable of maintaining the nuclear installation in a safe state or bringing the nuclear installation to a safe state after an earthquake ground motion larger than the SL-2 occurs at the site and of maintaining it there for an agreed time (e.g. 72 hours). The individual SSCs needed to accomplish each of the success paths are then identified and become the basis for the rest of the SMA analysis. This list of items is known as the selected SSCs [1].[10]

The SMA defines and evaluates the seismic capacity of each of the SSCs on the success path(s). In many installations several paths may exist. For example, under the NRC guidance on IPEEE, two success paths (primary, alternate) have to be selected so that they involve, to the maximum extent possible, systems, piping runs and components that differ between both paths [18]. For purposes of the present publication, the regulatory body, in conjunction with the licensee, specifies the number of success paths to be considered.

For the SMA, capacities of SSCs are defined as HCLPF values. In a probabilistic sense, the HCLPF capacity is a 95% confidence of a 5% frequency of failure or an equivalent mean confidence of a 1% frequency of failure when subjected to an earthquake motion defined by frequency characteristics of the RLE. Although defined conceptually in a probabilistic sense, HCLPF values are almost always calculated by deterministic methods, as explained below.

[10] The selected SSC list is sometimes known as the safe shutdown equipment list (SSEL) or the seismic equipment list (SEL).

Deterministic guidelines have been developed and demonstrated to yield the approximate probabilistic definition. For the SMA, the procedures are such that seismic engineers without training in probabilistic methods can routinely calculate the HCLPF capacities. This is in contrast to the expertise needed to develop fragility functions for the SPSA.

Quantification of the plant HCLPF capacity for the SMA can be achieved relatively simply by evaluating the success paths given the HCLPF capacity values of the SSCs comprising them. In summary, the SMA method comprises the following steps (see Section 5.1.2 for a description):

(1) Selection of the RLE (see Section 4.2);
(2) Selection of the assessment team;
(3) Plant familiarization and data collection (see Section 3);
(4) Selection of success path(s);
(5) Determination of seismic response of structures for input to capacity calculation;
(6) Systems walkdown to review preliminary success path(s), select success path(s) and SSCs;
(7) Seismic capability walkdown;
(8) HCLPF capacity calculations (SSCs and plant level);
(9) Peer review, enhancements and documentation [11, 15].

The 'bottom line results' of a well executed SMA comprises estimates of the seismic capacities of each of the SSCs analysed, from which estimates are derived of the seismic capacities of the necessary safety functions and then of the one or more success paths, leading ultimately to an estimate of the seismic capacity of the plant as a whole. In practice, a typical SMA is usually structured so that the estimated seismic capacities of many of the SSCs under consideration are lower bounds for the capacities rather than realistic estimates. The SMA end products of interest include the following:

— Plant HCLPF capacity, which means the ground motion level at or below which there is a high confidence of successfully achieving the defined end state for the necessary time frame;
— Identification of high seismic capacity SSCs with reference to the RLE;
— HCLPF capacities of selected SSCs;
— HCLPF capacities of success paths;
— Identification of low capacity SSCs (weak links);
— Identification of operations that lead to low plant HCLPF values;
— Information for upgrading decisions.

5.1.2. Elements of the seismic margin assessment

The elements of the SMA, as given in the EPRI guidance [11], and potential enhancements as may be required by the regulatory body, are discussed in this section (see Fig. 1).

5.1.2.1. Selection of the review level earthquake

The RLE defines the earthquake ground motion for which the SMA is to be conducted. Considerations for selection of the RLE have already been given in Section 4.2. In general, the RLE is not the acceptance criteria for the seismic

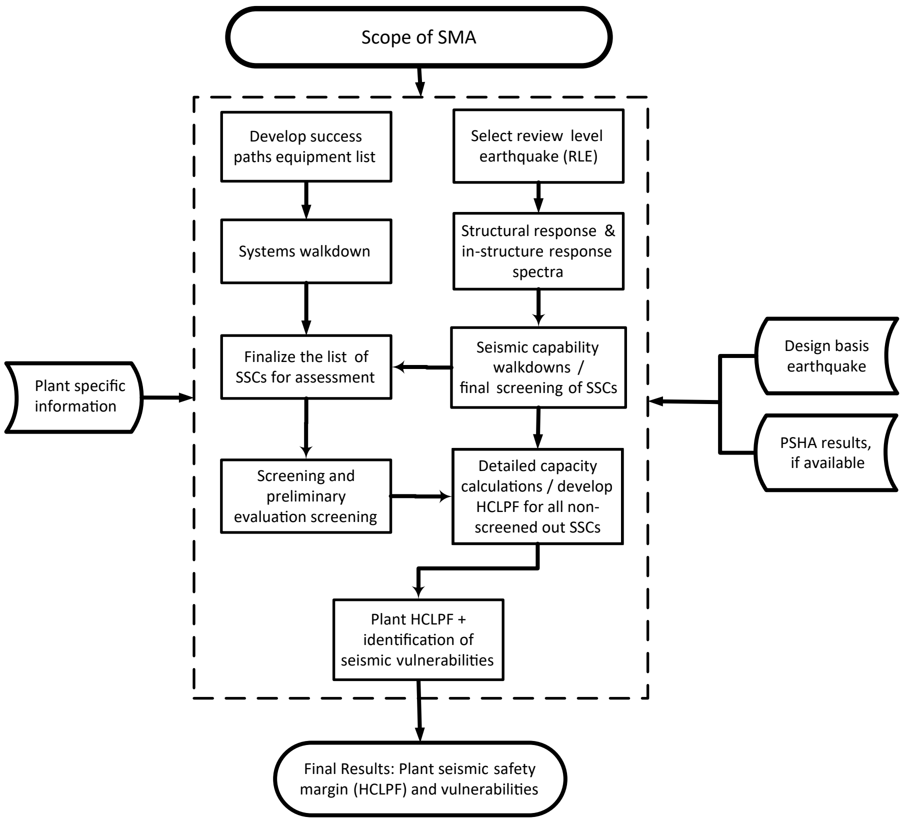

FIG. 1. General workflow of the seismic margin assessment (SMA) method. Note: HCLPF — high confidence of low probability of failure; PSHA — probabilistic seismic hazard analysis; SEL — selected structures, systems and components.

safety evaluation, nor is it a new design earthquake. It is a working tool used to evaluate whether the existing nuclear installation can perform safely, with an acceptable margin, during and after an earthquake ground motion larger than the one used for the design of the facility. Paragraph A–4 of NS-G-2.13 [1] states:

> "The definition of the review level earthquake for the SMA is required at initiation of the evaluation, but it is not dependent on the results of a probabilistic seismic hazard analysis. The review level earthquake defines a screening level in the evaluation process. Most of the procedures developed and implemented to date have defined two screening levels: peak ground accelerations of 0.3g, which corresponds to 0.8g spectral acceleration for 5% damping, and of 0.5g, which corresponds to 1.2g spectral acceleration for 5% damping. ...At the 0.3g screening level, many SSCs are screened out of the process on the basis of their demonstrated robustness to seismic loading conditions. Of course, conditions or caveats are imposed to allow the screening. For the 0.5g screening level, there is a significant increase in the scope of the SSCs to be individually evaluated."

Therefore, if the earthquake of interest is larger than 0.3g PGA, an SPSA may be the most cost effective approach regardless of the end metrics of interest. For those SSCs that are not screened out, additional analyses are necessary to determine their HCLPF seismic capacities. Whichever method is used to select the RLE, it always needs to be recognized that the HCLPF capacities computed in the SMA process are conditional to the response spectra defined for the RLE.

5.1.2.2. Selection of the seismic review team

The SRT is the group with the technical responsibility for the SMA. It is normally assisted by a number of seismic capability engineers, who perform background work under the direction of the SRT to support the decisions made by the SRT. The SRT is a multidisciplinary team made up of individuals with the following expertise [1]: senior systems engineers with knowledge of the installation's systems, in particular frontline and support systems to address safety functions; operations personnel with experience in the operation of the systems and location of selected SSC list items; and seismic capability engineers (civil, mechanical, electrical, instrumentation and control, fire, internal flood). The SRT needs to incorporate licensee personnel, to the maximum extent possible, so that results and insights obtained during the SMA can be utilized in installation operation, seismic upgrading and accident management. Generally, the SRT consists of three to five members who possess the following qualifications:

(a) Knowledge of the failure modes and performance of structures, tanks, piping, process and control equipment and active electrical components during strong earthquakes;
(b) Knowledge of nuclear design standards, seismic design practices and equipment qualification practices for nuclear installations;
(c) Ability to perform fragility/margins type capability evaluations including structural or mechanical analyses of essential elements of nuclear installations;
(d) General understanding of SPSA systems analysis and conclusions;
(e) General knowledge of the installation's systems and functions.

It is not necessary that each member of the team has a capability in all of these areas or extensive seismic experience for all of the elements identified in the success paths being considered. Collectively, however, the SRT needs to be strong in all of these areas. A good team would include systems engineers, plant operations personnel and seismic capability engineers.

Senior seismic capability engineers are responsible for the seismic capability walkdowns and for screening out SSCs from further evaluations for the SMA. They define additional effort to be expended on evaluations of individual SSCs, for those components not screened out (i.e. aspects of HCLPF capacity calculations, including acquisition of design data, construction or installation data), 'as-is' configuration (including seismic spatial systems interaction hazards) and calculation procedures for HCLPF calculations. Seismic capability engineers perform their functions in the field and in the office.

Systems engineers need to identify all reasonable alternate means to bring the installation to a stable and safe condition. They also need to identify all elements that comprise the frontline and support system components together with the associated electrical, fluid and pneumatic systems for each of these success paths. The systems engineers have the principal responsibility for selecting one or more success paths for which the seismic capability is to be assessed in detail.

Plant operations personnel on the SRT need to be intimately knowledgeable about normal and emergency operating procedures and operator responses to abnormal situations. These experts need to be aware of the instrumentation and actuation systems needed to support those operator actions that may be necessary to accomplish the safe shutdown objectives associated with the preferred and alternative success paths selected.

5.1.2.3. Systems analysis and selection of structures, systems and components

The systems engineers initially need to review the installation design documents and familiarize themselves with the installation design features. Information is contained in the final safety analysis report, piping and instrumentation drawings, electrical one-line drawings, plant arrangement drawings, topical reports and plant specifications. Representative lists of safety functions, frontline systems that perform the functions, support systems and of components and dependency matrices between frontline and support systems need to be reviewed.

The starting points for many installations are seismic studies (with generic observations and conclusions) that have been performed previously for similar installations. These studies need to be made more plant specific by systems personnel with specific expertise in the installation. Plant operations personnel familiar with the systems are the logical choice to perform a pre-screening of any representative lists. These engineers need to be able to conduct the following:

(a) Identify the important installation functions;
(b) Identify the frontline and supporting systems needed to perform necessary functions for installation shutdown;
(c) Identify alternate sequences to maintain the nuclear installation in a safe state or bringing the nuclear installation to a safe state (success path logic diagrams);
(d) Identify the elements of each system in each of the success paths.

Two success paths with independence of SSCs to the extent possible are often selected. For nuclear power plant evaluations, it is usually necessary that one of these two paths is capable of mitigating the consequences of a small loss of coolant accident (LOCA). It is important that this initial screening be closely monitored by members of the SRT and thoroughly documented.

The primary success path needs to be the path for which it is judged easiest to demonstrate a high seismic margin and the path which the plant operators would employ after a large earthquake, based upon procedures and training. The primary success path needs to be a logical success path consistent with plant operational procedures. Remote success paths unlikely to be used may have higher seismic margins, perhaps exceeding the RLE; however, their selection is inadvisable. The alternate paths need to involve operational sequences, systems piping runs and components different from those in the preferred path. The alternate paths need to contain levels of redundancy of the same order as that of the primary success path.

It has become common practice to use a pre-existing internal events Level 1 PSA to cover most of these system analysis activities. When using systems models developed for a Level 1 PSA for developing the components in each of the success paths, the following points have to be taken into account:

— Internal PSA models usually include only active components, since the probability of random failure of passive components is negligible. For the SMA, both active and passive components need to be included in the assessment, since the RLE could lead to failure of both types of component. In fact, some types of passive component tend to have low HCLPF capacities (e.g. above ground vertical tanks).
— Internal PSA models can be very detailed. They can have a separate representation of several items that are different components of the same item of equipment (e.g. limit switches, motor of a motor operated valve). For the SMA, the different components of the same equipment item need to be grouped together in a single item for seismic assessment (rule of the box, see Ref. [40]).
— The rule of the box does not apply to relays, motor starters and other electromechanically actuated devices that could 'chatter' during the RLE.

At this point, the systems engineers will be ready for the systems and element selection walkdown.

5.1.2.4. Systems and elements selection ('success paths') walkdown

The systems and elements selection walkdown is an initial walkdown carried out by the systems engineers, one or more plant operations experts and at least one seismic capability engineer. The purposes of the walkdown are to review the completeness of the selected SSCs associated with the primary and secondary success paths and to review the previously developed plant system models (candidate success paths) for obvious RLE evaluation problems and anchorage or seismic spatial system interaction issues. The following information needs to be provided by the systems engineers to the seismic capability engineers prior to the seismic capability walkdown:

(a) The selected SSC list of the primary and alternate success paths that are to be evaluated in the SMA, together with all important elements in these paths;
(b) Components in each success path, clearly marked on plant arrangement drawings;
(c) Instrumentation necessary for safe shutdown;

(d) List of relays and contactors for which seismic induced chatter needs to be precluded.

5.1.2.5. Review of plant seismic design information

Prior to the seismic capability walkdown, the installation seismic design documents need to be reviewed by all or part of the SRT or by a seismic capability engineer of the licensee under the direction of the SRT. The purpose of the review is to determine conformance of the individual elements of the installation design with screening guidelines. This review includes:

(a) Seismic sections of the final safety analysis report;
(b) Sample equipment qualification reports;
(c) Sample equipment specifications;
(d) Seismic analyses conducted for the purpose of defining IRS;
(e) IRS provided as response spectra to equipment vendors;
(f) Relay chatter documentation;
(g) Representative equipment seismic anchorage analyses and designs;
(h) Seismic qualification review forms, if available;
(i) Any topical reports associated with seismic issues.

Prior to the seismic capability walkdown, a summary of all the review items has to be provided to the SRT. The SRT needs to be familiar with the installation design basis prior to the walkdown. A thorough understanding of the seismic design basis and approaches used for equipment qualification and anchorage is necessary for a credible screening of elements for the RLE.

5.1.2.6. Development of realistic in-structure response spectra

A realistic median centred response to the RLE of the structures and equipment that comprise the proposed success paths is estimated in this task to facilitate screening of structures and equipment and evaluation of seismic HCLPF capacities of screened-in SSCs. In-structure responses at 84% non-exceedance probability (NEP), if the RLE occurs, are to be used [41]. Note that the median response will give 84% NEP if the RLE has been defined at the 84% NEP. In-structure responses could be obtained either by scaling of the DBE design analysis responses or by performing new analyses. Recent studies have further identified the applicability of each of the approaches:

(a) For rock founded structures subjected to an RLE with broad frequency content in the low frequency range (<10 Hz) and assuming soil–structure interaction is not a dominant phenomenon, scaling of design analysis responses is feasible;

(b) For soil founded structures, for which soil–structure interaction is an important phenomenon, the preferred approach is a reanalysis of at least a sample of the important structures to provide median cantered IRS conditional on the occurrence of the RLE at the site.

Damping values for different components and stress levels are given in Appendix II.

5.1.2.7. Preparatory work prior to seismic capability walkdown

The following information has to be provided by the systems engineers to the seismic capability engineers prior to the seismic capability walkdown:

(a) List of the primary and alternate success paths that are to be evaluated in the SMA, together with all the important elements in these paths (this list is referred to as the selected SSCs);
(b) Components in each success path, clearly marked on plant arrangement drawings;
(c) Instrumentation necessary for safe shutdown;
(d) List of relays and contactors for which seismic induced chatter needs to be precluded.

During the seismic capability walkdown, the SRT needs estimates of realistic IRS resulting from the RLE, as described above. During the walkdown, judgements can only be made on the adequacy of seismic ruggedness with an understanding of the seismic demand at the RLE level and a measure of equipment anchorage capacity. To this end, it is advisable to develop preliminary assessments of the capacity of equipment and equipment anchorage based on design information, seismic qualification margins and simple calculations using the RLE IRS developed in the previous step.

In addition, the potential for soil liquefaction, consolidation and slope instability is assessed considering the seismic sources in the site region and soil conditions. The objective is to assess whether soil failures are likely at the RLE level and to estimate the potential consequences on buildings, buried piping and ground mounted components such as tanks. The real issue is the estimate of the consequences of soil failure on the selected SSCs, not simply that soil failure modes could occur.

5.1.2.8. Seismic capability walkdown

The seismic capability walkdown is the responsibility of the SRT, assisted by seismic capability licensee engineers. A systems engineer who was engaged in the system and element selection walkdown and a person knowledgeable in installation operations also need to accompany the SRT. The seismic capability walkdown needs to concentrate on rooms that contain elements of the success paths previously selected by the systems engineer. The SRT also has to be aware of seismic spatial interaction effects and to make note of any deficiencies as they are generally an indicator of a lack of seismic concern on the part of plant operations and design personnel. The purposes of the seismic capability walkdown are:

(a) To screen from the SMA review all elements for which they estimate HCLPF capacities to exceed the RLE level, based upon their combined experience and judgement and use of earthquake experience data as appropriate;
(b) To define potential failure modes for elements that are not screened out and the types of review analysis that need to be conducted;
(c) To gather installation data necessary for further analyses;
(d) To list all potential systems interaction issues that need further evaluation as related to individual SSCs, including gross failures of non-seismically designed structures and components that could cause failure of the selected SSC items.

Each item is to be reviewed by at least two members of the SRT. Decisions to screen out items need to be unanimous. Otherwise, concerns have to be documented on walkdown forms for further review. All decisions to screen out are documented on walkdown forms. One form of the seismic capacity screening criteria is contained in Ref. [11]. Tables 2-3 (for civil structures) and 2-4 (for equipment and subsystems) of Ref. [11] contain inclusion rules and applicable caveats for the screening. It should be noted that ground motion levels in terms of the 5% damped peak spectral acceleration are used in the screening criteria because the spectral acceleration is a better descriptor of the potential for earthquake damage than the PGA. Note that these screening tables assume that anchorage capacity and spatial interaction issues are assessed separately.

Components that are inaccessible could be evaluated by alternative means such as photographic inspection or reliance on seismic reanalysis. If several components are similar and are similarly anchored, then a sample component from this group could be inspected for the purpose of qualifying the group. The ‘similarity basis’ is developed during the seismic capability preparatory work by reference to drawings, calculations or specifications and confirmed during the walkdown, if possible.

In addition to the detailed walkdown of items on the selected SSCs, area walkdowns to evaluate the potential for seismic fire and seismic flood issues are necessary. Seismic fire walkdowns need to be performed by a team of seismic capability engineers supplemented by an expert in fire issues, including ignition sources, combustibles, fire extinguishing systems, capacity of fire barriers, fire and smoke capacities of SSCs. Similarly, seismic internal flood walkdowns need to be performed by a team of seismic capability engineers supplemented by an expert in internal flood issues.

A major part of the walkdown is devoted to the evaluation of equipment anchorage, which typically consists of expansion bolts installed in concrete, cast in place bolts embedded in concrete, and welds to steel members embedded in concrete. Generic anchorage calculations for typical anchorage configurations and equipment types need to be made prior to the walkdown in order to assist the SRT with making screening decisions in the field. All anchorage for equipment needs to be analysed by either generic bounding or by analysis for individual equipment items. Generic bounding evaluation of equipment is preferred since it can be used to screen out whole classes of equipment. This minor effort performed prior to walkdowns ultimately saves time by narrowing the scope of the SMA work (see Section 5.1.2.8).

The walk-by of distribution systems, such as piping, cable trays, conduit, and heating, ventilating and air-conditioning ducting, need to be handled on an area basis (i.e. experienced seismic capability engineers, with realistic in-structure seismic demand results in hand, review the installations on an area basis). All of the accessible areas have to be visited. If seismic concerns are identified, more detailed assessments need to be performed. Major potential issues for distributions systems are due to non-seismically designed or detailed distribution systems whose failure could induce failure of selected SSCs, for instance non-seismically designed or specified fire piping systems.

For each of the elements that are not screened by the SRT walkdown and for each spatial interaction issue raised by the SRT, it may be necessary to gather field data. The amount of data to be gathered is dependent upon the amount of documentation that exists prior to the walkdown. The level of existing documentation is established during the seismic capability walkdown preparatory phase. In particular, the SRT will determine during the walkdown whether the documentation accurately describes element anchorage details and seismic support details. If discrepancies are found, they are noted for further evaluation.

Documentation of the walkdowns is essential. Seismic evaluation work sheets (SEWS) are one form of documentation of the walkdowns [11, 40]. SEWS are structured forms, one for each item in the component list (selected SSCs), that need information entered about the component being evaluated, such as:

(a) Name, type and manufacturer;
(b) Physical condition;
(c) Its function during and/or after the earthquake motion;
(d) Seismic demand, anchorage and attachments;
(e) Seismic systems interaction hazards;
(f) Important caveats to be met for applicability of the earthquake experience database and generic test databases.

SEWS need to contain field notes and photographs (see Appendix V for a sample SEWS). A guiding principle for recording observations and decisions is that if more than two minutes are spent on the evaluation, notes need to be made. The SEWS may be completed in two phases, with the completion of a screening verification data sheet (SVDS) as Phase 1. This Phase 1 effort is also referred to as 'walk-by', which looks for outliers, lack of similarity, anchorage that is different from that shown on drawings or prescribed in criteria for that component, potential systems interaction issues, situations that are at odds with the team members' experience and other areas of seismic concern (see Appendix V for a sample SVDS). If concerns exist, then the limited sample size for more thorough inspection in the Phase II preparation of the SEWS needs to be increased accordingly. As an alternative to the two phase approach, SEWS may be completed for all items on the selected SSC list.

5.1.2.9. Relay chatter review

In the context of seismic assessments, the concept of relay is more general than in electrical engineering. For seismic purposes, a relay is any electromechanical device (as opposed to purely mechanical or manual), with movable contacts, which could change state ('chatter') during seismic shaking. Typical examples are devices that use coils to open or close contacts following low voltage DC or AC electrical signals. They are used for control logic or circuit protection (e.g. motor starters fall into this extended concept of relays).

When a full scope SMA is carried out, the system analysis described in Section 5.1.2.3 needs to provide a list of 'essential' relays. In this context, a relay is 'essential' if contact chatter during the earthquake can prevent a safety action from taking place during or after the earthquake. Relays correctly mounted on properly anchored cabinets can be assumed not to be damaged by the earthquake if the cabinet has sufficient structural capacity. Contact chatter does not physically damage the relay.

Typically, there are only a small number of essential relays within a plant. However, the effort necessary to identify those relays can be very substantial. Therefore, it is common practice that the list of relays coming out of the systems analysis is an envelope of the actual list. Only when a relay is found to be of low seismic capacity, is it checked to determine whether it is actually an essential relay. Relay evaluation in the context of SMA can be made following EPRI guidance (see Refs [11, 42]).

There is no documented seismic experience about relay chatter. Hence, relay HCLPF capacities need to be obtained from seismic qualification test results or from generic capacity reports (e.g. generic equipment ruggedness spectra [43]). To confirm HCLPF capacities, a relay walkdown has to be performed by the SRT in order to verify that the relays are mounted in the same way as in the qualification tests and to exclude the possibility of seismic spatial interactions.

5.1.2.10. Seismic capacity evaluation

At the completion of the walkdowns, a relatively small list of items is expected to remain for which a detailed review is needed. For these items, the SRT needs to document what needs to be reviewed (e.g. anchorage, support details, seismic qualification test data). Experience has shown that most of the SMA work will be concerned with support and anchorage details.

For those components that need to be reviewed, the realistic seismic demand associated with the RLE will be available from the results of the work described in Section 5.1.2.6. This seismic demand will be specified in terms of in-structure (floor) response spectra at the base of the component. Once this demand is established, the next step is to compare it to the demand used in the seismic qualification of the component (i.e. the SL-2 required response spectrum). When the RLE demand, throughout the frequency range of interest, is less than or approximately equal to the design demand for which the component has been previously designed and qualified, no further work is necessary to demonstrate capability to withstand the RLE.

In those instances where the RLE demand significantly exceeds the design demand in an important frequency range, or where the component has not had previous seismic qualification, seismic HCLPF capacity evaluations for the component are necessary. Capacity evaluations can be performed analytically for items such as equipment anchorage and components designed by analysis, or can be performed by comparison with generic equipment qualification or fragility test data for functional failure modes of electromechanical equipment. If an analysis is necessary to determine the seismic HCLPF capacity of a component, the conservative deterministic failure margin (CDFM) approach discussed in Ref. [11] or the fragility analysis method may be used.

The CDFM used in determining a HCLPF capacity by analysis needs acceptance criteria. Such criteria are less conservative than design basis acceptance criteria, in that best estimate material properties, rather than specified minimum values, may be used and stress-strain behaviour into the inelastic region for ductile type failure modes are permitted. One such inelastic behaviour criterion is given by the use of inelastic energy absorption factors, F_{μ}, as defined in the standard ASCE/SEI 43-05 [30]. Inelastic energy absorption factors for evaluation of existing nuclear installations are presented in Appendix II.

Another possibility for inelastic behaviour criteria is given by the use of strain or displacement limits, which normally necessitates non-linear analysis. For structural systems, displacement limits are commonly specified as allowable drift ratios, to be compared with computed interstory drifts (see Appendix II for drift ratios for evaluation of existing nuclear installations). HCLPF capacities are documented for all elements in the primary and alternate success paths that have capacities less than the specified RLE. The element with the lowest HCLPF capacity in a success path establishes the seismic HCLPF capacity for the path. The higher seismic HCLPF capacity of the primary and alternative success paths is the seismic HCLPF capacity of the installation as a whole.

For nuclear power plants with two success paths, if at least one path can mitigate a small LOCA, but the small LOCA mitigation path has a higher HCLPF capacity than the other path, the HCLPF capacity of this path becomes the plant HCLPF capacity. However, in the case where only one success path can mitigate a small LOCA and this path also has a lower HCLPF capacity than the other path, then the plant HCLPF capacity is governed by the small LOCA success path HCLPF capacity.

5.1.3. Enhancements

In addition to the criteria outlined in the previous subsections, the following enhancements to the SMA may be necessary:

(a) Selection of alternative success paths: The regulatory body may determine how many alternative success paths are necessary to add redundancy to the process (e.g. for nuclear power plants, the NRC required two paths to be evaluated with at least one of the two adequate to mitigate a small LOCA). One approach is to identify several potential success paths and then select one or more from the total.
(b) Treatment of non-seismic failures and human actions: The identification of non-seismic failures and human actions in the success paths may be necessary. The success paths are chosen based on a screening criterion applied to non-seismic failures and human actions needed. It is important

that the non-seismic failures and human actions identified have low enough failure probabilities so as not to affect the seismic capabilities of the success paths.

(c) Evaluation of containment and containment systems: For nuclear power plants, the identification of vulnerabilities that involve early failure of containment functions including containment integrity, containment isolation, prevention of bypass functions and some specific systems that are included in the success paths may be necessary.

5.1.4. Documentation

Typical documentation of the results of the SMA is a report with the following:

(a) Methodology and assumptions of the assessment;
(b) Selection of the RLE;
(c) Composition and credentials of the SRT;
(d) Verification of the geological stability at the site;
(e) Detailed system descriptions used in developing the success paths, system notebooks and other data;
(f) Success paths selected, justification or reasoning for the selection, HCLPF capacity of path and controlling components;
(g) Walkdown report summarizing findings and system wide observations, if any;
(h) Table of selected SSCs with screening (if any), failure modes, seismic demand and HCLPF values tabulated;
(i) Operator actions needed and the evaluation of their likely success;
(j) Containment and containment system HCLPF capacities for nuclear power plants (if necessary);
(k) Treatment of non-seismic failures, relay chatter, dependences and seismic induced fire and flood.

5.2. SEISMIC PROBABILISTIC SAFETY ANALYSIS[11]

5.2.1. General

As stated in Section 2, the goal of an SPSA for a nuclear power plant is to provide an estimate of the overall frequency of a predetermined plant damage,

[11] This section is based on Ref. [2].

such as CDF or LRF, initiated by an earthquake. Hence, in contrast with the SMA method, the SPSA provides results that allow integration of the seismic risk with the risk resulting from internal or other external events.

The SPSA methodology has evolved since the 1990s along with the development of PSA methodologies for internal events (see Refs [8, 9, 12, 15, 44, 45] on various stages of development and ASME/ANS RA-Sa–2009 [16]). Reference [44] details the implementation of the SPSA methodology. To perform an SPSA, a PSHA for the site of interest is necessary (see Section 4.1). Paragraph A–4 of NS-G-2.13 [1] recommends that "It is helpful to have the results of the probabilistic seismic hazard analysis at the initial stage [of the programme] to guide the evaluation. If these results are not available at the start, they need to be available shortly thereafter", since they are needed for the screening of SSCs and computation of the seismic structural response. Paragraph 5.24 of NS-G-2.13 [1] states:

> "The system models of the internal event PSA should be modified for initiating events and for the responding system behaviour, that is, the front-line and support systems that are called into action to prevent the progression of the initiating event to core damage or to other undesirable end states."

A frontline system is a system that is capable of directly performing one of the accident mitigating functions (e.g. reactivity control, core heat removal). A support system is a system that provides a support function for one or more of other systems (e.g. electric power, cooling). Modelling of the initiating event and the response of systems is most often done with event trees; the responding systems are described by fault trees. Paragraph 5.24 of NS-G-2.13 [1] states that "In all cases, event trees and fault trees should be modified to account for seismic induced failures, that is, by adding basic events that represent the failure of SSCs due to seismic loading conditions." Typical initiating events for SPSAs are loss of off-site power, LOCAs of various sizes and transients. Paragraph 5.24 of NS-G-2.13 [1] concludes:

> "On the basis of a combination of engineering assessments and judgement, the experts of the assessment team should act to limit the number of initiating events to those that are credible. Fragility functions...should be derived for the SSC failure modes identified by fragility analysts. System models representing the containment [and other accident containment mitigation] systems should be appended to the sequences leading to core damage, where required."

Boolean expressions of system behaviour are developed and quantified. In summary, the SPSA comprises the following steps (see Fig. 2 and note that some of the steps have common elements with the SMA methodology):

- Seismic hazard assessment (see Section 4.1);
- Selection of the assessment team;
- Plant familiarization and data collection (see Section 3);
- Systems/accident sequence analysis leading to event trees/fault trees modelling and SSC identification;
- Determination of seismic response of SSCs for input to fragility calculations;
- Seismic capability walkdown;
- Fragility calculations for SSCs;
- Risk quantification;
- Peer review, enhancements and documentation [10, 11, 15].

The SPSA end products of interest are insights derived from the model and modelling process and the quantitative end state metrics of CDF and LERF or other failure end state. Frequently in nuclear power plants, failure of containment or containment bypass serves as a surrogate for LERF (i.e. a reasonable representative of LERF since a large release entails a breach of the containment). In summary, the SPSA end products of interest include the following:

- An appreciation of accident behaviour.
- An understanding of the most likely accident scenarios induced by earthquakes.
- Identification of dominant seismic risk contributors (components, systems, sequences, procedures).
- Seismic fragilities of SSCs and seismic margin as defined by HCLPF capacity values.
- Range of earthquakes that contribute most significantly to the seismic risk. This level is often in the range of 2–3 times the DBE.
- Seismic risk as defined by CDF or LERF as point estimates and as probability distributions representing confidence estimates.
- Comparison of seismic risk with risks from other events (internal events, fire).
- Understanding the relative likelihood of CDF and LERF due to earthquakes.
- Identification of importance of non-seismic failures and operator actions.
- Identification of potential modifications to the facility (physical and operational) and the surrounding area (physical and administrative). Quantification of risk reduction if implemented.

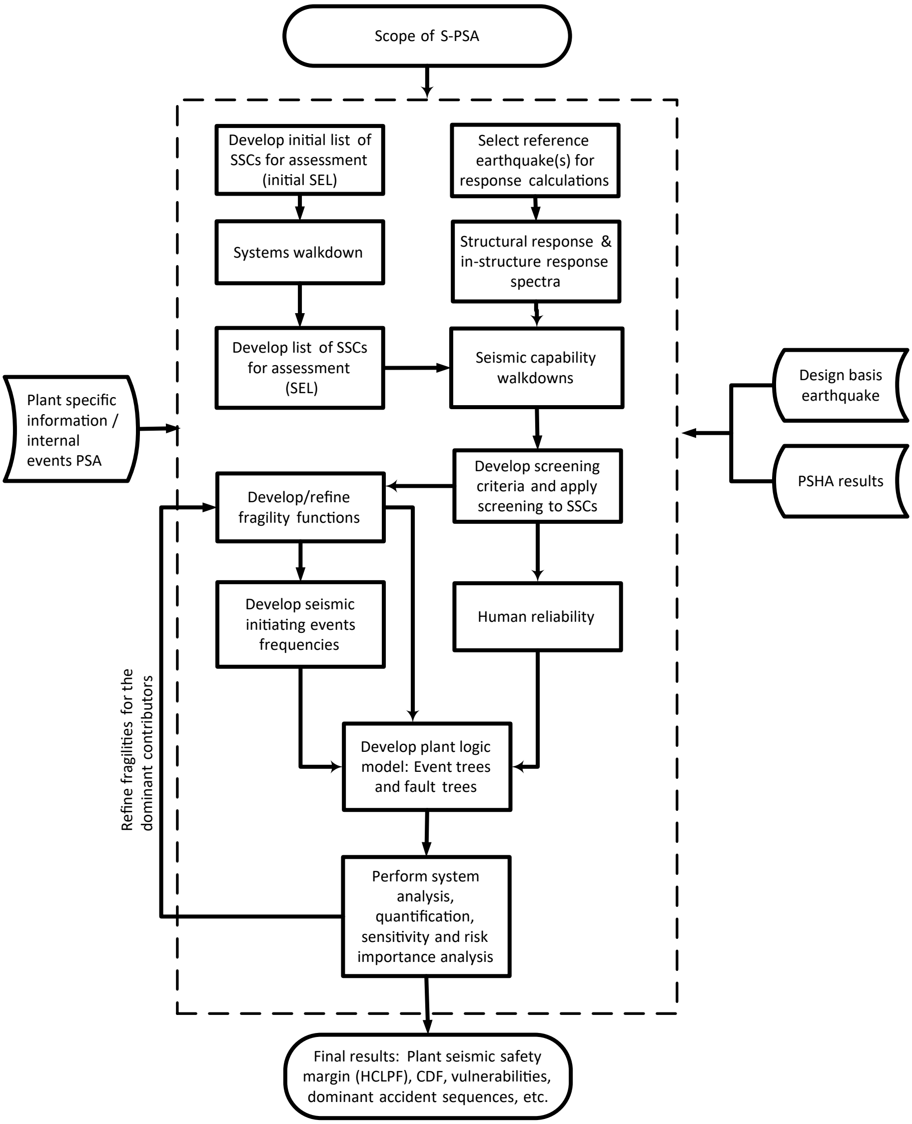

FIG. 2. General workflow of the seismic probabilistic safety assessment (SPSA) method. Note: CDF — core damage frequency; HRA — human reliability analysis; LERF — large early release frequency; OP — operator actions; PSHA — probabilistic seismic hazard analysis; SEL — selected structures, systems and components; UHS — uniform hazard spectra.

SPSA is resource intensive. For the SPSA, fragility functions of the SSCs in the event trees (initiating event frequencies) and in the fault trees are needed. These probabilistic definitions of fragility are usually developed with significant contributions from experts in the field and component failure data. In general, fragility function development involves specialized skills not necessarily available from the licensee's engineering staff.

5.2.2. Elements of the seismic probabilistic safety assessment

5.2.2.1. Seismic hazard assessment

To perform the SPSA, a PSHA for the site of interest is necessary (see Section 4.1 for a description of PSHA methodology). The PSHA is resource intensive and it is generally carried out by teams of specialized professionals (e.g. seismologists, geologists, geotechnical engineers) different from the team developing the SPSA. Paragraph 4.8 of NS-G-2.13 [1] states:

> "Generally the results of the site specific probabilistic seismic hazard assessment include seismic hazard curves defining the annual frequency of exceedance (often referred to as the annual probability) of a ground motion parameter (e.g. peak ground acceleration), associated response spectra (e.g. uniform hazard spectra) and characteristics of the dominant source parameters (e.g. magnitude and distance from the site)."

Differentiation between sources allows the selection of recorded ground motions with appropriate characteristics, if needed in the development of the seismic response task.

PSHA documentation needs to specify the control point for the ground motion defined by the hazard curves (e.g. the bedrock, a fictitious hard rock outcrop, at free field, at foundation level), since significant differences can exist from one point to the other, especially in soil sites. Generally, the SPSA will incorporate the effects of aleatory and epistemic uncertainties into the risk quantification. Hence, the aleatory and epistemic uncertainties in the seismic hazard need to be quantified.

5.2.2.2. Seismic equipment list

The first step of the SPSA is the selection of a set of SSCs for seismic capability assessment, which are then included in the selected SSC list. The selected SSCs represent the seismic basic events for which the seismic fragility parameters have to be determined. The internal events PSA component list can be used to provide the initial list of components that may potentially be important in the mitigation of seismic events. The selected SSCs include only those components relevant for the SPSA. The development of the selected SSCs is an iterative process that consists of sequences of screenings and additions:

(1) Identify risk significant components. This step involves the identification of all components modelled in the current internal events PSA. At this stage, an initial screening of components is performed. It is a generally accepted practice to remove from the selected SSCs those systems modelled in the PSA that are of low capacity or to provide a minimum mitigation potential in the PSA. These systems are usually the balance of plant systems that are not seismically designed. They usually include part of the component cooling system, instrument air, active equipment without backup power. Justification for the reduction in the selected SSCs is given by the following reasons:
   - The selected SSCs represent the equipment for which a seismic fragility will be evaluated. This effort is resource intensive and it needs to be minimized to the maximum extent possible.
   - Some non-seismically designed systems could have low seismic capacity and, consequently, could provide little reduction in the SPSA damage states frequency. In this case, it would be reasonable to keep them out of the analysis.
   - Off-site power is usually of low seismic capacity and it is beyond the boundaries of the nuclear installation. Hence, it can be reasonable to assume loss of off-site power. Since it is a controlling event for the operation of systems without backup power, these systems will play no role and can be screened out of the analysis.
   - Systems that generally need many support systems to operate could be also eliminated from the analysis when any of those support systems is considered to have a low seismic capacity. They increase the scope of the components to be evaluated while, at the same time, the many non-seismically designed support systems reduce the potential benefit of including them in the model.

   PSA model runs need to be performed to assess the value of these systems in the mitigation of seismic events. These model runs are performed on the internal PSA model and represent the conditional damage states frequencies with the systems assumed failed. This provides the PSA analyst with an order of magnitude estimate of the mitigation potential of these SSCs.

(2) Add passive SSCs needed to perform the required safety functions (e.g. tanks, heat exchangers, piping) or that may interact or produce failure of the internal PSA components (e.g. building structures). Passive SSCs are usually not included in the internal events PSA models. This activity usually

involves the review of piping and instrumentation diagrams and electrical one-line drawings.

(3) Group components. Internal events PSA models can be very detailed. They can have a separate representation of several items that are different components of the same item of equipment (e.g. limit switches, motor of a motor operated valve). For the SMA, the different components of the same equipment item need to be grouped together in a single item for seismic assessment (‘rule of the box’, see Ref. [40]).

(4) Eliminate inherently robust equipment. Passive in-line equipment may be considered rugged or more rugged than the line itself. Hence, there is no need to include these components as individual items in the selected SSC list, for example manual valves or dampers that do not need actuation (i.e. to change state) to perform the intended functions).

After these first activities, an initial selected SSC list will be available for the subsequent seismic capability tasks. Later, this list is screened on the basis of generic seismic capacities. Those components considered to be seismically rugged are screened out after the design review and plant walkdown to verify seismic ruggedness.

5.2.2.3. Structural response

The objective of this step of the methodology is to develop the seismic demand at the location of the items included in the selected SSC list. This aspect of the methodology is generally quite well developed and understood. The analyst usually starts with earthquake motions that are postulated to arrive at the local site. These motions can be in the form of uniform hazard spectra for specified frequencies of occurrence (typically between 10^{-5} and 10^{-4} and events per reactor year), provided by PSHA results. To obtain the seismic demand for items in the selected SSC list, it is necessary to develop IRS for each elevation in each important building, to represent the seismic input for each SSC for which seismic fragility calculations are necessary.

On soft soil, the soil–structure coupling can significantly affect the structure seismic response [46]: “For example, it is necessary to account for such factors as soil shear modulus and damping. Soil-structure interaction models developed over the years are quite reliable if all of the relevant site factors have been considered” (see Ref. [47]). It is important for the analyst not to directly use the models used in the design [46]:

> “these often contain conservatisms or other unrealistic assumptions which cannot serve as a realistic representation of behaviour in an actual earthquake.

> “...The analyst needs to develop a structural model for the building, unless a model developed earlier, such as in the original design or for the safety analysis report, can be relied on. ...
>
> “In developing realistic floor spectra, it is typical to use linear dynamic analysis for the structure, and then to account for non-linear effects by estimating the inelastic energy absorption capacity of each component, so that the response for the equipment item represents the floor spectrum modified to account for how each equipment item responds in frequency space. The modifications account for several factors specific to each item such as damping and modal response combination — all of which have variability which must be included in the analysis.”

Scaling of the original seismic design results (IRS, stress analysis) is acceptable if well justified with respect to the scope and objectives of the seismic re-evaluation programme. The OECD Nuclear Energy Agency (OECD/NEA) [46] conclude that “While uncertainties certainly exist in this aspect of the seismic-PSA analysis, arising from both variabilities [of analysis parameters] and modelling approximations, the analytical approaches for the several topics are all generally well-developed”. It is beyond the scope of the present document to cover the details of each aspect of the methodology (see ASCE 4-98 [47] for more information on the technical issues).

5.2.2.4. Plant walkdown

SPSA walkdowns are similar to seismic capability walkdowns in the SMA method (see Section 5.1.2.8), and they are performed and documented in the same way. Hence, the details will not be repeated here. The OECD/NEA [46] reports:

> “There is a broad consensus among seismic-PSA analysts that the plant walkdown is the most crucial aspect of the entire process. By using a well-planned and effectively executed walkdown, the analysis team can develop vital information about the plant configuration, specific spatial relationships, anchorages, and other features that cannot be found any other way. Furthermore, if a good walkdown is not performed, neither the seismic-capacity analyst nor the systems analyst can properly perform the required work.
>
> “A walkdown team usually consists of expertise drawn from at least the following areas: seismic-fragility-analysis, systems-analysis, and plant operations/maintenance. Sometimes, the walkdown teams can consist of

several representatives of each area, although having too many individuals on a walkdown can lead to a clumsy and inefficient evaluation.

> "For the seismic-capacity team, the crucial benefit of the walkdown is that they can determine, for each important item (structure or component), whether that item is 'typical' of its generic category, or somehow atypical or even unique. If it is judged to be 'typical', then information from the broad class in which the item fits can usually be used, eliminating the need for special analysis. If an unusual component or structure is identified, it can be given the special attention that it deserves."

The plant walkdown provides the basis for the final screening of the selected SSCs. Those components considered seismically rugged will either be eliminated from the assessment or receive a simplified treatment in subsequent tasks (e.g. simplified fragility analyses, grouping into surrogate elements within fault trees). The OECD/NEA [46] reports:

> "The documentation of the walkdown's findings is an important aspect, not only for archival reasons, but more importantly because the documentation is needed by both the seismic-capacity and systems-analysis engineering teams....

> "Evaluation: Because a large number of seismic-PSA walkdowns have been performed, and there exists excellent guidance on how to perform and document a walkdown, the methodology for seismic-PSA walkdowns should now be considered very mature. The guidance is sufficiently detailed, and the number of teams that have accomplished an excellent walkdown is large enough, that a new team should not have difficulty in learning how to perform a satisfactory walkdown."

5.2.2.5. Seismic failure modes and fragility analysis

Fragility analysis is an intrinsically probabilistic methodology, because it produces a probability of failure of a particular component as a function of the 'hazard parameter' expressed in PGA or spectral acceleration. The scope of fragility analysis is given, in principle, by the items included in the selected SSC list, which may contain several hundreds of items (sometimes thousands). In order to reduce the effort without affecting the quality of the results, several iterations and successive screenings are performed to identify the significant contributors to the CDF or LERF.

A first screening is carried out at the initial systems analysis stage, based on system considerations (see Section 5.2.2.2). A second screening is carried out after the plant walkdown, based on the seismic capacity of the components. For this second screening, the screening level needs to be set in terms of hazard parameters (PGA, spectral acceleration) in such a way that SSCs with capacity greater than the screening level will not have significant contribution to the CDF or LERF, and therefore all these SSCs will have fragility parameters corresponding to the screening level (surrogate element). The OECD/NEA [46] reports:

> "When analysing any specific structure or component, there are two different aspects of the analysis: the definition of 'failure' and the determination of the fragility.
>
> "...'Failure' must be defined before a seismic capacity can be determined."

The process of defining 'failure' is based on first identifying the performance criteria of the selected SSC item, including its required performance during the earthquake shaking (if any) and after the shaking has stopped. Once the performance criteria are established, fragility functions can be developed. Failure "usually does not include minor structural damage" [46]. The definition of performance criteria and [46]:

> "what constitutes 'failure' must be made by the structural analyst on a case-by-case basis, with the advice of a competent systems analyst, and taking into account the specific safety equipment and safety functions that would be vulnerable. Sometimes more than one failure mode must be considered in the analysis. The walkdown is an essential part of the engineering determination of what 'failure' means, because drawings often cannot properly capture the actual configuration of adjacent vulnerable items, nor reveal damage such as erosion that might affect a structure."

The failure of active equipment could be a recoverable malfunction (the function can be restored by operator actions, e.g. reset relays, manual start of diesels) and non-recoverable (associated to physical damage that cannot be repaired during the mission time). As with structural failures, the decision about which failure modes to consider needs to be made with the advice of a competent systems analyst. Guidance on assigning failure modes is available in the various methodology guides [11, 41, 48, 49]. The OECD/NEA [46] reports:

> "One key outcome of the multi-year effort to compile and understand earthquake-experience data is that some important categories of equipment

are now known to be generically quite rugged. This knowledge was first developed as part of the work of the Seismic Qualifications Utility Group [24]...and is also embedded in a set of screening tables for seismic capacity, that can be found in the NRC and EPRI seismic margin reports [11, 50].... Using these screening tables and the SQUG insights, fragility analysts can screen out certain items as rugged provided that various conditions are met...[and verified during the plant walkdown].”

Note that intensive use of the so called rule of the box is made when using the screening tables [40].

It became a common practice to develop first preliminary fragility parameters based on design review (design scaling), generic seismic capacity data or simplified fragility analysis and, after identification of the significant seismic risk contributors, to refine fragility functions for those contributors using detailed fragility analysis (see Refs [41, 49] on detailed fragility analysis). Simplified fragility analysis based on Kennedy’s Hybrid Method [9] is presented in Appendix IV. Detailed fragility analysis is traditionally performed only for a small number of components, usually 40–50 components.

5.2.2.6. Seismic probabilistic safety assessment systems analysis methodology

The SPSA systems analysis work is broadly similar to traditional PSA systems analysis for internal initiators. It uses the same tools and types of data and the same way of setting up the analysis and solving it numerically. Every aspect of the methodology is fully within the routine capability of PSA systems analysts. However, it is important to note that SPSA is different from an internal PSA in several important ways:

(a) Earthquakes could cause initiating events different from those considered in the internal events PSA.
(b) There are different types of failure mode for the same component. It is commonly assumed that the SSC seismic failure modes follow the most limiting case of the component failure identified in the fragility analysis.
(c) Passive systems are affected, and location of PSA components and seismic spatial interaction needs to be considered.
(d) All possible levels of earthquake together with their frequencies of occurrence and consequential damage to plant systems and components need to be considered.
(e) Earthquakes could produce relay chatter leading to spurious activation and deactivation of components and systems misalignment.

(f) Recovery actions and associated human errors need to consider specific plant conditions and operator stress level following an earthquake.
(g) Earthquakes could simultaneously damage multiple redundant components. This major common cause effect needs to be properly accounted for in the seismic risk quantification.

For the initially identified list of important SSCs, the fragility analysts provide the preliminary estimates of capacity to the systems analysts for their consideration in developing the systems models. These preliminary capacities will determine which structures and equipment are damaged by the various seismic initiating events (as a function of hazard parameter in terms of PGA or spectral acceleration). The OECD/NEA [46] reports:

> "The systems analyst must then take into account issues such as the random (non-earthquake-caused) likelihood that other vital equipment might be out-of-service due to testing, maintenance, operator error, or failure; possible correlations among failures; and the procedures used by the operators, including their ability to recover certain earthquake-damaged or failed equipment, or to substitute other equipment, or to perform the needed safety function another way.
>
> "...the systems analysts and the seismic-capacity analysts should have been working together from the start to screen out certain potential issues, develop input information on others, and help each other to focus on the issues deemed important. There will have been several iterations in any well-executed seismic-PSA study.
>
> "At the center of the systems analysis work is developing one or more accident sequence event trees, that include the various functions or systems needed for safe shutdown, possible operator prevention and recovery actions, and the like. The success-or-failure numerical values on the event-tree branch points are then worked out using either data or inputs from fault trees. If we assume that the analyst has access to a completed internal-initiators PSA (which should almost always be the situation), then direct use can be made of such vital information as the random failure data, the operating crew's procedures, and the support-system matrix."

In order to develop seismic event trees, first seismic initiating events have to be determined, and the internal events PSA event trees have to be modified to accommodate seismic initiating events. Seismic initiating events could be different from those used in internal events PSA. The failure modes are different and all need to be considered, such as.

- — Potential seismic failure of the passive systems;
- — Seismic interactions;
- — Relay chatter;
- — Simultaneous damage of equipment;
- — Correlation between different seismic induced failures and concomitant events, such as seismic induced flood and fire.

The internal events PSA initiating events list and grouping need to be reviewed for seismic considerations and for the inclusion of potentially new seismically induced initiating events. Initiating events are described by an initiating event frequency. For SPSA, the seismic initiating events are events that occur as a function of the excitation level described by the seismic hazard curve and its discretization. For instance, loss of off-site power is expected to occur starting at low excitation levels and quickly has a conditional frequency of occurrence of 1.0; small LOCA will likely occur only at high excitation levels and the frequency of occurrence will reflect this. These initiating events, if not adequately mitigated by the safety systems, may lead to unacceptable damage states to the plant (e.g. core damage). The goal is to identify the SSCs that could be linked to internal initiating events, thereby taking advantage of the existing internal event models. The internal event models of greatest interest are those that contribute significantly to the plant seismic damage states and for which detailed fragility analysis is necessary.

An example of acceleration ranges that could be selected for identifying seismic initiating events, for a facility with seismic design basis corresponding to a PGA of 0.15g, is as follows (with the damage indicated at each level of shaking given only for illustration purposes):

- — No seismic initiating event has been defined for a zero PGA less than 0.05g: Based on the experience for these seismic events, no failure will occur in nuclear installations.
- — Between 0.05g–0.15g: Potential failure of some categories of non-seismically qualified components. Loss of off-site power and relay chatter may occur.
- — Between 0.15g–0.30g: Potential failure of some categories of non-seismically qualified components and weak links of seismically qualified SSCs.
- — Between 0.30g–0.50g: Extensive failures of some categories of non-seismically qualified components and potential failure of seismically qualified SSCs. May also include potential failure of some passive systems.

— Between 0.50g–0.75g: Multiple failure of some categories of non-seismically qualified components SSCs and potential multiple failure of all active and passive systems.
— Between 0.75g–1.0g: Severe damage to most of the structures systems and components (low frequency hazard event).

Seismic initiator analysis consists of the identification of the relation between the group of seismic initiated failures (e.g. loss of off-site power, small LOCA, transients) and accident initiators transients that lead to core damage or external releases. This evaluation needs to be conducted for each acceleration range considered in the analysis. The evaluation of seismic initiators comprises the following steps:

(1) Define seismic initiators and associated frequencies as seismic events producing a PGA within specified acceleration ranges convolved with the seismic fragility of the induced failure that can develop in an accident sequence. Seismic initiating event frequencies can be calculated as:

$$F = \int_{a_1}^{a_2} \frac{\mathrm{d}H(a)}{\mathrm{d}a} F(a)\mathrm{d}a$$

where a_1 and a_2 define the acceleration range corresponding to a specific seismic initiating event.
(2) Develop general screening criteria and screening limits, based on the seismic design basis and applicable hazard curves.
(3) Analyse the group of potentially failed components by each seismic initiator and correlate them with the internal initiators (define seismic sequences corresponding to each seismic initiating event).
(4) Review seismic initiating scenarios for completeness and consider correlations between seismic induced failures (including seismic induced flood and fire).

After seismic initiating events have been selected, the following steps are necessary to complete the SPSA system analysis:

(i) Develop or adapt the existing event trees and fault trees based on defined seismic initiators and seismic basic events.
(ii) Insert the seismic basic events and associated logic in the fault tree models.
(iii) Perform system analyses and generate the Boolean equations (minimal cut sets) for each seismic initiating event.

It is important to emphasize that SPSAs typically identify not only accident sequences involving one or more seismic induced failures, but also sequences involving a combination of seismic failures, human errors and non-seismic failures, such as ‘random’ failures or maintenance unavailability. It is often found that accident sequences of this latter type are as important as the sequences involving only seismic failures. The OECD/NEA [46] reports that “If fault trees from an internal-initiator PSA analysis are used, they must be modified somewhat to account for location correlations and to introduce different seismic failure modes.”

5.2.2.7. Seismic probabilistic safety assessment database

SPSA databases need to be developed to facilitate the insertion of seismic events and logic into the internal events PSA model. SPSA databases can be developed in a spreadsheet format. The databases will ultimately be imported into an integrated external event access file, and the following database tables will need to be developed.

(a) Selected SSC/component table

A typical entry in this table needs to include the following:

— Component identification (e.g. tag);
— Building location;
— Building elevation;
— Associated basic event or gate in the internal event PSA model;
— Component description;
— Internal event PSA event or gate description;
— Seismic initiating event;
— Seismic basic event;
— Bounding fragility estimate for component;
— Detailed fragility value for component (if needed).

The following is a list of the minimum information to be contained in the selected SSC list in order to build the model most efficiently and correctly:

— SSC identification (e.g. tag);
— SSC description;
— System;
— Location;
— Median capacity;

— Composite variability;
— Description of fragility/failure mode (i.e. pump fails to run, pump fails with pressure boundary rupture, tank ruptures, wall or structure collapse).

(b) Structural interaction table

A typical entry in this table needs to include the following:

— Structure identification;
— Component identification;
— Structure description;
— Seismic initiating event;
— Structure basic event;
— Bounding fragility estimate for structure;
— Detailed fragility value for structure (if needed).

(c) Soil interaction table

A typical entry in this table needs to include the following:

— Soil location identification or soil category identification;
— Component or structure identification;
— Soil description;
— Seismic initiating event;
— Soil basic event;
— Bounding fragility estimate for soil;
— Detailed fragility value for soil (if needed).

(d) Seismic correlation table

A typical entry in this table needs to include the following:

— Seismic correlation basic event;
— Component identification;
— Seismic correlation description;
— Bounding fragility estimate for seismically correlated basic event;
— Detailed fragility value for seismically correlated basic event (if needed).

(e) Seismic initiator table

A typical entry in this table needs to include the following:

— Seismic initiating event identification;
— Internal event PSA initiating event identification;
— Seismic initiating event description;
— Internal event PSA initiating event description;
— Seismic initiating event frequency (from seismic hazard curve).

(f) Seismic human failure event table

A typical entry in this table needs to include the following:

— Internal event PSA;
— Human failure event identification;
— Seismic initiating event identification;
— SPSA human failure event identification;
— Internal event PSA human failure event description;
— Seismic human failure event description;
— Internal event PSA human failure event probability;
— Seismic human failure event probability.

The information in the selected SSC list then needs to be translated into basic events to be modelled in the seismic fault trees (see Fig. 3.).

5.2.2.8. Seismic quantification

Quantification is the last activity of an SPSA. During quantification, the fragility corresponding to each system failure and accident sequence is calculated by convolving component fragilities according to minimal cut sets Boolean equations developed by the systems analysis.

The annual probabilities (frequencies) of system failures and accident sequences are represented as the union of cut sets. The following expressions give upper bounds for the probability of a union of cut sets (the *j*-th cut set being indicated by C_j):

$$P(\text{sequence, endstate}) \leq 1 - \prod_{j=1}^{N} \left(1 - P\left(C_j\right)\right)$$

$$P(\text{sequence, endstate}) \le \sum_{j=1}^{N} P\left(C_j\right)$$

For discrete intervals of accelerations, the convolution between fragility and hazard functions is illustrated in Fig. 4.

Assuming the following seismic initiating events SI_1,..., SI_4, the convolution integral can be approximated using discreet intervals as follows:

$$P_F = \sum \left(F\left(a_{\mathrm{SI}_i}\right) \mathrm{d}a \right) \frac{H_{i+1}(a) - H_i(a)}{\mathrm{d}a} \quad \text{or}$$

$$P_F = \sum_{i=1,4} \left(H_{i+1}(a) - H_i(a)\right) F\left(a_{\mathrm{SI}_i}\right)$$

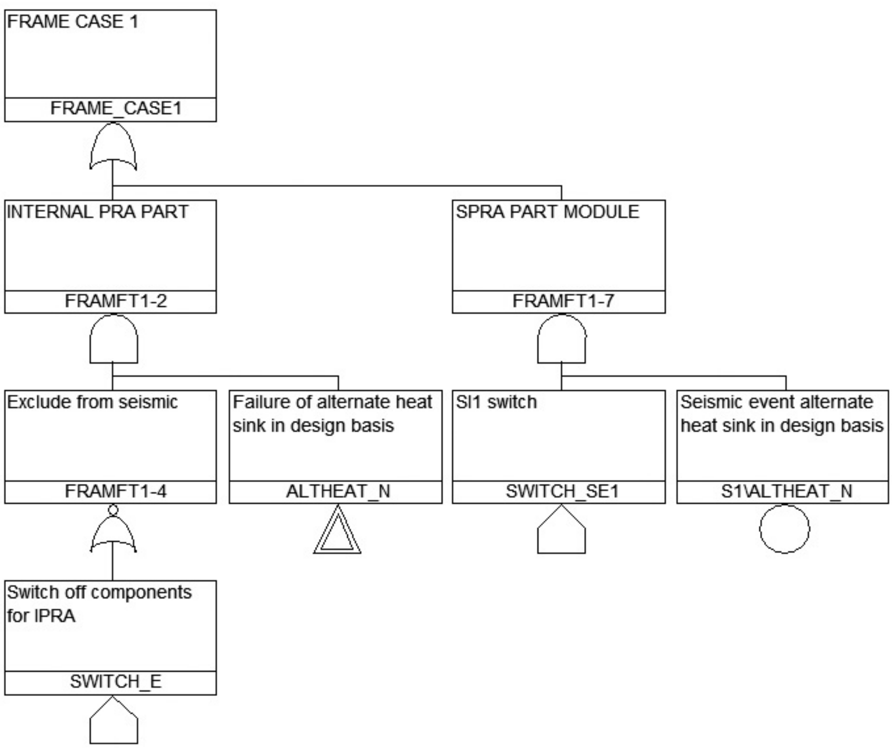

FIG. 3. Modification of fault trees for insertion of seismic basic events and associated logic. Note: IPRA — integrated probabilistic risk assessment; PRA — probabilistic risk assessment; SPRA — seismic probabilistic risk assessment.

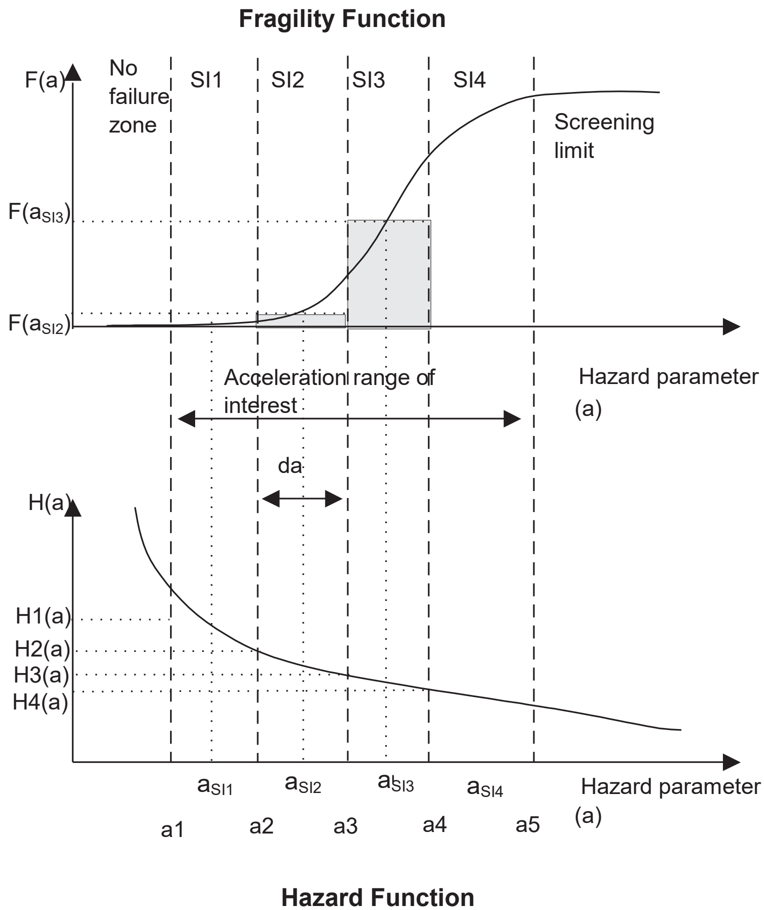

FIG. 4. Convolution between fragility and hazard functions.

where $F\left(a_{\mathrm{SI}_i}\right)$ represents the conditional probability of failure corresponding to SI_1,…, SI_4, and $H_{i+1}(a) - H_i(a)$ represents the seismic initiating event frequency for SI_1,..., SI_4. The plant level fragility curves can be evaluated by combining the component fragility curves according to a Boolean summation of the relevant cut sets. The CDF distribution is obtained by convolving the plant level fragility function with the derivative of the hazard function. For some applications, if only the point estimate of the CDF is needed, the process can be simplified significantly and may become similar to internal events PSA quantification.

5.2.3. Special issues

5.2.3.1. Correlations among failures

The OECD/NEA [46] reports:

"Typically, the PSA analysis will assume complete correlation in the response for nearby and similar equipment that is subject to the same floor motion. However, different equipment types, even if located in close proximity, are usually assigned only minor (if any) response correlation. Furthermore, even high response correlation does not always imply high capacity correlation, which would arise most obviously when, for example, two valves come from the same manufacturer and the same assembly line, with adjacent model numbers.

"The difficulty is that there is only very limited experimental information on correlations, from either testing or actual earthquakes, upon which to rely. ...[12]

"To overcome the problem, the usual fallback approach is to perform a sensitivity analysis, for example assuming complete correlation and then complete independence and ascertaining what difference these two assumptions make."

Variability in response and correlations are developed based on the results in Ref. [51]. The rules for assigning response correlation are shown in Table 2.

The NRC [51] reports:

"The correlation between any two component failures is computed from the following expression:

$$\rho_{12} = \frac{\beta_{\mathrm{R1}}\beta_{\mathrm{R2}}}{\sqrt{\beta_{\mathrm{R1}}^2 + \beta_{\mathrm{F1}}^2}\sqrt{\beta_{\mathrm{R2}}^2 + \beta_{\mathrm{F2}}^2}}\rho_{\mathrm{R1R2}} + \frac{\beta_{\mathrm{F1}}\beta_{\mathrm{F2}}}{\sqrt{\beta_{\mathrm{R1}}^2 + \beta_{\mathrm{F1}}^2}\sqrt{\beta_{\mathrm{R2}}^2 + \beta_{\mathrm{F2}}^2}}\rho_{\mathrm{F1F2}} \qquad (2\text{-}16)$$

[12] The methodology for coping with correlations is well developed (see Ref. [44]).

in which:

ρ_{12}	=	correlation coefficient between the failures of components 1 and 2
β_{R1}, β_{R2}	=	standard deviations of the logarithms of the response of components 1 and 2
β_{F1}, β_{F2}	=	standard deviations of the logarithms of the fragilities (capacities) of components 1 and 2
ρ_{R1R2}	=	correlation coefficient between the responses of components 1 and 2
ρ_{F1F2}	=	correlation coefficient between the fragilities (capacities) of components 1 and 2”.

5.2.3.2. Post-earthquake operator response and human errors

Operator actions explicitly modelled in the SPSA model need to be identified and reviewed. Human error probabilities may need to be re-evaluated

TABLE 2. RULES FOR ASSIGNING RESPONSE CORRELATION
(table 2-1 of Ref. [51], ZPA — zero period acceleration)

Rule	Text
1	Components on the same floor slab and sensitive to the same spectral frequency range (i.e. ZPA, 5–10 Hz or 10–15 Hz) will be assigned a response correlation = 1.0
2	Components on the same floor slab and sensitive to different ranges of spectral frequencies will be assigned a response correlation = 0.5
3	Components on different floor slabs (but in the same building) and sensitive to the same spectral frequency range (i.e. ZPA, 5–10 Hz or 10–15 Hz) will be assigned a response correlation = 0.75
4	Components on the ground surface (outside tanks, etc.) need to be treated as if they were on the grade floor of an adjacent building
5	Ganged valve configuration (either parallel or series) will have a response correlation = 1.0
6	All other configurations will have a response correlation equal to zero

with consideration of the impact of seismic conditions on the ability of the operator to perform several actions. Human error probabilities may need to be adjusted by factors, based on consideration of the following:

- Additional workload and stress (above that for similar sequences not caused by seismic events);
- Uncertainties in event progression (e.g. cue availability, timing concerns);
- Effect of seismic failures on mitigation and on response actions and recovery activities (e.g. accessibility restrictions, possibility of physical harm, local operator actions might no longer be possible, manual action might not be possible due to failure of specific components);
- Specific operator action aids and training (e.g. procedures, training exercises).

Revision of modelling of operator actions can include the following activities:

- Perform cut sets reviews to confirm the validity of the inserted seismic logic structures;
- Perform initial human failure analysis to identify human failure dependencies and determine more realistic probabilities for human failure events and recoveries;
- After re-quantification, perform importance and sensitivity analyses to determine risk significant components, structures and human actions;
- Use the importance and sensitivity analysis results to develop a prioritized list of components and structures that need detailed fragility analysis;
- Use the importance and sensitivity analysis results to develop a prioritized list of human failure events that need detailed human failure analysis and recovery analysis.

5.2.3.3. Relay chatter

Relay chatter assessment within SPSA follows basically the same process given in Section 5.1.2.9 for the SMA method. Systems analysis needs to provide the list of essential relays to be considered in the study. Failure of these relays needs to be considered in the fault trees that model the plant logic.

In the context of the SPSA method, relay chatter is a failure mode for which a fragility analysis needs to be made. Fragility analysis for relays follows the guidelines of Section 5.2.2.5. However, since relay chatter is a functional failure, fragility analysis needs to be based in the results of tests, either specific or generic (e.g. generic equipment ruggedness spectra [43]). The consistency between test conditions (e.g. position within the cabinet, orientation) and actual plant conditions is checked during the relay walkdown.

5.3. PSA BASED SEISMIC SAFETY ASSESSMENT

The PSA based SMA (NRC method [10]) was developed as a semiprobabilistic simplification of the full SPSA method. The main difference to the deterministic SMA (EPRI method [11]), described in Section 5.1, is the system analysis philosophy. The PSA based SMA method works in the 'failure space'. It uses the event tree/fault tree approach to delineate accident sequences. SSC selection and the computation of plant margin are based on the identified accident sequences. On the other hand, the deterministic SMA method works in the 'success space'. It uses the concept of 'success path' for the selection of SSCs and the computation of the seismic margin.

The core of the methodology remains the same for both methods, that is, the selection of the RLE (see Section 5.1.2.1), the review of plant seismic design information (see Section 5.1.2.5), the development of realistic IRS (see Section 5.1.2.6), the seismic capability walkdown (see Sections 5.1.2.7 and 5.1.2.8) and the relay chatter review (see Section 5.1.2.9) are basically the same in both methods.

The effort needed to develop a PSA based SMA is less than the effort needed for a full SPSA. However, it is larger than the effort needed for a deterministic SMA. The payback is that the PSA based SMA gives a better insight about the contributions to the seismic risk and it allows for a consistent consideration of random failures and human errors. In the following, only the differences with the deterministic SMA methodology described in Section 5.1 are discussed.

In recent years, the PSA based SMA method has been used extensively to justify seismic safety margins of new designs before the nuclear installation is actually built. For those cases, the RLE is set equal to the seismic design response spectra scaled by a factor corresponding to the target seismic margin (e.g. 1.5 or 1.67). Since the PSA based SMA method has been applied only to a very few existing nuclear installations, the following mainly refers to the application to new standard designs.

5.3.1. System and accident sequence analysis

Xu [20] reports:

> "The design-specific system and accident sequence analysis for a PRA-based SMA can be performed consistent with the Capability

Category I requirements of Section 5-2.3 of the ASME/ANS PRA standard [16], with the exceptions that the analysis should not be based on site- and plant-specific information and that it should not rely on an as-built and as-operated plant. ...The analysis should consider random equipment failures, seismic interactions, and operator actions as applicable.

"The plant systems analysis must focus on those sequences that lead to core damage or containment failures".

Examples include:

— All seismic induced initiators (transients, LOCA of various sizes, or others appropriate to the standard design);
— Complete logic structures, enhanced from internal event/fault trees to capture seismic failures;
— Consideration of non-seismic failures;
— Fully developed sequences important for CDF and LERF.

Xu [20] reports:

"In addition, the analysis should consider at-power (full-power), low-power, and shutdown modes. Note that the intent of the term 'all sequences'... is to capture significant contributions to plant-level seismic risk. Notes to Table 5-2.3-2(a) of the ASME/ANS PRA standard [16] provide some specific guidance on identifying seismic-caused initiating events based on past seismic PRA experiences. These initiators could be used as a starting point for developing associated seismic accident sequences for standard designs. In general, the design of SSCs for standard designs should have accounted for the risk-significant sequences for potential sites. This implies that failure of safety-related SSCs should most likely control the seismic sequences developed in the PRA-based SMA for a standard design. However, if the system analysis identifies a seismic sequence as important based on operating experiences in past seismic PRAs and if the seismic capacity of the sequence is controlled by nonsafety SSCs, further investigation may be required, which may lead either to design changes to upgrade these SSCs to safety-related SSCs or to an enhanced treatment to ensure an adequate sequence-level HCLPF capacity.

"A seismic equipment list (SEL) should document the SSCs associated with the accident sequences that will require seismic fragility evaluation for determining sequence-level HCLPF. As inferred from the previous paragraph, the SEL developed for standard designs should most likely contain the safety-related SSCs."

5.3.2. Fragility analysis

Xu [20] reports that screening of rugged SSCs can be performed based on the seismic design response spectra with its peak ground acceleration (PGA) scaled by a factor of corresponding to the target seismic margin (e.g. 1.5 or 1.67) and that "The basis for the screening needs to be adequately documented". After screening out rugged components, the seismic fragility evaluation of SSCs in the selected SSC list can be performed using Kennedy's hybrid method [9]. More detailed fragility analysis (i.e. capability category II requirements of Ref. [16]) needs to be used for the identified significant contributors that control the plant HCLPF.

Seismic input needs to account for the structural amplifications caused by the supporting structures, including soil–structure interaction effects and supporting systems and to incorporate an additional seismic margin factor, as appropriate. Xu [20] reports:

> "When applicants use generic data (such as test data, generic seismic qualification test data, and test experience data) to support the seismic fragility analysis, they should provide justification to demonstrate that the generic data are consistent and applicable to SSCs within the scope of the certified design application. For equipment on the SEL that must be qualified by seismic qualification tests, the applicant can use the procedure described in E.5 of EPRI Report 1002988, 'Seismic Fragility Application Guide,' issued December 2002 [48], for developing fragilities; however, the probability of failure at a ground motion equal to 1.67 times the CSDRS, including consideration of testing uncertainties, should be less than 1 percent. Note that the numerical value for the plant HCLPF is determined at the sequence level, not at the component level. Therefore, given the component and system redundancies, only those components in the cutsets whose capacities are deemed to control the sequence-level HCLPF capacity must meet the numerical limit of 1.67 times the CSDRS.
>
> ....
>
> "In accordance with DC/COL-ISG-020 [13], the HCLPF value for an SSC should be determined corresponding to a 1-percent failure probability on the mean fragility curve. ...The plant-level HCLPF capacity should be determined based on the sequence-level HCLPF values for all sequences as

identified in the design-specific plant system and accident sequence analysis. The NRC considers the Min-Max method [10] acceptable for computing sequence-level HCLPF values[13]. The plant-level HCLPF is therefore the lower bound of the sequence-level HCLPF values."

5.4. OFF-SITE SEISMIC INDUCED EFFECTS

The previous sections focus on the safety assessment against on-site effects. However, the seismic safety assessment would not be complete without considering potential off-site effects These include mainly flooding as the result of tsunamis, seiches and upstream dam failures. However, other effects such as large landslides or seismically induced accidents in nearby facilities are also possible.

The team performing the seismic assessment following any of the methods described in Section 5 needs to assess potential off-site seismic concerns using available data (see Section 3.4.1.2). When an off-site effect is considered to be potentially relevant, the vulnerability to the effect will normally be analysed by other specialists. This is the case, for example, with seismically induced external floods or external fires. Further information on assessing vulnerability to these induced effects can be found in Ref. [52].

6. MODERATE AND LOW HAZARD INSTALLATIONS

6.1. CATEGORIZATION

As discussed in Section 3.2, nuclear installations are typically divided into high, moderate and low hazard categories, as a function of their potential unmitigated releases of radioactive material or waste as it affects the health of installation and site workers, the public and the environment.

High hazard installations are those where unmitigated release of radioactivity would likely have unacceptable consequences for workers, for the public or for the environment. They are typically those installations where there

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[13] Following the min-max approach, the plant level HCLPF capacity is determined from component level HCLPFs using the Boolean expression for seismically induced core damage and taking the minimum HCLPF among 'OR' events and maximum HCLPF among 'AND' events.

is a potential for reactor core melt and hydrogen generation, either inside a power reactor or outside the reactor or containment structure, with a large radioactive inventory. Seismic safety evaluation of existing high hazard installations needs to follow the guidelines given in Section 5.

Typical moderate nuclear hazard installations include:

(a) Research and isotope production reactors and installations storing, processing, examining and testing nuclear material or waste with inventories below the high hazard category defined national regulatory authorities;
(b) Installations for storage or processing of spent nuclear fuel more than 3–5 years old (collocated either at nuclear power plants or at independent storage or processing installations), including installations for storage of spent fuel that needs only passive or natural convection cooling;
(c) Storage or processing facilities for nuclear material in the nuclear fuel cycle (e.g. conversion facilities, uranium enrichment facilities, fuel fabrication facilities, reprocessing plants) not in the high or low hazard categories.

Low hazard nuclear installations include installations with low levels of radioactive waste or radioactive material and typically include low level waste burial sites, incinerators and installations of the nuclear fuel and isotope production cycle not in the high or moderate hazard categories (see Ref. [53] for quantitative criteria that can be used for the categorization of nuclear facilities).

The hazard categorization of an installation generally defines the highest safety classification or seismic categorization of SSCs contained within the installation. As a result of the three installation hazard categories, high, moderate and low, a graded approach is typically used for developing the seismic loads, acceptance criteria and analytical procedures applied to the evaluation of existing controls and SSCs located in the three different hazard categories as well as administrative procedures and controls which protect individuals or the environment from accidental unmitigated release of large and dangerous quantities of highly radioactive materials or waste.

An example of seismic design categories (SDC) used in the United States of America is presented in Appendix I. Hazard categorization of a US nuclear installation as high (SDC-5, SDC-4), moderate (SDC-3) or low (SDC-2, SDC-1) uses a graded approach as a function of the installation construction quality and characteristics of their radioactive inventory and potential for a radioactive release [30].

6.2. SELECTION OF METHODOLOGY

In principle, evaluation of moderate and low hazard installations can use any of the methods presented for high hazard installations (see Section 2.2). In addition to the considerations in Section 2.2.4, the following points need to be taken into account when selecting the methodology:

(a) It is unlikely that an internal events PSA is available for a moderate or low hazard installation. In that case, the systems analysis part of the methods will have to be developed from scratch.
(b) With regard to seismic hazard, it is also unlikely that a full scope site specific PSHA is available for a moderate or low hazard installation. As a consequence, assessment of seismic risk metrics will only be possible in an approximate way.
(c) When installation seismic safety depends exclusively on passive functions, such as maintaining structural integrity or pressure boundaries, SMA methods usually provide the most appropriate and cost effective approach.

In the following sections, it is assumed that the SMA method is selected.

6.3. REVIEW LEVEL EARTHQUAKE

As discussed in Section 4.2, the RLE is not a new design earthquake. It is usually set higher than and different from the earthquake that was specified in the original design of the nuclear installation. The RLE needs to be sufficiently larger than the DBE to ensure that the SMA challenges the capacity of the SSCs so that the HCLPF capacity of the installation can be determined and the ‘weak links’ can be identified.

If a site specific seismic hazard assessment is available, the guideline for moderate hazard installations is that the RLE be defined with a mean probability of exceedance of around (1–4) × 10^{-4} per reactor year. For low hazard installations, the guideline is a mean probability of exceedance of around (0.4–1) × 10^{-3} per reactor year for the RLE. In most States, the seismic design requirements applicable to conventional building structures, distribution systems and components are typically associated with life safety, and mean probability of exceedance of the specified seismic action vary in the range of (0.4–2) × 10^{-3} per reactor year.

When a site specific hazard assessment is not available, the RLE may be selected based on site independent or broadly site dependent (rock, medium soil, soft soil) response spectral shapes, such as those of Ref. [36]. A zero PGA to anchor the spectral shape can be selected by careful extrapolation of national seismic hazard maps included in regular building codes, taking into account that those maps are usually given for probabilities of exceedance of about $(0.4–2) \times 10^{-3}$ per reactor year.

6.4. SELECTION OF SSCS TO BE EVALUATED

Selection of SSCs for seismic evaluation needs to be based on the identification of the safety functions needed to prevent significant releases of radionuclides. SSCs that perform a safety function and are therefore needed to reach a desired end state after the earthquake need to be selected and evaluated for their capacity to resist the RLE demand. Hence, the process is parallel to the one described for high hazard installations in Section 5.1 and it can also be based on the identification of ‘success paths’ for achieving safe and stable conditions. The final outcome of the activity is the selected SSC list for seismic capacity assessment.

The design of high hazard installations generally includes a minimum of two physically separated redundant safety trains each capable of performing the main safety functions. Sometimes, only one train is needed for the evaluation to meet the RLE. No such redundancy is necessary in the design or evaluation of moderate/low hazard category installations.

6.5. SSC EVALUATION AND WALKDOWN

The seismic capability walkdown is also to be considered an integral part of the program of seismic safety evaluation of SSCs for moderate and low hazard installations. No grading of the walkdown procedures that constitutes a part of the methodology for seismic safety evaluation is considered. The plant walkdown procedures of Section 5.1.2.8 need to be applied.

Walkdowns can also play an additional role in the documentation of the seismic adequacy of SSCs for installations where no seismic design or analysis has been performed or for which modifications have been implemented without adequate design documentation.

6.6. SEISMIC MARGIN ASSESSMENT

As for high hazard installations, after the walkdown a list of SSCs will remain for which a detailed review is necessary. The remaining SSCs will have been screened out from the assessment. For SSCs not screened out, the SRT needs to document exactly what needs to be reviewed (anchorage, support details, seismic qualification test data).

For those components that need to be reviewed, a realistic seismic demand associated with the RLE needs to be available. This seismic demand will be specified in terms of in-structure (floor) response spectra at the support points of the distribution system or component. Once this demand is established, the next step is to compare it with the HCLPF capacity of the component. For distribution systems or components supported at two or more points, it is normally necessary to consider differential displacements of the support points (seismic anchor motion) as well as seismic inertia stresses developed in the distribution system or component.

For moderate and low hazard installations, procedures for computing HCLPF capacities are the same as those for high hazard facilities (see Section 5.1). However, acceptance criteria can be different, since the necessary performance is generally different. HCLPF capacities are documented for all elements in success paths that have capacities less than the specified RLE. The element with the lowest HCLPF capacity in a success path establishes the seismic HCLPF capacity for the path and for the installation as a whole.

For moderate hazard nuclear installations, it is also necessary to determine the properties of the foundation media (as shown in Appendix I). This is typically done by use of in situ blow count testing and borehole sample content in laboratory evaluations. It generally does not involve down hole or cross hole in situ foundation media testing for shear wave velocities as would be performed for high hazard installations.

The classification of the foundation media is then used to modify generic seismic ground response accelerations at a standardized damping value (i.e. 5% critical damping). For SSCs in moderate and low hazard installations, inelastic response to earthquake motion is generally permitted, since moderate inelastic deformation is usually compatible with the performance of the required safety functions (e.g. seismic design categories SDC-3, SCD-4 and SDC-5 in Ref. [30]; see Appendix I).

For some mechanical pressure retaining distribution systems and components, pseudoelastic stresses beyond yield stresses are permitted, as is the case for Service Levels C and D as specified in the ASME Boiler and Pressure Vessel Code [31] for the design basis SL-2 earthquake as well as for the evaluation basis RLE.

6.7. SEISMIC INSTRUMENTATION

In general, there is no need for strong motion earthquake recorders at moderate and low hazard nuclear installations. However, experts strongly recommend their use, at least in the free field of the site, in order to facilitate a decision to shut down or restart the facility after an earthquake is felt.

7. CONSIDERATIONS FOR UPGRADING

As a result of the existing installation seismic evaluation, there may be a subset of the SSCs selected for evaluation that do not meet the acceptance criteria for the newly evaluated RLE (SMA) or that have a very significant contribution to seismic risk (SPSA). This information needs to provide the basis for decision making as to the necessity of performing physical upgrades of SSC installation and updating its documentation. These upgrades need to be prioritized as a function of their importance to safety, with consideration of the cost–benefit for the installation, for implementation purposes.

For installations that were not originally seismically designed, or for which seismic design played a relatively minor or unimportant role, or for any of the reasons indicated in Section 2.1.2, an ‘easy fix programme’ needs to be developed to address easily identified vulnerabilities within a short time. In such a programme, plant wide upgrades are instituted, such as simple positive anchorage of all safety related equipment or additional lateral bracing for safety related mechanical or electrical components or distribution systems where in lateral bracing spacing exceeds around three times the deadweight support spacing recommended by the distribution system design codes or standards. Seismically induced physical interaction effects between safety or non-safety and lower safety class SSCs also need to be evaluated.

Modifications need to be designed in accordance with currently applicable nuclear safety related SSC construction codes and standards for high and moderate hazard nuclear installations. For low hazard installations, national conventional construction codes and standards may be used — except that the RLE earthquake induced forces are used rather than the building code earthquake. For the design of modifications of seismic category 1, 2 or 3 SSCs (see NS-G-1.6 [3]), the seismic input, which defines the seismic demand, may be different from that defined by the national building code for the design of conventional structures.

The design for seismic upgrades needs to consider the available layout, space and maintenance needs and the working environment (radiation and chemical exposure). Upgrade concepts also need to accommodate the existing configuration, to the extent possible, and the in-service examination testing and maintenance requirements for any upgrades.

7.1. STRUCTURE AND SUBSTRUCTURE UPGRADES

Any upgrading, repair or strengthening of the selected structures and substructures need to include the following major activities:

(a) Preliminary design of the upgrades, including comparison of different alternatives including such expedience as a ‘quickly fixed,’ or adding of additional lateral bracing to components;
(b) Static or dynamic analysis of upgraded structures;
(c) Verification of the adequacy of relaxed acceptance criteria;
(d) Detailed analysis of the upgrades that demonstrate design adequacy.

Upgrading options are defined on the basis of walkdown inspections and an evaluation of the seismic capacity of the as-is structures. Preliminary concepts need to be developed for the upgrading of different parts of the structures or substructures. The final upgrading concept is determined by evaluating alternative feasible upgrading measures (or options).

The type of upgrading of existing structures or substructures depends on the additional seismic capacity that is needed. Local upgrades may be needed in the case of small deficiencies in seismic capacity such as may be developed by increased localized seismic loads applied to individual beams, columns, slabs and walls developed as a result of the response of supported systems or components not considered in the original design. However, a global strengthening of structures or substructures may be necessary in the case of low seismic capacity of an existing structure or substructure. In the case of global upgrades, the dynamic behaviour of the existing overall structure or substructure may need modification. As a consequence, the effects of the structure and substructure upgrades on the seismic adequacy of supported distribution systems and components need to be evaluated. When a global upgrade of the structure or substructure is necessary, the need for a dynamic analysis to generate new IRS and displacements has to be evaluated. In general, this generation of IRS would be necessary if, in the global structural model, the difference in stiffness to mass ratio of the upgraded global structure exceeds about 10% of that of the original ratio. If this is necessary because of a proposed upgrade, the foundation and soil capacity also need to be evaluated as discussed in Section 5 for high hazard structures and in Section 6 for moderate and low hazard structures. An upgrading fix for a particular lower bound SSC is also often applied to SSCs of the same class as a generic fix.

7.2. MECHANICAL AND ELECTRICAL DISTRIBUTION SYSTEMS AND MECHANICAL COMPONENTS UPGRADES

7.2.1. Pressure retaining mechanical distribution systems and components

Pressure retaining mechanical distribution systems and components are typically divided into two categories: ambient design temperature (<66°C), where thermal or flexibility analysis is not necessary; and elevated temperature above these ranges. In addition, there are different standards for construction of lower design pressure components (e.g. <0.4 MPa) and for higher pressure components. Typical low/moderate pressure and temperature components are tanks, heating, ventilation and air-conditioning equipment, chillers and other mechanical pressure retaining components.

The resultant fundamental frequency for moderate and low pressure retaining types of component is less than 15 Hz, such that the flexible seismic response of such components and their support or anchorage is amplified and may be the dominant load on such components. Pressure retaining mechanical distribution systems, regardless of their design pressure characteristics, are typically flexible.

For a high pressure component, the fundamental frequency of the pressure retaining boundary part of the component with respect to flexibility is generally above 15 Hz, such that the seismic response of the component is primarily dependent on the component's supports and anchorage stiffness effects on frequency. In general, the seismic stresses induced in the pressure retaining part of a high pressure component is a small fraction of the total allowable design stress in the component, which generally is not the case for moderate and low pressure design components. As a result, seismic upgrades for low and moderate pressure retaining components may involve upgrades to the component as well as its support and anchorage; while for high pressure components, seismic upgrades are generally only applicable to the component's support and anchorage system.

The effect of a flexible response of distribution systems and components in determining seismic loads on them for 5% damping typically varies from an amplification factor of about 2.5, based on ground response spectra, up to about 6.0 for IRS, as compared to ground or floor accelerations, respectively, where the structure is single response, mode dominated.

7.2.2. Other mechanical components

This category includes mechanical transport devices, cranes, hoists and other types of lifting or transport device. Cranes and hoists that use wire rope lift devices are typically very low frequency with trolleys that are high frequency (>15 Hz). Whether or not the bridge supporting the trolley is rigid or flexible is a function of whether or not they are considered loaded or unloaded when subjected to an earthquake. In general, when such devices are considered loaded or unloaded is a function of how often they are loaded in service. When this loaded duration is less than about 2% of the time, it could be considered unloaded when subjected to the RLE load. However, it may have to be evaluated as loaded when assessed against a more frequent, lesser earthquake.

7.3. ELECTRICAL AND INSTRUMENTATION AND CONTROL COMPONENTS

Electrical components are susceptible not only to physical damage (fracture or inelastic deformation) but also to malfunction. Examples of malfunction are relay chatter or liquid level trips, which are not the result of damage but may need resetting as a result of the seismic event.

Such electrical component malfunctions are considered more susceptible to high frequency seismic input motion in the frequency range above 15 Hz than are structures, mechanical and electrical distribution systems and mechanical components. Therefore, the seismic upgrade for electrical and instrumentation and control components needs to consider the potential for malfunction as well as damage in these higher frequency ranges, which would have little effect on the behaviour of structures, distribution systems and mechanical components.

8. MANAGEMENT SYSTEM FOR SEISMIC SAFETY EVALUATION

8.1. GENERAL

Paragraph 8.1 of NS-G-2.13 [1] states:

> "The management system applicable to all organizations involved in seismic safety evaluation should be established and implemented before

the start of the seismic safety evaluation programme.... The management system should cover all processes and activities of the programme for seismic safety evaluation, in particular, those relating to data collection and data processing, field and laboratory investigations, and analyses and evaluations that are within the scope of this Safety Guide."

Generally, a formal full quality assurance programme of these activities as may be part of a current design basis quality assurance or management design programme is not necessary; however, appropriate levels of checking and documentation are needed. These procedures need to be part of the management system (see Section 5 for the required documentation for the SPSA and SMA approaches). Two other important aspects of the management system are the peer review and the configuration control.

8.2. PEER REVIEW

Peer review is an essential element of the seismic evaluation of existing installations. Reference [17] provides specific guidance on the activities of the peer review team and forms the basis for the following. The purpose of the peer review is to provide an independent review of the SPSA or SMA, to ensure concurrence with the applicable state of practice in the nuclear industry. The composition and qualifications of the peer review team are important as discussed below. The number of reviewers depends on the skill set of the individuals selected. In some cases, individuals may cover many of the elements of the SPSA or SMA based on their expertise. For example, systems expertise is commonly coupled with risk quantification expertise, especially for licensees with active PSA groups. Selection of reviewers and the time spent in review need to be based on the specific evaluation. A peer review may take 5–10% of the programme execution resources. Performance of the peer review may be based on an overall review of the end results of each element and a review of a sample of the detailed analyses/calculations — the sample being large enough to provide confidence to the peer reviewer that methodologies and parameters are being appropriately implemented.

Reference [16] identifies the following peer review aspects for SPSAs and SMAs:

— The peer review team needs to have combined experience in the areas of systems engineering, seismic hazard, seismic capability engineering and SPSAs or seismic margin methodologies. The reviewers focusing on the seismic fragility work need to have participated in training sessions on fragility and HCLPF analyses as instructors or students. These team members need to have demonstrated equivalent experience in seismic walkdowns.

— The peer review team needs to evaluate whether the seismic hazard study used in the SPSA is appropriate for the site and has met the relevant recommendations in NS-G-2.13 [1] or other pertinent guidelines.
— The peer review team needs to evaluate whether the seismic initiating events are properly identified, the SSCs are properly modelled and the accident sequences are properly quantified. The peer review team needs to ensure that the seismic equipment (selected SSC) list is reasonable for the installation considering the type, design vintage and design of the installation.
— The peer review team needs to evaluate whether the seismic response analysis used in the development of seismic fragilities appropriately represents the median centred response conditional on the ground motion occurring. Specifically, the review needs to focus on the input ground motion (e.g. uniform hazard spectra or disaggregated seismic hazard), structural modelling including soil–structure interaction effects, parameters of structural response (e.g. structural frequencies and damping, soil stiffness, damping) and the reasonableness of the calculated seismic response.
— The peer review team needs to review the seismic walkdown of the plant to ensure the validity of the findings of the SRT on screening, seismic spatial interactions and the identification of critical failure modes. This review needs to be performed on a sampling basis by selecting a sample of components to review and perform the review in the installation.
— The peer review team needs to evaluate whether the methods and data used in the fragility analysis of SSCs are adequate for the purpose. The peer review team needs to perform independent fragility calculations of a selected sample of components covering different categories and contributions to CDF and LERF.
— The peer review team needs to evaluate whether the seismic quantification method used in the SPSA is appropriate and provides all the results and insights needed for risk informed decisions. The review needs to focus on the CDF and LERF estimates and uncertainty bounds and on the dominant risk contributors.

Reference [16] identifies the following peer review aspects for SMA:

— The peer review team needs to have combined experience in the areas of systems engineering, seismic hazard, seismic capability engineering and SPSAs or seismic margin methodologies. The reviewers focusing on the seismic capacity determination need to have participated in training sessions on fragility and HCLPF analyses as instructors or students. These team members need to have demonstrated equivalent experience in seismic walkdowns.

— The peer review team needs to evaluate whether the RLE used in the SMA is appropriate for the site and has met the relevant requirements of the regulatory body.
— The peer review team needs to evaluate whether the success paths are chosen properly and reflect the systems and operating procedures in the installation and that the preferred and alternative paths are reasonably redundant. The peer review team needs to ensure that the selected SSC list is reasonable for the installation considering the type, design vintage and design of the installation.
— The peer review team needs to evaluate whether the seismic response analysis used in the development of seismic margins appropriately represents the median centred response conditional on the RLE occurring. Specifically, the review needs to focus on the input ground motion, structural modelling including soil–structure interaction effects, parameters of structural response (e.g. structural frequencies and damping, soil stiffness, damping) and the reasonableness of the calculated seismic response for the RLE.
— The peer review team needs to review the seismic walkdown of the plant to ensure the validity of the findings of the SRT on screening, seismic spatial interactions and identification of critical failure modes. This review needs to be performed on a sampling basis by selecting a sample of components to review and perform the review in the installation.
— The peer review team needs to evaluate whether the methods and data used in the seismic margin analysis of components are adequate for the purpose. The review team needs to perform independent HCLPF calculations of a selected sample of components covering different categories and contributions to plant margin.
— The peer review team needs to evaluate whether the SMA method used is appropriate and provides all the results and insights needed for decision making. The review needs to focus on the HCLPF capacities of components and success paths and on the components that govern the seismic margin of the plant.

8.3. CONFIGURATION CONTROL

The licensee needs to implement a configuration management programme to ensure that, in the future, the design and construction of modifications to SSCs, the replacement of SSCs, maintenance programmes and procedures and operating procedures do not invalidate the results of the implemented programme of seismic safety evaluation. This is an essential aspect of the seismic evaluation in order to maintain the conclusions of the programme.

8.4. QUALITY ASSURANCE

The quality assurance programme as part of management system for seismic evaluation of existing installation needs to comply with IAEA safety standards and needs to be developed in accordance with IAEA Safety Standards Series No. GSR Part 2, Leadership and Management for Safety [54], and needs to define the programme structure, the responsibilities, the processes involved, a quality survey plan, the verification and control of the work process, qualification of service suppliers, document control and control of non-conformities. In other words, it needs to include all applicable elements from the general quality assurance to the seismic evaluation programme.

Appendix I

EXAMPLES OF SEISMIC DESIGN CATEGORIES AND SITE FOUNDATION CLASSIFICATION

Table 3 gives an example of guidelines for the development of seismic design categories (SDC) used in the United States of America, as a function of radioactive release and the release of other hazardous substances. Table 4 provides examples of site foundation classifications.

TABLE 3. GUIDANCE FOR SEISMIC DESIGN CATEGORIZATION (SDC) OF STRUCTURES, SYSTEMS AND COMPONENTS (SSCs) BASED ON UNMITIGATED RADIOACTIVE RELEASES IN EXISTING NUCLEAR INSTALLATIONS

Category	Worker	Public
SDC-1*	No radiological or chemical release consequences but failure of SSCs may place facility workers at risk of physical injury	No consequences
SDC-2	Lesser radiological or chemical exposures to workers than those in SDC-3 below in this column; this corresponds to the criterion that workers will experience no permanent health effects	Lesser radiological and chemical exposures to the public than those in SDC-3 below in this column, supporting that there are essentially no off-site consequences
SDC-3	0.25 Sv < dose < 1 Sv AEGL2, EPRG2 < concentration < AEGL3, ERPG3** Concentrations may place emergency facility operations at risk, or place several hundred workers at risk	0.05 Sv < dose < 0.25 Sv AEGL2, ERPG2 < concentration < AEGL3, ERPG3**
SDC-4	1 Sv < dose < 5 Sv concentration > AEGL3, ERPG3**	0.25 Sv < dose < 1 Sv > 300 mg soluble U intake concentration > AEGL3, ERPG3**

TABLE 3. GUIDANCE FOR SEISMIC DESIGN CATEGORIZATION (SDC) OF STRUCTURES, SYSTEMS AND COMPONENTS (SSCs) BASED ON UNMITIGATED RADIOACTIVE RELEASES IN EXISTING NUCLEAR INSTALLATIONS (cont.)

Category	Worker	Public
SDC-5	Radiological or toxicological effects may be likely to cause loss of facility worker life	1 Sv < dose concentration > AEGL3, ERPG3**

* 'No radiological or chemical release consequences' or 'no consequences' means that material releases that cause health or environment concerns are not expected to occur from failures of SSCs assigned to this category.

** For acute exposure guideline levels (AEGL) and emergency response planning guidelines (EPRG) for toxic chemicals, see Refs [55, 56].

TABLE 4. SITE FOUNDATION CLASSIFICATIONS

Site class	Shear wave velocity $\bar{v}_s$ (m/s)	Blow count	Undrained shear strength $\bar{s}_u$ (kPa)
A. Hard rock	>1530	n.a.*	n.a.*
B. Rock	765–1530	n.a.*	n.a.*
C. Very dense soil and soft rock	360–765	>50	95
D. Stiff soil	180–360	15–50	48–95
E. Soft clay soil	65	<15	48
F.	Any profile > 3 m of soil with the following characteristics: Plasticity index, PI > 20 Moisture content, $w \geq 40\%$ Undrained shear strength, $\bar{s}_u < 24$ kPa		

* n.a.: not applicable.

In the United States of America, land based moderate and low hazard installations and the SSCs contained therein are regulated by either the NRC or the Department of Energy. For moderate and low hazard SSCs located at nuclear power plants, such as for radioactive waste storage or processing, NRC Regulatory Guide 1.143 [57] establishes three safety classes: IIa, IIb and IIc. Safety class IIa SSCs are designed for a DBE that is one half the SL-2 (SSE) acceleration defined for the high hazard SSCs associated with reactor and actively cooled spent fuel. However, safety class IIa SSCs generally need to meet the same acceptance criteria defined for the high hazard SSCs.

For the low hazard SSCs (NRC safety classes IIb and IIc), the equivalent of the standard ASCE/SEI 7-10 Standard, Minimum Design Loads for Buildings and Other Structures [58], needs to be met, with safety class IIb SSCs being designed with an importance factor greater than 1.0, and safety class IIc SSCs being designed using an importance factor of 1.0.

Moderate hazard SSCs are generally designed to SDC-3 requirements contained in ANSI/ANS-2.26-2004 [29], ASCE 4-98 [47] and ASCE/SEI 43-05 [30]; and high hazard SSCs are limited to large, greater than 200 MW(e) reactor safety related SSCs and are generally designed to seismic SDC-4 or SDC-5 requirements contained in the same standards [29, 30, 47].

Appendix II

SUGGESTED DAMPING VALUES AND INELASTIC ENERGY ABSORPTION FACTORS

II.1 DAMPING VALUES FOR STRUCTURES, SYSTEMS AND COMPONENTS

Typical damping values used in linear elastic analyses for determining seismic loads for SSCs are presented in Table 5 as a function of the average demand to capacity ratio (D_e/C). The D_e/C ratios are indicators of the level of distress produced by the seismic action on the resisting components. The ratios are computed on an element basis (C = design code capacity, D_e = total elastic demand, including non-seismic loads); judgement is to be exercised in order to identify the average ratio for the whole SSC.

Damping values in the third column ($D_e/C \geq 1.0$) can normally be used for evaluating seismic induced forces and moments in structural members by elastic analysis. However, when the structural members need to develop essentially elastic behaviour, with not even minor damage, then the values in the central column ($0.5 \leq D_e/C \leq 1.0$) are best practice. In any case, if structural design is governed by elastic buckling considerations, damping values in the first column of Table 5 ($D_e/C \leq 0.5$) need to be used.

When structural analyses are performed for generation of IRS, selection of the appropriate damping values necessitates a consideration of the actual level of demand in the structural members. In lieu of iterative analyses to determine the actual level of demand and associated damping values, damping values in the first column of Table 5 ($D_e/C \leq 0.5$) can be used without further justification. Values in the other two columns can be used, if justified.

If a non-linear inelastic response analysis is performed that explicitly incorporates the hysteretic energy dissipation, damping values in the first column of Table 5 ($D_e/C \leq 0.5$) will normally be used to avoid the double counting of the hysteretic energy dissipation, which would result from the use of higher damping values. Values in the central column ($0.5 \leq D_e/C \leq 1.0$) may be used if they can be justified.

II.2 INELASTIC ENERGY ABSORPTION FACTORS

SSCs can be assessed against the specified seismic action using either displacement based criteria or strength based criteria. Seismic load combinations

TABLE 5. TYPICAL DAMPING VALUES FOR LINEAR ELASTIC ANALYSIS *(adapted from Ref. [30])*

Type of component	Damping (% of critical)		
	$D_e/C \leq 0.5$	$0.5 \leq D_e/C \leq 1.0$	$D_e/C \geq 1.0$
Welded and friction bolted structures	2	4	7
Bearing bolted metal structures	4	7	10
Pre-stressed concrete structures (without complete loss of pre-stress)	2	5	7
Reinforce concrete structures	4	7	10
Reinforced masonry shear walls	4	7	10
Piping	5	5	5
Distribution systems			
Cable trays 50% or more full and in-structure response spectrum zero period acceleration of 0.25g or greater	5	10	15
For other cable trays, cable trays with rigid fireproofing and conduits	5	7	7
Massive, low stressed mechanical components (pumps, compressors, fans, motor)	2	3	*
Light welded instrument racks	2	3	*
Electrical cabinets and other equipment	3	4	5**
Liquid containing metal tanks			
Impulsive mode	3	3	4
Sloshing mode	0.5	0.5	0.5

* May not be stressed beyond code capacity *C*.

** May be used for anchorage and structural failure modes that are accompanied by at least some inelastic response. Damping values in the first column ($De/C \leq 0.5$) need to be used for functional failure modes such as relay chatter or relative displacement issues that may occur at a low cabinet stress level.

for strength based acceptance criteria in the evaluation of existing nuclear installations will normally consider an inelastic energy absorption factor F_μ. Energy absorption in the inelastic range of response of structures and equipment to earthquake motions can be very important. Ignoring this effect in the evaluation of existing nuclear installations can lead to unrealistically low estimates of seismic capacity.

Generally, an accurate determination of inelastic behaviour necessitates dynamic non-linear analyses using direct integration of the equations of motion. The use of inelastic energy absorption factors is a simplified method to consider non-linear structural response using linear elastic analysis procedures. Following this approach, structural response is determined from either response spectra or time history dynamic analyses with the input excitation consistent with the elastic response spectra. The resulting elastically computed member forces are reduced by member specified inelastic energy absorption factors to give the inelastic demand. The inelastic demand is combined with concurrent non-seismic demand and the resulting total demand is compared with the capacity computed according to the strength prediction equations of the applicable design code (e.g. American Concrete Institute, American Institute of Steel Construction, Eurocodes, ASME [31]), for the limit state compatible with the intended function of the component. Following this approach, the total demand acting on an element is the sum of non-seismic demand, D_{NS}, and seismic demand, D_S, per the following load combination:

$$D = D_{NS} + \frac{D_S}{F_\mu}$$

where

- D is the total demand acting on element;
- D_{NS} is the non-seismic demand acting on an element, and includes the effects of dead, live, equipment, fluid, snow and at-rest lateral soil loads;
- D_S is the calculated seismic demand using an elastic analysis approach (by either response spectrum or time history analysis);

and F_μ is the inelastic energy absorption factor. Conservative F_μ factors can be taken from Table 6.

Note that, for simplicity, constant values are given in the table. In nature, however, these factors are frequency dependent and the values given in the table will not be conservative when the predominant structural response occurs at a frequency significantly greater than the upper limit of amplified acceleration region of the response spectrum defining the seismic action.

The use of inelastic energy absorption factors implies the acceptance of some level of inelastic deformation, which needs to be compatible with the intended function of the SSC being evaluated. Typically, the following criteria are acceptable in the evaluation of existing nuclear installations:

— For structures, a limited permanent distortion is allowed as far as it corresponds to ductile deformation.
— For pressure retaining mechanical equipment or distribution systems, a small to moderate inelastic energy absorption can be allowed, provided that no brittle materials or connections are present.
— For active components for which a change of state is necessary only after the earthquake to perform the intended safety functions, a small inelastic energy absorption can be allowed.
— For active components for which a change of state is necessary during the earthquake to perform the intended safety functions, no inelastic energy absorption is allowed ($F_{\mu} = 1$).

The values given in Table 7 are consistent with these criteria.

An alternative to the application of the F_{μ} factor as a divisor to elastically computed structural members or section forces, moments or stresses is the application of the F_{μ} factor to the elastic acceleration response spectral shape as described in section 6.1.6 of Ref. [59] and Ref. [36].

II.3. DRIFT LIMITS FOR STRUCTURAL SYSTEMS

As mentioned above, structural systems can also be assessed against the specified seismic action using displacement based criteria. Following this approach, interstory drift is computed and then compared with an allowable value. The computation of these drifts normally involves non-linear time history analyses for the specified seismic action and concurrent non-seismic loads. The analyses need to take into account cracking of concrete and plastic yielding of reinforcing bars or, in structural steel structures, the formation of plastic hinges. Allowable drift limits are specified in the form of a maximum story drift ratio, which is the interstory drift divided by the story height (see Ref. [30]).

In the evaluation of existing nuclear installations, a limited permanent distortion is acceptable in structural systems, as far as it corresponds to ductile deformation. For shear wall systems, drift ratios of 0.004–0.005 are normally acceptable. For steel braced frames, drift ratios of 0.005 are acceptable. Greater values could be used if moderate or large permanent distortions are acceptable in view of the systems and components supported by the structural systems.

TABLE 6. TYPICAL F_μ VALUES FOR THE SEISMIC RE-EVALUATION

Component	F_μ value
Structures[a]	
Concrete column axial component	1.00–1.15
Concrete columns flexural component	1.25–1.50
Concrete columns shear component	1.00–1.25
Concrete beams where flexure dominates	1.50–1.75
Concrete beams where shear dominates	1.25–1.50
Concrete connections	1.25–1.50
Concrete shear walls	1.50–1.75
Steel column axial component	1.00
Steel columns flexure component	1.15–1.35
Steel columns shear component	1.00–1.25
Steel beams where flexure dominates	1.50–2.00
Steel beams where shear dominates	1.00–1.25
Steel connections	1.25–2.00
Equipment[b,c]	
Vessel	1.25
Heat exchanger	1.25
Coolers	1.25
Chillers	1.25
Tanks (vertical)	1.25
Tanks (horizontal)	1.15
Pumps	1.25
Fans	1.25
Valves	1.25
Dampers	1.25
Filters	1.50
Glove boxes	1.35
Electrical boards	1.50
Electrical racks	1.50
Electrical cabinets	1.50
Distribution systems	
Butt joined groove welded pipe	1.25–1.50/2.0[(c)]
Socked welded pipe	1.25–1.50
Threaded pipe	1.15–1.25
Conduit	1.25–1.50
Instrument tubing	1.25–1.50
Cable trays[c]	1.50–2.00
Heating, ventilation and air-conditioning duct	1.00

TABLE 6. TYPICAL F_μ VALUES FOR THE SEISMIC RE-EVALUATION (cont.)

Component	F_μ value
Equipment supports[b]	1.25–1.50

[a] These inelastic energy absorption factors, F_μ, are applicable to equipment functioning in a passive mode during the earthquake. For active components, which need to change state during the earthquake, take $F_\mu = 1$.

[b] These components are normally designed to structural steel design code allowable, which are typically limited to 0.8–1.0 times the code specified minimum material yield stress (σ_y); hence, they are allowed a somewhat higher inelastic energy absorption factor as compared to components designed to pressure vessel design codes, where allowable stresses can be as high as 2.0 σ_y.

[c] Larger F_μ values can be justified and used on the basis of experimental feedback. In such cases, evidence needs to be provided that fatigue ratcheting (low cycle accumulation of plastic strain) failure mode is prevented.

Appendix III

BACKGROUND AND TERMINOLOGY FOR THE INELASTIC ENERGY ABSORPTION FACTORS

III.1. DUCTILITY CAPACITY AND DUCTILITY DEMAND

The elastic, perfectly plastic stress-strain relationship is shown in Fig. 5, where ε_e is the elastic yield strain of the material, ε_u is its ultimate strain (or rupture limit) and ε_{ad} is the allowable strain for the purpose of safety assessment. In practice, ε_u is a random variable. For the purposes of this publication, ε_{ad} is chosen so that $P(\varepsilon_u < \varepsilon_{ad}) \leq 5\%$. Ductility μ of the failure mode governed by the elastic, perfectly plastic relationship of Fig. 5 is commonly defined by:

$$\mu = \frac{\varepsilon_u}{\varepsilon_e}$$

Hence, the allowable ductility or ductility capacity for the purposes of a safety assessment is:

$$\mu_c = \frac{\varepsilon_{ad}}{\varepsilon_e}$$

An oscillator made of a material with the stress-strain behaviour experiences a strain history $\varepsilon(t)$ when subjected to the acceleration time history $a(t)$ as seismic input (see Fig. 5). If the maximum absolute value of $\varepsilon(t)$ is ε_{max}, the ductility demand is given by:

$$\mu_c = \frac{\varepsilon_{max}}{\varepsilon_e}$$

Assuming the acceleration time history $a(t)$ is scaled to $a_e(t)$ so that the scaled time history results in a ductility demand μ_d = 1, the available margin over $a_e(t)$ for the oscillator is given by a factor λ. The scaled time history $\lambda a_e(t)$ produces a maximum strain $\varepsilon_{max} = \varepsilon_{ad}$ or, in other words, produces a ductility demand μ_d equal to the ductility capacity μ_c.

The factor λ depends not only on the ductility capacity μ_d but also on the frequency content of the excitation $a_e(t)$ and the dynamic properties of the oscillator, such as natural frequency and damping. There is not a closed form

FIG. 5. Elastic perfectly plastic stress-strain relationship.

solution that covers all possible cases. In fact, the computation of the λ values was the basis for the development of the inelastic response spectrum [36].

Note that, if the scaled acceleration time history $\lambda a_e(t)$ is used as input for a linear elastic oscillator, it will result in a maximum stress of $\lambda\sigma_e$, whereas the maximum stress is only σ_e in the actual elastic, perfectly plastic oscillator. Hence, maximum stresses computed using a linear elastic model are divided by λ to produce the actual maximum stresses. On the other hand, the scaled acceleration time history $\lambda a_e(t)$ used as input for a linear elastic oscillator results in a maximum strain of $\lambda\varepsilon_e$, whereas the maximum strain will be $\mu_c\varepsilon_e$ in the actual oscillator. Hence, maximum strains computed using a linear elastic model are multiplied by (μ_c/λ) to produce the actual strains.

III.2. INELASTIC ENERGY ABSORPTION FACTORS

Inelastic energy absorption factors F_μ provide approximate values of λ factors, eliminating the need for non-linear time history analyses. In the early developments of the inelastic response spectrum, the following values were proposed for an excitation with a frequency content as commonly found in earthquake engineering [36]:

— $F_\mu = \mu$ if the natural frequency of the oscillator < 2 Hz;
— $F_\mu = (2\mu - 1)^{0.5}$ if the natural frequency of the oscillator is 2–8 Hz;
— $F_\mu = 1$ if the natural frequency of the oscillator > 33 Hz;
— F_μ has a linear transition 8–33 Hz.

A further step is to simplify the approach by introducing a non-frequency dependent F_μ factor, which is suggested in Appendix II. Note that this simplified approach could be unconservative for very stiff components or when there is a shift in the frequency content of the excitation (e.g. based isolated structures).

Appendix IV

HYBRID METHOD FOR FRAGILITY CALCULATIONS[14]

Two methods are available to calculate fragilities of SSCs for use in an SPSA model (see Refs [11, 41, 48, 49]):

(a) The hybrid method [11], where the HCLPF capacity is calculated first using the conservative deterministic failure margin (CDFM) approach and the median capacity with an assumed (experience based) composite variability is then calculated from the HCLPF;
(b) The separation of variables approach [41, 48, 49], where the median capacity is calculated and the randomness and uncertainty variabilities (logarithmic standard deviations) are then calculated in a detailed manner from various parameters.

The hybrid method is a simpler method and it is acceptable for generating fragilities within an SPSA for SSCs with a low to moderate influence in the plant level fragility. For critical SSCs with a potential large contribution to risk, the separation of variables approach is preferred.

In the hybrid method, the HCLPF capacity computed by the CDFM method is assumed to correspond to a 1% failure probability in the mean fragility curve ($C_{1\%}$). Then, an estimate of the composite logarithmic standard deviation β_C and its components of random variability β_R and uncertainty β_U are used to produce the corresponding fragility curve. Typically, β_C lies within the range of 0.3–0.6 [9]. Actually, when all of the sources of variability discussed in Ref. [49] are appropriately taken into account, it is not possible to obtain an estimated β_C less than approximately 0.3.

The basis for the hybrid method is provided by the fact that the annual probability of failure for any SSC is relatively insensitive to β_C. This annual probability (seismic risk) can be computed with appropriate precision from the CDFM capacity and an estimate of β_C. Kennedy [9] shows that the computed seismic risk at $\beta = 0.3$ is approximately 1.5 times that at $\beta = 0.4$, while at $\beta = 0.6$, the computed seismic risk is approximately 60% of that at $\beta = 0.4$. Table 7 provides recommended values for β_C, β_R and β_U.

The β_C values in Table 7 are based on Ref. [61] and on average have a slightly conservative bias (i.e. slightly low β_C on average). Since random variability β_R is primarily due to ground motion variability, a constant β_R value of 0.24 is

[14] Appendix IV is based on section 6.4 of Ref. [60].

TABLE 7. EPRI RECOMMENDED β_C β_C, β_R AND β_U VALUES TO BE USED IN THE HYBRID METHOD

Structure, system or component type	β_C	β_C	β_U
Structures and major passive mechanical components mounted at low elevation with structures	0.35	0.24	0.26
Active components mounted at high elevations in structures	0.45	0.24	0.38
Other structures, systems and components	0.40	0.24	0.32

recommended irrespective of the SSC being considered. The recommended β_U values are back calculated from the recommended β_C and β_R values.

Since the hybrid method involves less effort than the separation of variables approach, a possible strategy is to generate first all the necessary fragilities using the hybrid approach. The derived fragility parameters are used in the systems model to convolve with the hazard. For those SSCs that are determined to be the dominant risk contributors or are risk significant in the seismic accident sequences, estimates of median capacity ($C_{50\%}$) and variabilities (β_U and β_R) are then produced using the separation of variables approach. Finally, the integration is repeated using these corrected fragilities.

Appendix V

TYPICAL WALKDOWN FORMS

The two forms typically used to document seismic capability walkdowns are the screening verification data sheet (SVDS) and the seismic evaluation work sheet (SEWS). The SVDS is a list with a row for each item in the selected SSC list. The SVDS can be used for documenting the 'walk-by' and as a summary of the SEWS. The sample SVDS included in this appendix is taken from Ref. [27]. The SEWS is used to document a detailed inspection, as opposed to a walk-by, of an individual item included in the selected SSC list. There are as many SEWS as detailed inspections are carried out by the team performing the assessment. The sample SEWS included in this appendix is taken from Ref. [40], which includes other templates for different classes of equipment (see Ref. [11] for other examples).

SCREENING VERIFICATION DATA SHEET

Sheet 1 of ______

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

Notes: Enter applicable notation: Y = Yes; N = No; U = Unknown; N/A = 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 need to sign.

Print or type name

Signature

Date

EXAMPLE OF SEISMIC EVALUATION WORK SHEET
(FOR HORIZONTAL PUMPS)

SEISMIC EVALUTION WORK SHEET (SEWS) Sheet 1 of 2

Equip. ID No.: ______________________ Equip. Class 5 – Horizontal pumps

Equip. description: ______________________

Location: ________ Bldg: ________ Floor elev.: ______ 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 m 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 Bounding spectrum 1.5 × bounding spectrum	 DOC BS ABS
5. Demand based on: Ground response spectrum 1.5 × ground response spectrum In-structure response spectrum	 GRS AGS IRS
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 different 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

C. ANCHORAGE

1. Appropriate equipment characteristics determined (mass, centre of gravity, natural frequency, damping, 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, 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

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?

COMMENTS

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

Evaluated by: ______________________________________ Date:_________________

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DEFINITIONS

The definitions provided may not necessarily conform to definitions adopted elsewhere for international use.

accident sequence. Representation in terms of an initiating event followed by a combination of system, function and operator failures or successes of an accident that can lead to undesired consequences, with a specified end state (e.g. core damage or large early release). An accident sequence may contain many unique variations of events (minimal cut sets) that are similar.

accident sequence analysis. The process to determine the combinations of initiating events, safety functions and system failures and successes that may lead to core damage or large early release.

aleatory uncertainty. The uncertainty inherent in a non-deterministic (stochastic, random) phenomenon. Aleatory uncertainty is reflected by modelling the phenomenon in terms of a probabilistic model. In principle, aleatory uncertainty cannot be reduced by the accumulation of more data or additional information (sometimes called 'randomness').

basic event. An event in a fault tree model for which no further development is necessary because the appropriate limit of resolution has been reached.

composite variability. The composite variability includes the randomness variability (β_R) and the uncertainty (β_U). The logarithmic standard deviation of composite variability, β_c, is expressed as $(\beta_R{}^2 + \beta_U{}^2)^{0.5}$.

conservative deterministic failure margin (CDFM) method. The method described in Ref. [12], with which the seismic margin of the component is calculated using a set of deterministic rules that are more realistic than the design procedures.

containment failure. Loss of integrity of the containment pressure boundary from a core damage accident that results in unacceptable leakage of radionuclides to the environment.

core damage. Uncover and heat-up of the reactor core to the point at which prolonged oxidation and severe fuel damage are anticipated and involving enough of the core to cause a significant release.

core damage frequency (CDF). Expected number of core damage events per unit of time.

dependency. Requirement external to an item and upon which its function depends.

design basis earthquake (DBE). In some States, this term is used as equivalent to safe shutdown earthquake. However, in States where standard plant designs are used, the design basis earthquake can be larger or much larger than the safe shutdown earthquake.

distribution system. Piping, raceway, duct or tubing that carries or conducts fluids, electricity or signals from one point to another.

drift. Horizontal displacement of a given location of a structural system. Differences in horizontal displacement between consecutive horizontal diaphragms 'stories' in a structural system are indicators of potential damage in the vertical components of the system (e.g. columns or walls). For structural assessment purposes, drift is commonly expressed as a 'drift ratio', which is the story displacement with respect to the immediate lower story divided by the story height.

earthquake intensity. Measure of the observable effects of an earthquake at a particular place. Commonly used scales to specify intensity are the Modified Mercalli Intensity, Rossi–Forel, Medvedev–Sponheuer–Karnik and Japanese Meteorological Agency scales.

earthquake magnitude. A measure of the size of an earthquake. It is related to the energy released in the form of seismic waves. Magnitude means the numerical value on a standardized scale such as but not limited to moment magnitude, surface wave magnitude, body wave magnitude or Richter magnitude scale.

epistemic uncertainty. The uncertainty attributable to incomplete knowledge about a phenomenon that affects our ability to model it. Epistemic uncertainty is reflected in ranges of values for parameters, a range of viable models, the level of model detail, multiple expert interpretations and statistical confidence. In principle, epistemic uncertainty can be reduced by the accumulation of additional information, also called 'modelling uncertainty'.

failure probability. The likelihood that a structure, system or component will fail to operate upon demand or fail to operate for a specific mission time.

failure rate. Expected number of failures per unit of time expressed as the ratio of the number of failures to a selected unit of time.

fractile hazard curves. A set of hazard curves used to reflect the uncertainties associated with estimating a seismic hazard. A common family of hazard curves used in describing the results of a probabilistic seismic hazard analysis is curves of fractiles of the probability distributions of the estimated seismic hazard as a function of the level of the ground motion parameter.

fragility. Fragility of a system, structure or component is the conditional probability of its failure at a given hazard input level. The input could be earthquake motion, wind speed or flood level. The fragility model used in seismic probabilistic safety assessment is known as a double lognormal model with three parameters, A_m, β_R and β_U which are, respectively, the median acceleration capacity, the logarithmic standard deviation of randomness in capacity and the logarithmic standard deviation of the uncertainty in the median capacity.

ground acceleration. Acceleration at the ground surface produced by seismic waves, typically expressed in units of *g*, the acceleration at the Earth's surface due to gravity.

hazard. The physical effects of a natural phenomenon such as flooding, tornado or earthquake that can pose potential danger (e.g. the physical effects such as ground shaking, faulting, landsliding and liquefaction that underlie an earthquake's potential danger).

high confidence low probability of failure (HCLPF) capacity. A measure of seismic margin, defined in seismic probabilistic safety assessment as the earthquake motion level at which there is a high (95%) confidence of a low (at most 5%) probability of failure. Using the lognormal fragility model, the HCLPF capacity is expressed as $A_m\exp(-1.65(\beta_R + \beta_U))$. When the logarithmic standard deviation of composite variability β_c is used, the HCLPF capacity could be approximated as the ground motion level at which the composite probability of failure is at most 1%. In this case, HCLPF capacity is expressed as $A_m\exp(-2.33\beta_c)$. In deterministic seismic margin assessments, the HCLPF capacity is calculated using the CDFM method.

internal event. An event originating within a nuclear power plant that, in combination with safety system failures, operator errors, or both, can affect the operability of plant systems and may lead to core damage or large early release. By convention, loss of off-site power not caused by an external event is considered to be an internal event and internal fire is considered to be an external event.

large early release frequency (LERF). The frequency of those accidents involving rapid and unmitigated large release of airborne fission products from containment into the environment before the implementation of off-site emergency response such that there is potential for early health effects.

large release. A release of radioactive material for which off-site protective actions that are limited in terms of times and areas of application are insufficient for protecting people and the environment.

limit state. The limiting acceptable condition of a structure, system or component. A limit state can be defined in terms of a maximum acceptable displacement, strain, ductility, stress or any other parameter indicative of the distress introduced by the seismic action.

peak ground acceleration (PGA). Maximum value of acceleration displayed on an accelerogram; the largest ground acceleration produced by an earthquake at a site.

point estimate. Estimate of a parameter in the form of a single number.

probabilistic safety assessment (PSA). A comprehensive, structured approach to identifying failure scenarios, constituting a conceptual and mathematical tool for deriving numerical estimates of risk. Three levels of probabilistic safety assessment are generally recognized. Level 1 comprises the assessment of failures leading to determination of the frequency of core damage. Level 2 includes the assessment of containment response, leading, together with level 1 results, to the determination of frequencies of failure of the containment and release to the environment of a given percentage of the reactor core's inventory of radionuclides. Level 3 includes the assessment of off-site consequences, leading, together with the results of level 2 analysis, to estimates of public risks (also referred to as a probabilistic risk assessment).

randomness. The variability in seismic capacity arising from the randomness of the earthquake characteristics for the same acceleration and to the structural response parameters that relate to these characteristics response spectrum — a curve calculated from an earthquake accelerogram that gives the value of peak response in terms of acceleration, velocity, or displacement of a damped linear oscillator (with a given damping ratio) as a function of its period (or frequency).

review level earthquake (RLE). An earthquake larger than the plant SL-2 and is chosen in seismic margin assessment for initial screening purposes. Typically, the RLE is defined in terms of a ground motion spectrum. Note: A majority of plants in the Eastern and Midwestern United States of America have conducted SMA reviews for an RLE of 0.3g peak ground acceleration anchored to a median NUREG/CR-0098 spectrum [33].

safe shutdown earthquake. Synonymous with an SL-2 earthquake as defined in Ref. [33]. It is an earthquake for which the plant's design needs to ensure the fundamental safety functions.

safety related. Structures, systems and components that are relied upon to remain functional during and following design basis events to ensure: (i) the integrity of the reactor coolant pressure boundary; (ii) the capability to shut down the reactor and maintain it in a safe shutdown condition; or (iii) the capability to prevent or mitigate the consequences of accidents which could result in potential off-site exposures comparable to the applicable exposures established by the regulatory authority.

screening criteria. The values and conditions used to determine whether an item is a negligible contributor to the probability of an accident sequence or its consequences.

seismic margin. Seismic margin is expressed in terms of the earthquake motion level that compromises plant safety, specifically leading to severe core damage. The margin concept can also be extended to any particular structure, function, system, equipment item or component for which 'compromising safety' means sufficient loss of safety functions to contribute to core damage either independently or in combination with other failures.

seismic margin assessment (SMA). The process or activity to estimate the seismic margin of the plant and to identify any seismic vulnerabilities in the plant.

seismic source. A general term referring to both seismogenic sources and capable tectonic sources. A seismogenic source is a portion of the Earth assumed to have a uniform earthquake potential (same expected maximum earthquake and recurrence frequency), distinct from the seismicity of the surrounding regions. A capable tectonic source is a tectonic structure that can generate both vibratory ground motion and tectonic surface deformation such as faulting or folding at or near the Earth's surface. In a probabilistic seismic hazard analysis, all seismic sources in the site region with a potential to contribute to the frequency of ground motions (i.e. the hazard) are considered.

seismic spatial interaction. An interaction that could cause an equipment item to fail to perform its intended safety function. It is the physical interaction of a structure, pipe, distribution system or other equipment item with a nearby item of safety equipment caused by relative motions from an earthquake. The interactions of concern are: (i) proximity effects; (ii) structural failure and falling; and (iii) flexibility of attached lines and cables.

selected structures, systems and components (selected SSCs). The list of all SSCs that need evaluating in the seismic safety evaluation of a facility. This term was also used in NS-G-2.13 [1] to designate safe shutdown equipment list.

spectral acceleration. Generally given as a function of period or frequency and damping ratio (typically 5%). It is equal to the peak relative displacement of a linear oscillator of frequency f attached to the ground, multiplied by the quantity $(2\pi f)^2$. It is normally expressed in units of g or cm/s^2.

success path. A set of functions/components that can be used to bring the plant to a stable condition and maintain this condition for an agreed period of time, normally for at least 72 hours.

support system. A system that provides a support function (e.g. electric power, control power, cooling) for one or more other systems.

system failure. Termination of the ability of a system to perform any one of its design functions. Note that failure of a line/train within a system may occur in such a way that the system retains its ability to perform all its required functions; in this case, the system has not failed.

systems analysis. That portion of the external events probabilistic risk assessment analysis that applies to evaluating the impact of external events within the plant probabilistic safety assessment model. In this context, systems analysis encompasses the tasks relating to identification of the structures, systems and components to be included in the analysis, event sequence modelling, analysis of the failure of individual system functions within the sequences and the integration and quantification of the overall probabilistic risk assessment model.

uncertainty. A representation of the confidence in the state of knowledge about the parameter values and models used in constructing the probabilistic risk assessment.

uniform hazard response spectrum. A plot of a ground response parameter (e.g. spectral acceleration, spectral velocity) that has an equal likelihood of exceedance at different frequencies.

walkdown. Inspection of local areas in a nuclear power plant where structures, systems and components are physically located in order to ensure accuracy of procedures and drawings, equipment location, operating status and to review environmental effects or system interaction effects on the equipment that could occur during accident conditions. For seismic probabilistic safety assessment and seismic margin assessment reviews, the walkdown is explicitly used to confirm preliminary screening and to collect additional information for fragility or margin calculations.

ABBREVIATIONS

ASME	American Society of Mechanical Engineers
ANS	American Nuclear Society
CDF	core damage frequency
CDFM	conservative deterministic failure margin
CSDRS	certified seismic design response spectra
DBE	design basis earthquake
EPRI	Electric Power Research Institute
EUR	European Utility Requirements document
HCLPF	high confidence of low probability of failure
IPEEE	Individual Plant Examination of External Events
IRS	in-structure response spectra
LERF	large early release frequency
LOCA	loss of coolant accident
LRF	large release frequency
NRC	Nuclear Regulatory Commission
OECD/NEA	OECD Nuclear Energy Agency
PGA	peak ground acceleration
PSA	probabilistic safety assessment
PSHA	probabilistic seismic hazard analysis
RLE	review level earthquake
SC	seismic class
SEWS	seismic evaluation work sheet
SL-2	Seismic Level 2 earthquake
SMA	seismic margin assessment
SPSA	seismic probabilistic safety assessment
SRT	seismic review team
SSC	structure, system and component
SVDS	screening verification data sheet

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