Seismic high-frequency content loads on structures and components within nuclear facilities

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Abstract

Sweden is generally considered to be a low seismicity area, but for structures within nuclear power facilities, the safety level demand with respect to seismic events are high and thus, these structures are required to be earthquake-resistant. The seismic hazard is here primarily considered to be associated with near-field earthquakes. The nuclear power plants are further founded on hard rock and the expected ground motions are dominated by high frequencies. The design earthquake considered for the nuclear facilities has an annual probability of $10^{-5}$ events, that is, the probability of occurrence is once per 100,000 years. The focus of the study is the seismic response of large concrete structures for the nuclear power industry, with regard not only to the structure itself but also to non-structural components attached to the primary structure, and with emphasis on Swedish conditions. The aim of this licentiate thesis is to summarize and demonstrate some important aspects when the seismic load is dominated by high frequencies. Additionally, an overview of laws, regulations, codes, standards, and guidelines important for seismic analysis and design of nuclear power structures is provided.

The thesis includes two case studies investigating the effect of seismic high-frequency content loads. The first study investigates the influence of gaps in the piping supports on the response of a steel piping system subjected to a seismic load dominated by high amplitudes at high frequencies. The gaps are found in the joints of the strut supports or are gaps between the rigid box supports and the pipe. The piping system is assessed to be susceptible to high-frequency loads and is located within the reactor containment building of a nuclear power plant. The stress response of the pipe and the acceleration response of the valves are evaluated. The second study investigates the effect of fluid-structure interaction (FSI) on the response of an elevated rectangular water-containing concrete pool subjected to a seismic load with dominating low and high frequencies, respectively. The pool is located within the reactor containment building of a boiling water reactor at a nuclear power plant. The hydrodynamic pressure distribution is evaluated together with the stress distribution in the walls of the tank.

From the two case studies, it is evident that the response due to a seismic load dominated by high frequencies and low frequencies, respectively, is different. Although the seismic high-frequency load may be considered non-damaging for the structure, the effect may not be negligible for non-structural components attached to the primary structure. Including geometrical non-linear effects such as gaps may however reduce the response. It was shown that the stress response for most of the pipe elements in the first case study was reduced due to the gaps. It may also be that the inclusion of fluid-structure interaction effects changes the dynamic properties of a structural system so that it responds significantly in the high frequency range, thus making it more vulnerable to seismic loads dominated by high frequencies. In the second case study, it was shown that even for a seismic load with small amplitudes and short duration, but with dominating high-frequency content, as the Swedish $10^{-5}$ design earthquake, the increase of the dynamic response as fluid-structure interaction is accounted for is significant.

**Keywords:** Nuclear power plant, Earthquake, Seismic high frequencies, Fluid-structure interaction, Piping, Concrete, Pool
Sammanfattning

Sverige anses generellt vara ett område med låg seismicitet, men för strukturer inom kärnkraftsanläggningar är kraven på säkerhet med avseende på seismiska händelser hög och således måste dessa strukturer vara jordbävningssäkra. Den seismiska risken är här i första hand förknippad med jordbävningar i närområdet. Dessutom är kärnkraftverken grundlagda på hårt berg, varför de förväntade seismiska markrörelserna domineras av höga frekvenser. Den dimensionerande jordbävningen för de kärntekniska anläggningarna har en årlig sannolikhet på $10^{-5}$ händelser, det vill säga, sannolikheten för inträffande är en gång på 100 000 år. Fokus för studien är den seismiska responsen hos stora betongkonstruktioner inom kärnkrafts industrin, med hänsyn, inte bara till själva strukturen, utan även till icke-strukturella komponenter infästa i den primära strukturen, och med betoning på svenska förhållanden. Syftet med denna licentiatuppsats är att sammanfatta och demonstrera några aspekter som är viktiga när den seismiska belastningen domineras av höga frekvenser. Dessutom tillhandahålls en översikt av lagar, normer, standarder och riktlinjer som är viktiga för seismisk analys och dimensionering av strukturer inom kärnkraftsindustrin.


Från de två fallstudierna är det tydligt att responsen till följd av en seismisk belastning dominerad av höga frekvenser respektive låga frekvenser är olika. Även om den seismiska högfrekventa lasten kan anses vara icke-skadlig för själva byggnadsstrukturen, kan effekten bli relativt stor för icke-strukturella komponenter infästa i den primära strukturen. Inkluderande av geometriska icke-linjära effekter såsom mellanrum kan dock reducera responsen.

Den första fallstudien visade att spänningsresponsen för de flesta av rörelementen reducerades till följd av mellanrummen. Det kan också vara så att inkluderandet av fluid-struktur interaktionseffekter förändrar de dynamiska egenskaperna hos ett struktureellt system så att det responderar signifikant i det högre frekvensområdet, vilket gör det mer sårbar för seismiska belastningar som domineras av höga frekvenser. Den andra fallstudien visade att även för en seismisk last med små amplituder och kort varaktighet, men dominerad av höga frekvenser, som den svenska dimensionerande $10^{-5}$ jordbävningen, är ökningen av den dynamiska responsen då hänsyn tas till fluid-struktur interaktion betydande.

Nyckelord: Kärnkraftverk, Jordbävning, Seismiska höga frekvenser, Fluid-struktur interaktion, Rörsystem, Betong, Bassäng
Preface

The research presented in this licentiate thesis was carried out at the Department of Civil and Architectural Engineering at KTH Royal Institute of Technology, Stockholm, within the Division of Concrete Structures, between February 2011 and May 2014. Supervisors of the work were Professor Anders Ansell and PhD Richard Malm, and I am sincerely grateful for their guidance, assistance, support and never-ending encouragement throughout the project.

The work has been possible thanks to Vattenfall AB who financially has supported the project, which I am very thankful for. I also wish to express my thanks to Patrik Gatter and Richard Malm, at the time employed by Vattenfall, who initiated this project.

Part of the work reported in this thesis was conducted while I was visiting Westinghouse Electric Company, Cranberry Twp, PA, USA, and I wish to express my thanks to them for their assistance and support.

Furthermore, I would like to thank the members of the reference group and my colleagues, at KTH as well as at Vattenfall, for their time, valuable discussions and comments on the work. I am also very grateful for the valuable comments on the writing by Mikael Hallgren.

Last, but not least, I would like to express my deepest gratitude to my husband and my son for just being!

Stockholm, May 2014

Cecilia Rydell
List of appended publications

Paper I


The case study and the writing of the paper were done by Rydell. The work was supervised by Malm and Ansell who also assisted with comments on the writing. Part of the work reported in the paper was conducted while Rydell was visiting Westinghouse Electric Company, Cranberry Twp, PA, USA. Before submittal, the manuscript was reviewed and approved by Westinghouse Electric Company.

Paper II


The calculations and the analysis of the results were done by Rydell while the writing of the paper was done by Rydell and Ansell, who also supervised the work. Gasch and Eriksson were part of the initial Re&D project that the study in this paper is an independent continuation of. Gasch and Eriksson also contributed with comments on the writing.

Paper III


The writing of the paper was mainly done by Rydell with comments from Gasch, Facciolo, Eriksson and Malm. Rydell contributed with knowledge on seismic analysis. Facciolo contributed with knowledge on fluid mechanics. Gasch and Eriksson contributed with knowledge on advanced finite element modelling and performed the majority of the numerical simulations of the study. Malm coordinated the study and contributed with knowledge on advanced structural analysis.
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Bibliography  

Appended papers
1. Introduction

In the design of ordinary buildings in Sweden the structural dynamic effects of earthquakes do not have to be considered and the topic of earthquake-resistant design is thus not covered by conventional codes and standards. For nuclear power structures, these effects can however not be neglected, neither in the case of buildings nor in the case of associated electrical and process equipment. Within the nuclear power industry there is a demand for a sound knowledge of seismic analysis and design, and it has been further actualized with the Fukushima Dai-ichi nuclear power plant accident in 2011.

1.1 Background

There are three operating nuclear power plant sites in Sweden, Forsmark, Ringhals, and Oskarshamn, that together account for approximately 40 – 50 % of the Swedish electricity production. Vattenfall is the main owner of Forsmark and Ringhals, whereas Oskarshamn is owned by E.ON and Fortum. Until 2005, there was also an active nuclear power plant site at Barsebäck, which now is awaiting decommissioning. Apart from these power generating plants there are also other nuclear facilities, such as for example the nuclear fuel factory in Västerås, owned by Westinghouse Electric Company, the interim storage facility for spent nuclear fuel (Clab) near Oskarshamn, and the final repository for short-lived radioactive waste (SFR) in Forsmark. The Swedish Nuclear Fuel and Waste Management Company, SKB, is responsible for Clab and SFR. The three nuclear power plant sites in operation have a total of ten reactors; three at Forsmark, four at Ringhals and three at Oskarshamn. The three reactors at Oskarshamn, O1, O2 and O3, are all boiling water reactors (BWR) and they were put into operation in 1972, 1974 and 1985, respectively. Also at Forsmark the three reactors, F1, F2 and F3, are of BWR type and they have been in operation since 1980, 1981 and 1985, which makes Forsmark the newest nuclear power plant site in Sweden. Ringhals is the largest site and also different from the others in that three of the reactors, R2, R3 and R4, are pressure water reactors (PWR). These were put into operation 1975, 1981 and 1983, respectively. R1 is a BWR and has been in operation since 1976. The two reactors at Barsebäck were put into operation 1975 and 1977 and shut down in 1999 and 2005, respectively. For more information on the two main types of nuclear power reactors, BWR and PWR, see for example the Swedish Radiation Safety Authority (SSM, 2014a).

Sweden has been, and still is, considered a low seismicity region and when the nuclear reactors were built in the 1970’s and in the beginning of the 1980’s the shaking from earthquakes were not considered in the design. Forsmark 3 and Oskarshamn 3 are the only reactors in Sweden originally designed to resist earthquakes. The older reactors were not originally designed for this scenario and general requirements regarding earthquake-resistance were enforced only in 2005 when the regulation ”Konstruktion och utförande av kärnkraftreaktorer”, SKIFS 2004:2, was issued (today SSMFS 2008:17). That is, verifications with regard to earthquake-resistance had to be made afterwards and the licensees got until 2013, in some cases until 2015, to meet these requirements (SSM, 2012). In connection with plant modifications, seismic loads have however been considered since the mid 1990’s.

The question of earthquake-resistance was actualized with the large earthquake and subsequent tsunami that hit the Tohoku region of Japan on March 11, 2011. For the Fukushima Dai-ichi nuclear power plant the consequences were extremely serious and for three of the six reactors at the site the core melted (a meltdown), leading to substantial radioactive releas-
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The event has had far-reaching consequences on the safety awareness within the nuclear power industry. For nuclear power plants within the European Union the European Council requested that nuclear risk and safety reassessments, so-called stress tests, should be performed, see ENSREG (2011). The Swedish national report on the stress tests was finished at the end of 2011 (SSM, 2011). After having reviewed the stress-tests performed by the Swedish licensees, SSM concludes that further analyses have to be made for some of the reactors to find out if they fully can withstand a major earthquake.

During almost 20 years, it was prohibited to undertake preparatory actions for the construction of new nuclear power reactors in Sweden. In 1987, it was enforced by law that no new nuclear power reactors were allowed to be built. This inhibited the technical development within nuclear power safety and not until 2006 the regulations were changed, not prohibiting preparatory actions for the construction of new nuclear power reactors in Sweden to be undertaken. In 2010, the Swedish Parliament (Sveriges Riksdag) took a decision that changed the direction of Swedish nuclear policy; the decision allows new nuclear power reactors to replace old ones taken out of operation.

As a means of acquiring more knowledge on seismic analysis and design, Vattenfall AB initiated this doctoral project, with working title “Structural Dynamics – Seismic Design Civil Works”. The work is undertaken as an industrial doctoral project at KTH Royal Institute of Technology, Stockholm. The work reported in this thesis has been conducted mainly at KTH but also at Westinghouse Electric Company, Cranberry Township, USA.

1.2 Aims and goals

The focus of the doctoral project is seismic design of large concrete structures for the nuclear power industry, with regard not only to the structure itself but also to non-structural components, such as mechanical and electrical equipment, and with emphasis on Swedish conditions. The aim of this licentiate thesis is to summarize and demonstrate some aspects important when dealing with seismic high-frequency content loads, that is, with dominating frequencies above 10 Hz, which characterizes Swedish earthquakes. An additional aim is to put forth a brief outline of applicable regulations, standards and guidelines for seismic analysis and design of structures within nuclear facilities.

1.3 Contents of the thesis

The thesis consists of three appended papers and a main text, which provides some background knowledge and puts the papers in a context. The two primary papers of the thesis, Paper I and Paper II, investigate the effects of seismic high-frequency content loads on the response of non-structural components and structures, respectively. In Paper I a piping system within a reactor containment building is studied and in Paper II an elevated rectangular water-containing pool, also located within a reactor containment building, is investigated. The study in Paper II includes the effects of dynamic fluid-structure interaction and in the complementary Paper III a review of seismic fluid-structure interaction methods is presented.

The thesis is divided into seven main chapters. Chapter 2 gives an overview of laws, regulations, codes, standards and guidelines important for the seismic analysis and design of structures within nuclear power facilities. The documents listed in Chapter 2 are generally not included in the Bibliography, but the references given ought to be sufficiently clear to pre-
vent any ambiguities. Chapter 3 provides the background on causes and occurrences of earthquakes and the concept of response and design spectra are described. The chapter also includes an outline of the design response spectra used for the Swedish nuclear power plants. Chapter 4 provides the basics of seismic analysis. The different analysis methods are shortly described, together with the concept of damping. Further, the chapter includes a section on dynamic fluid-structure interaction and a section on seismic excitation. In Chapter 5 the appended papers are summarized and Chapter 6 contains a discussion on some important aspects that are of concern for the studies contained in the thesis. In the closing Chapter 7 conclusions are drawn and suggestions for further work are given.
2. Regulations, codes and standards

This chapter lists some laws, regulations, codes, standards and guidelines relevant to seismic design and analysis of nuclear power facilities in Sweden. Since American codes and standards have been used in the analysis and design within the Swedish nuclear power industry, special attention is paid to the presentation of American regulations, codes, and standards. For the Swedish documents the original titles are given, with translations in brackets. The outline is focused on the analysis and design of structures and not particularly on systems and components. For the analysis and design of systems and components additional codes and standards would be of relevance.

2.1 International

The International Atomic Energy Agency (IAEA) issues international safety standards that provide support for states to meet their obligations under international law. The regulation of safety is however ultimately a national responsibility. The IAEA Safety Standards Series are composed of three categories: Safety Fundamentals, which provide the basis for the Safety Requirements, Safety Requirements, which states the requirements that must be met to ensure the protection of people and the environment and Safety Guides, which provide recommendations and guidance on how to fulfil the safety requirements. In Table 2.1 are examples of Safety Guides governing seismic design issued by IAEA.

Table 2.1 Selection of IAEA Safety Guides governing seismic design.

<table>
<thead>
<tr>
<th>Denotation</th>
<th>Title</th>
</tr>
</thead>
<tbody>
<tr>
<td>NS-G-1.6</td>
<td>Seismic Design and Qualification for Nuclear Power Plants</td>
</tr>
<tr>
<td>NS-G-2.13</td>
<td>Evaluation of Seismic Safety for Existing Nuclear Installations</td>
</tr>
<tr>
<td>NS-G-3.3</td>
<td>Evaluation of Seismic Hazards for Nuclear Power Plants</td>
</tr>
<tr>
<td></td>
<td><em>Superseded by SSG-9</em></td>
</tr>
<tr>
<td>SSG-9</td>
<td>Seismic Hazards in Site Evaluation for Nuclear Installations</td>
</tr>
<tr>
<td></td>
<td><em>Supersedes NS-G-3.3</em></td>
</tr>
</tbody>
</table>

2.2 Sweden

Any company that is authorized to operate nuclear facilities in Sweden is obliged to comply with the legislations "Lag (1984:3) om kärnteknisk verksamhet" and “Strålskyddslag (1988:220)” decided by the parliament (riksdagen) and the legislations “Förordning (1984:14) om kärnteknisk verksamhet” and “Strålskyddsförordning (1988:293)” decided by the government (regeringen). The Swedish Radiation Safety Authority (Strålsäkerhetsmyndigheten, SSM) issues regulations that are more detailed and accompanied with general recommendations, “Strålsäkerhetsmyndighetens föreskrifter om säkerhet i kärntekniska anläggningar (SSMFS 2008:1)” and “Strålsäkerhetsmyndighetens föreskrifter om konstruktion och utfö-
rande av kärnkraftsreaktorer (SSMFS 2008:17)” include requirements on earthquake-resistance. The primary laws and regulations governing nuclear activities in Sweden are listed in Table 2.2.

The industry standard that have been used to this day for the design of structures at nuclear facilities in Sweden is “Dimensioneringsregler för byggnader vid kärntekniska anläggningar, DRB:2001” (DRB, 2001). The DRB:2001 primarily refers to “Boverkets konstruktionsregler, BKR” (BKR, 1998) and regarding seismic analysis and design reference is made to the American industry standard, “ASCE 4-98 Seismic analysis of safety-related nuclear structures and commentary” (ASCE, 1998). For conventional buildings, the BKR was replaced by the Eurocodes in 2011. To reflect this transition from BKR to Eurocodes, SSM has recently issued the report “2014:06 Dimensionering av nukleära byggnadskonstruktioner (DNB)” (SSM, 2014b). A selection of codes, standards and reports used in the earthquake resistant analysis and design in Sweden are listed in Table 2.3.

Table 2.2 Selection of Swedish laws and regulations that govern nuclear activities in Sweden.

<table>
<thead>
<tr>
<th>Denotation</th>
<th>Title</th>
</tr>
</thead>
<tbody>
<tr>
<td>SSMFS 2008:1</td>
<td>Strålsäkerhetsmyndighetens föreskrifter om säkerhet i kärntekniska anläggningar (Translation by SSM: The Swedish Radiation Safety Authority’s Regulations concerning Safety in Nuclear Facilities)</td>
</tr>
<tr>
<td>SSMFS 2008:17</td>
<td>Strålsäkerhetsmyndighetens föreskrifter om konstruktion och utförande av kärnkraftsreaktorer (Translation by SSM: The Swedish Radiation Safety Authority’s Regulations concerning the Design and Construction of Nuclear Power Reactors)</td>
</tr>
</tbody>
</table>
Table 2.3 Selection of codes, standards and reports used in the earthquake resistant
analysis and design in Sweden.

<table>
<thead>
<tr>
<th>Denotation</th>
<th>Title</th>
</tr>
</thead>
<tbody>
<tr>
<td>DRB:2001</td>
<td>Dimensioneringsregler för byggnader vid kärntekniska anläggningar</td>
</tr>
<tr>
<td></td>
<td>(Free translation: Design rules for buildings at nuclear facilities)</td>
</tr>
<tr>
<td></td>
<td>(Translation by Boverket: Design regulations, BKR)</td>
</tr>
<tr>
<td>ASCE 4-98</td>
<td>American Society of Civil Engineers, ASCE 4-98 Seismic analysis of safety-related nuclear structures and commentary</td>
</tr>
<tr>
<td>Regulatory Guide 1.60</td>
<td>Design response spectra for seismic design of nuclear power plants</td>
</tr>
<tr>
<td>SKI Technical Report 92:3</td>
<td>Characterization of seismic ground motions for probabilistic safety analysis of nuclear facilities in Sweden, Summary report</td>
</tr>
<tr>
<td>Report 21007024-1</td>
<td>Site-specific ground motion characterization for Simpevarp and Ringhals, 1995</td>
</tr>
<tr>
<td>Report 2014:06</td>
<td>Dimensionering av nukleära byggnadskonstruktioner (DNB)</td>
</tr>
<tr>
<td></td>
<td>(Free translation: Design of nuclear building structures)</td>
</tr>
<tr>
<td>SS-EN 1992-1-1</td>
<td>Eurokod 2: Dimensionering av betongkonstruktioner - Del 1-1: Allmänna regler och regler för byggnader</td>
</tr>
<tr>
<td></td>
<td>(Eurocode 2: Design of concrete structures. General rules and rules for buildings)</td>
</tr>
</tbody>
</table>

2.3 USA

The governing regulations for the operation of nuclear facilities in USA are the regulations issued by United States Nuclear Regulatory Commission (NRC), Title 10, Code of Federal Regulations (10 CFR). Some parts that address seismic design are given in Table 2.4. Regulatory guidance is given in the Standard Review Plan (NUREG-0800) and the accompanying Regulatory Guides. The guidance documents present procedures and methods that are acceptable to the NRC staff but do not contain regulatory requirements. Some parts of the Standard Review Plan addressing seismic design are given in Table 2.5, and in Table 2.6 some Regulatory Guides of interest are given.

When it comes to codes and standards there are several. In Table 2.7 are listed some that are of relevance in the seismic design of structures within nuclear facilities. For seismic analysis procedures the industry standard ASCE 4-98 (ASCE, 1998) issued by the American Socie-
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...ty of Civil Engineers (ASCE) is commonly used. However, this standard is not officially endorsed by NRC and when the standard is used a case-by-case review has to be made. The standard is currently under revision and was planned to be published in 2013, but still has not been re-published.

Many of the Regulatory Guides and the codes and standards listed in Table 2.6 and 2.7, respectively, have been, and still are, used, in Sweden, and in the recently issued Report 2014:06 (SSM, 2014b) reference is made to several of these.

<table>
<thead>
<tr>
<th>Denotation</th>
<th>Title</th>
<th>PART 50</th>
<th></th>
<th>PART 52</th>
<th>Licenses, Certifications, and Approvals for Nuclear Power Plants</th>
<th>Appendix D – Design Certification Rule for the AP1000 Design</th>
</tr>
</thead>
<tbody>
<tr>
<td>Part 100</td>
<td>Reactor site criteria</td>
<td>Appendix A to Part 100 – Seismic and Geologic Siting Criteria for Nuclear Power Plants</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 2.5 Selection of Standard Review Plan (NUREG-0800) parts governing seismic design.

| Denotation | Title | SRP 2.5.1 | Basic Geological and Seismic Information | SRP 2.5.2 | Vibratory Ground Motion | SRP 3.2.1 | Seismic Classification | SRP 3.7.1 | Seismic design parameters | SRP 3.7.2 | Seismic System Analysis | SRP 3.7.3 | Seismic Subsystems Analysis | SRP 3.8.1 | Concrete Containment | SRP 3.8.4 | Other Seismic Category 1 Structures |
Table 2.6 Selection of NRC Regulatory Guides governing seismic design.

<table>
<thead>
<tr>
<th>Denotation</th>
<th>Title</th>
</tr>
</thead>
<tbody>
<tr>
<td>RG 1.29</td>
<td>Seismic design classification (2007)</td>
</tr>
<tr>
<td>RG 1.60</td>
<td>Design response spectra for seismic design of nuclear power plants (1973)</td>
</tr>
<tr>
<td>RG 1.61</td>
<td>Damping values for seismic design of nuclear power plants (2007)</td>
</tr>
<tr>
<td>RG 1.92</td>
<td>Combining modal responses and spatial components in seismic response analysis (2012)</td>
</tr>
<tr>
<td>RG 1.122</td>
<td>Development of floor design response spectra for seismic design of floor-supported equipment or components (1978)</td>
</tr>
</tbody>
</table>
  *(Endorses ASME BPVC, Section III, Division 2)* |
| RG 1.142   | Safety-related concrete structures for nuclear power plants (other than reactor vessels and containments) (2001)  
  *(Endorses ACI 349)* |
| RG 1.165   | Identification and characterization of seismic sources and determination of safe shutdown earthquake ground motion (1997)  
  *(Withdrawn 2010 and replaced by RG 1.208)* |
| RG 1.166   | Pre-Earthquake Planning and Immediate Nuclear Power Plant Post-earthquake Actions (1997) |
| RG 1.208   | A performance-based approach to define the site-specific earthquake ground motion (2007) |
Table 2.7 Selection of codes and standards governing seismic design.

<table>
<thead>
<tr>
<th>Denotation</th>
<th>Title</th>
</tr>
</thead>
<tbody>
<tr>
<td>ASME BPVC-III-2 - 2010</td>
<td>ASME Boiler &amp; Pressure Vessel Code Section III: Rules for Construction of Nuclear Facility Components – Division 2: Code for Concrete Containments¹</td>
</tr>
</tbody>
</table>
| ACI 349-97 | Code Requirements for Nuclear Safety-Related Concrete Structures & Commentary²  
(For the design and construction of safety-related concrete structures other than reactor vessels and containments.)  
(Endorsed by Regulatory Guide 1.142) |
| ASCE 4-98 | Seismic Analysis of Safety-related Nuclear Structures³  
(Not officially endorsed by NRC) |
| ASCE 7-10 | Minimum Design Loads for Buildings and Other Structures³ |
| ASCE 43-05 | Seismic Design Criteria for Structures, Systems and Components in Nuclear Facilities³  
(Not officially endorsed by NRC) |
| ANSI/ANS-2.23-2002; R2009 | Nuclear Plant Response to an Earthquake⁴ |
| ANSI/ANS-2.27-2008 | Criteria for Investigations of Nuclear Facility Sites for Seismic Hazard Assessments⁴ |
| ANSI/ANS-2.29-2008 | Probabilistic Seismic Hazard Analysis⁴ |

¹ Issued by American Society of Mechanical Engineers (ASME).  
² Issued by American Concrete Institute (ACI).  
³ Issued by American Society of Civil Engineers (ASCE).  
⁴ Issued by American National Standards Institute / American Nuclear Society (ANSI/ANS).
3. Earthquakes

In this chapter a brief outline of the causes and occurrences of earthquakes is given, followed by a descriptive section on ground motions and response spectra. The use of different seismic design response spectra for the nuclear power plants in Sweden is thereafter presented.

3.1 Causes and occurrences

An earthquake is a transient shaking of the Earth’s surface as a consequence of a sudden release of energy stored in the crust of the Earth. For an earthquake to occur a mechanism is needed to supply the energy and stress the material, and the material needs to be brittle and able to store significant amounts of energy. There are different causes of earthquakes, but the majority of earthquakes in the world occur due to plate tectonics (Bolt, 2001). The theory of plate tectonics describes the movements of the outermost part of the earth, the lithosphere, which extends to a depth of approximately 80 km. Plate boundaries can either be divergent (moving away from each other), convergent (moving towards each other) or transform (slide past each other). Earthquakes are however not limited to the plate boundaries, but also occur within continental plates, so called intraplate earthquakes. The closest plate boundary to Sweden is the mid-Atlantic ridge, located more than 1,000 km away. For more on earthquake causes and mechanisms, see for example Bolt (2001) and Shearer (2009).

Sweden is today considered a low seismicity area. The biggest earthquake in the vicinity of Sweden in modern time is the 1904 magnitude M_S 5.4 Oslofjord earthquake (Bungum et al., 2009). The latest earthquakes greater than magnitude 5 are the 2004 Kaliningrad earthquakes which were of magnitude M_w 5.0 and 5.2. These events were not expected since they occurred in an area with low seismicity (Gregersen et al., 2007). Apart from these earthquakes, six earthquakes with magnitudes between 4 and 5 and several smaller earthquakes have occurred throughout Sweden since earthquakes commenced to be detected instrumentally in Sweden in 1904 (SNSN, 2014). Figure 3.1 shows the known earthquakes in Sweden and its vicinity between 1375 and 2005. The occurrence and magnitudes of earthquakes before 1904 are based on historical documents. For maps showing earthquakes after year 2000, see the website of the Swedish National Seismic Network (SNSN, 2014). The northeast coast, the area west of Lake Vänern and a number of the glacially induced faults in the north of Sweden are today areas with relatively high seismic activity, as can be seen in Figure 3.1. Although Sweden is considered an area with low seismic activity today, Bödvarsson et al. (2006) points out that a low historical seismic activity not necessarily means that the area will continue to show a low rate of activity. This is due to the episodic characteristic of seismic activity. Large earthquakes, some possibly with magnitudes above M_w 8, have occurred in the north of Sweden approximately 10,000 years ago (Bödvarsson et al., 2006). The likely cause of these events is believed to be the de-glaciation at the end of the ice age.

The largest earthquake ever measured is of magnitude 9.5 and occurred in Chile in 1960 (USGS, 2014). The latest earthquake of magnitude 9 occurred in March 2011 in the Tohoku region near the East coast of Honshu, Japan.

In the preceding paragraphs, some different magnitudes defining the size of earthquakes have been used. The most common magnitude scales are the Richter local magnitude scale M_L, the surface wave magnitude M_S, the body wave magnitude m_b and the moment magnitude M_w. The magnitude scales are related to the amplitude of the observed ground motion and are logarithmic. That is, one unit increase in magnitude means that the amplitude of the
Seismic wave increases by a factor of 10 and the energy released by the earthquake increases by a factor of 32. For a comprehensive description of magnitude scales, see for example Shearer (2009).

The shaking from an earthquake is made up of three different types of seismic waves, see Figure 3.2. The first is the primary wave or the P wave, which is the wave type travelling fastest. The P wave is a longitudinal wave, alternatingly compressing and dilating the ground in the direction of wave propagation. The second is the shear wave or S wave, sometimes also referred to as the secondary wave. The S wave shakes the ground back and forth in the direction perpendicular to wave propagation. Both the P and the S waves are body waves that move within the interior of the Earth. P-waves can propagate through both solid and liquid material, whereas S-waves only can propagate through solid material. The third type of waves is the surface waves, which can be either of Rayleigh or Love type. The surface waves propagate more slowly than the body waves.

The amplitudes of the seismic waves change as the waves travel through the Earth. This is due to the reflection and transmission coefficients at discontinuities/interfaces in the ground and at the ground surface, and to different impedance of different layers of material. Depending on the impedance ratios between the layers, the wave amplitudes will either be amplified or damped. For a seismic wave moving into a material with lower density and lower wave speed the amplitude will increase, making soft soils overlying bedrock very dangerous from a seismic perspective. The seismic signals are furthermore attenuated. That is, energy is lost as the waves propagate through the Earth as a result of geometrical dispersion of wave fronts (energy distributed on wider area/greater volume) and due to intrinsic attenuation (internal friction). For both P and S waves the attenuation increases with increasing frequency. The attenuation of S waves is however larger than for P waves. For more on this topic, see e.g. Bolt (2001), Shearer (2009) and Bodare (1997).

The seismic hazard in Sweden is primarily considered to be associated with near-field earthquakes, as is the case for many intraplate regions (SKI, 1992). The nuclear power plants are further founded on hard rock and the expected ground motions are dominated by high frequencies. The phenomenon of high amplitudes at high frequencies is not unique for Sweden. In McGuire et al. (2001) it is shown that hard rock sites in the central and eastern United States have higher amplitudes at high frequencies, above 10 Hz, and lower amplitudes at low frequencies compared to standard spectral shapes traditionally used, e.g. Regulatory Guide 1.60 spectra (USNRC, 1973). Atkinson et al. (2007) further show that typical spectra for moderate-seismicity hard rock sites in the eastern North America have higher amplitudes at high frequencies (above 10 Hz) and lower amplitudes at low frequencies, compared to the modified standard spectrum suggested for new nuclear power sites on hard rock (Elghohary et al., 2003).
Figure 3.1 Known earthquakes in Sweden and its vicinity from 1375 to 2005. The small circles have a radius of 100 km from Forsmark and Simpevarp (Oskarshamn), respectively. The big red and blue circles have a radius of 650 km from Forsmark and 500 km from Oskarshamn, respectively. From Bödvarsson et al. (2006).
Figure 3.2  Seismic waves: Longitudinal, shear and Rayleigh waves. Not shown in the figure is the surface Love wave, which movement essentially is the same as that of the shear wave. Redrawn from Dowding (1996).

3.2 Ground motions and response spectra

3.2.1  Time histories

Seismic ground motions are recorded by accelerometers, resulting in accelerograms, that is, time histories of the variation of acceleration, see e.g. Figure 4.1 in Section 4.6. The ground motion at a certain site depends on site geology, distance to fault, magnitude of earthquake, source characteristics, geology along the travel path of the seismic waves etc. Recorded time histories generally constitute one of the components in assessing the seismicity of a certain site and in the development of seismic design response spectra. Many time histories used in analysis of structures and components are however not recorded, but generated to be design spectrum compatible with regard to power spectral density functions. These time histories are often referred to as artificial or synthetic time histories. The terminology is not consistent and may vary from document to document. Synthetic is for example sometimes used about
time histories that are simulated using a seismological source model. Time histories have three important characteristics; amplitude, frequency content and duration.

### 3.2.2 Seismic response spectra

An earthquake response spectrum is a means of characterizing a ground motion with respect to structural response. It describes the maximum response (displacement, spectral velocity or spectral acceleration) of a damped single-degree-of-freedom oscillator due to a specific ground motion as a function of its natural frequency of vibration. A response spectrum has a jagged shape, see for example Figure 4.3 in Section 4.6. For the design of new structures, or for verification of existing structures, the jagged response spectrum is not an appropriate tool. For these purposes a smooth seismic design response spectrum is needed, see for example Figure 3.3. The seismic design response spectrum should be representative of both past recorded ground motions and expected future ground motions at the site. Many different seismic events are thereby covered by the design response spectrum. The development of a design response spectrum is generally based on statistical analysis of response spectra for a set of ground motions, either recorded at the site, or from a site with similar characteristics with respect to soil conditions at the site, epicentral distance to fault, magnitude of earthquake etc. The seismic design response spectrum is a specification of the level of seismic design force, or displacement, as a function of natural frequency and damping, see e.g. Chopra (2001) and Housner (1982).

Ground response spectra can be either site-independent or site-specific. The site-independent ground response spectra have a fixed shape and are generally defined for different damping ratios. The ground conditions and the magnitude of earthquake expected determine the applicability of the site-independent ground response spectra for a certain site. If the presupposed requirements are not met, site-specific ground response spectra have to be developed.

The lowest frequency at which the response acceleration is considered to be the same as the peak ground acceleration (PGA) is sometimes referred to as the lowest frequency of the rigid range of the response spectra or the frequency corresponding to the “Zero Period Acceleration” (ZPA). When site-independent spectra are scaled to a certain site PGA, this is the acceleration referred to.

The seismic design response spectra used for the nuclear power plants in Sweden are the US NRC Regulatory Guide 1.60 (USNRC, 1973) spectra and the spectra defined in SKI Technical Report 92:3 (SKI, 1992), see Figure 3.3.

**US NRC Regulatory Guide 1.60**

The design response spectra according to Regulatory Guide 1.60 (USNRC, 1973) are valid for sites underlain by rock or soil deposits and are defined for different levels of damping, with damping factors ranging from 0.5% to 10%. The control points are set at 0.25 Hz, 2.5 Hz, 9 Hz and 33 Hz and for natural frequencies of vibration above 33 Hz the response acceleration is considered to be the same as the impact acceleration. The response spectra are given for maximum horizontal ground acceleration of 1 g with corresponding maximum ground displacement of 36 inches (0.914m). The spectra are thereafter to be linearly scaled in proportion to the maximum ground acceleration relevant to the specific site. Amplification factors are given for the control points for the different levels of damping. The design re-
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Response spectra have been developed by analyzing, evaluating and statistically combining a number of response spectra of past significant earthquakes in the western United States. The spectra are not hazard consistent, that is, the annual probability of exceedance of the response spectra differs with natural frequency as opposed to the Swedish uniform hazard spectra (SKI, 1992). The vertical response spectrum is 2/3 of the horizontal spectrum for frequencies less than 0.25 Hz and for frequencies higher than 3.5 Hz it is unity. For frequencies between 0.25 Hz and 3.5 Hz it goes from 2/3 to unity.

**SKI Technical Report 92:3**

The response spectra given in SKI Technical Report 92:3 (SKI, 1992) are based on Japanese ground response spectra, modified to fit a Swedish typical hard rock site, in this case the Ringhals site. Site-specific spectra for the geological conditions at Barsebäck were also established. In the development of the spectra, the predominant statistical contribution comes from near-field earthquakes with hypocentral distances of approximately 10 to 30 km and of magnitude 5 to 6. The duration of the strong motion is corresponding to the duration of the shear wave face of this category of earthquakes. Spectra are given for three levels of occurrence frequencies, $10^{-5}$, $10^{-4}$ and $10^{-7}$ annual events, and are referred to as uniform hazard spectra. For each occurrence frequency, spectra are given for different damping ratios, ranging from 0.5% to 10%. For the spectra, the zero period acceleration frequency is 50 Hz. It is indicated that the qualification levels for the nuclear plants are set at $10^{-5}$ annual events (or once per 100,000 years) for safe shutdown and core cooling limit and $10^{-7}$ annual events (or once per 10 million years) for the containment integrity. The spectra in SKI (1992) have shown to be applicable also for the Forsmark and the Oskarshamn sites although the study summarized in SKI (1992) did not cover the site-specific ground motion characterizations for these sites. After the issuing of SKI (1992) additional studies were made regarding the site-specific rock conditions of the Swedish sites, resulting in a 15% reduction of the hard rock spectra (SKI, 1992) for the Ringhals and the Oskarshamn sites (VBB, 1995).

**Spectra used for the different nuclear reactors in Sweden**

This section gives a brief outline of how seismic concerns have been taken into account for the different reactors in Sweden and which seismic spectra that have been used. For more information, see e.g. SSM (2011).

Forsmark 1 and 2 were not originally designed for earthquakes, but verifications with regard to safe shutdown after an earthquake have been made using the Swedish earthquake (SKI, 1992) with a probability of $10^{-5}$ events per year. With reductions to account for site-specific conditions (VBB, 1995) this results in PGA of 0.11g and 0.09 g in the horizontal and vertical directions, respectively. For the seismic design of Forsmark 3 the spectra from Regulatory Guide 1.60 were used, scaled to 0.15 g and 0.10 g for horizontal and vertical PGA, respectively.

The two oldest of the three reactors at Oskarshamn have originally not been designed for earthquake-resistance. Oskarshamn 1 has been upgraded to withstand the Swedish earthquake (SKI, 1992) with probability of $10^{-5}$ events per year. Reductions to account for site-specific conditions (VBB, 1995) have been made here as well, resulting in PGA of 0.11g and 0.09 g in the horizontal and vertical directions, respectively. Oskarshamn 2 was to be verified to fulfill the same requirements at latest December 31, 2012. Oskarshamn 3 was originally
designed for the seismic design spectra according to Regulatory Guide 1.60, scaled to 0.15 g and 0.10 g for horizontal and vertical PGA, respectively.

None of the reactors at Ringhals has originally been designed for earthquake loads, but since the beginning of the 1990’s plant modifications important for reactor safety have been done considering the Swedish earthquake (SKI, 1992) of probability $10^{-5}$. After the stress tests the plants have been requalified for this condition, which was to be completed by the end of 2013.

Mitigation systems, to ensure pressure relief of the reactor containment and limited release of radioactive material in case of a severe accident, were installed at all plants during the 1980’s following a government decision. The mitigation systems were designed to withstand the seismic design response spectra according to Regulatory Guide 1.60, scaled to 0.15 g and 0.10 g for horizontal and vertical PGA, respectively.

The original design spectrum used in Sweden, Regulatory Guide 1.60 spectrum anchored at 0.15 g for the horizontal direction, is approximated to correspond to the probability range of $10^{-8}$ to $10^{-6}$ annual events per site in the natural frequency range 2–5 Hz (SKI, 1992).

![Design response spectra](image)

**Figure 3.3** Design response spectra according to Regulatory Guide 1.60, scaled to PGA 0.15g, and Swedish $10^{-5}$ design spectrum with PGA 0.11g, that is, reduced according to VBB (1995). For the horizontal direction and damping ratio 5%.

### 3.2.3 Earthquake levels

In several regulations and codes, two levels of earthquakes are considered; serviceability earthquake and design earthquake. The serviceability earthquake level is associated with operational requirements and the design earthquake level is often associated with safe shutdown requirements. Both IAEA (SSG-9) and US NRC (Appendix S 10CFR50) states that two levels of design basis earthquakes shall be evaluated for each plant, one corresponding to the safe shutdown earthquake (SSE) and one corresponding to operating basis earthquake (OBE). Given is also a lower peak ground acceleration limit for the design earthquake, being 0.1 g. In Sweden PGAs have been used that are less than 0.1g, when reduction according to VBB (1995) has been done. The regulations issued by SSM are very general and only states that the design of the plant shall be immune to such events or circumstances that may affect
the barriers or deep defense safety functions. In the general recommendations appended to the regulations examples of such events are given, where earthquake is one. Thus, it is not specified what earthquake levels that should be used. The industry standard DRB:2001 (DRB, 2001) only includes earthquake-induced loads in the accidental load combination. That is, a design earthquake level is used. SKI (1992) does not specify an earthquake level associated with operational requirements either. Other levels might however be defined in the Safety Analysis Report (SAR), which is the document containing the plant specific requirements.
4. Seismic analysis

This chapter presents some concepts and methods important in seismic analysis. First, the basics of seismic structural dynamics are presented. Thereafter, seismic analysis methods, modelling of damping, dynamic fluid-structure interaction and the seismic excitations are presented. Section 4.1 is a recapitulation of the corresponding section in Gasch et al. (2013).

4.1 Seismic structural dynamics

The basic equation of dynamic equilibrium for a damped linear elastic one-degree-of-freedom system is (see e.g. Chopra, 2001):

\[ f_i + f_d + f_s = 0 \]  \[ 4-1 \]

The first term is the \textit{inertia force}, the second the \textit{damping force}, and the third the \textit{spring force} (or \textit{restoring force}). The inertia force is related to the total acceleration \( \ddot{u}_t(t) \) of the mass, the damping force to the relative velocity \( \dot{u}(t) \) and the spring force to the relative displacement \( u(t) \), as given below:

\[ f_i = m\ddot{u}_t(t) \quad f_d = c\dot{u}(t) \quad f_s = ku(t) \]  \[ 4-2 \]

The total displacement of the mass, \( u_t(t) \), when subjected to seismic excitation, is the sum of the relative displacement between the mass and the ground, \( u(t) \), and the ground displacement, \( x_g(t) \):

\[ u_t(t) = u(t) + x_g(t) \]  \[ 4-3 \]

Substituting Eq. 4-2 and the second derivative of Eq. 4-3 in Eq. 4-1, and omitting the dependency on \( t \), gives:

\[ m(\dddot{u} + \ddot{x}_g) + c\ddot{u} + ku = 0 \]  \[ 4-4 \]

which can be rewritten as:

\[ m\dddot{u} + c\ddot{u} + ku = -m\dddot{x}_g \]  \[ 4-5 \]

Eq. 4-5 governs the response of a damped linear elastic one-degree-of-freedom system subjected to seismic excitation. The right hand side is \textit{the equivalent seismic force}. Consequently, the response of a damped linear elastic multi-degree-of-freedom system subjected to seismic excitation is governed by \textit{a system of differential equations}:

\[ m\dddot{u} + c\ddot{u} + ku = -m(1)\dddot{x}_g(t) \]  \[ 4-6 \]
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**m**  Mass matrix  
**ü**  Relative acceleration  
**c**  Viscous damping matrix  
**û**  Relative velocity  
**k**  Stiffness matrix  
**u**  Relative displacement  
{1}  Column vector of ones  
\( \ddot{x}_g(t) \)  Ground acceleration

The solution to the differential equations governs the relative displacements \( u(t) \) (absolute displacement minus ground displacement) wherefrom the stresses can be calculated. To note is that the forces acting on the system due to seismic excitation depend on both the seismic excitation itself and the response of the system/structure.

The nature of the problem (damping, linear/nonlinear etc.) will decide what method to use in solving for the displacements. Three methods are briefly described below; two based on modal superposition and one using direct time integration.

### 4.2 Mode superposition methods

Provided that the system is classically damped (damping matrix is diagonal) and its response is within its linear elastic range, the equations of motion can be transformed into a set of uncoupled modal equations. The response can thus be computed for each mode and then the contributions from all the modes can be combined (superposed) to give the total response. Here two procedures are presented; the modal dynamic time history method and the response spectrum method. In the first method, the seismic excitation is represented by a ground acceleration time history giving a time history response. The second method uses a response spectrum describing the seismic excitation, and with this method, only the peak response is obtained.

#### 4.2.1 Modal dynamic time history method

In the first step of the modal dynamic time history analysis the natural frequencies of free un-damped vibration are determined together with the corresponding mode shapes, which describe the shape of the displacements for each mode. The dynamic response is then evaluated by taking the mode-shape vector times the modal coordinate, describing the time variation of the displacements, and summing these modal displacements.

**Free vibration**

The natural frequencies of vibration, \( \omega_n \) \((n = 1, 2, \ldots)\), and mode shapes, \( \phi_n \) \((n = 1, 2, \ldots)\), for the un-damped linear elastic system are determined by studying the free vibration of the system (see e.g. Chopra, 2001). The equation of motion is thus:

\[
 m\ddot{u} + ku = 0 \quad [4-7]
\]
The free vibration of this system in one of its natural modes can be described by:

\[ u(t) = Y_n(t)\phi_n \]  \hspace{1cm} [4-8]

where \( Y_n(t) \) describes the time variation of the displacement (modal coordinates) and, assuming a simple harmonic motion, can be expressed as:

\[ Y_n(t) = A_n \cos \omega_n t + B_n \sin \omega_n t \]  \hspace{1cm} [4-9]

Substituting Eq. 4-8 and 4-9 and its second derivative in the equation of motion, Eq. 4-7, gives:

\[ [k - \omega_n^2 m]\phi_n = 0 \]  \hspace{1cm} [4-10]

which has nontrivial solutions if:

\[ \text{det}[k - \omega_n^2 m] = 0 \]  \hspace{1cm} [4-11]

By calculation of the determinant the natural frequencies of vibration are obtained, whereby the corresponding modes can be determined. The natural frequency of damped vibration, \( \omega_{dn} \), is related to the natural frequency of un-damped vibration, \( \omega_n \), through the damping ratio for each mode, \( \xi_n \):

\[ \omega_{dn} = \omega_n \sqrt{1 - \xi_n^2} \]  \hspace{1cm} [4-12]

**Response to dynamic loading**

The dynamic response \( u(t) \) of a damped linear elastic multi-degree-of-freedom system is obtained by the mode superposition technique and expressed as (see e.g. Chopra, 2001):

\[ u(t) = \sum_{n=1}^{N} \phi_n Y_n(t) \]  \hspace{1cm} [4-13]

Substituting Eq. 4-13 and its derivatives (for mode \( n \)) in the equation of motion, Eq. 4-6, gives the modal equation of motion:

\[ M_n \ddot{Y}_n(t) + C_n \dot{Y}_n(t) + K_n Y_n(t) = -\ddot{\phi}_n^T m(1) \ddot{x_g}(t) \]  \hspace{1cm} [4-14]

where:

\[ M_n = \phi_n^T m \phi_n \quad C_n = \phi_n^T c \phi_n \quad K_n = \phi_n^T k \phi_n \]  \hspace{1cm} [4-15]
The term \( \phi_n^T m(1) \) is referred to as the modal earthquake excitation factor \( L_n \). \( M_n, C_n, K_n \) and \( P_n(t) \) are called the generalized mass, generalized damping, generalized stiffness and generalized force for the \( n \)th mode. The generalized viscous damping, \( C_n \), is related to the damping ratio for each mode, \( \xi_n \), by:

\[
\xi_n = \frac{C_n}{2M_n\omega_n}
\]

where \( \omega_n \) is the \( n \)th natural frequency of vibration for the system without damping.

The modal coordinates \( y_n(t) \) are evaluated using a numerical method such as for example Newmark’s method. The total displacement response is then the summation of the responses for every mode according to Eq. 4-13.

### 4.2.2 Response spectrum method

In the response spectrum analysis the seismic excitation is described by a response spectrum, see e.g. Section 3.2.2. Just as for the modal dynamic time history analysis the natural frequencies of vibration and the corresponding mode shapes for the different natural modes of vibration have to be calculated in a first step. The maximum relative displacement in mode \( n \) is then given by:

\[
y_{n,\text{max}} = \phi_n L_n \frac{S_a(\xi_n, \omega_n)}{M_n\omega_n^2}
\]

where \( S_a(\xi_n, \omega_n) \) is the spectral acceleration given by the response spectrum. The maximum base shear is given by:

\[
v_{n,\text{max}} = \frac{L_n^2}{M_n} S_a(\xi_n, \omega_n)
\]

where \( L_n^2/M_n \) is the effective modal mass \( \Gamma_r \), which summed over all modes gives the total mass. To obtain the overall maximum of derived quantities, such as base shear, the square root of the sum of squares (SRSS) of all modal maximum values can be used. The SRSS is however not always the best to use and there are other methods to combine the modal contributions, see for example Regulatory Guide 1.92 (USNRC, 2006) for other methods.

### 4.3 Direct time integration method

The method of direct time integration can be used when the system has nonlinearities (e.g. material and geometrical nonlinearities) that have to be accounted for. This method uses a numerical time-stepping procedure to solve the coupled system of differential equations of motion. The response of the non-linear system, with \( f_s(u, \dot{u}) \) being the spring force dependent on both displacement and velocity (compare with Eq. 4-6), is governed by (see e.g. Chopra, 2001):
\[ \mathbf{m} \ddot{\mathbf{u}} + \mathbf{c} \dot{\mathbf{u}} + \mathbf{f}_s(\mathbf{u}, \dot{\mathbf{u}}) = -\mathbf{m} \{1\} \ddot{x}_g(t) \]  

[4-19]

with initial conditions at \( t = 0 \):

\[
\mathbf{u} = \mathbf{u}(0) \quad \text{and} \quad \dot{\mathbf{u}} = \dot{\mathbf{u}}(0)
\]

[4-20]

The ground motion is given as discrete values \( \ddot{x}_g(t_i) \) with time interval:

\[
\Delta t_i = t_{i+1} - t_i
\]

[4-21]

The procedure starts with the known responses \( \mathbf{u}_i, \dot{\mathbf{u}}_i \) and \( \ddot{\mathbf{u}}_i \) at time \( i \) satisfying:

\[
\mathbf{m} \ddot{\mathbf{u}}_i + \mathbf{c} \dot{\mathbf{u}}_i + \{\mathbf{f}_s\}_i = -\mathbf{m} \{1\} \ddot{x}_g(t_i)
\]

[4-22]

By the use of a numerical time-stepping procedure the responses \( \mathbf{u}_{i+1}, \dot{\mathbf{u}}_{i+1} \) and \( \ddot{\mathbf{u}}_{i+1} \) at time \( i + 1 \) can be calculated, the responses satisfying:

\[
\mathbf{m} \ddot{\mathbf{u}}_{i+1} + \mathbf{c} \dot{\mathbf{u}}_{i+1} + \{\mathbf{f}_s\}_{i+1} = -\mathbf{m} \{1\} \ddot{x}_g(t_{i+1})
\]

[4-23]

With \( i = 1, 2, 3, \ldots \) the responses at every time instance can then be calculated. To find the unknown responses \( \mathbf{u}_{i+1}, \dot{\mathbf{u}}_{i+1} \) and \( \ddot{\mathbf{u}}_{i+1} \) three matrix equations are needed. Two of these are derived either from finite difference equations for the velocity and acceleration vectors or from an assumption on how the response varies during a time step. The third equation is the equation of motion given above, Eq. 4-22 or 4-23, for a selected time instance. For the time instance being the current time \( i \) the method is called explicit and for the time being at the end of the step \( (i + 1) \) the method is called implicit.

Examples of numerical methods that are used are Central difference method, Newmark’s method, Average acceleration method and Wilson’s method. The method to be chosen depends on if the system is classically or non-classically damped and linear or non-linear.

The direct time integration method is used in both Paper I and Paper II, following an implicit time integration scheme.

### 4.4 Damping

An important parameter in seismic response analysis is the damping, which accounts for the dissipation of energy through different mechanisms. Explicitly modeling all the damping mechanisms of a structure is not feasible but the damping has to be idealized in dynamic analysis of real structures. Different theoretical models of damping exist and in structural dynamics, a viscous damping model is often employed. In mode superposition methods, the damping ratio for every mode can be specified, whereas for direct time integration methods the complete damping matrix has to be defined. A frequently implemented procedure to develop a classical damping matrix from specified modal damping ratios is the Rayleigh damping procedure. The Rayleigh damping is a mass and stiffness proportional damping and the damping ratio for the \( n \)th mode can be expressed:

\[ \eta_n = \frac{\alpha_n}{\omega_n} \]

where \( \alpha_n \) is the damping ratio of the \( n \)th mode and \( \omega_n \) is the circular frequency of the \( n \)th mode.
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\[ \xi_n = \frac{\alpha}{2\omega_n} + \frac{\beta \omega_n}{2} \]  \hspace{1cm} [4-24]

where \( \alpha \) and \( \beta \) are the Rayleigh damping factors, relating to the mass and the stiffness, respectively, and \( \omega_n = 2\pi f \) is the circular natural frequency of the \( n \)th mode, see e.g. Clough and Penzien (2003). As can be seen from Eq. 4-24 the damping ratio is a function of the natural frequencies and thus imposes different damping for the different natural frequencies of the system.

Damping values to be used in seismic analysis of structures at nuclear power facilities can be found in Regulatory Guide 1.61 (USNRC, 2007). This regulatory guide has been adopted also in Sweden and the recently issued guideline for design of structures within the Swedish nuclear power industry (SSM, 2014b) refers to this. The damping values are reproduced in Table 4.1 for the operational basis earthquake (OBE) and for the safe shutdown earthquake (SSE) analysis, respectively. Regulatory Guide 1.61 provides damping values also for piping. For water, the industry standard ASCE 4-98 (ASCE, 1998) suggests a damping ratio of 0.5%.

In Paper I, Rayleigh damping is used for the steel pipe, corresponding to approximately 3% of critical damping.

In Paper II, Rayleigh damping is used for the concrete structural parts and the coefficients \( \alpha \) and \( \beta \) are chosen to give structural response corresponding approximately to modal damping of 3%, corresponding to pre-stressed concrete for the lower stress-level (OBE). No damping is applied to the water.

Table 4.1 Damping ratio values from Regulatory Guide 1.61 (USNRC, 2007).

<table>
<thead>
<tr>
<th>Material type</th>
<th>Damping ratios for OBE</th>
<th>Damping ratios for SSE</th>
</tr>
</thead>
<tbody>
<tr>
<td>Welded steel and friction bolted steel</td>
<td>0.03</td>
<td>0.04</td>
</tr>
<tr>
<td>Bearing bolted steel</td>
<td>0.05</td>
<td>0.07</td>
</tr>
<tr>
<td>Reinforced concrete</td>
<td>0.04</td>
<td>0.07</td>
</tr>
<tr>
<td>Pre-stressed concrete</td>
<td>0.03</td>
<td>0.05</td>
</tr>
</tbody>
</table>

4.5 Fluid-structure interaction

The basic idea of dynamic fluid-structure interaction in a water-containing tank is that there are two components of hydrodynamic pressure; one that is related to the inertia force of the water produced by the impulsive movement of the walls, and one related to the oscillation of the fluid. The two components are referred to as the impulsive pressure term and the convective pressure term, respectively. The impulsive pressure term is directly proportional to the acceleration of the walls, and the convective term arises as a consequence of the impulsive pressure term. In the simplified analytical method by Housner (1963) the impulsive term is represented by a lumped fluid mass connected rigidly to the tank walls at a specific height, whereas the convective pressure term is represented by a lumped fluid mass connected with springs to the tank walls at a specific height.
One main difficulty in fluid-structure interaction analysis is to couple the two different domains; fluid and structure. The unknown field variables of the fluid domain are often fluid velocity and fluid pressure and of the structural domain displacements. Fluid-structure interaction can be accounted for in several different ways, ranging from rather simple analytical methods to highly complicated numerical methods relying on advanced software. Among the analytical methods can be mentioned Westergaard (1931), Housner (1954, 1957 and 1963), Epstein (1972), and Faltinsen (1978). More advanced numerical methods include Eulerian fluid methods, Lagrangian fluid methods and Arbitrary Lagrangian Eulerian (ALE) fluid methods. For a brief compilation of the methods see Paper III and for a more thorough compilation see Gasch et al. (2013).

In the benchmark example of a rectangular water-containing tank subjected to a base motion presented in Gasch et al (2013) and in Paper III, an evaluation of different FSI methods was done. From the results, it was concluded that the numerical methods with acoustic fluid elements, describe the influence of the fluid on the structural response, which is the main interest for the study in Paper II, sufficiently well. For the study reported in Paper II, an acoustic linear wave formulation was therefore chosen to account for the fluid-structure interaction effects. In the next section the basics of the linear acoustic wave formulation used are given.

4.5.1 Linear acoustic wave formulation

The theory of fluid dynamics constitutes the basis from which the equations of acoustic wave propagation are derived, with assumptions made regarding the acoustic medium, see e.g. Reynolds (1981). The linear acoustic wave formulation used in Paper II is expressed with the fluid pressure as the independent variable and is derived from the equations stating the conservation of momentum and the conservation of mass. Assuming that the fluid is homogeneous and isotropic, its viscosity is neglected, the flow is irrotational, the amplitude of the acoustic disturbance is small, and the wave propagation process is adiabatic and reversible, the equations of conservation of momentum and mass can be expressed:

\[
\rho_f \frac{\partial \mathbf{u}}{\partial t} + \nabla p = 0 \tag{4-25}
\]

and:

\[
\frac{\partial p}{\partial t} + K_f \nabla \mathbf{u} = 0 \tag{4-26}
\]

where \( \rho_f \) is the fluid density, \( \mathbf{u} \) is the fluid particle velocity, \( t \) is the time, \( p \) is the excess pressure in the fluid and \( K_f \) the bulk modulus of the fluid:

\[
K_f = \rho_f c_f^2 \tag{4-27}
\]

where \( c_f \) is the wave speed. The acoustic wave equation, with the fluid pressure as the independent variable, is obtained by combination of Eq. 4-25 and 4-26, giving:
\[
\frac{\partial^2 p}{\partial t^2} - c_f^2 \nabla^2 p = 0
\]

[4-28]

The natural frequencies and corresponding mode shapes are obtained from the eigenvalue problem, as defined in Abaqus (Dassault Systèmes, 2012):

\[
\begin{bmatrix}
K_s & S_{fs}^T \\
0 & K_f
\end{bmatrix}
\begin{bmatrix}
\mathbf{u} \\
\mathbf{p}
\end{bmatrix}
+ \omega^2
\begin{bmatrix}
M_s & 0 \\
-S_{fs} & M_f
\end{bmatrix}
\begin{bmatrix}
\mathbf{u} \\
\mathbf{p}
\end{bmatrix}
= 0
\]

[4-29]

where \(K_s, K_f, M_s\) and \(M_f\) are the stiffness and mass matrices of the structure and the fluid, respectively, \(\mathbf{u}\) and \(\mathbf{p}\) are the displacement and pressure vector, respectively, and \(\omega\) is the circular natural frequency of vibration. The coupling between the acoustic and the structural domains at the acoustic-structural interface is defined by the coupling matrix \(S_{fs}\). On this interface, the displacements of the structure and the fluid normal to the boundary are coupled, while the displacements tangential to the boundary are uncoupled.

### 4.6 Seismic excitation

As outlined in Section 3.2 the seismic excitation can be represented by either a set of time histories or a set of response spectra. The studies presented in Paper I and II, both use time histories since direct time integration analyses are employed. Response spectra are only presented to enhance the understanding of the dynamic characteristics of the input time histories, but are not used directly in the analyses.

**Paper I**

The seismic excitation of the piping system in Paper I is represented by displacement time histories applied at the supports of the system. For each support three time histories are applied simultaneously, one for each orthogonal direction. The time histories are obtained from a previously performed soil-structure interaction analysis of a nuclear power plant island for a design earthquake dominated by low frequencies and for an earthquake dominated by high frequencies.

**Paper II**

The seismic excitation of the elevated rectangular water-containing pool in Paper II is represented by acceleration time histories applied at the base of the supporting concrete structure. The analyses are performed with the time histories applied simultaneously for the three orthogonal directions.

Two sets of ground motion time histories representative of earthquakes dominated by relatively low frequencies and relatively high frequencies, respectively, are used in the analyses. The first set consists of records from the El Centro earthquake (Vibrationdata, 2014) that occurred in the Imperial Valley of California on May 18, 1940. The second set consists of time histories (CREA, 2007) generated to match the Swedish design response spectra with annual probability of \(10^{-5}\) events and damping of 5% (SKI, 1992). In the report documenting
the time histories for the Swedish design earthquake (CREA, 2007) a suite of acceleration
time histories are generated; three sets representing the three orthogonal directions, where
each set consists of five statistically independent time histories. The averaged spectrum of
each set of time histories matches the target design response spectrum. The time histories
used in Paper II are denoted H1-06, H2-06 and V1-06 in CREA (2007).

The El Centro time histories are 53.78 seconds long and represented by 2690 time points
with spacing of $\Delta t = 0.02$ seconds, see Figure 4.1. The Swedish time histories are 10 seconds
long and represented by 2001 time points with spacing of $\Delta t = 0.005$ seconds, see Figure 4.2.
Figure 4.3 shows the response spectra for the Swedish and the El Centro earthquake, respec-
tively, for damping ratio of 3%, generated from the used time histories. The figures are
shown as a complement to the figures presented in Paper II.
Figure 4.1  Time histories from the El Centro earthquake. Observe the different scales for the Swedish and the El Centro time histories.

Figure 4.2  Time histories from the Swedish earthquake. Observe the different scales for the Swedish and the El Centro time histories.
Figure 4.3  Response spectra for the Swedish and the El Centro earthquake, respectively, for damping ratio of three percent.
5. Summary of appended papers

Paper I: Piping system subjected to seismic hard rock high frequencies

Cecilia Rydell, Richard Malm and Anders Ansell

Submitted to Nuclear Engineering and Design in April 2014.

This paper investigates the influence of gaps in the piping supports on the response of a steel piping system subjected to a seismic load dominated by high amplitudes at high frequencies. The gaps are found in the joints of the strut supports or are gaps between the rigid box supports and the pipe. The piping system is assessed to be susceptible to high-frequency loads and is located within the reactor containment building of a nuclear power plant. Vertically the system spans 12.2 m, but the total length of the piping system is approximately 55 m. The system has 15 restraints and five large valves. A comparison between the responses due to the seismic high-frequency content load is made for a non-linear model including gaps and a linear model with rigid supports (i.e. without gaps). For reference, the responses are also compared to the response due to a seismic low-frequency content load for a linear model with rigid supports. For the high-frequency content load, the response for different support gap sizes is also investigated. It is concluded that considering the gaps in the analyses reduces the von Mises stress response for most of the pipe elements. It is also shown that the gaps do not have the same overall favorable effect on the valve acceleration response.

Paper II: Stresses in water filled pools within nuclear facilities subjected to seismic loads

Cecilia Rydell, Tobias Gasch, Daniel Eriksson and Anders Ansell

Submitted to Engineering Structures in April 2014.

This paper is a case study of the effect of fluid-structure interaction (FSI) on the response of an elevated rectangular water-containing concrete pool subjected to a seismic load. The pool is located within the reactor containment building of a boiling water reactor at a Swedish nuclear power plant. The hydrodynamic pressure distribution against the walls of the tank is evaluated together with the stress distribution in the walls of the tank. The structure is subjected to seismic loads with dominating low-frequency and dominating high-frequency content, respectively. The first being representative of an earthquake at the West coast of North America and the second of a design earthquake in Sweden (SKI, 1992). The effect of the seismic loads with different characteristics and the effect of additional cross-walls in the pool are investigated. It is shown that including the water content of the pool in the analyses generally lowers the natural frequencies of the vibration modes, but at the same time also lowers the effective masses of the modes. That is, the coupled fluid-structure system has more significant modes in the high-frequency range compared to a model without water, hence, for frequencies at which the Swedish earthquake has significant energy. It is also shown that the insertion of additional cross-walls increases the hydrodynamic pressure and the maximum principal stress distribution on the outside of the outer walls. It is concluded that the consideration of FSI is important also for seismic loads with dominating high-frequency content.
In this paper a brief review of some different methods to account for fluid-structure interaction (FSI) in seismic analyses of civil engineering structures at nuclear facilities is presented. The FSI methods range from simplified analytical methods to advanced numerical methods, including arbitrary Lagrangian-Eulerian (ALE) fluid methods, Eulerian fluid methods and Lagrangian fluid methods. Special interest is paid to the ALE fluid methods, to which the acoustic fluid description can be considered to belong. The methods are further evaluated in a benchmark example of a tank subjected to a periodic excitation at its base. The results from the analyses are verified against a performed shake table experiment documented in the literature. From the analyses, it can be concluded that the inclusion of FSI effects is essential in the evaluation of the structural response due to a periodic dynamic load. It is also shown that the simplified methods, where the hydrodynamic effects are accounted for by the introduction of a mass-spring system, give higher structural stresses than methods with the fluid described with continuum elements. On the basis that the focus is the assessment of the global response of the structure, many methods use unreasonably complex descriptions of the fluid domain. In many cases, methods using acoustic formulations for the fluid give acceptably accurate results.
6. Discussion

This chapter contains a discussion on some important aspects that are of concern for the studies covered in the thesis. The main focus is on seismic high frequencies, which was the concern of Paper I and Paper II, and fluid-structure interaction, which was the concern of Paper II and Paper III.

6.1 Practical use of regulations and standards

In the seismic analysis and design of structures within the nuclear power industry several laws, regulations, codes and standards have to be observed as outlined in Chapter 2. A major difference between USA and Sweden is that the legislations regarding the operation of nuclear facilities are very detailed and comprehensive in USA, whereas in Sweden they are more general and not very detailed. Regarding seismic analysis methods, the industry standard used to this day, DRB:2001 (DRB, 2001), and the recently issued design guideline (SSM, 2014b) both refer to the American industry standard ASCE 4-98 (ASCE, 1998). Regarding acceptance criteria and general requirements for earthquake-resistant design DRB:2001 primarily refers to Boverkets Konstruktionsregler BKR (BKR, 1998), whereas the new guideline primarily refers to Eurokod 2 (2008) and ASME Sect III Div 2 (ASME, 2010). Parts of ASME Sect III Div 2 has however been implemented in the past as well in the design of reactor containments. In Sweden, a consistent set of codes and standards that govern the seismic analysis and earthquake-resistant design for nuclear power facilities does not exist. Instead, the nuclear facilities have to rely on a mixture of codes and standards, which in itself implies that it can be difficult to get a clear picture of the actual safety margins and may increase the risk of making mistakes due to the human factor. To note is that all the plant-specific requirements are documented in the Safety Analysis Report (SAR). This document specifies all the laws, regulations and other requirements issued by SSM together with codes, standards and guidelines that are governing for the plant.

6.2 Seismic high frequencies

The studies reported in Paper I and Paper II both investigate the effect of seismic high frequencies; the first study being concerned with the response of non-structural components in the form of a piping system, the second with fluid-structure interaction effects in an elevated rectangular pool. Both the piping system and the elevated pool are located within a reactor containment building of a nuclear power plant. In Paper I the seismic high-frequency content load is representative of a central and eastern United States earthquake, whereas in Paper II the high-frequency content load corresponds to a Swedish design earthquake with annual probability of $10^{-5}$ events (SKI, 1992).

The earthquakes expected in Sweden are characterized by relatively high amplitudes at high frequencies and low amplitudes at low frequencies, characteristics that are similar to hard-rock sites of eastern North America although the amplitudes are different. The standard design response spectra that have been used in the design for many nuclear power plants in USA, and also in Sweden, is according to Regulatory Guide 1.60 (USNRC, 1973), which is based on a limited number of strong motion recordings at sites in California, USA. Observed seismic ground motions for eastern North American hard rock sites, for both small and intermediate earthquakes, have shown to have significant high-frequency content compared to
western North American rock ground motions of comparable magnitudes and distances (McGuire et al, 2001). It is also shown that spectral shapes of central and eastern United States rock sites have significant exceedances in the high frequency range, above 10 Hz, compared to western United States spectral shapes. Further, Atkinson and Elgohary (2007) developed a standard spectral shape for hard rock sites of moderate seismicity in eastern North America. The standard spectral shape exhibited higher amplitudes at high frequencies and lower amplitudes at low frequencies compared to the modified standard spectrum that has been proposed for future nuclear power plant rock sites in Canada.

The effect of seismic high frequencies on structures, systems and components (SSCs) within nuclear power facilities have been a topic for research during the last decades, and have been actualized with the release of the new seismic source characterization model for central and eastern United States for nuclear facilities (EPRI, 2012). The new seismic source characterization model together with the ground motion predictive equations (EPRI, 2004 and 2006a), result in ground motion response spectra with even higher amplification in the high frequency range than previously has been expected. In Paper I, the effect of this amplified ground motion was investigated.

There are several phenomena that may reduce the seismic high frequencies. One is the effect of nonlinearities, including material nonlinearities as well as geometrical nonlinearities. The study in Paper I focused on the gaps in piping supports, that is, geometrical nonlinearities, and its effect on the piping response when subjected to a seismic high-frequency load. The study showed that the support gaps decreased the stress response for most pipe elements. Other nonlinearities that could be included are cracking of the concrete in the primary supporting structure and the nonlinearity of the material of pipe supports and the pipe itself. However, in the finite element analysis in Paper I the stress in the pipe elements stayed within the linear elastic regime. The Electric Power Research Institute (EPRI) has issued reports on high-frequency seismic effects. EPRI (1993) suggested that the ground motion design spectra can be modified (reduced) to account for seismic high frequency nonlinear effects, hence, to be used for an equivalent linear model. EPRI (2005) investigated the effect of negligible inelastic behavior on the response due to high frequency motions. EPRI (2007a) presented both empirical and analytical evidence that short duration, high frequency excitations are non-damaging to power plant structures and equipment and suggested that a limited number of evaluations should be performed to support this statement for e.g. piping systems.

In Paper I the following studies on the influence of high-frequency content loads on piping response was referenced, Vayda (1981), Lockau et al. (1984), Steinwender et al. (1984) and Youtsos (1989a, 1989b). Apart from these studies, Yoshida (1989) performed a numerical study of a piping system on a rack structure and corresponding shake table test. It was concluded that the seismic response of both rack structure and piping system was reduced due to the gaps and the friction in the supports. A more recent study by Mohammed and Mekky (2012) investigated the effect of revised seismic spectra for eastern North America, with high amplification in the high-frequency range, on the response of non-structural components and piping. First the response of a two-degree-of-freedom system, representing the primary building structure and the supported secondary component, was evaluated. Secondly, a multi-supported piping system attached to a moment resisting frame structure was evaluated using finite-element analysis. The stress ratios of the piping system due to the revised seismic ground motion spectra, with high amplifications in the high frequency range, were shown to be slightly higher than the stress ratios obtained due to the design basis earthquake of standard spectral shape.
Another phenomenon that may reduce the seismic high frequencies is the effect of seismic wave incoherence, a spatial variation of ground motion affecting the input motion at the foundation of a structure. There are two sources of seismic wave incoherence; local wave scattering and wave passage effects. The scattering of waves is due to the heterogeneous nature of soil and rock along the propagation paths of seismic waves, and wave passage effects are due to differences in arrival times of the seismic waves over the foundation because of the inclination of waves. The effect of incoherence is greater for seismic ground motions with higher frequencies, and for larger foundations. The Electrical Power Research Institute (EPRI) has engaged in a research program on the effects of seismic wave incoherence and has issued comprehensive reports on the topic, see e.g. EPRI (2006b) and EPRI (2007b). Including the effect of seismic wave incoherence in the soil-structure analysis of nuclear power plant structures seems to be a widely adopted technique in the efforts to reduce the effect of seismic high frequencies and has gained acceptance by the US Nuclear Regulatory Commission, see the US NRC Interim Staff Guidance COL/DC-ISG-01 (USNRC, 2008).

Apart from the reports by EPRI, several recent studies investigate the effects of seismic wave incoherence on the response of nuclear power plant structures, e.g. Elkhoraibi et al. (2014), Tseng et al. (2014), Johnson et al. (2010), Ghiocel et al. (2009), and Xu and Samaddar (2009). Elkhoraibi et al. (2014) demonstrated the use of probabilistic and deterministic soil structure interaction analysis for a typical nuclear industry structure subjected to seismic low and high frequency loads, respectively. Analyses were performed with and without considering seismic ground motion incoherence effects. It was concluded that for rock sites subjected to seismic high-frequency content ground motions the incoherence effects are significant, reducing the in-structure response spectra in the higher frequency range for the studied structure. Tseng et al. (2014) investigated the effect of seismic wave incoherence on the response of the auxiliary control building of a nuclear power plant at a hard rock site in central and eastern United States. It was concluded that the effect of including incoherence in the soil-structure interaction analysis reduces the in-structure response spectra amplitudes in the high-frequency range, above 10 Hz, and this reduction increases with frequency. Johnson et al. (2010) studied the effect of seismic ground motion incoherence on in-structure response spectra for a nuclear power plant island founded on rock and subjected to a high frequency ground motion. It was concluded that it is very important also for hard rock sites subjected to seismic high-frequency ground motions that soil-structure interaction effects including incoherence is considered, since fixed base analysis may yield too conservative results. Ghiocel et al. (2009) studied the effect of seismic wave incoherence for the AP1000 nuclear island complex. It was concluded that the consideration of seismic wave incoherence gives significant effects on the response for rock as well as soil sites, but the reductions of the in-structure response spectra amplitudes were larger for the rock sites. Xu and Samaddar (2009) evaluated the response of a typical containment structure of a nuclear power plant with regard to the effect of seismic wave incoherence. It was shown that the responses in the high-frequency range, above 10 Hz, are reduced significantly, when incoherence effects are considered.

It seems that there is a general perception that the effect of seismic wave incoherence is important to consider for hard rock sites subjected to high frequency ground motions and do reduce the in-structure response spectra in the high-frequency range, above 10 Hz. This effect was however not considered for the studies reported in Paper I and Paper II. In Paper I only the effect of geometrical nonlinearities was considered for the piping system and the in-structure time histories were obtained from a previously performed coherent seismic soil-structure interaction analysis. As was reported in Ghiocel et al. (2009) the consideration of seismic wave incoherence do reduce the in-structure response spectra significantly in the
high-frequency range, above 10 Hz, for the AP1000 nuclear island complex for a hard rock site. Frequencies at which the piping system in Paper I has significant vibration modes.

Generally, high acceleration amplitudes in the high-frequency range are associated with small displacements, as opposed to high acceleration amplitudes in the low-frequency range that generally are associated with large displacements, see e.g. Lockau et al. (1984). Three published studies that report empirical findings after seismic high-frequency events were referenced in Paper I. Here a brief summary is given on the findings. The study by Chen et al (1988) is concerned with the Leroy, Ohio, magnitude 5 earthquake that occurred on January 31, 1986, in the vicinity of the Eastlake and the Perry power plants. Records of the earthquake were obtained at the Perry power plant 17 km from the epicenter. The records show high frequency, high acceleration, low velocity and small displacements and exceeded the design response spectra in the 20 Hz region. Plant inspections concluded that permanent plant structures and equipment were not damaged due to the earthquake. The US Nuclear Regulatory Commission (NRC) also evaluated the effects of the Leroy, Ohio earthquake on the Perry nuclear power plant (Bernero et al., 1988). They arrived at the conclusion that the plant had adequate safety margins to accommodate the recorded seismic motions although exceedances in the 20 Hz region. Whorton (1988) studied the effect of high frequency, high amplitude and low energy earthquake for the V.C Summer Nuclear Station in South Carolina, USA. In 1978 and 1979 reservoir induced earthquakes occurred due to the filling of the Monticello Reservoir close to the power plant. The calculated response spectra from the recorded motions exceeded the design response spectra in the greater than 10 Hz region. It was shown that adequate margin existed for equipment and components when subjected to these reservoir-induced earthquakes.

When it comes to assessing the damage potential of an earthquake the cumulative absolute velocity (CAV) parameter is sometimes employed. US NRC has issued Regulatory Guide 1.166 (USNRC, 1997), which specifies requirements for plant shutdown after a seismic event. The seismic event is quantified by a response spectrum check and a CAV check. The operational basis earthquake (OBE) is considered to be exceeded if the generated response spectra from the three components of the recorded free-field ground motion exceeds either the OBE spectral acceleration, or 0.2 g, whichever is greater, in the frequency interval 2–10 Hz, or the OBE spectral velocity, or 0.15 m/second, whichever is greater, in the frequency interval 1–2 Hz and the CAV exceeds 0.16g-seconds. Both requirements have to be met, or significant plant damage has to have occurred, for a plant shutdown to be required. Andersson et al. (2007) assessed the seismic risk in Sweden by evaluating the cumulative absolute velocity for the Swedish design earthquake (SKI, 1992) with annual probability of $10^{-5}$ events. The spectral acceleration exceeded 0.2 g in the frequency interval 2–10 Hz, but the spectral velocity in the frequency interval 1–2 Hz was well below 0.15 m/second and the calculated CAV-value was less than 0.16 g-seconds. It was thus concluded that the Swedish $10^{-5}$ design earthquake poses little risk of damaging Swedish nuclear power plants. It was also shown that the Swedish $10^{-5}$ design earthquake corresponds to a Regulatory Guide 1.60 earthquake scaled to peak ground acceleration of approximately 0.03–0.05 g with regard to damage potential (same CAV). The CAV measure might be of special interest for seismic high-frequency content loads, since these may have large acceleration amplitudes at high frequencies, but which are associated with low velocities and low displacements, and thus may be considered to be non-damaging for nuclear power plant structures. The Electric Power Research Institute (EPRI) has issued reports also on CAV, e.g. EPRI (1991) and EPRI (2006c). EPRI (1991) presented a standardization of the CAV measure. EPRI (2006c) developed a model for CAV to be used in determining the effects of small magnitude earthquakes on seismic hazard analyses for nuclear power plants.
6.3 Fluid-structure interaction, FSI

The phenomena of fluid-structure interaction for liquid-containing tanks have been extensively studied in the past, but much of the focus in research has been on cylindrical liquid storage tanks, especially when it comes to elevated tanks. In the published literature, studies on elevated rectangular concrete water-containing tanks, of the type studied in Paper II, are scarce. Further, the effect of seismic high-frequency content loads, with characteristics of the kind found in Sweden, on the dynamic response of this type of tanks does not seem to have been specifically addressed in the published literature.

The basis for the analytical methods to account for fluid-structure interaction in water tanks included in many codes and standards to this day is the analytical methods developed by Housner (1954), (1957) and (1963). Housner studied the dynamic behavior of water tanks, including elevated tanks, and developed a simplified method with lumped impulsive and convective fluid masses to account for the fluid-structure interaction. In Paper II, reference was given to some recent studies that deal with the dynamic behavior of rectangular liquid-containing tanks subjected to ground movements at their base and to studies that concern the seismic behavior of elevated liquid-containing tanks. Here these studies are briefly summarized.

Studies on tanks subjected to ground movements at their base, include, amongst other, work by Kianoush and Chen (2006), Ghaemmaghami and Kianoush (2010), Kianoush and Ghaemmaghami (2011), Hashemi et al. (2013), Livaoglu (2008) and Virella et al. (2008). Kianoush and Chen (2006) studied the response of concrete rectangular liquid storage tanks with respect to the effect of vertical ground acceleration at its base and concluded that it is of great importance to include the vertical component of ground motion for this type of tanks. Further, it was pointed out that it may be especially important in the case of near-field earthquakes. Ghaemmaghami and Kianoush (2010) investigated the effect of wall flexibility on the dynamic response of a ground-supported rectangular concrete liquid containing tank under seismic ground motion and showed that the impulsive terms of the base shear and the base moment are increased due to wall flexibility compared to rigid walls, whereas the convective terms are comparatively independent of wall flexibility. Kianoush and Ghaemmaghami (2011) studied the seismic response of concrete rectangular liquid tanks with respect to seismic input with different frequency content. The tanks, one shallow and one tall, were subjected to the ground movement at their base and soil-structure interaction was accounted for. For the shallow tank, it was concluded that the earthquake with a high-frequency content results in the highest impulsive response, presuming a rigid foundation, while the smallest response was obtained for a low-frequency content earthquake. Earthquakes with intermediate-frequency content increased the response of the tall tank most. The parameters evaluated were base shear, base moment and sloshing height. Hashemi et al. (2013) proposed an analytical method to determine the dynamic behavior of a partially filled ground-supported flexible rectangular container subjected to horizontal seismic ground motion. It was noted that the significant convective (sloshing) natural frequencies are much lower than the significant impulsive frequencies for a concrete liquid storage tank, and it was concluded that the convective and the impulsive terms of the response therefore could be considered independent of each other. It was also concluded that the hydrodynamic pressure at the middle of the wall is larger for a flexible tank than for a rigid tank. Livaoglu (2008) investigated the dynamic behavior of a rectangular ground-supported tank when subjected to two different earthquakes considering soil-structure interaction. The analyses were performed for tanks with two different wall thicknesses, one considered flexible and one rigid, for different soil conditions, and with and without embedment. The effects of FSI were considered using the two-mass
approximation by Housner. Virella et al. (2008) investigated the sloshing modes and their contribution to the pressure distribution in 2D rectangular tanks, considering linear and non-linear wave theory. Ground-supported tanks with different liquid height to tank width ratios were analyzed. To find the natural sloshing periods and mode shapes of the fluid-structure system the response to horizontal harmonic ground motions of varying loading frequency was calculated using time integration analysis methods. From the comparative study it was concluded, that considering nonlinear wave theory does not have significant effects on the pressure distribution on the walls of the rectangular tanks compared to linear wave theory.

Studies on elevated tanks include, amongst others, Livaoglu and Dogangün (2006), Dutta et al. (2009), Moslemi et al. (2011) and Shakib and Omidinasab (2009). Livaoglu and Dogangün (2006) reviewed simplified seismic analysis methods for elevated tanks considering soil-structure interaction. The applicability of the different methods was evaluated by analyses of ten different models with increasing complexity. The FSI was considered by either a two-mass analytical model or a finite element model with a lumped or added mass approach. It was noted that the seismic behavior of elevated tanks is dominated by the impulsive mode of vibration. Dutta et al. (2009) studied the dynamic behavior of cylindrical reinforced concrete elevated water tanks with shaft and frame staging, respectively, with the main focus being the effect of soil-structure interaction. Moslemi et al. (2011) investigated the seismic response of a liquid-filled conical shaped elevated thin-walled tank with shaft staging. Finite element analyses were performed and the liquid was modeled with displacement-based fluid elements. The convective and the impulsive parts of the response were attained separately and it was shown that the convective terms of the base shear and the base moment are relatively small compared to the impulsive terms. It was further noted that their peak values do not occur simultaneously, wherefore the consideration of sloshing either might decrease or increase the total response. Shakib and Omidinasab (2009) studied the seismic performance of an elevated circular concrete water storage tank with regard to seismic input of different characteristics. It was concluded that the seismic load with predominantly low-frequency content excites the first convective mode, resulting in large vertical displacements of the free water surface, while the seismic loads with predominantly high-frequency content excites the first impulsive mode, causing high base shear forces, overturning moments and displacements of the tank roof and floor.

The choice of modelling the fluid-structure interaction using a linear acoustic wave formulation for the fluid media was based on the findings in Gasch et al. (2013). The study concluded that for the purpose of global response evaluation of a structure of the type studied in Paper II, many numerical methods use an unnecessarily complex description of the fluid domain. The method of using an acoustic description of the fluid was concluded to give sufficiently good results. The acoustic method is further relatively simple to implement and the resolution of the fluid domain can be kept rather low compared to more advanced methods, leading to relatively short calculation times.

The acoustic wave formulation in Abaqus 6.12-1 (Dassault Systèmes, 2012) has drawbacks however, in that it does not include body forces. That is, the hydrostatic pressure has to be manually applied as a pressure load on the fluid-structure interface. This implies that even if surface gravity waves (sloshing effects) are accounted for, the change of the hydrostatic pressure distribution will not automatically change due to a change in water level height. In other finite element software, other acoustic wave formulations, including body forces, do exist. To note is that the inclusion of body forces only is important when dealing with fluids with significant density, as, for example, water.
The study in Paper II only includes the impulsive components of the hydrodynamic pressure against the walls of the elevated pool. The surface gravity waves, the convective components of the hydrodynamic pressure, are not accounted for, but a free surface is assumed with a zero pressure boundary condition. Previous studies, for example Livaoglu and Dogangün (2006) and Moslemi et al. (2011), showed that the overall dynamic behavior of elevated tanks was governed mainly by the impulsive modes of vibration. The hydrodynamic pressure distribution against the tank walls was however not evaluated in these studies but only the base shear and the base moment of the fluid-structure system.

When using a linear acoustic wave formulation for the fluid in a dynamic fluid-structure interaction problem of a rectangular tank the effect of gravity surface waves (sloshing) can be accounted for by the implementation of another boundary condition for the free surface of the fluid, see e.g. Virella et al. (2008) and Kianoush and Ghaemmaghami (2011). Considering sloshing using linear wave theory (acoustic wave formulation) gives a good representation of the pressure distribution on the walls of the tank and the nonlinearity of the surface waves does not give significant effects on the pressure distribution against the walls (Virella et al., 2008). To note is that the consideration of the nonlinearity of surface waves require the use of nonlinear wave theory and thus acoustic elements cannot be used to model the fluid.

As a complement to the figures of accumulated effective mass and the table with the ten first natural frequencies in Paper II, the ten first vibration modes of the elevated tank are here given in Figure 6.1 and 6.2 for the model with included partition walls, with and without water. The first two modes (mode 1 and 2) are the fundamental vibration modes in the length- and width-direction, respectively, of the elevated pool system and activate significant parts of the effective masses in each direction. The third mode is a pool torsional mode activating only an insignificant part of the total mass. Starting from the fourth mode, the vibration modes change when water is included. For the coupled fluid-structure system, the fourth mode is the first mode with a significant acoustic component. The next mode with a significant acoustic component is the seventh mode. One of the more important modes in accurately representing the hydrodynamic pressure against the walls is mode number 17, with a vibrational motion somewhat similar to mode number 7, and with a natural frequency of 24.5 Hz. As was noted in Paper II the inclusion of water generally lowers the natural frequencies, but also lowers the effective masses for the modes in the lower frequency range. Hence, the response of the coupled fluid-structure system depends more significantly on modes in the high-frequency range. It was also shown in Paper II that the accumulated acoustic effective masses rises faster when partition walls are added compared to a model without partition walls. Hence, modes with lower, although still relatively high, frequencies become more important to the hydrodynamic pressure response. The effect was seen when comparing the hydrodynamic pressure response against the outer long pool wall for a model with included partition walls to a model without, for the two types of earthquakes. The stiffening of the long wall due to inclusion of partition walls resulted in an increase of the hydrodynamic pressure. The result may seem to be in conflict with previous studies investigating the effect of flexible tank walls compared to rigid tank walls, see e.g. Ghaemmaghami and Kianoush (2010) and Hashemi et al. (2013). The first study concluded that flexible walls result in increased base shear and base moment compared to rigid walls. In the second study, it was concluded that the hydrodynamic pressure in the middle of the wall increases due to the flexibility of the wall. In these studies, comparisons are made of a flexible wall to a completely rigid wall. This is not the case in Paper II, where the walls of both models are flexible, although rather rigid considering the thickness of the walls. Stiffening the long walls of the pool by inserting additional partition walls does not make the wall rigid in that sense, but changes the dynamic properties of the fluid-structure system. That is, dynamic pressure amplification
occurs at the middle of the walls for both models, see e.g. modes number 4 and 7 for the coupled fluid-structure model with included partition walls.

![Eigenmodes 1–5 for structure only (a) and coupled fluid-structure system (b).](image)

**Figure 6.1** Eigenmodes 1–5 for structure only (a) and coupled fluid-structure system (b). The fluid elements are not shown for the coupled fluid-structure system.
Figure 6.2  Eigenmodes 6–10 for structure only (a) and coupled fluid-structure system (b). The fluid elements are not shown for the coupled fluid-structure system.
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7. Conclusions and further work

From the two studies in Paper I and Paper II, it is evident that the response due to a seismic load dominated by high frequencies is different to the response due to a seismic load with low-frequency content. Although the seismic high-frequency load may be considered non-damaging for the structure, the effect may not be negligible for non-structural components attached to the primary structure. It may also be that the inclusion of fluid-structure interaction effects changes the dynamic properties of a structural system so that it gets significant response in the high frequency range.

In the study on the effect of gaps in the supports of a piping system, Paper I, it was shown that the gaps had a positive effect on the stress response for most pipe elements when the system was subjected to a seismic high-frequency load, hence, the stresses were decreased due to the gaps. However, considering the geometrical nonlinearity of gaps in isolation does not qualify the piping system, that is, the obtained stresses are still higher for most elements when subjected to the seismic high-frequency load compared to the response when subjected to the AP1000 Certified Seismic Design Response Spectra. To note is that the evaluated stress parameter in the study was the von Mises stress, since this is a frequently used stress measure for steel. Typically, the moment stress according to ASME BPVC Section III (ASME, 2010) is used for piping within nuclear power facilities. For the stress response to further decrease, other phenomena would also need to be accounted for, e.g. seismic wave incoherence as discussed in Section 6.2.

In Paper I only one specific piping system was evaluated for a specific seismic load. To be able to draw more general conclusions on the outcome more piping systems would need to be investigated and preferably subjected to seismic loads dominated by high frequencies but with slightly different characteristics. Future works related to the high-frequency response of piping systems could also include the evaluation of possible new failure and qualification criteria when the seismic load is dominated by high frequencies.

In the study on fluid-structure interaction, Paper II, it was shown that the relative dynamic stress increase when including the water of the pool was larger for the Swedish high-frequency content seismic load than for the low-frequency content load. Thus, it is indicated that it might be even more important to consider fluid-structure interaction for a seismic high-frequency content load than for a low-frequency content load. Albeit the seismic load dominated by high frequencies may not significantly affect the structure itself when water is not included, the stress increase as water is included may be substantial. Even for a seismic load with small amplitudes and short duration as the Swedish 10⁻⁵ design earthquake, which supported by the CAV value would be non-damaging to nuclear power plant structures, it gives a significant increase of the dynamic response when fluid-structure is accounted for.

The study in Paper II, as noted in Section 6.3, only includes the impulsive term of the hydrodynamic pressure against the walls of the elevated pool. The effect of surface gravity waves (convective pressure term) is not accounted for. Previous studies on elevated tanks have shown that the overall dynamic behavior, base shear and base moment, is governed by the impulsive term. Future works would need to investigate if this is true also for the hydrodynamic pressure distribution against the walls of the elevated rectangular tank. It would also be of interest to investigate if the effect of including the convective term is different for a seismic low- respectively high-frequency content load.
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Bibliography


ASCE (1998). “Seismic analysis of safety-related nuclear structures and commentary”. ASCE Standard 4-98. American Society of Civil Engineers (ASCE), Reston, USA.


Seismic high-frequency content loads on structures and components within nuclear facilities


Dassault Systèmes (2012). “Abaqus 6.12 online documentation”, Dassault Systèmes Simulia Corp, Providence, RI, USA.


EPRI (1993). “Analysis of high-frequency seismic effects”, Electric Power research Institute (EPRI), Palo Alto, California, USA.

EPRI (2004). “CEUS ground motion project final report”, Electric Power Research Institute (EPRI), Palo Alto, California, USA.


EPRI (2006a). “Program on Technology Innovation: Truncation of the lognormal distribution and value of the standard deviation for ground motion models in the Central and Eastern United States”, Electric Power Research Institute (EPRI), Palo Alto, California, USA.


EPRI (2006c). “Program on Technology Innovation: Use of cumulative absolute velocity (CAV) in determining effects of small magnitude earthquakes on seismic hazard analyses”, Electric Power research Institute (EPRI), Palo Alto, California, USA.

EPRI (2007a). “Program on Technology Innovation: The effects of high-frequency ground motion on structures, components, and equipment in nuclear power plants”, Electric Power research Institute (EPRI), Palo Alto, California, USA.

EPRI (2007b). “Program on Technology Innovation: Effects of spatial incoherence on seismic ground motions”, Electric Power research Institute (EPRI), Palo Alto, California, USA.


Housner, G.W., Jennings, P.C. (1982). “Earthquake design criteria”, Earthquake Engineering Research Institute (EERI), Berkeley, California, USA.


Seismic high-frequency content loads on structures and components within nuclear facilities


SSM (2012). ”Sammanfattning av säkerhetsutvärderingar (stresstester) av svenska kärn tekniska anläggningar”, Strålsäkerhetsmyndigheten (SSM). (In Swedish)


*I Reference found only in Paper I.
*II Reference found only in Paper II.
*III Reference found only in Paper III.
Appended papers
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