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NUCLEAR ENERGY AGENCY
COMMITTEE ON THE SAFETY OF NUCLEAR INSTALLATIONS

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DIFFERENCES IN APPROACH BETWEEN NUCLEAR AND CONVENTIONAL SEISMIC STANDARDS WITH REGARD TO HAZARD DEFINITION

CSNI INTEGRITY AND AGEING WORKING GROUP

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

The mission of the NEA is:

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

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

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

COMMITTEE ON THE SAFETY OF NUCLEAR INSTALLATIONS

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

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

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

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

FOREWORD

At the Committee on the Safety of Nuclear Installations (CSNI) meeting in June 2003, a proposal of the CSNI Working Group on the Integrity and Ageing on the differences in approach between nuclear and conventional seismic standards with regard to hazards definition was approved. A previous paper published by the group addresses differences in methodologies and highlighted directions where the nuclear industry should tend to maintain its capabilities and leadership.

There is a perception, mainly in some of the European countries that nuclear seismic hazards and design standards may be lagging behind developments in similar standards for conventional facilities. Adequate answer to such perception, need the examination of the following aspects and their significance on the seismic assessment of structures and components:

- The safety philosophy behind the seismic nuclear and conventional standards.
- The differences in approach regarding the seismic hazard definition.
- The differences in approach regarding the design and the methods of analysis.

These topics are examined in this report. Description of the conventional and the nuclear approaches in NEA member countries were collected and included as appendices to this report.

ACKNOWLEDGEMENTS

Gratitude is expressed to the delegates of the CSNI Working Group on the Integrity of Components and Structures for providing the description of their conventional and nuclear approaches with regard to hazard definition. Special thanks to Mr. Ali Djaoudi (Tractebel, BLG) leader of the writing group composed of Prof. Pierre Labbe (EDF SEPTEN, FR), Dr. Andrew Murphy (USNRC, USA) and Dr. Yoshio Kitada (JNES, JPN) for preparing the report.

EXECUTIVE SUMMARY

The Committee on the safety of Nuclear Installations (CSNI) of the OECD-NEA co-ordinates the NEA activities related to maintaining and advancing the scientific and technological knowledge base of the safety of nuclear installations.

The Integrity and Ageing of Components and Structures Working Group of the CSNI is responsible for work related to the development and use of methods, data and information to assess the behaviour of materials and structures. It has three sub-groups, dealing with the integrity of metal components and structures, ageing of concrete structures, and the seismic behaviour of structures.

The CSNI has been actively working in the area of seismic behaviour over the last years. It has completed a series of three workshops on the definition of seismic input motions (BNL, USA 1999 – Istanbul, TK, 2002 – Tsukuba, JPN, 2004), and a report synthesizing the outcomes is under preparation in a way to indicate what the CSNI has achieved and to emphasize the better synergy between geologist, seismologist and engineers. The CSNI also started a study on NPPs and other facilities that have experienced an earthquake.

The CSNI, at its meeting in June 2003, agreed to initiate an activity aimed to identify any difference between nuclear and non-nuclear conventional standards and their potential significance with regard to seismic hazards and design methods.

There was a perception, mainly in some of the European countries that nuclear seismic hazard and design standards may be lagging behind developments in similar standards for conventional facilities. Adequate answer to such perception, need the examination of the following aspects and their significance on the seismic assessment of structures and components:

- The safety philosophy behind the seismic nuclear and conventional standards.
- The differences in approach regarding the seismic hazard definition.
- The difference in approach regarding the design and the methods of analysis.

These topics are examined in this report. Appendices A to H of this report contain a brief description of the conventional and the nuclear approaches in the NEA member countries: Belgium, Canada, Czech Republic, Germany, Japan, South Korea, Spain,and USA.

The following general conclusions can be drawn:

- The approach adopted by the nuclear seismic standards is more conservative and more reliable - in particular for meeting the continued operation criteria - than the recommended by the currently applicable force based conventional seismic codes.
- Introduction of the displacement based method considered as an advanced technique by the conventional industry (need for accurate modelling and introduction of the actual nonlinearities more realistic design performance assessment), is neither considered as efficient nor sufficiently conservative to be used for the nuclear structures and components. Use of displacement based methods by the nuclear industry need a definition of new acceptance criteria.

- Considering the difference in approaches on safety objective, hazard definition and the methodologies, one cannot consider that nuclear seismic hazards determination and design standards are lagging behind developments in similar standards for conventional facilities.

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1. SCOPE

There is a perception, mainly is some European countries that nuclear seismic hazards determination and design standards may be lagging behind developments in similar standards for conventional facilities.

Adequate answer to such perception, need the examination of the following aspects and their significance on the seismic assessment of structures and components:

1. The safety philosophy behind the seismic nuclear and conventional standards.
2. The differences in approach regarding the seismic hazard definition.
3. The differences in approach regarding the design and the methods of analysis.

These topics are examined in this paper. Appendices A to H attached to this paper contain a brief description of the conventional and the nuclear approaches written by each of the following participant countries: Belgium, Canada, Czech Republic, Germany, Japan, South Korea, Spain and USA.

2. INTENDED SAFETY GOALS OF THE NUCLEAR AND CONVENTIONAL STANDARD

2.1 Conventional facilities

In general, conventional seismic codes contain the minimal requirement destinated primarily to safeguard against major structural failure and loss of life not to limit damage at a certain acceptable degree or maintain function. In others words, crossing to elastoplastic domain are allowed. This aspect is clearly stated in the previous version of the UBC Code (Ref. 3). Even if it is no more so clearly stated, new version of this code the IBC Code (Ref. 2) still follow the same safety philosophy (Importance factors for buildings and other structures, seismic design categories). In the European seismic standard Eurocode 8 (Ref. 4), the requirements are clearly postulated: human lives protection and damage limitation.

Conventional seismic codes deal mainly with buildings and certain important "structure like" equipment e.g. bridges, tanks and stacks… For others components, the requirement are quiet lacking in some codes or are limited to the loading definition without proper consideration to the functional aspects.

2.2 Nuclear facilities

Nuclear seismic rules require that structures, systems and components important to safety to withstand the effects of earthquakes. The requirement states clearly that for equipment important to safety, both integrity and functionality should be ascertained. In others words, incursions in the elastoplastic domain is not allowed.

Appendix H to part 100 of the US-CFR regulations (Ref. 6) defines 2 seismic levels, OBE and SSE with specific safety requirement defined for each level.

The *Operating Basis Earthquake Ground Motion (OBE)* is the vibratory ground motion for which those features of the nuclear power plant necessary for continued operation without undue risk to the health and safety of the public will remain functional.

The *Safe Shutdown Earthquake Ground Motion (SSE)* is the vibratory ground motion for which certain structures, systems, and components important to nuclear safety must be designed to remain functional.

The same global safety approach is applied in many European and Asian states which follow the NRC regulations.

IAEA Safety Guide (Ref. 1) defines the SL-2 and the SL-1 which can be compared to the SSE and OBE

French regulations, defines two levels with a specific safety requirement for each: the SMS (Séisme Majoré de Sécurité (Ref. 11) comparable to the SSE. The 1/2 SDD (Demi Séisme de Dimensionnement) comparable to the OBE. The same approach is followed in Japan where 2 seismic levels are adopted, S1 and S2 with safety requirement defined as follows:

- S1: The maximum design earthquake which is determined basically from earthquake history and active faults.
- S2: The extreme design earthquake beyond the maximum design earthquake which is determined from the seismological point of view.

Nuclear seismic standards related with safety have more extended scope than the conventional ones, they includes buildings, structures and electromechanical active or passive equipment. Other structures not related to safety like the turbine building are, however, out of scope of the nuclear standards.

3. SEISMIC HAZARD DEFINITIONS

3.1 Probability of occurrence - Return period

The design seismic action is generally selected on the basis of the annual probability of exceedance (return period). There is a major difference on the return period definition for design earthquakes between the conventional and the nuclear seismic standards.

As pointed out above, the nuclear standards define 2 levels of earthquake, with 2 different levels of probability of occurrence: the OBE and the SSE as defined in the US and some Asian and European countries.

The return period of used acceleration to anchor the design spectra in conventional standards like the Eurocode 8 is based on the 10% probability of exceedance in 50 years e.g. 475 years return period which, in moderate seismicity regions, corresponds closely to the PGA of OBE defined in the nuclear standards. The option of other return period is considered in this code through the factor for structures critical for the safety of the public.

The return period corresponding to the SSE peak ground acceleration defined in nuclear seismic standards is based on the 0.5% to 0.05% probability of exceedance in 50 years e.g. 10 000 to 100 000 years return period.

The return periods of the S1 and the S2 in the Japanese nuclear standard (JEAG) are defined indirectly by taking into account the recurrence activities of the related faults as being 10 000 years for S1 and 50,000 years for S2 respectively.

The IAEA Safety Guide (Ref. 12) defines the SL-2 level as very low probability of occurrence corresponding to ultimate safety requirement and the SL1 as less severe more likely earthquake which has different safety implications than the SL-2.

3.2 Seismic hazards definitions methodologies

3.2.1 Conventional seismic standards

In conventional seismic codes, seismic hazard has been obtained by classical probabilistic approaches focused to define averaged acceleration (usually PGA) to be used with free field standard response spectra. The shape of such spectra is established through statistical analysis of acceleration time histories and adapted to local soil data (soil categories). Recommendations for the definition of site specific are given by certain codes (IBC 2000, Eurocode 8). In the Eurocode 8 this spectra are then scaled to the EPA (effective peak acceleration) which is defined as 0.75* PGA (Peak Ground Acceleration).

3.2.2 Nuclear seismic standards

From a general point of view, to analyze seismic hazard and define the SSE, nuclear seismic rules gives more structured framework procedures than conventional codes. In the US and some European countries, seismic hazard is performed by a deterministic approach to obtain the worst earthquake scenario in order to define the SSE. More recently, some countries have developed a new PSHA approach to obtain spectra

with uniform probability in order to check and to guarantee the conservatism of the defined SSE. Two procedures are followed to define the shape of the SSE design response spectra (the OBE is defined like a fraction of the SSE):

1. To use standard seismic response spectra like the USNRC RG 1.60 in the US and some Asian and European Countries or the "Oshaki" spectra (currently under revaluation process).
2. Using site specific data to develop a site specific response spectrum.

4. DIFFERENCES IN APPROACH REGARDING THE DESIGN METHODS

4.1. Buildings and structures

4.1.1 Conventional facilities

The codes and standards are also based on the use of the force based methodology.

The basic assumption used in this methodology is that the energy dissipation by post elastic deformation is allowed, while the calculation is performed on the elastic domain by an artificially lowered seismic loading. The methodologies propose to use dynamic analysis methods (modal and spectral, time-history). Simplified analyses are proposed for structures under certain conditions of regularity and symmetry.

A key feature on the seismic assessment of conventional structures is the use of the so called "ductility factor" or "behaviour factor". The use of such factor to lower intentionally the actual seismic load is consistent with the safety philosophy of the conventional seismic standard which is the "non collapse" rather than the integrity and/or the operability of the structures of components. Consequently, applying the same design spectrum will lead to significantly lower seismic loading for conventional structures than for nuclear structures. A source of potential inaccuracies is that application of the behaviour factor need a thorough check of the local and global dissipative capacity of the structure. It has to be noted that such verification is often not correctly treated; Experience show that the behaviour factor is somehow often applied on a "blind" way to the whole structure. In the experience of real earthquakes, there are many instances of serious damages due to the application of the behaviour factor in local zones where the dissipative capacity is questionable. For instance, shear in anchor bolts, pullout in anchor rods without sufficient elongation length, shallow adhesive or expansive anchors bolts, shear lugs, action of braces in foundations pads etc. These cases show clearly that the design based on the conventional codes may lead to serious structural damage. In our view, there is a need to revise the existing codes to include an adequate consideration of the "behaviour factor".

4.1.2 Nuclear facilities

Seismic design of nuclear facilities to OBE and SSE earthquakes is based on dynamic analysis methods using the modal and spectral theories or direct integration methods assuming a linear behaviour of the material. It's assumed that the structure remain in the elastic domain; energy dissipation by incursion in non elastic domain is not allowed. The structural verification is therefore based on the application of the full magnitude of the seismic action. This is coherent with the safety philosophy stated in § 2.2 since structures important to safety must remain functional after an SSE seismic event. Conservative static equivalent method is also used for structures for which the structural behaviour can be approximated by an SDOF system. All methodologies are "force based method".

For recently (post 1975) designed or seismically re-evaluated nuclear facilities, dynamic analyses of buildings and structures are performed on the basis of modern methods using all the "state of the art" on FEM modelling, modal and spectral analysis, soil-structure Interaction.

Variability and sensitivity studies (input motions, broadening of response spectra, soil parameters and concrete characteristics) are considered.

4.2 Components

4.2.1 Conventional standards

In general, the conventional national seismic standards don't include clear methodologies or requirement to assess seismically the components. The IBC 2000 includes the components but the methodology and the criteria are still limited to the evaluation of the acceleration and consequently the applied forces which are still lowered by the behaviour factor. Such requirements don't guarantee the functionality or even the integrity of the component provided that post-elastic deformation is allowed.

4.2.2 Nuclear standards

To meet the safety objectives requiring the continued operation during or during and after an earthquake, the nuclear standards define stringent requirement destined to duly qualify those items to earthquake. The Standards as the IEEE 344 defines the recommendations to assess seismically the 1E electrical components by analysis, test and the combination of test and analysis. The ASME QME gives requirement to assess by analysis, by test or by combination of test and analysis the mechanical active equipment such as valves and pumps. Analysis is used when structural requirement governs the seismic qualification, while for active and complex components or parts; the shake table tests are required.

5. PERSPECTIVES

5.1 Conventional facilities

It will be noted, that in the recent versions of the conventional codes the definition of the behaviour factors tends to be less approximately defined than it were in the past. More warnings and more requirements pushing to a rational use of this coefficient are given. For instance, in the Eurocode 8, distinction between dissipative and non dissipative structures is clearly established and the behaviour factor for non dissipative structures factor is lowered (1.0 to 1.5).

In general, the intent of these recommendations is to use more rationally and more carefully the ductility factor but not to eliminate it.

Regarding the safety philosophy, some evolution is being introduced in the new version of the Eurocode 8 by considering the "damage limitation" criteria for higher probability of occurrence earthquake (Return period years).

In October 1997, the FEMA Publication 273 went with a new approach to check the seismic behaviour of existing building. The FEMA defined a new methodology based on the displacement determination rather than in the force (Displacement based method). Nonlinear Static Procedure (NSP) and Nonlinear Dynamic procedure; in those methods, the model incorporating inelastic material response is displaced to a target displacement, and resulting deformations and forces are determined. The target displacement is intended to represent the maximum displacement likely to be experienced during the design earthquake. Consideration of the actual non linear behaviour of the structure is expected to give more accurate information on its post elastic deformation capacity, the uncertainties due to the ductility factor are reduced and the local as well as the overall performance of the structure are more adequately assessed.

5.2 Nuclear facilities

Experience from past and relatively recent earthquakes and new developments to deal with the uncertainties in more effective way are being incorporated, the nuclear safety rules affecting definition of the free field SSE experienced significant evolution in some countries like France and USA.

Provided that the nuclear industry is currently based on up to date dynamic analysis and that the responses should remain in the elastic linear domain, there is no special arguments to change significantly the current methodology standards. The question is whether the nuclear industry should or should not adopt the displacement based method and if yes, to specify the limitations (II/I interactions for instance) and the specific criteria.

6. CONCLUSIONS

The approach adopted by the nuclear seismic standards is more conservative and more reliable (in particular for meeting the continued operation criteria) than this recommended by the currently applicable force based conventional seismic codes. The arguments behind this are:

- The safety target is the conservation of the integrity and the function of structures and components important to safety rather than damage limiting or collapse avoidance stated in conventional standards.
- The structures are designed to remain in the elastic zone, incursion on inelastic zones is not allowed as it is in the conventional standards.
- To analyse seismic hazard and define the SSE, nuclear seismic rules gives more structured framework procedures than conventional codes.
- In certain cases the Ground Response Spectra defined by conventional standards shows higher acceleration values in certain frequency range than those defined in the nuclear standards; such discrepancy cannot be regarded as a consideration of lower hazard for the nuclear facilities but it is duly justified by the use of updated and more realistic seismic siting parameters.
- Definition of two seismic levels versus one in the conventional facilities, the return period corresponding to the SSE is well beyond this defined in conventional standards and makes the nuclear safety related structures and components designated to withstand to higher seismic level.
- Response spectra defined in the conventional structures are in general defined for large areas and are less accurately determined locally.
- SSI aspects and sensitivity study of soil parameters are more thoroughly evaluated in the nuclear approach.

Introduction of the displacement based method considered as an advanced technique by the conventional industry (need for accurate modelling and introduction of the actual nonlinearities more realistic design performance assessment), is neither considered as efficient nor sufficiently conservative to be used for the nuclear structures and components. Use of displacement based methods by the nuclear industry need a definition of new acceptance criteria. However, this method may present some benefits for the evaluation of the II/I interactions.

Considering the difference in approaches on safety objective, hazard definition and the methodologies, one cannot consider that nuclear seismic hazards determination and design standards are lagging behind developments in similar standards for conventional facilities.

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Appendix A.

BELGIUM

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1. Conventional standards

1.1 Applicable standard and seismic hazard mapping

Application of seismic code to conventional structures is relatively recent in Belgium. The applicable code is the ENV 1988 -1-1 (Eurocode 8) which is required for collective use buildings and structures. For private installations, the code is recommended but alternate seismic requirements or alternate seismic codes may be specified (e.g. LNG terminal).

The map of seismic hazards established in the Belgian NAD (National Application Document) attached to the Eurocode 8 is based on the probabilistic methodology developed by Cornel (1968) which is based on 3 assumptions:

- Exponential distribution of the magnitude of seismic events (Gutember-Richter).
- Time of recurrence following a Poisson distribution.
- Uniform distribution of the seismicity inside the zones defined as seismic sources.

Seisrisk III computer program were used to establish the hazard map in terms of accelerations on bedrocks. The seismic data are those given in the ORB (Observatoire Royale de Belgique) earthquakes catalogue (1998). Median + 1 standard deviation Ambraseys (1995) attenuation law was used for the hazard evaluation.

1.2 Ground response spectra definition

The seismic hazard is defined by a Design Ground Response Spectra as defined in this code; the Peak Ground Acceleration (PGA) is defined and mapped in the Belgian NAD (Figure 1). Belgium is a low seismic area; three zones are defined with the following range of PGA values:

- Zone 0 with no significant seismic acceleration.
- Zone 1 with PGA = 0.05 g.
- Zone 1 with PGA = 0.1 g.

In the Eurocode the seismic design response spectra is based on the effective ground acceleration (EPA) spectra rather the PGA. The design acceleration value is given by: $a_g \times \gamma_I \times S$

Where *ag* is the EPA set equal to 0.7xPGA in the Belgian NAD,
γ_I is the importance category of the construction; (longer return period associated with higher importance factor),
S is the soil factor.

If we consider a conventional building, with the highest importance (γ_I=1.4 as recommended by the Eurocode) to be erected in the Tihange site (Soil category B) which host three NPPs, the acceleration to be considered is: 0.7×0.05×1.4×1 = 0.05g; this value corresponds to the ZPA of the OBE as defined for the NPPs. The horizontal Ground Response Spectrum to be considered to such conventional building, with the Tihange soil condition, is given in figure 2 (curve 1).

The same assumption in Doel site (Soil category C) lead to the same acceleration level (0.05 g) and the horizontal Ground Response Spectrum with the Doel soil condition is given in figure 3 (curve 1).

2. Nuclear standards

2.1 Applicable standards

The applicable codes and standards are US NRC Regulatory Guides and the Standard Review Plans (SRPs) relative to seismic sitting and SS&C's seismic qualification are applicable for Belgian NPPs.

2.2 Ground response spectra definition

There are two NPP sites with 3000 MW capacity each: Doel site located in the North West and Tihange site in the South East. The Design Response Spectra are defined as follows:

- OBE Earthquake RG 1.60 - 0.05 g for both sites.
- SSE Earthquake RG 1.60 - 0.1 g for the Doel site.
- SSE Site Specific Earthquake - 0.17 g for the Tihange site.

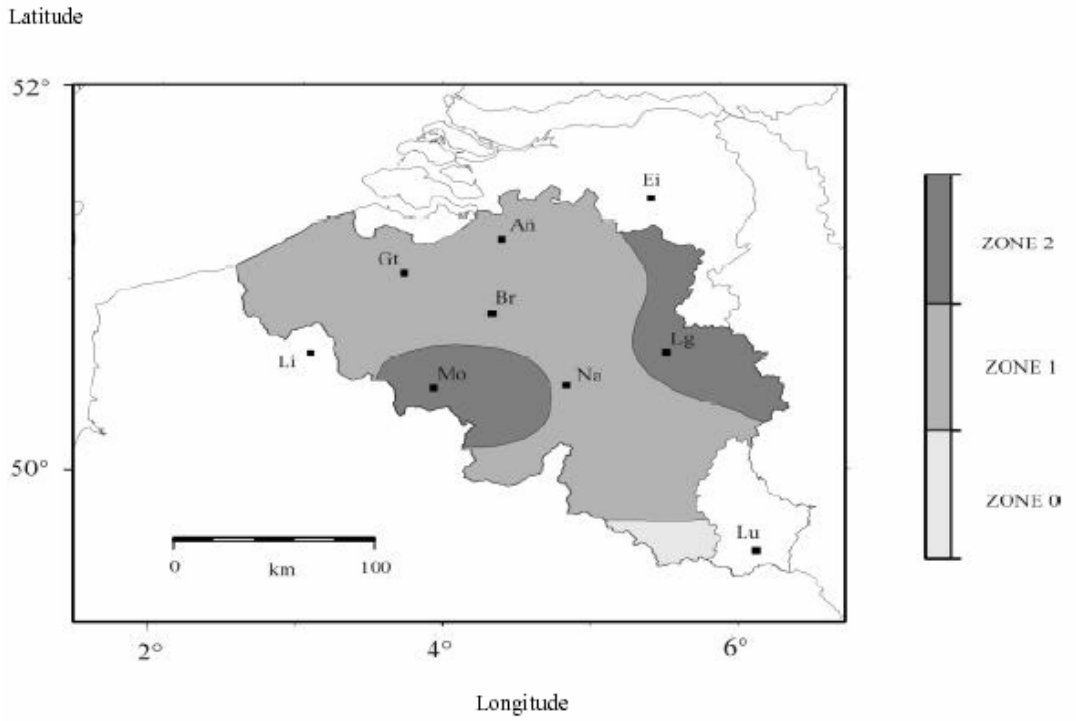
Curves (2) in figures 2 and 3 give the OBE horizontal Ground Response Spectrum respectively for Tihange and Doel sites, Curves (3) in the same figures gives the SSE horizontal Response Spectrum.

In Doel site, the SSE PGA was set to the minimal value recommended by the RG - 1.60 (0.1 g). In Tihange site, the PGA of 0.17 g was determined for the seismic revaluation by a deterministic seismic hazard assessment method based on:

- The historical and instrumental seismicity of the region.
- Shifting of the epicenter of the maximal historical event of the identified seismotectonic areas to closest edge of the site.
- Assuming an attenuation law.

The amplification ratios were evaluated by a statistical study (mean + one standard deviation) of seismic recordings made in equivalent local soil conditions in Europe and in the USA.

Figure 1. **Seismic hazard map of Belgium**



Zonation map:
(1) The seismic zonation of Belgium is given at Figure 1. Belgium has 3 zones where the design ground acceleration at the bedrock level (PGA ou Peak Ground Acceleration are respectively:
Seismic Zone 0: No significant acceleration
Seismic Zone 1: PGA = 0.05 g (0.50 m/s^2)
Seismic Zone 2: PGA = 0.10 g (1.00 m/s^2)

Figure 2. **Belgium – design ground response spectra for conventional and nuclear standards Tihange site**

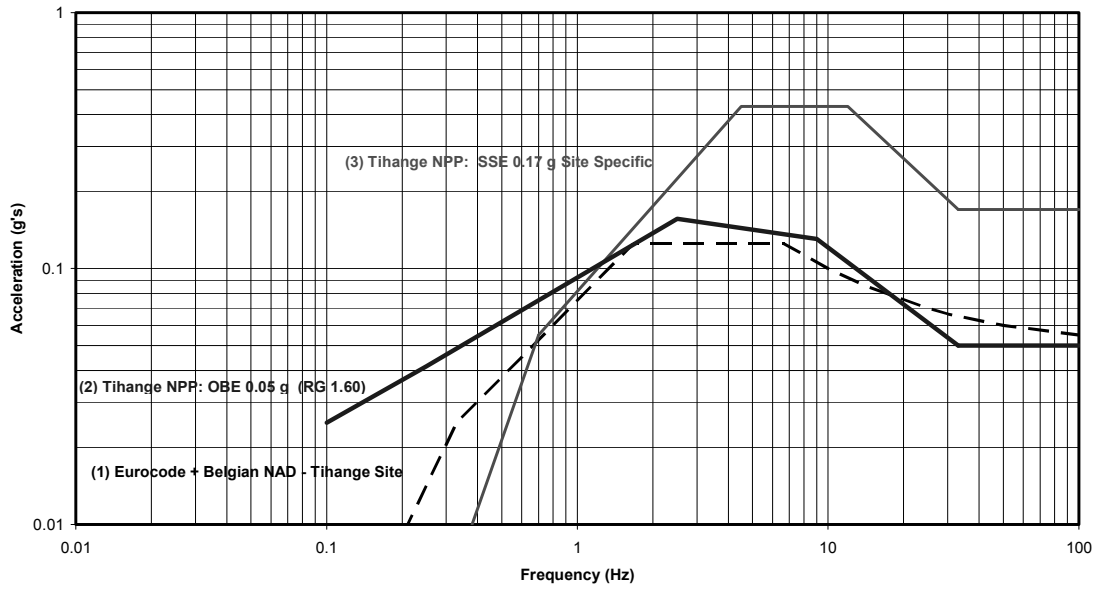
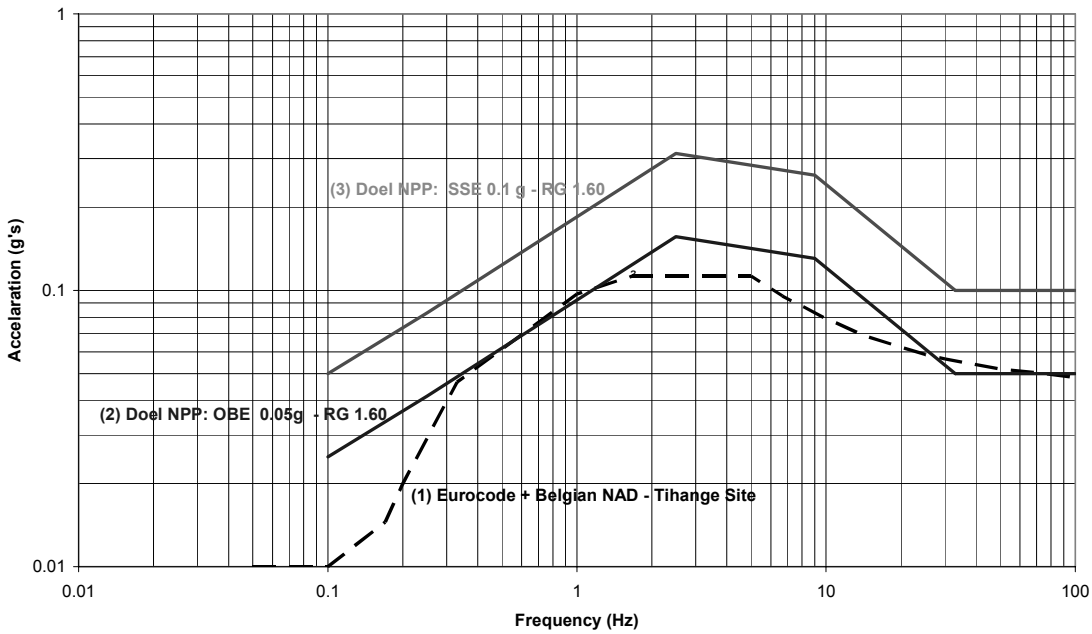


Figure 3. **Belgium – Design ground response spectra defined by conventional and nuclear standards Doel site**



3. Comparison of the nuclear and conventional seismic loadings - Case study

In order to compare the seismic design loads using the conventional and nuclear standards the magnitude of the Base Shear are estimated using the respective applicable spectra. For the sake of simplification, the structures are assumed to be symmetric, regular, dissipative and behaving ideally as an SDOF system.

The acceleration is determined for two current fictive types of buildings:

1. Relatively stiff structure where lateral loads are resisted by shear walls.
2. Framed concrete structure with infill masonry walls.

Application of the Eurocode 8 + NAD for Conventional Standard

	Assumed frequency (Hz)	Behaviour factor (q)	Spectral acceleration (g)	Elastic Base shear (without application of q factor)	Design Base Shear
Building (1)	1.5	3.75	0.1	0.1*W	0.03*W
Building (2)	7	3	0.125	0.125*W	0.04*W

Application of the Nuclear Standard

	Frequency (Hz)	Spectral acceleration (g)	SSE Design Base Shear	SSE Design Base Shear
Building (1)	1.5	0.1	0.1*W	0.13*W
Building (2)	7	0.125	0.13*W	0.41*W

4. Conclusions

The Belgian NPPs structures and components are designed for significantly higher seismic loads than the conventional counterparts. The SSE GRS defined for Nuclear installations covers significantly the conventional GRS except in the frequency range from 0.1 Hz to 0.6 Hz for the Tihange site. This is due to the fact that the site study during seismic revaluation shows a relatively stiff soil foundation non sensitive to very low frequencies. For both nuclear sites, the OBE GRS defined for comparable return period with conventional standards are similar or above the conventional spectra except in the high frequency range beyond 20 Hz.

Appendix B.

CANADA

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1. Conventional standards

1.1 Applicable standard and seismic hazard mapping

The seismic code applicable to conventional facilities in Canada is the National Building Code of Canada (NBCC 2005), (Ref. 1).

The fourth generation of seismic hazard maps for Canada (NBCC 2005) is based upon the work of Basham et al., (Ref. 2 and 3), which established the third generation of seismic hazards maps (NBCC 1995). The Cornell-McGuire methodology, (Ref. 4), is utilized but the new model includes for the first time the effects of uncertainty.

The fourth generation of seismic hazard maps is established by the Geological Survey of Canada, (Ref. 5), Figures 1 to 4.

1.2 Ground response spectra definition

In NBCC 2005, the seismic hazard is defined by the Uniform Hazard Spectra (UHS). The UHS 5% damped spectral accelerations, $S_a(T)$, are calculated for four periods (2.0 sec, 1 sec, 0.5 sec, and 0.2 sec or, in term of frequencies, 0.5 Hz, 1 Hz, 2 Hz and 5 Hz) for the median ground motion for a probability of exceedance of 2% in 50 Years (return period of 2 475 years). The accelerations are tabulated for more than 650 localities in Canada, (Ref. 5).

The design ground spectral accelerations, S(T), are obtained from the UHS spectral acceleration $S_a(T)$ after the application of acceleration or velocity based site coefficients F_V or F_a. F_V is for low and intermediate frequency range and F_a is for high frequency range. S(T) is defined using linear interpolation as follows:

S(T)	$= F_a S_a(0.2)$	for $T \leq 0.2$ s ($f \geq 5$ Hz).
	$= F_V S_a(0.5)$ or $F_a S_a(0.2)$ whichever is smaller	for $T = 0.5$ ($f = 2$ Hz).
	$= F_V S_a(1.0)$	for $T = 1.0$ s ($f = 1$ Hz).
	$= F_V S_a(2.0)$	for $T = 2.0$ s ($f = 0.5$ Hz).
	$= F_V S_a(1.0)/2$	for $T \geq 4.0$ s ($f \leq 0.25$ Hz).

For a conventional building at a site, near Toronto, Ontario, Canada, the design ground response spectral accelerations, as defined above, are shown in Figure 5. The reference soil case C (very dense soil or soft rock, 360 m/s $< V_S <$ 760 m/s is average shear wave velocity) is assumed. The coefficients F_a and F_V for this soil case are equal to 1.0.

2. Nuclear standards

2.1 Applicable standards

The codes and standards applicable to nuclear structures are the set of Standards CSA-N289 (CSA – Canadian Standard Association), (Ref. 6 to 10). (Ref. 7 and 8) provide the definition of the seismic ground motion and the standard design ground response spectra. The standard, (Ref. 7), provides two approaches to define Design Ground Response Spectra (DGRS): probabilistic and deterministic.

2.1.1 Probabilistic approach

The definition of the design seismic ground motion, (Ref. 7), is based on the spatial, temporal and parametric characterization of the earthquakes in the region. Probabilistic estimates of the risk at the site are applied using the regional seismicity in the form of zones of earthquake occurrence. These estimates take into account the magnitudes, epicentral distances and other parameters of the earthquakes producing the predominant contribution to risk as well as available strong motion recordings typical of the predominant earthquakes. The approach includes investigation of variations of expected mean ground motion parameters that may be produced in earthquake source parameters, regional attenuation proprieties and site effects as well as the possibility of significant earthquakes on active or potentially active geologic structures in the vicinity of the site.

2.1.2 Deterministic approach

To include additional margins in some situations when the probabilistic approach does not provide sufficient reliability in determining earthquake inputs, the deterministic approach can be utilized, (Ref. 7). The deterministic approach shall be used if the seismicity model and/or the attenuation relations are sufficiently uncertain and the potential for significant earthquakes in the vicinity of the site is not accounted for in the regional seismicity model.

2.2 Ground response spectra definition

According to (Ref. 7), the Design Basis Seismic Ground Motion (DBSGM) means the seismic ground motion at the site that represents the potentially severe effects of earthquakes in the region and that has sufficiently low probability of being exceeded during the lifetime of the plant that, when considered in relation to other engineering design margins, provides adequate assurance against an earthquake induced failure.

The Design Basis Earthquake (DBE) means an engineering representation of the DBSGM expressed in the form of response spectra or time-histories and employed for the seismic qualification of structures, systems and equipments.

Two Standard Ground Response Spectra, representing DBE, for soil and rock site conditions, Figure 6, are given in terms of spectral amplification factors in (Ref. 8).

The DBE Standard Ground Response Spectra for a nuclear power plant (NPP) located at the same site are anchored to 0.3 g, Figure 6. The plateau of the DBE soil spectrum is between 1.5 and 7 Hz with the spectral acceleration of 0.875 g. For the purpose of this comparison, the CSA nuclear spectra it is presented in Figure 5 along with NBCC 2005 spectra for the same site near Toronto, Ontario, Canada.

For the reference frequency of 5 Hz, the spectral acceleration of median confidence level Uniform Hazard Spectrum with the probability of exceedance of 2% in 50 years (return period of 2475 years) is 0.23 g according to NBCC 2005. On the other hand, the DBE spectral acceleration for the same frequency is 0.875 g using the nuclear standard. This spectral acceleration corresponds to the 95th percentile confidence level Uniform Hazard Spectrum with the probability of exceedance of 1% in 100 years (return period of 10 000 years), (Ref. 11).

For the reference frequency of 2 Hz, the NBCC 2005 Uniform Hazard Spectral acceleration is 0.11 g. The DBE spectral acceleration is 0.875 g (the plateau of the spectrum). This spectral acceleration corresponds to the Uniform Hazard Spectrum with the probability of exceedance of 0.5% in 100 years (return period of 20 000 years) with the confidence level higher than 95th percentile, (Ref. 11).

Thus DBE response spectrum has significantly lower probability with significantly higher confidence level than NBCC 2005 Uniform Hazard Spectrum. This reflects the additional conservatism applied in defining the seismic input to nuclear power plants (NPPs). The level of conservatism is frequency dependent and it is higher for low frequencies.

Figure 1. **Sa (0.2) for Canada (median values of 5% damped spectral acceleration for Site Class C and a probability of 2% / 50 years)**

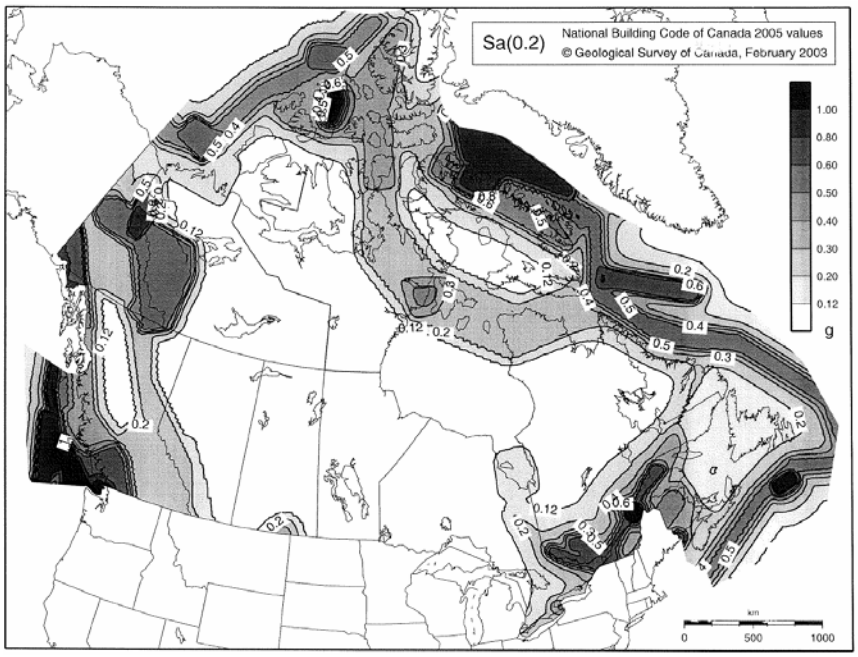


Figure 2. **Sa (0.5) for Canada (median values of 5% damped spectral acceleration for Site Class C and a probability of 2% / 50 years)**

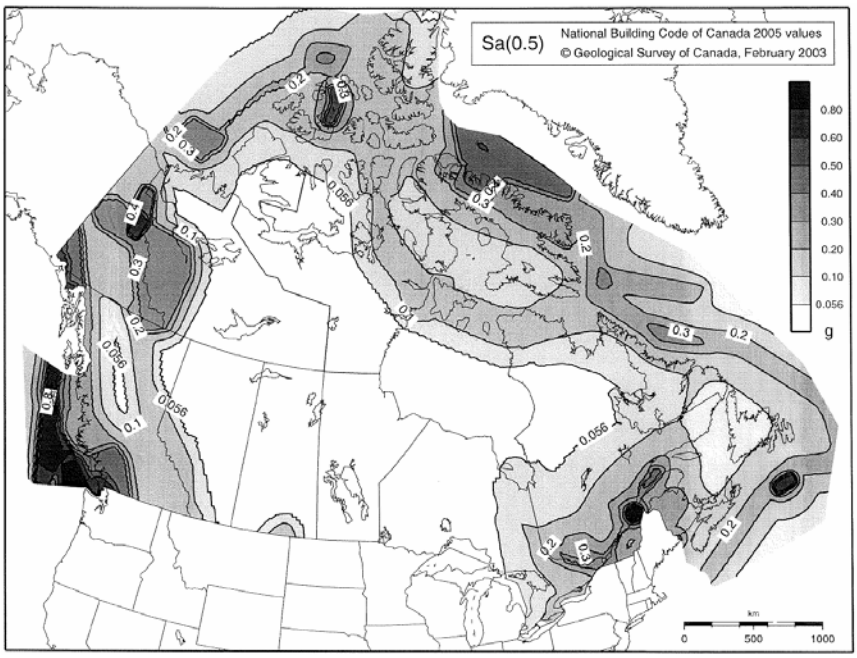


Figure 3. **Sa (1.0) for Canada (median values of 5% damped spectral acceleration for Site Class C and a probability of 2% / 50 years)**

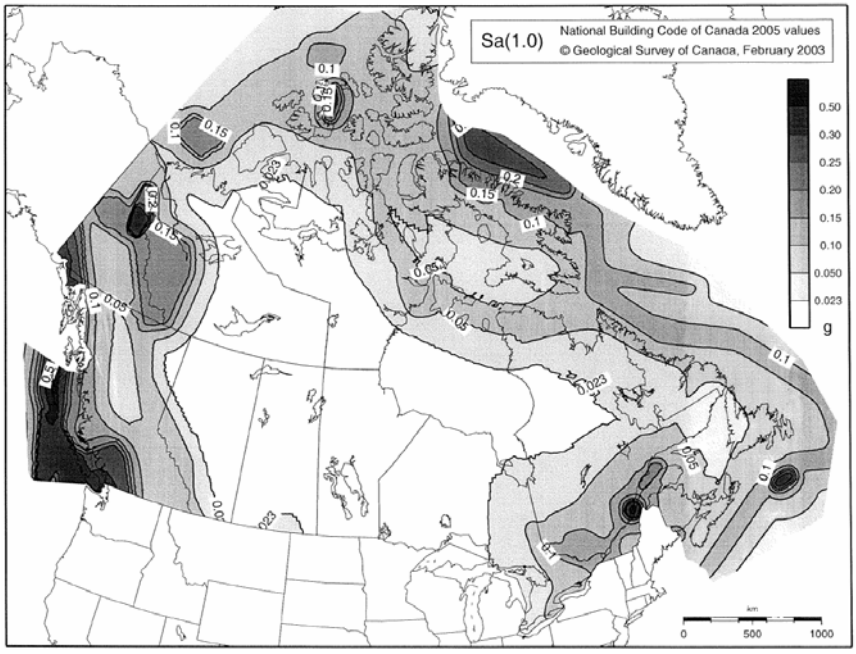


Figure 4. **Sa (2.0) for Canada (median values of 5% damped spectral acceleration for Site Class C and a probability of 2% / 50 years)**

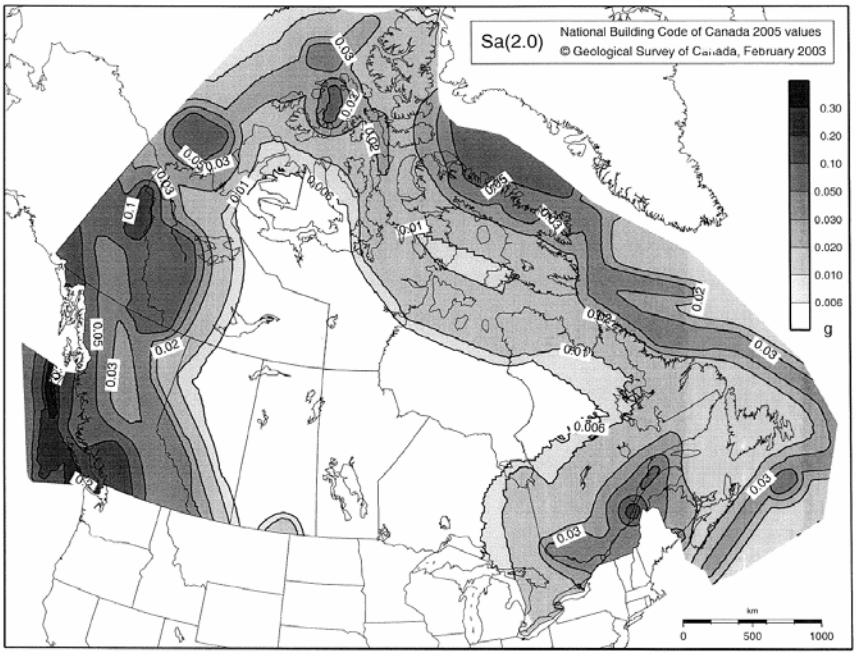


Figure 5. **Comparison of design spectra (5% damping) very dense soil site**

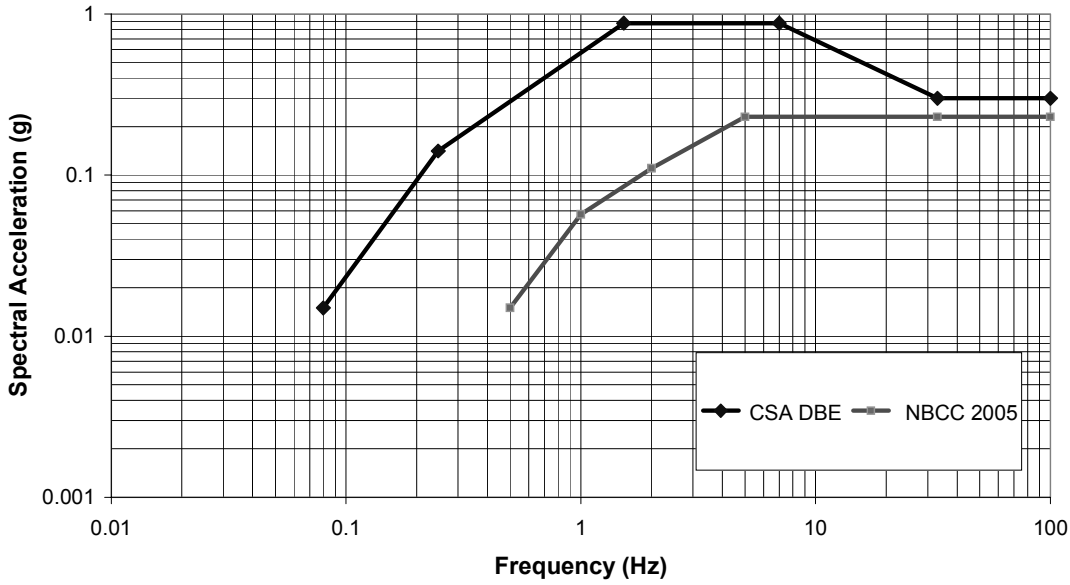
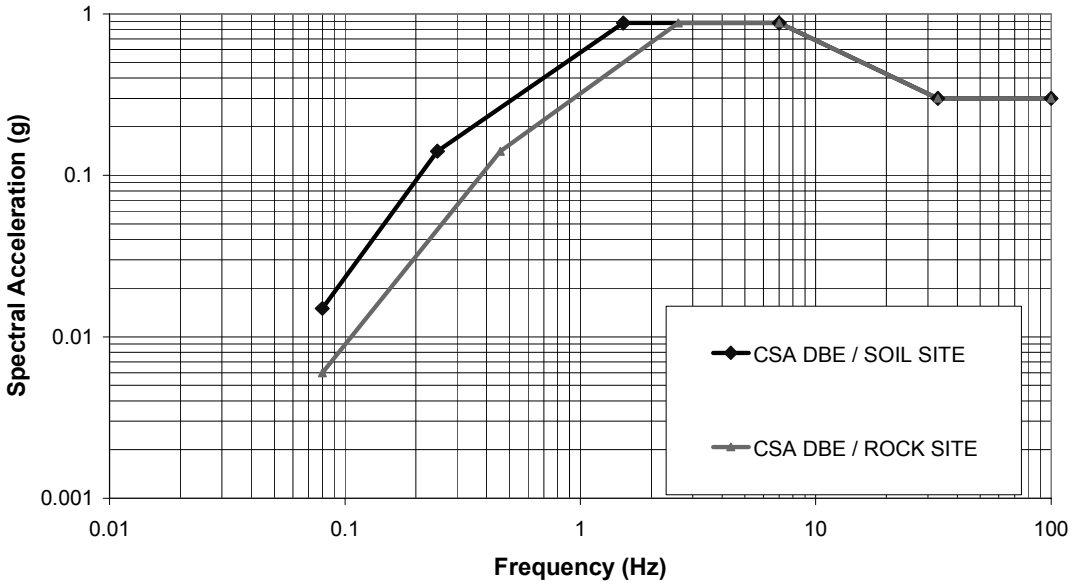


Figure 6 **CSA DBE – design spectra (5% damping) for soil and rock site**



3. Comparison of the nuclear and conventional seismic loadings – Case study

The comparison of loadings according to the nuclear and conventional seismic standards is performed for simple, regular reinforced concrete structures that can be represented by Single Degree of Freedom model with 5% of structural damping. For the purpose of this comparison, the frequency of two structures are assumed as follows:

- A concrete shear wall structure with a frequency of 5 Hz (T = 0.2 sec).
- A concrete frame structure with a frequency of 2 Hz (T = 0.5 sec).

3.1 Application of NBCC 2005 for conventional buildings

The minimum lateral earthquake force is:

$$V = S(T) I_E M_v W / (R_d R_o)$$

where :

S(T) is Design spectral acceleration as defined in Section 1.2
I_E is Importance factor for Earthquake Loads and Effects
R_d is Force Modification Factor,
R_o is System Overstrength Factor
M_v is Higher Mode Factor
W is the weight of the building

If the frequency of the structure is less than 0.5 Hz (T=2.0 sec), V shall not be taken less than $V = S(2.0) I_E M_v W / (R_d R_o)$.

For the Seismic Forces Resisting Systems with an R_d equal to or greater than 1.5, V need not to be taken greater than $V = \frac{2}{3} S(0.2) I_E M_v W / (R_d R_o)$.

In this comparison, one importance factor (Post Disaster Facility, Importance Factor for Earthquake Loads equal to 1.5) and two ductility levels (Moderate Ductility and Ductile Structure) are used.

The concrete shear wall structure

Structure 1	I_E	Moderate ductility		Ductile structure		Higher Mode Factor
		R_d	R_o	R_d	R_o	M_v
Post disaster	1.5	2.0	1.4	3.5	1.6	1.0

The concrete frame structure

Structure 2	I_E	Moderate ductility		Ductile structure		Higher Mode Factor
		R_d	R_o	R_d	R_o	M_v
Post disaster	1.5	2.5	1.4	4.0	1.7	1.0

Application of NBCC 2005 for Conventional Buildings

	Assumed Frequency (Hz)	$M_v I_E/(R_d R_o)$	Spectral Acceleration (g)	NBCC 2005 Design Lateral Force	Max NBCC 2005 $V_{max} = \frac{2}{3} S(0.2) I_E M_v W / (R_d R_o)$
Structure 1	5		0.23		
Mod.Duct.		0.535		0.123W	0.082W
Ductile		0.268		0.062W	0.041W
Structure 2	2		0.11		
Mod.Duct.		0.429		0.047W	0.066W
Ductile		0.221		0.024W	0.034W

3.2 Application of CAN3-N289.3 for Nuclear Power Plants

	Assumed Frequency (Hz)	Spectral Acceleration (g)	DBE Design Lateral Force
Structure 1	5	0.875	0.875W
Structure 2	2	0.875	0.875W

4. Conclusions

The comparison of seismic input motion and seismic induced loadings for conventional structures and nuclear power plants is presented for a given site near Toronto, Ontario, Canada.

Two Seismic Force Resisting Systems are considered: concrete shear wall (Structure 1) and concrete frame structure with reinforced masonry infill (Structure 2), each of them for two ductility levels (Moderate ductility and Ductile). Both of them are considered as Post Disaster Facilities with the Importance factor for Earthquake Loads and Effects equal to 1.5.

The maximum design spectral accelerations, in the present case study, for the conventional structures are at 3.8 times lower than for nuclear ones. The conventional spectrum is enveloped by the nuclear in each point. The Uniform Hazard Spectrum accelerations are with a probability of exceedance of 2% in 50 years (return period of 2475 years) and a median confidence level. The probability of exceedance and the confidence level of the DBE nuclear Standards Ground Design Spectrum are different for each frequency. The DBE nuclear Standard Ground Response Spectrum has significantly lower probability with significantly higher confidence level than NBCC 2005 Uniform Hazard Spectrum. The level of conservatism is frequency dependent and it is higher for low frequencies structure (i.e. 2 Hz) than for high frequency structures (i.e. 5 Hz).

Taking into account Force Modification and Over strength Factors and the Importance Factor for Earthquake Loadings, applicable to conventional buildings, the seismic induced loadings are:

- 11 to 21 times higher for the nuclear structure than for the conventional concrete shear wall structure (Structure 1 – with a frequency of 5 Hz), depending on the level of ductility.
- 19 to 36 times higher for the nuclear structure than for the conventional concrete frame structure with reinforced masonry infill (Structure 2 – with a frequency of 2 Hz), depending on the level of ductility.

As a general conclusion, a NPP, for a given site near Toronto, is designed for significantly higher seismic loadings than conventional buildings designed according to the National Building Code of Canada 2005.

It should be noted however, that for nuclear buildings, the structure is expected to behave elastically up to the DBE level, i.e. no credit is taken for ductile behaviour, while for conventional buildings, full credit can be taken for ductility thus allowing some acceptable level of damage to the structure.

5. References

1. National Building Code of Canada – NBCC 2005.
2. Basham, P.W.,Weichert, D.H., Angelin, F.M., and Berry, M.J., 1982 New Probabilistic Strong Seismic Ground Motion Maps of Canada: a Compilation of Earthquake Source Zones, Methods and Results. Earth Physics Branch Open File 82-83, 202 pp.
3. Basham, P.W.,Weichert, D.H., Angelin, F.M., and Berry, M.J., 1985 New Probabilistic Strong Seismic Ground Motion Maps of Canada. Bull. Seism. Soc. Am., v.75, p.563-595.
4. McGuire, R.K., 1993 Computations of Seismic Hazard In Giardini, D., and Basham, P.W. (Eds) Global Seismic Hazard Assessment Program, Annali di Geofisica.
5. Geological Survey of Canada, Open File 4459, John Adams and Stephen Halchuk, 2003.
6. CAN3-N289.1-M80 General Requirements for Seismic Qualification of CANDU Nuclear Power Plants.
7. CAN3-N289.2-M81 Ground Motion Determination for Seismic Qualification of CANDU Nuclear Power Plants.
8. CAN3-N289.3-M81 Design Procedures for Seismic Qualification of CANDU Nuclear Power Plants.
9. CAN3-N289.4-M86 Testing Procedures for Seismic Qualification of CANDU Nuclear Power Plants.
10. CAN3-N289.5-M91 Seismic Instrumentation Requirements for Seismic Qualification of CANDU Nuclear Power Plants.
11. Seismic Hazard Analysis Results and Sensitivity Seismic Hazard in Southern Ontario, Geomatrix, August 1997.

Appendix C.

CZECH REPUBLIC

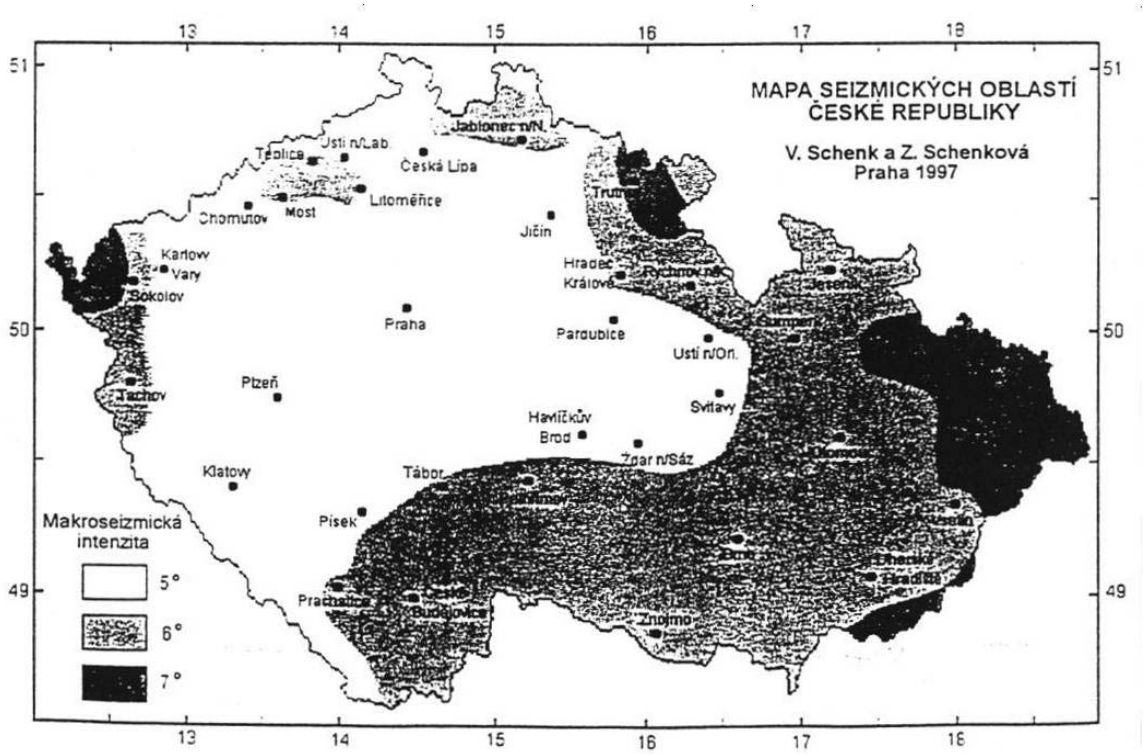
Table of contents

1. Conventional standards

1.1 Applicable standard and seismic hazard mapping

In Czech Republic is in force standard ČSN 73 0036 "Seismic Loads of Buildings", Revision 2 (May 1998). It is very old standard, Revision 0 was issued in 1973. The intensity of earthquake is classified according Medvedev – Sponhauer – Kárník scale 1984. Acronym MSK 84 is introduced. The Eurocode 8 – ENN 1988-1-1 replace this standard in the next future. At present is in the phase of adoption. Zones of maximum expected macroseismic intensity are illustrated in Fig. 1. Note that exist together 6 zones with macroseismic intensity higher than 6° MSK 84.

Figure 1. **Map of seismic zones**



Some information about registered earthquakes:

1) 6° 1902.
2) Seismic swarms 1897, 1900, 1901, 1903, 1904, 1908, 1911, 1936, 1962, 1985-1986, 1991; 7° only in 1908 and 1985.
3) 7° 1901.
4) 7° ÷ 8° 1786.
5) ≤ 6°, 1590 and 1876.
6) ≤ 6°, epicentre in Austria and North Italian Alpen.

1.2 Ground response

The ČSN 73 0036 code is based on the static equivalent method. The resulting seismic force is given by the equation

$$F_k = K_c\, m_k\, g \quad (1)$$

where K_c is the resulting seismic coefficient. The equation for the K_c takes the form

$$K_c = K_z\, \delta\, \eta_k\, \psi \quad (2)$$

where *Kz* represent the importance of a building, level of MSK 84 a soil characteristic
δdynamic coefficient

$$\delta = \frac{0.75}{T_0}$$

$T_0 < 0.33\ s \Rightarrow \delta = 2.25;\ T_0 > 1.25\ s \Rightarrow \delta = 0.6$

T_0 period of the building

η_k coefficient of the mode shape in the point k

The following equation is valid

$$\eta_k = \frac{u_k \sum_{i=1}^{n} m_i\, g\, u_i}{\sum_{i=1}^{n} m_i\, g\, u_i^2} \quad (3)$$

ψ damping of the building. The following values are introduced

= 1 for buildings with very high damping

= 1.5 for building with very low damping

2. Nuclear standards

2.1 Applicable standards

The applicable codes and standards are as follows:

- US NRC Regulatory Guides.
- Standard Review Plan, Section 3.7.1 ÷ 3.7.4, 3.10.
- ČSN/IEC 980 Recommended Practices for Seismic Qualification of Electrical Equipment of the Safety System for Nuclear Generating Stations.
- NS-G-1.6: Seismic Design and Qualification for Nuclear Power Plants.
- NS-G-3.3: Evaluation of Seismic Hazard for NPPs.

2.2 Ground response spectra definition

They are two operated NPPs in Czech Republic

i. NPP Dukovany 4 × 440 MW VVER, Model 213.
ii. NPP Temelin, 2 × 1000 MW VVER, Model 320.

ad i)

This NPP is located in South Moravia (6th zone) and is operated since 1985 (unit 1), 1986 (units 2 and 3) and 1987 (unit 4). Originally was constructed as non-seismic. The seismic upgrading was started in 1996 and three artificially accelerogram with PGA = 0.1 g have been used as recommended by the IAEA 50-SG-D15 Guide. The SMA methodology has been applied.

ad ii)

This NPP is located in South Bohemia (zone 5) and is operated since 2004. The PGA level of the SSE has been set to 0.1 g as recommended by the IAEA 50-SG-D15 Guide. For the generation of free-field response spectrum the following accelerograms from fifth past earthquakes have been used, see next Table 1.

Table 1. **Selected accelerograms for NPP Temelin after normalisation to PGA = 0.1 g**

Symbol	Original accelerogram	Components	a_{max} [g]
A	Carigliano 23.11.1980 18:34:54	H1 H2 V	0.1 0.1 0.07
B	San Severo 23.11.1980 18:34:54	H1 H2 V	0.1 0.1 0.07
C	USA, West region 04.09.1955	H1 H2 V	0.1 0.106 0.044
D	USA, West region 22.03.1957	H1 H2 V	0.082 0.103 0.037
E	USA, West region 22.03.1957	H1 H2 V	0.084 0.055 0.044

For each accelerogram has been generated free-field ground spectrum. Resulting response spectrum has been calculated using “median + sigma” approach, i.e. the non-exceeding probability is 84%. Results obtained are illustrated in Figures 2 and 3.

Figure 2. **SSE free-field ground spectrum in vertical direction**

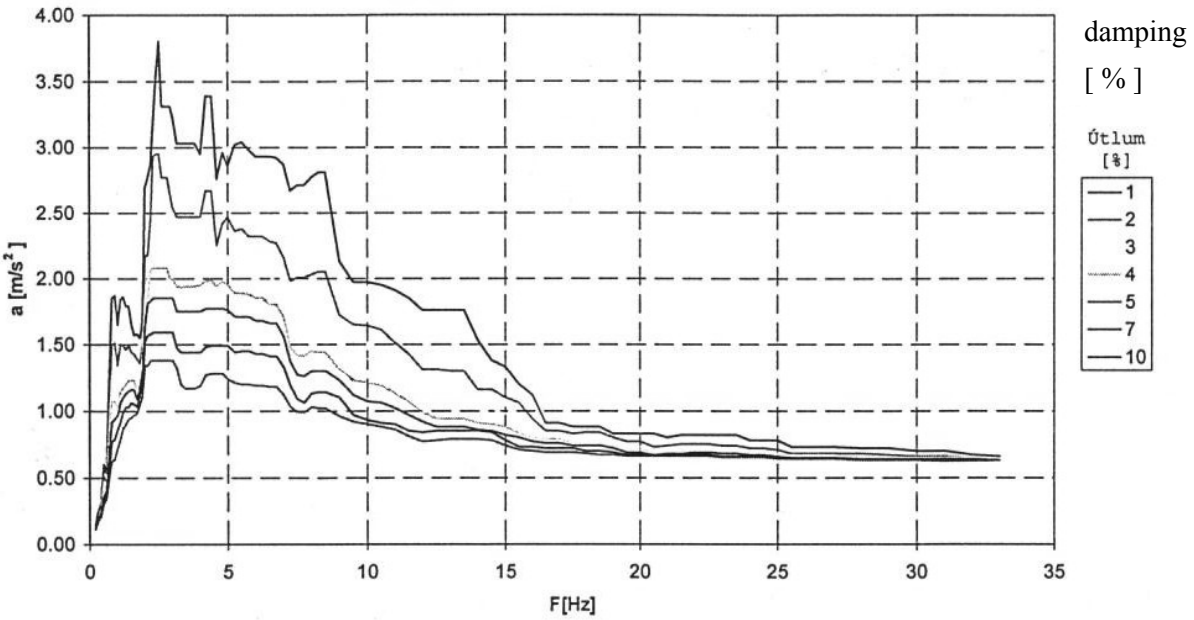


Figure. **3: SSE free-field ground spectrum in horizontal direction**

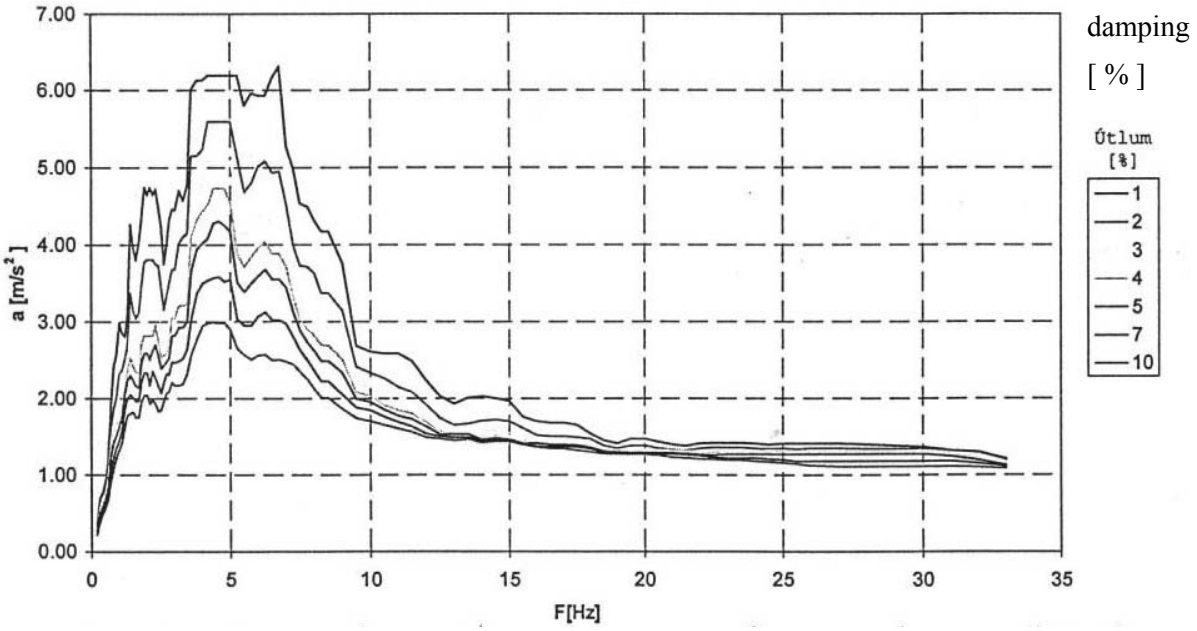
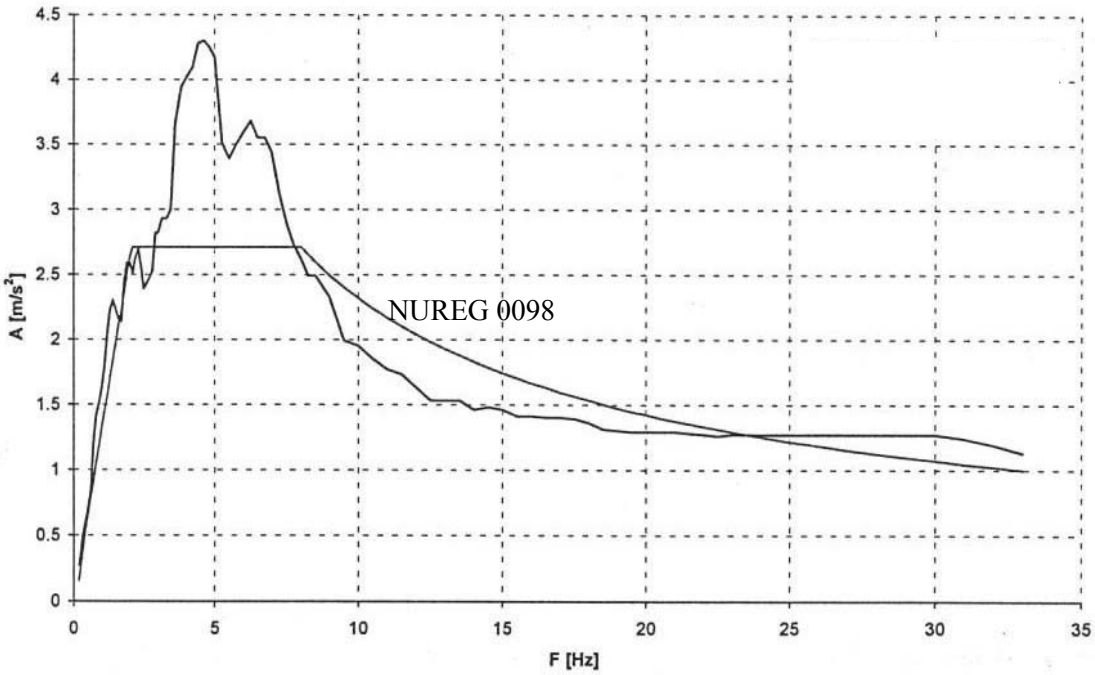


Figure 4 compares the Temelin SSE free-field ground spectrum in horizontal direction with standard spectrum according NUREG 0098. The 5% damping is used. The certain degree of conservatism is evident.

Figure 4. **Comparison of NUREG 0098 and Temelin free-field ground spectra**



3. Comparison of the nuclear and conventional seismic loadings – Case study

Since in Czech Republic the static equivalent method is used for conventional buildings, the following approach will be used:

- Two current fictive types of buildings identical with Belgian case study.
- A single degree of freedom system, length of the cantilever is 12 m, attached mass m = 10^3 kg.
- Rock soil,
- Behaviour factor is not introduced (is not included in CSN 70 0036).

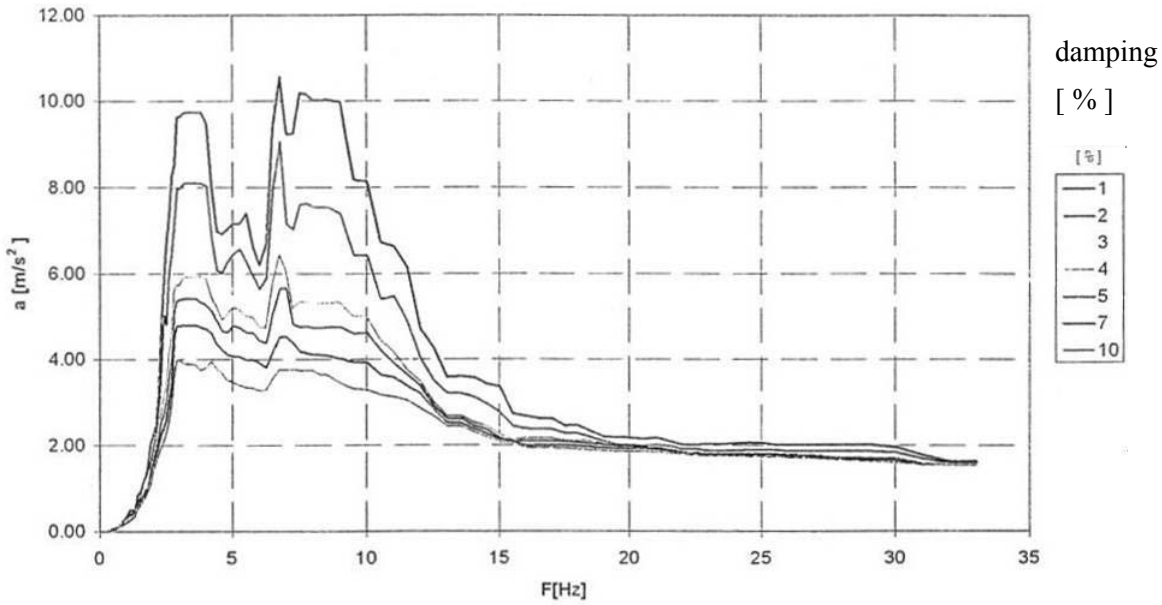
Application of CSN 73 0036 (see Eq. (2)), PGA = 0.1 g

	Assumed frequency [Hz]	Coefficient			Damping factor ψ	Resulting seismic coeff K	Seismic force F [kN]
		Kz	δ	ηk			
Building (1)	1.5	0.03	1.125	1	1	0.03375	337.5
Building (2)	7	0.03	2.25	1	1	0.0675	675

Application of Nuclear Standard (site Temelin), PGA = 0.1 g

	Assumed frequency [Hz]	Spectral acceleration (see Fig. 4)	Seismic force [N]
Building (1)	1.5	0.5	500
Building (2)	7	4.2	4200

Figure 5. **Temelin floor response spectrum in horizontal direction, floor level + 12m**



4. Conclusions

The macroseismic intensity in Czech Republic is very low, expected level is 6° MSK 64. For both operating NPPs the PGA level has been chosen 0.1 g in accordance with IAEA Safety Guide NS-G-1.6. Free-field ground acceleration is determined either using NUREG 0089 or World Bank accelerograms related to soil conditions. We suppose that Eurocode 8 will be in force during 2007.

Appendix D.

GERMANY

Table of contents

1. Conventional standards

1.1 Applicable standard and seismic hazard mapping

Since 1957 the seismic design of conventional structures in Germany is based on the industrial standard DIN 4149. There have been several amendments of this standard converging towards Eurocode 8. The latest version was issued in April 2005 (DIN 4149 - 04/2005) (Ref. 1).

The application of the code is limited to buildings and structures in German earthquake prone areas (zones 0-3). The map of the seismic zones given in the new code (Figure 1) is derived from a probabilistic methodology based on three assumptions:

- Exponential distribution of the magnitude of seismic events (Gutenberg-Richter).
- Return periods following Poisson distribution.
- Uniform distribution of the seismicity inside the zones.

Figure 2 shows the deep geology classes (R = rock, T = transition, S = sediment basin) within the areas belonging to the seismic zones.

1.2 Ground response spectra definition

The seismic hazard of each seismic zone (0-3) is defined by a design response spectrum, which is generated for each zone (defined range of intensities) with constant design ground acceleration (a_g) for the reference spectrum period (T_R) of 475 years.

According to DIN 4149 the calculated value of ground acceleration is given by $a_g \cdot \gamma_I \cdot S$, where γ_I(0.8-1.4) is the importance category (I-IV) of the construction and S (0.75-1.5) is the soil factor.

The value of the soil factor depends on the combination of the deep geology class with one of the three subsoil classes (A, B, and C). Possible combinations are: A-R, B-R, C-R, B-T, C-T, and C-S.

The soil factor S is valid for the whole frequency range of the spectrum. There are two major effects on the spectrum: (i) scaling of the calibration value ($a_g \cdot S$) for zero period acceleration, i.e. the site specific acceleration, and (ii) compensation of particularities in the shape of the spectrum (spectral amplification factor). The spectral amplification value as reference value is $\beta_0 = 2.5$ for 5% viscose damping.

Figure.1. **Zoning map of German earthquake regions according to the amended DIN 4149**

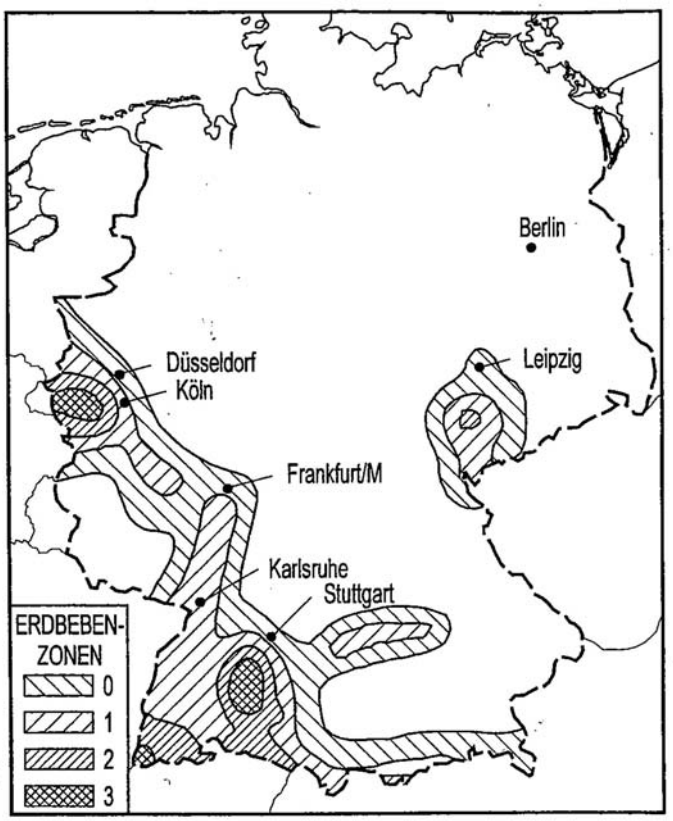
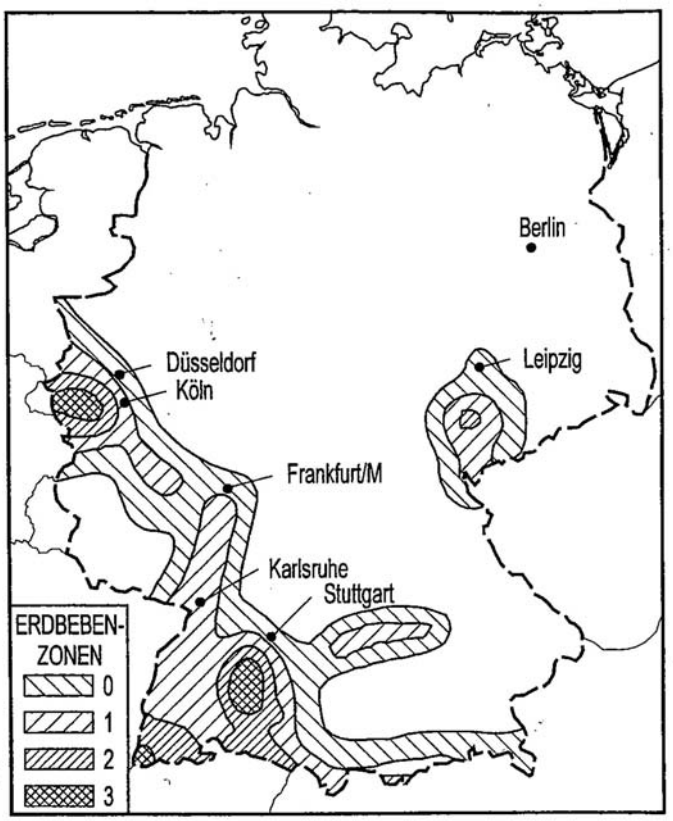


Figure 2. **Deep geology classes in the German seismic zones according to the amended DIN 4149**



2. Nuclear standards

2.1 Applicable standards and seismic hazard map

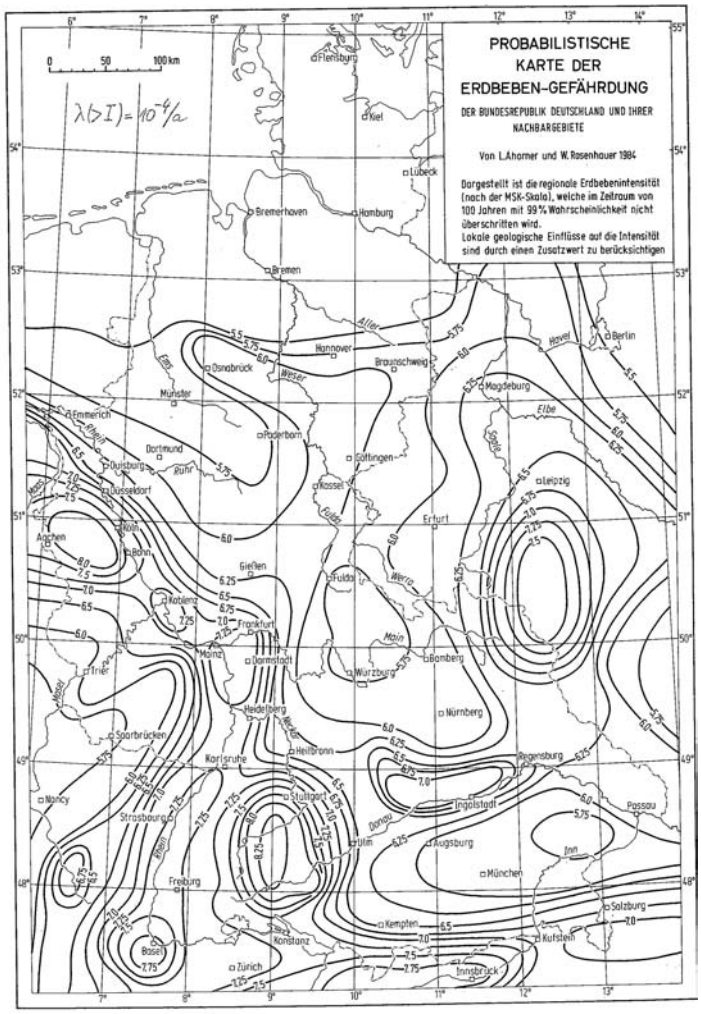
The applicable code for nuclear installations in Germany is the safety standard KTA 2201 "Design of Nuclear Power Plants against Seismic Events", which consists of six parts. For the issue discussed here Part 1 (Ref. 2), Part 2 (Ref. 3), and Part 3 (Ref. 4) are of particular interest.

At present, an amendment of Part 1 is under way due to recommendations of the Reactor Safety Commission (RSK) (Ref. 5). In view of the amendment of the conventional code, some modifications might also be necessary in KTA 2201 Part 3.

KTA 2201 Part 1 requires the seismic hazard to be evaluated (mainly deterministically) for each site individually taking into account the most severe earthquake that might occur in the surroundings of the site (radius r = 200km) in accordance with scientific knowledge.

Based on the recommendations of the RSK and the long standing seismological practice the amended version of KTA 2201 Part 1 will (probably) also require a probabilistic assessment of the seismic hazard. Typical exceedance probabilities for earthquake intensities are of the order 10^{-4} - 10^{-5}/a. As an example for the definition of seismic hazard for NPP Figure.3 shows isolines of earthquake intensity for an exceedance probability of 10^{-4}/a.

Figure.3. **Isolines of German earthquake intensities with a return period of 10000 years (Ref. 6)**



2.2 Ground response spectra definition

In general, site specific free-field response spectra are used.

As mentioned before, the concept given in the current KTA standard is essentially deterministic. But in its amended version it will incorporate some probabilistic aspects in accordance with the recommendations of the RSK and the current engineering practice (estimation of ground response spectra (Ref. 7),). It is, e.g., intended to associate the percentile values of the free-field response spectra and the exceedance probabilities for the design basis earthquake: 84% <=> 10^{-4}/a and 50% <=> 10^{-5}/a.

The free-field response spectra are derived on a statistical basis from the registered seismograms which are associated with the earthquake intensity classes and subsoil classes corresponding to the site. These spectra are smoothed according engineering judgement and finally converted into ground design spectra using appropriate fractile values.

3. Comparison of the nuclear and conventional seismic loadings - Case study

The following comparison has a threefold aim:

1. It will show that the ground response spectrum estimated according to nuclear standards results in higher requirements (acceleration) compared to the conventional approach.
2. The differences between spectra (for the same range of intensities I = 6.5 - 7.0 (MSK)) obtained according to nuclear and conventional standards will be revealed.
3. The agreement between the 84% spectrum with exceedance probability 10^{-4}/a and the 50% spectrum with exceedance probability 10^{-5}/a (used in the nuclear field) will be shown.

All ground response spectra have been derived as horizontal elastic spectra with 5% damping.

For the response spectrum according to DIN 4149 (labelled (1) in Figure.4.) the following parameters were chosen:

- $a_g = 0.4 m/s^2$ for seismic zone 1 ($6.5 \leq I(MSK) < 7.0$), exceedance probability $2.1 \cdot 10^{-3}$/a
- soil factor S=0.75 for the subsoil combination C-S, with T_A=0.04s (25Hz)T_B = 0.1 s (10 Hz), T_C = 0.5 s (2.0 Hz), T_D = 2.0 s (0.5 Hz)
- $\gamma_I = 1.4$ as the highest importance category (IV) of the construction
- $\beta_0 = 2.5$ as reference value for spectral amplification

For nuclear structures the smoothed realistic response spectra (based on statistical evaluations) for the range of intensities I = 6.5 - 7.0 (MSK) are adopted after Hosser (Ref. 7). These spectra denoted (2) to (5) in Figure.4, have the following characteristics:

- (2) 84% spectrum for I = 6.5 (MSK)
- (3) 84% spectrum for I = 7.0 (MSK), exceedance probability $2.5 \cdot 10^{-4}$/a
- (4) 50% spectrum for I = 7.75 (MSK), exceedance probability $2.0 \cdot 10^{-5}$/a

These three spectra are horizontal component spectra.

- (5) 50% spectrum for I = 7.75 (MSK), exceedance probability $2.0 \cdot 10^{-5}$/a, horizontal resultant spectrum

In view of the aims mentioned above we come to the following conclusions:

1. The spectrum used for nuclear design (3) results in significantly higher accelerations in the whole frequency range than the one for conventional buildings (1).

2. For the same range of intensities (I = 6.5 - 7.0 (MSK)) the nuclear spectra (2) and (3) yield higher acceleration values than the conventional spectrum (1) except for the frequency range from 0.5Hz to 0.8Hz where spectrum (2) is marginally lower than spectrum (1) at the considered site.
3. At the site under consideration, the SSE (DBE) spectra (for nuclear facilities), i.e. the 84% (3) and 50% (4) spectra, show some differences. For the frequency range ≥2.5Hz the 84% spectrum (exceedance probability 2.5·10-4/a) yields somewhat higher accelerations than the 50% spectrum (exceedance probability 2.0·10-5/a). This difference might become important for structures and mechanical components (Ref. 8) on higher floors due to building filtering effects. It should be mentioned that in general 84% spectra were used for the design of German NPP.

3.1 Description of the response spectra (Fig. 4)

1st consideration

The component spectrum (4) and the resultant spectrum (5) are 50% percentiles with exceedance probability $2\cdot10^{-5}$/a and I=7.75 (MSK). They are horizontal SSE (DBE) spectra used for nuclear structures. Spectrum (1) is a horizontal spectrum with a exceedance probability of $2.1\cdot10^{-3}$/a derived according to conventional standards. In both cases the considered site is the same.

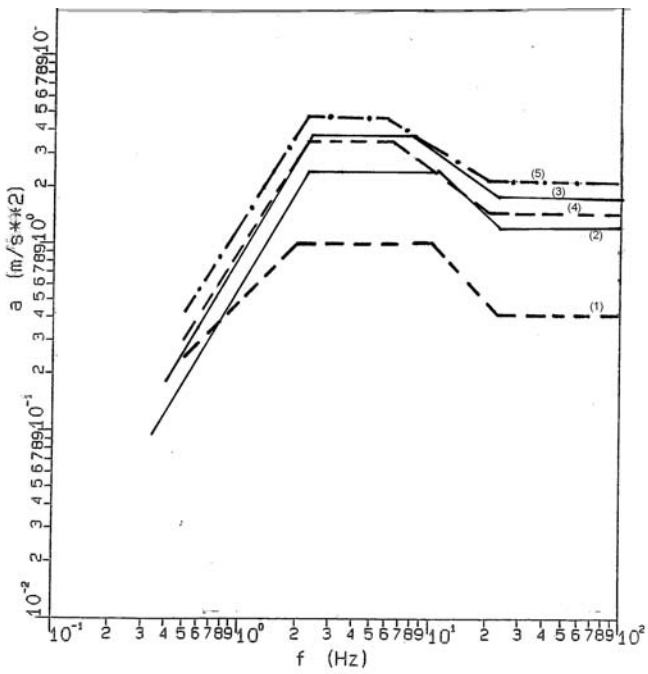
2nd consideration

The 84% spectra (2) and (3), applicable for nuclear design at intensities I=6.5 (MSK) and I=7.0 (MSK; exceedance probability $2.5\cdot10^{-4}$/a), show a big difference in comparison to spectrum (1) which is applicable for conventional structures (same intensity range 6.5≤I (MSK) < 7.0).

Conclusion

Both considerations reveals a great degree of conservatism of the spectra which are used for nuclear structures in comparison to those used for conventional structures.

Figure 4. **Comparison of the elastic ground response spectra for nuclear and conventional structures at an exemplary site in Germany**



4. Conclusions

Site specific SSE (DBE) ground response spectra obtained according to the German nuclear standards (KTA 2201, Part 1-3) result in higher requirements (with respect to ground acceleration) than spectra obtained according to the conventional seismic standard DIN 4149 for the same site.

5. References

1. DIN 4149, Bauten in deutschen Erdbebengebieten - Lastannahmen, Bemessung und Ausführung üblicher Hochbauten, April 2005.
2. KTA 2201.1, Design of Nuclear Power Plants against Seismic Events, Part 1: Principles, June 1990.
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7. Hosser D., Gutachterliche Stellungnahme G88069, Realistische seismische Lastannahmen für die Bauwerke des Kernkraftwerks KWB, Block A, January 1989 (revised April 1989).
8. KTA 2201.4, Design of Nuclear Power Plants against Seismic Events, Part 4: Requirements for Procedures for Verifying the Safety of Mechanical and Electrical Components against Earthquakes, June 1990.

Appendix E.

JAPAN

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Japan is an earthquake prone country; therefore there have been many standards for seismic design of industrial installations and systems since in the early time of the 20th Century.

The a seismic design of nuclear power plant in Japan had started when the introduction of the first nuclear power plant in 1958. The current "Examination Guide for a seismic Design of Nuclear Power Reactor Facilities" was established in 1978 (it is partly revised in 1981).

Table 1 shows a comparison of current a seismic design criteria between generic building structures and nuclear power plant installations". As it can be seen in this table, safety related important nuclear power plant building and structures has been given excellent seismic performances as compared with the generic buildings.

Through the experiences of big earthquakes including the 1995 Hyogo-ken Nanbu Earthquake (the Kobe Earthquake) in Japan, the latest knowledge related to earthquakes and earthquake ground motions has been accumulated. By taking into account these advanced technologies, experiences and knowledge, it is commonly recognized by Japanese experts on seismology and seismic engineering that some upgrading of the “Examination Guideline” is necessary to enhance the reliability of seismic safety of nuclear power plants.

Standing on this background, a taskforce for performing works to upgrade the “Examination Guideline” has been being organized in Nuclear Safety Commission of Japan. In this upgrading, key points that are currently discussing are as follows:

1. The fundamental way of thinking about the safety evaluation of the seismic design.
2. Classification of safety importance in the seismic design.
3. Earthquake ground motion for use in the evaluation of the seismic safety.
4. Evaluation of the seismic safety of the facilities.

However, because some strong earthquake ground motions struck some of nuclear power plant sites recently, discussion is heating therefore it might be difficult to converge the discussions in a short time.

Also after the 1995 Hyogo-ken Nanbu Earthquake, following typical seismic design standards for conventional structures and industrial facilities are also upgraded:

1. Specifications for highway bridges 2002, Part 5 Seismic Design, (in Japanese), 2002.
2. Seismic design code for railway structures, (in Japanese), Railway Technical Research Institute, 1999.
3. Technical Criteria for River Works Design Version (I), (II), 1997.
4. Guideline for the Seismic Design of High-pressure Gas Facilities, 1998 and 2000.
5. Technical Standards for Port and Harbor Facilities in Japan, 1999.
6. Guideline of Countermeasures against Earthquake for Water Facility, 1997.
7. Guideline of Countermeasures against Earthquake for Sewerage Facility, 1997.
8. Comprehensive Countermeasures against Earthquake for Government Facilities, 1996.
9. Guideline for Structural Design of School Buildings, 1996.
10. Technical Standard for Structural Calculation of Response and Limit Strength of Buildings, 2000.

In this document, three typical cases of the definition of seismic hazard out of the standard described above including the examination guide for a seismic design of nuclear power reactor facilities, currently under the discussion for upgrading, are introduced for comparison purposes. These are:

a. Technical Standard for Structural Calculation of Response and Limit Strength of Buildings, 2000.
b. Specifications for highway bridges 2002, Part 5 Seismic Design, 2002.
c. Guideline for the Seismic Design of High-pressure Gas Facilities, 1997.
d. Examination Guide for a seismic Design of Nuclear Power Reactor Facilities, (currently under discussion).

Table 1. **Comparison of a seismic design criteria between generic building structures and nuclear power plant Installations**

	Generic Structure	High Rise Building (Over 60 meters)	Nuclear Reactor Building
Structural Characteristics	The first natural periods for intermediate and low rise building: RC: 0.02*H sec SC: 0.03*H sec Wood, 2F: 0.3 sec H: height of building.	Soft structure The first natural periods: 0.06-0.1(*N) sec N: Number of stories	Building and structure should be rigid. Natural period of reactor building is about 0.15 sec (However, soil structure interaction system is about 0.2 sec.).
Installation Soil	Mainly soil surface, Alluvial deposit	Direct support or pile support to diluvial formation	Safety important building and structure should be supported on bedrock.
Seismic Importance Classification			Safety related installations are categorized as, A, B, and C, taking into account their importance.
Supposed Earthquake			Based on the study on the past earthquakes and active faults, the earthquake which affect the site mostly is supposed. As Class: Extreme design earthquake. A Class: Maximum design earthquake
Design Earthquake Ground Motion	Static (it is supposed that seismic intensity IV-V for the primary design, and VI for the secondary design)	Strong motion data, e.g., El-Centro, Taft, Hachinohe, and Tokyo-101 are applied	As Class: Extreme design earthquake ground motion. A Class: Maximum design earthquake ground motion.
Horizontal Seismic Force	Primary design (strength design): Static analysis using base shear coef. of 0.2. Secondary design (confirmation of holding capacity): Static analysis using base shear coef. of 1.0.	Seismic force evaluated by dynamic earthquake response analysis, Level 1: Maximum velocity amplitude of 25/cm. Level 2: Maximum velocity amplitude of 50/cm.	A Class: The larger force either dynamic seismic force evaluate by the analysis using S1 earthquake ground motion and three times of the primary design force for generic building. As Class: Dynamic seismic force evaluate by the analysis using S2 earthquake ground motion.
Vertical Seismic Force	A half of horizontal road is taken into account for highway pier and bridge of Shinkansen line.		A half of constant horizontal force is taken into account and applied simultaneously with horizontal force in the disadvantaged direction.

Table 1. **Comparison of a seismic design criteria between generic building structures and nuclear power plant Installations** (Cont'd)

	Generic Structure	High Rise Building (Over 60 meters)	Nuclear Reactor Building
Acceptance Condition	Primary design: Within elastic response region. Secondary design: Though partial failure occurred, never to be collapsed.	Level 1: Within elastic response region (story deformation angle of 1/200). Level 2: Non-linear response region (story deformation angle of 1/100).	For S1 Earthquake: within elastic response region. For S2 Earthquake: elasto-plastic response having enough margins for deformation capacity and having proper safety margins; equipment and piping doesn't have excessive deformation, cracks and failure.

1. Newly introduced performance-based seismic design provisions to the building standard law (partially quoted form reference. 1)

After the 1995 Hyogoken-nanbu Earthquake (EQ), based on many lessons learnt about EQ preparedness, disaster response, seismic design, upgrading of existing buildings and introduction of new technologies, the seismic provisions of the Building Standard Law of Japan (BSLJ) were significantly revised in 2000 from existing prescriptive into performance-based type to enlarge the alternatives of structural design, particularly for the application of newly developed materials, structural elements, structural systems and construction.

In the revised provisions, the precise definitions for structural performance requirements and verification procedures are specified on the basis of definite response and limit values. The provisions should be applicable to any kind of materials and any type of buildings on condition that the material properties are clearly defined and the behaviour of a building structure is properly predicted.

The basic concept for seismic design spectra of EQ motions in the verification procedures is:

1. The basic design spectra defined at the engineering bedrock.
2. The evaluation of site response from geotechnical data of surface soil layers. The verification procedures apply the equivalent linearization technique using an equivalent single-degree-of-freedom system and the response spectrum analysis.

1.1 Requirement for seismic performance

The new procedures deal with the evaluation and verification of structural performance at a set of limit states under dead, live and snow loads, and wind and earthquake forces. Two limit states need to be considered for building structures to protect the life and property of the occupants against earthquake motions; life safety and damage limitation. For this purposes, two sets of earthquake motions are considered; maximum earthquake motions to be considered and once-in-a-lifetime earthquake motions, each having different probability of occurrence. The effects of the design earthquake motions were kept at the same levels as the design seismic forces in the previous provisions.

The level of maximum earthquake motions to be considered corresponds to the category of requirement for life safety and is assumed to produce the maximum possible effects on the structural safety of a building to be constructed at a site. This earthquake motion level corresponds approximately to that of the highest

earthquake forces used in the conventional seismic design practice, representing the horizontal earthquake forces induced in buildings in case of major seismic events.

The level of once-in-a-lifetime events corresponds to the category of requirement for damage limitation of a building and is assumed to be experienced at least once during the lifetime of the building. This earthquake motion level corresponds approximately to the middle level earthquake forces used in the conventional seismic design practice, representing the horizontal earthquake forces induced in buildings in case of moderate earthquakes.

It is difficult to apply the design seismic forces to new structural systems and construction such as seismic isolation and structural-control buildings, and to take into account the seismic behaviour of surface soil deposits. Therefore it was concluded that the seismic design should start with defining the input earthquake ground motions. This coincides with the framework of the performance-based structural engineering aiming at the flexible design. The new seismic design procedures including the design earthquake response spectrum have been introduced to BSLJ.

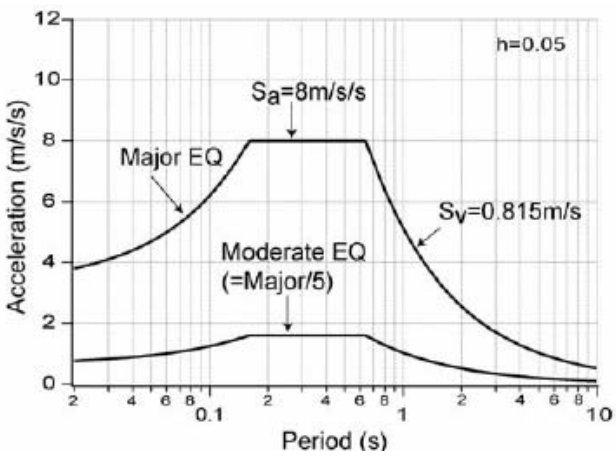
1.2 Design response spectrum at engineering bedrock

The EQ ground motion used for the seismic design at the life-safety limit state is the site specific motion of an extremely rare earthquake that is expected to occur once in approximately 500 years. A soil layer whose shear wave velocity is not less than about 400 m/s is assumed to be the engineering bedrock. The basic design EQ acceleration response spectrum, S0, of the seismic ground motion at the exposed (outcrop) engineering bedrock is shown in Figure. 1 and given in Eq. (1).

$$\begin{aligned} S_0(T) &= (3.2 + 30T) && \text{for} \quad T < 0.16 \\ S_0(T) &= 8.0 && \text{for} \quad 0.16 \le T < 0.64 \\ S_0(T) &= \frac{5.12}{T} && \text{for} \quad 0.64 \le T \end{aligned} \qquad (1)$$

where, S_0: basic design acceleration response spectrum at the exposed (outcrop) engineering bedrock (m/s^2), and, T: natural period (s). 'Major EQ' and 'Moderate EQ' in Figure. 1 correspond to the maximum earthquake motions to be considered and the once-in-a-lifetime earthquake motions, respectively.

Figure 1. **Basic design earthquake acceleration response spectra at exposed engineering bedrock**



The level of the earthquake ground motion used for the seismic design at the damage-limitation limit state should be reduced to a fifth of that for life safety. These response spectra at the engineering bedrock are applied in the design of all buildings such as conventionally designed buildings and base isolated buildings.

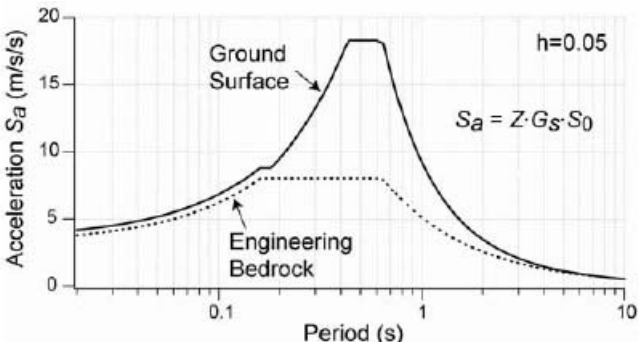
1.3 Design response spectrum at ground surface

Multiplying the response spectrum at the engineering bedrock by the surface-soil-layer amplification factor, G_s, the design EQ response spectrum at the ground surface, S_a, is obtained as shown in Figure. 2 and expressed by Eq. (2).

$$S_a(T) = ZG_s(T)S_0(T) \qquad (2)$$

where, Sa: design acceleration response spectrum at ground surface (m/s2), Z: seismic zone factor of 0.7 to 1.0, Gs: surface-soil-layer amplification factor, and, T: natural period (s).

Figure. 2. **Design earthquake acceleration response spectrum at ground surface**



Gs to be determined here is the ratio of response spectra. *Gs* is calculated based on the strain-dependent shear stiffness and damping ratio of soil as given by Eq. (3):

$$\begin{aligned}
G_S &= G_{S2}\frac{T}{0.8T_2} && \text{for} \quad T \le 0.8T_2 \\
G_S &= G_{S2} + \frac{G_{S1} - G_{S2}}{0.8(T_1 - T_2)}(T - 0.8T_2) && \text{for} \quad 0.8T_2 < T \le 1.2T_1 \\
G_S &= G_{S1} && \text{for} \quad 0.8T_1 < T \le 1.2T_1 \\
G_S &= G_{S1} + \frac{G_{S1} - 1.0}{\frac{1}{1.2T_1} - 0.1}\left(\frac{1}{T} - \frac{1}{1.2T_1}\right) && \text{for} \quad 1.2T_1 < T
\end{aligned} \qquad (3)$$

where, *Gs*: surface-soil-layer amplification factor, *Gs*1: *Gs* value at the period of *T*1, *Gs*2: *Gs* value at the period of *T*2, *T*: natural period (s), *T*1: predominant period of surface soil layers for the first mode (s), and, *T*2: predominant period of surface soil layers for the second mode (s). Minimum value of *Gs*: 1.5 for $T \le 1.2T1$ and 1.35 for $1.2T1 < T$ at the damage-limitation limit state, and 1.2 for $T \le 1.2T1$ and 1.0 for $1.2T1 < T$ at the life-safety limit state. The factors of 0.8 and 1.2 in the period classification such as 0.8*T*1, 0.8*T*2 and 1.2*T*1 in Eq. (3) are introduced to consider the uncertainties included in the soil properties and the simplified calculation.

2. Seismic design of highway bridges (partially quoted form reference. 2)

The Hyogo-ken-Nanbu (Kobe) Earthquake of January 17, 1995, caused destructive damage to the highway bridges. Based on the lessons learned from the Kobe Earthquake, the seismic design specifications for highway bridges were significantly revised in 1996. The intensive earthquake motion with a short distance from the inland-type earthquakes with Magnitude 7 class as the Kobe Earthquake has been considered in the seismic design.

2.1 Basic principles of seismic design

The bridges are categorized into two groups depending on their importance; standard bridges (Type-A bridges) and important bridges (Type-B bridges). Seismic performance level depends on the importance of bridges. For moderate ground motions induced in the earthquakes with high probability to occur, both A and B bridges should behave in an elastic manner without essential structural damage. For extreme ground motions induced in the earthquakes with low probability to occur, the Type-A bridges should prevent critical failure, while the Type-B bridges should perform with limited damage.

In the Ductility Design Method, two types of ground motions must be considered. The first is the ground motions which could be induced in the interplate-type earthquakes with magnitude of about 8. The ground motion at Tokyo in the 1923 Kanto Earthquake is a typical target of this type of ground motion. The second is the ground motion developed in earthquakes with magnitude of about 7-7.2 at very short distance.

Obviously the ground motions at Kobe in the Hyogo-ken nanbu earthquake is a typical target of this type of ground motion. The first and the second ground motions are called as Type-I and Type-II ground motions, respectively, are employed as the same as 1996 JRA Specifications. The recurrence time of the Type-II ground motion may be longer than that of the Type-I ground motion.

Table 2 shows the performance matrix including the design earthquake ground motion and the Seismic Performance Level (SPL) provided in the revised Seismic Design Specifications in 2002. There is no revision on this basic principle from the 1996 Specifications. The two level ground motion as the moderate ground motions induced in the earthquakes with high probability to occur (Level 1 Earthquake) and the intensive ground motions induced in the earthquakes with low probability to occur (Level 2 Earthquake).

Table 2. **Seismic performance matrix**

<table>
<tr><th colspan="2">Type of Design Ground Motions</th><th>Standard Bridges (Type-A)</th><th>Important Bridges (Type-B)</th></tr>
<tr><td colspan="2">Level 1 Earthquake: Ground Motions with High Probability to Occur</td><td colspan="2">SPL 1: Prevent Damage</td></tr>
<tr><td rowspan="2">Level 2 Earthquake: Ground Motions with Low Probability to Occur</td><td>Interplate Earthquakes (Type- I)</td><td rowspan="2">SPL 3: Prevent Critical Damage</td><td rowspan="2">SPL 2: Limited Damage for Function Recovery</td></tr>
<tr><td>Inland Earthquakes (Type- II)</td></tr>
</table>

Note: SPL = Seismic Performance Level.

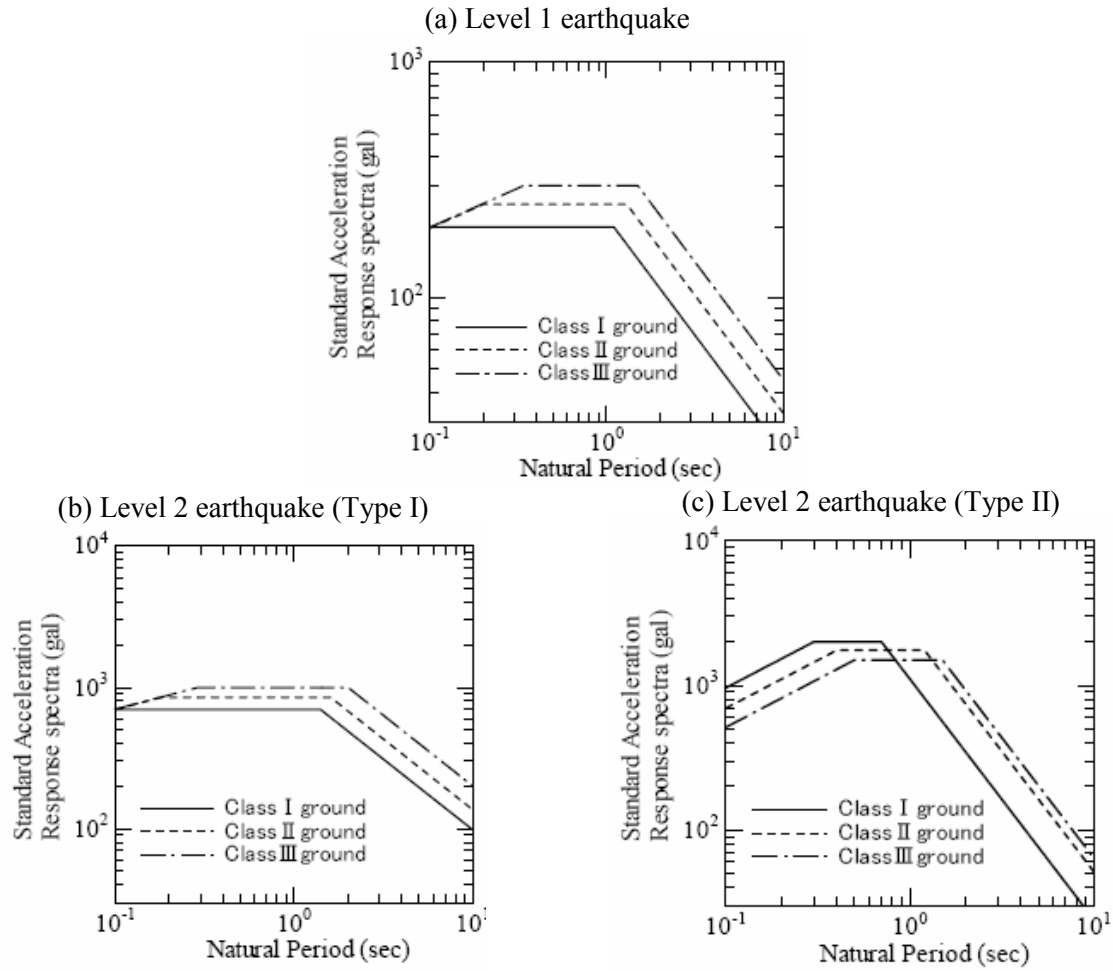
The Level 1 Earthquake provides the ground motions induced by the moderate earthquakes and the ground motion considered in the conventional elastic design method is employed. For the Level 2 Earthquake, two types of ground motions are considered. The first is the ground motions which are induced in the interplate-type earthquakes with the magnitude of around 8. The ground motion at Tokyo in the 1923 Kanto Earthquake is a typical target of this type of ground motion. The second is the ground motion developed in earthquakes with magnitude of around 7 at a very short distance. The ground motion at Kobe during the Hyogo-ken-Nanbu Earthquake is a typical target of this type of ground motion. The first and the second ground motions are named as Type-I and Type-II ground motions, respectively.

Figure 3 shows the acceleration response spectrums of the design ground motions.

In the 2002 revision, the design ground motions are named as Level 1 Earthquake and Level 2 Earthquake. One more important revision on the design earthquake ground motion is that the site-specific design ground motions shall be considered if the ground motion can be appropriately estimated based on the

information on the earthquake including past history and the location and detailed condition of the active faults, ground conditions including the condition from the faults to the construction sites. To determine the site-specific design ground motion, it is required to have the necessary and accurate information on the earthquake ground motions and ground conditions as well as the verified evaluation methodology of the fault-induced ground motions. However, the area to get such detailed information in Japan is very limited so far. Therefore, the continuous investigation and research on this issue as well as the reflection on the practical design of highway bridges is expected.

Figure 3. **Design acceleration spectrum**



3. Seismic design code for high pressure gas facilities (partially quoted form reference. 3)

The Great Hyogoken-nanbu Earthquake occurred in 1995. Some facilities and piping systems were damaged due to ground displacement (settlement and/or lateral movement) induced by liquefaction. Learning from the earthquake disaster, the Seismic Design Code was amended on March 25, 1997, after reviews and investigations by the "Seismic Safety Promotion Committee". Main items of the new code for High Pressure Gas (HPG) facilities are as follows:

1. Introduction of two-step earthquake assessments
   The Basic Disaster Management Plan revised by the Central Disaster Management Council, July 1995, requires "two-step earthquake assessments" Level 1 earthquakes and Level 2 earthquakes are defined and the required seismic performances are stipulated.

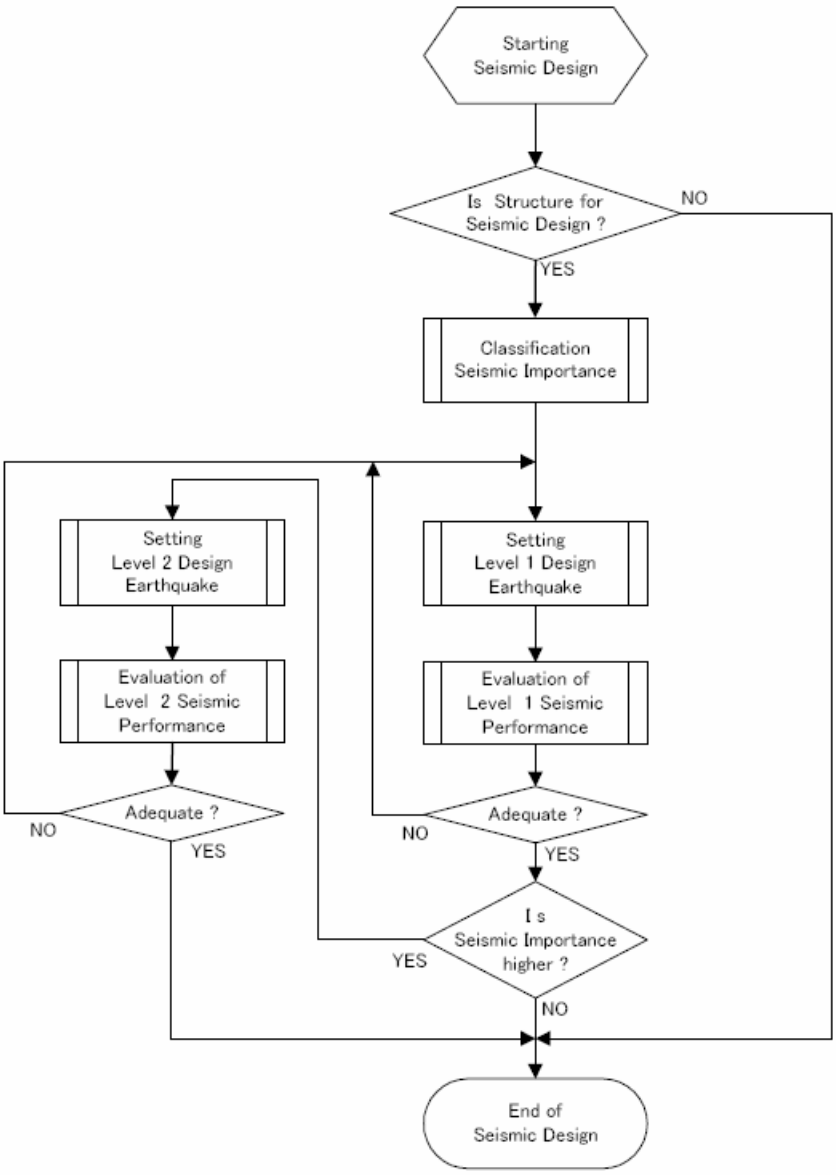
2. Seismic design code for liquefaction of soil.
   Regarding foundations, damage is caused by ground liquefaction. The required performance against ground liquefaction is added to the Seismic Design Code.

3. Seismic design code for piping systems
   Regarding piping, which was beyond the scope of the previous Seismic Design Code, because some damage was incurred, the seismic required performance of piping is added in the new code.

3.1 Introduction of two step assessments

As the first item above, "two-step earthquake assessments" for HPG facilities, the seismic design flow diagram is presented in Figure. 4. Towers, vessels, tanks, piping and their supporting structures and foundations of HPG facilities must be designed to resist earthquakes.

Figure 4. **Seismic design procedure**



All of these "Seismic Design Structures" are classified according to Seismic Importance (Ia, I, II, III). In the 1st-step seismic assessment, the maximum ground acceleration of Design Basis Earthquake is specified in accordance with Seismic Importance.

The performance of the seismic design structures during and after a Level 1 earthquake must be such that the facilities maintain their operational functions.

The 2nd step assessment is executed only for facilities of higher Seismic Importance (Ia, I). The maximum ground acceleration of Design Basis Earthquake is specified in accordance with the Seismic Importance.

The performance of the seismic design structures during and after a Level 2 earthquake must be such that the facilities shall not cause loss of human life.

3.2 The design base earthquake

The Level 1 earthquake is defined as a probable strong earthquake that occurs during the service life of the plant. The maximum ground acceleration of Level 1 earthquake for design (called "Level-1 Design Basis Earthquake" or "L-1 DBE") is defined as 300 gal in the most seismically active areas of Japan, the Kanto and Tokai regions.

The Level 2 earthquake is defined as a possible strongest earthquake with extremely low probability of occurrence.

The definition of Design Base Earthquakes is as follows. The peak ground accelerations, PGA, of a Design Base Earthquake are defined as:

$$\alpha_H = 150\mu_k \beta_1 \beta_2 \beta_3 \quad \text{(Gal.)} \tag{4}$$

$$\alpha_V = 75\mu_k \beta_1 \beta_2 \beta_3 \quad \text{(Gal.)} \tag{5}$$

where, α_H and α_V : the peak maximum horizontal and vertical ground accelerations of Design Base Earthquake, respectively. $\beta 1$: the Seismic Importance factor of the seismic design structure, and its value is in the range 1.0 to 0.5, $\beta 2$: the Seismic Zoning factor, and its value is in the range 1.0 to 0.4 for Level 1 and 1.0 to 0.7 for Level 2 earthquakes, $\beta 3$: the ground multiplication factor in surface soil layers, and its value is 2.0, but 1.4 for hard rocks. μ_k: the Earthquake level factor, whose value is 1.0 for Level 1 earthquakes, and 2.0 or greater for Level 2 earthquakes.

Dynamic characteristics are determined from the "Response spectrum as multiplication factor" for Seismic Zoning and the soil structure. The maximum ground acceleration of L-2 DBE is the value from 420 to 600 Gal in accordance with Seismic Zoning. The stipulation is based on the following discussions.

In the case of HPG facilities, the entire country is mapped as four zones, which is compatible with the Japanese Building Code. However, when considering Level 2 earthquakes, it is not economically feasible for all facilities in a zone to be designed to resist the highest acceleration to be expected in the same zone.

Locations where highest acceleration is expected are seismically limited to near active faults. The accelerations at the other most locations in the same zone are of lower levels. The DBE in this code is defined by the acceleration expected at the larger area in the same zone. HPG facilities located at sites where acceleration higher than the defined level is expected would be designed according to the owner's discretion.

In the Kobe earthquake, the maximum ground acceleration is 818 Gal., as reported by the Kobe Ocean Meteorological Observatory. However, no serious damage of nearby HPG facilities was observed, which were designed to resist DBE of 240 Gal. Now seismology is developing and advancing but it is difficult to

determine a reliable and economically feasible DBE that can be universally applied to all HPG facilities under the jurisdiction of the HPG safety law.

Many seismologists say that there are no observations of active faults associated with earthquakes of JMA Magnitude less than 6.5. Therefore, such earthquakes could occur anywhere in Japan. The minimum ground surface acceleration of Level 2 earthquakes is stipulated in the Code to be 420 Gal., which acceleration is expected in an area near the epicentre of a Magnitude 6.5 earthquake.

4 Upgrading of "examination guide for a seismic design of nuclear power plant (partially quoted form reference. 4)

Japanese "Examination Guide for A Seismic Design of Nuclear Power Reactor Facilities" (Ref. 5) is now under the process of upgrading by the Nuclear Safety Commission of Japan. The major points of the upgrading are related to the new developments of seismology, the safety concept for public understanding, and the reflection of the government policy to handle "Seismic margins of nuclear facilities" (Ref. 6) The works and discussions are currently still on going. The followings are recommended to describe in the "Examination Guideline" by the Nuclear Safety Commission:

1. The safety function of the important facilities including safety protection facilities should never be spoilt even if the plant is attacked by the earthquake ground motion presumed to occur in quite small probability from the viewpoint of the geology, the geological structure around the site, and the seismology within a certain period of service life of the NPP facilities.
2. The above facilities should be designed to have suitable safety margins based on the existence of the certainties in determining the above earthquake ground motion and the uncertainties (dispersion) in the seismic capacities of the NPP facilities.

4.1 Classification of safety importance in seismic design

Through the works for upgrading of the present "Examination Guideline", it is required to evaluate the residual risk taking into account the existence of the uncertainty in the seismic capacities of the facilities and the uncertainties in determining a design earthquake ground motion as small as possible. For such a risk, it has been said that the risk is kept small enough by taking enough margins in the detailed design of the facilities against the seismic load by the design earthquake ground motion of S_2. Also it is considered desirable to decrease the residual risk from the viewpoint of improving the safety much more. Based on this concept, it is proposed to revise the classification methodology in the "Examination Guideline" that the class A component, which has a function to redundant accident conditions when accident occurs, is changed over to the present class As so that the whole structures, systems, and components in the class A will be categorized into the present class As. For the name of every class, it is proposed to take "seismic class I, II, and III" in expression to avoid confusion. The functional importance classification of the NPP facilities in seismic design based on the way of thinking described above is shown in Table 3 .

Table 3. **The Functional Importance Classification of Facilities in Seismic Design**

Seismic Class	Function
Seismic Class I	A SSC* which has radioactive materials inside or a SSC directly related to other SSC having radioactive materials inside, then the function loss of the SSC might be a cause of radioactive material release in the atmosphere. Also a SSC needed to avoid radioactive materials release and a SSC needed to shutdown the reactor when accident occur or a SSC needed to reduce an influence by the radioactive material release in the atmosphere in addition those influence and the effect are large.
Seismic Class II	SSCs whose influence and effect is small as compare to the above mentioned phenomena in the seismic class I.
Seismic Class III	SSCs other than the seismic classes of I and II

* SSC: structures, systems and components

4.2 Earthquake ground motion for use in the evaluation of seismic safety

It is proposed to treat the earthquake ground motion for use in the evaluation of seismic safety. The earthquake ground motion of Ss is defined as an earthquake ground motion presumed to occur, or possibly occur, though its possibility is quite small, around the site from the viewpoint of seismology and earthquake engineering within a certain period of the plant life.

The earthquake ground motion of Ss is proposed to be designed based on the following:

1. It should be taken into account of past earthquake ground motion and ground motion caused by active faults. “Seismo-techtonic” knowledge is also considered for reference.
2. It should be taken into account as earthquake ground motion to be considered at least as “earthquake ground motion presumed without specifying the seismic sources”. It is presented that the common way of thinking to determine a response spectrum based on a probabilistic study and/or past earthquake records obtained in the neighbourhood of epicentres without seismic fault in the inland cluster earthquakes.
3. The probability of the ground motion level of Ss is checked after design.
4. Earthquake ground motion in the vertical direction at free field should be also determined.

Figure 5 shows the current proposed methodology for determining the design ground motion of Ss.

4.3 Evaluation of the seismic safety of the facilities

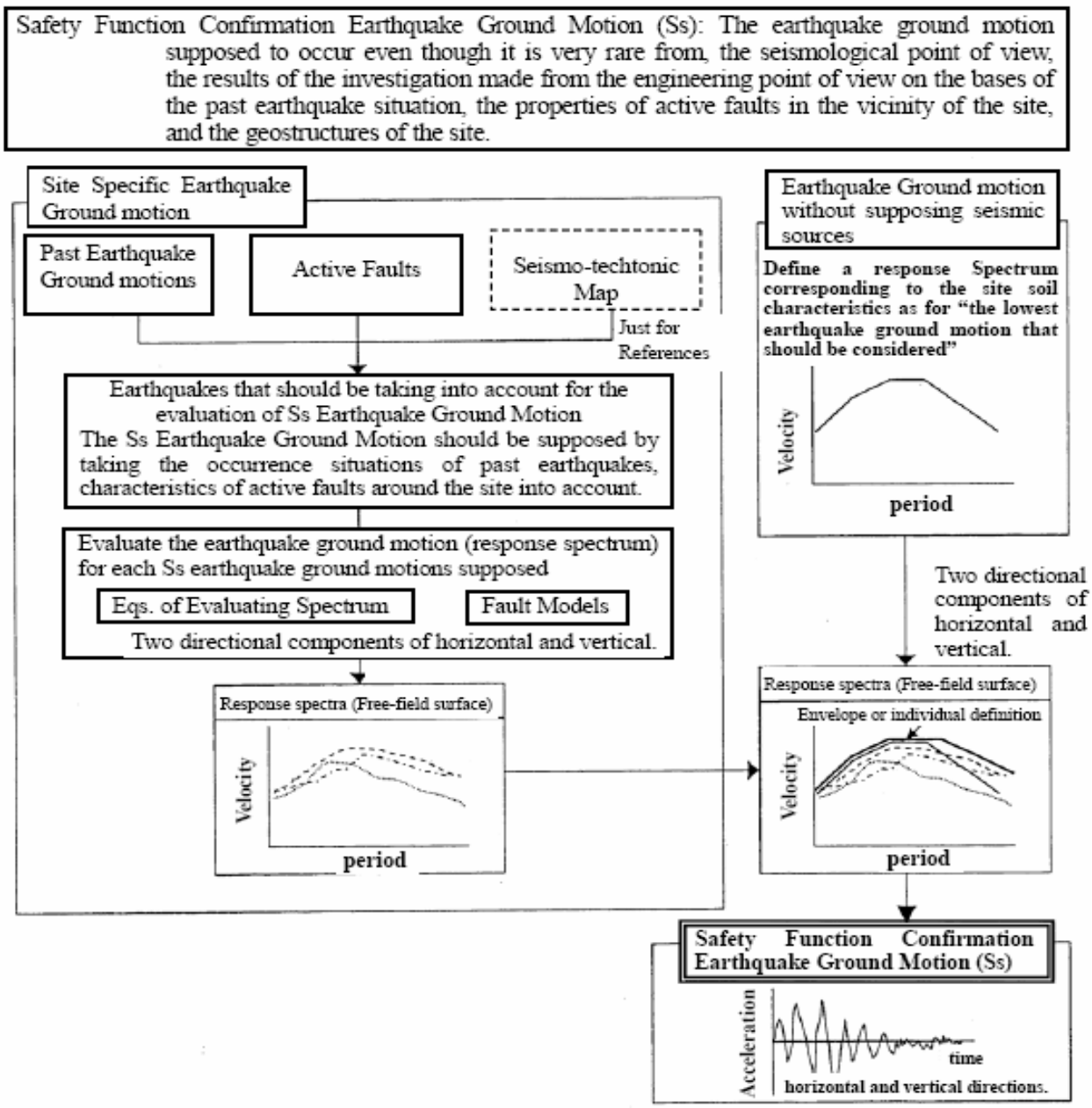
Followings are proposed to evaluate the facilities based on the fundamental way of thinking:

1. The facilities of seismic class I should not be spoilt their safety functions by the seismic load of Ss. The way of thinking for the load combination of building, structure, piping, and equipment and the allowable state limit are followed after those for the current standard earthquake ground motion of S_2.
2. It is proposed to evaluate the safety margins existing in buildings, structures, equipment and piping. The evaluation should be performed with suitable technique, which can take into account the uncertainties in determining the earthquake ground motion of Ss and the uncertainties in the quantitative seismic capacity of the facilities.

In order to evaluate the “Safety Margins”, there are two methodologies, the conventional deterministic methodology and the newly introduced probabilistic methodology. Therefore before evaluating actual

safety margins, it should be judged independently which methodology is taken at a stage of the detailed design, and then the evaluation should be formed properly.

Figure.5 **A Flow chart for generating design earthquake Ground Motion**



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Appendix F.

SOUTH KOREA

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1. Conventional standards

1.1 Applicable standard and seismic hazard mapping

The different seismic design codes are prepared for buildings and industrial facility structures: e.g. design code (KBCS, Korean Building Code-Structural) for conventional buildings, design code for highway bridges, etc.).

The map of seismic hazards was established utilizing historical and instrumental earthquake catalogue in Korean Peninsula based on the probabilistic methodologies developed by Frankel (1995) and Cornell (1968) and the probabilistic seismic hazard methodology developed by EPRI (1986). Several attenuation laws were used for the hazard evaluation.

1.2 Ground response spectra definition

The seismic hazard is defined by a design ground response spectrum (DGRS) as defined in the conventional standards. Two zones are defined based on the seismic hazard map for 500-year return period with the following seismic zone factors (A): 0.11g for Zone 1 and 0.07 for Zone 2.

While the methods for determining the DGRS for the seismic design of conventional structures are different from each other, the basic concepts are the same as follows:

The resultant DGRS is given by

$$S_a(T) = a_g \times \gamma_I \times S \times C_s(T)$$

where $S_a(T)$ is the DGRS, a_g is equal to $A \times g$, γ_I is the importance category of the construction, S is the soil factor, and $C_s(T)$ is the seismic response coefficient.

If we consider a conventional building, with the highest importance (γ_I =1.5 as recommended by the KBCS) to be erected in the Korean NPP sites (Soil class A [hard rock site] or B [rock site]), the horizontal DGRS to be applied to the seismic design of such a conventional building, with the soil conditions, is given in Figure 1 (curves 1 and 2). The DRGS by the current KBCS, which has been revised in this year, has spectral acceleration values around two times higher than the previous ones and 1.5 times higher than the ones by the design code for highway bridges, in the case of Soil class B.

2. Nuclear standards

2.1 Applicable standards

The following standards are used as the applicable standards in the seismic design of all safety-related structures of nuclear facilities:

- US NRC Regulatory Guides and the Standard Review Plans relating to seismic siting and seismic design and qualification of the structures, systems and components (SSC) are applicable for Korean NPPs.
- Canadian CSA standards had been exceptionally applied to the Wolsong NPPs since they were imported from Canada.

2.2 Ground response spectra definition

There are four NPP sites in Korea: Kori (four units), Wolsong (four units), Yonggwang (six units), and Ulchin (six units) sites. The design ground response spectra are defined as follows except for Kori unit 1:

- OBE Earthquake RG 1.60 - 0.1 g for the Kori, Yonggwang, and Ulchin sites.
- SDE (equivalent to OBE) Earthquake CSA (CAN3-N289.3) - 0.1 g for the Wolsong site.
- SSE Earthquake RG 1.60 - 0.2 g for the Kori, Yonggwang, and Ulchin sites.
- DBE (equivalent to SSE) Earthquake CSA (CAN3-N289.3) - 0.2 g for the Wolsong site.

The design spectra applied to Kori unit 1, for which the construction permit had been issued in 1972, were derived from the 1940 El Centro earthquake scaled to the design PGA.

Curves (3) and (4) in Figure 1 show the OBE and SSE horizontal DGRS (5% damping) from RG 1.60. Figure 2 shows the OBE (4% damping) and SSE (7% damping) horizontal DGRS from RG 1.60 since 4% and 7% damping ratios are used in the seismic design of reinforced concrete (RC) structures of NPP facilities for the OBE and the SSE, respectively, based on RG 1.61. For reference, the SSE horizontal DGRS from RG 1.60 (5% and 7% damping) and the CSA standard (5% damping) are compared in Figure 3 since 5% damping ratio is used in the seismic design of the RC structures by the CSA standard.

For all site, the SSE PGA of 0.2 g was determined by a deterministic seismic hazard assessment method based on:

- The historical and instrumental seismicity of the region.
- Shifting of the epicenter of the maximal historical event of the identified seismotectonic areas to closest edge of the site.
- Assuming an attenuation law.

Figure.1 **Design ground response spectra by conventional and nuclear standards at NPP sites for 5% damping**

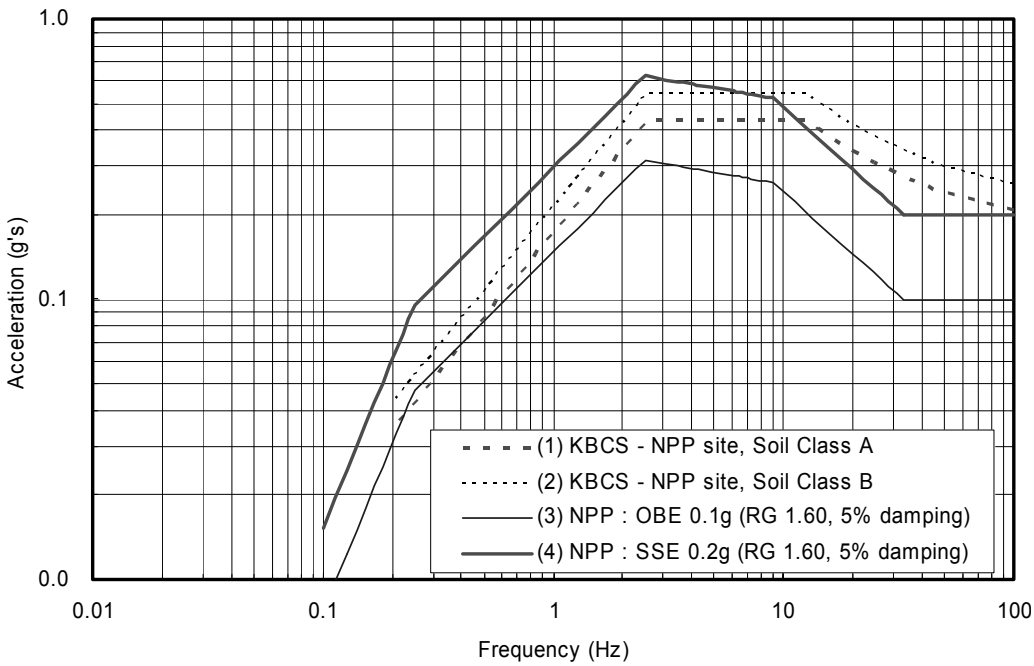


Figure 2. **Design ground response spectra by conventional and nuclear standards at NPP sites for different damping ratios**

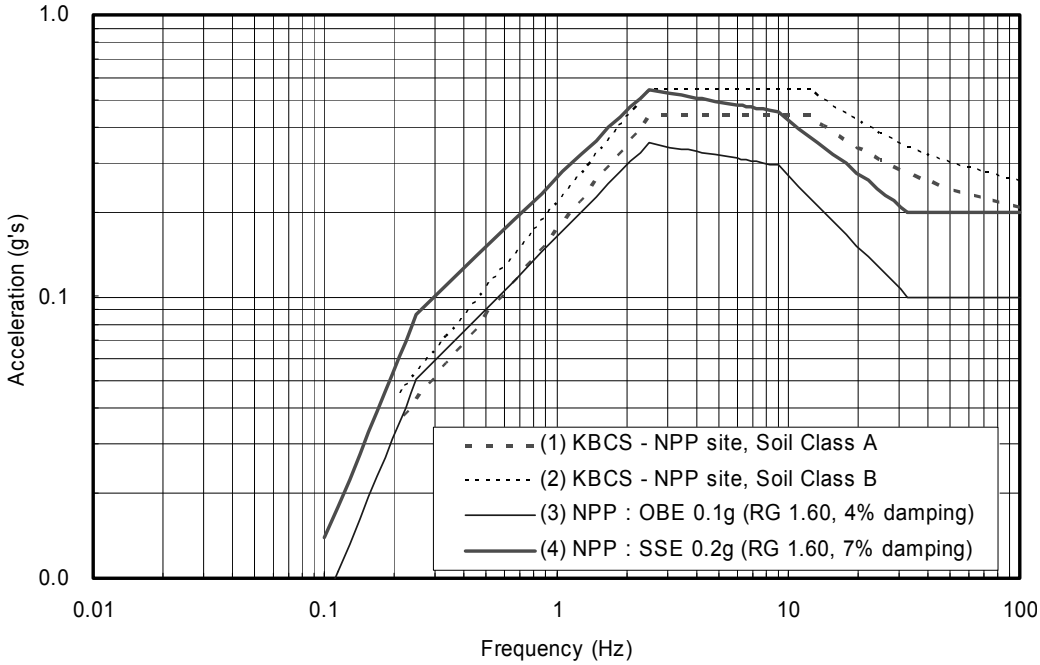
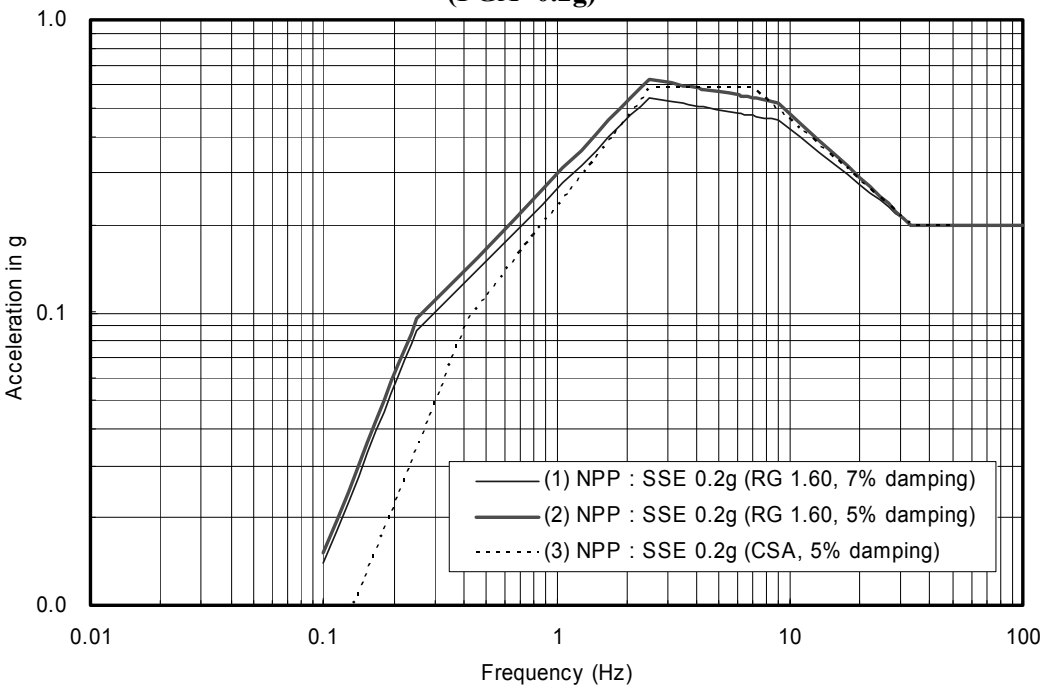


Figure 3. **Design ground response spectra by RG 1.60 and CSA standard (PGA=0.2g)**



3. Comparison of the nuclear and conventional seismic loadings - Case study

In order to compare the seismic design loads using the conventional and nuclear standards, the magnitude of the Base Shear are estimated using the respective applicable spectra. For the sake of simplification, the structures are assumed to be symmetric, regular, dissipative and behaving ideally as an SDOF system.

The acceleration is determined for two current fictive types of buildings:

1. Relatively stiff structure where lateral loads are resisted by shear walls.
2. Framed concrete structure with infill masonry walls.

Application of KBCS for the Conventional Standard (Soil Class B)

	Assumed frequency (Hz)	Response modification coefficient (R)	Spectral acceleration (g)	Elastic Base shear (without application of q factor)	**Design Base Shear**
Building (1)	1.5	4.5	0.329	0.329*W	**0.073*W**
Building (2)	7	3	0.549	0.549*W	**0.183*W**

Application of the Nuclear Standard (RG 1.60, 7% damping)

	Assumed frequency (Hz)	Spectral acceleration (g)	**SSE Design Base Shear**
Building (1)	1.5	0.362	**0.362*W**
Building (2)	7	0.471	**0.471***W

4. Conclusions

The Korean NPPs structures and components are designed for much higher seismic loads than the conventional counterparts even though the DGRS for conventional buildings is higher than the SSE DGRS defined for nuclear installations in the high frequency range for Korean NPP sites. The OBE DGRS for nuclear installations is lower than the DGRS for conventional buildings for the NPP sites.

5. References

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Appendix G.

SPAIN

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1. Conventional standards

1.1 Applicable standard and seismic hazard mapping

Application of seismic code to design structures in Spain is considered from 1968. The current applicable code is the NCSE-02[1] which is required as a minimum for collective, industrial, and private use buildings and structures.

The current map of seismic hazard established in Spain is based on the probabilistic methodology developed by Cornell (1968) which is based on 3 basic assumptions:

- Exponential distribution of the seismic events magnitude (Guttenber-Richter law).
- Time of recurrence following a Poisson distribution.
- Uniform distribution of the seismicity inside the zones defined as seismic sources.

The seismic hazard in Spain has been established in terms of free field acceleration in rock, through updated seismic data given by the IGN (Instituto Geográfico Nacional) earthquakes catalogue. A Spanish intensity (European Macroseismic Scale) attenuation law was used for the hazard evaluation, and an average of basic horizontal acceleration was obtained in Gals by using the correlation: $\log a_b = 0{,}30103 \times I - 0{,}2321$. The basic acceleration "a_b" was calculated in all Spanish municipalities for a 10% probability of exceedance in fifty years (corresponding with a 475 return period, or a 2‰ annual probability of exceedance). Low period influence of larger earthquakes in Southwest of Iberia from the distant Gorringe Bank source are considered by the contribution coefficient K which values can range from 1.0 to 1.3. Both indicated parameters, the acceleration "a_b" and the coefficient "K", are mapped in figure 1. Requirements of the NCSE-02 code are not applicable for sites with "a_b" values below 0.04g. Minimum free field seismic design loads are defined by the ground response spectra anchored with the design acceleration "a_c" as defined in NCSE-02.

1.2 Ground response spectra definition

The design acceleration a_c is defined by the expression: $a_c = a_b \times \rho \times S$;

where, "ρ" is the structure risk coefficient (range from 1.0 for normal importance to 1.3 for special importance), and.

"S" is the soil effect coefficient which values can range from 0.80 to 1.44 depending on $a_b \times \rho$ results, and the ground coefficient "C" which values can range from 1.0 for ground with $v_s > 0.75$ km/s to 2.0 for ground with $v_s \leq 0.20$ km/s.

Ground response spectra shape for a critical damping of 5% are defined by the following expressions:

- for $T < T_A \Rightarrow \alpha(T) = 1 + 1.5 \times T \div T_A$;
- for $T_A \leq T \leq T_B \Rightarrow \alpha(T) = 2.5$; and
- for $T > T_B \Rightarrow \alpha(T) = K \times C \div T$

  where: "T_A" and "T_B" are the cut shape characteristic periods,
  "α(T)" is the spectral ordinate for a period "T", and
  the respective factors "K" and "C", are the contribution coefficient and the ground coefficient mentioned in paragraph 1.1. The NCSE spectral shapes for different "C" and "K" values are showed in figure 2.

1. To visit the IGN web site (www.geo.ign.es) to obtain the current version.

To obtain spectral shapes for critical damping different than 5%, are interpolate "α(T)" values for $T < T_A$, and for $T \geq T_A$ "α(T)" values are multiply by the factor "ν", where $\nu = (5 \div \omega)^{0.4}$, and "ω" is the critical damping percentage of the structure. The vertical spectral shapes are obtained by multiplying for 0.7 the respective horizontal ones.

2. Nuclear standards

2.1 Applicable standards

The applicable seismic codes and standards to all Spanish NPPs are: the USNRC Appendix H of 10CFR100, and the Standard Review Plan and Regulatory Guides referring sitting, design of safety structures, and SSCs seismic qualification.

2.2 Ground response spectra definition

In Spain there are nine nuclear sites: seven sites with nine operating reactors (total capacity of 7461 MWe) and one plant under decommission (Vandellós I, 480 MWe reactor), a fuel facility (Juzbado plant), and a low-medium radioactive waste disposal (El Cabril). Figure 1 shows their locations, and in tables 1 to 5 are shown the main data related to their seismic design, including vertical loads.

Through figures 3 to 7, are presented the SSE_H free field ground response spectra used as design basis in all Spanish nuclear installations, except for José Cabrera because of the plant will finish operation at April 2006 and will apply decommission and closure plans.

For a comparative purpose, in figures 3 and 4, are also included the NCSE-02 free field ground horizontal response spectra required as minimum seismic load for designing structures in Ascó, Cofrentes and Vandellós municipalities. According with the NCSE-02 criterion, no seismic design is required for building structures in Almaraz , José Cabrera, Stª. Mª. de Garoña, and Trillo municipalities, because of "a_b" values obtained from seismic hazard analysis are below 0.04 g.

Table 1. **Main data of Spanish NPPs under operation**.

NPP Reactor	Power (MWe)	Origin of Technology	Previous Permit	Construction Permit
JOSÉ CABRERA	160	U.S.A. - PWR Westinghouse	March 1963	June 1964
S. Mª GAROÑA	466	U.S.A. – BWR3 MARK I – G. E.	August 1963	May 1966
ALMARAZ, I	930	U.S.A. - PWR Westinghouse	October 1971	July 1973
ALMARAZ, II	930	U.S.A. - PWR Westinghouse	May 1972	July 1973
ASCÓ, I	940	U.S.A. – PWR Westinghouse	April 1972	May 1974
ASCÓ, II	966	U.S.A. - PWR Westinghouse	April 1972	Mars 1975
COFRENTES	994	U.S.A. – BWR6 MARK III – G.E.	November 1972	Sep. 1975
VANDELLÓS, II	1009	U.S.A. - PWR Westinghouse	March 1976	Dec. 1980
TRILLO	1066	Germany - PWR K.W.U.	September 1975	Aug. 1979

Table 2. **Main data of Spanish fuel cycle facilities**.

Facility	Facility Type	Capacity	Origin of Technology	Relevant Permits
JUZBADO	Fuel to BWR, PWR and VVER	400 UTn/Year	U.S.A/Spain	Dec. 1980
VANDELLÓS I	NPP under decommission	480 MWe	France/GCR	Nov. 1989 Jan. 1998
EL CABRIL	LMRW Disposal	60 000 m^3	France/Spain	Oct. 1989

Table 3. **Site seismic design basis of newer Spanish NPPs under operation**.

	Safe Shutdown Earthquake		**Operating Basis Earthquake**	
NPP	**PGA_H - PGA_V**	**Spectra**	**PGA_H - PGA_V**	**Spectra**
ALMARAZ, I II[2]	0.10g - 0.067g	Newmark H	0.05g - 0.033g	Newmark H
ASCÓ, I-II	0.13g - 0.087g	Newmark H	0.07g - 0.047g	Newmark H
COFRENTES	0.17g	RG-1.60 H - V	0.085g	RG-1.60 H - V
VANDELLÓS, II	0.20g	RG-1.60 H - V	0.10g	RG-1.60 H - V
TRILLO	0.12g - 0.085g	RG-1.60 H	0.06g - 0.425g	RG-1.60 H

Table 4. S**ite seismic design basis of older Spanish NPPs under operation**

	Safe Shutdown Earthquake		**Operating Basis Earthquake**	
NPP	**PGAH - PGAV**	**Spectra**	**PGAH - PGAV**	**Spectra**
J. CABRERA	0.07 g - 0.047g	Nureg/Cr-0098 H	0.035g - 0.023g	Nureg/Cr-0098 H
S. Mª GAROÑA	0.10g	RG-1.60 H - V	0.05g	RG-1.60 H - V

Table 5. **Site seismic design basis of Spanish fuel cycle facilities**

	Safe Shutdown Earthquake		**Operating Earthquake**	
Facility	**PGAH - PGAV**	**Spectra**	**PGAH - PGAV**	**Spectra**
JUZBADO	0.15 g	Pseudostatic Analysis	-----	-----
EL CABRIL	0.24g	RG-1.60 H – V	0.12g	RG-1.60 H - V

2. One diesel building was designed as CN Trillo with a USNRC R.G. 1.60 horizontal spectrum anchored to adopted SSE PGA_{H-V}

Figure 1. **Seismic hazard map of Spain. Red dots show all Spanish nuclear installations under operation**.

Figure 2. **NCSE-02 free field ground horizontal response spectra for different C and K values**.

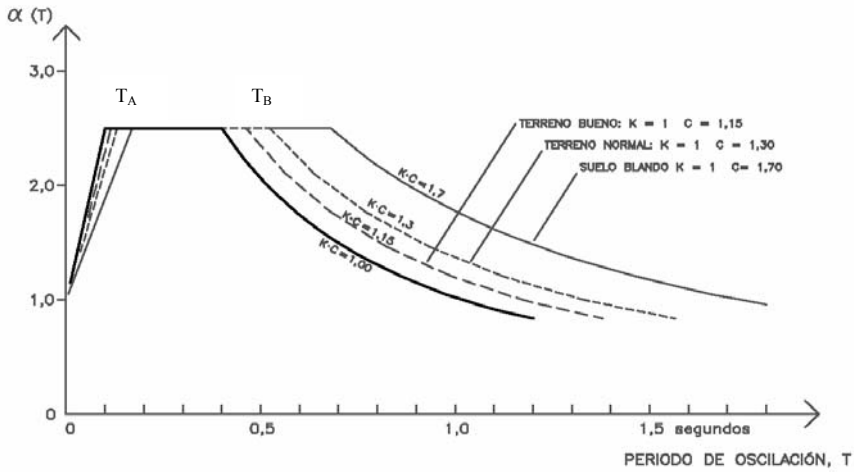


Figure 3. **Free field ground response spectra of Cofrentes NPP (SSE_H) and NCSE-02 code in Cofrentes municipality**

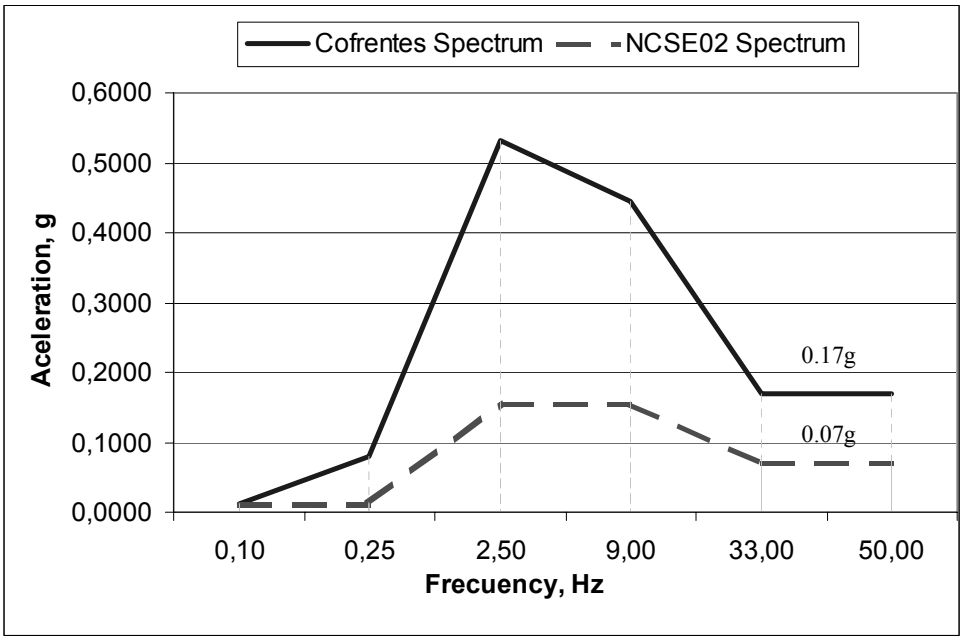


Figure 4. **Free field ground response spectra of Ascó and Vandellós NPP (SSE_H) and NCSE-02 code in both municipalities**.

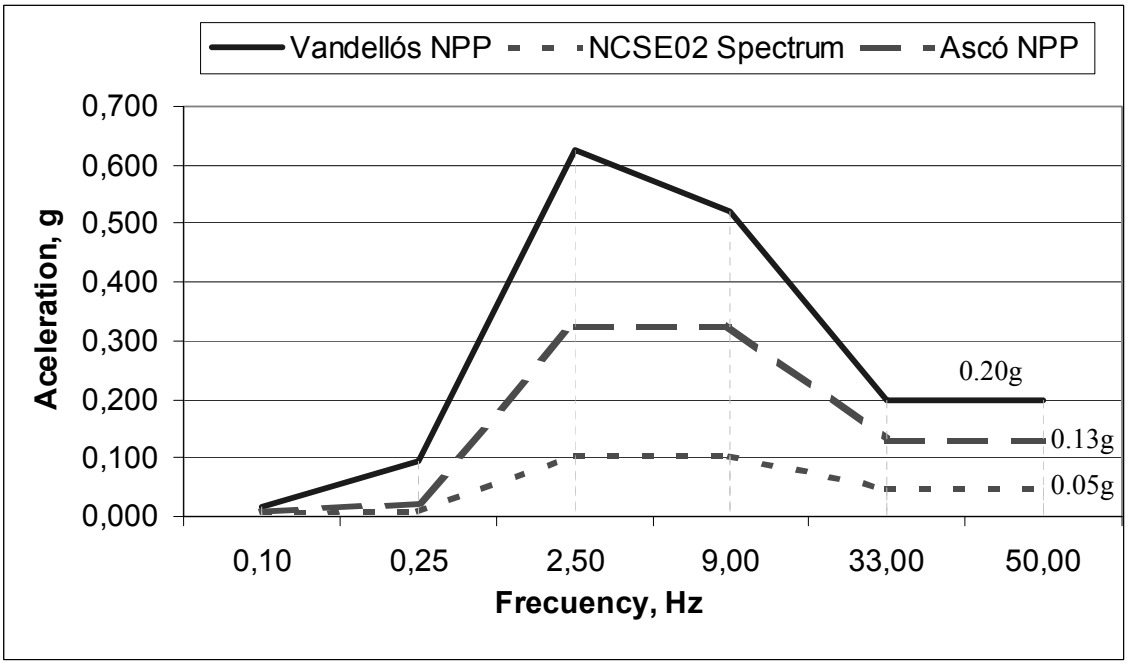


Figure 5. **Free field ground response spectra of Almaraz NPP (SSE_H). Seismic loads from NCSE-02 are not required in Almaraz municipality, because of "ab" value are below 0.04 g**

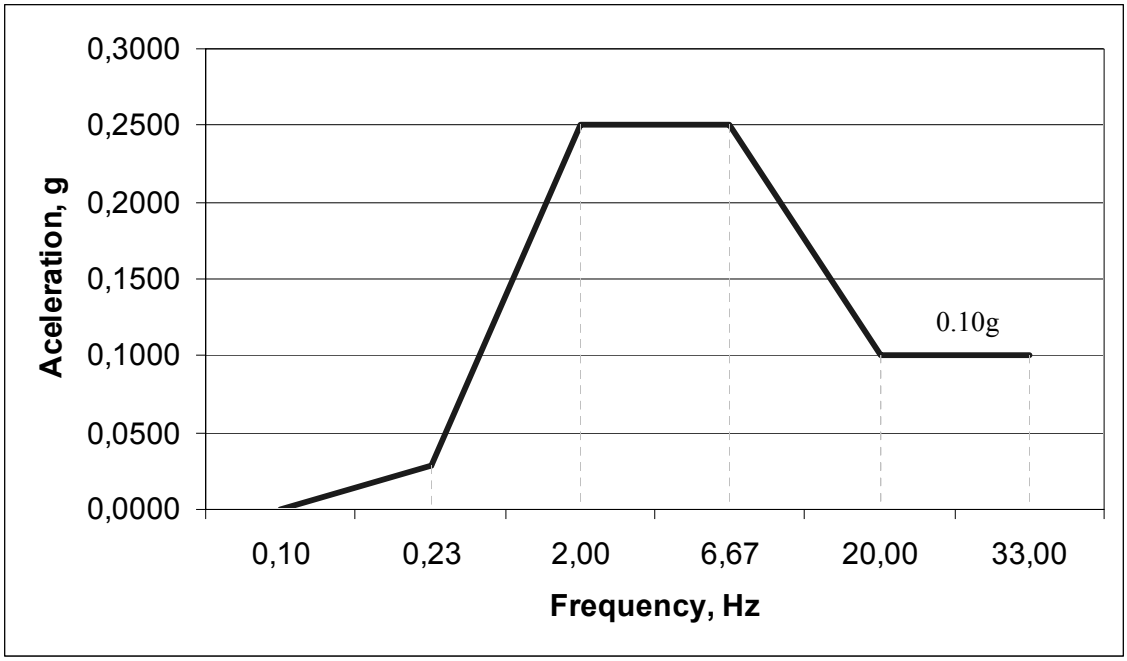


Figure 6. **Free field ground response spectra of St^a. Mª de Garoña NPP (SSE_H). Seismic loads from NCSE-02 are not required in Garoña municipality, because of "ab" value are below 0.04 g**

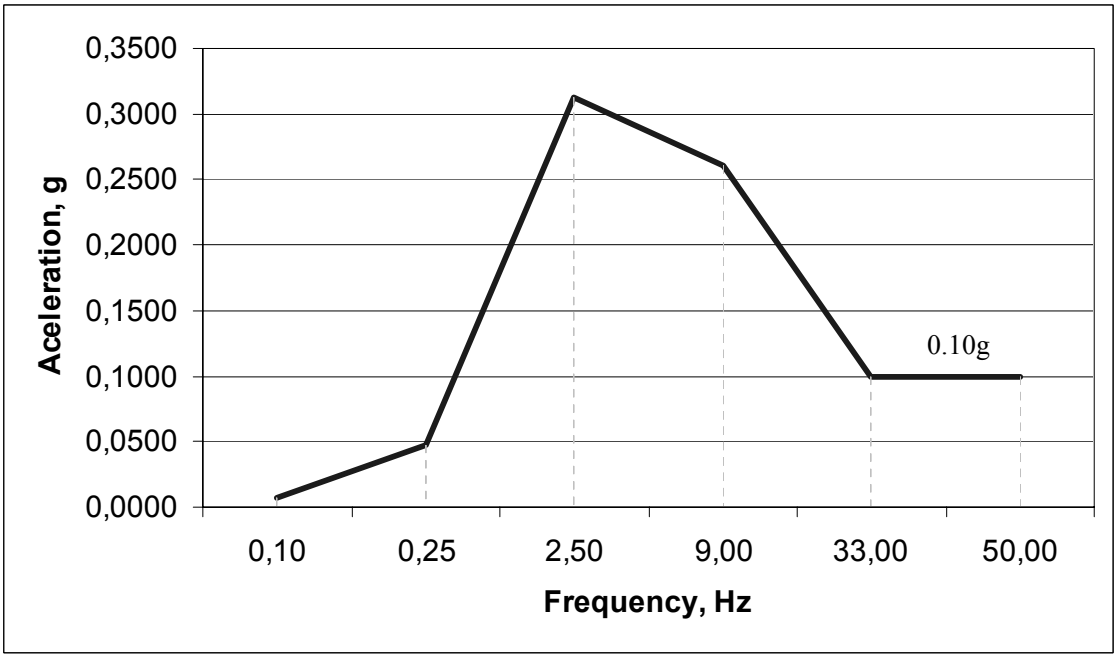
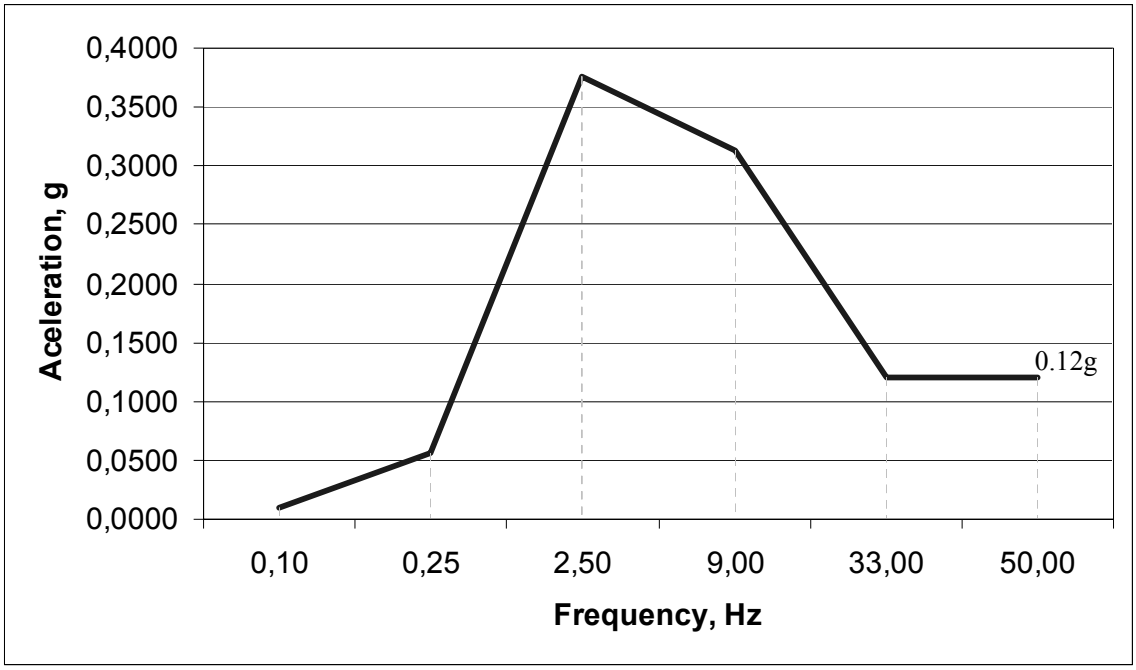


Figure 7. **Free field ground response spectra of Trillo NPP (SSE_H). Seismic loads from NCSE-02 are not required in Trillo municipality, because of "ab" value are below 0.04g**



3. Comparison of the nuclear and conventional seismic loadings - Case study

In order to compare the seismic design loads using the conventional and nuclear standards the magnitude of the Base Shear are estimated using the respective applicable spectra. For the sake of simplification, the structures are assumed to be symmetric, regular, dissipative and behaving ideally as an SDOF system.

The seismic load is determined for a two current fictive types of buildings:

1. Relatively stiff structure where lateral loads are resisted by shear walls.
2. Framed concrete structure with infill masonry walls.

Application of the Spanish NCSE-02

Building	Assumed frequency	Spectral acceleration	Behaviour factor (q)	Elastic base shear, before use the q factor	**Design Base Shear**
1	1.5 Hz	0.100g	3.00	(0.100 × W) ÷ 3	**0.033 × W**
2	7.0 Hz	0.125g	3.00	(0.125 × W) ÷ 3	**0.042 × W**

Application of the Nuclear Standard

Building	Frequency	Spectral acceleration	**SSE Design Base Shear**
1	1.5 Hz	0.100g	**0.33 × W**
2	7.0 Hz	0.125g	**0.42 × W**

4. Conclusions

Through figures 3 and 4 is shown a comparison between the free field seismic loads required by nuclear standards to the operating Spanish NPPs, and those required by the Spanish NCSE-02 code to building structures in the nuclear site municipalities (Cofrentes, Ascó, and Vandellós). Figures 5 to 7 only show the free field seismic loads required by nuclear standards, because of "a_b" values are below 0.04 g in Almaraz, Stª Mª de Garoña, and Trillo municipalities, and applying seismic loads of the NCSE-02 are not required.

Nuclear plant SSCs are designed in Spain for significantly higher seismic loads than the conventional counterparts (~ one order of magnitude in the previous case study). The SSE defined as design basis for the Spanish nuclear installations, covers significantly loads from the conventional Spanish NCSE-02 code in all free field ground response spectra frequency range. Additionally, a PGA of 0.1 g must be considered like minimum value to anchor the SSE design spectra of all Spanish nuclear installations.

Appendix H.

UNITED STATES

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1. Standards for conventional structures

1.1 Applicable standards

For design of conventional structures in the United States, different seismic codes are in effect in the different states and local municipalities to estimate seismic demands and design requirements for load combinations. In the majority of the states (particularly those east of the Rocky Mountains), the code being used is the International Building Code (Ref. 1). The code currently in force in many of these states is the IBC-2000 version, although the IBC-2003 version is available for incorporation into the local building code. In some municipalities, the governing seismic code is the Uniform Building Code (Ref. 2). The IBC code is the more recent code development as compared to the UBC, which has been available for many years for use by local municipalities.

Both codes are based on a design concept for near-collapse conditions. For a given seismic hazard definition and site condition, seismic demands are computed on the basis of relatively simplified elastic assumptions to evaluate structural response. These elastic element loads are then significantly reduced by means of global "response modification" factors that are selected based upon building type, building configuration and joint ductility considerations. These reduced element loads are then used to design individual structural elements. The use of these reduced design loads is intended to incorporate significant nonlinear behaviour in the structural response. An attempt is made in these codes to consider performance-based design concepts in their formulation by the use of "Importance Factors". The purpose of these factors is to increase the design seismic demands for more critical facilities where acceptable nonlinear behaviour is intended to be reduced to ensure post-event operability of the facility.

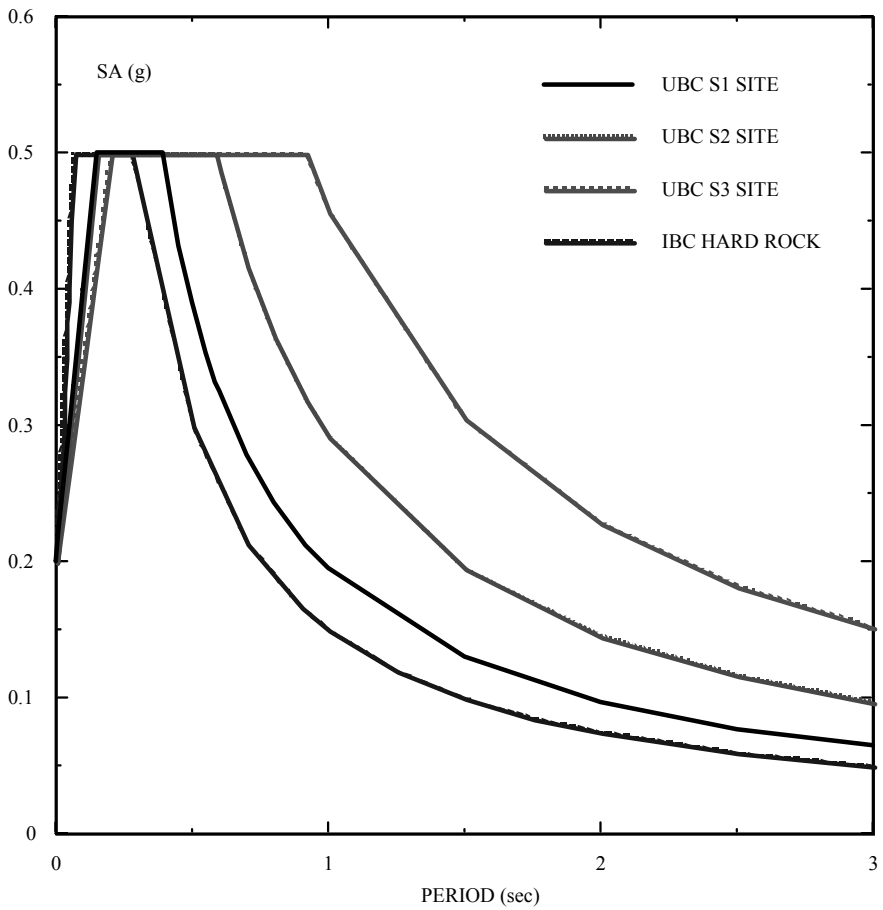
The designs based on the IBC and UBC building codes only consider near-collapse situations; that is, the purpose of the design for ordinary facilities is to ensure that the structure, although potentially seriously damaged, does not collapse onto occupants. For more important facilities considered for design following building code requirements, the use of the Importance Factor leads to designs expected to be less damaged for the same seismic environment and therefore better able to be operable following the design event. The agencies charged with developing and maintaining these design guides incorporate modifications as improved analysis methods or new seismic empirical data become available. The latest recommendations contained in FEMA 450 (Ref. 3), for example, have been updated from previous versions of the publication. The objective is to upgrade the IBC Code in relatively short periods as consensus among design professions is reached.

In recent years, the performance-based code concepts have been significantly extended by considering a broader range of acceptable behaviour for states from near-elastic to near-collapse structural. The purpose of this extension is to allow the performance-based design concept to consider other design issues. For example, for critical facilities where issues such as radiation confinement may be of concern, a more extensive performance-based design procedure has been developed and is described in the American Society of Civil Engineers (ASCE) Design Guide 43-05 (Ref. 4). The procedure has been incorporated into Department of Energy directives for design of nuclear storage and processing facilities. The process developed considers five seismic design categories of increasing seismic hazard (defined in terms of lower probabilities of non-exceedance or longer return periods) and four acceptable deformation states. The structural deformation states consider responses varying from near-elastic (essentially undamaged) behaviour to near-collapse. The designer then must choose appropriate design factors based on the expected performance required to satisfy the design criteria.

1.2 Ground response spectra definitions

The seismic hazard definition used for design of conventional facilities covered by the building codes is obtained from maps and spectral shapes presented in the specific codes. Figure 1 presents a comparison of response spectral shapes (all scaled to 0.2 g PGA) from the UBC and IBC codes. In the older versions of the UBC code, spectral shapes were modified based on relatively general assessments of site characteristics. The S1 site is considered a relatively stiff rock or soil site, but not a rock site with especially hard rock associated with eastern US locations. The S2 site is somewhat softer and the S3 site even softer. All the spectra are scaled to a specified PGA appropriate for a given region based on a relatively old seismic hazard maps (1980 vintage). For the more recent IBC Code, a given spectral shape is shown for a relatively hard rock site condition. As may be noted, the primary difference is to shift spectral peaks to longer periods as site characteristics get softer. For the IBC Code, however, the mapped characteristics are (a) based on more recent seismic hazard evaluations generated by the U.S. Geological Survey, and (b) scaled the shapes based on two parameters (a short period and one second period amplification factors). The modifications in FEMA 450 also incorporate an additional spectral shape parameter. These parameters are also selected based on site condition as well as structural design category (SDC). It should be noted that SDC as used in the IBC is not the same as the SDC categories defined in ASCE 43-05.

Figure 1. **Standard spectral shapes from building codes**
5% damped conventional spectral shapes (file: spectra.crd)



The 5% damped design response spectral parameter used for design of conventional facilities in the older UBC codes (PGA) was selected at a hazard level associated with a 10% probability of exceedance in

50 years (or the so called 500 year event). In the later versions of the IBC Code, the basic design spectral parameters are selected at a hazard level of 2% in 50 years (or the so-called 2 500-yr seismic event). Following the recommended IBC procedure, this spectrum is then scaled by a factor of 2/3 for design of conventional facilities. This results in a spectrum that is different from that for the 500-year event, with the difference being a function of the specific site location. For some Central and Eastern US (CEUS) sites, this spectrum may be closer to the 1 000-yr events rather than the 500-year event. For the case where an Importance Factor of 1.5 is used for design of special facilities, the basic design ground motion is therefore associated with the 2 500-yr hazard.

For the special storage facilities considered in ASCE 43-05, the basic design response spectrum is based upon the 2 500-yr return period. The consideration of the additional conservatism needed to cover design issues such as facility confinement, requiring a reduced acceptable strain state, is accounted for in the selection of the various parameters used the designs.

2. Nuclear standards

2.1 Applicable standards

The design of nuclear power plants (NPPs) follows, as expected, a much more rigorous design and evaluation process than conventional or even other critical storage facilities since the consequence of failure is a much more serious issue. The primary guidance documents were developed by the U.S. Nuclear Regulatory Commission and consist of the Standard Review Plan NUREG-0800, Ref. 5) and associated Regulatory Guides (such as RG 1.60, Ref. 6). These guidance documents provide significant detail on design and analysis procedures required for safe system designs. Other developments undertaken by NRC over the years have addressed a number of issues with respect to plant design. Fortunately, this effort continues to improve assessment of uncertainties in these methods. More recent evaluations of site seismic hazard have been developed by the U. S. NRC and have made use of probabilistic hazard estimation to address issues of consistency in probability of non-exceedance in development of design response spectra (Ref. 7).

The primary differences in process between design of NPPs and more conventional facilities has to do with (a) the much longer return period used to develop the parameters of the design response spectrum and (b) the requirement to not allow any inelastic behaviour in the structural responses. The result of these two differences generally leads to extremely robust designs for Category I facilities.

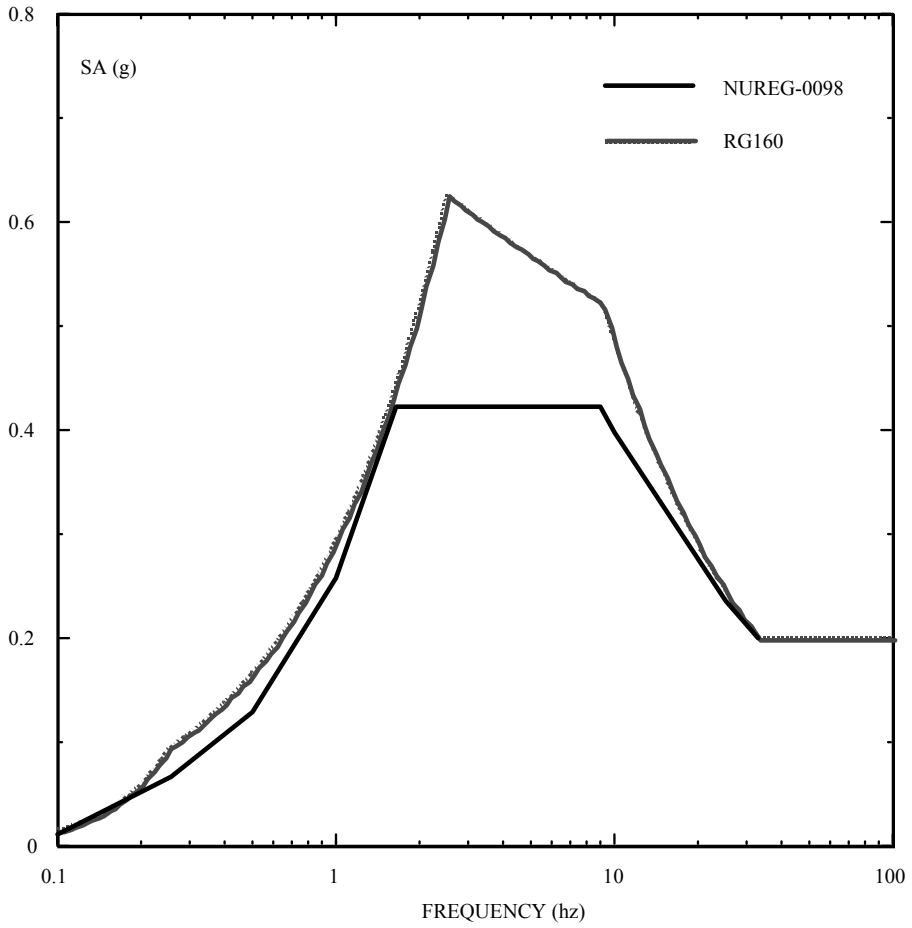
2.2 Ground response spectra definitions

The basic ground motion design spectrum has typically been defined at a probability of non-exceedance set at a level of median $1*10^{-5}$, which is approximately equivalent to mean $1*10^{-4}$ (mean 10 000-yr event). This compares with the 500-yr event used for ordinary structures or 2 500-yr event used for the some critical facilities. In the graded approach described in ASCE 43-05, the 10 000-yr event is considered only for the highest design category. Coupled with the requirement of elastic response, the NRC design process then corresponds to the most stringent design conditions considered in the graded approach used for design of critical facilities by other U.S. agencies.

In the original designs approved for older nuclear power plants, the basic input design response spectrum and corresponding enveloping ground motions were based on Reg. Guide 1.60 spectral shapes scaled to peak ground acceleration (PGA) selected to match the site seismic hazard. This spectral shape was selected independently of site condition (rock or deep soil, for example) and specified PGA. Figure 2 presents a plot of this spectral shape (scaled to a PGA of 0.2 g) together with the NUREG-0098 shape (Ref. 8). This recommended spectrum was developed later in time and was used in re-evaluations of some older plants. Both are considered appropriate shapes to represent large magnitude events. As may be noted, the Reg.

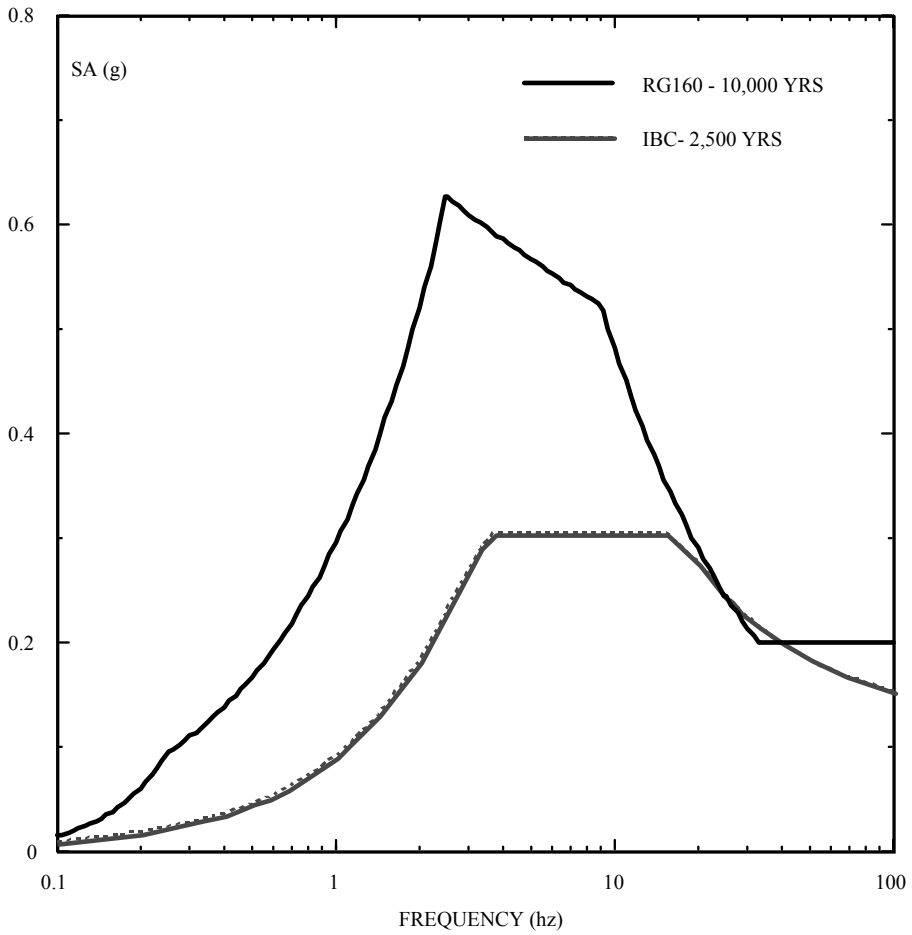
Guide 1.60 spectrum is significantly more conservative than the 0098 spectrum, particularly at frequencies less than 10 Hz, the frequency range of interest from a damage potential point of view. However, in addition to not being able to characterize anticipated site-specific behaviour, these spectral shapes are hazard inconsistent; that is, unfortunately, from a hazard consistent point of view, these spectral shapes are not consistent; that is, the return period associated with the spectra is frequency dependent.

Figure 2. **Comparison of deterministic spectra for large magnitude events**
5% damped spectral shapes used for evaluation of nuclear structures (file: spectra.crd)



It is difficult to compare the Reg. Guide and NUREG-0098 spectra with building code spectra without somehow accounting for the difference in return period. The comparison is obviously site-specific since it depends on the seismic hazard at a given location. Figure 3 presents a comparison of the 10 000-yr Reg. Guide 1.60 spectrum with a corresponding 2 500-yr IBC spectrum, using a scaling relationship from a particular location. As may be noted, the conservatism incorporated into the NPP design spectrum as compared to a typical building code design, even for a critical facility, is obvious.

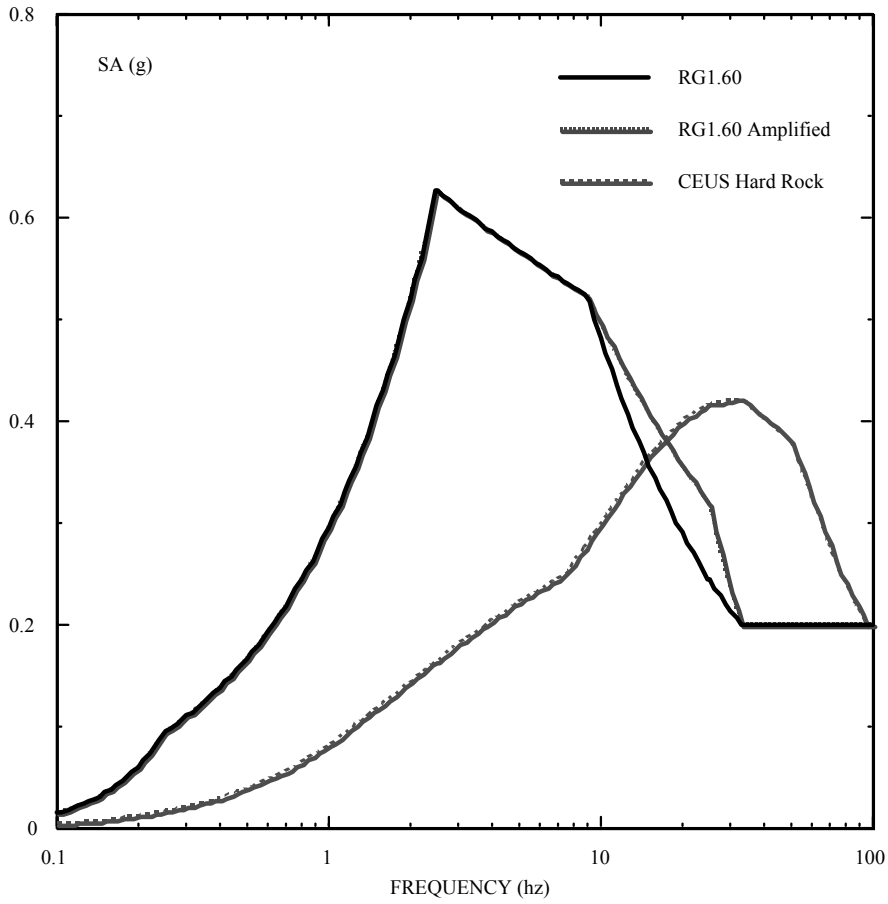
Figure 3. **Approximate relation of a RG 1.60 spectrum with a building spectrum**
(5% damped spectra comparison of RG 1.60 and building code estimate (file: spectra.crd))



In recent years, evaluations of the seismic hazard have improved as the empirical earthquake data set has increased, particularly with respect to measurements from larger magnitude events at many different sites. In addition, the increased confidence in site response analysis has allowed for the ability to transfer the empirical data to sites with significantly different crustal velocity profiles. Recent studies (Ref. 9) have used this information to generate spectral shapes that are considered appropriate for various parts of the U.S. The implications of this development are extremely significant and now require more serious consideration of appropriate design ground motion inputs for NPPs. It has been understood for some time that the higher velocity crustal rocks associated with CEUS sites lead to much higher frequency content in surface ground motions than captured in the Reg. Guide 1.60 spectrum. The developments in (Ref. 9) make an attempt at quantifying this difference.

An example of this development is shown in Figure 4. The low frequency RG 1.60 spectral shape is shown and is first compared with a similar spectrum amplified to some extent in the frequency range from 9 Hz to 33 Hz. This modified Reg. Guide shape was proposed for evaluation of one of the advanced standard plant designs. The modified spectrum was only modestly increased in this frequency range beyond 9 Hz. Based on the recent work in Ref. 9, the spectral shape for CEUS hard rock sites is seen to exhibit significantly greater high frequency content, extending to frequencies as high as 80 Hz. This may have an important consequence for design of equipment housed within the plant. Of even more importance for design of the primary load path elements of the structure is the significantly reduced ground motion levels at frequencies below 10 Hz that will impact the seismic structural and strength demands on these elements.

Figure 4. **New spectral shapes based on site-specific hazard evaluations**
(Comparison of spectra amplified in high frequency (file: spectra.crd))



3. Conclusions

This short summary comparing building code design processes with that typically used for NPPs clearly indicates the significantly increased conservatism in the NPP designs. This conservatism stems from two separate effects, namely, the significantly longer return period associated with the input design response spectrum as well as the restriction of the element design to consider only elastic behaviour. This additional conservatism alone adds factors of as much as five as compared with building code designs, when comparing similar input spectra. Recent advances for performing hazard consistent design spectra developments has led to significant issues that must be evaluated to come to similar confidence in facility safety. The issues associated with these developments stem from the potential for significantly increased high frequency content of design ground motions, with a corresponding decrease in low frequency content in the important frequency range from 1 Hz to 10 Hz.

4. References

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