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**NUCLEAR ENERGY AGENCY  
COMMITTEE ON THE SAFETY OF NUCLEAR INSTALLATIONS**

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**Specialist Meeting on the Seismic Probabilistic Safety Assessment of Nuclear Facilities**

**Jeju Island, Republic of Korea  
6-8 November 2006**

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\* \* \*

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## **NUCLEAR ENERGY AGENCY**

The OECD Nuclear Energy Agency (NEA) was established on 1<sup>st</sup> February 1958 under the name of the OEEC European Nuclear Energy Agency. It received its present designation on 20<sup>th</sup> April 1972, when Japan became its first non-European full member. NEA membership today consists of 28 OECD member countries: Australia, Austria, Belgium, Canada, the Czech Republic, Denmark, Finland, France, Germany, Greece, Hungary, Iceland, Ireland, Italy, Japan, Luxembourg, Mexico, the Netherlands, Norway, Portugal, Republic of Korea, the Slovak Republic, Spain, Sweden, Switzerland, Turkey, the United Kingdom and the United States. The Commission of the European Communities also takes part in the work of the Agency.

The mission of the NEA is:

- to assist its member countries in maintaining and further developing, through international co-operation, the scientific, technological and legal bases required for a safe, environmentally friendly and economical use of nuclear energy for peaceful purposes, as well as
- to provide authoritative assessments and to forge common understandings on key issues, as input to government decisions on nuclear energy policy and to broader OECD policy analyses in areas such as energy and sustainable development.

Specific areas of competence of the NEA include safety and regulation of nuclear activities, radioactive waste management, radiological protection, nuclear science, economic and technical analyses of the nuclear fuel cycle, nuclear law and liability, and public information. The NEA Data Bank provides nuclear data and computer program services for participating countries.

In these and related tasks, the NEA works in close collaboration with the International Atomic Energy Agency in Vienna, with which it has a Co-operation Agreement, as well as with other international organisations in the nuclear field.

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## **COMMITTEE ON THE SAFETY OF NUCLEAR INSTALLATIONS**

The NEA Committee on the Safety of Nuclear Installations (CSNI) is an international committee made of senior scientists and engineers, with broad responsibilities for safety technology and research programmes, and representatives from regulatory authorities. It was set up in 1973 to develop and co-ordinate the activities of the NEA concerning the technical aspects of the design, construction and operation of nuclear installations insofar as they affect the safety of such installations.

The committee's purpose is to foster international co-operation in nuclear safety amongst the OECD member countries. The CSNI's main tasks are to exchange technical information and to promote collaboration between research, development, engineering and regulatory organisations; to review operating experience and the state of knowledge on selected topics of nuclear safety technology and safety assessment; to initiate and conduct programmes to overcome discrepancies, develop improvements and research consensus on technical issues; to promote the co-ordination of work that serve maintaining competence in the nuclear safety matters, including the establishment of joint undertakings.

The committee shall focus primarily on existing power reactors and other nuclear installations; it shall also consider the safety implications of scientific and technical developments of new reactor designs.

In implementing its programme, the CSNI establishes co-operate mechanisms with NEA's Committee on Nuclear Regulatory Activities (CNRA) responsible for the programme of the Agency concerning the regulation, licensing and inspection of nuclear installations with regard to safety. It also co-operates with NEA's Committee on Radiation Protection and Public Health (CRPPH), NEA's Radioactive Waste Management Committee (RWMC) and the NEA's Nuclear Science Committee (NSC) on matters of common interest.



**SPECIALIST MEETING  
ON THE SEISMIC PROBABILISTIC SAFETY ASSESSMENT OF NUCLEAR FACILITIES  
SEOGWIPPO ICC, JEJU ISLAND, KOREA, 6-8 NOVEMBER 2006**

**SUMMARY AND CONCLUSIONS**

**A. Introduction**

This is a summary report from the "Specialists Meeting on Seismic Probabilistic Safety Assessment (SPSA) of Nuclear Facilities", held in Jeju, Korea on 6-8 November 2006. About 75 specialists from 15 countries attended. The Meeting was sponsored by the OECD Nuclear Energy Agency in cooperation with the International Atomic Energy Agency, and hosted by two Korean institutes, KAERI (Korea Atomic Energy Research Institute) and KINS (Korea Institute of Nuclear Safety.) This was the second meeting on SPSA organized by the NEA, following the first one in Tokyo in August 1999.

The main objectives of the Meeting were to review recent advances in the methodology of SPSA, to discuss practical applications, to review the current state of the art, and to identify methodological issues where further research would be beneficial in enhancing the usefulness of the methodology. Applications of the Seismic Margin Assessment methodology (SMA), a methodology related to SPSA, were also discussed. One specific objective was to compare the situation today with the situation at the time of the 1999 workshop, and to develop a set of findings and recommendations that would update those from that earlier workshop.

**B. Benefits of seismic PSA**

There was a consensus at the Specialists Meeting that SPSA is now in widespread use throughout the nuclear-power industry worldwide, by the operating nuclear power plants (NPPs) themselves, by the various national regulatory agencies, and by the designers of new NPPs. It was also widely agreed that it can systematically accomplish several very important objectives; specifically, it can contribute:

- To understanding the seismic risk arising from NPPs.
- To understanding the safety significance of seismic design shortfalls.
- To prioritizing seismic safety improvements.
- To evaluating and improving seismic regulations.
- To modifying the seismic regulatory/licensing basis of an individual NPP.

**C. Developments since the last meeting in 1999**

Compared to the situation in 1999, when the first Workshop was held in Tokyo, there have been significant expansions in the use of SPSA in many different areas. Among those discussed during this meeting are:

- There have been a number of applications in new areas – new since 1999, or at least areas where almost no work was being done earlier but now some important new approaches have been developed. The most important of the new applications reported on during the Meeting were related to designing

of new advanced NPPs, studies leading to revisions in regulations, studies of the seismic risk at multiple-unit sites, studies of post-earthquake evacuation and emergency planning issues, and studies of the impact of aftershocks on the results.

- There has been a major expansion in applications by the regulatory agencies, not only in the number of national agencies using SPSA but also in the ways that these agencies are using SPSA (see below).
- A major full-scale probabilistic seismic hazard analysis (PSHA) of nuclear power plant sites in Switzerland (called PEGASOS) has been conducted using a large group of Swiss and international experts in the fields of seismology and geology.
- Probabilistic seismic hazard analysis and performance-based design approaches have been used in the selection of the Safe Shutdown Earthquake as part of the current Early Site Permit application and US Nuclear Regulatory Commission (NRC) review process in the United States.
- A number of design certifications for standard plants have been issued by the US NRC. These have used the PSA based Seismic Margin Assessment methodology to demonstrate acceptable seismic margin and to identify any system-level seismic vulnerabilities. The Finnish regulatory agency STUK has required seismic PSAs at the design and construction phases of the new reactor under construction in that country.
- Two important developments are the American Nuclear Society's methodology standard (2003), and the new methodology standard by the Atomic Energy Society of Japan (2006).
- Around the world, applications of SPSA have identified a few places where methodological weaknesses exist, or have reinforced what was already known about these weaknesses. This has led to progress in addressing some of these weaknesses, or at least to a better understanding of their significance (see below).

#### **D. Regulatory framework for SPSA in various countries**

Some countries provided detailed information on their regulatory framework for using seismic PSA. Many other countries also provided some information in their papers as background for conducting SPSA. These pieces of information can be summarized as follows.

- Most countries are performing seismic PSA under some type of national policy. The national policies are in some countries expressed in regulatory rules and in others as recommendations by the regulatory bodies.
- In almost all cases, information obtained from SPSAs on core damage sequences and their contributions was used for identifying weaknesses and evaluating the effectiveness of proposed improvements, and was finally reflected in actual plant modifications.
- In several countries, the regulatory requirements on seismic design include probabilistic requirements for determining design basis earthquakes, or requirements to present the annual frequencies of earthquake motions that exceed the design basis.
- In one country (the U.S.), SPSA is now being used in many areas of rule making, including various risk-informed applications and guidance for seismic siting and design of NPPs.

#### **E. Areas where methodology weaknesses were identified**

During the Meeting, a small number of important weaknesses in SPSA methodology were identified. None of these are new, all having been widely recognized for many years. However, for some of the weaknesses, extensive discussions during the Meeting provided insights into how the community of SPSA practitioners could improve matters. The major areas of weakness are discussed below:

### ***E1. Probabilistic seismic hazard analysis***

Although a PSHA report from Japan was a very useful contribution as an example from a high seismicity area, the main discussions at the Meeting concentrated on PSAs conducted in areas with low-to-moderate seismicity.

Problems in PSHA studies for regions with low to moderate seismicity may arise from the fact that, due to the small number of strong-motion earthquakes in such regions, attenuation relationships must start with those taken from other regions with available strong motions. This could lead to inconsistencies or to large uncertainties, depending on experts' choices. Consequently, the PSHA hazard distributions in such low-to-moderate seismicity areas may have this large uncertainty, so that the mean values become comparable in size to PSHA mean hazard results obtained from areas with relatively high seismicity.

Much discussion occurred on this topic. The main results of this discussion are summarized as follows:

- PSHA must be performed in as realistic a way as possible, in order to reach a probabilistic result in the form of a realistic distribution, including all of the uncertainties.
- In addition, PSHA results should be compared to all available observations, especially for return periods where records are available, in order to get an objective comparison and to improve the confidence in the results, at least in that range of return periods.
- Finally, an extensive comparison should be performed between PSHA results conducted in different regions, including low to moderate seismicity regions as well as high seismicity regions, especially in the range of return periods where observed data are available.
- A well-executed PSHA would normally include these three desirable features described above, as for instance required in the ANS Standard. However, discussions at the meeting revealed that this may not always be the case. Since seismic hazard analysis forms a key element of seismic PSA with major safety and cost impact on nuclear power plants, future PSAs are encouraged to meet these requirements.

### ***E2. Human reliability analysis in SPSA***

The meeting observed that one vast area of continuing uncertainty is in quantifying the response of the operating crew and emergency organizations after earthquakes. Partly, the problem is generic with all human reliability analysis: uncertainties and the lack of data. However, there are specific characteristics of earthquakes that make post-earthquake actions more difficult to analyze and quantify. Among these characteristics are physical and mental consequences of a seismic shock. Such consequences are due in part to the damage and accessibility of equipment, consequential events such as fires likely to increase the workload, problems with multiple units potentially experiencing different consequences, conflicting goals of the government authorities in case of a large earthquake, accessibility to the site and on the site, personnel worrying about consequences to their families, and so on. Collecting information from conventional industrial sites after large earthquakes may be a good way to increase our knowledge about operator and emergency-organization responses after seismic events.

### ***E3. Treatment of correlations in SPSA***

Starting with the very first SPSAs in the early 1980s, analysts have struggled with the problem of how to quantify the correlations in the failures of similar equipment or similar structures due to the earthquake. Everyone accepts that some correlations certainly exist, for example in the response of two identical pumps located near each other, or arising from the identical design and construction of two identical shear walls. Quantifying these correlations has seldom been done in a rigorous way: analysis is complex, testing has

produced ambiguous insights at best, and the experience data base from real earthquakes is difficult to interpret. The analysts have usually used sensitivity studies to identify where the numerical results are sensitive, but they have also usually assigned large uncertainties to the numbers.

The experience with the seismic PSAs for existing U.S. plants is that the seismic Core Damage Frequency is usually dominated by one or more low-seismic capacity singletons, i.e. single contributors to a PSA cut set. In these cases, therefore, the impact of correlation was judged to be small, if any. The situation may be different in future in situations where the design basis earthquakes are higher and the goal is to demonstrate a relatively low seismic risk. Assuming perfect dependence between redundant co-located components may be too conservative in situations where low-capacity singletons are avoided by design.

Discussions during the Meeting did not provide any novel insights about how to improve the treatment of correlations. This area therefore remains one where uncertainties in the numerical bottom-line results of SPSAs will remain large.

#### **F. Recommendations**

The Meeting concluded that there is an urgent need for a comparison of PSHA results from countries with high, medium and low seismicity because of the differences that have been identified. The problem is especially important for countries with low and medium seismicity, because little actual earthquake data exist for them. A recommendation was developed that a well-focused workshop would be a suitable way to do this comparison. The seismic-behaviour sub-group of the NEA CSNI Working Group on Integrity of Components and Structures was identified as a relevant group for this activity, but other arrangements might also be used.

Any continuation activity should preferably include all stakeholders: plant owners, regulators, PSA managers, systems analysts and fragility analysts, because PSHA experts need to be aware of the impact of their studies, uncertainties and judgments.

The Meeting also gave a clear recommendation that because of the rapid progress in using seismic PSA, the state-of-the-art report issued in 1997 by the NEA CSNI Working Group Risk should be updated. Also, developing a short document about the latest situation for generalists, for example a technical opinion paper, should be considered.

The general consensus of the attendees, based on the discussion during the closing session, was that this Meeting fully met its objectives. The hope was widely expressed that another such Meeting should be held but spaced more closely than the 7-year gap between this one and the earlier one in Tokyo.

Specialist Meeting on the Seismic Probabilistic Safety Assessment of Nuclear Facilities, Seogwipo ICC,  
Jeju Island, Korea, 6-8 November 2006

Hosted by the Korea Atomic Energy Research Institute (KAERI) and  
the Korea Institute of Nuclear Safety (KINS)  
In co-operation with the International Atomic Energy Agency (IAEA)

**Opening Session**  
**by Chairs U.C. Lee and Dr. P. Pyy**

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U.C. Lee (Professor, University of Seoul, Korea, chairman)

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## Opening Session

Welcomes addresses

WGRISK Representative, J-M Lanore, chair, NEA CSNI Working Group on Risk

Application of Seismic PSA in the United States:

Activities since the Tokyo Wokshop, Invited

M. Drouin, A. J. Murphy, and N. Chokshi, US NRC

Revision of Examination Guideline

for Seismic Design of Nuclear Power Reactor Facilities of Japan

T. Yonomoto, T. Nakatogawa, Y. Maeda, NSC Japan, Invited

Welcome Address  
by Takanori TANAKA  
Deputy Director for Safety and Regulation  
OECD Nuclear Energy Agency

Honorable Chairman, Dear colleagues, ladies and gentlemen,

It is a great pleasure to me to be with you today here on the beautiful Jeju Island to open this meeting on behalf of the OECD NEA. Really, it is very difficult to imagine a better place for a meeting about assessment of seismic phenomena than this ancient volcanic island, and I congratulate the Korean organizers, the KINS and KAERI, for their choice of venue. At the same time, I have been told that the only volcanic phenomenon that we may see is the eruption show organized regularly by the Lotte Jeju hotel, which also confirms that the place is well chosen and safe.

I am very happy to see that the NEA has been able to gather together such a large amount of prominent experts and participants from all the interest groups, regulators, licensees, researchers and general managers, for a second time already. As many of you remember, the NEA and its Committee on Safety of Nuclear Installation, namely sponsored a workshop on seismic risk in Tokyo in August 1999. The objective, as for this meeting, was to discuss seismic probabilistic safety assessment and seismic margin methodologies for nuclear installations. The workshop report [1] provided a valuable summary of the state of the art of seismic PSA at that time, and included findings and recommendations to guide the future work of both researchers and analysts. I have been told that a lot of progress has been made since then and that we are going to hear news about some of these developments, standards and guidelines during this meeting. Also, I know that Madame Lanore, the chair of the Working Group Risk of the CSNI, will tell you some words about the detailed achievements of the WGRISK in the area and, later on, Dr. Andrew Murphy will reflect on the CSNI Working Group on Integrity of Components and Structures (IAGE) seismic work.

When we think about seismic PSA, it is the study about uncertainties related to seismic phenomena and their effects on the nuclear installations. As you well know, there are many different types of uncertainties involved starting from seismic phenomena, their frequency, ground motion, secondary phenomena, impact on the NPPs, emergency response in extreme cases where infrastructure and people have suffered, etc. Some of these uncertainties are rather large and difficult to quantify. However, seismic PSA offers a tool to put all of them in context and, by using it, to mitigate consequences of seismic phenomena in a rational manner.

The results of seismic PSA have enabled both plant managerial personnel and regulators to concentrate their efforts and resources where safety may be improved at a reasonable cost. For example, I am aware that seismic PSA / margin studies often identify anchorage problems with electrical equipment, pumps, large tanks, and relay chatter. These problems can usually be fixed without large expense. Also, One also has to remember that a it is not only a major catastrophe that we are talking about – sometimes even rather small earthquakes can cause loss of important systems if the systems are not designed to tolerate them.

Seven years have passed since the last large NEA meeting in the area of seismic PSA. There have been new guidelines and standards emerging during that time. Consequently, the time is now ripe for a specialist meeting to discuss these advances, develop new findings and recommendations and provide the community of seismic risk analysts with a forum for discussing the current status of the field. I am also very happy that the meeting is co-organized with the IAEA. This brings even more weight to its outcomes.

Ample time has been reserved for discussion during this meeting. Please, use the opportunity and voice your opinions. Especially, I want to use this opportunity to make a personal wish: apart from providing a forum for information exchange, I would be very glad if the meeting could bring well grounded messages to the international organizations about where the field is going, where current activities are sufficient and where they should do work in future. The more detailed these messages are, the better can we help you in future to realize them.

Mr. Chairman, let me thank you for chairing this important Specialists' meeting. Similarly, I again wish to show my gratitude to the Korea Atomic Energy Research Institute (KAERI) and the Korea Institute of Nuclear Safety (KINS) for hosting this meeting. Organizing such a large meeting is always a challenge but what I have seen today shows that you have been very successful. Dr. Pyy from my staff will be with you throughout the meeting and I ask you to turn to him in case of any matter you feel important.

With these words, let me wish you all a very productive and successful meeting on behalf of the NEA. Thank you for your attention.

## Seismic PSA WGRisk Activities

J.-M. Lanore  
Chairman of the NEA CSNI Working Group on Risk



Agence pour l'énergie nucléaire  
Nuclear Energy Agency

OECD

# SEISMIC PSA WGRISK ACTIVITIES

Jeanne-Marie LANORE (IRSN - France)  
Pekka PYY (NEA)

Earthquake could contribute significantly to the Risk.

- Seismic Risk Assessment is an important activity for WGRisk for many years.

Jeju Island - November 2006



Agence pour l'énergie nucléaire  
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# WGRISK ACTIVITIES - GENERAL

- Main mission
  - Advance the understanding and utilisation of PSA in ensuring continued safety of nuclear installations and in improving regulatory decision making by introduction of risk assessment related tools in member countries.
- Methods and products
  - Exchange of information (A report grouping all the information provided by member countries was provided in 2002 and will be updated in the near future.)
  - State-of-the-Art Reports.
  - Participation to data collection.
  - Technical Opinion Papers.
  - Organisation of workshops (This seismic PSA Workshop, for example!)



## WGRISK ACTIVITIES FOR SEISMIC PSA

- Past activities
  - SOAR (March 1998)
  - Workshop (August 1999)
  - TOP (2002)
  - Main conclusions:  
despite several uncertainties (frequency of the initiating event, correlations among failures, human errors...), the seismic PSA could provide very useful insights to complement the traditional deterministic methods.



## WGRISK ACTIVITIES FOR SEISMIC PSA

- Recent insights
  - The updated report on "PSA use and development in member countries" indicates a large number of new activities in the field of SPSA:
  - Requirements in a regulatory framework.
    - Development of SPSA.
    - Use of SPSA results for several plant safety improvements.
    - Research and development related to SPSA.



## THIS WORKSHOP

It is now a good time for a workshop on this topic.

We wish in particular to obtain:

- New knowledge for reducing the uncertainties...
- New examples of good practices and applications...
- Many exchanges and contacts, especially between specialists of different fields...

Many thanks for the organisation!

Welcome to all the participants!

## Application of Seismic PSA in the United States Activities since the Tokyo Workshop

Mary Drouin, Andrew J. Murphy<sup>1</sup>, and Nilesh Chokshi  
U. S. Nuclear Regulatory Commission  
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### **Abstract**

In 2003, the American Nuclear Society (ANS) issued a standard entitled, “External Events PRA Methodology” (ANSI/ANS-58-21-2003). This ANS standard is one in a series of standards relevant to the probabilistic risk analysis methods which either have been published or are under preparation. The primary objective of these standards is to facilitate the use of risk-informed approaches. The NRC supports this facilitation process by publication of regulatory guides which outline the criteria for PRA technical acceptability.

While the ANS standard covers both natural external events, such as, earthquake, high winds, and external floods, and humanity-generated external events, such as, airplane crashes, explosions from nearby industrial sources, and impact of nearby transportation activities, the focus in this paper is on seismic events. Generally, the requirements for PRA analysis of accident sequences initiated by a seismic event that might occur while a nuclear power plant is at full power are described. The requirements are for both a Level 1 and for a limited Level 2 PRA sufficient to evaluate the large early release frequency. Requirements are also provided for a seismic margins assessment (SMA). The requirements for both the EPRI and the NRC SMAs are described along with screening analysis criteria and criteria for demonstrably conservative or bounding analysis<sup>2</sup>.

In February, 2004, the NRC published Regulatory Guide 1.200, for trial use, entitled, “An Approach for Determining the Technical Adequacy of Probabilistic Assessment Results for Risk-Informed Activities” (NRC, 2004a). Regulatory Guide 1.200 describes an acceptable approach for determining that the quality of a PRA is sufficient to provide confidence in the results such that the PRA can be used in regulatory decision making. The guidance is intended to be consistent with NRC’s PRA Policy Statement and additional detailed guidance in Regulatory Guide 1.174, “An Approach for Using Probabilistic RISK Assessment in Risk-Informed Decisions on Plant-Specific Changes to the Licensing Basis”. It is also intended to reflect and endorse guidance provided by standard-setting and nuclear industry organizations. “Draft Guide (DG) 1138 (Appendix C to RG 1.200) (NRC, 2004b) was issued in August 2004; this DG provides the staff position on ANSI/ANS-58-21-2003.”

### **Background**

This paper is intended to provide an update of the development and application of seismic probabilistic risk analysis in the United States since the topic was last addressed in Tokyo in August, 1999, OECD, (1999). At that time one of my co-authors addressed the initial lessons learned from the seismic portion of the Individual Plant Examination for External Events program (IPEEE). Since then the final conclusions from that program were published as NUREG-1742, *Perspectives Gained from the Individual Plant examination of External Events (IPEEE) Program*, USNRC (2002a). The overall perspectives gained included:

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1. Corresponding Author, Senior Technical Advisor, U.S. Nuclear Regulatory Commission, Mail Stop T-10-D-20, Washington, DC USA, 301-415-6011, ajm1@nrc.gov
  2. The NRC staff is in the early stages of developing guidance of the treatment of uncertainties and alternate methods in risk-informed decisionmaking.

1. The IPEEE program has been generally successful in meeting its objectives.
  - a. Licensees develop an appreciation of the plant's severe accident behaviour.
  - b. Licensees understand the most likely accident sequences at full power.
  - c. Licensees gain a qualitative understanding of likelihood of core damage and fission product release.
  - d. Licensees reduce overall likelihood of core damage and fission product release by cost-effective improvements, hardware or procedural.
2. Earthquake have been identified as major contributors to CDF.
3. A majority of plant owners have implemented cost-effective improvements due to IPEEE program.
4. A direct comparison of CDF estimates between plants is not appropriate because of variability in input and modeling assumptions.
5. IPEEE results have the potential to support risk-informed regulatory activities.

### **PRA Quality Developments at the NRC**

The basis for the U.S. Nuclear Regulatory Commission (NRC) policy on the use of probabilistic risk analysis (PRA) can be found in a Policy Statement issued in 1995, USNRC, 1995. That Policy states that "... The use of PRA technology should be increased to the extent supported by the state of the art in PRA methods and data and in a manner that complements the NRC's deterministic approach." Since publication of this final policy statement, numerous uses have been made of PRA technology to support regulatory actions, including modification of the NRC's reactor safety inspection program and initiation of efforts to modify reactor safety regulation and regulatory guidance. Consequently, confidence in the information and conclusions drawn from a PRA is an important issue, in that the accuracy of the technical content must be sufficient to justify the specific results and insights that are used to support the regulatory decision under consideration.

During the initial implementation of PRA methods and results to support applications for specific regulatory decisions, the question of the quality of the PRA analysis and results arose. These questions can be best identified through comments developed during the regulatory review process, such as those from the ACRS.

In 2002, the NRC Commission provided guidance to the staff on stabilizing PRA quality expectations and requirements and instructed the NRC staff to develop a plan to implement this guidance, USNRC (2003). The Commission endorsed a phased approach to PRA quality to allow the continued practical use of risk-informed methods and continued progress toward adoption of state-of-the-art methodologies. For an application-specific phase, as an initial phase, it was noted that since almost all licensees have participated in the industry peer review process and have evaluated their PRAs against the NEI guidance document, NEI (2006), PRA quality above this basic level has likely already been achieved. The Commission considers that this level of PRA quality is not fully satisfactory for the longer term since it would not promote efficiency in PRA reviews or fully support standards development activities. Although this approach is still adequate for the intended regulatory decisions, it is not justifiable for the long term and should eventually be phased out once more desirable approaches are achievable. This initial outline of a phased approach to PRA quality was composed of:

1. An application-specific phase for PRA quality.
2. An issue-specific phase for PRA quality.
3. An all-application phase for PRA quality.
4. A fully-quantified phase of PRA quality.

To implement this Commission guidance, the staff was directed to develop an action plan that would define a practical strategy for a phased approach to PRA quality. Further in the plan the staff was to discuss the resolution of specific technical issues, such as model uncertainty, treatment of seismic and other external events, and human performance issues.

In 2004, the Commission published an action plan for the implementation of the Commission's phased approach to PRA quality, USNRC (2004c). In the development of the plan the staff made extensive use of stakeholder input. Drafts of the plan were circulated internally to the Commission and externally through the NRC public web site. The staff held public meetings on the draft plan on three different occasions. They met with the Advisory Committee on Reactor Safety and received comment letters from US professional societies, nuclear industry representatives, and the general public. In general, these stakeholders expressed their support for the adequacy of the plan.

The action plan for stabilizing PRA quality follows the three phases outlined by the Commission as above and identifies the staff activities required to support the phased approach. (The feasibility of Phase 4 will be addressed following the successful achievement of Phase 3.). Phase 1 of the action plan represents the current situation, where guidance on PRA quality is general, and staff review of the base PRA supporting the regulatory activity is performed on a case-by-case basis according to the Guidance in Regulatory Guide 1.174, NRC (2002b). Phase 2 takes advantage of the large amount of current work that has been and is being performed to develop PRA standards and for the NRC staff to develop regulatory guidance, which are being developed on varying and different schedules (See Tables 1 and 2 for a summary of this work.); and Phase 3 occurs when the PRA standards and the associated regulatory guides endorsing the standards are complete and final. As a result in this varying schedule for PRA standards, risk-informed activities will transition to Phase 2 on different schedules according to which scope items are significant to the regulatory decision being considered. Phase 3 provides a regulatory framework for the development of a PRA that will be of sufficient quality to support all current and anticipated applications; Phase 3 will be completed by December 31, 2008.

In implementing the phased approach, the action plan calls for specific application types to be defined and the necessary guidance documents identified and developed. Additional guidance documents will be developed on a schedule that is dependent on when the industry consensus standards are available. To fully implement the phased approach, a process for prioritizing and scheduling review of licensee submittals is required. This prioritization process must balance the need to use staff resources effectively and efficiently with the need to provide incentives for licensees to develop more complete PRA models to improve the overall quality of the PRAs and to provide bases for regulatory decisions. Because the development of guidance documents will be achieved over an extended period of time, the NRC staff intends to continue to use other opportunities to monitor the scope, level of detail, and technical adequacy of licensee PRAs, i.e., their overall quality. The NRC staff, also, plans to work closely with the industry consensus writing organization in development of needed standards and to develop the associated regulatory guidance. (See Tables 1 & 2 for status of standards and regulatory guidance.)

**Table 1. Status of Consensus PRA Standards**

Item	Scope	Responsibility	Status
Risk Characterization	Level 1	ASME	Available and endorsed in RG 1.200
	Level 2 (LERF)	ASME	Available and endorsed in RG 1.200
	Level 2 (full)	ANS	Under development
	Level 3	ANS	Under development
Operating Modes	Full power	ASME	Available for Level 1 and LERF
	Low power and shutdown	ANS	Under development (projected draft in 2005)
Initiating Events	Internal (transients, LOCAs, floods)	ASME	Addressed for Level 1 and LERF in ASME standard
	Internal (fires)	ANS	Under development
	External (seismic, winds, floods, other)	ANS	Available

Phase 2 of the PRA quality action plan began to approach fruition with the publication for trial use of Regulatory Guide 1.200, "An Approach for Determining the technical Adequacy of Probabilistic Risk Assessment Results for Risk-Informed Activities", USNRC (2004b). This regulatory guide describes one acceptable approach for determining that the quality and, thus, the adequacy of a PRA is sufficient to provide confidence in the results, such that the PRA can be used to support a specific regulatory decision for light-water reactors. The guidance is intended to be consistent with the NRC's PRA policy statement and the more detailed guidance in Regulatory Guide 1.174. It is also intended to reflect and endorse guidance provided by standard-setting and nuclear industry organizations. When used to support an application for a licensing decision by the NRC, use of this regulatory guide will obviate the need for an in-depth review of the PRA by NRC reviewers, allowing them to focus their review efforts on key assumptions and areas identified by peer reviewers as being of concern and relevance to the application. Consequently, this guide will lead to a more focused and consistent review process.

**Table 2. Status of Guidance Documents**

Application	Document	Originator	Status
License amendment	RG 1.174	NRC	Rev. 1
In-service testing (ST)	RG 1.175	NRC	Rev. 0
Graded QA	RG 1.176	NRC	Rev. 0
Technical specifications	RG 1.177	NRC	Rev. 0
In-service inspection (ISI) of piping	RG 1.178	NRC	Rev. 1
Technical adequacy of PRA	RG 1.200	NRC	Rev 0 issued for trial use, Rev 1 for use to be issued Dec 2006
Endorsement of NEI-00-04	RG 1.201	NRC	Rev. 0
Guidance for categorization of structures, systems and components (SSCs) by risk-significance	NEI-00-04	NEI	Rev. D
Guidance for PRA Peer Review Process and self-Assessment Process	NEI-00-02	NES	Rev. 1
Endorsement of NEI-00-02	RG 1.200	NRC	Appendix B of RG 1.200

This guidance was issued as a draft for public comment in 2002. While the comments were generally positive, the remaining comments were best resolved by testing the guidance against actual applications. Thus, this regulatory guide was issued for trial use for plant applications.

In September 2006, the NRC made Draft Regulatory Guide DG-1161 publicly available. This draft guide is Proposed Revision1 of Regulatory Guide 1.200. Subsequent to its issuance for trial use in February, 2002, five trial applications were conducted. DG-1146 incorporates the lessons learned from those trial applications. In addition, the appendices to this draft guide have been revised to address the changes made in the professional society PRA standards and nuclear industry PRA guidance documents.

The focus of this workshop for which this paper has been prepared is seismic PRA. Thus, the preceding material was all a prelude to a status report on the NRC activities. In March, 2003, the American National Standard Institute approved the publication of ANSI/ANS-58.21-2003, a national consensus standard prepared by a working group of the American Nuclear Society. This standard details the requirements for producing a quality PRA for external events, including events initiated by earthquake. While the interest of this workshop is in seismic initiated events, the standard covers both natural external events, such as, high winds, and external floods, and human-made external events, such as, airplane crashes, explosions at nearby industrial facilities, and impacts from nearby transportation activities. External fires are specifically excluded because they are being treated by another standard.

The standard also addresses the requirements for a seismic margins assessment (SMA), but the scope of SMA requirements is limited to analysis of nuclear power plant seismic capacities following either the EPRI method, which was used extensively by industry for the IPEEE program, or the LLNL method. Including the SMA methods is appropriate because SMA employs many of the same tools used in a seismic PRA. The utility of a SMA analysis is restricted to a limited set of risk-informed applications.

This paper will not cover additional characteristic of ANSI/ANS-58.21-2003 because there is a paper specifically about the standard in these proceedings by R.M. Ravindra, 2006.

The application of the results IPEEE is to partially justify the selection of a particular target performance goal of  $1 \times 10^{-5}$ . This goal is used in the development of guidance for a risk-informed approach to seismic siting, Murphy et al., 2006.

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## Revision of Examination Guideline for Seismic Design of Nuclear Power Reactor Facilities of Japan

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### ***Summary***

The Nuclear Safety Commission (NSC) approved the examination guideline for the seismic design of nuclear power reactor facilities on September 19 in 2006, which was revised through the five-year intense discussions in the subcommittee composed of the specialists in the field of earthquake (see Appendix). The main motivation of the revision was the reflection of the significant technical advancements occurred after the former guideline was accepted in 1981 and partially revised in 2001. The advancements include the technologies for the geotechnical investigation for the active faults, the numerical prediction of the strong earthquake motion, and the earthquake-proof design such as seismic isolation, which were enhanced especially after Hyogo-ken Nambu earthquake that caused significant damage in the city of Kobe. The new guideline will be applied to the regulation for the reactor installment and construction in the future. For the existing plants, NSC recommended to the Nuclear and Industrial Safety Agency (NISA) to take action so that the utilities make an appropriate review on the seismic safety of the nuclear facilities based on the new guideline.

The guideline is used to provide the basis of the judgement in the licensing process for the seismic design policy of the nuclear power plant, and describes the basic procedures to determine the design-basis-ground-motion (DBGM), the classification of the nuclear facility components and systems according to the importance to prevent the radiological hazard, the principle of the seismic design, and the load combination and allowable limit. The basic policies of the guideline are summarized as follows:

The DBGM (denoted by “Ss” in the guideline) should be determined conservatively through the consideration of the possible ground motions caused by sufficiently scarce earthquakes with clearly-defined source-location or those without clear ones.

Classification of the nuclear components and systems shall be established according to the importance to avoid radiological hazards to the environment.

The nuclear components and systems classified in the highest importance level shall be designed to maintain safety functions against DBGM.

Buildings and structures shall be settled on the grounds having sufficient supporting capability.

Regarding the introduction of the seismic PSA to the guideline, the subcommittee concluded that the seismic PSA is not matured sufficiently enough to provide the quantitative basis for the regulatory judgement. In stead of the quantitative use of the PSA, the subcommittee recommended to analyze the exceedance probability of DBGM to provide complemental information to discuss the adequacy of the conservatism in determining DBGM.

The subcommittee also concluded that the qualitative use of the exceedance probability in the licensing process is helpful to provide opportunities to find and solve problems in calculating the probability, promote the study related to the seismic PSA, and contribute the future risk informed regulation.

## *Appendix*

### Examination Guideline for Seismic Design of Nuclear Power Reactor Facilities

Provisional translation

(Secretariat of the Nuclear Safety Commission)

Prepared for the OECD/NEA-CSNI Workshop Specialist Meeting  
on the Seismic Probabilistic Safety Assessment of Nuclear Facilities  
Jeju Island, Korea, 6-8 November 2006

## **1. Introduction**

This guideline is provided to show the basis of the judgment for adequacy of the seismic design policy in the standpoints to ensure seismic safety at the Safety Examination related to the application for the establishment license (includes the application of alteration of an establishment license) of the individual light water power reactor.

The former “Examination Guideline for Seismic Design of Nuclear Power Rector Facilities (decided by the Nuclear Safety Commission (NSC) on 20 July 1981 and revised on 29 March 2001, hereinafter referred to as “Former Guideline”)” was the guideline which was revised based on the state of arts of evaluating methods of static seismic force etc. by the NSC in July 1981, which had been provided in September 1978 by the Atomic Energy Commission. And it was partially revised in March 2001.

This time, overall revision of Former Guideline has been conducted by reflecting accumulated new seismological and earthquake engineering knowledge and remarkable improvement and development of seismic design technology of nuclear power reactor facilities.

Incidentally, this guideline shall be revised to reflect the coming new knowledge and experiences suitably according to accumulation of new findings.

## **2. Scope of Application**

This guideline shall be applied to the nuclear power reactor facilities (hereinafter referred to as “Facilities”).

Nevertheless, basic concept of this guideline could be referred to other type nuclear reactor facilities as well as other nuclear related facilities.

Incidentally, if some part of application contents could not comply with this guideline, it would not be excluded if it reflected technological improvements or developments and seismic safety could be ensured farther than satisfying this guideline.

## **3. Basic Policy**

A part of Facilities designated as important ones from the seismic design points shall be designed to bear seismic force exerted from earthquake ground motion and to maintain their safety function, which could be postulated appropriately to occur but very scarcely in the operational period of Facilities from the seismological and earthquake engineering standpoints such as geological features, geological structure, seismicity, etc. in the vicinity of the proposed site.

Moreover, any Facilities shall be designed to bear the design seismic force sufficiently which is assumed appropriately for every classification in the seismic design from the standpoint of radiological effects to the environment which could be caused by earthquake.

Besides, buildings and structures shall be settled on the grounds which have sufficient supporting capacity.

(Commentary)

I. Regarding Basic Policy

(1) Regarding determination of earthquake ground motion in the seismic design

In the seismic design, it shall be based on the principle that ‘the ground motion which could be postulated appropriately to occur but very scarcely in the operational period of Facilities and are feared affecting severely to Facilities’ shall be planned out adequately, and that, on the premise of this ground motion, the seismic design shall be conducted not to give any risk of serious radiological exposure to the public in the vicinity of Facilities from the external disturbance initiated by an earthquake.

This policy is equal to the ‘basic policy’ in Former Guideline which is required to the seismic design with the provision of ‘nuclear power reactor facilities shall maintain seismic integrity against any postulated seismic force assumed so sufficiently that no earthquake would induce significant accidents’.

(2) Regarding existence of “Residual Risk”

From the seismological standpoint, the possibility of occurrence of stronger earthquake ground motion which exceed one planned out on the above-mentioned (1) can not be denied. This means, in determination of seismic design earthquake ground motion, the existence of “Residual Risk”(defined as such a risk that, by extension of the effect of the ground motion which exceeds the planned out design ground motion of Facilities, impairing events would occur to Facilities and the event in which massive radioactive materials diffuse from Facilities would break out, or the result of these events would cause radiological exposure hazards to the public in the vicinity of Facilities).

Therefore, at the design of Facilities, appropriate attention should be paid to possibility of occurrence of the exceeding ground motion to the planned out one and, recognizing the existence of this “Residual Risk”, every effort should be made to minimize it as low as practically possible not only in the stage of design basis but also in the following stages.

#### 4. Classification of Importance in Seismic Design

Importance in seismic design of Facilities shall be classified into the followings from the standpoints of the possible impact of radiation to the environment caused by earthquake corresponding to the categories of Facilities.

##### 4.1 Classification on Function

S Class: Facilities containing radioactive materials by themselves or related directly to Facilities containing radioactive materials, whose loss of function might lead to the diffusion of radioactive materials to the environment, Facilities required to prevent the occurrence of those events and Facilities required to mitigate the consequences resulting from the diffusion of radioactive materials in the occurrences of those accidents, and also whose influences are very significant,

B Class: Facilities of the same functional categories as above S Class, however whose influences are relatively small,

C Class: Facilities except for S or B Class, and ones required to ensure equal safety as general industrial facilities.

## 4.2 Facilities of Classes

Facilities of Classes are shown as follows by the above classification of the importance in the seismic design.

### 1) S Class Facilities

- i) Equipment/piping system composing of the ‘reactor coolant pressure Boundary’ (the definition is the same that is described in other Examination Guidelines for Safety Design of Light Water Nuclear Power Reactor Facilities).
- ii) Spent fuel storage pool.
- iii) Facilities to add the negative reactivity rapidly to shutdown the reactor and Facilities to preserve the shutdown mode of the reactor.
- iv) Facilities to remove the decay heat from the reactor core after reactor shutdown.
- v) Facilities to remove the decay heat from the reactor core after the failure accident of reactor coolant pressure boundary.
- vi) Facilities to prevent the propagation of radioactive materials directly as the pressure barrier at the failure accident of reactor coolant pressure boundary.
- vii) Facilities, except for those in the category □) above, to mitigate the diffusion of radioactive materials to the environment at the accident which involves the release of radioactive materials.

### 2) B Class Facilities

- i) Facilities connected directly to reactor coolant pressure boundary and containing radioactive materials by themselves or have possibility to contain radioactive materials.
- ii) Facilities containing radioactive materials. Except for those whose effect of radiological exposure to the public due to their break is smaller enough to compare with annual exposure limit at the outside of the peripheral observation area, because of its small inventory of containing radioactive materials or of the difference of the type of storage system.
- iii) Facilities related to the radioactive materials except radioactive wastes and have possibility to give excessive radiological exposure to the public and the operational personnel from their break.
- iv) Facilities to cool the spent fuels.
- v) Facilities except for those of S Class, to mitigate diffusion of radioactive materials to the environment at an accident which involves the release of radioactive materials.

### 3) C Class Facilities

Those Facilities not belong to above S or B Class.

## 5. Determination of Design Basis Earthquake Ground Motion

The ground motion to be established as the basis of the seismic design of the Facilities shall be planned out adequately as the ground motion to be postulated to occur but very scarcely in the operational period of Facilities from the seismological and earthquake engineering point of view relating to geology, geological structure, seismicity, etc. in the vicinity of the proposed site, and to be feared making a serious impact to Facilities. (Hereinafter this ground motion is referred to as “Design Basis Earthquake Ground Motion Ss” or “DBGMs Ss”).

DBGMs Ss shall be planned out on the following principles.

- 5.1 DBGMs Ss shall be planned out as following two types of earthquake ground motions in horizontal direction and vertical direction on the free surface of the base stratum at the proposed site, relating to
  - 5.2 “Site specific earthquakes ground motion whose source to be identified with the proposed site” and
  - 5.3 “Earthquake ground motion whose source not to be identified” mentioned below.

5.2 Site specific earthquakes ground motion whose source to be identified with the proposed site shall be planned out on the following principles.

- 1) Taking account of the characteristics of active faults and the situation of earthquake occurrences in the past and at present in the vicinity of the proposed site, and classifying the earthquakes by the pattern of earthquake occurrence etc. plural number of earthquakes which are feared making severe impact to the proposed site shall be selected (hereinafter referred to as "Investigation Earthquakes").
- 2) Following items shall be taken into account concerning the 'characteristics of the active faults around the proposed site' in above-mentioned 1).
  - i) The active faults considered in the seismic design shall be identified as the one of which activities since the late Pleistocene epoch can not be denied. Incidentally, judgment of the faults can depend upon whether the displacement and deformation by the faults exist or not in the stratum or on the geomorphic surface formed during the last interglacial period.
  - ii) The active faults shall be investigated sufficiently by integrating geomorphological, geological and geophysical methods, etc. to make clear the location, shape, activity of the active faults, etc. according to the distance from the proposed site.
- 3) For any Investigation Earthquakes selected in above-mentioned 1), following evaluations of earthquake ground motion both i) with response spectra and ii) by the method with fault models shall be conducted, and DBGM Ss shall be planned out from respective Investigation Earthquakes.  
Incidentally, in evaluating the earthquake ground motion various characteristics (include the regional peculiarity) according to the pattern of earthquake occurrences, seismic wave propagation channel, etc. shall be taken into account sufficiently.
  - i) Evaluation of earthquake ground motion with response spectra for respective Investigation Earthquakes, response spectra shall be appraised by applying appropriate methods and the design response spectra shall be evaluated on these spectra, and earthquake ground motions shall be evaluated in considering the earthquake ground motion characteristics such as duration time, time dependent change of amplitude-enveloping curve suitably.
  - ii) Evaluation of earthquake ground motion by the method with fault model for respective Investigation Earthquakes, earthquake ground motions shall be evaluated by settling the seismic source characteristics parameters with appropriate methods.
- 4) Uncertainty (dispersion) concerned with the evaluation process of the DBGM Ss in above-mentioned 3) shall be considered by applying the appropriate methods.

5.3 Earthquake ground motion whose source not to be identified shall be planned out on the following principle.

Design Earthquake Ground Motions shall be planned out by collecting the observation records near the source which are obtained from past earthquakes inside the inland earth's crust, of which the source can not be related directly to any active faults, settling the response spectra based on those records by taking account of the ground material characteristics of the proposed site, and adding consideration of the earthquake ground motion characteristics such as the duration time, time dependent change of amplitude-enveloping curve, etc. suitably to these results.

## (Commentary)

## II. Regarding to determination of DBGM Ss.

## (1) Regarding the characteristics of DBGM Ss.

In Former Guideline, regarding design basis earthquake ground motion two categories of “Earthquake Ground Motion S1” and “Earthquake Ground Motion S2” were required to be planned out, however in this revision both these motions were integrated, and enhancement of selection of Investigation Earthquakes, evaluation of ground motion etc. were strived for DBGM Ss.

This DBGM Ss is the premise ground of the seismic design to ensure seismic safety of Facilities and, in planning it out, its adequacy should be checked sufficiently according to the latest knowledge in the specific examination.

## (2) The interpretation of the terminology regarding determination of DBGM Ss are as follows.

- 1) ‘Free surface of the base stratum’ is defined as the free surface settled hypothetically without any surface layer or structure and as the surface of base stratum postulated to be nearly flat with considerable expanse and without eminent unevenness to plan out design basis earthquake ground Motion.

‘Base stratum’ mentioned here is defined as a solid foundation of which shear wave velocity  $V_s$  exceeds 700m/s, and which has not been weathered significantly.

- 2) ‘Active faults’ are defined as faults which moved repeatedly in recent geological age and have also possibility to move in the future.

## (3) Regarding the principle of determination DBGM Ss

- 1) In selecting Investigation Earthquakes, the characteristics of active faults and the situation of earthquake occurrence in the past and at present should be investigated carefully, and furthermore existing research results concerned with distribution of middle, small and fine size of earthquakes in the vicinity of the proposed site, stress field, pattern of earthquake occurrence (including shape, movement and mutual interaction of the plate) shall be examined comprehensively.

- 2) Investigation Earthquakes shall be selected depending on the classification considering the pattern of earthquake occurrence etc. as follows.

## i) Inside Inland Earth’s Crust Earthquake

‘Inside inland earth’s crust earthquake’ is defined as the earthquake which occurs in the upper crust earthquake generation layer and includes one which occurs in the rather offshore coast.

## ii) Inter-plates Earthquake

‘Inter-plates earthquake’ is defined as one which occurs in the interfacial plane of two mutually contacting plates.

## iii) Inside Oceanic Plate Earthquake

‘Inside oceanic plate earthquake’ is defined as one which occurs inside a subducting (subducted) oceanic plate, and is classified into two types,

‘Inside subducting oceanic plate earthquake’ which occurs near the axis of sea trench or in its rather offshore area, and ‘Inside subducted oceanic plate earthquake (Inside slab earthquake)’ which occurs in the land side area from the vicinity of the axis of sea trench.

- 3) The evaluation method using fault model should be regarded as important in the case of earthquake whose source is near the proposed site and process of its failure could be supposed to make large impact to evaluation of the ground motion.

- 4) In consideration of ‘uncertainty (dispersion) concerned with the determination process of DBGM Ss’, appropriate method should be applied considering the cause of uncertainty (dispersion) and its extent which are supposed to make large impact directly to plan out DBGM Ss.

- 5) The principle of determination of 'Earthquake ground motion whose source not to be identified' is implied that, if the detailed investigation would be conducted sufficiently considering the situation etc. in the vicinity of the proposed site, it could not be asserted to evaluate all earthquakes inside inland earth's crust in advance which could have still the possibility to occur near the proposed site, therefore this earthquake should be considered commonly in all applications in spite of the results of the detailed investigation around the proposed site. The validity of DBGM Ss planned out by materializing this principle should be confirmed specifically in checking on the latest information at the time of each application. Incidentally, on that occasion, probabilistic evaluation could be referred as the needs arise regarding the ground motion near the source generated from the source fault which does not indicate any clear trace on the ground surface.
- 6) Regarding 'Site specific earthquakes ground motion whose source to be identified with the proposed site' and 'Earthquake ground motion whose source not to be identified', the exceedance probability of respective earthquakes should be referred in each safety examination from the standpoint that it is desirable to grasp that the response spectra of each seismic ground motion planed out correspond to what extent of the exceedance probability.
- 7) In the case that the necessary investigation and evaluation are implemented in selection of Investigation Earthquakes and determination of DBGM Ss, existing materials etc. should be referred in considering the accuracy of them sufficiently. If different result would be obtained compared with the existing evaluation results, its reason should be shown clear.
- 8) Regarding the ground which supports the structures of Facilities and Facilities themselves, if the peculiar frequency characteristics could be found in the seismic response, it should be reflected to determination of DBGM Ss as the needs arise.
- (4) Regarding evaluation of the faults which assumed as the source of earthquake
- 1) As investigation of the active faults is the basis of the evaluation concerning the faults which is assumed as the source of earthquake, appropriate investigation should be implemented combining adequately the survey of existing materials, tectonic geomorphologic examination, the earth's surface geological feature examination, geophysical examination, etc. according to the distance from the proposed site. Especially in the area near the proposed site, precise and detailed investigation should be applied. Incidentally extent of the area near the proposed site should be decided suitably considering the relation etc. with DBGM Ss planned out as 'Earthquake ground motion whose source not to be identified'.
  - 2) Regarding active folds, active flexures, etc. these should also be the object of investigation in above-mentioned 1) as well as the active faults and should be considered in the evaluation of the faults assumed to be the source in accordance with their dispositions.
  - 3) The dispositions of the faults should be evaluated appropriately grasping the under ground structure etc. depending on the regional situation. Incidentally, the special consideration should be required if the earthquake should be assumed from the dispositions of faults in the area where the faults are indistinct.
  - 4) In the case, the scale of earthquake shall be postulated from the length of the fault etc. by applying the empirical formula, the scale should be evaluated adequately considering the special features etc. of the empirical formula.
  - 5) Uncertainty shall be considered appropriately in assumption of the characteristics of the source, in the case that sufficient information could not be obtained to settle the source characteristics parameter including the shape evaluation of the fault to be assumed as the source even by implementing investigation of the active faults.

## 6. Principle of Seismic Design

### 6.1 Primal Policy

Facilities shall be designed to fulfill the following primal policies of the seismic design for respective categories of Class.

- 1) Respective Facilities of S Class shall maintain their safety functions under the seismic force caused by DBGM Ss. And also shall bear the larger seismic force loading of those caused by “Elastically Dynamic Design Earthquake Ground Motion Sd” or the static seismic force shown below. (Hereinafter Elastically Dynamic Design Earthquake Ground Motion Sd is referred to as “EDGM Sd”.)
- 2) Respective Facilities of B Class shall bear the static seismic force shown below. And, as for the Facilities those are feared of resonating with earthquake, the influence shall be evaluated.
- 3) Respective Facilities of C Class shall bear the static seismic force shown below.
- 4) In respective items shown above, the integrity of upper Class Facilities shall not be impaired by the damage of the lower Class Facilities.

### 6.2 Evaluation Method for Seismic Force

The seismic force for seismic design of Facilities shall be evaluated with the following methods.

- 1) Seismic forces caused by DBGM Ss  
Seismic force caused by DBGM Ss shall be evaluated by applying DBGM Ss in combining horizontal seismic force with the vertical seismic force appropriately.
- 2) Seismic forces caused by EDGM Sd  
EDGM Sd shall be established based on DBGM Ss with the technological judgments. And the seismic forces caused by EDGM Sd shall be also evaluated in combining horizontal seismic forces with the vertical seismic force appropriately.
- 3) Static seismic force  
Evaluation of the Static seismic force shall be based on the followings.
  - i) Buildings and structures  
Horizontal seismic force shall be evaluated by multiplying the seismic story shear coefficient Ci by the coefficient corresponding to the importance classification of the facilities as shown below, and multiplying the weight at the above height of the story concerned.

S Class	3.0
B Class	1.5
C Class	1.0

Here, Ci of the seismic story shear coefficient shall be obtained in putting the standard shear coefficient Co to be 0.2, considering the vibration characteristics of the buildings and structures, categories of the ground, etc.

As for the facilities of S Class, both horizontal and vertical seismic forces shall be combined simultaneously in the most adverse fashion. The vertical seismic force shall be evaluated with the vertical seismic intensity which is obtained by putting the seismic intensity 0.3 as a standard, and by considering the vibration characteristics of buildings and structures, categories of the ground, etc. However the vertical seismic coefficient shall be constant in the height direction.

- ii) Components and piping system

The seismic force of respective Classes shall be evaluated with the seismic intensities which are obtained by multiplying the seismic story shear coefficient Ci in above-mentioned i) by the coefficient corresponding to the importance classification of the Facilities as the horizontal seismic intensity, and by increasing the horizontal seismic intensity concerned and the vertical seismic intensity in above-mentioned i) by 20% respectively.

Incidentally, horizontal seismic force shall be combined with the vertical seismic force simultaneously in the most adverse fashion. However, vertical seismic forces shall be assumed to be constant in the height direction.

(Commentary)

**III. Regarding the Design Principle**

**(1) Regarding the necessity of establishment of EDGM Sd**

In Former Guideline, the design basis earthquake ground motion should have been planned out classified as two categories of Earthquake Ground Motion S1 and Earthquake Ground Motion S2 corresponding to the seismic importance classification of the buildings, structures, components and piping system, however in this revision, the determination of DBGM Ss shall only be required. In the seismic design concept to ensure seismic safety of Facilities, it is the basic principle that the safety functions of the seismically important Facilities shall be maintained under the seismic forces by this DBGM Ss.

In addition to confirm maintenance of seismic safety functions of the Facilities under this DBGM Ss with higher precision, establishment of EDGM Sd, which is closely related with DBGM Ss from technical standpoint, is also required to be prescribed.

**(2) Regarding establishment of EDGM Sd**

The concept of ‘to bear the seismic force’ which prescribed in the Article 6. in this Guideline means that Facilities as a whole are designed in the elastic range on the whole to a certain seismic force.

In this case, design in the elastic range means to retain the stress of respective parts of the Facilities under the allowable limits by implementing stress analysis supposing the facilities as the elastic body.

Incidentally, the allowable limits shown here, does not require strict elastic limits and requires the situation that the Facilities as a whole should retain in elastic range on the whole even though the case in which the Facilities partially exceeds the elastic range could be accepted.

Although respective S Class Facilities are required ‘to bear the seismic force’ by EDEGM Sd, this EDGM Sd is established based on the technological judgment.

The elastic limits condition is the condition that the impact which the Earthquake Ground Motion makes to the Facilities and the situation of the Facilities can be evaluated clearly, and that it makes a grasp of maintenance of seismic safety functions as a whole of the Facilities under the seismic force by DBGM Ss more reliable by confirming that the Facilities as a whole retains in elastic limits condition on the whole under the seismic force by EDEGM Sd.

Namely EDEGM Sd assumes a part of the roles which the Design Earthquake Ground Motion S1 of Former Guideline used to be attained in the seismic design.

EDGM Sd should be established by multiplying DBGM Ss by coefficients obtained on the technological judgment in considering the ratio of input seismic loads for the safety functional limits and the elastic limits for the respective Facilities and their composing elements. Here, in evaluating the coefficient, the exceedance probability which is referred in the determination of DBGM Ss would be consulted.

The concrete established value and reason of establishment of EDGM Sd should be made clear sufficiently in respective specific application.

Incidentally, the ratio of EDGM Sd and DBGM Ss (Sd/Ss) should be expected larger than a certain extent in considering the characteristics required to EDGM Sd, and should be obtained not to be less than 0.5 as an aimed value.

In addition, EDGM Sd would be established specifically to respective elements which compose the Facilities depending on the difference of their characteristics to be considered in seismic design.

Incidentally, regarding to B Class Facilities, ‘as for Facilities that are feared resonating with seismic force loading, the influence shall be evaluated’, the earthquake ground motion applied to this evaluation would be established with multiplying EDGM Sd by 0.5.

(3) Regarding the evaluation of the seismic force by DBGM Ss and EDGM Sd

In case that the seismic force by DBGM Ss and EDGM Sd are evaluated based the seismic response analysis, the appropriate analytical methods should be selected and suitable analytical consideration should be settled based on the sufficient investigation in considering to the applicable range of response analysis methods, applicable limits, etc.

Incidentally, in the case ‘free surface of the base stratum’ is very deep compared with the ground level on which Facilities would be settled, amplification characteristics of the ground motion on the ground level above free surface of the base stratum should be investigated sufficiently and be reflected to the evaluation of the seismic response as the needs arise.

(4) Regarding static seismic force

Evaluation of the static seismic force should be depended upon 1) and 2) shown below.

In addition, regarding to the buildings and structures, the adequate safety margin of retained horizontal strength of buildings and structures concerned should be checked to maintain the retained horizontal strength required relating to the importance of Facilities, and the evaluation of retained horizontal strength required should be complied to the 3) shown below.

1) Horizontal seismic force

- i) The datum plane for evaluation of horizontal seismic force should be the ground surface in principle. However, if it is needed to consider the characteristics such as the constitution of the building and the structures and the relation to the surrounding ground around Facilities, the datum plane should be provided appropriately and be reflected to the evaluation.
- ii) Horizontal seismic force applied to aboveground part from the datum plane should be obtained to be the total of the seismic forces acted on the part concerned in accordance with the height of the building and the structure and be calculated with the following formula,

$$Q_i = n \cdot C_i \cdot W_i$$

Where

$Q_i$ : Horizontal seismic force acting on the part in question.

n: Coefficient in accordance with importance classification of facilities (Earthquake-proof S Class 3.0, Earthquake-proof B Class 1.5, Earthquake-proof C Class 1.0).

$C_i$ : Seismic story shear coefficient, it depends on the following formula,

$$C_i = Z \cdot R_t \cdot A_i \cdot C_o$$

Where:

Z: Zoning factor (to be 1.0, the regional difference is not considered).

$R_t$ : A value representing vibration characteristics of building to be obtained by the appropriate calculation methods specified in standards and criteria which are assumed to be adequate for safety. Here, ‘the appropriate calculation methods in standards and criteria which are assumed to be adequate for safety’ corresponds to the Building Standard Law etc.

However, if the value which expresses the vibration characteristics and is evaluated considering the structural characteristics of buildings and structures, and the response characteristics and situation of the ground in the seismic condition would be confirmed

	<p>to fall short of the value calculated by the methods in the Building Standard Law etc. it could be reduced to the evaluated value by this method (but equal to or not less than 0.7).</p> <p>Ai: A value representing a vertical distribution of seismic story shear coefficient according to the vibration characteristics of building, to be calculated by the appropriate methods specified in standards, criteria and the other appropriate methods as is like Rt,</p> <p>Co: Standard shear coefficient (to be 0.2),</p> <p>Wi: Total of fixed loads and live loads supported by the part in question.</p>
iii)	<p>Horizontal seismic force which acts on the parts of the buildings and structures under the datum plane should be evaluated by following formula,</p> $Pk = n \cdot k \cdot W_k$ <p>Where:</p> <p>Pk: Horizontal seismic force acting on the part in question.</p> <p>n: Coefficient in accordance with importance Classification of Facilities (Earthquake-proof S Class 3.0, Earthquake-proof B Class 1.5, Earthquake-proof C Class 1.0)</p> <p>k: Horizontal seismic coefficient by the following formula</p> $k \geq 0.1 \cdot \left[ 1 - \frac{H}{40} \right] \cdot Z$ <p>Where:</p> <p>H: Depth of each under part from the datum plane; 20 ( m ) at depths of &gt;20 m</p> <p>Z: Zoning factor (to be 1.0, the regional difference is not considered )</p> <p>W<sub>k</sub>: Summation of dead loads and live loads of the part concerned.</p> <p>Incidentally, in the case if the value would be calculated in evaluating the vibration characteristics suitably by considering the structural characteristics of buildings and structures, and the response characteristics and situation of the ground in the seismic condition, it would be the value calculated by this method.</p>
2)	<p>Vertical seismic force</p> <p>The vertical seismic force in the evaluation of the static force to Earthquake-proof S Class Facilities should be evaluated with the vertical seismic intensity by the following formula,</p> $Cv = R_v \cdot 0.3$ <p>Where:</p> <p>Cv: Vertical seismic intensity.</p> <p>R<sub>v</sub>: A value representing the vertical vibration characteristics of the building, to be 1.0. However, based on special investigation or study, if it would be confirmed to fall short of 1.0, it would be reduced to be the value based on the results of investigation or study (but equal to or not less than 0.7).</p>
3)	<p>Retained horizontal strength required</p> <p>Retained horizontal strength required should be evaluated specified in the method in standards and criteria which are accepted to be adequate for safety.</p> <p>Here, the standards and criteria which are accepted to be adequate for safety correspond to the Building Standard Law etc.</p> <p>Incidentally, in evaluation of retained horizontal strength required, the coefficient regarding the importance classification of the facilities which is multiplied by the seismic story shear coefficient should be settled to be 1.0 in all the case of Earthquake-proof S, B, C Class and standard shear force coefficient Co which is used in this case should be provided to 1.0.</p>

## 7. Load combination and allowable limit

The basic concepts about combination of loads and allowable limit shall be considered in assessing adequacy of design principle regarding seismic safety as follows:

(1) Buildings and structures

1) Earthquake-proof S Class Buildings and Structures

i) Combination with DBGM Ss and allowable limit

Regarding the combination of normal loads and operating loads with the seismic forces caused by DBGM Ss, the buildings and structures concerned shall have sufficient margin of deformation acceptability (deformation at ultimate strength) as a whole, and adequate safety margin compared to the ultimate strength of buildings and structures.

ii) Combination with EDGM Sd and allowable limit

Regarding resulted stress in combining the normal loads and operating loads imposed with the seismic loads caused by EDGM Sd or Static seismic force, allowable unit stress specified in standards and criteria assumed to be adequate for safety shall be established as the allowable limits.

2) Earthquake-proof B, C Class Buildings and Structures

Regarding resulted stress in combining the normal loads and operating loads imposed with Static seismic forces, allowable unit stress in above-mentioned 1) ii) shall be established as the allowable limits.

(2) Components and piping system

1) Earthquake-proof S Class Components and Piping System

i) Combination with DBGM Ss and allowable limits

The functions of Facilities shall not be affected by the occurrence of excessive deformations, crack and failure, even if the most part of structures would reach yield condition and the plastic deformation would occur, with respective resultant stress due to combined respective loads which occur in the normal operating condition, unusual transient condition in operation and accident condition with the seismic loads caused by DBGM Ss.

As for the active components etc., acceleration limit etc. for retaining of function shall be established as the allowable limit, which is confirmed by the verification test etc. regarding the response acceleration caused by the DBGM Ss.

ii) Combination of EDGM Sd with allowable limits

The yield stress or the stress with equivalent safety to this shall be established as allowable limits to respective resultant loads due to combined loads at normal operating condition, unusual transient condition in operation and accident condition imposed with the seismic loads caused by EDGM Sd or Static seismic force.

2) Earthquake-proof B, C Class Components and Piping System

The yield stress or the stress with equivalent safety to this shall be established as allowable limits to respective resultant loads due to combined loads in normal operating condition and unusual transient condition in operation imposed with the seismic loads caused by Static seismic force.

(Commentary)

#### IV. Regarding Load Combination and Allowable Limit

The interpretation of the combination of loads and allowable limits should be based on the followings.

- (1) Regarding ‘respective loads which occur in unusual transient operation and accident’, if the load acted on by the events which are feared being caused by the earthquake and the loads, even if which are not feared being caused by the earthquake but being caused by the events which continue in long term if they would occur once, should be considered to be combined with the seismic load.

However, even if the load is ‘a load which occurs in accident’, considering the relation between occurrence probability of this accidental event and the duration time, and the exceedance probability of the earthquake, the load caused by this event needs not be considered to be combined with the seismic loads if the probability that the both of them occur simultaneously is extremely small.

- (2) Regarding the allowable limits for combination of buildings and structures with EDGM Sd etc. though it was required to be established as the ‘allowable unit stress specified in standards and criteria assumed to be adequate for safety’, this standards and criteria correspond concretely to the Building Standard Law etc.
- (3) ‘Ultimate strength’ in the terms regarding combination of the buildings and structures with DBGM Ss means the bounding maximum bearing load in reaching the condition, which is considered as the ultimate condition of the structures, where deformation and strain of the structure would increase remarkably by adding the load to the structure gradually.
- (4) Regarding the allowable limit of components and piping system, though the basic principle requires to maintain the resulted stress under the ‘yield stress or equivalent safety situation’, this situation corresponds concretely to the situation specified in the ‘Technical Standards on Structures etc. of Nuclear Power Generation Facilities etc.’ which is prescribed in the Electricity Utilities Industry Law.

### 8. Consideration of the accompanying events of earthquake

Facilities shall be designed regarding the accompanying events of earthquake with sufficient consideration to the following terms.

- (1) Safety functions of Facilities shall not be significantly affected by the collapses of the inclined planes around Facilities which could be postulated in the seismic events.
- (2) Safety functions of Facilities shall not be significantly affected by the tsunami which could be postulated appropriately to attack but very scarcely in the operational period of Facilities.

**Topical Area 1.**  
**Regulatory Framework and Objectives of SPRA and SMA Studies**

Development of a Risk-Informed Approach to Seismic Siting and Design  
by US NRC, A. J. Murphy, G. Bagchi, S. Ali, M. Shah,  
H. Graves, C. Munson, Yong Li, and V. Thomas, , Invited

***Technical session***

***Chairs A. Murphy (US NRC) and C.J. Lee (KINS)***

Methodology and Use of PSA/SMA results  
P. Hessel, CNSC, Canada (to be presented by T. Sung)

Assessment Of Seismic Risk – Pilot Study  
S. Andersson, R. Roberts, J. Lundwall, SA Ingenjorsbyra,  
Uppsala University, Ringhals AB, Sweden

The use of the Krško NPP Seismic Probabilistic Safety Assessment  
at the Slovenian Nuclear Safety Administration  
M. Uršič, A. Stritar, D. Vojnovič, A. Mühleisen, Slovenian Nuclear Safety Administration

Seismic Risk Analysis and Design in Finland - A Regulatory View  
J. Sandberg, P. Välikangas and Y. Hytönen, STUK, Finland

## Development of a risk-informed approach to seismic siting and design

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### *Abstract*

In 1996 and 1997, the U.S. Nuclear Regulatory Commission published a new Geologic and Seismic Siting Criteria, 10 CFR Part 100.23 and a series of supporting regulatory guides. Regulatory Guide 1.165, "Identification and Characterization of Seismic Sources and Determination of Safe Shutdown Earthquake Ground Motion" is one of this series and contains the basic guidance for the development of the safe shutdown earthquake ground motion response spectra which actually governs the seismic design of nuclear power plants. This guidance was effectively untested until three utilities applied for early site permits in about 2004. In the same time frame the Electric Power Research Institute (EPRI) completed a comprehensive new study of the attenuation of seismic ground motion for the central and eastern United States (EPRI, 2004). Using this ground motion attenuation model in the EPRI probabilistic seismic hazard assessment (PSHA) methodology, which is specifically acceptable to the NRC under the above regulatory guide, there were a number of exceedances of seismic response spectra for available certified nuclear power plant designs. These exceedances were in the high frequency portion of the spectra and may not represent challenges to the facility.

To address the issue of these exceedances, the Nuclear Energy Institute, acting for the US industry, proposed a revision of the regulatory guidance based on a risk-informed, performance-based approach described in a standard of the American Society of Civil Engineers and Structural Engineering Institute, ASCE/SEI 43-05. The proposed standard is entitled "Seismic Design Criteria for Structures, Systems, and Components in Nuclear Facilities" and was finalized in July, 2005. In addition to changing the basis of the regulatory guide from uniform hazard consistent to risk consistent, NEI has proposed several improvements for the PSHA methodology. This paper describes the risk-informed, performance-based approach and technical basis of the ASCE/SEI 43-05 standard, the review issues with it, and the details of improvements to the PSHA methodology. The NRC staff is planning to have a draft of a new regulatory guide available for public comment late in 2006.

### **Background of current siting rules and standards**

This section describes the technical materials that assisted in the development of the proposed risk-informed and performance-based regulatory guidance for seismic siting of nuclear power plants in the United States. These materials include NRC regulation and regulatory guidance, Department of Energy (DOE) seismic standards, and industry consensus standards. The proposed new NRC regulatory guidance is an alternative to the current Regulatory Guide 1.165, "Identification and Characterization of Seismic Sources and Determination of Safe Shutdown Earthquake Ground Motion", USNRC (1997a), which was available for the four early site permit applications (Clinton, North Anna, Grand Gulf, and Vogtle) under various stages of NRC review in the Fall of 2006. Reg. Guide 1.165 is proposed to remain an acceptable method for determining the safe shutdown earthquake ground motion response spectra.

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### **10 CFR Part 100.23 and Reg Guide 1.165**

After many years (1973 through 1997) of experience with Appendix A to 10 CFR Part 100, “Seismic and Geologic Siting Criteria for Nuclear Power Plants”, the NRC acknowledged both the successful utilization of this regulation and the technical and adjudicatory difficulties in its implementation. These observations led to its replacement with the shortened, streamlined rule found in 10 CFR Part 100.23. This new rule accomplished a number of very important changes: a) it removed the prescriptive guidance found in the rule itself and moved the guidance to a stand-alone regulatory guide, Reg. Guide 1.165; b) it acknowledged the uncertainty in the seismic/geologic knowledge base and requires the uncertainties be estimated through an appropriate analysis; and c) it explicitly accepted the use of probabilistic seismic hazard analysis (PSHA) to satisfy this uncertainty requirement.

The guidance on developing the safe shutdown earthquake ground motion (SSE), which is characterized by a site response spectrum, was placed in Reg. Guide 1.165. The guide was published in March 1997. It contains; 1) guidance on the geological and seismological investigations that should be carried out to fully characterize the nuclear power plant (NPP) site and its environs; 2) guidance on acceptable PSHA databases and methodologies (it explicitly names both the Lawrence Livermore National Laboratory (LLNL) (Berneuter et al, 1989) and Electric Power Research Institute (EPRI, 1989) methods as acceptable.); 3) description of a reference probability that provides uniformity of seismic hazard exposure between operating NPPs and future NPPs (the reference probability is set to a median annual value of  $1 \times 10^{-5}$ ), 4) description of the derivation of the reference probability and its use in obtaining the SSE; and 5) guidance on ensuring that the PSHA databases are current

### **Reg Guide 1.60**

The companion regulatory guidance for Appendix A to 10 CFR part 100 to determine the seismic design response spectra from the safe shutdown earthquake spectra is Regulatory Guide 1.60, "Design Response Spectra for Seismic Design of Nuclear Power Plants", USNRC (1973). In 1997, the NRC staff initiated research to develop a technical basis for the revision and update of the guidance to build on the development of PSHA guidance in Reg Guide 1.165. This research is documented in NUREG/CR reports (McGuire et al., 2001 and McGuire et al., 2002). These two reports are pivotal in providing a technical basis for development of hazard- and risk-consistent seismic spectra through the modification of uniform hazard response spectra (UHRS) by considering the site-specific slope of the hazard curves. The research also examined the available methods for modifying the UHRS for site-specific soil conditions; four methods were evaluated and characterized based on their approach. The four methods were demonstrated for two sites, one in California and the other in South Carolina; the results were discussed with the advantages and issues with each being noted. The research also developed a database of the strong ground motions by magnitude and distance, from source to recorder, for potential use in a revised regulatory guide on design response spectra.

### **Department of Energy Standard 1020**

In 2002, DOE published a revision to its seismic siting standard, “Natural Phenomena Hazards, design and Evaluation Criteria for Department of Energy Facilities”. “This natural phenomena hazard standard ... provides criteria for design of new SSCs (structures, systems and components) and for evaluation, modification, or upgrade of existing SSCs so that DOE facilities safely withstand the effects of natural phenomena hazards, such as earthquakes, extreme winds, and flooding”. It was the performance-based approach that DOE took to the issue of seismic siting that was adapted by the American Society of Civil Engineers and Structural Engineering Institute (ASCE/SEI) in their standard that interested the NRC staff. The DOE standard expressed this approach in the statement: “the intent is to control the level of conservatism in the design/evaluation process such that; (1) the hazards are treated consistently; and (2) the

level of conservatism is appropriate for SSC characteristics related to safety, environmental protection, importance, and cost.” While the ASCE/SEI standard, discussed in the following section, only addressed seismic hazards, it followed the DOE approach of addressing seismic hazards consistently through a performance goal for facility characteristics across the entire country.

#### **ASCE/SEI 43-05**

In July, 2005, the ASCE/SEI published a standard for the seismic design of safety-related structures, systems, and components (SSCs) in a broad spectrum of nuclear facilities, but not specifically addressing commercial nuclear power plants. This standard was developed for DOE and its nuclear facilities. In view of the current trend, within both the conventional and nuclear design and construction industries, toward the use of risk-informed and performance-based methods in various consensus codes and standards, the ASCE/SEI decided to move in this direction as well. The resultant standard is “Seismic Design Criteria for Structures , Systems, and Components in Nuclear Facilities” is referred to as “ASCE/SEI 43-05”. Without specifically addressing commercial NPPs, the standard contains a number of technical concepts that could be useful in design guidance for commercial NPPs. Two of the most important and useful concepts are the development of a technical basis for a performance-based approach to determination of a site-specific, safe shutdown earthquake ground motion response spectra and a technical basis for a target performance goal.

A third prime concept utilized in the standard is that of the frequency of the onset of significant inelastic deformation. (Although the concept is not referred to by the acronym, FOSID by ASCE/SEI, the acronym is being frequently used within the technical community.) ASCE/SEI is making use of this concept instead of the previous standard, seismic core damage frequency (SCDF) that has been the term of reference in seismic probabilistic risk assessments (PRAs) or probabilistic safety assessments (PSAs). FOSID as a limit state ensures that a nuclear power plant SSCs may have localized inelastic response, but they will remain essentially within elastic limits globally. Whereas, the seismic core damage , as a limit state, occurs when there are plant system and component failures. NEI has performed seismic core damage frequency analysis based on seismic hazard and idealized fragility functions for 28 sites and established that the FOSID performance target is conservative, such that the seismic hazard based on the FOSID performance target would limit the SCDF to less than  $5 \times 10^{-6}$  per year.

#### **Public interactions during development of new regulatory guide**

In addition to being required by law, the NRC makes use of interactions with the public during the development of its regulations and regulatory guidance as a matter of good procedure and practice. (One of the NRC’s Strategic Goals is to ensure openness in its regulatory process.) There are two commonly used mechanisms to obtain public input to inform the process; these are public meetings, which are formally announced, through the *Federal Register*, and public comment periods during which draft copies of the material are available. These are also formally announced through the *Federal Register*. In a series of public meetings, the NRC staff interacted extensively with the nuclear industry during the development of the subject regulatory guidance.

#### **Nuclear Energy Institute Activities**

As the U.S. nuclear power generating industry moved toward a resurgence in the development of new generating capacity, the Nuclear Energy Institute (NEI), representing the nuclear industry as a whole, undertook the task of identifying and resolving regulatory issues potentially affecting the efficient licensing of nuclear power plants. While acknowledging the strong points of the current guidance and its up-to-date elements, NEI identified two elements that were potential impediments to the goal of efficient licensing. These two significant impediments are: the current guidance does not take into specific consideration the advances that have been made in the modeling of ground motion propagation in the central and eastern

United States (CEUS) since the EPRI and LLNL PSHA methods and databases were used in R.G. 1.165 in 1997 and the current guidance does not make use of the development of risk-informed, performance-based procedures for determination of site-specific safe shutdown earthquake response spectra.

NEI undertook its New Plant Seismic Issues Resolution Program to develop technical information to specifically inform the NRC process to develop the subject regulatory guidance. This information was provided in a series of reports provided to the NRC and publically available on the NRC website, USNRC (2006).

NEI drafted an Integration Report (EPRI, 2006a) in which it provided a series of recommendations for industry that it proposed to be included in the new seismic siting guidance. These recommendations are to include guidance on:

1. Generic updating of earthquake recurrence and ground motion elements.
  - 1a. Implementation of the cumulative absolute velocity (CAV) filter.
  - 1b. Implementation of the EPRI ground motion model (EPRI, 2004).
2. Implementation of the ASCE/SEI standard 43-05 to determine the safe shutdown earthquake response spectra.
3. Determination of the response of local site geology to seismic waves.
4. Determination of site-specific, risk-informed seismic design response spectra.
5. Determination of the control point location for the site-specific safe shutdown response spectra and the seismic design response spectra.

### ***Basis for Performance-Based Approach and Target Performance Goal***

NEI noted that the NRC, through two documents, made known its desire to move specifically to the use of risk-informed and performance-based principles in new regulations and regulatory guidance (USNRC, 1995 and USNRC, 1998). Building on this NRC commitment to utilize these regulatory principle, NEI advocated the use of ASCE/SEI 43-05 as the basis for a performance-based regulatory guide with a specific mean annual target probability goal of  $1 \times 10^{-5}$ . Through a series of presentations and tutorials at public meetings, NEI and its consultants provided technical material to justify the use of the ASCE 43-05 approach and the target probability USNRC, (2006).

### ***Cumulative absolute velocity filter***

With the introduction of recent ground motion attenuation models for the CEUS into modern PSHA, there has been a tendency of increasing level of ground motion, particularly in the high frequency end of the response spectrum, EPRI (2006b). EPRI attributed a significant portion of this observed increase to the influence of small magnitude earthquakes in the PSHAs. Earlier research has shown that small magnitude earthquakes are not damaging to well-engineered structures, such as those found in nuclear power plants. NEI has proposed using a CAV value of 0.16 g-sec as a filter level to remove the effect of small magnitude earthquakes from a PSHA. O'Hara and Jacobson (1991) have demonstrated that a CAV value of 0.16 g-sec is a level of ground motion observed to be non-damaging the buildings of good design and construction. This is the same value used by the NRC in Reg Guide 1.166, "Pre-Earthquake Planning and Immediate Nuclear Power Plant Operator Postearthquake Action", USNRC (1997b), to check if the nuclear power plant has experienced an operating basis earthquake (OBE).

### ***Public meetings***

Between May, 2005 and September, 2006, the NRC conducted seven public meetings, several of which were of two day duration to collect technical information. This information was used by the NRC staff to inform the preparation of the subject new regulatory guide. Additional public meetings are anticipated after the completion of the planned public comment period.

### **Technical basis for new regulatory guidance**

During the most recent public meeting the NRC staff stated that the staff was developing a regulatory guide that included the following features.

1. The ASCE/SEI 43-05 performance-based approach.
2. An annual mean target performance goal of for seismically induced onset of significant inelastic deformation, i.e., a FOSID goal of  $1 \times 10^{-5}$ .
3. Optional use of CAV filtering for PSHA.
4. Generic soil amplification approaches as outlined in NUREG/CR-6728, McGuire (2001) - procedures, 2A, 2B, 2A/3, & 4.

### ***Target performance goal***

The NRC staff has examined the various facets of the intertwined issues associated decisions with a choice of a target performance goal. The staff has tentatively concluded that the approach described in ASCE/SEI standard 43-05 for SDB-5D (Seismic Design Basis-5D), which targets a mean annual FOSID value of  $1 \times 10^{-5}$  is an acceptable approach. The ASCE standard is a national consensus standard and applicable portions of the standard have been thoroughly reviewed and evaluated by the NRC staff and its consultants (Braverman et. al. 2006). The target performance goal is based, in part, on the Reference Probability of a median  $1 \times 10^{-5}$  used in Reg Guide 1.165 and on the typical frequencies found for existing NPPs examined in the NRC IPEEE program, USNRC (2002). The NRC staff also reviewed EPRI reports regarding the assessment of existing NPP design bases ground motions for the updated seismic hazard using PSHA. Based on this review, the staff tentatively (subject to public comment) concluded that the ASCE 43-05 approach using the FOSID mean annual target performance goal of  $1 \times 10^{-5}$  is reasonable.

### ***Publication***

As of September 29, 2006, the NRC staff anticipates publication of the subject regulatory guide as a draft for public comment in about October or November 2006, and the current plan is to publish the final version in March 2007.

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## Methodology and Use of PSA/SMA Results

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Canadian Regulatory Standard S-294 (PSA for Nuclear Power Plants) requires all licensees to perform a Level 2 PSA for internal and external events. However, for external events the standard allows the licensee to choose other methodologies, provided the other methodologies are acceptable to the Regulator, the Canadian Nuclear Safety Commission (CNSC).

Canada is also working on a draft regulatory document “Design Requirements for Design of Nuclear Power Plants”, defining the regulatory requirements for new reactors’ design, includes safety goals. These safety goals address the risk arising from all possible events, including earthquakes.

Presently, several licensees and designers have requested acceptance of a PSA-Based Seismic Margins Assessment instead of a Seismic PSA.

Before accepting such a methodology, the CNSC has analysed the capabilities of both methodologies (seismic PSA and PSA-Based SMA) for their ability to demonstrate the nuclear power plants meets the Safety Goals. One of the identified problems with SMA is its capability to demonstrate that the risk from seismic events is low enough so that the safety goals are met.

The proposed paper will present the results of this analysis and discuss the problems found. A specific issue will be how to interpret, in a PSA-based SMA, the “mixed cutsets”, i.e. these cutsets that combine seismically induced failures and random component failures.

## Assessment of Seismic Risk in Sweden – Pilot Study

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### ***Abstract***

This pilot study investigates a number of methods and criteria with regard to their potential in assessing the seismic risk for Swedish nuclear power plants.

It is shown that a “Swedish” earthquake according to SKI TR 92:3 with a frequency of exceedance of  $< 10^{-5}/\text{yr}$  (E5) satisfies the criteria of NRC Reg. Guide 1.166 indicating that its effects do not exceed intensity VI according to the modified Mercalli scale. MMI VI is the intensity at which slight damage begins to appear in poorly built buildings implying that the risk of damage to a nuclear plant from an E5-earthquake can be considered very low even if the plant has been built without any consideration given to seismic loads.

As part of this pilot study an integrated ground motion measure, CAV, damage potentials have been compared for a “Swedish” and a “Western United States” earthquake according to SKI TR 92:3 and Reg. Guide 1.60 respectively, which are very different in frequency content. It is found that the “Swedish” earthquake needs to be scaled by a factor of 2 - 4 to get the same damage potential (= equal CAV-value) as a “Western United States” event. The result indicates that the Swedish E5-earthquake with a PGA-value of 0.11g is comparable to a Reg. Guide 1.60-earthquake with a PGA-value of about 0.03g-0.05g, which is considered to be low/non-damaging by most seismic experts. (PGA = *Peak Ground Acceleration*.) The significant parameter controlling this relation is the expected longer time of duration for the American loadings.

In conclusion, the results of the pilot study can motivate significant changes of presently applied requirements and criteria with regard to seismic risk in Sweden.

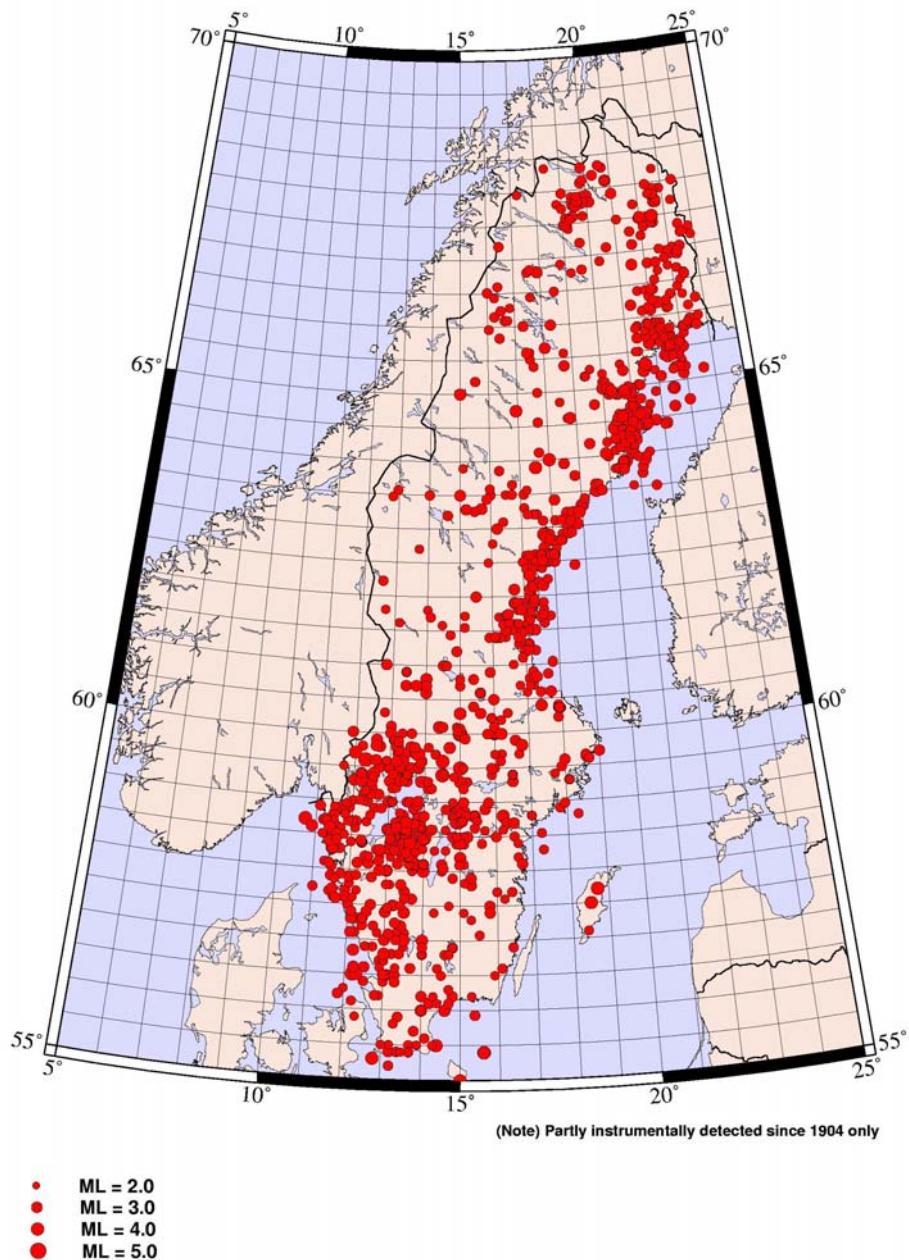
### **Introduction**

Swedish earthquake risk is dominated by near-field earthquake events and is characterized by high acceleration responses at high frequencies. This fact causes problems when applying standard methods (including SQUG and SMA) for seismic re-evaluation. The current study aims at investigating the seismic risk for Swedish nuclear plants and to find out whether there are reasons to revise today’s view of seismic risk.

### **The “Swedish” earthquake**

Sweden’s bedrock mainly consists of ”the Baltic Shield”. These rocks are very old and because of the recurring ice ages, sedimentary sequences are relatively rare and thin. As crystalline rocks often extend almost to the surface, surface seismic velocities are high. The damping of seismic signals is low in these rocks, which means that the observable effects of earthquake vibrations can travel longer distances than in other geological environments, especially at higher frequencies. At the same time there are no strong focusing- and amplification effects such as those that can be created in e.g. sedimentary basins, with large vibrations as a consequence. The fact that Sweden is a country of low seismicity further reduces the risk. Figure 1 shows all known historical earthquakes in Sweden of magnitude over 2.

Figure 1. Earthquakes in Sweden 1375 to 2003

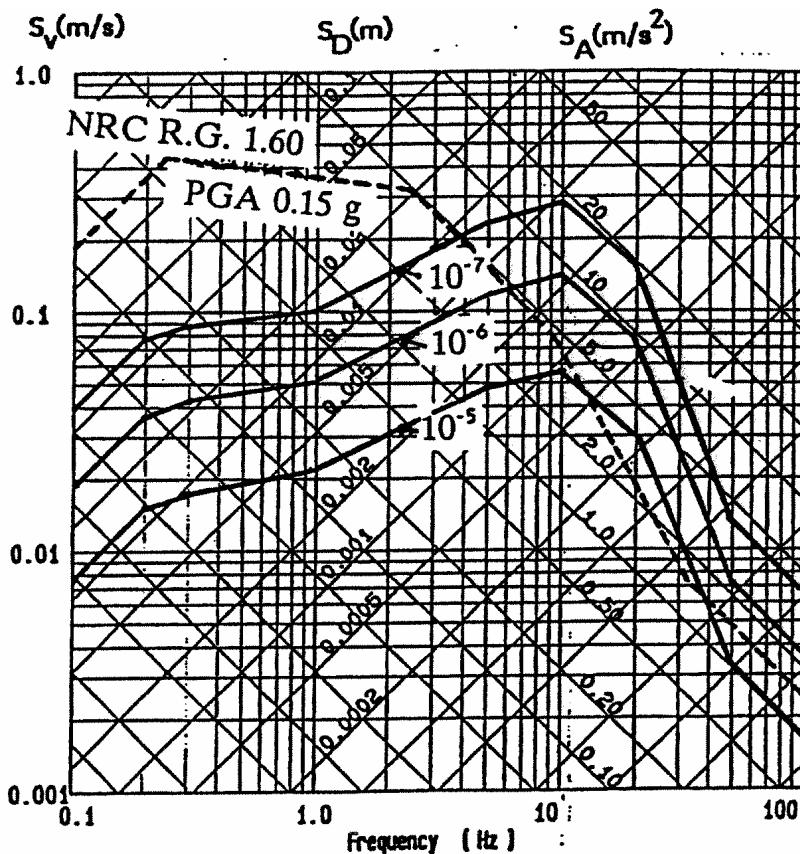


As a result of the work carried out in the late 80's with the probabilistic characterisation of seismic ground motions, uniform hazard ground response spectra for various exceedance frequencies (E-5, E-6, E-7) were outlined [1]. The intention was to provide a basis for a systematic risk analysis covering various combinations of load probabilities and conditional probabilities of failure and damage, if required.

It was found that the seismic risk was mainly governed by earthquakes with hypocentral distances less than 30 km.

In Figure 2, the response spectrum according to Reg. Guide 1.60 is compared to ref. [1] envelope ground response spectra.

Figure 2. Response spectrum comparison



### The CAV-concept, OBE-exceedance

The criteria of NRC Reg. Guide 1.166 [2] aim at defining an earthquake which empirically is not dangerous to industrial plants. MMI VI was chosen as a measure of such an earthquake and it was found that the so called CAV-value (Cumulative Absolute Velocity) was best suited in correlating observed damage and measured ground motion.

CAV is calculated by:

$$\text{CAV} = \int |a(t)| dt$$

where  $a(t)$  = time history of ground acceleration.

NRC have issued a Regulatory Guide, 1.166; "Pre-Earthquake Planning and Immediate Nuclear Power Plant Operator Post-earthquake Actions", March 1997. To characterize a certain earthquake it requires both a response spectrum check and a check of the CAV-value.

The spectrum check involves a check of maximum spectral acceleration in the frequency range 2-10 Hz, and a check of maximum spectral velocity in the frequency range 1-2 Hz; in both cases a damping value of 5% is assumed.

If the spectral acceleration exceeds the OBE-spectrum values, 1/3 of the SSE-spectrum values or 0.2 g, whichever is greater or if the spectral velocity exceeds the OBE-spectrum values, 1/3 of the SSE-spectrum values or 6 inches per second (0.15 m/s), whichever is greater and at the same time the calculated CAV-value exceeds 0.16 gsec, it shall be assumed that OBE was exceeded and act accordingly.

### **CAV based on SKI 92:3 E-5 envelope spectrum**

The criteria are simple to apply and to get a perspective on the Swedish earthquake, spectra and time histories according to SKI TR 92:3 [1] have been studied. The spectra which specify the  $10^{-5}$ /yr-level do exceed 0.2 g at frequencies above 7 Hz. However, the spectral velocity at frequencies  $< 2$  Hz is only about 0.03 m/s and furthermore the calculated CAV-value falls below 0.16 gsec. According to these criteria an E5-earthquake consequently has a damage potential corresponding to  $MMI \leq VI$ .  $MMI VI$  is the intensity at which slight damage begins to appear in poorly built buildings implying that the risk of damage to a nuclear plant from an E5-earthquake can be considered very low even if the plant has been built without any consideration given to seismic loads. An earthquake of this intensity would not require reactor shutdown.

### **CAV for Western United States seismic loadings versus Swedish loading**

As part of this pilot study an integrated ground motion measure, CAV, damage potentials have been compared for a "Swedish" and a "Western United States" earthquake according to SKI TR 92:3 and Reg. Guide 1.60 respectively, see ref. [3]. These earthquakes are very different in frequency content. It is found that the "Swedish" earthquake needs to be increased by a factor of 2 - 4 to get the same damage potential (= equal CAV-value) as the "Western United States" event. The result indicates that the Swedish E5-earthquake with a PGA-value of 0.11g is comparable to a Reg. Guide 1.60-earthquake with a PGA-value of about 0.03g-0.05g, which is considered to be low/non-damaging by most seismic experts. (PGA = *Peak Ground Acceleration*.) The significant parameter controlling this relation is the expected longer time of duration for the American loadings.

### **The relation between CAV and PGA for "typical" Swedish bedrock**

Theoretical studies of the relation between CAV and PGA for "typical" Swedish bedrock have been carried out. For a given earthquake PGA increases faster than CAV the closer the epicenter we get. This is natural since PGA is a measure of a single maximum while CAV is a more integrated measure of the ground motion and the energy distribution in time increases with distance during the travel of the seismic wave through the inhomogeneous Earth. Since risk estimates, especially those based on PGA, are dominated by near-field events and since it seems obvious that CAV is a better measure of damage risk, it follows that risk estimates based on PGA will exaggerate the seismic risk for Swedish nuclear plants.

The numerical simulations are relatively few and the study should be regarded as preliminary. A number of parameters can be changed in the model including depth, size and orientation of the earthquake and the structure of the Earth. Adjustments of these parameters would of course change the calculated PGA- and CAV-values. Using reasonable parameter values for Swedish conditions the model shows that at a distance of 20 km a PGA-value of 0.1 g corresponds to a CAV-value of about 0.1 gsec. Despite the limitations of the model we achieve a surprisingly good comparison with [1] where the earthquake load is dominated by events within a radius of about 30 km and where a PGA-value of 0.11 g corresponds to a CAV-value of 0.13 gsec. Compared to published data from USA this implies a risk picture with a relatively low CAV in relation to PGA, a conclusion fully in compliance with the judgement that larger, more distant earthquakes are more important for risk in USA than in Sweden.

### **Conclusions**

Using the parameter CAV it is found that the damage potential of the Swedish E5-earthquake corresponds to intensity VI of the modified Mercalli scale. Thus the risk of damage to well-built a nuclear plant from an E5-earthquake can consequently be considered very low even if it has been built without any consideration given to seismic loads. The only known components where uncertainty remains are some relays of older type which have proved to be relatively acceleration sensitive when tested, especially in "undrawn" condition, and if they are important to safety they need a closer study (or need to be exchanged).

Summing up we arrive at a picture indicating that earthquake events do not constitute any significant risk for Swedish nuclear plants. The benefit of resources spent on reducing seismic risk will therefore be very ineffective from a cost point of view, which should be taken into account when formulating seismic requirements. Similar observations evidently have been made in other countries, the following quotation from [4] is one example: *"In general, concerning the re-evaluation methodologies, when mean probabilities of exceedance at the 10<sup>-4</sup>/a level are less than 0.1g peak ground acceleration, considerations should be given to developing reduced scope Seismic Margin Assessment procedures. The existing procedures which were developed on a site specific basis for moderate — 0.12–0.33g peak ground acceleration sites may not be appropriate or cost benefit effective for such low seismicity sites and new approaches should be developed for a realistic, but safe, analysis of the protection against external events."*

In conclusion, the results of the pilot study can motivate significant changes of presently applied requirements and criteria with regard to seismic risk in Sweden.

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## The use of the Krško NPP Seismic Probabilistic Safety Assessment at the Slovenian Nuclear Safety Administration

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### ***Abstract***

The hazard of seismic event was already recognized during the design and construction phase of the Krško NPP. Due to this the Krško NPP was designed and constructed as a robust, seismically resistant plant. In the recent years extensive studies, considering more recent geologic, seismologic, geophysical and geodetic investigations of the plant region have been performed in the scope of the Periodic Safety Review (PSR). Studies resulted in the Probabilistic Seismic Hazard Assessment (PSHA) which was used for a complete update of the seismic Probabilistic Safety Assessment (PSA). The last seismic PSA study of the Krško NPP was conducted in 2004. An improved seismic PSA model in the study brought the seismic model closer to a real NPP configuration. That was important because it increased reliability of results, sensitivity studies,...

The Slovenian Nuclear Safety Administration (SNSA) uses the PSA results as a source of information and the PSA models as a background for performing analyses. The SNSA also uses PSA studies for informing the expert community. Next, the PSA sensitivity studies are used to value further plant modifications. Furthermore, SNSA decided to extend the use of PSA at SNSA. The result will be a construction of a new internal PSA information system based on the PSA models, which will primarily be used in the risk-informed decision making process.

### **Introduction**

The seismic hazard was recognized early, during the design and construction phase of the plant. Consequently, the Krško NPP was designed and constructed as a robust, seismically resistant plant. Further examination of seismic faults and their reevaluation in the region of the plant and plant's various modifications led to the necessity for renewed plant safety verifications. Besides, an important objective was the identification of possible cost-effective improvements to support the decisions for various plant changes and upgrades.

The most important challenge of the plant safety verification is management of safety margins built into the plant due to relatively large uncertainties related to seismic hazard. The regulator should be able to identify parameters which are critical for maintaining adequate safety margins. This objective is pursued through a review of the Krško NPP seismic analyses.

In the internationally accepted safety philosophy the Periodic Safety Review (PSR) is a comprehensive review for verification that an operating NPP remains safe when judged against current safety objectives and practices [1]. The PSR also enables verification that adequate arrangements are in place to maintain an acceptable level of safety [1]. Next, it is considered as an additional measure to deal with the cumulative effects of plant ageing, modifications, operating experience and technical developments over long time-scale period [1]. Furthermore, the PSR as a tool enables the NPP identification of issues important for improving nuclear safety. Finally, it also gives to the regulatory body an overall view of the safety status of the plant.

One of the important parts of the PSR is also a review of seismic hazard. The guidance for performing a safety assessment at the Krško NPP was based on the International Atomic Energy Agency (IAEA) safety report [2 – 3]. The purpose of the seismic safety assessment was to prove safe NPP operation during

operating basis earthquake (OBE) and fulfilling safety requirements at safe shutdown earthquake (SSE). According to international practice the seismic PSA method was selected for safety assessment at the Krško NPP. The last seismic Probabilistic Safety Assessment (PSA) study was conducted in 2004.

The aim of this paper is to present evolution of the Krško NPP safety assessment due to seismic hazard on the basis of studies made by other authors [4 – 8]. The main stress of the report is upon results of the 2004 seismic PSA study. The paper also describes the use of the Krško NPP PSA results, models, data, reports..., at the SNSA now and in the near future.

### **Evolution of the Krško NPP safety assessment due to seismic hazard BEFORE 2002 and its use at the sns**

#### ***Original design***

The Krško NPP is located in a seismically active region. For this reason extensive studies based on detailed geologic, seismologic, geophysical and geodetic investigations were carried out in the period between 1964 and 1975. With deterministic analyses of collected data the value of 0.3 g was accepted for the design ground acceleration at the foundation level [4]. Therefore, in the original design of the Krško NPP, response horizontal free-field spectrum from US NRC Regulatory Guide 1.60 scaled to 0.3 g peak ground acceleration (PGA) was used for SSE [4, 9].

The construction of the plant began in 1975. The Westinghouse 2-loop pressurized water reactor (PWR) design was chosen. Testing started in 1981 and since 1983 the Krško NPP has been in commercial operation.

#### ***Leak-before-break concept***

In 1998, the SNSA ordered a study of stress analyses of the primary cooling loop in the Krško NPP and evaluation of applicability of the leak-before-break (LBB) concept. The analyses [5] were made by the Laboratory for Numerical Modeling and Simulation, (LNMS), Faculty of Mechanical Engineering, University of Ljubljana. The shell-type numerical model of the cooling system and relevant structural components (reactor vessel, steam generators, reactor cooling pumps, auxiliary lines of radius 8" and more) was made [5]. The model was used to evaluate static and dynamic loads defined by the design operating conditions [5]. The floor response spectra based on the original design was used in the application of the LBB concept, which has to take into consideration also the SSE and OBE seismic loads [5, 10]. The method of the stress calculation was based on the ASME code [5]. The results of the analyses showed sufficient persistency of the primary piping, which can preserve the integrity of the pressure boundary even in the most critical loading conditions [5].

#### ***PSHA and seismic PSA methodology***

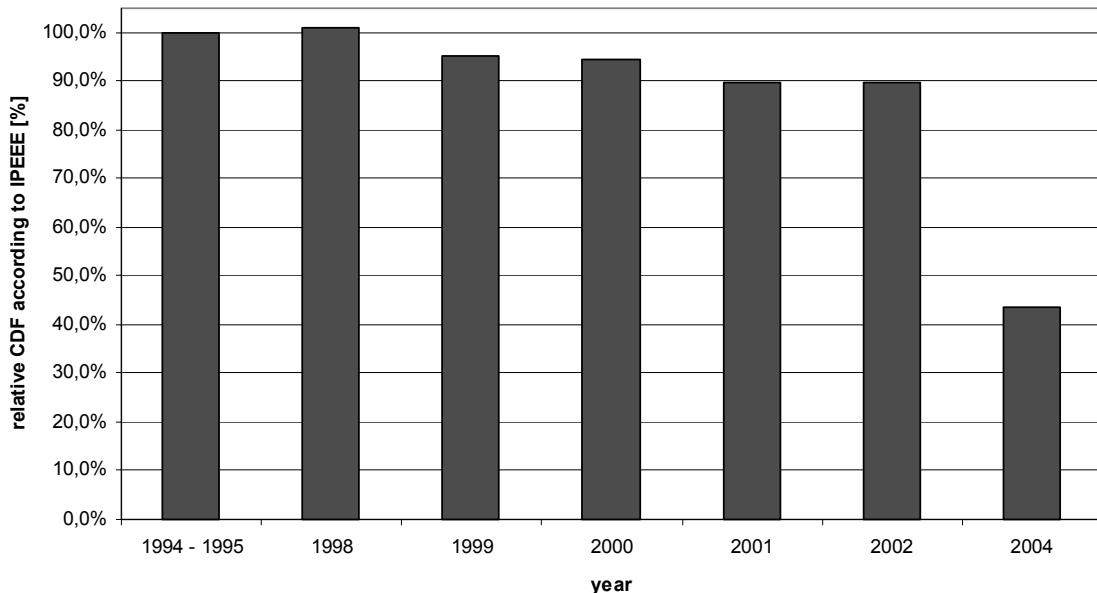
In the nineties a reevaluation of seismic hazard was made on the basis of Probabilistic Safety Hazard Assessment (PSHA) methodology, which was developed in the eighties and was also recommended by the International Atomic Energy Agency (IAEA). PSHA is a method which provides information in terms of frequencies of seismic events with various levels of ground motion typically represented by PGA [11]. The PSHA study was made in 1994 by the Institute of Structural Engineering, Earthquake Engineering and Construction Informatics (IKPIR), Faculty of Civil and Geodetic Engineering, University of Ljubljana. The reevaluation was based on the existing geologic, seismologic, geophysical and geodetic information [4]. The procedures and results were reviewed by the IAEA.

It was concluded from the 1994 PSHA study findings and recommendations given by the IAEA, that there was insufficient information regarding local faulting [2, 12]. Therefore, the SNSA and the Krško NPP decided to implement a program of additional geological and geophysical investigations in the vicinity of the power plant [2].

Due to new knowledge about seismic risk in the 1994 PSHA study, new inputs were used in an Individual Plant Examination of External Events (IPEEE) which also included the seismic PSA [13]. The seismic PSA was selected as a tool for assessment of the risk to which a NPP and its environment are exposed due to earthquakes [11]. The seismic PSA has been also proven to be very useful in identifying and assessing weak points in a plant's response to seismic events [11]. In the scope of the IPEEE the seismic PSA was performed both for a PSA level 1 to assess core damage frequencies (CDF) and for a level 2 to assess release categories (RC) frequencies induced by the earthquake.

The evolution of the seismic contribution from the IPEEE model (1994 – 1995) to the 2002<sup>1</sup> model is shown in Figure 1. A comparison of the PSA level 1 results, which remained essentially unchanged, indicates almost unchanged PSA level 1 model of seismic initiating events [6, 13 – 18]. The last 2004 seismic PSA study results gave the first significant reduction in seismic CDF (Section 3.2).

**Figure 1. Relative change of seismic CDF according to the results of IPEEE model (1994 – 1995). The reduction of CDF in 2004 was significant and was a result of the 2004 seismic PSA study. [6, 13 – 18]**



## 2004 Seismic probabilistic safety assessment study

As part of the ongoing PSR of the Krško NPP, a study was conducted to address seismic design and a new PSHA [6]. The 2002 PSHA study was performed by IKPIR [7]. Due to findings in the new 2002 PSHA study a complete update of the seismic PSA of the Krško NPP was conducted in 2004 [6]. The seismic PSA study was performed by the ABS Consulting in accordance with the new ANS Standard for seismic probabilistic risk assessment [6]. A target was set that updated seismic PSA should correspond to capability category II of the mentioned standard, which required additional review, calculations and extension of the model with respect to the IPEEE model [2]. The study has been reviewed by IRSN (Institut de Radioprotection et de Sûreté Nucléaire, France) [19].

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1. The NPP Krško PSA model from 2002 [18] presents the update of the model version from 2001 [17]. The 2002 model reflects changes of Instrument Air System [18]. The 2001 model included the knowledge of plant design and configuration status at the end of 2001 Outage and of plant history up to the end of the year 2000 [17].

### **From 2002 PSHA to 2004 seismic PSA**

The 1994 PSHA study revealed a lack of information regarding capable faults<sup>2</sup> at the Krško NPP site and its vicinity [4]. With regard to insufficient information, additional investigations were carried out in the periods 1994 – 1996 and 1999 – 2000 [4]. Extensive studies, considering more recent geologic, seismologic, geophysical and geodetic investigations, were performed in the scope of the PSR process ending in the new 2002 PSHA study [7]. Compared to the 1994 PSHA results the new PSHA predicts higher seismic hazard at the Krško NPP site. Higher estimates are mainly due to seismic sources with larger potential magnitudes at a smaller distance to the site and influence of local soil conditions [7].

In the design process the designed ground acceleration 0.3 g was conservatively applied also to the foundation level. Current practice is to deconvolute acceleration from the free field to the foundation level [7]. The PGA value determined in the 2002 PSHA study was 0.56 g at the free field and 0.38 g at the foundation level (for the return period of 10.000 years) [7].

2002 PSHA results were used as input data for performing 2004 seismic PSA study.

### **2004 seismic PSA model**

The seismic PSA methodology is schematically shown in Figure 2. To quantify the plant response from an earthquake, a fault and event trees are used to model the plant functions. Inputs for a model are probabilistic seismic hazard description and fragility<sup>3</sup> of essential structures, systems and components in the model [6]. Combining fragility curves with random and human error failures results in different plant damage states. Seismically induced CDF is given by the integration of the annual probability of occurrence of the different earthquake PGA values and the conditional probabilities of occurrence of each plant damage state results. Bridge trees are used to describe failure modes of the containment caused by seismic events [6]. A bridge tree links the Level 1 plant damage states and the Level 2 release categories.

Due to the increase in the seismic hazard (PSHA study), the increase in the calculated seismic CDF was expected. For this reason additional new equipment were installed and changes of the model were preformed in the preparation phase of analysis. Conservatism in the plant model and data were also decreased.

The evaluation reveled that beside a significant effect of increased seismic hazard more detailed model of relay (electro-mechanical relays, motor starter contactors, breakers and switches) behaviour during a seismic event also had a moderate impact on the increase of seismic CDF [2]. Nevertheless, as shown in Figure 1 the seismic CDF was significantly reduced in the 2004 seismic PSA study. The main changes in the seismic PSA model affecting the reduction of the calculated seismic CDF were [2, 6, 8]:

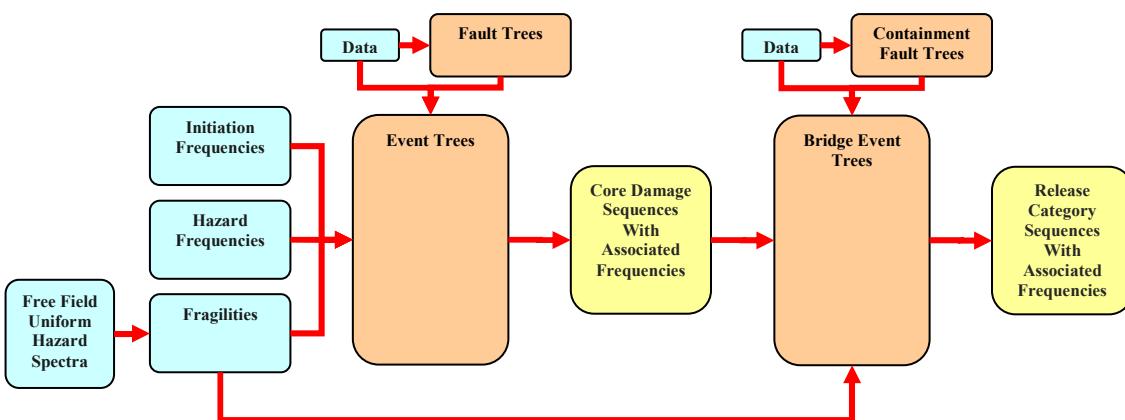
- Significant affect was due to the model change, where the 110 kV off site power system in parallel with the 400 kV system is now taken as seismically independent.
- Significant changes in the plant were made during the modernization program in the year 2000 when the steam generators were replaced. The steam generator replacement involved changes in piping routing and supports. Consequently a re-analysis of most of the primary and secondary systems in the

- 
2. According to US NRC regulation 10 CFR Part 100 [20], a capable fault is a fault which has exhibited one or more of the following characteristics:
    - Movement at or near the ground surface at least once within the past 35,000 years or movement of a recurring nature within the past 500,000 years.
    - Macro-seismicity instrumentally determined with records of sufficient precision to demonstrate a direct relationship with the fault.
    - A structural relationship to a capable fault according to first or second characteristics of this paragraph such that movement on one could be reasonably expected to be accompanied by movement on the other.
  3. Fragility is a conditional probability of seismically induced failures [11].

containment was made. The results of the new analyses were factored into the development of fragilities of the primary system piping and equipment. The results were also used to replace a generic small and medium break loss of coolant accident fragility used in the original seismic PSA with the plant specific fragility. The effect was significant.

- Moderate influence was due to the increased screening level for the calculated equipment capacity.
- Use of plant specific diesel generator control panel fragility also had moderate impact.
- Several changes due to repeated or new plant specific fragility calculations and more exact accident sequences modeling had small impact on CDF reduction. The most important changes were due to: new inverters fragility; diesel generator fuel tank fragility recalculation; equipment fragilities; refueling water storage tank (RWST) fragility recalculation; condensate storage tank fragility recalculation; more precise station blackout, anticipated transient without scram (ATWS) and loss of essential service water modeling; incorporation of positive displacement pump into the model of reactor cooling pump seal injection; incorporation of emergency boration into the model; incorporation of the concrete shield wall around the RWST as a secondary water barrier in the event of failure of the RWST into the model; and improvements of the logical model.

Figure 2. Seismic PSA methodology [6]



### 2004 seismic PSA results and main conclusions

Seismic PSA has been performed both for PSA level 1 and for level 2. Risk Spectrum was used to quantify the seismic event trees for each of the seismic initiating events. The main contributions to seismic CDF (over 80% of the total seismic CDF) are caused by earthquakes stronger than the design basis earthquake [6].

An importance analysis was preformed to identify the dominant contributors to seismic CDF. The importance was expressed as the change in CDF when the event was removed from the analysis. The most significant seismic initiating events are seismic station blackout (61.2% change in CDF), seismic loss of off-site power (13.5% change in CDF) and seismic ATWS (10.6% change in CDF). The most important seismic failure events are due to seismic failure of diesel generator control panel (24.6% change in CDF) and seismic failure of the condensate storage tank (11.3% change in CDF). Importance of the operator's actions (fail to switch valve alignment from the condensate storage tank to essential service water brings 5.8% change in CDF) as well as importance of non-seismic failures, where the main contributor is both diesel generators (diesel generator not recovered before core uncovering brings 26.3% change in CDF, diesel generator 1 fails to run 18.6% change in CDF, diesel generator 2 fails to run 18.5% change in CDF,...), were estimated. [6]

In case of the Krško NPP the PSA Level 2 results are represented by twelve release categories. They are grouped into three major categories of a release. Relative contributions are shown in Table 1. The “small release” is the most dominant among all of them. Large contribution in the “small release” is mainly due to

the importance of station blackout type events and ATWS events resulting from a seismic initiator. The “large release” is a very low probability event and is mainly due to containment bypass in earthquakes stronger than 1.1 g. [2, 6]

Uncertainty analyses were also preformed with the combination of uncertainty in the seismic hazard, the fragilities, random failure and human reliability. The base case seismic CDF was based on the mean seismic curve and mean seismic fragility curves for systems, structures and components. The predicted seismic CDF increased by about 24 % if also uncertainty in random failures and human error probability were included. Uncertainty in the seismic hazard and fragility was determined to have a bigger effect than uncertainty in random failure and human reliability. [6]

The main conclusions from 2004 seismic PSA study were:

- The updated seismic PSA met the requirements for a Capability Category II seismic probabilistic risk assessment as defined in the ANS standard for external event probabilistic risk assessment.
- The seismic PSA successfully addressed all seismic issues identified during PSR.
- Even though the seismic hazard increased, the seismic CDF significantly decreased (Figure 1).
- Large early release frequency was shown to be a very low probability event.
- The Krško NPP was designed and constructed as a robust plant with sufficient reserve in the case of a seismic event.
- The power plant and its operation is safe.

**Table 1. Major release categories.**  
Only influence of the seismic event is considered. [6]

<b>Major Release Category</b>	<b>Seismic Conditional Probability [%]</b>
Very small	10.6
Small	84.0
Large	5.4

### **The use of The Krško NPP Seismic PSA at the SNSA**

The SNSA uses results from the seismic PSA study to evaluate further plant modifications, for guidance of inspection work, to inform management and public about nuclear safety,...

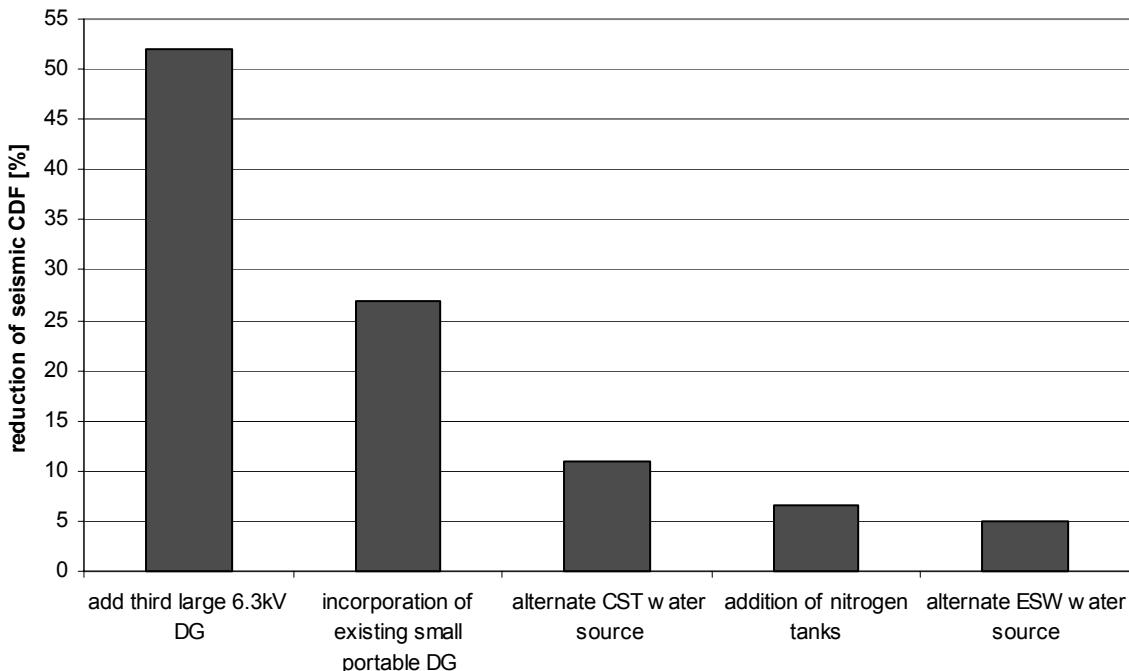
#### ***Further plant modifications***

Seismic PSA enables NPP risk assessment in case of an earthquake. The result is a quantitative measure of seismically induced reactor core damage and a measure of release into the environment. The study can also measure the value of further plant modifications.

Sensitivity studies indicating the value of further plant modifications were performed in the 2004 seismic PSA study. Modifications like additional third independent full size diesel generator, incorporation of existing small portable diesel generator (DG) to the power positive displacement pump and battery charger, implementation of backup to existing condensate storage tank (CST), addition of nitrogen tanks for operation of pressurizer power operated relief valves and implementation of backup to the existing essential service water (ESW) system, were evaluated. The results are shown in Figure 3.

It was evaluated that especially the addition of third large 6.3kV or incorporation of the existing small portable diesel generator would significantly reduce the seismic risk [6]. Due to significant safety benefit the SNSA showed strong interest in the additional diesel generator to increase the Krško NPP safety in case of a seismic event and also other events with loss of offsite power. Therefore, the Krško NPP is preparing a new detailed analysis to choose the combination of the most cost-effective modifications.

**Figure 3. Potential CDF changes due to plant modifications.**  
Only influence of seismic event contribution is considered [6].



### ***Informing public community***

As it is SNSA's desire to openly and regularly communicate with the public, the SNSA prepares reports which are also available to the public. Especially the last seismic PSA study, prepared as part of PSR, was extensively presented to the wider expert community. An extensive abstract of the 2002 PSHA [7] and the 2004 seismic PSA [6] studies was first separately prepared by the experts [4, 8] in cooperation with the SNSA. Next, the extensive abstract was combined with an extensive report "The analysis of seismic sensitivity of the Krško NPP during Periodic Safety Review 2001-2004" [2] and its abridged version. Furthermore, the evolution of the Krško NPP safety assessment due to seismic hazard, with focus on 2004 seismic PSA study results, was presented at an international conference [21].

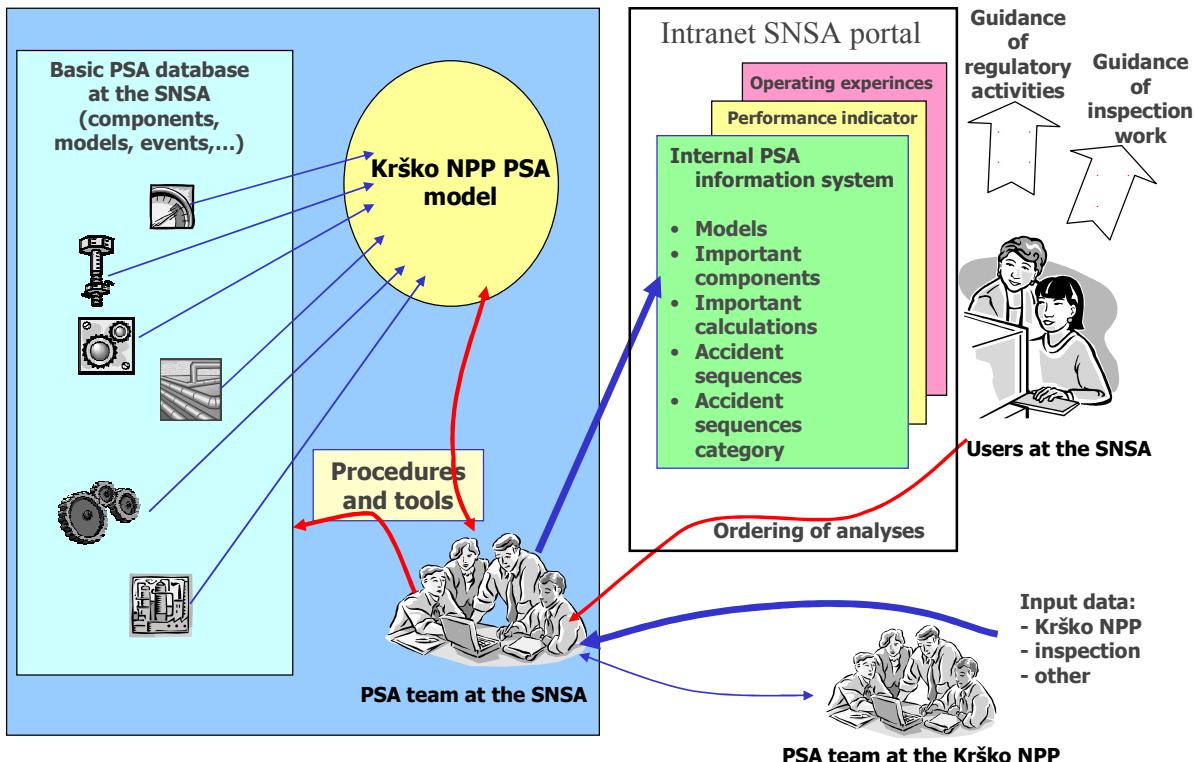
### ***Future use of PSA at the SNSA***

The SNSA has decided to extend the use of results and models of the seismic PSA and also other internal and external events at the SNSA in the near future. The result will be a new internal PSA information system as part of an intranet SNSA portal which was developed in 2004 [22].

The internal PSA information system will be based on the PSA models, data, results, reports and all other relevant PSA information. It will be the central tool for data evaluation and planning of regulatory activities and inspection work. The system will also be used in the risk-informed decision making process to complement the deterministic approach and other requirements (legal, regulatory, cost-benefit...) and to provide additional insights that would otherwise not be available [23]. Use of integrated approach can lead to an improved decision making process that can improve safety and lead to a more efficient and cost effective use of resources [23].

The future role of the internal PSA information system at the SNSA is schematically presented in Figure 4.

Figure 4. Schematically presented use of the PSA at the SNSA



The position of the internal PSA information system in the process of PSA use is also shown. The internal PSA information system is part of the intranet SNSA portal. The internal PSA information system will be used in the decision making process (modification assessment, plant operation safety,...), for inspection planning, for regulatory activities planning, for importance indication of findings during inspection,... conclusion

The 2002 PSHA study pointed out higher seismic hazard at the plant site in comparison with the 1994 PSHA study and the one taken into account during the construction of the plant. The higher seismic hazard is mainly due to seismic sources with larger potential magnitudes at a smaller distance to the site and influence of local soil conditions. Nevertheless, the values of the 2002 PSHA study at the foundation level can be compared with the values used in the original design. Results of the 2004 seismic PSA study showed that even though the seismic hazard defined in the 2004 PSHA has increased, the CDF is lower than previously assumed (Figure 1). The CDF has decreased due to new equipment added to enhance safety, addition of some systems previously assumed to be unavailable after a seismic event, and removal of some conservatism in the plant model and data.

Sensitivity studies were performed in the 2004 seismic PSA study. It was evaluated that especially the addition of the diesel generator would significantly reduce the possibility of radioactive releases to the environment in case of a seismic event and also in case of initiating event with loss of offsite power. The SNSA showed high support for the diesel generator addition. The Krško NPP is now preparing a new detailed analysis to choose the combination of the most cost-effective further plant modifications.

Besides using PSA studies to assess further plant modification, the SNSA also recognized many other benefits of the PSA studies. First, the PSA is being used as a source of information and for performance of analyses. Next, the SNSA also uses PSA studies for informing the wider expert community on the Krško NPP safety. Especially the last seismic PSA study was extensively presented. Furthermore, the SNSA decided to extend the use of PSA at the SNSA in the near future. The result will be the construction of a

new internal information system based on the PSA models, which will be used in the risk-informed decision making process (modification assessment, plant operation safety, ...), for inspection planning, for regulatory activities planning, for importance indication of findings during inspection,...

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# Seismic Risk Analysis and Design in Finland

## A Regulatory View

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### ***Abstract***

In Finland seismic activity is low. When the operating four units were built in the late 1970's there were no regulatory requirements for seismic design of NPPs. Later the seismic risk studies have revealed some significant vulnerabilities related to, e.g., supports of electrical cabinets and battery racks. The main risk contributors have been eliminated by plant modifications.

Current Finnish regulations require seismic design and seismic risk analyses for new units and for major modifications of the existing units. A design basis ground response spectrum corresponding to 100 000 year recurrence period has been determined for the NPP sites. In Olkiluoto 3 (EPR 1600) currently under construction all safety significant systems, structures and components will be qualified against seismic events. The design of the new unit to withstand a collision of a large passenger aircraft also requires dynamic structural analyses.

### **Introduction**

Finland is situated in the north-western part of Europe and belongs to the Baltic shield. Finland is in an intraplate area where seismic activity is quite low. The most severe instrumentally registered earthquake in the vicinity of the Finnish sites is the magnitude 4.9 earthquake in Estonia in 1976. Altogether about ten earthquakes with magnitude higher than 4.5 have been observed in Finland.

There are four operating nuclear power plant units in Finland: the TVO power company has two 840 MWe BWR units supplied by Asea Atom at the Olkiluoto site on the south-eastern coast of Finland and Fortum (formerly IVO) has two 500 MWe VVER 440/213 units at the Loviisa site on the southern coast of Finland. The units were commissioned between 1977 and 1982. Seawater is used as the ultimate heat sink at both plants, and all units have been constructed on hard crystalline bedrock. In addition, a 1600 MW EPR (European Pressurized Water Reactor) is under construction at the Olkiluoto site. The plant is supplied by a consortium of AREVA NP, formerly Framatome ANP, (nuclear island) and Siemens (turbine island). The construction permit for Olkiluoto 3 (OL3) was granted by the Finnish Government in early 2005 [1,2].

### **Seismic design requirements**

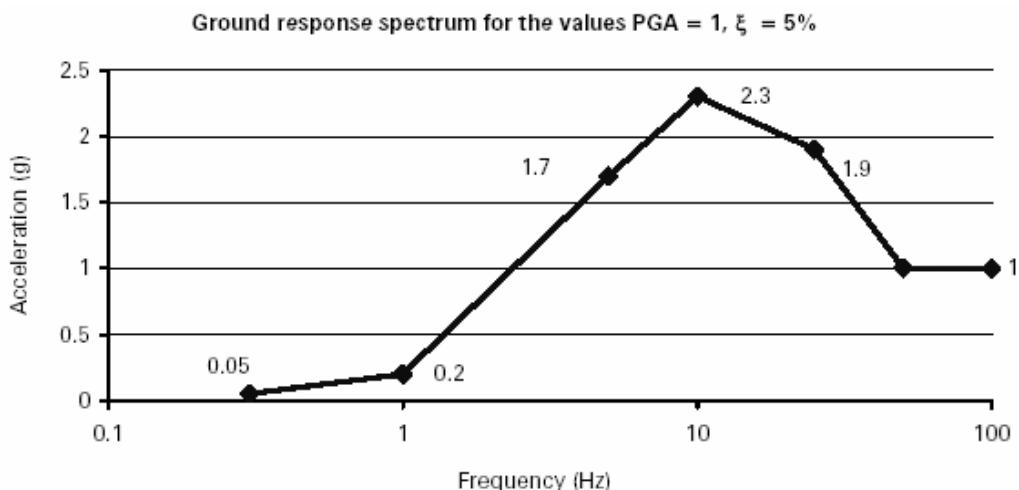
When the operating Finnish NPP units were built there were no regulatory requirements on seismic design, and earthquake loads were not considered separately in the design. Later seismic risk analyses have revealed some seismic vulnerabilities which have been mainly removed with plant modifications.

Nowadays the seismic requirements on design of nuclear power plants are set forth in the regulatory guide YVL 2.6 which has been last revised in 2001 [3]. The guide also requires that seismic events are analysed in PSA.

The design basis earthquake shall be determined so that its recurrence interval is 100 000 years on the 50% confidence level. The design basis earthquake is characterized by a ground response spectrum. The ground response spectrum shall be presented and justified by the licence holder or applicant, and it is reviewed by the safety authority STUK.

The ground response spectrum submitted by TVO has been determined with a statistical procedure described in [4] using an earthquake catalog covering a period of about 600 years [5]. The spectrum, Figure 1, is peaked at relatively high frequencies because the Finnish bedrock is composed of hard crystalline rock. The calculated horizontal peak ground acceleration is about 0.065 g at Loviisa and 0.085 g at Olkiluoto. According to the IAEA recommendations [6], the design PGA is taken as 0.1 g at both sites. As the Finnish NPPs are founded on bedrock, no calculations for soil amplification are required.

**Figure 1. The design ground response spectrum**  
with damping ratio 5 % determined for southern Finland, YVL 2.6 [3].  
In Olkiluoto PGA = 0.085 g and in Loviisa PGA = 0.065 g.



Due to the low seismic activity, only the safe shutdown earthquake is relevant in Finland. In areas with higher seismic activity, an operating basis earthquake with recurrence period comparable to the operating life of the plant is defined as another seismic design value. The normal operation of the plant should be possible in the case of an operating basis earthquake. In Finland the operating basis earthquake would be so weak that it would not result in any practical requirements for the plant systems.

As a novel feature, OL3 is designed to withstand an airplane crash (APC) of large passenger jet. The main challenges of APC are the strength of the outer walls against the direct impact and the design of openings against the external fuel fire. In addition, the effects of induced vibrations on safety systems have to be considered. As seismic design loads are low in Finland, the AP induced vibrations may be dominant especially for high frequencies.

In practice, the design and qualification of structures and equipment against induced vibrations involves a set of analyses and qualification tests:

1. An acceleration time history corresponding to the ground response spectrum will be defined for the base slab of the building. The design basis ground response spectrum, Figure 1, is based on seismic studies and statistical analysis. This is also the case for an airplane collision to the nuclear island, since there will be a protective shield which isolates the reactor building, fuel building and certain safeguard buildings from the direct impact of an airplane.
2. Structural dynamic analyses will lead to corresponding floor response spectra of buildings.
3. A method to combine all relevant load cases and location possibilities of equipment is to collect them under frequency dependent envelope spectra. One or a set of envelope spectra are used as criteria for structural design and qualification of structures and equipment.
4. Design and qualification of structures and equipment is based, on the other hand, on corresponding vibration response investigation in order to ensure that all relevant frequencies and modes are covered.

5. Stability of structures and buildings will be checked against the highest accelerations in the critical frequency areas.
6. Dynamic time dependent design and/or testing loads are calculated from the envelope spectra (inverse Fourier transformation).
7. Finally the proper dynamic behavior of structures and equipment will be verified against calculated design and/or testing loads.
8. In case testing has been used in qualification, vibration response investigation will be repeated by testing in order to ensure, that no serious structural breaking has occurred and that the testing has proceeded as planned so that also the last test is describing the design situation

Seismic loads shall also be considered in the design of major plant modifications and in renewal projects. For example, the seismic design of new buildings in the Loviisa automation renewal project is described in [7].

## **Seismic PSA in Finland**

### ***The general PSA approach***

The possibilities of probabilistic methods in nuclear safety management were recognized by the Finnish authorities and licensees in the early 1970's. In 1984 STUK formally required that the Finnish licensees perform PSA studies. It was also required that the PSA studies are performed as in-house projects by the licensee personnel to ensure development of expertise and commitment to PSA and its applications. Outside consultants were to be used only for special tasks. The goal for each plant was a full scope living PSA model which is easy to use and is kept constantly up-to-date. These decisions laid the foundation for the present use of PSA in risk informed regulation by the authority STUK and in risk informed safety management by the licensees.

Living PSA models have been developed for both the Olkiluoto and Loviisa plants. At the moment, Level 1 studies for full power operation cover internal events, area events (fires, floods), and external events such as harsh weather conditions, and seismic events. For shutdown and low power states Level 1 PSA covers internal events and some area and external events. Level 2 studies cover internal events and some area events for full power operation states.

The Olkiluoto PSA model has been implemented on the SPSA program which has been developed at STUK. The Loviisa PSA was originally developed with the CAFTA program, but the model has recently been moved to Risk Spectrum.

STUK and the licensees have an agreement for introducing the Living PSA as a common information platform. According to the agreement the identical, reviewed PSA model is used for resolution of safety issues both by the licensee and STUK. The use of the same PSA model gives a common basis for discussions between the authority and the licensees on risk-related issues. A prerequisite for the use of a common model is a thorough review of the PSA models by the authority, with the assistance of consultants in some special cases.

The requirements on PSA are set forth in the Regulatory Guide YVL 2.8 originally issued by STUK in 1987 and last revised in 2003. The guide emphasizes the Living PSA approach. PSA covers the whole life cycle of a nuclear power plant. It is formally integrated in the regulatory process of nuclear power plants already in the early design phase and it is to run through the construction and operation phases all through the plant service time [8].

The Guide YVL 2.8 includes probabilistic safety requirements:

- The core damage frequency shall be less than  $10^{-5}/\text{year}$ .
- The large release frequency shall be less than  $5 \cdot 10^{-7}/\text{year}$ .

The “large release” is defined as 100 TBq of Cs 137 which corresponds to only about 0.015 per cent of OL3 Cs 137 inventory. This is actually the release limit for severe accident management design defined in Government Decision on general regulations for the safety of nuclear power plants, VNp 395/1991 [3].

As concerns a new plant unit, a concise plant specific design phase PSA including Levels 1 and 2 is required as a prerequisite for issuing construction license, and a complete Level 1 and 2 PSA is a condition for issuing the operating license. The design phase PSA should cover all groups of initiating events for power operation and shutdown states.

In the regulatory review of a PSA by STUK it is essential to find out, whether the probabilistic safety requirements are fulfilled and whether there are any dominant risk factors which could be removed with design modifications. As regards seismic PSA, it is also important evaluate the fragility of structures and components of at accelerations higher than the design basis. To reach the probabilistic requirements with  $10^{-5}$ /year design basis PGA frequency of occurrence, it is essential that the fragility of components is small up to about double the design basis PGA.

### ***Seismic PSAs for the operating units***

#### *Loviisa*

The seismic PSA for the Loviisa VVER 440 units was done in the early 1990's. The work was carried out by the utility using the assistance of the US consulting company Earthquake Engineering Inc. (EQE), especially for component seismic fragility evaluations.

The seismic hazard curves for different confidence levels were determined with the EQRISK program [9]. The information on past earthquakes included 1839 events covering the period 1467-1984. Some results of the seismic hazard calculation are shown in Table 1.

Table 1. Seismic hazard with 50 % confidence level  
at the Loviisa site according to the seismic PSA (1992)

Frequency (1/a)	PGA (g)
$10^{-5}$	0.012
$10^{-4}$	0.025
$10^{-3}$	0.048
$10^{-2}$	0.095

The potentially vulnerable structures and components were identified in plant walk-throughs by a group including plant experts and seismic experts from EQE. Examples of components with high or moderate seismic fragility are the feedwater tank, deionizing tank, demineralised water tank, emergency feedwater lubricant and some DC power systems. Building structures were estimated to be quite resistant to the seismic acceleration considered possible in Finland.

Some of the most severe vulnerabilities were removed immediately. For example, the supports of DC system batteries were not originally designed for any dynamic loads and were immediately strengthened.

The feedwater tank is a long horizontal tank situated on a relatively high elevation and the supports are not designed for seismic loads. Due to the location of the feedwater tank close to the main control room, it is estimated that the failure of the tank would always lead to core damage. The contribution of the feedwater tank failure is about 50 % of the seismic CDF. In addition, 19 other seismic induced initiating events were identified. The initiating even frequencies were calculated with a fault tree using the seismic hazard information and component vulnerabilities. It was assumed that simultaneously with the seismic initiating

events the safety systems for their management are lost and no accident chains were included in the PSA model. The total seismic CDF was estimated as  $3.6 \cdot 10^{-6}$ /year which is about 4 per cent of the total CDF of the Loviisa units.

The current operating licence of the Loviisa units will terminate at the end of 2007, and the application for renewal of the licence is expected in late 2006. During the review of the application STUK will also consider the seismic resistance of the Loviisa units and assess, if the seismic PSA can still be considered sufficient. The methods used in the Loviisa PSA were quite advanced in the early 1990's. Since then some updates have been made in the seismic hazard estimates but so far no significant deficiencies have been identified in the Loviisa seismic PSA.

### *Olkiluoto 1 and 2*

The seismic PSA for the operating units at Olkiluoto was reported in 1997. The work was done in cooperation by the utility TVO and EQE International Inc., which had an important role in all phases of the seismic PSA. The methods complied with the IAEA, EPRI and NRC guidance [10, 11, 12]. The SEISRISK III program [13] was used in Olkiluoto PSA for the determination of the seismic hazard curves.

According to the first estimates, the seismic CDF  $2.5 \cdot 10^{-5}$ /year was dominating the risk profile. The high seismic risk was due to inadequate supports and anchorage of batteries, DC/AC converters and electric cabinets. After immediate improvements, the seismic CDF was reduced to  $4.4 \cdot 10^{-6}$ /year.

The current seismic CDF estimate is  $5 \cdot 10^{-6}$ /year or 30 % of the total CDF  $1.69 \cdot 10^{-5}$ /year (power operation and shutdown states). The most important contributor is relay chatter which may lead to loss of emergency core cooling systems or their inadvertent isolation and inadvertent pressure suppression. Due to the large number of relays, the elimination of risks due to relay chatter would require quite extensive modifications and has not been carried out so far. However, the problem will be removed in connection with the Olkiluoto 1 and 2 control systems will be renewal project which will be started in the near future.

### *Seismic PSA for OL3*

In principle, Guide YVL 2.8 requires that the design phase PSA includes seismic events. However, seismic PSA was not submitted to STUK in connection with the construction licence applications. Instead, the design phase PSA included only a preliminary plan for seismic PSA and qualitative arguments based on the plant supplier's experience that with proper detailed design the probabilistic safety goals can be reached also regarding seismic events. Based on regulatory review of the seismic design and the low seismic hazard in Finland, the approach was approved by STUK.

Later the plant supplier AREVA NP has drawn up a detailed seismic PSA plan. The seismic PSA will use the state of the art methodology as described in Appendix B of ANSI/ANS 58.21 "External-events PRA methodology". The key elements of the seismic PSA, as presented by AREVA, are:

1. Seismic hazard analysis for determining the frequency of occurrence of different peak ground acceleration at the site. The seismic hazard of the site is available from the seismic PSA performed for OL1/OL2 and from the recent updates of the hazard analysis described in [14].
2. Seismic fragility evaluation which estimates the conditional probability of failure of important structures and components whose failure may lead to core damage (or other unacceptable damage).
3. Systems and accident sequence analysis which models the various combinations of structural and equipment failures that could initiate and propagate a seismic core damage sequence.
4. Risk quantification which assembles the result of the seismic hazard, fragility and system analyses to estimate the frequencies of core damage and plant damage states.

Since the proposed new plant shall be originally designed for seismic events and the seismic qualification is required of the safety system components, it is to be expected that the seismic risk analyses will not reveal serious vulnerabilities comparable to those identified in the operating NPP units.

## Conclusions

Seismic risk analyses of the operating Finnish nuclear power plant units have shown that seismic risks may be significant even in a region with low seismic activity if seismic loads have not been considered in the design. The most serious risks have been due to inadequate anchorage or supports of electric and electronic equipment. It has been possible to remove most of these risks by relatively simple hardware modifications. For new power plant units, extensive seismic design analyses, qualification and risk studies will be carried out.

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**Topical Areas 2 and 4.  
Lessons Learnt from Applications of SPRA and Uses of PSA/SMA Results**

Use of Insights from Seismic PSA for NPP Paks  
A. Bareith (VEIKI, Hungary), Invited

***Technical session***

***Chairs R. Budnitz and J.E. Yang (KAERI)***

Lessons learned from seismic PSA - Level 1 and Level 2 at NPP Goesgen  
J-U. Klügel, NPP Goesgen-Daeniken, Switzerland

Insights from PSA Based Seismic Margin Assessment  
of the Advanced CANDU Reactor™  
U. Menon, P. Santamaura, H. Shapiro, T. Ramadan, M. Elgohary, AECL, Canada

Seismic Risk Analysis of a CANDU Containment Building  
for Probability Based Scenario Earthquake  
I.K. Choi, KAERI, Korea

## Use of Insights from Seismic PSA for NPP Paks

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### ***Abstract***

A level 1 seismic PSA was performed in one of the last stages of the seismic safety enhancement programme for the Paks nuclear power plant of Hungary. In addition to quantifying core damage frequency from earthquakes, identification and ranking of plant vulnerabilities was an important objective of the analysis to help risk reduction if seen necessary. The level 1 seismic PSA for NPP Paks covers full power operation. The results from the baseline analysis showed that seismic events were a dominant risk contributor. Therefore the PSA models were used to evaluate the progressive risk improvements that may be achieved from implementation of seven potential seismic upgrades. Most of these upgrades have been implemented in pursuit of safety but further improvement is still seen feasible. This paper presents an overview of the Paks seismic PSA with focus on the use of insights for safety improvement purposes.

### **Introduction**

In the original plant design little – if any – consideration was given to seismic loads for the four VVER 440, type 213 reactors of the Paks nuclear power plant of Hungary. This was attributable to the fact that the seismic hazard of the site had been underestimated, a problem that typically showed up in the design of the VVER 440 reactor series. By the end of the 1980s seismic hazard reviews and analyses indicated much higher seismic hazard than the hazard level assumed in the original design. This finding led to a comprehensive programme on more detailed assessment of seismic safety and on improving seismic resistance of the Paks plant. Initially, the approach adopted in this programme was a combination of seismic margin assessment (SMA) and the use of experience-based methods (SQUG) with the following major steps and achievements:

- Re-evaluation of seismic hazard for the site including geotechnical survey, analysis of soil liquefaction, etc.
- Elaboration of post-earthquake shutdown and heat removal technology with identification of structures, systems and components essential to ensure safe shutdown conditions after a seismic event.
- Installation of seismic instrumentation and elaboration of pre-earthquake preparedness and post-earthquake actions.
- Evaluation of seismic capacity for systems, structures and components included in the safe shutdown and heat removal technology.
- Upgrading measures necessary to ensure the required level of seismic resistance/capacity by taking into account the priorities and the feasibility of corrective actions.

Seismic re-evaluation of the plant has been carried out for the design basis earthquake (SSE or SL-2 level) in accordance with international practice. In the design of the Paks NPP an operational base earthquake (OBE or SL-1 level) was not determined and considered. So the earthquake level for safe continuous operation could be determined on the basis of the experiences from re-qualification of safety and safety related systems.

Level 1 seismic probabilistic safety assessment (PSA) became an integral part of the seismic safety enhancement programme for Paks in its last developmental stages.

## Objectives of Paks seismic PSA

According to the requirements of the nuclear safety authority, PSA for all types of initiating events (internal and external) and plant operational modes is mandatory in Hungary. However, the motivation for the Paks seismic PSA was not born merely from this requirement but there was also an overall interest in evaluating the effectiveness of safety measures taken in the safety enhancement programme. Thus the objectives of the level 1 seismic PSA were defined as follows:

1. Quantification core damage frequency attributable to seismic events.
2. Identification of important contributors to seismic risk (initiating events, accident sequences, component failures, human errors).
3. Identification and ranking of plant vulnerabilities based on qualitative insights and quantitative results of the study.
4. Development of recommendations, if necessary, for safety improvement including feedback to the seismic safety programme.

Objectives number 3 and 4 were most important with respect to the use of PSA insights for the purposes of the already established seismic safety enhancement programme.

## Methodological aspects

Currently the seismic PSA for Paks covers plant operation at full power<sup>1</sup>. Unit 3 out of the four units of Paks was selected as the reference unit for the analysis. In addition to the risk of core damage, containment performance was also evaluated to enable future extension to a level 2 PSA. The fundamental parts of the analysis process were identical to those followed mostly in seismic PSAs [1] including

- Assessment of seismic hazard.
- Analysis of seismic response.
- Development of seismic fragilities for safety related structures systems and components.
- Development of accident sequence and system models for seismic-induced plant transients.
- Computation of core damage frequency.
- Uncertainty and sensitivity analyses...

Subsequent calculations on risk reduction from proposed seismic upgrades appeared an important element of the study following the baseline analysis. Each analysis step was a considerable challenge due to the fact that hardly any detailed seismic PSA has ever been performed for a VVER plant.

### *Assessment of seismic hazard*

Probabilistic seismic hazard was assessed for the site of NPP Paks for very low probability events (return period of  $10^2$ - $10^8$  years) to provide input to

- Determining seismic ground accelerations and frequencies for the PSA.
- Calculation of seismic fragilities for plant structures and components.

Peak ground acceleration (PGA) and uniform hazard response spectrum (UHRS) at bedrock were calculated for three return periods. The non-linear effective stress methodology was then applied in site response analysis to propagate the bedrock spectra to the surface and to assess liquefaction potential. A logic tree formulation was used to incorporate input parameter uncertainties.

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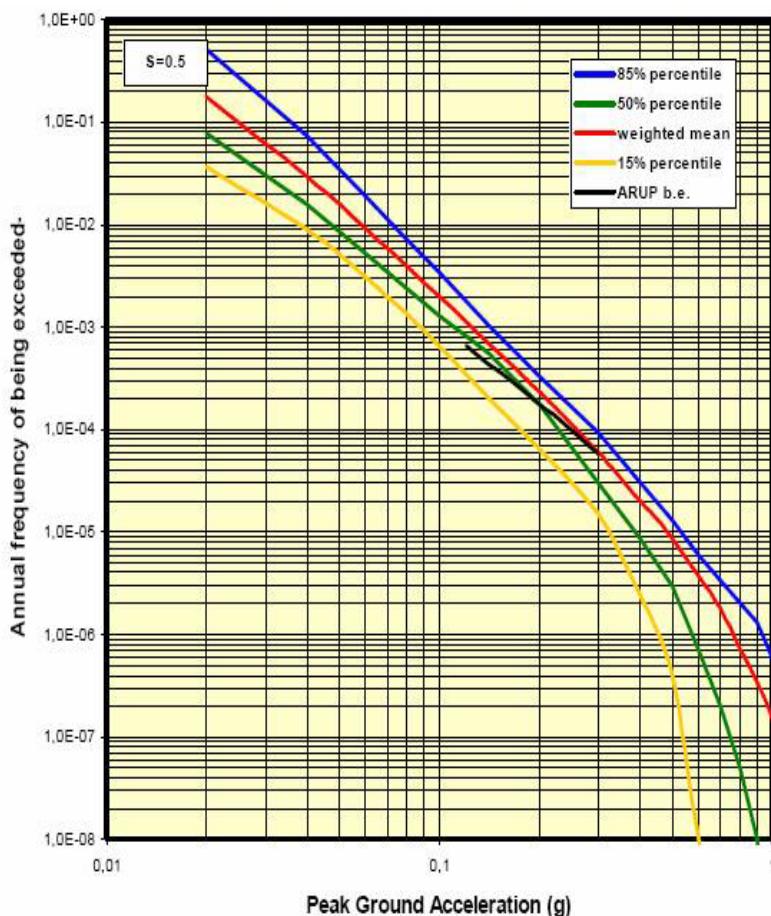
1. Seismic PSA for low power and shutdown conditions is ongoing, as of October 2006.

All available historical data and instrumental earthquake information were considered for the probabilistic seismic hazard assessment. A comprehensive earthquake catalogue with more than 20,000 entries was compiled and homogenised from different regional databases. Three alternative area source models were created with 20, 25 and 15 source zones, respectively, and earthquake recurrence potential was estimated particularly within each of these areas.

During re-evaluation of the site the most discussed question was whether the tectonic structures on the site had been active during the recent tectonic regime. The conclusion was that the probability of recent activity and the existence of a capable fault are very low. In order to prove this finding a micro-seismic monitoring network has been installed at Paks. No micro-seismic event has been registered around the site since the start of its operation. Although neither historical seismicity nor recent local monitoring data supported any fault activity, an additional 10% probability was assigned to possible local fault sources.

The original seismic hazard defined for the upgrading was developed for a 10,000 year return period event. In order to conduct seismic PSA, the hazard had to be re-defined throughout a broad range of return periods; consequently, the hazard study was completely redone by a local institute. The Paks site is soft soil, and subject to liquefaction at acceleration levels exceeding that of the 10,000-year return period. Consequently, defining the hazard at the surface required several non-linear soil response analyses, considering the state of liquefaction, to propagate the seismic motion at bedrock up to the surface. Site response analysis was carried out to propagate the bedrock spectra to the surface taking account the effect of site-specific geotechnical and soil parameters. Although, dedicated analyses were performed on soil liquefaction potential. Fragility development then had to account for this non-linear soil column response.

**Figure 1. Hazard Curves for PGA at Paks as given in [2]**



As shown in Figure 1, PSHA has resulted in PGA and spectral acceleration values for all the assessed return periods for the bedrock outcrop at the site [2]. The results of the study match pretty well with those given earlier in another study [3], namely PGA=0.25 g at around 10,000 years return period which is the defined design basis earthquake for NPP Paks used in the seismic margin assessment.

It was not practical to quantify the PSA models using continuous families of seismic hazard curves (and associated equipment fragility distributions). Instead, seven acceleration ranges were selected to define seismic initiating events. The lower bound is 0.07g, this corresponds to the lowest seismic HCLPF capacity for all structures and equipment at the time of the analysis. The upper bound of 1.0g (PGA) is the highest acceleration evaluated in seismic hazard analysis.

The bounds of the acceleration ranges for the seven initiating events (see Table 1) were selected to ensure that the seismic hazard curves remained approximately linear throughout a range. The intervals are progressively larger to account for the fact that the frequencies change more slowly at higher accelerations. In addition to mean exceedance frequencies, use was made of the whole family of hazard curves from seismic hazard assessment.

**Table 1. Seismic initiating event acceleration ranges**

<b>Initiating Event</b>	<b>Acceleration Range (g), PGA</b>	<b>Mean Frequency (Event/Year)</b>
SEIS1	0.07 - 0.10	$2.69 \cdot 10^{-3}$
SEIS2	0.10 - 0.15	$1.08 \cdot 10^{-3}$
SEIS3	0.15 - 0.22	$3.16 \cdot 10^{-4}$
SEIS4	0.22 - 0.32	$8.71 \cdot 10^{-5}$
SEIS5	0.32 - 0.48	$2.35 \cdot 10^{-5}$
SEIS6	0.48 - 0.70	$4.76 \cdot 10^{-6}$
SEIS7	0.70 - 1.00	$8.99 \cdot 10^{-7}$

#### ***Assessment of seismic response and seismic fragilities***

The analysis of seismic response based on the results of finite element evaluation of structures and floor response spectra were already available from SMA for practically all levels of interest within the buildings of safe shutdown components. Comprehensive walk-downs were conducted to examine all structures and plant locations that contain mechanical equipment, piping, switchgear, electrical equipment, instrumentation and control cabinets, and cables that may affect any systems or functions that are analysed in the PSA. Systematic screening criteria were applied during the walk-downs to determine whether the seismic capacity of an examined item is sufficiently high to justify no need for additional evaluation. As a result of an initial evaluation process, many structures and components were assigned to three screening groups, based on their assessed capacity [4]:

- Structures, mechanical equipment, electrical equipment, relays and cabinets that satisfy a predefined higher screening capacity criteria (HCLPF=0.53g PGA, median capacity:  $A_m=1.34g$ )
- Mechanical equipment that satisfy a lower screening capacity criteria (HCLPF=0.35g PGA,  $A_m=0.89g$ )
- Tested relays (contact devices) and cabinets that satisfy the relay screening capacity criteria (HCLPF=0.27g PGA,  $A_m=0.73g$ ).

The three screening groups are included in the PSA models to ensure completeness of the seismic risk quantification process. The failure fractions for each group were determined by their respective screening fragility distributions. The functional impacts from each group were determined by the most limiting combination of failures, based on the specific structures and equipment that are included in the group. In this manner, the PSA quantification process accounted for seismic failures of all relevant structures and equipment in the plant, and no items were omitted from the analysis process.

More detailed analyses were performed to determine the capacities of specific structures, mechanical equipment, electrical equipment, cabinets and relays that did not satisfy any of the initial screening criteria. The fragility distributions determine the likelihood of failure for each structure or component in the PSA models, as a function of the seismic acceleration. In some cases, similar types of equipment were combined in a single group for quantifying the PSA models because they were found to have similar seismic capacities and were expected to fail at approximately the same acceleration.

In addition to the analyses of structural and equipment capacities, site-specific analyses were performed to evaluate the likelihood of various degrees of soil liquefaction that may occur and the potential damage to structures, equipment, buried piping, cables, etc.

Over and above the one higher screen and the two lower screen groups, the following separate seismic failure groups were defined – with the associated fragilities – for modelling and quantification in PSA:

- 27 groups of mechanical equipment, grouping based on equipment type and/or location.
- 9 groups of electrical and I&C cabinets, grouping based on cabinet location.
- 20 groups of electrical and I&C relays (contact devices), grouping based on relay type.
- 11 structural failures.
- 2 degrees of liquefaction, based on the differences in consequences.

Fragilities were developed using the standard separation of variables approach and they were mostly based on existing deterministic analyses conducted during the upgrading. The focus was on consequences of liquefaction, non-ductile failure modes of steel structures and spatial systems interactions identified during the walk-down.

### ***Development of PSA model***

A baseline seismic PSA model was developed in order to construct an event tree structure that integrates event trees developed earlier within the internal initiator PSA study and specific earthquake-induced transients into a generic model that reflects the specifics of an earthquake for the reference state of the plant [5]. Extensive use was made of the existing level 1 PSA models for internal initiators throughout model development to ensure consistency in terms of modelling approaches, depth and level of detail. The baseline accident sequence models were developed in the following major steps:

- Identification of transient initiating failures and additional system, train or component level failures and degradations that can be caused by any combination of equipment failures selected for a group, establishment of a list of transient initiating failures that can be caused by an earthquake.
- Development of functional event trees for single transient initiating failures.
- Development of a generic event tree for modelling plant responses to an earthquake with combinations of single and multiple transient initiating failures.

When the *transient initiating failures* and the mitigating system/component failures from earthquakes were identified equipment level failures within a fragility group were assumed fully correlated. These failures were determined by an evaluation of failure consequences for each component within a group. The impact of block wall collapse on electrical cables was taken into consideration in this process. Also, failures that can be caused by the simultaneous occurrence of different group failures were identified too.

A comprehensive and detailed analysis of electrical circuit diagrams was performed to determine the consequence of I&C failures including seismic induced chattering of contact devices. The failures identified can be recoverable or unrecoverable. In general, relay failures may be recoverable. The likelihood of recovery depends on the type of relay, its failure mode, the possibility of remote or local actions to operate the affected equipment, and the overall plant conditions after the failure occurs. Recovery from cabinet failures is

generally much more difficult, because failure of the cabinet anchorage will damage the internal relays and the cabinet cable connections. In these cases, recovery of specific components almost always requires local operation of the affected equipment. Mechanical failures and direct failures caused by structural failure or liquefaction were considered unrecoverable. It is noted however that even if all the failures in a group are identified recoverable their simultaneous occurrence and the earthquake itself may result in such a mental and/or physical load to the operators that they may become practically unrecoverable. This aspect of recovery was dealt with in detail during model development.

A list of transient initiating failures that can potentially occur due to an earthquake was established. Most of the transient initiating failures had their equivalents among the internal event PSA initiating events, but there were some that had not been modelled in the internal event PSA. The latter included transient initiating failures like inadvertent closure of all main gate valves, inadvertent closure of all steam generator isolation valves and others. These were not modelled in the internal event PSA due to the low likelihood of occurrence, but their likelihood has become notable due to some earthquake specific effects like e.g. relay chatter.

In order to develop *functional event trees for single transient initiating failures* the seismic induced transients were examined to determine whether plant responses were designed to be the same for random and for seismic-induced events or not. The major finding of the analysis is that plant responses and the mitigation process appear virtually the same for random initiating events and for seismic-induced transients except for the loss of all 6 kV busbars event. New event trees were developed for this transient and for other earthquake specific events, e.g. inadvertent operation of multiple valves not yet included in the internal events PSA.

A *generic event tree* was developed for the potential seismic-induced plant transients (with combinations of multiple transients) in the final step of accident sequence modelling. Transient initiating failures were modelled as event tree headers to enable explicit modelling of their combined effects. All the core damage event sequences from all the single transient initiating failures were combined as a last, complex header in the generic event tree.

The generic event tree in the baseline study contains 577 event sequences with three major types of event tree headers:

- The first header represents seismic failures in the higher screen groups, an event that would lead to core damage.
- Headers of the second type model seismic induced transients one by one with the associated combinations of the underlying failure events as fault trees
- The third type of header represents an OR connection of core damages sequences from all transient initiating failures in a large fault tree.

Failures caused by the different seismic groups were modelled so that the seismic-induced failure of a component was put into an OR connection with its random failure event. Specific boundary conditions were applied so that the model could be run with or without seismic failures.

### ***Computation***

In view of the information about seismic hazard and component fragilities, failure fractions for the seismic induced failures were calculated by convoluting the seismic hazard and fragility curves. In addition to calculating the failure fraction point estimates as mean values, uncertainties in failure fractions were calculated using Monte Carlo simulation. Each failure fraction represents the mean conditional likelihood for a given seismic induced failure at the designated seismic acceleration [6].

The seismic PSA model was developed using the Risk Spectrum PSA software [7]. However, use of another, separate computational tool was necessary to determine seismic failure fractions. Mean failure

fractions obtained by this tool as well as the frequencies of the seismic initiating events were put into the Risk Spectrum based seismic PSA model, and point estimate core damage frequencies were calculated for each range separately. In many cases, for high acceleration ranges in particular, conditional core damage probability was calculated (given the seismic initiator) instead of core damage frequency because frequency calculations yielded incorrect results for high failure probabilities that frequently appear in the high acceleration ranges. In those ranges the conditional core damage probability is close to unity. The final results for core damage frequency were derived by multiplying conditional core damage probabilities with the frequency of seismic acceleration for a given range.

Uncertainty calculations were done using the same convolution code as the one applied for determining seismic failure fractions. The minimal cutsets generated by Risk Spectrum were used for this purpose.

### **Baseline results**

The core damage frequency estimates from the baseline seismic PSA for NPP Paks represents the plant state at the time of the analysis (year 2002). It includes the effect of implemented upgrades but it also gives credit to improvements that had detailed design documentation and implementation plan in place. Table 2 summarises the mean CDF estimates for the different acceleration ranges from the baseline seismic PSA.

As to the major contributors to seismic risk, structural failures of bolted connections in the turbine hall were found dominant. These failures are assumed to cause building collapse and subsequently steam and feedwater header breaks with total loss of feedwater so that the so-called closed loop secondary heat removal cannot be ensured. Although an emergency feedwater system can provide cooling but in an open loop only that is not capable of long term cooling, i.e. cooling after depletion of water resources. Primary feed-and-bleed through the emergency core cooling system with cooling from service water would be the only means of heat removal. However, the collapse of turbine building can endanger pipelines in the service water system too.

Structural failures in the main building complex (reactor hall and electrical galleries) and failures of non-tested I&C devices and cabinets also show up as important risk factors in the baseline analysis.

**Table 2. Point estimate results from baseline seismic PSA for NPP Paks**

ID	Acceleration Range		Core Damage Frequency, 1/year
	from (g), PGA	to (g), PGA	
SEIS1	0.07	0.10	$4.22 \cdot 10^{-6}$
SEIS2	0.10	0.15	$7.18 \cdot 10^{-5}$
SEIS3	0.15	0.22	$1.17 \cdot 10^{-4}$
SEIS4	0.22	0.32	$7.26 \cdot 10^{-5}$
SEIS5	0.32	0.48	$2.34 \cdot 10^{-5}$
SEIS6	0.48	0.70	$4.76 \cdot 10^{-6}$
SEIS7	0.70	1.00	$8.99 \cdot 10^{-7}$

### **Evaluation of potential upgrades**

Since the core damage frequency from earthquakes were found significantly higher than that from other initiating events, it was seen necessary and useful to examine whether feasible and practicable measures could be proposed for risk reduction. Therefore the PSA models were used to evaluate the progressive risk improvements that may be achieved from implementation of seven potential seismic upgrades.

The upgrade cases were modelled in the seismic PSA at fault tree level, i.e. the fault trees and the associated input data were modified to represent the effects of an upgrade or a combination of upgrades on

vulnerability of structures and components to seismic events. Following is a brief discussion on the technical contents of the upgrades and the associated benefits in terms of improved CDF figures.

- Upgrade 1: turbine building bolted connections and reactor hall / longitudinal electrical gallery bolted connections

Upgrade 1 increases the structural capacity of the turbine building bolted connections and the reactor hall / longitudinal electrical gallery bolted connections. The revised PSA is based on the assumption that the seismic capacities of these connections are increased to the extent that failures of these structures are now governed by buckling of the vertical frames. The effects from this upgrade reduce the total core damage frequency by a factor of approximately 3, compared with the baseline results.

- Upgrade 2: masonry block walls

Upgrade 2 increases the structural capacity of all interior masonry block walls that are located near PSA equipment and cables. Correlated failures of these walls are modelled in the baseline PSA by a separate structural failure event. The revised PSA is based on the assumption that the seismic capacities of all walls in this group are increased to at least the lower screening capacity for other plant structures and mechanical equipment (i.e., HCLPF = 0.35g, median = 0.89g). The effect from this upgrade reduces the total core damage frequency by approximately 4 %, compared with the baseline results.

- Upgrade 3: untested relays and cabinets

Upgrade 3 increases the capacities for all untested relays and cabinets that affect any equipment in the PSA models. Correlated failures of these relays and cabinets are modelled in the baseline PSA by a single element that includes all groups of specific relays and cabinets that are described in [5]. The revised PSA is based on the assumption that the seismic capacities of all kinds of cabinet anchorage are increased to at least the lower screening capacity for other plant structures and mechanical equipment (i.e., HCLPF = 0.35g, median = 0.89g). The revised analysis is also based on the assumption that all relays are either tested to demonstrate that their capacities meet or exceed the minimum screening capacity for all other tested relays (i.e., HCLPF = 0.27g, median = 0.73g), or the relays are replaced with qualified components that exceed this capacity. The combined effects from upgrades 2 and 3 reduce the total core damage frequency by approximately 9 %, compared with the baseline results.

- Upgrade 4: diesel generator building 14cm block walls

Upgrade 4 increases the structural capacity of the diesel generator building 14cm interior block walls. The revised PSA is based on the assumption that the seismic capacity of these walls is increased to at least the lower screening capacity for other plant structures and mechanical equipment (i.e., HCLPF = 0.35g, median = 0.89g). The combined effects from upgrades 1, 2, 3 and 4 reduce the total core damage frequency by a factor of 5.6, compared with the baseline results.

- Upgrade 5: turbine building structural bracing

Upgrade 5 increases the structural capacity of the turbine building vertical braced frame. The revised PSA is based on the assumption that the combined seismic capacities of the turbine building frame and the bolted connections are increased to at least the lower screening capacity for other plant structures and mechanical equipment (i.e., HCLPF = 0.35g, median = 0.89g). The combined effects from upgrades 1, 2, 3, 4 and 5 reduce the total core damage frequency by a factor of approximately 8, compared with the baseline results.

- Upgrade 6: diesel generator building 30cm block walls

Upgrade 6 increases the structural capacity of the diesel generator building 30cm block walls. The revised PSA is based on the assumption that the seismic capacity of these walls is increased to at least the lower screening capacity for other plant structures and mechanical equipment (i.e., HCLPF = 0.35g, median = 0.89g). The effect of this upgrade is evaluated in combination with upgrades 1, 2, 3, 4, 5 and 7 below.

- Upgrade 7: air compressor building

Upgrade 7 increases the structural capacity of the air compressor building. The revised PSA is based on the assumption that the seismic capacity of this building is increased to at least the lower screening capacity for other plant structures and mechanical equipment (i.e., HCLPF = 0.35g, median = 0.89g). The combined effects from upgrades 1, 2, 3, 4, 5, 6 and 7 reduce the total core damage frequency by a factor of 10, compared with the baseline results.

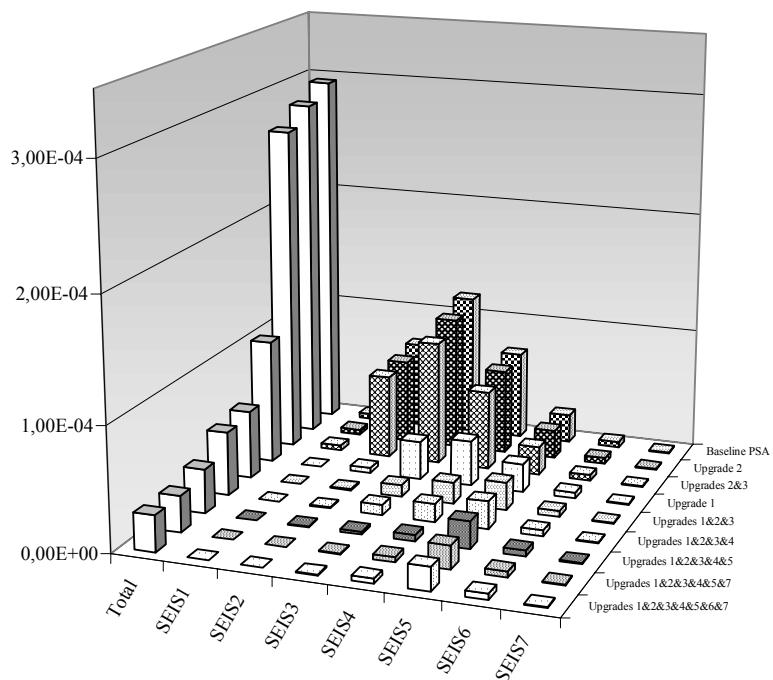
Table 3 and Figure 2 provide an overview of the core damage frequency from the baseline PSA results and from each upgrade analysis. The table also shows the contributions from all failures that occur in each of the seven seismic acceleration ranges. This summary is useful to compare the total risk reduction that is achieved from each upgrade, and to examine how the most important risk contributors shift from lower accelerations to higher accelerations as progressive upgrades are implemented.

If all the upgrades are implemented, then the most important risk contributors are determined by failures of offsite power at high accelerations that also disable onsite operator performance and may prevent effective response from offsite personnel, and extensive soil liquefaction that damages the underground service water piping. The results from this analysis show that the residual seismic risk after combined implementation of all the upgrades is strongly dominated by extensive soil liquefaction and by seismic failures that occur at accelerations well beyond the 0.25g design basis earthquake. Also, approximately 4% of the revised core damage frequency is caused by failures that occur below the 0.25g design basis earthquake.

**Table 3. Summary of CDF estimates from upgrade analyses**

Seismic Initiating Event	Acc. Range, g	Baseline PSA	Upgrade 1	Upgrade 2	Upgrades 2&3	Upgrades 1 to 3	Upgrades 1 to 4	Upgrades 1 to 5	Upgrades 1 to 5 & 7	Upgrades 1 to 7
SEIS1	0.07 - 0.10	$4.22 \cdot 10^{-6}$	$2.13 \cdot 10^{-10}$	$4.22 \cdot 10^{-6}$	$4.22 \cdot 10^{-6}$	$2.13 \cdot 10^{-10}$				
SEIS2	0.10 - 0.15	$7.18 \cdot 10^{-5}$	$4.08 \cdot 10^{-6}$	$6.93 \cdot 10^{-5}$	$6.88 \cdot 10^{-5}$	$9.05 \cdot 10^{-7}$	$8.26 \cdot 10^{-7}$	$8.26 \cdot 10^{-7}$	$1.91 \cdot 10^{-7}$	$1.91 \cdot 10^{-7}$
SEIS3	0.15 - 0.22	$1.17 \cdot 10^{-4}$	$3.16 \cdot 10^{-5}$	$1.10 \cdot 10^{-4}$	$1.02 \cdot 10^{-4}$	$9.61 \cdot 10^{-6}$	$8.52 \cdot 10^{-6}$	$2.28 \cdot 10^{-6}$	$8.67 \cdot 10^{-7}$	$8.67 \cdot 10^{-7}$
SEIS4	0.22 - 0.32	$7.26 \cdot 10^{-5}$	$3.71 \cdot 10^{-5}$	$6.99 \cdot 10^{-5}$	$6.49 \cdot 10^{-5}$	$1.72 \cdot 10^{-5}$	$1.47 \cdot 10^{-5}$	$5.29 \cdot 10^{-6}$	$4.04 \cdot 10^{-6}$	$3.91 \cdot 10^{-6}$
SEIS5	0.32 - 0.48	$2.34 \cdot 10^{-5}$	$2.31 \cdot 10^{-5}$	$2.34 \cdot 10^{-5}$	$2.34 \cdot 10^{-5}$	$2.25 \cdot 10^{-5}$	$2.25 \cdot 10^{-5}$	$2.21 \cdot 10^{-5}$	$1.89 \cdot 10^{-5}$	$1.89 \cdot 10^{-5}$
SEIS6	0.48 - 0.70	$4.76 \cdot 10^{-6}$	$4.76 \cdot 10^{-6}$	$4.76 \cdot 10^{-6}$	$4.76 \cdot 10^{-6}$	$4.76 \cdot 10^{-6}$	$4.76 \cdot 10^{-6}$	$4.75 \cdot 10^{-6}$	$4.75 \cdot 10^{-6}$	$4.74 \cdot 10^{-6}$
SEIS7	0.70 - 1.00	$8.99 \cdot 10^{-7}$	$8.99 \cdot 10^{-7}$	$8.99 \cdot 10^{-7}$	$8.99 \cdot 10^{-7}$	$8.99 \cdot 10^{-7}$	$8.99 \cdot 10^{-7}$	$8.99 \cdot 10^{-7}$	$8.99 \cdot 10^{-7}$	$8.99 \cdot 10^{-7}$

**Figure 2. Expected CDF reduction from upgrades**



## Current status

The results of the Paks seismic PSA and the conceptualised upgrading measure, in particular were given to serious consideration by the utility. By now all practicable structural improvements can either be regarded as completed or are in the implementation phase. The upgrade concerned with untested relays is an open issue that is still under discussion. A more detailed elaboration of the problem is expected by comparing the list of untested devices with that of the qualified ones to hopefully reduce the scope of relays that need to be looked at. Also, the seismic PSA model will be used to identify the most important elements within the relatively long list of untested relays. Based on the implemented upgrades and also on the fact that the PSA model has undergone an update in a living PSA procedure, the current results of the seismic PSA for Paks can be summarised as given in Table 4. Further risk reduction may be expected by resolution of the issue on untested relays and cabinets.

**Table 4. Results of seismic PSA following improvements**

Acceleration Range			Core Damage Frequency, 1/year
ID	from (g), PGA	to (g), PGA	
SEIS1	0.07	0.10	$2.58 \cdot 10^{-8}$
SEIS2	0.10	0.15	$1.54 \cdot 10^{-6}$
SEIS3	0.15	0.22	$1.50 \cdot 10^{-5}$
SEIS4	0.22	0.32	$2.15 \cdot 10^{-5}$
SEIS5	0.32	0.48	$2.27 \cdot 10^{-5}$
SEIS6	0.48	0.70	$4.76 \cdot 10^{-6}$
SEIS7	0.70	1.00	$8.99 \cdot 10^{-7}$

## Conclusions

PSA was used as a tool to quantify the effectiveness of measures taken in the seismic safety enhancement programme for the Paks NPP to ensure adequate defence of the plant against a strong seismic motion. Although the level 1 PSA results from the baseline analysis showed a high contribution of earthquakes to core damage risk, the PSA model and the associated analyses proved to be useful in identifying plant vulnerabilities to seismic events and in developing recommendations for improvements. Most of these improvements have already been made in consideration to the associated risk reduction.

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## Lessons learned from seismic PSA – Level 1 and Level 2 at NPP Goesgen

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### *Abstract*

The elaboration of the first PSA for NPP Goesgen was performed from 1990 to 1994. From the beginning the PSA for Goesgen was a full scope level 1/ level 2 PSA including not only internal but also external events and hazards as well as shutdown configurations (level 1+). Seismic PSA was included from the beginning of PSA work at Goesgen. The original seismic PSA of 1994 was based on the state of the art at that time mainly influenced by the methodology developed in the USA for seismic PRA due to the selected contractor PLG (now ABS Risk Consulting). In 2001/2003 and in 2005/2006 updates of the seismic PRA have been performed to accommodate new regulatory requirements and changing boundary conditions with respect to the evaluation of the seismic hazard. All updates included a full scale re-evaluation of fragilities. The seismic PSA has been implemented as a supporting tool for the evaluation of the plant's specific seismic upgrade program. The update of 2005/2006 involved the preliminary incorporation of the new seismic hazard derived from the PEGASOS project (a full scale level 4 SSHAC-analysis, SSHAC, 1997) in the sense of a "What –if–else" analysis. The uncertainty analysis performed as a part of all PSA-studies resulted in very flat distributions for the seismic core damage frequency and large early release frequency. The wide span character of these distributions do not allow to perform meaningful (in the sense of mathematical) decision-making. The huge uncertainties associated with the input regarding the PSA-studies – the results of the probabilistic seismic hazard analysis – have been identified as the root cause of this observation. These uncertainties are partially attributable to a mathematically incorrect formulation of the hazard integral used in PSHA (SSHAC, 1997) leading to an increasing mathematical error with decreasing probabilities of exceedance of ground motion.

### **Introduction**

NPP Goesgen completed its first PSA (Level 1 and 2, shutdown, external and area events) in 1994. The seismic PRA was performed based on the state of the art at that time mainly influenced by the methodology developed in the USA for seismic PRA. The main contractor for this study was PLG (now ABS Risk Consulting). Based on the results of a review by the regulator completed in 1999 a full scale update of the seismic PSA-study was completed in 2001 involving a full reevaluation of fragilities of structures and components. This study was updated in 2003 to incorporate new component reliability data. The seismic PSA has been implemented as a supporting tool for the evaluation of the plant's specific seismic upgrade program. For this purpose detailed sensitivity and uncertainty analyses have been performed. The upgrades performed included:

- Upgrade of 58 masonry walls in the electrical building.
- Upgrade of the main control room light panel ceiling.
- Upgrade of the emergency feedwater building.
- Replacement of control room boards (in the bunkered system as well as in the main control room) by boards with improved seismic qualification.
- Improvement of the anchorage of electric control and switchgear cabinets.
- Re-qualification of safety class electrical equipment (ongoing).

In 1998/1999 extended discussions mainly driven by American consultants raised some concerns about the evaluation of the site specific seismic hazard used in the earlier Goesgen PSA study. NPP Goesgen itself raised some concerns about the large uncertainties associated with the results of seismic PSA. Therefore, a research project for the development of a new site specific seismic hazard following the SSHAC-procedures (SSHAC, 1997) at its most elaborated level 4 was supported by NPP Goesgen. Goesgen expected a decrease in the uncertainties associated with the seismic hazard. In 2004 preliminary results of the PEGASOS-project were available showing even more uncertainties than previous studies. Based on a request by the regulator, these results were implemented as a part of a sensitivity analysis in the sense of a “what if –else” analysis. Methodology, results and lessons learned from the 15 years of experience of seismic PSA at Goesgen are described below.

### **Overview on the methodology of seismic PSA for NPP Goesgen**

The methodology used for the development of the seismic PSA for Goesgen included the following steps:

- Assessment of the site specific seismic hazard and development of the annual frequency of seismic initiating events.
- Assessment of the plant response during an earthquake involving.
  - The selection of safety important buildings, structures and components.
  - The evaluation of the load capacity of the selected buildings, structures and components (fragility analysis).
  - The evaluation of the consequences of the failure of buildings, structures and components including direct impacts (primary faults) and dependent failures caused by the primary faults (secondary faults).
  - The development of a plant logic model representing the possible failure consequences of an earthquake (PRA model).
- Quantification of the seismic initiating events with the help of the developed model.
- Sensitivity and uncertainty analysis.

The general methodology did not change during the periodic updates, but the degree of detail considered as well as the degree of sophistication of the calculation methods used changed greatly.

### ***Site specific seismic hazard***

The Goesgen Seismic PSA is still based on the traditional assumption (originated in the U.S.A) that uniform hazard spectra and the hazard curves derived as a result of a probabilistic seismic hazard analysis (PSHA) can be used directly as an input for a seismic PSA. This is a tribute to the current regulation practice although it is known that this assumption leads to a deviation from risk analysis approaches usually applied in PSA (Klügel, 2005). The original seismic PSA completed in 1994 was based on a PSHA-study performed by Basler & Hofmann based on the probabilistic seismic hazard maps of the Swiss Seismological Service available at that time with some site-specific adjustments. From the perspective of SSHAC-procedures, the PSHA performed corresponded to a SSHAC level 1 approach. Characteristic features of the study were:

- Application of the logic tree method to consider epistemic uncertainties (2 different source models (zonation), 2 different magnitude-frequency relations, 3 different values for upper bound magnitudes and 3 different attenuation equations were considered).
- Use of a specific set of attenuation equations (from literature), partially intensity-based and converted to peak ground acceleration.
- Use of a parabolic magnitude-frequency recurrence model adapted from a Gutenberg-Richter-Model to describe intensity recurrence transformed to a corresponding magnitude-frequency relationship.

For the quantification of the seismic hazard functions the BHSEIS-code, an adaption of an earlier code of McGuire,(1977) was used.

The randomized variability of the attenuation equations (called by people “aleatory” variability) was restricted within the limits of the interval  $(\mu - \sigma, \mu + \sigma)$ . Today it is known that this interval corresponds exactly to the limits of application of the approximative calculation of the probability of exceedance of a specified hazard level implemented into the McGuire-codes (replacement of the calculation of a multivariate probability of exceedance by a univariate approximation, Klügel 2006). In the late 90ies it was still believed that the considered interval of random variability does not fully reflect the total amount of “aleatory” variability to be considered in a PSHA.

After the completion of the first seismic PSA at Goesgen some problems associated with the developed hazard curves occurred. The most important issues were:

- Lack of clear definition of the hazard at the foundation level of the safety –relevant buildings (used as the reference level for the fragility analysis);
- Numerical representation of a total of 108 hazard curves in the RISK-software (RISKMAN $\circledcirc$ ) – at that time limited to 9 hazard curves;
- Need of hazard extrapolation into the range of very low probabilities of exceedance (seismic risk was found to be driven by this extrapolated range) and in the range of low peak ground acceleration (due to the vulnerability of some structures (masonry walls) affecting the performance of risk-relevant components);
- Concerns about the large uncertainties associated with the seismic hazard analysis results (the calculated mean intersected with the 85%-quantile);

For the update of the seismic PSA in 2001/2003 the hazard curves were adjusted to the reference ground level and to reduce the numerical error introduced by the reduction of 108 hazard curves to a total of 9 in the RISKMAN $\circledcirc$  software.

Figure 1 shows a comparison of the different representation of the hazard curves. The following abbreviations are used – NDS – revised model, mean; DPD – original hazard of the 1994 study, mean; Median – median of the revised hazard, 85%-SC – 85% quantile of the original hazard).

Due to the large uncertainties associated with the seismic hazard which showed some significant effect on the PSA-results (see section 3) NPP Goesgen decided to support a large-scale PSHA-research project, the PEGASOS-study, a full-scope SSHAC level 4 study (Klügel, 2005b) for all sites of Swiss nuclear power plants. The goal of this research project from the Goesgen perspective was to incorporate the best available knowledge into the evaluation of the seismic hazard to reduce uncertainties and to provide more reliable bounds for maximum accelerations to be expected at the Goesgen site at low probabilities of exceedance. This research project was requested by the regulator based on extended consultancy with US NRC and US-based consultants with technical background related to the realization of similar projects in the U.S.A.

In 2004 the first and preliminary results of the PEGASOS-project became available which subsequently have been implemented into the seismic PRA model of Goesgen as an intermediate hazard (for PSA-applications) and as part of a detailed sensitivity analysis (based on the UHS).

Figure 1. Comparison of seismic hazard curves (1994 – 2001)

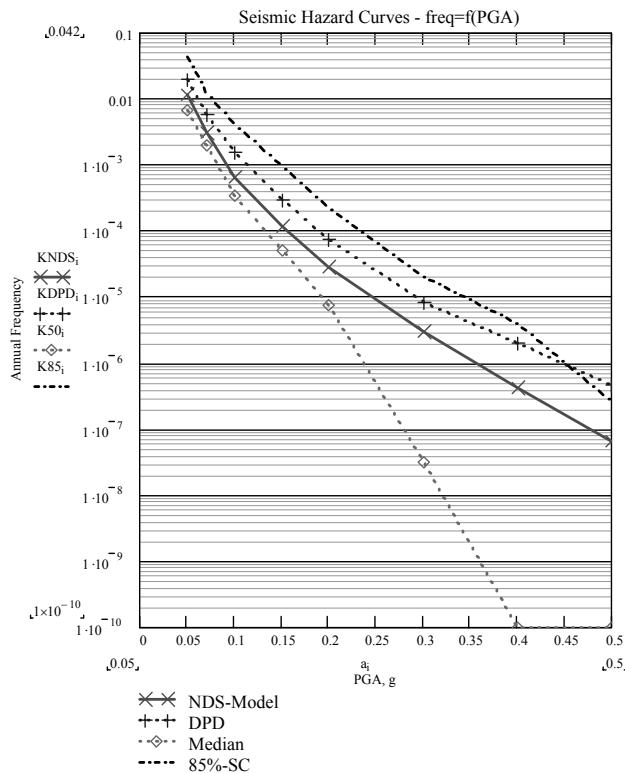
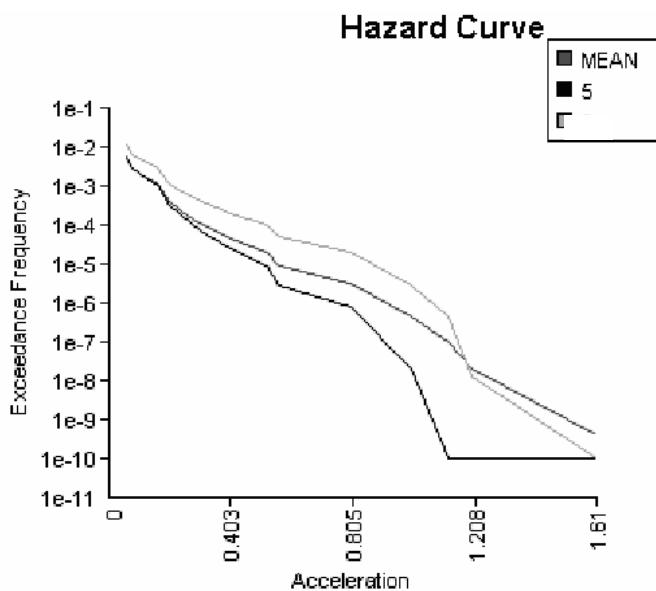


Figure 2 shows the intermediate hazard curves in use at NPP Goesgen for PSA applications derived on the basis of the preliminary PEGASOS results (mean, curve 5 – median, curve 8 – 95%-quantile) by using a reduction factor for all quantiles of 20% (justified by the great uncertainties observed). The curves are truncated at a probability of exceedance of  $10^{-10}/a$ .

Figure 2. Intermediate hazard curves derived from the preliminary PEGASOS-results



### **Fragility analysis**

The methodology for fragility analysis also followed in general the methodology used in the U.S.A at the same time (e.g. EPRI, 1991). The list of safety-specific equipment developed during the first seismic PSA has not been changed by much except for the consideration of plant modifications. The scope of the fragility analysis was gradually expanded. While for the first seismic PSA-Study generic fragilities were employed to a very large extent, the amount of fragilities obtained by explicit analysis was largely increased for the update of the seismic PSA in 2001(2003). The contribution of screened out components to core damage frequency (CDF) and to large early release frequency (LERF) was explicitly considered by top events in the seismic event tree binned directly to core damage with or without loss of the confinement function. Many safety-specific components have been lumped together to build a meaningful top event representing the same functional impact on the plant logic model.

For the first seismic PSAs (1994, 2001) the most simple analysis method was selected which was based on a comparison of the best-estimate seismic hazard with the design basis taking into account plant specific material data and safety margins in the design of the corresponding structures and components with respect to critical failure modes (safety factor approach). The analysis of 1994 did not consider probabilistic soil-structure interactions, nor were the latter considered in the original design. The structural models used were developed in the seventies and therefore very simple (e. g. simple “stick-spring”-models). The uncertainties (variability and epistemic uncertainty) were presented in the usual form of a product of two lognormal variables. The insufficient scope of soil-structure analysis of the analyses of 1994 and 2001 was implicitly compensated by the simplicity of the structural models used and by the conservative modelling assumption to fail all equipment of the same type in all of the safety redundancies of the same type (fully coupled failure). Therefore, the overall fragility analysis approach of the first Goesgen PSA-studies was conservative although the method of fragility analysis from a current perspective must be regarded as obsolete. Nevertheless it was found that the fragility of a human operator is the most important fragility, because the guaranteed failure of all operator actions including accident management actions starting from a certain acceleration (or damage) level is the dominating contributor to core damage or large early releases. In our opinion, this aspect is underestimated in the current U.S. “equal risk” design approach for new nuclear power plants, because the failure behavior of human beings is nonlinear and cannot be extrapolated easily from one hazard level (probability of exceedance of a design basis UHS) to another.

The request to implement the preliminary PEGASOS-results to perform a “What-if-else”-analysis (“what if the PEGASOS-results were true”) in an updated PSA –model led to the need to re-perform the fragility analysis completely. This work is near to completion. The contractor for this work was replaced (in the old studies EQE, ABS Risk Consulting, now Rizzo & Associates with S&A Europe). The difference to the previous analyses consists in:

- A full-scope soil structure analysis.
- A full re-evaluation of the load capacity of structures (9 buildings).
- A complete new evaluation of probabilistic floor-response spectra using coupled (including equipment masses) soil-structure analysis models.
- A total of 6 different fragilities cases to address differences in the spectral shape of different scenario – earthquakes (controlling events) in dependencies of distance range and probability of exceedance.
- A full scope seismic walkdown of all safety relevant structures.
- An increased scope of detailed analysis (e.g. decompose the old screening level top events of the seismic PSA model into a larger set of contributing components).
- Re-evaluation of main piping systems (calculation of seismically induced steam generator tube rupture, LOCA, failures of pressure vessels (e.g. steam generators)) (Note that seismically induced ATWS has been considered from the beginning).
- Recalculation of fragilities of upgraded structures and components.

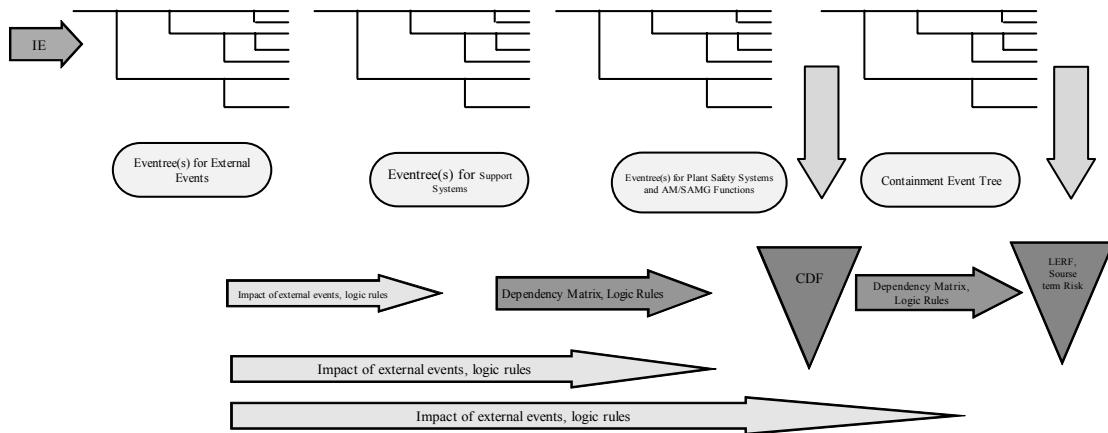
In addition, a special software tool was developed by Rizzo & Associates to allow inter- or extrapolation of fragility analysis with changes in the seismic demand. This is necessary because it is already known that the PEGASOS-results do not represent the correct geotechnical conditions of the Goesgen-site due to a calibration error with respect to the shear wave velocity of the geological bedrock at Goesgen (error of more than 1 000 m/s). Therefore, the whole analysis will be repeated in the near future again.

At present preliminary fragility results (before refined, detailed analysis) for more than 270 calculation units are available. The preliminary results indicate some large underestimation of the seismic capacity of the most important buildings like the reactor building and the bunkered special emergency building in the old fragility analysis. They also show some additional safety margin of electrical equipment. The upgrades (see section 4) were also considered in the analysis. The preliminary results are used in the currently available sensitivity analysis PSA model which provides the basis for further model refinements.

### **Plant response model**

The first step in developing a plant model consisted in the selection of critical buildings, structures and components. The scope of buildings, structures and components which were modelled in the seismic PRA model was largely expanded with each update of the seismic PRA. The same applies for the number of components for which detailed fragility analysis was performed. For the Goesgen PSA the linked event tree approach is used. This approach has proven to be very comfortable for developing external hazards models, because the complete plant logic structure with the dependencies of safety systems on their support systems can be used. On the other hand it requires a larger model concerning the scope of modelled systems in comparison with linked fault trees. All important systems, which in case of a failure could cause a plant transient should be included in the model. Goesgen uses an integrated level 1 and level 2 model. Figure 3 gives an overview of the general structure of the integrated Goesgen PSA-model.

**Figure 3. Structure of the integrated Goesgen PSA model (Level1+Level2)**



The seismic event tree is one of the pre-trees of the PSA-model used to represent the impact of external events on the plant response.

The seismic event tree (SEISET) was modified considerably since the first seismic PSA. The number of seismic top events was increased by more than a factor of 3 since the first study. In the actual version (PEGASOS sensitivity analysis model) it contains 55 top events to quantify 16 seismic initiators (8 seismically induced transients, 8 seismically induced transients with failure of the reactor scram function (ATWS) including induced small LOCAs and direct core damage scenarios representing the screening levels).

For the detailed seismic model addressing the disaggregation of the PEGASOS-hazard into 3 distance ranges, the seismic event tree model will be expanded to allow for the quantification of up to 84 seismic

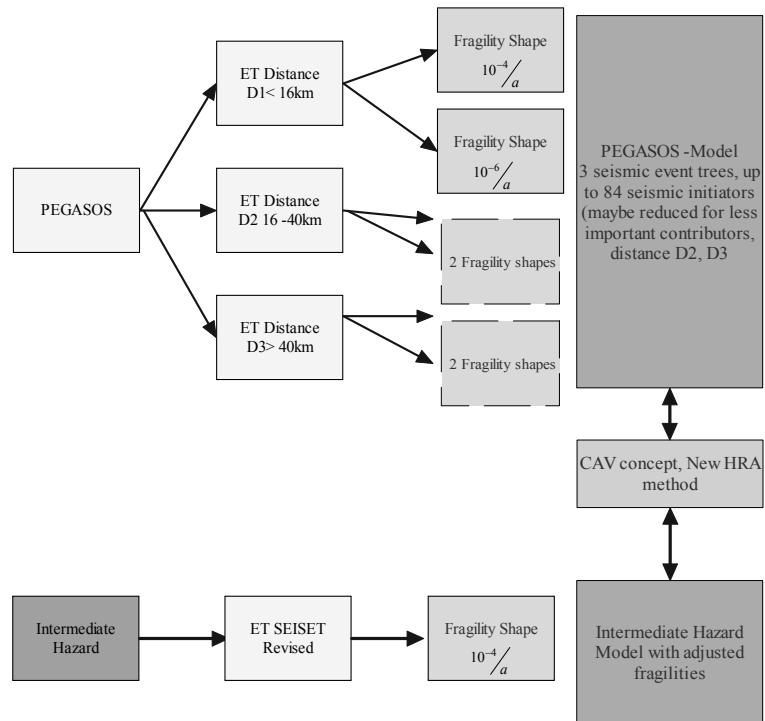
initiators in six different fragility cases. Because the seismic hazard derived from PEGASOS is based on preliminary PSHA results, the new model is structured to enable the use of either the intermediate hazard model (for PSA-applications) or the full PEGASOS-sensitivity model. Figure 4 illustrates the structure of the refined PSA model which is currently before completion. The sensitivity model currently used follows in general the structure of the intermediate hazard model with the difference that the full PEGASOS-hazard is used in the analysis.

The logic rules used in the seismic model (in the linked event trees) must be developed based on an assessment of the consequences of the failure of any given seismic top event in the seismic event tree. The development of these rules is closely linked to the definition of failure and damage states. This question is the most complicated one with regard to the development of a meaningful plant response model. Most of seismic PSAs use conservative approaches for deriving failure and damage criteria.

For the seismic PSAs at Goesgen NPP, structures were considered to have failed when inelastic deformations of the structure under seismic load potentially interfere with the operability of equipment supported by the structure. These limits on inelastic absorption capacity (ductility limits) are estimated to correspond to the onset of significant structural damage, but not necessarily to structure collapse.

This failure criterion is very conservative, especially for passive functions (for example buildings housing safety-specific equipment), because these functions can be maintained even in the case of very high plastic deformations.

**Figure 4. Structure of new seismic PSA model**



Concerning the impact of failed seismic top events it is necessary to differentiate between primary failures – the failure of the seismic top events itself causes a (partial) failure of a safety function (for example for pumps) and secondary failures – dependent failures of other equipment caused by the failure of the seismic top event (for example a loss of coolant accident caused by a failure of piping, or a short circuit in electrical cable caused by the collapse of some structure which even may lead to a fire as a consequence of an earthquake).

For the Goesgen seismic PSAs up to now it was assumed, that a failure of a building will result in the complete failure of all equipment situated in the building (very conservative, because the fragility weren't calculated for a collapse case). For other structures it was assumed that all equipment in the closer surrounding (or below) will fail as a consequence of the failure of the structure (with conditional probability 1). A special case study was performed for dependent failures caused by the failure of masonry walls to assess the conditional probability of secondary failures caused by direct mechanical impacts, induced fires, debris loads and dust effects (in electrical rooms). A statistical representative "best estimate" distribution was developed for this conditional probability.

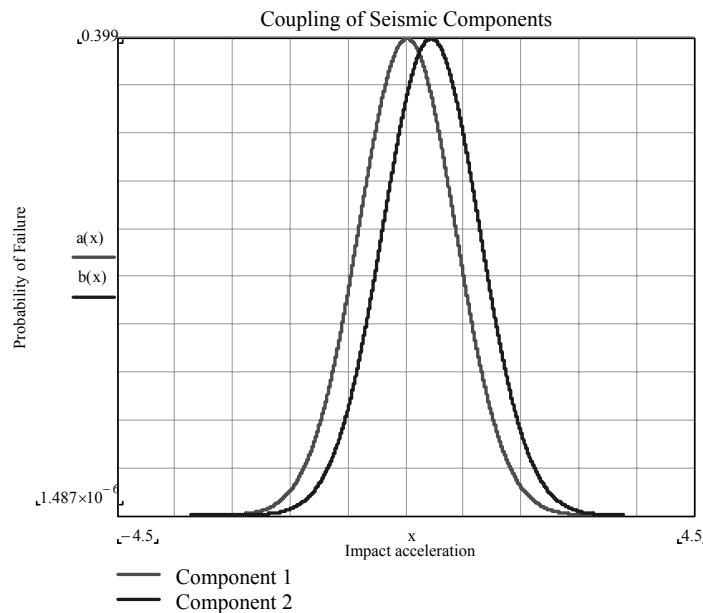
Another problem of deriving a plant logic model consists in the coupled behavior of failure modes of different parts of equipment. Generally an earthquake from the point of view of a PRA analyst can be looked at as a typical common mode event, because all buildings, structures and components are exposed to the impact of the earthquake at the same time. The problem is that with available PRA tools, it is possible to model the coupled failure behaviour of seismic components only in a very simplified way, usually either as completely coupled or as uncoupled. Neither of both is correct. Figure 5 shows an illustration for the case of partially coupled seismic components.

To address this issue special common cause failure models were developed to allow partial coupling between components in different redundancies of the same safety system. Criteria for using a common cause failure model instead of full coupling of components are:

- Distance between components (to consider wave incoherency).
- Orientation of components of the same type.
- "Individuality" of anchorage of components of the same type.

The MGL-approach can be used for the CCF-models. In the Goesgen PSA it is frequently reduced to a simple beta-factor model.

**Figure 5. Coupling of seismic components, illustration**



### Overview on results and sensitivity studies

The results of the Goesgen seismic PSA did vary strongly in the past. Table 1 shows the results of mean core damage frequency and LERF (if available) obtained with different models to demonstrate this observation.

**Table 1. Overview on results of (mean) CDF of different seismic PSA models for Goesgen**

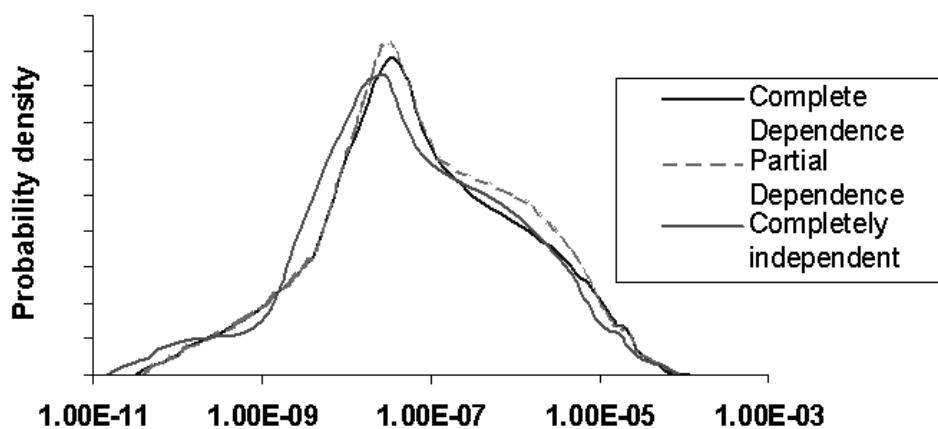
Model Year	1994	2001	2003	2005	2006
CDF, [1/a]	1.8E-7	1.4E-5 <sup>1</sup> 5.5E-6 <sup>2</sup> 1.8E-6 <sup>3</sup>	1.5E-6 <sup>4</sup>	6.3E-6 (intermediate hazard)	2.0E-5 (Full PEGASOS – hazard sensitivity)
LERF, [1/a]	<1E-8	<1E-7	<1E-7	1.32E-8	<1E-7

At present seismic hazard contributes about 90-99% of the total core damage frequency at NPP Goesgen, while it contributes only 4 -18% of the large early release frequency. This means that the risk contribution measured in health effects of the environment is significantly lower than the contribution to core damage. This difference is caused by the large pressure load capacity of the Goesgen containment and the observation that most of the seismic core damage sequences represent rather slow transient.

The large scatter of results can only partially be explained by modelling deficiencies or the seismic plant upgrade program. Methodological improvements of the PSA-model typically led to a reduction of the core damage frequency, while this reduction is annihilated by the numerically increasing hazard (of course we didn't face any significant earthquake in the mean time). While after the study of 1994 some significant vulnerabilities were discovered (weak masonry walls used for fire protection, total 453 walls) which explain the increase in seismic core damage frequency in the study of 2001, the increase between 2001/2003 and 2005/2006 is fully explained by the increase of the seismic hazard which is currently only partially compensated by model improvements (CAV concept, new HRA-methodology, still preliminary fragilities). According to our preliminary estimates the full PEGASOS hazard model (according to figure 4) will lead to another reduction of the CDF by more than a factor 2. It is important to note that the rather small adjustment of the seismic hazard in 2001 (table 1) led to a reduction of the CDF of almost a factor of 3. How can this behaviour of the results be explained then?

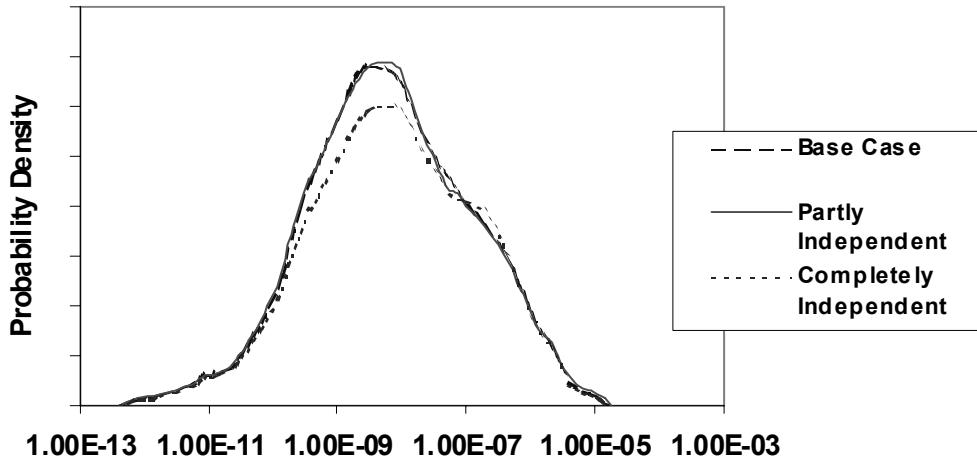
The root cause of the problem is the huge uncertainties associated with the results. Figures 6 and 7 show the uncertainty distribution for core damage frequency or large early release frequency of one of models. So what is driving the uncertainties?

**Figure 6. Uncertainty distribution of core damage frequency (2003)  
sensitivity case CCF of 380V switchgear**



1. Before plant upgrades and without adjustment of the seismic hazard.
2. Adjustment of the seismic hazard (see figure 1).
3. After seismic upgrades of masonry walls and main control room ceiling.
4. New component reliability data.

Figure 7. **Uncertainty distribution of large early release frequency (2003)**  
sensitivity case CCF-model for 380V switchgear cabinets



Simple sensitivity analysis shows that the main contributor to the uncertainty is the seismic hazard. An increase in the mean of the seismic hazard by a factor of 2 increases the core damage frequency by a factor of 5 to 10 (in dependence of whether recovery actions are possible or not).

The true root cause for the uncertainties is the seismic hazard model of Cornell-McGuire which was used for the seismic hazard analysis and which is also recommended in the SSHAC-procedures. This model is derived based on an incorrect calculation of the probability of exceedance of ground motion levels which leads to a systematic error in the hazard calculation (Klügel, 2006). According to the PSHA approach it is requested to calculate the conditional probability of exceedance of ground accelerations (spectral or pga) for fixed magnitude and distance values using an attenuation law as the basis. Equation (1) shows the most simple case - an attenuation law, where spectral acceleration depends only on magnitude and distance:

$$\ln(S_a(m, r)) = g(m, r) + \varepsilon\sigma \quad (1)$$

The spectral acceleration  $S_a(m, r)$  is a random variable, which depends on the function  $g(m, r)$ .  $g(m, r)$  is a function of the random variables magnitude  $m$  and  $r$  and therefore defines a random variable which we denote as  $\Gamma$ .  $\sigma$  is the standard deviation of the attenuation equation (we assume for simplicity reasons that we are dealing with an empirical attenuation equation) and the parameter  $\varepsilon$  defines the confidence intervals of the attenuation equation. In case of a measurement,  $\varepsilon$  has a Student's t-distribution with  $n-1$  degrees of freedom. For large  $n$  it is usually approximated by a normal distribution with a zero mean and the standard deviation  $\sigma$ . Therefore, equation (1) represents an equation for calculating the random parameter  $\ln(S_a(m, r))$  – spectral acceleration – as a function of three random variables (variates) once we include the error term of the attenuation equation in our probabilistic model. Equation (1) is obviously not a conditional probability density function, but rather represents the functional dependency of the random spectral acceleration on the random variates, magnitude, distance and measurement error. We have to apply the laws of multivariate theory of probability to calculate the conditional probability of exceedance of a certain hazard level  $z$  for a given set of parameters  $m$  and  $r$  by developing the joint probability density distribution for the spectral acceleration and relating it to the marginals of  $m$  and  $r$  (assuming independence between  $m$  and  $r$ ). In case of dependency between  $m$  and  $r$  we have to apply the total probability theorem to obtain the required conditional probability density distribution. This leads to the need to trace all dependencies between the parameters of the probabilistic model used (e.g. over all nodes of our logic tree) resulting in a mathematical problem that obtains such a

large scale that it represents an insurmountable obstacle for any practical applications. Therefore, the PSHA model was simplified by assuming that  $g(m, r)$  is constant and all the randomness of the problem is concentrated in the error term  $\varepsilon\sigma$ . The analogy to a measurement can be used assuming that  $g(m, r)$  is the central value of the confidence interval of the result of a measurement. Therefore, the random variable  $\Gamma = g(m, r)$  was effectively replaced by the (hopefully) best estimate of the expected value of  $\Gamma$ , by its mean. This is correct in an approximate way, because of Chebyshev's inequality (equation (2)):

$$\Pr(|X - \mu| \geq \varepsilon\sigma) \leq \frac{1}{\varepsilon^2} \quad (2)$$

Indeed all data observations prove this inequality and indeed the deviations from the mean can be approximated by a normal distribution, leading to the model of a lognormal distribution for spectral attenuations once we convert from the logarithmic scale to the usual number scale. As a result of the simplification we obtain equation (3) for the probabilistic model:

$$\ln(S_a(m, r)) = E(g(m, r)) + \varepsilon\sigma \quad (3)$$

The nice thing about this simplification is that now it is possible to reduce the size of the problem. By multiplying the simplified equation (3) with the probability density function of  $\varepsilon$ , performing integration and converting the resulting expression to the complementary probability distribution function (usually the complementary normal distribution function) analysts were able to separate the randomness from the “quasi-deterministic” calculation of ground motion calculation. This is the model used in SSHAC (SSHAC, 1997)

Unfortunately, this simplifying replacement is completely incorrect from the point of view of mathematics because a random parameter (here  $\Gamma$ ) is replaced by a number, by its expected value. There is a hard price to pay for this simplification – the introduction of a systematic error. We can show this by replacing the distribution  $g(m, r)$  by a series (assuming that the development into a series is possible, which is the case here) around its expected value  $E(g(m, r))$ .

$$g(m, r) = E(g(m, r)) + \Delta(m, r) \quad (4)$$

Here  $\Delta(m, r)$  is a nontrivial (not equal to 0) random variable describing the deviations of  $g(m, r)$  around its expected value. Replacing  $g(m, r)$  in (1) by equation (4) we obtain

$$\ln(S_a(m, r)) = E(g(m, r)) + \Delta(m, r) + \varepsilon\sigma \quad (5)$$

Replacing  $\ln(S_a(m, r))$  in equation (3) by equation (5), we obtain as a result

$$\Delta(m, r) = 0 \quad (6)$$

This is obviously wrong by the way  $\Delta(m, r)$  was defined. So we obtained a statement of the same logical value as “x equals 0 for any x” as the result of the simplifications made in the traditional PSHA model.

## Conclusions

The main conclusion from our sensitivity analysis is that the great uncertainties observed in the results of seismic PSA can be traced down to a crude mathematical error in the calculation of the probability of exceedance of ground accelerations in the Cornell-McGuire (SSHAC) PSHA-model. This observation has the consequence that a mature methodology for a mathematically correct probabilistic evaluation of seismic hazard is not available at present. It will be one of the tasks of the PEGASOS refinement project to develop such a methodology, probably based on a robust Monte-Carlo procedure.

Despite these problems, which are mostly related to risk quantification issues, the experience gained at Goesgen with the use of insights from seismic PSA contains some positive features. It helped to perform a systematic seismic upgrade program eliminating the last potential vulnerabilities in the design of the plant. From a quantitative perspective it can be concluded that the seismic risk of the plant is significantly lower than predicted by the currently available methodology which is mathematically incorrect.

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## Insights from Probabilistic Safety Assessment-Based Seismic Margin Assessment of the Advanced CANDU Reactor<sup>®</sup>

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### **Abstract**

A probabilistic safety assessment-based seismic margin assessment (PB-SMA) is planned for the Advanced CANDU Reactor-1000<sup>®</sup> (ACR-1000<sup>®</sup>)<sup>1</sup> design. The ACR-1000<sup>®</sup> has several systems that are seismically qualified, such as the main control room, emergency feedwater, Class III diesel generators, and essential cooling water. This paper describes the seismically qualified design features of the ACR-1000<sup>®</sup> and summarizes the methodology of the PB-SMA, including the results and insights gained from a Level 1 PSA-based seismic margin assessment of the ACR-700<sup>®</sup>.

The PB-SMA was performed during the early design stages of the ACR-700<sup>®</sup>, and the potential seismic failure modes and component weak links/functionality were considered. Non-seismic failures, human errors, and their conditional probabilities were also considered in the analysis. Estimates of seismic capacities of structures, systems, and equipment have been developed using the ACR-700 seismic design criteria and qualification criteria, the past seismic experience, and the recent seismic PSAs and SMAs. With these estimates and the ACR-specific seismic event trees and fault trees, an overall plant seismic margin measured in terms of the HCLPF (high confidence of low probability of failure) capacity of 0.5g peak ground acceleration is reasonably achievable for the ACR design. Appropriate design provisions and conservatism in seismic capacity evaluation will be made to support a HCLPF capacity of 0.5g in the ACR-1000<sup>®</sup> design. This compares well with the design basis earthquake (DBE) of 0.3g peak ground acceleration for the ACR design.

The results obtained in past ACR-700<sup>®</sup> analyses show a robust design that provides a strong foundation for the ACR-1000<sup>®</sup> design development currently underway.

### **1. Introduction**

International experience has shown that earthquakes may be a significant contributor to plant risk. It has been accepted that seismic events must be routinely included in PSAs. NUREG/CR-2300 [1] provides general procedural guidance for conducting a seismic PSA. NUREG-1407 [2] contains general procedures and submittal guidance for conducting external event analyses, including seismic events. The report [2] was written by the U.S. Nuclear Regulatory Commission (USNRC) for the individual plant examination of external events (IPEEE) for current nuclear power plants in the United States. The report states that two assessment methods are acceptable – either seismic margin or seismic PSA. The NRC method for performing a seismic margin review is documented in NUREG/CR-4334 [3]. Additional guidelines for performing such reviews are presented in NUREG/CR-4482 [4].

More recently the American Nuclear Society has developed a standard for performing seismic PSAs and seismic margin reviews [5], and these high-level requirements are considered in the methodology used for the PB-SMA of the ACR.

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1. ACR-700<sup>®</sup> and ACR-1000<sup>®</sup> (Advanced CANDU Reactor<sup>®</sup>) are registered trademarks of Atomic Energy of Canada Limited (AECL).

The International Atomic Energy Agency (IAEA) has also issued a document that provides an overview of seismic PSA [6]. The Canadian Nuclear Safety Commission's (CNSC) pre-consultation draft, "Requirements for Design of Nuclear Power Plants," stipulates the requirement to include external events in the probabilistic safety assessment. It also states "For external events, the designer may propose an alternative analysis method to conduct the assessment."

Performance targets are being set for the ACR. In particular, seismic performance targets (i.e., HCLPF capacities) are assumed for equipment, some of which have already been verified. Some equipment will be verified during the detailed design stage.

The PSA-design interface involves two-way communication between the PSA group and design groups. To assist in the ACR design process, the PSA group sends fault tree and event tree analyses reports to designers and safety analysts for their formal review and comments. The PSA group identifies and informs the designers of any deficiencies and vulnerabilities in the ACR systems and assists in the evaluation of possible design solutions. The PSA group gives the assumptions for the seismic performance targets to designers. The designers will then verify the seismic capacity during and after the seismic design.

A Level 1 PSA-based seismic margin assessment (PB-SMA) for the ACR-700<sup>®</sup> design has been performed using the USNRC seismic margin assessment methodology as per SECY-93-087 [7]. The potential seismic failure modes and component weak links/functionality are considered. Non-seismic failures, human errors, and their conditional probabilities are also considered in the analysis. The objective of the analysis is to show that the ACR design has a seismic margin measured in terms of the HCLPF (high confidence of low probability of failure) capacity that exceeds 0.50g peak ground acceleration. This ensures there is sufficient margin above the ACR's design basis earthquake (DBE) of 0.3g.

The PB-SMA consists of similar steps to a seismic PSA but does not include the steps for developing seismic hazard and integration of the hazard curves with the rest of the analysis. The PB-SMA method eliminates the large uncertainty of the seismic hazard curve and its impact on the analysis, which is particularly useful for a standard reactor design such as the ACR that could potentially be built at any site in future.

The PSA-based seismic margin assessment was conducted during the design phase of the ACR-700<sup>®</sup>. A PB-SMA is planned for the ACR-1000<sup>®</sup> design.

## 2. ACR design

The ACR design is based on horizontal fuel channels surrounded by heavy water moderator, the same as with all CANDU<sup>®2</sup> reactors. The major innovation in the ACR is the use of low-enriched uranium fuel and light water as coolant. The safety systems provided in the ACR are as follows:

- Two fast-acting, fully capable, diverse, and separate shutdown systems, which are physically and functionally independent of each other and from the reactor regulating system.
- Emergency core cooling system (ECC), consisting of the emergency coolant injection system and the long-term cooling system (LTC).
- Containment system (strong containment structures (steel-lined), containment isolation system, containment heat removal system, etc.).

The safety enhancements made in the ACR-1000<sup>®</sup> encompass the safety margins and the performance and reliability of structures, systems, and components (SSCs) important to safety. Figures 1 and 2 show the ACR-1000<sup>®</sup> reactor building cross-section and the two-unit ACR-1000 plant layout. Further description of the ACR-1000 is provided in AECL's "ACR-1000 Marketing Brochure" [8].

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2. CANDU<sup>®</sup> (CANada Deuterium Uranium) is a registered trademark of Atomic Energy of Canada Limited (AECL).

The ACR-1000 has a seismically qualified main control room, DBE-B-qualified emergency feedwater (EFW) system, reserve water system (RWS), seismically qualified support system such as Class III standby diesel generators, essential cooling water (ECW) system, and essential service water (ESW) system.

### **3. Methodology**

The PB-SMA is based on an event/fault tree approach to delineate the accident sequences. High confidence of low probability of failure (HCLPF) capacities obtained for each end state of the seismic event tree are used instead of accident frequencies. The methodology of PB-SMA used for the ACR-700<sup>®</sup> is applicable to the ACR-1000<sup>®</sup>.

The internal event PSA model developed by AECL for the ACR-700<sup>®</sup> was the starting point for this seismic margin analysis. Seismic event trees are developed by considering all potential sequences leading to severe core damage and limited core damage. For the systems modelled in these event trees, seismic fault trees are constructed based on the system flow sheets and success criteria. Seismic-induced failures of components appearing in these fault trees are collected as the safe shutdown equipment list (SSEL) for seismic margin analysis. The safe shutdown equipment is currently being designed as either seismically qualified or non-seismically qualified. The non-seismically qualified structures and equipment at a minimum meet the requirements of the National Building Code of Canada. This earthquake level is called generic design earthquake (GDE).

The code requirements are also consistent with the 2003 International Building Code and the American Society of Civil Engineers ("Minimum Design Loads for Buildings and Other Structures, ASCE 7-02" [11]).

The seismic event trees and fault trees are solved to derive cutsets that lead to severe core damage (SCD). These cutsets contain both seismic-induced failures, random unavailabilities of equipment, as well as operator error probabilities. The significant cutsets that are either seismic failures only or a combination of seismic failures and random failures (mixed cutsets) are identified. Using the "min-max" approach specified in the NRC seismic margin methodology, the sequence level or plant level seismic margin is calculated for SCD.

#### ***3.1 Fragility analysis***

Fragility analysis characterizes conditional probability of failure of important structures, systems, and components whose failure may lead to unacceptable damage to the plant (e.g., core damage) given occurrence of an external event.

A probabilistic approach is taken to estimate the median seismic capacity ( $A_m$ ) of each component as well as its composite variability ( $\beta_C$ ). Using these two values, the HCLPF capacity of the component is derived. The median seismic capacity ( $A_m$ ) and composite variability ( $\beta_C$ ) are estimated using a combination of ACR-700 design criteria, generic data, and other relevant sources.

The conservative deterministic failure margin (CDFM) method estimates the seismic capacities in terms of HCLPF values based on the following equation:

$$\text{HCLPF (CDFM)} = (\text{Seismic Capacity}/\text{Seismic Demand at Review Level Earthquake (RLE)})^* \text{ RLE}$$

Seismic capacities are defined as a 95% exceedance probability, including the non-linear behaviour of the equipment, and the seismic demand is based on the 84% exceedance probability estimated for RLE. Both the fragility analysis and the CDFM method are used for deriving HCLPF. Results of the preliminary fragility analysis for sample structure and equipment of the ACR-700<sup>®</sup> are shown in Table 1.

**Table 1. Results of Preliminary Fragility Analysis  
for Sample Structure and Equipment of the ACR-700®**

<b>System</b>	<b>Component</b>	<b>HCLPF</b>
Civil	Reactor Building Containment Structure	1.00g
Process	Steam Generator	0.50g
Reactor and Fuel Handling	Calandria Assembly	0.50g
Instrumentation & Control	Secondary Control Area (SCA) Panels – Relay & Meter	0.41g

These HCLPF component capacities are considered to be reasonably achievable for the ACR when designed and constructed using the ACR seismic design criteria and current design and qualification methods. In fact, it is expected that these capacities will be exceeded in the ACR plant.

### **3.2 System analysis**

The first step in the analysis is to develop the safe shutdown equipment list (SSEL). These are components that are necessary to perform the safety functions and include both front line and support systems. The support systems, such as electrical power, cooling water, and instrument air, provide services to the front line systems. The SSEL also includes items that may fail during an earthquake, which may lead to an initiating event (IE). The IEs identified in the internal events PSA must be reviewed and taken into account to ensure that all potential initiating events are covered in the SMA.

The internal events PSA fault trees do not provide a complete list of equipment for the SMA; structural items must be added to the list, for example, electrical panels and cabinets, walls, and buildings. For each safety function, the safety system(s) and its components must be identified.

Generally speaking, manual valves, check valves, small relief valves, and other passive equipment are not included in the SSEL. Based on earthquake experience data and past seismic evaluations, these are judged to be seismically rugged. However, during a seismic walk-down, these items must be checked. The ACR will use solid-state relays, considered to be seismically rugged, and relay chatter analysis is not performed.

The sources of information for the SSEL include the seismic qualification equipment list, the basic event list for the internal events PSA, design or operational flow sheets, and elementary wire drawings. Component information that is required for items on the SSEL includes the component identification, description, redundancy, component location (room and elevation), type/class of component, normal operating position, fail-safe position, manufacturer, power supply (control and power), and any other conditions that may apply.

Other special considerations that must be taken into account when compiling components in the SSEL include the following:

- Identification of active components that are required for the isolation of potential diversion paths.
- Identification of any block walls that are marginally reinforced or not reinforced that may fall and fail safety equipment.

A simplified approach for modelling such components is followed, as described below. The goal of this approach is to include such components within the model in a way that best enables sensitivity analysis of alternate seismic design requirements. The types of components considered include the following:

- Active valves.
- Cables.
- Piping and passive valves.

These component types may be non-seismically qualified, DBE-A-qualified, or DBE-B-qualified. Therefore, HCLPF capacities are estimated for each of these nine categories of components; i.e., for three component types and three levels of qualification.

It is necessary to be familiar with the electrical system – its layout, bus hierarchy, and cabinet and panel naming conventions, etc. The impact on the plant of the seismic failure of cabinets and panels is assumed and will be confirmed at a later date when design details are fully available.

### ***3.3 Assumptions***

Several key assumptions are made to perform this analysis due to insufficient plant-specific design information:

- Seismically initiated small loss of coolant accidents (LOCAs) are assessed to possibly occur at seismic levels of 0.50 g, and the impulse lines are judged to have a high seismic capacity.
- Non-seismically qualified structures are designed with enough separation and will not impact or collapse on seismically qualified structures.

### ***3.4 Development of seismic event trees and fault trees***

The methodology for developing seismic event trees is somewhat different from that of the internal events PSA. Mitigating and support systems may fail at the same time as a result of the seismic event. In this respect, the methodology is different than for the internal events PSA, where the initiating event (IE) generally affects one component or system at a time unless it is a support system failure.

The event tree structure describes the combination of system successes and failures that can result in the design basis accidents and/or core damage. The structure reflects system interrelationships and accident phenomenology that determine whether the sequences lead to severe core damage. In association with an assumed seismic level, the seismic event trees are used to perform accident sequence quantification to derive the HCLPF capacity of the final state (end state) of a particular accident sequence.

Two sets of event trees must be developed. The first set of event trees (seismic pre-tree) is used strictly to determine the seismic-induced initiating events. As such, the sequences in the seismic initiating event trees terminate in either a success state, a damage state in which the reactor core has partially or fully disassembled, or some seismic-induced initiating events.

The 2<sup>nd</sup> set of event trees is to delineate the plant behaviour in response to the seismic initiators. The 2<sup>nd</sup> set of event trees is developed by modifying the internal event trees to reflect the seismic-induced conditions.

For this assessment, the internal event trees developed for a loss of all Class IV power are adapted. The loss of Class IV power trees separately considers sequences with and without small LOCAs and large LOCAs. The loss of Class IV power trees also considers safety-related and non-safety-related systems. The responses of non-safety-related systems, which may still function following a loss of all off-site power, are incorporated even if they are not seismically qualified. This permits sensitivity studies to be performed in which such systems are upgraded to be seismically qualified.

The plant damage states (PDS) are the same as those of the internal events at-power PSA for this analysis [10].

### ***3.5 Human reliability analysis***

The models used for human reliability analysis for the ACR-700<sup>®</sup> PB-SMA are similar to those used for the PSA for internal events. The methods prescribed in the accident sequence evaluation program human reliability analysis (ASEP HRA) procedure [9] are used for both pre-initiator and post-initiator actions.

Adjustments are made to these procedures in the assignment of human error rates to account for the perceived increased stresses the operators would experience following the occurrence of a large earthquake.

#### 4. Results

Two cases are analyzed to determine the ACR-700<sup>®</sup> plant HCLPF capacities:

- a) Base Case: The auxiliary feedwater (AFW) is in the turbine building and not seismically qualified.
- b) Sensitivity case: Emergency feedwater system is located in the reactor auxiliary building (RAB) and seismically qualified.

For the base case ACR-700<sup>®</sup> design – reviewing cutsets with solely seismic failures – the plant HCLPF for severe core damage is 0.5g. At 0.5g, the dominant sequences are large LOCA sequences with failure of seismically qualified electric power that would potentially result in severe core damage. A large LOCA may result from seismic failure of several fuel channel assemblies or of the pressurizer.

For the base case – when mixed cutsets involving seismic failures and non-seismic failures are considered – the plant HCLPF for severe core damage and for limited core damage is also 0.5g. This indicates a robust design in which unusual operator actions are not relied upon to achieve the stated plant HCLPF capacity.

For the sensitivity case – which analyzes a seismically qualified EFW system located in the RAB – the plant HCLPF capacities are the same as those for the base case for both severe core damage and for limited core damage (LCD).

For the sensitivity case – when mixed cutsets involving seismic failures and non-seismic failures are considered – the HCLPF capacities are the same as those for the base case. The only difference is a slight increase in the highest-ranked mixed cutset failure probability for severe core damage. The HCLPF capacity for small LOCA is assumed to be 0.5g.

#### 5. Insights

Several insights are obtained from this PB-SMA of the ACR-700<sup>®</sup> that provide feedback to the ACR design:

1. In the event that small LOCAs are to occur at lower seismic levels, a key sequence to consider involves seismic failures of the non-seismically qualified AFW system combined with failure of the operators to align the RWS for gravity feed of the steam generators due to the limited time available. In this case, seismically qualified EFW ensures higher HCLPF capacity than the HCLPF capacity, non-seismically qualified AFW.
2. The loss of long-term cooling in the LOCA recovery mode following a large LOCA limits the seismic margin capacity of the ACR-700<sup>®</sup> to 0.5g.
3. The moderator cooling system, though designed to not collapse following a DBE, is assessed at having a fairly high functional capacity. For selected PB-SMA sequences, the moderator cooling system provides an effective cooling means to limit the sequence consequence to limited core damage.
4. The seismically qualified buildings are all assigned high seismic capacities. Examples include the reactor auxiliary building (0.6g), control building (0.6g), and the seismically qualified diesel generator building (0.60g). By contrast, the non-seismically qualified buildings are assigned much lower HCLPF capacities. Examples include the turbine building (0.35g), crane hall (0.35g), and the service building (0.35g). As noted previously, it is important that seismically qualified buildings are designed to ensure that they are not susceptible to impacts from non-seismically qualified buildings at lower than 0.6g in order to maintain the assessed HCLPF capacity of the seismically qualified building.

## 6. Conclusions

The PB-SMA during the early stages of the ACR-700® design has played an important role in estimating its seismic margin in terms of HCLPF capacity. This paper summarizes the results and insights gained from a Level 1 PSA-based SMA for the ACR-700®. With these estimates and the ACR-specific seismic event trees and fault trees, an overall plant seismic margin measured in terms of the HCLPF capacity of 0.5g peak ground acceleration is reasonably achievable for the ACR design. This indicates a robust design in which unusual operator actions are not relied upon to achieve the stated plant HCLPF capacity. This HCLPF capacity compares well with the design basis earthquake of 0.3g for the ACR. Appropriate design provisions and conservatism in seismic capacity evaluation will be made to support an HCLPF capacity of 0.5g in the ACR-1000® design. The results obtained in this analysis show a robust design that provides a strong foundation for the ACR-1000® design development currently underway.

## Nomenclature

ACR	Advanced CANDU Reactor	LCD	Limited Core Damage
AECL	Atomic Energy of Canada Limited	LTC	Long-Term Cooling
CANDU	CANada Deuterium Uranium	NSQ	Non-Seismically Qualified
CNSC	Canadian Nuclear Safety Commission	NUREG	Nuclear Regulatory Guide
DBE	Design Basis Earthquake	PB-SMA	PSA-Based Seismic Margin Assessment
ECC	Emergency Core Cooling	PDS	Plant Damage State
ECW	Essential Cooling Water	PSA	Probabilistic Safety Assessment
ESW	Essential Service Water	RAB	Reactor Auxiliary Building
EFW	Emergency Feedwater	RWS	Reserve Water System
HCLPF	High Confidence of Low Probability Failure	SCD	Severe Core Damage
GDE	Generic Design Earthquake	SSCs	Structures, Systems, and Components
IE	Initiating Event	SSEL	Safe Shutdown Equipment List
IPEEE	Individual Plant Examination of External Events	USNRC	United States Nuclear Regulatory Commission
LOCA	Loss of Coolant Accident		

## Acknowledgments

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Figure 1. ACR-1000 Reactor Building Cross-Section

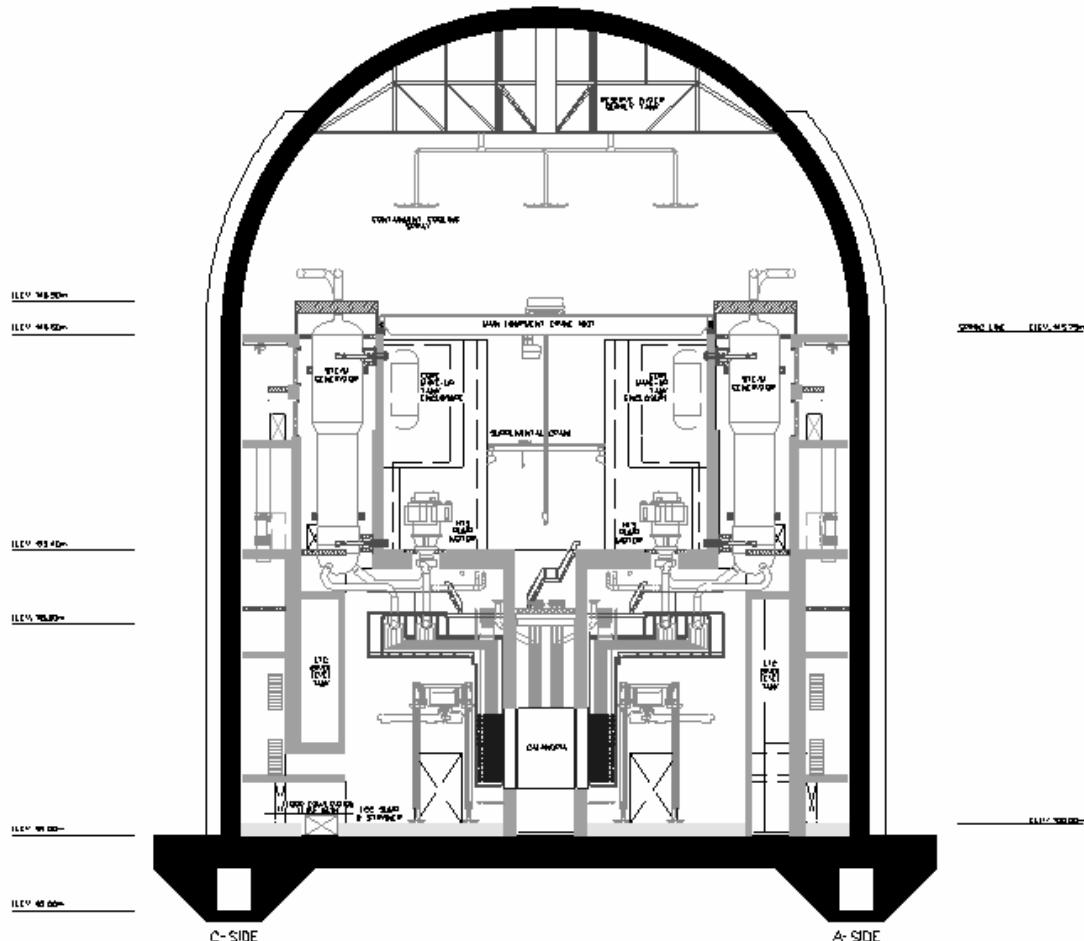
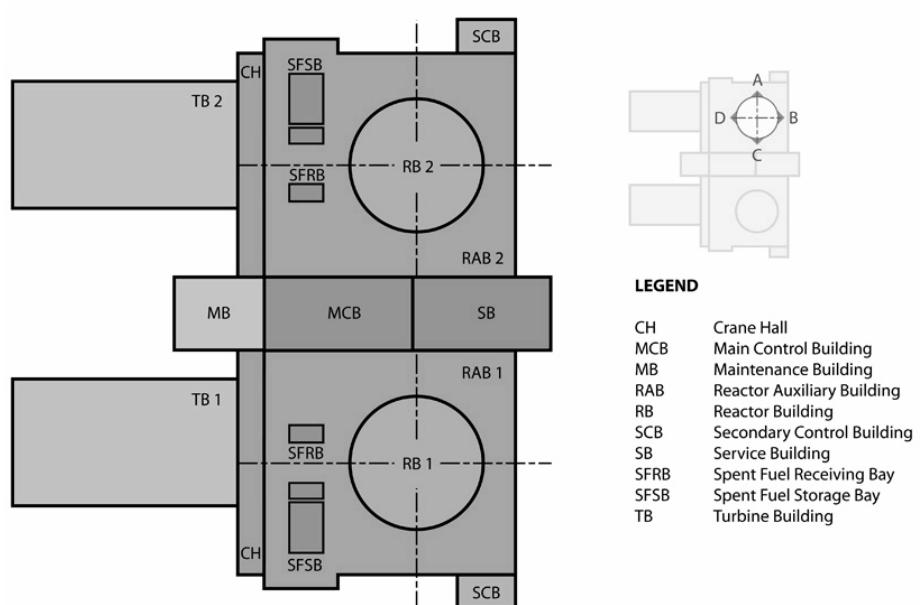


Figure 2. ACR-1000 Two-Unit Plant Layout



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# Seismic Risk Analysis of a CANDU Containment Building for Probability Based Scenario Earthquake

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## *Abstract*

In this study, the seismic risk of CANDU containment buildings was estimated by performing nonlinear seismic analysis for scenario earthquakes. The lumped mass model of the containment building was used for a nonlinear dynamic time history analysis. The tri-linear skeleton curve was used for the nonlinear behavior of the prestressed concrete containment building. In order to estimate the inelastic nonlinear response of the containment, the maximum point oriented model was used for the hysteretic rule of the shear deformation. The scenario earthquakes were defined as the potential earthquakes which can occur at the Korean NPP sites. The probability based scenario earthquake was estimated for the Wolsung NPP site and used for the seismic risk analysis of the containment building.

## 1. Introduction

The standard response spectra proposed by US NRC [1] and Canadian standard [2] have been used as design earthquake for the Korean nuclear power plant structures and CANDU nuclear power plants in Korea. However the seismic safety of a nuclear power plant can not be secured by considering only the design basis earthquake as more knowledge on the seismological situation of a nuclear power plant site is gained through the advances in geosciences.

In Korea, a survey on some of the Quaternary fault segments near Korean nuclear power plants is ongoing [3]. Some Quaternary fault systems being surveyed are located near the nuclear power plants in Korea. It is likely that these faults will be identified as active ones. If the faults are confirmed as active ones, it will be necessary to reevaluate the seismic safety of the nuclear power plants located near the faults. Considering this site condition, near-fault earthquakes can occur at a nuclear power plant site near the potential active faults.

A probabilistic seismic hazard analysis (PSHA) of all the nuclear power plant sites in Korea was performed as a part of a probabilistic seismic risk assessment (SPRA). In the SPRA studies, the seismic hazard curves from the probabilistic seismic hazard analysis were used for the calculation of the core damage frequency (CDF). But, in the seismic fragility analysis and seismic margin assessment, several kinds of response spectra were used as an evaluation spectrum, such as the median site specific spectrum and NUREG-0098 spectrum [4]. It can cause a large uncertainty and incorrect results in the seismic risk of a nuclear power plant.

In this study, the seismic risk analysis based on nonlinear dynamic time-history analyses was performed to investigate the seismic safety of the containment buildings subjected to the probability based scenario earthquakes.

## 2. Seismic fragility result of past SPRA

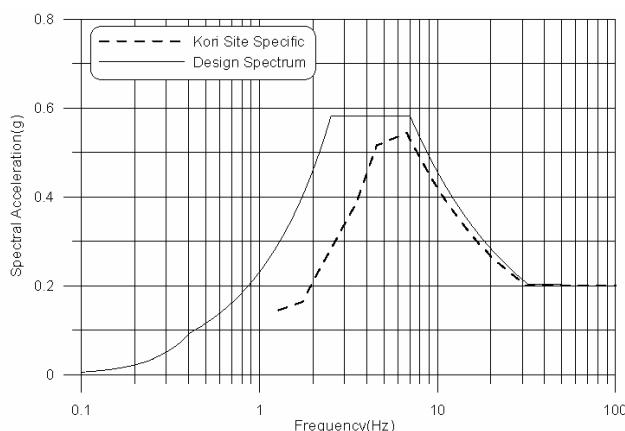
The seismic fragility analysis in the past SPRA study was performed by using the Zion method. The evaluation response spectrum is a Kori site-specific median response spectrum developed about fifteen years ago. The seismic response used in the fragility analysis was the result of a design analysis, which was performed by using a response spectrum analysis method. The nonlinear behavior of the reinforced concrete

containment building was considered by using an inelastic energy absorbing factor. Table 1 shows the summary of the safety factors and their logarithmic standard deviation for a randomness and uncertainty. The spectrum shape factor considered in the safety factor was 2.037 based on the Kori site-specific median spectrum. Figure 1 shows the comparison of the design spectrum and the evaluation spectrum.

**Table 1. Seismic fragility result for the Wolsung NPP containment building**

SSE(g)	Combined safety factor	Median capacity (g)	Randomness	Uncertainty	HCLPF (g)
0.2	17.81	3.562	0.30	0.40	1.122

**Figure 1. Comparison of the design spectrum and evaluation spectrum**



### 3. Probability based scenario earthquake

The probabilistic seismic hazard analysis for most of the Korean NPP sites has been completed. The hazard consistent earthquake scenario is developed as the probability based earthquake scenario by using the existing results of the probabilistic seismic hazard analysis. The scenario earthquake is specified in terms of the earthquake magnitude, M, and its distance, R, from the site under consideration. The probability based scenario earthquake is developed by the de-aggregation of the probabilistic seismic hazard analysis results according to the procedures of the US NRC R.G. 1.165 [5]. The spectral shape for the scenario earthquake is developed by using the attenuation equations adopted in PSHA. The near-fault ground motion effect is incorporated into the response spectra, since the potential active fault is located near a nuclear power plant site. Near-fault ground motions have caused considerable damage in the vicinity of seismic sources during recent earthquakes. This is due to the pulse-type ground motion, which has a large amount of input energy.

#### 3.1 Probabilistic seismic hazard analysis

The probabilistic seismic hazard analysis was performed for the NPP site. The team approach developed by EPRI (Electric Power Research Institute) was adopted for the hazard analysis. Three seismicity expert teams and one attenuation team were established to obtain the PSHA input parameters. At least one non-seismologist was included in each seismicity team. However, in the attenuation team, only one expert recommended several different attenuation equations with a weight [6].

Figure 2 shows one of the seismic source maps which was used for the evaluation of a seismicity by the expert team. Table 2 shows the attenuation equations recommended by the expert. As shown in this table, the attenuation expert recommended three attenuation equations for a peak ground acceleration and three attenuation equations for a spectral acceleration with an individual weight.

Using these PSHA input data proposed by the expert teams, the PSHA was performed for the site. Figure 3 shows the seismic hazard curves for the example NPP site.

Figure 2. Example seismic source map for PSHA

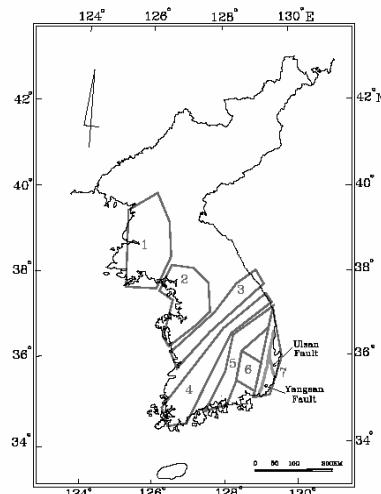
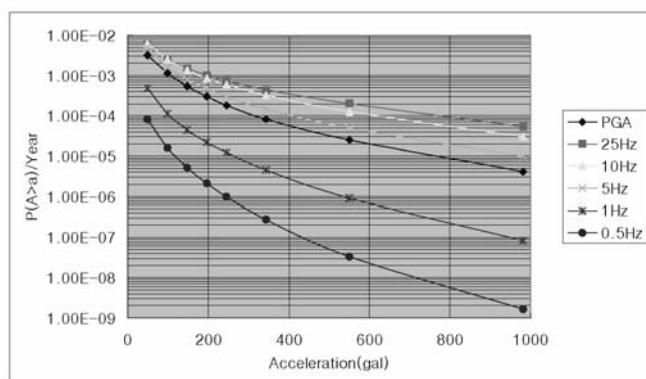


Table 2. Attenuation equations for PSHA

Ground Motion Measure	Model #	Description	Var.	Minimum Distance	Weighting		References
PGA	1	South Korea	0.6	0 km	0.5	Sum =1	Baag, 1998
	2	Central & Eastern North America		0 km	0.3		Toro, Abrahamson and Schneider, 1997
	3	North China		0 km	0.2		Zhixin, Xiaobai and Jingru
SA (Spectral Acceleration)	4	Central & Eastern North America		0 km	0.5	Sum =1	Toro, Abrahamson and Schneider, 1997
	5	South Korea		0 km	0.3		Baag, 1998
	6	Eastern North America		0 km	0.2		Atkinson and Boore, 1995

Figure 3. Seismic hazard curves for Wolsung NPP site



### 3.2 Probability based scenario earthquakes

The seismic hazard was de-aggregated to determine the dominant magnitudes and distances at the prescribed exceedance level. In this study, the seismic hazard was de-aggregated at 1Hz, 5Hz, and 10Hz at the  $10^{-5}$  exceedance level according to the U.S. NRC Regulatory Guide 1.165 [5]. According to the guide, the seismic hazard should also be de-aggregated at 2.5Hz. But the ground motion attenuation equations proposed by the expert did not include the equation for 2.5Hz. The fractional contribution of the magnitudes and distance bin to the total hazard for 1Hz was used for the development of a low frequency scenario earthquake. Because the contribution of the distance bins greater than 100km contained less than 5% of the total hazard for the 1Hz, additional calculations to consider the effects of distant and larger events were not needed.

Figure 4 shows the contribution of the magnitude and distance bins for 1Hz and the average of the 5Hz and 10Hz, respectively. The scenario earthquakes for the example site were determined based on the contribution. Table 2 shows the magnitude and distance of the scenario earthquakes for the example Korean NPP site. As shown in this table, the magnitudes and distances of the scenario earthquakes are very similar. It may be due to the small contribution of the distant earthquakes of the 1Hz scenario earthquake.

Figure 4. Contribution factors

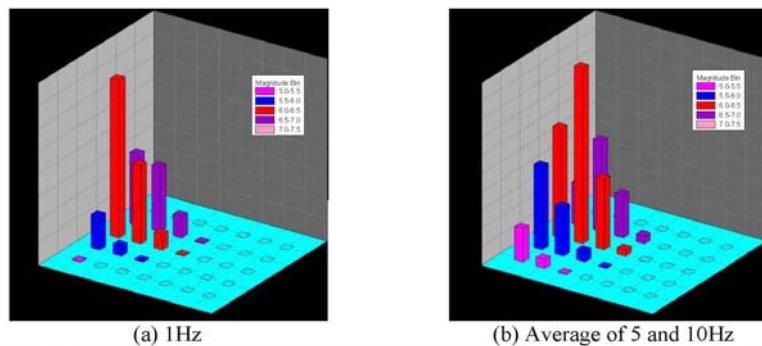


Table 2. Probability based scenario earthquakes

1Hz	5-10Hz
M6.4, 9.0Km	M6.2, 13.0km

### 3.3 Near-fault ground motion effects

These features of near-fault ground motions are generally not considered in the seismic design of nuclear power plant structures and components. Many researches have been performed to identify the characteristics of near-fault ground motions [7,8,9]. Ohno et al. [10] showed the range where the near-fault rupture directivity effect is dominant and proposed a method to correct the predefined response spectrum considering this effect. Figure 7 shows the FN (Fault Normal) to FP (Fault Parallel) response spectral ratio from 37 records of the 11 strong earthquakes. These strong motion records were strongly affected by the near-fault directivity effect. Based on this study, Nishimura et al. [11] propose a correction factor to modify the response spectrum. The correction factor,  $\lambda(T_i)$ , can be obtained from the following equations.

$$\lambda(T_i) = 1 \quad \text{for } T_i = T_D \quad (1)$$

$$\lambda(T_i) = 10 \log(2.5) \cdot \log(T_i/T_D)/\log(T_H/T_D) \quad \text{for } T_D < T_i \quad (2)$$

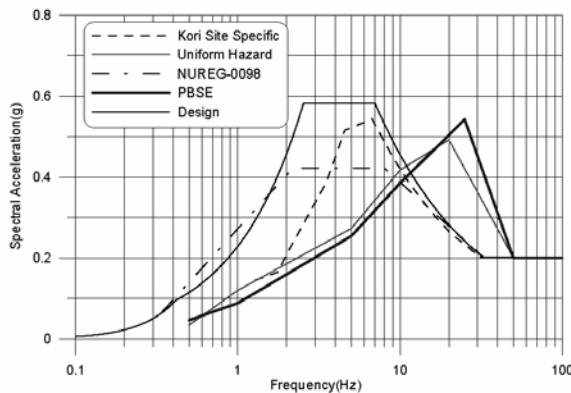
where,  $T_i$  denotes the period.  $T_D (= 0.33\text{sec})$  and  $T_H (= 5\text{sec})$  are the control points of the design ground response spectrum. These equations show that only the spectral acceleration for a long period greater than 0.33 sec is increased due to the near-fault rupture directivity effect. And, it is assumed that the spectral acceleration amplification due to the near-fault rupture directivity effect does not appear in a short period range shorter than 0.33 sec.

Using the proposed equation, the spectral shapes for the scenario earthquakes from the attenuation equations were modified to incorporate the near-fault rupture directivity effect. Figure 5 shows the modified response spectral shape for the PBSE with the design and other evaluation spectrum.

#### 4. Seismic fragility analysis for PBSE

In Korea, several types of containment buildings were constructed. In this study, two types of the containment building were considered for the safety evaluation. The first one is a prestressed concrete containment building which is a typical containment structure for the Korean standard NPP (KSNP). Most of the NPP in Korea are KSNP. The second one is a prestressed concrete containment building which is a typical containment structure for CANDU NPP. There are 4 units of CANDU NPP in the south-eastern part of the Korean peninsula.

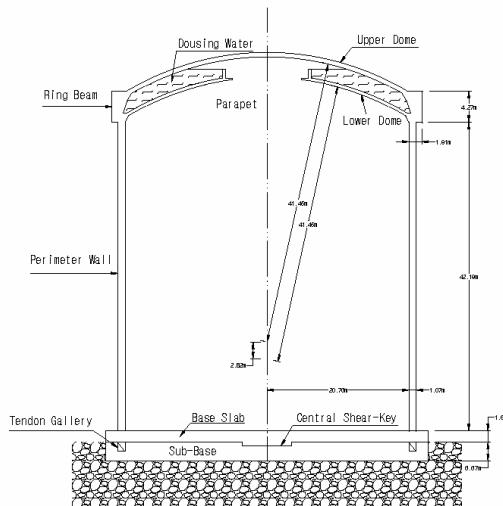
Figure 5. PBSE response spectrum with design and other evaluation spectrum



#### 4.1 Characteristics of CANDU containment building

The CANDU containment building in Korea, which houses the nuclear reactor and safety related equipments, is a prestressed, post tensioned reinforced concrete structure [12]. Figure 6 shows the schematic view of a CANDU containment building. The containment building of a CANDU nuclear power plant has some specific features, such as a dousing tank, dousing frame, ring beam, etc., when compare with that of a PWR plant. The CANDU containment contains dousing water of 1,560m<sup>3</sup> in an elevated tank around the building dome for a powerful pressure suppression, not like the PWR sprays.

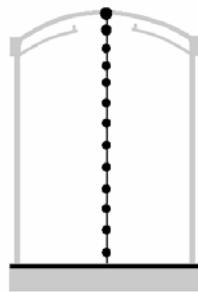
Figure 6. CANDU containment building



#### 4.2 Analysis model

Lumped mass models were developed for the nonlinear dynamic time history analysis. The mass of the dousing water in the CANDU containment building was assumed to be attached to the adjacent parts of the domes and the ring beams in each of the analysis model. The fundamental frequencies obtained from the finite element models and the lumped mass models are summarized with the fundamental frequencies described in the design reports [12]. Figure 7 shows the lumped mass model of the CANDU containment buildings.

Figure 7. Lumped mass model for the nonlinear dynamic analysis



#### 4.3 Nonlinear hysteretic model

In this study, the tri-linear approximation shown in Figure 8 was used for the shear behavior of the containment building. These skeleton curves are of the lower part of the containment shell. The turning points for the shear stress and strain relationship were determined based on the EPJR (Electric Power Joint Research) method [13]. In order to perform an elasto-plastic seismic response analysis, based on the tri-linear skeleton curve, the maximum point oriented model was used as the hysteresis rule for the repeated unloading and loading processes. Figure 9 shows the hysteresis rule of the maximum point oriented model. The moment-curvature relationship is assumed as linear elastic, since the containment building is a very thick reinforced concrete structure and its dominant failure is a shear failure at the bottom of the containment shell.

Figure 8. Tri-linear skeleton curve

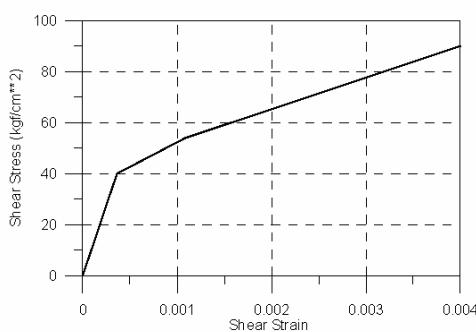
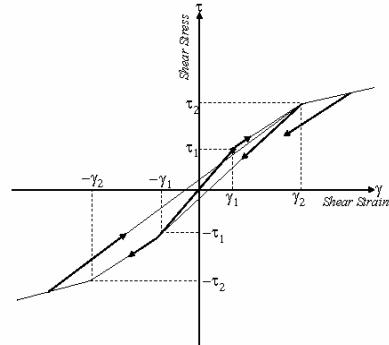
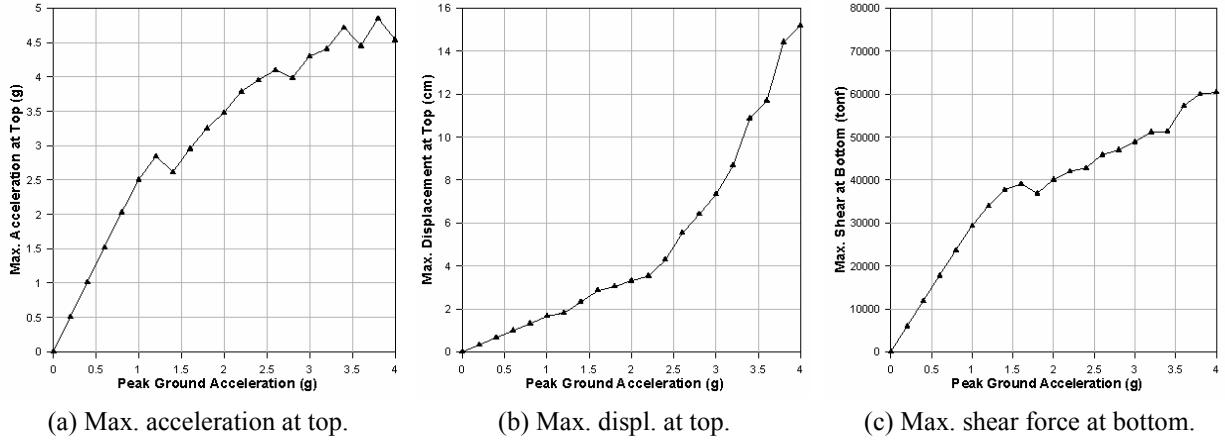


Figure 9. Maximum point-oriented model



#### 4.4 Response of containment buildings

The nonlinear inelastic dynamic analyses were performed to investigate the seismic response of the containment buildings for the PBSE. Figure 10 shows the maximum responses of the containment buildings according to the peak ground acceleration (PGA) of the input ground motions.

**Figure 10. Maximum Responses According to the PGA**

## 5. Fragility calculation

### 5.1 Fragility calculation method

The safety margin of a member of the structure for the present level of a seismic intensity is defined as a ratio of its seismic capacity to its seismic response. Safety margin  $S_f(a)$  concerning some seismic response parameters for a given seismic intensity  $a$  is defined as the ratio of its seismic capacity  $R$  to the seismic response parameter  $S(a)$ , i.e.

$$S_f(a) = R/S(a) \quad (3)$$

and the probability of a failure is:

$$P_f(a) = \text{Prob}(R/S(a) < 1) \quad (4)$$

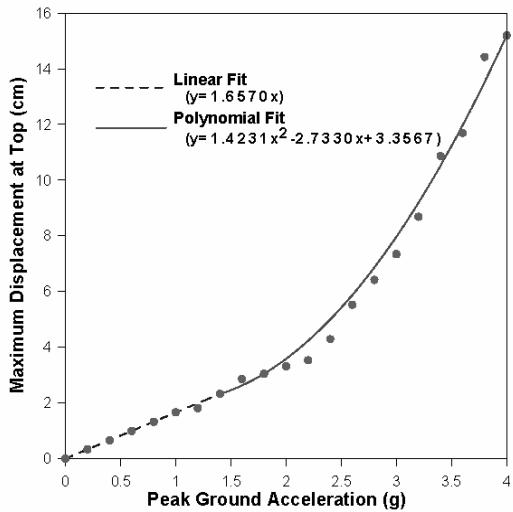
The seismic fragility of a structure represents probabilistically the capability of a ground motion to cause a structural damage. The probability of a failure of a structure  $P_f(a)$  at any non-exceedence probability level  $Q$  can be obtained from the following equation.

$$P_f(a) = \phi\left(\frac{\ln(S_m(a)/C_m) + \beta_U \phi^{-1}(Q)}{\beta_R}\right) \quad (5)$$

where  $\phi(\cdot)$  is the standard Gaussian cumulative distribution function,  $a$  is a peak ground acceleration as a ground motion parameter,  $\phi^{-1}(\cdot)$  is the inverse of the standard Gaussian cumulative distribution function,  $s_m(a)$  and  $c_m$  are the median seismic response at a given ground acceleration  $a$ , and the median seismic capacity, respectively, and  $\beta_R$  and  $\beta_U$  are the lognormal standard deviations of the randomness and uncertainty of  $s_m(a)$  and  $c_m$ , respectively.

In order to calculate the median displacement response at a PGA level, a regression analysis was performed. Figure 11 shows the regression analysis results for the maximum displacement response of the CANDU containment building at the top.

Figure 11. Regression analysis of max. displacement response



## 5.2 Failure criteria

In the fragility analysis, it is very important to estimate various failure modes. The ultimate shear stress and bending moment capacity at the lower part of the containment shell were generally used as the dominant failure modes of the containment building. This conventional method was based on a linear elastic seismic analysis. The effect of a non-linear behavior of a structure is considered by using the inelastic energy absorbing factor [14]. In this study, the non-linear seismic time history analyses for the near-fault ground motions were performed to estimate the non-linear behavior of the containment building for strong ground motions.

The top displacement is used as the damage index of the containment building. A top displacement is very useful for the damage index of a containment building, since a containment building is a very simple cantilever type structure. The damage index of the containment building is obtained from the results of a push-over analysis. The ultimate displacement of the containment building was 18.35cm at the top [15].

## 5.3 Fragility curves

The logarithmic standard deviations for a randomness was estimated based on the statistical analysis of the containment response for 30 artificial earthquake time histories which envelop the target response spectrum. The estimated logarithmic standard deviation for a randomness was 0.10 [16]. The logarithmic standard deviation of 0.32 is used for the uncertainty of the displacement capacity. This value was proposed for the ultimate strain of a cylindrical wall by Ozaki et al. [17].

Figure 12 shows the fragility curves of the containment building for the PBSE. The realistic seismic capacity of the structures and equipments can be expressed by the fragility curves or HCLPF (High Confidence of Low Probability of Failure) capacity. HCLPF capacity in the SPRA is defined mathematically as a 95% confidence of less than a 5% probability of a failure. This HCLPF capacity is generally used as an index to represent the seismic capacity of the structures and equipments in a NPP. The HCLPF capacity of the CANDU containment building is 2.155g. This HCLPF capacity is almost two times as large as that from a past SPRA study. Figure 13 shows the comparison of the mean fragility curves from this study and past SPRA study. As shown in this figure, the fragility of this study is lower than that of the past SPRA study at the small and intermediate earthquake ground motion level. The fragility for the PBSE increases rapidly at the strong ground motion level due to a nonlinear response.

Figure 12. **Fragility curves of the CANDU containment building**

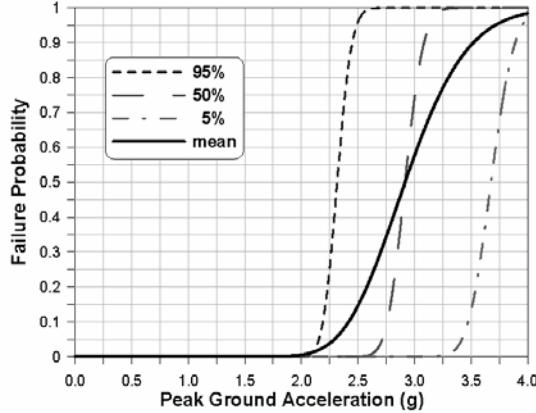
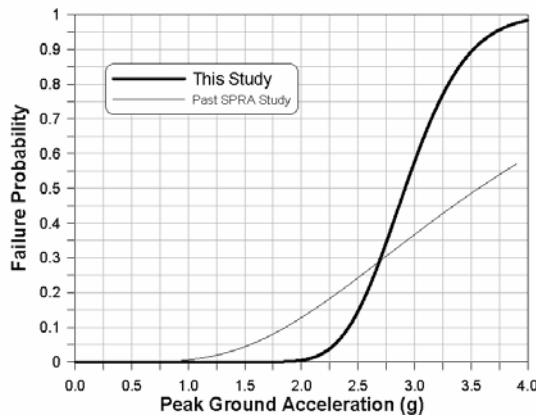


Figure 13. **Comparison of the mean fragility curves**



## 6. Seismic risk analysis

For a nuclear power plant component, a mean probability of an unacceptable performance can be obtained by a convolution of the seismic hazard and seismic fragility curves. The probabilistic seismic risk can be obtained from the following equation.

$$P_F = - \int_0^{\infty} \left( \frac{dH(a)}{da} \right) P_f(a) da \quad (6)$$

Where  $H(a)$  is the seismic hazard curve and  $P_f(a)$  is the probability of a failure (fragility) for a given ground motion amplitude  $a$ , which captures both the response and capacity uncertainties.

The mean frequency of the CANDU containment building is 2.70e-7/year. The mean frequency from the past SPRA study is 1.09e-6. The probabilistic seismic risk from the past SPRA study is greater than that from this study. This means that the past SPRA study was performed by using a conservative evaluation response spectrum and did not reflect the seismological condition of the Wolsung NPP site according to the advances of the geosciences.

## 7. Conclusion

In this study, the probabilistic seismic risk analysis of the CANDU containment building was performed based on the displacement responses obtained from the inelastic nonlinear seismic analyses for the PBSE.

The evaluation response spectrum plays an important role in a seismic risk analysis. In Korea, several kinds of evaluation response spectra have been used for the seismic fragility analysis and the seismic margin assessment. Even though the probabilistic seismic hazard curve was used for the seismic risk evaluation, the evaluation response spectra developed from a seismic hazard analysis have not been used for a seismic fragility analysis and a seismic margin assessment due to a large uncertainty of the seismic hazard. However, it is necessary to use the evaluation response spectra developed from a seismic hazard analysis to preserve the consistency in a seismic fragility analysis and a seismic risk analysis.

It can be concluded from this study that the realistic evaluation response spectrum is essential for a evaluation of the realistic seismic risk of NPP SSCs.

### **Acknowledgement**

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### **Topical Area 3. Methodology**

Evaluation on some gaps between theoretical approach and techniques for practice in seismic PSA code

H. Shibata, Professor Emeritus, University of Tokyo, Japan, Invited

Procedure for the evaluation of damaging effects of earthquakes – J-U. Klügel, NPP Goesgen-Daeniken, Switzerland

Seismic PSA methodology for multi-unit sites

T. Hakata, NSC, Japan

Japan: accident sequence study for seismic event at the multi-unit site

K. Ogura, M. Fukuda, M. Sakagami, K. Ebisawa, JNES

The radiological consequence analysis by evacuation model for earthquake event of a BWR plant

K. Funayama, S. Sumida, M. Kajimoto and M. Sakagami, JNES, Japan

Potential uses of seismic PSA for risk management of NPPs – a review of recent studies at JAEA

K. Muramatsu, Q. Liu, K. Onizawa, JAEA, Japan

#### ***Technical session: Hazard analysis***

Probabilistic seismic hazard assessment for OL3 plant site in Finland

P. Varpasuo, Fortum Nuclear Services Ltd, Finland

Development of seismic PSA methodology considering aftershock

H. Tsutsumi, H. Nanaba, S. Motohashi, K. Ebisawa, JNES, Japan

An innovating approach to address uncertainties in probabilistic seismic hazard assessment

E. Viallet, C. Martin, C. Durouchoux, P. Labbé, P. Sollogoub, D. Sicilia, N. Humbert, EdF/GEOTER/CEA, France

A probabilistic seismic hazard assessment based on logic tree utilizing expert judgment

H. Nanba, H. Kameda, K. Ebisawa, M. Sakagami, K. Irino, JNES, Japan

Probabilistic evaluation of near-field ground motions due to buried-rupture earthquakes caused by undefined faults

S. Motohashi, JNES, Japan

#### ***Technical session: Ground motion***

PEGASOS: a PSHA for the Swiss nuclear power plants – Some comments from the managerial point of view

P. Zuidema, NAGRA, Switzerland

Recent advances in developing site-specific ground motion attenuation relations for the nuclear power plant sites in Korea

K-H. Yun, D-H. Park, KEPRI, Korea

Probabilistic approach to design earthquake ground motions based on fault rupture scenarios

K. Dan, M. Muto, N. Hata, J. Miyakoshi, M. Kaneko, Y. Suzuki,

J. Kanda, Ohsaki Research Institute, Inc.; The University of Tokyo, Nagoya University, Japan

Determination of fragility curves of aging plants by ambient vibration measurements

H. Wenzel, G. Achs, VCE Holding, Austria

#### ***Technical session: Fragility***

A Study on comparison of seismic capacities by probabilistic approach and deterministic approach

J-G. Song, S-K. Kang

Usage of information entropy in updating seismic fragilities

A. Yamaguchi, Osaka University, Japan

Development of probabilistic safety assessment considering slope collapse by earthquake

H. Sugino, K. Mori, H. Tsutsumi, M. Sakagami, JNES, Japan

Development of the fragility curve of the refuelling water storage tank

R. Bertrand - J.M. Rambach - C. Lincot, IRSN, France

Seismic fragility capacity tests on nuclear power plant

M. Sakagami, N. Takamatsu, T. Iijima, JNES, Japan

Seismic fragility evaluation of electrical power systems and influence evaluation of seismic base isolation on seismic risk

K. Sano, H. Tsutsumi, K. Tanabe, M. Sakagami, K. Ebisawa, JNES, Japan

#### ***Special session: Standards***

Development of American national standard on external event PRA methodology

M.K. Ravindra, ABS Consulting, USA, Invited

Outline of seismic PSA implementation standards on the Atomic Energy Society of Japan

M. Hirano, T. Nakamura, K. Ebisawa, Japan, Invited, Kansai Electric Power Co / JNES

## Evaluation on Some Gaps between Theoretical Approach and Techniques for Practice in Seismic PSA Code

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**Key words:** Seismic-PSA, computational modeling, evaluation of uncertainty on modeling, evaluation of quality of PSA code

### ***Abstract***

To evaluate the probability of occurrence of vital failures and accidents in a light water nuclear power plant by a distractive earthquake event, we should analyze various steps from events caused by an earthquake to failure of primary cooling water systems and other protective systems from the view point of their final critical states. In these processes, we use various technical approaches in each step of analysis or evaluation.

The author wants to discuss on the effect of selecting an approach in each step in relation to the seismic design process. The Seismic PSA is based on the following knowledge as well as the design process: Results obtained from mathematical modeling, with data or without data, semi-empirical model, application of highly complicated simulation soft ware, alternative simplified methods and so on based on observations on actual events.

In usual process of Seismic PSA, each step of computation uses various models selected from those mentioned above. To complete a PSA study, we need uniform quality approaches for each step of evaluation in the treatment of uncertainty as stochastic characteristics. For example, if we use a deterministic relation in some step, we need to evaluate its uncertainty. However, by a different type of approaches, we should expect the different random nature, in general. We should examine the uncertainties in each step and compare them through the whole process of seismic PSA.

The American Nuclear Society (ANS) and the Atomic Energy Society of Japan (AESJ) have recently developed two seismic PSA standards, which describe accepted methodologies in two countries. Although they have difference in style of description, that is, the ANS standard has 126 pages, on the other hand, the draft of AESJ has approximately 600 pages to provide sufficient guidance information, both of them suggest that there are many steps for which several allowable methods exist. The variations of accepted methods for individual steps are not always small. Most extreme case, the author believes, in Japanese procedure which appears in the AESJ Standard is that for floor response. Although it comes from the design procedure defined in the Seismic Design Guideline, JEAG 4601 originally, the method of generating the floor response curve in the design guideline was an ordinary response analysis method using a deterministic time history and there were no sense of stochastic evaluation in origin. For S-PSA evaluation, we should examine those processes in design as mentioned above step by step. However, this is not an easy task because the number of pages of the original Seismic Design Guideline is huge, that is, its size is approximately 2 000 pages. This was partly translated into a NUREG report (NUREG/CR-6241).

In this article, the author tries to refer some examples which have been discussed in the recent revision of the Licensing Criteria for seismic design by the Nuclear Safety Commission of Japan and the Technical Guideline for seismic design by the Japan Electric Association. Those were 1) the criteria of collapsing of full cross-section of primary cooling piping, 2) the seismic accumulative failure and wave (loading) numbers and 3) results of equipment obtained by high acceleration shaking tests and their variations.

Back to the theoretical model of the wave number in 2) as an example, in 1950s, for response analysis of a vessel in the sea, a stochastic technique has been developed, but very few people have introduced such approach into Seismic PSA field. The idea of applying the concept of random vibration to ground motions was presented in 1965. This might be applicable to response analysis generally, and it is useful to make clear the stochastic nature for Seismic PSA analysis.

The author wants to give an overview of the total performance of the practice step by step but because of the space and time, he will refer to only items above mentioned.

The author is planning to summarize it for SMiRT-19, next year based on this article. In the SMiRT paper, he will try to discuss some details on one or two selected typical examples in relation to the design practice from the same view point as that we make in Seismic PSA. However, in the case of Seismic PSA, most of all hazard data come from natural phenomena. We should calibrate them by the observed results, and mathematical models should be updated on the basis of increased knowledge on true phenomena themselves. And the design process is affected by this estimation, the expected hazard.

The author is going to evaluate our knowledge which we introduce for the regulation, the seismic safety guideline in the future, and to find the critical points, which we should improve it in next chance to obtain the more substantial design result in the sense of the total safety.

## Procedure for the Evaluation of Damaging Effects of Earthquakes

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### **Abstract**

From 2000 to 2004 the Swiss nuclear power plants sponsored a large scale probabilistic seismic hazard analysis following the SSHAC-procedures at its most elaborated level 4. This research project was intended to be used as a support of the update of the Goesgen plant-specific PSA. A detailed review based on the development of site-specific controlling earthquakes indicated that the results obtained included the effects of non-damaging earthquakes, which may potentially cause significant accelerations at the plant site not exceeding the acknowledged limits of an operational earthquake as defined in NRC RG 1.166 by a threshold value of 0.16 gs for the cumulative absolute velocity (CAV). Therefore, a detailed procedure was developed to eliminate non-damaging earthquakes from the results of a SSHAC-level 4 PSHA. The procedure allows to calculate the probability of exceedance of the damaging threshold of an operational earthquake. The implementation of the procedure in the site specific seismic PSA leads to a significant reduction of the calculated core damage frequency.

### **1. Introduction**

From 2000 to 2004 the Swiss nuclear power plants sponsored a large scale probabilistic seismic hazard analysis following the SSHAC-procedures (SSHAC, 1997) at its most elaborated level 4. This research project was intended to be used to support the update of the Goesgen plant-specific PSA. The mathematical PSHA model used in modern PSHA studies according to (Abrahamson, 2006) can be represented as:

$$\nu(S_a > z) = \sum_{i=1}^{n_{\text{source}}} N_i(M_{\min}) \int \int \int f_{mi}(M) f_{Ri}(r, M) f_{\varepsilon} P(S_a > z | M, R, \varepsilon) d\varepsilon dR dM \quad (1)$$

Equation (1) represents the usual annual frequency of events, leading to a spectral acceleration  $S_a$ , exceeding a hazard value  $z$ . It is evaluated by summing up the contributions of all relevant sources and by performing source specific integration over magnitude, distance and the error term (called aleatory variability) of the attenuation equation, multiplying the source specific frequency density distributions with the conditional probability of exceedance of the specified hazard level  $z$ . The conditional probability of exceedance is calculated based on the corresponding attenuation equation (or a set of equations in case of the use of logic trees). The attenuation equation can have the following format:

$$\log(S_a) = g(M, R, X_i) + \varepsilon \quad (2)$$

with the error term expressed as the multiple of the standard deviation  $\varepsilon = a\sigma_{\log}$  of the attenuation model.

The standard deviation  $\sigma_{\log}$  reflects the variability of measurement conditions under which the data points (including the data points used for the measurement of magnitude and location) used for the regression were obtained.  $X_i$  represents additional explanatory variables (or classification properties for the specific travel path from the seismic source to the site) of the attenuation model, which may or may not be considered in the model. Examples for these additional explanatory variables are:

- Site conditions (e. g. shear wave velocity, depth of surface layer).
- Topographical and directivity effects.
- Hanging wall and footwall effects.

- Fault style.
- Aspect ratio of the seismic source.
- Material properties of the travel path of seismic waves.

The result of PSHA using equation (1) is clearly driven by the number of standard deviations considered as the boundary condition for the integral over  $\varepsilon$ . The number of standard deviations considered is in principle unlimited, although physical boundaries (e. g. maximal ground motion) can be provided. Therefore, the conclusion is that the hazard integral (1) can converge to infinity (this means it does not converge at all) or to a maximum ground motion level set by the analyst in advance. From Chebyshev's inequality:

$$\Pr\left(\left|X - E(X)\right| \geq a\sigma\right) \leq \frac{1}{a^2} \quad (3)$$

in conjunction with (1), it follows directly that the results of a PSHA are driven by the recordings of statistically rare time-histories, which frequently were recorded under measurement conditions completely different from the site of interest. It is obvious that the traditional PSHA (SSHAC, 1997) represents a worst-case methodology, leading systematically to ambiguous results. Furthermore, it can be shown that the derivation of the hazard integral (1) is based on the simplifying assumption of replacing the random variable  $\Gamma = g(m, r, X_i)$  in equation (2) by an estimate of its expected value  $E(g(m, r, X_i))$ . This is mathematically incorrect. This simplifying approach can be considered as a meaningful approximation as long as the uncertainty bounds in integral (1) are lower than  $\varepsilon = 1\sigma$  (a direct consequence of Chebyshev's inequality).

Furthermore, Vanmarcke & Lai (1980) have shown that for a given content of elastic energy (expressed by the ARIAS-intensity (Arias, 1970)) the strong motion duration reduces almost proportionally with the increase in the peak values of peak ground acceleration. This observation may have two possible consequences:

- a) The spectral shape of the earthquake excitation (response spectrum) is the same as for a large magnitude event causing the same spectrum. Then, the reduction of the strong motion duration means that the number of zero-crossings (a useful damage index) is reduced. Therefore, the damage impact is considerably reduced and not adequately described by the response spectrum in comparison to a strong earthquake.
- b) The spectral shape of the earthquake excitation (response spectrum) is different from a large magnitude event maintaining a similar or even a larger number of zero-crossing. Due to the reduction of the strong motion period, this means that the frequency content of the earthquake excitation is shifted towards higher frequencies, which usually are not transmitted into the robust structures of NPP buildings (first natural frequencies are typically in the range between 2 and 8 Hz).

This means that PSHA-results as obtained by the methods prescribed in national standards (Budnitz et al, 2003) and dominated by the recordings of rare time-histories are not suitable for direct practical applications like the design or risk analysis of nuclear power plants.

Therefore, the need arises to introduce some procedures reducing the impact of the incorrect mathematical approach used in current PSHA-methodology to avoid an overly conservative risk assessment for nuclear power plants. This is required as long as not more advanced methods for probabilistic seismic risk analysis (Klügel et al, 2006) are introduced into national standards.

## **2. General description of the procedure for elimination of non-damaging earthquakes**

The PEGASOS-project consisted of 4 separated subprojects:

- SP1 “Source Characterisation” with four different expert groups.

- SP2 “Ground Motion Characteristics” developing the hazard on generic rock – 5 experts.
- SP3 “Site Response Characterisation” propagating the hazard from generic rock to site surface and different subsurface layers – 4 experts.
- SP4 “Hazard Quantification” – performing computational tasks. Hazard quantification was performed with FRISK88M (discrete probability approach, Rice & Miller (1983)).

The procedure for the elimination of non-damaging earthquakes consists of the following steps:

1. Disaggregation of the hazard in terms of probability of exceedance for different distance ranges, different spectral frequencies and for each of the single expert opinion combinations taking different disaggregation procedures into account (US NRC RG 1.165 and Bazzuro & Cornell, 1999). This allows to develop an epistemic uncertainty distribution of controlling events for each of the considered distance ranges ( $D1 < 16\text{km}$ ,  $D2 16\text{-}40\text{km}$ ,  $D3 > 40\text{km}$ ). The results were presented in terms of magnitude and distance pairs, representing the discrete bivariate epistemic probability distribution of controlling (scenario) seismic events. Scenario events were developed for each single expert opinion combination. Disaggregation was performed on rock (limitation of the FRISK88M code used in the project). Table 1 shows an example of the format of the bivariate discrete probability distribution of the scenario earthquakes as developed from hazard disaggregation.
2. Selection/development of site-specific empirical correlations for the cumulative absolute velocity CAV. Analysis of influencing model parameters (e.g. focal depth) and assessment of epistemic uncertainty. According to NRC RG 1.166 and the underlying supporting investigations, the damaging threshold of an earthquake is defined by a CAV threshold value of 0.16 gs. Correlations have to be developed including the regression error interpreted as spatial randomness of earthquake time histories at different locations (combines inter-event and intra-event variability).
3. Development of a Monte Carlo (Bootstrap) procedure for correlated sampling of controlling events and estimation of CAV-threshold exceedance.
4. Development of a composite epistemic uncertainty distribution of the CAV threshold value for the different distance ranges.
5. Development of a composite epistemic uncertainty distribution for the Uniform Hazard Spectrum (UHS).
6. Application of the composite distribution either for a direct adjustment of the hazard curves or in the PSA-model for the evaluation of the split fractions used to calculate the exceedance of the damaging threshold of earthquakes.

**Table 1. Format of the bivariate discrete probability distribution  
as used for the bootstrap procedure – example for distance range D1 (<16km),  
spectral frequency 1Hz and exceedance probability of 10<sup>-3</sup>/a**

Hazard ex-ceedance probability	Disaggregation frequency	Expert of SP1	Expert of SP2	Disaggregation method 1 (NRC)		Disaggregation method 2 (Bazzuro & Cornell, 1999)	
				M	Distance km	M	Distance km
1E-03	1 Hz	EG1a	Bommer	5.25	13.6	5.15	14.2
1E-03	1 Hz	EG1a	Bungum	5.75	13.6	5.55	14.2
1E-03	1 Hz	EG1a	Cotton	5.25	13.6	5.05	14.2
1E-03	1 Hz	EG1a	Sabetta	5.25	13.6	5.25	14.2
1E-03	1 Hz	EG1a	Scherbaum	5.25	8.8	5.35	14.2
1E-03	1 Hz	EG1b	Bommer	5.25	12.0	5.25	15.0
1E-03	1 Hz	EG1b	Bungum	5.75	12.0	5.35	10.6
1E-03	1 Hz	EG1b	Cotton	5.25	12.0	5.15	15.0
1E-03	1 Hz	EG1b	Sabetta	5.25	12.0	5.25	15.0
1E-03	1 Hz	EG1b	Scherbaum	5.25	12.0	5.35	15.0
1E-03	1 Hz	EG1c	Bommer	5.25	13.6	5.05	15.4
1E-03	1 Hz	EG1c	Bungum	5.25	10.4	5.35	11.8
1E-03	1 Hz	EG1c	Cotton	5.25	13.6	5.05	15.4
1E-03	1 Hz	EG1c	Sabetta	5.25	13.6	5.05	15.4
1E-03	1 Hz	EG1c	Scherbaum	5.25	13.6	5.25	15.4
1E-03	1 Hz	EG1d	Bommer	5.25	12.0	5.25	11.4
1E-03	1 Hz	EG1d	Bungum	5.75	12.0	5.45	11.4
1E-03	1 Hz	EG1d	Cotton	5.25	12.0	5.15	11.4
1E-03	1 Hz	EG1d	Sabetta	5.25	12.0	5.15	11.4
1E-03	1 Hz	EG1d	Scherbaum	5.25	12.0	5.35	11.4

### 3. Empirical equations for the calculation of the cumulative absolute velocity (CAV)

The development of a model or of empirical equations for the calculation of the cumulative absolute velocity is a key step of the procedure for the elimination of non-damaging earthquakes. In an ideal case, the same set of data shall be used for the development of an empirical equation for CAV as was used for the development of the attenuation model of the PSHA. In case of the PEGASOS-project this was not feasible because the original data used for the development of the different attenuation models (a total of 15 models were considered by the experts) applied in the project were not retrieved from the authors of the different models. Therefore, it was necessary to perform a detailed estimation of epistemic uncertainties associated with the use of different models or different data sets.

#### 3.1 European equations for CAV

Kostov (2005) derived a set of attenuation equations for CAV for different distance ranges, using the database of European earthquake time histories. At present, Kostov's work is the only publicly available information on the dependence of CAV on magnitude and distance for European conditions. Please note that Kostov was not able to provide a classification of the ground characteristics for the measurement points of the European earthquake recordings used.

The equation for near-site low to moderate magnitude earthquakes obtains the following form:

$$\log(CAV) = -3.55 + 0.606M_s - 0.461\log(R) + P\sigma \quad (4)$$

with the distance measure  $R$  based on the epicentral distance  $D_{epi}$  and the focal depth  $h$ .  $M_s$  stands for surface magnitude. The uncertainty is given as  $\sigma = 0.21$ , but due to the need to extrapolate to larger magnitude values an additional epistemic component shall be considered. Therefore, a value of **0.25** is used (corresponds to the typical values for attenuation equations for spectral accelerations).

$$R = \sqrt{(D_{epi}^2) + h^2} \quad (5)$$

A second equation was developed by Kostov to calculate CAV in dependence of magnitude and distance for the more distant regional sources. This equation can be used to evaluate the exceedance probability of the critical CAV-value of 0.16 gs for the more distant seismic sources (for the associated controlling earthquakes):

$$\log(CAV) = -2.88 + 0.44M_s - 0.565\log(R) + \sigma P \quad (4a)$$

The associated value for  $\sigma$  is higher than for equation (1),  $\sigma = 0.37$ .

The epicentral distance can be calculated from the Joyner-Boore distance  $R_{JB}$ , used in the PEGASOS project from the following equation (Klügel, 2005):

$$D_{epi} = R_{JB} + \varepsilon \quad (6)$$

$$\varepsilon = -0.86548 + 0.0206M^{2.8861} \quad (7)$$

$M$  here is the moment magnitude. The conversion of moment magnitude  $M_w$  to surface wave magnitude  $M_s$  is based on the equation of Ekström and Dziewonski [4]:

$$M_s = -3.391 + 1.563M_w \quad (8)$$

To apply Kostov's equation to the PEGASOS-results, it was necessary to develop a statistical model for the distribution of focal depths in dependence of magnitude. Only the experts of SP1 provided some estimate for the focal depth of earthquakes in Switzerland, assuming a modal value of the focal depth distribution of 10 km based on observations of minor (low magnitude) earthquakes. This information was used to develop a Gamma-distribution for focal depths with magnitude dependent mean and variation, considering the statistical dependence of focal depth on magnitude discussed by Toro (2003) as part of the PEGASOS-project. Table 2 gives an overview of the characteristics of this distribution.

Table 2. Parameters for the magnitude dependent focal depth distribution [km]

Magnitude ( $M_w$ )	5%	Median	Mean	95%
5.0	0.77	2.58	2.985	6.03
6.0	4.87	9.13	9.661	15.2
7.0	16.4	24.3	25	34.3

The gamma-distribution obtained was used to calculate a randomised set of focal depths, corresponding to the magnitude value of the randomly selected scenario earthquake.

### 3.2 Development of site-specific empirical correlations for CAV from the WUS-database (EPRI 2005)

Two different sets of empirical equations were developed from the published WUS (Western United States and all over the world) database (EPRI, 2005). These published data sets are strongly classified with respect to the relevant ground characteristics in terms of the governing shear wave velocity ( $V_{S30}$ ). They contain a large amount of data from sites with similar values of shear wave velocity as the Gösgen site (in the range between 300 m/s and 500 m/s). A total of 1541 datasets were found to match the range of shear wave velocities measured for the Goesgen site (Anatolian Geophysical, 2006).

The first set of equations was developed following the regression shape suggested by Kostov, replacing the focal depth  $h$  by a free regression parameter, due to the lack of information on focal depths in the WUS database (EPRI, 2005). The following regression shape was selected:

$$\log(CAV) = b_1 + b_2 M + b_3 \log(R) + b_4 R + b_5 (M - 5.0)^2 \quad (9)$$

$$R = (D_{JB}^2 + h^2)^{0.5} \quad (10)$$

with the changed meaning of  $h$  as stated above. A similar correlation was developed for pga using the same datasets.

The obtained coefficients are presented in table 3.

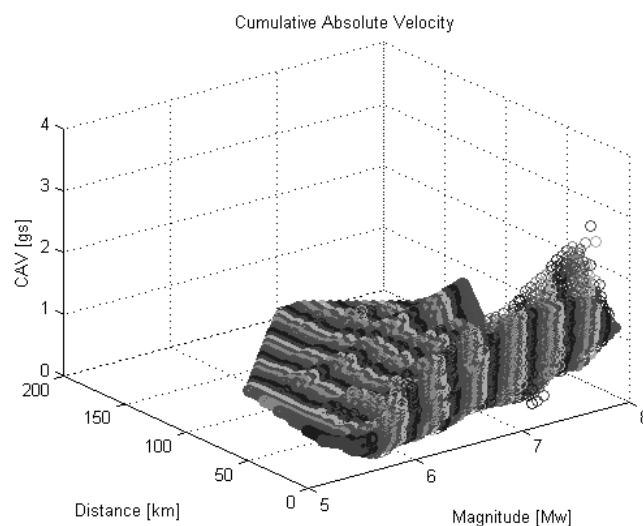
**Table 3. Regression coefficients and standard deviation for the second set of equations (300 m/s < Vs < 500 m/s).**

Parameter	b1	b2	b3	b4	b5	h	$\sigma$
log(CAV)	-5.9266	1.3811	-2.0284	0.0036717	-0.26281	17.952	0.37226
log(pga)	-3.2922	0.73851	-1.4135	0.0036421	-0.17693	11.94 <sup>1</sup>	0.22051

Figure 1 and figure 2 (residual plot, absolute deviations between data points and equation (9)) indicate that the selected regression shape can be regarded as appropriate. The equation (9) with the parameters given in table 3 is abbreviated below as WUS-correlation 1.

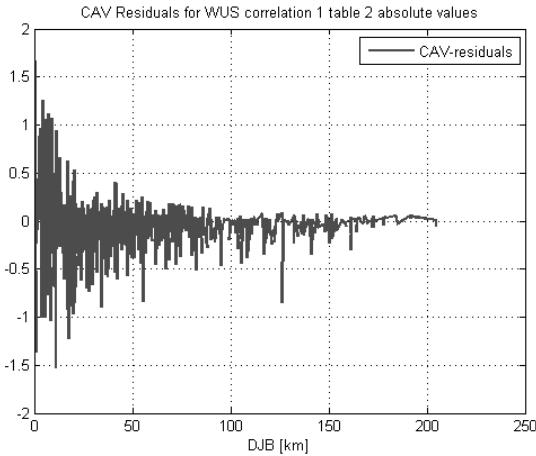
A second interpretation of the data given in (EPRI, 2005) representing Goesgen site conditions was developed following an idea of (EPRI, 2005) using uniform duration (Bolt, 1973) as a scaling parameter. A correlation between CAV, pga and the uniform duration of an earthquake can be suggested in analogy to a paper of Vanmarcke & Lai (1980), who established a correlation between the strong motion duration, the pga and the Arias-intensity. In this analogy, the uniform duration can be considered as equivalent to the strong motion duration according to Vanmarcke & Lai (1980).

**Figure 1. Plot of data derived from the WUS database (EPRI, 2005) in dependence of distance and magnitude**



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1. The regression resulted in a negative value for the “artificial focal depth” – the solution for this parameter is not unique because of the square root of the sum of squares in the equation. The positive branch of the solution was selected.

Figure 2. Plot of CAV residuals in dependence of distance from the WUS database (EPRI, 2005)



The approximate correlation of Vanmarcke for the strong motion duration is:

$$s_0 = 7.5 \frac{I_A}{pg a^2} \quad (11)$$

Here,  $I_A$  is the Arias-intensity and  $s_0$  the strong motion duration. The Arias-intensity is based on the square values of the acceleration time history, while CAV is based on the absolute values of the acceleration time history, so equation (11) indicates an almost linear dependence between CAV and the strong motion (or uniform) duration. Therefore, the following regression shapes appears to be meaningful:

$$CAV = a_1 * pga^{a_2} * d_u^{a_3} + a_4 \quad (12)$$

$$\log(d_u) = c_1 + c_2 M + c_3 \log(R) + \frac{c_5}{(1+pg a)} \quad (13)$$

Here,  $d_u$  is the uniform duration. Because the calculation of CAV is defined for time windows of length of 1s for a meaningful regression, it was necessary to consider only datasets with a uniform duration of a length of at least 1s in the regression for the uniform duration. Nevertheless, 1064 datasets were retained for the regression analysis. Tables 4 and 5 show the coefficients for equations (12) and (13), obtained by nonlinear regression. The results of the regression confirm the almost linear dependency between CAV and the uniform duration for a fixed pga-value ( $a_3$  is close to 1). Equations (12) and (13) are abbreviated below as WUS correlation 2.

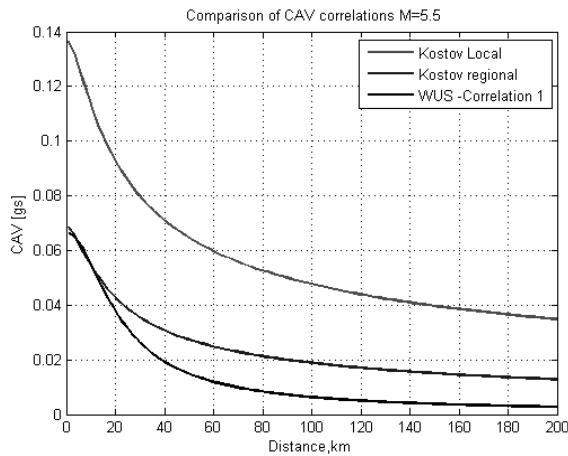
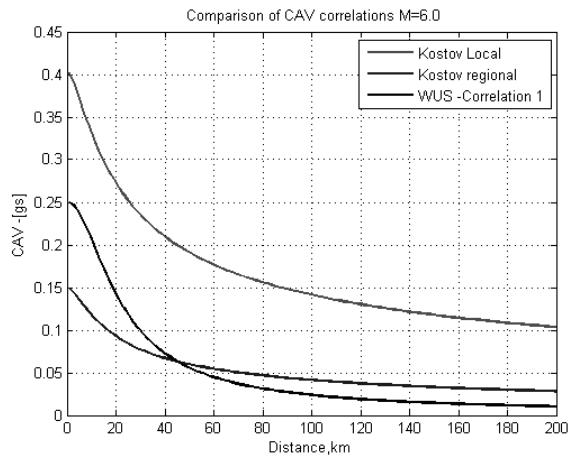
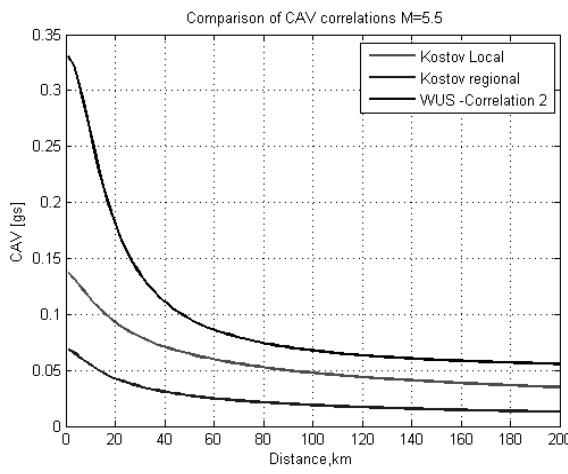
Table 4. Coefficients of the alternative regression equation (12) for CAV

Parameter	a1	a2	a3	a4	$\sigma$
CAV	0.13435	0.34454	0.96193	0.042448	0.060404

Table 5. Coefficients of the regression equation (13) for uniform duration

Parameter	c <sub>1</sub>	c <sub>2</sub>	c <sub>3</sub>	c <sub>4</sub>	c <sub>5</sub>	$\sigma$
d <sub>u</sub>	-0.099174	0.32521	-0.90306	14.992	-0.014763	0.2217

Figures 3 to 5 show a comparison of the different equations for the calculation of CAV. The comparison shows that some significant differences can be observed between the different equations. It is worth noting that equations (12) and (13) will result in some overestimation of CAV, because a larger amount of explanatory variables was used (and considered as uncorrelated in the regression). The multi-step regression procedure used increases the regression error and leads to a convolution effect (increased mean as a pay-off for reduced variability of the residuals).

**Figure 3. Comparison of Kostov's equations with WUS correlation 1 for magnitude 5.5****Figure 4. Comparison of Kostov's equations with WUS correlation 1 for magnitude 6.0****Figure 5. Comparison of Kostov's equations with WUS correlation 2 for magnitude 5.5**

The equations obtained were used to calculate the probability of exceedance for the damaging threshold of earthquakes as defined in NRC RG 1.166 (CAV > 0.16 gs). For the WUS-correlation 2, an additional intermediate step is required to consider the reduction of the datasets used by eliminating short strong

motion records (below 1s) statistically. Therefore, the procedure for calculating the exceedance probability of the damaging threshold had to be adjusted by weighting the exceedance of CAV > 0.16 gs with the probability that a uniform duration larger than 1s is observed.

$$P_{DL} = P_{CAV>0.16\text{gs}} P_{du \geq 1s} \quad (14)$$

Here,  $P_{DL}$  is the probability of exceedance of the damaging threshold,  $P_{CAV>0.16\text{gs}}$  is the probability of exceedance of CAV=0.16 gs obtained by the direct use of WUS correlation 2 and  $P_{du \geq 1s}$ . Simple linear approximations in dependence of magnitude were obtained for the different distance ranges D1, D2, D3.

#### 4. Development of the epistemic uncertainty distribution for the probability of exceedance of the damaging threshold of earthquakes

The exceedance probability of the damaging threshold of earthquakes was calculated for each of the developed equations, according to the bootstrap procedure discussed in section 2. Because the relative performance of the different equations may change in dependence of magnitude and distance (see for example figures 3 and 4), the epistemic uncertainty distribution was derived in the form of a lognormal distribution assigning the calculated maximal exceedance probability of the damaging threshold for each of the hazard probabilities (abbreviation for exceedance probability of a given hazard level) to the 95% quantile of the lognormal distribution and the minimal value to the 5% quantile. This leads to a conservative stretching of the resulting epistemic uncertainty distribution towards the upper and lower tails increasing the total amount of uncertainty considered in the model. Interpolation between the calculated values of the exceedance probability of the damaging threshold for the different hazard probabilities (which were considered in the hazard disaggregation) was performed using Hermit polynomials. This procedure was used to develop epistemic uncertainty distributions for each of the considered distance ranges (D1, D2, D3). On the basis of the distributions developed, a composite uncertainty distribution for the uniform hazard spectrum obtained from the PEGASOS-results was developed by weighting the distributions for the separate distance ranges with their relative contribution to the total hazard:

$$w_i(H, f) = \frac{P_{UHS,i}(a_{UHS,tot}|(H, f))}{H} \quad (15)$$

where  $w_i(H, f)$  is the weight for the distance range  $i$  associated with the hazard probability  $H$  (e. g. for  $10^{-4}/\text{a}$ ) and the spectral acceleration  $f$ , and  $P_{UHS,i}(a_{UHS,tot}|(H, f))$  is the exceedance probability of the spectral acceleration  $a$ , corresponding to the hazard probability  $H$  and the spectral frequency  $f$ . Figure 6 shows an example of the weights obtained by equation (15). Figure 7 shows an example of the resulting epistemic uncertainty distribution for distance range D1 (near site sources), while figure 8 shows the resulting composite epistemic uncertainty distributions for the exceedance probability of the damaging threshold of earthquakes for different spectral frequencies (mean values) with respect to the UHS.

Figure 6. Frequency dependent weighing factors according to equation (15), hazard probability 10<sup>-4</sup>/a

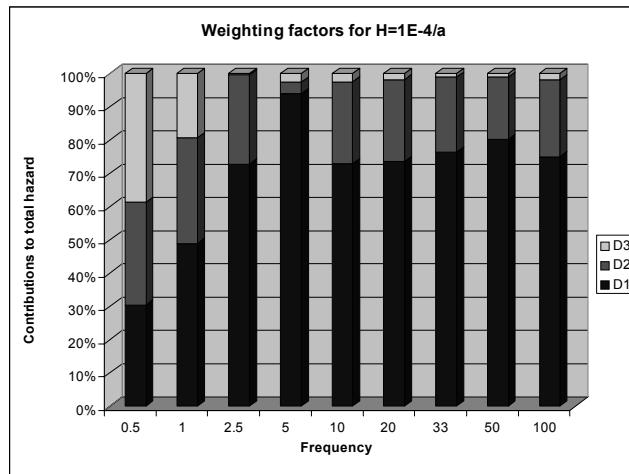


Figure 7. Composite epistemic uncertainty distribution for the probability of exceedance of the damaging threshold for distance range D1 (<16km)

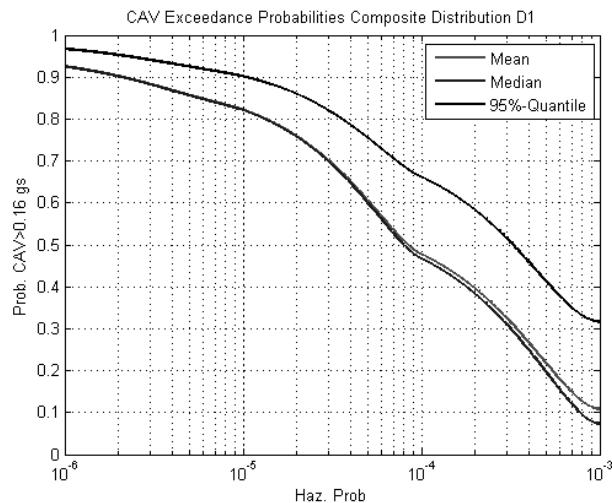
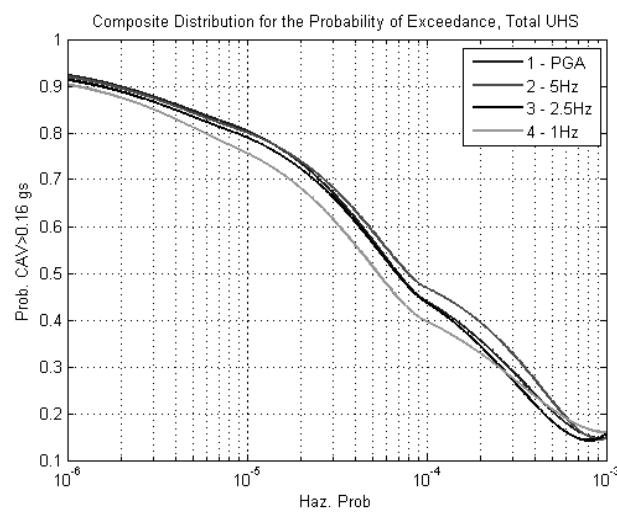


Figure 8. Frequency dependent probabilities of exceedance for the damaging threshold of earthquakes for the PEGASOS UHS for NPP Goesgen (mean values)



## 5. First application and conclusions

A detailed procedure for the elimination of non-damaging earthquakes from the results of a SSHAC level 4 PSHA study has been developed. The procedure takes full account of the existing and currently not fully reducible epistemic uncertainties with respect to both the uncertainty of the scenario earthquakes as well as to the uncertainty associated with the calculation of CAV. A key element of the procedure consists in the use of a correlated bivariate Monte Carlo bootstrap technique to address the epistemic uncertainties associated with the definition of scenario earthquakes and the use of multiple equations for the calculation of CAV. The results have been successfully implemented into the Goesgen specific seismic PRA model. The first sensitivity analysis performed indicates that a reduction of the mean core damage frequency of about 30% is to be expected, due to the implementation of the developed procedure. In another application, the current safe shutdown earthquake, defined as the mean hazard for a exceedance probability of  $10^{-4}/a$  (hazard probability) derived from a PSHA, was confirmed as appropriate for the design of the plant.

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## Seismic PSA Methodology for Multi-Unit Sites

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### 1. Introduction

The Nuclear Safety Commission of Japan (NSC) issued the Interim Safety Goals in December 2003 [1] and Performance Goals for Nuclear Power Plants in June, 2006 [2]. The performance goals require consideration of effects of external initiating events as well as internal initiating events, and require consideration of effects of multiple nuclear power plants for multi-unit sites. The draft of 10 CFR 53 (alternative to 10 CFR 50) by the US NRC requires also assessment of integrated site risks for new nuclear power plants to meet the NRC Quantitative Health Objectives as well as for individual plants. [3].

Multi-unit sites have probability of simultaneous multiple plant damages especially at seismic events. In such a case, site consequence could be larger than that from a single reactor plant accident. In order to analyze such situations, seismic PSA methods for multiple nuclear power reactors, up to 9 reactors were developed by the author [4]. This paper presents the essential models and discusses also the effects of accident management measures by mutual supports in multi-unit sites with sample analyses.

This study is by the author, not by the NSC.

### 2. Regulatory Requirements on Site Risks for Multiple Nuclear Reactor Site

The performance goals (subsidiary goals:  $CDF < 10^{-4}$  /year and  $CFF < 10^{-5}$ /year) for nuclear power plants by Nuclear Safety Commission of Japan require that risk assessment should include internal and external initiating events and the effects of multiple nuclear power plants in a site. Japan is earthquake-prone-country and seismic risks could be dominant. The requirements are applied to all nuclear reactor plants, including operating plants to meet the NSC safety goals. The document states that summation of risk metrics (CDF and CFF) of the multiple nuclear power plants in a site is acceptable and assessment by Level-3 PSA for the whole plants in a site against the QHO is also acceptable.

The draft document of the US NRC approaches alternative to 10 CFR 50 (10 CFR 53) [3] state as follows:

One question raised about the safety goals is whether they apply to a single unit or to the whole site, i.e. should the integral risk from multiple units meet the safety goals in the NRC. The documents for new framework of a risk-informed and performance-based alternative to 10 CFR Part 50 indicate that the newly proposed framework position is that the integrated risk posed by all new reactors at a single site should not exceed the risk expressed by the NRC QHOs as well as individual risk of each new reactor. It is stated that current PRAs are usually performed for a single unit or sometimes for two sister units, and the PRAs for modular reactor designs, which may operate multiple modular units at a site with a centralized control room, need to address potential interactions among the multiple units for both internal and external initiating events.

### 3. Seismic PSA methods for multiple nuclear power plants in a site

Multiple nuclear power plant sites have potentials of simultaneous multiple nuclear plant damages at external events, especially at earthquakes. Such seismic risks in multi-unit site could be dominant for nuclear power plant site in severe seismic area. The frequency of occurrence of severe accidents at a site per site-year is generally between the mean and the sum of all the plants.

The maximum number of nuclear reactors in one site is 8 in the world and 7 in Japan. Seismic PSA models for multiple units, up to 9 units, were developed and the models have been programmed as a computer code CORAL-reef. The objectives of the methods are to compute probability distribution of M simultaneous plant damages out of N nuclear power plants in a site and to obtain integrated site risks i.e. CDF and CFF per site year in addition to per reactor-year.

### **3.1 Seismic PSA methods for multiple nuclear power plants**

#### *(1) Integration of individual plant risks in a site*

It is generally known from existing detailed seismic PRA that certain limited numbers of key risk significant structures and components and accident sequences dominate seismic PRA results. To make integrated seismic PSA for as many as up to 9 units per site practically tactful and efficient, the models are based on risk information and risk insights available from detailed or screening PSAs for the representing reactor plants at the site. The risk significant components are usually identified in PSA by importance analysis using Fussel-Vesley Importance or Risk-Reduction values, etc. The key or dominant components and accident sequences are modeled and the other non-dominant sequences are lumped together as non-dominant residues, if desired.

#### *(2) Fault-tree linked monte carlo methods are adopted, since Monte Carlo approach is the best to deal correctly the correlation in complex multiple plant systems and components in parallel (AND) and in series (OR) configuration.*

#### *(3) Grouping reactor plants*

Twin units may have the same design and the same architecture built close each other generally in symmetric layout and may have some degree of correlation between the components, systems and structures between the two units. Such adjacent plants are grouped together and assumed to have the same design, configurations and characteristics from the point of seismic risk assessment. Since Monte Carlo computation can assign different values to seismic parameters, the identical reactor plants grouped may still behave differently from each other. On the other hand reactor plants with different design and architecture are assigned to different groups of similar plant designs. Plants in different groups are assumed basically to have no correlation with each other, except for some specific components or devices (such as shared systems) which may have correlation beyond the groupings. Up to 9 units in 3 groups are simulated at maximum.

#### *(4) Time-dependent analysis*

Short-term seismic safety is governed mainly by the seismic loads and the capacity or fragility of structures and equipments, whereas long-term seismic safety depends on non-seismic availability of continuous operation of safety systems necessary to maintain safety shutdown state.

#### *(5) Uncertainty analysis*

Seismic PSA has generally fairly large uncertainty mainly from hazard analysis because earthquakes have stochastic characteristics and accurate prediction of occurrence of earthquakes is difficult. As far as the seismic hazards can be assumed common to all reactors in a site, the uncertainty of hazard data may not be enlarged for integration of site risks. The important parameters specific to multi-unit seismic PSA are common-mode effects of earthquake i.e. correlation coefficients for structures and components across units. The realistic values of correlation coefficients of systems and components response and fragility, especially across reactors, have to be determined from the structural analysis, experiments and engineering judgment. The uncertainty of the correlation coefficients should be included in the uncertainty analysis.

### 3.2 Multivariate correlation models for Monte Carlo computation

Algorithms for computing correlation for Seismic PSA have been developed, such as in the SEISM code [5], and the SECOP-2 code [6][7], etc. Those models have been essentially developed to deal with correlation of components inside a plant.

In this study the following basic equations are developed to represent multivariate correlation for complex systems of multiple reactors by Monte Carlo approach:

$$S_i = \beta_i [\sum R_{Cij} \sqrt{\rho_{ij}} + R_i \sqrt{1 - \sum \rho_{ij}}]; \sum \rho_{ij} = <1 \quad \dots (1)$$

Where:

$S_i$  = deviation of response or fragility for component  $i$

$\beta_i$  = standard deviation of response or fragility of components  $i$

$\rho_{ij}$  = correlation coefficient for response and fragility between  $i$  and  $j$

$R_{Cij}$  = random number of log-normal probability distribution common to the  $i$  and  $j$  components.

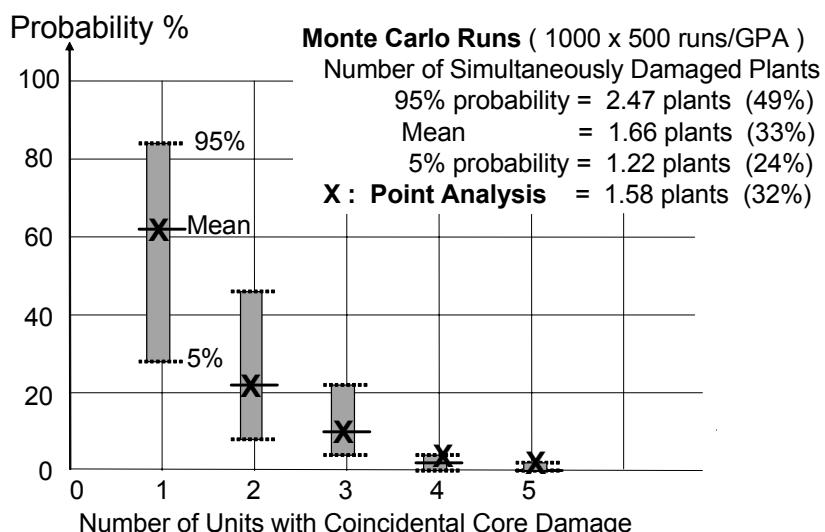
$R_i$  = random number of log-normal probability distribution of  $i$ -th component.

The adequacy and correctness of the equation (1) were ascertained by mathematical derivation and verified by sample computer analyses [4].

### 3.3 Sample analysis of 5 unit site

The result of a sample analysis of the probability of simultaneous multiple nuclear power reactor failures is shown in Figure 1. The mean number of reactors with simultaneous core damage is 1.66. The core damage frequency per site-year, which is defined as frequency of occurrence of core damage of any reactor in the site, is about 3.0 times the mean CDF per reactor- year in the case.

Figure 1. Probabilities of coincidental core damages



### 4. Safety design and accident management measures for multi-unit site – Mutual support

One of the advantages of multi-unit site may be the possibility of mutual supports at accident to prevent severe accidents. Effects of accident management by mutual supports are analyzed.

#### 4.1 Accident management measures

Most of nuclear power plants have provisions for accident management (called AM hereafters). The typical AM measures are Diverse Reactor Scram, Alternative Emergency Core Cooling Water Supply, Depressurization of Reactor Coolant System and Feed-and-Bleed, Alternative Containment Cooling, Containment Venting and Electrical Tie-lines between units, etc.

Those measures are usually designed for internal initiating events and utilize even non-safety non-seismically-qualified systems or equipments in the unit, which may not function at severe seismic events.

AM measures for seismic events had better use mutual supports of risk significant safety functions by seismically qualified preferred safety systems or equipments in the multiple units, such as:

- Electrical Tie between units.
- Ties of Emergency Feed-water Supply (i.e. CST) between units.
- Ties of Refueling water ( in RWST) between units.
- Ties of Service (or Sea) Water Systems between units.
- Etc.

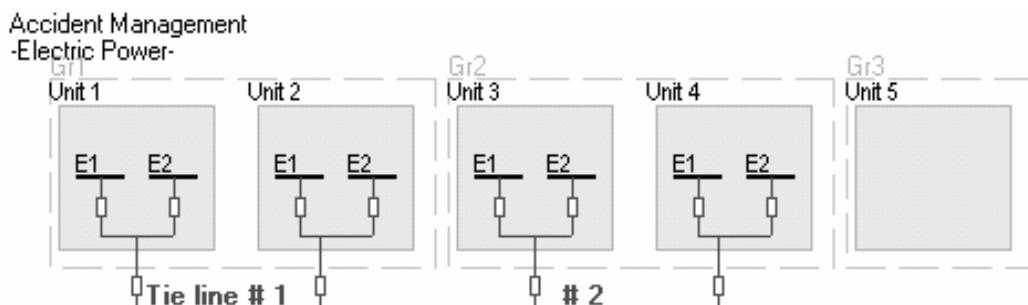
Seismically qualified shared independent devices such as shared emergency power sources are effective, but AM by mutual supports are analyzed in this study.

#### 4.2 Sample analysis

Sample analyses were conducted by the CORAL-reef code for an imaginary 5 unit (PWR) site to assess the effects of the following AM ties between units:

- 1) Electrical Buss-Ties between two units (see Figure 2).
- 2) Ties of Condensate Storage Water Tank Discharge Lines (CST).
- 3) Ties of Refueling Water Storage Tank Discharge Lines (RWST for ECCS water source).

Figure 2. Emergency buss ties between units



The results are shown in Table 1 and Figure 3.

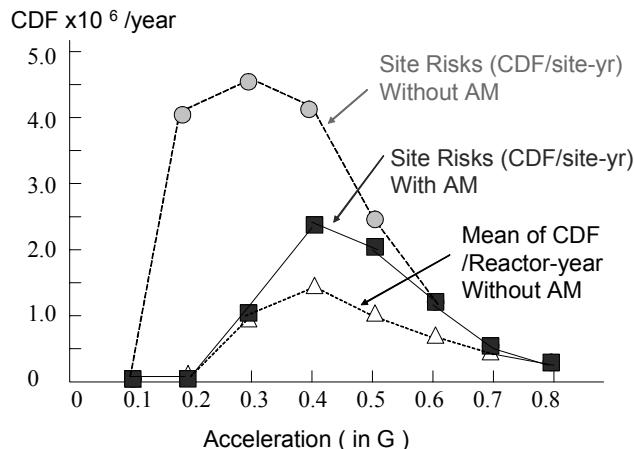
Table 1. Results of sample analysis for AM effects

Case No.	Accident Management	Units 1,2*	Unit 3-4**	Unit 5***	Mean	Site
1	None	0.94E-5	1.6E-6	1.2E-6	4.6E-6	1.40E-5
2	Ties of electrical buss	-1%	-3%	-	0%	0%
3	Ties of electrical buss, CST	-23%	-23%	-	-22%	-24%
4	Ties of electrical buss, CST, RWST	-42%	-30%	-	-38%	-47%

Note: \*: Old twin units (Plant designs and seismic parameters are taken from a PWR plant analyzed in NUREG-1150)

\*\*: Twin Units improved (HCLFP of fragility of dominant safety components > SSE)

\*\*\*: New plant (HCLFP of fragility of safety components > SSE, 4 trains, RWSP inside CV, etc. )

**Figure 3. Effect of accident management measures by ties between units**

Following are concluded from the sample analysis:

- 1) Effects of Electrical Ties by themselves are not large, probably because the nuclear reactor plants are already designed to withstand loss of all AC power for some period of time and fragility of CST and AFW pumps, etc. dominates the risks associated with loss of off-site power.
- 2) Combination of ties of electrical busses and CST discharge lines reduces CDF/reactor-yr by about 20%. The AM provision can increase residual heat removal capability.
- 3) Ties of three AM measures (Electrical Busses, CST and RWST) reduces CDF/reactor-year by about 30 to 40% and the integrated site risk (CDF/ site-year) by about 50%.  
The AM by ties of risk significant safety systems or components are more effective for lower acceleration ranges of earthquake motions as shown in Figure 3. This can practically eliminate risks from earthquakes below the design basis earthquake motions, which is quite important
- 4) Increase of site risks from increase of number of the units in a site can be compensated for by features of mutual supports to some extent.  
The ratio of CDF per site-year over CDF per reactor-year was about 2.6. The reduction of CDF per site-year by the mutual supports of the ties was about 50% as shown in Table 1. That means the site risk of CDF per site-year was reduced to 1.3 times the mean of CDF per reactor year without the AM for the case analyzed.

It should be noted that additional analyses showed the effects of the ties were negated (zero), when the correlation coefficients of the tied systems between two units were assumed 1 (complete correlation). The effects of the mutual supports depend on the correlation of the tied systems.

## 5. Risk metrics for integrated site risks

The current safety design and regulatory practice are primarily based on individual reactors and the risks are assessed in the term of reactor-year. It is however not reasonable that there are no provision for cooperative defense of safety for multiple nuclear power plants in a site, so that intact reactors support failed reactors.

New regulatory framework which requires consideration of integrated site risks will ensure site risks directly and encourage safety design and accident management measures by mutual supports among nuclear power plants in multi-unit sites. New risk metrics of CDF per site-year, LERF and CFF per site-year have to be looked at in addition to risk metrics per reactor-year.

## 6. Conclusion

Following are concluded by this study on seismic PSA for multi-unit sites:

- New policy that requires assessment of integral site risks for multi-unit sites in addition to individual reactor risk per reactor year is or will be introduced in Japan and in USA. This needs seismic PSA method for multi-unit site.
- The ratio of CDF/site over mean CDF/reactor may be less than the total number of the units in a site for seismic events. (probably in the range of 60% plus /minus 10%)
- Features of mutual supports e.g. ties of risk significant preferred safety systems or components among units might have possibility of reducing the site risks much especially for lower ranges of GPA of earthquake motions.
- New risk metrics in terms of per site year, such as CDF, LERF and CFF per site-year are recommended to be looked at in seismic PSA.
- Mutual supports among multiple nuclear power plants are expected to effectively improve site safety.

This paper presents the essential models for seismic PSA methods developed for multiple nuclear power plants in a site (CORAL-reef). The sample analysis, which was made for imaginary 5 unit site with AM mutual supports, shows the effects of reducing CDF per site-year, compensating for some of the increase of site risks from multi-unit site.

Use of seismic PSA methods for multi-unit site could assess the site risks more reasonably and make safety design more reasonable.

The approaches will contribute also to safety design of fleets of future advanced modular-type small reactors.

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## Japan: Accident Sequence Study for Seismic Event at the Multi-Unit Site

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### ***Abstract***

One or more units of a multi-unit nuclear power plant (NPP) could fail simultaneously at seismic event depending on the seismic ground motion and its influence to units of the site. The approach proposed here is to analyze the multi-unit accident sequences with core damage frequency (CDF) at seismic event explicitly by best applying and improving the existing technology and providing interfacing capability of the Level 2 and 3 parts of PSA.

The identified accident sequence at each unit, the end state of which could be failure (damaged core) as well as success (intact core), is linked mutually and a set of these sequences are conditionally quantified. If all potential accident sequences would be conditioned mutually, quite a number of accident sequences have to be analyzed. To circumvent such an unnecessary and quite resource-intensive burden, a screening process is effective and important for this approach. Therefore two-stage screening method was developed for the approach proposed here. The first and second stage screenings were applied to initiating events and accident sequences respectively. In these screening processes the correlation analysis of seismic-induced component failures was necessary and important, and was performed in use of representative response and capacity correlation factors, in application on which the floor response spectra were used for structures and components. The correlation analysis results were applied to quantify the concurrent seismic-induced failure probability.

The above-mentioned approach was applied to an example of twin-unit BWR5 site for verification purpose. The preliminary results revealed that the evaluation of correlation factors or concurrent seismic-induced failure probabilities affected significantly on the dominant accident sequences and therefore it is important to improve the development of the correlation factors as long as detailed consequence evaluation at a multi-unit site would be required in future application.

### **1. Introduction**

Progresses of methodology and data development for seismic probabilistic safety assessment (PSA) being endeavored by governmental organizations and utilities in Japan have advanced and increased quality of the seismic-induced risk analysis. It became obvious from internal and external event PSAs that the risk induced by earthquake is one of the most important concerns in Japan. The most of the existing methodology and data developed for seismic PSA are primarily intended for application to single unit NPPs. The seismic motion, however, affects on units more than one simultaneously depending the seismic event and its influence and the most of the Japanese NPPs are multi-unit. Need and its importance of an analytical capability of multi-unit NPPs have increased for the seismic risk with the progress of technology. The existing approaches [1,2] have contributed to this area: the approaches targeted and enabled the analysis of the core damage frequencies (CDFs) at a multi-unit site.

The present work deals with improvement of analytical capability including interfacing function of Level 2 and 3 seismic PSA, application and verification of the methodology, and identification of important issues to be addressed and resolved through application study[3].

## 2. Methodology

The fundamental and key elements of the methodology are analytical capabilities and required data for Seismic Hazard, Fragility and Accident Sequence Quantification analyses. The essence of the technology and the provisions devised for analytical capabilities of multi-unit NPPs are described below. It should be noted that the provisions dedicated to multi-unit NPP analysis are: analysis for correlation of seismic-induced failures of structures and components among units, quantification of concurrent failure probability derived from correlation factors, pertinent screening to circumvent resources, and identification of each quantified accident sequence.

### 2.1 Seismic hazard analysis

A realistic evaluation of uncertainty is of critical importance in the seismic hazard analysis. Methodologies and data for seismic hazards have been investigated, developed and applied to domestic NPPs, taking account of the uncertainties and the various site characteristics considering all potential seismic sources that affected on the site. A lot of efforts have been made in the seismic hazard analysis area by NUPEC and JNES and the Ref. [4] presents most updated and integral information.

It has been recognized and stressed that the expert opinions are quite important for modeling and quantifying uncertainties included in the seismic hazard evaluation. These experts have much knowledge about the potential seismic sources of active faults, earthquakes along the boundary of oceanic plate and continental plate and the historical earthquake data, as well as empirical attenuation laws. A logic tree model, that reflected the elicitation and the integration of these expert opinions for the uncertain terms of hazard model, had been applied to develop the seismic hazard. The technical workshop for the logic tree model and its application [5] was held and its validity was discussed and recognized. The seismic hazard at a site, using the logic tree model, was applied here.

### 2.2 Fragility analysis

Failure probability distributions, to which best-estimate (BE) and specific response and capacity for the respective building or component were applied, were developed. Many fragility analysis reports have been published from NUPEC and JNES, and the Ref. [6] summarizes most recent and integral information.

For multi-unit NPP analysis the BE responses of specific buildings and components to the seismic ground motion at unit were analyzed, using the Monte Carlo response analysis method considering modeling uncertainties directly. The effect of adjacent buildings, and concurrent failure due to system and component interaction, i.e. response correlation, was considered through the development of the fragility.

The response correlation factors for important buildings and components [7] were analyzed according to the statistical analysis. Definition of the bivariate correlation factor of X1 and X2,  $\rho_{X1X2}$ , is,

$$\rho_{X1X2} = \frac{\text{cov}(X1, X2)}{\sigma_{X1} \cdot \sigma_{X2}} \quad (1)$$

where cov and  $\sigma$  are covariance and standard deviation respectively. Let R and S be realistic response and capacity with log-normal distribution respectively, and let the variable X be,

$$X = \ln\left(\frac{R}{S}\right) \quad (2)$$

Here X is a performance function with the normal distribution. When X>0, a component fails due to the seismic motion. Substituting X into Eq. (1) on the assumption that R and S are independent, that is,  $\rho_{RS} = 0$ ,

$$\rho = \frac{\beta_{R1}\beta_{R2}}{\sqrt{\beta_{R1}^2 + \beta_{S1}^2}\sqrt{\beta_{R2}^2 + \beta_{S2}^2}} = \rho_{R1R2} + \frac{\beta_{S1}\beta_{S2}}{\sqrt{\beta_{R1}^2 + \beta_{S1}^2}\sqrt{\beta_{R2}^2 + \beta_{S2}^2}}\rho_{S1S2} \quad (3)$$

where,

$\rho_{R1R2}$ ,  $\rho_{S1S2}$  = response and capacity correlation factor between components 1 and 2 respectively.

$\beta_{Ri}$ ,  $\beta_{Si}$  = lognormal standard deviations of the response and capacity of component, i, respectively.

Then the seismic-induced concurrent failure probability of components can be calculated from the following equation.

$$P(\alpha) = (2\pi)^{-1}(|\mathbf{V}|)^{-\frac{1}{2}} \int_{-\infty}^{u_j} \int_{-\infty}^{u_k} \exp\left\{-\frac{1}{2} \mathbf{p}^T(\alpha) \cdot \mathbf{V}^{-1} \cdot \mathbf{p}(\alpha)\right\} dx_j \cdot dx_k \quad (4)$$

Here  $\mathbf{V}$  is the symmetry matrix of the correlation factors derived from Eq. (3),  $\mathbf{p}$  is the column vector of seismic-induced failure probability of each component at the acceleration,  $\alpha$ , and the upper limit of integration range,  $u$ , can be defined by the response and capacity distribution parameters.

The uncertainties of the fragility were identified and classified by aleatory and epistemic ones that arise due to the random or stochastic nature of the event and due to the paucity of knowledge on the event respectively.  $\beta_{Ri}$  and  $\beta_{Si}$  above includes these aleatory and epistemic uncertainties.

### 2.3 Accident Sequence Analysis

Hierarchical event tree model, where initiating events were developed in order of consequence, was applied to identify and quantify the seismic-induced initiating events. Seismic-induced component failures that were able to induce an initiating event potentially were identified and modeled. Then each initiating event occurrence probability on condition of the seismic event was quantified. Except for the initiating events that induce the core damage directly, each initiating event was linked with a system event tree, where accident mitigation systems were developed and the end states showed the damaged or intact core. Minimal cutsets for each accident sequence were derived from the initiating event, system fault tree and event tree models through logical calculation, and were applied to quantify the accident sequence, i.e. the conditional core damage probability. The core damage frequency (CDF) was obtained through the integration of the seismic hazard and the conditional core damage probability as follows.

$$CDF_i = \int_{a_{min}}^{a_{max}} -\frac{dH(a)}{da} \cdot CDP_i(a) da \quad (5)$$

Here,  $CDP_i(a)$  is the conditional core damage probability of the accident sequence, i, at maximum seismic acceleration, a. And  $H(a)$  is the annual exceedance frequency where a seismic ground motion of a certain acceleration, a, on the bed rock will occur in any 1-year period. Total CDF for single unit can be derived from the summation of all CDFs [8].

$$CDF_{Total} = \sum_i CDF_i \quad (6)$$

The accident sequences both for with and without core damage cases have to be considered for individual unit of the multi-unit site. That is, an accident sequence at Unit 1 is conditional on any accident sequences

at Unit 2, 3, etc. inside the site. Here the  $i$ th accident sequence of unit 2 is conditional on the  $i$ th accident sequence of unit 1,  $AS_i(a; U_1)$ , and is shown as  $AS_i(a; U_2 | U_1)$ . The  $i$ th accident sequence that occurs concurrently at unit 1 and 2 is shown as:

$$AS_i(\text{Unit 1 and 2}) = AS_i(a; U_1) \cdot AS_i(a; U_2 | U_1) \quad (7)$$

Here each  $AS_i(a)$  consists of minimal cutsets to the failures of structures and components and the  $i$ th accident sequences of unit 1 and 2 may or may not be identical. Supposing that  $N$  units locate at the site, the  $i$ th accident sequence at all units is,

$$AS_i(\text{all units}) = AS_i(a; U_1) \cdot \prod_{k=2}^N AS_i(a; U_k | U_{k-1}) \quad (8)$$

Eq. (8) is the logical product, and the seismic-induced concurrent structure and component failures to initiating events and mitigating systems for all units have to be considered in the logical multiplication. Then, a set of minimal cutsets of  $AS_i(\text{all units})$  is able to be derived from Eq. (8).

When the conditional probabilities of the seismic-induced concurrent structure and component failures to initiating events and mitigating systems are calculated for multi-unit analysis, correlation factors to these failures from the Eq. (3) are applied, using response and capacity correlation factors

These accident sequences at multi-unit NPP include both core damaged and intact sequences. If all potential accident sequences would be conditioned mutually, a large number of accident sequences have to be analyzed. A screening process in this approach is important in order to avoid such unrealistic approach that requires quite large resources. Regarding analytical capability for multi-unit NPP two-stage screening was developed for the approach proposed here. The first and second stage screenings were applied to initiating events and accident sequences respectively. In these screening processes the correlation analysis of seismic-induced structure and component failures was necessary and important as above described, and was performed in use of representative response and capacity correlation factors for buildings, structures and components.

Applying the  $i$ th conditional accident sequence probability,  $ASP_i(\text{all units})$ , the core damage frequency of the  $i$ th accident sequence is derived from the following equation.

$$CDF_i = \int_{a_{\min}}^{a_{\max}} -\frac{dH(a)}{da} \cdot ASP_i(\text{all units}) da \quad (9)$$

Note that  $ASP_i(\text{all units})$  depends on seismic acceleration,  $a$ . The total CDF at multi-unit site is obtained from Eqs. (6) and (9).

For multi-unit NPP analysis the accident sequences at all units with a CDF are identified with  $AS_i(U_1)$  and  $AS_i(U_k | U_{k-1})$  ( $k=2, N$ ), that occur simultaneously, to interface with Level 2 and 3 PSA; i.e. the corresponding source terms with containment failure frequency will be able to be analyzed.

Plant design information and data are necessary to develop the hierarchical initiating event tree, system event trees and fault trees. Seismic-induced structure and component failures as well as random failures and human errors have to be included in these trees.

### 3. Sample analysis

A twin-unit site with BWR5 units was selected as an example.

Specific seismic hazard curve with uncertainty at the dual-unit site was developed according to the method and data described above. Seismic-induced failure probability distributions of the structures and components in cooperation with seismic hazard analysis were developed, being based on the capacity data and the response data depending on the acceleration of seismic ground motion. The seismic-induced concurrent failure probabilities for structures and components at each unit had been developed and were applied in the analysis too [8].

The responses to the loss of off-site (LOSP) events at both units were assumed to correlate perfectly; that is, the conditional LOSP probability at unit B was assumed to be 1.0 when LOSP occurred at unit A. The response correlation factor for structures and components with identical design, that contributed to the initiating event and were installed in the different building, e.g., the containment vessels at unit A and B, was assumed to be 0.8. When the design characteristics were different, e.g., the containment vessel at unit A and the reactor vessel at unit B, the assumed response correlation factor was 0.6. When structures and components were located outside building, the response correlation factor was assumed to be 0.8, independent of units. The response correlation factor between structures or components inside building and those outside building was assumed to be 0.0; that is, both were assumed to have no correlation. These response correlation factors have been analyzed and verified for structures and components installed in different buildings within a unit, and the applicability of these factors for the buildings at different units was discussed and concluded to be appropriate. Meanwhile the capacity correlation factor of  $\rho_{F1F2}$  for all correlated structures and components was assumed to 0.0.

Mitigation system models to the internal event PSA were applied, adding the seismic-induced structure and component failures. Random failure data except for seismic-induced failure data were based not on domestic database but on worldwide database. But the contribution of the seismic-induced failure was large enough to neglect the contribution of random failure, and then the applied model here was adequate to evaluate the seismic risk even if random failure data based on the domestic database will be applied in future.

A computer code that has been developed in NUPPEC (currently in JNES) was modified and applied to quantify the accident sequences at dual-unit site. The acceleration due to seismic ground motion ranged from 300 through 1,300 gal.

#### 4. Results

The preliminary analysis results revealed that the contribution of concurrent core damage accident sequence frequencies at both units was approx. 10% and that the contribution of combined accident sequences for damaged core at a unit and intact core at another unit was approx. 90%, as shown in Figure 1. The contribution of concurrent core damage sequence frequencies at both units was very small because earthquake attenuation was large enough that component random failures rather than seismic-induced structure and component failures dominated CDF as the characteristics of the applied BWR5 model. Then, the concurrent core damage sequences will be expected to contribute more and more if damping of soil is small and seismic ground motion affects on a site more strongly. The contribution of concurrent core damage sequences was, however, the larger as the seismic acceleration was the larger. Total CDF of the twin-unit site was approx. 1.8 times as large as that of single unit. The top rank sequences were as follows:

a. Core damage at individual unit

Seq. at Unit 1: LOSP occurred, being followed by loss of two emergency diesel generators (EDGs) due to the concurrent loss of fuel oil tanks outside building chiefly, and loss of the containment heat removal.

Seq. at Unit 2: LOSP occurred but the core remained intact.

b. (Simultaneous) Core damage at both units

Seq. at Unit 1: LOSP occurred, being followed by loss of all three EDGs (an EDG for High Pressure Core Spray System) due to the concurrent intake failure; i.e. the station blackout.

Seq. at Unit 2: Same with the accident sequence at Unit 1.

Top ten dominant sequences are shown in Table 1.

**Figure 1. Initiating event contribution to core damage frequency**

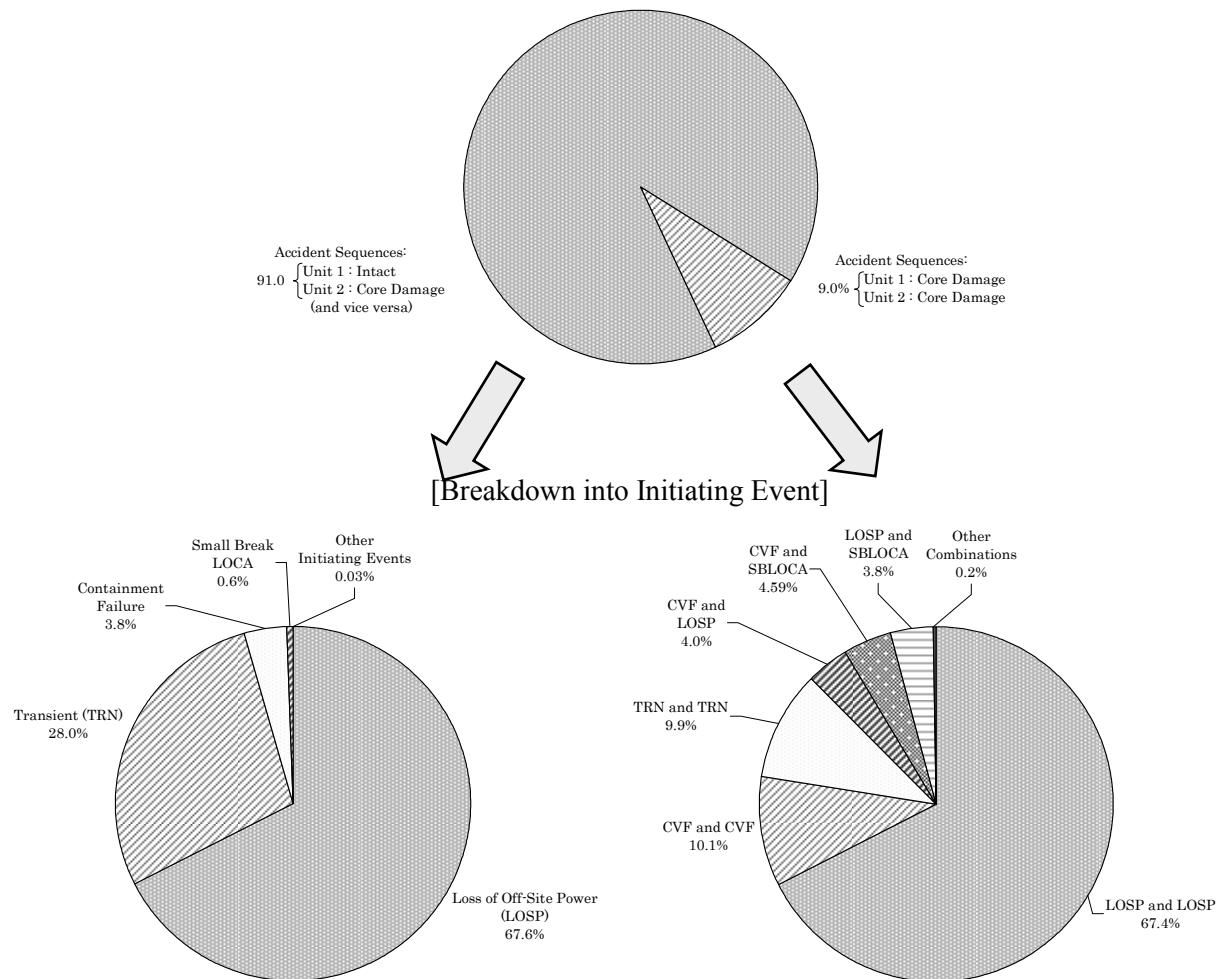


Table 1. Dominant accident sequences

Rank	Accident Sequence at Unit 1	Accident Sequence at Unit 2	Contribution (%)
1	LOSP*DG-A/B Failure*Loss of Recovery of Long-Term Offsite Power	LOSP (Intact Core)	15.4
2	Transient*Loss of Long-term Heat Removal	Transient (Intact Core)	15.3
3	LOSP*DG-A/B Failure*HPCS Failure*Loss of Recovery of Long-Term Offsite Power	LOSP (Intact Core)	12.6
4	LOSP* DG-A/B Failure*HPCS Failure*RCIC Failure	LOSP (Intact Core)	10.9
5	Transient*Loss of High and Low Pressure ECCSs	Transient (Intact Core)	7.9
6	LOSP*DG-A Failure* Loss of Recovery of Long-Term Offsite Power*Loss of Long-term Heat Removal	LOSP (Intact Core)	7.0
7	LOSP*DG-B Failure* Loss of Recovery of Long-Term Offsite Power*Loss of Long-term Heat Removal	LOSP (Intact Core)	6.7
8	LOSP*DG-B Failure*HPCS Failure*Loss of Recovery of Long-Term Offsite Power*Loss of Long-term Heat Removal	LOSP (Intact Core)	3.0
9	Seismic-induced Containment Failure	LOSP (Intact Core)	2.1
10	LOSP* DG-A/B Failure*HPCS Failure*RCIC Failure	LOSP* DG-A/B Failure*HPCS Failure*RCIC Failure	1.7
Total Contribution above			82.5

LOSP: Loss of Offsite Power; DG: Diesel Generator; HPCS: High Pressure Core Spray; RCIC: Reactor Core Isolation Cooling; ECCS: Emergency Core Cooling System

## 5. Summary

Core damage accident sequences at a multi-unit site were able to be identified and quantified along the conditional sequence linkage approach proposed and applied in this paper. The contribution of CDF at both damaged units was insignificant, depending strongly on site specific characteristics. These results will be able to interface with the part of Level 2 and 3 seismic PSA and will make it possible to analyze the seismic-induced fatality risk at a multi-unit site in precise, based on the identified and classified accident sequences for a multi-unit site.

The analysis results mainly focused on the coincident failure probability estimation for illustration purpose. Correlation factors for the structures, systems and components (SSCs) installed among buildings should be quantified and evaluated precisely, if a best estimate of plant damage configuration is necessary. These correlation factors for failures of SSCs among different plants enumerate:

- (1) The correlation factors among SSCs will be evaluated in detail but resource intensive. Preliminary results of the dominant contributors to CDF will help prioritization and selecting target SSCs; e.g., concurrent seismic-induced EDG failures discussed here.
- (2) Accompanying events of fire, flooding, slope failure and so on that could be induced by seismic event, have to be considered appropriately.
- (3) An aging effect has to be considered appropriately.

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## The Radiological Consequence Analysis by Evacuation Model for Earthquake Event of a BWR plant

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### **1. Introduction**

The MACCS2 code has been extensively applied to analyze radiological consequences for typical BWR and PWR plants in the Level 3 PSA program at Safety Analysis and Evaluation Division (SAED) of Japan Nuclear Energy Safety Organization (JNES) [1][2].

The present study examined radiological consequences for a typical BWR with a modified Mark-II containment under the seismic condition referring to results of Level 2 PSA for a typical BWR with Mark-II in Japan [3]. Source terms and frequencies of their release categories, which were presented with dominant plant damage states (PDS) and major containment failure modes, were used based on the results of Level 2 PSA [3]. Individual risk depends strongly on evacuation models.

In the present study, three evacuation models with different protection actions were considered to analyze radiological consequence (health effect in terms of early and late fatalities) referring to a domestic nuclear disaster drill, and the study in the United States [4].

### **2. Interface between Level 2 PSA and Level 3 PSA**

In the Level 2 PSA, all accident sequences leading to the containment failure were classified release categories with plant damage states, the containment failure modes and release pathways to the environment based on the Level 1 PSA results, consisting of core damage accident sequences for each earthquake acceleration. The typical accident sequence was selected among release categories, and the source term was calculated in terms of radionuclides release to the environment, character and condition, radionuclides amount, release timing, release duration and release energy. Then the frequencies of release categories, calculated results of source term and multiplied with frequency of all accident sequences classified with release category each earthquake acceleration were transferred to Level 3 PSA.

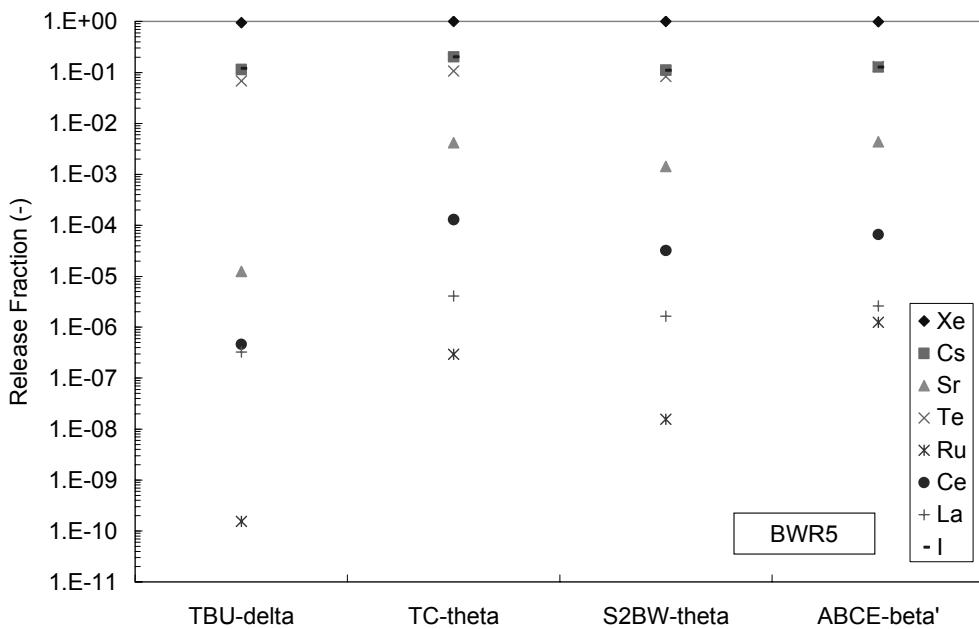
### **3. Radiological consequences analysis**

#### **3.1 Analytical conditions**

The source terms were determined for the seismic condition with the results of Level 2 PSA of typical BWR-5 plant (1,100MWe) with a modified Mark-II containment [3]. The analysis cases are shown in Table 1. The release fractions of radionuclides to the environment of each analysis case [3] are shown in Figure 1.

**Table 1. Analysis cases**

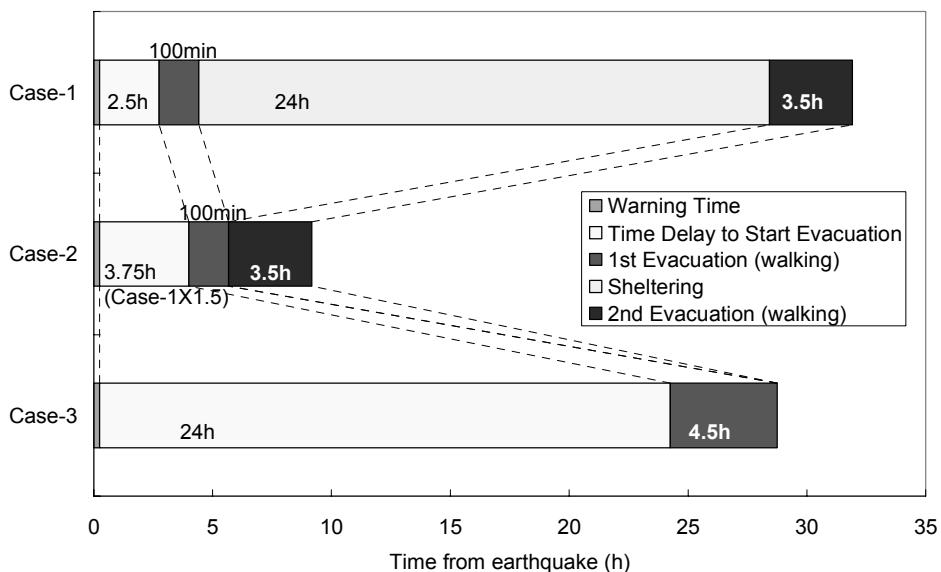
symbols	Containment failure modes
TBU-delta	Overpressure failure by steam and non-condensable gas accumulations at transient with loss of all AC and DC powers
TC-theta	Overpressure failure by steam accumulation at the transient without scram
S2BW-theta	Overpressure failure by the steam accumulation at the small LOCA with loss of residual heat removal system
ABCE-beta'	Direct failure by earthquake and large LOCA without scram

**Figure 1. Release fractions of radionuclide to the environment**

The protection actions in early phase were sheltering, evacuation and intake of tablets of potassium iodine. Intake of tablets of potassium iodine is effective in a competing situation to evacuate, but it was not considered in the present study.

Three evacuation models studied are depicted in Figure 2.

An emergency planning zone (EPZ) that is provided by the disaster prevention guideline is 8 – 10 km radius from the reactor site, but a 10km radius assumed in the present study.

**Figure 2. Evacuation models in the present study**

A declaration of “nuclear emergency” that is provided by the 15<sup>th</sup> article of the nuclear disaster special measures law was assumed to be issued at the event of the earthquake’s occurrence.

Case-1 is based on actual condition of evacuation at the earthquake, while the evacuation models of Case-2 and Case-3 are studied from the concept of NUREG/CR-4551 [4], but the delay time and evacuation velocity were assumed by referring the information of domestic nuclear disaster drill and actual condition of evacuation at the earthquake. No sheltering is assumed in these models.

In Case-1, the residents within the 10km radius from the reactor site start to move to evacuation places at an evacuation velocity (2km/h, on foot) after a delay time from the warning time. The residents are taking sheltering for 24 hours at the evacuation places. After sheltering, the residents start to evacuate outside of EPZ at evacuation velocity (2km/h, on foot).

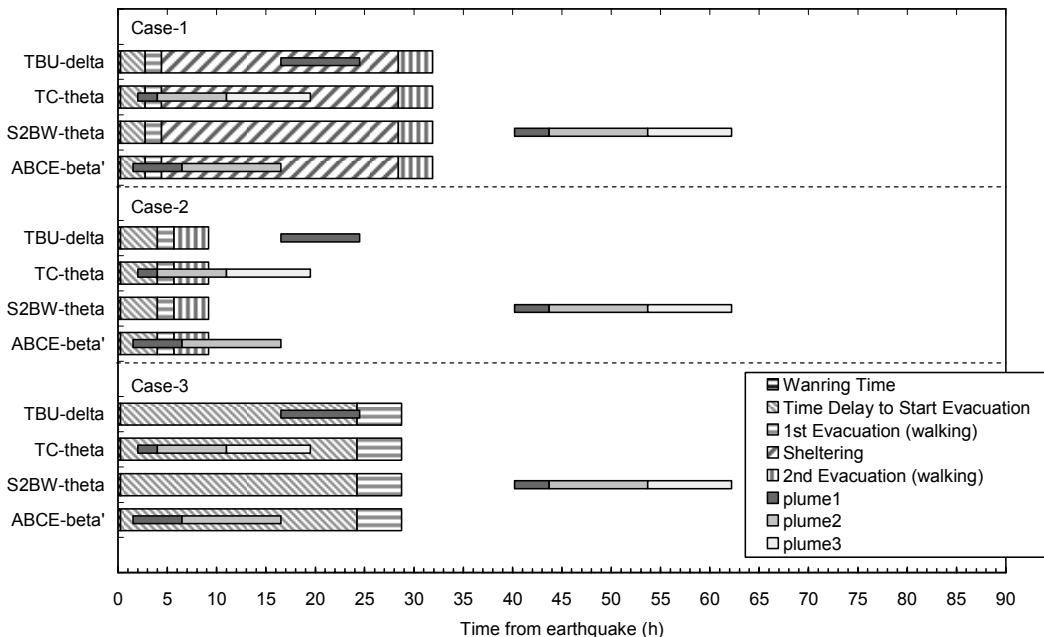
Case-2 is applied to the case with weak earthquake (PGAs less than 0.6g), and the delay time is 1.5 times longer the normal delay time and the evacuation velocity is half of the normal evacuation velocity (4km/h, on foot).

Case-3 is applied to the case with strong earthquake (PGAs greater than 0.6g), and the residents within the 10km radius stay in-place for 24 hours and then relocate.

All the evacuation models in the present study are assumed to move radially from the reactor site.

These evacuation models and timings of radionuclide releases to the environment of each analysis case are shown in Figure 3.

**Figure 3. Timings of evacuation initiation and radionuclide releases to the environment**



In TBU-delta, radionuclides start to release significantly with the containment failure caused by accumulation of steam and non-condensable gases after core damage. The radionuclide releases in TC-theta, S2BW-theta and ABCD-beta' start with reactor core damage. In the theta modes, however, the containment failures occur before core damage.

The release rate and duration of radionuclides of Level 2 PSA results [3] are simulated with a multiple plume model of MACCS2, but the tailing parts are neglected.

The time delay to start evacuation is determined by the time to receive warning, the time before leaving the office, the time to return home and the time before leaving home. It was based on the nuclear disaster drill in Japan.

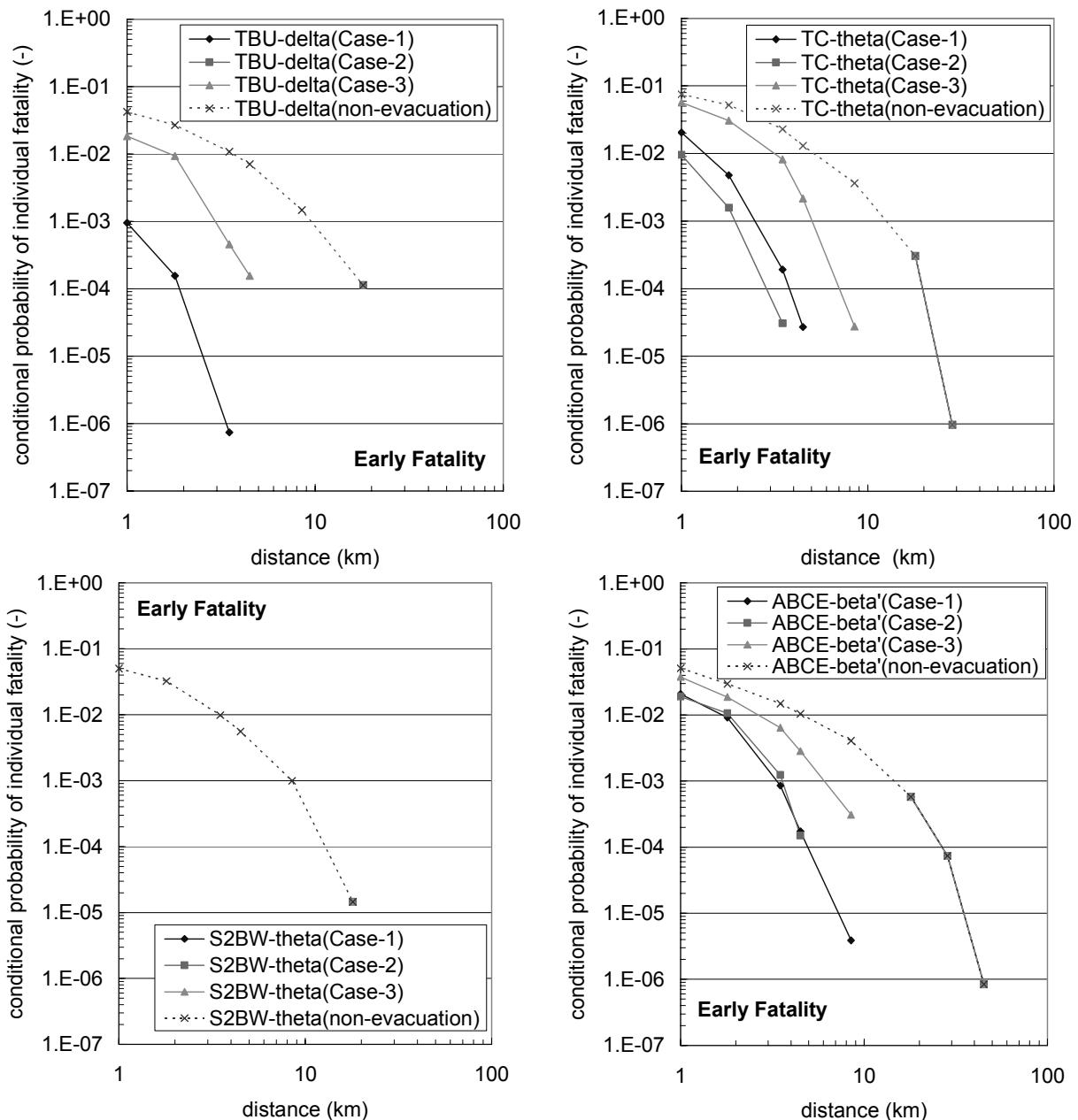
### 3.2 Reduction of Individual Fatality Probability by Evacuations

Among many parameters of radiological consequence as health injure and economic damage, conditional probabilities of individual fatality in the present study were chosen as an index to confirm reduction of radiological consequence by evacuation.

The conditional probability of individual fatality was defined by conditional probability of individual fatality in case of the occurrence of an accident.

The calculated results were shown in Figures 4 and 5.

**Figure 4. Conditional probability of individual fatality (Early fatality)**



**Figure 5. Conditional probability of individual fatality  
(Cancer fatality)**

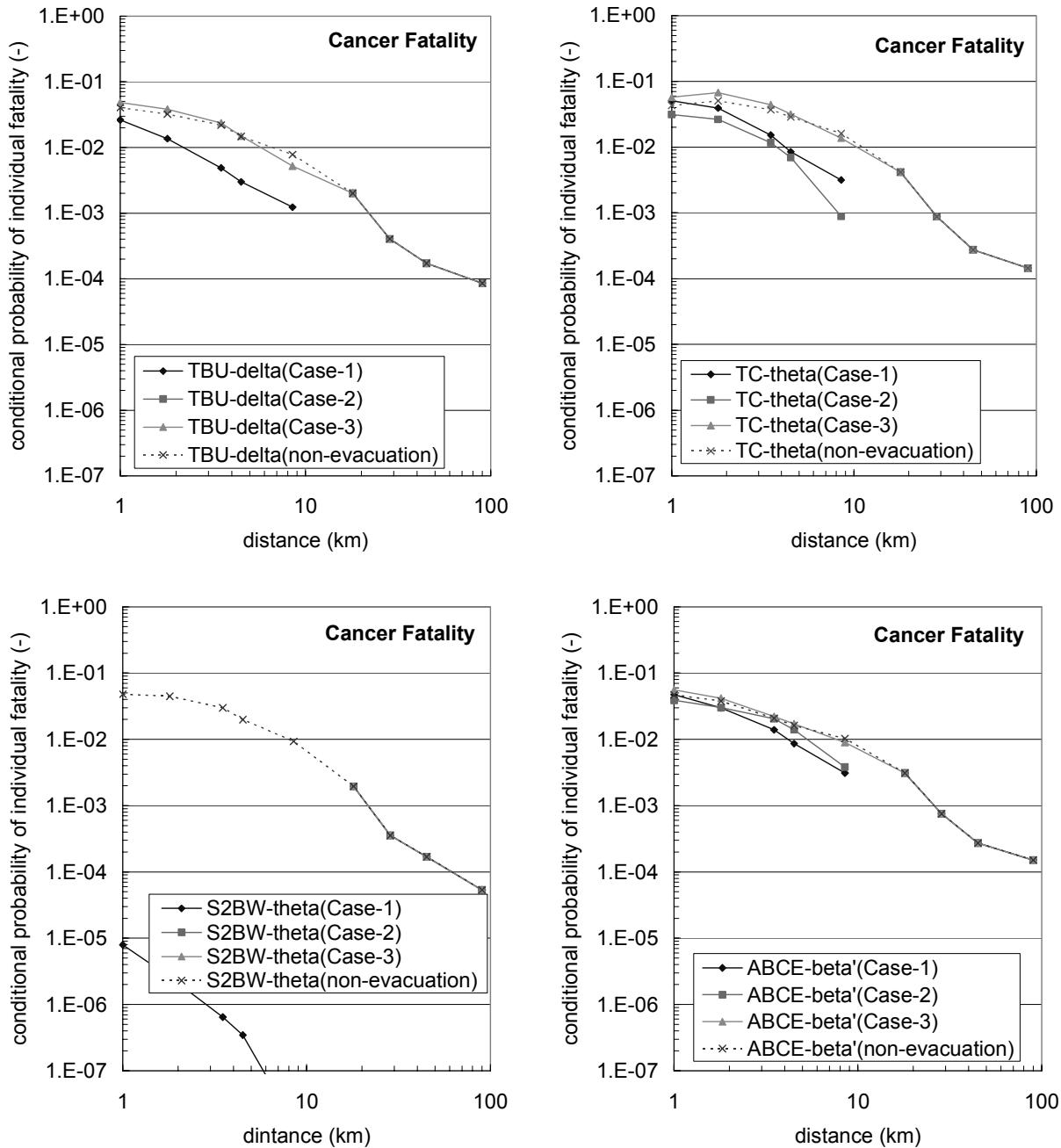


Figure 4 shows the conditional probability of individual fatality (Early fatality), and Figure 5 shows the conditional probability of individual fatality (Cancer fatality). The conditional probability of individual fatality at 1km form the reactor site was shown in Table 2. Among three evacuation models Case-2 became the highest reduction effect, while the Case-3 showed lowest.

**Table 2. Conditional probability of individual fatality at 1km from the site**

	Early Fatality				Cancer Fatality			
	non-evacuation (A)	Case-1 (Case-1/A)	Case-2 (Case-2/A)	Case-3 (Case-3/A)	non-evacuation (B)	Case-1 (Case-1/B)	Case-2 (Case-2/B)	Case-3 (Case-3/B)
TBU-delta	4E-02	1E-03 (0.02)	—	2E-02 (0.4)	4E-02	3E-02 (0.7)	—	5E-02 (1.2)
TC-theta	8E-02	2E-02 (0.3)	1E-02 (0.1)	6E-02 (0.8)	4E-02	5E-02 (1.2)	3E-02 (0.7)	6E-02 (1.3)
S2BW-theta	5E-02	—	—	—	5E-02	8E-06 (0.0002)	—	—
ABCE-beta'	5E-02	2E-02 (0.4)	2E-02 (0.4)	4E-02 (0.7)	5E-02	5E-02 (1)	4E-02 (0.8)	6E-02 (1.2)

In Case-1, the conditional probability of individual fatality (Early fatality) was reduced to 0 – 0.4 of the non-evacuation case. In addition, the conditional probability of individual fatality (Cancer fatality) became 0.0002 – 1.2 of the non-evacuation case.

In Case-2, the conditional probability of individual fatality (Early fatality) was reduced to 0 - 0.4 of the evacuation model. The conditional probability of individual fatality (Cancer fatality) was reduced to 0 - 0.8 of the non-evacuation model.

In Case-3, the conditional probability of individual fatality (Early fatality) was reduced to 0 - 0.8 of the evacuation model. The conditional probability of individual fatality (Cancer fatality) became 0 – 1.3 of the non-evacuation model.

Since the S2BW-theta case had enough time to evacuate in all evacuation models, the conditional probabilities of individual fatalities (Early fatality and Cancer fatality) became very small.

Considering Case-2, TBU-delta had also enough time to evacuate, so the conditional probabilities of individual fatalities (Early fatality and Cancer fatality) became negligible small.

However, since TC-theta and ABCE-beta' had not enough time to evacuate in Case-1 and Case-3 as shown in Figure 3, the reduction effect by evacuation became small.

#### 4. Summary

The present study focused on the analyses of radiological consequences for dominant accident sequences of earthquake events provided by the JNES Level 2 PSA for BWR plant with a modified Mark-II containment, and provided valuable insights regarding evacuation issues and its safety implication.

However, a continuous effort is needed for practical applications, because seismic events are fundamentally site specific nature. It includes the site specific evacuation model and organizational provision for emergency response activities against earthquake events.

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## Potential Uses of Seismic PSA for Risk Management of NPPs A Review of Recent Studies at JAEA

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### *Abstract*

Aiming at contributing to the discussions on the use of seismic PSA for risk management of nuclear power plants, this paper reviews three recent studies made at the Japan Atomic Energy Agency(JAEA) in the field of ageing management, accident management and emergency planning.

The first study is the development of component reliability analysis procedure considering seismic conditions conducted as part of the material degradation and ageing management research program. In order to contribute to ageing management strategies, JAEA developed a procedure that combines the probabilistic fracture mechanics analysis code with seismic hazard analysis. The second and third studies were performed as part of the research program on seismic risk management. The second study is on the core damage sequences and frequencies of two BWR units of same design located in the same site. This study indicated that the cross connection of emergency electric power between two units, which has been implemented as part of accident management procedures in Japan on the basis of internal event PSA, might be quite effective also for seismic conditions if credit can be taken on the fragility of connection system. The third study is on the damage to the road and buildings in the vicinity of an NPP caused by earthquake motions. This study suggested that, in emergency planning for NPPs, the reference earthquake motions for efficient estimation of the damages to the society could be determined from a level 1 seismic PSA as those of seismic motion intensity that contributes most to the core damage frequency.

This paper gives an over view of these studies and technical issues to be studied for extending the uses of seismic PSA for these new fields.

### **1. Introduction**

Methodologies of probabilistic safety assessment for events initiated by earthquakes (seismic PSA) were basically established in 1980s and have been applied to nuclear power plants (NPPs) over the world. However the use of results from seismic PSA seems to be rather limited as compared to the PSAs for internal events. In the recent years, application of PSAs for internal events is growing fast in many areas including, for example, risk-informed testing, inspections, technical specifications and decision making in safety regulations in general. On the other hand, most typical uses of seismic PSA seems to be in the area of identification of weak points and reducing risks by modifications in design of structures, systems, and components (SSCs) and operating procedures after earthquakes. Many examples in this area appeared in the summary report on the IPEEE program of the USNRC. One promising area suggested in the last OECD/NEA/CSNI workshop on seismic PSA held in Tokyo in 1999 was the risk-based seismic design [1]. Aiming at contributing to the discussions on the further use of seismic PSA, this paper reviews three recent studies made at the Japan Atomic Energy Agency(JAEA) in the field of ageing management, accident management and emergency planning, which have not been discussed extensively in the past.

## 2. Structural reliability evaluation of aged components in nuclear power plants based on probabilistic fracture mechanics (PFM)

### 2.1 Background and objectives

Older light water reactors (LWRs) in Japan have been operating for over 30 years. Measures need to be taken against decreases in the structural reliability induced by aging phenomena such as stress corrosion cracking (SCC). Furthermore, in the process of the revision of the seismic design guideline of the nuclear safety commission, it was recommended for the utilities to assess the risk of earthquake motions that exceeds the design basis earthquake motions by the use of appropriate method such as seismic PSA. In order to evaluate the structural reliability of aged components in LWRs, the probabilistic fracture mechanics (PFM) analysis is rational method and most suitable because the uncertainties in aging degradation and predicting seismic motion can be quantified.

### 2.2 Approach

In JAEA, we have developed a failure probability analysis method for aged piping based on PFM analysis and latest knowledge on aging, and a seismic hazard analysis method which takes into consideration the uncertainty in predicting seismic motion. Computer codes PASCAL-SC [3] for PFM analysis and SHEAT-FM [4] for hazard analysis was developed for these calculations. Although these methods were developed separately, we have recently established a structural reliability evaluation methodology merging them together.

In the first phase, the seismic hazard, i.e., occurrence frequencies of seismic motion of various intensities are evaluated based on the information on past seismic histories around LWR plants and the prediction values of seismic intensity (e.g. peak ground acceleration) at the plants. The seismic intensity is predicted by a fault model, made considering the break process of a fault and propagation characteristics inside the crusts. In the next phase, failure probabilities on aged piping for various seismic motions are calculated by the PFM analysis. In this analysis, SCC and fatigue crack extension by seismic load are calculated considering the scatter and uncertainties in crack initiation and growth rate, residual stress and material properties.

Failure probability is calculated with the random variables expressing uncertainty by a Monte Carlo method. The structural reliability of aged piping against seismic motion can be evaluated by multiplying the failure probability as a function of seismic intensities by the occurrence probabilities obtained from their frequencies.

Figure 1 shows the over-all flow of this procedure. Figure 2 is the procedure of seismic hazard analysis. Figure 3 shows the procedure of fragility evaluation.

Figure 1. Seismic reliability evaluation of aged piping

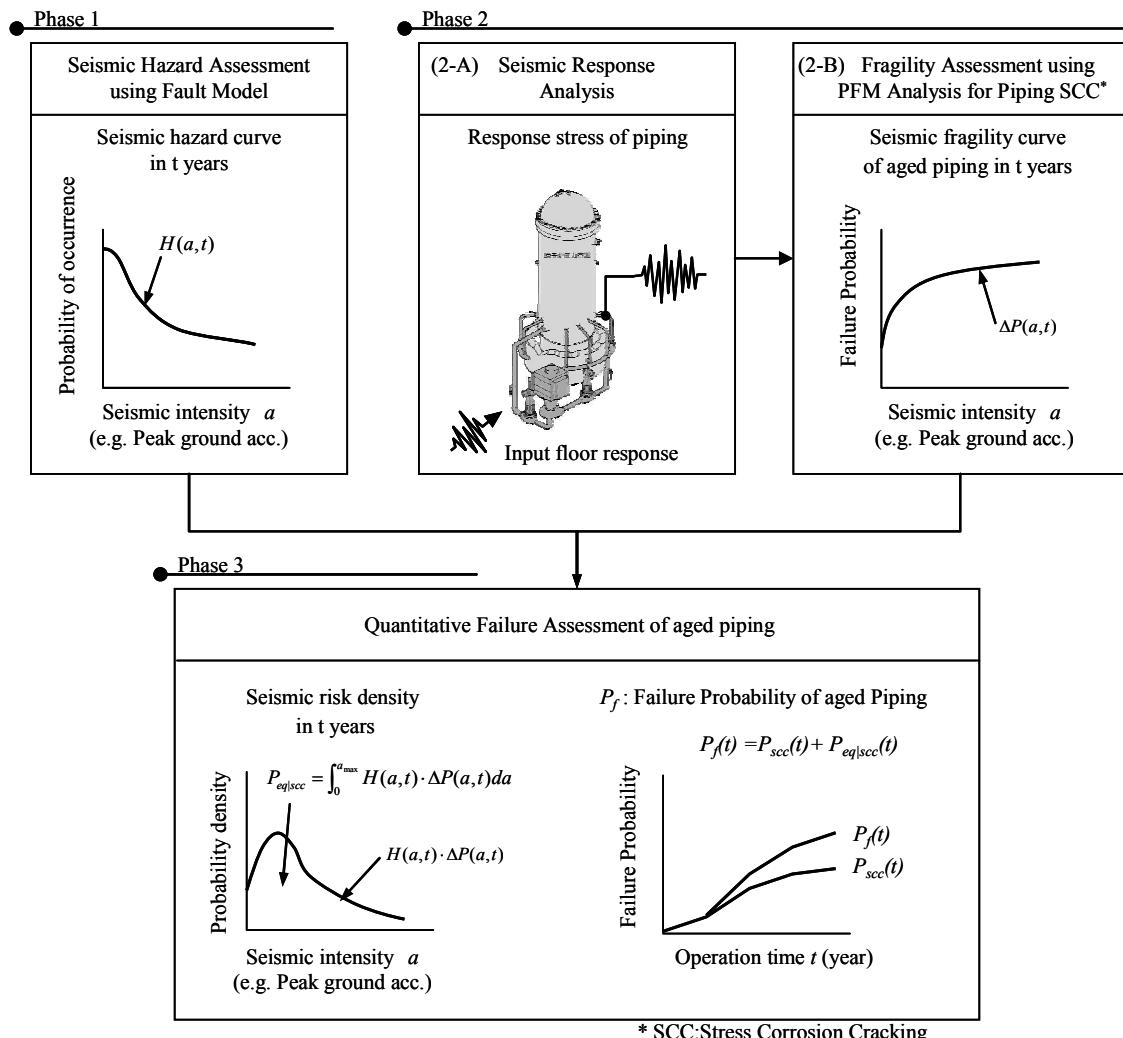
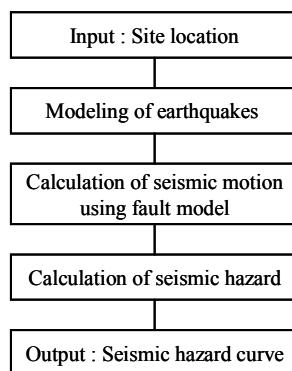
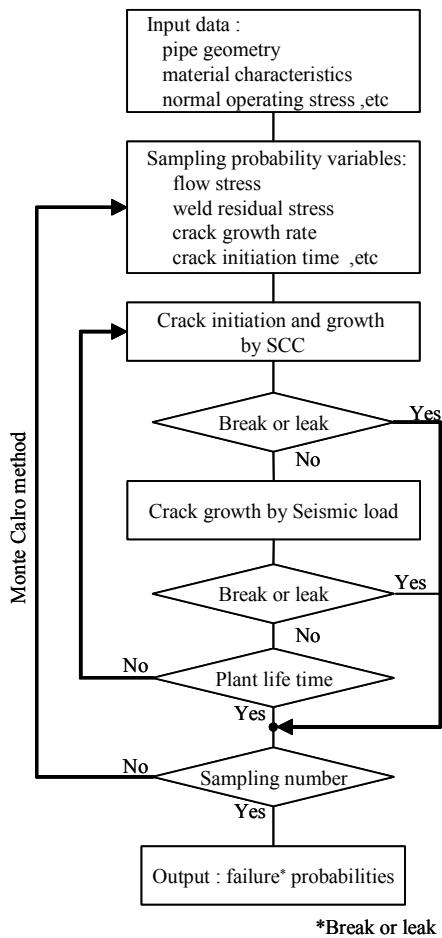


Figure 2. The procedure of seismic hazard evaluation



**Figure 3. Main flowchart of failure probability evaluation for pipping SCC by Monte Carlo method**



In the structural reliability evaluation of aged components under seismic motion, the prediction of the material degradation is important and this study made it possible by the development and application of a probabilistic fracture mechanics code.

### 2.3 Results

The usefulness of the above procedures was confirmed by an application to a specific component and the failure probabilities were predicted. A welded joint of a BWR primary recirculation line (PLR) pipe was taken as a test problem. Ageing phenomenon considered in this analysis is stress corrosion cracking. A series of time history analyses for soil, building, and piping provided seismic loading conditions for the PASCAL-SC code.

Some of the results are shown in Figures 4, 5, and 6. Figure 4 shows the fragility curves for seismic load at different ages obtained from PFM analyses. Figure 5 shows the seismic risk density as a function of seismic motion intensity at different ages. Figure 6 shows the failure probability of the welded joint by earthquake motions by 40, 50, and 60 years from the start of operation.

The details of the study have been published in reference [2].

Figure 4. Seismic fragility curves

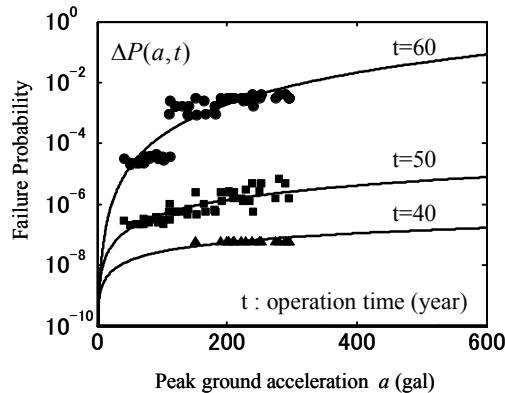


Figure 5. Seismic risk densities

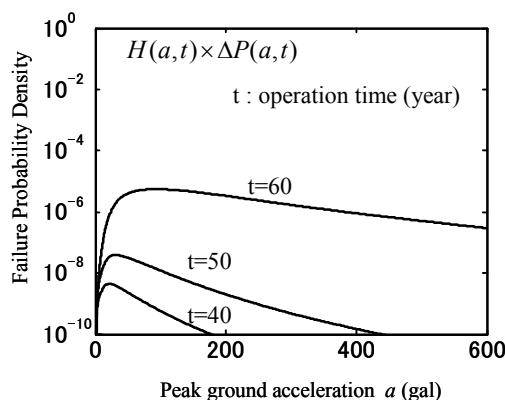
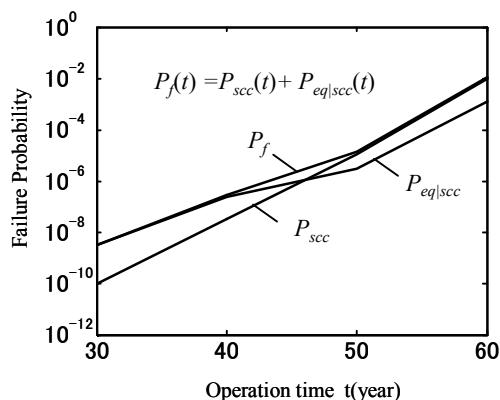


Figure 6. Example of failure probability of a welded



Joint of PLR piping using a BWR model plant.  
The point at  $t = 30$  was determined assuming.  
 $P_{sc} = 1 \times 10^{-10}$  and  $\Delta P(a) = 1 \times 10^{-10}$  (constant).

## 2.4 Technical issues that needs further study

Followings are areas that need further studies to realize effective uses of this type of analysis.

- Establishment of more practical evaluation methods for the effect and accuracy of in-service inspections.

- Development of logical frame work for the use of component reliability assessment. A potential approach is to establish quantitative acceptance criteria for the increase in component failure probability or contribution to core damage frequency that can be used for screening of components that must be considered in ageing management.

### **3. Accident management after earthquakes**

#### ***3.1 Background and objectives***

Japan Atomic Research Agency (JAEA, previously Japan Atomic Energy Research Institute (JAERI)) has been conducting the methodology development of seismic PSA since 1986. Included in the development are the SHEAT (Seismic Hazard Evaluation for Assessing the Threat to a Facility Site) code for seismic hazard analysis [5] and the SECOM2 (Seismic Core Melt Frequency Evaluation Code, version 2) code for systems reliability analysis[6]. The seismic PSA methodology and tools have been applied to a generic BWR plant (Model Plant) [7] and the applicability of this methodology has been confirmed. The results from this PSA (Model plant seismic PSA) were presented in the last OECD/NEA/CSNI workshop. Using the products from this development, JAEA started the research on seismic risk management in 2001, where trial applications of seismic PSA were conducted to propose effective ways of using seismic PSA for managing and reducing seismic risk of nuclear power plants. Later, in the discussions on the revision of the guidelines for seismic design evaluation of nuclear power reactors in Japan, the existence of residual risk and the importance of efforts to reduce such risks as low as reasonably practicable was pointed out.

Since there are generally more than one NPP units located in one site in Japan. It is necessary to perform seismic PSA on multiple units in the same site to understand the effect of earthquake on CDFs and core damage sequences of multiple units. If two units are completely independent from each other, the frequency of core damage in at least one unit will be 2 times of the CDF of a single one. However, in the case of earthquake, the union of the CDFs of the two units will be less than 2 times of the CDF of a single one because earthquake is a common cause event that cause simultaneous core damage of both units. In addition, the correlation of component failure will also have an effect on the union of the CDF of two units. The importance of correlation of component failure in seismic PSA has been recognized. In an earlier study it was pointed out that the difference between assuming complete correlation and complete independence could lead to an order of magnitude difference in CDF[8]. Though the effect of correlation of components on CDF may not be that significant[9], the effect of correlation of components on the CDF of multiple units needs to be investigated.

The objectives of this study was to understand the core damage sequences in two units and study the effectiveness of some accident management measures implemented in Japan. The cross connection of emergency diesel generators (EDGs) between adjacent units, which was introduced on the basis of internal event PSAs, was examined by a sensitivity calculation.

#### ***3.2 Approach***

Twin hypothetical units located in the same site were taken as examples and the effect of the correlation of component failure and the cross connection of EDGs on CDF and core damage sequences of these two units were analyzed using SECOM2. A unique feature of the SECOM2 code is the method of quantification of accident sequence frequencies. It uses the DQFM (Direct quantification of fault trees by Monte Carlo Method) Method, which allows exact treatment of the effect of correlations of failure if the set of correlation coefficients for responses and capacities are given and the sampling numbers are enough. This is difficult for usual MCS (minimul cut set) – based codes. The correlation coefficients for the base case calculation in this study were determined by the use of rules for assigning response correlations in NUREG-1150.

To study the effect of correlation as well as the effect of cross connection of EDSs on seismically induced CDF and core damage sequences, four cases were designed as shown in Table 2. Case 1a and 1b assumed no correlations and Case 2a and 2b assumed correlations as described in Table 1. No correlations were assumed for capacities. Case 2a and 2b assumed that cross connections will be successful if an EDG is intact.

**Table 1. Rules for assigning response correlation in NUREG-1150 program**

- 
1. Components on the same floor slab, and sensitive to the same special frequency range (i.e., zero period acceleration (ZPA), 5-10 Hz, or 10-15 Hz ) will be assigned response correlation coefficient 1.0
  2. Components on the same floor slab, sensitive to different ranges of spectral acceleration will be assigned response correlation coefficient 0.5.
  3. Components on different floor slabs (but in the same building) and sensitive to the same spectral frequency range (ZPA, 5-10Hz or 10-15Hz) will be assigned response correlation coefficient 0.75.
  4. Components on the ground surface (outside tanks, etc) shall be treated as if they were on the grade floor of an adjacent building.
  5. “Ganged” value configurations (either parallel or series) will have response correlation coefficient 1.0
  6. All other configurations will have response correlation equal to zero.
- 

**Table 2. Four cases analyzed in this study**

	Condition of correlation			Whether there is cross connection of EDGs or not?
	Correlation of responses in the same unit	Correlation of responses between different units	Correlation of capacity	
Case 1a	Independent	Independent	Independent	No
Case 1b	Independent	Independent	Independent	Yes
Case 2a	Rules of Nureg-1150	Rules of Nureg-1150*	Independent	No
Case 2b	Rules of Nureg-1150	Rules of Nureg-1150	Independent	Yes

\*: In NUREG-1150, the correlation of response of components in different buildings was defined to be 0. To be conservative, the correlation coefficient of components in the same building was applied to that of components in different buildings of the same type in this study.

### 3.3 Results

The results from the four cases are shown in Table 3. In the case of correlation of responses of components not considered, when the cross connection of EDGs between the two units was available, the CDF of a single unit was  $1.16 \times 10^{-5}$ /Reactor · Year. The frequency of core damage in at least one unit and the frequency of simultaneous core damage of both units were  $1.88 \times 10^{-5}$ /Reactor · Year, and  $4.33 \times 10^{-6}$ /Reactor Year, which were about 1.6 times and 0.4 times of the CDF for a single unit, respectively.

**Table 3. Frequency of core damage**

	CDF of a single unit (/Reactor · Year)	Frequency of core damage in at least one units (/Reactor · Year)	Frequency of simultaneous core damage of both units (/Reactor · Year)
Case 1a	$2.29 \times 10^{-5}$	$4.07 \times 10^{-5}$	$5.51 \times 10^{-6}$
Case 1b	$1.16 \times 10^{-5}$	$1.88 \times 10^{-5}$	$4.33 \times 10^{-6}$
Case 2a	$2.99 \times 10^{-5}$	$4.76 \times 10^{-5}$	$1.27 \times 10^{-5}$
Case 2b	$1.97 \times 10^{-5}$	$2.78 \times 10^{-5}$	$1.13 \times 10^{-5}$

In the case of correlation of responses of components being considered, when the cross connection of EDGs between the two units was available, the CDF of a single unit was  $1.97 \times 10^{-5}$ /Reactor · Year. The frequency of core damage in at least one unit and the frequency of simultaneous core damage of both units were  $2.78 \times 10^{-5}$ /Reactor · Year, and  $1.13 \times 10^{-5}$ /Reactor · Year, which were about 1.4 times and 0.6 times of the CDF for a single unit, respectively.

No matter correlation of responses of components was considered or not, when the cross connection of EDGs of the two units were available, the frequency of simultaneous core damage of both units was less than that with the cross connection of EDGs not available. However, the contribution ratio of the sequences that led to simultaneous core damage increased as correlation of responses increased. In addition, the frequency of core damage in at least one unit, the CDF of single unit and the frequency of simultaneous core damage of both units were decreased compared to those with the cross connection of EDGs not available. Further, the frequency of core damage in at least one unit with the cross connection of EDGs considered was lower than the CDF of a single unit. It was suggested that the cross connection of EDGs could be an effective way to decrease the frequency of core damage of multiple units and is worth detailed examination.

### ***3.4 Technical issues that needs further study***

The present analysis suggests that seismic PSAs will provide important information on the candidates of accident management measures that are potentially effective for reducing seismic risks. However in order to confirm the effectiveness the cross connection of EDGs studied here, procedures and capacities of the current design needs to be verified. Further studies on the validity of assumptions on the correlation coefficients are also desirable.

As for the planning on accident management strategies, following types of studies for specific plants would be necessary.

- Detailed studies on the accident progression and related systems and human actions for the core damage and containment failure sequences identified by seismic PSA.
- Systematic sensitivity calculations on the effectiveness of various candidates for accident management strategies.

## **4. Use of level 1 seismic PSA results for determination of representative earthquakes for emergency planning**

### ***4.1 Background and Objectives***

The third study was performed in the same program as the second, the research program on seismic risk management. The objectives of this study was to obtain basic information on the damages to the road and buildings and human life in the vicinity of an NPP caused by earthquake motions. Such estimates would be useful for modeling development for the effectiveness of emergency actions in level 3 PSAs and for discussions on the application of seismic PSA in assistance to emergency planning.

### ***4.2 Approach***

This work consists of the following subtasks.

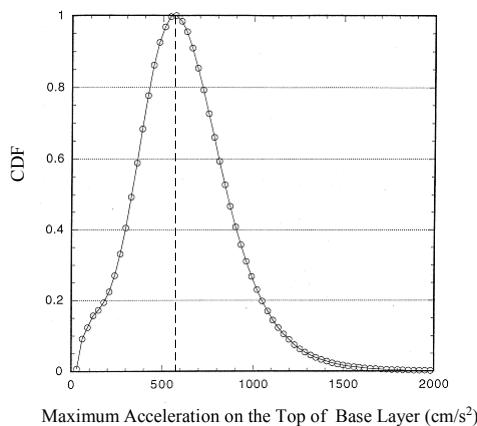
- (1) Proposal of approach for determining representative earthquakes for estimation of seismically caused damages to the society around the NPP site.
- (2) Seismic hazard analysis for a model site.
- (3) Survey of existing methodologies for estimating damages to the buildings, roads and human life caused by earthquakes and application to a model site.

As for the determination of reference earthquakes, it was proposed that the reference earthquakes should be selected from those seismic sources that give seismic motion intensity level at the NPP site contributing most to the core damage frequency of the NPP. This information is obtained from a level 1 seismic PSA. Using earthquake sources selected this way, the damages to the society around an NPP site were estimated.

#### **4.3 Results**

The Model plant seismic PSA performed at JAEA, which was used in the second study, was used for this study also. Figure 7 shows the density of the core damage frequency as a function of the intensity of the seismic motion (peak acceleration at bed rock). From this figure, the most important range of seismic motion intensity was determined to be around 570 Gal. The plant was assumed to be located at the Tokai establishment of JAEA.

**Figure7. Normalized CDF density as a function of peak acceleration at bedrock**



Several seismic sources that give the above intensity level were selected as reference earthquakes. References [10] to [19] were used for the estimation of damages to the society due to the earthquakes.

Results for one of the reference earthquake sources which was assumed to be 10 km under the site was as follows. This earthquakes caused the maximum velocity of 43cm/s on the top of the base layer, and 82 cm/s on the ground level, which was equivalent to the seismic intensity 6+ (6 upper).

Since the liquefaction of ground due to the earthquake can lead to damage of building and make the roads impassable, the potential for liquefaction was examined. It was shown that the risk of liquefaction of the vicinity of this site can be high and should be examined. As to the roads, depending on the width of the roads, the probability of traffic impassability was different. But generally it was high (more than 40%), which means that it might be unsuitable to use these roads for evacuation by car. As to the probability of bridge impassability, the probability of impassability of the bridge nearest to the model site was 33.0%. Its traffic might be impossible or be restricted.

In addition, the percentage of collapsed buildings was below 4%, the percentage of deaths and severely injured persons was below 0.2%. As to the percentage of the injured, it was about 15% of the total population.

#### **4.4 Technical issues that needs further study**

This study suggested that, in emergency planning for NPPs, the reference earthquake motions for efficient estimation of the damages to the society could be determined from the seismic PSA as those of seismic motion strength that contributes most to the core damage frequency.

It should be noted however that although this study showed the usefulness of level 1 seismic PSA for determining reference earthquakes, such results may depend strongly on the plant-specific fragility and that the estimated damages shown here are based on relatively simple models and needs further examination

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**Technical Session: Hazard Analysis**

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## Probabilistic seismic hazard assessment for OL3 plant site in Finland

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### **Abstract**

The main objective of this paper is to describe the Probabilistic Seismic Hazard Assessment methodology in order to investigate the overall seismic site characteristics to be followed in the construction phase Probabilistic Safety Assessment of Olkiluoto 3 Nuclear Power Plant in Finland. In addition, the paper describes the three major elements of a seismic Probabilistic Safety Assessment: probabilistic seismic hazard analysis (PSHA); seismic fragility analysis (SFA); and seismic systems analysis (SSA) including risk quantification. For each element, the methodology and software tools are described. For example, seismic hazard curves and ground motion spectra for OL3 will be developed and presented.

### **1. Introduction**

The purpose of the present work is the estimation of seismic hazard in the territory of the nuclear power plant Olkiluoto3 in Eurajoki.

Because there are no registered strong motion acceleration recordings of earthquakes in Finland, the earthquake recordings from Saguenay and Newcastle regions from Canada and Australia were taken as sources of initial data because of their geological and tectonical similarity to Fennoscandia.

The probabilistic seismic hazard assessment consists of three parts: 1) source effects, 2) path effects, 3) site effects. The site effects phase of the study is not relevant to this study because the prospective target sites are located on solid bedrock.

Theoretical bases of determination of seismic hazard, questions of seismicity of a southern part of Finland, initial data on earthquakes and techniques of their processing, are considered below.

### **2. Regional seismicity in Fennoscandia**

Finland is situated on the Baltic shield, which is the one of the seismically quietest areas in the world. According to the fault plane solutions of earthquakes the push from the North Atlantic Ridge in the NW-SE direction seems to be the major stress related to the seismicity of Finland. Other factors of the stress field, such as glacial rebound and local seismotectonics, are more local.

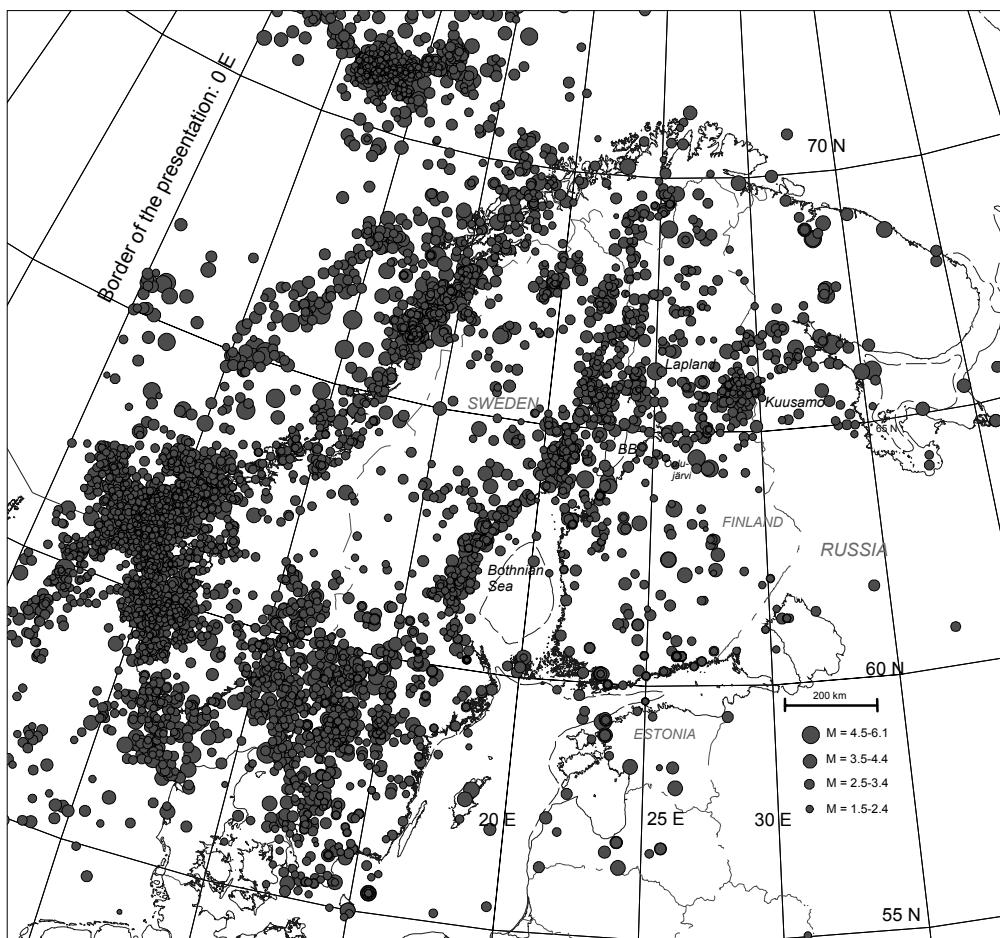
Earthquake recurrence rates in Fennoscandia are very low if compared with plate boundary regions worldwide. Nonetheless, Fennoscandia is an active seismic region, albeit at low earthquake recurrence rates and with relatively low magnitudes.

The earthquake catalogue for Northern Europe (FENCAT), maintained by the Institute of Seismology of the University of Helsinki, was used in this study [1]]. This catalogue encompasses the whole of Fennoscandia and adjacent areas inside a window of about 55-80°N and 10°W-45°E. The catalogue includes all documented earthquakes in the region since 1375.

Although instrumental earthquake observations started in Finland in the 1920s, local short period recordings started in 1956 [2]. The events in Finland and in Fennoscandia have been predominantly instrumentally located since the mid 1960s [2]. The instrumental magnitudes are based on the Richter's classical local magnitude scale,  $M_L$ , modified for the Fennoscandian region. The uncertainty of macroseismic magnitudes is assumed to be 10% at best [2].

Figure 2-1 shows that southern Finland, which is the target area of this study, is characterized by relatively low seismicity. The most active belts of seismicity close to it are the Swedish coast from the Bothnian Sea to the Bothnian Bay, western Lapland and the northern Bothnian Bay-Kuusamo region.

**Figure 2-1. Distribution of the earthquake epicentres in northern Europe since 1375 according to FENCAT**



### 3. Theoretical bases of probabilistic seismic hazard assessment (PSHA)

In the execution of the given work the seismic hazard assessment methodology described in reference [3] and [4] was used. The theory, on which the methodology of references 3 and 4 is based, has been developed for a number of years by Cornell [5] (1968) and Cornell [6] (1974) Merz and Cornell [7] (1973).

The PSHA theory as presented in reference 4 can be submitted in the basic form as the Theorem of total probability

$$\text{Equation 3-1} \quad P[A] = \iint P[A | S \text{ and } R] f_s(S) f_r(R) ds dr.$$

In Equation 3-1  $P$  indicates the specified probability,  $A$  is the event whose probability is sought,  $S$  and  $R$  are continuous independent random variables that have an effect on the value of  $A$ . In other words the probability that  $A$  occurs can be determined by multiplying the conditional probability of  $A$ , when  $s$  and  $r$  are given, times the independent probabilities of  $s$  and  $r$  and integrating over all investigated of  $s$  and  $r$ . Variables  $s$  and  $r$  represent earthquake size in used measure and distance from the site of interest. Random size and random location of the events are taken into account in the method. At an estimation of continuous probability of logarithm of ground acceleration exceeding some value  $i$  at the given station, the normal distribution assumption of reference [8]. The average size of continuous distribution of logarithm of ground acceleration is defined as

$$\text{Equation 3-2} \quad M_I(S, R) = C_1 + C_2 * S + C_3 * \ln(R + h),$$

In Equation 3-2  $C_1, C_2, C_3$  are constant factors and  $S$  is the size of earthquake and  $R$  is epicentral distance in km.  $h$  is depth of a seismic source in km. This equation is called the attenuation equation of ground motion. The standard deviation of logarithm of ground acceleration  $\sigma_I$  is generally constant and independent from  $S$  and  $R$ . Applying normal distribution and Equation 3-2, we have

$$\text{Equation 3-3} \quad P[A | S \text{ and } R] =$$

$$P[I > i | S \text{ and } R] = \Phi^*((i - C_1 - C_2 * S - C_3 * \ln(R + h)) / \sigma_I)$$

In Equation 3-3  $\Phi^*$  is the cumulative standard normal distribution. Characteristic ground motion variable is generally assumed to be log-normally distributed. Thus the logarithm of this variable is normally distributed. This normality assumption is fundamental to the methodology. The number  $N_m$  of earthquakes having magnitude greater than  $m$ , occurring in the source area is assumed to be of the form [9].

$$\text{Equation 3-4} \quad \log_{10} N_m = a - b m,$$

In Equation 3-4, parameters  $a$  and  $b$  are constants describing the seismicity of a particular source area. The constant  $b$  describes relative distribution of small and large magnitude events; larger values of  $b$  indicate relatively fewer large events, and vice versa. Assuming, that the sizes of consecutive earthquakes in the source area are independent, it follows from Equation 3-4 that the cumulative distribution magnitude for each event is given by the following equation

$$\text{Equation 3-5} \quad F_M(m) = k [1 - \exp(-\beta(m - m_0))]; m_0 < m < m_1,$$

In Equation 3-5,  $m_0$  is the lower-bound magnitude,  $m_1$  is the upper bound-magnitude, which can be developed from properties of the source area, constants  $\beta$  and  $k$  are given by

$$\text{Equation 3-6} \quad \beta = b * \ln 10; k = [1 - \exp(-\beta(m_1 - m_0))]^{-1}$$

From the Equation 3-5 follows that the density function on magnitude is given by

$$\text{Equation 3-7} \quad f_M(m) = \beta k * \exp(-\beta(m - m_0)); m_0 < m < m_1$$

By substituting Equation 3-3 and Equation 3-7 into Equation 3-7 and equating  $s$  to  $m$  the exceedance probability of logarithmic acceleration  $i$  is at the site is obtained as

$$\text{Equation 3-8} \quad P[I > i] = \iint_{R=m_0}^{m_1} \Phi((i - C_1 - C_2 * m - C_3 * \ln(R + h)) / \sigma_I) * \beta * k * \exp(-\beta * (m - m_0)) * f_R(r) dm dr$$

With the aid of some algebraic manipulation, the integration on magnitude in Equation 3-8 may be performed analytically in closed form and resulting equation will be as follows

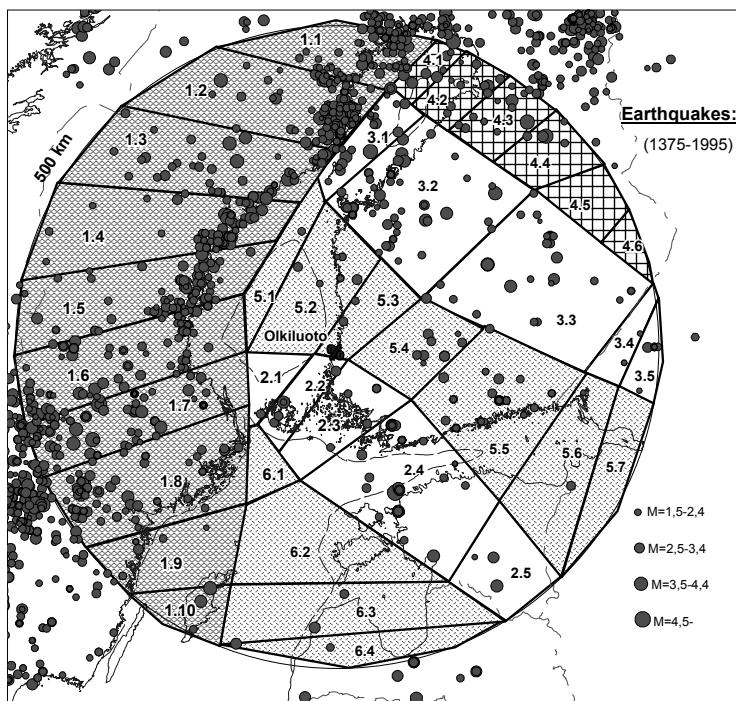
$$\text{Equation 3-9} \quad P[|>i] = \int_0^R \{(1-k)\Phi^*(z/\sigma_l) + k*\Phi^*(z'/\sigma_l) + \\ k*(r+h)^{\beta C_3/C_2} * \exp(-i\beta/C_2 + \beta*C_1/C_2 + \beta*m_0 + \beta^2\sigma_l^2/2/C_2^2) * [\Phi^*((z-\beta\sigma_l^2/C_2)/\sigma_l) - \Phi^*((z'-\\ \beta\sigma_l^2/C_2)/\sigma_l)]\} f_R(r) dr$$

In Equation 3-9 constants  $z$  and  $z'$  are defined in reference 7. The theory given in above formulas Equation 3-1- Equation 3-9 is only given as reference material. The actual numerical calculations are carried out with the aid of EQRISK-program [4] and with the aid of SEISRISK III -program [3]. In the work reported in this document reference [4] was only used for calibrating and verifying the input parameter values given to algorithm of reference [3]. This was carried out in the beginning of the work in order to validate the obtained results. Later on reference [3] was solely used to carry out the numerical computations reported in this document. In the following section the most important features of the algorithm used in reference [3] are described.

#### 4. Division of the source area to source zones and source area delineation for the olkiluoto site

The data of seismicity of territory around Olkiluoto with radius 500km are investigated. This source area is divided into six source zones presented in Figure 4-1.

Figure 4-1. The source zone division for Olkiluoto site embedded on the Fennoscandian epicentral for time window 1375-1995 and for magnitudes greater than 1.5. Schematic model for the program SEISRISK III [3]



Except the source zone 1 (eastern Sweden), the source division resembles division presented for the Loviisa data set. As mentioned, over 70 % of the events in the Olkiluoto data set are within the source zone 1. It is obviously completely different to the other seismic source zones of this study. However, the completeness analysis of the Finnish data seems to be valid in eastern Sweden as well. Therefore, the same time windows have been applied when the annual magnitude frequency relation was estimated. The b-value obtained (1.173) fit well the reference value for Sweden ( $b_6=1.26$ ).

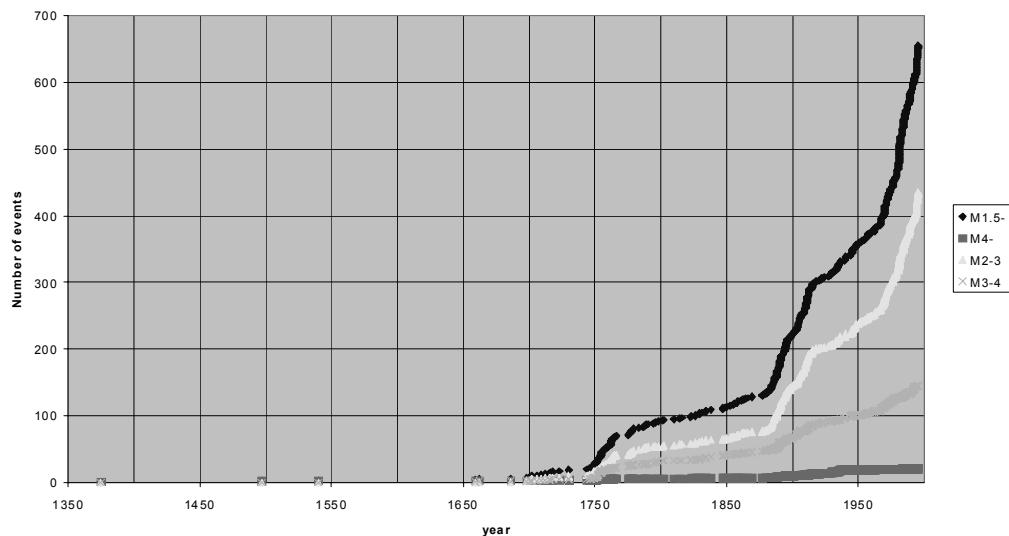
**Table 4-1. Seismicity parameters for the source zones of the Olkiluoto data set.**  
 Usually parameters  $a_{rel}$  and  $b_{rel}$  are scaled as for the Loviisa data set, but  $a_{rel}$  for Sweden is scaled to correspond the surface area of zone 1 and  $b_{rel} = b_6$ .  $M_{max}$  = observed maximum magnitude.

Source zone	b	a	$a_{rel}$	$M_{max}$
1. Eastern Sweden	1.173	3.460	3.642*	5.1
2. Å-P-P Zone	0.731	1.097	1.073	4.9
3. B-L Zone	0.782	1.459	1.192	4.6
4. Bothnian Bay - S. Kuusamo	0.829	1.121	0.810	4.7
5. SFQZ	1.166	2.065	1.230	3.2
6. Latvia	0.818**	1.033**	1.033	3.3

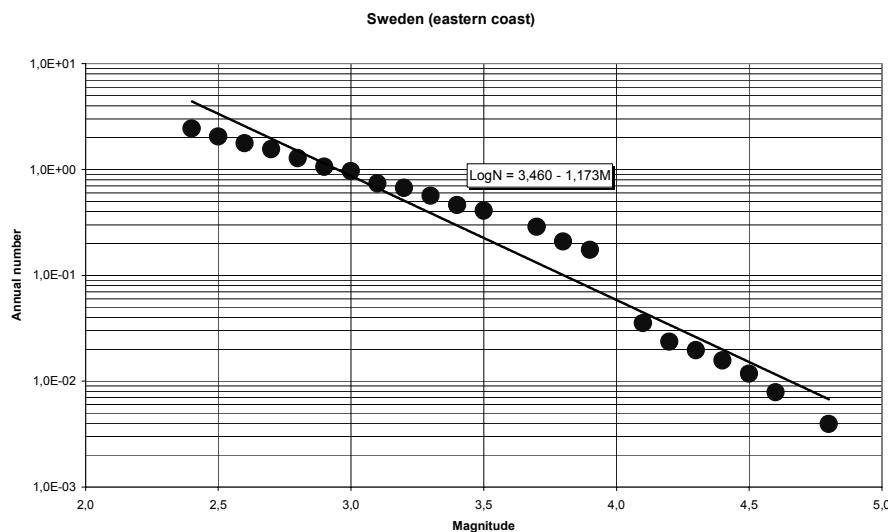
Note: \*: Kijko, A., Skordas, E., Wahlström R. and Mäntyniemi P., 1993. Maximum Likelihood Estimation of Seismic Hazard for Sweden, Natural Hazards, Vol. 7, 41-57.

\*\*: Lack of data:  $a = a_{rel}$ , and  $b = b_{rel}$

**Figure 4-2. Cumulative number of seismic events of the source zone 1 (eastern Sweden) in four groups:  $M \geq 1.5$ ,  $M = 2 - 3$ ,  $M = 3 - 4$  and  $M \geq 4$ .**



**Figure 4-3. The Richter magnitude-frequency diagram for eastern Sweden (source zone 1)**



## 5. Attenuation of ground acceleration

### 5.1 Development of the Dahle<sup>1</sup> attenuation model

The Dahle attenuation function form with minor modifications is adopted for the attenuation relationship used in this work. In the following sections the development of the functional form of Dahle attenuation relationship is explained.

The determination of significant site-source distance is important in probabilistic seismic hazard analysis. The hypo-central distance is therefore defined as

$$\text{Equation 5-1} \quad R = (d^2 + h^2)^{1/2}$$

In Equation 5-1 d is epicentral distance and h is depth of focus. The epi-central distances and the depths of focus are obtained from data set presented in Table 5-1 and Table 5-2. Surface wave magnitudes,  $M_s$ , were chosen for the magnitude definition for development of Dahle attenuation model. In Dahle model the maximum of the two horizontal components was selected to represent the ground motion parameters to be modelled. The following parametric expression for the size of motion for a harmonic wave in infinite elastic half-space is adopted for the regression problem:

$$\text{Equation 5-2} \quad A = A_0 / R^b \exp(aM + qR)$$

In Equation 5-2 A is the observed ground motion amplitude, R is the hypo-central distance, M is magnitude and  $A_0$ , a, b and q are constants to be determined. Linearization of Equation 5-2 then yields

$$\text{Equation 5-3} \quad \ln A = \ln A_0 + aM - b\ln R + qR$$

The Dahle attenuation model adopts a geometrical spreading model for ground motion cut-off in the vicinity of source as follows:

$$\text{Equation 5-4} \quad \begin{aligned} G(R, R_0) &= R^{-1} && \text{for } R < R_0 \\ G(R, R_0) &= R_0^{-1} (R_0/R)^{5/6} && \text{for } R > R_0 \end{aligned}$$

The recommended value for  $R_0$  is 100 km. as a distance at which the spherical spreading for S waves is overtaken by cylindrical spreading for Lg waves. Using the geometric spreading model adopted may conveniently be written as

$$\text{Equation 5-5} \quad \ln A - \ln G(R, R_0) = c_1 + c_2 M + c_4 R \\ \text{where } c_1, c_2 \text{ and } c_4 \text{ are coefficients to be determined}$$

The main reason why the Dahle attenuation model was not accepted as such in this study was large standard deviation values over twice the commonly used values. Second reason was that the spectral attenuations were developed for pseudo-velocity spectrum and not for acceleration response spectrum, which was needed for the purpose of current task. Thirdly, the frequency band used in Dahle model was not sufficiently wide for the purposes of current study. It could be expected that that spectral accelerations over 10 Hz would be of interest in developing the design spectra for geologic conditions where the bedrock outcrops were the expected level of grade at prospective site and the Precambrian rock had very high stiffness qualities.

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1. Dahle A., Bungum H., Kvamme L. B., Attenuation Models inferred from Intra-plate Earthquake Recordings, Earthquake Engineering and Structural Dynamics, Volume 19, 1125-1141(1990).

## 5.2 Development of ground motion model for Olkiluoto3 NPP site

On the basis of the work and considerations given in the Sections 5.1 and 5.2 ground motions were estimated from an attenuation relationships of the following form,

$$\text{Equation 5-6} \quad \begin{aligned} \ln(y) &= c_1 + c_2 M + c_3 \ln((R+h)^{1/2})^{-1} + c_4 ((R+h)^{1/2}) \quad \text{for } R < R_0 \\ \ln(y) &= c_1 + c_2 M + c_3 \ln((R+h)^{1/2}) + c_4 ((R+h)^{1/2}) \quad \text{for } R > R_0 \end{aligned}$$

In Equation 5-6,  $y$  is the strong motion parameter of interest;  $M$  is earthquake magnitude,  $R$  is the distance from the earthquake epicenter to the site;  $h$  is the depth of earthquake focus; and  $c_1$ ,  $c_2$ ,  $c_3$  and  $c_4$  are regionally dependent coefficients. This form is a mix of the Dahle model and a widely used model that is described in reference [4]. The  $R_0$  used in Equation 5-6 was chosen to be 100 km.

Attenuation of strong shaking is not only dependent on distance to the source and earthquake magnitude, but is also dependent on the type of earthquake source and of geologic site conditions as was explained in the Sections 5.1 and 5.2. In this study, the ground acceleration registrations used for evaluating coefficients  $c_1$ ,  $c_2$ ,  $c_3$  and  $c_4$  were selected from those geological and tectonic regions that were judged to be similar to the investigated area. The second principle for choosing these areas was the availability of registrations. By use this procedure the Saguenay region from Eastern Canada and the Newcastle region from Australia were chosen. These are both moderate seismicity, intra-plate regions and the registrations were observed on the bedrock. In case of Saguenay, the bedrock was of Precambrian formations, similar to Fennoscandia, but in case of Newcastle the rock formations were sedimentary rocks. This difference in rock formation was the weakness of Newcastle data in respect of its similarity to Fennoscandia, but in other respects also this area was similar to Fennoscandia. The reason for selecting these similar areas as the source of basic data for attenuation is that there are no strong motion acceleration recordings available from Fennoscandia.

In Fennoscandia, the earthquakes occur mainly at depths from 5 km to 20 km. The (80%) majority of the hypocenters are in the depth range of 10-20 km [1]. In spite of the small amount of shallow earthquakes they are important when seismic risk is concerned. Therefore the Newcastle events (depths <5 km) complete essentially the data of Eastern Canada with depths from 10 km to 30 km.

All magnitude scales are designed to be as compatible as possible with the original local magnitude ( $M_L$ ). Unfortunately, this is rarely the case. In addition, different regions have their own modified  $M_L$ -scale. In Fennoscandia, for example, the  $M_L$  estimations vary 0.1 - 0.5 unit between different regional formulae applied, on the average. In addition, in most of the local magnitude scales, the standard deviation of individual station magnitudes is about 0.2 units. The uncertainties related to magnitude estimates are taken into account by adding 0.1 and 0.5 units to the values of maximum magnitude estimated in Section 4.

For the determination of coefficients, which are included in Equation 5-6, the method of optimization described in reference [10] is used. The method implemented in the algorithm of reference [10] consists in the nonlinear curve-fitting problem that is solved with the aid of the least-squares method.

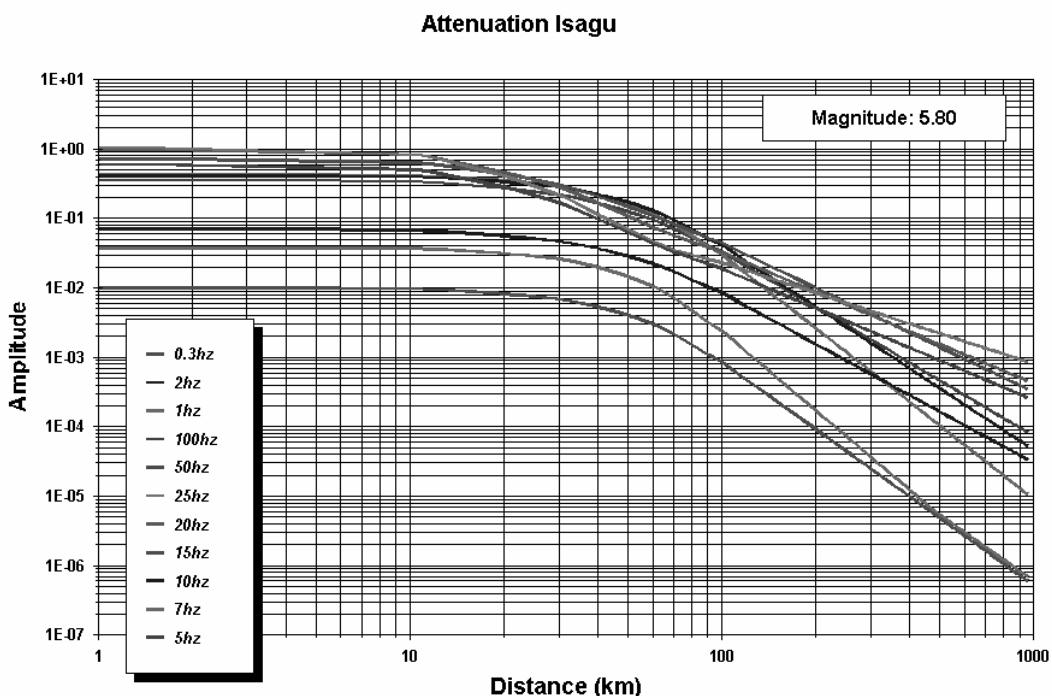
The initial data for Saguenay events consist of digitized, three component acceleration registrations for eleven Saguenay events, four Miramichi events and some additional registrations for Nahanni1 and Nahanni2 events. The digital SMA recordings for Newcastle events consist of four three component recordings. These events were converted to g units from Strong Motion Acceleration recordings by conversion coefficient described in reference [11]. Synthesized recording for Kelunji earthquake complemented the instrumentally recorded Newcastle events. The Kelunji synthesized recording was amplified to magnitude level of 5.6 from the instrumentally recorded event the magnitude 2.3 by methods described in the reference [12].

The processing of these materials was carried out by Microsoft Excel software. First, the acceleration histories were plotted on the basis of the recordings. From these plots we choose the ten-second time windows corresponding maximum acceleration peaks. From these acceleration plots the response spectra for 5% damping were computed with the aid of software from reference [13].

The spectral ordinates are calculated for longitudinal and transversal earthquake components. The investigated frequency values were 0.3, 1, 2, 5, 7, 10, 15, 20, 25 and 97 Hz. The magnitude value for the Saguenay event was 5.8 and the depth focus was 29 km.

The Kelunji synthesized recording was used for spectral calculations after the scaling of the magnitude to the value of 5.6 and the distance to the value of 10 km. The magnitude scaling was carried out according to the relations given in reference [14] and the distance scaling according to the relations given in reference [11]. The Miramichi recordings used for spectral calculations were scaled for same magnitude and same depth value as the recordings of Saguenay event with the aid of relations in the reference [14]. The Kelunji recordings were used to supplement the four Ellalong recordings that are called together the Newcastle recordings in this report because of their vicinity to town of Newcastle in New South Wales, Australia. The two recordings of the Miramichi event were used to supplement the ten Saguenay recordings that are called Saguenay recordings in this report after the name of the main event. In the following figures the attenuation relations for the logarithm of peak ground acceleration or various spectral accelerations are given as well as the attenuation for the peak ground acceleration ordinates. The figures are for longitudinal and transversal components of Saguenay and Newcastle event, which for the purposes of this study are regarded as independent recordings. For reliable determination of the coefficient  $c_2$  the magnitude data represented in Saguenay-Newcastle data set was too scarce. For that reason the  $c_2$  coefficient was determined on the basis of reference [15]. The attenuation curves for Saguenay longitudinal component are given next:

Figure 5-1. Spectral attenuation fit for Saguenay longitudinal component, Damping 5%, Magnitude 5.8



The attenuation is used as input for SEISRISK III [3] in table form. The parameter in these attenuation tables is the earthquake magnitude and the argument is the hypo-central distance R. All attenuation relationships are developed for solid bedrock and all site effects are excluded from this investigation because the feasible target sites are solid rock sites.

The depth of the source information is included in the Saguenay-Newcastle data set. The source depth varies from 1.4 to 29 km, which covers the range of expected source depths in the Olkiluoto3 earthquake catalog source area circles.

The total amount of acceleration records used in the study is 36. This number is considered quite adequate for the purposes of the study. The chosen records were compared to those used in Dahle study<sup>2</sup> and EPRI guidelines [16].

The aim in choosing the available intra-plate recordings was not obtain largest possible number but the best possible relevance to the target regions geological and seismological conditions: pre-cambrian solid rock, moderate seismicity about the same level to be expected in Fennoscandia, intra-plate conditions

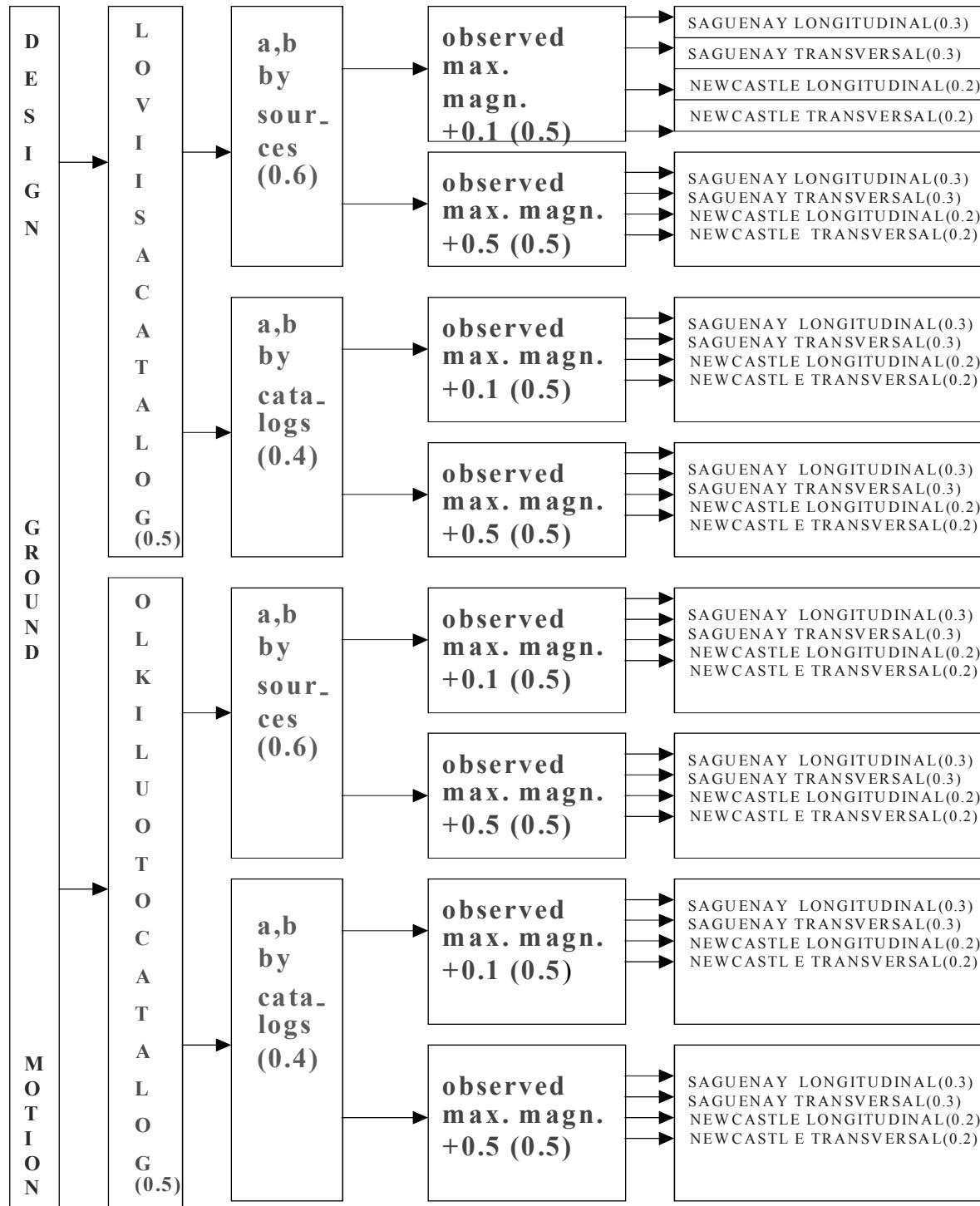
## **6. Decision tree for the treatment of uncertainties**

The code basis for the ground motion estimation in probabilistic seismic hazard studies stipulates the median spectra for mean return period of 100 000 years [17]. The decision three used in the treatment of uncertainties in this study was as follows:

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2. Dahle A., Bungum H., Kvamme L. B., Attenuation Models inferred from Intra-plate Earthquake Recordings, Earthquake Engineering and Structural Dynamics, Volume 19, 1125-1141(1990).

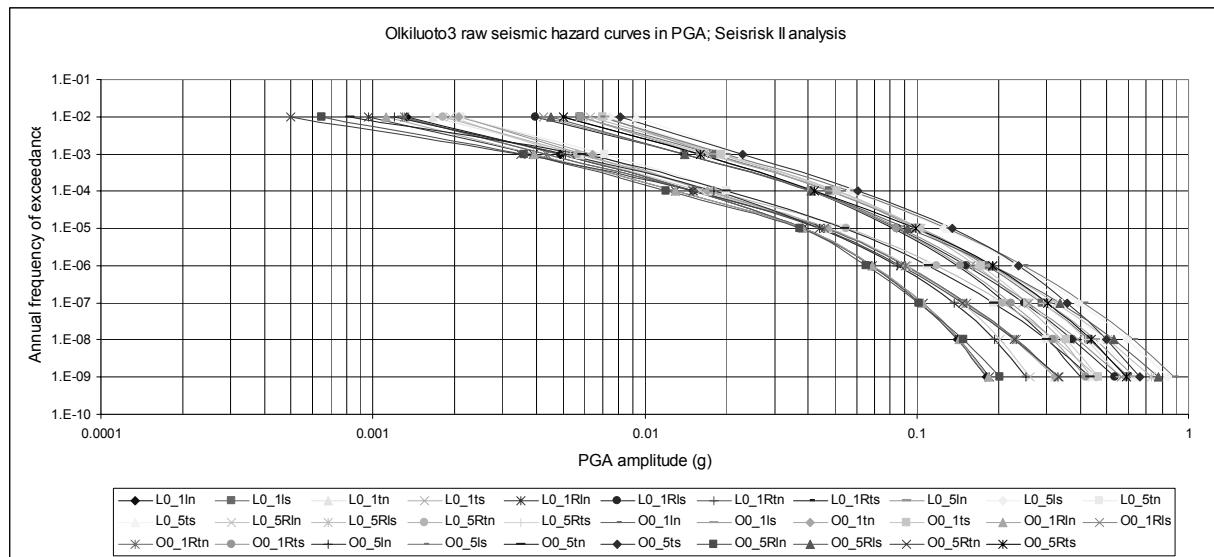
Figure 6-1. Logic tree structure for treating uncertainties



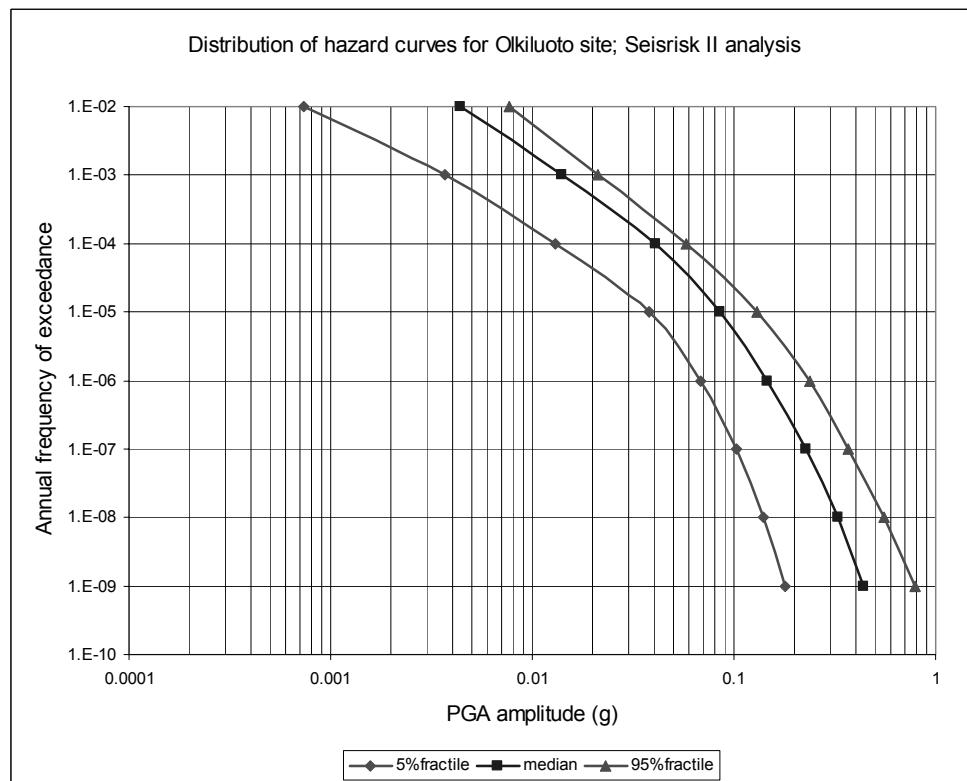
## 7. Results

By preparing the initial data according to the previous section, the further analysis was carried out on Fortran Computer Program for Seismic Risk Analysis described in reference [3]. The resulting raw site hazard curves as well as median hazard curve with 5% percentile and 95% percentile and ground response spectra for annual exceedance probability of 1.0E-5 are given in the following figures:

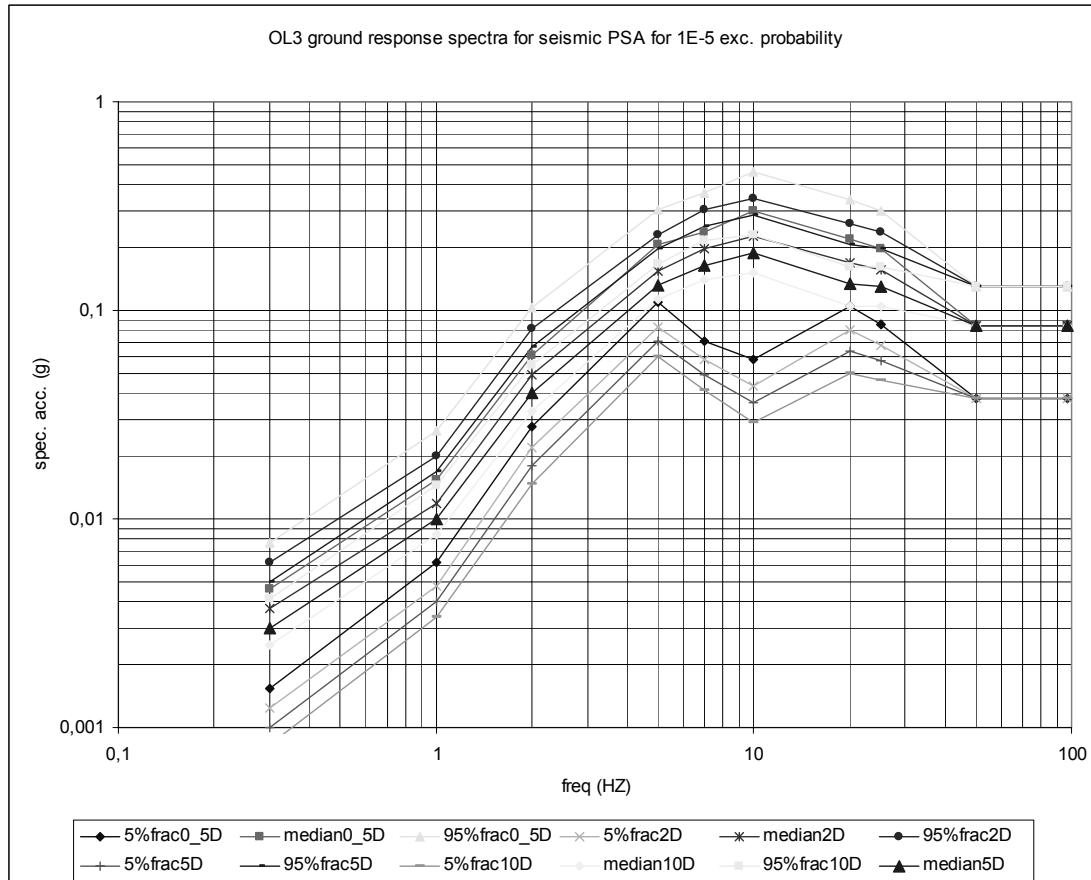
**Figure 7-1. Seismic Hazard in PGA for Olkiluoto3 site**  
*Presented are the 32 raw hazard curves forming the hazard distribution at site according to the decision tree presentation of Figure 6-1*



**Figure 7-2. Hazard curves with confidence bounds (5%, median, 95%) for Olkiluoto site**



**Figure 7.3. The ground response spectra for OL3 with uncertainty bounds for annual exceedance probability of 1.0E-05.**  
*The median spectrum shape and 5% and 95% percentiles for damping values of 0.5%, 2%, 55 and 10% are given.*



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# Development of Seismic PSA Methodology Considering Aftershock

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**Keywords:** Seismic PSA, Aftershock, Seismic hazard, Fragility, Accident sequence

## 1. Introduction

When a large earthquake (mainshock) has occurred around nuclear power plant, it is generally followed by a number of large aftershocks. Therefore, the safety of power plant against the main shock and aftershocks should be assessed. We have developed the following seismic PSA methodology for nuclear power plant considering large aftershocks:

- 1) Seismic hazard analysis of aftershocks: It is possible to predict the aftershock activity by combining the evaluating the number of aftershock following mainshock and the frequency of aftershock of a certain magnitude. The seismic motions due to aftershocks are evaluated by attenuation formula or fault model considering the spatial distribution of aftershocks in the aftershock region.
- 2) Seismic fragility evaluation: Structures and components fragility can be evaluated by considering seismic motions due to mainshock and aftershocks that will occur within a short period of time.
- 3) System reliability analysis: The system reliability analysis of a nuclear power plant is taken into account for additional accidents by aftershocks while the reactor is establish cold shutdown after scram by a mainshock.

This paper is organized in two sections. In the first section, the seismic PSA methodology for aftershock is explained. In the second section, an example of the seismic hazard analysis for aftershock based on earthquake observation records is presented. In this analysis the prediction model of aftershock activity were verified and the influence of the seismic hazard of the aftershock was also examined by comparing with seismic hazard of the mainshock.

## 2. Seismic PSA methodology considering aftershocks

Figure 1 shows an overview of a seismic PSA method for aftershocks. As shown in this figure, the seismic PSA methodology for aftershocks consists of a seismic hazard analysis, evaluation of the fragility of structure and component, and system analysis. This procedure is basically same as a seismic PSA method for main shocks. However, because the seismic PSA for aftershocks should take into account the damage due to mainshock and aftershocks in the fragility evaluation and system analysis, the evaluation details are more complicated than the PSA taking only mainshocks into account. Therefore, when the effects of aftershocks on seismic hazards or fragility cannot be neglected compared to those of the main shock, full-scope PSA for aftershocks should be conducted as shown in Figure 2.

The definition of aftershocks in this methodology is described in following section.

Figure 1. Overview of seismic PSA methodology for aftershocks

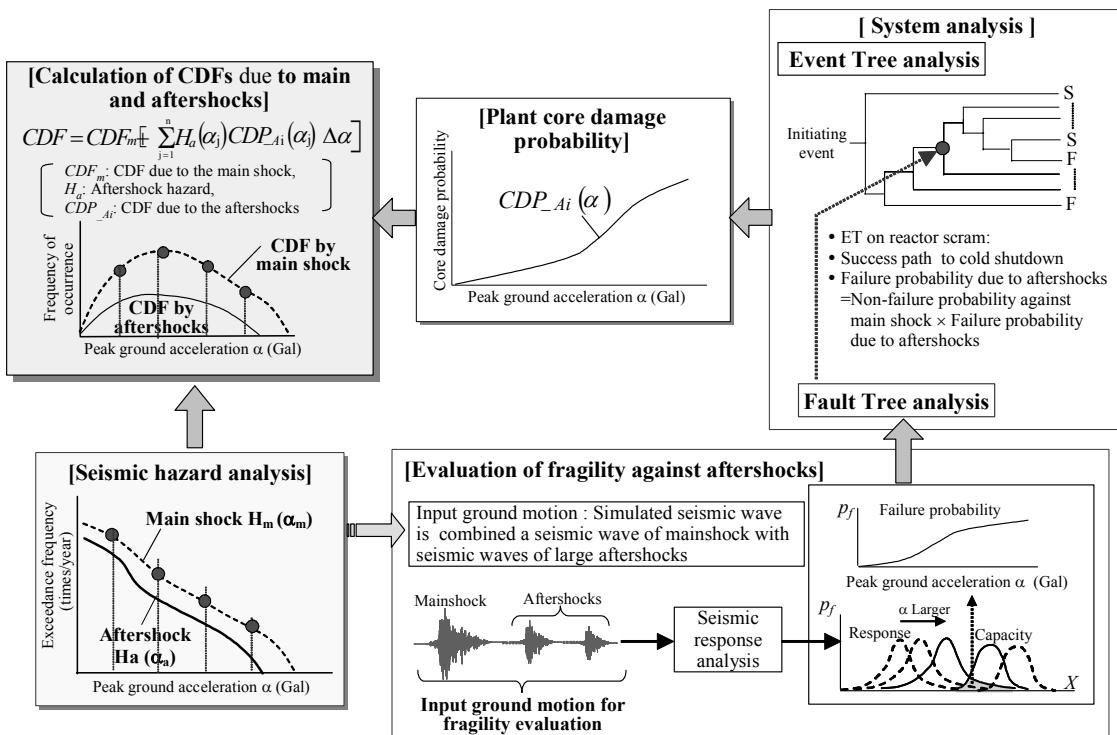
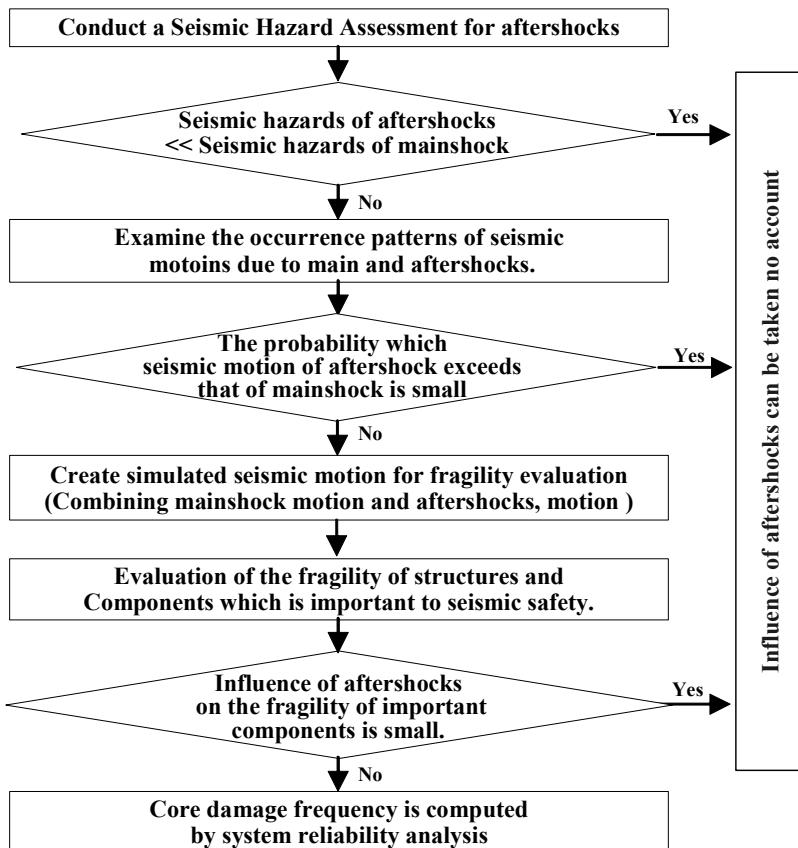


Figure 2. Seismic PSA considering the influence of aftershocks



## 2.1 Seismic hazard analysis for aftershocks

The largest one of earthquakes which occurs in a focal region is defined as the mainshock, and smaller earthquakes following the mainshock are defined as aftershocks. An objective evaluation period in this seismic PSA is the period during which the stable condition (cold shutdown) of the reactor is established following scram by mainshock.

Figure 3. Seismic hazard analysis methodology for aftershocks

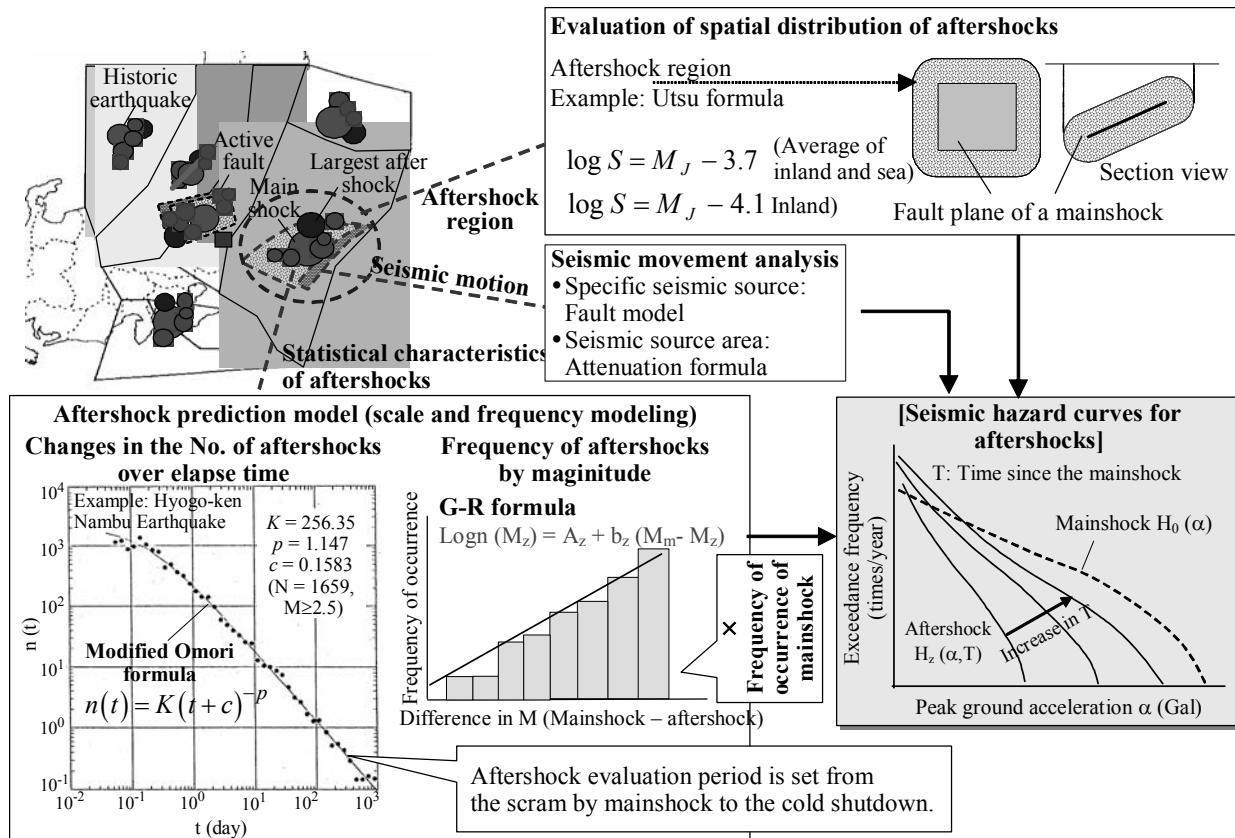


Figure 3 shows a procedure of the seismic hazard analysis. To evaluate the seismic hazard for aftershock, it is necessary to make a stochastic prediction model of aftershocks activity. The four basical terms that are required to make this prediction model are:

- (1) Number of aftershocks during the evaluating period in the seismic PSA.
- (2) Frequency that an aftershock of a certain magnitude will occur.
- (3) Magnitude of the largest aftershock.
- (4) Spatial distribution of aftershocks focus.

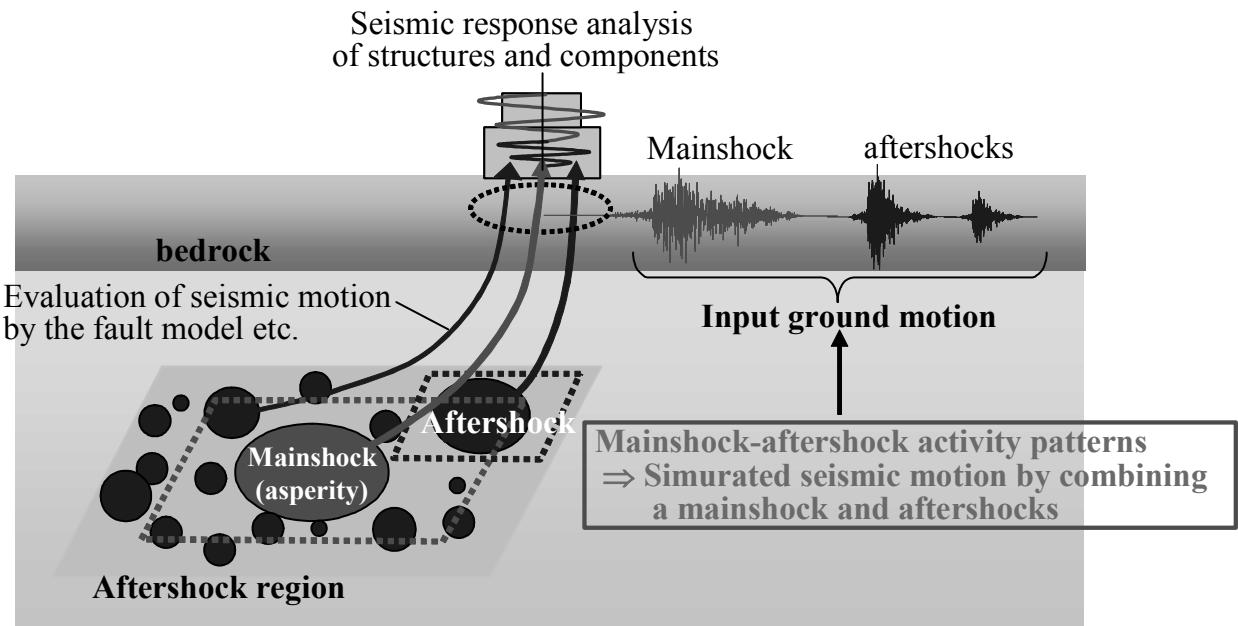
The numbers of aftershocks after mainshock can be predicted using modified Omori formula [1,2]. The frequency that an aftershock of a certain magnitude will occur can be predicted by using Gutenberg-Richter (G-R) formula [3]. By combining the modified Omori formula and the G-R formula, it is possible to predict the probabilistic aftershocks activity [4]. The largest aftershock magnitude is predicted based on an statistical investigation of difference  $D (=M_m - M_a)$  between the mainshock magnitude( $M_m$ ) and largest aftershock one( $M_a$ ) by Utsu [2]. The value of  $D$  is provided in relationship with  $M_m$  and its average value is 1.4.

It is possible to designate the locations where the aftershocks will occur within the aftershock region which are estimated by the Utsu formula [2] with the mainshock magnitude as parameter. In this method the aftershocks are assumed to distribute at random throughout the aftershock region. The seismic motion of aftershock is evaluated by attenuation formula or fault model according to the magnitude and location of aftershock.

## 2.2 Seismic fragility evaluation

The seismic fragility of structures and components is evaluated by the assumption that a nuclear power plant receives the seismic motions by a mainshock and the aftershocks within a short period of time continually. Under this assumption the simulated seismic motion as input ground motion is expressed by combining a minshock seismic motion with several large aftershock motions (see Figure 4). The combination of a mainshock and the aftershocks with great impact on nuclear power plant are selected from combination patterns of mainshock and aftershocks occurred in various seismic sources such as earthquakes in sea area or shallow inland earthquakes.

Figure 4. Fragility evaluation considering seismic motion due to main and aftershocks



The seismic responses of reactor building and components are computed by using above simulated seismic waves, and the fragility curves are estimated as the conditional frequency of failure for a given level of input ground motion. The response analysis models of the building and components are the same models used in the analysis taking only a mainshock into account.

## 2.3 System reliability analysis

Once component fragilities are available, core damage frequency (CDF) can be computed using the event trees and fault trees developed in the system reliability analysis stage. This is done in same basic procedure as in the seismic PSA for mainshock. The item to consider in the system analysis for aftershocks is as follows:

The accident events to consider within event trees are those occurred since scram induced by mainshock to the cold shutdown as shown in Figure 5 (an example of PWR plant). The success paths to establish the cold shutdown in accident sequences are selected for evaluation, excluding failure paths to extend the core damage by the main shock (see Figure 6).

Figure 5. Predicted plant operating states following aftershocks (PWR plant)

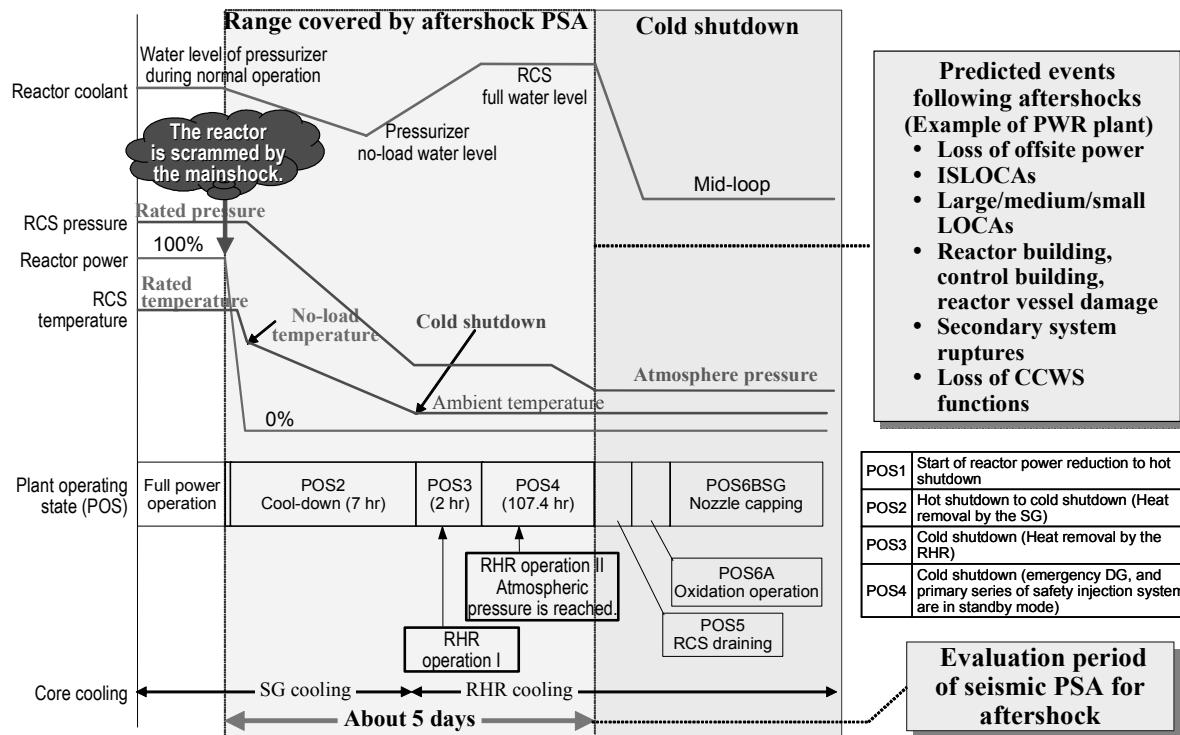
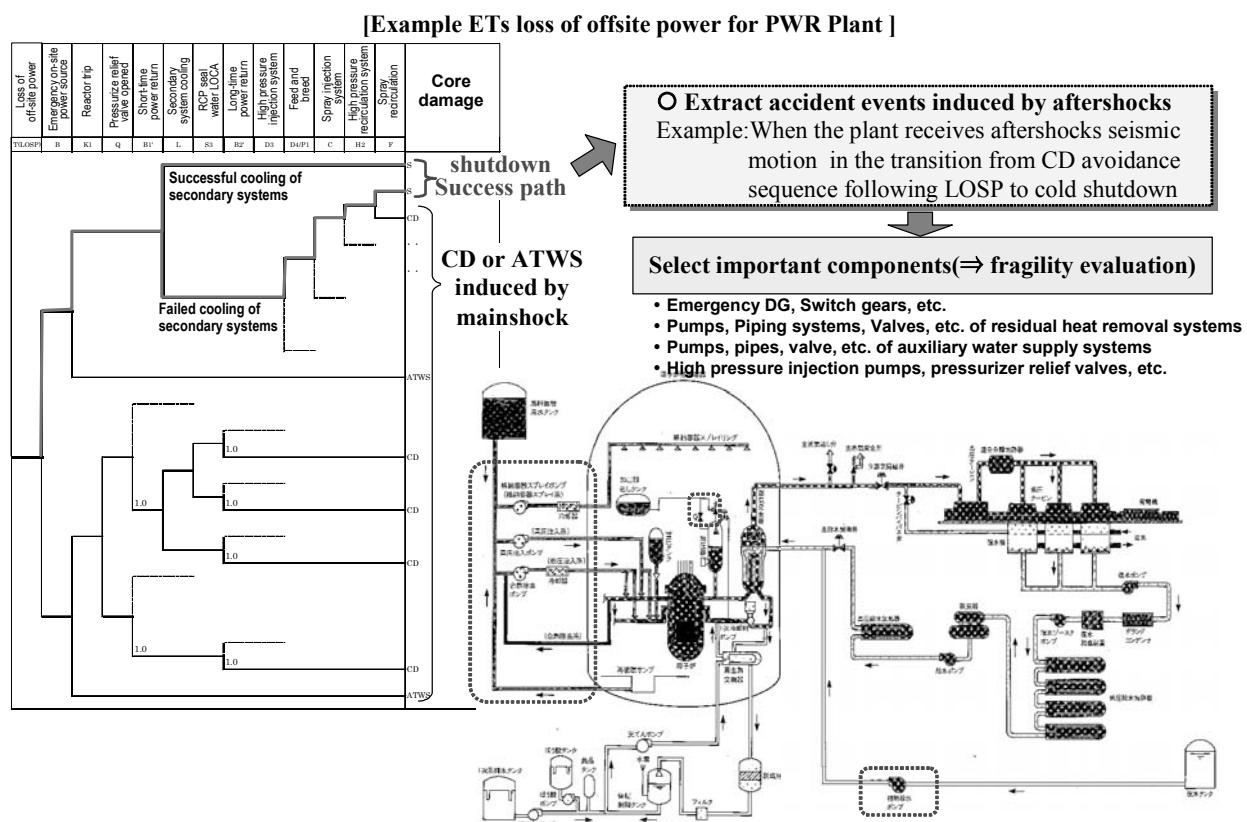


Figure 6. Analysis of accident sequences following aftershocks (PWR plant)



Failure probability due to aftershocks in fault tree analysis is evaluated by multiplying the non-failure probability of the building and components against the mainshock and the failure probability induced by aftershocks. CDF induced by aftershocks is computed using the fragility curve considering the influence due to mainshock and aftershocks and hazard curve for aftershocks.

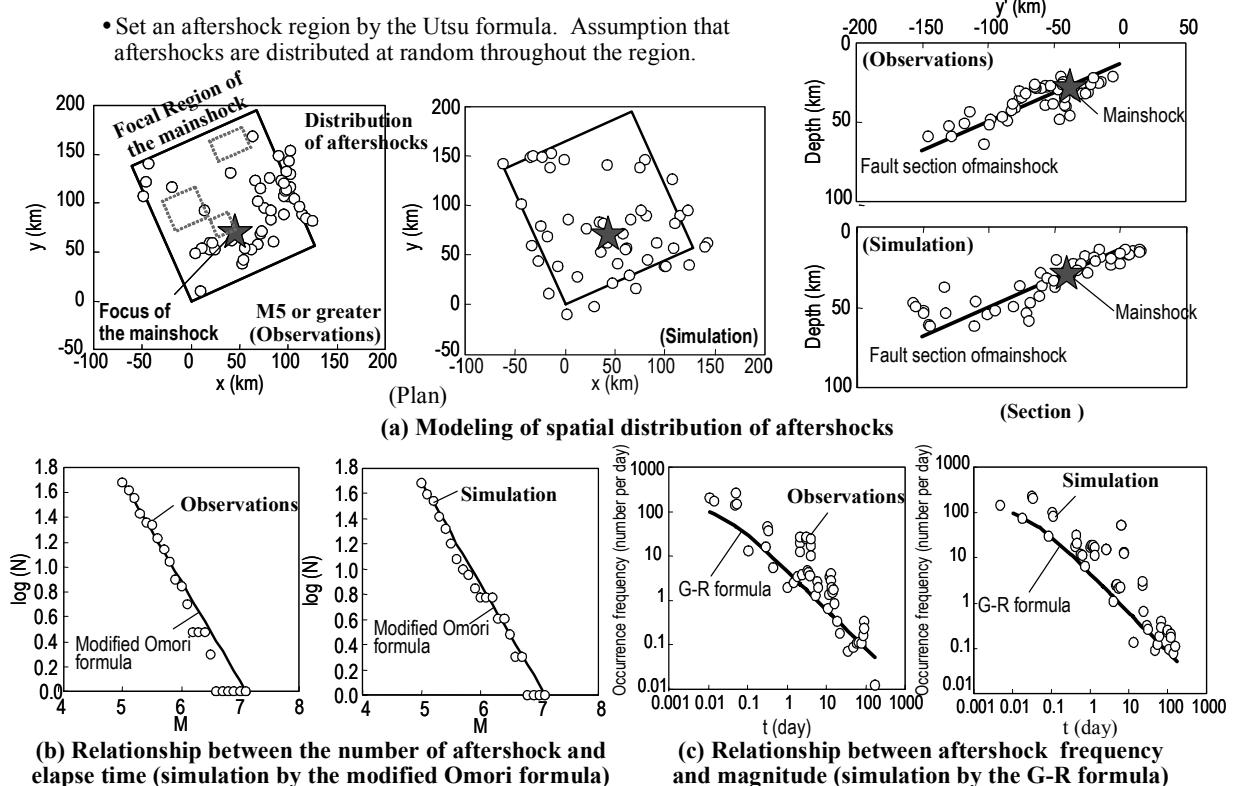
### 3. Example of seismic hazard analysis for aftershocks using observation records

In order to confirm the applicability of the prediction model and procedure in seismic hazard analysis for aftershocks, trial analysis was performed using observation records of the 2003 Tokachi-oki Earthquake. The influence of the seismic hazard of the largest aftershock was also examined by comparing with seismic hazard of the mainshock in this trial analysis.

#### 3.1 Simulation of the aftershock activity based on the aftershock prediction model

Figure 7 shows the simulation results of the aftershocks activity by above prediction model. Figure 7 gives the number of aftershocks with elapse time following the mainshock and frequency of aftershocks by magnitude. In the same figure the spatial distribution of aftershocks in focal region are also given. The numbers of aftershocks and frequency by magnitude were simulated using the modified Omori formula and G-R formula respectively, and the simulation results corresponded to observation records. On the other hand, the simulated spatial distribution of aftershocks which was assumed to locate at random throughout the aftershock region was different from the observed distribution. The observed aftershocks often occur in the northeast side near the edge of aftershock region, and it is seemed that the aftershocks do not occur near the asperity area. To obtain the more real aftershock distribution, it is necessary to research the relationship between aftershocks distribution and asperity location, left at the mainshock rupture and three dimensional shape of the focal region, etc. about more earthquake records.

Figure 7. Simulation of an aftershock occurrence model for the 2003 Tokachi-oki earthquake

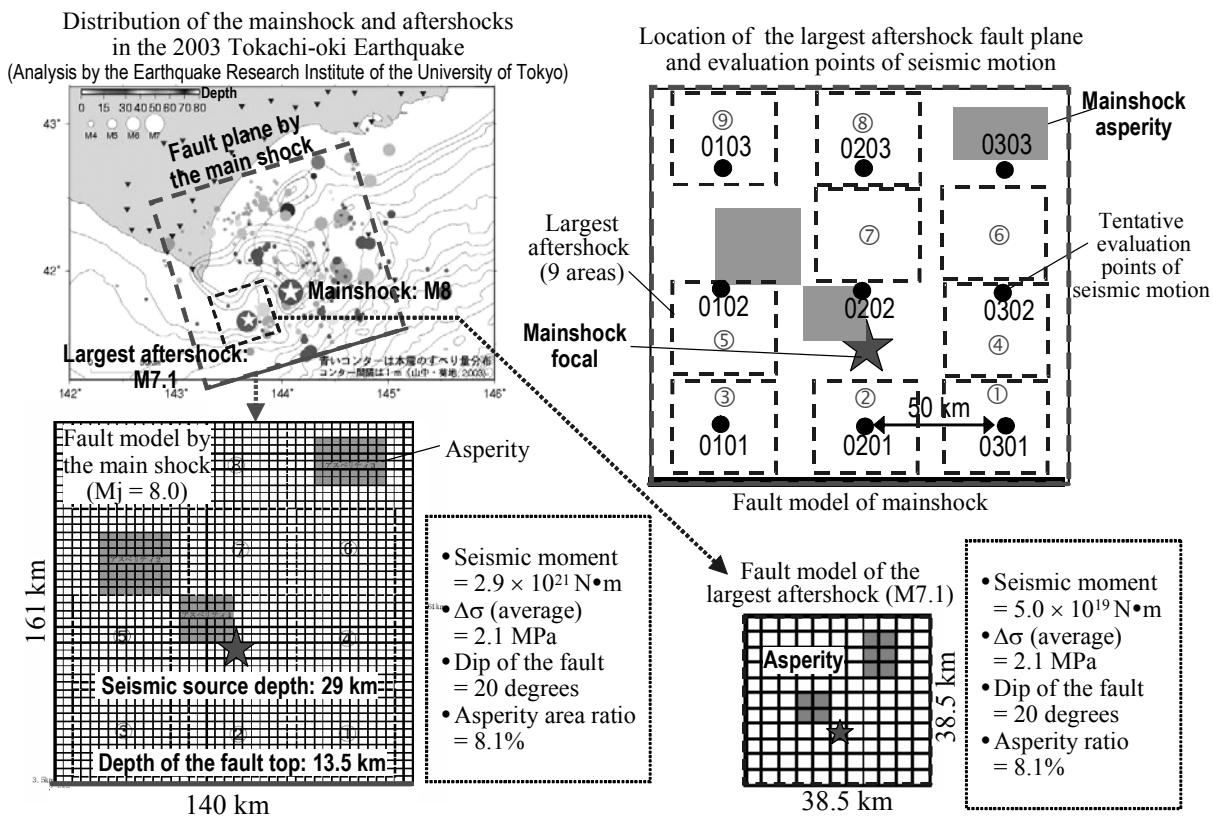


### 3.2 Trial analysis of seismic hazard of mainshock and largest aftershock

#### (1) Analysis conditions

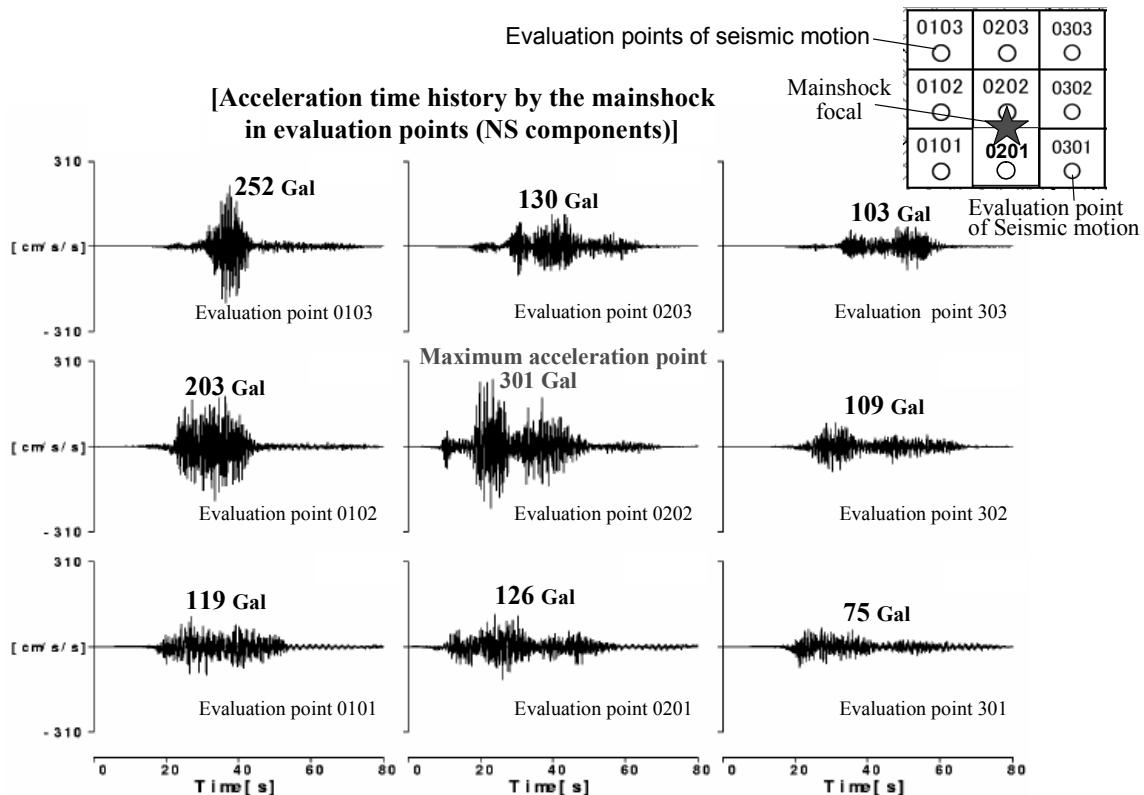
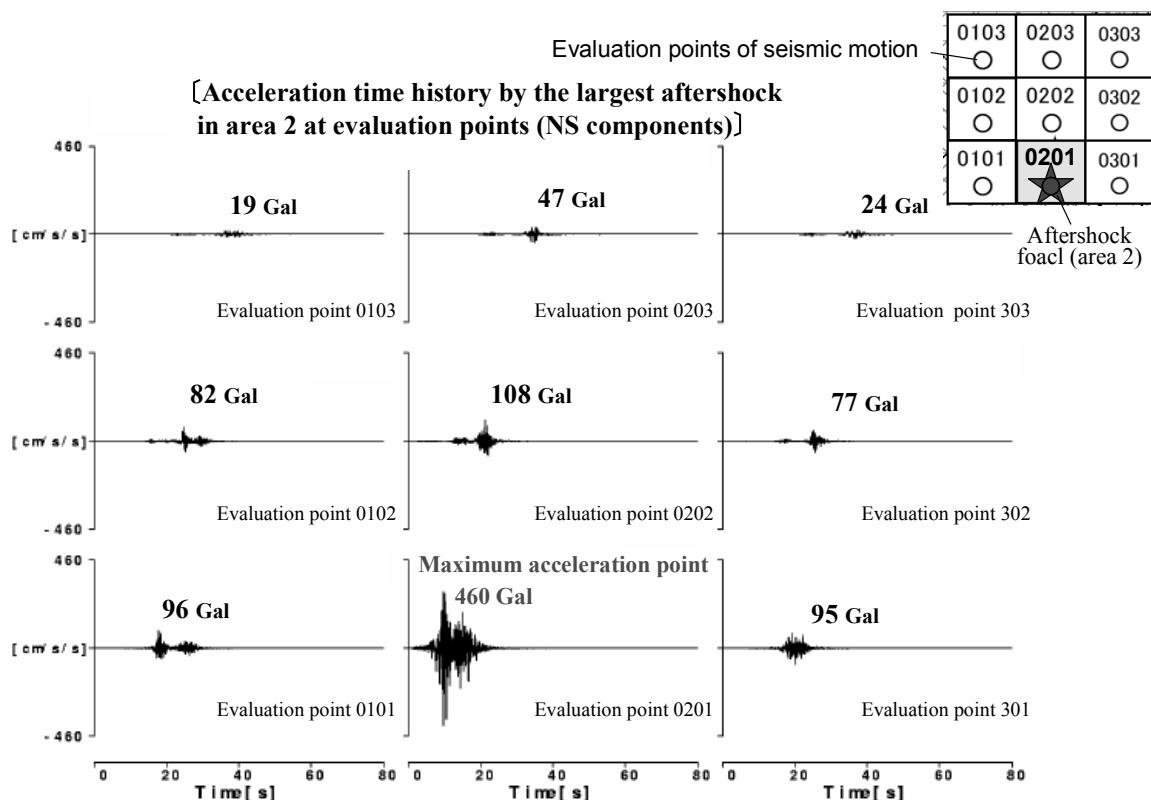
As shown in figure 8 in this analysis the mainshock magnitude (M8.0), the largest aftershock magnitude (M7.1) and focal region of mainshock was determined based on actual observation records of the 2003 Tokachi-oki Earthquake. It was assumed that the largest aftershock occurs with same probability at either in 9 areas within the aftershock region excluding the asperity areas of the main shock. The seismic motions of mainshock and largest aftershock were evaluated at 9 tentative points above the focal region of mainshock, and seismic motions were evaluated using a fault model<sup>5)</sup>. Subsequently, the exceedance probabilities of the seismic motion by mainshock and by largest aftershock were evaluated and both exceedance probabilities were compared at 9 points.

Figure 8. Analysis conditions of seismic motion due to mainshock and aftershocks by fault model

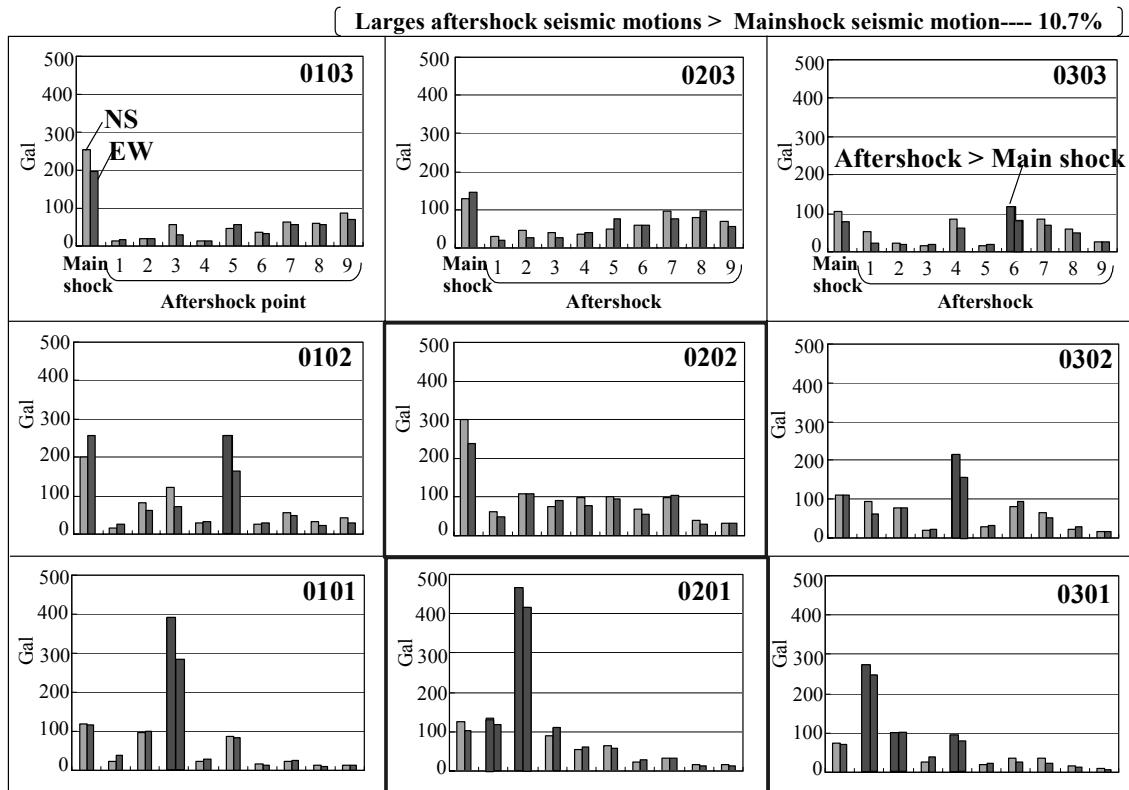


#### (2) Analysis results

Figure 9 shows the results of analysis of seismic motions due to the mainshock at 9 evaluation points. Figure 10 shows the analyzed seismic motions at 9 evaluation points due to the largest aftershocks set in the area 2. As shown in these figures, when the largest aftershock occurs in area 2, the largest peak acceleration in all analysis case is observed at evaluation point 0201 near the aftershock area, and the seismic motions is greater than that of the mainshock. Figure 11 compares the peak accelerations of the main shock and the largest aftershock at each evaluation points. In this figure when the largest aftershock is located in near the surface, the peak acceleration of the aftershock at the evaluation points near the aftershock exceeds that of the mainshock, and the ratio of appearance of such cases is about 11% in all analysis cases.

**Figure 9. Results of seismic motion due to the mainshock by fault model****Figure 10. Results of seismic motion due to the largest aftershock by fault model**

**Figure 11. Peak ground acceleration at evaluation points due to the mainshock and largest aftershocks**



**Figure 12. Mainshock and largest aftershock hazard analysis results**

Headquarters for Earthquake Research Promotion: Set a 30-year exceedance probability of 80% for the main shock based on the probability of earthquakes in the area along the Chishima Trench

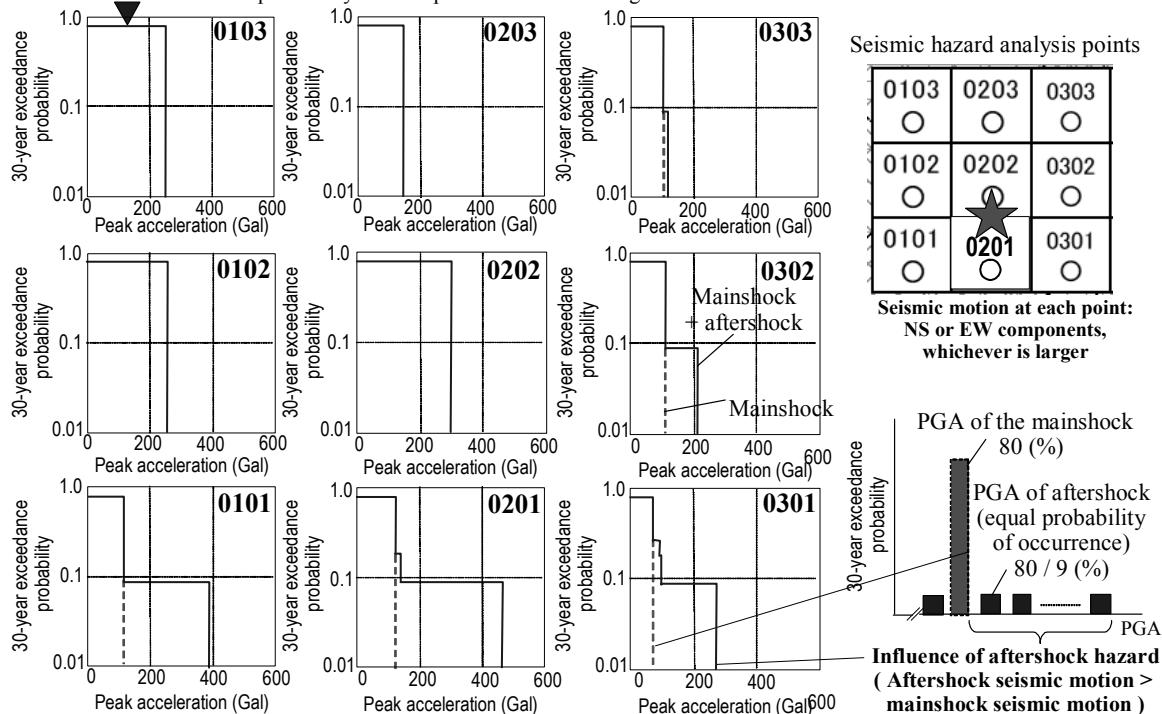


Figure 12 shows seismic hazard curves for each evaluation points based on the above analyses. The seismic hazard curves were assessed by setting the 30-year exceedance probability of the 2003 Tokachi-oki Earthquake to be 80% based on earthquake research<sup>5)</sup> of the Headquarters for Earthquake Research Promotion. In these seismic hazard curves, the exceedance probability of peak ground acceleration increases at the evaluation points where the peak acceleration of the largest aftershock exceeds that of the mainshock. It is supposed that aftershocks influence on seismic hazards at a high seismic motion level (with a low probability area), and that the influence of aftershock appears in limited area near the large aftershock.

#### 4. Conclusion

The conclusion of this report is summarized as follows:

- 1) As a seismic PSA methodology considering aftershocks, the methodology of seismic hazards analysis for aftershocks, fragility evaluation of structures and components against mainshock and aftershocks, and system reliability analysis by considering additional accidents by aftershocks following scram by mainshock were summarized.
- 2) Based on the observation records of the main and aftershocks of the 2003 Tokachi-oki Earthquake, the simulation was carried out using the aftershock prediction model. It is confirmed that the modified Omori and G-R formulas can be used to accurately simulate the number of aftershocks and occurrence frequencies of aftershocks by magnitude.
- 3) In order to confirm the applicability of the prediction model and procedure in seismic hazard analysis for aftershocks, trial analysis was performed using observation records of the 2003 Tokachi-oki Earthquake. The influence of the aftershock hazard of the aftershock was also examined by comparing with seismic hazard of the mainshock. The exceedance probability of peak ground acceleration increases at the evaluation points where the peak acceleration of the largest aftershock exceeds that of the mainshock. It is supposed that aftershocks influence on seismic hazards at a high seismic motion level (with a low probability area), and that the influence of aftershock appears only in limited area near the large aftershock.

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## An Innovating Approach to Address Uncertainties in Probabilistic Seismic Hazard Assessment

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### ***Abstract***

Since the basic work of Cornell, many studies have been conducted in order to evaluate the probabilistic seismic hazard (PSHA) of nuclear power plants. In general, results of such studies are used as inputs for seismic PSA. Such approaches are nowadays considered as well established and come more and more used worldwide, generally in addition to deterministic approaches, typically for risk informed studies, at design stage or for evaluation of existing nuclear power plants.

Nevertheless, some discrepancies have been observed recently in some PSHA, especially from studies conducted in areas with low to moderate seismicity. The lessons learned from these results lead to conclude that, due to uncertainties inherent to such a domain, some deterministic choices (concerning input data for instance) have to be taken and, depending on expert judgment and choices, may lead to strong differences in terms of seismic motion evaluation.

In that context, the objective of this paper is to point out some difficulties that may appear in the development of PSHA studies and to propose an approach that may be used to orient expert judgment and address epistemic uncertainties. The key point, which corresponds to the innovating point of the process, is the use of instrumental experience to update the results of a PSHA using observations that remain consistent with the real regional seismicity, as recorded.

For this purpose, we present first some basic considerations on the current practice in PSHA studies and some resulting difficulties that one may face at different steps of the study. Then we describe an approach that may be used to update the results of the PSHA, which is based on a Bayesian updating technique including real observations as conditional events, with their own probabilistic distribution.

The results presented here point out that a PSHA must be conducted in a real probabilistic spirit that is totally different from a deterministic approach (the choice of “best-estimate” or “median” input data instead of “conservative” ones is one of the key points). In addition, logic tree procedure, which seems to be the most appropriate way do account for epistemic uncertainties, does not quantify the variability on the physical parameter itself but quantify variability on expert opinion. This may also lead to an important bias in a PSHA study.

Finally, the lesson learned is that results from PSHA may be strongly different from real seismicity, as recorded, especially depending on previous considerations. Then, the comparison to the instrumental experience data appears to be necessary to address such difficulties. In that context, the use of the Bayesian updating technique presented in this paper may become a necessary tool to address epistemic uncertainties in PSHA and its performances could allow to get PSHA more rugged and consistent with observations.

## 1. Context and objectives of the study

Since the basic work of Cornell, many studies have been conducted in order to evaluate the probabilistic seismic hazard (PSHA) of nuclear power plants. In general, results of such studies are used as inputs for seismic PSA. Such approaches are nowadays considered as well established and come more and more used worldwide, generally in addition to deterministic approaches, typically for risk informed studies, at design stage or for evaluation of existing nuclear power plants.

Nevertheless, some discrepancies have been observed recently in some PSHA [KLÜ-05], especially from studies conducted in areas with low to moderate seismicity. The lessons learned from these results lead to conclude that, due to uncertainties inherent to such a domain, some deterministic choices (concerning input data for instance) have to be taken and, depending on expert judgment and choices, may lead to strong differences in terms of seismic motion evaluation.

In that context, the objective of this paper is not to describe in detail the PSHA overall approach but its objective is to point out some difficulties that may appear in the development of PSHA studies, and to propose an approach that may be used to orient expert judgment and address epistemic uncertainties. The key point, which corresponds to the innovating point of the process, is the use of instrumental experience to update the results of a PSHA using observations that remain consistent with the real regional seismicity, as recorded.

This paper is divided in two parts. The first part presents some basic considerations on the current practice in PSHA studies and identifies some difficulties that one may face at different steps of the study, especially concerning input data selection. The second part describes an approach that may be used to update the results of the PSHA, which is based on a Bayesian updating technique including real observation as conditional events, with their own probabilistic distribution.

## 2. Part 1 - Basic considerations on the current practice in PSHA and associated difficulties

As the objective of this paper is not to describe in detail the PSHA overall approach, we only focus hereafter on some specific aspects of a PSHA that could lead to difficulties (implying deterministic choices) and we try to quantify their consequences.

### 2.1 Seismic motion characterization: Random uncertainty?

We would like to discuss first on the characterization of the variability of the seismic motion. In particular, two question may raise related to this factor:

- Q1: Where sigma (i.e. standard deviation associated to the attenuation relationship) comes from?
- Q2: How many sigma to integrate in the PSHA (1, 2, infinite ...)?

#### *Elements of answer to Q1*

Concerning Q1, it seems to be obvious that sigma comes from the (random) variability of the seismic motion itself. Nevertheless, it is also obvious that in the strong motion databases that are used to determine attenuation relationships, the characteristics (magnitude and distance) of the events are determined with a uncertainty which may be significant (typically 0.5 for the magnitude, even more if different types of magnitudes are used and it may also be the case for the distance, especially for near field events). These uncertainties have a direct impact on the sigma of the attenuation relationship.

To give an order of magnitude, for an attenuation relationship which would have a coefficient 0.3 on the magnitude dependent term ( $\log \text{PGA} \sim 0.3 M$ ), and a sigma of 0.3 (which is a common value), a 0.5

uncertainty on the value of the magnitude given in the database would lead to a significant contribution on the global standard deviation which should therefore decrease the sigma of 0.25 on the random variability of the seismic motion itself, due to combination of uncertainties.

As this value of sigma has a direct (and important) effect on the PSHA, the “best-estimate” value of the sigma to be used should be carefully assessed.

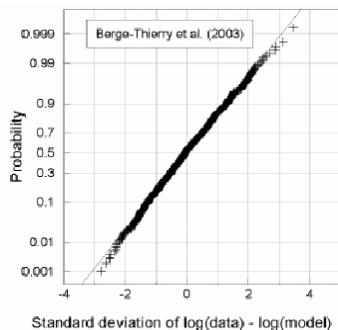
Unfortunately, only a few studies are available on that point.

#### *Elements of answer to Q2*

In a pure probabilistic spirit, the first answer should be to integrate the sigma to infinite. Nevertheless, the fact that seismic motion distribution follows a lognormal law is an assumption and should be always checked with real data. Depending on strong motion databases, this verification may not be possible over 2 to 3 sigma (see Figure 1).

This is an important fact to point out. Even if it may be shown that over 3 sigma, the impact may not be so important, the integration should be limited between 2 and 3 times sigma in a PSHA.

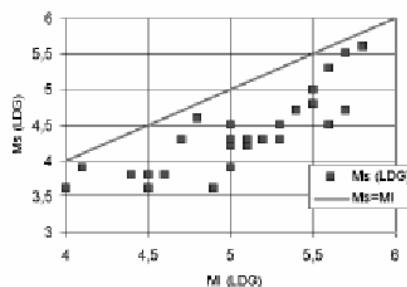
**Figure 1. Comparison between lognormal distribution (records and model)**



Another point we would like to discuss now concerns the equivalence of magnitude that may be necessary to assess in PSHA, due to historical and instrumental seismicity, and magnitude used in attenuation relationships.

This leads to choices that can be taken in a probabilistic spirit or in a deterministic spirit. For instance, the choice to take a “conservative” relationship between  $M_s$  and  $M_{L(LDG)}$  (see Figure 2), which may be the appropriate choice for a deterministic hazard evaluation for a NPP for instance, may lead to a bias in a probabilistic approach and will directly over-estimate the median value of the ground motion.

**Figure 2. Equivalence between different types of magnitude: which relationship to use?**



According to the PSHA methodology, best-estimate data should be kept at each stage of the process in order to keep the “median” estimation spirit of the PSHA, as expected.

### **Conclusion on the characterization of the seismic motion**

Concerning variability of seismic motion, some choices have to be made (among the possible ones) and may have a strong impact on the results of a PSHA. The most important request is to keep “best estimate” datas, to remain consistent with the PSHA philosophy. In that situation, the value of sigma associated with attenuation relationships may be slightly reduced to account for the random variability of the seismic motion only, its range of integration should also be carefully chosen: typically between 2 and 3 sigma but no more, and the potential relationship used to transform different types of magnitudes should be also a best-estimate one.

*This fact should be considered in a PSHA in order to get a “real” median value, as expected.*

## **2.2 Selection of attenuation relationship for a given area: Epistemic uncertainty?**

As one can expect, attenuation relationships come from areas with moderate to high seismicity (where datas are available). However, they may be used for PSHA in areas with low to moderate seismicity. In that situation especially, their applicability (even if there may not be another way to proceed) is to be seriously assessed.

In addition, it may be obvious to say that very important differences may be observed from different attenuation relationships (see one illustration in Figure 4, § 3.2).

In a PSHA, this aspect is accounted for based on expert judgment. The knowledge of the real seismicity of the given area and the content of attenuation relationships is then an important factor. Practically, this is usually accounted for by means of logic trees that allow to take into consideration all possible judgments.

Nevertheless, one must keep in mind that this logic tree approach allows to quantify variability on experts' judgment only but not the variability on the physical parameter itself.

This epistemic uncertainty is expected to be reduced, with increase of knowledge.

### **Conclusion on the choice of an attenuation relationship**

The choice of attenuation relationships is one of the most important choices in a PSHA and may lead to high differences in the results. In that situation, it is important to point out that the logic trees procedure which seems to be the most appropriate way to account for epistemic uncertainties does not quantify the variability on the physical parameter itself but quantify variability on expert opinion. This may lead to an important bias in a PSHA study.

*The current practice should then be improved in order to reduce the epistemic uncertainty on the physical parameter itself.*

## **2.3 Conclusion**

Although well established, some basic steps of PSHA are still under discussion. But most of the difficulties come from the choice of input data and the way to account for uncertainties, which is different from the way to address them in a deterministic approach.

- The most important request is to keep “best estimate” data, at each stage, to remain consistent with the PSHA philosophy, and to obtain a “real” median estimation.
- It is also important to propagate variability but trying to separate random uncertainty from epistemic ones.
- Finally, it must be kept in mind that logic trees allow to quantify the variability on expert judgment but not the variability on the physical parameter itself, this should be improved.

The next part of this paper is to propose a method to address this last.

### 3. Part 2

#### An innovating approach to address epistemic uncertainties using real earthquake records

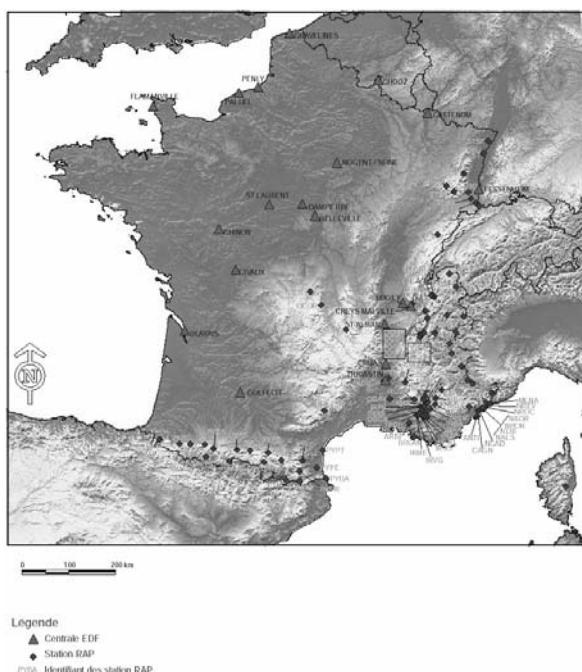
##### 3.1 Context and available data

This paragraph proposes a method to address epistemic uncertainties using real seismicity, as recorded.

This method is based on a preliminary statement that results of a PSHA may not be consistent with real data (as recorded) especially depending on the potential problems discussed previously. This fact is discussed in paragraph 3.2. Then the method is described in paragraph 3.3. Finally, a first application is presented in paragraph 3.4 and discussed in paragraph 3.5.

For the illustration of this situation, a case study is used. A PSHA is performed on the French metropolitan territory and records from EDF PWR NPP network and national earthquake network [RAP] are used (see Figure 3).

**Figure 3. Map of French metropolitan territory and location of earthquake stations**



The comparison is performed for the period of time consistent with the time of available records: 5 to 10 years for RAP stations and 10 to 25 years from EDF NPP stations.

To compare results between PSHA prediction and experience events, the number events with a PGA higher than a selected value are compared between the prediction and the real observation.

##### 3.2 PSHA study – Impact of uncertainties and comparison to available experience data

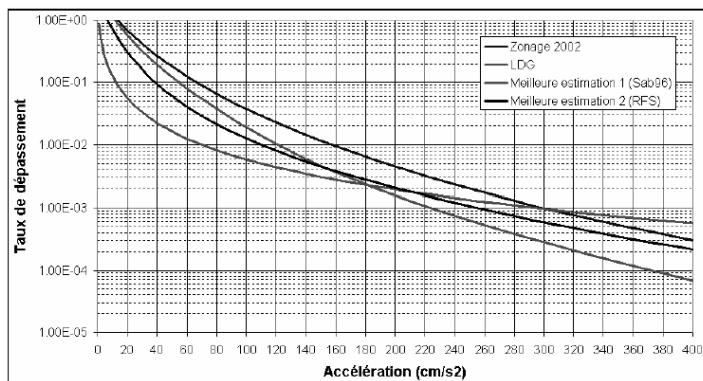
The PSHA is performed based on usual practice. The input datas are selected considering the previous discussion (see § 2).

For this application some attenuation relationships are used and will be considered is a logic tree approach, see Table 1.

**Table 1. Description of model used in the study**

	Description des modèles historiques				
	H1	H2	H3	H4	H5
<i>Catalogue de sismicité Catalogue ZPGF de l'étude 2002, sur les 25 dernières années.</i>					
Loi d'atténuation	Ambraseys (1995)	Ambraseys (1995)	Sabetta et Pugliese (1996)	RFS 2001-01	RFS 2001-01
Transformation $M_{LDS}/M_s$	non	Nicolas et al. (2000)	non	non	Nicolas et al. (2000)
Magnitude minimum			4.0		
Grille de calcul			0.3°		
Valeur calculée			Médiane		

For each attenuation relationship, taken individually, it is possible to perform a PSHA and make an inter-comparison. The results are illustrated in Figure 4.

**Figure 4. Illustration of the results of a PSHA depending on attenuation relationships**

This shows the potentially large impact of such choices in the probabilistic hazard evaluation (factor of 2 on PGA or 10 on return period).

Then it is possible to compare the predictions to the observations, case by case. These results (from 2 among the previous models) are shown in Table 2.

**Table 2. Comparison of number of events higher than 0.01 g (model and observation)**

Network	Number of events with PGA > 0.01g		Observations	
	Model			
	H4	H5		
EDF PWR (total number of events)	85	12	1	
RAP stations (total number of events)	87	27	14	

This comparison shows that, depending on choices, the differences between prediction and observation may be very important.

#### **Conclusion on the impact of uncertainties and comparison to available experience data.**

The lesson learned from this part is that results from PSHA may be strongly different from real seismicity, as recorded, especially depending on choices on input data that always depend on expert judgment.

*Then, the use of instrumental experience data appears to be necessary to address such difficulties.*

### 3.3 A way to address uncertainties: the use of a Bayesian updating technique

Based on previous results, the objective here is to use records to update the prediction in order to get results more consistent with experience.

The method used is based on a classical Bayesian updating technique, used for many years in reliability field, especially in mechanics.

The basic consideration is simple:

***“based on a probabilistic evaluation of a given parameter, what is the most probable values that this parameter could take (associated with a given confidence level) considering the available observations (including their uncertainties) as conditional events ?”***

The technique used here was developed by Madsen [MAD-85] and its performance and pertinence have been already confirmed [HEI-99].

It uses the Bayesian theorem of conditional probability:

$$P(A|B) = \frac{P(B|A)P(A)}{P(B)}$$

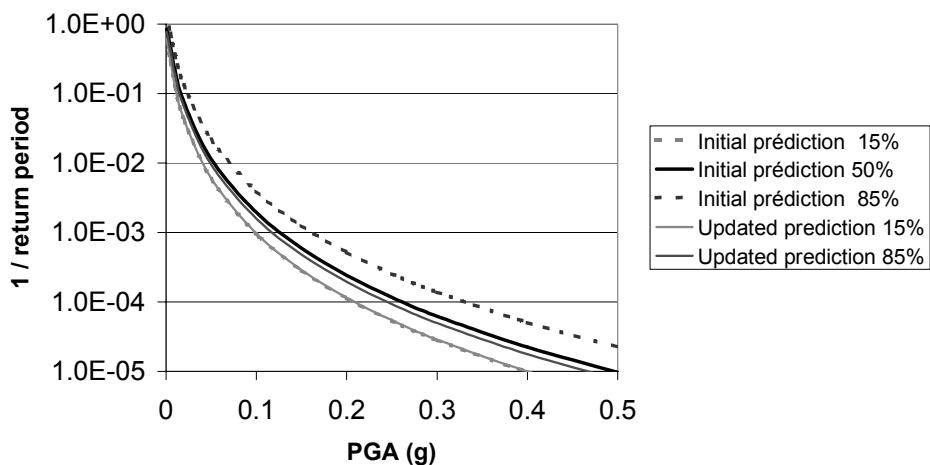
In our case, the “realistic” range of the PGA is defined by the 15% - 85% confidence level of the predicted PGA. The probability that reaches both limits may be calculated for each selected return period. The observed data are then taken into account as conditional events, with their own probabilistic distribution.

The updated probabilities are obtained from a classical FORM method within the Bayesian updating framework developed in [MAD-85] and solved for each selected fractile (15 – 85% in our case) by a dichotomy method.

### 3.4 First application: feasibility exercise

As a first application of this method, the updating technique is carried out using the results of the complete logic tree from the study described in § 3.2. The results obtained are shown in Figure 5.

Figure 5. Comparison between initial prediction and updated one



One can observe the potential effectiveness of the updating process which reduces significantly the scattering of the initial prediction and also “re-center” the prediction around fractiles which are not the median value of the initial prediction.

These results are discussed in the next paragraph.

### ***3.5 Discussion around the updating technique***

First of all, it is important to indicate that the updating technique does not modify any of the input data or assumption of the initial prediction. The technique only accounts for observations in order to identify the most probable results among the initial ones.

Then, one can notice that the techniques accounts for random uncertainties as far as these random uncertainties are included in the observations (this is the case by considering different stations in different areas and different time of observation). This is one of the reasons that lead to an updated prediction with still variability (in other mechanical studies dealing with other parameters with a low random uncertainty, such as delayed strain in PWR RC containment for instance, the updated technique lead to results with a low variability).

Nevertheless, it must be told that a potential limitation of the current exercise could come from the relatively low period of observations (due to low seismicity in France and relatively “young” seismic network). But some other countries like Japan and USA for instance may have a larger data base and should not be limited by that fact. In addition, for our case, the time of observation is continuously increasing (for instance, the available data should double within 5 next years!) and it is obviously possible to “update” the updating including new data.

Nevertheless, low data is better than no data at all!

#### **Conclusion on the use of the Bayesian updating technique**

Our conclusion concerning the use of the Bayesian updating technique is that it may become a real interesting tool to address epistemic uncertainties in PSHA and its performances could allow to get PSHA more rugged and consistent with observation.

***Then, updating technique may become one necessary step in PSHA methodologies.***

## **4. Conclusions and perspectives**

The objective of this study was to point out some difficulties in PSHA studies and to propose an approach that may be used to orient expert judgment and address epistemic uncertainties.

The most important conclusions that we would like to point out are the following:

- First of all, it may be important to keep “best estimate” data, to remain consistent with the PSHA philosophy. In that situation, the value of the standard deviation associated with attenuation relationships may be slightly reduced to account for random uncertainty only, its range of integration should also be carefully chosen: typically between 2 and 3 time sigma but no more, and the potential relationship use to transform different types of magnitudes should also a best-estimate one.
- The choice of attenuation relationships is one of the most important choices in a PSHA and may lead to high differences in the results. In that situation, it is important to point out that the logic trees procedure which seems to be the most appropriate way do account for epistemic variability does not quantify the variability on the physical parameter itself but quantify variability on expert opinion. This may lead to an important bias in a PSHA study and this should be improved.

- Then, the lesson learned is that results from PSHA may be strongly different from real seismicity, as recorded, especially depending on choices on input data that always depend on expert judgment. In that situation, the comparison to the instrumental experience data appears to be necessary to address such difficulties.
- In that context, the use of the Bayesian updating technique may be an excellent tool to address such uncertainties and could become a necessary tool to address epistemic uncertainties in PSHA in order to obtain results more rugged and consistent with observations.

One other conclusion of this study is the effectiveness of sharing experience between seismologists and structural engineers which should be systematized in future for PSHA but also for other fields such as characterization of seismic motion for instance.

In term of perspectives, and based on the results presented here, we plan to perform the following actions:

- Performing a large scale PSHA (typically French metropolitan territory) including the updating technique using a more systematic process including observed data from EDF nuclear PWR and national accelerometric network and accounting for potential soil effect that could modify observation and / or prediction (soil effects for seismic hazard and SSI effects for network).
- Using historical seismicity and vulnerability to obtain some additional data especially in the range of 500 to 1000 year of return period in order to use all the available observations in the updating process.

As a final opinion, we are convinced that such updating techniques may become one necessary step in PSHA methodologies and could be a real opportunity to share experience between seismologists and engineers.

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## A Probabilistic Seismic Hazard Assessment based on Logic Tree Utilizing Experts Judgment

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### **1. Introduction**

There are various uncertainties in the process of Probabilistic Seismic Hazard Assessment (PSHA), a task in the Seismic Probabilistic Safety Assessment (SPSA). Uncertainties are classified into factors associated with randomness existing in the nature (Aleatory Variability) and factors associated with a shortage of human knowledge or recognition (Epistemic Uncertainty). While the former can be quantified by stochastic modeling, how to qualify the latter is a challenging issue.

This paper introduces the basic policy of JNES for uncertainties in SHA for nuclear plant sites and an example of SHA based on a logic tree (LT). In addition, it shows a method to quantify epistemic uncertainties measured on the basis of the accumulated experience of JNES to date, namely the method to utilize experts' judgment, with an outline of the result of the applicability assessment currently underway.

In the meantime, JNES was established in October 2004. It should be noted that, of the activities shown below, those implemented before the establishment of JNES were performed by the Nuclear Power Engineering Corporation (NUPEC), whose results and so on have been taken over by JNES. (JNES established in October 2004 through institutional recognition of former NUPEC.)

### **2. Basic Policy of PSHA at JNES**

JNES have aimed to develop a logic tree method considering the condition of JAPAN by modifying the method which developed at United States of America. And, the basic policies of PSHA at JNES were as below:

- ① Based on response spectrum [Break away from based on peak ground acceleration (PGA)].
- ② Consider the state of seismological knowledge. (Incorporate the simulation technique of strong motion base on seismic source mechanism into SPSA.)
- ③ Utilizing experts' judgment to a logic tree.

The various activities implemented by JNES on the development of the method utilizing experts' judgment are briefly introduced below.

### **3. Logic Tree Implementation in PSHA**

#### **3.1 PGA-based Evaluation (1996 -1998)**

SHA was implemented for three representative nuclear plant sites in Japan with attenuation relations for peak ground acceleration (PGA) by use of LTs. For these sites, there was a presumption of existence of large

epistemic uncertainties in the estimation models for seismic frequency of regional earthquake sources by magnitude, the selection of attenuation formulas, separation of the seismic area, selection of database for active faults, and estimation method for the magnitude of earthquakes occurring in the active faults. Accordingly, expert opinions were collected on the above items, with experts of various scientific and engineering fields gathered together. In addition, a questionnaire survey was conducted with experts. Finally, the technical integrator (TI) developed LTs on the epistemic uncertainty to analyze the seismic hazard. A paper discussing the study process was presented to OECD/NEA Seismic PSA Workshop (1999) [1].

### **3.2 Spectrum-based Evaluation (1999 - 2006)**

JNES has been assessing the PSHA technique using attenuation relations to the acceleration response spectrum. Five sites were targeted for the assessment in total from Sites 1 to 5. Sites 1, 3, and 5 are BWR-type nuclear power plants, whereas Sites 2 and 4 are PWR-type nuclear power plants. The SHA was conducted by extracting important epistemic uncertainty factors for each site, collecting experts' opinions through a questionnaire survey, and developing LTs based on the above results with the estimator as a TI.

The selection of attenuation relations and setting of the upper limits of seismic ground motion were specified as the epistemic uncertainty common to individual sites. Important epistemic uncertainties specific to each site are as shown in Table 1.

The detailed assessment process is discussed in Section 3.1.

**Table 1. Important epistemic uncertainty in each site**

Site	Epistemic Uncertainties
Common	Selection of attenuation relations, setting of upper limits of earthquake motion
Site 1	Estimation of the length and magnitude of the active fault located in the vicinity of the site, separation of the source region
Site 2	Estimation of the active fault with low certainty in the vicinity of the site
Site 3	Estimation of a sequential inter-plate earthquake in the vicinity of the site
Site 4	Estimation of the sea area fault group belonging to the grand active fault (MTL) in the vicinity of the site
Site 5	Estimation of sequential inter-plate earthquakes in the vicinity of the site

### **3.3 Fault model (1999 - 2005)**

On the other hand, a technique to estimate strong motions caused by an active fault using a fault model has been established in recent years. Accordingly, PSHA was conducted for Site 3 through the use of a fault model. In addition to the repeated occurrence of inter-plate earthquakes in the vicinity of Site 3, the Tokai Earthquake (Soutei Tokai Earthquake) is forecasted to occur beneath the site. Although those earthquakes may occur sequentially, the estimation of the recurrence time includes epistemic uncertainty. Subsequently an LT was developed based on the expert opinions to conduct SHA. The detailed process is discussed in Section 3.2.

### **3.4 Verification Efforts on the Methodology**

JNES held a workshop on uncertainties in PSHA on April 19, 2004 to collect the opinions of researchers and practitioners in Japan and the United States. As a result, it was commonly recognized that the use of an LT for epistemic uncertainty in PSHA is effective.

In light of those experiences, JNES proposed a method to utilize experts' judgment (Expert Utilization Level 1 - Expert Utilization Level 3). In addition, JNES reflected the utilization method in the SPSA

Standard of the Atomic Energy Society of Japan [2], which will be published for the public comments. At present, the method to utilize experts' judgment is under study for applicability. The details of the study are discussed in Chapter 4.

#### 4. Trial PSHA for NPP Sites

##### 4.1 Seismic Hazard of Site 4 (Using Attenuation Relations)

JNES analyzed the seismic hazard of Site 4 by use of attenuation relations concern to the acceleration response spectrum.

There is a grand active fault (Median Tectonic Line (MTL)) in the vicinity of Site 4 extending over approximately 300 km from east to west. Site 4 is estimated to be influenced by the fault group located in the sea area in front of the site (sea area fault group) and a part of the fault group close to the site in the terrestrial area (terrestrial area fault group). Since be located so close to the site, the sea area fault group was estimated to exert a significant influence on the seismic hazard of the site.

The Headquarters for Earthquake Research Promotion of the Ministry of Education, Culture, Sports, Science and Technology (hereinafter referred to as the "Headquarters for Promotion") estimated the fault length resulting from the concurrent activity of the sea area fault group and the terrestrial area fault group to be 130 km, the mean earthquake recurrence interval to be 1,000 years to 2,900 years, and the chronological period of the latest seismic activity to the 16th century.

In accordance with the above findings, JNES implemented the SHA in the following steps based on expert opinions:

###### (1) First step

The segmentation and activity level were set up based on the estimation of the Headquarters for Promotion. The mean earthquake recurrence interval was treated as an epistemic uncertainty. Prior to the PSHA with the sensitivity analysis on the mean earthquake recurrence interval was implemented to select branches of provisional LT-1.

An expert suggested regarding the segmentation that there was another approach that assessed the seismic hazard dealing with the sea area fault group and the terrestrial area fault group separated, contrary to the approach of the Headquarters for Promotion that assessed the seismic hazard with both fault groups combined as 130 km in fault length.

The sensitivity analysis revealed that the terrestrial area fault group is estimated to exert little influence on the seismic hazard, because it is located so far away from the site. For the sea area fault group, however, it was revealed that collection of research data is insufficient for estimating seismic activities. An expert also pointed out that there is little evidence for the occurrence of earthquakes in the 16<sup>th</sup> century.

###### (2) Second step

It was decided that the segmentation, the mean earthquake recurrence interval, and the chronological period of the latest seismic activity will be dealt with as an epistemic uncertainty based on the findings and experts' opinions in the first step. And the provisional LT-2 was constructed to implement the second PSHA.

The following comments were given by an expert: "It may be possible to use the activity level obtained from a trench survey for the terrestrial area fault group as a guide for the assessment of the activity level of the sea area fault group," and "As for attenuation relations, it may be possible to use an attenuation relation based on the shortest distance, R, for which the earthquake motion is estimated to be larger proportionally to the fault length.".

Based on the comments of experts given in the second step, the provisional LT-3 was constructed.

(3) *Third step*

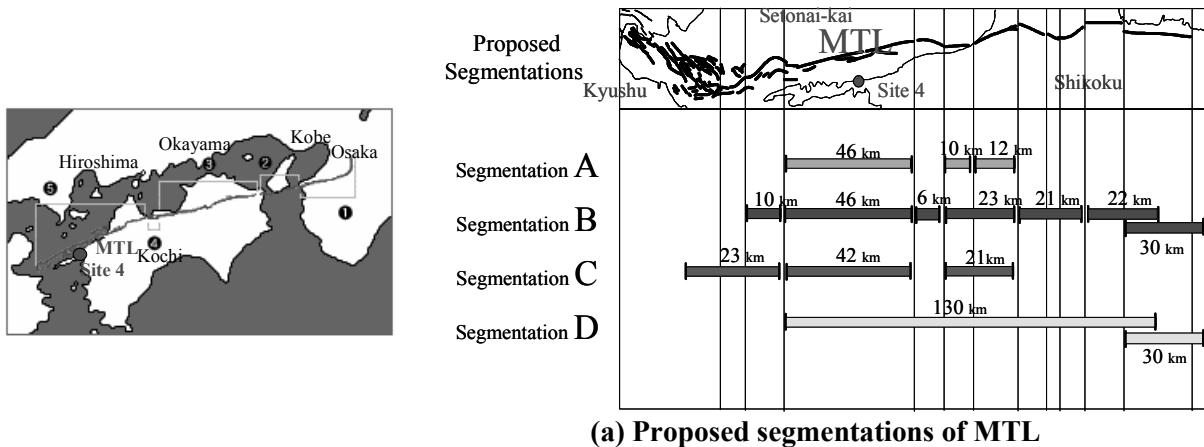
A questionnaire survey was conducted on the provisional LT-3 with experts. The questionnaire was presented to them with the sensitivity analysis results on individual branching items attached as reference data.

Regarding the assessment of the activity of the sea area fault group, the answer obtained from the experts was a suggestion to use the activity level of the terrestrial area fault group as a guide. It was recommended by an expert specialized in the field of attenuation relation to “apply the latest attenuation relation based on the shortest distance, R.”

The final LT was constructed with the branching items of the provisional LT-3 reassessed based on the results of the questionnaire. The seismic hazard was analyzed based on the final LT. The final LT and the results of the PSHA are shown in Figure 1.

It was recognized anew that, when the influence of epistemic uncertainty is extremely large and complicated, it is important to utilize experts' judgment for the formulation of an LT to secure accountability and transparency in SHA.

Figure 1. Probabilistic seismic hazard of site 4



#### ■ Seismic source model of MTL

Evaluate organization	Length	Recurrence Interval (year)	Recent event
Sui-hon	130km	1,000~1,400	D.C.16
	130km	2,500~2,900	D.C.16
	130km	2,500~2,900	before 2400 year
Ehime Pref.	46km	1,400~1,600	after D.C.12
	46km	3,500~4,000	after D.C.10
AIST	42km	2,500~3,500	after D.C.13
	42km	1,000	after D.C.13
	110km	2,500~2,900	before 2,400 year
An Expert			

Sui-hon : The Headquarters for Earthquake Research Promotion of the Ministry of ECSST  
 AIST : Advanced Industrial Science and Technology

#### ■ Attenuation relation (for crust)

- Tai-toku-I (Xeq,  $\zeta = 0.53$ )
- Abrahamson & Silva (R,  $\zeta = 0.43$ )
- Takahashi et al. (2004,R,  $\zeta = 0.62$ (PGA))

#### (for Intra and inter plate)

- Tai-toku-I (Xeq,  $\zeta = 0.53$ )
- Takahashi et al. (2004,R,  $\zeta = 0.62$ (PGA))

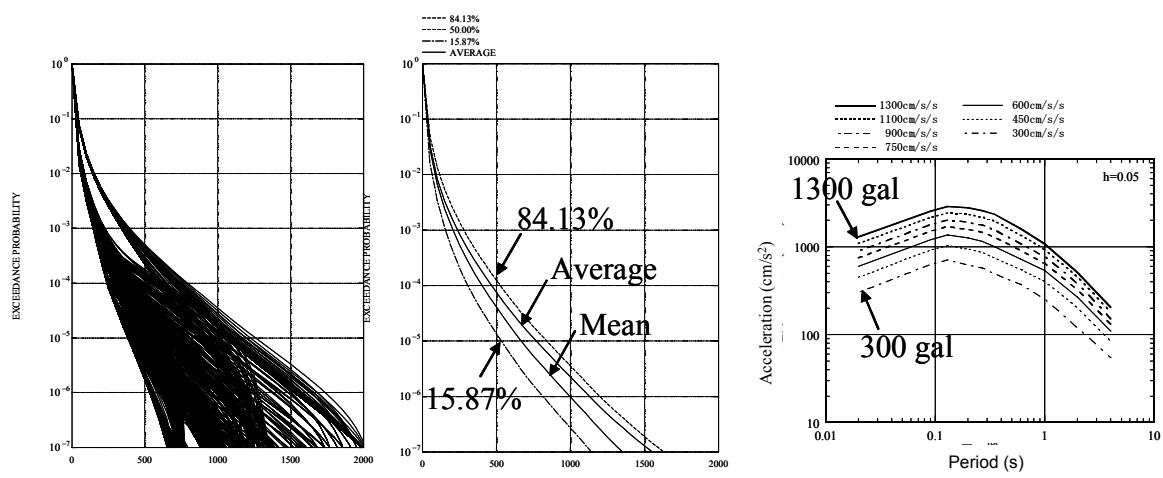
#### (for offshore faults of MTL)

- Tai-toku-I (Xeq,  $\zeta = 0.53$ )
- Abrahamson & Silva (R,  $\zeta = 0.43$ )

#### ■ Upper limit of ground motion

- Median+2· $\zeta$
- Median+3· $\zeta$
- Median+∞· $\zeta$

(b) Logic Tree of Site 4



(c) Seismic hazard of each path of LT

(d) Seismic hazard

(e) Uniform hazard spectrum

#### **4.2 Seismic Hazard of Site 3 (Using a Fault Model)**

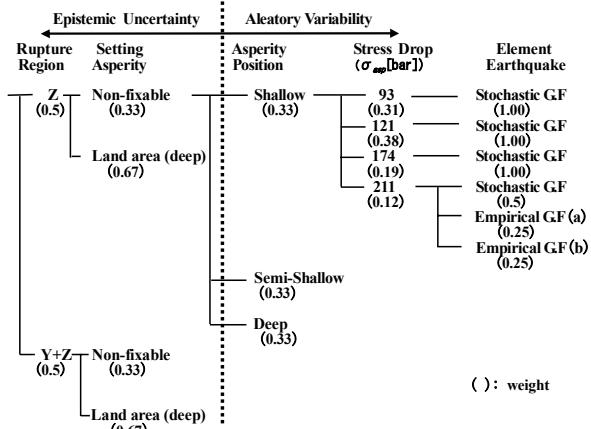
A PSHA was implemented for the great inter-plate earthquakes with rupture regions along the NANKAI Trough by use of a fault model. The idea on rupture region and on setting asperity were treated as epistemic uncertainty and the positions /stress drops of asperities, rupture velocities, and element earthquakes were treated as aleatory variability. Figure 2(a) shows the LT applied to the estimation. The level of expert opinion utilization in this PSHA corresponds to Level 1 in the table 2.

Two branches relating to the idea on setting asperities were introduced; a case of setting asperities in a land area (deep) with a weight of two-thirds and a case of setting it non-fixable with a weight of one-third. The former was assigned with a larger weight than the latter because, given that plate coupling in a land area is relatively strong, it is highly likely that it will become an asperity of an earthquake occurring in future. On the other hand, models of rupture Region (Z) and Region (Y+Z) along the NANKAI Trough (see Figure 2(b)) were introduced with an equal weight of one-half.

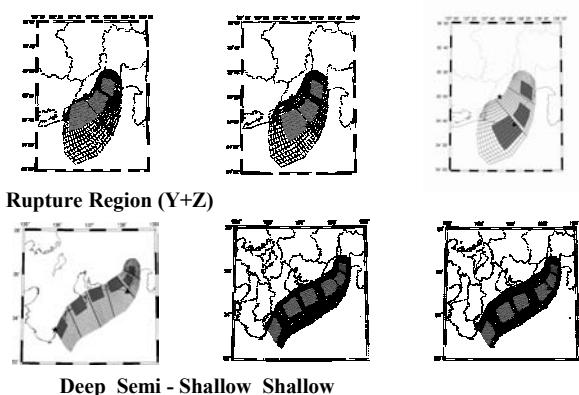
The positions of asperities treated as aleatory variability were modeled by the following three categories, viz. shallow, semi-shallow, and deep, as shown in Figure 2(c), with equal weights of one-third. For the stress drops of asperities, weights were assigned in accordance with the probability distribution based on the study results by Somerville et al. [3] on the ratio of asperity area to fault rupture area, as shown in Figure 2(d). For element earthquake, artificial ground motion as stochastic Green's function was applied except the case when stress drop is 211 bar. In the case of 211 bar, an equal weight of one-half was assigned to an artificial ground motion and two small-event records as empirical Green's function. In the latter case, the weight 0.25 was assigned to each element earthquake. The variability in rupture velocity was expressed by setting up 21 sets of seismic wave generation points distributed randomly within the sub-faults and assigning an equal weight of one-twenty-first for each set.

The seismic hazard was assessed for the great inter-plate earthquake at Site 3 by use of the logic tree discussed above and the earthquake occurrence probabilities in the respective rupture regions shown in Figure 2(e). The results are shown in Figure 2(f). The seismic hazard curve indicated that the annual exceedance probability of ground motion is higher for the rupture Region (Z) than for the Region (Y+Z) due to the difference in earthquake occurrence probability and that the annual exceedance probability becomes higher in the range of strong ground motion in the case of setting asperities non-fixable than the case of setting it in a land area.

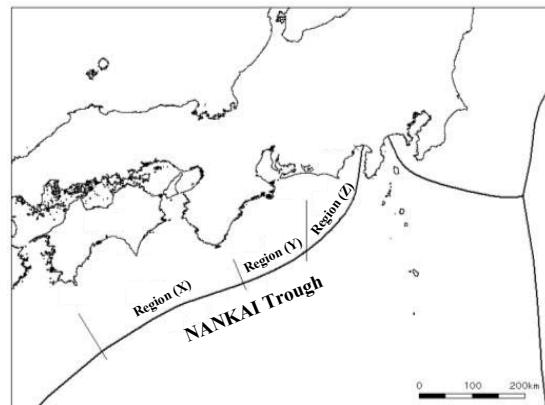
Figure 2. Probabilistic seismic hazard assessment for Site 3



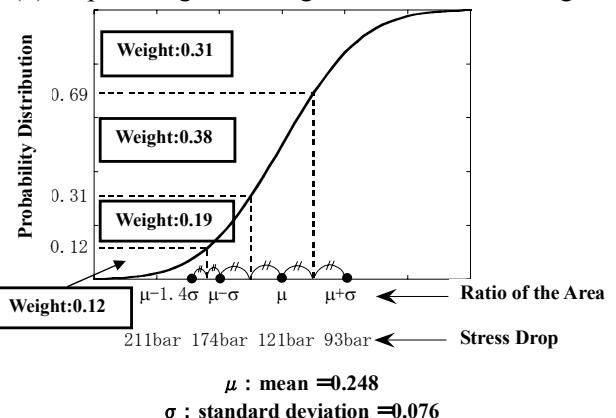
(a) Logic tree for the great inter plate earthquake Rupture Region (Z)



(c) Asperity position in fault model



(b) Rupture regions along the NANKAI Through

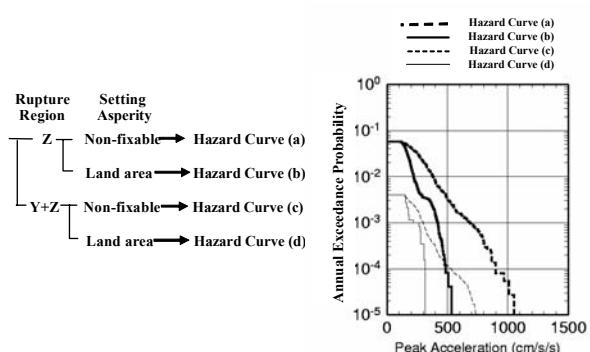


(d) Weighting for stress drop of asperity

Rupture Region	Occurrence Interval (year)	Scatter	Latest Activity Year (A.D.)	Annual Occurrence Probability (%)
Z	118.8	0.16~0.24	1854	5.7
Y+Z	86.4	0.18~0.24	1944	0.4

Assessment Year: 2000 A.D.

(e) Earthquake occurrence probabilities



(f) Seismic hazard curves evaluated by logic tree

## 5. JNES efforts for methodological enhancement of LT scheme

### 5.1 Formulation of utilizing experts' judgment

JNES recognized the several technical problems and the problems on the process of treatment of uncertainties in PSHA. Their problems are shown as below.

- ① Technical problems.
  - Evaluation of segmentation and of active fault.
  - Evaluation of the relation among an observed fault on the surface of the earth and earthquake source fault.
  - Settlement of the upper and lower layer of seismic zone.
  - Evaluation of the sea area fault.
  - Settlement of the source parameter of a fault.
- ② The problems on the process of treatment of uncertainties.
  - Harmony among science and engineering.
  - Utilizing expert judgment in order to the degree of difficulty.
  - Possessing the information jointly among experts.

JNES formulated a method of utilizing experts' judgment according to the degree of technical difficulty in LT formulation in fiscal 2006 based on their so-far accumulated experience. Table 2 shows the utilization method. In addition, a method of selecting experts, TI, and TFI was formulated together. These expert utilization methods were formulated by referring the utilization method discussed in the SSHAC Report [4] as a guide.

**Table 2. Method of Utilizing Experts' Judgment**

<ul style="list-style-type: none"> <li>■ Setting of degree of technical difficulty and expert utilization standards in logic tree formulation In formulating a logic tree, any of the following three stages of expert utilization standards will be set up considering the degree of technical difficulty of the epistemic uncertainty factors targeted for assessment. Given that the operation procedure will vary greatly according to the expert utilization level, the selection of the utilization level should be made considering the reliability and accountability associated with the logic tree to be formulated.</li> </ul>
<p><b>a) Expert utilization level 1</b> This level will be applied when the influence of the seismic hazard on uncertainties is relatively small. A TI will formulate a logic tree by assessing the community distribution (objective distribution of uncertainties at the time comprehensively assessed by a scientist group) based on literature review and his/her own experience.</p>
<p><b>b) Expert utilization level 2</b> This level will be applied when the influence of the seismic hazard on uncertainties is relatively large and expert opinion on important items varies among experts. A TI will formulate a logic tree by improving and narrowing down the models and estimating the community distribution in and communication with experts by contacting model proponents or related experts to seek their views reasons or through discussions and so on with these experts gathered together.</p>
<p><b>c) Expert utilization level 3</b> This level will be applied when the uncertainty factors of the seismic hazard are widely ranged and diverse, are considered to be important and complicated, and whose assessment requires substantial time, expense, human resources, and, in some cases, various researches. A TFI will formulate a logic tree by organizing an expert panel, utilizing the experts not as model proponents but as objective estimators for uncertainties, and integrating the community distribution estimated by the panel in a fair manner.</p> <p>The roles of the TI in Expert Utilization Level 1 and Expert Utilization Level 2 may be assumed by the seismic hazard evaluator by him/herself.</p>

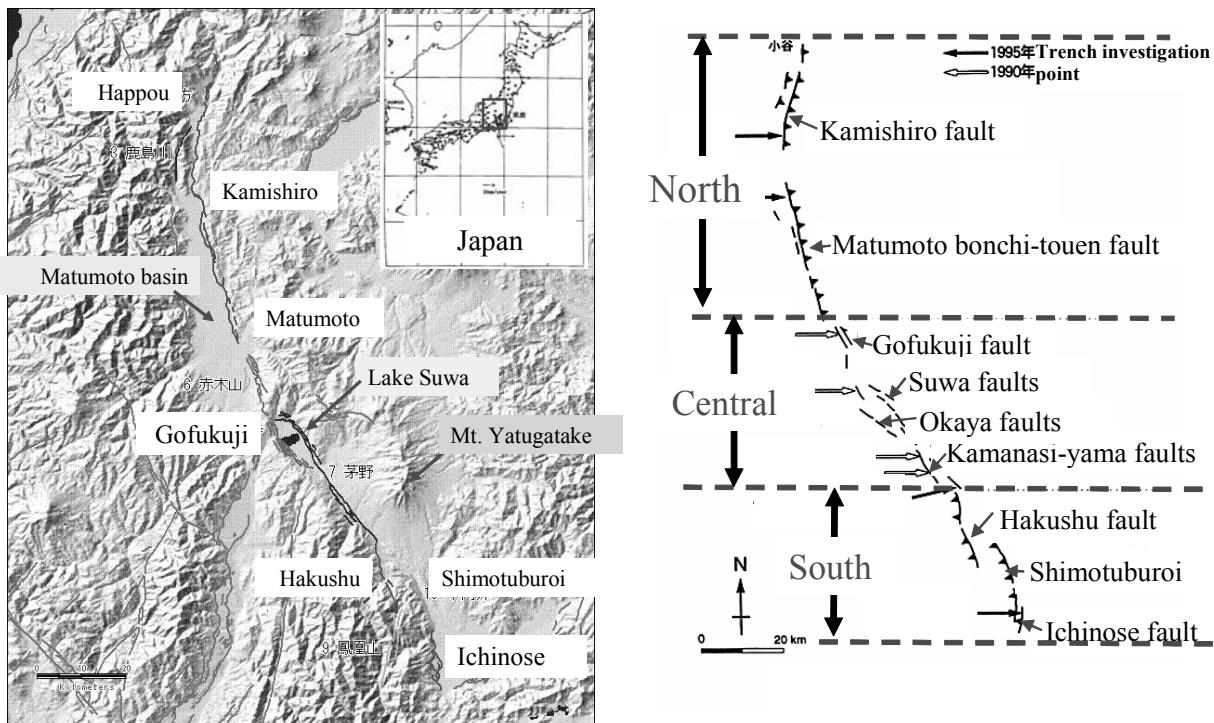
TI: Technical Integrator; a technical manager for logic trees

TFI: Technical Facilitator/Integrator, a technical manager for logic trees and organizer integrating experts' opinions

## 5.2 On-going WG activities

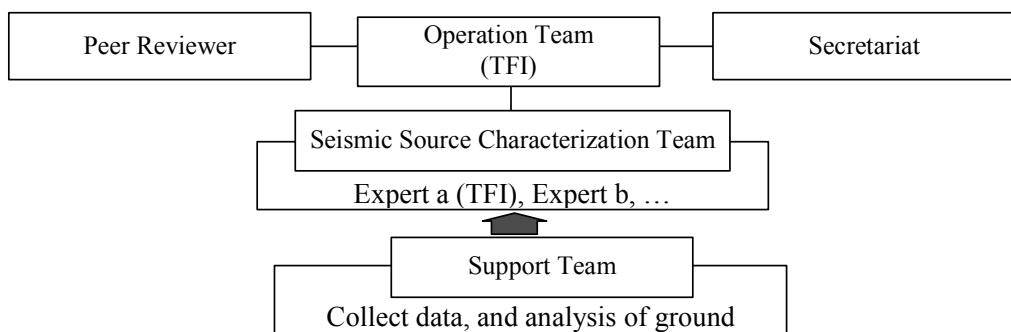
JNES is currently in the process of a feasibility study for Expert Utilization Level 3 of the method of utilizing experts' judgment. The study subject is the Itoigawa-Shizuoka Tectonic Line (ISTL) shown in Figure 3. The ISTL is a grand active fault located approximately in the center of Japan. ISTL is divided into three segments (northern segment, middle segment, and southern segment). The ISTL includes epistemic uncertainties in terms of the estimation of the segmentation, dip angle, activity time, etc. In the segmentation, in particular, experts' opinions vary on whether the southern segment will become active at the same time as the northern segment and the middle segment.

Figure 3. Itoigawa-Shizuoka Tectonic Line (ISTL)



### (1) Implementing Organization

An expert team on uncertainties in the model for the ISTL was organized. The expert team implements discussion and so on concerning epistemic uncertainty under the management of the TFI. The study process and study results of the expert team are reviewed by peer reviewers.



**1) Selection of TFI and Their Roles**

TFI organized an expert panel to utilize individual experts not as model proponents but as objective estimators for uncertainties. TFI will fairly integrate the distribution of experts' opinions (community distribution) concerning the epistemic uncertainty evaluated by the panel to develop an LT.

**2) Selection of Experts and Their Roles**

Six experts were selected in total. The experts' roles include expressing their own opinions as individual experts (including underlying data and evaluations), expressing their views on community distribution, and participating in discussions among experts.

**3) Objective of Discussions among Experts**

This discussion process intends to summarize the distribution of the final opinions of experts in the form of an LT in discussion among experts, with information about the epistemic uncertainty shared among them.

**4) Peer Review**

This process reviews whether studies have been implemented in the right direction and with the proper manner in accordance with its objective. The peer reviewers will perform an interactive review with issues raised to the TFI and/or expert group on the study approach if any as needed.

**5) Operation Team**

The operation team will coordinate matters, processes and so on concerning the holding of discussions among experts.

**6) Support Team**

The support team will collect basic data and perform sensitivity analysis providing the results to the expert team as reference data.

**(2) Discussion among experts and questionnaire surveys**

Five rounds of discussion among experts and two questionnaire surveys were conducted from December 2005 to October 2006.

**1) The First Questionnaire Survey.**

The first questionnaire survey investigated the interpretation and opinions of experts on the following basic items:

- Interpretation of a seismogenic fault zone.
- Definition of a grand active fault.
- Estimation of earthquake magnitude.
- Geometrical forms and underground structures.
- Reference data useful in evaluating the distribution of the ISTL.
- Seismic activity range of earthquakes generated in the ISTL.

**2) The First Discussion among Experts.**

Experts' opinions on the respective survey items were introduced based on the results of the first questionnaire, and discussions were conducted on them with the objective of sharing the same recognition among the experts.

**3) The Second Discussion among Experts**

The discussion was conducted on the summary of the survey items in the first questionnaire based on the first discussion among experts.

4) The Second questionnaire Survey

Experts' opinions were collected on how understanding should be encouraged regarding the Time-Space Diagram of ISTL (a diagram with the chronological periods of earthquake activities indicated by trench survey points) [5] formulated by the Japan Society of Civil Engineers based on the trench survey.

5) The Third Discussion among Experts

The discussion was conducted on the summary of the second questionnaire that surveyed the interpretation and so on of the survey results of adjacent trenches. The Active Fault Assessment Method and Procedure (Draft), which was compiled with previous experts' opinions collected, was presented by the TFI and discussion was conducted on this. An explanation was given by an expert on the formulation of an earthquake scenario based on survey results of active faults and the dynamic simulation of fault rupture. The results of the sensitivity analysis of ground motion by use of attenuation relations based on the advice of experts were reported by the support team for reference.

5) The Fourth Discussion among Experts

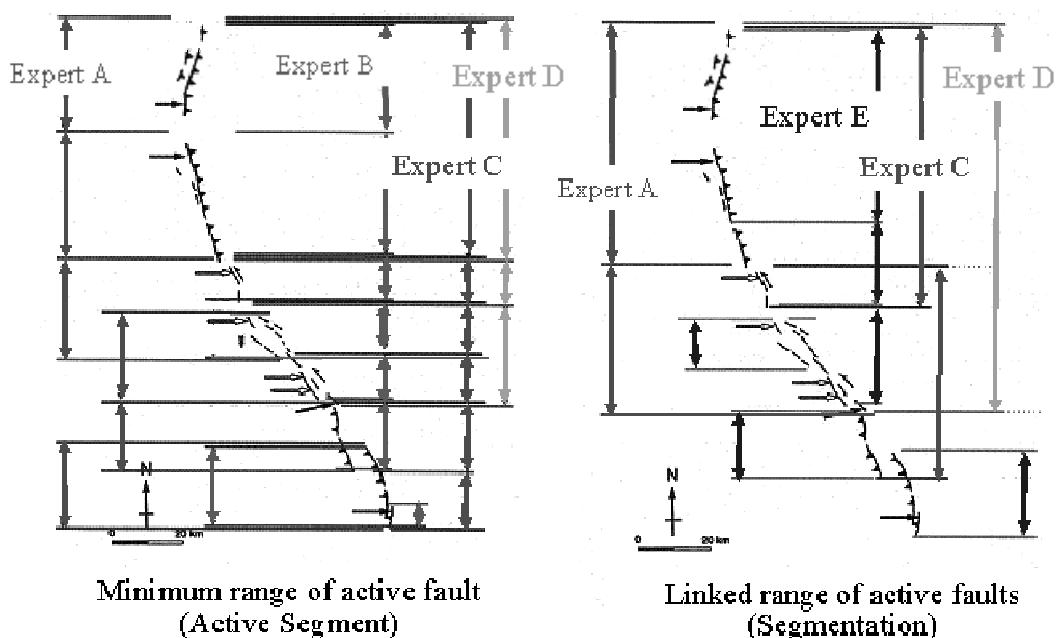
The discussion was conducted on the complex motion of active faults. The combined movement patterns of active faults likely to be based on experts' opinions were presented by the support team in response to a request from the TFI, and discussion was conducted on whether consideration needs to be given to all the combined movement patterns. An explanation was given by an expert on the estimation of the sense of fault displacement and the magnitude and frequency of earthquakes, regarding the possible extent of earthquake estimation based on trench data and the relationship between earthquake source faults and surface earthquake faults.

5) The Fifth Discussion among Experts

Currently, a fifth discussion among experts is expected to be held to evaluate the distribution of the experts' judgment on epistemic uncertainty of the ISTL for the development of the LT.

The examples of current expert judgment about ISTL are shown in Figure 4. Expert judgments are different about the minimum range of active fault, the linked range of active faults and etc.

**Figure 4. Example of Experts Judgement about ISTL**



## 6. Conclusion

Uncertainties associated with PSHA include aleatory variability and epistemic uncertainty. In order to secure accountability and transparency in the results of PSHA, it is important to deal with epistemic uncertainty appropriately. As a method of dealing with epistemic uncertainty, an LT technique with experts' judgment utilized is effective.

JNES has been analyzing seismic hazards by use of attenuation relations, and by developing an LT concerning the epistemic uncertainty associated with earthquakes at five sites and seismic ground motion estimation. In addition, seismic hazards have been analyzed by use of fault models and by developing an LT concerning the epistemic uncertainty associated with hypocenter parameters for a site with a huge interplate earthquake estimated in the vicinity.

Based on the above study results, JNES formulated the Method of Utilizing Experts' Judgment (Expert Utilization Level 1, 2, and 3) in accordance with the influence level of the epistemic uncertainty and the level of complexity of assessment and so on associated with PSHA.

Currently, JNES is in the process of developing an LT by obtaining the distribution of experts' judgment concerning the epistemic uncertainty of the ISTL based on Expert Utilization Level 3.

In the current applicability study, the prospect was obtained that an LT could be formulated with high accountability by utilizing experts' judgment for epistemic uncertainty as a result of the understanding and strong will of the TFI and the experts.

The JNES efforts for enhancing the PSHA methodologies including the logic tree scheme are intended to meet the needs for SPSA that will be a reality in the design process of Nuclear Power Reactor Facilities, issued by the Nuclear Safety Commission of JAPAN. Such a perspective comes from the new Examination Guideline for Seismic Design for NPP revised in September 2006.

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## Probabilistic Evaluation of Near-Field Ground Motions due to Buried-Rupture Earthquakes Caused by Undefined Faults

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### ***Abstract***

The Nuclear Safety Commission of Japan (NSC) was discussing the revision of Safety Examination Guideline for Seismic Design of Nuclear Power Plants of Japan, and finished the discussion and revised them in September 19, 2006. In the discussion, one of the main issues was the design earthquake motion due to close-by earthquakes caused by undefined faults. This paper proposes a probabilistic method for evaluating ground motions and their predicted frequencies by covering variations of earthquake magnitude and location of undefined faults by strong motion simulation technique (hybrid technique) based on fault models, and evaluated probabilistic response spectra (uniform hazard spectra) due to close-by earthquakes. These uniform hazard spectra evaluated by the hybrid technique are compared with those evaluated by an empirical approach. The response spectra with a damping factor of 5% at 0.02s simulated by the hybrid technique are about 160, 340, 570, and 800 cm/s/s for annual exceedance probabilities of  $10^{-3}$ ,  $10^{-4}$ ,  $10^{-5}$ , and  $10^{-6}$ , respectively, which are in good agreement with the response spectra evaluated by the empirical approach. It is also recognized that the response spectrum proposed by Kato et al. (2004) as the upper level of the strong motion records of buried-rupture earthquakes corresponded to the uniform hazard spectra of annual exceedance probability of  $10^{-5}$  -  $10^{-4}$ .

### **1. Introduction**

The 1995 Hyogo-Ken Nambu Earthquake in Japan (Japan Meteorological Agency magnitude  $M_J=7.3$ ) caused the Hanshin-Awaji earthquake disaster, and intensive research on this earthquake and strong motion records promoted the Nuclear Safety Commission of Japan to review the examination guideline for seismic design of nuclear power plants. This disaster also promoted the Japanese Diet to make a new law to establish the Headquarter for Earthquake Research Promotion in the Japanese Government.

According to recent earthquake research, the main issues of the review of the previous guideline were strong ground motions due to close-by earthquakes caused by undefined faults, intra-slab earthquakes that generate larger short-period seismic motions than inland earthquakes or subduction earthquakes, and probabilistic evaluation of strong ground motions (seismic hazard). This is because that the magnitude of close-by earthquakes was specified by M6.5 in the previous guideline, but larger-magnitude inland earthquakes caused by undefined faults actually occurred in Japan. And the intra-slab earthquakes and their strong motions were not considered explicitly in the present guideline, and an evaluation of “residual risk” based on the concept that it cannot deny a possibility to occur the over design earthquake motion become a social requirement for the safety assessment of nuclear power plants.

For the first issue, Kato et al. (2004) proposed a deterministic design ground motion based on an upper level of strong motion records on an outcrop due to close-by inland earthquakes of undefined faults. Their approach is convincing for using actual records, however, it has a limitation of the records with regard to variation of magnitudes of the earthquakes and location of undefined faults.

Hence, this paper proposed a probabilistic method for covering these variations using a strong motion simulation technique based on fault models, and calculated probabilistic response spectra due to close-by earthquakes caused by undefined faults.

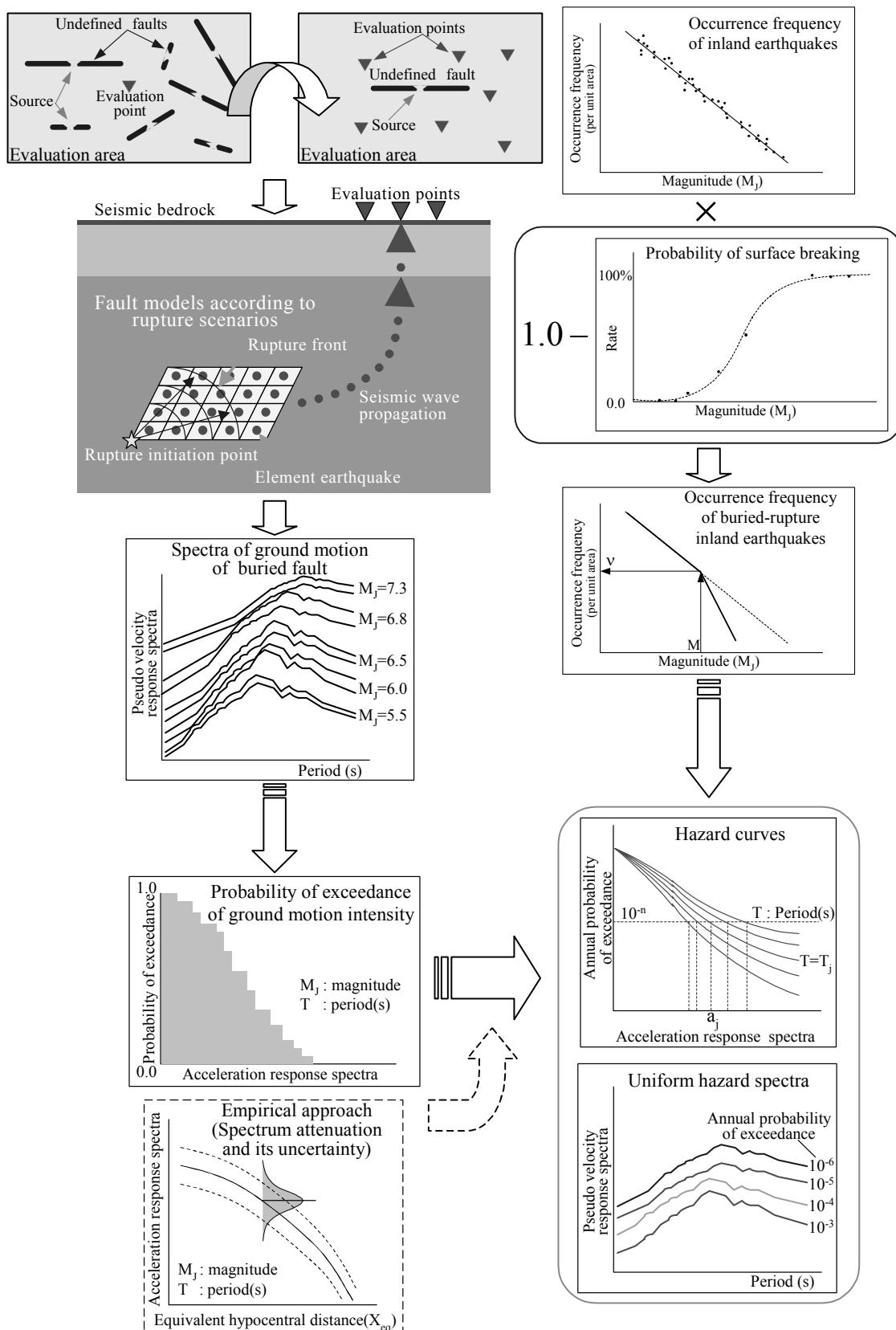
## 2. Flow of procedure

We adopted the procedure proposed by Dan et al. (2001) for probabilistic evaluation of near-field ground motions due to buried-rupture earthquakes caused by undefined faults. This procedure consists of four steps: (1) earthquake occurrence modeling, (2) fault rupture scenario modeling, (3) strong motion simulation, and (4) hazard curve calculation.

Figure 1 shows the procedure to evaluate the probability of annual exceedance of response spectra for buried-fault rupture earthquakes from the relation between the magnitude and the averaged occurrence frequency for inland earthquakes and the relation between the magnitude and the surface breaking probability analytically estimated from the surface deformation by fault (Kagawa et al., 2005). In the procedure, we assumed that the occurrence of buried-fault rupture earthquakes is a Poisson process.

As shown in the upper left of Figure 1, we considered that the distribution of undefined faults around the evaluation point is equivalent to the distribution of evaluation points around the undefined fault. The strong ground motions were simulated at every evaluation points (on seismic bedrock).

Figure 1. Flow chart of procedure



### 3. Methods

#### 3.1 Annual occurrence frequency of buried-fault rupture earthquakes

We obtained the following Gutenberg-Richter model of the annual cumulative number of earthquakes per  $314\text{km}^2$  in terms of the magnitudes of inland earthquakes from the JMA earthquake catalogue:

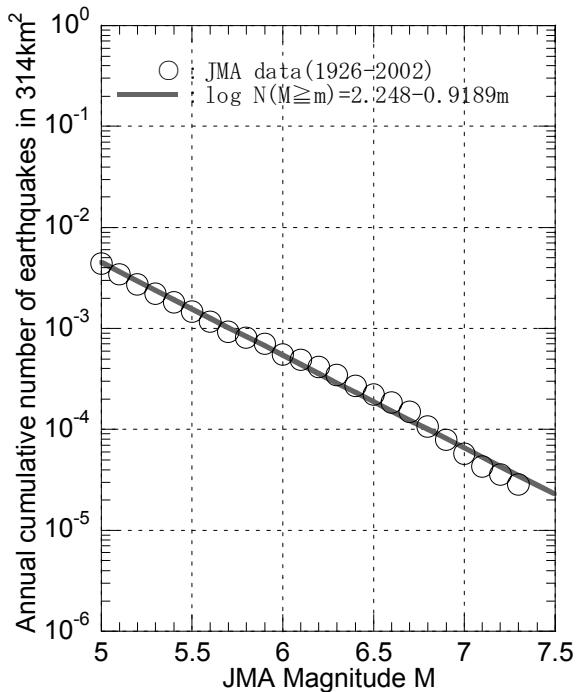
$$\log_{10} N(M \geq m) = 2.248 - 0.9189m \quad (\text{Eq. 1})$$

In order to obtain this Eq.1, we used 617 data of Japanese inland earthquakes in a source area of 571,000  $\text{km}^2$  with magnitudes larger than 5 and hypocentral depths shallower than 25km in the JMA earthquake catalogue from 1926 to 2002. The epicenters of the earthquakes are shown in Figure 2, and the model of the annual cumulative number of earthquakes are shown in Figure 3.

**Figure 2. Source area model and distribution of epicenters of inland earthquakes**



**Figure 3. Annual cumulative number of occurrence of inland earthquakes in  $314\text{ km}^2$**

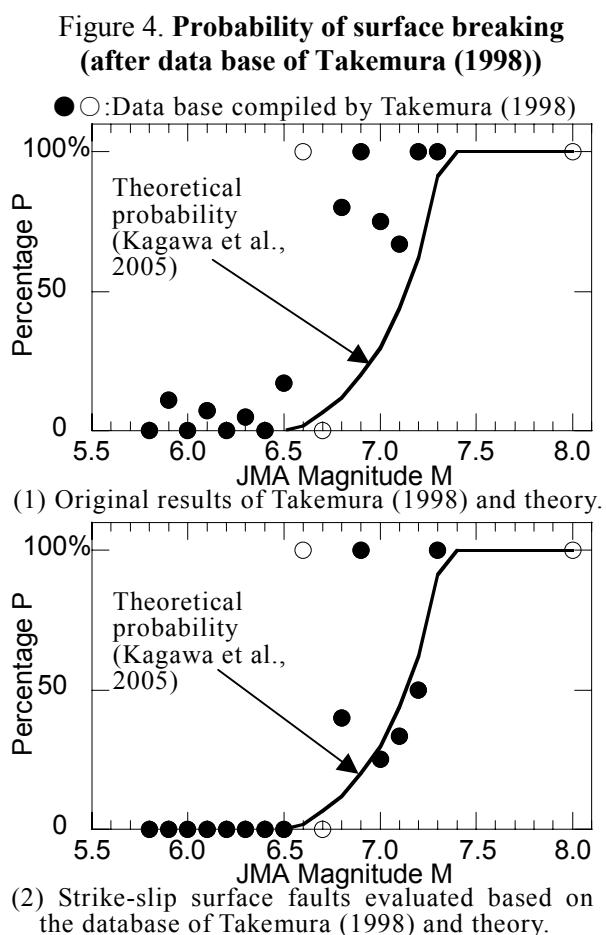


Kagawa et al. (2005) proposed a probability of surface break by faults,  $p_{sf}(M)$ , in terms of magnitude and depth of fault based on the eternal displacement criteria of 5 cm on the bedrock surface after analytical study on the various strike-slip faults by referring to the actual earthquakes. The probability of surface break for strike-slip fault is shown in Figure 4. The upper part of Figure 4 compares with the percentage of surface break based on database of actual earthquakes identified by Takemura (1998) which is including the data of strike-slip faults and dip-slip faults. The open circles indicate that the total number of the events is only one for the corresponding magnitude. The lower part of Figure 4 compares the percentage of surface break of only strike-slip case evaluated from the database compiled by Takemura (1998) with the same theoretical result. As shown, they are in good agreement, and also recognize that the surface break of strike-slip faults may not occur under magnitudes of 6.5 and occur over 7.3.

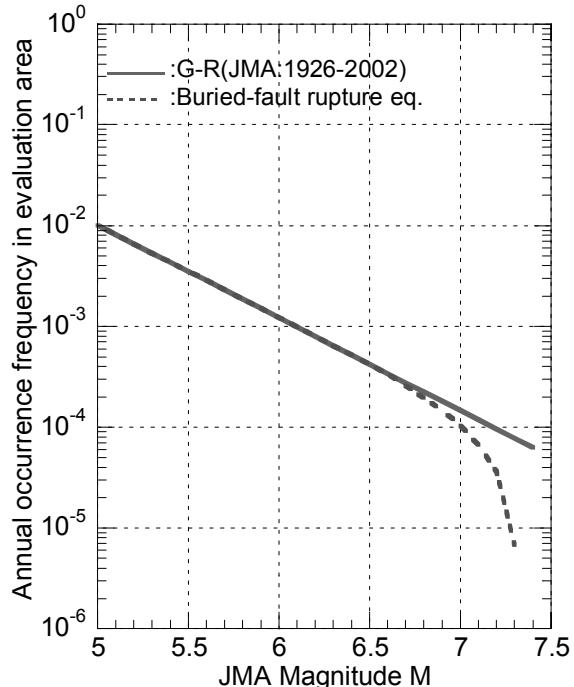
Consequently, the annual occurrence frequency of buried fault earthquakes with magnitude  $M=[m, m+0.1]$  can be obtained from the annual occurrence frequency of earthquakes and the probability of the surface break as Eq. 2.

$$\nu(M) = [N(M \geq m) - N(M \geq m + 0.1)] \times [1 - p_{sf}(M)] \cdot (1/\text{year}/314\text{km}^2) \quad (\text{Eq. 2})$$

Figure 5 shows the annual occurrence frequency of buried-fault rupture earthquakes in the evaluation area (dotted line). It is shown that the annual occurrence frequency of buried-fault rupture earthquakes decreased rapidly for magnitudes larger than 7.0.



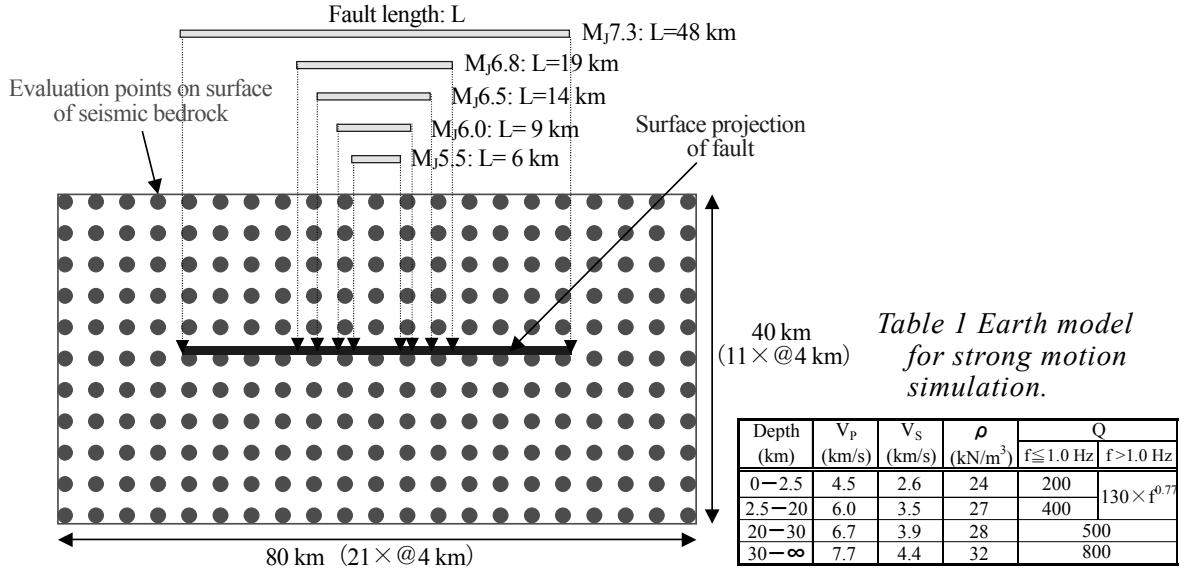
**Figure 5. Annual occurrence frequency of buried-fault rupture earthquakes in the evaluation area ( $3696\text{ km}^2$ )**



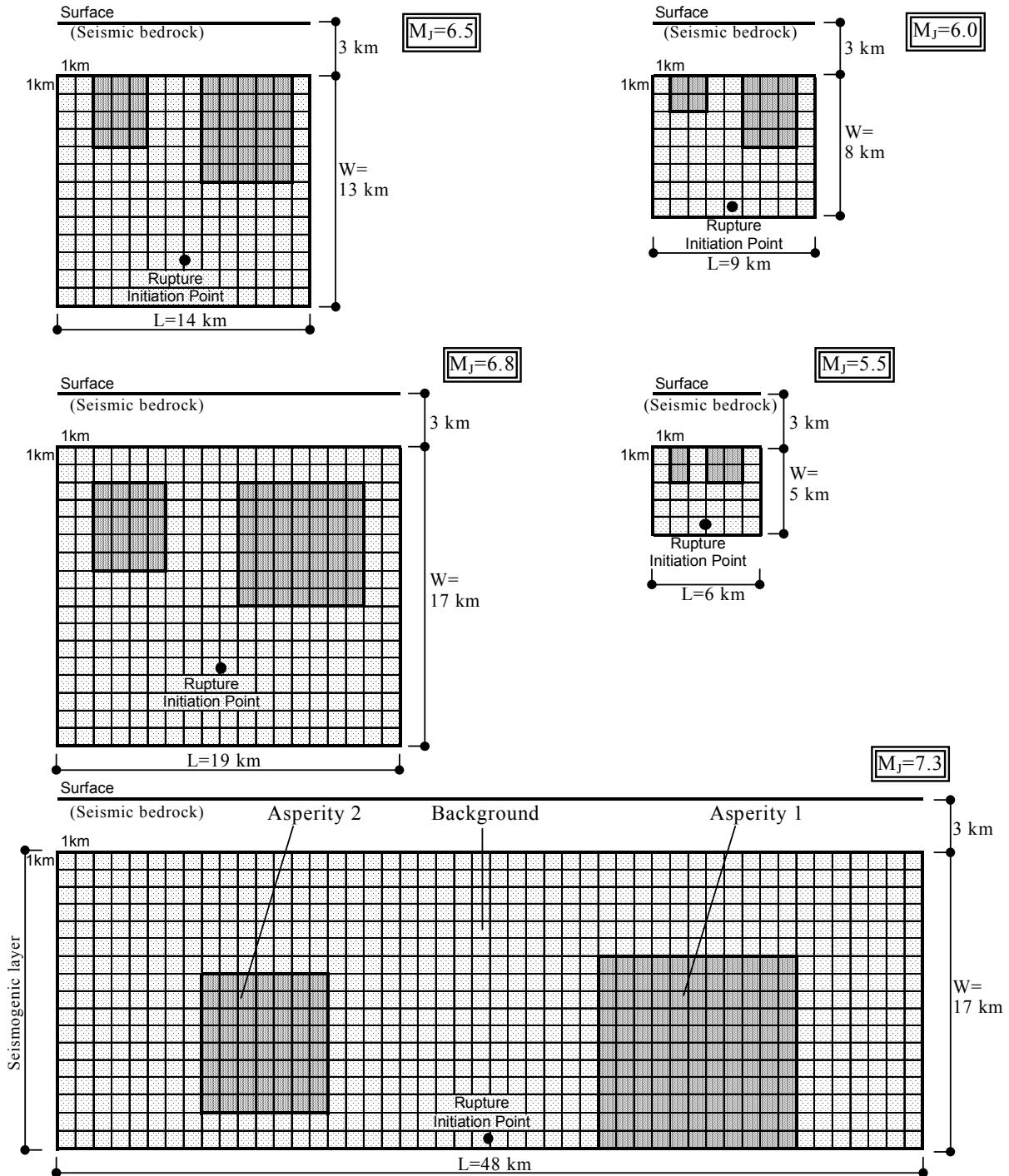
### 3.2 Fault rupture scenarios and fault models

We assumed 40 earthquakes scenarios without surface rupture for strike-slip faults with magnitudes of 5.5, 6.0, 6.5, 6.8, 7.1, and 7.3, considering variations of depth (shallow, intermediate, and deep) and stress drop (high, middle, and low) of the asperities. Then, we established fault models for each scenario based on the recipe proposed by Irikura and Miyake (2001) for strong motion prediction.

Figure 6 shows the evaluation surface area ( $40\text{km} \times 80\text{km}$ ) and the fault position for magnitudes of 5.5, 6.0, 6.5, 6.8, and 7.3. The earth model (velocity, density, and Q) is shown in Table 1. Figure 7 shows an example of the characterized scenario fault models with middle stress drop. An example of fault parameters is listed in Table 2.

**Figure 6. Evaluation area and fault position****Table 2. Example of fault parameters of shallow fault and high stress drop**

Parameters	Japan Meteorological Agency magnitude				
	5.5	6.0	6.5	6.8	7.3
Strike	N090E (EW direction)				
Dip	90° (vertical)				
Rake	0° (left-lateral strike-slip)				
Moment magnitude	M <sub>w</sub>	5.5	5.9	6.3	6.5
Seismic moment	M <sub>0</sub> × 10 <sup>-7</sup> [N· m]	2.00E+24	7.94E+24	3.16E+25	7.24E+25
Upper depth of the fault	[km]	3	3	3	3
Lower depth of the fault	[km]	8	11	16	20
Area of the entire fault	S [km <sup>2</sup> ]	29.3	73.6	185.0	321.5
Mean depth of the larger asperity	H [km]	4.0	4.5	5.5	6.0
Short-period level	A × 10 <sup>-7</sup> [N· m/s <sup>2</sup> ]	3.74E+25	5.93E+25	9.40E+25	1.24E+26
Area of the asperities	S <sub>asp</sub> [km <sup>2</sup> ]	4.7	11.8	29.6	51.4
Area of the larger asperity	S <sub>1</sub> [km <sup>2</sup> ]	3.3	8.2	20.7	36.0
Area of the smaller asperity	S <sub>2</sub> [km <sup>2</sup> ]	1.4	3.5	8.9	15.4
Area of the background	S <sub>b</sub> [km <sup>2</sup> ]	24.6	61.9	155.4	270.1
Fault length	L [km]	6	9	14	19
Fault width	W [km]	5	8	13	17
Length of the larger asperity	L <sub>1</sub> [km]	2	3	4	6
Width of the larger asperity	W <sub>1</sub> [km]	2	3	5	6
Length of the smaller asperity	L <sub>2</sub> [km]	1	2	3	4
Width of the smaller asperity	W <sub>2</sub> [km]	1	2	3	4
Upper depth of the larger asperity	[km]	3	3	3	4
Rupture initiation point	Deeper point in the center of the fault				
Rupture propagation velocity	[km/s]	2.7			
Rupture propagation mode		Radial			
Rise time	τ [s]	0.26	0.41	0.64	0.85
Shear-wave velocity	β [km/s]	3.5			
Rigidity	μ × 10 <sup>-1</sup> [N/m <sup>2</sup> ]	3.31E+11			
Averaged slip on the entire fault	D [cm]	21	33	52	68
Slip ratio	α	1.96			
Slip on the larger asperity	D <sub>1</sub> [cm]	45	71	113	149
Slip on the smaller asperity	D <sub>2</sub> [cm]	29	47	74	97
Slip on the background	D <sub>b</sub> [cm]	17	27	42	56
Seismic moment of the larger asperity	M <sub>01</sub> × 10 <sup>-7</sup> [N· m]	4.89E+23	1.95E+24	7.74E+24	1.77E+25
Seismic moment of the smaller asperity	M <sub>02</sub> × 10 <sup>-7</sup> [N· m]	1.37E+23	5.46E+23	2.17E+24	4.98E+24
Seismic moment of the background	M <sub>0b</sub> × 10 <sup>-7</sup> [N· m]	1.37E+24	5.45E+24	2.17E+25	4.97E+25
Effective stress on the larger asperity	σ <sub>1</sub> × 10 <sup>-1</sup> [MPa]	191	191	191	191
Effective stress on the smaller asperity	σ <sub>2</sub> × 10 <sup>-1</sup> [MPa]	191	191	191	191
Effective stress on the background	σ <sub>b</sub> × 10 <sup>-1</sup> [MPa]	24	24	24	24
Length of the calculation region	[km]	80			
Width of the calculation region	[km]	40			

**Figure 7. Fault models for earthquakes of shallow fault and middle stress drop**

### 3.3 Hybrid technique for strong motion simulation

Strong ground motions on seismic bedrock were simulated for the fault models by a hybrid technique combining a theoretical method and a semi-empirical method (e.g. Kamae et al., 1998; Kagawa, 2004).

This technique adopted the stochastic Green's function method (Kamae et al., 1990) for the short period range and the discrete wave-number method (Bouchon, 1981) for the long period range. Here,  $f_{\max}$  (cut-off

frequency in the high frequency range) was taken in the parameters of 84<sup>th</sup> percentile (11.9Hz), 50<sup>th</sup> percentile (7.0Hz), and 16<sup>th</sup> percentile (4.7Hz) estimated by the spectrum fitting study on 120 actual records. In addition, strong ground motions other than sample magnitudes (M5.5, 6.0, 6.5, 6.8, 7.3) were calculated by interpolation or extrapolation between the magnitude range of 5.0 - 7.3 by 0.1 step. Furthermore, we assumed that the depth of the asperities was distributed uniformly in the seismogenic layer (depth from -3km to -20km), the strong ground motions were also calculated by interpolating those of shallow, intermediate, and deep asperity cases.

Finally, the relation between uniform hazard spectra and annual exceedance probabilities were obtained by analyzing about 1,100,000 acceleration response spectra of ground motions.

### **3.4 Empirical approach to strong motion simulation**

We also estimated the earthquake ground motions on the bedrock surface by the empirical approach proposed by Nishimura et al. (2001) using the parameters of earthquake magnitude and equivalent hypocentral distance, which is proposed to evaluate the response spectra of design-basis earthquake ground motions on an outcrop. But Noda et al. (2002) found that the response spectra by Nishimura et al. (2001) for shallow inland earthquakes overestimate the observations, and proposed a correction factor.

Considering this correction factor, strong ground motions were also calculated by the empirical approach for above 40 fault models used in the hybrid technique. Here, we also took into account near-fault rupture directivity effect in the empirical approach.

The relation between the uniform hazard spectra and the annual exceedance probabilities were estimated in the same way as the above hybrid technique by using the acceleration response spectra obtained by the empirical approach, where deviation of 0.53 for the natural-logarithmic normal distribution was considered, following to Noda et al. (2002).

## **4. Result**

### **4.1 Comparison of seismic ground motions simulated by hybrid technique and empirical approach**

We simulated strong ground motions on the bed rock surface by 40 fault models at the 231 (=11x21) evaluation points, shown in Figure 6.

An example of horizontal peak accelerations (values at 0.02s of the acceleration response spectrum with damping factor of 5%) on the bedrock surface for magnitudes of 6.5 obtained by the hybrid technique is shown in Figure 8, comparing that by the empirical approach,. The peak accelerations close to the fault by the hybrid technique were larger than those by the empirical approach, which is considered to be affected by the effective stress on the large asperity. However, the peak acceleration contours obtained by the hybrid technique were similar to those obtained by the empirical approach.

And the horizontal response spectra (damping factor:5%) simulated by the hybrid technique was compared with those simulated by the empirical approach. Figure 9(1)-(2) shows average and average plus/minus standard deviation of the pseudo velocity response spectra simulated by the hybrid technique comparing with those by the empirical approach, for magnitudes of 6.5 and 6.8. As shown in Figures 9, the average and the average plus/minus standard deviation of the response spectra simulated by the hybrid technique were consistent with those simulated by the empirical approach at the same magnitude, especially in the period range shorter than 0.5s.

**Figure 8. Contour of estimated peak accelerations on seismic bedrock  
for MJ=6.5 with shallow fault and middle stress drop**

(a) Fault normal

M6.5(FN)	-40.0	-36.0	-32.0	-28.0	-24.0	-20.0	-16.0	-12.0	-8.0	-4.0	0.0	4.0	8.0	12.0	16.0	20.0	24.0	28.0	32.0	36.0	40(km)
20.0	14.1	11.4	19.0	29.6	25.2	55.4	89.8	68.0	99.3	127.5	100.0	97.7	87.3	76.0	67.1	54.7	37.9	29.7	23.6	21.0	18.8
16.0	13.3	13.6	16.1	25.4	26.7	53.6	62.5	105.3	89.6	102.6	153.2	114.4	112.4	103.5	79.7	50.2	53.9	28.8	21.3	18.9	11.6
12.0	14.1	15.0	19.6	29.6	43.8	42.2	57.0	66.8	133.7	193.0	141.9	146.6	122.6	95.3	57.4	39.1	27.1	22.2	15.8	13.6	
8.0	12.2	18.8	21.6	26.7	34.4	37.5	50.5	123.7	142.9	210.5	277.6	228.0	156.9	181.6	110.3	56.6	44.8	35.8	28.4	17.9	16.5
4.0	15.6	20.6	19.1	23.2	36.9	49.1	75.7	143.0	230.4	341.8	411.5	357.2	476.6	235.3	87.2	73.5	57.7	36.8	27.5	21.0	15.8
0.0	13.4	17.4	22.6	30.6	43.6	52.1	64.6	160.8	272.5	375.1	626.7	710.0	632.7	305.1	111.1	78.4	71.2	39.6	28.2	26.0	15.4
-4.0	14.5	18.9	20.4	24.6	31.9	42.5	85.1	108.7	258.3	240.0	218.5	295.0	279.5	203.0	166.6	73.9	43.8	35.0	29.9	20.2	17.0
-8.0	13.5	17.6	21.8	24.2	22.7	44.2	55.6	124.2	123.2	213.4	184.6	149.4	164.1	150.1	116.5	54.2	38.8	28.3	20.1	14.2	19.1
-12.0	15.4	13.4	15.7	21.0	37.4	41.0	53.6	106.9	115.2	126.5	126.2	104.7	141.7	101.1	86.3	65.1	43.2	29.9	21.1	17.4	16.5
-16.0	11.3	18.1	21.2	29.1	39.5	50.8	67.8	84.4	109.4	94.5	124.3	102.8	120.5	96.2	66.2	57.7	30.9	29.4	20.5	15.6	18.1
-20.0	16.0	19.4	24.0	25.9	30.8	38.4	72.8	92.1	77.8	67.5	89.9	79.6	69.1	88.9	81.2	54.7	49.5	21.6	21.2	20.6	17.1

(b) Fault parallel

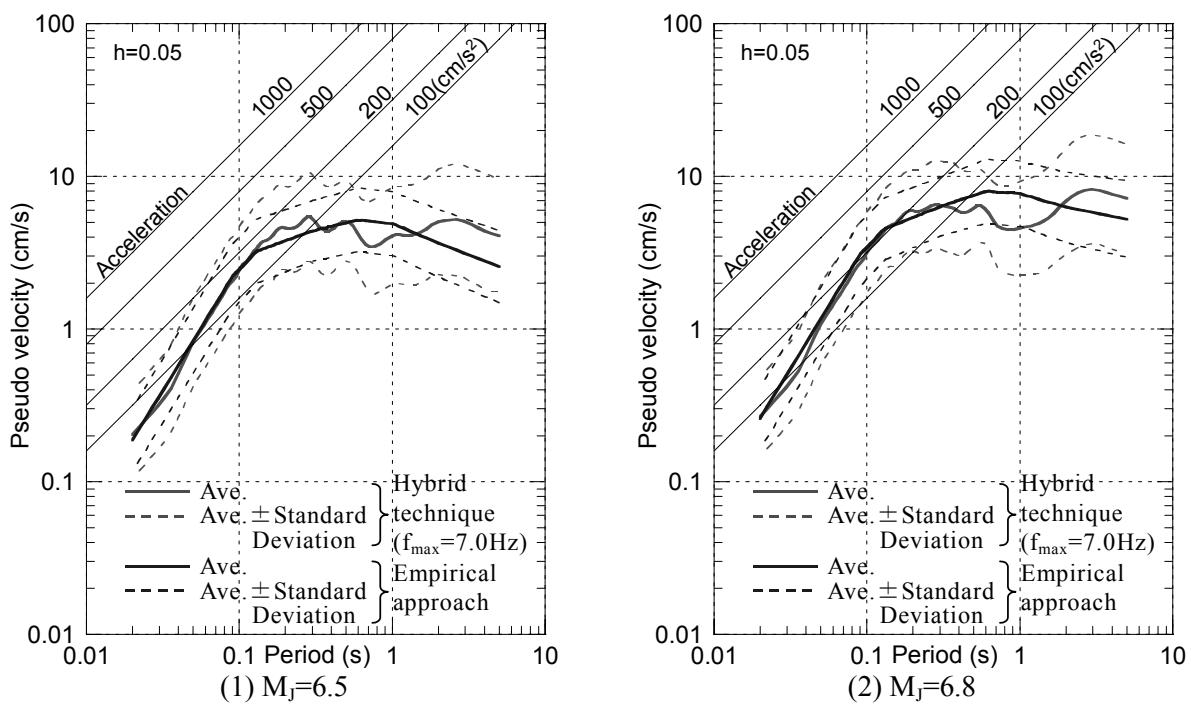
(1) Hybrid technique ( $f_{max}=7.0\text{Hz}$ )

M6.5(FP)	-40.0	-36.0	-32.0	-28.0	-24.0	-20.0	-16.0	-12.0	-8.0	-4.0	0.0	4.0	8.0	12.0	16.0	20.0	24.0	28.0	32.0	36.0	40(km)
20.0	23.5	21.6	32.1	33.8	45.3	34.4	45.2	52.1	49.4	53.4	77.2	56.4	55.6	46.4	40.9	42.2	56.4	42.5	35.3	29.8	33.5
16.0	34.4	32.7	38.0	49.1	41.7	48.8	89.3	62.3	61.0	84.3	104.0	75.5	79.4	66.1	55.1	57.4	49.8	48.8	43.7	32.5	28.3
12.0	23.0	45.8	55.3	57.8	64.7	74.5	69.6	100.6	88.8	133.5	152.1	125.0	115.8	132.7	95.0	78.6	59.6	48.7	40.6	29.1	31.5
8.0	32.2	39.6	54.5	38.4	64.2	77.7	90.1	127.7	277.1	214.5	249.3	252.7	299.5	142.2	124.7	74.0	78.6	60.1	41.2	34.4	36.3
4.0	34.9	43.5	37.3	46.1	60.3	62.5	100.0	118.7	251.6	415.6	390.2	656.0	470.3	218.6	106.3	91.7	73.2	57.5	48.1	39.7	32.2
0.0	34.2	44.6	41.0	41.8	45.0	71.4	117.3	125.2	169.6	248.5	340.5	521.5	292.9	209.4	101.7	89.8	87.8	60.6	44.3	39.2	32.7
-4.0	28.9	30.3	34.9	63.1	50.7	69.5	88.6	162.0	160.2	279.2	250.4	337.1	274.5	276.6	123.4	72.9	73.4	60.3	53.8	31.9	32.4
-8.0	29.5	42.9	40.5	52.4	58.2	77.1	115.3	77.1	134.5	150.5	169.2	224.2	142.8	175.5	130.9	77.2	82.7	64.4	41.4	27.4	24.4
-12.0	36.8	40.7	44.2	38.4	45.9	64.4	85.6	66.9	64.3	93.1	117.4	107.4	112.8	86.2	67.2	62.0	75.4	46.8	30.6	30.9	32.9
-16.0	25.4	24.6	31.5	34.9	46.8	54.0	56.7	50.7	58.4	75.5	72.5	86.3	54.9	47.1	51.1	40.1	42.3	39.0	34.8	28.4	27.5
-20.0	29.1	42.9	32.7	31.9	35.8	37.1	38.7	44.4	40.5	55.9	70.9	63.6	51.0	34.9	43.2	37.3	35.2	32.6	41.3	29.1	22.7

(2) Empirical approach

M6.5sh	-40.0	-36.0	-32.0	-28.0	-24.0	-20.0	-16.0	-12.0	-8.0	-4.0	0.0	4.0	8.0	12.0	16.0	20.0	24.0	28.0	32.0	36.0	40(km)
20.0	27.2	30.9	35.3	40.7	45.8	51.5	57.8	64.6	71.1	76.3	79.3	79.2	75.9	70.3	63.6	56.7	50.4	44.8	39.6	34.4	30.1
16.0	28.7	32.9	38.1	43.9	50.0	57.3	66.0	75.9	86.2	95.1	100.5	100.4	94.7	85.2	74.5	64.5	55.9	48.7	42.8	37.0	32.0
12.0	30.0	34.7	40.6	46.6	54.0	63.3	75.1	89.8	106.4	122.3	132.9	133.2	122.1	104.8	87.5	72.9	61.4	52.4	45.4	39.3	33.6
8.0	31.0	36.1	42.3	48.9	57.4	68.7	84.1	105.3	132.8	160.7	180.7	183.1	161.7	129.6	101.4	80.9	66.2	55.5	47.4	41.1	34.9
4.0	31.5	36.9	43.3	50.3	59.5	72.3	90.9	119.1	161.4	209.3	248.5	260.8	210.8	153.3	112.5	86.5	69.4	57.4	48.7	42.0	35.6
0.0	31.7	37.0	43.5	50.5	60.0	73.1	92.4	122.8	170.9	230.1	277.4	298.8	229.6	159.4	115.2	87.8	70.1	57.8	48.9	42.2	35.8
-4.0	31.3	36.5	42.9	49.7	58.6	70.8	88.0	112.9	147.9	183.9	212.6	218.2	185.6	142.7	107.6	84.1	68.1	56.6	48.1	41.7	35.3
-8.0	30.5	35.4	41.6	47.8	55.8	66.1	79.8	97.4	118.8	140.4	154.8	155.7	140.7	116.7	94.5	77.0	63.9	54.0	46.5	40.3	34.3
-12.0	29.4	33.8	39.4	45.3	52.0	60.3	70.5	82.5	95.6	107.4	114.9	114.9	107.0	94.3	80.8	68.7	58.7	50.6	44.1	38.2	32.9
-16.0	28.0	31.9	36.7	42.4	47.9	54.3	61.8	69.9	78.1	84.9	88.9	88.8	84.5	77.2	68.8	60.5	53.1	46.8	41.4	35.7	31.1
-20.0	26.4	29.8	33.9	38.7	43.7	48.7	54.1	59.7	65.0	69.1	71.4	71.3	68.8	64.4	58.9	53.2	47.8	42.9	37.7	33.0	29.1

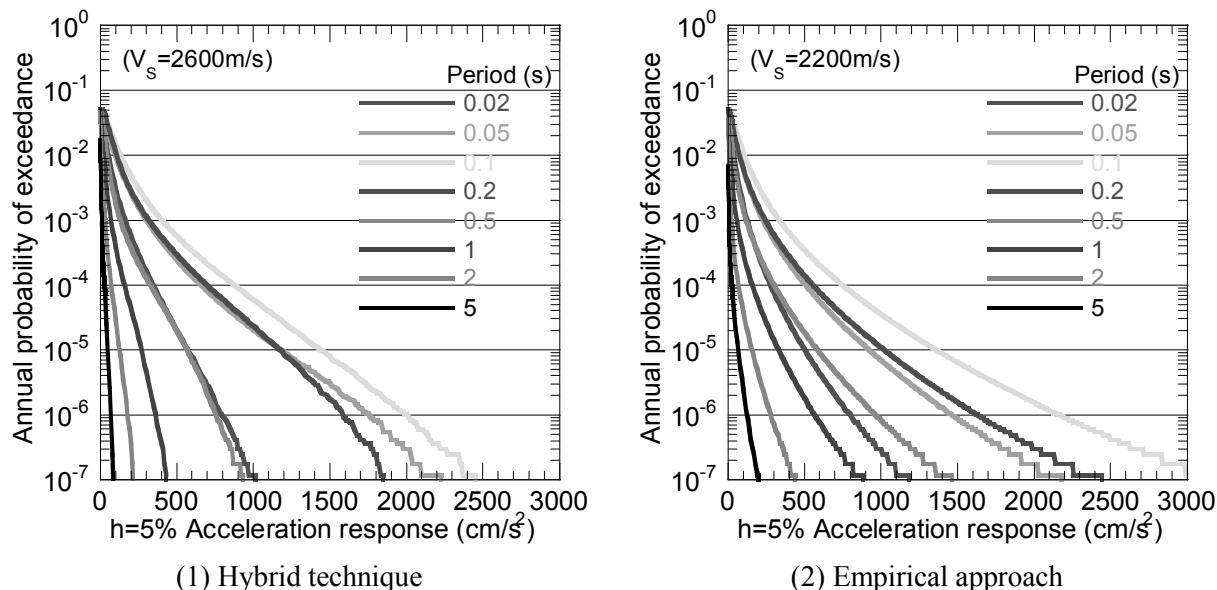
**Figure 9. Horizontal pseudo velocity response spectra on seismic bedrock evaluated by hybrid technique and empirical approach**



#### 4.2 Seismic hazard curves and uniform hazard spectra

Figure 10 shows the seismic hazard curves for several spectral periods evaluated by the hybrid technique comparing with those by the empirical approach. As can be seen, both of the curves were close at annual exceedance probabilities higher than  $10^{-5}$ .

**Figure 10. Hazard curves of horizontal-acceleration response spectra on seismic bedrock evaluated by hybrid technique and empirical approach**



The annual exceedance probabilities of the horizontal response spectra at 0.02s simulated by the hybrid technique and the empirical approach are summarized in Table 5. The horizontal response spectra with a damping factor of 5% at 0.02s simulated by the hybrid technique were about 160, 340, 570, and 800 cm/s/s at annual exceedance probabilities of  $10^{-3}$ ,  $10^{-4}$ ,  $10^{-5}$ , and  $10^{-6}$ , respectively. And those by the empirical response spectrum were about 130, 280, 500, and 800 cm/s/s, respectively.

**Table 5. Annual probability of exceedance of horizontal-acceleration response spectra**

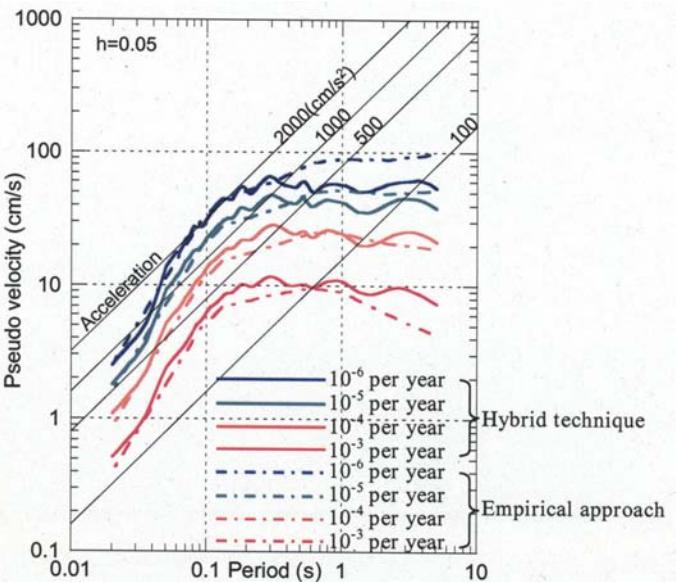
Annual probability of exceedance	Horizontal-acceleration response spectra (T=0.02 s)	
	Hybrid technique	Empirical approach
$10^{-3}$	160 cm/s <sup>2</sup>	130 cm/s <sup>2</sup>
$10^{-4}$	340 cm/s <sup>2</sup>	280 cm/s <sup>2</sup>
$10^{-5}$	570 cm/s <sup>2</sup>	500 cm/s <sup>2</sup>
$10^{-6}$	800 cm/s <sup>2</sup>	800 cm/s <sup>2</sup>

For a certain annual exceedance probability, the ground motion spectrum (on the seismic bedrock) is extracted from the hazard curves for various spectral periods. These spectra are then plotted at their respective spectral periods to form the uniform hazard spectra.

Using this process, the horizontal uniform hazard spectra were evaluated from the hybrid technique and the empirical approach.

Figure 11 shows these horizontal uniform hazard spectra at annual exceedance probabilities of  $10^{-3}$ ,  $10^{-4}$ ,  $10^{-5}$ , and  $10^{-6}$ ; and the solid lines are the horizontal uniform hazard spectra evaluated from the hybrid technique, and the dash-dotted lines are those evaluated from the empirical approach. The uniform hazard spectra evaluated by the hybrid technique show good agreement with those evaluated by the empirical approach, but some differences are seen in the uniform hazard spectra for the annual exceedance probability of  $10^{-6}$  in the period range longer than 0.5s.

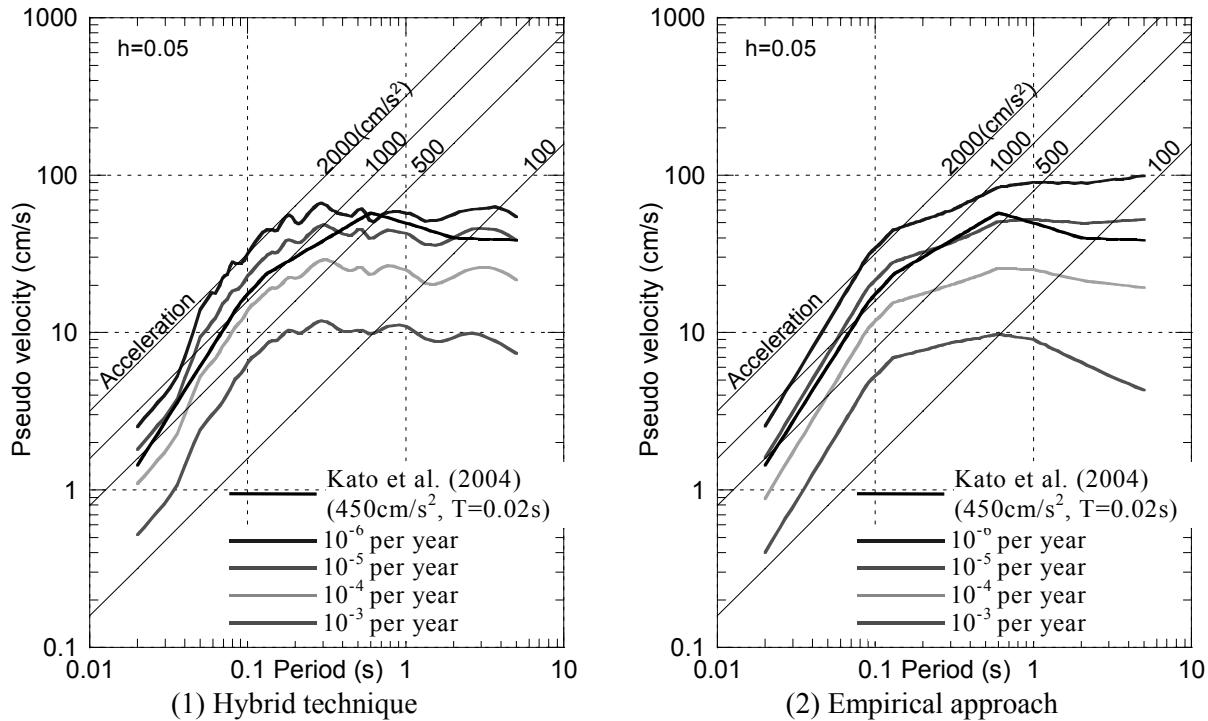
**Figure 11. Uniform hazard spectra of the horizontal strong motion caused by buried-rupture earthquakes in evaluation area**



On the other hand, Kato et al. (2004) proposed an upper level of strong motion spectra using records on an outcrop due to inland earthquakes caused by undefined faults. They used 30 strong motion records observed at 12 stations on rock sites from 9 earthquakes to determine the upper strong motion spectra for undefined faults. The estimated peak ground acceleration is 450cm/s/s on seismic bedrock.

Finally, the uniform hazard spectra by the hybrid technique and the empirical approach were compared with their proposed response spectra in Figure 12. As shown in the figure, their upper level of the response spectra corresponds to the uniform hazard spectra of the annual exceedance probability of  $10^{-5}$  and  $10^{-4}$ .

**Figure 12. Uniform hazard spectra of the horizontal strong motion caused by buried-rupture earthquakes and the upper level of the strong motion records proposed by Kato et al. (2004)**



## 5. Conclusions

We proposed a method to evaluate the probabilistic ground motion spectra due to close-by inland earthquakes caused by undefined faults covering the variation in magnitude and location of undefined faults, and calculated uniform hazard spectra by a strong motion simulation technique based on fault models. Moreover, we also calculated the uniform hazard spectra by empirical approach proposed by Nishimura et al. (2001) and Noda et al. (2002) in the same way.

Comparing the horizontal uniform hazard spectra evaluated by the hybrid technique and those evaluated by the empirical approach, both of them are in good agreement. And the response spectra with a damping factor of 5% at 0.02s simulated by the hybrid technique were about 160, 340, 570, and 800cm/s/s at annual exceedance probabilities of  $10^{-3}$ ,  $10^{-4}$ ,  $10^{-5}$ , and  $10^{-6}$ , respectively, which were also in good agreement with the response spectra evaluated by the empirical approach.

And the deterministic response spectrum proposed by Kato et al. (2004) as the upper level of the strong motion records for undefined faults corresponded to the uniform hazard spectra of the annual exceedance probability of  $10^{-5}$  and  $10^{-4}$ .

## 6. Acknowledgments

The Ministry of Economy, Trade and Industry, Japan sponsored this research. We would like to express our appreciation to Dr. Sci. Kojiro Irikura, President of Irikura Research Institute, for his guidance in the strong motion simulation technique based on asperity models. We would also like to thank Dr. Kazuo Dan, Ohsaki Research Institute, Inc. and Dr. Yasuhiro Ohtsuka, Kobori Research Complex, Inc. and Dr. Takao Kagawa, Geo-Research Institute for evaluating the uniform hazard spectrum and for fruitful discussions.

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***Technical Session: Ground Motion***

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## PEGASOS: a PSHA for the Swiss nuclear power plants Some comments from the managerial point of view

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### **1. Introduction**

In the nineties, the Swiss Federal Nuclear Safety Inspectorate (HSK) identified the need to update the seismic hazard assessments for the Swiss NPPs and requested in 1998 from the Swiss NPP operators to prepare a new hazard study that would satisfy SSHAC level 4 (SSHAC, 1997).

Because of its long experience in earth science issues, Nagra (the National Co-operative for the Disposal of Radioactive Waste) was entrusted by swissnuclear ('the sponsor'), on behalf of the utilities, to plan, organise and perform such a study – the PEGASOS project (Probabilistische Erdbeben-Gefährdungs-Analyse für KKW-Standorte in der Schweiz). The project relied to a large extent on external services: more than 20 experts from 7 European countries, about 25 Swiss and foreign specialists and consultants providing the experts with databases, computational and modelling support, including a team of experienced PSHA procedural specialists from the US (e.g. the Technical Facilitators / Integrators, TFIs) and - last but not least - the Swiss Seismological Service providing the key contributions to the PEGASOS earthquake catalogue (SED, 2002). The expert teams contained both recognised experts from within Switzerland (including experts from the Swiss Seismological Service on an 'ad personam' basis) and from abroad (mainly Europe but also from the US). The study was closely followed by the Swiss Federal Nuclear Safety Inspectorate through a participatory review panel (HSK, 2004). Preparatory work for the study started in 1999. The project itself began in early 2001 and was completed in summer 2004 with costs of approx. 10 Mio CHF. Although the project was concluded quite a while ago, the use of its results in full PSAs is still under discussion and, thus, the overall meaning of the results cannot yet be judged.

The aim of this paper is to provide a summary of the PEGASOS study (Nagra, 2004) and of the current status of the different activities related to this study – i.e. the conclusions from the participatory review by the Swiss Federal Nuclear Safety Inspectorate and the analyses of the results of the PEGASOS study by the sponsors.<sup>1</sup> However, it has to be mentioned that the author himself was not directly involved in any of these studies but has followed the development over the full duration from some distance. The paper thus only summarises findings that are documented in more detail in the material mentioned, including information received from some of the persons directly involved and makes no judgments on any of the findings. The author takes the full responsibility for any incorrect interpretation of the information mentioned.

### **2. The methodology used and its implementation**

A PSA for a NPP requires among other external events also the consideration of very low probability earthquakes. Considering earthquakes with probabilities as low as  $10^{-7}/a$  is a special challenge due to the lacking or very limited database to develop the different elements of a PSHA (earthquake recurrence, maximum magnitudes, regional attenuation of strong ground motions, site response). In Switzerland, a

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1. The full PEGASOS project report (Vol. 1 – 6, Nagra, 2004) and the final review report of the regulator (HSK, 2004) as well as a note documenting the position of the sponsors with respect to the results of the PEGASOS study (swissnuclear, 2006) including key references (Cooke, 2005; Meng, 2005; Ravindra, 2004; Proseis, 2005; Yilmaz et al., 2006), have been made available to the participants of the workshop for their personal use.

country with low to moderate seismicity, the assessment of strong ground motion attenuation is particularly difficult due to the sparse regional data base (no observed earthquakes of the size corresponding to low or very low probabilities). The problem therefore requires informed extrapolation of available information from Switzerland and elsewhere in order to develop the input for the hazard calculations. According to the SSHAC level 4 methodology, a structured approach is used to elicit the information from qualified experts to arrive at the view of the informed scientific community with special emphasis on a systematic quantitative evaluation of uncertainties. This requires: (i) building a team covering all relevant areas with experts that are sufficiently qualified to represent the current state of knowledge of the scientific community in their respective fields; (ii) providing the experts with the information they need (however, with collecting only very limited new field data in the course of the project); (iii) conducting a process that ensures that a balanced view of the required input is developed for calculating the seismic hazard. In the PEGASOS project this also included a careful evaluation of and distinction between both aleatory (randomness) and epistemic (lack of knowledge) uncertainty.

Another important element of the methodology is the participatory review (in the case of the PEGASOS project by a panel of the safety authorities without the involvement of the sponsor) with the aims of (i) ensuring that the project plan and the SSHAC-PSHA process are properly implemented; (ii) verifying that the relevant data and state-of-the-art methods are considered, (iii) verifying that key intermediate and final products / results appear generally reasonable.

Through a thorough selection process, starting with a short-list of over 100 expert candidates, 21 were finally engaged to develop the input for the hazard study. They formed the core of the project team of about 45 persons, among them many recognised European experts in the fields of seismology, geology, geophysics, and earthquake engineering as well as experienced US PSHA-experts (the TFIs and support specialists).

The hazard analysis involved four main tasks, which were attributed to four separate subprojects: the characterisation of seismic sources, the characterisation of ground motion attenuation, the characterisation of site response and finally the performance of the seismic hazard calculations. The project was conducted through a series of workshops, complemented by individual elicitation meetings between the experts and the TFIs and work that the experts had to do by themselves. The workshops provided primarily the opportunity for the experts to share data, present their views and interpretations and receive feedback from their colleagues. Although the timing and topics varied somewhat among the subprojects, the major goals and purposes of the five workshops (WS) were similar: WS-1 focussed on elicitation training and the identification of key issues and data needs, WS-2 on methodologies and the evaluation of models, WS-3 on providing the experts with feedback on their preliminary assessments, WS-4 on revised assessments and the exposure to expert and hazard sensitivity feedback. Then, a last opportunity was offered to the experts to modify their input. Thereafter, the final hazard calculations were performed. WS-5 concluded the study with the presentation of the final results. As specified in the project plan, no participation/interference of the NPP PSA specialists was allowed with respect to the development of the PSHA input by the experts.

The project provided input to the experts according to their expressed wishes and needs (data-bases, special database products and presentations, supporting calculations and special studies - in total 57 computation projects and dedicated studies on poorly understood or controversial issues were commissioned by the experts). Within the PEGASOS project some new procedural and technical aspects were introduced: 1) the incorporation of site response in a SSHAC level-4 study (with assessment of site models and uncertainties by expert elicitation); 2) the explicit assessment of maximum ground motions; 3) 'hazard input documents' (HIDs) to formalize the interface between expert assessments and hazard calculations; 4) the provision of extensive, subproject-specific databases (incl. GIS) with auxiliary software and 5) a QA-system specifically developed for the project.

The systematic and detailed evaluation of epistemic uncertainties led to a very large number of different branches in the logic trees. This resulted in an enormous number of calculations, which presented a big challenge and required ‘pinching’ under the observation of very stringent rules.

The project was completed within budget and the planned time frame. According to the scope of the project, after having completed the final hazard calculations no further work was done, e.g. to refine the underlying databases.

### 3. Results & key scientific issues

The project resulted in a wealth of information and produced hazard results of a type and in a format in accordance with the specifications of the utilities and as agreed by the authorities in their review of the PEGASOS project. The calculated hazard levels are comparable to those derived in recent studies for other areas with low to moderate seismicity. For example, the calculated peak rock accelerations corresponding to a mean hazard at  $10^{-4}$  annual probability are in the range of 0.23 – 0.27 g. This compares well with the hard rock accelerations of 0.1 to 0.4 g obtained by the USGS for much of the eastern United States using a similar approach.

According to the results the most important contributor to the epistemic uncertainty in seismic hazard is uncertainty in the rock ground motion attenuation, followed by uncertainty in site response and then uncertainty in source characterisation. This situation is common for hazard results in regions with limited strong-motion data and reflects the wide range of candidate ground motion models that the experts considered to be potentially applicable to Switzerland. Both the ranking of uncertainty sources and the absolute size of the uncertainties are also observed in other recent studies. The 16<sup>th</sup> to 84<sup>th</sup> fractile range of the PGA (peak ground acceleration) hazard on rock at a probability level of  $10^{-4}/a$  corresponds to a factor of 20, as compared to 30 for Yucca Mountain (with significant uncertainty in the characteristics of the Basin and Range earthquake ground motions) and about 5 to 10 for the central and eastern United States where there is only limited ground motion data, but where the tectonic environment is well known and extensive ground motion modelling studies have been carried out.

The deaggregation of hazard into magnitude-distance- $\varepsilon^2$  bins shows that with increasing ground motion the contribution from larger, more distant events decreases rapidly and the hazard is dominated by close-by moderate events. In accordance with the deaggregation results the sensitivity studies show that seismic sources located close to the sites provide the largest contribution to the hazard for the probability levels of primary interest ( $< 10^{-4}/a$ ).

An expert-to-expert comparison of (interim) PEGASOS results showed a good degree of consistency among the experts for the characterisation of seismic sources, ground motion attenuation and site response. Thus, the observed large epistemic uncertainties for ground motion attenuation are not due to different interpretations by the experts but rather reflect the key issue that the experts were facing: Do the particularly low ground motions observed for small earthquakes in Switzerland imply that the ground motions from large magnitude earthquakes in Switzerland will also be lower than in other parts of Europe? The ground motion experts gave this issue considerable thought when they formulated their interpretations and models and concluded that there was not a strong technical basis for ground motions in Switzerland being significantly smaller than in other areas in Europe.

Compared to older site-specific studies from the mid-eighties to the mid-nineties, the PEGASOS hazard results are much higher (e.g. PGA corresponding to the median hazard at  $10^{-4}$  annual probability, by a

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2.  $\varepsilon$  is a measure that indicates which part of the distribution of epistemic uncertainty contributes to the corresponding hazard

factor of 1.7 – 2.7). The PEGASOS team undertook a careful analysis of the previous studies to establish why the hazard results were higher and came to the conclusion that the higher hazard could be attributed to a single reason: the aleatory variability of the ground motion had not been correctly included in the former hazard calculations. While the non-inclusion of the aleatory ground motion variability was common practice in the seventies and eighties, it is no longer considered to be acceptable today. As a consequence, the PEGASOS analysts recalculated the old studies, using the same input and software, with the only difference that the aleatory ground motion variability was included in the hazard integral. Using the original input including the original ground attenuation model (but including aleatory variability), this resulted in hazards that were equal to or even higher than the PEGASOS hazards.

At the end of the project the project team recommended an in-depth analysis of the most promising ways to reduce epistemic uncertainty through a discussion between the TFIs and (preferably) the original experts. The PEGASOS project team has already identified some areas of potential refinements, especially with respect to reduction in ground motion uncertainty. This includes the collection of a limited number of site-specific data to better characterise ground motion sites in Switzerland ( $V_s$  and kappa) and the study of the 2003 St. Dié, the 2004 Besançon and other recent moderate magnitude earthquakes to help constrain the Swiss-specific ground motions for moderate magnitudes.

#### **4. The view of the authorities**

The project was followed by a review team of the Swiss Federal Nuclear Safety Inspectorate (HSK) and the review team provided valuable input at specific milestones of the project which was taken into account in the subsequent work. In their final report (HSK, 2004) the safety authorities came to the conclusions that: (i) the project had achieved the safety authority's requirement of a SSHAC level 4 PSHA; (ii) the project had in a number of respects defined a new state-of-the-art; (iii) the results produced provided the best site-specific PSHA estimates now available; (iv) the results were consistent and compatible with PSHA insights derived elsewhere; (v) a reasoned evaluation of uncertainties had been produced, consistent with expectations and other state-of-the-art studies. However, the review team pointed out that the epistemic hazard evaluated appeared quite diffuse and that it might be possible to somewhat better constrain the epistemic uncertainty in PSHA results. The review team identified areas for refinement that may reduce epistemic uncertainty and narrow the divergence between mean and median PSHA results. Finally, the review team recommended that (vi) any refinements should be conducted with quality standards and methodologies compatible with those adopted in PEGASOS.

The report also provided an overview on specific comments and issues and also pointed out potential refinements – which however, according to the safety authorities, do not put the results of the PEGASOS study in question. The areas mentioned are very briefly summarised below:

- Clarification of magnitude-dependent distribution of hypocentral depths.
- Modelling of ruptures that extend beyond the source boundaries.
- Additional assessments of maximum magnitude and their effect on hazard.
- Use of lower values for lower-bound magnitude.
- Increasing attention to near-source mechanics, near-source ground motions, and integrated effects.
- Integrated assessment of rock ground motions and soil amplifications with the potential to reduce epistemic uncertainty (instead of independent assessment).
- Documentation of information on important soil failure modes and liquefaction.
- Better explaining the interface issues concerning seismic sources, ground motion and soil amplification with the potential to somewhat reduce epistemic uncertainty.

As a first step – to achieve better insights concerning reasonableness and assurance of the numerical PSHA results and their uses – the authorities considered it important to perform sensitivity studies that produce the set of 80 “mean” hazard curves (for each NPP site, and each soil location and rock location) derived from the

possible combinations of the individual expert/expert team mean assessments. These then should be discussed with respect to their significance in relation to the intended application (seismic design, PSA studies).

## 5. The view of the sponsors

After completion of the project, the sponsors started with a review of the results. The initial review focussed on some simple plausibility checks (validation of the results in the sense of quality assurance, including comparisons with results from other studies) as well as on some mathematical issues (applicability of the used models and expert opinion aggregation methods). The review comments (geophysical part) were peer reviewed by independent experts (seismologists and geologists not involved in the PEGASOS project) to incorporate additional information from the technically informed community. The first review by the sponsor resulted in the conclusion that the preliminary PEGASOS-results at their current state are not suitable for use in a PSA (Klügel, 2005a). The review was continued in 2005 and the following work has been accomplished:

- Decomposition of the calculated hazard into the separate 80 ‘expert opinion’ combinations and evaluation of the differences in the calculated hazard and in the shape of the hazard spectra. The sensitivity analysis performed showed some remarkable differences between the different single ‘expert opinion’ combinations (the total hazard was decomposed into the set of the single 80 hazard curves and spectra) both with respect to the shape of the uniform hazard spectra as well as to the absolute values of the hazard (Proseis, 2005).
- Commissioning of a "Comparison Study of Earthquake Hazard Curves" (Ravindra, 2004) which showed remarkable differences with other recent seismic hazard studies.
- Reviews of the expert opinion aggregation method (Cooke, 2005, Meng, 2005) have shown some difficulties with giving equal weight to all experts. The use of alternative aggregation methods (Bayesian, usually regarded as a benchmark for the evaluation of the outcome of expert elicitation processes, see Kahneman, et al, 1982) did lead to significantly different results (Klügel, 2005b).
- The sponsors also commissioned a detailed geotechnical analysis (Yilmaz et al., 2006) for one site, revealing substantial differences in the shear wave velocity profile in comparison to what was used in the PEGASOS project.

After some discussions with the safety authorities, the sponsors have continued their studies and plan a workshop at the end of November 2006 in Switzerland that will involve some of the experts who took part in the PEGASOS project but also other experts as well as the safety authorities. The sponsors plan a study, designed to refine the results of the PEGASOS study under consideration of the outcome of that workshop. With respect to the PEGASOS study, the current concerns of the sponsors are related to the following issues (swissnuclear, 2006):

- Plausibility of the hazard curves based on comparisons with benchmark tests and with curves from other European studies.
- Ground motion attenuation equations and site effects: the use of world-wide data vs. data from the study region for parameterisation of the corresponding equations, the use of multidimensional simulation of seismic wave propagation, etc.
- Seismic source: lower bound magnitude, maximum magnitude distribution; earthquakes as poissonian or non-poissonian process, etc.
- Methodological issues when dealing with very low probability events (probabilities as low as 10<sup>-7</sup> per year): propagation of uncertainties, use of expert input (aggregation of expert opinion and calibration of subjective probabilities), etc.

The different issues mentioned above led the sponsors to the conclusion that significant refinements of the calculated hazards are necessary before they could be used in formal PSAs. These issues are currently undergoing an independent external review. In addition, the sponsors are preparing proposals for the refinement of PEGASOS.

## 6. Summary and conclusions

With the PEGASOS project, a PSHA has been completed according to the SSHAC level-4 recommendations. The most important factors for achieving a study according to SSHAC level 4 are considered to be:

- The availability and willingness of a range of experts who belong to the recognised specialists in the disciplines relevant to PSHA to participate in a very demanding project.
- The ability of the project management and especially of the so-called technical facilitators and integrators (TFIs) to keep the work of the experts “on track”.
- The importance of supporting infrastructure with extensive and easily accessible databases, the possibility of inviting resource experts to provide in-depth information on specific issues and the possibility to order supporting computations and special studies.
- The importance of adequate QA measures (e.g. for defining and documenting the input to the hazard calculations).
- The availability of an extensive earthquake catalogue for Switzerland together with the availability of corresponding experience and expertise in Switzerland (importance of the Swiss Seismological Service).
- A methodology that has proven to be feasible albeit demanding with respect to the amount and quality of resources needed.

After completion of the study, the sponsors evaluated the plausibility of the calculated hazard, analysed the different elements of the study and made a critical review of the methodology. This analysis, which also considers the results of some studies commissioned by the sponsors, led the sponsors to the conclusion that significant refinements of the calculated hazards are necessary before they could be used in formal PSAs. Some of the issues criticised by the sponsors have been published in the literature (Klügel, 2005a). This has led to controversy and disagreement between the sponsors and some experts not involved in the PEGASOS study on the one side, and the experts involved in PEGASOS and some members of the technical community on the other side (see *Engineering Geology*, Vol. 82, 2005 with contributions by Krinitzsky, Lomnitz, Wang and Budnitz et al., Musson et al., respectively). The issues raised by the sponsors and by the review team of the safety authorities will be discussed at a workshop in November 2006 in Switzerland.

The background material (PEGASOS project reports, the final report by the review team of the Swiss Federal Nuclear Safety Inspectorate and a note documenting the view of swissnuclear - the sponsor of the PEGASOS study) is made available to the participants of this workshop to provide them with the relevant information to allow them to contribute to this discussion. The author of this paper considers it important that at least some of the controversial issues can be resolved and that for the others a clear understanding of the different positions will be developed, especially for those concerning the possible refinement of the results of the PEGASOS study - proposals for refinements have been made by the PEGASOS project team, the review team of the safety authorities and the sponsors. It is hoped that this paper and the documents made available to the participants of this workshop will help in this process.

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## Recent Advances in Developing Site-Specific Ground Motion Attenuation Relations for the Nuclear Power Plant Sites in Korea

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### ***Abstract***

Site-specific GM (ground-motion) attenuation relations for four Nuclear Power Plant sites in Korea were developed by regression analyses on the multiple simulations of PGA-values obtained by the RVT simulation method adopted in the stochastic GM method. Multiple simulations were made at distances of 1, 5, 10, 20, 50, 75, 100, 200, and 400 km. At each distance, four magnitudes were used: M 4.5, 5.5, 6.5, and 7.5. RVT simulation method was implemented by using the site-specific characteristics of site and path effect identified by analyzing the large spectral data set from small earthquakes and by using the regional source characteristics of a few moderate-to-large earthquakes occurring in and around the Korean Peninsula. In particular, the development of site-specific GM attenuation relations is possible by directly using the observed data from the seismic station located just right at the site of interest without considering the geotechnical properties of the foundation. These relations were compared with some observed data available up to now and the result of which showed good agreement each other supporting the validity of the relations. The result of this study is quite promising in that the characteristics of small earthquakes regarding the site and path effect can be used to predict strong ground-motion, although there still remains some issues that still have to be resolved such as of nonlinear behavior of the structural foundation and uncertainty of source scaling relations which are expected for large earthquakes.

### **1. Introduction**

PSHA (Probabilistic Seismic Hazard Analysis) is a well known established procedure that should be performed in low-to-moderate seismicity area such as Korea in order to provide reasonable levels of the ground-motion parameters for aseismic design of critical facilities constructed at a site. PSHA incorporates every conceivable uncertainties inherent in the evaluations of seismic source characterization and ground-motion (GM) attenuation relations by adopting a logic tree approach. Aggregate of multiple hazard curves, each of which corresponds to one branch of the logic tree, is the final product of the PSHA representing the total uncertainty of the available geoscience information and their interpretations among the expert groups. The total uncertainty should be minimized as far as we can. Otherwise it could lead to unnecessary overestimation of the aseismic design level regarding the ground-motion. In Korea, PSHA has been mainly conducted to systematically evaluate the seismic safety of NPP sites.

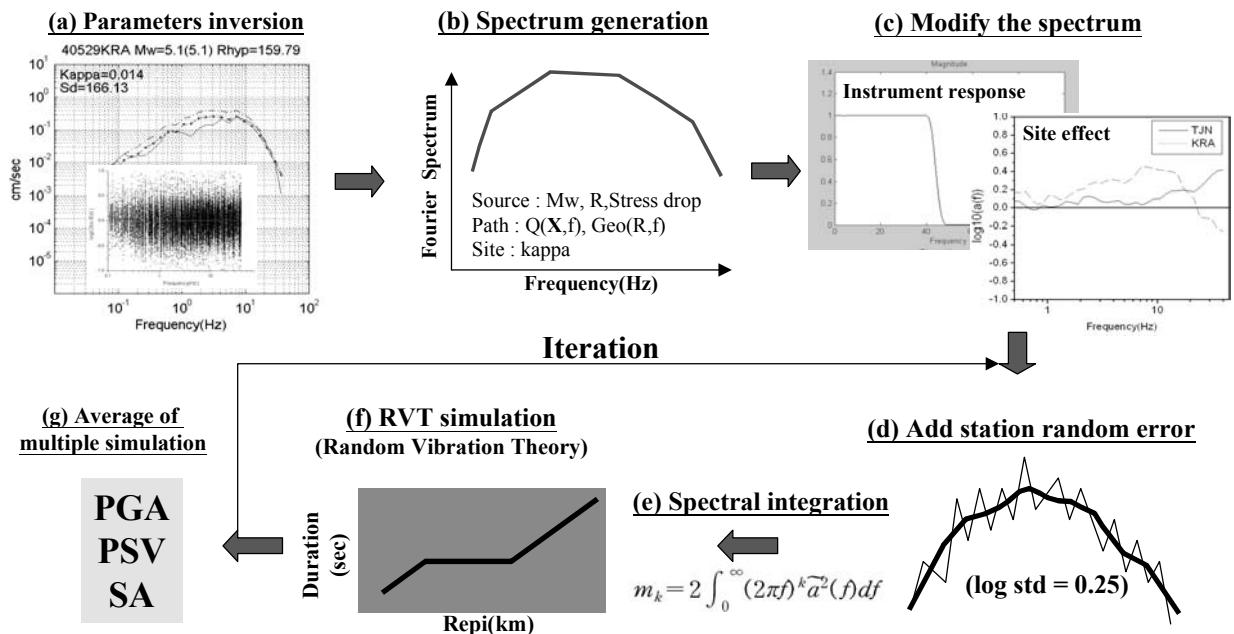
Based on the previous results of the PSHA, it was found that the uncertainty of GM attenuation relations was a significant contributor to the total uncertainty. This was an expected result because since there were no strong-ground motions available in Korea to which the GM attenuation relation should be calibrated, the experts diversify in their opinions on the use of the GM attenuation relations and their parameters. In this respect, we have conducted a research project to develop site-specific GM attenuation relations for NPP sites as a way of reducing the uncertainty in GM attenuation relations and finally to obtain site-specific response spectra compatible with a target probability according to the world wide practice. In this study, I will introduce an evaluation procedure to develop a site-specific GM attenuation relation, its preliminary result, and the results of comparative study with the observed data.

## 2. Methodology

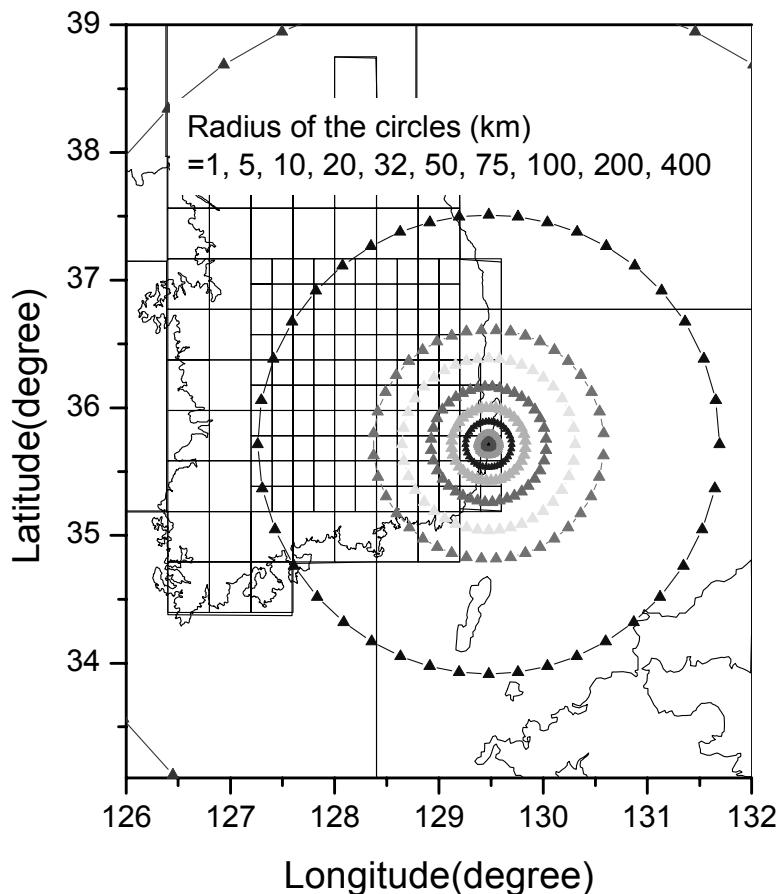
Site-specific GM attenuation relations for NPP sites were developed by a regression analysis on the multiple simulations of the stochastic GM method (Boore, 2003) that adopts RVT (random vibration theory). For this, the stochastic GM parameters regarding the source, site and path effect was inverted based on the large data set that consists of horizontal shear-wave Fourier spectra calculated for small earthquakes occurring in and around the Korean Peninsula. This data set has been accumulated since 1992 and most of the data come from the earthquakes occurring since 1999 when a modern digital seismic network in Korea was actually established with dense distribution of seismic stations around the nation. We used a site-specific stochastic GM parameters regarding site and path effect except the source effect. To consider site-specific path effects for a site, 2-D tomography of  $Q_0$  and  $\eta$  was conducted based on the large spectral data set with respect to the nonparametrically derived geometrical spreading model. As a result, we have obtained **KEPRI04**  $Q(Q_0(\underline{X}), \eta(\underline{X}), f)$  model that is capable of representing the laterally varying  $Q$  across the nation. The 159 different sizes of grid cells were used for 2-D  $Q$  tomography as shown in Figure 1. In particular, the site-response of the seismic station located right at the NPP site of interest was obtained based on the residuals from the simultaneous inversion of the parameters and used as input for RVT simulation. These seismic stations called “representative station” are WSA, UJA, KRN, and YGA for the Wolsung, Uljin, Kori, and Yeongwang NPP sites respectively. Brune’s stress drop, which is a key parameter of the earthquake source, was estimated to be 170 bars for the earthquakes of  $M > 4.0$  occurring in and around the Korean Peninsula.

The stochastic GM method that adopts a RVT simulation was implemented as shown in Figure 1 by modifying the Boore’s simulation program (Boore, 1996). The stochastic GM method has been successfully used worldwide in simulating the high-frequency ground-motion above 1Hz for engineering purposes. Especially, the randomness of the spectra was incorporated into generating the synthetic Fourier spectra to take into account the uncertainty that cannot be accounted for by the parameters adopted in the stochastic GM model.

**Figure 1. Procedure of a RVT simulation to generate synthetic data set of PGA, PGV and SA based on the inverted parameters of the stochastic GM model**



**Figure 2. Spatial distribution of 159 grid cells used for 2-D Q tomography and source-site geometry used in multiple RVT simulations for the WSA site. The earthquakes are assumed to occur at a given distance and azimuth direction from the site to generate synthetic data set of PGA and spectral acceleration.**



A simulation consistent with the procedure shown in Figure 1 has been performed multiple times for various cases of uniformly distributed magnitudes, distances, and azimuth directions from the site (see Figure 2) by using the site-specific stochastic GM parameters which were summarized in Table 1. GM attenuation relations have been obtained by fitting to the multiple simulation results a mathematical form of equation (1) given as a function of magnitude and distance. In particular, for large earthquakes of  $M_w > 6.5$ , empirically derived scaling relations of magnitude-dependent geometrical spreading (Abramson and Silva, 1997) and magnitude-dependent depth (Atkinson and Silva, 2000) were used as inputs for the stochastic GM simulation. The cross-over distance of  $R_0=50\text{km}$  was used in equation (1) to reflect the change of geometrical spreading models based on the result of the study (Yun *et al.*, 2002).

**Table 1. Values of the stochastic GM model parameters for RVT simulation used to develop site-specific GM attenuation relations**

Parameters \ NPP	Kori	Uljin	Wolsung	Yeongwang
Stress Drop (bars)	170 bars (Mw=4.5, 5.5, 6.5, 7.5)			
Q(f)	<b>KEPRI04</b> $Q(Q_0(\underline{X}), \eta(\underline{X}), f)$ model used			
Site effect (station)	KRN (KINS)	UJA (KEPRI)	WSA (KEPRI)	YGA (KEPRI)
Geo(R,f)	<ul style="list-style-type: none"> <li>Empirical geometrical spreading model used.</li> <li>Magnitude dependent geometrical spreading model used*</li> </ul>			
Distance (R)	<ul style="list-style-type: none"> <li>Magnitude dependent depth (h) used to calculate the hypocentral distance  <math>-\log h (\text{km}) = -0.05 + 0.15 \times M_w</math> (Atkinson and Silva, 2000)</li> </ul>			
Kappa0 ( $\text{sec}^{-1}$ )	0.015	0.021	0.010	0.013
Duration	$1/f_c + 0.05 \times R$ for $R < 100\text{km}$ Choi's 2002 model for $R > 100\text{km}$			
Randomness of spectrum	$0.25 \log 10$			
$\langle R_{00} \rangle$	0.55			
F	2.0			
V	0.707			
$\rho(\text{g/cm}^3)$	2.7			
$\beta(\text{km/sec})$	3.5			

\*: Empirically obtained geometrical spreading model was rescaled to reflect the distance saturation by using the following equation. (Abramson and Silva, 1997)

$$R^{-\text{Slope}(M)}/R^{-\text{Slope}(4.5)} \text{ for } R < R_0, R^{-\text{Slope}(M)/2}/R^{-\text{Slope}(4.5)} \text{ for } R > R_0, \text{ Slope}(M) = a + b \times (M-6.5) \\ (R = \text{hypocentral distance (km)}, R_0 = 50\text{km}, a = 1.0296, b = -0.0422)$$

$$\ln(Y) = P1 + P2 \cdot M_w + (P3 + P4 \cdot M_w) \cdot \ln(R_{rup} + e^{P5}) + P6 \cdot (M_w - 6)^2 + P7 \cdot \ln(\min(R, R_0)) + P8 \cdot \ln(\max(R, R_0)) \quad (1)$$

where  $R = \sqrt{(R_{rup})^2 + 9.8^2}$  for  $M_w \leq 6.5$   
 $= \sqrt{(R_{rup})^2 + 9.8^2} \cdot e^{2(-1.25+0.227 \cdot M_w)}$  for  $M_w > 6.5$

$R_0 = 50\text{km}$

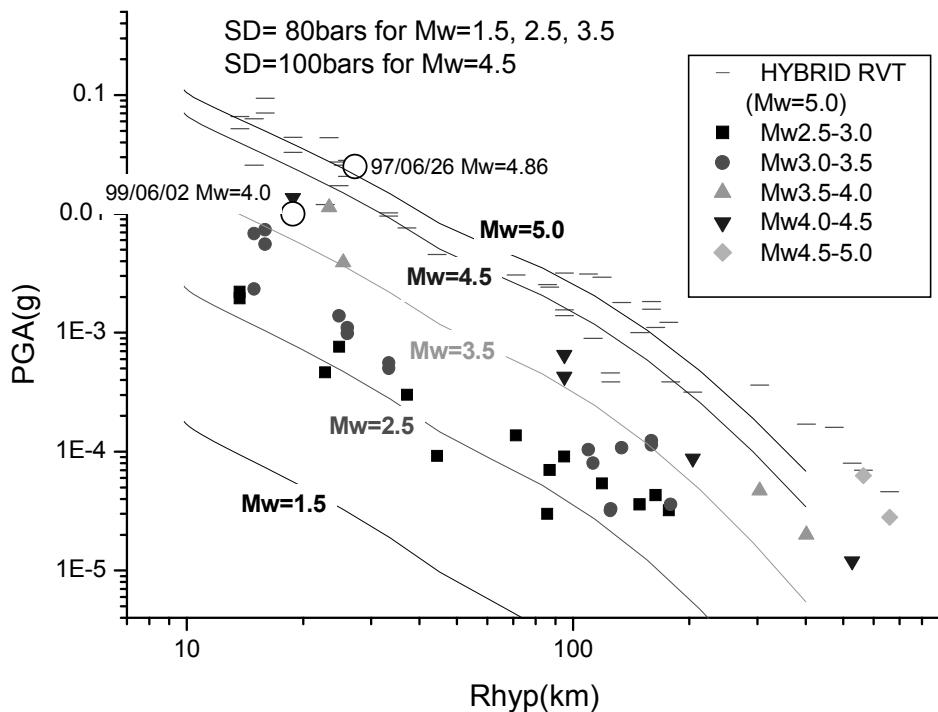
$R_{rup}$  = Rupture distance (km)

$M_w$  = moment magnitude

### 3. Validation of the procedure and evaluation of the result

Before comparison with the actual data, the procedure explained by a schematic diagram shown in Figure 1 is validated through comparison with other published GM attenuation relations in Korea for which the parameters of stochastic GM model are available. We applied this procedure to derive GM attenuation relations by using the same stochastic GM model parameters of theirs. The resultant GM attenuation relations were almost the same with the corresponding GM attenuation relations published by them. When we used the parameters listed in Table 1, we were able to reproduce the observed data from each site with an error of  $\sigma_{\log 10}=0.1$ . In addition, we applied this procedure to the strong GM data recorded for 17 earthquakes recorded worldwide and succeeded in predicting the observed PGA-values with an error of  $\sigma_{\log 10}=0.29$ .

**Figure 3. Comparison of the site-specific GM attenuation relation (instrument corrected) for WSA site with the observed PGA-values at the WSA and hybrid RVT simulated result.**

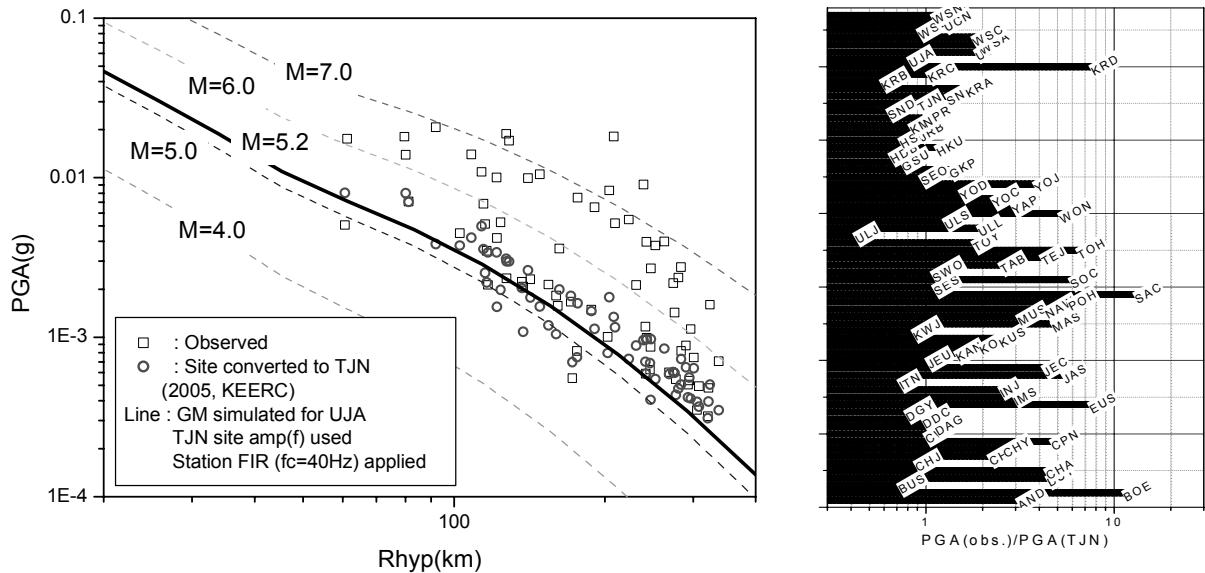


The GM attenuation relations for 4 NPP sites were developed following the procedure described in Figure 1 and the regression analysis by using equation (1). The shapes of the site-specific response spectrums for a certain magnitude-distance pair were found to differ considerably with one another, which is mainly the result of using the different Kappa0 values in Table 1 and different site-responses of the seismic stations specific to a NPP site. Also we have noticed that there is a great difference in the attenuations of waves coming from the off-shore and the inland for the 3 NPP sites located on the east shore.

To evaluate the GM attenuation relations developed through the above procedure, we compared these relations with some observed data. These include the data from small earthquakes actually observed at the concerned site, one moderate earthquake in Korea which recently occurred after the dense seismic network in Korea was fully in operation, and two large crustal earthquakes of Japan. For the proper comparison, it is import to precondition the simulated data used for the development of GM attenuation relations by applying the same instrument response of the observed data or to correct the observed data so that they are compatible with the simulated data in terms of the site conditions.

First, the GM attenuation relation for WSA was developed for small earthquakes by taking into account decreasing trend of the stress drops according to the decrease of earthquake magnitude. This relation is compared with the observed data from small earthquakes recorded at the WSA seismic station and two triggered records at the seismic monitoring system of Wolsung NPP as in Figure 3. From Figure 3, we can find that the developed GM attenuation relation reflects the observed attenuation pattern in distance fairly well.

**Figure 4. Comparison of the site-specific GM attenuation relation (instrument corrected) for UJA site with the observed PGA-values and PGA-values which are converted to represent the TJN site response**

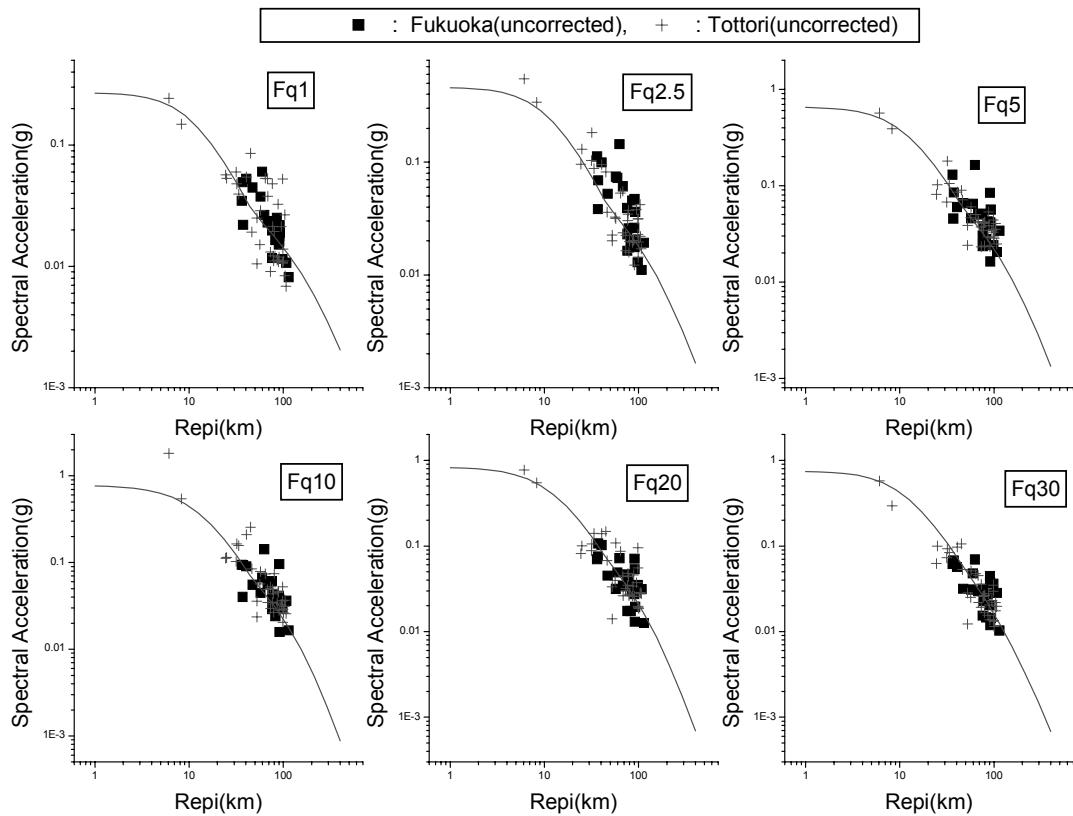


Another important earthquake of  $M_w=5.1$  occurred in 2004 offshore Uljin in the East Sea (Sea of Japan). This earthquake was recorded at nationwide seismic stations and provided the valuable data in the aspect of earthquake engineering which can be used to calibrate the GM attenuation relations in Korea. Unfortunately, raw data from the seismic stations cannot be used as a reference dataset for the evaluation purposes because of prominent site-response at some of the recorded sites. Beside no geophysical and geotechnical information of the seismic stations were available to classify the raw data according to a site condition. So a process which we called “site-conversion” was implemented through which previously estimated site-responses of all the seismic stations were deconvolved and convolved in frequency domain with a site-response of a reference station TJN (Yun *et al.*, 2005a). Figure 4 shows the comparison between the site-specific GM attenuation relation developed for UJA site and PGA-values of the unprocessed observed data and of the processed data that were converted to represent the TJN site response. As shown in Figure 4, the GM attenuation relation for the UJA represent the site-converted PGA-values quite well while it is only fitted to the lower limit of the data distribution of the unprocessed PGA-values.

Also comparison with the observed data from the Fukuoka earthquake has been made based on the premise that the source characteristics of the Fukuoka earthquake is not significantly different from that of a scenario earthquake of the similar magnitude which is likely to occur in the future in and around the Korean Peninsula. In particular, the comparison with the Fukuoka earthquake has been made possible based on the recent study (Yun *et al.*, 2005b). This study showed that there was good agreement among source spectra estimated based on the data from seismic networks of Korea at large distances ( $190\text{km} < R < 500\text{km}$ ) and KiK-net downhole and Hi-net near the source region ( $R < 100\text{km}$ ), implying that the site response at the seismic basement of the Korean seismic network is similar to the site responses at KiK-net downhole and Hi-net stations near the epicenter. This fact validates not only the source spectrum estimated by using only the Korean earthquake data but also path and site responses used for the source estimation.

Based on this result, the observed data from two crustal earthquakes occurring in the south-west of Japan was directly compared with the GM attenuation relation for WSA developed for a seismic basement. These two shallow crustal earthquakes are Fukuoka earthquake (05/03/20,  $M_w=6.6$ ) and Tottori earthquake (00/10/06,  $M_w=6.6$ ). Figure 5 is the result of comparision which shows almost perfect agreement with each other.

**Figure 5. Comparison of the site-specific GM attenuation relation (site-response removed) for WSA site with the observed PGA-values at deep borehole sensors of KiK-net of Japan for Fukuoka earthquake (05/03/20, Mw=6.6) and Tottori earthquake (00/10/06, Mw=6.6)**



#### 4. Conclusion and discussion

In this study, we developed site-specific GM attenuation relations for four NPP sites by regression analyses on the multiple simulations of PGA-values obtained by the RVT simulation method adopted in the stochastic GM method (Boore, 2003). Multiple simulations were made at distances of 1, 5, 10, 20, 50, 75, 100, 200, and 400 km. At each distance, four magnitudes were used: M 4.5, 5.5, 6.5, and 7.5. RVT simulation method was implemented by using the site-specific characteristics of site and path effect identified by analyzing the large spectral data set from small earthquakes and by using the regional source characteristics of a few moderate-to-large earthquakes occurring in and around the Korean Peninsula. In particular, the development of site-specific GM attenuation relations was possible by directly using the observed data from the seismic station located just right at the site of interest without considering the geotechnical properties of the foundation. In addition, lateral variation of Q was taken into account in developing the GM attenuation relations by the use of the result from the 2-D Q tomography study around the Korean Peninsula. These relations were compared with some observed data available up to now and the result of which showed good agreement each other supporting the validity of the relations.

Although the result of this study is quite promising in that the characteristics of small earthquakes regarding the site and path effect can be used to predict strong ground-motion, there still remains some issues that still have to be resolved such as of nonlinear behavior of the structural foundation and uncertainty of source scaling relations which are expected for large earthquakes. For this, the characteristics of the strong ground-motion data worldwide should be more thoroughly studied in light of

reducing the uncertainty involved in developing GM attenuation for large earthquakes based on the characteristics of small earthquakes. These issues are particularly important because these are related to the quantification of variability in the earthquake ground-motion that is another integral element of the GM attenuation relation which however was left out of this study. We plan to estimate this variability in the ground-motion based on the hybrid data set which is generated by adding the observed error bias from a parametric inversion for the small earthquakes to the synthetic spectra for large earthquakes. And more comprehensive and detailed investigation of the site condition of the seismic stations installed in Korea should be conducted with a top priority for application of the result of this study to another target site.

For the time being, it is recommended that the strong ground-motion data of time histories recorded at deep downhole of KiK-net can be used as an input motion at the basement of foundations and convolved with the site-response for seismic analysis of the structures.

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## Probabilistic Approach to Design Earthquake Ground Motions Based on Fault Rupture Scenarios

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### *Abstract*

In the conventional earthquake resistant design of nuclear power plants in Japan, the input ground motions were generated according to the design spectra specified with the magnitudes and the distances. Neither the fault parameters nor the exceedance probability were explicitly indicated in the design spectra. Recent seismological developments have enabled us to estimate the parameters of the fault rupture scenarios and also the occurrence probability of each scenario for major active faults in Japan. The new regulatory guide for aseismic design of nuclear power reactor facilities, revised by the Atomic Safety Commission this September, requires to take into account of these parameters for the design earthquake ground motions.

The purpose of this study is to propose a procedure to generate design earthquake motions based on fault rupture scenarios for any life-time exceedance probability in particular in the range of  $10^{-2}$  to  $10^{-3}$ .

At first, we categorized the earthquakes in and around Japan as follows:

- Category I: inland earthquakes caused by active faults recognized on the surface.
- Category II: large plate-boundary earthquakes in the subduction zones.
- Category III: large intra-slab earthquakes in the subducting plates.
- Category IV: inland earthquakes caused by blind faults not recognized on the surface.
- Category V: moderate and small plate-boundary earthquakes in the subduction zones.
- Category VI: moderate and small intra-slab earthquakes in the subducting plates.

Next, we chose the KiK-net site of Shizugawa, Miyagi Prefecture, as a test site because the National Research Institute for Earth Science and Disaster Prevention had proposed the earthquake occurrence model in and around Miyagi Prefecture and because good records were available at this site to get the high accuracy of the strong motion prediction.

After preliminary seismic hazard analysis based on the empirical relation of peak ground velocities with the magnitudes and the distances, we found that the earthquakes of the Categories II, IV, V, and VI had predominant influence on the site. So, we assumed 5,333 fault rupture scenarios for those earthquakes, predicted strong ground motions due to the rupture scenarios, evaluated the occurrence probability of each scenario, and obtained the seismic hazard curves for the peak accelerations, the peak velocities, and the response spectra at various natural periods of structures.

Finally, in order to show some examples of design earthquake ground motions, we selected 64 predicted motions with the 50-year exceedance probability of  $10^{-2}$  to  $10^{-3}$ , averaged their response spectra, and generated 64 artificial motions with the same response spectrum as the averaged one and with the same phase as that of the 64 selected motions.

## 1. Introduction

In the conventional earthquake resistant design of nuclear power plants in Japan, the input earthquake ground motions were generated according to the design spectra specified with the magnitudes and the distances. Neither the fault parameters nor the exceedance probability were explicitly indicated in the design spectra.

On the other hand, recent seismological developments have enabled us to estimate fault parameters and also the occurrence probability for inland earthquakes caused by major active faults and large plate-boundary earthquakes in Japan [1]. The new regulatory guide for aseismic design of nuclear power reactor facilities, revised by the Atomic Safety Commission this September, requires to take into account of these parameters for the design earthquake ground motions. Furthermore, it requires to evaluate the exceedance probability of the design earthquake ground motions to prepare for measuring the seismic safety of the nuclear power plants.

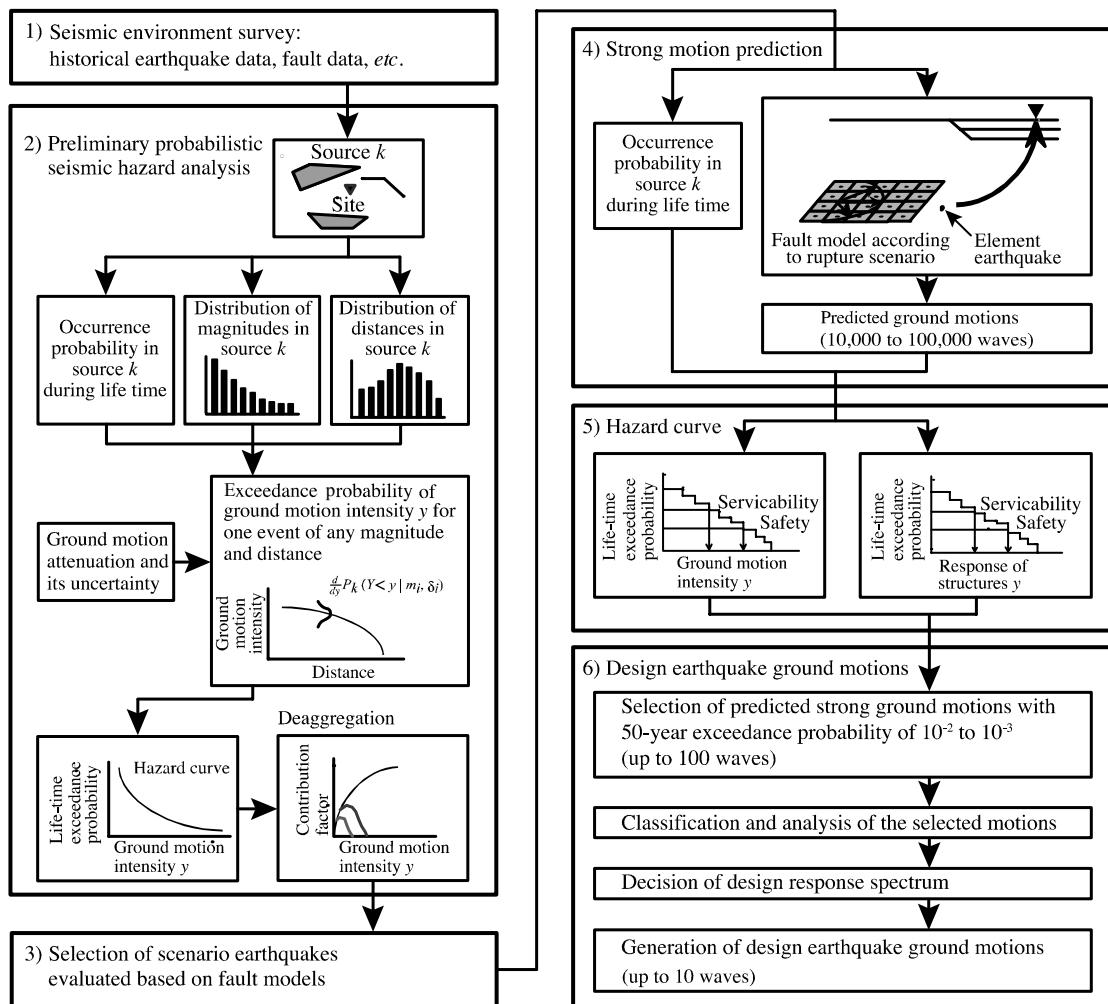
More than a few probability seismic hazard maps are available, but the findings for scenario earthquake faults are not taken into account. The purpose of this study is to propose a procedure to generate design earthquake motions considering scenario earthquake data for any life-time exceedance probability in particular in the range of  $10^{-2}$  to  $10^{-3}$ . Number of simulations is limited in this study, but we introduce the framework of the procedure and show some examples of the design earthquake ground motions for the usefulness of the procedure.

## 2. Flow chart of procedure for probabilistic approach to design earthquake ground motions

Figure 1 shows the flow chart of our procedure for probabilistic approach to design earthquake ground motions based on the fault rupture scenarios and their occurrence probability. This flow chart consists of the following six steps:

- 1) Seismic environment around the site should be surveyed in prior to any analysis or evaluation. The materials are historical earthquake data, active fault data, plate boundary data, and so on.
- 2) A preliminary probabilistic seismic hazard analysis is performed for the site based on the earthquake occurrence model and the attenuation relations among ground motion intensities, the magnitudes, and the distances from the site to the source. The contribution of each source is calculated by deaggregation method of Kameda *et al.* [2].
- 3) Scenario earthquakes are selected accordingly to the contribution.
- 4) Fault rupture scenarios are considered in the scenario earthquakes, and the occurrence probability of each fault rupture scenario is evaluated. Then, fault models are assumed accordingly to the fault rupture scenarios for predicting ground motions. The total of the predicted ground motions are 10,000 to 100,000, because we focus on the life-time exceedance probability of  $10^{-2}$  to  $10^{-3}$ .
- 5) Since each predicting ground motion has its occurrence probability, by assembling all possible ground motions, preliminary hazard curves are replaced by those based on scenario earthquakes with more detailed information. For measuring intensity, the response spectrum might be preferred to the peak ground acceleration or the peak ground velocity.
- 6) Once seismic hazard curves are formed, some simulated earthquake ground motions can be selected at required exceedance probabilities corresponding to specified criteria. For example, Figure 1 shows the required 50-year exceedance probability of  $10^{-2}$  to  $10^{-3}$ , corresponding to the annual exceedance probability of  $10^{-4}$ , and the wave number of up to 100. Then, the selected ground motions are classified by the earthquake type of inland earthquakes, plate-boundary earthquakes, and intra-slab earthquakes and by the magnitude, because the physical differences may be found in the frequency contents of the different classes of the ground motions. After the analysis of the frequency contents, the design response spectra are decided, and the design earthquake ground motions are generated.

**Figure 1. Flow chart of proposed procedure for probabilistic approach to design earthquake ground motions**



### 3. Test site

The accuracy of the information on the historical earthquakes and the active faults varies at present in Japan, and the uniform accuracy cannot be expected in calculating seismic hazard curves. Hence, in this study, we chose a test site where we could calculate a seismic hazard curve including rare strong ground motions in the view points of the earthquake occurrence model and the strong motion prediction procedure.

We focused on how to consider the scenarios of the fault ruptures, and categorized the earthquakes in and around Japan as follows [3]:

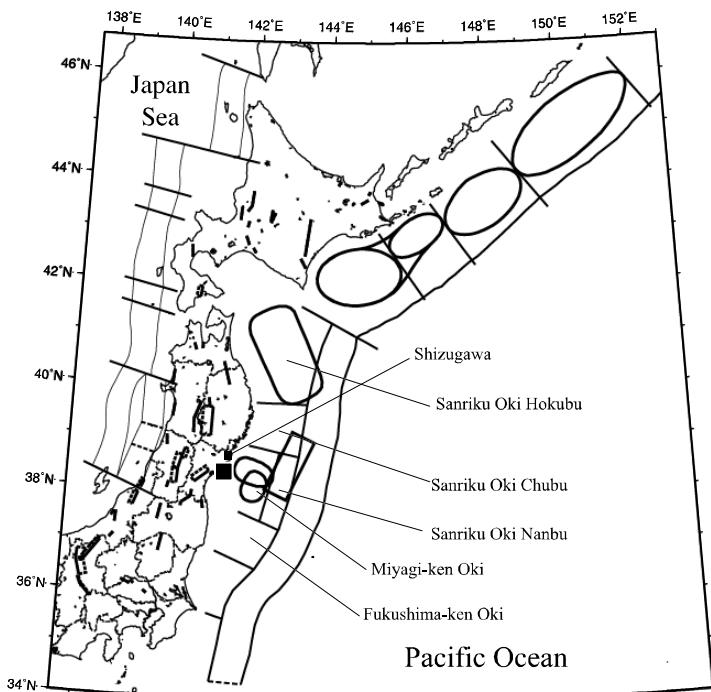
- Category I: inland earthquakes caused by active faults recognized on the surface.
- Category II: large plate-boundary earthquakes in the subduction zones.
- Category III: large intra-slab earthquakes in the subducting plates.
- Category IV: inland earthquakes caused by blind faults not recognized on the surface.
- Category V: moderate and small plate-boundary earthquakes in the subduction zones.
- Category VI: moderate and small intra-slab earthquakes in the subducting plates.

It is preferred that the test site is located in the region where all the six categories of the earthquakes occur because of easy extension of the methodology to other sites.

On the other hand, the accuracy of the strong motion prediction is expected to be improved when the earthquake records are observed at the site, because we can examine the source, path, and site characteristics based on the records.

Consequently, we chose the KiK-net site of Shizugawa (38.6386N, 141.4463E), Miyagi Prefecture, because the National Research Institute for Earth Science and Disaster Prevention [3] had proposed the earthquake occurrence model in and around Miyagi Prefecture and also because good records were available at this site. Figure 2 shows the location of the test site with the active faults (Category I) and the source areas of the large plate-boundary earthquakes (Category II) taken from Okumura *et al.* [4].

**Figure 2. Location of the test site of Shizugawa with the active faults (Category I) and the source areas of large plate-boundary earthquakes (Category II) taken from Okumura *et al.* [4]**



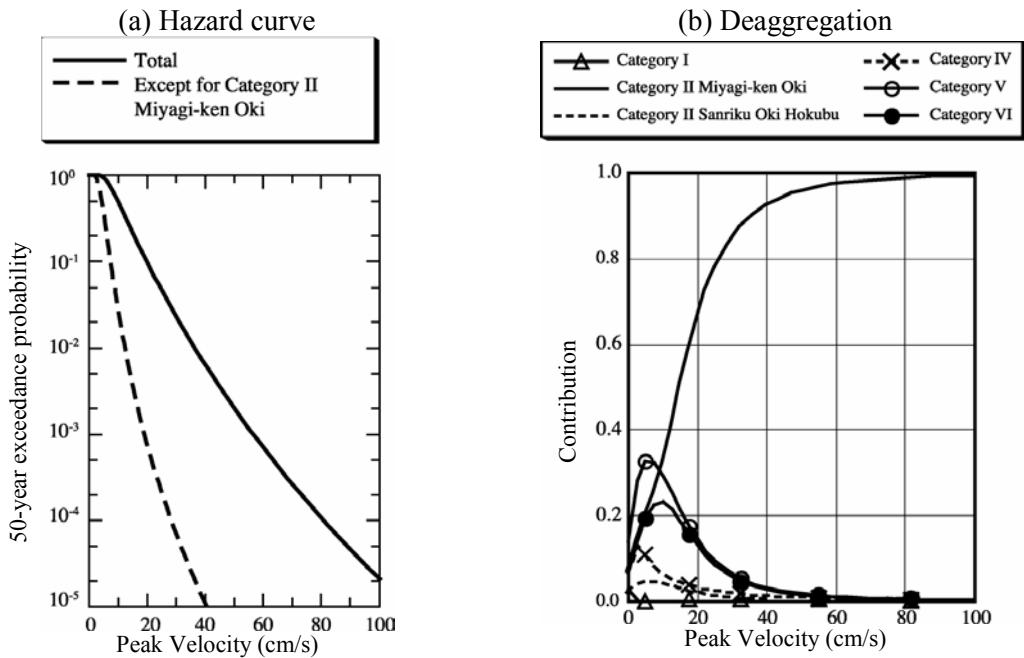
#### 4. Preliminary seismic hazard analysis and selection of scenario earthquakes

We carried out a preliminary seismic hazard analysis to select scenario earthquakes, using peak velocities on the outcrop ( $V_s=860$  m/s) at the site as ground motion intensities. We adopted an attenuation relation among peak velocities on the engineering bed rock ( $V_s=600$  m/s), magnitudes, and distances by Si and Midorikawa [5] and a correction factor for local path effects in northern Japan by Morikawa *et al.* [6], and modeled the variation by the log-normal distribution with the logarithmic standard deviation of 0.53. Here, we considered the soil amplification factor between the engineering bed rock ( $V_s=600$  m/s) to the outcrop ( $V_s=860$  m/s) by Matsuoka and Midorikawa [7].

Figure 3(a) shows the results of the 50-year exceedance probability of the peak velocity. Figure 3(b) shows the deaggregation results: the contribution of each group of earthquakes, clearly indicating that the Miyagi-ken Oki earthquakes of Category II are predominant in the wide peak velocity range.

The preliminary results of the probabilistic seismic hazard analysis in Figure 3 indicated that the Miyagi-ken Oki earthquakes of Category II were predominant and that moderate and small earthquakes of Categories IV, V, and VI were not negligible. Hence, we selected the earthquakes of these four categories as scenario earthquakes.

**Figure 3. Preliminary results of probabilistic seismic hazard analysis at the KiK-net site of Shizugawa in Miyagi Prefecture, Japan, for selecting scenario earthquakes**



## 5. Fault rupture scenarios and prediction of strong ground motions

### 5.1 Large plate-boundary scenario earthquakes in the subduction zone (Category II)

We adopted scenarios of the fault ruptures in the Miyagi-ken Oki earthquakes of a magnitude of 8 class in Category II proposed by the National Research Institute for Earth Science and Disaster Prevention [3]. The National Research Institute for Earth Science and Disaster Prevention [3] assumed scenarios of the fault ruptures in the areas A1, A2, and B [8] shown in Figure 4(a) and some combinations of the three areas. The scenarios and the moment magnitudes  $M_W$  were assumed as follows:

- $M_W$  7.5 in the area A1 only,
- $M_W$  7.4 in the area A2 only,
- $M_W$  7.8 in the area B only,
- $M_W$  8.0 in the area A1+B,
- $M_W$  8.0 in the area A2+B,
- $M_W$  8.1 in the area A1+A2+B.

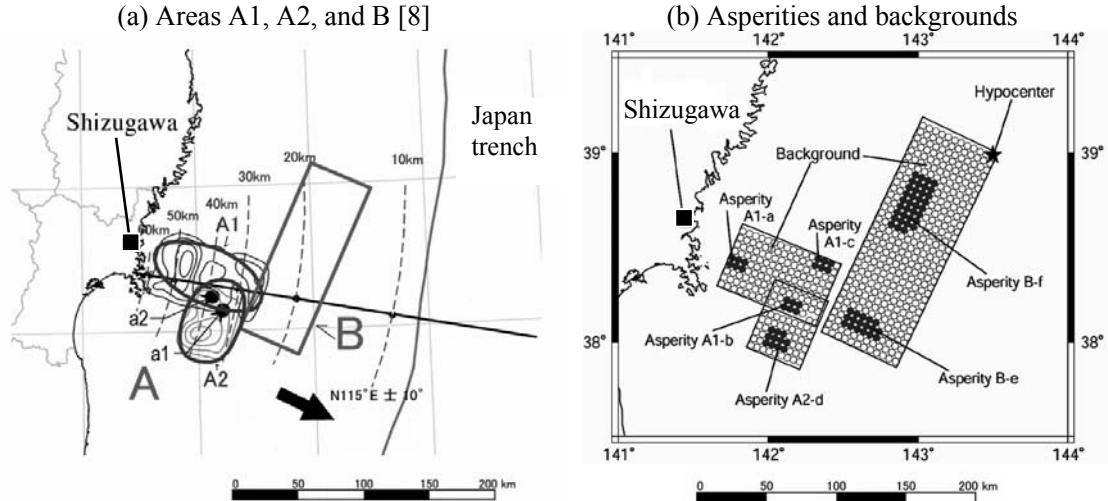
The combinations of the earthquakes and their occurrence probability in the next 50 years were calculated from the historical data by the National Research Institute for Earth Science and Disaster Prevention [3] as listed in Table 1.

Figure 4(b) shows the fault model of the earthquake with a magnitude of 8.1 in the area A1+A2+B based on the Headquarters of Earthquake Research Promotion [9], consisting of six asperities and three backgrounds. The fault parameters were evaluated by the procedure of Irikura *et al.* [10].

We assumed 378 fault models for A1, A2, B, A1+B, A2+B, and A1+A2+B with 5 % variability of the short-period level [11] and with 21 cases of variable locations of the point source at each sub-fault. Here, the short-period level is the flat level of the acceleration source spectrum in short-period range. So, we devided the 50-year occurrence probability in Table 1 by 63, and assigned the value to each fault model.

We adopted a semi-empirical method to predict large-earthquake ground motions by utilizing small earthquake records as empirical Green's functions [12]. Figure 5 shows the acceleration motion, the velocity motion, and the pseudo velocity response spectrum for the fault model in Figure 4(b).

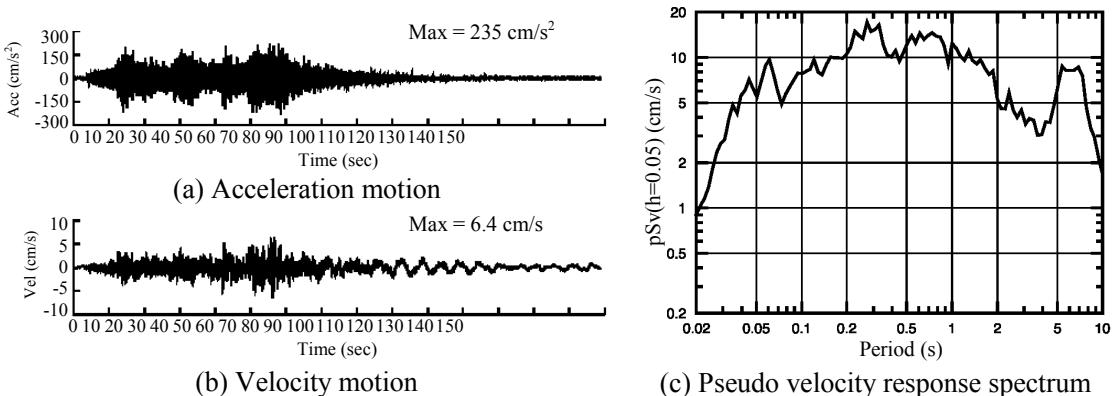
**Figure 4. Example of the fault models for the large plate-boundary earthquakes in the subduction zone (Category II)**



**Table 1. Combinations of the large plate-boundary scenario earthquakes and their occurrence probability in the next 50 years (Category II) [3]**

No.	Combination	50-year occurrence probability	No.	Combination	50-year occurrence probability
1	non	0	12	A2, A2	0.0074
2	B	0	13	A1, A1, B	0.063
3	A1	0.013	14	A1, A2, B	0.13
4	A2	0.013	15	A2, A2, B	0.063
5	A1, B	0.11	16	A1+B, A1	0.042
6	A2, B	0.11	17	A2+B, A1	0.042
7	A1+B	0.073	18	A1+A2+B, A1	0.042
8	A2+B	0.073	19	A1+B, A2	0.042
9	A1+A2+B	0.073	20	A2+B, A2	0.042
10	A1, A1	0.0074	21	A1+A2+B, A2	0.042
11	A1, A2	0.015		Total	1

**Figure 5. Predicted results from the fault model in Figure 4(b)**



## 5.2 Inland scenario earthquakes caused by blind faults not recognized on the surface (Category IV)

We adopted the procedure by Motohashi *et al.* [13] for modeling the source of inland earthquakes caused by blind faults for evaluating their 50-year occurrence probability. The 31 fault rupture scenarios and their 50-year occurrence probability are listed in Table 2.

In order to evaluate the 50-year occurrence probability, we calculated the annual occurrence frequency  $\Delta N(M_J)$  by equations (1) and (2) at first.

$$\log N(M_J) = 3.103 - 0.9M_J \quad (1)$$

$$\Delta N(M_J) = N(M_J - 0.05) - N(M_J + 0.05) \quad (2)$$

Here, equation (1) is Gutenberg-Richter's relationship between the annual occurrence cumulative frequency  $N(M_J)$  and the Japan Meteorological Agency magnitude  $M_J$  in the area of 5,412 km<sup>2</sup> around the test site, which was obtained by the procedure of the Headquarters for Earthquake Research Promotion [15].

Next, we calculated the annual occurrence frequency of the earthquakes caused by blind faults in the occupied area, 5×5=25 km<sup>2</sup> in this paper, by multiplying the annual occurrence frequency of equation (2) and the non-appearance probability of the faults on the surface and also multiplying the annual occurrence frequency and the ratio of the occupied area 25 km<sup>2</sup> to the entire area 5,412 km<sup>2</sup>.

Since we assumed three different depths of the asperities, shallow, middle, and deep, we divided the annual occurrence frequency into three. Also, since we assumed three different effective stresses on the asperities, high, middle, and low, we divided the annual occurrence frequency by normal probabilistic distribution. Here, high, middle, and low effective stresses corresponded to average plus standard deviation, average, and average minus standard deviation in the database by Somerville *et al.* [14].

We assigned the annual occurrence frequency to each fault rupture scenario, and we calculated the 50-year occurrence probability  $p_{50}$  by equation (3).

$$p_{50} = 1 - e^{-50\nu} \quad (3)$$

Here,  $\nu$  is the annual occurrence frequency for each fault rupture scenario.

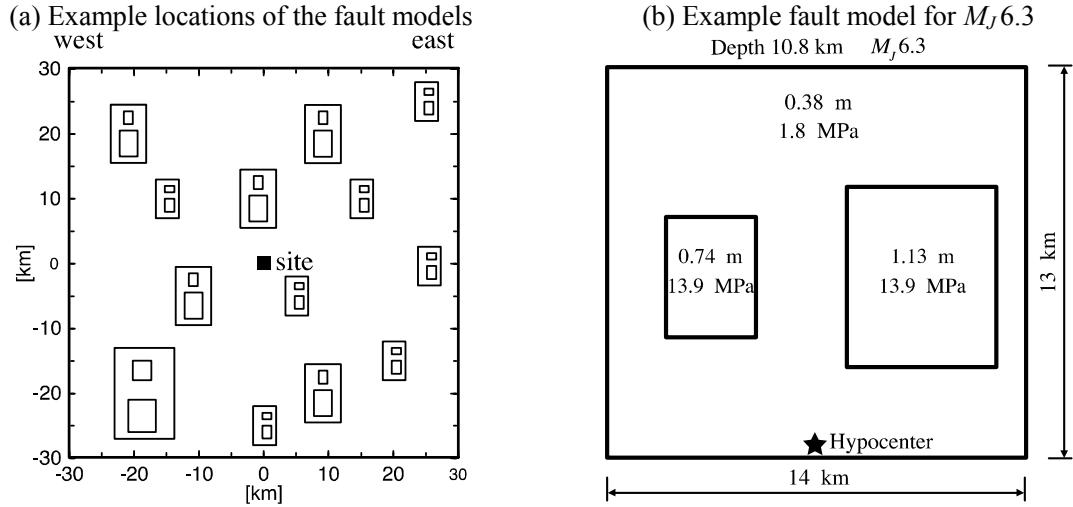
Figure 6(a) shows example locations of the fault models for the inland earthquakes caused by blind faults, and Figure 6(b) shows an example fault model for  $M_J$  6.3. We located the fault models every 5 km around the test site, totaling 121 locations.

Figure 7 shows the acceleration motion, the velocity motion, and the pseudo velocity response spectrum for the example fault model in Figure 6(b).

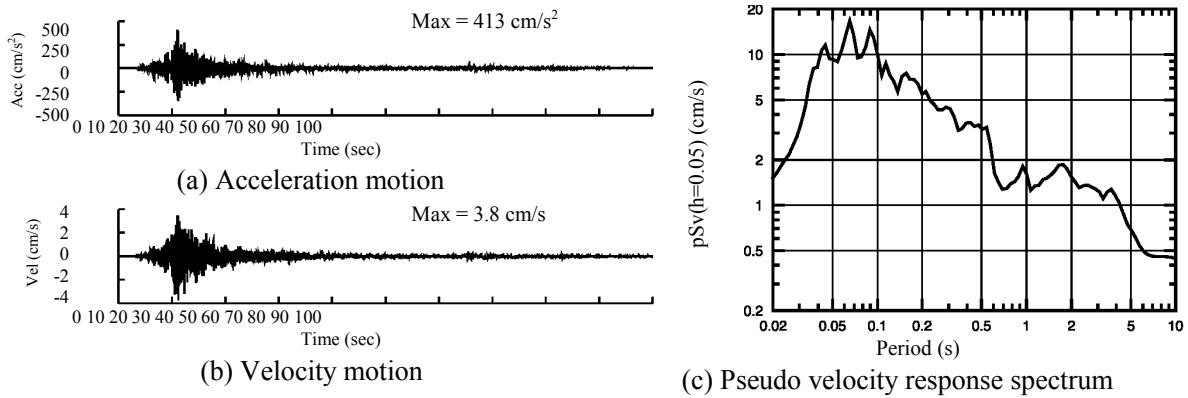
**Table 2. Inland scenario earthquakes caused by blind faults not recognized on the surface (Category IV) and their 50-year occurrence probability per 25 km<sup>2</sup> around the site.**

No	$M_J$	Depth of asperities	Effective stress	50-year occurrence probability	No	$M_J$	Depth of asperities	Effective stress	50-year occurrence probability	No	$M_J$	Depth of asperities	Effective stress	50-year occurrence probability
1	5.5	Shallow	High	1.739E-04	12	6.0	Shallow	Low	1.142E-04	23	6.5	Middle	Middle	2.269E-05
2	5.5	Shallow	Middle	2.131E-04	13	6.0	Middle	High	1.142E-04	24	6.5	Middle	Low	1.851E-05
3	5.5	Shallow	Low	1.739E-04	14	6.0	Middle	Middle	1.400E-04	25	6.5	Deep	High	1.851E-05
4	5.5	Middle	High	1.739E-04	15	6.0	Middle	Low	1.142E-04	26	6.5	Deep	Middle	2.269E-05
5	5.5	Middle	Middle	2.131E-04	16	6.0	Deep	High	1.142E-04	27	6.5	Deep	Low	1.851E-05
6	5.5	Middle	Low	1.739E-04	17	6.0	Deep	Middle	1.400E-04	28	6.6	Middle	Low	8.109E-06
7	5.5	Deep	High	1.739E-04	18	6.0	Deep	Low	1.142E-04	29	6.7	Middle	Middle	9.941E-06
8	5.5	Deep	Middle	2.131E-04	19	6.5	Shallow	High	1.851E-05	30	6.8	Shallow	High	4.055E-06
9	5.5	Deep	Low	1.739E-04	20	6.5	Shallow	Middle	2.269E-05	31	6.8	Deep	High	4.055E-06
10	6.0	Shallow	High	1.142E-04	21	6.5	Shallow	Low	1.851E-05					
11	6.0	Shallow	Middle	1.400E-04	22	6.5	Middle	High	1.851E-05					

**Figure 6. Examples of the fault models for the inland scenario earthquakes caused by blind faults not recognized on the surface (Category IV).**



**Figure 7. Predicted results for the example fault model in Figure 6(b).**



### 5.3 Moderate and small plate-boundary scenario earthquakes in the subduction zone (Category V)

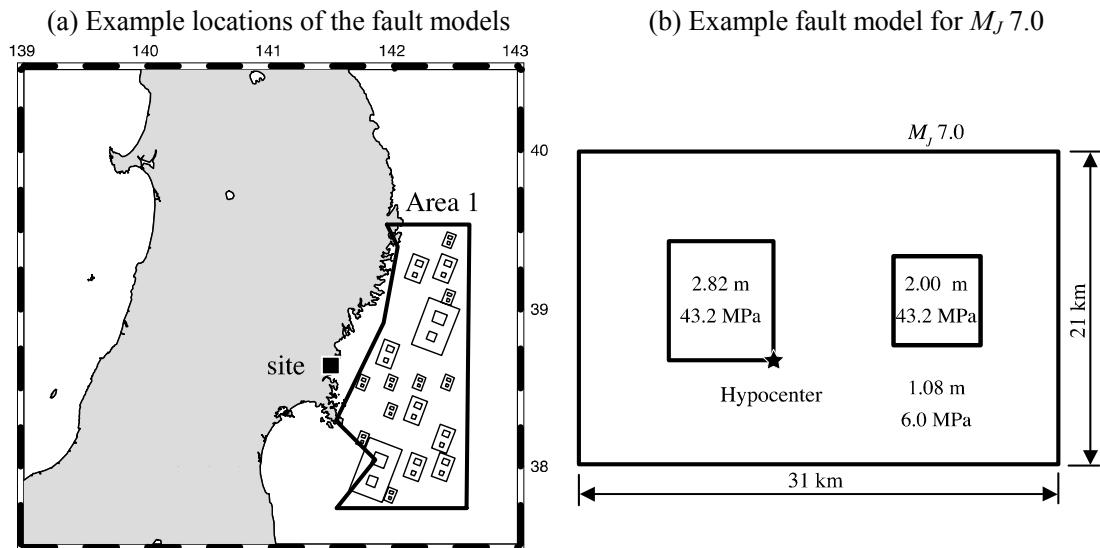
We adopted the Area 1 in Figure 8(a) for the source area of the moderate and small plate-boundary earthquakes in the subduction zone (Category V) based on the Headquarters of Earthquake Research Promotion [14]. We assumed fault rupture scenarios of the plate-boundary earthquakes with magnitudes of 5.5 to 7.2 as listed in Table 3. We evaluated the occurrence probability based on Gutenberg-Richter's relationship for the Area 1 by the same procedure as that we had applied to the inland earthquakes caused by blind faults. Here, Gutenberg-Richter's relationship was written by equation (4).

$$\log N(M_J) = 4.764 - 0.9M_J \text{ for Area 1 (19,843 km}^2\text{)} \quad (4)$$

The fault parameters for each rupture scenario were evaluated by the procedure of Irikura *et al.* [10]. Figure 8(a) shows example locations of the fault models for the moderate and small plate-boundary earthquakes, and Figure 8(b) shows an example fault model for  $M_J 7.0$ . We located the fault models every 20 km in the Area 1, totaling 34 locations.

Figure 9 shows the acceleration motion, the velocity motion, and the pseudo velocity response spectrum for the example fault model in Figure 8(b).

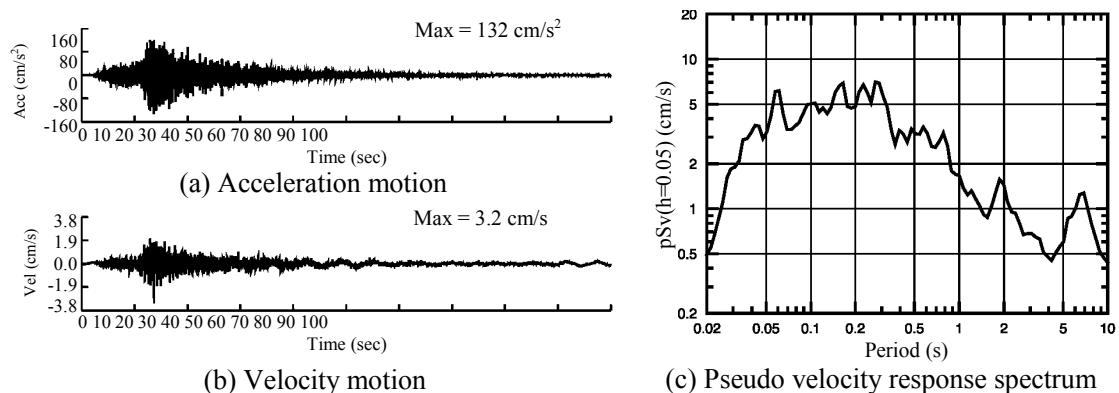
**Figure 8. Examples of the fault models for the moderate and small plate-boundary scenario earthquakes in the subduction zone (Category V).**



**Table 3. Moderate and small plate-boundary scenario earthquakes (Category V) and their 50-year occurrence probability per  $584 \text{ km}^2$  in the Area 1.**

$M_J$	Area 1 50-year occurrence probability / $584 \text{ km}^2$
5.5	3.885E-01
6.0	3.080E-01
6.5	1.224E-01
7.0	3.271E-02
7.1	7.194E-03
7.2	5.851E-03

**Figure 9. Predicted results for the example fault model in Figure 8(b).**



#### 5.4 Moderate and small intra-slab scenario earthquakes in the subducting Pacific Ocean plate (Category VI)

We adopted the Area 2 and the Area 3 in Figure 10(a) for the source area of the moderate and small intra-slab earthquakes in the subducting Pacific Ocean plate (Category VI) based on the Headquarters of Earthquake Research Promotion [15].

We assumed scenarios of fault ruptures of the intra-slab earthquakes with magnitudes of 5.5 to 7.2 as listed in Table 4. We evaluated the occurrence probability based on Gutenberg-Richter's relationship for each area by the same procedure as that we had applied to the inland earthquakes caused by blind faults. Here, Gutenberg-Richter's relationship was written by equations (5) and (6).

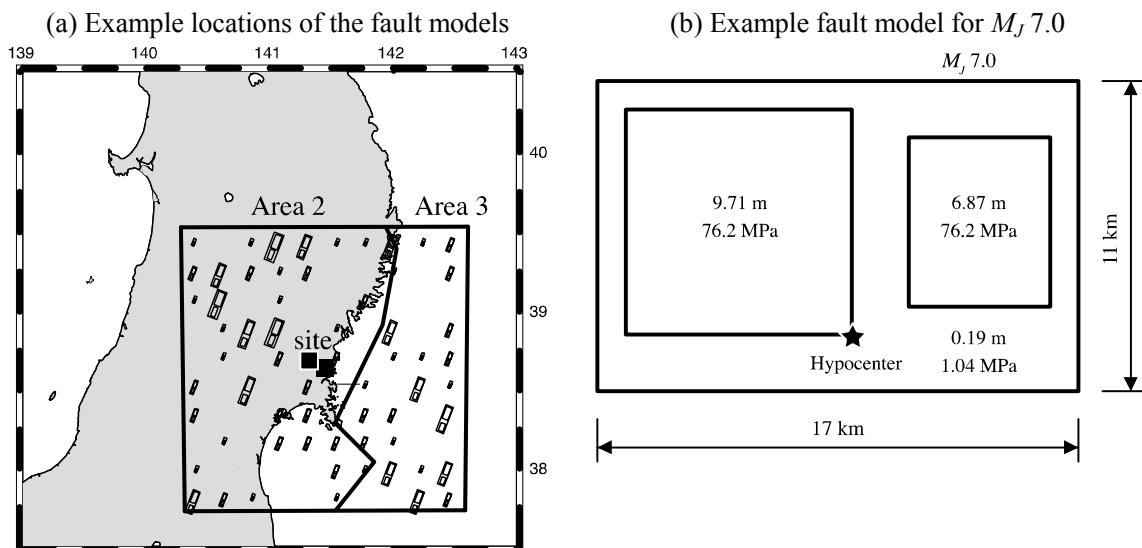
$$\log N(M_J) = 4.271 - 0.9M_J \text{ for Area 2 } (32,953 \text{ km}^2) \quad (5)$$

$$\log N(M_J) = 3.506 - 0.9M_J \text{ for Area 3 } (19,843 \text{ km}^2) \quad (6)$$

The fault parameters for each rupture scenario were evaluated by the procedure of Dan *et al.* [16]. Figure 8(a) shows example locations of the fault models for the moderate and small plate-boundary earthquakes, and Figure 8(b) shows an example fault model for  $M_J$  7.0. We located the fault models every 20 km in the Area 2 and the Area 3, totaling 100 locations.

Figure 9 shows the acceleration motion, the velocity motion, and the pseudo velocity response spectrum for the example fault model in Figure 8(b).

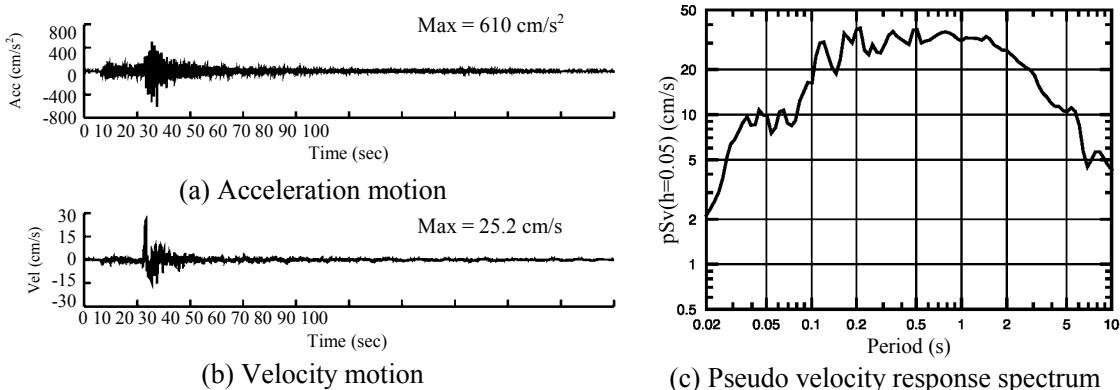
**Figure 10. Examples of the fault models for the moderate and small intra-slab scenario earthquakes in the subducting Pacific Ocean plate (Category VI).**



**Table 4. Moderate and small intra-slab scenario earthquakes (Category VI) and their 50-year occurrence probability per 499 km<sup>2</sup> in the Area 2 and per 584 km<sup>2</sup> in the Area 3.**

$M_J$	Area 2 50-year occurrence probability / 499 km <sup>2</sup>	Area 3 50-year occurrence probability / 584 km <sup>2</sup>
5.5	7.817E-02	2.681E-02
6.0	5.910E-02	2.013E-02
6.5	1.539E-02	5.164E-03
6.6	3.361E-03	1.124E-03
6.7	2.733E-03	9.133E-04
6.8	2.222E-03	7.424E-04
6.9	1.807E-03	6.035E-04
7.0	1.469E-03	4.906E-04
7.1	1.194E-03	3.988E-04
7.2	9.706E-04	3.242E-04

**Figure 11. Predicted results for the example fault model in Figure 10(b).**



## 6. Seismic hazard curves based on fault rupture scenarios

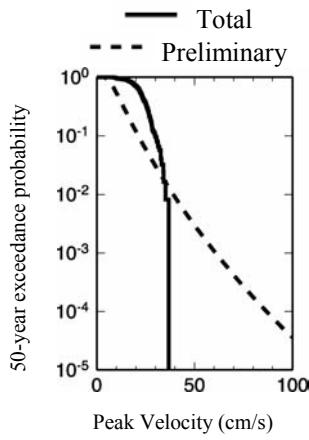
We predicted 10,666 ground motions in the NS and EW directions from the 5,333 fault models. Since each scenario had its own occurrence probability, we could calculate different kinds of the seismic hazard curves for the peak ground acceleration (PGA), the peak ground velocity (PGV), or the response spectrum at various periods. These were improved hazard curves from the preliminary results shown in Figure 3. Uncertainties expressed by the variability of the attenuation formula could be reduced by the present predicted ground motions with more detailed fault models. Once the scenario-earthquake-based hazard model was formed, design earthquake ground motions could be selected corresponding to the specified exceedance probability. For example, Figure 12 shows the hazard curve of PGV from the predicted strong ground motions. It also shows the preliminary hazard curve of PGV. The scenario-earthquake-based hazard curve had higher PGV in the range of high 50-year exceedance probability and lower PGV in the range of low 50-year exceedance probability than the preliminary hazard curve obtained by the attenuation formula.

In order to show some examples of design earthquake ground motions, we adopted the seismic hazard curve for pseudo velocity response spectrum pSv with a damping factor of 5 % at the period of 0.2 seconds in Figure 13. We assumed that the design criterion for the earthquake ground motions was  $10^{-4}$  in the annual exceedance probability, and then we selected predicted motions with the 50-year exceedance probability of  $10^{-2}$  to  $10^{-3}$ , corresponding to the pseudo velocity response spectrum of 33 cm/s to 54 cm/s at 0.2 seconds.

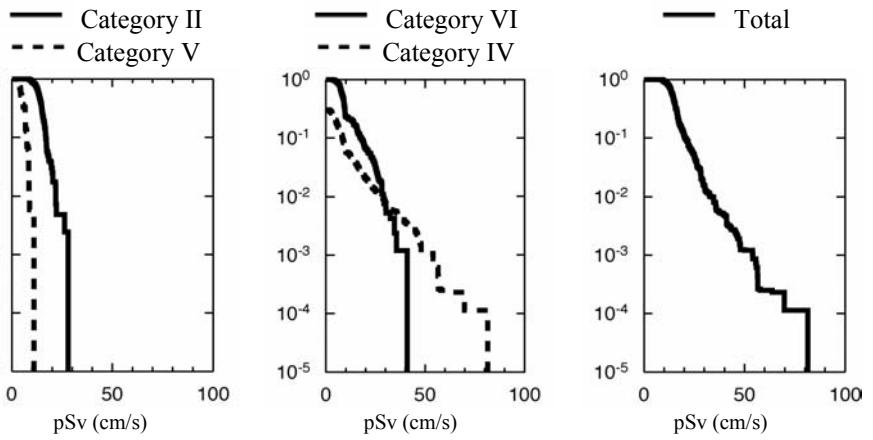
The selected motions consisted of 61 motions from the inland earthquakes caused by blind faults and 3 motions from the intra-slab earthquakes. Since the magnitude of the selected motions from the inland earthquakes were 5.5, 6.0, 6.5, and 6.8, we classified the motions into four groups. Meanwhile, the magnitude of the selected motions from the intra-slab earthquakes were 6.9 and 7.1, we treated them as one group.

Figure 14 shows the selected pseudo velocity response spectra for each class. Significant difference is observed between the spectra of the inland earthquakes caused by blind faults and those of the intra-slab earthquakes, but not observed among the spectra of the different magnitude for the inland earthquakes. Hence, we averaged the spectra in Figures 14(a) to 14(d) for the inland earthquakes caused by blind faults. As for the intra-slab earthquakes, we averaged the spectra in Figure 14(e).

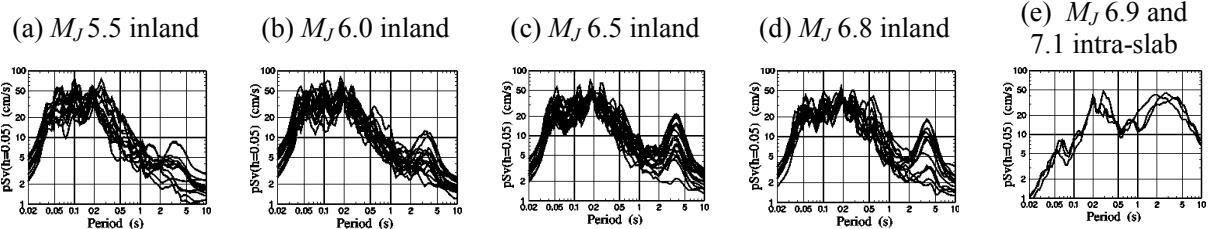
**Figure 12. Hazard curves of PGV**



**Figure 13. Hazard curves of pSv at the period of 0.2 seconds**



**Figure 14. Selected pseudo velocity response spectra with 50-year exceedance probability of  $10^{-2}$  to  $10^{-3}$  at the period of 0.2 seconds from the inland earthquakes caused by the blind faults not recognized on the surface (Category IV) and from the intra-slab earthquakes (Category VI).**



## 7. Generation of design earthquake ground motions

Figure 15(a) shows the averaged spectrum for the inland earthquakes caused by blind faults. We generated 61 artificial motions with the averaged spectrum in Figure 15(a) and with the phases of the selected motions for the inland earthquakes. Any of the 61 artificial motions are available for the design earthquake ground motions. Figures 15(b) and 15(c) show examples of the design earthquake ground motions for the inland earthquakes caused by blind faults.

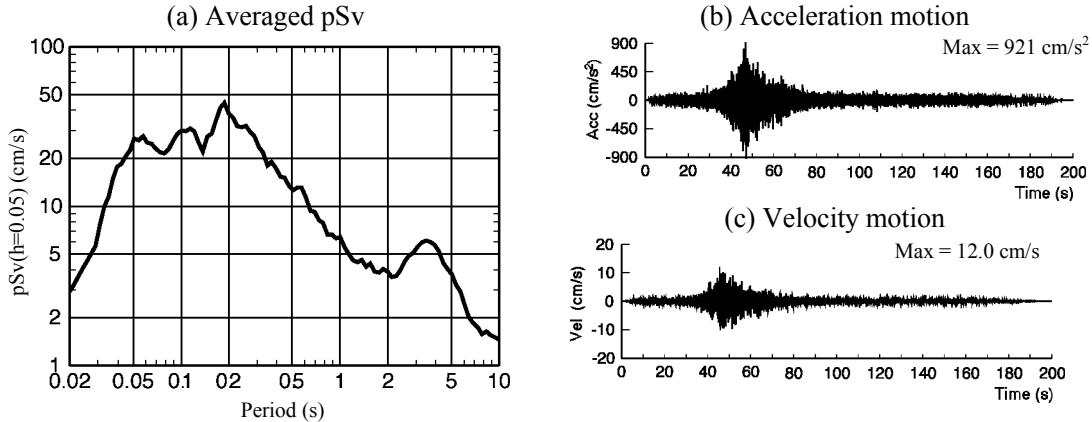
Figure 16(a) shows the averaged spectrum for the intra-slab earthquakes, and Figures 16(b) and 16(c) show examples of the design earthquake ground motions.

Figures 15 and 16 show that the pseudo velocity response spectra at 0.2 seconds are almost equal to each other while the PGA from the inland earthquake is 3 times larger than the PGA from the intra-slab

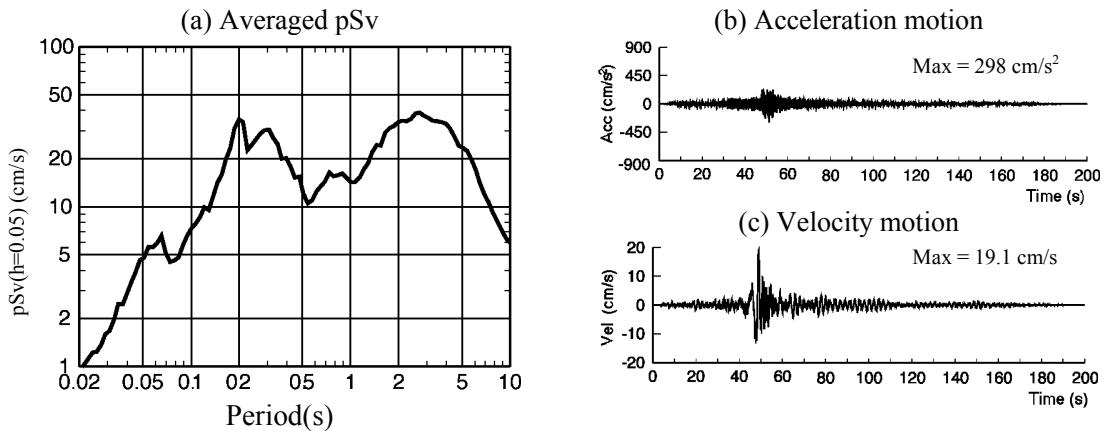
earthquake and the PGV from the inland earthquake caused by blind faults is smaller than the PGV from the intra-slab earthquake.

Because the PGA of  $921 \text{ cm/s}^2$  from the inland earthquake is rather too large for the PGV of  $12.0 \text{ cm/s}$ , we should investigate the  $f_{max}$  [17] of the element earthquakes used as empirical Green's function in the further study.

**Figure 15. Averaged spectrum of those in Figures 14(a) to 14(d) and generated design earthquake ground motions with the averaged spectrum for the inland earthquakes caused by blind faults.**



**Figure 16. Averaged spectrum of those in Figure 14(e) and generated design earthquake ground motions with the averaged spectrum for the intra-slab earthquakes**



## 8. Conclusions

We proposed a procedure for obtaining probabilistic design earthquake ground motions based on scenario earthquakes, and presented two examples of the design earthquake ground motions at the test site of Shizugawa in Miyagi Prefecture, Japan. Uncertainties expressed by the variability of attenuation formula could be reduced by the predicted ground motions with more detailed fault models. The scenario-earthquake-based hazard curve had higher PGV in the range of high 50-year exceedance probability and lower PGV in the range of low 50-year exceedance probability than the preliminary hazard model by the attenuation formula.

Our procedure explicitly indicated the parameters for the fault rupture scenarios and the exceedance probability of the design earthquake ground motions for nuclear power plants.

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# Determination of Fragility Curves of Aging Plants by Ambient Vibration Monitoring

**Helmut WENZEL<sup>1</sup> and Günther ACHS<sup>2</sup>**

## ***Summary***

Seismic hazard analysis has become more and more sophisticated in the last few decades. An enormous amount of research in the field of seismology led to a careful handling with seismic demand. Knowing that many existing nuclear power plants (not only around Austria) are situated in previously “quiet” zones provides interesting but difficult challenges for earthquake engineers. The first fundamental idea of the proposed method is to get better information about the subsoil condition to reduce the epistemic uncertainties by free field tests around a building or plant. Furthermore the consequences of the aging of building materials, additional structural components or reduced stiffness can hardly be estimated by calculation. Therefore it is absolutely indispensable to run measurements on real structures. The proposed method which combines the capacity spectrum method with in-situ measurements should therefore be a requirement for each part of a nuclear power plant. Finally under consideration of all aspects a reliable capacity curve for the building structure should be evaluated, which represents the structural behaviour under a realistic seismic demand.

## **1. Introduction**

During the last few decades various techniques and methodologies for seismic hazard analysis were developed. According to their reliability they are more or less time-consuming and therefore expensive. Even if they are using highly sophisticated algorithms the lack of input data can be problematic for all of them.

One of the biggest unknown parameters in determining the seismic response of structures is the change of the dynamic behaviour with elapsed time. The reasons for these changes are the aging material and the structural discrepancies with the initial state. Even if the aging of the material itself can be considered very well, arisen inelastic zones (e.g. cracks) can hardly be handled. Similar to the unknown of the aging material there is always a big uncertainty in managing structural and non structural components of a building. It is not economical to consider each change in an overall seismic hazard analysis.

The proposed method can evaluate a prediction for the changes in the fragility curve with periodic measurements (to get the changes in the dynamic behaviour).

## **2. Current outline of epistemic uncertainties**

The major benefit of the probabilistic seismic hazard analysis is to reduce the epistemic uncertainties wherever it is possible. Most of the uncertainties in the data themselves are aleatory (because they are subjected to random sampling). The epistemic uncertainties of course can be reduced by learning about unknown parameters and intensive development. In the following, some of the most interesting findings and topics of new research work for Austria (especially for the Vienna Basin) are listed.

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## 2.1 Source characterization

The considerations about active and non-active faults have been extensively enhanced in the last few years. According to some publications [Hinsch,R., 2003; Decker, K., 2005] the risk of an earthquake which could be higher than any of the so far documented events is enormous.

### 2.1.1 Seismological considerations

In former times the considerations of historic earthquakes in Austria were constricted to written notes. Therefore it was evident that former events were not regarded for recurrence rates. Scientists of the Vienna University [Hinsch,R., 2003] found out that there are some potential capable faults along the Vienna Basin. They investigated the slip rates of the Vienna Basin Transfer Fault and determine an underestimation of the earthquake potential. According to their evaluation they calculated a possible earthquake of a magnitude M = 6.1 with a recurrence rate of 800 years.

### 2.1.2 Size considerations

In another publication [Decker, K., 2005] a detailed map of active faulting and quaternary basin subsidence in the Vienna Basin can be found. With this detailed map it is possible to identify the actual capable faults and their maximum magnitude.

## 2.2 Attenuation of earthquakes

One of the biggest challenges concerning the path of seismic waves from the source to the site would be to develop an accurate attenuation relationship for moderate seismic areas. In the past it was hardly possible to develop attenuation rules for Europe. Due to the lack of recording stations and strong motion recordings it was nearly impossible to obtain enough data to run a reliable regression analysis.

During the last few years a part of the PEER Next Generation Attenuation (NGA) Project [Campbell, K. W., 2006] was assigned to develop new relationships for active shallow crustal regions in Europe. The general form of the attenuation law is given as [Campbell, K. W., 2006]:

$$\ln Y = f_1(M) + f_2(R) + f_3(F) + f_4(HW) + f_5(S) + f_6(D) + \varepsilon_T$$

with:  $f_1(M)$  ... dependent on magnitude.

$f_2(R)$  ... dependent on source-to-site distance.

$f_3(F)$  ... dependent on style of faulting.

$f_4(HW)$  ... dependent on hanging-wall effects.

$f_5(S)$  ... dependent on shallow site conditions (linear and nonlinear).

$f_6(D)$  ... dependent on sediment depth (shallow and 3-D basin effects).

The equation can be seen as a first estimation of an attenuation relationship for active shallow crustal regions which need to be evaluated to have confirmation for the application to European countries.

## 2.3 Improvements

To improve the accuracy of probabilistic methods it is indispensable to obtain as much information as possible. Therefore the proposed SEISMID-technology can be very useful in miscellaneous fields. On the one hand the developed seismic hazard maps for cities can be very useful for urban power plants, on the other hand they can be adopted for wide spread nuclear plants, where inhomogeneous subsoil is obvious. The proposed methodology will be presented and introduced in section 3.

Furthermore aging of plants accumulates a very dangerous type of uncertainty. The consequences of the aging of building materials, additional structural components or reduced stiffness can hardly be estimated by calculation. The proposed method which combines the capacity spectrum method with in-situ measurements should therefore be a requirement for each part of a nuclear power plant. To introduce this method, section 4 gives an example from bridge engineering and section 5 should connect the method to the problem of aging power plants.

### **3. Characterization of local soil conditions**

#### **3.1 Introduction**

The influence of local soil conditions on the structural behaviour of buildings under seismic loading is a well-known scientific fact. On the one hand soft soil layers can adversely affect the reflection, refraction and amplification of seismic shear waves; on the other hand the low natural frequency of the soil-structure system can cause resonance effects.

The prediction of site-effects is generally done by numerical simulations, assuming that the conditions of the local subsoil are well-known. The large number of uncertainties (soil properties, layer thickness, inclination of soil layers, groundwater table, structural parameters, etc.) could be easily reduced by additional measurements, which can be implemented very quickly. If the obtained information is assembled to an overall probabilistic seismic hazard analysis the reliability could be increased extensively.

#### **3.2 H/V – Amplification ratio**

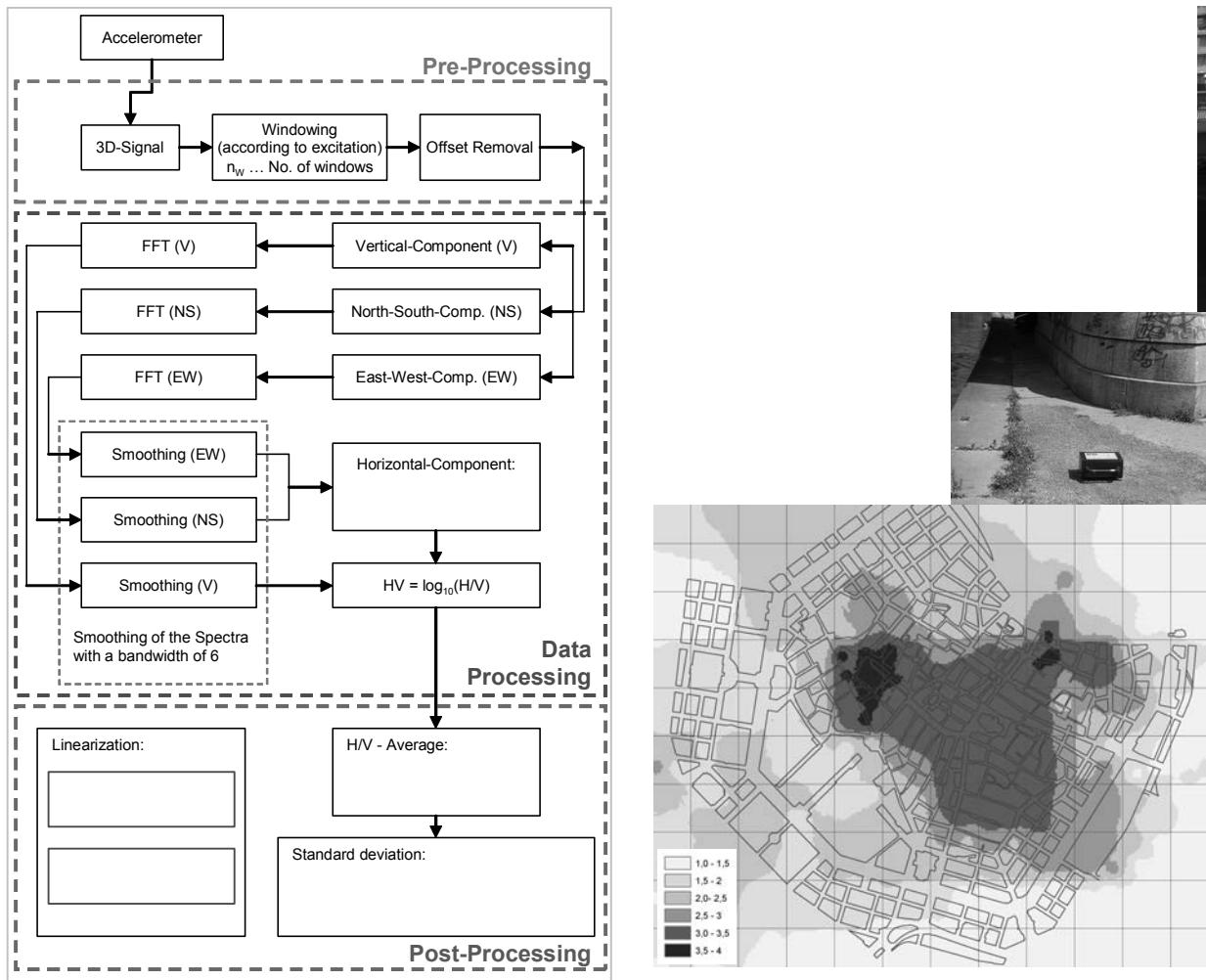
The proposed method, developed and adjusted during the national research project SEISMID, uses the H/V-Method according to Nakamura [Nakamura, Y., 1989]. It is a very effective and economic method, whereby a meaningful local amplification factor is determined. At the present time VCE is trying to implement the obtained information in geographical information systems [Wenzel, H., 2006]. Therefore the measured amplification values are devolved into a seismic hazard map.

With the presented method the following local soil parameters can be defined:

- Basic natural frequency  $f_0$  of the soil-structure system.
- Knowing the basic natural frequency  $f_0$ , either the shear wave velocity  $v_s$  or the layer thickness can be calculated, if one of both values is known.
- Determination of the amplification factor (the peak of the H/V-Spectra can be transferred into an amplification factor for seismic shear waves).

The input for the H/V-Computation can consist of transient or ambient vibration signals. Usually in the proposed project it was more beneficial to use ambient excitation because the large number of samples which are needed for statistical evaluation would, in case of transient excitation, lead to unacceptable economical disadvantages.

**Figure 1. Flow chart for the determination of the H/V-amplification ratio (a); Measurements of the H/V-ratio in Vienna (b); Seismic hazard map for the 1<sup>st</sup> district (c).**



### 3.3 Predominant wave path - Boundary conditions

As local subsoil layers may vary a lot it is indispensable to get information about the different wave-path directions. As a further step this information should be included in the evaluation of fragility curves in probabilistic seismic hazard approaches.

To specify the different possible wave path directions it is necessary to drive measurements with a constant input to get the following information:

- Determination of various frequency spectra for each measuring point.
- Calculation of relative and absolute transfer function.
- Evaluation of the frequency-dependent damping of the subsoil.

Supplementary to this important data it is easy to get additional knowledge of the coherences of subsoil and the building structure. The following information may be very important to calibrate numerical models where soil-structure interaction is considered:

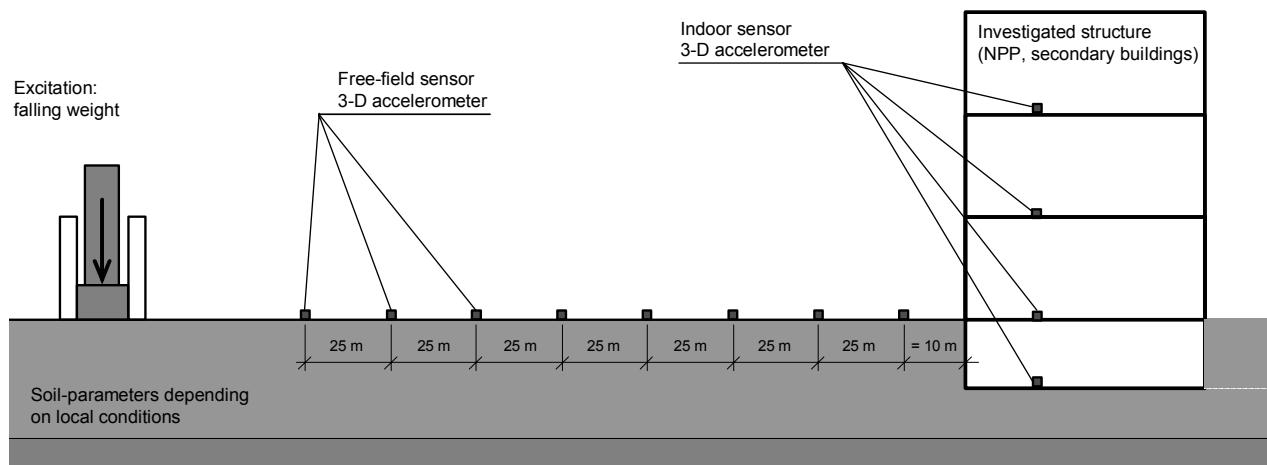
- Calculation of transfer functions for subsoil and structure.
- Evaluation of parameters for the boundary conditions of FE-analysis (spring and damper).
- Determination of soil-structure-interaction effects.

Due to the fact that it is necessary to position secondary accelerometers inside the building it is obvious that also structural components can be determined easily:

- Evaluation of the leading values for floor response spectra.
- Calculation of transfer functions for each storey and for the global system.
- Natural frequencies and mode shapes.

The use of different information of measurements can be very economical and is therefore a future-orientated method.

**Figure 2. In-situ test set-up for measurements to determine the predominant wave path**



### 3.4 Implementation in PSHA

The use of probabilistic seismic hazard analysis provides a powerful tool in which the large number of uncertainties can be identified and combined to obtain a realistic and complete view of the seismic exposure of a structure. In the present case we can extract more detailed information about the site conditions as well as for the attenuation of earthquake waves. Combined with the current research work of seismologists (earthquake source characterization) an overall deterministic approach could be developed.

## 4. Capacity curves for bridges under seismic demand

### 4.1 Introduction

Several methodologies have been developed in recent years to perform adequate and accurate seismic hazard analysis for critical facilities. Bridges are one of the most sophisticated but also highly vulnerable structures. On the other hand due to their well defined pattern they can be easily reviewed and analyzed. As a result of their massive girder construction and at the same time slender and high piers they are very vulnerable against horizontal forces. Since wind excitation is particularly dangerous at the construction stage the proper horizontal loads in the final stage have to be regarded from the seismic exposure.

As a fact of the very economic analysis the dimensions of wide span bridges closely cover the demands. Therefore the capacity of horizontal load-bearing structures is limited. Hence it can be concluded that even in low seismicity areas a simple but very effective and accurate seismic hazard analysis is needed.

In the following, a vulnerable bridge is selected and an assessment methodology is described which can be used to calculate a realistic bearing capacity under an actual possible earthquake demand.

Figure 3. Schottwien bridge at the Semmering pass in Lower Austria (S6 Motorway Bridge)

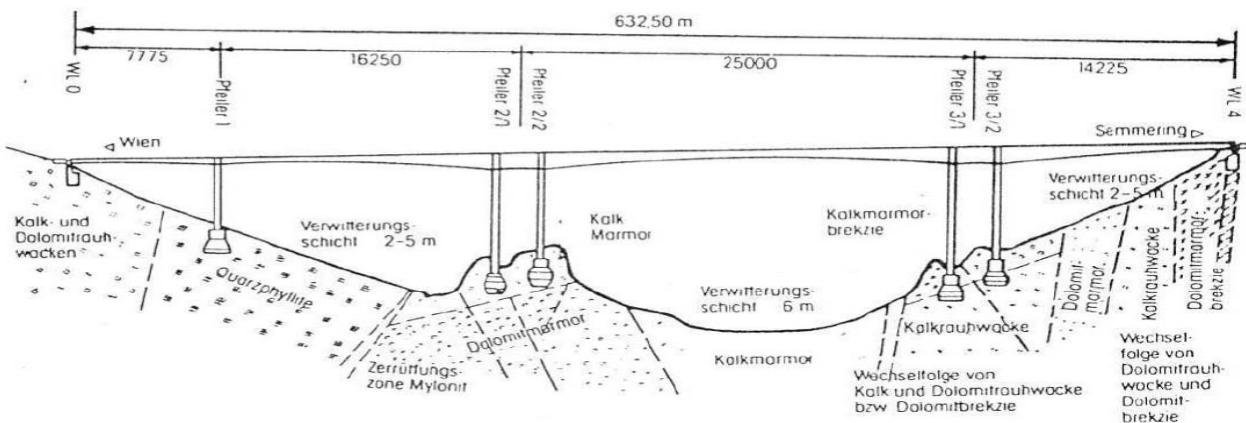


#### 4.2 Local conditions

The local soil conditions of wide span bridges can vary a lot, see e.g. Figure 4. Therefore it is necessary to get information of the subsoil at every pier. This information can be outstanding, particularly due to the fact that seismic waves are reflected, refracted and amplified at the boundaries of different soil-layers. If most of the piers and abutments are erected on rock it can be even disastrous for the global equilibrium if some of the piers are erected on soft soil layers. Therefore it is necessary to identify the local soil amplification factors for the whole bridge environment.

To estimate the local amplification factor it is prevalent to use the H/V-Method according to Nakamura [Nakamura, Y., 1989] due to the fact that it is a very simple and user-friendly practice. The method is convenient for almost every region in Europe because it was successfully applied in countries with moderate seismicity where no strong motion data were available. Therefore the input for the H/V-Computation can also consist of transient or even ambient excited signals.

Figure 4. Geologic fundamentals of the Schottwien bridge



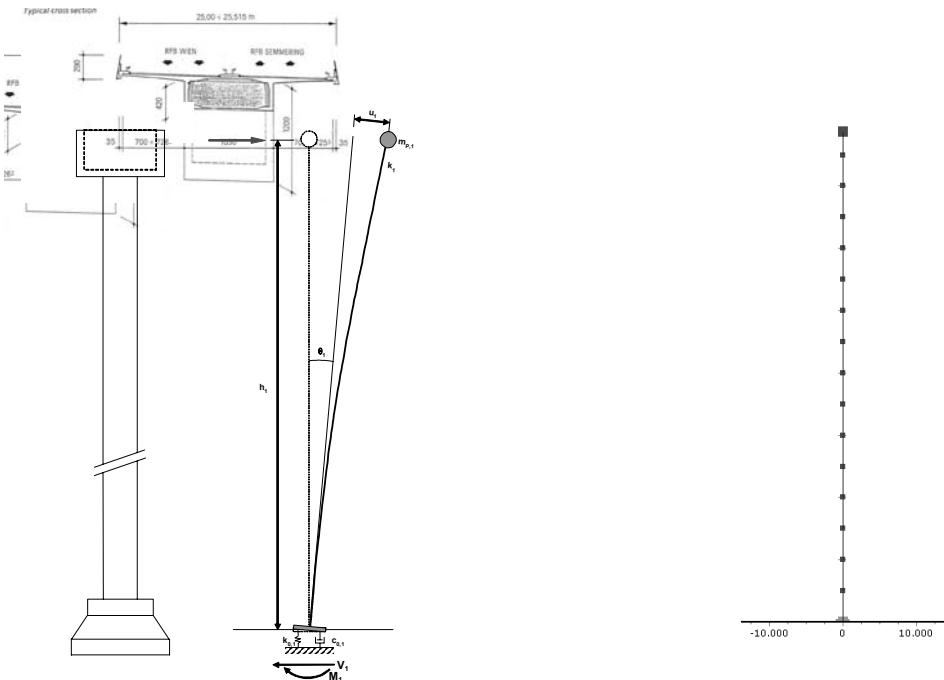
#### 4.3 Pushover curve

Before the pushover analysis can be accomplished an adequate model has to be defined. Therefore the bridge pier can be modelled as a SDOF-system.

The point mass is calculated by the contributing part of the bridge girder and its build-ups. The generalisation of the system is shown in figure 6.

The analysis was performed using the displacement-based adaptive pushover technique to estimate the seismic capacity of reinforced concrete structures [Pinho, R., 2006].

Figure 5. Numerical model of Pier 3, and implementation [SeismoStruct]



#### 4.4 Demand curve

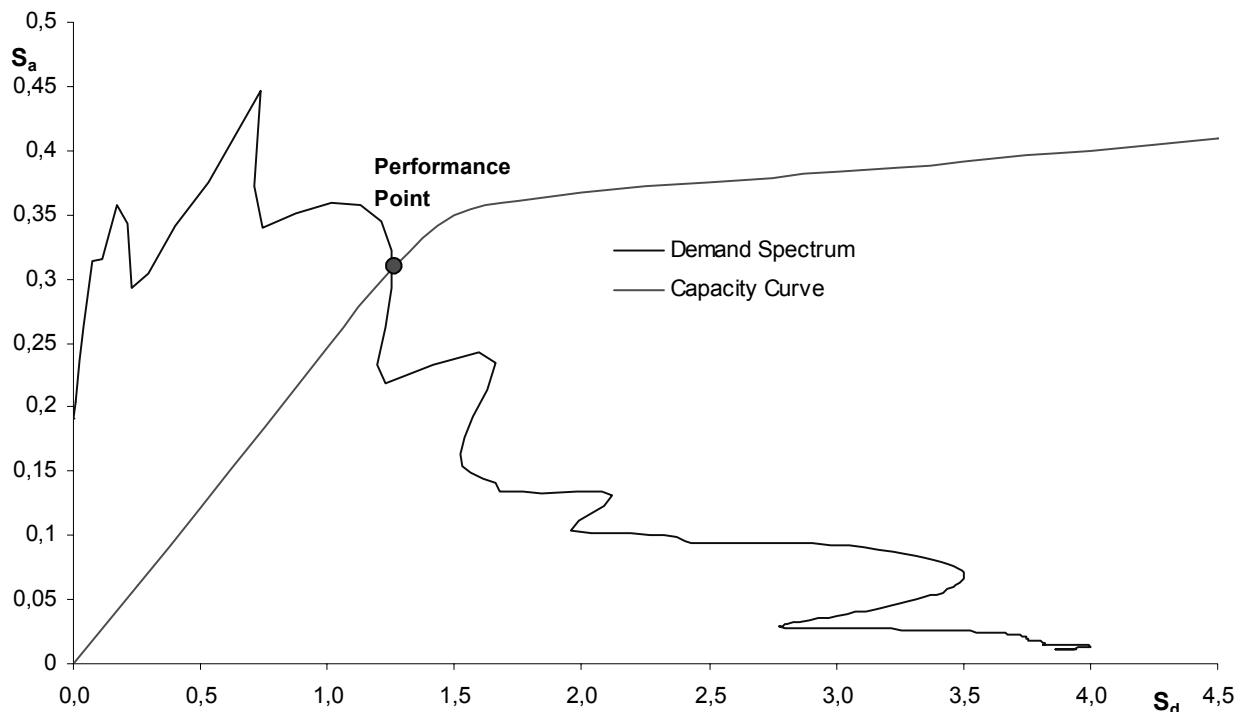
The demand curve is calculated using an adequate attenuation relationship [Campbell, K. W., 2006], using the nearest capable fault with a certain probability for the highest earthquake event.

For the consideration of the subsoil in the configuration of the demand spectrum, existing building codes and regulations specify different subsoil classes. Unfortunately the layers of the subsoil have local variations which cannot be taken into account in conventional seismic hazard analysis. Therefore it would be helpful to evaluate local amplification factors and include their influence in the demand spectrum.

#### 4.5 Capacity curve – Performance point

If the intersection of the capacity curve for the structure and the determined demand spectrum, the so-called performance point of the structure is located in the elastic area, the condition of the structure can be represented as suitable to resist a maximum feasible earthquake.

Figure 6. Example of an elastic performance point



Whether the performance point is in the inelastic range, further considerations have to be taken into account:

- Representation of the inelastic behaviour.
  - Steel – plastic hinges.
  - Reinforced concrete – cracks.
- Realistic representation of aging of material.

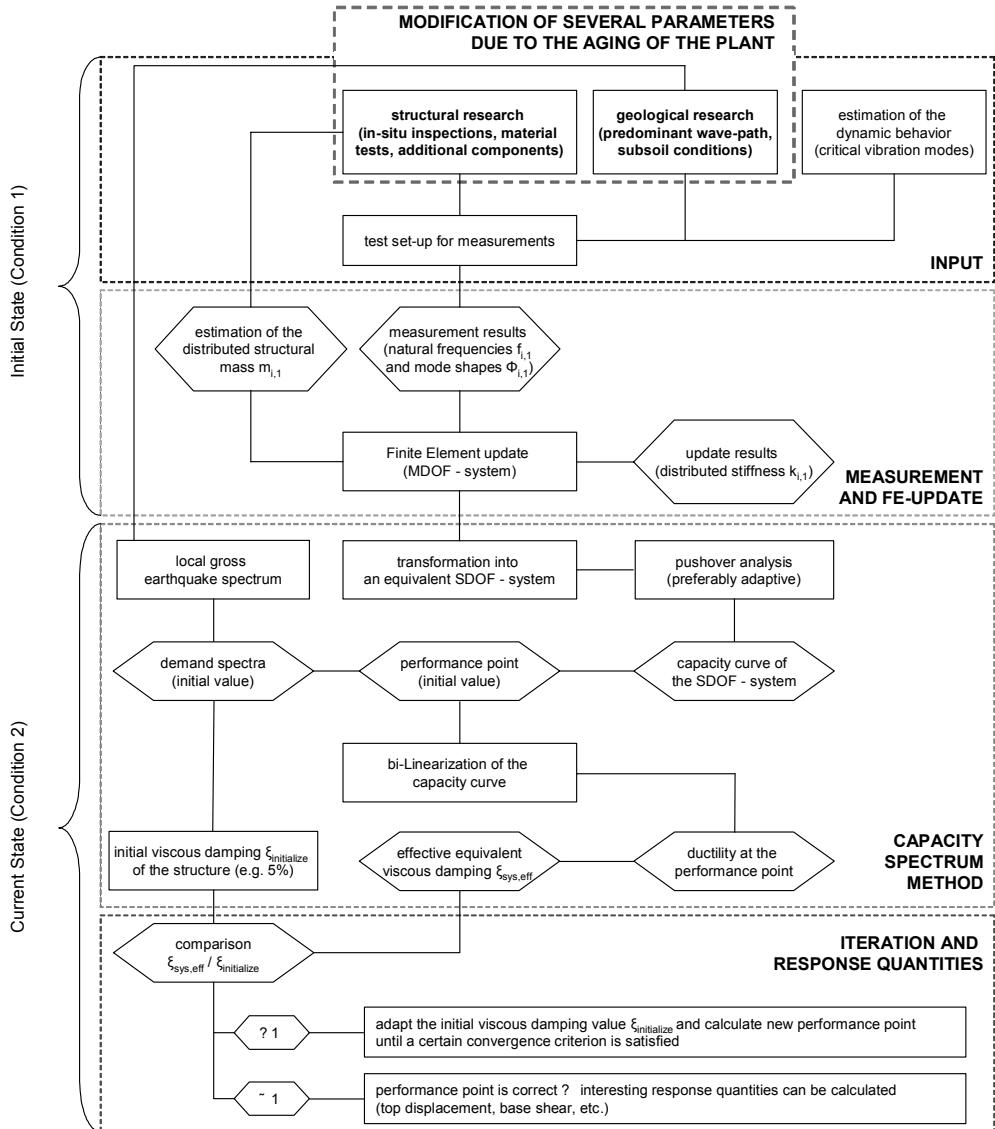
## 5. Capacity curves for aging plants

### 5.1 Introduction

Aging of plants may cause serious problems for the structural and dynamic behaviour. Due to the deteriorating conditions of building materials and additional structural and non structural parts the dynamic behaviour is changing drastically.

The identification of this modification can be easily done by dynamic measurements. Afterwards the actual bearing capacity with regard to all aging effects can be determined using the capacity spectrum method. In the following figure the methodology of the determination of capacity curves for aging plants is described.

**Figure 7. Flow chart – determination of capacity curves for aging plants**

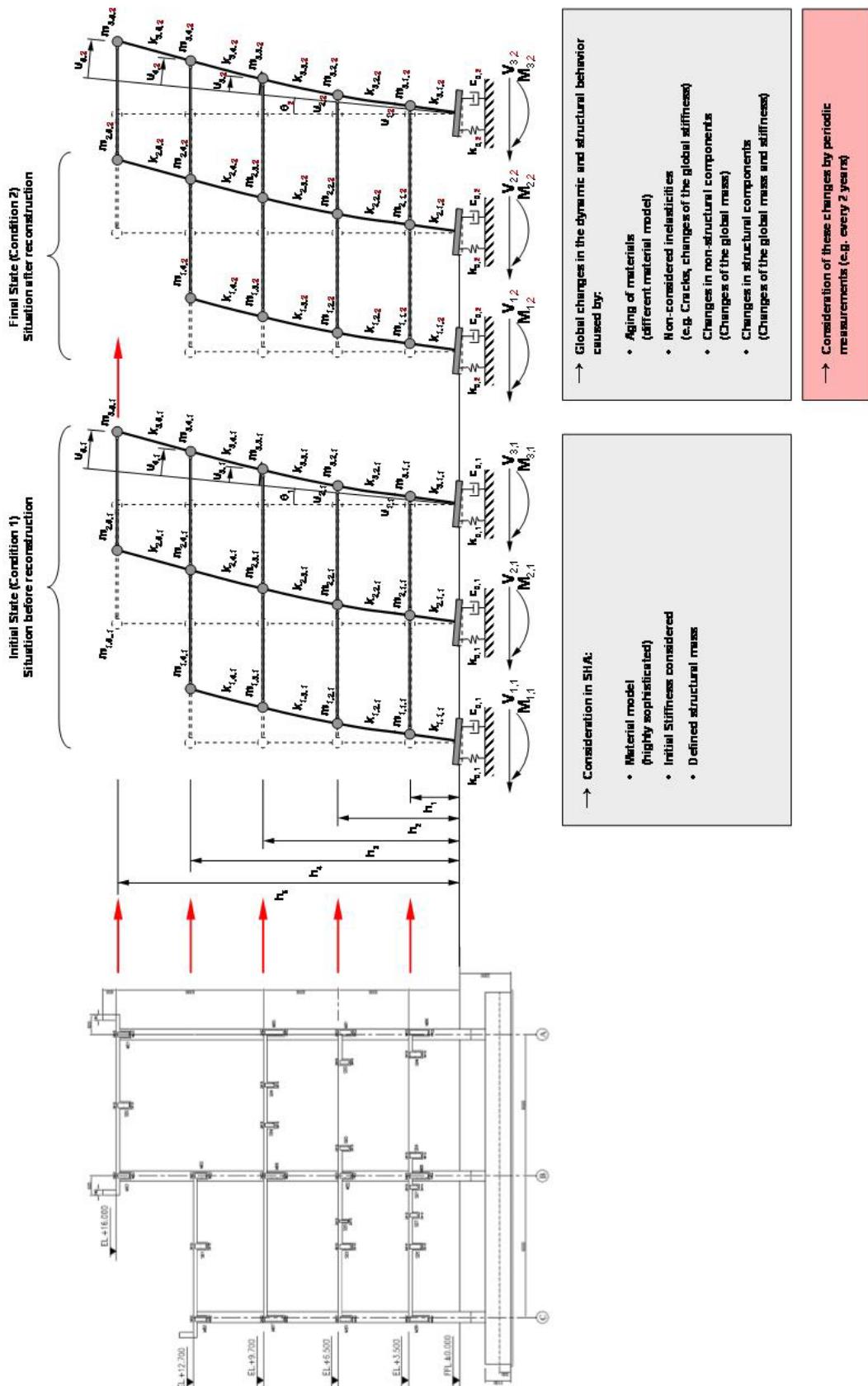


## 5.2 Pushover analysis – Capacity curve

Compared to the adaptive capacity spectrum method for bridge structures the analysis for plants is more sophisticated because the mass and stiffness distribution of the structures does not have to be uniform. In general we have to consider a frame structure and transform it to an MDOF-System, as shown in figure 9.

The analysis is preferably performed using the displacement-based adaptive pushover technique. The currently used pushover techniques are performed using monotonically increasing lateral forces with invariant distribution until a certain displacement target is reached [Pinho, R., 2006]. The basic assumption is that the response of the system is controlled by the fundamental mode shape which is not changing with structure yields and plastic hinges. The innovative adaptive pushover analysis is computing the shape of the load vector in each analysis step and therefore can account for problems arising from the disadvantages of conventional methodologies.

Figure 8. Transformation of a plant structure to an MDOF-System



After the pushover analysis is performed, in a first step it is necessary to transform the MDOF-System into an equivalent SDOF-System in order to determine the capacity curve. Therefore it is necessary to define the equivalent system displacements  $\Delta_{sys,k}$  at each analysis step k.

$$\Delta_{sys,k} = \frac{\sum_i m_i \Delta_{i,k}^2}{\sum_i m_i} \quad \text{with: } m_i \quad \dots \text{ floor mass}$$

$$\Delta_{i,k} \quad \dots \text{ floor displacement}$$

Afterwards the equivalent system mass  $M_{sys,k}$  can be defined at each analysis step:

$$M_{sys,k} = \frac{\sum_i m_i \Delta_{i,k}}{\Delta_{sys,k}}$$

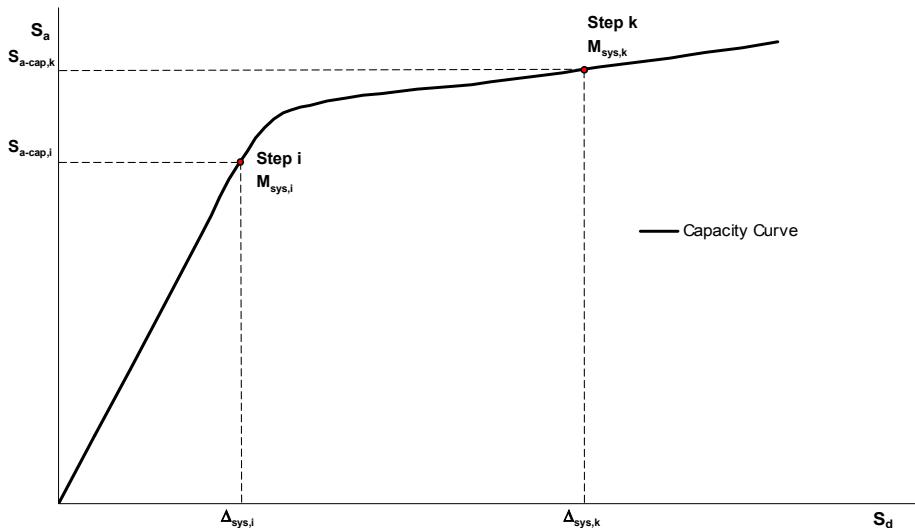
With the equivalent system mass  $M_{sys,k}$  and the calculated base shear  $V_{b,k}$ , the equivalent system acceleration  $S_{a-cap,k}$  can be defined:

$$S_{a-cap,k} = \frac{V_{b,k}}{M_{sys,k} g} \quad \text{with: } V_{b,k} \quad \dots \text{ base shear at each analysis step k}$$

$$g \quad \dots \text{ acceleration of gravity}$$

Subsequently the capacity curve of the equivalent SDOF - System can be plotted.

**Figure 9. Capacity Curve of the equivalent SDOF-System**



### 5.3 Capacity design – Demand spectrum

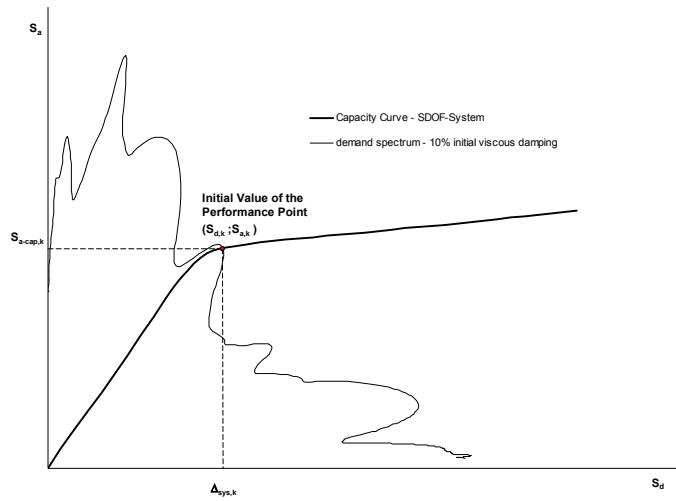
To compute the performance point, it is necessary to calculate the demand spectrum of the investigated building structure. The following considerations have to be taken into account to calculate a realistic seismic demand (according to the definition of probabilistic seismic hazard analysis):

- Source considerations (active and capable faults).

- Size characterization (maximum magnitude, duration).
- Recurrence rate.
- Attenuation relationships.
- Site effects (soil amplification, consideration of soil-structure interaction effects).

If all of these terms are observed correctly in a probabilistic approach, a seismic demand curve calculated with an initial viscous damping value  $\xi_{initialize}$  can be plotted against the capacity curve of the equivalent SDOF-System.

Figure 10. Capacity curve of the equivalent SDOF-system



The corresponding spectral reduction factor can be calculated from the initial viscous damping value  $\xi_{initialize}$  as follows:

$$\eta_{initialize} = \sqrt{\frac{10}{5 + \xi_{initialize}}}$$

In the next step of the evaluation, the computed capacity curve of the equivalent SDOF-System has to be bi-linearised.

After the calculation of the ductility  $\mu_{sys}$  of the equivalent SDOF-system, the effective viscous damping  $\xi_{sys,eff}$  of the system can be computed:

$$\mu_{sys} = \frac{x_m}{x_y} \quad \text{with: } x_m \dots \text{maximum deformation}$$

$x_y \dots \text{deformation at the yielding point}$

$$\xi_{sys,eff} = 5 + \frac{100}{\pi} \left( 1 - \frac{1}{\sqrt{\mu_{sys}}} \right) \quad \text{with: } \mu_{sys} \dots \text{ductility of the equivalent SDOF-system}$$

In the last step the effective viscous damping  $\xi_{sys,eff}$  of the system has to be compared with the initial viscous damping value  $\xi_{initialize}$ . The iteration has to be repeated until a certain convergence criterion is satisfied.

$$(\xi_{sys,eff} - \xi_{initialize}) \leq \varepsilon \text{ with: } \varepsilon \dots \text{convergence criterion (e.g. 0.001)}$$

Finally the response quantities of interest can be computed in correspondence with the Performance Point deformation level (storey drift).

## 6. Correction of existing fragility curves

To express the vulnerability of building structures it is common to develop fragility curves, which provide the probability of exceeding a prescribed level of damage for different ranges of ground motion intensities. Their main and outstanding purpose is to express the overall risk to the building structures from potential earthquakes.

To develop the fragility curves, statistical simulation approaches (e.g. Monte Carlo) are used containing uncertainties of ground motion parameters and structural components. Although these evaluations are highly sophisticated, there are still some uncertainties which could be reduced by more detailed investigations. Some of them can be covered by the above described measurements.

### 6.1 Structural components

Using the numerical model introduced in figure 8, the inelastic behaviour of the structure does not have to be reduced to the connections of beams and hinges (former numerical models were often dealing with plastic hinge locations at these zones). The inelastic behaviour of structures can vary a lot, depending on the structural age and former damages. Therefore the actual inelastic zones can be hardly verified without in-situ measurements.

The aging of the material itself can be modelled in the numerical simulations, but the actual conditions can only be verified by drilling cores or global measurements of the structure. With this information the numerical simulations could be calibrated.

### 6.2 Ground motion parameters

The proposed method to identify local soil amplifications can be used either to calibrate the duration of the earthquake event (resonance effects) or the peak ground acceleration. It is obvious that the probability of exceedance is increasing with lower duration and lower peak ground acceleration [Tantala, M. W., 2002]. Therefore it is very important to appreciate the possible duration and the peak ground acceleration as accurately as possible.

## 7. Summary and perspectives

The development of Probabilistic Seismic Hazard Analysis has involved many difficulties in the approach of the right uncertainties. The analysis of different parts of the methodology led to very accurate fragility curves which represent a certain probability of exceedance. Therefore it is obvious trying to reduce the uncertainties as far as possible.

One of the most important parameters is the aging of the building structure. Not only is the material behaviour changing with time, there are also various structural and non-structural changes which can be hardly recognized in a numerical model. With the proposed method it would be possible to give a snapshot of the current state of the structural building which covers all (visible and invisible) changes.

In addition to some structural parameters local soil conditions can vary a lot. To avoid major uncertainties for possible site effects, the above mentioned measurements can also be used to identify the amplification factor. The big advantage of the proposed method is that the measurements are economical and simultaneously to that they implicate very important information for an accurate PSHA.

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***Technical Session: Fragility***

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## A Study on Comparison of Seismic Capacities by Probabilistic Approach and Deterministic Approach

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***Abstract***

Seismic Performances of nuclear power plants are being performed by Seismic Probabilistic Safety Assessment (SPSA) and Seismic Margin Assessment (SMA). The major difference between these two methodologies is the approach, which of SPSA is based on the probabilistic approach, whereas which of SMA is based on the deterministic approach in evaluating capacities of structures and equipment. As for Korean nuclear power plants, SPSA has been used for new power plants but recently SMA has been taken for old power plants. This paper shows the quantitative correlation between the two methodologies by evaluating and comparing seismic capacities described as a HCLPF (High Confidence of Low Probability of Failure) on the same equipment using the two methodologies respectively. A model used for comparing and analyzing is a CCW surge Tank which is located on EL.165ft in Auxiliary Building. A HCLPF value of SMA is about 1.33 times ( $0.64g/0.48g = 1.33$ ) higher as that of SPSA and the value of 1.33 becomes a correlation factor of two methodologies in this study.

**1. Introduction**

Seismic Performances of nuclear power plants are being performed by Seismic Probabilistic Safety Assessment (SPSA) and Seismic Margin Assessment (SMA). The major difference between these two methodologies is the approach, which of SPSA is based on the probabilistic approach, whereas which of SMA is based on the deterministic approach in evaluating capacities of structures and equipment. As for Korean NPPs, SPSA has been used for new power plants but recently SMA has been taken for old power plants. This paper shows the quantitative correlation between the two methodologies by evaluating and comparing seismic capacities on the same equipment using the two methodologies respectively. The reason why equipment is chosen in this paper is that seismic capacity depends on equipment when earthquake occurs because the capacity of structure is much higher than that of equipment in nuclear Power Plants.

**2. Models for evaluating**

Models used for comparing and analyzing are as bellows. The equipment is placed on the 4<sup>th</sup> slab (EL.165ft) of the structure.

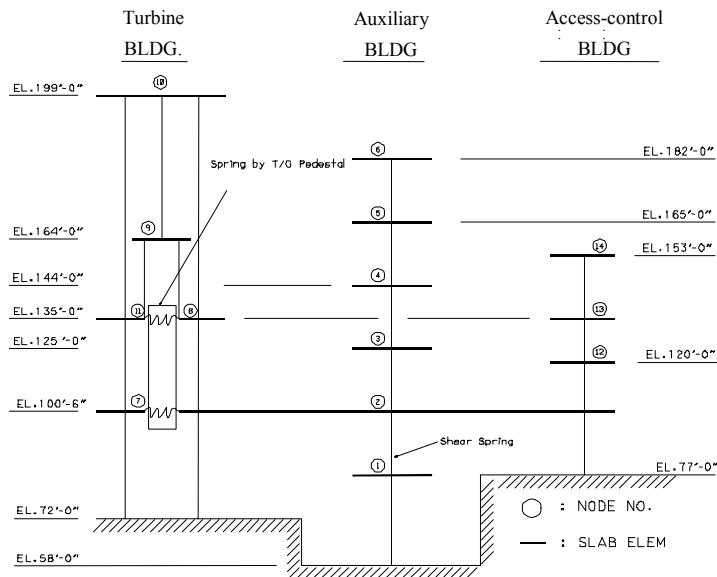
**2.1 Structure model**

The Auxiliary Building which belongs to seismic category I is a reinforced concrete structure with multi-story. The Auxiliary Building is composed of Primary and Secondary Auxiliary Buildings that share common slabs and shear walls. The methods used to analyze and design the shear walls of each building are the same. As planned, the Auxiliary Building is rectangular in shape, with the exception that the east side of the Primary Auxiliary Building wraps around the Containment Building with a 210 degree arc and is separated from the Containment Building by a 2-inch seismic isolation gap. The Auxiliary Building is approximately 218 feet wide, 324 feet long and 124 feet high.

The Auxiliary Building is modeled with the Turbine Building and Access Control Building which are seismic category II structures as one combined unit because they share common slabs at elevation 100'-6"

and below. The floor slab and shear wall/bracing structure of the Auxiliary-Turbine-Access Control Building Complex is modeled as a slab-shear spring system in which the structural mass is considered is concentrated at each main floor slab elevation. The slabs, in general, are assumed as infinitely rigid in their own planes and are interconnected by weightless linear elastic springs which simulate the stiffness of shear walls or steel braced frames in the building complex. The fundamental natural frequencies of the Auxiliary Building are 7.26Hz (EW), 7.66Hz (NS). Figure 1 is showing this model.

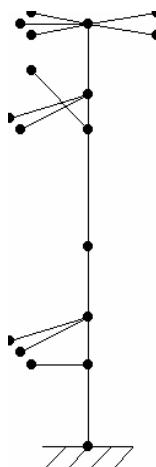
**Figure 1. Seismic model for structure**



## 2.2 Equipment model

The CCW surge Tank which is located on EL.165ft in the Auxiliary Building of Yong Gwang nuclear power plant is used for evaluating equipment. This equipment is a vertical tank with the length 29.5ft, the diameter 96ft, and the weight 93.7 kips. The fundamental frequency of this equipment is 14.5Hz by eigenvalue analysis. Figure 2 is showing this model.

**Figure 2. Seismic Model for Equipment**



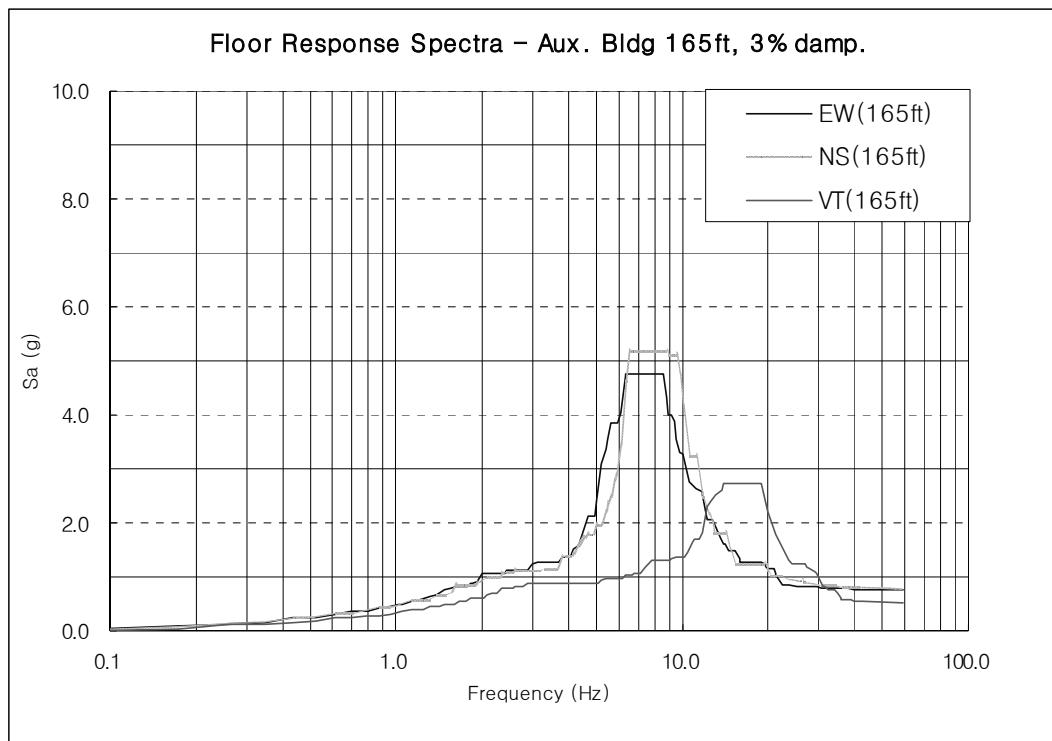
### 3. Evaluating method

To evaluate this equipment, first of all, failure mode has to be decided. Anchor bolt failure and concrete cone failure of anchorage are decided as governing failure mode based on long engineering experiences. In both approaches, the seismic loadings are only combined with normal operating loads (NOL) which would be expected to occur concurrently with earthquake and use load factors of unity for loadings and the only self weight of equipment is considered as normal operating loads. In analyzing seismic responses, CQC method is used for mode combination and 100-40-40 rule which is recommended by Newmark is used for three earthquake component combination.

#### 3.1 Seismic capacity by SPSA

First of all, the floor response spectra on which the equipment is located are gained and then seismic analysis of the equipment is performed using these floor response spectra and the equipment responses are calculated finally. The design damping ratios (7% with structure, 3% with equipment) and evaluation damping ratios (10% with structure, 5% with equipment) are used in this analysis and table 1 is showing the seismic demand of anchor bolt by these responses. Figure 3 is showing the Floor Response Spectra located at equipment.

Figure 3. FRS located at CCW Surge TK



#### 3.2 Seismic capacity by SMA

Seismic responses should be calculated by reanalysis using RLE (Review Level Earthquake) as input motion, but the scaling approach using design responses by DBE (Design Basis Earthquake) may be used to calculate RLE responses when the general shapes of DBE and RLE ground response spectra are similar and SSI (Soil Structure Interaction) effect is excluded. In this study the scaling factor is derived by the following.

$$S_F = \frac{S_{a10\%RLE\_bldg\_freq}}{S_{a7\%DBE\_bldg\_freq}} \cdot \frac{S_{a5\%\_equip\_freq}}{S_{a3\%\_equip\_freq}}$$

where:  
 $S_{a10\%RLE\_bldg\_freq}$ : Spectral Acceleration at building frequency by RLE  
 $S_{a7\%DBE\_bldg\_freq}$ : Spectral Acceleration at building frequency by DBE  
 $S_{a5\%\_equip\_freq}$ : Spectral Acceleration at equipment frequency by FRS  
 $S_{a3\%\_equip\_freq}$ : Spectral Acceleration at equipment frequency by FRS

$$S_F := \frac{0.492g}{0.465g} \cdot \frac{1.50g}{1.60g}$$

$$S_F = 0.99$$

Figure 4 and Table 1 is showing the DBE vs RLE ground response spectra and the seismic demands of anchor bolt by these responses.

Figure 4. DBE vs RLE

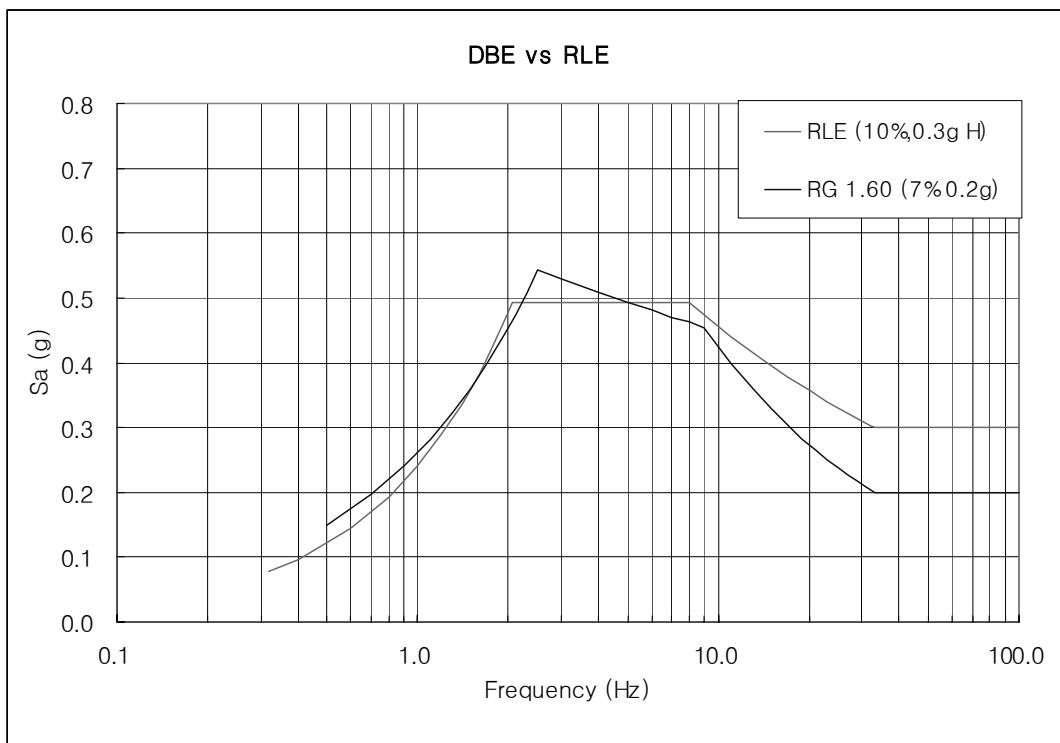


Table 1. Seismic demands by two methodologies

	SPSA	SMA	Stress
FtNOL	-1852 psi	-1852 psi	Max.tensile stress for normal operating load
FtEQ	21656 psi	21439 psi	Max.tensile stress for earthquake load
FsNOL	0 kips	0 kips	Max.shear stress for normal operating load
FsEQ	2153 psi	2131 psi	Max.shear stress for earthquake load

#### 4. Results of evaluation

The detailed process of calculation for each methodology is omitted and the results are included in this paper in Table 2.

Table 2. Seismic capacities of equipment by two methodologies

<b>Methodology</b>		<b>SPSA</b>			<b>SMA</b>
Strength Factor		$F_S$	$\beta_R$	$\beta_U$	$F_S$
Failure Mode	Shear Tension Interaction Failure	5.74	—	—	3.76
	Concrete Cone Failure (govern)	3.80	0	0.34	2.14
Inelastic Energy Absorption Factor		1	0	0	1
	<b><math>F_{EC}</math></b>	3.8	0	0.34	2.14
Equipment Response Factor		$F_S$	$\beta_R$	$\beta_U$	$F_S$
Qualification Method Factor		1	0	0	
Spectral Shape Factor		1.48	0	0.01	
Damping Factor		1.17	0	0.12	
Modeling Factor		1.02	0	0.13	
Mode Combination Factor		1	0.1	0	
Earthquake Component Combination Factor		1	0.05	0	
	<b><math>F_{RE}</math></b>	1.76	0.11	0.17	
Structural Factor		$F_S$	$\beta_R$	$\beta_U$	$F_S$
Spectral Shape Factor		1.03	0.19	0.06	
Damping Factor		1.15	0.04	0.14	
Modeling Factor		1	0	0.19	
Modal Combination Factor		1	0.05	0	
Earthquake Component Combination Factor		1	0.05	0	
Horizontal Earthquake Direction Factor		1	0.15	0	
Soil Structure Interaction Factor		1	0	0	
Ground Motion Incoherence Factor		1.02	0.03	0.04	
Structural Inelastic Response Factor		1	0	0	
	<b><math>F_{RS}</math></b>	1.22	0.26	0.25	
PGA		0.2g (SEE)			0.3g (RLE)
	<b>HCLPF</b>	<b>0.48</b>			<b>0.64</b>

#### 4.1 Seismic demands

As shown in table 1, seismic demands by SMA are somewhat smaller than those by SPSA because RLE using evaluation damping is larger than DBE using design damping ( $0.492g/0.465g = 1.058$ ), whereas FRS using evaluation damping is smaller than that using design damping ( $1.50g/1.60g = 0.937$ ).

#### 4.2 Equipment capacity

Equipment capacity factor is made up of strength factor and inelastic energy absorption factor, and inelastic energy absorption factor is unity because the concrete cone failure is brittle. So equipment capacity depends on strength factor, which is ratio of strength capacity and seismic demands. Seismic demands by SPSA are smaller than those by SMA as described above, strength capacity by SPSA is larger than that by SMA because strength capacity by SPSA is based on median value (50%) whereas strength capacity by SMA is based on code-specified (95%). That's why the value of SPSA is about two times higher as that of SMA as shown in Table 2.

### 4.3 HCLPF

In evaluating seismic capacities by SPSA, a HCLPF value is functions of median ground acceleration capacity and log normal deviation consist of randomness and uncertainties. This HCLPF value means 95% confidence of 5% probability of exceedance. On the other hand in evaluating capacities by SMA, a HCLPF value means about 84% confidence of 16% probability of exceedance. So in this equipment, a HCLPF value of SMA is about 1.33 times ( $0.64g/0.48g = 1.33$ ) higher as that of SPSA as shown in Table 2 because of difference of confidence level. The value of 1.33 becomes a correlation factor of two methodologies in this equipment.

## 5. Conclusion

The relationship between the two HCLPFs is defined as a variable of horizontal component response spectrum shape, so 1.22 to 1.35 of factors can be derived [4]. That is, the Square Root of Square Sum (SRSS) combinations of these randomness and uncertainty ( $\beta_{rs}$ ) are determined by 0.2 and 0.3 for the west of the U.S. and the east of the U.S. respectively. Therefore 1.22 (i.e.,  $e^{0.2}$ ) and 1.35 (i.e.,  $e^{0.3}$ ) of factors are introduced. 1.33 of the factors achieved by this study is existed in between them.

In this paper evaluative methods for each methodology are studied and HCLPF values are compared and analyzed in order to evaluate correlation of two methodologies. This study will be helpful to recognize exactly the concept of seismic capacities which is inherent in each methodology and to induce the quantitative correlation of the two methodologies in evaluating seismic capacities of individual components.

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# Usage of Information Entropy in Updating Seismic Fragilities

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## 1. Introduction

Following the revision of the guidelines for seismic design evaluation of power reactor facilities in September, 2006, The Nuclear and Industrial Safety Agency (NISA) has directed nuclear energy companies to evaluate “residual risk” and report evaluation results to NISA in order to contribute to the examination of the possible introduction of probabilistic safety assessments in safety regulations. [1]

Whole the spectrum of seismic-induced scenario is taken into consideration in the current seismic probabilistic safety assessment (SPSA) of a nuclear power plant (NPP). Therefore, seismic fragilities should be prepared for every failure mode of all the safety related equipment and components considered in the SPSA of the NPP in order to quantify the residual seismic risk. Generally, a large number of systems and equipment are involved in the SPSA model. A problem is that our knowledge and data necessary for the fragility evaluation are not always sufficient and it is a difficult task to prepare the complete seismic fragility dataset specific to the NPP under consideration.

Preliminary evaluation is performed using generic seismic fragilities first of all and risk-dominant sequence and related systems and equipment are identified. According to the preliminary results, seismic responses and fragilities are refined with regard to the components in the dominant sequences if the seismic behaviour is not well-understood and the uncertainty is large. To update the seismic fragilities, additional seismic qualification test and/or detailed analysis with the best-estimate methodology are performed. The additional test and/or analysis should be designed so that the results are useful and cost-effective for the fragility quantification and updating process. Therefore, we need a quantitative measure to evaluate the importance of the test and/or analysis.

The present study is directed to updating process of the seismic fragilities. The information entropy theory has been applied to the seismic fragility evaluation. The theory is well-known in the statistic science. The entropy and its expected value are appropriate measures of the importance of additional information such as a seismic test or analysis. The information entropy tells us how the current fragility estimate is updated in the most efficient way. In other words, additional test or analysis for the seismic fragility update should be designed so that the entropy becomes the maximum. The procedure suggests a reasonable guideline for the fragility updating.

In section 2, the seismic fragility model is described and features of the uncertainty in the fragility are discussed. The information entropy is explained in section 3 as well as the relationship with the probability density function of the seismic capacity. Numerical example is given in section 4 to show the effectiveness of the information entropy. A Bayesian method that is a reasonable way for the fragility update [2,3] is used in the fragility updating process.

## 2. Fragility function

The seismic fragility is defined as a failure probability on condition that an earthquake occurs. A double logarithmic normal distribution shown in Figure 1 is commonly used to describe the seismic fragility

curves. With the peak ground acceleration (PGA)  $\alpha$  as the intensity parameter of an earthquake, the fragility function  $F(\alpha)$  is expressed as:

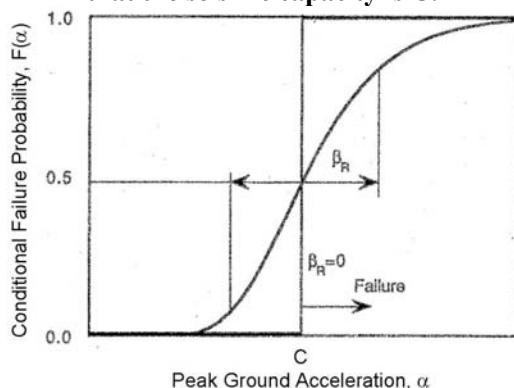
$$F(\alpha) = \Phi\left[\frac{\ln(\alpha/C)}{\beta_R}\right], \quad (1)$$

where  $\beta_R$  and  $\Phi(\cdot)$  are variability of the fragility associated with random uncertainty and the cumulative standard normal distribution function, respectively.  $C$  is the seismic capacity of equipment corresponding to a confidence level. From Equation (1), it is seen that the seismic capacity is the PGA level at which the seismic fragility or seismic-induced failure probability equals 0.5. The relationship of the PGA and the component failure probability given by Equation (1) is schematically shown in Figure 1.

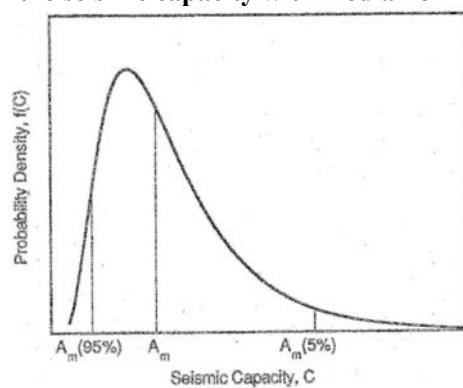
The probability density function (PDF) of the seismic capacity is also lognormal:

$$f(C) = \frac{1}{\sqrt{2\pi}\beta_U C} \exp\left[-\frac{1}{2}\left(\frac{\ln(C/A_m)}{\beta_U}\right)^2\right]. \quad (2)$$

**Figure 1. Failure probability on condition that the seismic capacity is  $C$ .**



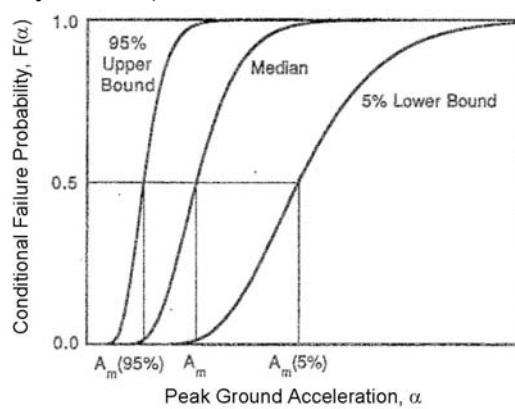
**Figure 2. Probability density function of the seismic capacity with median of  $A_m$ .**



Here,  $A_m$  and  $\beta_U$  are the median value and logarithmic standard deviation of the seismic capacity, respectively. The variability is attributed to the inadequateness of our knowledge concerning the seismic capacity. The PDF of the seismic capacity is shown in Figure 2.

Comparing Equation (1) and Equation (2), we can interpret that  $\beta_R$  is the variability of the normalized seismic input level by the seismic capacity, i.e.  $\alpha / C$ . On the other hand,  $\beta_U$  is the variability of the seismic capacity normalized by the median capacity, i.e.  $C / A_m$ . If we use various confidence levels of the seismic capacity, we can draw many fragility curves. Figure 3 shows the median (50% confidence), 5% confidence, and 95% confidence fragility curves as a function of the PGA.

**Figure 3. Seismic fragility curves (median, 5% and 95% bounding) with median of  $A_m$**



### 3. Information entropy

One can update the equipment fragility by performing shaking table tests or structural response analyses, for example. It is noted that the shaking table test is costly. The detailed analysis using high performance computer is also expensive as well. Thus we have to decide the most cost-effective test or analytical conditions based on the engineering judgment.

Let us consider a situation where a shaking table test is performed at acceleration level A. If an analyst believes the seismic capacity of the component is C, and the random uncertainty is  $\beta_R$ , the failure probability would be given by Equation (1). Hence the probability that the shaking test results in failure is evaluated by:

$$F(A|C) = \Phi\left[\frac{\ln(A/C)}{\beta_R}\right]. \quad (3)$$

We see that Equation (1) is interpreted as a conditional probability given that the seismic capacity is C.

It is interesting to know the importance or worth of new information from the viewpoint of the fragility updating. Here let us consider an identifier k of the new information. The importance of the information k can be evaluated by:

$$V_k = -\ln L_k, \quad (4)$$

where  $V_k$  is the importance and  $L_k$  is the likelihood. The likelihood  $L_k$  is the probability that a piece of evidence k is observed. Hence, in this example,  $L_k = F(A|C)$  if the shaking table test results in a component failure and  $L_k = 1 - F(A|C)$  if the component does not fail. Hereafter, we name  $V_k$  the logarithmic likelihood.

The logarithmic likelihood varies from 0 to infinity. If the component is very rigid and seismic-resistant, the likelihood of component failure is nearly zero and the logarithmic likelihood is infinite. It implies that the failure of very resistant component is a rare event. Viewing it from the other side, we understand that an occurrence of a rare event is very valuable and the importance of the information is significant.

On the other hand, the logarithmic likelihood is zero if the failure probability or the likelihood is unity. We take it granted for that a very fragile component should fail in the shaking table test and no noteworthy information is obtained from the test. Therefore, the logarithmic likelihood is zero which implies the test result is almost meaningless or the information value is negligibly small. It can be said that the logarithmic likelihood is a useful measure to judge the test result is notable or not after test after the test.

Since we do not know the test results in advance, the expected value of the logarithmic likelihood with respect to all the possibilities is a point of concern. It is defined as the information entropy, E. We assume the test result is either fail or success. The information entropy is defined as:

$$E = -F(A|C)\ln F(A|C) - \{1 - F(A|C)\}\ln\{1 - F(A|C)\}. \quad (5)$$

When N components are tested at the same acceleration level, there are  $N + 1$  possibilities. Thus the general expression of the entropy is:

$$E = -\sum_{k=0}^N L_k \ln L_k, \quad (6)$$

where  $L_k$  is the likelihood that k components out of N fail and is calculated by the binomial distribution:

$$L_k = \binom{N}{k} F(A|C)^k \{1 - F(A|C)\}^{N-k}. \quad (7)$$

The entropy (expected value in terms of all the possibilities) is an appropriate measure to judge the test program is useful or not for fragility analysts before the test is performed.

In the discussion mentioned above, the equipment failure probability is given by Equation (3) on condition that the seismic capacity is  $C$ . As can be seen from Equation (2), the seismic capacity is an uncertain variable with median capacity  $A_m$  and logarithmic standard deviation  $\beta_U$ . Therefore, the information entropy depends on our prior understanding on the seismic capacity.

It is reasonable that the expected information entropy with respect to the seismic capacity should be utilized to evaluate the importance of the additional information such as a test and analysis. The expected values of the logarithmic likelihood and the information entropy are given by:

$$\bar{V}_k = \int_0^\infty V_k f(C) dC = - \int_0^\infty f(C) \ell n L_k dC, \text{ and} \quad (8)$$

$$\bar{E} = \int_0^\infty E f(C) dC = - \int_0^\infty f(C) \sum_{k=0}^N L_k \ell n L_k dC, \quad (9)$$

respectively.

The expected entropy is interpreted as the importance of a test at acceleration level  $A$  before the test is performed for the analyst who believes the seismic capacity PDF is  $f(C)$ . On the other hand, the expected logarithmic likelihood is the importance of the test result that  $k$  components failed at acceleration level  $A$  for the analyst who believes the seismic capacity PDF is  $f(C)$ .

#### 4. Numerical example

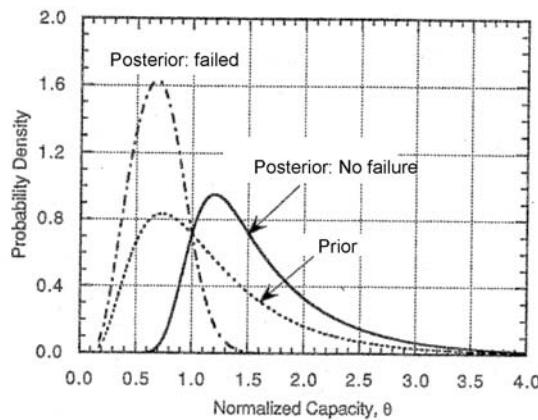
Based on the discussion in section 3, the importance of the additional information on the seismic fragility update is quantified by the expected values of the logarithmic likelihood and the information entropy.

Figure 4 shows the results of the Bayesian update of the PDF of a seismic capacity. The likelihood function is binomial as in Equation (7). The prior PDF is given by Equation (2). The prior fragility parameters are assumed to be:

$$A_m = 2.0, \beta_R = 0.3, \beta_U = 0.4 \quad (10)$$

The seismic capacity level is normalized with respect to the median capacity, i.e.  $\theta = C/A_m$  in Figure 4. Let us consider a shaking table test performed at the median capacity level ( $\theta = 1$ ). Two cases are considered in Figure 4; the chained line in Figure 4 shows the case in which the component failed; the solid line corresponds to the successful test with no failure. It is seen that the PDF of the seismic capacity is substantially changed according to the information obtained from a single component test at the median capacity PGA level.

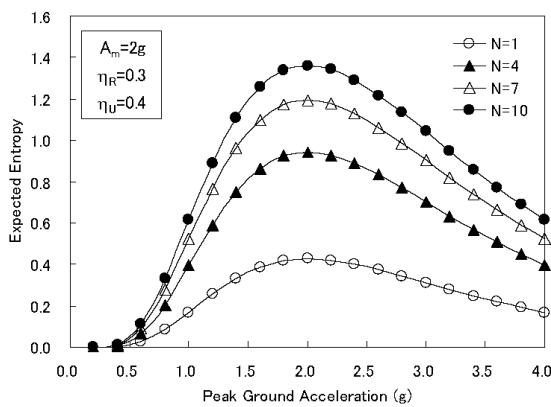
Figure 4. Bayesian update: prior and posterior PDFs of the seismic capacity



Here a question arises about the fragility qualification by a shaking table test. What acceleration level should be selected in the shaking table test? How many components should be tested? It is emphasized that the information entropy gives the answer to the questions.

Figure 5 shows the expected information entropy for the shaking table test of a component with the fragility parameters given by Equation (9). The horizontal axis indicates the acceleration level at which the test is performed. Four curves drawn in Figure 5 indicate different number of components tested, that is  $N=1, 4, 7$  and  $10$ . It is seen that the expected entropy reaches maximum at the median acceleration level at  $2g$  regardless to the number of components. It is consistent with our intuition. If the test acceleration level is low, we expect most of the tested components survive. On the other hand, many will fail if the acceleration level is too high.

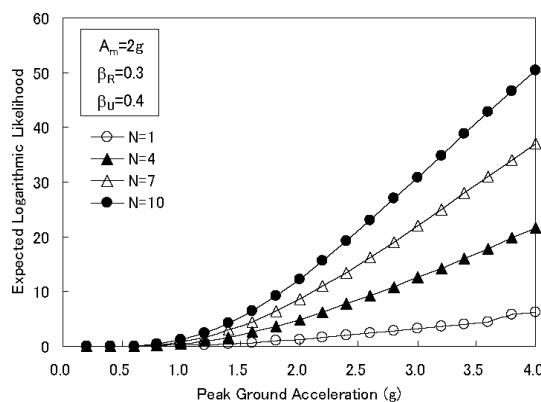
**Figure 5. The expected entropy of a shaking table test as a function of the test acceleration level and number of tested components**



We see another important tendency from Figure 5. As the number of tested components increases, the increment of the expected entropy is reduced. We can suggest that limited number (one or two) of components is tested at the median capacity level. Afterward, another acceleration level should be selected according to the updated PDF of the median capacity.

Figures 6 and 7 show the expected logarithmic likelihood for the components with the same fragility parameters as the case in Figure 4. Four curves correspond to the number of tested components, that is  $N=1, 4, 7$  and  $10$ .

**Figure 6. Expected likelihood of all the components survival in a shaking table test as a function of the test acceleration level and number of components**



**Figure 7. Expected likelihood of all the components failure in a shaking table test as a function of the test acceleration level and number of components**

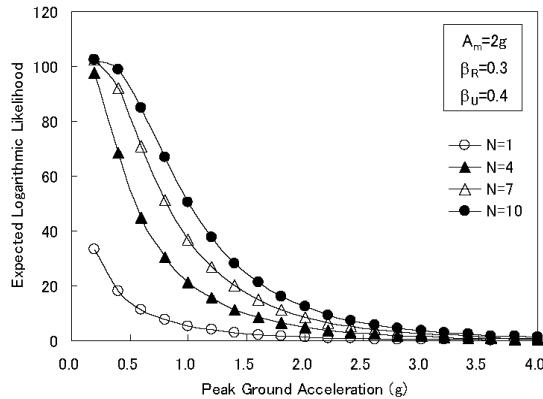


Figure 6 shows the results when all the components survive in the test. The expected logarithmic likelihood for successful test results is monotonically increasing. The results come up to our expectation. It is easily understood that the observation is very significant if the components survive at higher acceleration level than the median capacity. At the acceleration level lower than 1.0g (half of the median capacity), the expected logarithmic likelihood is very small. It is a matter of course that the seismic qualification test at the acceleration level lower than the median capacity is almost meaningless.

The expected logarithmic likelihood when all the components failed in the test is shown in Figure 7. This figure shows opposite tendency to the all component survival case. Figures 7 and 6 are extreme cases that all components fail or all components survive, respectively. Figure 5 is the intermediate between the two extreme cases.

At present, we can evaluate the importance of additional information for updating the seismic fragility. Here we verify how the information entropy works and seismic fragility is updated using two different shaking table tests results and the Bayesian method. As to the Bayesian method, the details are given in Ref [3].

Let us consider a shaking table again at the median capacity level. The component tested has the same seismic fragility parameters as in Equation (10). If one component is tested and it survives, the expected logarithmic likelihood (importance of the observation) is calculated as 1.22 (see Figure 6; N=1; PGA=2g). Now we consider the component test at lower acceleration than the median capacity level. To achieve the same value of the expected logarithmic likelihood, we should use four components at 1.282g and all of them survive. The former test (one component test at 2g acceleration level) is named Case 1 and the latter one (four components test at 1.282g) is Case 2 in the following.

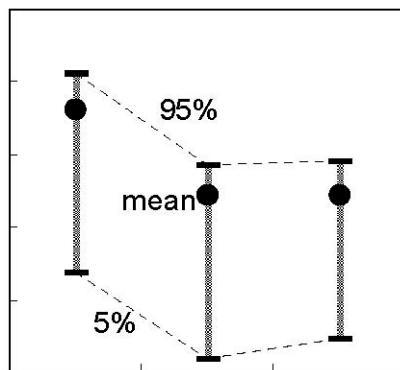
Table 1 shows the Bayesian update results using the test information. From the evidence that the components do not fail in the test, the median capacity increases and the uncertainty is reduced. The median capacity increment is larger in Case 1 and we are confident that the capacity is higher than we considered in advance of the test. On the other hand, the reduction of the uncertainty is larger in Case 2 (from 0.4 to 0.255) than in Case 1 (from 0.4 to 0.286). We are more confident that the failure probability at the test level (1.28g) should be smaller than we expected in the prior fragility because no failure occurs in the test. In other words, we have obtained enough evidence to decrease the failure probability at low acceleration level. It is the reason that more reduction of the uncertainty is observed in Case 2 than in Case 1.

**Table 1. Prior and posterior fragility parameters**

	Prior	Posterior	
		Case 1 (One test at 2g)	Case 2 (Four tests at 1.28g)
$A_m$ (g)	2.0	2.548	2.424
$\beta_R$	0.3	0.3	0.3
$\beta_U$	0.4	0.286	0.255

Now let us investigate how the two cases influence on the annual frequency of the component failure. Although the frequency depends on the characteristics of the hazard curve, we use typical seismic hazard curves for NPPs and the results seem to be general. The posterior fragility curves are convoluted with the hazard curves to obtain the annual frequency of failure. The results

**Figure 8. Annual Frequency of failure evaluated for prior fragility and posterior fragilities (Cases 1 and 2)**



## 5. Conclusions

In the present study, information entropy is applied to the seismic fragility evaluation and updating process. Expected information entropy is a useful measure for estimating the importance or usefulness of a seismic qualification test or analysis (before the results are obtained). On the other hand, the expected logarithmic likelihood is an appropriate measure and can be used to estimate the importance of evidence when we know the results.

Additional seismic qualification test and/or additional detailed analysis with the best-estimate methodology are performed to update the seismic fragilities. According to the information entropy and logarithmic likelihood, one can predict how the current fragility estimate is updated in the most efficient way. In other words, the seismic fragility is to be updated so that the entropy becomes the maximum. The procedure suggests a reasonable guideline for the fragility update by performing additional test and analysis. If we have many observations (test results and analysis results) concerning the seismic fragility, we just need to calculate the expected logarithmic likelihood. If it is large enough, the observation should be used in the fragility update process.

It is concluded that the information entropy is effective and useful measure in seismic fragility update. Bayesian method can be used for updating the seismic fragility using selected information by the expected entropy and logarithmic likelihood.

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## Development of Probabilistic Safety Assessment Considering Slope Collapse by Earthquake

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### ***Abstract***

In Japan, a part of nuclear power plants is surrounded by the land slope. In such a nuclear power plant, it is necessary to evaluate the stability of the slope under the deterministic seismic condition, and to ascertain that the plant is kept in a safe condition even if collapse of the slope might occur due to earthquakes.

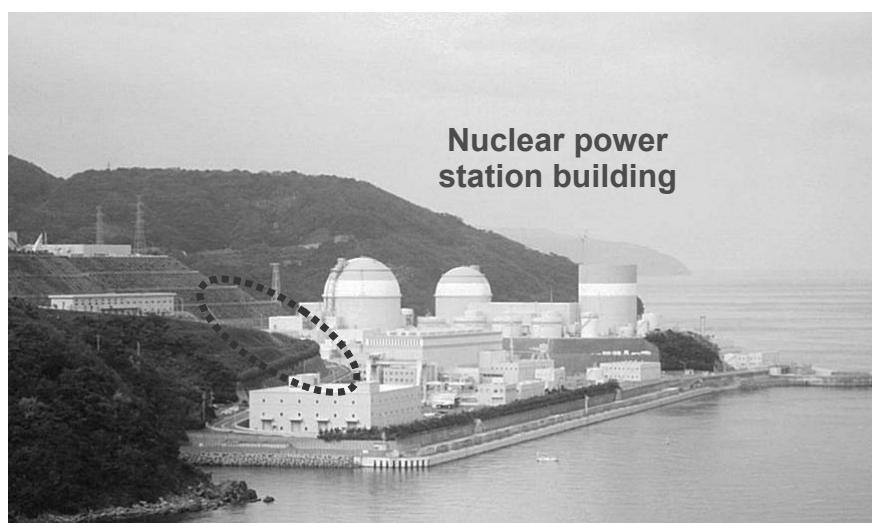
A probabilistic safety assessment methodology considering the slope collapse (Slope PSA) has been developed as a part of seismic PSA at Japan Nuclear Energy Safety Organization (JNES). This method consists of slope collapse hazard evaluation, fragility evaluation (evaluation of secondary influences on nuclear facilities) and system reliability analysis. In the slope collapse hazard evaluation, probabilities of slope stability, failure modes, collapse behavior, rock reach area and shock force to nuclear facility are analyzed and calculated.

This report describes the slope collapse hazard evaluation procedure and sample calculations for a model slope, and also results of parametric studies for the rock block behavior analysis.

### **1. Introduction**

Nuclear power plants in Japan are often located near coastlines because of their need to acquire large amounts of cooling water. As shown in Photo 1, some of them stand close to the land slopes. The worry is that such nuclear power plants would suffer secondary damage, due to the collapse of surrounding slopes and the impact of rock blocks on the buildings following ground motion in an earthquake.

**Photo 1. Nuclear power plant located close to slope topography**



The revised version of the Examination Guideline for Seismic Design of Nuclear Power Reactor Facilities (herein the Revised Guideline), issued by the Nuclear Safety Commission, regards the collapse of the surrounding slope due to earthquake ground motion to be an earthquake-associated phenomenon and requires nuclear power plants to ensure that the safety capability of their facilities will sustain no serious impact, even if the slopes collapse. The Revised Guideline, based on the notion that the probability of an earthquake whose earthquake ground motion exceeds that of the design standard cannot be denied, suggests the existence of “residual risk” of an earthquake and requests efforts to rationally minimize such risk.

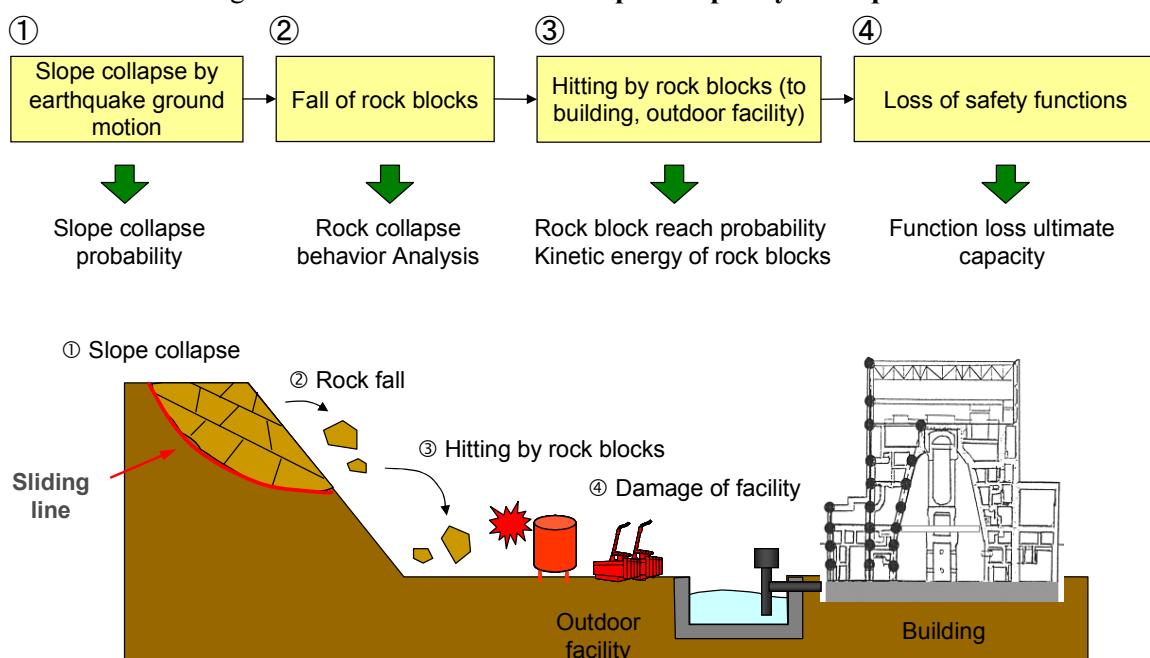
To understand the level of residual risk by earthquake for a nuclear power plant, all accident scenarios must be considered. It is important for sites where slopes exist near nuclear power plants to consider scenarios of accidents involving slope collapse.

The Japan Nuclear Energy Safety Organization (JNES) is developing a seismic PSA method that takes slope collapse into account as a technique to evaluate the earthquake residual risk.

## 2. Outline of the seismic PSA methodology considering slope collapse

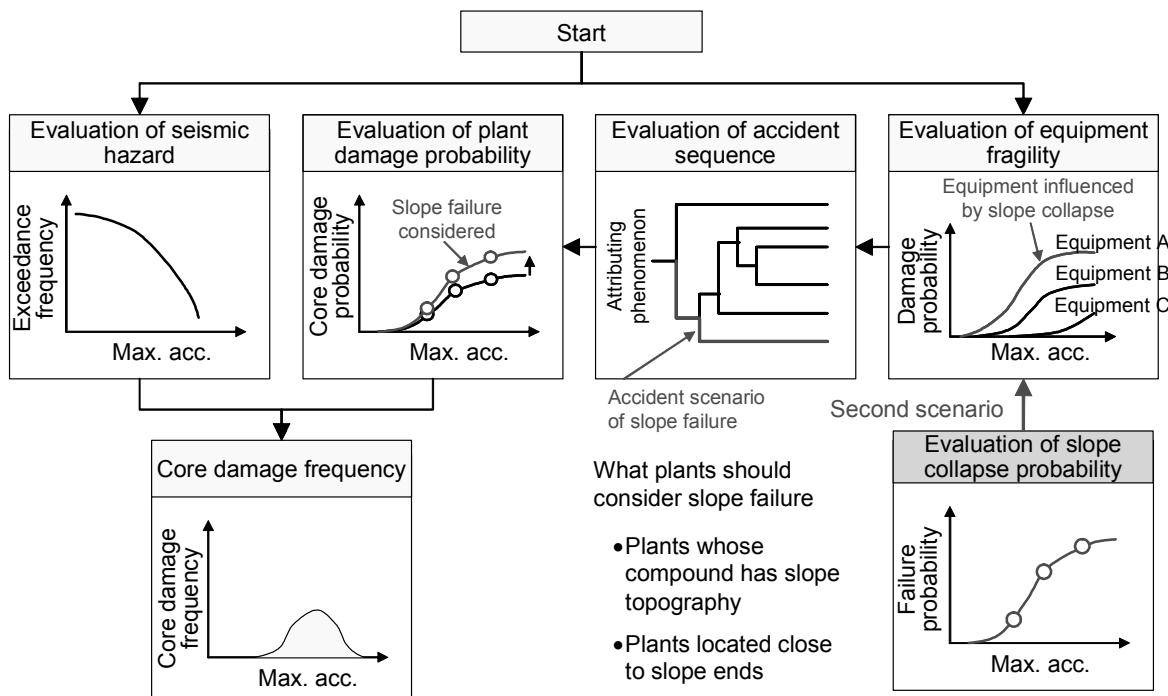
Figure 1 shows a schematic diagram of a slope collapse accident scenario for a nuclear power plant, following the occurrence of an earthquake. The scenario envisages a slope having collapsed due to earthquake ground motion triggering the fall of rock blocks on the slope, which then roll down and hit the nuclear power plant building and important outdoor equipment. The subsequent damage to equipment, both inside and outside the building, renders the safety capability non-operational. Specific cases of equipment damage include the secondary impacts of slope collapse, such as loss of external power sources due to damage sustained by the external substation or loss of the capability to take in cooling water, as rock blocks bury the water intake pit. For a seismic PSA that considers slope collapse, it is important to establish techniques to evaluate the probability of slope collapse due to earthquake ground motion, rock behaviour, the probability of rock reach, rock kinetic energy, and the safety capability loss limit strength of each structure against the impact of rock blocks with respect to the said accident scenario.

Figure 1. Accident scenario of slope collapse by earthquake



The schematic flow of a seismic PSA considering slope collapse is shown in Figure 2. The conventional seismic PSA consists of three factors, namely seismic hazard evaluation, equipment fragility evaluation, and accident sequence evaluation, and calculates the core damage frequency based on the earthquake ground motion. However, when the secondary impact by slope collapse is considered, it is necessary to evaluate the slope collapse probability, depending on the strength of the earthquake ground motion, and also evaluate the fragility of equipment hit by the rolling rock blocks. The occurrence probability of an initiating event, triggered by slope collapse due to earthquake ground motion of a certain magnitude is calculated as “(slope collapse probability)  $\times$  (rock arrival probability)  $\times$  (building damage probability by rock hit).”

**Figure 2. Schematic flow of seismic PSA considering slope collapse**



### 3. Slope collapse probability assessment by earthquake ground motion

Methods to evaluate the probability of collapse of the surrounding slope by earthquake ground motion were developed, based on conventional slope stability assessment methods, such as the sliding surface method and dynamic FEM which were used as part of seismic design. The slope collapse probability evaluation method was then applied to a model slope to calculate the collapse probability.

The following discussion is focused on the slope collapse probability evaluation method using the dynamic FEM technique.

#### (1) Evaluation method

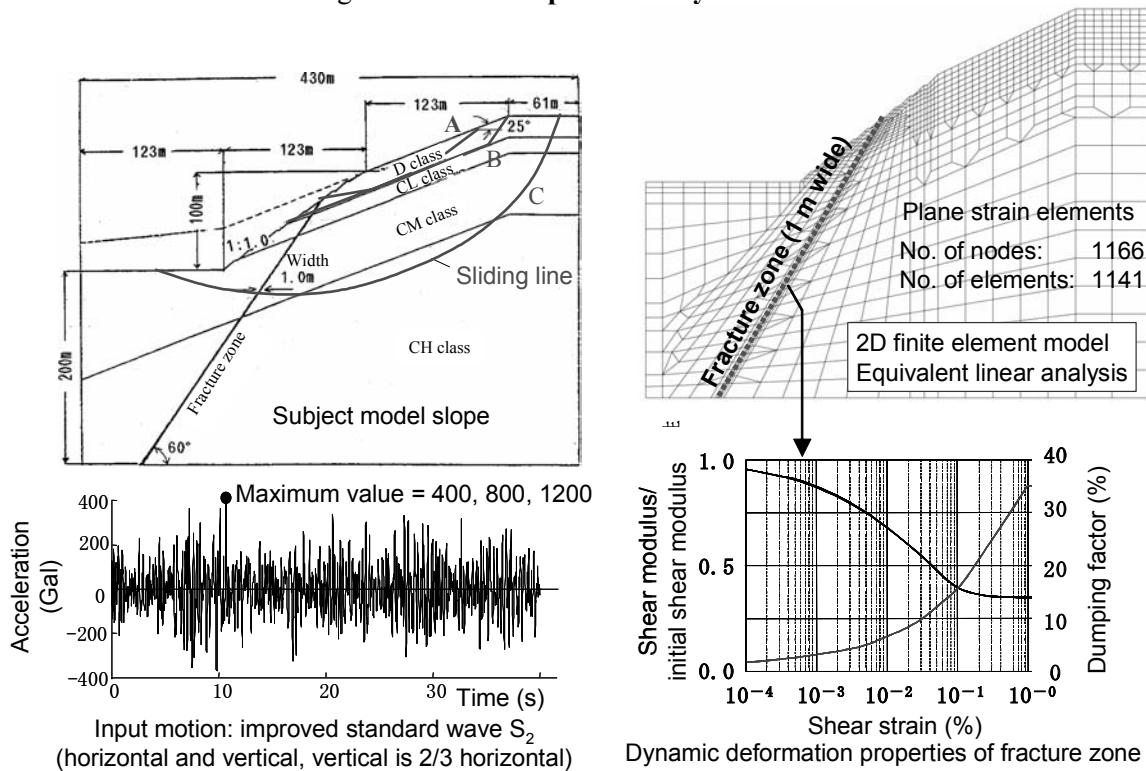
The slope stability evaluation method, using the dynamic FEM technique, calculates the total sum of the response shear force and shear resistance force, when an earthquake occurred, of the elements on an assumed sliding surface every time. Then the stability is evaluated based on the ratio (sliding safety factor = shear resistance force / response shear force). The same concept applies to the slope collapse probability evaluation, which means collapse is judged when the sliding safety factor is below 1.0. Since material properties of the ground constituting a slope are intrinsically dispersive, the probability distribution model is assumed to express physical properties of the ground. Then, stability is evaluated for multiple analytical

models with different ground properties which are sampled using Latin Hypercube Sampling (LHS) method. Collapse probability is finally evaluated based on the ratios that cause the sliding safety factor to descend below 1.0.

### (2) Evaluation conditions

Figure 3 shows the model slope to be evaluated and the analytical conditions. The model slope is a hard rock slope, partly containing a crush zone, and the sliding surface is assumed as shown in the figure. The dynamic deformation properties are modeled for Class D rock mass and the crush zone, and other strata are regarded as elastic. With this in mind, seismic response analysis is conducted using the equivalent linear method. Of the ground properties, focus is put on the shear modulus and cohesive force, and dispersion is considered. Note that the same value is used for ground properties in each stratum.

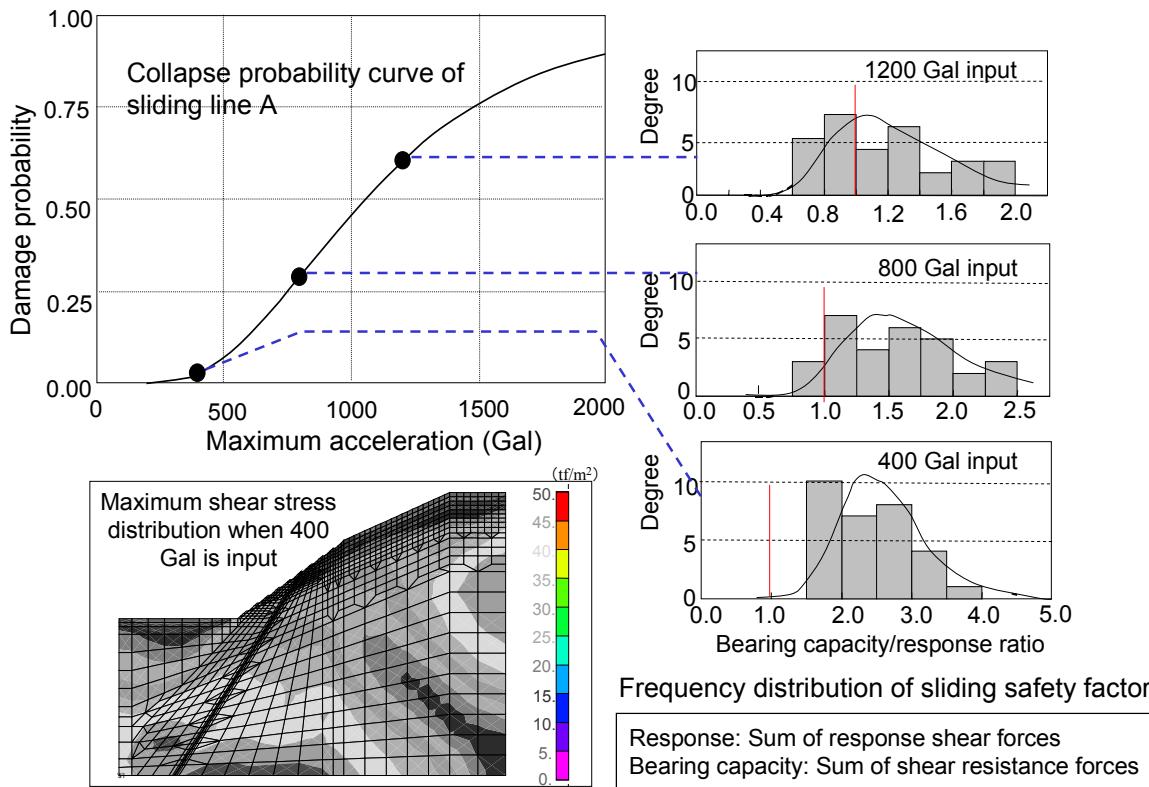
Figure 3. Model slope and analysis conditions



The input earthquake ground motion used is the improved standard wave in a highly earthquake-prone zone, following which analysis is made with the maximum acceleration in the horizontal direction being 400, 800 and 1200 gal respectively and that in the vertical direction being 2/3 of the figure in the horizontal direction.

### (3) Evaluation result

Figure 4 shows some of the analysis results. Of the assumed three sliding surfaces, Surface A shows the highest probability of collapse, as shown in this figure. It is indicated that one can quantitatively show the slope collapse probability relative to the size of the input earthquake ground motion and that the slope collapse probability increases with the rise in the input earthquake ground motion.

**Figure 4. Result of slope failure probability analysis (sliding line A)**

#### 4. Rock collapse behaviour analysis after slope collapse

To evaluate the secondary impact of rock blocks on a nuclear power plant building and equipment, following the collapse of a surrounding slope by earthquake ground motion, an distinct element method analytical code UDEC was introduced as a technique to calculate the behavior and reach distance of the collapsed rock blocks and the impact force of the rocks on the facility. The distinct element method is widely used in behavior analysis of discontinuum, such as falling rock, and is suited for analysis of rock slope collapse, where many discontinuous surfaces, such as joints or cracks, are found.

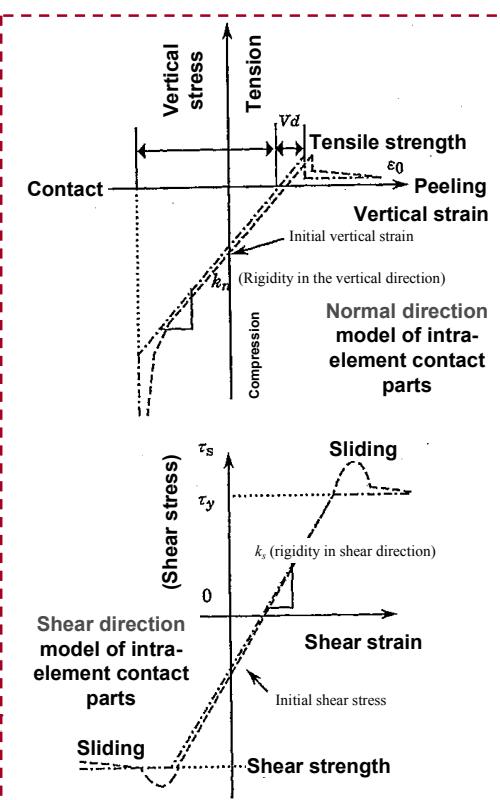
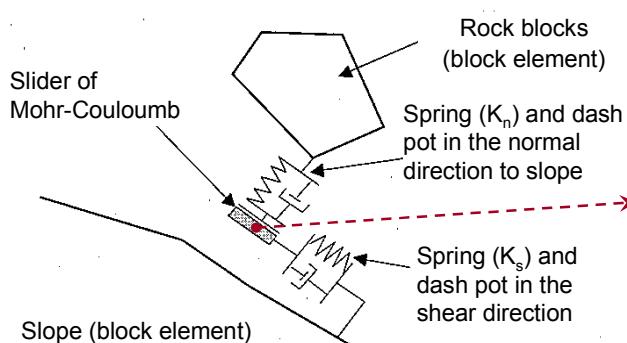
Now the outline of the analysis method of rock collapse behavior using the distinct element method is presented, followed by an explanation of the results of the simulation analysis of the collapse case conducted to verify the validity of the UDEC code and an example of analysis of the rock collapse behavior for a model slope.

##### **(1) Analysis procedure of distinct element method**

Analysis of rock collapse behavior using the distinct element method is conducted in the following procedure. Firstly, whether elements are connected, separated or sliding should be judged by comparing the Mohr-Coulomb fracture criterion and the total of dead load stress and seismic load stress. Then, if they are judged to be separated, the elements become separated on the contact surfaces and collapse. If judged to be sliding, they slide as if in contact with each other. If judged to be connected, they will not cause any collapse or sliding. Those non-linear characteristics on the contact surfaces of elements should be modeled by the spring element, the occurrence of which is caused in the normal direction and the shear direction that connects elements. Figure 5 shows a schematic diagram of the distinct element method.

Figure 5. Schematic diagram of distinct element method

- Contact mechanism that considers discontinuity of elements and uses spring and dash pot in the normal direction and shear direction of elements for modeling.



## (2) Simulation analysis for actual rock collapse event

To verify the validity of the distinct element method, simulation analysis was conducted to reproduce collapse cases actually previously observed.

The subject collapse case used here is the collapse of rock mass (natural collapse) observed in the western part of the Amadoribashi-nishi area in Wakayama Prefecture. Measuring devices were installed at this spot as well as many rock slopes in various parts of Japan. Those devices are used to carry out fixed-point observation tests and measure behavior immediately before and after a collapse event. The western part of the Amadoribashi-nishi area is one that has successfully provided detailed data. The impression illustration of the rock collapse, drawn based on video that actually recorded the collapse, is shown in Figure 6. At this point, two collapse events were observed at an interval of two weeks. The first involves the collapse of the lower part, while the second one is the collapse of the upper, with both involving cases of toppling collapse.

Figure 6. Impression illustrations of the rock collapse at Amadoribashi-nishi area

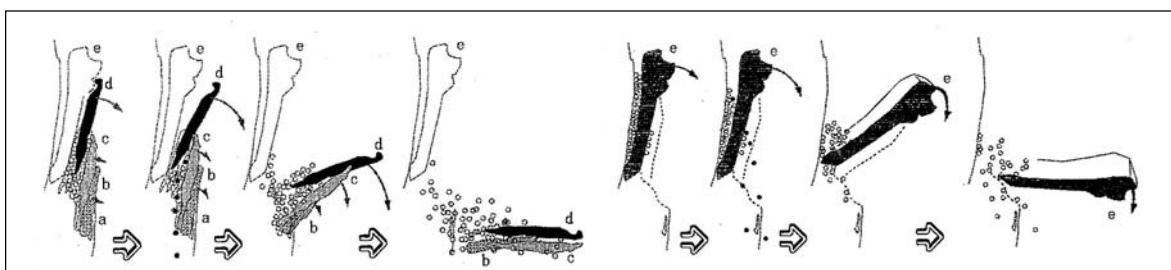
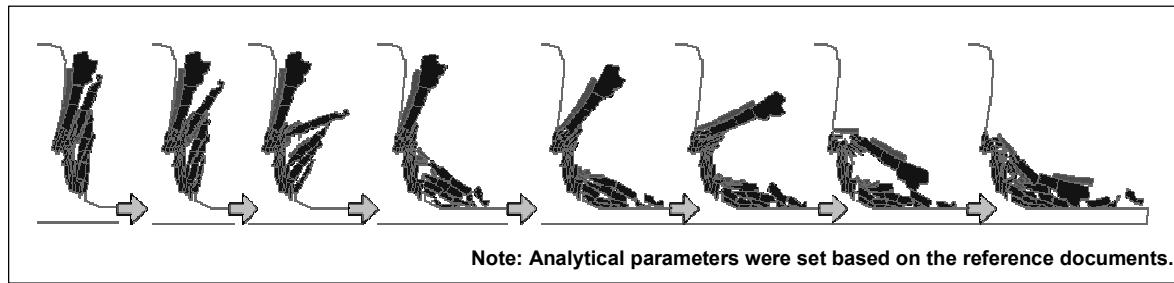


Figure 7, meanwhile, is the result of the simulation analysis using the distinct element method. The shape and physical properties of the rock element block and the physical properties of discontinuous surfaces were set based on the data of the research by Monma et al<sup>1)</sup>. Comparing the analysis result with the impression illustration, two stages of collapse are found to have been successfully reproduced. Thus, the reproduction of an actual collapse event is effective in identifying analytical parameters for which the setting of physical properties of discontinuous surfaces is difficult. The results of the reproduction test will serve as useful information for analysis to predict any collapse of a slope.

**Figure 7. Result of simulation by distinct element method**



### (3) Simulation analysis for a model slope collapse

Next, an analysis example of rock collapse behavior for a model slope is presented. The subject slope is one adjacent to an actual nuclear power plant, modeled into a slope measuring 30 m in both height and width. Figure 8 shows the analytical model. For the area of collapse, a virtual sliding surface was assumed, based on the seismic stress distribution obtained from previously conducted dynamic FEM analysis. The shapes of rock elements in the area of collapse were divided into hexagonal or quadrilateral elements. Elements outside the collapse area were treated as rigid bodies, while the side and bottom surfaces were seen as fixed boundaries. For input earthquake ground motion, meanwhile, the maximum acceleration of the horizontal and vertical motions were standardized to 800 gal and 427 gal, respectively, and both horizontal and vertical motions were simultaneously input. As the ground that constitutes the slope includes many uncertain factors, such as physical properties, stratigraphic structure, and the presence of cracks, and is therefore intrinsically wide in dispersion, parametric analysis was conducted by changing the major parameters, such as the shape of rock elements, the friction coefficient of the ground surface, and the bond strength of elements.

**Figure 8. Analytical model**

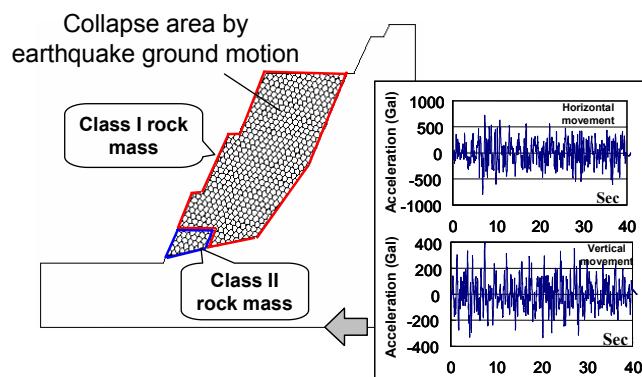


Figure 9 is an example of the result of rock collapse behavior analysis. Some analytical results obtained seem inappropriate in terms of engineering, depending on the above parameter settings, which indicates that the analysis result is strongly dependent on parameters and that such dependency makes the analysis

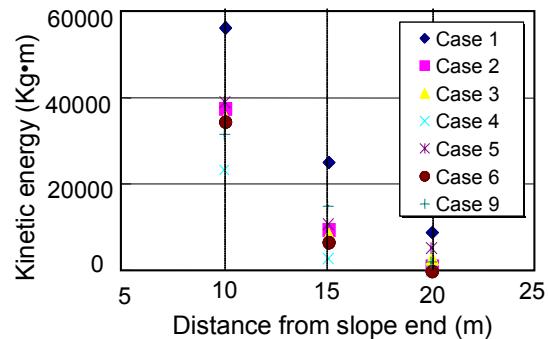
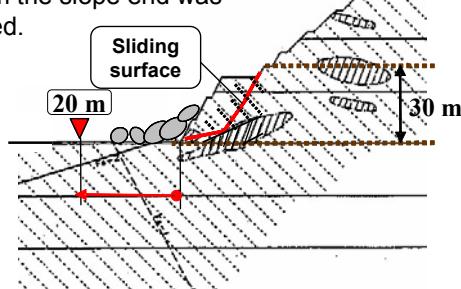
itself unstable. The difficulty in and importance of setting parameters in the distinct element method are recognized anew. Figure 10 shows an example of the relationship between the distance from the slope end and the cumulative dynamic energy of rock blocks in the case, outside parametric analysis cases, where the collapse condition is judged to be appropriate. The knowledge that the kinetic energy of rock blocks decreases with the increase in distance from the slope end and the fact that such the kinetic energy can be quantitatively understood will represent useful information to evaluate the bearing capacity of buildings against impact by rocks.

Figure 9. Example of rock collapse shape after slope failure



Figure 10. Cumulative kinetic energy of rock blocks

Kinetic energy of rock blocks that reached the position 10m, 15m, or 20m from the slope end was calculated.



## 5. Evaluation of secondary effect of the impact by rocks on the building

In order to understand the secondary impact of falling rocks striking the building following slope collapse, the impact of the rock blocks hitting was calculated and the response analysis conducted for the building hit by the rocks. Figures 11 and 12 show the locational relationship between the slope and the building and the mass model of the building, respectively. The subject slope is the model of the actual slope used in the previous analysis. The side wall of the building is assumed to be located 8 m from the slope end. The impact force of the rocks striking was calculated as a dynamic load using the distinct element method, by assuming the hit was concentrated on positions 8.9 m and 16.5 m from the ground surface. Next, the calculated dynamic impact force was input to the corresponding nodes of the building's mass model to carry out a response analysis of the building. In the rock collapse behavior analysis, the shape of the rock elements, the coefficient of friction of the ground surface, and the bond strength of the elements were changed as in the previous chapter to consider the dispersion of the properties of the ground constituting the slope.

Figure 11. Locational relationship between slope and building

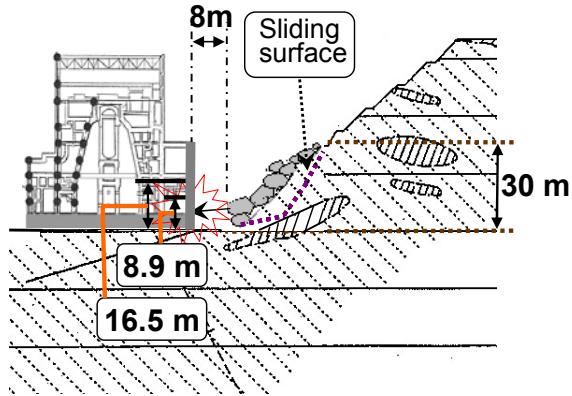
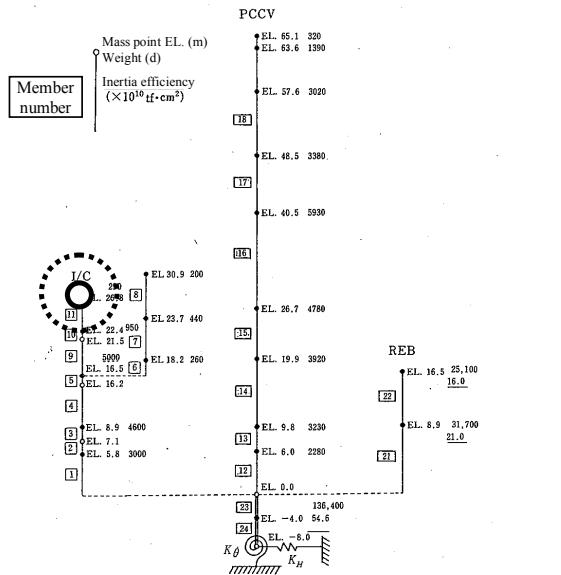
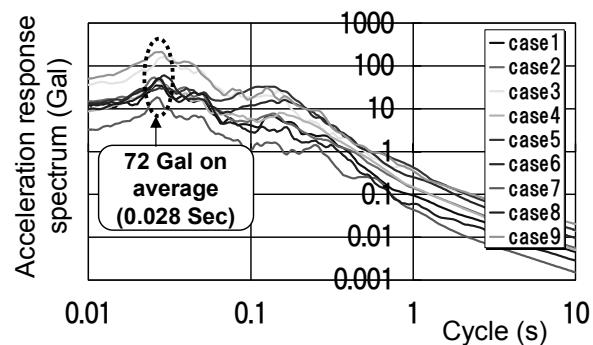


Figure 12. Building of mass model



An example of the analysis result is shown in Figure 13, which is the acceleration response spectra of the building node. The peak is found to have appeared at around 0.03 seconds and the average acceleration was about 78 gal. These results suggest that the influence does not seem so serious when compared with a response by the design earthquake ground motion. However, this review was simplified when compared with the building modeled into a mass system, and the local stress sustained by the building side wall, upon the impact of the rocks, was not taken into consideration. This detailed review will be necessary in future.

Figure 13. Acceleration response spectrum of building



## 6. Conclusion

The author is developing a probabilistic safety assessment that considers slope collapse caused by earthquake ground motion as a technique to evaluate the residual risk of an earthquake for a nuclear power plant with surrounding slopes. This paper provides the overall view of the evaluation technique and reports on the slope collapse probability evaluation method, rock collapse behavior analysis after slope collapse, and a secondary effect evaluation method for buildings hit by falling rocks. The studied methods introduce the distinct element method to analyze rock collapse behavior and generate a certain satisfactory outcome. However, as mentioned earlier, the analysis results are strongly dependent on the input parameters, which can make the analysis itself unstable. Attention needs to be paid to how to set input parameters.

The author intends to plan and implement a large-scale test in order to improve the precision of rock collapse behavior analysis and understand the collapse behavior of actual slopes.

## Reference

- 1) MONMA et al., Study of the application of discontinuous deformation analysis to evaluate the mechanism of rock slope instability, Journal of geotechnical engineering, No.757/□-66, 45-55, 2004.

# Development of the Fragility Curve of the Refuelling Water Storage Tank

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## 1. Introduction

The Institut de Radioprotection et de Sûreté Nucléaire (IRSN) carried out a Probabilistic Safety Assessment (PSA) of the standard French 900 MWe PWR. The general objective of this PSA is to supplement the deterministic analyses on which the reactor design is based in order to get better appraisal of nuclear risks. This PSA handles internal events and fire. To complete this PSA, IRSN carried out a study to examine the interest and the feasibility of a seismic PSA. This study showed that the equipment fragility evaluation constitutes one important step of the seismic PSA. So IRSN decided, in an exploratory way, to carry out the fragility study of an item of equipment. The item of equipment chosen was a Refuelling Water Storage Tank (RWST) of a typical PWR plant. The objective of this study was to define the methodology to implement in order to elaborate the fragility curve of static equipment.

The purpose of the paper is to describe the methodology used to establish the fragility curve of the RWST, to present the results got as well the main insights underlined by this fragility study.

## 2. Methodology

The methodology is drawn from Ravindra's works (ref. [1]) which are based on the research of the level of a solicitation leading to a postulated equipment failure. Concerning the RWST, the failures taken into account in the study are the loss of vessel integrity and/or its anchors. The seismic fragility of the RWST is defined by its conditional probability of failure induced by mechanical loading due to seismic motion of a given level. The seismic motion has been characterised by response spectra to the ground level (on the fragility curve this spectra is represented by the Peak Ground Acceleration - PGA).

The model used for the study consists in expressing the RWST fragility in terms of the best estimate of the median ground acceleration capacity **A<sub>m</sub>** that is factorized by two random variables **€<sub>u</sub>** and **€<sub>r</sub>** with unit medians. Thus, the ground acceleration **A** beyond which the RWST is damaged is given by:

$$A = A_m \cdot \epsilon_u \cdot \epsilon_r$$

**€<sub>r</sub>** represents the inherent randomness about the median and **€<sub>u</sub>** the uncertainty in the median value. We assumed that both **€<sub>r</sub>** and **€<sub>u</sub>** are lognormally distributed with logarithmic standard deviations **β<sub>r</sub>** and **β<sub>u</sub>**, respectively. These variables take into account the random feature of the seismic motion and structure resistance (tank + construction) as well as the uncertainties associated to each step of the seismic calculation carried out to determine the seismic loads and verify the design criteria. The first variable corresponds to the random uncertainties and the second to the knowledge insufficiency and the necessary simplificative assumptions for modelling.

The conditional probability of failure **P** for a given peak ground acceleration level **a** and for a given confidence level **Q** is given by:

$$P = \Phi \left[ \frac{\ln \left( \frac{a}{A_m} \right) + \beta_u \Phi^{-1}(Q)}{\beta_r} \right]$$

$\Phi$  is the standard Gaussian cumulative distribution function.

The fragility evaluation of the RWST follows two successive main steps:

- The failure modes definition.
- The evaluation of the seismic capacity of the tank for each failure mode.

The different modes of failure leading to the loss of integrity of the tank or to the rupture of its anchorages have been examined.

The seismic capacity has been evaluated for each failure mode and at the most unfavourable acceleration (spectral acceleration corresponding to the natural frequency of the tank). For estimating the parameters of the fragility curve ( $A_m$ ,  $\beta_r$  and  $\beta_u$ ), it is useful to introduce the variable  $F$ , called margin factor and defined by:

$$A = F * A_c, \text{ with } F = (\text{actual seismic capacity}) / A_{\text{median}}$$

$A$  is the ground acceleration capacity

$A_c$  is a scalar equals to the acceleration corresponding to the design earthquake.

$A_{\text{median}}$  is the actual response of the equipment (RWST) due to design earthquake.

$F$  can be expressed like:  $F = F_c * F_r$

$$F_c = (\text{actual capacity}) / A_{\text{dim}}$$

$$F_r = A_{\text{dim}} / A_{\text{median}}$$

$A_{\text{dim}}$  is the design response of the equipment (RWST) due to design earthquake.

The median margin factor  $F_m$  of the variable  $F$  can be directly related to the median ground acceleration capacity  $A_m$  of the variable  $A$  by:

$$A_m = F_m * A_c$$

For the RWST, the margin factor has been modelled as the product of the following margin factors:

$$F = F_c * F_{sr}$$

- a) Capacity factor  $F_c$ :  $F_c$  is the capacity factor, it is the product of the strength factor  $F_s$  by the ductility factor  $F_\mu$  ( $F_c = F_s * F_\mu$ ). The strength factor represents the ratio of ultimate strength to the loading calculated for the design earthquake.

$$F_s = \frac{P_l - P_n}{P_s - P_n}$$

Where  $P_l$  is the load inducing the equipment failure for the specific failure mode,  $P_n$  is the load corresponding to normal operation and  $P_s$  is the design total load (sum of seismic load corresponding to the design earthquake and to the normal operation load). The  $F_s$  factor corresponds to the margin taken into account during the design on the load calculation and on the numeric values considered like for instance the yield strength.

The ductility factor  $F_\mu$  is the margin induced by the fact that an earthquake represents a limited energy source and equipment can absorb substantial amounts of energy beyond yield without loss of integrity.

- b) Structure response factor  $F_{sr}$ : The structure response determined at the design stage is based on calculation using conservative deterministic response parameters. These parameters being random the actual structure response may differ substantially from the design calculated response. The factor  $F_{sr}$  takes into account this margin. It is modelled as the product of the following factors:

$$F_{sr} = F_{sa} * F_\delta * F_m * F_{cm} * F_{ec} * F_{ss} * F_{sd}$$

$F_{sa}$  is the spectral shape factor representing the difference between actual ground spectra and design ground spectra.

**F $\delta$**  is the damping factor representing the margin induced by difference between actual damping and design damping of the building.

**Fm** is the modelling factor representing the variability in floor response due to the different possible models.

**Fcm** is the mode combination factor accounting for variability in response due to the method used in combining dynamic modes of response.

**Fec** is the earthquake direction combination factor accounting for variability in response due to the method used in combining earthquake direction

**Fss** is the sol structure interaction factor representing the uncertainties linked to these phenomena.

**Fsd** is the factor to reflect the reduction with depth of seismic input.

The RWST being installed directly on the floor of the building, the factor margin of the equipment can be deduced directly from the acceleration of the floor at the natural frequency of the tank. Moreover, as far as only one acceleration value cannot describe the seismic behaviour of the tank, it is necessary to write:

$$F_{sr} = Q \text{ (Adim)} / Q(\text{Amedian})$$

**Q** is the torque: rocking moment and the shear and vertical forces at the tank foot. This quantity is function of several accelerations from response spectra of the floor supporting the RWST.

The variable **F** is then considered as a product of elementary independent variables involved into the successive submodels considered in the chain of computation from solicitation to ultimate resistance for a given failure. Allowing to mathematical properties of lognormal distributions of probability densities of these independent variables, the median value of the variable **F** is “simply” obtained as the product of the median value of each elementary variable of **F**, and the standard deviation of **F** ( $\beta_r$  and  $\beta_u$ ) are obtained by quadratic combination of the standard deviations of those elementary variables.

$$F_m = \prod_i (F_m)_i \quad \text{and} \quad \beta_u^2 = \sum_i (\beta_u^2)_i \quad \text{and} \quad \beta_r^2 = \sum_i (\beta_r^2)_i$$

For each supposed failure of RWST vessel and/or anchorage, it can be associated a set of fragility curves that are fully characterized by the following triplet ( $A_m$ ,  $\beta_r$ ,  $\beta_u$ ) for a given confidence level **Q**.

The determination of **Am** is made by any classical deterministic computation with all the means that are at our disposal (from analytical formulae up to non linear and temporal computations on FEM model), the real difficulty in such a study is to establish the values of the standard deviations  $\beta_r$  and  $\beta_u$  that are attached to each submodel considered in the decomposition of the above variable **F**.

### 3. Description of the RWS tank

The RWS Tank is a vessel made of stainless steel sheets giving an axisymmetrical shape with a cylindrical skirt resting on a flat bottom and topped by a spherical roof (see appendix 3). The skirt has a diameter of about 11,760 mm and a height of 16,200 mm and is constituted by several shells whose thickness varies from 17.5 mm (lower part of the tank) to 5 mm (upper part). The spherical roof ( $R=11,000$  mm) and the flat bottom are constituted by several welded sheets (5 mm and 6.5 mm thickness respectively). The tank is filled with 1600 m<sup>3</sup> of boron water up to 14730 mm height. The tank is anchored on a reinforced concrete floor (1000 mm thick) at level  $\pm 0.00$  of the fuel building by 36 anchor rods Ø80 mm located at the periphery of the bottom. The bottom is reinforced by an outer ring (60 mm thick and 300 mm width) resting on the concrete floor. The lower tip of the anchor rods is blocked under the floor by a nut that transmits the anchoring force to the concrete by compression through an interposed square steel plate. A manhole and several nozzles are disposed on the lower part of the skirt. The vessel is under atmospheric pressure due to a chimney through the roof.

During an earthquake, the tank is submitted by its supporting floor to alternate displacements characterized by the floor response spectra towards 3 orthogonal directions (2 horizontal, 1 vertical). The floor response spectra come from free field response spectra after a transfer through the structure of Fuel building.

#### 4. Ruin failure modes

The study has been conducted by IRSN with the help of the Bureau Coyne et Bellier (ref. [4]).

Several ruin mechanisms have been considered in order to establish the fragility of the tank under seismic solicitation. Among these mechanisms, nine have been estimated as representative of the physics of phenomena that could occur up to the ruin of such a vessel. It can be observed that during the study some postulated ruin mechanisms have been set aside (due to very high margin factor) and other new ruin mechanisms have been considered as more unfavourable.

The failure modes that are considered are the following ones (see appendix 3):

- Ruin A: Ruin by plastic lengthening of anchor rods.
- Ruin B: Ruin by concrete crushing under the blocking plate of anchor rods.
- Ruin C: Ruin by conical rupture of concrete around anchor rod.
- Ruin D: Ruin by tensile rupture of lower current part of the skirt.
- Ruin E1: Ruin by tensile rupture of lower part of the skirt in nozzles area 1.
- Ruin E2: Ruin by tensile rupture of lower part of the skirt in nozzles area 2.
- Ruin E3: Ruin by tensile rupture of lower part of the skirt in nozzles area 3.
- Ruin F: Ruin by elastic-plastic buckling of the skirt (elephant-foot buckling).
- Ruin G: Ruin by elastic buckling of the skirt.

#### 5. Margin factors and fragility curves

##### 5.1 Introduction

A margin factor is established for each ruin mechanism by using the physical model proposed by Housner (ref. [2]) (tank supposed rigid) and amended by Haroun (ref. [3]) (tank with its own flexibility).

It is convenient to share the **Fsr** margin factor into two subfactors: **Fsrf** that is the margin factor from the floor up to the equipment and **Fsrb** that is the margin factor from the ground up to the floor. Such dichotomy does facilitate the research of the failure modes, the subfactors of Fsrf are then limited to modelling subfactor and the damping subfactor of the floor itself.

$$\mathbf{F} = \mathbf{FS} * \mathbf{F}\mu * \mathbf{Fsrf} * \mathbf{Fsrb}$$

In the schemes were the forces developed by the seismic solicitation are balanced by the resisting forces developed by the structure at the postulated rupture, it is possible to share the margin factor into several margin subfactors, each one of them being dedicated to a specific variability (for instance: yield limit for the material, ductility account, geometric imperfections, modelling uncertainties, etc).

The tables in Appendix 1 sum up the characteristics of the different margin factors in terms of median value **Fi** and standard deviations  $\beta_{I,u}$  and  $\beta_{I,r}$  according to the here above decomposition.

The fragility curves are the expression of global margin factors, after their factorization by the PGA of design spectra.

## 5.2 Margin factor $F_s$ and $F_\mu$

The  $F_s$  and  $F_\mu$  margin factor are strongly dependent on ruin mechanism and on the post-elastic behaviour of materials up to rupture. The computation of their median value, and standard deviations are based on the semi-analytical model proposed by Haroun (some diagrams are used) completed by more refined non linear finite elements models.

Median value of  $F_s$  is obtained by considering, at limit equilibrium and from deterministic models, the balance of the seismic forces by the resistance strengths of materials involved in postulated rupture, the level of balanced forces being fixed by rupture criteria.

Median value  $F_\mu$  is obtained by using non linear models for beyond elastic deformations, in order to account for the plastic deformation and then the energy consumption.

The standard deviation  $\beta_r$  is deduced from the variability of mechanical characteristics (yield limit and Young modulus for steel, resistance in compression and in traction for concrete): their actual values are necessarily greater than the guaranteed ones.

The standard deviation  $\beta_u$  is deduced from the variability due to assumptions on lower limit of the values of post elastic deformation, that are describing the behaviour law, and on lower limits of ductility coefficient  $\mu$ , that are assumed.

The following table sums up the main assumptions necessary to evaluate the  $F_s$  and  $F_\mu$  margin factors.

	$F_s$	$\beta_r$	$\beta_u$	$F_\mu$	$\beta_r$	$\beta_u$
A	$f_y$ anchor + analytical model	$(f_y)m > (f_y)g$	Anchor location Diagram reading Modes uncoupling	$=(1-2\mu)^{1/2}$ By analytical non linear model	$(f_y)m > (f_y)g$	Shape of behaviour law $f_y = f_y(\epsilon)$ beyond elastic limit
B	$f_c$ floor + analytical model	$(f_c)m > (f_c)g$	Blocking plate dimension	$=1$	$(f_c)m > (f_c)g$	
C	$f_t$ floor + analytical model	$(f_t)m > (f_t)g$	Aperture of conical surface at rupture	(fragile rupture)	0	0
D	$f_y$ skirt + analytical model	$(f_y)m > (f_y)g$	Diameter and thickness of skirt Newmark vs. algebraic combination			
E1	$f_y$ skirt + analytical model	$(f_y)m > (f_y)g$	Diameter and thickness of skirt Approximation of analytical model	$=(1-2\mu)^{1/2}$ By FE non linear axisymmetrical model	Effect on buckling criteria of $(f_y)m > (f_y)g$	Lower limit of $\mu$ equal to 1,25
E2	$f_y$ skirt + FE model					
E3						
F	$f_y$ skirt + elastic plastic buckling criteria on analytical model	$(f_y)m > (f_y)g$	SRSS vs. algebraic combination			
G	$f_y$ skirt + elastic buckling criteria on analytical model	$(E_s)m > (E_s)g$	Diameter and thickness of skirt Diagram reading			

Notation used in the table:

$f_y$ : steel yield limit.

$E_s$ : steel Young's modulus.

$f_c$ : concrete compressive limit.

$f_t$ : concrete tensile limit.

$(f_x)m$ : actual mean limit value.

$(f_x)g$ : guaranteed limit value.

### 5.3 Margin factor $F_{sr,f}$

As above mentioned,  $F_{sr,f}$  margin factor relate only to the incidence of floor modelling through the margin subfactor  $F_{sr,f,m}$  and floor damping through the margin subfactor  $F_{sr,f,\delta}$  which are not independent from the mode of rupture. A detailed FE model of the supporting floor has been developed in order to quantify the weight of the implicit assumptions of an infinitely rigid support under the tank.

**$F_{sr,f,m}$ :** the stiffness of the floor, introduced by a FE model, modifies the natural frequencies of the tank and the resultant torque value at tank basis, for a given floor response spectrum. The median value is the ratio of such torque obtained by floor FE model on the torque value when the tank is rigidly supported. The standard deviation  $\beta_r$  is obtained by considering, on above ratio, the effect of variation of concrete Young modulus. The standard deviation  $\beta_u$  is obtained by considering, on above ratio, the effect of floor thickness variation.

**$F_{sr,f,\delta}$ :** the damping coefficient of the concrete, when combined with the damping coefficient of the tank and its content, tends to reduce the values of the floor response spectrum and thus the torque at tank basis. The median value is the ratio of the torque at tank basis obtained by reduced response spectrum on the torque value when the tank is rigidly supported. The standard deviation  $\beta_u$  is obtained by considering a variation of damping allowing the ratio to reach the unit value. The standard deviation  $\beta_r$  is obtained by applying a coefficient of 0.2 on the preceding standard deviation.

### 5.4 Margin factor $F_{sr,b}$

The margin factor  $F_{sr,b}$  is decomposed into subfactors according to the methodology. In order to quantify them, two types of computation have been used, a simplified analytic one corresponding to the model used for NPP design (i.e. stick model for building and soil-structure interaction by 3 dashpots) and a more refined one corresponding to actual best industrial possibilities (Finite Element code for building and Boundary Element code for soil-structure interaction). The mode of rupture of the equipment does not influence these margin factors, when considering the mass of the equipment with respect to the mass of the whole structure. Conversely, the study being focused on RWST failure modes, the computation chain from ground to floor is supposed to comply with viscous-elastic behaviour laws, even for high PGA values: the post-elastic behaviour of the building housing the tank, the ruin mechanisms of this building and its overturning are deliberately disregarded.

**$F_{sr,b,sa}$ :** the median value is the ratio, by refined model, of torque value at tank basis obtained when considering an enlargement of  $\pm 15\%$  of response spectrum on initial torque value; the standard deviation  $\beta_r$  is obtained by considering the effect, on the above ratio, of a 25% increase of concrete Young modulus; the standard deviation  $\beta_u$  is obtained by considering the effect, on above ratio, of a 33% reduction of soil characteristics.

**$F_{sr,b,\delta}$ :** the median value is obtained by considering the following two values: a first value is the ratio, by simplified model, of the torque value at tank basis with the conservatisms relating to the damping coefficients used at design stage on the torque value without the above conservatisms, the second value is the ratio, by refined model, of torque value with same conservative damping coefficients on the torque value with damping coefficients given by the refined model. The median value is the mean of preceding ratios. The standard deviation  $\beta_r$  is obtained by considering the effect of a 25% increase of concrete Young modulus on the ratio obtained by the simplified model; the standard deviation  $\beta_u$  is obtained by considering the effect of a 33% reduction of soil characteristics on above ratio by refined model.

- F<sub>srB,m</sub>**: the median value is the ratio of torque value at tank basis obtained by simplified model (design calculation) on torque value obtained by refined model; the standard deviation  $\beta_r$  is obtained by considering the effects, on above ratio, of a 25% increase of concrete Young modulus and of an increase from 7% up to 10% of the critical damping coefficient for concrete; the standard deviation  $\beta_u$  is obtained by considering the effects, on above ratio, of thickness and mass variation of the structure (within construction tolerances).
- F<sub>srB,cm</sub>**: the median value is the ratio of torque value at tank basis obtained by refined model with SRSS modes combination (used at design stage) on torque value obtained by refined model with CQC modes combination; the standard deviations  $\beta_r$  and  $\beta_u$  are neglected ( $=0$ ).
- F<sub>srB,ec</sub>**: the median value is the mean value of the ratios, for several floor response spectra, of torque value at tank basis obtained by quadratic direction on the torque value obtained by Newmark combination used at design stage (i.e. of  $QX \pm 0.4QY$  type when  $QX$  corresponds to a force induced by an earthquake in X direction); the standard deviation  $\beta_r$  is obtained by considering the effects, on above mean value, of a 25% increase of concrete Young modulus; the standard deviation  $\beta_u$  is obtained by considering the effect, on above mean value, of the more penalizing combination of  $QX \pm QY$  type.
- F<sub>srB,ss</sub>**: the median value is the ratio of torque value at tank basis obtained by simplified model (design calculation) on torque value obtained by refined model (with damping value of simplified model); the standard deviation  $\beta_r$  is obtained by considering the effects, on above ratio, of a 25% increase of concrete Young modulus; the standard deviation  $\beta_u$  is obtained by considering the effects of a 33% reduction of soil characteristics on torque value resulting from refined model (damping of simplified model being kept to same initial value).
- F<sub>srB,sd</sub>**: the median value is the ratio, by refined model, of the torque value at tank basis obtained with site response spectrum applied under building basemat on the torque value with the response spectrum applied under basemat after deconvolution-convolution in the soil from ground surface to basemat level of a signal which is consistent with the site response spectrum at ground level; the standard deviation  $\beta_r$  is obtained by considering the effects, on above ratio, of a 25% increase of concrete Young modulus; the standard deviation  $\beta_u$  is obtained by considering the effects, on above ratio, of a 33% reduction of soil characteristics.

## 5.5 Fragility curves

Fragility curves in terms of conditional probability of ruin vs. PGA level are developed, for each ruin mode, on the basis of above margin factors and on a design spectra of 0.2g:  $A = F * 0.2$  (g unit). They can be ranked according to their HCLPF value (High confidence of low probability of failure) given by  $HCLPF = A_m * \exp(-1,645 * (\beta_r + \beta_u))$ , where high confidence is rated to 95% and low probability to 5%. They are presented in Appendix 2.

## 6. Results and comments

### 6.1 Introduction

The results are presented in 4 tables in Appendix 1: table 1 for median values of margin factors, tables 2 and 3 for standard deviations ( $\beta_r$  and  $\beta_u$  respectively), table 4 for fragility curves and HCLPF values. The 3 first tables present the sharing in subfactors and their partial and global recomposition of the above values, rupture mode by rupture mode, the 4th table presents the recomposition of fragility curves parameters. Such detailing allows to bring some insights within the margin sources for the median value and within dispersive sources for the standard deviations.

## 6.2 Median values

- F<sub>s</sub>** values: a margin value less than 1 for B rupture means there is a misdimensioning of anchoring system, high values for C and D ruins mean they are improbable;
- F<sub>μ</sub>** values: values between 1 and 1.32 are likely values.
- F<sub>srf</sub>** values: values between 1 and 1.21 mean the dimensioning is made with reasonably precise computations, some light margin rests in damping provision for concrete.
- F<sub>srB</sub>** values: the values are independent from the rupture modes. The global value is quoted to 6.70: it means the main margin source comes from computation up to floor spectra. In F<sub>srB</sub> decomposition, it appears that 3 subfactors are actually responsible of high F<sub>srB</sub> value: F<sub>srB,m</sub> (model assumptions), F<sub>srB,ss</sub> and F<sub>srB,sd</sub> (relating to soil/structure interaction).

## 6.3 Standard deviation $\beta_r$

- $\beta_{s,r}$ :** low values comprised between 0.01 and 0.1, mainly due to the guaranteed value of yield strength of resisting material that is greater than the actual one.
- $\beta_{\mu,r}$ :** low values comprised between 0 and 0.033, due to guaranteed value of deformation at rupture of resisting material.
- $\beta_{srf,r}$ :** low values comprised between 0.011 and 0.019, due to the guaranteed value of yield strength of resisting material greater than the actual one.
- $\beta_{srB,r}$ :** high values (0.193) coming from radiative damping for soil/structure interaction.

## 6.4 Standard deviation $\beta_u$

- $\beta_{s,u}$ :** low values comprised between 0.016 and 0.11, mainly due to tolerances on location, dimensions and approximations for models.
- $\beta_{\mu,u}$ :** low values less than 0.043, mainly due to the realistic assumptions.
- $\beta_{srf,u}$ :** low values comprised between 0.03 and 0.085, due to floor thickness tolerance and conservatism on concrete damping assumption.
- $\beta_{srB,u}$ :** very high values (0.378) coming mainly from conservatisms on radioactive damping assumption for soil/structure interaction (0.148) and to soil characteristics reduction (0.307).

## 7. Lessons from present study

This type of study brings a transverse design analysis: the anchoring misdesign revealed through a design review of safety related tanks is naturally found again by such a study: the ruin B by concrete crushing under the blocking plate for anchor rod appears with a “margin factor” F<sub>s</sub> far from 1 (0.32), the applied successive margin factors raise the value up to a global margin factor equal to 2.59 which is far below the global margin factors of the other ruin mechanisms. The HCLPF of B ruin mechanism is equal to 0.19 that is less than the design spectra level (0.20): this is solved by increasing blocking plate dimensions.

The method allows a reliable appreciation of the beyond design margin: the margin is reasonably comfortable (misdesign being excepted).

The main sources of margin are located, by importance order, in the account of:

- Soil/structure interaction, by underdamped dashpot vs. Boundary Element Model.
- Building depth-in-soil, by convolution/déconvolution of seismic input signal from surface to basemat.

- Simplified model assumptions, by stick FE model vs. 3D FE model.
- Safety coefficients and guaranteed yield limits of resisting material.
- Energy consumption by ductility beyond elastic limit of resisting material.

The main sources of standard deviation are located, by importance order, in the account of:

- The damping uncertainty and randomness of soil under the structure housing the equipment.
- The soil characteristics uncertainty and randomness.
- The shape uncertainty of seismic signal.
- The randomness of yield limits of resisting materials.

When performing standard deviations evaluation, one has to bear in mind that due to their way of combination, small values of standard deviations are masked by the highest values.

When performing such probabilistic study, attention has to be paid on the greatest margins and standard deviations providers, i.e. soil/structure interaction.

Conversely, when performing seismic design of NPP, greater attention has to be paid on soil characteristics and on computation reliability that are decisive robustness factors of seismic capacity.

## **8. Conclusions**

The objective of this study has been to experiment a methodology in order to elaborate a fragility curve of one static equipment in view of defining the methodology to implement for carrying out a seismic Probabilistic Safety Assessment (PSA) for a standard French PWR 900 MWe NPP. The results confirmed that the equipment fragility evaluation in the frame of a seismic PSA supplements the deterministic analyses with transverse design review and allows better appraisal of margins allocation.

The fragility curve analysis is based on Ravindra's methodology which allows a reliable appreciation of the margin beyond design by evaluating a median value of best estimate resistance of equipment and the variations around the above value. This method offers the advantage of being relatively simple to set up and to reach an evaluation of the probability of failure of an item of equipment.

However, obtaining these values is a difficult and delicate work which must be carried out the most precisely possible and assumptions are to be kept in mind, in particular:

- Expert opinions are widely used in safety coefficients evaluation.
- The choice of lognormal distribution laws for parameters is more based on mathematical expedience than on rigorous theoretical principles.
- Numerous ruin mechanisms are to be considered in order to be convinced of their exhaustiveness.
- The use of response spectrum for earthquake characterization leads to neglect the energy brought to the structures and to neglect the role of solicitation cycles.
- The only use of PGA (acceleration at high frequency) to pilot the failure probability may be detrimental in fragility evaluation of equipment that are sensitive to solicitations at low frequency.

IRSN considers that coefficients and parameters used in fragility evaluation are to be ascertained by tests on shaking tables, by feedback from installations having experienced a strong earthquake and by experience gained from dynamic models, with finite elements method, of equipments. IRSN intends to launch a seismic PSA with systems involving more complex equipments (mechanical, electrical, pneumatic... active, passive...pumps, valves, steam generators) and feels the necessity to share the experience from seismic PSA when considering such complex equipments.

## References

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- [2] Housner, G. W., Dynamic pressures on accelerated fluid containers in *Bulletin Seismological Society of America*, Vol. 47, No. 1, 1957, pp 15-35
- [3] Haroun, M. A., Ellaihy, H. M., Model for flexible tank undergoing rocking in *Journal of Mechanical Engineering*, Vol. 111, No. 2., Feb. 1985
- [4] Coyne et Bellier, French civil engineering bureau

## Appendix 1.

Table 1. Median value  
of the margin factors  $F_i$ 

	Ruin mechanisms								
	A	B	C	D	E1	E2	E3	F	G
$F_s$	<b>2.77</b>	<b>0.32</b>	<b>5.66</b>	<b>9.76</b>	<b>1.75</b>	<b>2.23</b>	<b>1.68</b>	<b>1.23</b>	<b>1.83</b>
$F_\mu$	<b>1.28</b>	<b>1.00</b>	<b>1.00</b>	<b>1.32</b>	<b>1.32</b>	<b>1.32</b>	<b>1.32</b>	<b>1.32</b>	<b>1.32</b>
$F_{srf,\delta}$	1.310	1.310	1.310	1.110	1.150	1.310	1.250	1.060	1.010
$F_{srf,m}$	0.922	0.922	0.922	0.970	0.960	0.920	0.940	0.982	0.992
Combined $F_{RSF}$	<b>1.21</b>	<b>1.21</b>	<b>1.21</b>	<b>1.08</b>	<b>1.10</b>	<b>1.21</b>	<b>1.18</b>	<b>1.04</b>	<b>1.00</b>
$F_{srb,sa}$	1.020	1.020	1.020	1.020	1.020	1.020	1.020	1.020	1.020
$F_{srb,\delta}$	1.020	1.020	1.020	1.020	1.020	1.020	1.020	1.020	1.020
$F_{srb,m}$	2.100	2.100	2.100	2.100	2.100	2.100	2.100	2.100	2.100
$F_{srb,mc}$	1.075	1.075	1.075	1.075	1.075	1.075	1.075	1.075	1.075
$F_{srb,ec}$	0.940	0.940	0.940	0.940	0.940	0.940	0.940	0.940	0.940
$F_{srb,ss}$	1.650	1.650	1.650	1.650	1.650	1.650	1.650	1.650	1.650
$F_{srb,sd}$	1.840	1.840	1.840	1.840	1.840	1.840	1.840	1.840	1.840
Combined $F_{srb}$	<b>6.70</b>	<b>6.70</b>	<b>6.70</b>	<b>6.70</b>	<b>6.70</b>	<b>6.70</b>	<b>6.70</b>	<b>6.70</b>	<b>6.70</b>
<b>Partial combined margin factor</b> $F = F_S * F_\mu * F_{srf}$	<b>4.28</b>	<b>0.39</b>	<b>6.84</b>	<b>13.87</b>	<b>2.55</b>	<b>3.55</b>	<b>2.61</b>	<b>1.69</b>	<b>2.42</b>
<b>Global combined margin factor</b> $F = F_S * F_\mu * F_{srf} * F_{srb}$	<b>28.70</b>	<b>2.59</b>	<b>45.82</b>	<b>92.98</b>	<b>17.09</b>	<b>23.78</b>	<b>17.47</b>	<b>11.33</b>	<b>16.22</b>

Table 2. Standard deviations of margin  
factor from supporting floor  $\beta_{i,u}$ 

	Ruin mechanisms								
	A	B	C	D	E1	E2	E3	F	G
$\beta_{s,u}$	<b>0.016</b>	<b>0.035</b>	<b>0.051</b>	<b>0.024</b>	<b>0.207</b>	<b>0.110</b>	<b>0.122</b>	<b>0.031</b>	<b>0.027</b>
$\beta_{\mu,u}$	<b>0.043</b>	<b>0.043</b>	<b>0.000</b>	<b>0.030</b>	<b>0.030</b>	<b>0.030</b>	<b>0.030</b>	<b>0.030</b>	<b>0.030</b>
$\beta_{srf,\delta,u}$	0.079	0.079	0.079	0.033	0.043	0.079	0.067	0.019	0.003
$\beta_{srfm,u}$	0.030	0.030	0.030	0.030	0.030	0.030	0.030	0.030	0.030
Combined $\beta_{RSF,u}$	<b>0.085</b>	<b>0.085</b>	<b>0.085</b>	<b>0.045</b>	<b>0.052</b>	<b>0.085</b>	<b>0.073</b>	<b>0.036</b>	<b>0.030</b>
$\beta_{srb,sa,u}$	0.141	0.141	0.141	0.141	0.141	0.141	0.141	0.141	0.141
$\beta_{srb,\delta,u}$	0.307	0.307	0.307	0.307	0.307	0.307	0.307	0.307	0.307
$\beta_{srb,m,u}$	0.007	0.007	0.007	0.007	0.007	0.007	0.007	0.007	0.007
$\beta_{srb,mc,u}$	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
$\beta_{srb,ec,u}$	0.071	0.071	0.071	0.071	0.071	0.071	0.071	0.071	0.071
$\beta_{srb,ss,u}$	0.148	0.148	0.148	0.148	0.148	0.148	0.148	0.148	0.148
$\beta_{srb,sd,u}$	0.038	0.038	0.038	0.038	0.038	0.038	0.038	0.038	0.038
Combined $\beta_{srb,u}$	<b>0.378</b>	<b>0.378</b>	<b>0.378</b>	<b>0.378</b>	<b>0.378</b>	<b>0.378</b>	<b>0.378</b>	<b>0.378</b>	<b>0.378</b>
<b>Partial combined standard deviation</b> $\beta_{s,u} \oplus \beta_{\mu,u} \oplus \beta_{srf,u}$	<b>0.096</b>	<b>0.101</b>	<b>0.099</b>	<b>0.059</b>	<b>0.216</b>	<b>0.142</b>	<b>0.146</b>	<b>0.056</b>	<b>0.050</b>
<b>Global combined standard deviation</b> $\beta_u = \beta_{s,u} \oplus \beta_{\mu,u} \oplus \beta_{srf,u} \oplus \beta_{srb,u}$	<b>0.390</b>	<b>0.391</b>	<b>0.390</b>	<b>0.382</b>	<b>0.435</b>	<b>0.403</b>	<b>0.405</b>	<b>0.382</b>	<b>0.381</b>

Table 3. Standard deviations of margin factor from supporting floor  $\beta_{i,r}$

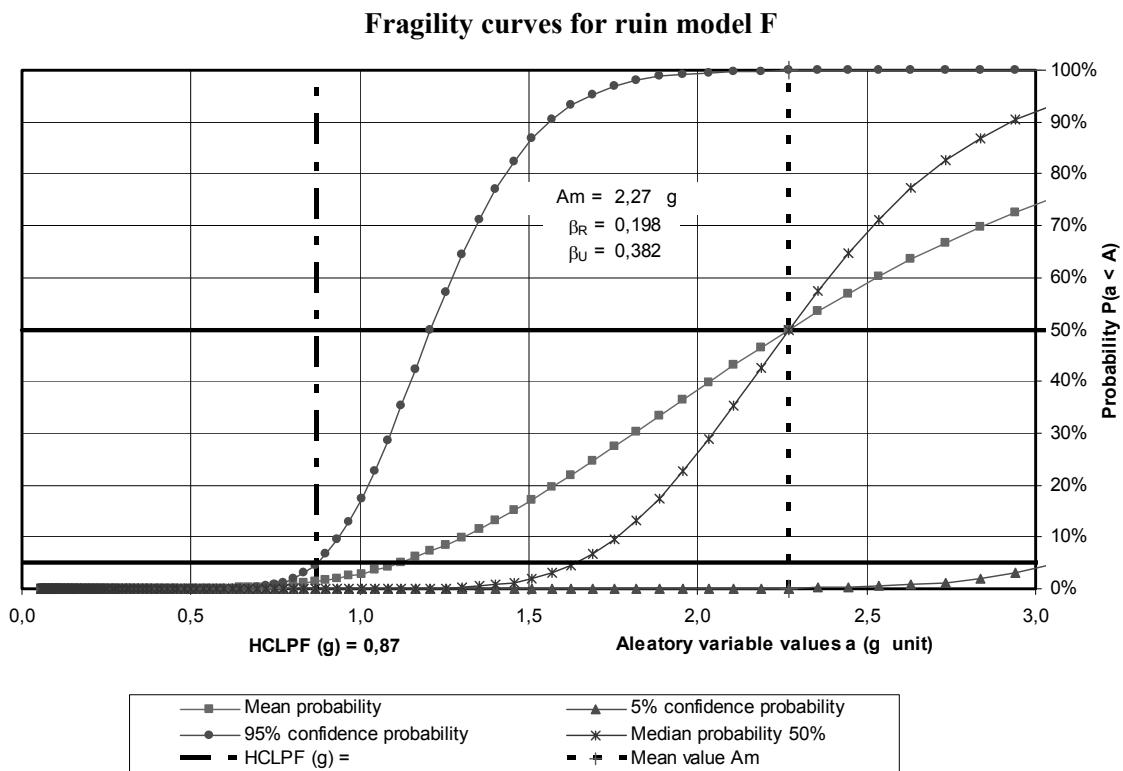
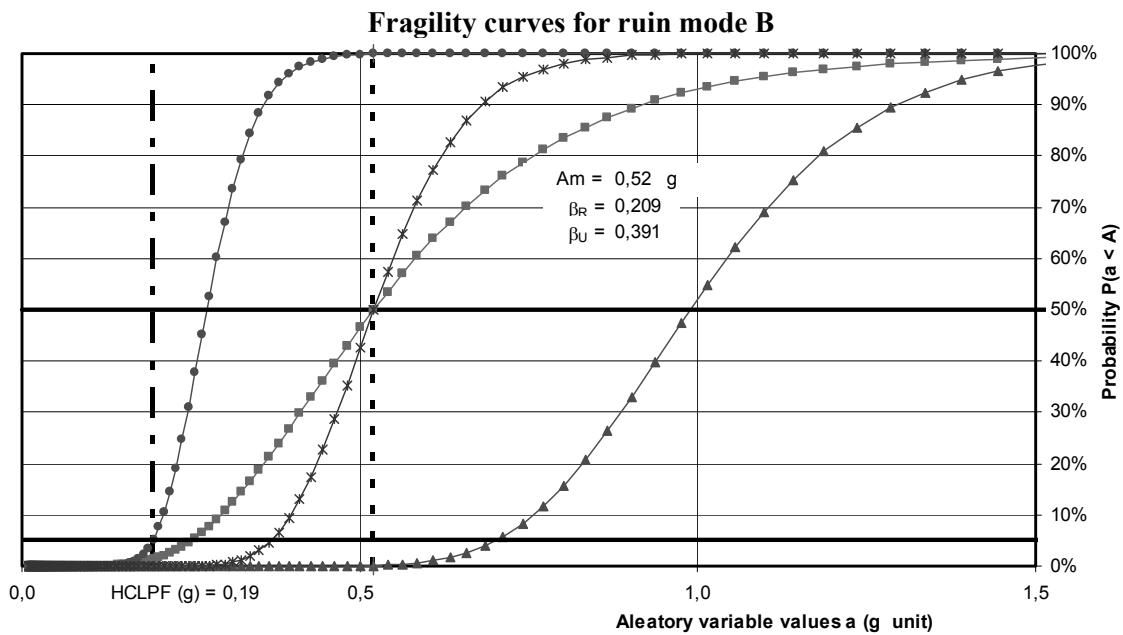
	Ruin mechanisms								
	A	B	C	D	E1	E2	E3	F	G
$\beta_{s,r}$	<b>0.053</b>	<b>0.078</b>	<b>0.047</b>	<b>0.092</b>	<b>0.115</b>	<b>0.085</b>	<b>0.093</b>	<b>0.031</b>	<b>0.011</b>
$\beta_{\mu,r}$	<b>0.015</b>	<b>0.015</b>	<b>0.000</b>	<b>0.033</b>	<b>0.033</b>	<b>0.033</b>	<b>0.033</b>	<b>0.033</b>	<b>0.033</b>
$\beta_{srf,\delta,r}$	0.016	0.016	0.016	0.007	0.009	0.016	0.013	0.004	0.001
$\beta_{srfm,r}$	0.011	0.011	0.011	0.011	0.011	0.011	0.011	0.011	0.011
<b>Combined <math>\beta_{Rsfr,r}</math></b>	<b>0.019</b>	<b>0.019</b>	<b>0.019</b>	<b>0.013</b>	<b>0.014</b>	<b>0.019</b>	<b>0.017</b>	<b>0.012</b>	<b>0.011</b>
$\beta_{srb,sa,r}$	0.013	0.013	0.013	0.013	0.013	0.013	0.013	0.013	0.013
$\beta_{srb,\delta,r}$	0.163	0.163	0.163	0.163	0.163	0.163	0.163	0.163	0.163
$\beta_{srb,m,r}$	0.042	0.042	0.042	0.042	0.042	0.042	0.042	0.042	0.042
$\beta_{srb,mc,r}$	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
$\beta_{srb,ec,r}$	0.026	0.026	0.026	0.026	0.026	0.026	0.026	0.026	0.026
$\beta_{srb,ss,r}$	0.047	0.047	0.047	0.047	0.047	0.047	0.047	0.047	0.047
$\beta_{srb,sd,r}$	0.076	0.076	0.076	0.076	0.076	0.076	0.076	0.076	0.076
<b>Combined <math>\beta_{srb,r}</math></b>	<b>0.193</b>	<b>0.193</b>	<b>0.193</b>	<b>0.193</b>	<b>0.193</b>	<b>0.193</b>	<b>0.193</b>	<b>0.193</b>	<b>0.193</b>
<b>Partial combined standard deviation</b>									
$\beta_{s,r} \oplus \beta_{\mu,r} \oplus \beta_{srf,r}$	<b>0.058</b>	<b>0.082</b>	<b>0.051</b>	<b>0.099</b>	<b>0.120</b>	<b>0.093</b>	<b>0.100</b>	<b>0.047</b>	<b>0.036</b>
<b>Global combined standard deviation</b>									
$\beta_r = \beta_{s,r} \oplus \beta_{\mu,r} \oplus \beta_{srf,r} \oplus \beta_{srb,r}$	<b>0.201</b>	<b>0.209</b>	<b>0.199</b>	<b>0.217</b>	<b>0.227</b>	<b>0.214</b>	<b>0.217</b>	<b>0.198</b>	<b>0.196</b>

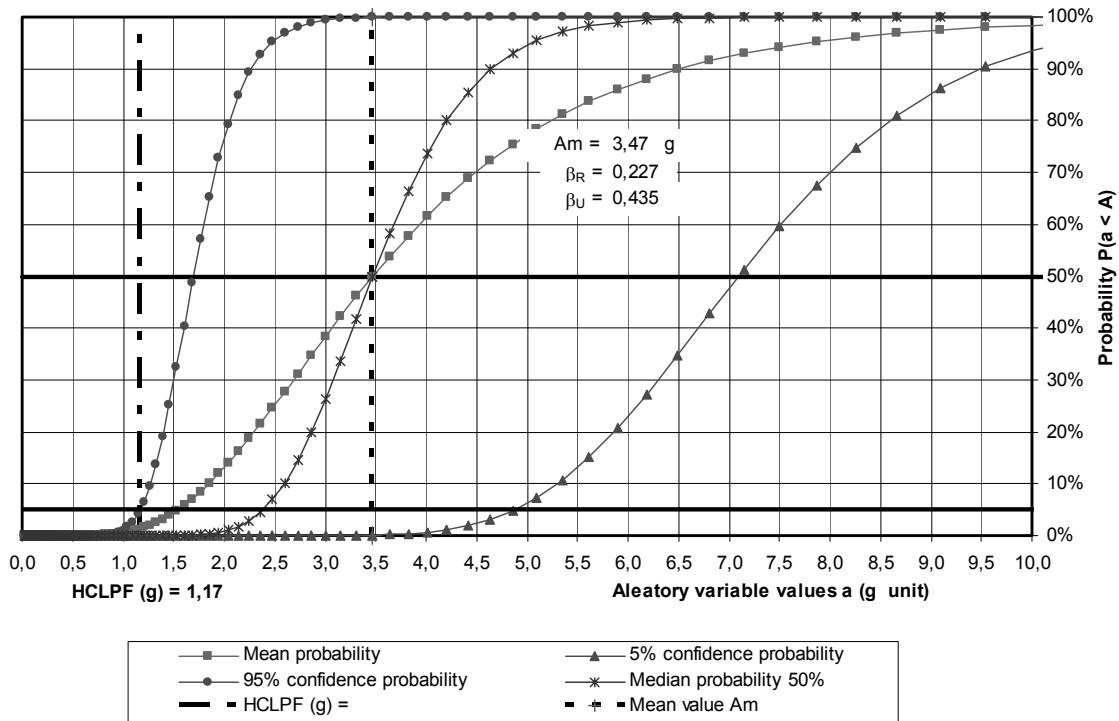
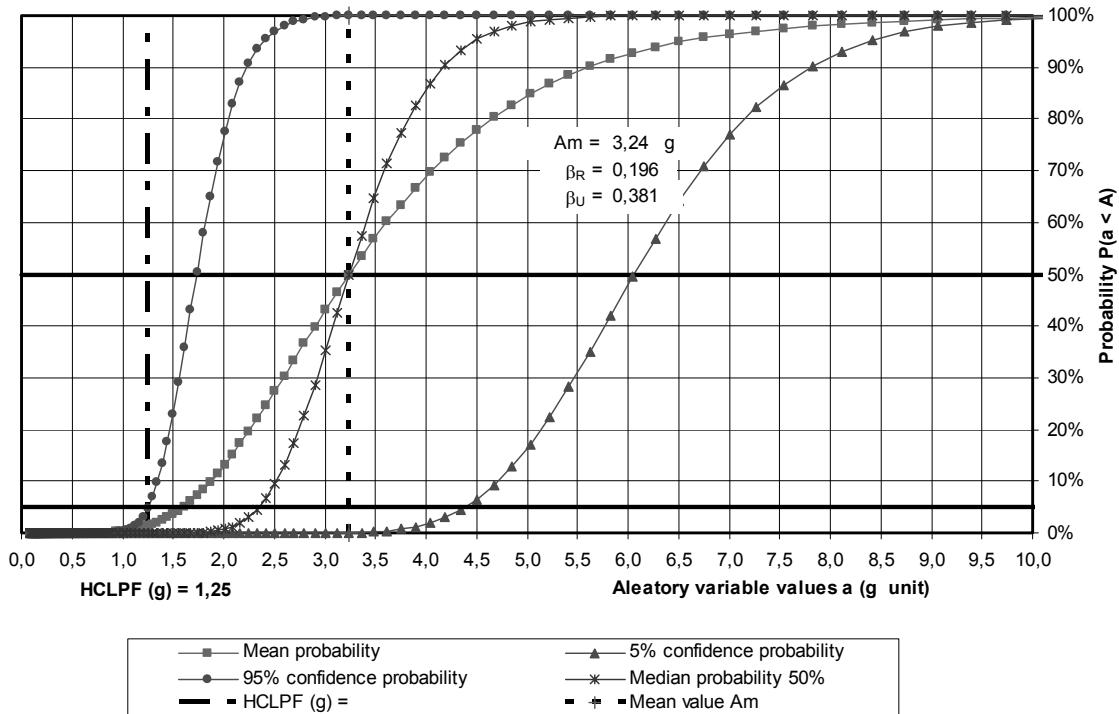
Table 4. HCLPF value (g unit)

	Ruin mechanisms								
	A	B	C	D	E1	E2	E3	F	G
(Am)c = $F_c * 0.2 = F_s * F_\mu * 0.2$ (g)	0.71	0.06	1.13	2.58	0.46	0.59	0.44	0.32	0.48
$\beta_r s \oplus \mu$	0.055	0.079	0.047	0.098	0.120	0.091	0.099	0.045	0.035
$\beta_u s \oplus \mu$	0.046	0.055	0.051	0.038	0.209	0.114	0.126	0.043	0.040
<b>Partial HCLPF (Tank+anchoring)</b>	<b>0.60</b>	<b>0.05</b>	<b>0.96</b>	<b>2.06</b>	<b>0.27</b>	<b>0.42</b>	<b>0.31</b>	<b>0.28</b>	<b>0.43</b>
$F_s * F_\mu * F_{srf} * 0.2$ (g)	0.86	0.08	1.37	2.77	0.51	0.71	0.52	0.34	0.48
$\beta_r s \oplus \mu \oplus srf$	0.058	0.082	0.051	0.099	0.120	0.093	0.100	0.047	0.036
$\beta_u s \oplus \mu \oplus srf$	0.096	0.101	0.099	0.059	0.216	0.142	0.146	0.056	0.050
<b>Partial HCLPF (Tank+anchoring+floor)</b>	<b>1.10</b>	<b>0.10</b>	<b>1.75</b>	<b>3.59</b>	<b>0.89</b>	<b>1.04</b>	<b>0.78</b>	<b>0.40</b>	<b>0.56</b>
$Am = F_s * F_\mu * F_{srf} * F_{srb} * 0.2$ (g)	5.74	0.52	9.16	18.60	3.42	4.76	3.49	2.27	3.24
$\beta_r s \oplus \mu \oplus srf \oplus srb$	0.201	0.209	0.199	0.217	0.227	0.214	0.217	0.198	0.196
$\beta_u s \oplus \mu \oplus srf \oplus srb$	0.390	0.391	0.390	0.382	0.435	0.403	0.405	0.382	0.381
<b>Global HCLPF (g unit)</b>	<b>2.17</b>	<b>0.19</b>	<b>3.47</b>	<b>6.95</b>	<b>1.15</b>	<b>1.72</b>	<b>1.26</b>	<b>0.87</b>	<b>1.26</b>
<b>Rank</b>	<b>7</b>	<b>1</b>	<b>8</b>	<b>9</b>	<b>3</b>	<b>6</b>	<b>5</b>	<b>2</b>	<b>4</b>

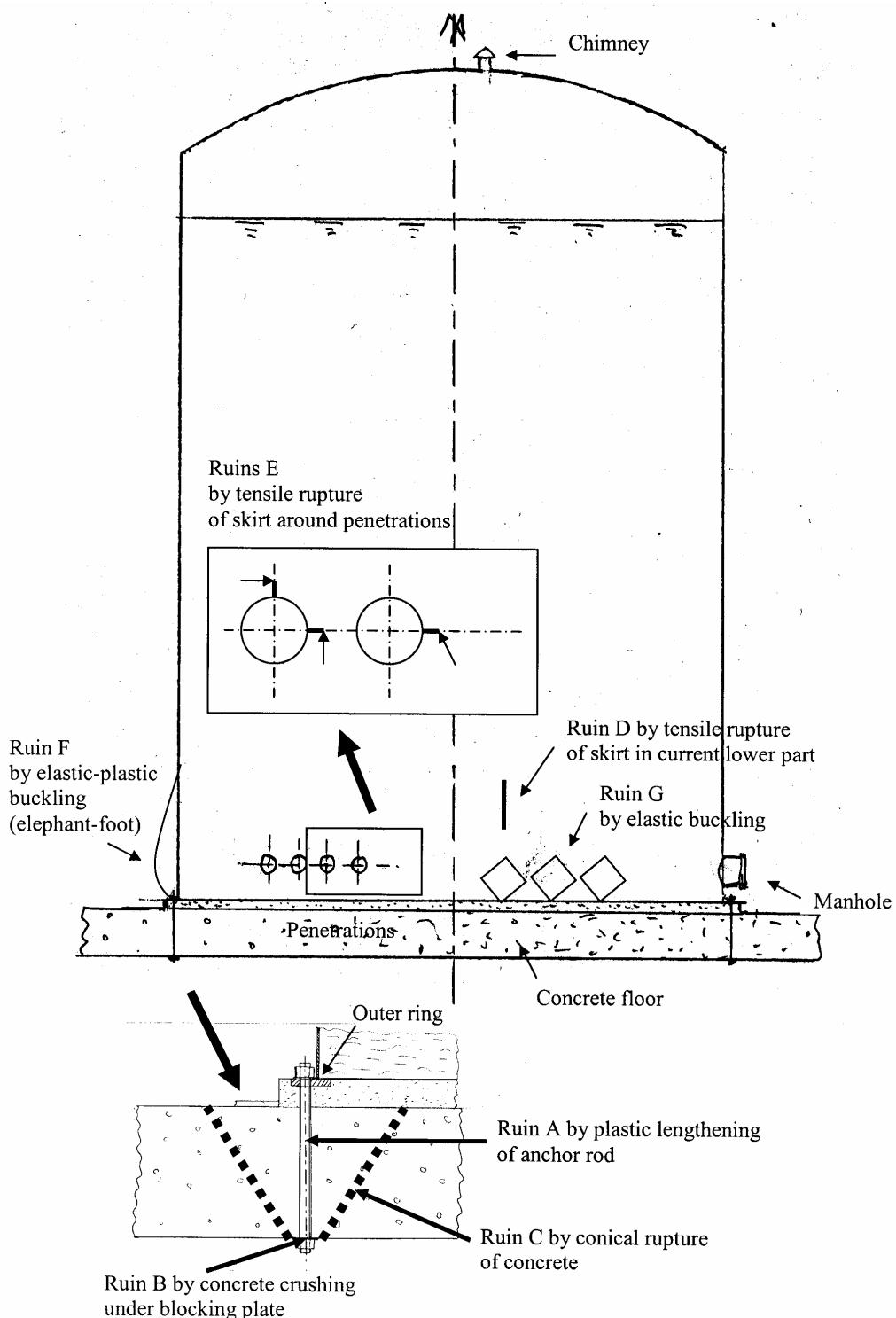
*Appendix 2.*

Fragility curves for the most probable rupture, according to HCPLF ranking



**Fragility curves for ruin mode E1****Fragility curves for ruin mode G**

## Appendix 3.

**RWS Tank typical shape and location of postulated failures**

## Seismic Fragility Capacity Tests of Components of Nuclear Power Plant

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### 1. Abstract

A seismic probabilistic safety assessment (PSA) is an useful method to evaluate residual risk of nuclear power plants (NPPs) that is designed with deterministic seismic design condition. For seismic PSA analysis of NPPs, seismic fragility capacity data of components are necessary, but the data of Japanese BWR and PWR components have not been sufficiently collected.

This paper describes the plan and current state of seismic fragility capacity tests on important NPPs components in JAPAN. Components for fragility capacity tests were selected considering the effect on core damage frequency (CDF) that was evaluated through preliminary seismic PSA of standard BWR and PWR. As results of the preliminary seismic PSA analyses, control rod (CR) system, vertical shaft pump, horizontal shaft pump, electric panels, valves and tanks were selected.

The seismic fragility capacity tests are conducted from Phase-1 to Phase-4. (Phase-1: horizontal shaft pump, electric panels, Phase-2: CR system of BWR and PWR plants, Phase-3: vertical shaft pump, Phase-4: valves and tanks)

The fragility capacity tests consist of two kinds of tests. One is life scale tests and the other is element tests. For the life scale tests, a life size model is tested using the TADOTSU shaking table with vibration amplification system, and critical acceleration, a failure mode are investigated. The element test is conducted on many samples, and critical acceleration, median and deviations are evaluated.

Phase-1 tests were already completed and critical acceleration and failure mode of horizontal shaft pump were collected. The critical accelerations and failure modes of electric panels, CR system of BWR and PWR plants and vertical shaft pump are now investigated. Phase-4 for valves and tanks is now being tested.

### 2. Introduction

Equipments of NPP are classified into the structures (piping, tank etc) and active components (pump, valve etc). Seismic capacities of the structures are obtained from their material strength data , and sufficient data have been collected .

For seismic PSA, realistic capacity data of active components are needed, but these data have not been sufficiently collected in Japan. So, we have used seismic proof tests data obtained in the past as the capacity data of these components. But in these seismic proof tests, acceleration levels were based on the design level. And they were not so high. So, we estimated capacity data of these components using the obtained qualified acceleration levels by the HCLPF method. Therefore, our seismic PSA analysis may be conservative and it is possible that if actual capacity data of important components are confirmed, reliability of seismic PSA is expected to increase. For such reasons, the seismic fragility tests on important components were being performed in JNES since 2002.

This paper described an overall test plan and some of the test results of horizontal shaft pump.

### 3. Overall test plan

#### 3.1 Test components

Components for the fragility tests were selected considering F-V (Fussell-Vesely) importance. Figures 3.1-3.4 and Figure 3.5 show the F-V Importance and RAW (Risk Achievement Worth) that were obtained by preliminary PSA analysis using current data. Considering that the importance of active components is high and active components are difficult to estimate their fragility capacity by analysis, the following components were selected for seismic fragility tests.

- Horizontal Shaft Pump.
- Electric panels (Control Panels, Relay Cabinets, Switch Gear etc).
- Control Rod (CR) System.
- Vertical Shaft Pump.
- Valves.
- Tanks.

The fragility tests are performed from Phase-1 to Phase-4. Horizontal shaft pump and electric cabinets are tested in Phase-1, CR system in Phase-2, vertical shaft pump in Phase-3 , Valves and Tanks in Phase-4.

Figure 3.1 F-V Importance of A-plant (BWR)

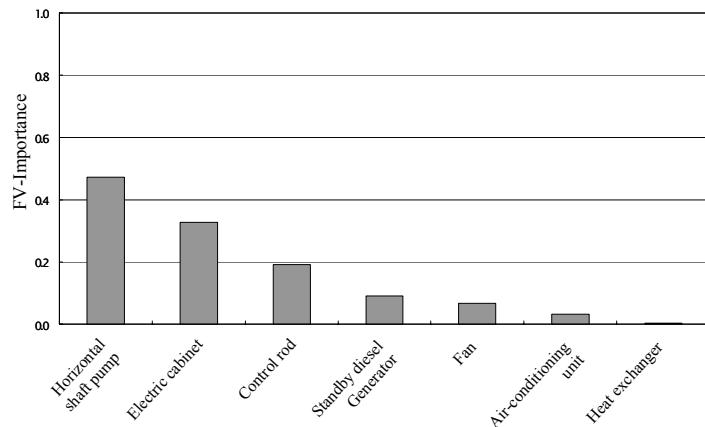
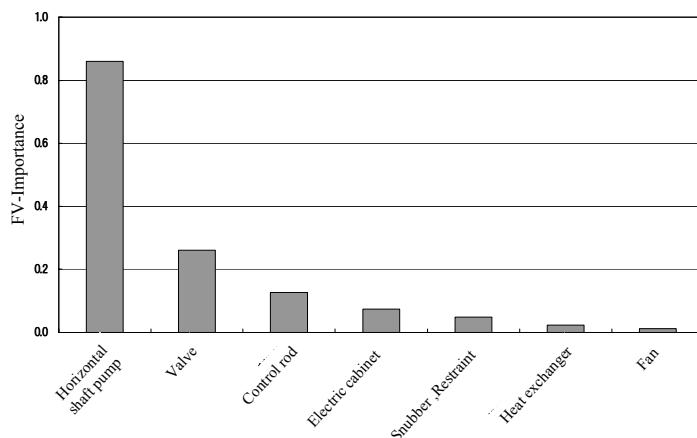
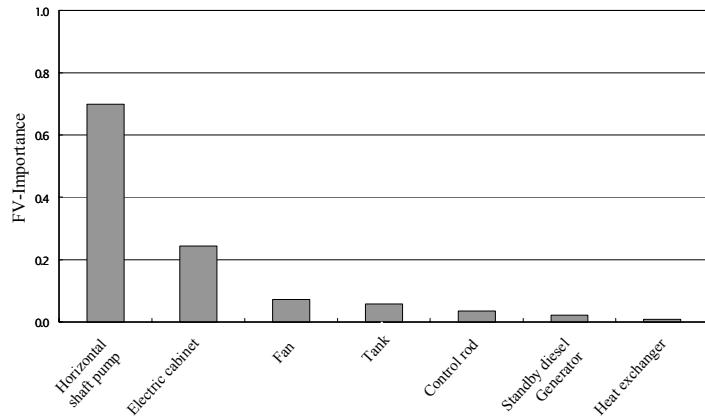


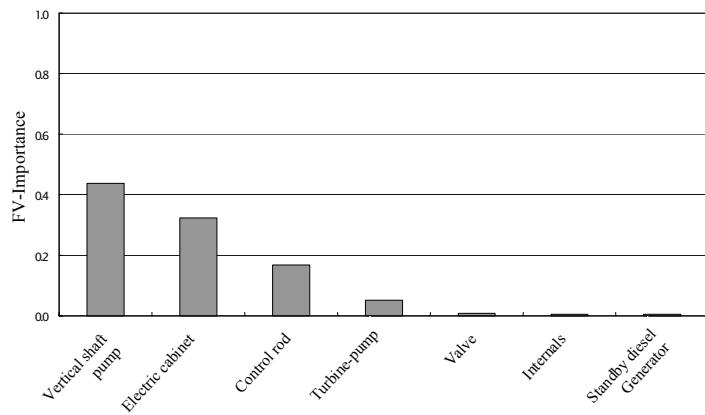
Figure 3.2 F-V Importance of B-plant (BWR)



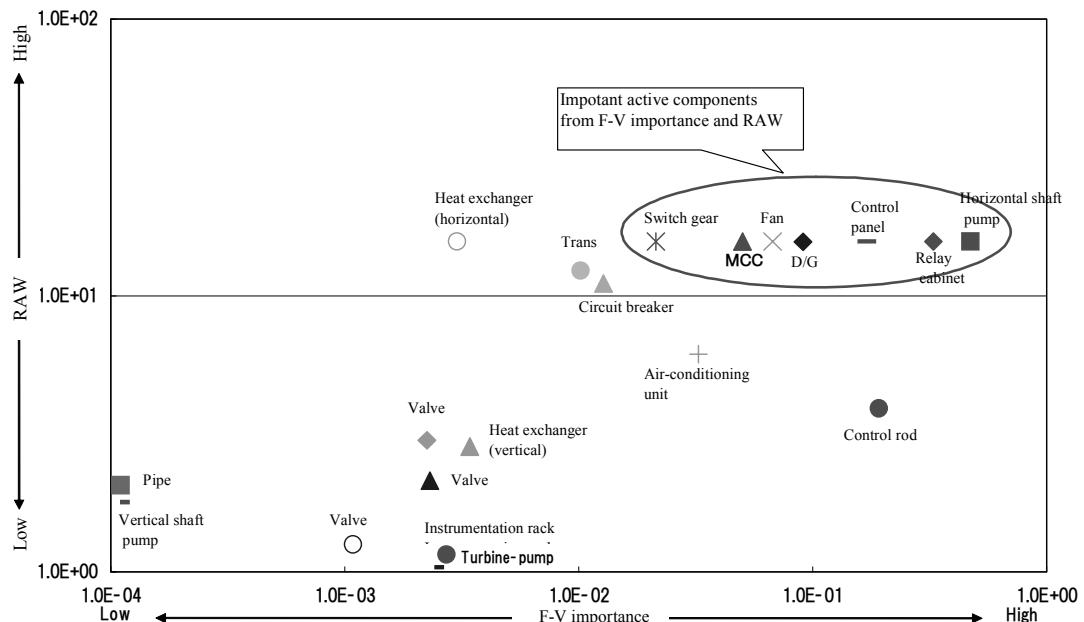
**Figure 3.3 F-V Importance of C-plant (BWR)**



**Figure 3.4 F-V Importance of B-plant (BWR)**



**Figure 3.5 RAW and F-V Importance of A-plant (BWR)**



### 3.2 Test schedule

The overall schedule of fragility tests is shown in table 3.1.

Phase-1: Life Scale horizontal shaft pump and Life Scale electric panels.

Phase-2: Life Scale but small number of fuel assemblies and control rods.

Phase-3: Life Scale vertical shaft pump.

Phase-4: Life Scale valves and scale model of tanks.

Table 3.1 Test schedule

	2002	2003	2004	2005	2006
Phase-1 Horizontal shaft pump Electric panels		Tests			
	Design/Manufacturing			Evaluation	
Phase-2 CR system		Tests			
	Design/Manufacturing			Evaluation	
Phase-3 Vertical shaft pump		Tests			
	Design/Manufacturing			Evaluation	
Phase-4 Tanks ,Valves		Design/Manufacturing			Tests Tests

## 4. Tests methods

### 4.1 Framework of tests

The fragility tests consist of two kinds of tests. One is life scale test and the other is element test. Main purpose of life scale test is investigation of critical acceleration and failure mode of component. The element test is conducted with many samples of parts that perform important function and have low seismic margin. The purpose of element test is to evaluate threshold acceleration of elements and its deviation. Component fragility capacity is estimated from the results of life scale test and element test.

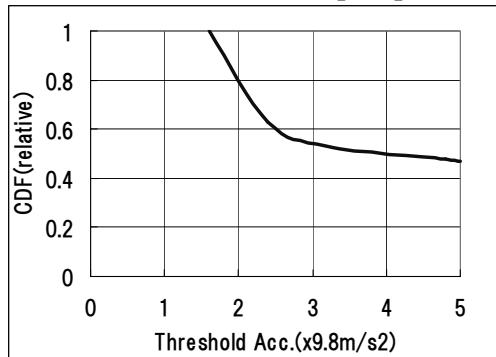
### 4.2 Acceleration amplifying system for life scale tests

An input acceleration level was determined based on the sensitivity analysis results of preliminary seismic PSA. Figure 4.1 shows a relationship between the critical acceleration of horizontal shaft pump and CDF values. From the results, it is seen that, if the critical acceleration of pump is higher than 3g, the CDF values is expected to decrease sufficiently.

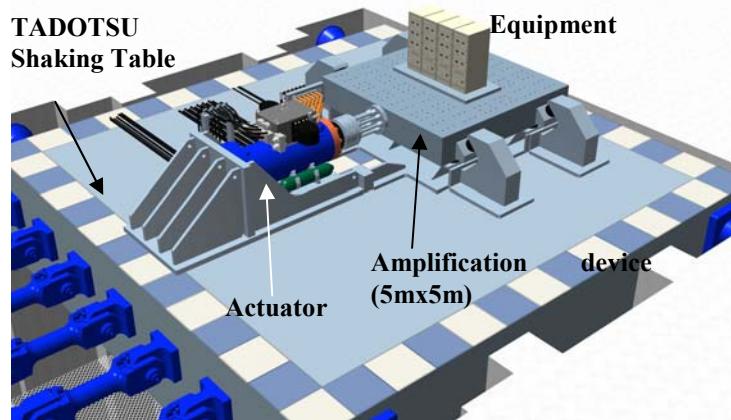
According to same analysis on electric panels, effective level of fragility capacity for reducing CDF sufficiently is over 5 or 6g. Consequently, maximum input acceleration was determined to be 6g-force in Phase-1 test.

The life scale test was carried out using the shaking table of TADOTSU Engineering Laboratory of Nuclear Power Engineering Corporation (NUPEC), but the original shaking table was not provided with a capability for high-level acceleration. So, an acceleration amplifying system was installed. Figure 4.2 shows an overview of the shaking table. The acceleration amplification device, 5m square table, is mounted on the TADOTSU shaking table. It moves in synchronization with TADOTSU shaking table. Its maximum loading capacity is 10ton and has a capacity of test acceleration up to 6g. The horizontal shaft pump and electric panels were tested separately.

**Figure 4.1 CDF vs. critical acceleration of horizontal shaft pump**



**Figure 4.2 Overview of shaking table**



## 5. Horizontal shaft pump test

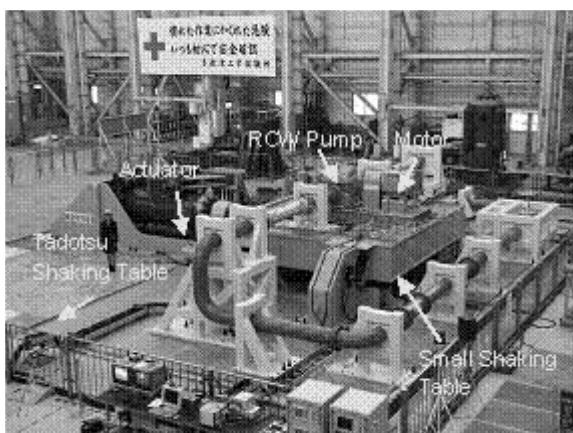
### 5.1 Life scale model test

#### (1) Test System

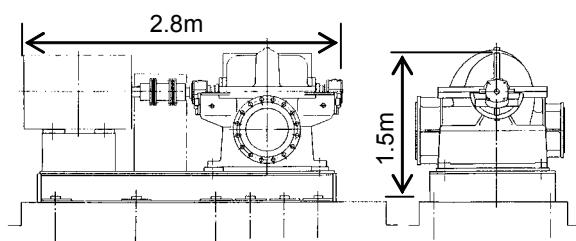
According to the preliminary PSA analysis, seismic damage to a horizontal shaft pumps of service water system causes an important effect on CDF. So a Reactor Building Closed Cooling Water (RCW) pump was selected as a typical horizontal shaft pump for the life scale test. Figure 5.1 shows an overview of the test system of horizontal shaft pump consisted of the RCW pump, electric motor, piping, valves, support structure, tank, power cabinets and instrument system. The RCW pump and electric motor are set up on the small shaking table.

The total length including pump and motor is about 2.8m, and height is about 1.5m and the total weight is 5.7ton. (Figure 5.2)

**Figure 5.1 Overview of test system**



**Figure 5.2 RCW pump for test**



## (2) Test conditions

An input acceleration level was increased from  $2 \times 9.8 \text{ m/s}^2$  to  $6 \times 9.8 \text{ m/s}^2$  with a step of about  $9.8 \text{ m/s}^2$ . The pump was tested both for operating and standby conditions and excitations were performed on axial and transverse directions.

## (3) Test results

Figure 5.3 shows the curve of flow rate and head. The dark rhombus marks are initial performance test data, white squares are data before the excitation test, and triangles are data after the excitation test. No decline was seen in pump performance. Figure 5.4 shows acceleration at the bearing case. Before the seismic excitation test (shown by “Input acceleration” of zero), the rattling acceleration was about  $10 \text{ m/s}^2$  to  $18 \text{ m/s}^2$ . After the seismic excitation test, the rattling acceleration increased slightly. But the difference was very small and no significant damage was found by post-test check of surface roughness.

Figure 5.3 Flow rate and head of the pump

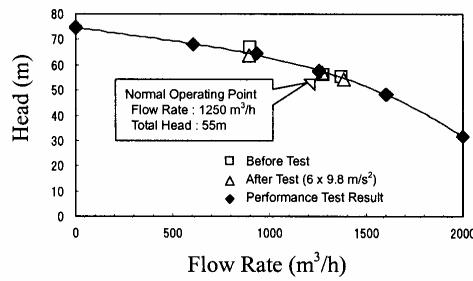
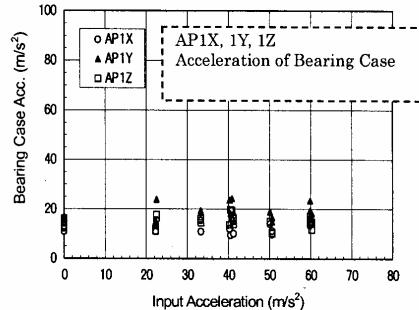


Figure 5.4 Acceleration at bearing case



## 5.2 Element tests

### (1) Elements for the test

In the element test, ball bearings, slide bearings and liner rings were investigated.

Liner ring: According to previous study, a seismic margin of the liner ring is lower than other parts, it is possible that the liner ring and the pump impeller might contact during transverse excitation.

Bearing: Bearings generally have a high seismic margin, but they are the most fundamental parts of rotational function and dynamic characteristics data are necessary to make dynamic analysis model of the horizontal shaft pump.

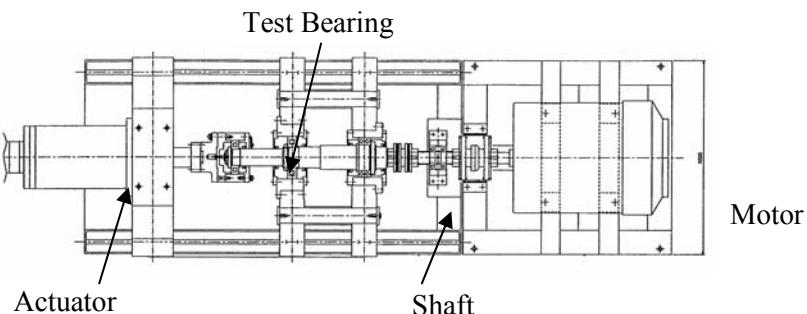
### (2) Test devices and conditions

In the element tests, sinusoidal excitation tests and seismic wave excitation tests were carried out.

Sinusoidal excitation tests: Dynamic properties of parts were tested under large input loads. The maximum input load was equivalent to over 10g-force acceleration.

Seismic wave excitation test: Fragility capacity and failure mode were evaluated. The maximum load was about 33 kN, which was equivalent to about 39g-force input acceleration for actual horizontal shaft pump. An element test device for thrust ball bearing is shown in Figure 5.5. It consisted of actuator, motor, shaft, support structure, and instrumentation system. The slide bearing test was carried out with a different device.

Figure 5.5 Test device of thrust ball bearing



### (3) Test results

Figure 5.6 shows accelerations of the bearing case of type 6310.

When the maximum load magnitude was lower than 20kN, the rattling acceleration did not change, but in the cases where load was higher than 20kN, the rattling acceleration increased significantly. This load level is approximately equivalent to 20g-force. Figure 5.7 compares the surface roughness of balls. Surface roughness of balls and internal structure of bearing were degraded by the vibration test. Similar degradation was found on other types of bearings.

Regarding the slide bearings, a slight degradation was found on the inner surface of radial bearing. Figure 5.8 shows the relationship between the input load and the torque of slide bearing after shaking test (60mm I.D.). The torque began to change at approximately input load of 20kN, which is equivalent to 21g-force seismic acceleration for the actual pump. An example of inner surface damage of the slide bearing is shown in Figure 5.9. This photograph was taken after the vibration test.

Such damage on the bearings does not cause loss of pump function immediately, but it would cause reduction of pump life. An important point is that such kind of damage may possibly happen under strong seismic load condition, that largely exceeds design or seismic PSA condition.

Figure 5.6 Acceleration of bearing(6310)

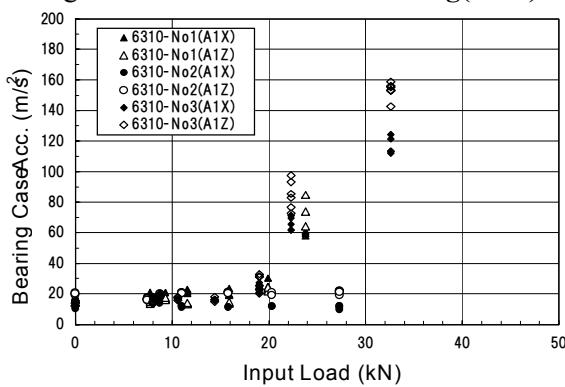
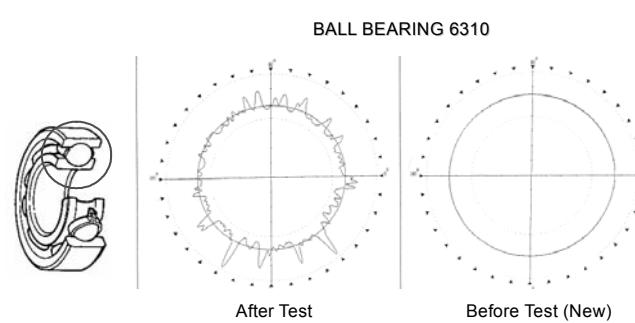
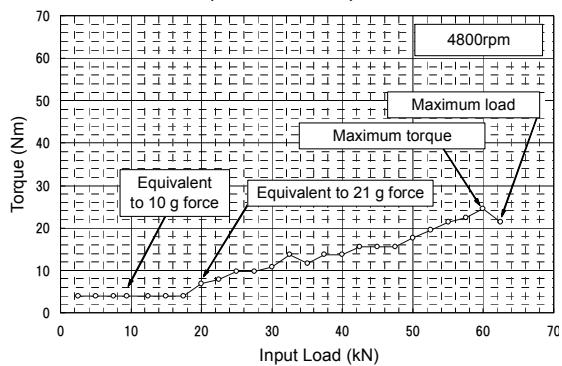


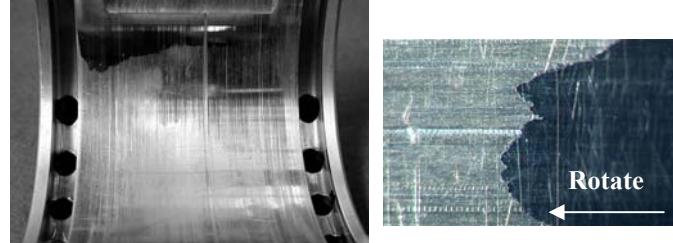
Figure 5.7 Surface roughness of ball (6310)



**Figure 5.8 Input load and torque  
(60mm I.D.)**



**Figure 5.9 Inner surface of slide bearing  
(60mm I.D.)**



## 6. Application of the test results to the fragility curve of horizontal shaft pump

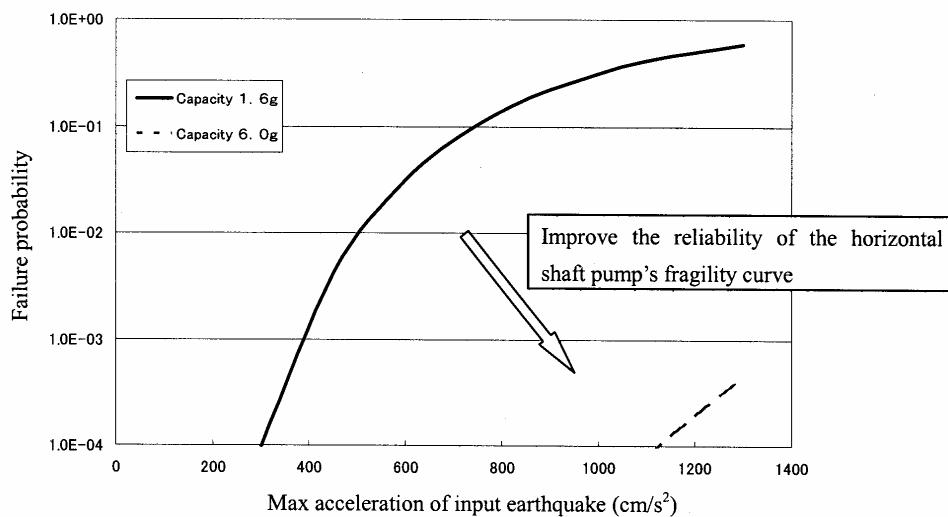
Fragility capacity acceleration of horizontal shaft pump was assumed to be 1.6g from previous vibration tests and had been used for a JNSE seismic PSA analysis.

From the results of the life scale test and the element test, it seems that horizontal shaft pump keep the functions at the seismic acceleration level of 6g.

Figure 6.1 shows the comparison of the fragility curves of horizontal shaft pump with its capacity value of 1.6g and 6g.

Using the fragility capacity test result (6.0g), the fragility curve of horizontal shaft pump is revised and the reliability of seismic PSA is also improved.

**Figure 6.1 Comparison of the fragility curve of horizontal shaft pump**



## 7. Conclusions

JNES have been evaluating seismic fragility capacities of important active components from Phase-1 to Phase-4. (Phase-1: horizontal shaft pump, electric panels; Phase-2: CR system of BWR and PWR plants; Phase-3: vertical shaft pump and Phase-4: valves and tanks).

In the Phase-1 the life scale test and the element test of the horizontal shaft pump were performed. In the life scale test, the acceleration amplifying system was used with the input acceleration up to 6g. But any loss of the pump function or damage on the main parts was not observed.

On the element test, small degradation was found on the surface of some bearings, which would reduce the pump life. But it happened under very high seismic acceleration that exceeded 10g force. The test results show that the realistic fragility capacity of horizontal shaft pump will be at least 6g , and higher than the current value (1.6g) .

The critical accelerations and the failure modes of the electric panels, CR system of BWR and PWR plants and the vertical shaft pump are now investigated. The valves and the tanks are now under testing. These fragility capacity data will be used for the seismic PSA, and it is expected that the reliability of seismic PSA analysis will increase.

#### *Acknowledgments*

This task has been performed by JNES as a national project with review of technical committee in JNES that is comprised of learned and experienced persons.

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# Seismic Fragility Evaluation of Electrical Power Systems and Influence Evaluation of Seismic Base Isolation on Seismic Risk

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## *Abstract*

This paper presents a methodology to evaluate the seismic fragility of the electrical power system including the transmission tower. This paper also presents the effectiveness of the seismic base isolation as method of decreasing seismic risk.

The failure probability of the transmission tower under the seismic condition was relatively low compared with the electrical transmission equipment in the switchyard. Therefore, the occurrence probability of LOSP will be determined by the failure of the transmission equipment in the switchyard.

In case studies, the effectiveness of the seismic base isolation to the emergency transformer with ceramic tube and the air blast circuit breaker located in the switchyard was evaluated. As the result, the failure probability of these components under a seismic condition was significantly decreased by the seismic base isolation.

## **1. Introduction**

In the seismic probabilistic safety assessment (PSA) of nuclear power plants, loss of offsite power (LOSP) is an initiating event that has a major influence on core damage frequency (CDF). The conventional seismic PSA assumed that LOSP was caused by failure of emergency transformer with ceramic tube or a circuit breaker in the site switchyard, and the effect of electrical power systems outside the nuclear power plant has not been discussed.

However, damage records of electrical transmission equipment in substations outside nuclear power plants have been reported in the 1995 Hyougo-ken Nanbu earthquake. In addition, many collapses of transmission towers have been reported in the 1999 Taiwan Chi-Chi earthquake. For more accurate evaluation of occurrence probability of LOSP it is necessary to evaluate the seismic fragility in the electrical power system including the transmission tower.

This report:

- Propose a methodology of the seismic fragility evaluation in the electrical power system by Fault Tree (FT) analysis based on the seismic functional failure probability of equipment.
- Evaluate the effectiveness of the seismic base isolation to the electrical transmission equipment in order to reduce the seismic risk due to LOSP.

## **2. Seismic fragility evaluation of electrical power system**

### ***2.1 Methodology of seismic fragility evaluation***

The authors assumed that the electrical power system consists of two pairs of transmission lines, the switchyard in the nuclear power plant is a starting point, and the substation is a relay point. There are transformers, circuit breakers, disconnecting switches and other equipment in the switchyard and substations. Figure 1 shows the outline of the electrical power system. A scenario to the loss of function of

the electrical power system was arranged into a systematic association chart to create the fault tree (FT) shown in Figure 2. In this study, the following electrical equipment was selected for seismic fragility evaluation: emergency transformer with ceramic tube, circuit breaker and transmission tower.

Figure 1. Schematic sketch of electrical power system

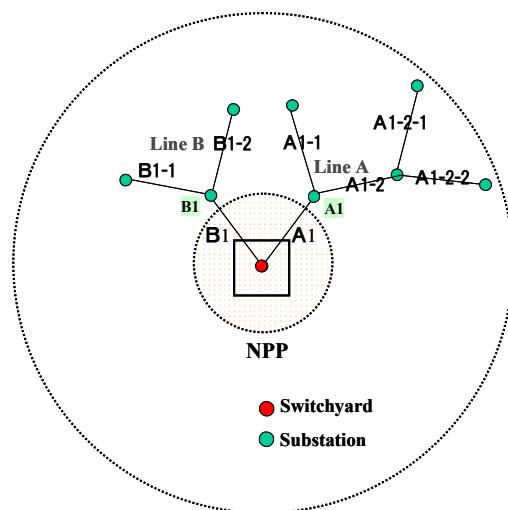
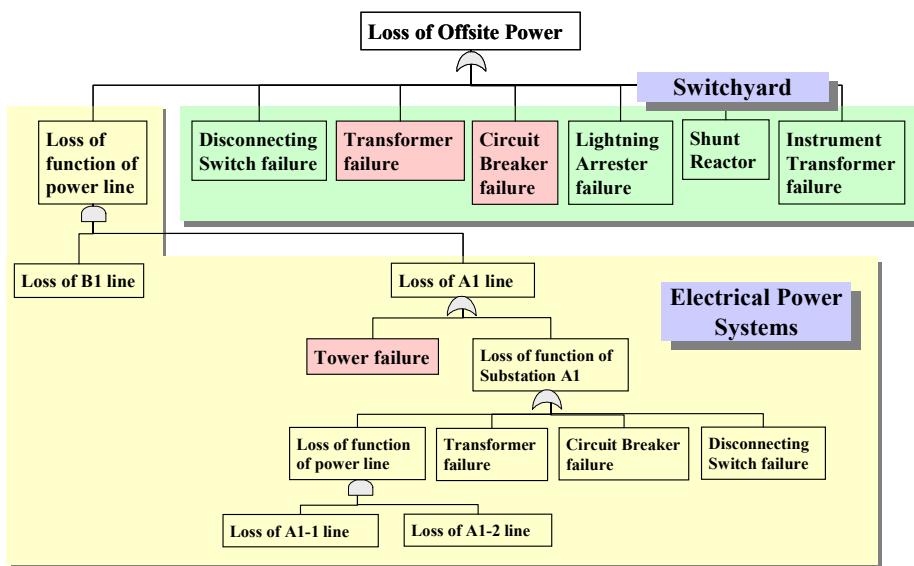


Figure 2. Fault tree of electrical power system



In this study, the seismic fragility of the transmission tower is calculated and compared with the seismic fragility of electrical equipment in the switchyard. Based on this, the influence of seismic fragility of the transmission tower on seismic fragility of the electrical power system is evaluated.

## 2.2 Seismic fragility evaluation of the transmission tower

In the capacity evaluation, the seismic margin corresponding to the failure mode of the tower was calculated and the minimum seismic margin was taken as the realistic capacity. The realistic response was calculated by dynamic analysis using the time history of design ground motion, the FEM model and the response factor. The response factor introduced as a measure of conservatism in seismic design analysis is defined as a ratio of conservative design response to realistic response. The response factor is represented

by a probability density function of the logarithmic normal distribution with median and logarithmic standard deviation. Seismic fragility of a UHV (Ultra-High Voltage: 1000-kV class) transmission tower was evaluated using a seismic capacity and realistic response.

### 2.2.1 Transmission tower

UHV Akagi Electric Transmission Tower was selected for seismic fragility evaluation [1-5]. Such a tower is much taller and contains large cross arms compared to conventional towers. A schematic diagram of UHV Akagi Test Line is shown in Figure 3 and the structure and dimensions of central Tower 2 is shown in Figure 4. The FEM model of the tower is shown in Figure 5. The model is a three tower model considering of one detailed tower model in the center. The tower foundation is much stiffer than its superstructure. There is little difference in the response in the tower where the foundation was assumed to be fixation and in the tower where the foundation and the ground were modeled [3]. Therefore the foundation of the transmission tower was assumed as fixation. The tower is comprised of tubular steel.

Figure 3. Schematic diagram of Akagi Test Line

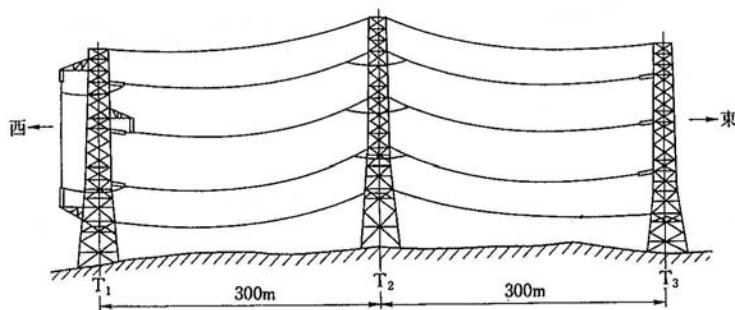


Figure 4. Structure of transmission tower

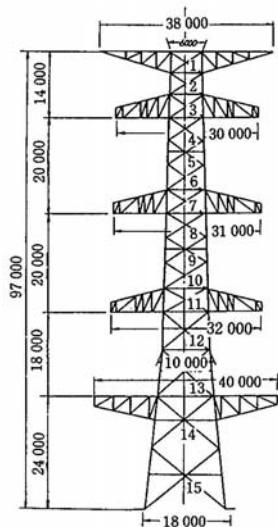
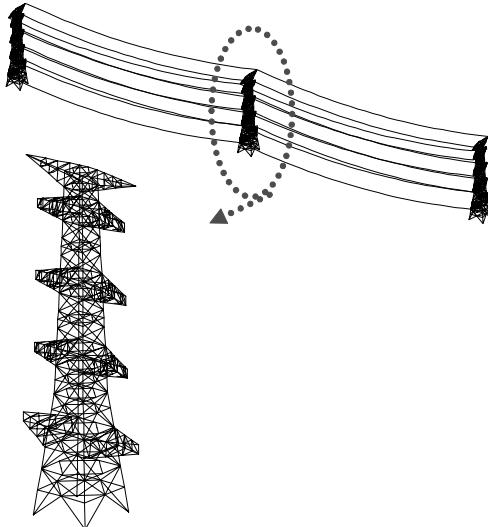


Figure 5. Analysis model



### 2.2.2 Capacity evaluation

#### (1) Failure modes, vulnerable parts and failure criteria

The failure mode was presumed the functional failure of tower. The failure criteria of tower were considered to be material strength (axial force, shear force and bending moment), buckling and falling. The functional failure of suspended insulators is not considered to be failure mode in this study. The suspended

insulators play the role to insulate the horizontal vibrations of the line system, so there is no difference of seismic response between the single tower model and the coupled system of tower and power line system including the suspended insulators [6]. In addition, there are no reports of the insulator damage due to the earthquake in Japan and the Chi-Chi earthquake of Taiwan. The failure criteria, ultimate value, response and margin (ultimate value/response) based on the response analysis results are shown in Table 1.

Table1. Results of response analysis

Failure Criteria	Vulnerable Parts	Ultimate Value	Response (300 gal)	Margin
Axial force	Foundation	460 N/mm <sup>2</sup>	120 N/mm <sup>2</sup>	3.8
Shear force		270 N/mm <sup>2</sup>	0.4 N/mm <sup>2</sup>	660
Bending moment		460 N/mm <sup>2</sup>	16 N/mm <sup>2</sup>	24
Buckling	Top	540 kN	76 kN	7.1
Falling	Foundation	13,000 tfm	3,200tfm	4.1

The failure criterion of tower was presumed to be axial force as shown in Table 1. The capacity corresponding to this criterion will be described as follows.

## (2) Capacity of Transmission Tower

The realistic capacity of the tower,  $f_C(X)$ , was evaluated by the following equation.

$$f(X) = \frac{1}{\sqrt{2\pi} \cdot \beta_C \cdot X} \cdot \exp \left\{ -\frac{1}{2} \left( \frac{\ln X - \ln M_C}{\beta_C} \right)^2 \right\} \quad (1)$$

where  $M_C$  : Median of structural strength (stress).

$\beta_C$  : Logarithmic standard deviation of structural strength (stress).

$X$  : Structural strength.

The structural strength was established based on the variation in the material strength of the steel. The logarithmic standard deviation of structural strength of the transmission tower was estimated from the data of past material tests [7] to be 0.1.

### 2.2.3 Response evaluation

The realistic response of the tower,  $f_R(\alpha, X)$ , was evaluated by the following equation.

$$f_R(\alpha, X) = \frac{1}{\sqrt{2\pi} \cdot \beta_R \cdot X} \cdot \exp \left[ -\frac{1}{2} \left\{ \frac{\ln X - \ln \left( M_R \cdot f \cdot \frac{\alpha}{\alpha_D} \right)}{\beta_R} \right\}^2 \right] \quad (2)$$

where  $M_R$  : Median of structural strength (stress caused by design ground motion).

$\beta_R$  : Logarithmic standard deviation of response of structural strength.

$f$  : Factor to consider response amplification due to ground characteristics.

$\alpha_D$  : Maximum acceleration of design ground motion.

$\alpha$  : Maximum acceleration of ground motion.

$X$  : Parameter of structural strength.

The design response of the tower was calculated by dynamic analysis using the design ground motion (maximum acceleration  $\alpha_D$  : 286 Gal) and the FEM model of the tower. For the response factor to consider the ground characteristics of the tower, the median and the logarithmic standard deviation of response factor were estimated to be 1.27 and 0.44, respectively. The realistic response of the tower assessed in a probabilistic way is shown in Table 2.

**Table 2. Realistic response of tower**

Evaluation Part	Response (N/mm <sup>2</sup> )	Response factor	
		Median	Logarithmic standard deviation
Foundation	120	1.27	0.44

#### 2.2.4 Evaluation Results of Functional Failure Probability

If the realistic response of the tower,  $f_R(\alpha, X)$ , is independent of the realistic capacity of the tower,  $f_C(X)$ , the functional failure probability,  $P_{CF}(\alpha)$ , can be expressed as follows:

$$P(\alpha) = \int_0^{\infty} f_R(\alpha, X) \left\{ \int_0^X f_C(X) dX \right\} dX \quad (3)$$

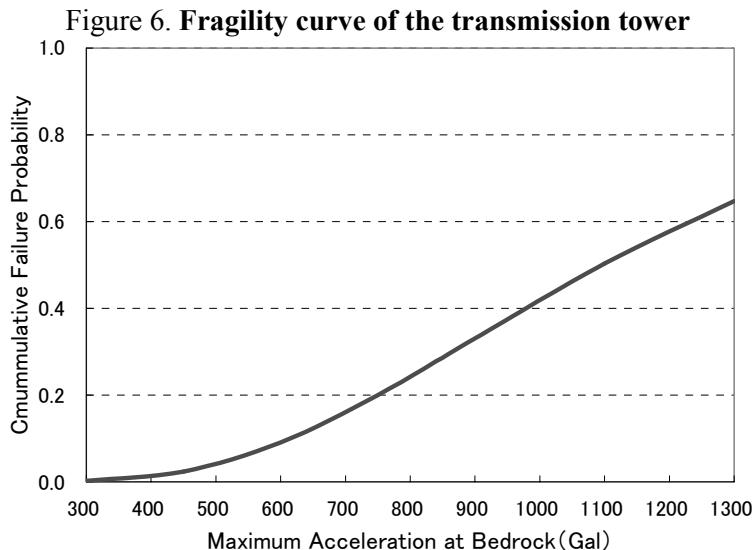
where  $f_R(\alpha, X)$  : Probability density function of the realistic response.

$f_C(X)$  : Probability density function of realistic capacity.

$\alpha$  : Maximum acceleration of ground motion at the bedrock.

$X$  : Variable of response or other parameter.

The fragility curve of the tower is shown in Figure 6.



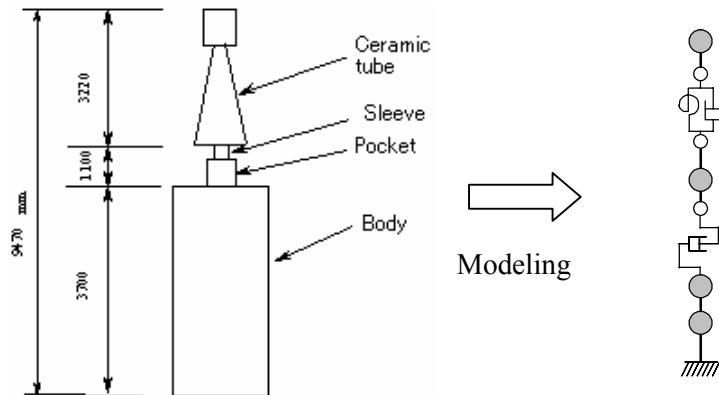
### 2.3 Seismic fragility evaluation of transmission equipment in the switchyard

#### 2.3.1 Equipment

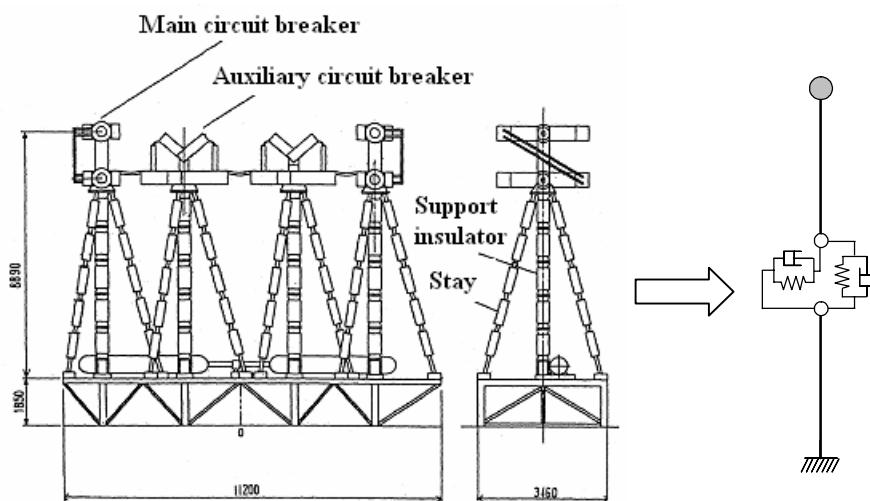
The emergency transformer with ceramic tube consists of three ceramic tubes and body. The total weight of the ceramic tubes and body is about 80 tons. The air blast circuit breaker consists of support insulators,

stay insulators, circuit breaker and foundation. The total weight is about 10 tons. The vibration analysis models of the emergency transformer with ceramic tube and the air blast circuit breaker are shown in Figures 7 and 8.

**Figure 7. Emergency transformer structure and vibration model**



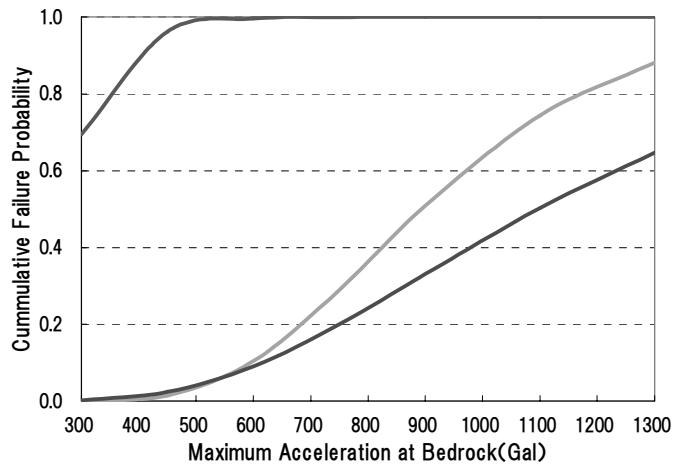
**Figure 8. Circuit breaker structure and vibration model**



### 2.3.2 Seismic fragility evaluation

The functional failure probability of the emergency transformer with ceramic tube and the air blast circuit breaker was evaluated using a seismic capacity and realistic response. As for the capacity and the response of this equipment, the data evaluated for standard type equipment in typical BWR plants were used. The fragility curves of the emergency transformer with ceramic tube and the air blast circuit breaker are shown in Figure 9 together with the fragility curve of the tower. The functional failure probability of the tower is relatively small compared to the functional failure probability of the transmission equipment in the switchyard, so the former probability is thought to have a small effect on the occurrence probability of LOSP. Therefore, the occurrence probability of LOSP will be determined by the failure of the transmission equipment in the switchyard. The transmission equipment in the switchyard are seismically fragile, and aseismic countermeasures are difficult to take, so application of the base isolation to the transmission equipment in the switchyard is considered an effective countermeasure to reduce the seismic risk associated with LOSP.

Figure 9. Fragility curves of the transmission equipments and transmission tower



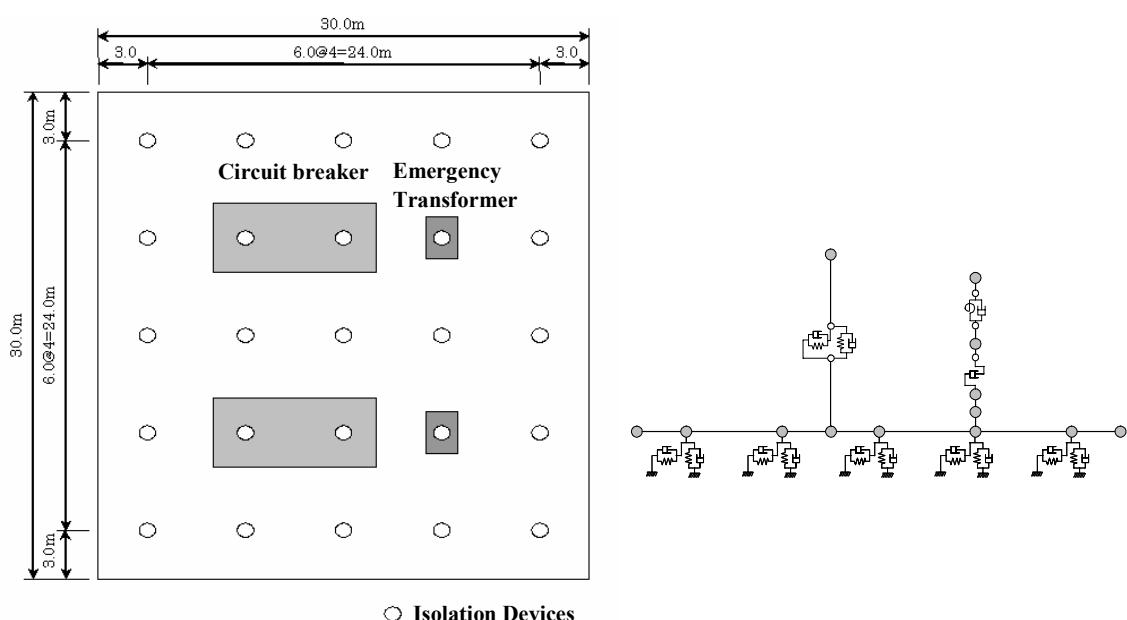
### 3. Influence evaluation of seismic base isolation on seismic risk

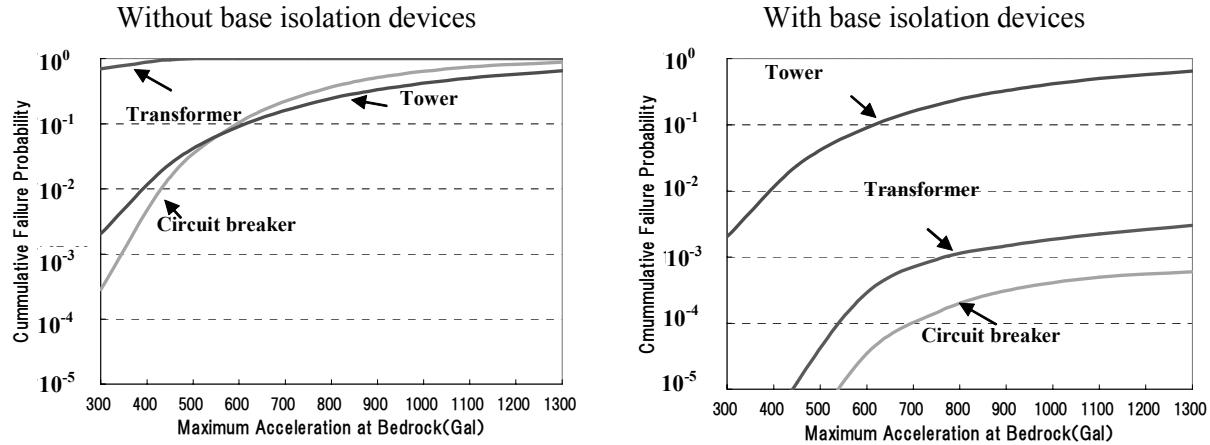
In order to reduce the functional failure probability of the electrical power system, the seismic base isolation was applied to the emergency transformer with ceramic tube and the air blast circuit breaker in the switchyard. The functional failure probability of the transmission equipment was calculated by using the characteristic of isolation device for the parameter, and effectiveness of the seismic equipment base isolation was confirmed.

#### 3.1 Evaluation conditions

The emergency transformer with ceramic tube and the air blast circuit breaker were seismically isolated by the high damping rubber bearing (HDRB). Assuming those two emergency transformers with ceramic tube and two air blast circuit breakers were on a concrete slab. The HDRB isolators were installed under the concrete slab. The seismically isolated structure is shown in Figure 10 and its analysis model is shown in Figure 11. The isolation devices are designed corresponding to the natural period of 2 to 3 seconds.

Figure 10. Layout of isolation devices and vibration model



**Figure 11. Fragility curves of the transmission equipments and transmission tower**

### 3.2 Seismic fragility evaluation

The functional failure probability of the emergency transformer with ceramic tube with base isolation devices and the air blast circuit breaker with base isolation devices was evaluated using a seismic capacity and realistic response. As for the capacity of these equipments with base isolation devices, the same data as equipment without base isolation in the section 2.3.2 was used. The realistic response was calculated by dynamic analysis using the time history of design ground motion and the multi-lumped mass vibration model with base isolation devices. The fragility curves of the emergency transformer with ceramic tube with base isolation devices and the air blast circuit breaker with base isolation devices are shown in Figures 10 and 11, respectively. And, these fragility curves are also shown together with the fragility curve of the transmission tower in Figure 11. The emergency transformer with ceramic tube is seismically fragile; its functional failure probability was calculated to be about 0.9 under range over 450 Gal. By applying the seismic base isolation however, the functional failure probability was greatly reduced. The absolute value of the functional failure probability of the air blast circuit breaker is small compared to the emergency transformer with ceramic tube; however, the reduction in the functional failure probability by introducing the seismic base isolation is large.

### 4. Conclusion

A methodology for estimating the functional failure probability of the electrical power system was proposed. In this methodology, the functional failure probability of the transmission tower was calculated and compared with the functional failure probability of electrical equipment in the switchyard. Based on this, the influence on seismic fragility of the electrical power system of seismic fragility of the transmission tower was evaluated. As the result, the failure probability of the transmission tower under the seismic condition was relatively low compared with the electrical transmission equipment in the switchyard. Therefore, the occurrence probability of LOSP will be determined by the failure of the transmission equipment in the switchyard. By applying the seismic base isolation to these equipments, the functional failure probability was greatly reduced. As a result, the prospect that the functional failure probability of the electrical power system was able to be decreased by applying the seismic base isolation to the electrical transmission equipment in the switchyard was obtained.

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*Special Session: Standards*

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## Development of American National Standard on External Event PRA Methodology

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### 1. Introduction

During the last ten years, the U.S. Nuclear Regulatory Commission and the U.S. nuclear utilities have been developing methods and requirements for risk-informed applications making use of probabilistic risk assessments (PRA) of nuclear power plants. Early in this process, it became clear that the existing PRAs were done with different objectives and methodologies by different analysts. For uniformity and consistency in future risk-informed applications, industry consensus standards on probabilistic risk assessments were deemed to be essential.

Currently, the following standards have been published or under preparation:

- ASME RA-S-2002: “Standard for Probabilistic Risk Assessment for Nuclear Power Plant Applications”, Addendum C, March 2005.
- ANSI/ANS-58.21-2003 “External-Events PRA Methodology” March 2003.
- ANS-58.22 “Low Power and Shutdown PRA Standard”.
- ANS – 58.23 “Fire PRA Methodology Standard”.
- ANS Level 2 and Level 3 PRA Standards.

The ASME Standard specifies the requirements for performing PRA for internal events under full power conditions. ANS 58.21 specifies the requirements for conducting PRA of external events under full power conditions. These standards have been published, whereas the other standards are under preparation.

ANS 58.21 was prepared by a working group comprised of the following individuals:

- Dr. R.J. Budnitz, Lawrence Livermore National Laboratory.
- Dr. N.C. Chokshi, U.S. Nuclear Regulatory Commission.
- W. Henries, Maine Yankee Atomic Power Company.
- Dr. M.K. Ravindra, ABS Consulting, (Current Chair of Working Group).
- Dr. J.D. Stevenson, J.D. Stevenson Consultants.
- T. Yee, Southern California Edison Company.

The development of this Standard was guided and approved by the Risk Informed Standards Committee (RISC) of American Nuclear Society. This committee has presently 22 members drawn from different sectors of the nuclear industry: utilities, reactor vendors, universities, research organizations, architect-engineers, consultants and the U.S. Nuclear Regulatory Commission. In addition, the Working Group had sent early drafts of the Standard to a select group of peers in different elements of the external event PRA; this peer group included industry experts like Professor Allin Cornell, Dr. Robert Kennedy, and Mr. David Moore. The final requirements in the Standard hence represent the consensus of the industry.

## 2. Scope and format

### 2.1 Scope

The scope of a PRA covered by ANS 58.21 is limited to analyzing accident sequences initiated by external events that might occur while a nuclear power plant is at nominal full power. It is further limited to requirements for (a) a Level 1 analysis of the core damage frequency (CDF) and (b) a limited Level 2 analysis sufficient to evaluate the large early release frequency (LERF). The scope of a seismic margin assessment (SMA) covered by this Standard is limited to analyzing nuclear power plant seismic capacities according to either the so-called Electric Power Research Institute method (EPRI, 1991) or the so-called U.S. Nuclear Regulatory Commission method (Budnitz, et al 1985).

External events covered within this Standard's scope include both natural external events (e.g., earthquakes, high winds, and external flooding) and human-made external events (e.g., accidental airplane crashes, explosions at nearby industrial facilities, and impacts from nearby transportation activities).

This Standard along with ASME standard and the Fire PRA methodology standard cover all potential accident initiators arising at nominal full power conditions except for human-induced security threats (e.g., sabotage).

The scope of the external events Standard includes not only traditional PRA analyses, which are intended to be realistic, but also screening analyses and demonstrably conservative or bounding approaches that use aspects of PRA methodology. Specifically, the scope of this Standard includes the widely used SMA methodology. SMA methods employ many of the same tools as a seismic PRA and could be used, as appropriate, for risk-informed applications.

As mentioned above, the requirements cover a limited Level 2 analysis sufficient to evaluate the large early release frequency (LERF). In analyzing external events that have the potential to impede effective emergency evacuation, the focus should be to examine whether any accident sequences that are not in the LERF category in the internal-events PRA model need to be included in that category for the particular external event being analyzed.

Although the Standard is written based mainly upon PRA and SMA methodologies and applications that have evaluated U.S. light-water nuclear power reactors (LWR) that are in commercial service, it is applicable with appropriate adaptations, to similar LWRs in the design and construction phase. Similarly, the Standard could be applied with appropriate adaptations, to other types of nuclear power reactors.

### 2.2 Format

The Standard is written following the format first used in the ASME-RA-S-2002. The standard is not written to support any specific application but it concerned only with the capability of a PRA to support an application. ASME has defined three capability levels (called "Capability Categories" I, II and III). For each technical area, three different supporting requirements are written to cover the three different capability categories. Table 1.3-1 of ASME Standard describes the basis that was used to differentiate among the three capability categories using one or more of the following three criteria: "scope and level of detail," "plant-specificity," or "realism." The goal is not to assign a single capability category to the entire PRA. Rather, different elements of a PRA can meet the supporting requirements in one of the three capability categories. For a specific risk informed application, only a few requirements may have to be met at certain capability category levels.

For each technical element of either a seismic PRA or a SMA, the Standard includes both high-level requirements and supporting requirements. The high-level requirements are a set of requirements that encompass beneath them all of the supporting technical requirements. The high-level requirements are general in their language acknowledging the diversity of approaches used in the existing PRAs. For each

high-level requirement, there are a number of supporting requirements focusing on the different aspects. The supporting requirement for each aspect varies with the capability category.

In conformance with the ASME standard, the supporting requirements are phrased in action-verb form. A unique feature of ANS 58.21 Standard is that it provides a commentary for each supporting requirement. The purpose of the commentary is to elaborate on the supporting requirement, point out acceptable methods and data, and provide references in the open literature.

The major elements of a seismic PRA are:

- Seismic Hazard Analysis.
- Systems Analysis including Quantification.
- Seismic Fragility Evaluation.

In the following sections, the high-level requirements and examples of supporting requirements for each of the above elements are given. The high-level and supporting requirements for SMA closely follow the EPRI guidance document (EPRI 1991) and are not discussed herein.

### **3. Seismic hazard analysis**

#### ***3.1 High-level requirements***

- A – Scope (HLR-HA-A): The frequency of earthquakes at the site SHALL be based on a site-specific probabilistic seismic hazard analysis (PSHA) (existing or new) that reflects the composite distribution of the informed technical community. The level of analysis SHALL be determined based on the intended application and on site-specific complexity.
- B – Data collection (HLR-HA-B): To provide inputs to the PSHA, a comprehensive up-to-date data base including: geological, seismological, and geophysical data; local site topography; and surficial geologic and geotechnical site properties, SHALL be compiled. A catalog of historical, instrumental and paleoseismicity information SHALL also be compiled.
- C – Seismic sources and source characterization (HLR-HA-C): To account for the frequency of occurrence of earthquakes in the site region, the PSHA SHALL consider all credible sources of potentially damaging earthquakes. Both the aleatory and epistemic uncertainties SHALL be considered in characterizing the seismic sources.
- D – Ground motion characterization (HLR-HA-D): The PSHA SHALL account for all credible mechanisms influencing estimates of vibratory ground motion that can occur at a site given the occurrence of an earthquake of a certain magnitude at a certain location. Both the aleatory and epistemic uncertainties SHALL be considered in characterizing the ground motion propagation.
- E – Local site effects (HLR-HA-E): The PSHA SHALL account for the effects of local site response.
- H – Use of existing studies (HLR-HA-H): When use is made of an existing study for PSHA purposes, it SHALL be confirmed that the basic data and interpretations are still valid in light of current information, the study meets the requirements outlined in A through G above, and the study is suitable for the intended application.
- I – Other seismic hazards (HLR-HA-I): A screening analysis SHALL be performed to assess whether, in addition to the vibratory ground motion, other seismic hazards, such as fault displacement, landslide, soil liquefaction, or soil settlement need to be included in the SPRA for the specific application. If so, the SPRA SHALL address the effect of these hazards through assessment of the frequency of hazard occurrence and/or the magnitude of hazard consequences.

- J – Documentation (HLR-HA-J): The PSHA SHALL be documented in a manner that facilitates applying the PRA and updating it, and that enables peer review

### 3.2 Example supporting requirements

<u><b>SEISMIC HAZARD ANALYSIS HIGH-LEVEL REQUIREMENT F: AGGREGATION AND QUANTIFICATION</b></u>			
Index No. HA-F	CAPABILITY CATEGORY I	CAPABILITY CATEGORY II	CAPABILITY CATEGORY III
HA-F1	In the final quantification of the seismic hazard, INCLUDE mean estimates. ADDRESS how this accounts for uncertainties.	In the final quantification of the seismic hazard, INCLUDE and DISPLAY the propagation of both aleatory and epistemic uncertainties.	
<b>NOTE HA-F1:</b> The seismic hazard quantification involves the combination of seismic source and ground motion inputs to compute the frequency of exceedance of ground motions at a site (i.e., the seismic hazard). Thus, the principal result of the PSHA is a set of seismic hazard curves that quantify the aleatory and epistemic uncertainties in the site hazard. This is typically presented in terms of a set of fractile seismic hazard curves, defined at specified fractile levels, and the mean hazard. Two acceptable approaches have been used to propagate epistemic uncertainties: logic tree enumeration and Monte Carlo simulation (EPRI, 1989 and Bernreuter et.al., 1989). For Capability Category I applications, use of a single mean curve may be appropriate.			
HA-F2	In the PSHA, INCLUDE appropriate sensitivity studies and intermediate results to identify factors that are important to the site hazard and that make the analysis traceable.		

## 4. Systems analysis including quantification

### 4.1 High-level requirements

- A – Completeness (HLR-SA-A): The seismic-PRA systems models SHALL include all important seismic-caused initiating events that can lead to core damage or large early release, and SHALL include all other important failures that can contribute significantly to CDF or LERF, including seismic-induced SSC failures, non-seismic-induced unavailabilities, and human errors.
- B – Adaptations Based on the Internal-Events Pra Systems Model (HLR-SA-B): The seismic-PRA systems model SHALL be adapted to incorporate seismic-analysis aspects that are different from corresponding aspects found in the full-power, internal-events PRA systems model.
- C – Plant Fidelity (HLR-SA-C): The seismic-PRA systems models SHALL reflect the as-built and as-operated plant being analyzed.
- D – Seismic Equipment List (HLR-SA-D): The list of SSCs selected for seismic-fragility analysis SHALL include all SSCs that participate in accident sequences included in the seismic-PRA systems model.
- E – Integration and Quantification (HLR-SA-E): The analysis to quantify CDF and LERF frequencies SHALL appropriately INTEGRATE the seismic hazard, the seismic fragilities, and the systems-analysis aspects.
- F – Documentation (HLR-SA-F): The seismic-PRA analysis SHALL be documented in a manner that facilitates applying the PRA and updating it, and that enables peer review.

## 4.2 Example supporting requirements

SA-E2	<p>In quantifying CDF and LERF frequencies, PERFORM the quantification on a cut-set-by-cut-set or accident-sequence-by-accident-sequence basis (or for defined groups of these), as well as on a comprehensive/integrated basis.</p> <p>Broad groupings MAY be used.</p>	<p>In quantifying CDF and LERF frequencies, PERFORM the quantification on a cut-set-by-cut-set or accident-sequence-by-accident-sequence basis (or for defined groups of these), as well as on a comprehensive/integrated basis.</p>
<b>NOTE SA-E2:</b> The intent of this requirement is to ensure that key information about each accident sequence (or cut set) is retained, rather than simply "lost" in the production of overall integrated values for CDF and LERF. Of course, it is common to group cut-sets of accident sequences when they are so similar that phenomenologically they cannot be distinguished very well; such grouping is entirely acceptable, if its basis is defined.		
SA-E3	<p>In the analysis, USE the quantification process to ensure that any screening of SSCs does not affect the results, taking into account the various uncertainties.</p>	<p><b>NOTE SA-E3:</b> SSC screening – the elimination from the model of SSCs – is done throughout the process of performing any PRA. A defined set of criteria must be developed and used to ensure that this screening does not eliminate elements of the model that should have been retained. The intent of this requirement is to ensure that the quantification process is used to check that the screening has not erroneously eliminated important SSCs. It is recognized that this type of work is an iterative process, in which approximate interim quantifications are done during which the screening decisions are checked, and only then is a final quantification done. There are many different approaches in current use among seismic-PRA analysts to accomplish this step. (Bohn and Lambright, 1988) contains a useful discussion on this aspect.</p>
<b>NOTE SA-E4:</b> As discussed earlier, treating earthquake-specific correlations and dependencies properly is vital to achieving a successful seismic-PRA. This requirement is intended to ensure that this issue is covered. A discussion of this type of correlation/dependency analysis is found in (Bohn and Lambright, 1988). See (REQ. SA-B3) where the requirement to deal with dependencies and correlations in initial screening is covered, and (REQ. SA-E7) where appropriate sensitivity analyses are required to explore these issues.		
SA-E4	<p>In the integration/quantification analysis, ACCOUNT for all significant dependencies and correlations that affect the results.</p> <p>The analysis MAY use generic dependency and correlation values if justified.</p>	<p>In the integration/quantification analysis, ACCOUNT for all significant dependencies and correlations that affect the results.</p> <p>USE plant-specific dependency and correlation values throughout.</p>

## 5. Seismic fragility evaluation

### 5.1 High-level requirements

- A – Realism (HLR-FR-A): The seismic fragility evaluation SHALL be performed to estimate plant-specific, realistic seismic fragilities of structures, systems and components whose failure may contribute to core damage and/or large early release.
- B – Screening (HLR-FR-B): If screening of high-seismic-capacity components is performed, the basis for the screening SHALL be fully described.
- C – Response (HLR-FR-C): The seismic fragility evaluation SHALL be based on realistic seismic response that the SSCs experience at their failure levels. Depending on the site conditions and response analysis methods used in the plant design, realistic seismic response MAY be obtained by an appropriate combination of scaling, new analysis and new structural models
- D – Failure Modes (HLR-FR-D): The seismic fragility evaluation SHALL be performed for critical failure modes of structures, systems and components such as structural failure modes and functional failure modes identified through the review of plant design documents, supplemented as needed by earthquake experience data, fragility test data, generic qualification test data, and a walkdown.
- E – Walkdown (HLR-FR-E): The seismic fragility evaluation SHALL incorporate the findings of a detailed walkdown of the plant focusing on the anchorage, lateral seismic support, and potential systems interactions
- F – Data Sources (HLR-FR-F): The calculation of seismic fragility parameters such as median capacity and variabilities SHALL be based on plant specific data supplemented as needed by earthquake experience data, fragility test data and generic qualification test data. Use of such generic data SHALL be justified.

- G – Documentation (HLR-FR-G): The seismic fragility evaluation SHALL be documented in a manner that facilitates applying the PRA and updating it, and that enables peer review

## 5.2 Example supporting requirements

<u>SEISMIC FRAGILITY EVALUATION HIGH LEVEL REQUIREMENT D: FAILURE MODES</u>			
Index No. FR-D	CAPABILITY CATEGORY I	CAPABILITY CATEGORY II	CAPABILITY CATEGORY III
FR-D1	IDENTIFY realistic failure modes of structures and equipment that interfere with the operability of equipment during or after the earthquake through a review of the plant design documents and the walkdown.		
	<b>NOTE FR-D1:</b> Note that sometimes failure modes such as drift and yielding MAY be more relevant for the functionality of attached equipment than gross structural failures (i.e., partial collapse or complete collapse).		
FR-D2	CONSIDER all relevant failure modes of structures (e.g., sliding, overturning, yielding, and excessive drift), equipment (e.g., anchorage failure, impact with adjacent equipment or structures, bracing failure, and functional failure) and soil (i.e., liquefaction, slope instability, excessive differential settlement), and EVALUATE fragilities for critical failure modes.		
	<b>NOTE FR-D2:</b> Published references and past seismic PRAs MAY be used as guidance. Examples include (Reed and Kennedy, 1994); (EPRI, 1991); (PG&E, 1988).		

## 6. NRC/industry review and revision of the standard

After the publication of the Standard, the U.S. Nuclear Regulatory Commission reviewed it for possible endorsement for use in risk-informed applications. A draft Regulatory Guide 1.200 Appendix C was issued with some important comments and several editorial comments. These have been resolved and a revision to the Standard is currently underway. At the same time, the RISC was reviewing the suggested revisions to the Standard. Some members expressed their concern about two issues: use of existing seismic hazard studies and the excessive burden of meeting the Standard.

On the issue of using the existing seismic hazard study results, the high-level requirement (HLR-HA-H) permits such a use as long as the basic data and interpretations are shown to be still valid. However, the concern is that a new hazard study may be too expensive and the results are quite uncertain. After much debate, the following note to the supporting requirement was agreed:

“When using the Lawrence Livermore National Laboratory/U.S. Nuclear Regulatory Commission or Electric Power Research Institute hazard studies, or another study done to a comparable technical level, the intent of this requirement is not to repeat the entire hazard exercise or calculations, unless new information and interpretations prepared to a comparable technical level that affect the site have been established and affect the usefulness of the seismic PRA for the intended application. Depending on the application, sensitivity studies, modest extensions of the existing analysis, or approximate estimates of the differences between using an existing hazard study and applying the newer one may be sufficient. Additionally, an educated assessment may be sufficient to demonstrate that the impact on the application of information or data that is less extensive than a new hazard study is not significant.”

On the issue of excessive burden of complying with the Standard, it was pointed out that most existing seismic PRAs done to meet the IPEEE requirements would not meet some of the supporting requirements of the Standard. The major goal of IPEEE was to qualitatively search of severe accident vulnerabilities in the plant. The goal of the Standard is to facilitate the review of a risk informed application using numerical risk metrics of CDF and LERF. Contrary to the claim that a seismic PRA that meets the Standard cannot be

performed, the working group showed that a number of existing seismic PRAs do, for the most part, meet Capability Category II requirements. In order to resolve this issue, the industry is launching on a pilot study which would serve the following purposes:

1. Assessment of the ability of the Standard to technically support selected applications.
2. Assessment of the overall practicality and cost of implementing the Standard for those applications.
3. Assessment of “good industry practice” versus the requirements of Capability Category II.

In the last three years, the seismic PRAs of Krško Nuclear Power Plant in Slovenia and Columbia Generating Station in the U.S. have been reviewed for conformance with the Standard. Krško seismic PRA was updated to meet the Capability Category II requirements in all aspects. Currently, the seismic PRA of Beznau Nuclear Power Plant in Switzerland is being updated to meet the Capability Category II requirements.

## **7. Acknowledgements**

The author would like to acknowledge the contributions of Dr. Robert Budnitz and Dr. Nilesh Chokshi; Dr. Budnitz was the Chairman of the Working Group during the development of the published Standard. He and Dr. Chokshi have provided the much needed input in resolving the many thorny issues during the consensus process.

## **8. References**

- American Nuclear Society (2003) “External Events PRA Methodology – an American National Standard”, ANSI/ANS-58.21-2003.
- Budnitz, R.J. et al (1985) “An Approach to the Quantification of Seismic Margins in Nuclear Power Plants,” NUREG/CR-4334, Lawrence Livermore National Laboratory, Livermore, California.
- EPRI (1991) “A Methodology for Assessment of Nuclear Power Plant Seismic Margin,” EPRI-NP-6041-SL, Rev. 1, Electric Power Research Institute, Palo Alto, California.

## Outline of Seismic PSA Implementation Standards on the Atomic Energy Society of Japan

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### **1. Introduction**

In September 2004, the Atomic Energy Society of Japan has established 3 working groups (Seismic Hazard Evaluation, Building & Component Fragility Evaluation, Accident Sequence Evaluation) under the Seismic PSA working group in Technical Group on Nuclear Power Plant in Standards Committee with the purpose to develop its Seismic PSA implementation standards. The standards were approved by the Seismic PSA working group in December 2005 and by Technical Group on Nuclear Power Plant in June 2006. A plan is currently going on to pend upon a resolution by Standards Committee scheduled in October and to enter the public review phase.

Meanwhile, the Nuclear Safety Commission of Japan established a working group for reviewing seismic design guidelines under the Technical Group on Nuclear Safety Standards and Guidelines in July 2001 so that the project to revise Examination Guideline for Seismic Design could be started. After the deliberation of almost five years, the final draft of the Examination Guideline for Seismic Design (Draft Seismic Guideline) was compiled in the working group meeting in April 2006. After a period of public comment submission for two months, the Guideline was decided in September<sup>1)</sup>. In the Guidelines, the risk by the extension of the effect of the seismic ground motion that exceeds the design basis seismic ground motion Ss is defined as "residual risk", and the efforts to minimize "the residual risk" as low as practically achievable should be made. Also, the Commission published the Draft Interim Report for Safety Goals in December 2003<sup>2)</sup> and the performance goal in May 2006<sup>3)</sup> as a part of the projects related to "the residual risk".

In this report, the process of the formulation activities as well as the formulation policies / composition related to the above-mentioned Seismic PSA implementation standards are shown. In addition, the main characteristics of specific items consisting the implementation standards are described.

### **2. Process of the formulation activities and the formulation policies / composition related to Seismic PSA implementation standards**

#### **(I) Process of seismic PSA working group activities**

The main Process of Seismic PSA working group activities are as below:

- 1) Investigation and review of the domestic and international seismic PSA methodologies (International: IAEA, OECD/NEA, NRC in the United States, etc. Domestic: previous Japan Atomic Energy Research Institute, Japan Nuclear Energy Safety Organization, industries, etc.) were conducted.
- 2) "External Events PRA methodology" standard developed by American Nuclear Society (ANS) were reviewed.
- 3) Based on above 1) and 2), "formulation policies for implementation standards" were developed, a table of contents composition was determined and adjustment was achieved for the items common to all three working groups.

## **(2) Formulation policies for implementation standards**

Main implementation policies are as below:

- 1) From the standpoint of seismic safety evaluation for the nuclear power plant, the review shall be conducted using a wide variety of proven methods, where evaluation of applicability for safety goal / performance goal, choice of AM counteractions against earthquake and confirmation of their effectiveness, refinement of seismic design method and confirmation of its efficacy etc. shall be included in the scope of operation so that the methods can be chosen which enable consideration for specific levels suitable for operation purpose.
- 2) To clarify the specific implementation procedures, standards shall be formulated in the form of specification. However, adequate consideration shall be given to reflect points of importance such as prevention of omitting high-level requirements and the confirmation of Japan-specific requirements.
- 3) The scope shall be the evaluation of core damage frequency (including preceding damage of containment vessel) in LWR operation and the identification of scenarios in the case of loss of containment function at the time of earthquake. By this identification of scenarios, it is possible to evaluate incidence of earthquake-related loss of containment function, source term, individual risks and others thereafter, using the level 2 and level 3 PSA implementation standards for internal events.
- 4) Upon listing as much as possible a wide variety of accident scenarios under seismic ground motion, screening shall be conducted on practical basis and scenarios for quantitative evaluation shall be chosen.
- 5) Uncertainty factors in each stage of the Seismic PSA shall be categorized into randomness uncertain factors related to natural phenomena and the epistemic uncertain factors related to insufficiency of knowledge and information, then the factors shall be clearly described and examples for each factor shall be shown as much as possible.
- 6) Many evaluation examples shall be described with the purpose to promote understanding and effective operation of implementation standards.

## **(3) Composition of implementation standards**

The implementation standards are consisted from following 8 chapters.

- Chapter 1. Scope of application.
- Chapter 2. Definition of technical terms.
- Chapter 3. Evaluation process.
- Chapter 4. Collection / Analysis of plant information and general analysis for accident scenarios.
- Chapter 5. Seismic Hazard Evaluation.
- Chapter 6. Building & Component Fragility Evaluation.
- Chapter 7. Accident Sequence Evaluation.
- Chapter 8. Documentation.

### **3. Outline of each chapter**

#### **3.1 Scope of application**

Scope of application for Chapter 1 is as below. The standards focus on the earthquake-related accident sequences that lead to serious core damage among the Probabilistic Safety Assessments (PSA) of nuclear power plant in operation states as their scope. They prescribe the requirements for the method to perform the level 1 PSA to obtain their frequencies and the method to identify the earthquake-related accident sequences that lead to containment damage as well as specific methods to satisfy these requirements based on the implementation procedure. However, the fire-, flood- and tsunami-related events that may occur as a result of an earthquake shall be excluded from the scope of application of these standards.

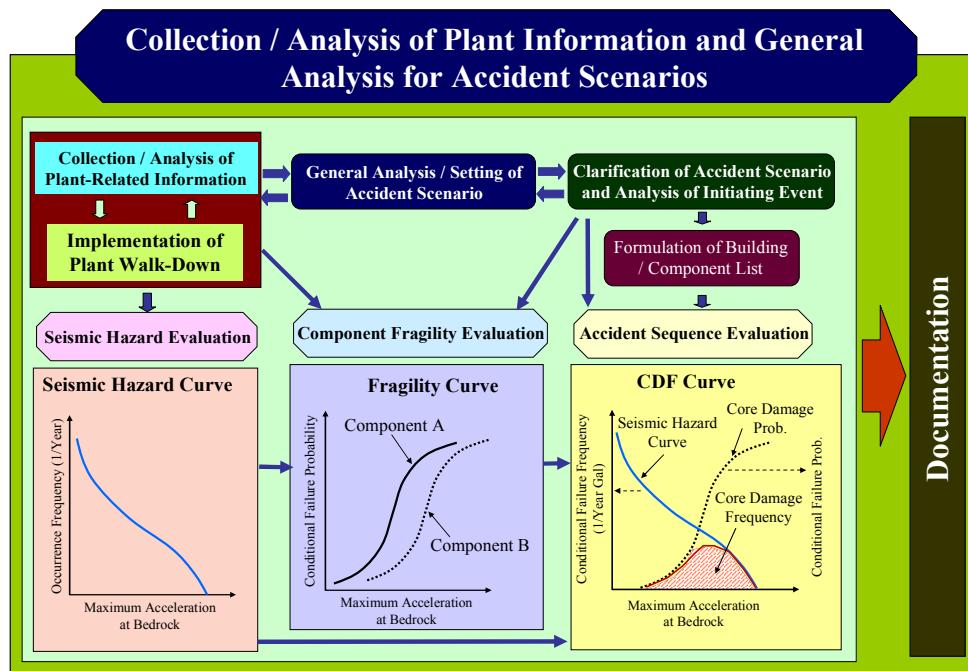
### 3.2 Definition of technical terms

The technical terms listed in Chapter 2 are those with high frequency of appearance or those with possibility of multiple interpretations.

### 3.3 Evaluation process

Chapter 3 “Evaluation process” is dedicated to take general view of the composition of the current Seismic PSA implementation standards as well as to facilitate understanding the mutual correlation among chapters from Chapter 4 to Chapter 8. The procedure of Seismic PSA is shown in Figure 1.

Figure1. Procedure of seismic PSA



In the section of “Collection / Analysis of plant information and general analysis for accident scenarios”, while collecting / analyzing information necessary for Seismic PSA and conducting plant walk-down, a wide variety of earthquake-specific accident scenarios are defined, initiating events that trigger core damage accidents are analyzed and a component list is formulated as a preparation of quantitative evaluation of core damage (defined as the status where maximum fuel cladding temperature reaches 1200°C), which is the most serious type of accident in nuclear power plant accidents.

In the section of “Seismic Hazard Evaluation”, by generating a model of location / size / occurrence frequency of earthquakes that may occur in vicinity of the site in the future, the exceedance frequency of seismic ground motion caused by the earthquakes at each strength level are obtained. In the section of “Building & Component Fragility Evaluation”, by using their realistic response and capacity, accumulated failure probability for each strength level of seismic ground motion are obtained. In the section of “Accident Sequence Evaluation”, accident scenarios that lead to core damage are analyzed, and, by using the seismic hazard evaluation result, fragility evaluation result and plant system information, occurrence frequency of core damage accident sequence is obtained. In these evaluations, while reflecting domestic seismic design information, adequate considerations are given to uncertainty related to evaluation models and database.

In “Documentation”, the ground / judgment of adopting a specific model or data in the process of evaluation is stated. As a consequence, uncertain factors that seriously influence to core damage, accident scenarios, mitigation systems and components are identified and clearly described. From the standpoint of assuring clarity of explanation and transparency, these elements are summarized in the report.

### ***3.4 Collection / analysis of plant information and general analysis for accident scenarios***

#### ***(1) Composition***

“Collection / Analysis of plant information and general analysis for accident scenarios” (Chapter 4) is described in the following 6 sections:

- 4.1 Analysis process.
- 4.2 Collection / Analysis of plant-related information.
- 4.3 Implementation of plant walk-down.
- 4.4 General analysis / setting of accident scenario.
- 4.5 Clarification of accident scenario and analysis of initiating event.
- 4.6 Formulation of building / component list.

Collection / Analysis of plant information and general analysis for accident scenarios are extremely important processes that should be conducted in order to prevent omission of any earthquake-specific accident scenario as well as to enable effective implementation of various evaluations such as the seismic hazard evaluation, fragility evaluation and accident sequence evaluation. These processes should take place prior to the above-mentioned evaluations.

#### ***(2) Main characteristics***

Main characteristics are as below:

- 1) Implementation of plant walk-down is introduced in order to supplement insufficient desk information and to confirm consistency of analysis models with the actual site situation.
- 2) Implementation of qualitative and quantitative screening is introduced based on setting / analysis of wide variety of accident scenario in order to prevent missing earthquake-specific accident scenarios.
- 3) Scenarios in which event process leading to core damage is unknown or in which screening is infeasible due to insufficient evaluation technique are recorded in a report to ensure transparency and clarity of explanation.

### ***3.5 Seismic hazard evaluation***

#### ***(1) Composition***

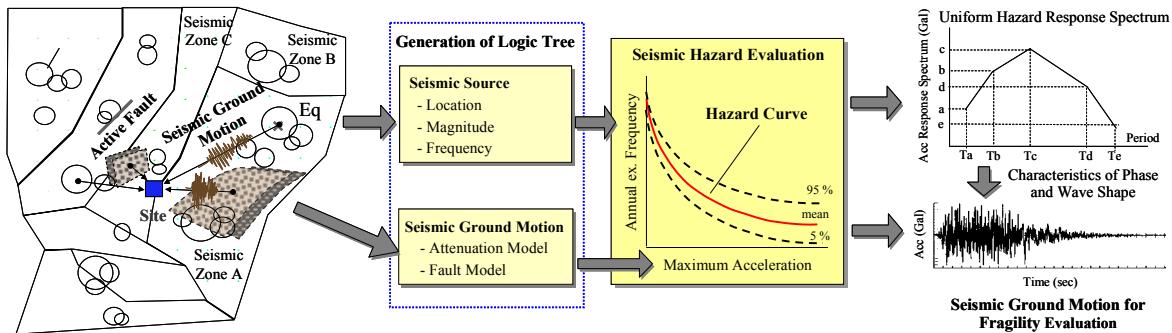
“Seismic Hazard Evaluation” (Chapter 5) is described in the following 7 sections:

- 5.1 Process of Seismic Hazard Evaluation.
- 5.2 Treatment of uncertainty in vertical motion and Seismic Hazard Evaluation.
- 5.3 Setting of seismic source model.
- 5.4 Setting of seismic ground motion propagation model.
- 5.5 Formation of logic tree.
- 5.6 Evaluation of seismic hazard.
- 5.7 Formation of seismic ground motion for fragility evaluation.

In “Process of Seismic Hazard Evaluation”, modeling of location / size / incidence of earthquakes that may occur in the vicinity of the site is conducted using active fault data and historical earthquake data as shown

in Figure 2. Then, the propagation of seismic ground motion caused by the earthquakes is evaluated based on distance attenuation model or fault model in order to obtain the relationship between the strength of seismic ground motion and the exceedance frequency / probability. The uncertainty in the modeling is treated by using a logic tree.

**Figure2. Procedure of seismic hazard evaluation**



## (2) Main characteristics

Main characteristics are as below:

- 1) Seismic source models are categorized into a specific seismic source model and an area seismic source model.
- 2) Either the distance attenuation model or the fault model can be used as a seismic ground motion propagation model. It is possible to set both horizontal seismic ground motion and vertical seismic ground motion.
- 3) Logarithmic-standard deviation to represent dispersion of seismic ground motion and the maximum value of seismic ground motion are set.
- 4) Uncertain factors related to the seismic source model and the seismic ground motion propagation model are categorized into randomness uncertain factors related to natural phenomena and the epistemic uncertain factors related to insufficiency of knowledge and information. Then, the uncertainty evaluation method is introduced by using a logic tree targeted for the latter factors (see Figure 3 for example).
- 5) A method is used where seismic ground motion for response analysis in the building / component fragility evaluation is evaluated based on the seismic uniform hazard spectrum.
- 6) To promote understanding of seismic hazard evaluation, quantitative evaluation results are indicated as much as possible.

**Figure3. Example of logic tree**

Seismic Source Characteristics				Seismic Ground Motion Characteristics			
Probability of Existence	Fault Length	Estimation Model of Magnitude	Source Location	Occurrence Frequency Model	Selection of Attenuation	Logarithmic Standard Deviation	Upper Limit of Acceleration
P1	x 1 km	Matuda Eq.	Uniform Distribution Shortest Distance	Poisson Process	Taitokui Eq.	$\sigma = 0.3$	Non
P2	x 2 km	Utsu Eq.			Annaka Eq.		Mean +2 $\sigma$
P3	x 3 km	Others	Non-Poisson Process	Fukusima and Tanala Eq.		$\sigma = 0.5$	Mean +3 $\sigma$

### 3.6 Building & component fragility evaluation

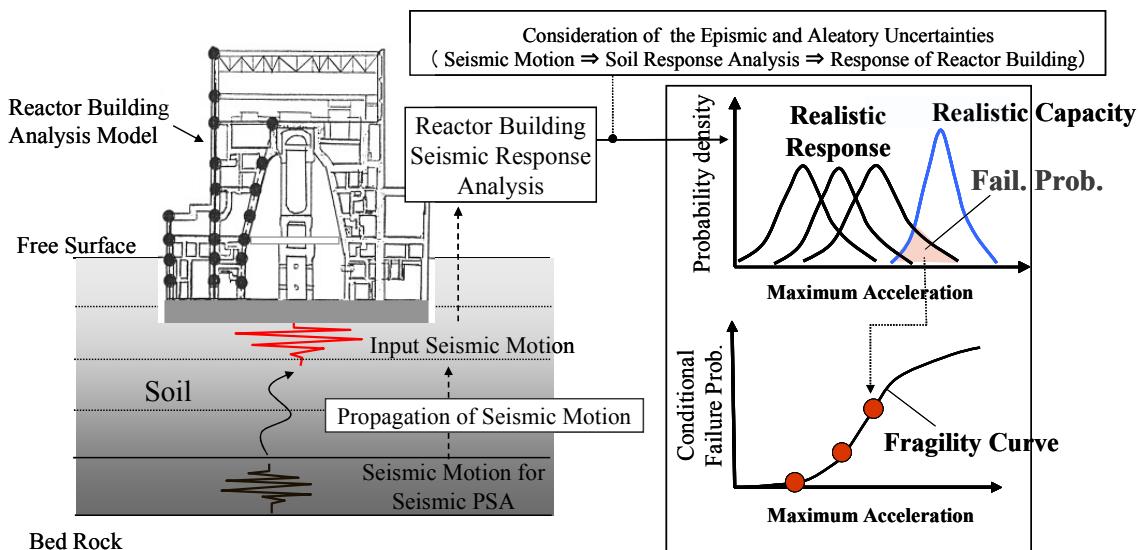
#### (1) Composition

“Fragility Evaluation” (Chapter 6) is described in the following 7 sections:

- 6.1 Fragility evaluation process.
- 6.2 Selection of evaluation target and failure mode.
- 6.3 Selection of evaluation method.
- 6.4 Evaluation of realistic capacity.
- 6.5 Evaluation of realistic response.
- 6.6 Evaluation of fragility curve.
- 6.7 Correlation of failures and fragility evaluation for seismic isolation type nuclear power plants.

In the fragility evaluation procedure, the relationship of the strength of seismic ground motion and the conditioned failure probability where realistic response of the buildings / components to the seismic ground motion should exceed their realistic capacity is obtained, which practically the accumulated curve of the conditioned failure probability (fragility curve) as shown in Figure 4. Both the realistic response and the realistic capacity are represented as median value / logarithmic-standard deviation by assuming that they are based on log-normal distribution. Uncertain factors related to response / capacity are categorized into the randomness uncertain factors related to natural phenomena and the epistemic uncertain factors related to insufficiency of knowledge and information. The logarithmic-standard deviation of the former factors is represented as the gradient of fragility curve, while that of the latter factors are represented as the curve width (such as 5, 50, 95%).

Figure4. Procedure of fragility evaluation



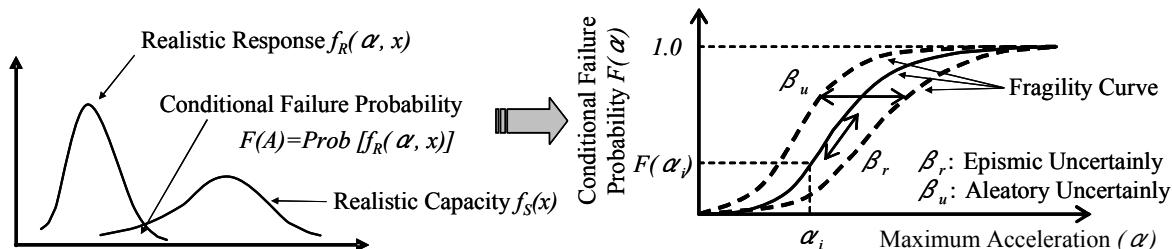
#### (2) Main characteristics

Main characteristics of the implementation standards are as below:

- 1) Methods of categorizing buildings / components in scope are indicated so as to enable accurate evaluation of core damage frequency based on the small number of buildings / components.
- 2) Any evaluation method can be chosen from “detailed evaluation”, “a moderate detailed evaluation” and “a simplified evaluation” depending on the objective of concern in the evaluation.

- 3) With the purpose to allow detailed capacity evaluation, methods of setting failure mode / failure section of the categorized buildings / components in detail are indicated.
- 4) Response evaluation method for vertical seismic ground motion is shown.
- 5) Correlation evaluation method is introduced in order to allow evaluation of simultaneous failure of multiple components under strong seismic motion.
- 6) Evaluation methods that can reflect scenarios that may lead to secondary core damage such as collapse of the slope in vicinity or damage of stacks are considered in addition to accident scenarios that directly lead to core damage.
- 7) Methods that can reflect aftershock, aging and seismic isolation structure are considered.
- 8) To promote understanding of the fragility evaluation, quantitative evaluation results are indicated as much as possible (Figure 5).

Figure 5. Illustration and concept of fragility evaluation



### 3.7 Accident sequence evaluation

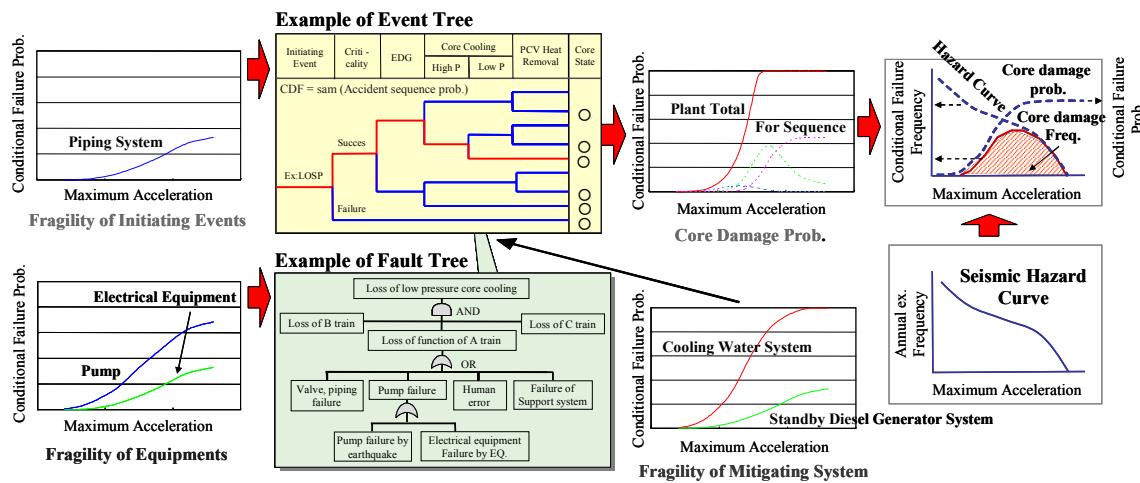
#### (1) Composition

“Accident Sequence Evaluation” (Chapter 7) is described in the following 6 sections:

- 7.1 Evaluation process.
- 7.2 Setting of initiating event.
- 7.3 Modeling of accident sequence.
- 7.4 Modeling of systems.
- 7.5 Quantitative evaluation of accident sequence.
- 7.6 Analysis of loss of containment function scenario.

In the procedure of accident sequence evaluation as shown in Figure 6, initiating events are determined first of all, and modeling of accident sequence is conducted as Event Tree (ET) for these initiating events. Next, modeling of the logic models for evaluating earthquake-related loss of function of the mitigation systems included in the ET heading is conducted. Then, by using the evaluation results of ET, FT, human errors, fragility evaluation and seismic hazard evaluation, probability / frequency of core damage for the concerned accident sequences that may lead to core damage is obtained considering the uncertainty. Finally, by multiplying these probabilities / frequencies, core damage probability / frequency (CDF) is obtained. If necessary, by performing importance analysis, important accident sequences, mitigation systems and components that dominantly contribute to CDF are obtained.

Figure6. Procedure of accident sequence evaluation and core damage frequency



## (2) Main characteristics

Main characteristics are as below:

- 1) The method of accident sequence evaluation is introduced that covers simultaneous failure of multiple components under strong seismic motion.
- 2) The method of evaluating mitigation operation under high-stress condition after strong seismic motion is introduced.
- 3) In order to identify accident sequences, mitigation systems and components that are critical in safety aspect and contribute significantly to CDF, methods to evaluate their importance such as Fussell-Vesely index, risk reduction worth and risk achievement worth are described.
- 4) Analysis contents of loss of containment function scenarios are indicated as a preparation of seismic level 2 PSA.
- 5) To promote understanding of accident sequence evaluation, quantitative evaluation results are indicated as much as possible.

## 3.8 Documentation

“Documentation” (Chapter 8) is prepared so as to satisfy the followings:

- Necessary information can be obtained in case of utilization in decision making etc.
- General image of evaluation can be understood and appropriateness of evaluation contents / results can be easily reviewed by professionals other than the evaluators.

## 4. Conclusion

The Seismic PSA technologies in our country are in the phase of utilization. The Atomic Energy Society of Japan Standards Committee has compiled requirements for Seismic PSA and specific methods to satisfy the requirements as the implementation standards. They are scheduled to be published within this year after receiving the public comment.

To conclude this report, the points to be addressed in the future for further refinement of the standards are mentioned below:

- Decrease of logarithmic-standard deviation representing the dispersion of seismic ground motion and a detailed modeling of setting the maximum limit need to be addressed.

- Detailed definition of fragility evaluation method reflecting the effect of aging and expansion of capacity data concerning local components / piping need to be addressed.
- Refinement of correlation evaluation model among multiple components as well as the refinement of evaluation method for core damage frequency in the multiple units site need to be addressed.

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