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Seismic design and analysis of safety-related nuclear structures in Sweden

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SSM perspective

Background

Following the severe accident in the Fukushima Dai-ichi nuclear power plant in Japan on March 11, 2011, the European Council decided to request stress tests to be performed on all European nuclear power plants. The European Commission, the European Nuclear Safety Regulators Group (ENSREG) and the Western European Nuclear Regulators Association (WENRA) were commissioned to develop the scope for the stress tests. It was decided to focus the stress tests and the peer review on three main topics where natural hazards including earthquake, tsunami and extreme weather was one of these topics.

One step in the European stress tests was the international peer review of each country's activities. On the basis of the international peer review and SSM's own review of the Swedish nuclear power plants, SSM has specified prioritized activities in "the Swedish action plan" with the intention to handle all plant weaknesses identified by the European stress tests.

Objectives

The main objective of the project was to identify and evaluate different approaches for the design and analysis of safety-related nuclear structures in Sweden with respect to severe earthquakes.

Results

The report presents the historical development of the seismic design for the U.S., France and Sweden with special focus on issues related to severe earthquakes beyond the design basis as well as important aspects concerning the design basis ground motions for the Swedish nuclear facilities.
The report provides recommendations on a revised model for seismic hazard assessments, on minimum requirements for seismic analysis of safety-related nuclear structures in Sweden as well as some recommendations for new structural design or redesign of existing structures. It also provides a proposal to address seismic margin assessments for severe earthquakes beyond the design basis.

The detailed recommendations on minimum requirements and safety assessments are focused on the safety-related building structures.

Need for further research

At present there is no need for further research in this area.

Project information

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Reference: SSM 2013/1651

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Contents

Executive summary

The severe earthquake and the subsequent tsunami that devastated the nuclear power plant at Fukushima Dai-ichi in Japan on March 11, 2011 has resulted in extensive international discussions and investigations as regards natural hazard assessments, and how to improve the existing safety evaluation methods for severe external events beyond the design basis.

In this report, the outcome of the stress tests of the European nuclear power plants is assessed, with special focus on earthquake effects on building structures at nuclear facilities. The Swedish action plan, which was developed after the stress tests, emphasize the needs to review and update the seismic design basis as well as the conditions and methods for seismic analysis and design. Additionally also, the methods for seismic margin assessments for ground motions exceeding the Design Basis Earthquake (DBE) need to be improved.

The historical development of the seismic design practice is reported for the U.S., France and Sweden. Important aspects regarding the design basis ground motions for the Swedish nuclear facilities are addressed and recommendations on a revised model for seismic hazard assessments are provided.

Seismic analysis methods and the seismic design process for new nuclear facilities as well as safety evaluation procedures for existing facilities are covered at a general plant level for safety-related Structures, Systems and Components (SSCs). However, detailed recommendations on minimum requirements and safety assessments are focused on the building structures.

A vast majority of the safety-related structures at the Swedish nuclear facilities consists of concrete shear walls and slab systems of general heavy proportions. For steel framework structures, the effects of wind and snow loads normally govern the design. Hence, detailed requirements on material properties and procedures for structural analysis as well as determination of failure modes and strength properties for seismic margin assessments are primarily addressed for load-bearing concrete structures.

John D. Stevenson, Consulting Engineer and Jean-Pierre Touret, Scanscot Technology France have provided essential input as regards the historical development of the seismic design basis in section 2.2 for the U.S. and in section 2.3 for France. They have also reviewed the other parts of the report.

Sammanfattning

Den svåra jordbävningen med den efterföljande tsunamin som ödelade kärnkraftverket i Fukushima Dai-ichi i Japan den 11 mars 2011 har resulterat i omfattande internationella diskussioner och utredningar avseende riskbedömningar av naturkatastrofer, samt hur man kan förbättra de nuvarande metoderna för säkerhetsvärderingar av svåra yttre händelser utanför design.

I denna rapport genomförs en utvärdering av resultaten från stresstesterna av de europeiska kärnkraftverken, med fokus på seismiska lasteffekter på byggnadskonstruktioner vid kärntekniska anläggningar. Den svenska handlingsplanen som togs fram efter stresstesterna betonar behovet att se över och uppdatera de seismiska dimensioneringsförutsättningarna samt villkoren och metoderna för seismisk analys och design. Dessutom bör också metoderna för bedömningar av säkerhetsmarginaler för markrörelser som överstiger den dimensionerande jordbävningen (DBE) förbättras.

Den historiska utvecklingen av de seismiska dimensioneringsprinciperna redovisas för USA, Frankrike och Sverige. Viktiga aspekter på de dimensionerande seismiska markrörelserna för de svenska kärntekniska anläggningarna behandlas tillsammans med rekommendationer avseende en reviderad modell för seismiska riskbedömningar.

Seismiska analysmetoder och den seismiska designprocessen för nya nukleära anläggningar samt procedurerna för säkerhetsbedömningar av befintliga anläggningar, behandlas på en övergripande anläggningsnivå för säkerhetsrelaterade byggnader, system och komponenter (SSC). Emellertid fokuseras de detaljerade rekommendationerna avseende minimikrav och säkerhetsbedömningar på byggnadskonstruktionerna.

Flertalet av de säkerhetsrelaterade byggnaderna vid de svenska nukleära anläggningarna består av betongväggar och bjälklag med grova dimensioner. För byggnader med bärande stålstommar blir ofta effekter av vind- och snölaster dimensionerande. Därför redovisas detaljkrav avseende materialparametrar och procedurer för strukturanalyserer samt bestämning av brottmoder och hållfasthetsvärden vid seismiska säkerhetsutvärderingar primärt för bärande konstruktioner av betong.

John D. Stevenson, Consulting Engineer och Jean-Pierre Touret, Scanscot Technology France har lämnat värdefulla bidrag avseende den historiska utvecklingen av de seismiska dimensioneringsförutsättningarna i avsnitt 2.2 för USA och i avsnitt 2.3 för Frankrike. De har också granskat de övriga delarna av rapporten.

1. Introduction

1.1 The European stress tests

1.1.1 General

Following the severe accidents which occured in the Fukushima Dai-ichi nuclear power plant, the European Council in March 2011 requested stress tests to be performed on all European nuclear power plants. The European Council invited ENSREG, the European Commision and WENRA to develop the scope for the stress tests. It was decided to focus the stress tests and the peer review on three main topics which were directly derived from the preliminary lessons learned from the Fukushima disaster:

- Natural hazards, including earthquake, tsunami and extreme weather
- Loss of safety systems
- Severe accident management

The stress tests and the peer review assessed these topics in a three step process. The first step required the operators to perform an assessment and set out proposals following the ENSREG specifications. The second step was for the national regulators to perform an independent review of the operators' assessments and issue requirements whenever appropriate. The last step was a peer review of the national reports submitted by regulators. The objectives of the peer review were to assess the compliance of the stress tests with the ENSREG specification, to check that no important issue has been overlooked and to identify strong features, weaknesses and relevant proposals to increase plant robustness in light of the preliminary lessons learned from the Fukushima accident. The operators submitted their final assessments in October 2011 and the regulators submitted their final reports in December 2011.

The peer review started in January 2012. The peer review was completed with a main report that includes final conclusions and recommendations at European level as well as country reports that included country-specific conclusions and recommendations. The report was approved by ENSREG and the European Council in April 2012. In a joint ENSREG/European Council statement the stress test report was accepted and it was agreed that an ENSREG action plan would be developed to track how well the recommendations were implemented. As part of the ENSREG action plan each national regulator generated a country-specific action plan.

1.1.2 European level recommendations

The peer review was structured in accordance with the three topics of the stress tests; natural hazards, loss of safety systems and severe accident management. The peer review identified four main areas of improvement to be considered at the European level as compiled in [1] and [2] and shown in Table 1.1.

Besides these four main recommendations, the peer review highlighted a number of other observations. For topic item no 1 (natural hazards), eight subtopics were highlighted, amongst which five addressed various recommendations on seismic-related issues, as shown in Table 1.2.

Table 1.1 – European level recommendations according to [1] and [2]

No	Area	Recommendations
1	European guidance on assesment of natural hazards and margins	The peer review Board recommends that WENRA, involving best available expertise from Europe, develop guidance on natural hazard assessments, including earthquake, flooding and extreme weather conditions, as well as corresponding guidance on assessment of margins beyond the design basis and cliff-edge effects.
2	Periodic safety review	The peer review Board recommends that ENSREG underline the importance of periodic safety review. In particular, ENSREG should highlight the necessity to reevaluate natural hazards and relevant plant provisions as often as appropriate but at least every 10 years.
3	Containment integrity	Urgent implementation of the recognized measures to protect containment integrity is a finding of the peer review that national regulators should consider.
4	Prevention of accidents resulting from natural hazards and limiting their consequences	Necessary implementation of measures allowing preventions of accidents and limitation of their consequencies in case of extreme natural hazards is a finding of the peer review that national regulators should consider.

Table 1.2 – Topic item no 1 (natural hazards) relating to seismic hazard according to [1] and [2]

Subitem	
Hazard frequency	The use of a return frequency of 10^{-4} per annum (0.1g minimum PGA for earthquakes) for plant review/back-fitting with respect to external hazard safety cases.
Secondary effects of earthquakes	The possible secondary effects of seismic events, such as flood or fire arising as a result of the event, in future assessments.
Seismic monitoring	The installation of seismic monitoring systems with related procedures and training.
Qualified walkdowns	The development of standards to address qualified plant walkdowns with regard to earthquake, flooding and extreme weather, to provide a more systematic search for non-conformities and correct them (e.g. appropriate storage of equipment, particulary for temporary and mobile plant and tools used to mitigate beyond design basis (BDB) external events).
External hazard margins	In conjunction with main recommendation 1 (European guidance on assesment of natural hazards and margins), the formal assessment of margins for all external hazards including, seismic, flooding and severe weather, and identification of potential improvements.

As regards the topic item “Hazard frequency”, the ENSREG peer review team states in [2] that a good practice is that external events should be addressed by designing to the hazard level consistens with a 10 000 year return period, i.e. an annual frequency equivalent to 10^{-4}. However there are some countries where the acceleration levels consistent with the perceived 10^{-4} yearly return frequency are very low. In these circumstances, IAEA guidance suggests that a minimum 0.1g horizontal PGA should be adopted.

As regards the topic item “External hazard margins”, the peer review process noted that the evaluation of margins beyond design basis (BDB) is not consistent in participating countries. The majority have made only a general claim that margins exists and therefore there is no information on the basis of which to consider effective potential improvements. Very few countries have determined cliff-edge effects and the associated protection improvements in the manner envisaged by ENSREG. There are well-established practices for assessing seismic margins BDB, referred to as seismic margin assessment (SMA). This appears similar to a deterministic method, although the acceptance criteria are derived from probabilistic fragility assessments. Alternatively, similar fragilities can be implemented in a seismic PSA. On the basis of this outcome, the peer review team recommended that WENRA, involving the best available expertise from Europe, should consider how to determine a consistent approach to margin assessments for external events.

1.1.3 The French action plan

After having received and evaluated the complementary safety assessments from the French operators, the French safety authority (ASN) concluded in the following general statements [4]:

- The natural disaster which struck the Fukushima Dai-ichi NPP confirms that, whatever the precautions taken in the design, construction and operation of nuclear facilities, an accident can never be completely ruled out.
- The licensee has the overall responsibility for the safety of its facilities while, on behalf of the State, ASN is responsible for regulating and monitoring nuclear safety, with the technical support of IRSN and its Advisory Committees. Pursuant to the law, ASN ensures that the safety of French civil nuclear facilities shall be maintained continuously, in particular through the periodic review process and the integration of experience feedback.

ASN considers that the continued operation of the French nuclear facilities requires with highest priority their robustness to extreme situations to be increased beyond existing safety margins. ASN is thus requiring the licensees to adopt a range of measures to provide the facilities with the means to deal with the following events:

- A combination of natural phenomena of an exceptional scale and exceeding those adopted in the design or the periodic safety review of the facilities.
- Severe accident situations following the prolonged loss of electrical power or cooling and liable to affect all the facilities at a given site.

Among these new provisions, ASN would in particular recommend the creation of a "hardened safety core" of essential SSCs and organizational arrangements making it possible to manage the fundamental safety functions in extreme situations, with the aim of preventing a severe accident, limiting large-scale radioactive releases if the accident cannot be controlled and enabling the licensee, even in extreme situations, to perform its emergency management duties. This will for example involve setting up a "bunkerized" emergency management centre with diesel electricity generator, and an ultimate backup water supply. The equipment to be included in this “hardened safety core” must be designed to withstand major events (earthquake, flooding, etc.), of a scale far in excess of those used to determine the strength of the facilities, even if not considered to be plausible.

In January 2014, the ASN adopted 19 resolutions [5] setting out additional requirements for implementation of the post-Fukushima “hardened safety core” in EDF’s NPPs. These resolutions specify the objectives and the contents of this “hardened safety core”, which shall comprise measures to:

- Prevent a severe accident affecting the core of the reactor or the spent fuel pool.
- Limit the consequences of an accident which could not be avoided, with the aim of preserving the integrity of the containment without opening the venting system. This aim of mitigating the consequences of an accident applies to all the phases of an accident.
- Enable the licensee to perform its emergency management duties.

This “hardened safety core” must be as independent as possible from the existing systems, more specifically with regard to I&C and electrical power supplies. The ASN resolutions specify the design rules to be adopted for the “hardened safety core” equipment. These rules must comply with the most demanding nuclear industry standards, used for the design and construction of installations requiring a high level of safety. Finally, they will lead EDF to determine the maximum hazards to be considered for the “hardened safety core” equipment, in particular for earthquake and flooding in order to ensure ultimate protection of the facilities. The hazard level for the earthquake is still pending, it should probably be consistent with BDBE used in the SMA. These resolutions will apply to all the NPPs in operation, as well as to the Flamanville 3 EPR reactor currently under construction.

Over and above the “hardened safety core”, EDF is required to comply with the following prescriptions:

- The “Nuclear rapid intervention force (FARN)”, fully operational no later than the end of 2014. This force can provide assistance to a damaged site by providing specialized teams to back up those of the plant concerned and mobile equipment to supply additional water and electricity. A number of modifications were therefore made to the reactors to make it easier to connect this equipment brought on-site by the FARN.
- Launch before 30th June 2013 the deployment of the 58 ultimate backup diesel generator sets for all the reactors, medium-power generator sets were added to each reactor.
- Additional training to its staff for intervention in the event of an earthquake and a severe accident.

With regard to the basic safety requirements concerning the consideration of seismic hazard, the prescriptions setting out additional requirements for deployment of the “hardened safety core” significantly reinforce the ability of the NPPs to withstand this risk. Finally, together with IRSN, ASN has begun to examine an update of the basic safety rule 2001-01 [40] concerning the determination of the seismic risk.

1.1.4 The Swedish action plan

One step in the European stress tests was the international peer review of each country’s activities. The main observations from the peer review of the Swedish activities [3] as regards plants assessments with respect to the Design basis Earthquake (DBE) can be summarized as follows:

- Licensing apply a DBE within a radius of twenty kilometers of a strength corresponding to a magnitude of approximately 6.0 on the Richter scale and with a probability of once per 100 000 years (10^{-5}).

- The assessment of the DBE uses a probabilistic approach based on a so called "average Fennoscandian seismicity function" accounting for site conditions of hard rock. Consideration of site effects leads to compute peak ground accelerations for the DBE by the reduction of the PGA related to the Swedish 10^{-5} earthquake by 15% to account for the favourable site conditions as all plants are sited on solid rock.
- It appears that the values of the DBEs for the different sites are close to IAEA's suggested minimum values at the background of the active deformation of Fennoscandia, which is proved by geodetic and paleoseismologic data.
- The full compliance of the reactors, originally not designed to withstand seismic loads, is expected in 2013 after the implementation of modifications (e.g. anchoring of mechanical components, emergency power supply) in accordance with the requirements on seismic safety, in force in 2005.

The main observations of the peer review group as regards assessment of plants beyond design basis earthquakes are as follows:

- The integrity assessments of the reactor containment, scrubber building and spent fuel pools (SFPs) are based on approximate calculation methods and engineering judgement on a best estimate basis due to the limited time available for the study.
- The integrity of reactor containments, SFPs and other important buildings are estimated to be preserved in case of the 10^{-7}-earthquake. However, there is need for refined analyses and further investigations before definite conclusions are possible. Such investigations should emphasize on evaluating margins to reach safe shutdown conditions.

According to SSM [3], the Swedish earthquake is based on observations and historical accounts of earthquakes in Fennoscandia for about 500 years, as well as comparisons with the occurrence of earthquakes in other low seismic regions in the world. Based on these facts, SSM estimated that fairly reliable predictions can be performed concerning the earthquakes that are likely to occur in Scandinavia in a 500 year geological time scale. However, this was questioned as a restriction by the peer review team, due to the fact that geodetic and paleoseismologic data which according to some researchers indicates continuous active uplift and deformation of Fennoscandia. Also, IAEA in SSG-9 [6] explicitly suggests the use of such data in low seismicity intraplate regions. SSM has agreed to consider the existing approach by taking into account the geodetic and paleoseismologic data.

On the basis of the international peer review and SSM's own review of the Swedish NPP plants, SSM has specified prioritized activities in "the Swedish action plan" [7], with the intention to handle all plant weaknesses identified by the EU stress tests. As regards earthquake hazards, actions are defined as shown in Table 1.3. These actions shall be finalised latest 2015.

Table 1.3 – Swedish action plan [7] regarding earthquake hazards

No	Item	Action
T1.LA.1	Seismic plant analyses	A return frequency of 10^{-5}/year (with a minimum peak ground acceleration of 0.1g) shall be used as a basis for plant reviews/back-fitting. The following actions shall be performed: - Further studies regarding the structural integrity of the reactor containments, scrubber buildings and fuel storage pools shall be performed. - The pipes between the reactor containment and the MVSS that allows a controlled pressure relief of the reactor containment shall be evaluated further.
T1.LA.2	Investigation regarding secondary effects of an earthquake	Investigations regarding secondary effects of an earthquake shall be performed. Fire analyses at Swedish NPPs are in general performed according to SAR but analyses of the effects of fire as a result of an earthquake have not been carried out at any of the Swedish NPPs. A more detailed analysis of earthquake induced flood, where for example leakage from broken water storage tanks and cracks in the cooling water channels are taken into account have to be included in the analyses regarding secondary effects.
T1.LA.3	Review of seismic monitoring	Seismic monitoring systems are installed at all Swedish sites. The utilities shall review the procedures and training program for seismic monitoring and implement them.
T1.LA.9	Investigations of external hazard margins	In conjunction with recommendation regarding flooding margin assessments, a formal assessment of margins for all external hazards including seismic, flooding and severe weather, and identification of potential improvements shall be performed. Weaknesses in the plants shall be identified. Regarding the seismic margins an evaluation of structures, systems and components against ground motions exceeding DBE shall be performed. Such evaluations shall emphasize on margins.
T1.RA.1	Research project regarding the influence of paleoseismological data	SSM will start up a research project regarding the influence of paleoseimological data on the existing model regarding frequency and strength of the ground response spectra constructed in the project SKI 92:3 [8] .

1.2 Scope of the report

A general conclusion from the topic “Natural Hazards” within the scope of the European stress tests, is the necessity for further activities on the Swedish nuclear facilities in order to develop the following issues:

- The approach to determine the seismic design basis, as well as the conditions and methods for seismic design, analysis and safety verification.
- Methods to address cliff-edge[1] effects and seismic margin assessments for ground motions exceeding the DBE.

In this report, the outcome of the stress tests of the European nuclear power plants is assessed, with special focus on earthquake effects on building structures at nuclear facilities.

The historical development of the seismic design practice is reported for the U.S., France and Sweden in section 2, with especial focus on issues related to severe earthquakes beyond the design basis.

In section 3, important aspects as regards the design basis ground motions for the Swedish nuclear facilities are addresses and recommendations on a revised model for seismic hazard assessments are provided.

Recommendation on minimum requirements for seismic analysis of safety-related nuclear structures in Sweden, in accordance with ASCE 4-98 [24] and IAEA SG-G-1.6 [50], are presented in section 4.

The different steps in the seismic design process are addressed at a general plant level in section 5, together with some recommendations for new structural design or redesign of existing structures.

Specific considerations regarding seismic safety evaluation of existing structures not designed against earthquakes are reported in section 6, together with a proposal to address seismic margin assessments for severe earthquakes beyond the design basis.

Seismic analysis methods, the seismic design process for new facilities and safety evaluation procedures for existing nuclear facilities are covered at a general plant level for safety-related Structures, Systems and Components (SSCs). However, detailed recommendations on minimum requirements and safety assessments are focused on the safety-related building structures.

A vast majority of the buildings at the Swedish nuclear facilities consists of concrete shear walls and slab systems of general heavy proportions. For steel framework structures, the effects of wind and snow loads normally govern the design. Hence, detailed requirements on material properties and procedures for structural analysis as well as determination of failure modes and strength properties for seismic margin assessments are primarily addressed for concrete structures.

[1] when a small deviation of a design plant parameter give rise to an abrupt worsened situation for the whole plant.

2. Historical development of the seismic design basis

2.1 General

The first generation of nuclear power facilities in the U.S., which were commissioned during the 1950s and the beginning of the 1960s, included only some general seismic design recommendations that would be applied to any building structure, without any detailed requirements. The rapid expansion of the nuclear power industry during the 1960s and 1970s was in fact one important reason for the development of the seismic design requirements applicable to safety-related nuclear structures, distribution systems and components. The new knowledge and experience in seismic engineering were soon reflected in new standards for NPPs.

Standards and guidelines for seismic design and analysis of NPPs have to a large extent been developed in the U.S. under the superintendence of the USNRC. Later on, these standards and guidelines were also adopted for nuclear facilities in many other countries. In order to better understand the principles of current design criteria for seismic analysis of safety-related nuclear structures and how to improve the assessments as regards beyond design issues for severe earthquakes, a historical retrospect of the development of the international design practice is presented in this chapter. The focus is on the developments in the U.S. and France. Some important perspective from the Swedish horizon is also considered.

2.2 The United States

2.2.1 Introduction

There is a hierarchy of requirement in the U.S. in order to regulate the seismic design of NPPs. These requirements are as follows:

- Federal Laws of the U.S. These are laws passed by the U.S. Congress. These laws provide the highest tier of requirement which are in broadly stated objectives and have the force of law and are mandatory.
- Code of Federal Regulations (CFR). These are requirements prepared by the USNRC intended to provide more specific requirements to implement the laws. The requirements also have the force of law and are mandatory.
- Regulatory Guide (RG) and Standard Review Plan (SRP) procedures. These are requirements that if followed would satisfy the USNRC's interpretation of Federal Laws and CFR. These are not mandatory, but if RG or SRP are not used or departed from, the designer must justify the difference to the satisfaction of the USNRC. It should be noted that the requirements of a Federal Regulation and provisions of a RG or SRP provisions typically become USNRC's policy one to three years before they are formally published.
- Codes & Standards for design and constructiorn. The most important standards for nuclear concrete structures are ASME B&PV Code, Sect III, Div 2 [9] for concrete reactor containments and ACI 349 [10] for other safety-related nuclear structures.

The evaluation of the seismic effects on safety-related nuclear structures can be divided into three basic design activities as follows:

- Define earthquake phenomena and resultant loads typically in the form of Peak Ground Acceleration (PGA) and response spectral shape.
- Identify procedures to convert earthquake loads to energized forces, stresses and deformations or strains in safety-related nuclear structures, distribution systems and components (SSCs).
- Provide acceptance criteria associated with the resultant generalized forces, deformation or strains.

These three activities will be discussed historically as they were developed for nuclear safety, from the initial static and dynamic deterministic based criteria to the current risk informed probabilistic criteria used in the U.S. today.

2.2.2 Historical development of seismic design ground motions

The earthquake design effort for NPPs in the U.S. was first contained in TID 7024 [11] published in 1963. TID 7024 [11] was a departure from conventional codes for building structures, mechanical and electrical distribution systems and components design. Prior to 1965 the U.S. Uniform Building Code [12] was used. For instance Connecticut Yankee NPP, whose design began in 1964, was initially designed for 0.03g static acceleration as specified by the then applicable Uniform Building Code [12]. Later the plant was evaluated for a 0.17g seismic PGA and response spectra. The TID 7024 [11] publication was prepared for the then U.S. Atomic Energy Commission by a group of industry experts.

The definition of seismic PGA was addressed by the publication of 10 CFR100 Appendix A [13], which defines the SSE, formally in 1973 but which had been in use since 1965. The SSE corresponds to the DBE for commercial NPPs and is defined as that earthquake which produces the maximum vibratory ground motion for which certain SSCs are designed to ensure the following:

- Integrity of the reactor coolant pressure boundary.
- Capability to shut down the reactor and maintain it in a safe shutdown condition, or
- Capability to prevent or mitigate the consequences of accidents, which would result in potential off-site exposures.

In 10 CFR100 Appendix A [13], the definition of seismic PGA applicable to the ground support of safety related nuclear structures was developed as follows:

For tectonic provinces without known capable fault locations, the largest historically recorded earthquake occurring within the tectonic province of the site was moved to the site. In general, the PGA which defined the anchor of the Housner shaped ground response spectra [14] in TID 7024 [11] was used to generate the seismic acceleration input to the foundation of safety-related nuclear structures (Seismic Category I) in the period before 1967.

In some instances, large historical earthquakes in the Eastern U.S. such as Charleston in 1886 and in the Central U.S. such as New Madrid in 1811-1812, governed over the local tectonic province historical earthquakes moved to the site.

TID 7024 [11] provided suggested reductions or attenuation of seismic intensities and associated PGAs as a function of the distance to the site from the hypocenter of earthquake in tectonic provinces other than the site province. These attenuation criteria were based on Southern California earthquake data. Subsequent studies for the Central/Eastern/Southern regions of the U.S. showed that the slope of the attenuation of seismic intensity with distance in California was too severe by a factor of 2 or more in the Central/Eastern/Southern U.S. There is continuing research into the attenuation parameter which is expected to be complete by the end of 2013. The current relationships were developed by EPRI in 2006.

Earthquake loads on SSCs are determined by the PGA to which the design basis ground response spectra is anchored, (acceleration in excess of at least 33Hz). The Housner shaped ground response spectrum [14] is based on an average of four measured strong motion earthquake response spectra. These spectra were used for NPP design in the U.S. between 1965 and 1967 and were replaced by the original and modified Newmark NBK response spectra which were used between 1968 and 1971. These spectra were then replaced by the Newmark Blume and Kapoor spectra in 1971. In 1973 the USNRC published the RG 1.60 [15] response spectra as shown in Figure 2.1 which formally replaced the Newmark NBK spectra, as discussed by Stevenson and Conan in [16].

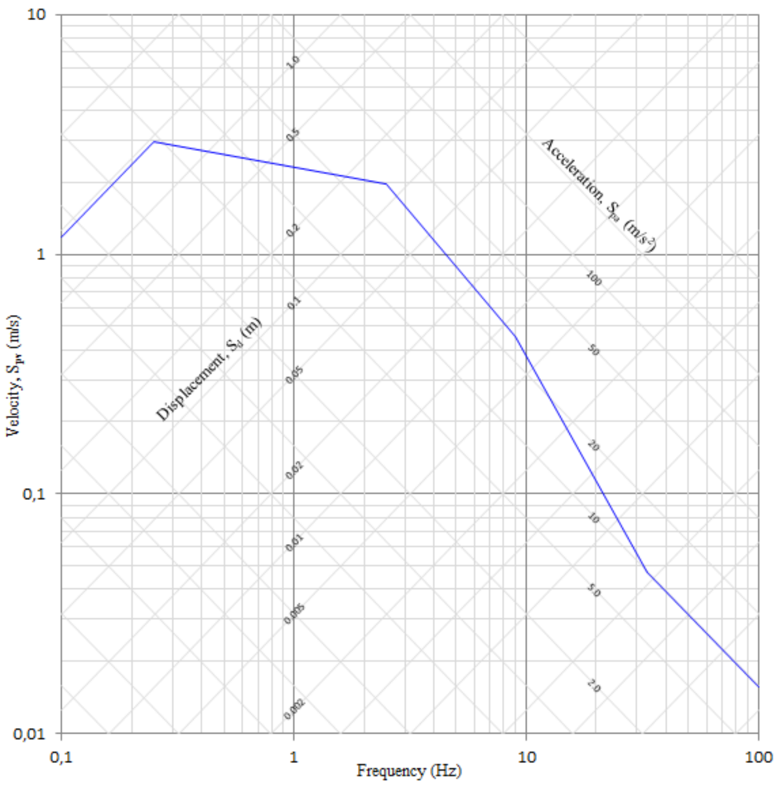

Figure 2.1 – Horizontal design response spectra (5% critical damping) according to RG 1.60 [15], scaled to 1g horizontal ground acceleration

The design response spectra as defined by Housner in [14] and by USNRC in RG 1.60 [15] are examples of so called *standard*, *generic* or *site-independent* spectra. The term *standard* here refers to response spectra that have been developed by statistical analysis of a set of strong motion data obtained within a wide range of distances of relatively large magnitude earthquakes and without specific consideration of the tectonic environment or the local subsurface conditions at the site being evaluated, see Figure 2.2.

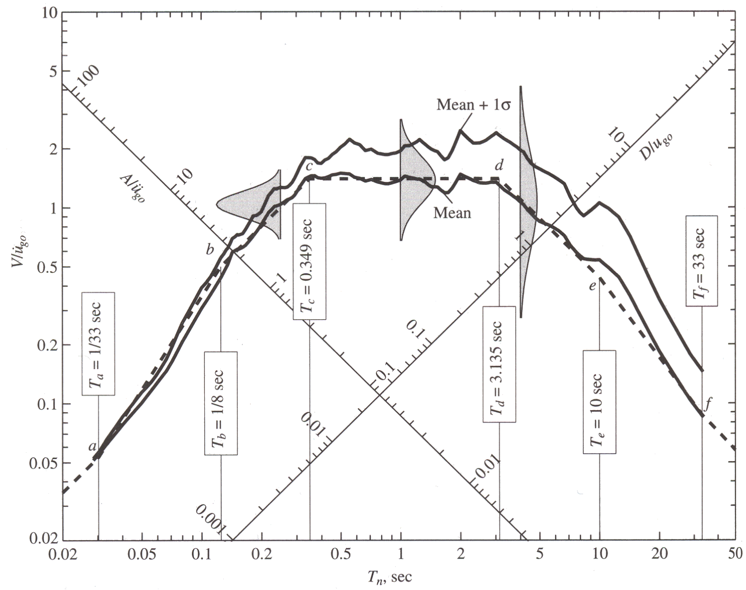

Figure 2.2 – Example of a standard or site-independent response spectra, developed from a statistical data from recorded earthquakes, according to [17].

Starting in the late 1980s, it began to be realized that the standard type of spectra were less appropriate for soft soil foundations characterized by soils having a low strain and shear wave velocity, and for sites susceptible to high frequency motions (where significant spectral amplification occurs at frequencies of 33 Hz and beyond) if systems and components are sensitive to such motion. For such conditions, site-specific response spectra based on a reference annual probability of exceedance approach started to be developed.

Site-specific spectra have the advantage of incorporating specific considerations of the tectonic environment and subsurface conditions at a site. The development of these spectra may be based on applicable response-spectral attenuation relationship or a statistical analysis of a selected set of strong motion data to be particularly applicable to the site, and/or on modeling and analysis of the effect of physical factors (earthquake source characteristics, geologic travel path, and local soil conditions) on ground motions at the site.

This recognition of the probabilistic nature of the seismic hazard is based on the need to define such risks probabilistically in order to meet overall safety goals of the USNRC. These goals were expressed probabilistically by the publication of 10 CFR100.23 [19] and 10 CFR 50 Appendix S [20] in 1996, later on resulted in the publication of the USNRC RG 1.165 [21] in 1997, in which a probabilistic basis for determining the DBE was provided.

It should be noted that the RG 1.165 [21] introduced several more details to the process of developing the DBE (SSE) requirements as follows:

- The requirement to develop a response spectra specifically applicable to the site rather than use of only a generic response spectra (i.e. RG 1.60 [15]).
- A distinction between a site spectral ground motion on a real or assumed site rock statum and that applied to plant structures.

- The establishment of the DBE response spectra at the median (50th percentile) 10^{-5}/yr. probability of exceedence level.[2]
- However, this RG 1.165 [21] continued to assume the dominate earthquake acceleration occurs in the 2 to 10 Hz frequency range characteristic of a large earthquakes in regions (as in California) where there are many capable faults identified.

It should be noted that the RG 1.165 [21] was withdrawn in 2010 and replaced by another RG 1.208 [22] published in 2007. RG 1.208 [22] is the current USNRC recommended procedures for developing the DBE (SSE) requirements in the U.S.

RG 1.208 [22] uses a performance-based approach instead of the reference probability approach as in RG 1.165 [21], in order to ensure that NPPs can withstand the effects of earthquakes with a desired performance. Further, the method consists of establishing a site-specific Uniform Hazard Response Spectra (UHRS) with spectra coordinates at each frequency having the same probability of occurrence and is based on the procedures developed in Chapter 2 of ASCE 43-05 [23].

2.2.3 Historical development of the structural modeling technique

Prior to 1965, equivalent static seismic loads on structures were defined on the basis of the fundamental frequency of the structure and an assumed flexibility based on a $\sqrt{(k/m)}$ relationship with m being the mass of the structure and k its shear and bending moment stiffness. Multi-degrees of freedom models using Housner response spectra [14] as input, and as well including springs representing SSI where a plant was founded on other than rock or very stiff soil, enhanced the modeling capability after 1965. At the end of the 1960s computer programs became available in the nuclear industry, permitting multi-degree of freedom stick- or beam-modeling technique and response spectrum modal analysis as well as developing in-structure response spectra using two- or three-dimensional shell or plate models in three-dimensional space as the basis for modeling and analyzing safety subsystems.

By the mid 1970s two- or three-dimensional shell or plate models were recognized as important to consider for unsymmetrical buildings and components, resulting in enhancements of the traditional stick- or beam-models to handle the summation of the co-directional responses from all three orthogonal earthquake excitation directions.

The first SSI-technique, using simple one-dimensional linear springs gave way to develop the soil impedance function method in the 1980s. Since the 1990s finite element modeling of the foundation media, coupled with the building model, has been developed and implemented in various computer software products.

In contradiction to static loadcases when load values are determined independent of the mathematic model of the structure, the magnitudes of the seismic loads are dependent on the dynamic properties of the structural system being modeled. This means that the requirements on the structural model and analysis must be more rigorous when dealing with seismic analysis compared to conventional static analysis. This simple fact, in combination with the development of more sophisticated 3D finite element models during the 1980s and 1990s, boosted the need for more consistent requirements on the seismic analysis technique.

The USNRC released its first versions of SRP 3.7.1 [25] and 3.7.2 [26] regarding seismic system design parameters and seismic system analysis in 1972 with revisions in 1981, 1989 and 2007 and the first versions of RG 1.92 [27] in 1976 for combining modal responses and spatial

[2] For typical seismic hazard curves the mean 10^{-4}/yr. equals the median 10^{-5}/yr. probability of exceedence.

components in seismic analysis, as well as RG 1.122 [28] in 1978 regarding acceptable procedures for developing in-structure response spectra. The first version of ASCE 4-98 Standard [24] was released in 1986 and included requirements for modeling and analysis of safety-related nuclear structures subjected to earthquake motions.

2.2.4 Seismic design classification

Seismic design classification for NPPs is addressed in RG 1.29 [29]. Those SSCs which have to be designed to withstand the effects of an SSE, are specified and designated as Seismic Category 1.

In RG 1.143 [30], in reference to radioactive waste stored at an NPP site, there are two safety classes identified; Safety Class RW-IIa, RW-IIb and a Non-Safety Class RW-IIc. These Safety Classes are a function of the radio nuclides and their quantities stored to include their gasous liquid or solid forms and are described in RG 1.143 [30]. The off-site radiological release criteria are set to 5 millisieverts (mSv) per year or an on-site dose of 50 mSv per year resulting from a postulated failure requiring a RW-IIa classification and any value of radiation release less than these values requiring Safety Class RW-llb.

The seismic load applicable to RW-IIa SSC is one half of the SSE load specified for Seismic Class I SSC as shown in Table 2 in RG 1.143 [30]. The seismic load specified for RW-llb and RW-llc are found in the ASCE 7-10 [32] Standard Risk Categories III and II respectively.

In RG 1.29 [29] there are references to SSE for fire protection system design in RG 1.189 [31], where the following statement is made on page 54: "The fire suppression systems should retain their original design capability for (1) natural phenomena of less severity and greater frequency than the most sever natural phenomena (approximately once in 10 years) such as tornadoes, hurricanes, floods, ice storms, or small-intensity earthquakes that are characteristic of the geographic region." This statement would result in an earthquake stated load smaller than specified for conventional SSCs by ASCE 7-10 [32].

The USNRC has recognized the need to be less prescriptive in the designation of Seismic Category I and relates such categories to safety-related and safety-significant risk categorization as contained in Figure 1 in RG 1.201 [33], as shown in Figure 2.3, as an alternative to the RG 1.29 [29] deterministic seismic classification of SSCs which did not consider the level of risk associated with a particular SSC failure.

In 10 CFR 50.69 [34], the application of safety-related and safety-significant categorization to NPPs is explained. To date this procedure for risk informed reclassification of SSC has been attempted on a trial basis related to maintenance, testing and examination activities for SSCs in a small number of existing NPPs. There is an active effort in the ASME B&PVC Section III Nuclear Component Design Code Committees to extend risk informed categorization to design activities. But to date, definitive design criteria based on risk informed assessment categorization has not been developed and it is expected it will take several years more.

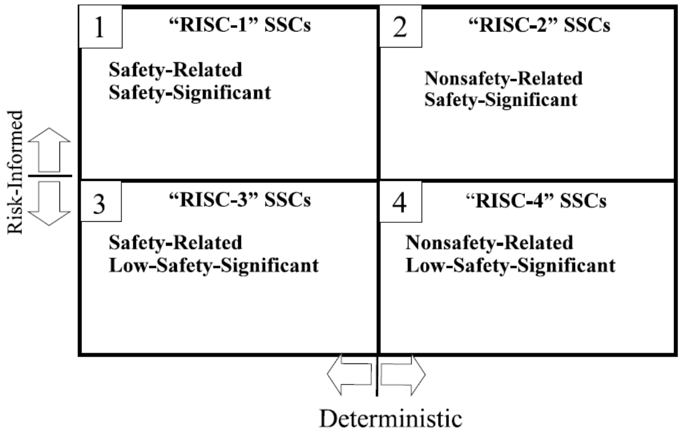

Figure 2.3 –Figure depicting the current safety-related versus nonsafety-related SSC categorization scheme with an overlay of the new safety-significance categorization, according to [33].

2.2.5 Codes and standards

ACI 318 [35] prescribes minimum requirements for all types of ordinary concrete buildings in the U.S. In general, the structural form consists of moment resisting frames designed for an essentially elastic response for all loads and load combinations except those associated with strong earthquake motions. ACI 318 [35] permits a seismic design based on loads corresponding to an inelastic response to earthquake ground motions. In order to secure that the structural elements can exhibit inelastic behavior during the translational earthquake motions, ACI 318 [35], chapter 21 provides minimum requirements on the reinforcing steel detailing.

ACI 349 [10] provides requirements for design of safety-related nuclear concrete structures. The predominant structural form is shear wall and slab construction of geneal heavy proportions. The structural elements are designed for an elastic behavior for all loads (except impulsive and impactive loads) and load combinations applicable to structures, distribution systems or containments in Limit State D according to ASCE 43-05 [23] or Seismic Category 1 according to RG 1.29 [29] including those associated with the DBE. The main reason to the choice of structural form and the elastic design principle is of course to ensure a robust design with large safety margins for SSC which provide for reactor safety and shutdown and spent fuel storage. It should be noted that SSC categorized in Limit State A, B or C according to ASCE 43-05 [23] or in Safety Class IIa and IIb according to RG 1.143 [30] are allowed to respond inelastically. But, as indicated in section 1.1 as well as Table C1-1 in ASCE 43-05 [23], SSCs in modern NPPs fall, almost exclusively, in the highest Seismic Design Category (SDC 5), hence being associated with an "essential elastic behavior" for the DBE event.

Even though ACI 349 [10] requires safety-related nuclear structures to be designed essentially elastic to earthquake loads, it provides minimum requirements for reinforcing steel detailing according to the requirements of chapter 21 in ACI 318 [35]. Besides maintaining the maximum possible compatibility between ACI 349 [10] and ACI 318 [35], the main reason for this approach is to provide additional assurance that structural integrity is maintained in the unlikely event of an earthquake beyond the design basis event DBE.

For mechanical components and distribution systems, elastically computed stress are used when the allowable stress is typical in the range of 1.6 and 2.0 times specified minimum yield stress.

For electrical distribution systems and components generally follow the civil engineering acceptance criteria for building (i.e. specified minimum yield stresss).

2.2.6 Consequences of Fukushima Earthquake on Design Basis Earthquake

2.2.6.1 Introduction

Following the severe accidents at the Fukushima Dai-ichi NPP, the USNRC decided to make additional improvements to its regulatory system in order to enhance the protection against accidents resulting from natural phenomena, mitigating the consequencies of such accidents and ensuring emergency preparedness. The USNRC's review of insights from the Fukushima Dai-ichi accident resulted in recommendations for enhancing the reactor safety as reported in [36].

Even though current USNRC regulations and associated regulatory guidance provide a robust regulatory approach for evaluation of site hazards asscociated with natural phenomena, this framework has evolved over time as new information regarding site hazards and their potential consequences has become available. As a result, the licensing bases, design, and level of protection from natural phenomena differ among existing operating reactors in the U.S., depending on when the plant was constructed and when the plant was licensed for operation. Over the years the USNRC has initiated several efforts to evaluate risks and potential safety issues resulting from these differences. However, the USNRC has not yet undertaken a comprehensive reestablishment of the design basis for existing plants that would reflect the current state of knowledge of current licensing criteria. As a result, significant differences may exist between plants in the way they protect against design-basis natural phenomena and the safety margin provided.

With regard to seismic hazards, available seismic data and models show increased seismic hazard estimates for some operating nuclear power plant sites, as reported in [36]. The state of knowledge of seismic hazards within the U.S. has evolved to the point that it would be appropriate for licensees to reevaluate the designs of existing nuclear reactors to ensure that SSCs important to safety will withstand a seismic event without loss of capability to perform their intended safety function. As seismic knowledge continues to increase, new seismic hazard data and models will be produced. Thus, the need to evaluate the implications of updated seismic hazards on operating reactors will recur and need to be reevaluated at appropriate intervals.

In order to ensure adequate protection from natural phenomena, consistent with the current state of knowledge and analythical methods as above, the USNRC initiated a number of actions, as reported in [36] and presented in Table 2.1. The outcome of Seismic Recommendation 2.1 and 2.3 are reported in the following sections.

Table 2.1 – USNRC's Recommendation 2 to enhance the reactor and spent fuel safety in the U.S [36].

Recommendation	
2.1	Order licensees to reevaluate the seismic and flooding hazards at their sites against current NRC requirements and guidance, and if necessary, update the design basis and SSCs important to safety to protect against the updated hazards.
2.2	Initiate rulemaking to require licensees to confirm seismic hazards and flooding hazards every 10 years and address any new and significant information. If necessary, update the design basis for SSCs important to safety to protect against the updated hazards.
2.3	Order licensees to perform seismic and flood protection walkdowns in accordance with their licensing basis to identify and address plant-specific vulnerabilities and verify the adequacy of monitoring and maintenance for protection features such as watertight barriers and seals in the interim period until longer term actions are completed to update the design basis for external events.

2.2.6.2 Recommendation 2.1: Seismic

The activity entitled *Recommendation 2.1: Seismic* was required by the USNRC to seismically re-evaluate and potentially increase the seismic resistance of existing NPP. This included development of a revised probabilistically based seismic hazard at the mean 10^{-4}/yr. probability of exceedence level to be instituted for each U.S. NPP site. This hazard for the various plant sites has involved a complete re-evaluation of the earthquake level to be considered as a function of the magnitude of the probabilistic defined earthquake hazard and its motion attenuation with distance from its epicenter or focus to the NPP site.

This re-evaluation and development of the seismic hazard earthquake is termed the Review Level Earthquake (RLE) and is currently underway. It is expected that both the shape of the ground response spectra and the PGA will change significantly from the spectral shape and PGA used as the plant seismic design licensing basis, i.e., higher acceleration at frequencies above 10Hz. This is particularly true for the Central and Eastern U.S. sites.

Also associated with this *Recommendation 2.1: Seismic* program will be the development of fragility curves for nuclear safety-related SSC. In the development of these fragility curves it is understood that the SSCs that are ductile will be allowed to respond into the inelastic range beyond yield of the SSC's material and that revised capability may be used in developing the individual SSC fragility curves.

Guidance for the development and performance of the *Recommendation 2.1: Seismic* program is contained in [37]. As part of the *Recommendation 2.1: Seismic* program is the so called Flex Program. This program provides for re-evaluation of existing installed SSC and for the procurement of additional SSC which are intended to mitigate or reduce seismic risk.

2.2.6.3 Recommendation 2.3: Seismic

The near-term program activity *Recommendation 2.3: Seismic* consisted of a review of the seismic design adequacy on a walkdown and walkby basis (evaluation based on the existing licensing basis for the NPP plant) which was completed based on existing outage schedules by

November 2012[13]. This was accomplished by a physical walkdown and walkby of a sample of nuclear safety related mechanical and electrical components (approximately 120 items) installed in the NPP. The Seismic Evaluation Guidance [38] used to implement this program was developed by EPRI and endorsed by the USNRC.

It should be noted a similar review of the flooding hazards, from all causes not just seismic, at each NPP site was also performed by review of current design basis flooding hazard levels, flood protection procedures and a physical plant walkdown of flooding mitigation and prevention structures systems and components.

2.2.6.4 Beyond Design Basis Earthquake

The USNRC has developed a requirement to evaluate a BDBE to assure that there is margin available to resist seismic load without failure of the SSC to perform required safety functions. The load selected is 1.67 times the DBE load. It should be understood that for most of North America the slope of most seismic hazard curves currently under review has an increase in acceleration of between 1.5 and 2.0 for each doubling of the return period in the 10^4 to 10^5 mean return period range in years. As a result of the NRC's 1.67 multiplication factor the BDBE probabilistically defined mean return period would be increased from 10 000 years to approximately 20 000 years.

Associated with this increase in seismic acceleration would be an increase in the applied acceptance criteria into the inelastic range beyond yield for ductile type structures. However, specific acceptance criteria into the inelastic range has not yet been published by the USNRC.

ASCE 43-05 [23] provides acceptance criteria into the inelastic range for ductile SSC in SDC 3 to SDC 5. SDC 1 and SDC 2 category SSC are equivalent to USNRC Safety Class RW Class IIc and IIb respectively as defined in RG 1.143 [30] and are designed to the loading requirements of ASCE 7-10 [32] for natural hazard loads and ACI 318 [35] for concrete structures.

It should also be observed that Section 1.1 as well as Table C1-1 in ASCE 43-05 [23] indicates that SSCs in modern NPPs fall, almost exclusively, in the highest Seismic Design Category (SDC 5) and is associated with an "essential elastic behavior" for design basis acceptance criteria.

From ASCE 43-05 [23] for elastic analyses and ductile structures in other than the limit state D category for SDC 3 to SDC 5, the total limiting capacity for an element shall be the yield stress or design code ultimate strength equal to or greater than the sum of non-seismic demand, D_{NS}, and seismic demand, D_s, per the following load combination, as appropriate:

For bending moment, in-plane shear, and axial load in diagonal bracing:

$$U \geq D_{\mathrm{NS}} + D_{\mathrm{S}}/ F_{\mu S}$$

For other axial loads, other shear loads, and torsion:

$$U \geq D_{\mathrm{NS}} + D_{\mathrm{S}}/ 1.0$$

where

U = Ultimate strength or specified minimum yield stress

D_{NS} = Non-seismic demand acting on an element. Non-seismic demand shall include the code effects of dead, live, equipment, fluid, snow and at-rest lateral soil loads.

[3] The Program extends beyond November 2012 for SSC which were not available for walkdowns because outages were not scheduled between July and November 2012.

D_s = Calculated seismic response to the DBE using an elastic analysis approach

$F_{\mu s}$ = System inelastic energy absorption factor for structural elements according to Table 5-1 in ASCE 43-05 [23].

The USNRC has not yet provided any acceptance criteria for Beyond Design Basis Event.

2.3 France

2.3.1 Introduction

The regulatory hierarchy of the safety requirements for safety-related nuclear structures in France can be described as follows:

- Laws of the French republic, to be passed by the French Parliament. The TSN (Transparancy and Security in Nuclear field) Law of June 2006 unifies previous laws and decrees, complemented with several decrees providing application details. According to the TSN Law, ASN is declared as an independent regulatory authority and responsible for technical and regulatory decisions, licensing and control of nuclear facilities, public information, management of emergency situations and advices to the French Government.
- Decrees, departmental orders and ASN decisions. Some examples are the environmental RTGE decree, the Regulation about working conditions and protection of workers health and the Decree regarding quality assurance in nuclear activities.
- RFS fundamental safety rules, ASN guides and Technical directives. The RFS and ASN guides are issued by ASN and define technically acceptable practice.
- Codes & Standards for design and construction. Typical French Codes & Standards for civil structures are the RCC-G covering the existing French NPPs and the ETC-C for EPR NPP design.

Following an earthquake, the objective of the protection of a NPP is to ensure that the safety functions needed to return and maintain the plant to a safe shutdown state are not unacceptably affected.

The SSCs required to achieve the safety objectives must be subject to seismic classification. SSCs necessary for the safety must be designed so that they are able to fulfil their functions, maintain their integrity or remain stable under the conditions caused by the seismic ground motions.

The basic steps in the earthquake design process of safety-related nuclear structures together with the applicable regulations, guidelines and design codes in France can be categorized as follows:

a. Determine the soil characteristics at the NPP site according to RFS-I.3.c (1984) [39].
b. Determine the seismic design ground motions according to RFS 2001-01 (2001) [40].
c. Seismic modeling and analysis:
    - Existing facilities according to RFS V.2.g (1985) [41].
    - New facilities (except EPR) according to ASN/Guide/2/01 (2006) [42].
    - EPR according to ETC-C, Appendix A.1 (2010) [43].
d. Seismic design and safety evaluation:

- Existing facilities according to RCC-G, volume 1-Design (1981 and 1985) [44].
- EPR according to ETC-C, Part 1 Design (2010) [43].

The earliest requirements for evaluating the seismic hazard in France was published in 1981 in RFS I.2.c [45]. These requirements were replaced by ASN in 1985 through RFS V.2.g [41]. A comprehensive review of RFS V.2.g [41] in 2006, resulted in new guidelines in ASN/Guide/2/01 [42]. These guidelines define the seismic design requirements and acceptable methods for civil works. The requirements for determining the seismic design ground motions are specified in RFS 2001-01 (2001) [40].

The development of the RCC-G [44] began in 1976 when it was decided to establish a working group involving EDF, FRAMATOME-CEA and the French Ministry of Industry, piloted by EDF, to examine the possibility of issuing detailed documents (initially called "Codes and Standards" and then, from 1978, "Rules of Design and Construction”) with the following objectives:

- To serve as a basis for contractual relations between licensees and suppliers
- To facilitate discussions with nuclear safety authorities

The ETC-C [43] is an evolutionary development of the RCC-G [44]. It was undertaken for the design and construction of EPR safety-classified buildings. The reasons for developing ETC-C [43] were as follows:

- It was necessary for the EPR to comply with requirements from both French and German regulations and practices
- New load cases were required to represent severe accident and more severe hazard conditions
- Changes were needed to take into account the Eurocodes in the design of structures
- Updated operational experience feedback as well as current updated safety analysis requirements had to be taken into account
- Updated knowledge of material and structure behaviour from laboratory and mock-up tests had to be incorporated

A previous edition of the ETC-C [43] was issued by EDF in April 2006 and serves as a reference document for the Flamanville 3 project. Since 2009, the ETC-C [43] development has continued under the lead of AFCEN resulting in revised editions in 2010 and 2012.

In ETC-C [43], the safety requirements are achieved through various specifications as regards analysis methods or criteria, such as: linear analysis, requirements to limit cracking in concrete structures, limitation on strains in materials, etc.

2.3.2 Historical development of seismic design ground motions

2.3.2.1 Seismic design ground motions for existing plants

There are in total 58 nuclear power units in operation in France. All of these are of PWR-type and were designed by Framatome. There are three major standard types of designs.

- CP0 and CPY design types (900 MWe). There are in total 34 units still in operation. They were designed and constructed in the 1970s and beginning of the 1980s.

- P4 and P’4 design types (1300 MWe). There are 20 reactors of these design types in operation in France. They were designed and constructed during the late 1970s, the 1980s and beginning of the 1990s.
- N4 design type (1400 MWe). Of this type, there are 4 reactors in operation in France. They were designed and constructed during the 1980s and the beginning of the 1990s.

Accordingly, the design of the existing French nuclear fleet were carried out mainly during the 1970s and 1980s, before any consistent framework of requirements as regards seismic ground motions was established. Therefore the seismic design basis were different for the different design types as follows:

- CP0 and CPY: For the design of the CP0 and CPY plant series, the spectral shape used was that known as the "EDF spectrum", defined as the smoothed mean of eight accelerograms recorded during five earthquakes of Californian origin. The accelerations were normalized according to the local seismicity PGA.
    - CP0: Bugey EDF spectrum anchored at 0.1 g PGA and for Fessenheim at 0.2g associated to local soil conditions.
    - CPY: EDF spectrum anchored at 0.2 g PGA associated to a range of soil’s conditions: 500 to 2000 MPa in terms of dynamic young modulus for the soil.
- P4 and P’4: The DBE for Paluel, the first P4 site, was changed during the course of its construction. At the beginning of construction in the late 1970s, the spectral shape used hitherto for the units was that of the "EDF spectrum". Later during construction, a new spectral shape was taken from that established by the USNRC in RG 1.60 [15], which was also adopted in France as the reference for the design of the 1300 MWe plant series. For the following P4 and P’4 reactors, EDF adopted the RG 1.60 [15] spectrum, normalized to 0.15 g ZPA as the standard DBE applicable to nuclear island design, compatible with the sites chosen for the reactors in this plant series. For the buildings, this led EDF to use the following in turn:
    - For a transitional period, the EDF spectrum anchored at 0.2 g PGA associated with a range of soil conditions: 500 to 15000 MPa in terms of dynamic young modulus for the soil.
    - The RG 1.60 [15] spectra anchored at 0.15 g PGA associated with a range of soil conditions: 500 to 15000 MPa in terms of dynamic young modulus for the soil.
- N4: For the DBE, the RG 1.60 [15] spectrum anchored at 0.15 g PGA was applied at Civaux site and RG 1.60 [15] spectrum anchored at 0.12 g PGA was applied at Chooz site associated with local soil conditions

2.3.2.2 Approach in the Basic safety rules

In 2001 more consistent requirements with regard to seismic design ground motions were established in the Basic Safety Rules in RFS 2001-01 (2001) [40]. The main objectives with this document were to:

- Ensure that safety-related functions being maintained during and after plausible earthquakes that could affect nuclear installations.
- Define acceptable methods for determining the vibratory ground motions to be considered in the seismic design basis.

The basic principles of the approach in RFS 2001-01 (2001) [40] can be summarized as follows:

- The approach is basicly deterministic and assuming that earthquakes comparable to historically known earthquakes are liable to occur in the future.
- The definition of the characteristics of “Maximum Historically Probable Earthquakes (MHPE) considered to be the most damaging earthquakes liable to occur over a period comparable to the historical period of approximately 1000 years.
- The definition of a SSE to account for uncertainties in MHPE, which may be complemented by paleoseismological evidences.

For an envisaged site, an intensity I(MHPE) is determined. In order to take account of uncertainties inherent in the determination of the MHPE characteristics, a fixed safety margin is defined as follows. For each MHPE, a SSE is defined, deduced from the MHPE by the following simple equation in terms of intensity (I)[4] on the site:

I (SSE) = I(MHPE)+1

Except for the particular case when the site is located in the immediate vicinity of an active fault with surface fractures, the SSE are considered as the most aggressive earthquake to be included in the design basis. The SSE can be preceded or followed by earthquakes capable of reaching the MHPE level.

Seismic motion is defined by the response spectra of the horizontal and vertical components of the motion on the surface of the site ground.

2.3.2.3 Seismic design motions for future plants

The design and qualification of SSCs in future plants, such as for instance the EPR, shall consider the EUR standard design spectra in Figure 4 in [46] as shown in Figure 2.4 scaled to 0.25 g horizontal ground acceleration. These standard design spectra can be used under condition that the range of soil characteristics forming the basis of the EUR spectra envelope the specific site soil conditions at the facility. Soil property data for soft, medium and hard soil conditions can be found in section 2.4-6.4.2.1 in [46]. The input ground motion shall be represented by either response spectra or artificial time histories based on a damping value of 5 %. The earthquake excitations shall be represented by two horizontal and one vertical input motions simultaneously. Additional requirements regarding seismic ground motions and seismic analysis are described in Appendix A.1 of ETC-C [43].

[4] Intensity scales measure the amount of shaking at a particular location

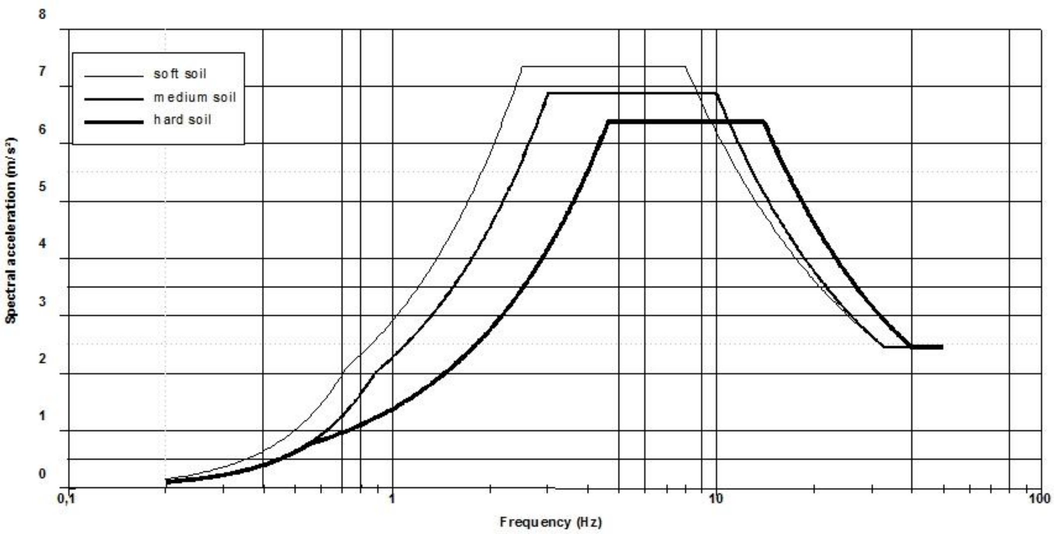

Figure 2.4 – EUR design basis ground motion spectra (horizontal, 5% damping) [46]

2.3.3 Evaluation of seismic safety margins

2.3.3.1 General

The methodology used in France to assess seismic hazards is based essentially on a deterministic approach. The most penalising historical event, MHPE, is considered, to which large margins are added as described in section 2.3.2.2. This approach is supplemented by probabilistic safety assessments (PSA), based on a systematic investigation of the different accident scenarios to determine the probability of unacceptable consequences.

PSA as regards external events have not yet been applied systematically at the French NPPs for external events. Seismic Probabilistic Safety Assessment (SPSA) studies were performed by EDF for the St Alban, Flamanville and Civaux NPPs. The SPSA of the St Alban site was developed within the scope of the third PSR of the 1300 MWe series (P4/P'4-series), and EDF's conclusion was that the PGA return period of the DBE obtained by EDF is equal to about 10 000 years.

In addition, EDF has incorporated some of the seismic safety operating experience feedback from the July 2007 earthquake at the Japanese NPP at Kashiwasaki-Kariwa, with especial consideration of defining the scope of seismic inspections and studying the consequences of a transformer fire.

The external hazards are reassessed periodically in the period safety reviews performed every 10 years. Moreover, the external hazards, particularly earthquakes and flooding, were the subject of a targeted reassessment as part of the European stress tests in 2011.

2.3.3.2 Approch for seismic margin assessments

Within the scope of the European stress tests, EDF identified three margin sources for its overall margin study:

- Margins between MHPE and SSE and between SSE and DBE
- Margins due to the response of the structure
- Margins due to the design criteria for the structures and equipment

A specific study was conducted for the Tricastin site (900 MWe), applying the SMA-method. In addition to the SMA-approach, EDF performed a SPSA for the St Alban site (1300 MWe), as mentioned in section 2.3.3.1. For the next PSR of the 900 MW plants it is considered to perform a robustness analysis based on a seismic PSA or SMA.

On the request of ASN, EDF performed some complementary safety assessments on the basis of the available information as follows:

- To give an evaluation of the level of earthquake beyond which the loss of fundamental safety functions or fuel damage (in vessel or pool) are inevitable.
- To identify weak points and cliff-edge effects.
- To propose measures to prevent these cliff-edge effects and reinforce the robustness of the facility.

EDF reviewed the seismic strength margins of the structures and equipment important to safety, in order to determine the level of acceleration for which, with a high level of confidence, the facility has a very low probability of failure (HCLPF).

EDF supplemented its general study with studies of equipment for which there could be performance discontinuities, together with proposed modifications or reinforcements.

Finally, EDF carried out the seismic inspection of a sample of the equipment needed to operate the unit in the event of total loss of off-site and on-site power supplies, whether or not seismic-classified, for all the NPPs in service.

2.3.3.3 Seismic margin assessment of existing plants

EDF used a plant series response spectrum as the design spectrum for all the reactors of the same series. For each site, EDF proposed a table of margin factors between the reassessed site SSE and the DBE, between 1 Hz and 6 or 10 Hz. EDF adopted margin values between 1 and 1.7 depending on the sites and the buildings considered.

EDF concluded that the seismic capacity of the containment and equipment which, in the event of failure, would compromise the safety functions, is at least 1.5 times larger than the forces and stresses resulting from the SSE. EDF considered that this level easily exceeds the seismic context of the sites, up to hazard values that are hardly plausible or implausible for these sites.

A robustness study regarding significant damage of the fuel assemblies identified no failure of the necessary safety systems for a hazard equivalent to 1.5 times the SSE.

Beyond the current safety requirements, EDF proposed additional measures to prevent the serious consequences of extreme situations, on a deterministic basis, regardless of their plausibility. EDF proposed defining a hard core of reinforced equipment aimed to prevent severe accidents and to avoid significant radioactive releases into the environment, over and above the current safety requirements, for the deterministic situations studied in the complementary safety assessments, as described in section 1.1.3.

2.3.3.4 Seismic margin assessment of new plants

The EPR was originally designed to withstand the DBE according to EUR [46]. This DBE is defined by standard design acceleration and a set of seismic spectra reflecting the European conditions.

- A set of three European DBE spectra is given for “hard”, “medium”and “soft”soil, as shown in Figure 2.4.

- A standard design acceleration level of PGA = 0.25 g is required for these three spectra.
- It shall be demonstrated that the standard plant remains in a safe condition for the whole range of parameters (soil conditions) for the DBE.

An Operating Basis Earthquake (OBE) has not been specified in the EUR [46].

The standard design of the EPR is not intended to envelop all possible combinations of national regulations and site conditions. If necessary, the designer may have to make modifications or additional studies to ensure the standard design is satisfactory for particular sites. The SSE is usually defined in accordance with international practice for an annual probability of exceedance of 10^{-4} (i.e. a return period of 10 000 years)

A site-specific seismic margin analysis of the structures and equipment shall be carried out, to ensure that adequate safety margins exist in the seismic design of the main structures and components beyond the design basis conditions.

Hence, it is necessary to choose a Review Level Earthquake (RLE). According to EUR [46], the design shall withstand potential earthquakes with a margin of 40 % on the horizontal PGA above the design SSE level. At present, a 60 % margin is targeted for the EPR. The objective shall be to establish the seismic capability of a minimum set of plant structures and systems needed to avoid core damage, then bring the plant to and maintain it in a safe shutdown state. This demonstration shall be made following a best-estimate methodology. The assessment shall identify the items without sufficient margins in the capacity of the design. For items without sufficient margins, a comprehensive SMA program shall be established using analyses and tests.

The SMA-RLE is defined for a lower annual probability of exceedance than the SSE, e.g.10^{-5}. By this approach it is demonstrated that there are no cliff-edge effects from earthquakes slightly exceeding the design basis.

2.3.3.5 Seismic margin assessment for UK-EPR

The seismic margin of the UK-EPR is assessed by a PSA-based SMA, following a methodology developed by the USNRC. This approach uses the PSA model to identify combinations of seismic equipment failures which could result in core damage, as well as combinations of seismic failures, random failures and human errors which contribute significantly to seismic risk. By identifying which equipment items and structures are of critical importance in seismic events, the analysis approach ensures that vulnerabilities in the design are identified allowing them to be corrected if necessary, thus helping ensure that the seismic risk is ALARP.

The UK-EPR is designed to resist an enveloping seismic event which is bounding for NPPs constructed in Western Europe, with a Design Basis Earthquake (DBE) corresponding to a 0.25 g peak ground acceleration in the horizontal direction, according to Figure 2.4. SSCs required for controlling and mitigating accidents are designed so that they are able to fulfil their functions, maintain their integrity or remain stable under the conditions caused by the seismic motion.

The detailed PSA-based SMA is performed for at-power states. A simplified approach is used for shutdown states. Also, at this first stage, internal hazards that might be caused by a seismic event, such as fire or flooding, are not analysed in detail and are not included in the PSA model supporting the SMA.

The purpose of the SMA is to show that the SSCs critical to achieving a safe shutdown state following an earthquake are designed with large safety margins so that they have a low probability of failure in the Review Level Earthquake (RLE), which has a PGA of 1.6 times that assumed for the DBE. The first step in this process is to define the ground motion spectrum for calculating the seismic capacities (fragilities) of the SSCs. The ground motion spectrum is a characteristic of the EPR site in question.

For the purposes of the SMA, the free field ground motion spectra used as input data for the estimation of seismic capacities of equipment and structures, are a bounding 'hard' and 'soft' site spectra derived by enveloping Uniform Risk Spectra (URS) for prospective UK new build sites.

To perform the SMA, it is necessary to produce a Seismic Equipment List (SEL) containing the SSCs whose seismic capacities need to be evaluated for the SMA. The seismic fragility analysis for these SSCs is then performed.

The SEL for the UK-EPR SMA is developed using expert judgements in combination with the Level 1 PSA model. The use of the PSA model to identify critical combinations of component failures serves to confirm the completeness of the SEL.

The fragility assessment of the SSC items in the SEL evaluates the PGA at which their response will exceed a threshold of acceptability for the characteristic motion spectrum adopted. As noted above, the motion spectrum depends on the ground conditions.

The fragility assessment of the SSCs considers the capacity to withstand ground motion of each component and its associated uncertainties. The capacity is defined as the free field PGA value for which the seismic response at the component location exceeds the component resistance capacity, resulting in the probability of failure of the SSC, i.e. the probability that the response exceeds a defined threshold. The PGA capacity of the SSCs is estimated using information on the plant design and ground parameters, test data from SSC qualification and fragility tests, data from generic seismic tests, earthquake experience results, material property data, etc. Where identical components occur in different redundant trains, the seismic capacity of all the components is conservatively set to that of the most vulnerable component, taking no benefit for the redundancy of the system. Similarly the seismic capacity of electrical cables is assumed to be the capacity of the cable tray anchorages, which is conservatively set to the capacity at the most seismically vulnerable anchorage location.

The SMA shows that the UK-EPR could tolerate a seismic event with 0.61g PGA without significant risk of a severe accident and release of radioactivity from the plant. Therefore, it has been demonstrated that the seismic capacity of the UK EPR is higher than the RLE defined as 1.6 times the Design Basis Earthquake (i.e. corresponding to $1.6 \cdot 0.25g = 0.4g$ PGA). The SMA has shown that there are no cliff edge effects for seismic events with magnitudes above that assumed in the design basis.

2.4 Sweden

2.4.1 Introduction

The Swedish Radiation Safety Authority (SSM) superintend the Swedish nuclear facilities as well as all other activities in the country within the areas of nuclear safety, radiation protection and nuclear non-proliferation. The SSM reports directly to the Ministry of the Environment.

The hierarchy of requirement in Sweden in order to regulate the design and analysis of NPPs can be categorized as follows:

- The Act on Nuclear Activities (Kärntekniklagen) and the Radiation Protection Act (Strålskyddslagen). These are laws passed by the Swedish Parliament.

- SSM Regulations (SSMFS). Regulations issued by SSM. These regulations are mandatory with the intention to provide more specific requirements on how to implement the Act on Nuclear Activities and the Radiation Protection Act.
- General recommendations to SSMFS. These general recommendations are issued by SSM. They are not mandatory but provides recommendations on the authority's view on the compliance with the Laws and the Regulations (SSMFS).
- Codes & Standards for design and construction. The most important standard for nuclear structures is applicable parts of the Eurocodes [52].

2.4.2 Historical development of the seismic design basis

The seismic activity in Scandinavia is low. Historically there are only a few registered events, which might have caused damage to an industrial facility [47]. Thus, earthquake load effects on buildings in Sweden have been regarded as negligible compared to other loads to be expected during the lifetime of a building. Accordingly, the design criteria for the oldest Swedish nuclear power facilities in the beginning of the 1970s did not include any requirements on structural integrity or maintaining safety functions due to earthquake ground motions.

Concurrently with an increasing safety consciousness in the nuclear power industry, there was an increased understanding that seismic effects must be regarded for the Swedish nuclear facilities. Thus, the design criteria for the latest Swedish NPPs, Forsmark unit 3 and Oskarshamn unit 3, designed at the end of the 1970s included seismic load requirements. Due to lack of statistical data regarding larger earthquakes in Scandinavia, design response spectra anchored at 0.15 g PGA horizontally and 0.10 g vertically according to USNRC RG 1.60 [15] were applied.

With the purpose of deriving ground motions to be used in the safety analysis of the Swedish nuclear power facilities, a joint project was initiated in the mid 1980s between the then Swedish nuclear safety authority (SKI) and the Swedish nuclear power licensees. The project results are presented in SKI Technical Report 92:3 [8]. In this report, envelope ground response spectra corresponding to a certain annual probability of exceedance (10^{-5}, 10^{-6} and 10^{-7}) are defined for typical Swedish hard rock sites.

SS-EN 1998-1:2004; Eurocode 8 [48] has been in force since 2009 and applies to design and construction of buildings and civil engineering works in seismic regions in Europe. The main purpose with this standard is according to SS-EN 1998 [48], section 1.1.1 to protect human lives, to limit damages and to secure that structures important for civil protection remain operational.

It must be observed that SS-EN 1998 [48] only includes complementary reuirements in addition to the requirements of the other relevant Eurocodes, to be applied for the design of structures in seismic regions. In this respect, SS-EN 1998 [48] is a complement to the other Eurocodes.

An important limitation with SS-EN 1998 [48] is that it, as for other parts of the Eurocodes, formally does not apply to nuclear power plants, offshore structures and large dam structures.

According to SS-EN 1998 [48], structures in seismic regions are recommended to be designed and constructed to withstand a design seismic action associated with a reference probability of exceedance of 10% in 50 years or a reference return period of 475 years. The hazard is described in terms of the value of the reference PGA, which may be derived from zonation maps found in National Annex.

According to the Swedish National Annex to SS-EN 1998 [48], the Swedish National Board of Housing, Building and Planning (Boverket) has not found it necessary to issue any regulations or recommendations with regard to seismic load effects, since SS-EN 1998 [48] would only be used in very specific cases where special expertise is required. The Swedish Transport Administration (Trafikverket) specifies as well its position in the Swedish National Annex to SS-EN 1998 [48]. Trafikverket states that earthquake loads do not need to be considered in Sweden, since the other parts of the Eurocodes normally ensure the strength and durability of the structure for those earthquake hazard levels that could arise in Sweden.

One fundamental question is what values of the reference PGA are applicable in Sweden? In [53], a hazard map corresponding to a mean return period of 475 years, i.e. the recommended reference period in SS-EN 1998 [48], is presented for Fennoscandia, as shown in Figure 2.5. From this map the horizontal PGA for the different NPP sites in Sweden can be identified, according to Table 2.2. As can be noticed in Table 2.2, the maximum horizontal PGA for a mean return perod of 475 years is 0.15 m/s^2 for Forsmark and Oskarshamn NPP and 0.25 m/s^2 for Ringhals site. Maximum PGA in Sweden arises in the territory inbetween Lake Vänern and the Norwegian border where it can reach values of 0.35 m/s^2.

According to SS-EN 1998 [48], a *low seismicity* case is defined as a region having a design PGA not greater than 0.78 m/s^2 (0.08g) and a *very low seismicity* case a region having a design PGA not greater than 0.49 m/s^2 (0.05g). It is also stated that in cases of *very low seismicity*, the requirements of SS-EN 1998 [48] need not to be observed. Hence, on the basis of Figure 2.5 and Table 2.2, it can be concluded that the whole Swedish territory can be classified as a *very low seismicity* case and the requirements of SS-EN 1998 [48] can be disregarded, as also are concluded by Boverket and Trafikverket in the National Annex to SS-EN 1998 [48].

However, the safety conditions for safety-related structures at NPP sites in Sweden differ from the conditions for bridges, conventional structures and industrial facilities. Safety-related structures at nuclear facilities house important safety systems, which in case of failure could result in severe and unacceptable consequences for the personnel, the off-site public or the environment. Hence, safety-related structures at nuclear facilities should be designed against external and internal hazards with higher reference return period than conventional structures and facilities. SSM has also in SSMFS 2008:17 [49] specifically mentioned earthquake as one of several natural events that the Swedish nuclear power plants must prove resistance to.

Within the framework of the large modernization and power uprate projects at the Swedish NPP sites during the recent 20 years, new installed SSCs have been designed and existing SSCs been evaluated against a DBE corresponding to an annual exceedance probability of 10^{-5} according to SKI Technical Report 92:3 [8] and shown in Figure 2.6.

Within the scope of the European stress tests, the resistance of the Swedish NPPs were assessed against hazardous natural events with lower annual exceedance frequencies than 10^{-5}. Assessments were performed for earthquakes, flooding and extreme weather conditions, as further described in section 1.1.4.

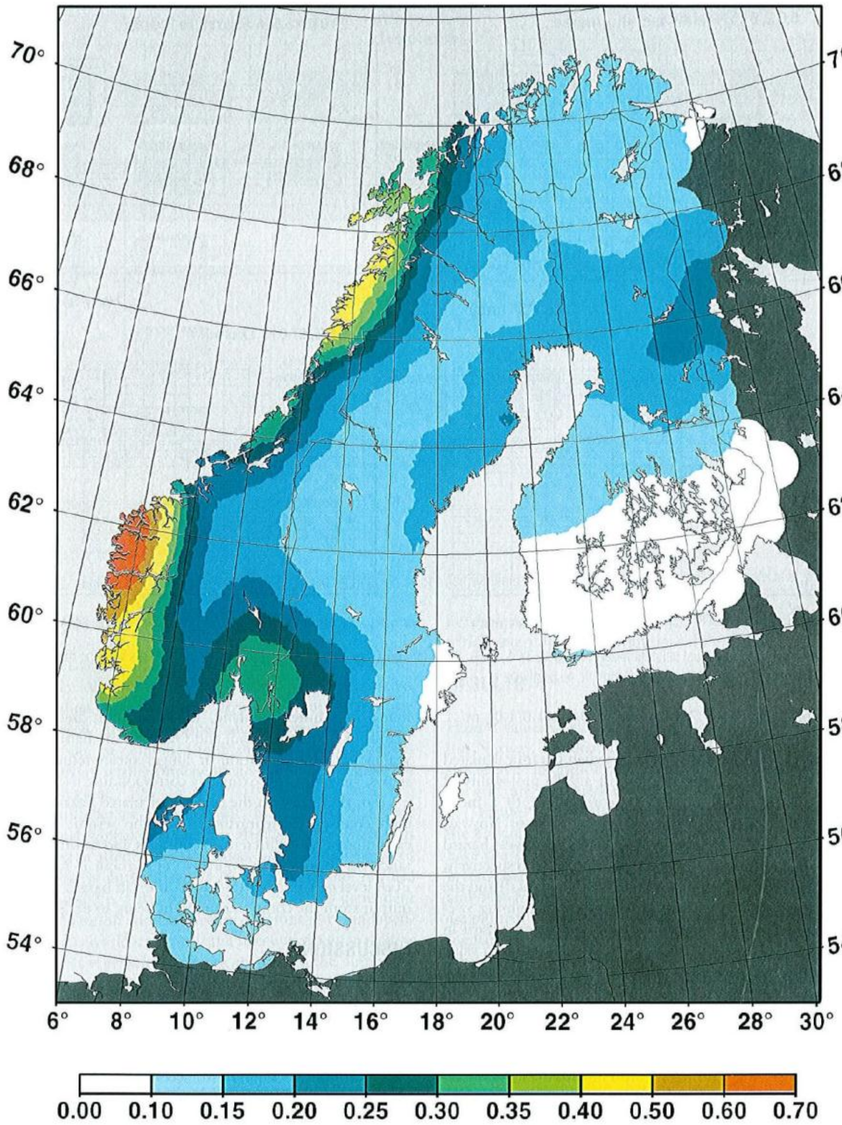

Figure 2.5 – Map of 90% probability of nonexceedence of horizontal PGA (m/s^2) in 50 years, corresponding to a mean return period of 475 years, from [53].

Table 2.2 – Maximum reference horizontal PGA (m/s^2) corresponding to a mean return period of 475 years for the Swedish NPP sites.

NPP Site	Maximum horizontal PGA
Forsmark	0.15 (m/s^2) = 0.015 g
Oskarshamn	0.15 (m/s^2) = 0.015 g
Ringhals	0.25 (m/s^2) = 0.025 g
Swedish territory	0.35 (m/s^2) = 0.035 g

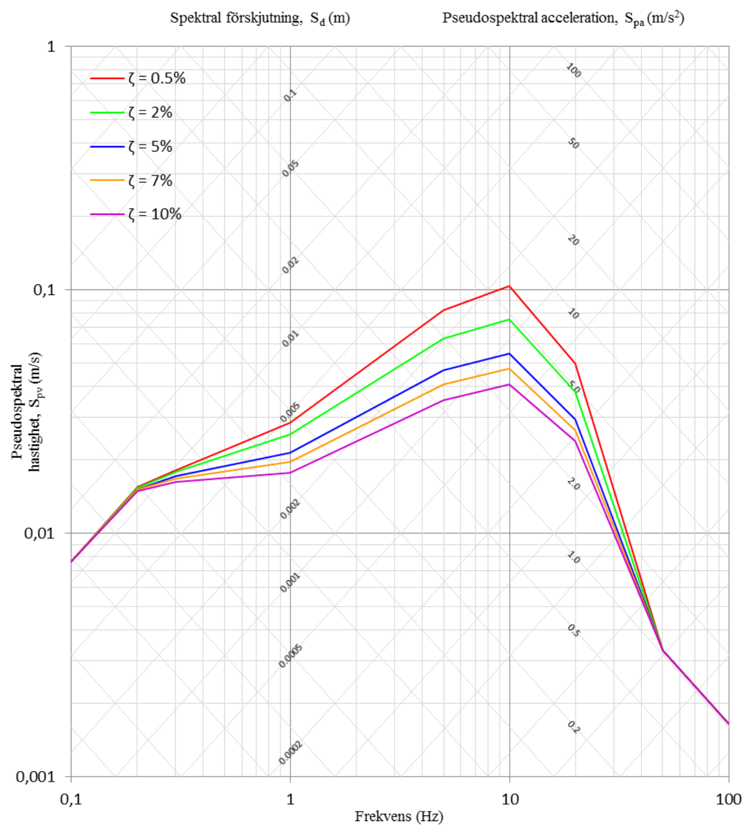

Figure 2.6 - Envelope ground response spectra for a typical Swedish hard rock site, corresponding to an annual exceedance probability of 10^{-5}.

2.4.3 Seismic design classification

At the Swedish nuclear facilities, those SSCs which are identified to possess safety functions necessary to be maintained during and after an earthquake, and even SSCs which although not maintaining safety functions but for which loss of function could jeopardize the function of a safety-related equipment, shall be designated a Seismic Category. There are three Seismic Categories (1, P and N) depending on type of safety function, as shown in Table 2.3.

Table 2.3 – Seismic design classification for SSCs at Swedish NPP

Seismic Category	Structures	Piping system	Pump/valve	Electrical components
1	Leak-tightness	Passive function 1)	Active function	Active function
P	Load-bearing function	Mechanical integrity	Mechanical integrity	-
N	No demand 2)	No demand 2)	No demand 2)	No demand 2)

1) Refers for instance to ensure free flow of water or steam.

2) No demand as regards leak-tightness, load-bearing function or mechanical integrity. But SSCs in Seismic Category N should not challenge any safety function of SSCs in Seismic Category 1 or P.

Examples of typically requirements for building elements in respective Seismic Category can be found in Table 2.4.

Table 2.4 – Examples on various types of requirements on building elements

Seismic Category	Safety function	Requirement (examples)
1	Leak-tightness	Leak-tightness over the steel liner in the containment vessel.
		Leak-tightness over the steel containment lid (BWR).
		Leak-tightness over equipment hatches and openings in the containment vessel.
		Leak-tightness over casing tubes around penetrations in the containment vessel.
		Leak-tightness between the primary and secondary compartment in BWRs.
		Leak-tightness over the steel liner in Spent Fuel Pools (SFP)
		Leak-tightness over building elements for protection against leakage from vessels in waste buildings.
		Leak-tightess of culverts, against leakage from enclosed piping containing radiactive waste in fluid phase.
P	Load-bearing function	Structural integrity of the load-bearing system.
		To provide support and to shield safety systems and components attached to the structural system.
N	No demand	No demand as regards leak-tightness, load-bearing function or mechanical integrity. But SSCs in Seismic Category N should not challenge any safety function of SSCs in Seismic Category 1 or P.

2.4.4 Codes and Standards

Within the scope of the modernization and power uprate projects at the Swedish nuclear facilities during the 2000s, conditions and evaluation criteria according to DRB:2001 [54] were generally used. In DRB:2001 [54], conditions and requirements for new design and for evaluation or analysis of existing safety-related structures at NPPs are formulated. The rules in DRB:2001 [54] were based on the then general Swedish building code, BKR [55], together with additional conditions and requirements specifically applicable for building structures at NPPs.

However, since 2011 all building activities in Sweden must comply to the Eurocodes [52], and the conditions and requirements in BKR [55] and DRB:2001 [54] are no longer applicable. The statute book of SSM does not yet include specific requirements and adequate guidance on how safety-related structures at NPPs should be handled in safety analyses of existing structures as well as in case of the design of new constructions. Therefore, the SSM together with the Swedish licensees commissioned Scanscot Technology AB to further develop DRB:2001 [54] to be based on the Eurocodes [52] instead. The scope of work was accomplished during 2012 and 2013 and the first version of DNB [51] was released in January 2014.

The scope of DNB [51] includes instructions regarding design and analysis of loadbearing concrete structures, covering reactor containments as well as other safety-related structures. The main aim with DNB [51] is to complement the requirements in the Eurocodes [52] for application at NPPs in Sweden. Thus, DNB [51] is based on the partial factor method and the principles of design in limit states, as specified in the Eurocodes [52] including the National Determined Parameters chosen by Swedish Authorities.

DNB [51] covers conditions, requirements and acceptance criteria for seismic design and analyses and can be applied for design of new nuclear structures as well as for evaluation of existing facilities. The scope and application of DNB [51] are described in more detail in section 4 to 6.

3. Design basis ground motions

3.1 General

In section 2.4.2 a survey of the historical development of the seismic design basis in Sweden is described. Recommendations regarding applicable hazard frequencies have been addressed in the European stress tests of the European NPPs, as described in section 1.1.2 to 1.1.4.

On the basis of the outcome of the survey and the European stress tests as well as applicable parts of the IAEA Safety Guides, some recommendations will be provided in the following as regards a reasonable seismic design basis for the Swedish nuclear facilities.

3.2 Design basis considerations

3.2.1 Basic requirements according to IAEA Safety Guides

In section 2 in IAEA SG-G-1.6 [50], it is stated that two levels (SL-1 and SL-2) of ground motion hazard should be evaluated for an NPP. SL-1 corresponds to a level with a mean annual exceedance frequency of $1{\cdot}10^{-2}$, while SL-2 corresponds to a mean annual exceedance frequency in the range of $1{\cdot}10^{-3}$ to $1{\cdot}10^{-4}$ or a median annual exceedance frequency in the range of $1{\cdot}10^{-4}$ to $1{\cdot}10^{-5}$. Additionally also for the SL-2 level, the PGA should be at least 0.1 g regardless of the actual seismic hazard.

When probabilistic seismic hazard analysis is used, either a reference annual exceedance frequency in the U.S. according to RG 1.165 [21] as discussed in section 2.2.2, or a performance based approach according to RG 1.208 [22] is allowed according to section 9.2 in IAEA SSG-9 [6]. As stated in section 9.4 in IAEA SSG-9 [6], a uniform hazard response spectrum shall be developed by determining response spectral ordinate values that correspond to annual exceedance frequencies on the basis of seismic hazard curves.

The SL-1 correspond to the Operating Basis Earthquake (OBE), which normally relates to operational requirements in case this is required by safety authorities. However, in IAEA SG-G-1.6 [50] the SL-1 is addressed only in relation to safety analysis and design considerations.

In this connection, it should be noted that in the U.S in seismic design of new standardize NPPs, the use of the OBE as a design basis is only necessary if the licensee defines the OBE as exceeding ⅓ of the SSE (IAEA SL-2), which has not been done for any new standard NPP licensed in the U.S to date.

3.2.2 Aspects as regards an eventual Operating Basis Earthquake (OBE)

According to IAEA SG-G-1.6 [50], SL-1 (OBE) corresponds to a seismic hazard level with a mean annual exceedance frequency of $1{\cdot}10^{-2}$ as stated in section 3.2.1. Curves of annual seismic hazard at 16%, 37%, 50% (median), 63% and 84% fractiles and the mean for the region with the highest hazard in Sweden, i.e. the region between Lake Vänern and the Norwegian border, are presented in [53], as shown in Figure 3.1. From this figure, the mean PGA can be determined to be approximately 0.15 m/s^2 = 0.015 g. This very low PGA level would only result in negligible effects on safety-related structures as well as on safety equipment housed in the buildings.

Hence, it can be concluded that an OBE as defined in IAEA SG-G-1.6 [50] would result in negligible effects on safety-related SSCs at nuclear facilities in Sweden and need therefore not be considered. Finland has similar seismic conditions as in Sweden and as a comparison, the OBE is not either required by STUK in their provisions for internal and external hazards at a nuclear facility (YVL B.7 [56]).

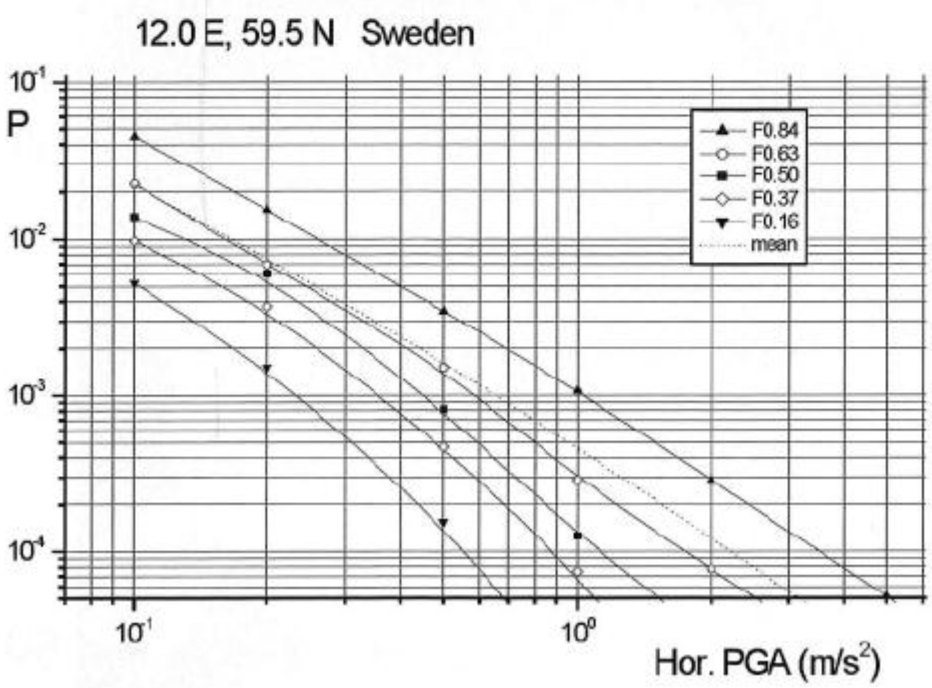

Figure 3.1 - Curves of annual seismic hazard (expressed in horizontal PGA) at 16%, 37%, 50% (median), 63% and 84% fractiles and the mean for the region with the highest hazard in Sweden, according to [53].

3.2.3 Requirements as regards the Design Basis Earthquake (DBE)

SL-2 corresponds to an earthquake level denoted as a Safe Shutdown Earthquake (SSE) or more generally designated as the Design Basis Earthquake (DBE).

As stated in section 3.2.1, level SL-2 corresponds to a mean annual exceedance frequency in the range of $1 \cdot 10^{-3}$ to $1 \cdot 10^{-4}$ or a median annual exceedance frequency in the range of $1 \cdot 10^{-4}$ to $1 \cdot 10^{-5}$, according to the recommendations in IAEA SG-G-1.6 [50]. The minimum PGA should be 0.1 g regardless of the actual seismic hazard at the site.

Within the scope of the European stress tests, the ENSREG peer review team recommended in [2] that external events should be addressed by designing to a hazard level corresponding to an annual exceedance frequency of $1 \cdot 10^{-4}$. In some countries the acceleration levels at the target annual exceedance level $1 \cdot 10^{-4}$ are very low and therefore ENSREG also recommended a minimum horizontal PGA of 0.1 g, with reference to the IAEA Safety Guides. It is not obvious from ENSREG's conclusions in [2] if the recommended annual exceedance level $1 \cdot 10^{-4}$ relates to the median (50% fractile) or the mean hazard. But judging from the fact that IAEA recommends the usage of mean values for probability level $1 \cdot 10^{-4}$ in IAEA SG-G-1.6 [50] , it seems reasonable that ENSREG refers to the mean curves as well.

In this connection it is important to distinguish between mean and median hazard values. As shown in Figure 3.1, the mean hazard values approximately target the 63% fractile curve for relatively large annual exceedance frequencies ($>10^{-3}$), while the mean hazard curves approach even larger fractiles for annual exceedance fractiles less than 10^{-3}. As stated in section 2.2.2, the mean annual exceedance frequency 10^{-4} approximately equals the median annual exceedance frequency 10^{-5}, which also is reflected in the IAEA recommendations as stated in section 3.2.1.

The median fractile curves can be considered as best estimates, while the 84% fractile is a conservative estimate consistent with general engineering practice in case of design. Historically in the U.S, there has been focus on generating design response spectra at the 84% fractile level. For instance, the design response spectra in RG 1.60 [15] are specified at that level.

3.2.4 Current design ground motions in SKI Technical Report 92:3

As stated in section 2.4.2, it can be concluded that the whole Swedish territory can be classified as a *very low seismicity* case and therefore the seismic requirements of SS-EN 1998 [48] can be disregarded, as also is concluded by Boverket and Trafikverket in the National Annex to SS-EN 1998 [48]. But of course, safety-related structures at nuclear facilities must be designed against external and internal hazards with much higher reference return period than conventional structures and facilities. SSM has also in SSMFS 2008:17 [49] specifically mentioned earthquake as one of several natural events that the Swedish nuclear power plants must prove resistance to.

The main purpose with project Seismic Safety in the beginning of the 1990's was to provide ground response spectra in accordance with Swedish geological and seismological conditions. The result from this project is presented in SKI Technical Report 92:3 [8], in which ground response spectra are presented for various annual exceedance probability levels (10^{-5}, 10^{-6} and 10^{-7}). Spectra were presented for typical hard rock conditions, but also specifically for the Barsebäck NPP, which was decommissioned in 2005.

The hard rock spectra developed within the scope of Seismic Safety and presented in SKI Technical Report 92:3 [2] were based on a number of accelerograms from registred earthquakes in Japan. These Japanese spectra were transformed to Swedish spectra considering the differences in fault mechanisms and geological conditions between these regions. However variations in wave velocities along the wave paths due to variations of the hardness of the rock were not considered. Later research showed that the Japanese recordings originated from sites were the hardness variation, characterized by the ratio v_{so}/v_{sb} between the shear wave velocities at the surface and the basement rock were considerable. Some of the Swedish licensees claimed that the influence of this hardness variation on the results presented in SKI Technical Report [2] could be considered by reducing the hard rock spectra values by 15%.

As stated in section 2.4.2, new installed SSCs have been designed and existing SSCs been evaluated against a DBE corresponding to an annual exceedance probability of $1 \cdot 10^{-5}$ according to SKI Technical Report 92:3 [8] and shown in Figure 2.6. This design hazard level can be compared with the corresponding hazard in south Finland, which also refers to an annual exceedance probability of $1 \cdot 10^{-5}$ at the median confidence level and with a PGA of 0.1g according to YVL B.7 [56], and shown in Figure 3.2.

In Figure 3.2, envelope ground response spectra corresponding to an annual exceedance probability of $1 \cdot 10^{-5}$ with a horizontal PGA of 0.11 g (i.e. the current Swedish DBE), are in the same figure compared to the following site-independent or site-dependent horizontal design response spectra at 5 % critical damping:

- Finnish STUK YVL Guide B.7, PGA=0.1 g
- USNRC RG 1.60, PGA=0.11 g
- Eurocode 8 (SS-EN 1998-1:2004), Type 2, Ground type A (rock), PGA = 0.11 g

- EUR, hard soil, PGA=0.25 g

The EUR ground response spectra, which form the design basis for the EPR, as described in section 2.3.2.3, have a PGA of 0.25 g and are thus enveloping all other design spectra with a PGA of 0.1 or 0.11 g. As can be noticed, the Swedish DBE spectra will give lower acceleration responses compared with all other design spectra in the important frequency range below 9 Hz, where the fundamental eigenfrequencies of the reactor containments and auxiliary buildings will arise. On the other hand, the Swedish DBE spectra will create higher responses in the frequency range above 9 Hz (except for the EUR spectra), thus being more vulnerable to certain safety installations primarily associated with electrical and instrumentation control devices in the buildings.

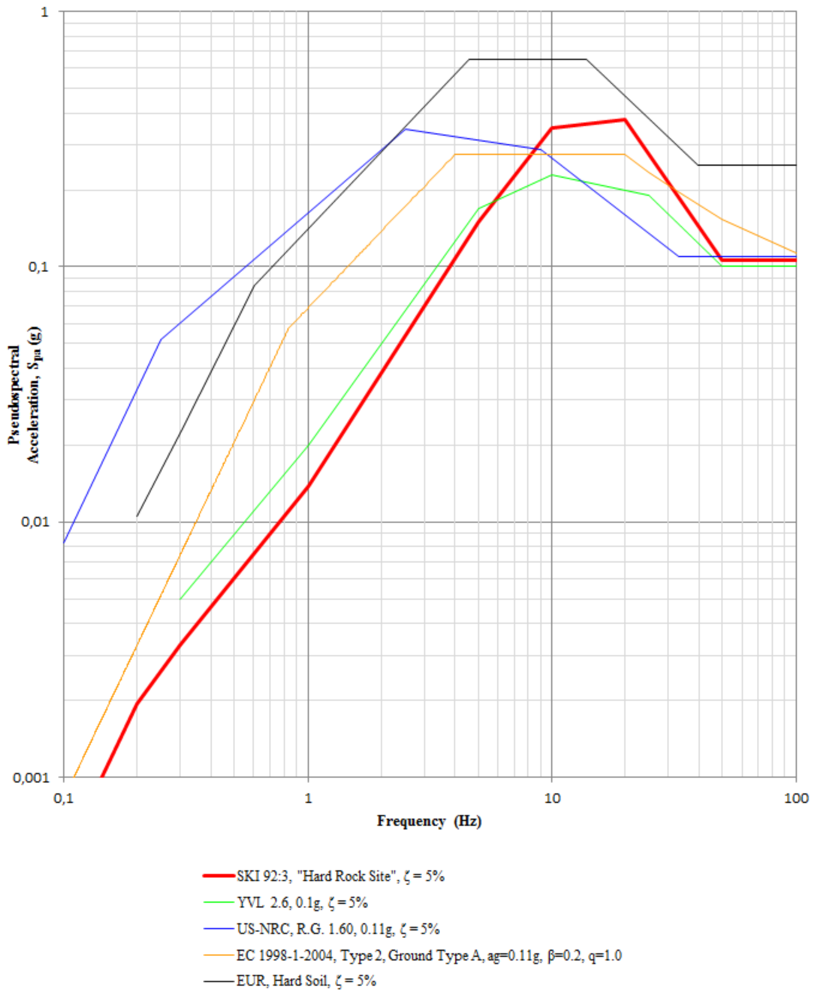

Figure 3.2 - Comparison between the Swedish DBE ground response spectra (10^{-5}) and other international well-known DBE response spectra.

SKI Technical Report 92:3 [8] was reviewed by ENSREG peer review team within the scope of the European stress tests. As stated in [3], ENSREG notified that the seismic hazard in SKI Technical Report 92:3 [8] is based on observations and historical accounts for about 500 years in Fennoscandia, together with transformed statistical data from earthquakes occurred in Japan. ENSREG questioned this short geological time scale, due to the fact that geodetic and paleoseismologic data indicates continuous active uplift and deformation of Fennoscandia, according to some researchers. Besides plate techtonics, isostatic rebound may be a main contributor in the seismogenic process in Fennoscandia, according to for instance [57]. One theory to the cause of earthquakes in Fennoscandia is the release of stresses built up and propagated from the North Atlantic Ridge versus stress adjustment connected to the postglacial land uplift.

According to section 3.25 in IAEA SSG-9 [6], all data on historical earthquakes, including also events for which no instrumental recording were possible, should be collected as far back in time as possible. Paleoseismic and archeological information on historical and prehistoric earthquakes should also be taken into account. (The term “paleoseismic” refers to the evidence of a prehistoric or historical earthquake displayed as for instance displacements on a fault or secondary effects as ground deformations). Paleoseismic studies may be particularly useful in areas for which historical earthquake records are lacking, like for instance in Fennoscandia. As stated in the Swedish action plan [7], SSM will start up a research program regarding the influence of paleoseismological data on the existing Swedish seismic design basis.

The approach for determining the hard rock spectra in SKI Technical Report 92:3 [8] was adopted in the 1980s and the applied statistical database, including the one from Japan, includes earthquake events until the 1970s. During the last 30 years the methodologies for addressing seismic hazard have been extensively developed and several European projects have been active at global and regional scales. The development of the Eurocodes during the 1990s and 2000s highlighted the need for development of a homogenous and unified probabilistic seismic hazard assessment procedure for Europe. This development has resulted in the first ever unified source model throughout the whole European-Mediterranean region and the generation of seismic hazard maps, expressing ground motion in different parameters, for different soil conditions and probability levels, some examples for Fennoscandia are shown in Figure 2.5 and Figure 3.1. The seismic hazard assessments for buildings and civil works in the Eurocodes [52] are based on these hazard maps.

Section 3.18 to 3.20 in IAEA SG-G-1.6 [50] defines how periodic safety reviews should be addressed at NPPs. As described in recommendation 2.2 in [36] USNRC recommends licensees to confirm seismic and flooding hazards every 10 years and address any new and significant information. In France, the external hazards are as well reassessed periodically in the period safety reviews performed every 10 years, as stated in section 2.3.3.1.

3.2.5 Recommendations on revised design ground motions

With reference to the discussions, evaluations and conclusions stated in previous sections 3.2.1 to 3.2.4, some recommendations are presented regarding a revised seismic design basis for defining the seismic input for safety analysis and design of nuclear facilities in Sweden.

As described in section 3.2.4 there are today a number of deficiencies in the characterization of seismic ground motions in SKI Technical Report 92:3 [8]:

- The applied statictical database is from the 1970s and are thus lacking input from seismic events from the last 30-40 years. According to international recommendations from safety authorities the seismic hazard assessments should be confirmed every 10 years and address any new and significant information.
- The applied methodology for addressing the seismic hazard during project Seismic Safety was probably in line with the scientific position at that time. But these methods have been exensively developed and unified at an European level during recent years.
- Within the scope of the European stress tests, ENSREG has questioned the very short geological time scale (500 years) for applied observations and historical accounts and the lack of paleoseismic data.

On the basis of these shortcomings the following recommendations are proposed:

- Replace SKI Technical Report 92:3 [8] with a new model for seismic hazard assessments on the basis of the last 20 years research and development within the subject field.
- Establish annual seismic hazard curves for different fractiles (16%, 37%, 50%, 63% and 84%) and the mean, for different regions in Sweden and for annual probability of exceedance in the range 10^{-2} to 10^{-7}.
- Develop new ground response spectra for various annual probability of exceedance, if possible also for the different NPP sites in Sweden.
- Anchor the Design Basis Earthquake (DBE) at a minimum PGA of 0.1 g.
- Confirm the seismic hazard assesments every 10 years.

Additional recommendation:

- An eventual Operating Basis Earthquake (OBE) with a recommended annual probability of exceedance according to IAEA SG-G-1.6 [50] would results in neglectible effects on SSCs and need not to be considered at nuclear facilities in Sweden.

4. Seismic analysis methods

4.1 General

There are a number of different handbooks covering various aspects regarding modeling and analysis of structural dynamic systems. ASCE 4-98 [24] is a standard which provides minimum requirements and acceptable methods for seismic analysis of safety-related nuclear structures. This standard provides a comprehensive survey of the seismic analysis process, also addressing requirements on seismic input and input for subsystem seismic analysis. ASCE 4-98 [24] covers in principle all applicable requirements in Regulatory Guides and Standard Review Plans issued by USNRC before 1998, for instance RG 1.61 [58], RG 1.92 [27], SRP 3.7.1 [59] and SRP 3.7.2 [60] and provide more extensive background information to the intentions behind the requirements compared to the official USNRC documents.

ASCE 4-98 [24] provides much more stringent and robust requirements on structural analysis methods, reflection the enhanced demands for nuclear facilities, compared to what is common practice in standards for conventional buildings, as for instance in SS-EN 1998 [48]. ASCE 4-98 [24] has undergone a major upgrading and has been approved for a revised publication in 2014 as an ASCE/ANSI Standard.

The recommendations in the following sections are based on the requirements of ASCE 4-98 [24] complemented with later provisions from USNRC issued after 1998. In order to comply with the Eurocodes [52], complementary input as regards material parameters and design considerations are also provided. These set of requirements are applicable in the design process of new structures as well as for seismic evaluation of existing facilities in Sweden. All proposed recommendations are in all essentials in accordance with applicable parts of IAEA SG-G-1.6 [50].

In this connection it is also worth mentioning that the technical recommendations in this section may also be applicable in part to the design of a nuclear facility against vibrational phenomena induced by sources other than erathquakes, as for instance explosions, aircraft crashes or accidents with high speed rotating machinery, as stated in section 1 of IAEA SG-G-1.6 [50].

4.2 Structural modeling

4.2.1 General requirements

ASCE 4-98 [24], section 3.1.1 defines some general basic requirements on modeling of structures.

4.2.2 Material properties

In linear elastic analysis of concrete structures, for calculation of eigenfrequencies as well as for determining sectional forces and moments in the structural elements, mean value of the modulus of elasticity (E_{cm}), according to the principles in SS-EN 1992-1 [61], section 5.4 can be used. The value of E_{cm} is then calculated according to SS-EN 1992-1 [61], Table 3.1.

Recommended value of the Poisson's ratio (υ) is 0.2 for uncracked concrete and 0 for cracked concrete according to SS-EN 1992-1 [61], section 3.1.3.

For eventual non-linear calculations, the general stress-strain diagram according to SS-EN 1992-1 [61] section 3.1.5 can be used.

4.2.3 Modeling of stiffness of concrete elements

For determination of the load effects, linear elastic analysis may be carried out under assumption of uncracked concrete cross-sections and mean value of the modulus of elasticity (E_{cm}). That is, the structural model can be based on the nominal geometrical properties of the concrete elements.

However, if a linear elastic analysis indicates extensive cracking in concrete elements, the reduced stiffness must be considered. Qualified engineering assessments are needed to address the stiffness reduction in an updated linear elastic calculation, whereby ASCE 4-98 [24], section 3.1.3 can provide guidance. An acceptable approach to consider cracked concrete properties can be to reduce the stiffness of the uncracked members by a reduction factor as described in ASCE 43-05 [23], section 3.4.1.

4.2.4 Modeling of mass distribution

The inertial mass properties in the load-bearing structures can be defined directly in the structural model through the geometrical properties of the structural elements and the density of the material. In addition to the structural mass, mass equivalent to a distributed floor load of 250 kg/m^2 could be included, to represent miscellaneous dead weights such as minor equipment, piping and raceways, according to SRP 3.7.2 [26]. The mass of major permanent installed equipment should be distributed over a representative floor area or included as concentrated lumped masses at the equipment locations.

The structural model used for determining the seismic response shall also include the mass of the quasipermanent part of the live load ($\psi_2 Q_k$). Guidelines for applicable values on ψ_2 for different load types can be found in SS-EN 1990 [62]. Participating part of the mass of the live load at floor slabs in nuclear facilities should be determined on a best estimate basis, but not less than 25% ($\psi_2 \geq 0.25$) of the specified design live load, in accordance with ASCE 43-05 [23], section 3.4.2.

4.2.5 Modeling of damping

Damping represents the structural ability to absorb energy when responding to dynamic loading. Damping is dependent on various factors such as type of connections between the structural elements, type of material and the stress levels during loading.

Applicable damping values to be used can be found in ASCE 4-98 [24], Table 3.1-1 for various types of material. In ASCE 4-98 [24], section 3.1.2.2, the principles for determining damping values for design and structural evaluation of structures are described, as well as for determining in-structure response spectra to be used for subsystem seismic analysis.

In this connection, it must be observed that USNRC in their latest version of RG 1.61 [58] from March 2007 has revised the damping values applicable when generating in-structure response spectra to safety equipment at low stress levels. Hence, damping values for different structural types can be determined according to Table 4.1. The principles for determining stress level 1 and 2 respectively as described in ASCE 4-98 [24], section 3.1.2.2 are in all essentials compatible with corresponding principles in RG 1.61 [58].

In practice, Stress Level 2 damping values may always be used in seismic design of structures, while Stress Level 1 is most often used in development of in-structure response spectra.

Table 4.1 – Modal damping ratios according to RG 1.61 [58], with stress level definitions according to ASCE 4-98 [24].

Structure type	Stress level 1	Stress level 2
Reinforced concrete	4 %	7 %
Prestressed concrete	3 %	5 %
Welded steel or bolted steel with friction connections	3 %	4 %
Bolted steel with bearing connections	5 %	7 %

4.2.6 Modeling of hydrodynamic effects

Hydrodynamic effects of large volumes of water in for instance fuel- and service pools and condensation pools can be considered in accordance with ASCE 4-98 [24], section 3.1.6. In this connection, the effects on the dynamical properties (eigenfrequencies) as well as the resulting load effects in the walls and floors of the pools and eventual separating walls between different pools should be considered.

ASCE 4-98 [24], section 3.1.6 provides acceptable methods for modeling hydrodynamic effects of water in pools. ASCE 4-98 [24], section 3.1.6.3 include examples on acceptable methods for determining convective and impulsive effects of water.

4.3 Seismic analysis

4.3.1 General

Following methods are acceptable to use when performing a seismic response analysis of safety-related structures at nuclear power plants:

1. The time history method
2. The response spectrum method
3. The equivalent static method

Minimum requirements for each method are described in the following.

4.3.2 Time history method

Time history analysis can be carried out using linear or non-linear analysis methods.

Modal dynamic time history analysis is the most common linear analysis method. The earthquake is then described in the form of acceleration-time histories. Requirements on the method are described in ASCE 4-98 [24], section 3.2.2.2.1. It must be observed that USNRC does not support ASCE 4-98 [24], section 3.2.2.2.1(f) regarding how many modes need to be included in the modal superposition. USNRC states that ASCE 4-98 [24], section 3.2.2.2.1(f) is non-conservative and recommend instead to apply RG 1.92 [27] for modal superposition and addressing that part of the mass not excited within the total modal mass (“missing mass”). Hence, ASCE 4-98 [24], section 3.2.2.2.1(f) ought to be used with care and if not all mass is included in the analysis, it need to be demonstrated that the effect of missing mass can be considered negligible.

As an alternative to the modal dynamic analysis, the direct integration method can be used, see ASCE 4-98 [24], section 3.2.2.2.2.

In case that geometrical non-linearities, for instance gaps between structural elements, have a significant impact on the response or where material non-linearities as for instance plasticity or friction occur, non-linear time-history methods can be applied. Requirements on these methods are described in ASCE 4-98 [24], section 3.2.2.3.

4.3.3 Response spectrum method

The response spectrum method enables calculation of the maximum response in the structure when excited by an earthquake defined in the form of a ground response spectrum. The calculation of maximum values is carried out by combining the maximum responses for the participating modes. In ASCE 4-98 [24], section 3.2.3, requirements on how to apply the response spectrum method are described. As regards the application of ASCE 4-98 [24], section 3.2.2.2.1(f), see section 4.3.2.

4.3.4 Equivalent static method

Equivalent static methods for determining seismic load effects in structures are allowed in national standards for simple structure with symmetric and uniform geometry and mass distribution. However, the method is inappropriate for structures with irregular shapes, likewise there are restrictions for the method to be used at nuclear facilities. In general, the primary usage of equivalent static methods are for simple estimates and feasibility assessments of results from more rigorous dynamic analysis for building structures and for component and distribution systems. The requirements on the use of equivalent static methods at nuclear facilities are presented in ASCE 4-98 [24], section 3.2.5.

4.3.5 Multiply-support systems

For structures or safety systems supported on different structures or different structural elements within a building, the effect of different in-signals must be considered according to ASCE 4-98 [24], section 3.2.6.

4.3.6 Combination of modal and component responses

Requirements on how modes and excitation directions shall be combined when the response spectrum method is applied and how the excitation directions shall be considered when the time history method is used are described in ASCE 4-98 [24], section 3.2.7. In this connection, it is important to emphasize the requirement that the three directional components of earthquake motion in a time-history analysis must be statistically independent in order for the three excitation directions to be applied simultaneously to the numerical model in one analysis. If the directions have a statistical dependency, each excitation direction has to be applied separately and the structural response should be adequately combined as described in ASCE 4-98 [24], section 3.2.7.2.

4.3.7 Soil-structure interaction

In comparison with other dynamic loads, the earthquake load can be characterized in terms of the ground motion rather than an applied external load. Ground response spectra or alternatively synthetic developed time histories describe the ground motion in the free field without any influence from the structure.

Depending on the characteristics of the earthquake, the foundation conditions and the dynamic properties of the structure, the actual motion of the foundation will deviate from the ground motion in the free field. For a light building with a flexible foundation slab founded on rock or on soil with high stiffness, the deviation will be negligible, since the building transfer only a small amount of energy to the environment through the foundation. On the contrary, a heavy building with a relatively stiffer foundation slab founded on softer soil conditions has a greater ability to radiate energy to the environment, causing the ground motion in the foundation slab to differ significantly from the motion in free field.

In case a significant difference can be expected between the motion in free field and the motion under influence from the structure, ASCE 4-98 [24], section 3.3 requires analysis to be performed by considering the interaction between soil and structure, i.e. Soil-Structure Interaction (SSI).

In ASCE 4-98 [24], section 3.3.1, it is required that SSI shall be considered for all structures not founded on rock or rock-like soil foundation material. A fixed-base support may generally be assumed when the structure is supported on rock or rock-like conditions, which approximately correspond to shear wave velocities > 1100 m/s. However, it should be verified that the interaction frequency for a model with a completely stiff structure in combination with discrete springs according to ASCE 4-98 [24], table 3.3-1 for a circular slab and table 3.3-3 respectively for a rectangular slab, is at least twice the fixed-base frequency in a model with a flexible structure. If the shear wave velocity > 2400 m/s, a fixed base assumption is accepted without any further verification, according to SRP 3.7.2 [26].

4.3.8 Input for subsystem seismic analysis

The scope of seismic design of conventional buildings includes primarily calculation of load effects and verification of sufficient capacity of the load-bearing structural elements. In addition for safety-related structures at nuclear facilities, the licensees also need to provide input for seismic analysis of safety equipment in the building in the form of in-structure response spectra or in-structure time histories at certain positions in the structure, normally at least at each floor level. In general, in order to provide in-structure response spectra with sufficient accuracy, the numerical model need to have higher geometrical resolution and more dense mesh to catch higher local eigenfrequencies. In ASCE 4-98 [24], section 3.4, requirementss on acceptable procedures for generating in-structure response spectra and time history motions are provided.

5. Seismic design of nuclear structures

5.1 General

In this section the seismic design process for nuclear facilities in Sweden is discussed. The different steps in the design process are described at a general plant level, but the detailed assessments and recommendations are focused on the safety-related structures.

The recommendations in this section are primarily applicable for new structural design or redesign of existing structures, but can as well be applied for evaluation of existing structures. Specific assessments and recommendations for existing structures not designed against earthquakes are addressed in chapter 6.

5.2 Seismic design classification

5.2.1 Seismic classification according to IAEA Safety Guides

Seismic classification is the process by which each individual SSC at a nuclear facility should be assigned to a specific seismic category, depending on its required performance during and after an earthquake, in addition to other required safety classifications. SSCs should be assigned into a number of categories in accordance with the principles of section 2.11 to 2.26 in IAEA SG NS-G-1.6 [50].

SSCs which should be designed to withstand the consequences of a SL-2 (DBE) earthquake are classified to the highest safety category, Seismic Category 1 (SC1). In particular, following items are classified as SC1:

- Items whose failure could directly or indirectly cause accident conditions as a consequence of a SL-2 earthquake.
- Items necessary for shutting down and maintaining the reactor in a shutdown condition, removing residual heat over a required period and monitoring parameters essential to these functions.
- Items necessary to prevent or mitigate non-permissible radioactive releases, for any postulated initiating events considered in the design basis, regardless of their probability of occurrence, i.e. the defence in depth approach.

For any item in SC1, an appropriate acceptance criterion should be established in the form of values of design parameters as regards leaktightness, structural integrity, passive or active function, etc,.

Some examples of items in SC1 associated with the reactor are:

- The primary coolant system
- The main steam- and feedwater systems
- The primary heat removal system
- The control rod drive system
- The emergency power supply, including diesel generators, auxiliaries and distribution systems.
- Instrumentation and control systems.

- Control rooms required for safe shutdown.
- The reactor containment.
- Structures and buildings which house or support systems for safe shutdown, power systems and instrumentation and control systems.

In IAEA SG NS-G-1.6 [50], items categorized to a Seismic Category 2 (SC2) mainly includes items whose failure due to collapse, falling or displacement as a consequence of a SL-2 earthquake may jeopardize items in SC1 and SC3, or items that may influence safety functions of items in SC1 and SC3 or safety related operator actions.

Collapse, falling or displacements caused by an earthquake may for example generate missiles due to failure of rotating machinery, pressure waves due to bursting tanks, blocking of emergency cooling lines, flooding or fire.

Some examples of SC2 items are:

- The turbine building
- The cooling water intake structures
- The emergency access roads

Seismic Category 3 (SC3) items should include all items that could pose a radiological hazard but that are not related to the reactor. Some examples of SC3 items:

- The spent fuel building when the fuel is no longer active (i.e. requires forced cooling)
- The radioactive waste building

Seismic Category 4 (SC4) items should include all items that are not in SC1, SC2 or SC3. Some examples on SC4 items are storage and workshop buildings and administrative buildings. Structures and buildings in SC4 could be designed in accordance with conventional non-nuclear building standards, such as for instance the Eurocodes [52].

5.2.2 Seismic classification according to USNRC Regulatory Guides

The basic approach for seismic design classification in the U.S. is described in section 2.2.4. The classification approach for NPPs follows RG 1.29 [29], while for radioactive waste at NPP sites, RG 1.143 [30] is applied. Basically, RG 1.29 [29] only use one seismic category, Seismic Category 1, which in general complies with SC1 according to IAEA SG NS-G-1.6 [50].

For radioactive waste management SSC in the U.S., RG 1.143 [30] applies. The safety classes RW-IIa and RW-Ib in RG 1.143 [30] are a function of radiological releases and thus representing a somewhat more sophisticated classification approach compared to SC3 in IAEA SG NS-G-1.6 [50] and a reduction of the SL-2 earthqauke motion is specified for design.

As also stated in section 2.2.4, the USNRC has recognized the need to be less prescriptive in the designation of SC1 items, with the aim of implementing a more safety-related and safety-significant risk categorization as described in RG 1.201 [33] as an alternative to the deterministic seismic classification approach in RG 1.29 [29]. However, this procedure for risk informed reclassification of SSCs has so far only been attempted on a trial basis at a small number of existing NPPs.

5.2.3 Seismic classification according to YVL Guides in Finland

The seismic classification approach to be applied at the Finnish nuclear facilities is addressed in section 3.4 in YVL Guide B.2 [63]. At the Finnish NPPs, each SSC shall be assigned to one of three seismic categories (S1, S2A and S2B) on the basis of the specified seismic resistance requirements.

Seismic Category S1 items shall maintain their leak-tightness, integrity, functionality and proper position in a loading situation caused by a DBE. Some items may be assigned only a certain feature, for instance leak-tightness, that need to be maintained. The requirements for Seismic Category S1 items are in all essentials in compliance with corresponding requirements for SC1 items as defined in IAEA SG NS-G-1.6 [50].

Seismic Category S2A comprises SSCs for which operability and integrity are not essential, but whose failure due to for instance collapse or falling or other reasons may jeopardize the safety-related operation and integrity of S1 items. The definition of Seismic Category S2A items complies with corresponding definition of SC2 items in IAEA SG NS-G-1.6 [50].

It is also mentioned in YVL Guide B.2 [63] that S1 and S2A items should have specifications as regards acceptance criteria for e.g. operability, integrity and leak-tightness, as also are addressed in IAEA SG NS-G-1.6 [50].

Seismic Category S2B shall comprise all other SSCs of a nuclear facility, corresponding to the definition of SC4 in IAEA SG NS-G-1.6 [50].

Seismic Category S1 in YVL Guide B.2 [63] does not distinguish between seismic categorization of SSC items related to the reactor and other items that could pose a radiological hazard to the site as do IAEA SG NS-G-1.6 [50] (SC1 and SC3 respectively).

5.2.4 Seismic classification at the Swedish nuclear facilities

The seismic classification approach in Sweden is described in section 2.4.3. There are three seismic categories (1, P and N). The designation of the categories (1, P, N) differ from corresponding in IAEA SG NS-G-1.6 [50] (SC1, SC2, SC3 and SC4) and in YVL Guide B.2 [63] (S1, S2A and S2B). But in practice there are a good coherence between the Swedish practice and international standards as discussed below.

IAEA SG NS-G-1.6 [50] strongly emphasize on that an appropriate acceptance criterion should be established in accordance with the required safety function. As examples of such acceptance criterion, IAEA SG NS-G-1.6 [50] specifically mention design parameters for functionality, leaktightness or maximum distortion. YVL Guide B.2 [63] also emphasize on specification of appropriate acceptance criteria for e.g. operability, integrity and leak-tightness. The Swedish seismic classification approach further addressing these issues by separating Seismic Category 1 (SC1) in IAEA SG NS-G-1.6 [50] or S1 in YVL Guide B.2 [63] into two categories; 1 (leak-tightness) and P (integrity) as shown in Table 2.3. Seismic Category 1 is the highest category and includes integrity as well.

This is a strong feature, which facilitates the seismic categorization process by more specifically addressing the type of acceptance criterion for each different SSC as illustrated in Table 2.3.

Sesmic Category N corresponds to Seismic Category SC2 in IAEA SG NS-G-1.6 [50] and Seismic Category S2A in YVL Guide B.2 [63].

In Sweden there are no specific category for SSC items whithout any specified safety function during a DBE earthquake. (SC4 in IAEA SG NS-G-1.6 [50] and S2B in YVL Guide B.2 [63]).

As for YVL Guide B.2 [63], the Swedish seismic classification approach does not distinguish between seismic categorization of safety items for reactor systems and items that could pose a radiological hazard not related to the reactor.

5.3 Design Basis Earthquake

5.3.1 General

In section 3, some basic conditions for determining design basis ground motions according to international standards are described and summarized. Also, recommendations on revisions of the characterizations of the seismic ground motions in SKI Technical Report 92:3 [8] are provided.

In this section, the main sources and references for establishing the Design Basis Earthquake (DBE) for the Swedish NPPs are analyzed and some conclusions are provided.

5.3.2 Design Basis Earthquake

In section 3.2.1 it is stated with reference to IAEA SG NS-G-1.6 [50] that the SL-2 (DBE) earthquake corresponds to a level with a mean annual exceedance frequency in the range of $1 \cdot 10^{-3}$ to $1 \cdot 10^{-4}$ or a median annual exceedance frequency in the range of $1 \cdot 10^{-4}$ to $1 \cdot 10^{-5}$. Additionally also for the SL-2 level, the PGA should be at least 0.1 g regardless of the actual seismic hazard.

In section 9.2 in IAEA SSG-9 [6], it is stated that the SL-2 (DBE) should be defined by means of appropriate spectral representations and time histories. The ground motions should be defined for free field conditions, at the level of ground surface or key embedment depths.

Design ground response spectra can be defined either as a standard or a uniform hazard response type of response spectrum as further described in section 2.2.2 or section 9.4 and 9.5 in IAEA SSG-9 [6]. A standard spectrum is obtained from various response spectra derived on the basis of earthquake records and engineering considerations, scaled to envelope the mean ground motion levels at low and high frequencies. A uniform hazard response spectrum is developed by selecting the values of the response spectral ordinates that correspond to the annual frequencies of exceedance of interest from the seismic hazard curves. As stated in section 2.2.2, the existing standard type of spectra are not appropriate for sites susceptible to high frequency motions, i.e. the situation in Sweden with the dominating contribution from near-field earthquakes. Even though it might be able to justify the usage of standard response spectra for certain special cases, it is today an internationally establish common practice to consider the probabilistic nature of seismic hazard and accordingly to apply site-specific uniform hazard response spectra for seismic safety evaluations. Therefore, usage of standard response spectra is not further discussed or evaluated.

According to RG 1.165 [21], a median based reference probability of $1 \cdot 10^{-5}$ is recommended for determining the DBE. In RG 1.208 [22], a performance-based approach according to ANSI/ANS 2.26 [64] and ASCE 43-05 [23] is recommended instead. For the highest Seismic Design Category class (SDC 5), it is recommended to use a mean annual exceedance frequency of $1 \cdot 10^{-4}$, for the minimum structural damage state, i.e. essentially elastic behavior. Essentially elastic behavior means that localized inelasticity might occur at stress concentration points, but the overall seismic response should be essentially elastic. As stated in section 2.2.2, for typical seismic hazard curves the mean annual exceedance probability $1 \cdot 10^{-4}$ equals the median annual exceedance probability $1 \cdot 10^{-5}$.

In section 1.5.3.5 in SS-EN 1990 [62], an accidental action is defined as an action, usually of short duration but of significant magnitude, that is unlikely to occur on a given structure during the design working life. Further it is stated that impact, snow, wind and seismic actions may be classified as variable or accidental actions, depending on the available information on statistical distributions. In section 4.1.1 in in SS-EN 1990 [62], it is stated that the design value of an accidental action should be specified for individual projects and that the design value for seismic actions should be assessed from the characteristic value or specified for individual projects.

Further as described in 2.4.2, non-nuclear structures in seismic regions in Europe are recommended to be designed to withstand a design seismic action associated with a reference probability of exceedance of 10% in 50 years or a reference return period of 475 years. It is also concluded that for hazards with such low reference return period, the whole Swedish territory can be classified as a very low seismicity case and the requirements of SS-EN 1998 [48] can be disregarded (i.e. earthquakes of such short return periods in Sweden are not damaging to conventional structures). But due to the fact that nuclear facilities must be designed against internal and external hazards with much higher reference return period (in the range of 10 000 to 100 000 years), the resulting seismic action should be classified as an accidental action according to the definitions in the Eurocodes [52], and could therefore be specified for individual projects, without considering the characteristic values.

DNB [51] includes complementary rules to the requirements in the Eurocodes [52] for application at NPPs in Sweden. The DBE load is in section 4.2.3 in DNB [51] defined as an accidental load and denominated E_{DBE}. Further in section 7.4.1 in DNB [51], E_{DBE} is specified as a seismic action corresponding to an annual exceedance frecuency of $1 \cdot 10^{-5}$ and defined in the form of envelope ground response spectra for a typical hard rock site in Sweden according to Appendix 1 in SKI Technical Report 92:3 [8]. As a comparison, the DBE in Finland is according to section 4.1 in YVL Guide B.7 [56] defined as having an annual exceedance probability less than $1 \cdot 10^{-5}$ on the median confidence level. The Finnish design ground response spectra is anchored at 0.1 g PGA and has a similar shape and spectra ordinate values as the spectra in SKI Technical Report 92:3 [8] at the same exceedance probability, as illustrated in Figure 3.2.

Some conclusions:

- The basic definitions of the DBE (SL-2) earthquake can be found in IAEA SG NS-G-1.6 [50] and IAEA SSG-9 [6].
- It is an internationally established common practice to consider the probabilistic nature of seismic hazard and accordingly to apply site-specific uniform hazard response spectra for seismic safety evaluations.
- The DBE earthquake corresponds to a level with a mean annual exceedance frequency in the range of $1 \cdot 10^{-3}$ to $1 \cdot 10^{-4}$ or a median annual exceedance frequency in the range of $1 \cdot 10^{-4}$ to $1 \cdot 10^{-5}$.
- According to the Eurocodes [52] and DNB [51], the DBE load is defined as an accidental load and denominated E_{DBE}.
- E_{DBE} corresponds to ground response spectra for a typical hard rock site with an annual exceedance probability of $1 \cdot 10^{-5}$ according to SKI Technical Report 92:3 [8].
- Some important remarks and conclusions as regards the deficiencies in the characterization of seismic ground motions in SKI Technical Report 92:3 [8] can be found in section 3.2.5.

5.4 Design Extension Earthquake

5.4.1 General

In the aftermath of the Fukushima nuclear accident, there has been ongoing activities and discussons in the nuclear community about how to take into account very unlikely external events beyond the design basis, in order to assess the existing safety margins as well as to identify potential improvements in the existing plants.

Issues related to severe earthquake effects beyond the design basis for new build safety-related structures are discussed in this section. For existing structures not designed against earthquakes, specific issues are addressed in chapter 6.

5.4.2 Design Extension Earthquake

Already within the Seismic Safety project during the 1980s, discussions were initiated as how to maintain certain very essential SSCs against seismic loads with extreme low probability of exceedance levels (10^{-7}), in addition to the requirements on the safe shutdown and cooling of the reactor to withstand a seismic load corresponding to an annual exceedance frequency of 10^{-5}. It was concluded that SSCs essential for the containment integrity, isolation and pressure relief should have a sufficient ultimate capability to withstand a seismic load corresponding to an annual exceedance frequency of 10^{-7}. However, no formal recommendations were established and the actions were limited to some minor pilot studies and assessments at some of the Swedish NPP units.

As part of the European stress tests during 2011, the Swedish licensees carried out calculations and engineering judgments for a probability level of 10^{-7} at a majority of the Swedish NPPs, in order to assess the robustness for severe earthquakes beyond the design basis. The integrity assessments were focused on the reactor containments, the scrubber buildings, the spent fuel pools and certain structural walls and slab elements housing safety functions essential for the containment and pressure relief function and the spent fuel pool integrity. However, these assessments were based on rough up-scaling calculation methods and engineering judgments owing the very limited time available, as also is notified by ENSREG in [3] and described in section 1.1.4. Therefore, further refined analyses and investigations are necessary before definite conclusions can be established.

According to section 2.39 in IAEA SG NS-G-1.6 [50], the seismic design should be carried out in accordance with the general recommendations in section 1 to 3 in [50], in order to provide sufficient margins for seismic events beyond the design basis to avoid severely abnormal plant behavior ("cliff edge effects"). A "cliff edge effect" has reference to when a small deviation of a design plant parameter give rise to an abrupt worsened situation for the whole plant.

Further, it is also important to create sufficient margins in the design against for instance future elevated seismic hazard levels, new modern standards for hazard assessments and more sopfisticated design and analysis methods.

As stated in section 1.1.3, the French safety authority (ASN) has recommended the French licensee (EDF) the creation of a "hardened safety core" of essential SSCs and organizational arrangements in order to manage the fundamental safety functions in extreme situations, aiming on preventing a severe accident and limiting large-scale radioactive releases if the accident becomes uncontrollable. This approach involves setting up a bunkerized emergency management centre with diesel generator and an ultimate backup water supply. The equipment to be included in the "hardened safety core" must be designed to withstand external events, including earthquakes, far beyond the design basis of the plant. ASN has adopted a number of resolutions, which specify the objectives of the "hardened safety core":

- Prevent a severe accident to affect the core of the reactor or the spent fuel pool.
- Limit the consequencies of an accident which could not be avoided, with the aim of preserving the integrity of the reactor containment without opening the venting system.
- Enable the licensee to perform its emergency management duties.

ASN has specified the design rules for the "hardened safety core" equipment to comply with the most demanding nuclear industry standards. The seismic hazard level is not yet determined but should probably be consistent with the BDBE for SMA, that is approximately 1.6 times the DBE level, which is applied for the EPR. As a comparison, USNRC has developed a requirement to evaluate essential SSCs to perform required functions for a seismic load of 1.67 times the DBE load, according to section 2.2.6.4.

In the Swedish DNB [51], accidental loads corresponding to very unlikely events (Design Extension Conditions) are specified. One such load is the Design Extension Earthquake (DEE), denominated E_{DEE}, according to section 4.2.4 in DNB [51]. Further in section 7.4.1 in DNB [51], it is stated that the seismic load level, E_{DEE} should be specified by the Swedish nuclear safety authority, SSM.

5.4.3 Recommendations on Design Extension Earthquake

DNB [51] provides recommendations to address consequences of earthquakes beyond design basis through the seismic load E_{DEE} under Design Extension Condition. Thus, it is now possible to design certain essential safety functions for a very unlikely earthquake, as was the intention already in the Seismic Safety project during the 1980s and further emphasized through the intention of the French safety authorities as regards the "hardened safety core" equipment.

Some recommendations as regards the Design Extension Earthquake:

- Determine a reasonable seismic hazard level for a Design Extension Earthquake (E_{DEE}). A reasonable hazard level could be approximately 1.6 to 1.7 times the DBE in line with ongoing discussions in the U.S. and France, as discussed in 2.2.6, 2.3.3 and 5.4.2. Another possibility could be to apply an annual exceedance probability of $1 \cdot 10^{-6}$ according to SKI Technical Report 92:3 [8], with a PGA of 0.23g, i.e. approximately 2 times the DBE. In order to comply with the recommendations provided in the Seismic Safety project and to create sufficient margins for future increased seismic hazard levels due to identified deficiencies in the characterization of seismic ground motions in SKI Technical Report 92:3 [8] as stated in section 3.2.5, it can be recommended to apply an annual exceedance probability of $1 \cdot 10^{-7}$ according to SKI Technical Report 92:3 [8] for certain essential safety-critical SSCs.
- The essential SSCs which need to be maintained for a E_{DEE} can be determined by means of a seismic classification, with similar procedures as for the DBE load.
- As regards acceptance criteria for an E_{DEE} , see section 5.6.

5.5 Seismic load combinations

General recommendations on how to combine the earthquake load with other loads can be found in section 2.27 to 2.30 in IAEA SG NS-G-1.6 [50]. However, these paragraphs only provide some overall recommendations for normal conditions, anticipated operational conditions and accident conditions respectively, and for each Seismic Category, without any specific recommendations.

In the Eurocodes [52], there are requirements for conventional buildings and industrial facilities on how to combine permanent, variable and accidental loads. These requirements should be followed for nuclear structures as well, together with additional requirements to consider essential safety issues for nuclear facilities. DNB [51] specifically provides such supplementary requirements in addition to corresponding requirements for ordinary building structures as covered in SS-EN 1990 [62].

The general load combination rule for seismic design situations according to section 6.4.3.4 in SS-EN 1990 [62] is:

$\Sigma G + P + \Sigma(\psi_2 \cdot Q) + E_{\mathrm{DBE}}$ where

ΣG = Sum of the different permanent loads

P = Prestressing load

$\Sigma(\psi_2 \cdot Q)$ = Sum of the different variable loads

E_{DBE} = Seismic load

In Table 4.6 in DNB [51], this general expression is further decomposed to the different individual loads, together with some specified nuclear-related loads:

$D + H_{\mathrm{gw}} + H_{\mathrm{ge}} + P_{\mathrm{p}} + \psi_2 L + \psi_2 S + \psi_2 W_q + \psi_2 \Delta T + \psi_2 H_{\mathrm{qw}} + \psi_2 H_{\mathrm{qe}} + M_d + E_{\mathrm{DBE}}$ where

D = Dead load of the structures and permanently installed equipment.

H_{gw} = permanent groundwater pressure at the mean water level and hydrostatic pressure from water in basins at normal operating conditions.

H_{ge} = Permanent soil pressure.

P_{p} = Prestressing forces.

$\psi_2 L$ = Quasi-permanent value of the live loads.

$\psi_2 S$ = Quasi-permanent value of the snow load.

$\psi_2 W_q$ = Quasi-permanent value of the wind load.

$\psi_2 \Delta T$ = Quasi-permanent value of climate-related temperature difference and temperature change.

$\psi_2 H_{\mathrm{qw}}$ = Quasi-permanent value of the difference between variable waterpressure and H_{gw}.

$\psi_2 H_{\mathrm{qe}}$ = Quasi-permanent value of the difference between variable soil pressure due to moving load and H_{ge}.

M_d = Process-related loads from pipe and process-system, differential pressures, temperature differences and loads resulting from relief valve or other high energy deviced actuation, during operation, outages or operational disturbances.

E_{DBE} = Seismic load DBE. Determined as the inertia effects from all excited mass, calculated from a dynamic analysis. Can be expressed as a/g $(D + \psi_2 L + \psi_2 S)$, where a represents the seismic acceleration of the mass.

The quasi-permanent or long-term value of a variable action $(\psi_2 \cdot Q)$ is determined so that the total period of time for which it will be exceeded is a large fraction of the reference period. Recommended values for some variable loads can be found in Table A1.1 in SS-EN 1990 [62] as follows:

Snow load (S):	$\psi_2 = 0.2$
Wind load (W_{q}):	$\psi_2 = 0$
Climate-related temperature load (ΔT):	$\psi_2 = 0$

Live loads (L) at slab structures in nuclear facilities: $\psi_2 > 0.25$ [5)]

For load H_{qw} and H_{qe},, in general at NPPs: $\psi_2 = 0$

The seismic load combination as above can then be condensed to following expression:

$D + H_{gw} + H_{ge} + P_{p} + 0.25L + 0.2S + M_d + E_{DBE}$

The US design standard ASME Section III, Div 2 [9] for reactor containments and ACI 349-06 [10] for other safety-related buildings are commonly used internationally as reference standards for design of safety-related structures at nuclear facilities. Hence, the seismic load combination rule in DNB [51] is here compared with corresponding combinations rules in ASME Section III, Div 2 [9] for reactor containments in Table 5.1 and ACI 349-06 [10] for other safety-related structures in Table 5.2, with load symbols in accordance with DNB [51].

Table 5.1 – Comparison of load combination rules between DNB [51] and ASME Section III, Div 2 [9] for reactor containments.

Code	**Permanent loads**				**Variable loads**						**Accidental loads**			
-	D	H_{gw}	H_{ge}	P_{p}	L	S	ΔT	H_{qw}	H_{qe}	M_d	P_a	ΔT_{a}	R	E_{DBE}
DNB	1.0	1.0	1.0	1.0	0.25	0.2	-	-	-	1.0	-	-	-	1.0
ASME 1)	1.0	1.0	1.0	1.0	1.0	-	1.0	1.0	1.0	1.0	-	-	-	1.0
ASME 2)	1.0	1.0	1.0	1.0	1.0	-	-	1.0	1.0	-	1.0	1.0	1.0	1.0

1) Load combination “Extreme environmental” according to Table CC-3230-1 in ASME [9].

2) Load combination Abnormal/extreme environmental ” according to Table CC-3230-1 in ASME [9].

Table 5.2 – Comparison of load combination rules between DNB [51] and ACI 349-06 [10] for other safety-related buildings.

Code	**Permanent loads**				**Variable loads**						**Accidental loads**			
-	D	H_{gw}	H_{ge}	P_{p}	L	S	ΔT	H_{qw}	H_{qe}	M_d	P_a	ΔT_{a}	R	E_{DBE}
DNB	1.0	1.0	1.0	1.0	0.25	0.2	-	-	-	1.0	-	-	-	1.0
ACI 349 1)	1.0	1.0	1.0	1.0	0.8	-	1.0	1.0	1.0	1.0	-	-	-	1.0
ACI 349 2)	1.0	1.0	1.0	1.0	0.8	-	-	1.0	1.0	-	1.0	1.0	1.0	1.0

1) Load combination (9-6) according to ACI 349-06 [10].

2) Load combination (9-9) according to ACI 349-06 [10].

After comparing the standards some differences can be distinguished:

- DNB/Eurocodes use an approach in which reduction factors are applied on the variable loads, reflecting the long-term values of these loads, in equivalence with the approach for determining the participating mass during the earthquake. In contrast, ASME [9] uses the full value of the live loads, despite the fact that the participating mass is reduced, while the live load according to ACI 349-06 [10] includes a minor reduction (20%) of the live load.
- It seems that neither ASME [9] nor ACI 349-06 [10] consider any snow load in combination with the seismic load.
- Climate-related temperature distributions are not considered in combination with seismic loads in DNB/Eurocodes.

5) To be determined on a best estimate basis, but not less than 0.25, see section 4.2.4.

- In contrast to ASME [9] and ACI 349-06 [10], effects of a pipe break accident need not to be combined with seismic loads in DNB/Eurocodes.

Regarding the approach in DNB/Eurocodes to consider the long-term quasi-permanent part of the variable loads in combination with the seismic loads is in line with the basic statistical principles of the standard, while ASME [9] and ACI 349-06 [10] use a more conservatively deterministic approach.

The same statistical design principles in DNB/Eurocodes are applied to disregard climate-related temperatures in combination with the seismic load.

Effects of a pipe break as a consequence of the seismic DBE load have not been adopted in the design basis for the Swedish NPP design. The design philosophy is instead to prove that the high-energy piping system can withstand the DBE load. The same design principle is adopted in Finland. In section 4.2.2 in YVL Guide B.7 [56], it is stated that the DBE loads need not to be combined with other accidental loads if it can be proved that the consequences of the seismic loads can be prevented by the SSC whose failure could initiate another accident (i.e. a pipe break). In practice in the U.S. today, Leak Before Break (LBB) is used to eliminate postulated pipe breaks in new NPPs.

The load combination principles for the DEE load are identical with the DBE load as shown in Table 4.7 in DNB [51]. Thus, the symbol E_{DBE} can simply be replaced by E_{DEE}. As for the E_{DBE}, pipe break must be proved to be prevented.

5.6 Seismic safety verification

5.6.1 General

Verification of the resistance of SSCs at a nuclear facility against earthquake loads can be executed by means of one of following methods or a combination of them:

- Experience based methods
- Testing
- Numerical simulations (dynamic analysis)

The experience-based methods consist mainly of assessments of existing facilities' resistance against actual strong motion earthquakes. These methods can be used on facilities not designed against earthquake load effects or facilities designed for a certain earthquake hazard level, but where the site need to be re-assessed for a more severe hazard level. The best known methods are SMA and SPSA.

Testing of components is carried out on shaking tables according to specified routines and for equipment that is difficult to evaluate by other methods. Most commonly, testing is done on electrical instrumentation and control components and devices.

The predominant method for seismic safety verification of building structures is numerical simulations by means of dynamic analyses.

5.6.2 Codes and Standards

As stated in section 2.4.4, all building activities in Sweden shall comply to the Eurocodes [52]. However, one important limitation with the Eurocodes [52] is that it formally does not apply to nuclear power plants, offshore structures and large dam structures. Additionally also, the statute book of SSM does not yet cover specific requirements and guidance for safety-related structures at nuclear facilities. Therefore, in order to remedy these deficiencies, SSM together with the Swedish licensees commissioned Scanscot Technology AB to develop DNB [51], as further described in section 2.4.4.

As described in section 2.4.2, SS-EN 1998-1 [48] applies to design and construction of buildings and civil engineering works in seismic regions in Europe. However, the whole Swedish territory can be classified as a *very low seismicity* region and the requirements of SS-EN 1998 [48] can be disregarded, as also are concluded by Boverket and Trafikverket in the National Annex to SS-EN 1998 [48]. Besides this formal non-validity, Eurocode 8 [48] does not provide sufficient stringent requirements to meet the high level of seismic safety requirements at nuclear facilities.

ASCE 43-05 [23] uses a modern seismic design approach for safety-related SSCs at a broad range of nuclear facilities. It uses a graded approach in developing the seismic design criteria. When subjected to the DBE, significant damage are accepted at Limit State A, while no damage and essentially elastic behavior are the goal at Limit State D. Performance goal are expressed as the mean annual probability of exceedance of the specified Limit State. In addition, all buildings are classified in a Seismic Design Category (SDC) developed in ANSI/ANS 2.26 [64], in order to set the design earthquake levels. Five SDCs have been established. The approach for assigning an SSCs SDC is based on the severity of the consequences of its failure to perform its safety functions as determined by safety analysis. Conventional buildings may be assigned to SDC-1 while nuclear power plants associated with reactor and spent fuel safety may be assigned SDC-5. In practice, all modern NPPs fall in SDC-5 with the Limit State D acceptance criteria “essential elastic behavior”.

ACI 349 [10] provides requirements for design of safety-related nuclear structures As described in section 2.2.5, ACI 349 [10] requires reinforcing steel detailing to provide ductility to the structures in accordance with chapter 21 in the code. The main reason for this requirement despite the “essential elastic behavior” criterion for the DBE load, is to provide additional assurance that the structural integrity is maintained in the unlikely event of an earthquake beyond the DBE or other unforeseen circumstances.

5.6.3 Seismic safety verification

The seismic design process can in general be described in three steps:

1. Establish the design earthquake level, i.e. DBE or DEE.
2. For each earthquake level assign each SSCs to any of the Seismic Categories (1, P or N).
3. Verify that the acceptance criteria for each SSCs are fulfilled, during and after the earthquake.

The procedures for step 1 is described in section 5.3 for DBE and in section 5.4 for DEE. Step 2 procedures are discussed in section 5.2. Recommendations as regards seismic safety verification of the safety-related structures are described below.

Minimum requirements for structural modeling and seismic analysis are described in section 4. It can be concluded that modern nuclear facilities should be classified as SDC-5 structures as defined in ASCE 43-05 [23] and thus be verified against the acceptance criteria “essential elastic behavior”. The requirements in chapter 21 in ACI 349 [10] have the same purpose as corresponding requirements in section 5 in Eurocode 8 [48], that is to ensure adequate ductility to dissipate energy without substantial reduction of the overall resistance of the building. However, Eurocode 8 [48] recommends concrete buildings to be designed for low dissipation capacity and low ductility in cases of low seismicity regions (as in Sweden). This means that building structures can be designed for the seismic design situations in principle with the same methods as for other accidental loading situations according to SS-EN 1992-1-1 [61], without any specific ductile reinforcement arrangement.

Design of safety-related structures in Seismic Category 1 and P for the DBE is carried out in accordance with requirements in the ultimate limit state in DNB [51] for the seismic load combination as described in section 5.5. In order to meet the “essential elastic behavior” criteria, the idealization of the behavior in the structural analysis is limited to linear elastic behavior, according to section 6.6.6.1 in DNB [51]. Idealization of the behavior in terms of “linear elastic behavior with limited redistribution” or “plastic behavior” should be avoided for the DBE load because of the cyclic character of the seismic load and the absence of ductile reinforcement. Eventual comprehensive concrete cracking is considered linear elastically in accordance with the priciples for stiffness reduction, as described in section 4.2.3.

Safety-related structures in Seismic Category N need not comply with any formal seismic safety requirements. But structures or structural elements in Seismic Category N should not jeopardize SSCs in Seismic Category 1 or P.

Design of structures in Seismic Category 1 and P for the Design Extension Earthquake (DEE) can be carried out in accordance with the same approach as for the DBE. The “essential elastic behavior” acceptance criteria and the non-ductile design philosophy as for the DBE applies as well.

6. Seismic evaluation of existing nuclear structures

6.1 General

6.1.1 General considerations

It is important to distinguish between seismic design and seismic safety evaluation, in that seismic design of SSCs is primarily related to the design stage of the installation prior to its construction and operation, whereas seismic safety evaluation is applied after the installation has been constructed. Seismic design can of course also be applied after its construction in case of design of a new SSC or upgrading of an existing SSC.

The seismic design procedures for new structural design as described in section 5 can be applied to existing structures as well. However, when evaluating existing structures, alternative methods are generally applied, as described in this section. The main reference is IAEA NS-G-2.13 [69], but also section A1.0 in ASCE 4-98 [24].

It is usually recognized that nuclear facilities designed against seismic loads in accordance with established international practice have an inherent capacity to resist larger earthquakes than considered in the design basis, as discussed in section 5.6.3. This inherent robustness is usually described as the “seismic design margin” and is a direct consequence of

a. the conservatism in the seismic design procedures and

b. the fact that other loads than the seismic load are governing for many of the SSCs.

For the Swedish existing nuclear facilities (except Oskarshamn unit 3 and Forsmark unit 3), the first reason (a) is not applicable since seismic design requirements were not included in the original design. In case of a seismic evaluation against earthquake loads, the second reason (2) is often at hand at the Swedish facilities, especially for load-bearing steel structures, where wind and snow loads usually are governing, or for the reactor containment vessels, for which the inherent over-preussurization capacity normally envelope the relatively moderate seismic load effects. Due to the fact that nuclear installations are designed for a wide range of internal and external accidental and natural hazards other than seismic events, the seismic design margin varies a lot for different locations and different SSCs in a facility.

6.1.2 Objectives of the seismic safety evaluation

In accordance with international accepted practice, systematic safety reassessments of a NPP shall be performed throughout its operational lifetime, taking operating experience and significant new safety information into account. A seismic safety evaluation of an existing nuclear facility should be performed in the event of any one of the following:

- Evidence that the seismic hazard at the site is significantly greater than the DBE, because of new seismological data or new methods for seismic hazard assessments.
- Regulatory requirements as regards for instance periodic safety reviews that take into account the state of the knowledge and the actual condition of the plant.
- Inadequate seismic design, due to for instance old and obsolete design solutions.
- New technical findings, such as vulnerability of selected SSCs.
- New experience from the occurrence of actual earthquakes.

- The need to address the performance of the facility for beyond design basis earthquakes, in order to demonstrate that no significant failure would occur for a slightly greater earthquake than the DBE, that is to prove that there is no “cliff-edge effect”.
- As part of a programme for long term operation of the plant.

It is important to clearly establish the purposes of the seismic evaluation before the process starts-up, because there are significant differencies in the evaluation procedures and acceptance criteria depending on the purpose of the evaluation. The main objectives of a seismic safety evaluation may include one or more of the following:

- To demonstrate the seismic safety margin beyond the original design basis and to confirm that there are no cliff edge effects.
- To identify weak links in the facility and its operations with regard to seismic loads.
- To identify and prioritize possible upgrades.
- To assess plant capacity parameters, for instance HCLPF-values, against regulatory expectations.

6.1.3 Selection of appropriate methodologies

There are two main approaches for evaluating the seismic safety of existing facilities; the Seismic Margin Assessments (SMA) and the seismic PSA (SPSA) methodology. The main differencies between these two methods are related to the modeling approach and the capacity evaluation. The SMA uses success paths for system modeling, while SPSA applies so called event trees or fault trees. Capacity evaluations of SSCs are performed in terms of HCLPF values in the SMA method, whereas capacity evaluations are made by probabilistically defined fragility functions in the SPSA method.

In general, the level of effort required is much higher for the SPSA-method than the SMA-method because of the extended scope of the SPSA even when done at the same detail level.

The approach for seismic safety evaluation in this report is limited to safety-related nuclear structures and structural elements. Hence, in section 6.3 and thereafter focus will be on applicable parts of the SMA-methodology to address issues related to seismic safety evaluation of safety-related structures and equipment-building interfaces at the existing Swedish nuclear facilities.

However, in section 6.2 for the safety evaluation of existing structures against the DBE, a conventional evaluation is recommended in accordance with the same procedures as for new structures, as described in section 5.6.3, for the DBE load.

Walkdown procedures and use of earthquake experience data are important parts of seismic safety assessments, especially as regards selecting the success paths and determining the seismic capacities of the selected SSCs. However, these activities are mainly related to the overall plant level and are not dealt with in this report.

6.2 Safety evaluation against the Design Basis Earthquake

According to the Swedish action plan [3] developed within the framework of the European stress tests, the existing Swedish NPPs should withstand the effects of a DBE with an annual exceedance probability of $1\cdot10^{-5}$ according to SKI Technical Report 92:3 [8]. For the DBE it is reasonable to apply a conventional evaluation of the seismic safety of building structures, according to the same procedures as for new structures as described in section 5.6.3.

It should be noted that for seismic evaluation of existing NPP distribution systems and components as described in the IAEA Safety Report No. 28 [65] development of success paths sampling, earthquake experience data and engineering judgement play a more significant role.

In this connection it is important to notice that in principle all existing safety-related structures at the Swedish NPPs were designed during the late 1960s to the beginning of the 1980s, in accordance with the then applicable design standard "Bestämmelser för betongkonstruktioner" (BfB) for concrete structures [66] and "Statens Planverks spännbetongnormer" SBN-S25.21[67] for prestressed concrete structures. The steel structures were designed according to Stålbyggnadsnormerna StBK-N1, N2 och N4 [68].

The design philosophy in BfB [66], SBN-S25.21[67] and StBK-N1, N2 och N4 [68], were based on the principles of certain specified "allowable stresses" for concrete and reinforcement. Load cases were defined as "ordinary load cases" for which low allowable stress values were used, whereas 20% higher allowable values could be used for "exceptional load cases". Earthquake loads were not included in the design basis and consequently no specific ductile reinforcement were required. In addition, requirements on shear reinforcement in BfB [66] were not as stringent as in the Eurocodes [52], thus further limiting eventual inherent ductility compared to the Eurocodes [52].

Considering the absence of ductile reinforcement in the existing Swedish nuclear facilities, it is reasonable to apply the "essential elastic" acceptance criteria as described in section 5.6.3 for new structures. It shall however be emphasized that the "essential elastic behavior" as stated in RG 1.208 [22] can be somewhat mitigated in the sense that localized inelasticity are accepted at stress concentration points, but the overall seismic response should be essentially elastic. In addition in SRP 3.7.2 [26], it is stated that the SRP acceptance criteria address linear elastic analysis with allowable stresses near elastic limits of the structures. However, for certain special cases (e.g., stability analysis and evaluation of as-built structures), reliance on limited inelastic/nonlinear behavior is acceptable when appropriate.

6.3 Seismic Margin Assessments

6.3.1 General

Besides evaluation of the seismic safety of existing structures against the DBE load, it might also be necessary to consider effects of beyond the design basis, in order to avoid any "cliff-edge effects", as described in section 5.4.2. As described in section 3.2.5 a number of deficiencies in the characterization of the seismic ground motions in SKI Technical Report 92:3 [8] is specified. For instance, the applied database is very old (from the 1970s) and has not been updated despite international recommendations to confirm/update the seismic hazard assessments every 10 years, and ENSREG has specifically also questioned the very short geological time scale (500 years) for applied observations and historical account as well as the lack of paleoseismic data.

Thus, it is quite likely that a future reassessment of the seismic hazard would result in increased hazard levels. Further as stated in section 5.4.2 it is also important to consider that new modern standards for seismic hazard assessments and more sophisticated analysis methods may indicate insufficient margins. Also, as will for instance be the case in France, there might be new requirements on verifying specified essential safety equipment for a seismic event beyond the design basis, as discussed in section section 5.4.2, not only for new structures but also for existing ones.

To sum up it might be necessary to provide safety evaluations and assessments for beyond design basis seismic events for the existing Swedish NPPs.

The main aim of a seismic safety evaluation of an existing nuclear facility is to determine the true state of the evaluated SSCs in terms of their required safety function and their seismic capacity and thus to assess the seismic safety margin. The approach used by the SMA methodology is to consider a reasonable higher seismic hazard level than the design basis and to associate this with realistic and best estimate values for the as-is condition of the SSCs. Hence, the inherent excess capacity of the SSCs can be accounted for.

The SMA approach is in general performed in six different steps:

- Define the objectives of the seismic safety evaluation.
- Determine the Review Level Earthquake (RLE).
- Selection of success paths and of selected SSCs.
- Seismic response analysis.
- Capacity assessments of the selected SSCs.
- Determination of utilization ratios for the selected SSCs

On the basis of the SMA approach with its different procedures and specifically also the Conservative Deterministic Failure Margin (CDFM) method as described in EPRI NP-6041-SLR1 [70], a proposal for a somewhat simplified deterministic method for seismic safety evaluation of safety-related nuclear structures is presented in section 6.3.2 to 6.3.7.

6.3.2 Objectives of the seismic safety evaluation

The first important step is to establish the main purposes of the seismic safety evaluation. In section 6.1.2, the four most common objectives are specified. The approach presented here is limited to a safety evaluation of the building structures in line with the following main objectives:

- To evaluate the seismic safety margin beyond the DBE and to confirm that there are no cliff edge effects.
- To identify weak links as regards maintaining required functions in the structures during and after an earthquake beyond the DBE.

In accordance with these objectives, it is important to determine the RLE level of the seismic ground motion, the safety functions to be ensured and the acceptance criteria for the identified critical failure modes to be prevented.

The methodology presented here is not dependent on any eventual SMA for the safety equipment in the plant, but can of course be one acitivity in a larger SMA scope at plant level. Anyway, the step to select the success paths and selection of essential SSCs ought to be determined on a plant level, to be able to identify essential safety equipment and their locations in the plant, as basis for defining the structural elements which must maintain supporting and/or shielding functions of these SSCs.

One disadvantage with the approach described above is that only two of the Swedish NPP units (Oskarshamn 3 and Forsmark 3) include seismic load in their design basis. This means that for the other units, a successful seismic evaluation against a post-defined DBE must first be executed in accordance with for instance the procedures in section 6.2.

6.3.3 Determine the Review Level Earthquake (RLE)

The Review Level Earthquake (RLE) defines a screening level in the evaluation process. The RLE should be defined with a sufficient margin over the original DBE, in order to ensure plant safety and to identify any essential "weak links" that may limit the overall capacity of the plant to withstand a seismic event beyond the original DBE.

Most of the procedures developed and implemented to date have defined two screening levels; PGA of 0.3 g and PGA of 0.5g. These PGA levels were established on the basis of seismic hazard values and earthquake experience data from the U.S.

According to section 2.2.6.4, the USNRC has recommended the US licensee to assure a safety margin for a Beyond Design Basis Earthquake (BDBE) corresponding to a load of 1.67 times the DBE. This BDBE correspond in probabilistic terms to a doubling of the mean return time from 10 000 years (DBE) to approximately 20 000 years. For the new EPR NPP plant, seismic margin assessments have been performed with a target margin of 1.6 times that assumed for the DBE, as stated in section 2.3.3.4 and 2.3.3.5.

The DBE at the Swedish nuclear facilities is defined in terms of a uniform annual frequency of exceedance response spectrum shape according to SKI Technical Report 92:3 [8], in contrast to for instance the standard type of response spectra in RG 1.60 [15]. The CDFM method as described in EPRI NP-6041-SLR1 [70] recommend to select the RLE in terms of a uniform hazard spectra in case the DBE is defined similarly, in order to achieve uniformity in the frequency domain between the DBE and RLE and in the safety margin assessments. Then, statements as regard seismic margins can be expressed in probabilistic terms of annual frequency of exceedance, as opposed to the deterministic PGA.

According to international practice, the RLE level often is determined to be approximately 1.6 to 2.0 times the DBE level. Thus, a first approach could be to apply a trial responce spectra with an annual exceedance probability of $1 \cdot 10^{-6}$ according to SKI Technical Report 92:3 [8], which correspond to a PGA of 0.23 g, that is 2.0 times the PGA of the DBE (annual exceedance probability of $1 \cdot 10^{-5}$). If too many essential structural elements fail in the evaluation at this level, it can be necessary to reduce the RLE level in a second step.

As stated in section 5.4.2, it was in the joint project Seismic Safety during the 1980s between the then Swedish nuclear safety authority (SKI) and the Swedish licensees concluded that SSCs essential for the containment integrity, isolation and pressure relief should have a sufficient ultimate capability to withstand a seismic load corresponding to an annual exceedance frequency of 10^{-7}. Consequently, the Swedish licensees carried out calculations and engineering judgements for a selection of the most important of these safety functions at an annual exceedance probability of 10^{-7}, within the scope of the European stress tests during 2011. Hence, in order to comply with the recommendations in the project Seismic Safety, but also to specifically consider future increased seismic hazard levels due to identified deficiencies in the characterization of seismic ground motions in SKI Technical Report 92:3 [8] as stated in section 3.2.5, it may be advisable to apply an annual exceedance probability of $1 \cdot 10^{-7}$ according to SKI Technical Report 92:3 [8] as RLE for certain essential safety-critical SSCs.

6.3.4 Selection of success paths and selected SSCs

The SMA uses a success path approach to determine which SSCs should be selected. A success path means a path of SSCs (selected SSCs) that can successfully bring the plant to the desired end state for the plant (e.g. safe shutdown). Other types of successful end states could for instance include the defence in depth and system redundancy for the nuclear reactors, and successful confinement of nuclear waste during and after an earthquake for a NPP and other nuclear facilities.

The most fundamental safety functions to be ensured for the RLE level are:

- Control of reactivity.

- Removal of heat from the core.
- Confinement of radioactive materials, control of operational discharges as well as limitation of accidental releases.
- Cooling of spent fuel.

In terms of structures and structural elements whose safety functions (confinement, support or shielding) need to be preserved, the following elements can be specified as a minimum:

- The reactor containment.
- The scrubber building.
- The spent fuel pools.
- The structural elements housing or shielding essential safety equipment to preserve the fundamental safety functions as above.

6.3.5 Seismic response analysis

For those SSCs selected as part of the safe shutdown success path, the seismic response should be determined for:

a. Evaluation of the structural capacity on the basis of required function to be maintained and the damage mode.
b. Generation of in-structure response spectra.

For each building and structure which has been defined as part of the selected SSCs, the safety functions to be maintained, the damage modes for these functions and associated acceptance criteria should be defined.

The evaluation procedure for determining the best estimate (median centred) of the seismic response should be defined. When using best estimate parameters it is acceptable to use Response Level 3 damping values according to Table 3.2 in ASCE 43-05, if a linear elastic analyses is used and if the structural elements are near yield. Response Level 3 is limited to the evaluation of the structural elements. For generation of in-structure response spectra Response Level 2 is used.

The normal procedures for the response calculations are through dynamic structural analysis, but in certain simple cases, scaling can also be accepted.

Evaluation of equipment-building interfaces (e.g. anchorages, anchor plates and anchor bolts) should be included in the evaluation of selected equipment and distribution systems. All dominant failure modes of these interfaces, such as failure of the anchorage or the substructure should be identified and evaluated on the basis of as-is conditions. The expected behavior of the supporting building structure elements should also account for local concrete cracking which may reduce the capacity of expansion anchor bolts.

6.3.6 Capacity assessments of the selected SSCs

In a SMA, capacities of selected SSCs are defined as HCLPF capacities. In probabilistic terms, the HCLPF capacity of an SSC is the earthquake level at which there is a high confidence (95%) of a low (5%) probability of failure. Although defined with a probabilistic approach, HCLPF values are almost always calculated by deterministic methods. Deterministic guidelines in the form of the CDFM-method as described in EPRI NP-6041-SLR1 [70], have been developed and demonstrated to yield the approximate probabilistic definition.

The required functions of the selected SSCs must be identified. For example, required functions of structures can be confinement, support and/or protection of other SSCs. Realistic failure modes of the SSCs, that is the inability of the SSCs to perform its required safety function due to inadequate seismic capacity, have to be determined. For structures with proven ductility capacity against cyclic loads, some non-linear behavior can be accepted, but at lower levels than for conventional industrial facilities. In seismic safety evaluations of SSCs, ageing degradation should be considered if the ageing effects reduce the seismic capacity of SSCs.

The as-is concrete classes used for the construction of the structures should be verified on the basis of existing plant specific tests and industry standards for concrete. Either destructive or non-destructive methods may be used. The actual material properties of the reinforcement steel should be used in the evaluation. Material properties should be available from existing test data.

For Swedish conditions it can be reasonable to accept characteristic values on concrete and reinforcement, i.e. to use f_{yk} for reinforcement and f_{ck} and f_{ctk} for concrete without any reduction to reflect design, and including effects of concrete ageing if appropriate.

In order to assess the vulnerability due to eventual non-compliancy of certain structural elements when using characteristic values, it might be reasonable to instead apply median centred strength property values in a second step.

The ductility of the Swedish nuclear structures to cyclic earthquake loads is limited because of the absence of ductile reinforcement arrangements according to international standards. However, limited non-linear assessments may be applied for exceptional cases when appropriate. Then, especial attention should be paid to ensuring that shear or other non-ductile failure modes can be excluded with sufficient margins for the investigated elements.

6.3.7 Determination of utilization ratios for the selected SSCs

The calculated load effects from the seismic response analysis as determined according to section 6.3.5 (i.e. demand) are compared to the capacities determined in section 6.3.6.

Thus, eventual weak links in the structures may be identified. Alternatively, if too many essential structural elements have failed, an updated SMA can be executed for a lower RLE level.

Finally, a seismic margin assessment for the building structures can be expressed in probabilistic terms, by interpolating between applicable typical seismic hazard curves for the different exceedance probability levels of the DBE and RLE respectively.

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Appendix 1: List of figures

Appendix 2: List of tabels

Appendix 3: List of Acronyms

ALARP	As Low As Reasonably Practicable
ASN	Autorité de Sureté Nucléaire
DRB:2001	Design Rules for Buildings
BFS 2011:10 – EKS 8	Boverkets föreskrifter och allmänna råd om tillämpning av europeiska konstruktionsstandarder (The Eurocodes)
BDBE	Beyond Design Basis Earthquake
BOVERKET	The Swedish National Board of Housing, Building and Planning
BWR	Boiling Water Reactor
CDFM	Conservative Deterministic Failure Margin
CFR	Code of Federal Regulations
DBE	Design Basis Earthquake
DRS	Design Response Spectra
DEC	Design Extension Conditions
DEE	Design Extension Earthquake
EKS	see BFS 2011:10 – EKS 8
EC	European Council
ENSREG	European Nuclear Safety Regulators Group
EPR	European Pressurized Reactor
EPRI	Electric Power Resaerch Institute
ETC-C	French Association for Design, Construction, and In-Service Inspection Rules for Nuclear Island Components, afcen, ETC-C EPR Technical Code for Civil Works
HCLPF	High Conficence of a Low Probability of Failure
H_D	Hazard exceedance probability
DNB	Dimensionering av Nukleära Byggnadskonstruktioner (Design of Nuclear Buildings)
IAEA	International Atomic Energy Agency
MHPE	Maximum Historically Probable Earthquake
MVSS	Multi-Venturi Scrubber System
NA	National Annex
NPP	Nuclear Power Plants
OBE	Operating Basis Earthquake

PGA	Peak Ground Acceleration
PSR	Periodic Safety Review
PSA	Probability Safety Assessment
PSAR	Preliminary Safety Analysis Report
RFS	Fundamental Safety Rule
RG	Regulatory Guide
RLE	Review Level Eartquake
SAR	Safety Analysis Report
SC	Seismic Category
SDC	Seismic Design Category
SKI	Statens Kärnkraftsinspektion (today SSM)
SFP	Spent Fuel Pool
SMA	Seismic Margin Assessment
SPSA	Seismic Probabilistic Safety Assessment
SRP	Standard Review Plan
SSC	Structures, Systems and Components
SSE	Safe Shutdown Earthquake
SS-EN	Swedish Eurocodes in general
SSM	The Swedish Radiation Safety Authority (Strålsäkerhetsmyndigheten)
SSMFS	Safety Regulations issued by SSM
TRAFIKVERKET	The Swedish Transport Administration
TSN	Transparancy and Security in Nuclear field
UHRS	Uniform Hazard Response Spectra
USNRC	United States Nuclear Regulatory Commission
WENRA	Western European Nuclear Regulators' Association
YVL Guides	Guides from the Finnish Radiation and Nuclear Safety Authority

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The Swedish Radiation Safety Authority has a comprehensive responsibility to ensure that society is safe from the effects of radiation. The Authority works to achieve radiation safety in a number of areas: nuclear power, medical care as well as commercial products and services. The Authority also works to achieve protection from natural radiation and to increase the level of radiation safety internationally.

The Swedish Radiation Safety Authority works proactively and preventively to protect people and the environment from the harmful effects of radiation, now and in the future. The Authority issues regulations and supervises compliance, while also supporting research, providing training and information, and issuing advice. Often, activities involving radiation require licences issued by the Authority. The Swedish Radiation Safety Authority maintains emergency preparedness around the clock with the aim of limiting the aftermath of radiation accidents and the unintentional spreading of radioactive substances. The Authority participates in international co-operation in order to promote radiation safety and finances projects aiming to raise the level of radiation safety in certain Eastern European countries.

The Authority reports to the Ministry of the Environment and has around 315 employees with competencies in the fields of engineering, natural and behavioural sciences, law, economics and communications. We have received quality, environmental and working environment certification.

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