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IAEA-TECDOC-1990

## **Methodologies for Seismic Soil–Structure Interaction Analysis in the Design and Assessment of Nuclear Installations**



**IAEA**

International Atomic Energy Agency

METHODOLOGIES FOR  
SEISMIC SOIL–STRUCTURE  
INTERACTION ANALYSIS IN THE  
DESIGN AND ASSESSMENT OF  
NUCLEAR INSTALLATIONS

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METHODOLOGIES FOR  
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INTERNATIONAL ATOMIC ENERGY AGENCY  
VIENNA, 2022

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For further information on this publication, please contact:

External Events Safety Section  
International Atomic Energy Agency  
Vienna International Centre  
PO Box 100  
1400 Vienna, Austria  
Email: [Official.Mail@iaea.org](mailto:Official.Mail@iaea.org)

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## FOREWORD

How the structure of a nuclear installation responds to an earthquake depends on the characteristics of the ground motion, the surrounding soil and the structure itself. The characteristics of free field ground motion can differ depending on the soil or rock present. The foundation motion can also differ from the free field motion with respect to the subsurface conditions due to soil structure interaction (SSI). As such, the foundation motion depends on the type of soil or rock and SSI effects.

SSI analysis is used in nuclear installation design and assessment to evaluate the effects of seismic ground motion on nuclear installation structures, systems and components important to safety. This analysis ensures that they are designed to withstand the effects of earthquakes without losing the capability to perform their safety functions.

In recent years significant experience has been gained with regard to the effects of earthquakes on nuclear installations worldwide. The SSI effects are evident, with significantly reduced motions from top of grade to basement level and modified motions in the structure accompanied by shifts to lower frequencies. The motions measured at nuclear installations demonstrate the important aspects of spatial variation of free field ground motion and SSI behaviour.

This publication provides detailed information on SSI phenomena and the current application of SSI modelling, simulation methodology and analysis methods for nuclear installations. It presents information on the selection and use of available SSI methodologies for the design and assessment of nuclear installations, examples of which are provided in the supplementary file available on-line.

This publication supports IAEA Safety Standards Series No. SSG-67, Seismic Design of Nuclear Installations, and No. NS-G-2.13, Evaluation of Seismic Safety for Existing Nuclear Installations.

The IAEA would like to express its appreciation to the contributors to the development and review of this publication. In particular, the IAEA gratefully acknowledges the contributions of A. Pecker (France), B. Jeremic and J.J. Johnson (United States of America) and N. Orbovic (Canada). The IAEA officer responsible for this publication was A. Altinyollar of the Division of Nuclear Installation Safety.

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# 1. INTRODUCTION

## 1.1. BACKGROUND

The response of a nuclear installation structure during an earthquake depends on the characteristics of the ground motion, the surrounding soil, and the structure itself. In general, the characteristics of free field ground motion on different soils and rock are different, and secondly the foundation motion differs from the free field motion with respect to the subsurface condition due to soil–structure interaction (SSI). As such, the foundation motion depends on the type of soil and/or rock and SSI effects.

The objective of SSI analysis is to calculate the seismic demand on individual structures, systems and components (SSCs) of nuclear installations for design and assessment purposes. To achieve this objective, the site is evaluated, the ground motion associated with the design basis earthquake (DBE) is defined and applied to the SSI model, and the seismic input is obtained at different floor levels.

A site is defined as soft soil, stiff soil, soft rock, hard rock, etc. and is generally classified on the basis of  $V_{s30}$  values, which represent the shear wave velocity of the top 30 metres of the ground. The higher the  $V_{s30}$  value, the harder is the soil/rock. It is widely used in ground motions predict equations and building codes.

For massive foundations and massive structures founded on soil and rock, with foundations on the surface, shallowly embedded, or deeply embedded, the foundation motion differs from that in the free field due to the coupling of the soil/rock and structure during the earthquake. This SSI results from the scattering of waves from the foundation (kinematic interaction) and the radiation of energy from the structure due to structural vibrations (inertial interaction). Because of these effects, the state of deformation (particle displacements, velocities, and accelerations) in the supporting soil/rock is different from that in the free field. As a result, the dynamic response of a structure supported on soft soil may differ substantially in amplitude and frequency content from the response of an identical structure supported on a very stiff soil or rock. The coupled soil–structure system exhibits a peak structural response at a lower frequency than would an identical rigidly supported structure. Also, the amplitude of structural response is affected by the additional energy dissipation introduced into the system through radiation damping and material damping in the soil and contacts. On stiff soil or on rock sites, the response of rigid nuclear structures will be higher due to lower damping and the peak will shift to the higher frequency side.

For engineering purposes, for light surface structures founded on rock or very stiff soils and subjected to ground motion with frequency characteristics in the low frequency range (i.e. in the frequency range of 1 Hz to 10 Hz), the foundation motion and the ground motion in the free field can be assumed to be the same; this means that the SSI is not an important phenomenon for such cases. In these cases, a fixed-base analysis can be justified. However, care is needed especially for structures subjected to ground motion with high frequency content, i.e. greater than about 20 Hz.

The input ground motions are acceleration time histories in the two horizontal directions and the vertical direction. Figure 1 shows an example of horizontal in-structure response spectra at the top of a typical nuclear power plant structure calculated assuming the structure is founded on four different site conditions ranging from rock ( $V_s = 1830$  m/s) to soft soil ( $V_s = 152$  m/s). Figure 1 clearly demonstrates the importance of SSI in the dynamic response of the structure

founded on different subsurface conditions as different seismic load (in-structure response spectra) is imparted to equipment, components, distribution systems, and supporting substructures.

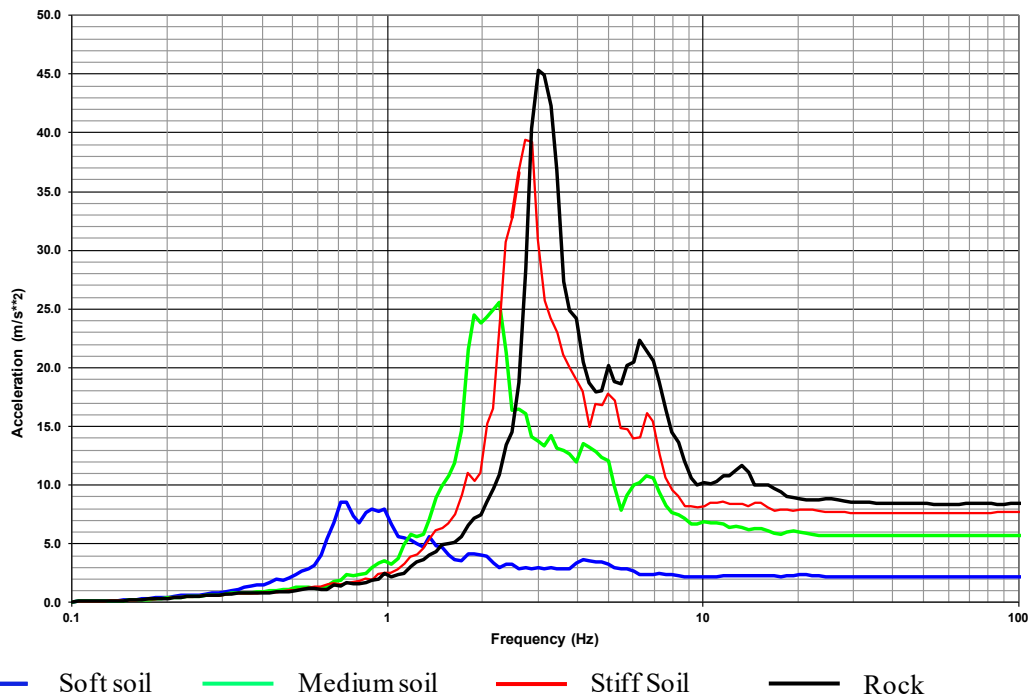


FIG. 1. Effect of soil stiffness on structure response of a typical nuclear power plant structure; response spectra at the top of shield building, horizontal direction; rock ( $V_s = 1830\text{ m/s}$ ), stiff soil ( $V_s = 762\text{ m/s}$ ), medium soil ( $V_s = 305\text{ m/s}$ ), and soft soil ( $V_s = 152\text{ m/s}$ ) (reproduced with permission courtesy of [James J. Johnson and Associates]).

This publication supports the recommendations provided in IAEA Safety Standard Series Nos SSG-67, Seismic Design of Nuclear Installations, and NS-G-2.13, Evaluation of Seismic Safety for Existing Nuclear Installations.

## 1.2. OBJECTIVE

The objective of the publication is to present a detailed treatise on SSI phenomena and the state-of-the-practice on SSI modelling, simulation methodology and analysis methods for nuclear installation<sup>1</sup>. It provides information on the selection and use of the available SSI methodologies for the design and assessment of nuclear installation structures.

This publication is intended for use by SSI analysis practitioners and reviewers. It is also intended for use by regulatory bodies responsible for establishing regulatory requirements, and by operating organizations directly responsible for the execution of the seismic safety assessments and upgrading programs.

## 1.3. SCOPE

The publication provides practical information to:

- (a) Describe the physical aspects of site, structure, and earthquake ground motion that lead to important SSI effects on the behaviour of SSCs;

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<sup>1</sup> Nuclear installation [1] is any nuclear facility subject to authorization that is part of the nuclear fuel cycle, except facilities for the mining or processing of uranium ores or thorium ores and disposal facilities for radioactive waste.

- (b) Describe the modelling of elements of SSI analysis that are relevant to calculating the behaviour of SSCs subjected to earthquake ground motion;
- (c) Identify, and if feasible quantify, the uncertainty associated with the elements of SSI analysis;
- (d) Review the state-of-practice for SSI analysis as a function of the site, structures, and ground motion definition of interest. Other important considerations are the purposes of the analyses, i.e., design and/or assessment;
- (e) Provide guidance on the selection and use of available SSI analysis methodologies for design and assessment purposes;
- (f) Identify sensitivity studies to be performed on a generic basis and a site specific basis to aid in decision-making (remove decision making as it concerns to all items here);
- (g) Provide a future view of the SSI analysis field;
- (h) Document the observations, recommendations, and conclusions of the SSI analysis.

This publication addresses a range of types of nuclear installation. It was originally derived for nuclear power plants (NPPs), and it can be used for other nuclear installations as applicable.

In the context of nuclear installation design and assessment:

- (a) Design includes new NPPs design, such as reference designs or certified designs of NPPs and the design and qualification of modifications, replacements and upgrades. to an existing facility.
- (b) Assessments encompass evaluations for beyond design basis earthquake (BDBE)<sup>2</sup> ground motions (typically seismic margin assessments (SMAs) or seismic probabilistic risk assessments (SPRAs) for new nuclear installations and existing nuclear installations. The results of assessments may lead to design changes or modifications.

#### 1.4. STRUCTURE

Section 2 presents design consideration and country practices. Section 3 presents an overview of the elements of SSI and refers to other sections in this publication for more in-depth and complete discussion. Section 4 gathers site configuration and soil properties, including an experimental description of soil behaviours, the iterative linear model, the physical parameters, calibration and validation of the soil constitutive models, the uncertainties to be considered and the spatial variability. Section 5 provides an overview of seismic hazard analysis (SHA) and the interfaces between the SHA team and the SSI analysis team. Section 6 summarizes the seismic wave fields and free field ground motions. It describes the recorded data of earthquakes motions and at the end of the section the seismic wave incoherence is also presented. Section 7 provides an overview of the site response analysis and seismic input, including the approaches, standards and site specific response spectra, as well as the time histories, uncertainties and limitations of time and frequency domain methods for free field ground motions. Section 8 introduces the methods and models for SSI analysis, and addresses first the basic steps for SSI analysis, followed by the direct methods including substructuring methods. Section 8 also introduces several SSI computational models and provides an overview of the probabilistic response analysis. Section 9 presents the seismic response aspects for design and assessment. It presents an overview of all features of the elements of the soil structure interaction analysis for nuclear installations. It draws extensively from the information presented in Sections 1–8 and includes additional relevant information/elaborations. Section 10 presents examples of available software focusing on the support required, training and quality assurance.

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<sup>2</sup> In addition to the two earthquake levels SL-1 and SL-2, defined and determined for design purposes, a more severe earthquake level exceeding the ones considered for design, derived from the hazard evaluation of the site, needs to be considered. For this earthquake level, referred to as the beyond design basis earthquake [2].

Annexes provide examples as a supplementary file for this publication and can be found on the publication's individual web page at [www.iaea.org/publications](http://www.iaea.org/publications). They are organized in 7 Annexes. Annex I presents seismic wave incoherence: a case study. Annex II provides an example of site response analysis. Annex III presents analysis of a pile foundation (for a bridge) by the substructure method. Annex IV provides examples of seismic response of an NPP on nonlinear soil and contact (slip and uplift). Annex V covers nonlinear analysis of a deeply embedded small modular reactor (SMR). Annex VI presents nonlinear time domain, 3-D, earthquake soil-structure interaction (ESSI) analysis of NPP, analysis procedures. Annex VII describes equivalent bulk modulus for unsaturated soil.

## **2. EVOLUTION OF SOIL-STRUCTURE INTERACTION ANALYSIS, DESIGN CONSIDERATIONS AND COUNTRY PRACTICES**

### **2.1. EVOLUTION OF SOIL-STRUCTURE INTERACTION ANALYSIS**

Soil Structure Interaction analysis originated in late 19th century and evolved and matured gradually during the first half of 20<sup>th</sup> century [3]. The theory of dynamic SSI began in 1936 with a publication by Reissner [4]. It then progressed rapidly, stimulated mainly by the needs of the nuclear power and offshore industries, by the debut of powerful computers and simulation tools such as finite elements, and by the needs for improvements in seismic safety.

Initially in the 1960s and early 1970s, SSI was treated with tools developed for calculating the effects of machine vibrations on their foundations, the supporting media, and the machine itself. Foundations were modelled as rigid disks of circular or rectangular shape. Generally, soils were modelled as uniform linear elastic half-spaces. Soil springs were developed from continuum mechanics principles and damping was modelled with dashpots. This approach addressed inertial effects only.

The beginning of the modern era in SSI analysis can be said to have begun with the publication of papers [5–7], which provided complete rigorous solutions to the problem of circular plates underlain by elastic half-spaces excited dynamically over a broad range of frequencies, and for a wide set of Poisson's ratios.

The decades of the mid 1960's to mid-1970's were also marked by the introduction of powerful digital computers together with versatile numerical methods — especially finite elements — both of which helped to radically change the research paradigm and shift its emphasis away from purely analytical methods. Thus, instead of continuing to solve highly idealized mathematical problems involving, for example, rigid circular disks welded onto perfectly homogeneous half-spaces, it became possible to address irregularly shaped, flexible foundations embedded in inhomogeneous or layered media, and account for complex effects such as the inelasticity of the soil. This was also the time when sophisticated computer programs such as SHAKE, LUSH, SASSI, and CLASSI appeared and — at least in the nuclear power industry — were viewed as the main instruments by which one could solve nearly any practical SSI problem.

In 1974, Kausel and Roesset [8] focused on key aspects of SSI and clarified the source of inconsistencies observed by alternate methods. This motivated the development of the so-called three-step solution, which provided the means to accomplish fully consistent comparisons between the results obtained by purely numerical models with finite elements and those by the lumped parameter method based on foundation impedances or “springs” together with seismic

motions prescribed underneath these springs; kinematic interaction, foundation stiffness and inertial interaction. These concepts are referred in [9].

During this period, nonlinear analyses of soil/rock media subjected to explosive loading conditions led to alternative calculation methods focused on nonlinear material behaviour and short duration, high amplitude loading conditions. Adaptation of these methods to seismic analysis was attempted with mixed results.

As one element of the U.S. Nuclear Regulatory Commission's (NRC) sponsored Seismic Safety Margin Research Program (SSMRP), the state of knowledge of SSI in 1980 was documented [10] and provided a framework for SSI over the 1980s and 1990s.

Soil Structure Interaction analysis methodologies evolved over the 1970s and 1980s. Simplified soil spring approaches continued to be used in various contexts. More complete substructure methods emerged, specifically developments in several direct approaches to performing the SSI analyses.

In the 1980s, emphasis was placed on the accumulation of substantial data supporting and clarifying the roles of the various elements of the SSI phenomenon. One important activity undertaken by the Electric Power Research Institute (EPRI), in cooperation with Taiwan Power Company (TPC), was the construction of two scale-model reinforced concrete nuclear reactor containment buildings (one quarter and one twelfth scale) within an array of strong motion instruments (SMART-1, Strong Motion Array Taiwan, Number 1) [11]. The structures were instrumented to complement the free field motion instruments of SMART-1.

The expectation was that this highly active seismic area would produce a significant earthquake with strong ground motion. The objectives of the experiment were to measure the responses at instrumented locations due to vibration tests, and due to actual earthquakes. Further, to sponsor a numerical experiment designed to validate analysis procedures and to measure free field and structure response for further validation of the SSI phenomenon and SSI analysis techniques. These objectives were generally accomplished although with some limitations due to the dynamic characteristics of the scale model structure compared to the very soft soil at the site.

Additional recorded data in Japan and the United States of America served to demonstrate important aspects of the free field motion and SSI phenomena [12]. The additional recorded data includes: (i) downhole free field motion demonstrating variations with depth, generally, reductions of motion with depth in the soil or rock; and (ii) recorded motions on embedded foundations indicating reduction of motions compared to motions recorded on the free surface demonstrating kinematic and inertial interaction (either separately or combined).

Significant progress was made in the development and implementation of SSI analysis techniques, including the release of the SASSI computer program (see Section 10), which continues to be in use today.

In the 1980s, scepticism persisted as to the physical phenomena of spatial variation of free field motion, i.e. the effect of introducing a free boundary (top of grade) into the free field system, the effect of construction of a berm for placement of buildings or earthen structures, and other elements. In addition, the lack of understanding of the relationship between SSI analysis 'lumped parameter' methods and finite element methods led to a US NRC requirement that SSI analysis be performed by 'lumped parameter' and by finite element methods and the results enveloped for design.

In the 1990s, data acquisition and observations, statements by experts, researchers, and engineering practitioners concluded that the two methods yield the same results for problems that are defined consistently. The EPRI/TPC work contributed to this clarity.



The methods implemented during these three decades and continuing to the present are linear or equivalent linear representations of the soil, structure, and interfaces. Although research and development in nonlinear methods has been performed and tools, such as Real ESSI, LS-DYNA and others (see Section 10), are have been implemented for verification, validation, and testing on realistic physical situations, adoption in design or assessments has yet to be done.

In recent years, significant experience has been gained on the effects of earthquakes on NPPs worldwide. Events affecting plants in high-seismic-hazard areas, such as Japan, have been documented in IAEA Safety Reports Series No. 66, Earthquake Preparedness and Response for Nuclear Power Plants [13]. In some cases, the SSI response characteristics of NPP structures have been documented and studied, in particular the excitation and response of the Kashiwazaki-Kariwa Nuclear Power Plant in Japan, due to the Niigataken-Chuetsu-Oki (NCO) earthquake (16 July 2007) [14]. Figure 2 shows the recorded responses of the free field top of grade measured motion compared to the motions recorded at the Unit 7 Reactor Building basement and the third floor. The SSI effects are evident – significantly reduced motions from top of grade to the basement level and reduced motions in the structure accompanied by frequency shifts to lower frequencies. These measured motions demonstrate the important aspects of spatial variation of free field ground motion and SSI behaviour.

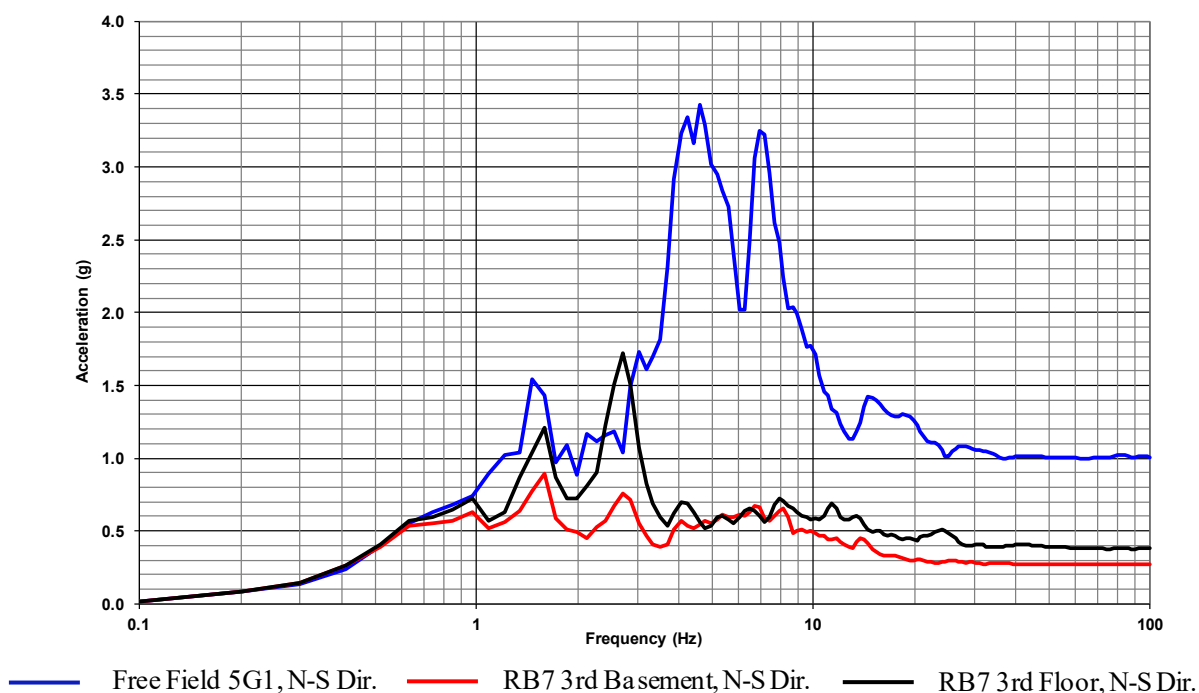


FIG. 2. Recorded motions at the Kashiwazaki-Kariwa Nuclear Power Plant in Japan, due to the Niigataken-Chuetsu-Oki (NCO) earthquake (16 July 2007) in the free field (top of grade) and in the Unit 7 Reactor Building basement, and third floor (reproduced from Ref. [14]).

## 2.2. DESIGN CONSIDERATIONS

Paragraphs 5.21 and 5.21A of IAEA Safety Standards Series No. SSR-2/1 (Rev. 1), Safety of Nuclear Power Plants: Design [15] require an adequate amount of conservatism to be introduced into the design process. The amount of conservatism is dependent on a performance goal to be established. Performance goals are established dependent on the importance to safety of the SSC and the consequences to personnel (on-site and public) and the environment of the failure of the SSC.

Safety objectives and performance goals:

- (a) May be defined probabilistically or deterministically;

- (b) May be specified at the individual SSCs level;
- (c) May be specified in terms of overall facility behaviour;
- (d) May encompass both design level and beyond design basis performance goals.

More information on safety objectives and seismic performance goal are provided in IAEA Safety Standard Series No. SSG-67, Seismic Design of Nuclear Installations [2].

In general, safety objectives and performance goals need to be consistent with the SSC design and qualification procedures that when implemented yield the overall objectives and goals. However, safety is the overriding priority. An example of overall risk-based performance goals in the USA is described in Section 2.3.1.

In general, the process of establishing a comprehensive approach to defining and implementing the overall performance goals and linking these to SSI analysis (and other elements of the seismic analysis, design, and evaluation process) are as follows:

- (a) Establish performance goals, or develop a procedure to establish performance goals, for seismic design and beyond design basis earthquake assessments for SSCs;
- (b) Divide the achievement of the performance goal into elements that contribute to the design process including SSI analysis;
- (c) Develop guidance for SSI modelling and analysis to achieve the performance goal.

An important element in this process is to establish the levels of conservatism in the current design and evaluation approaches to SSI.

## 2.3. NATIONAL PRACTICES

### 2.3.1. United States of America

Performance goals are established at the highest level of overall nuclear facility performance:

- (a) For NPPs, overall performance goals are risk based using risk metrics of mean annual core damage frequency (CDF) and mean annual large early release frequency (LERF); these goals are  $CDF < 1 \times 10^{-4}$  and  $LERF < 1 \times 10^{-6}$  [16];
- (b) For US Department of Energy (DOE) nuclear facilities, the overall performance goals are that confinement of nuclear material needs to be ensured at a mean annual frequency of failure of  $10^{-4}$  to  $10^{-5}$  [17, 18].

ASCE 4-16 [19], ASCE 43-05 [20] and U.S. DOE Standard [17, 18] define specific performance goals of SSCs in terms of DBE and in combination with BDBEs.

For the DBE, the goal of ASCE/SEI 4-16 [19] is to develop seismic responses with an 80% probability of non-exceedance. For probabilistic seismic analyses, the response with an 80% probability of non-exceedance is selected.

ASCE/SEI 4-16 [19] is intended to be used with the revision to ASCE/SEI 43-05 [20] to achieve specified target performance goal annual frequencies. To achieve these target performance goals, ASCE/SEI 43-05 [20] specifies that the seismic demand and structural capacity evaluations have sufficient conservatism to achieve both of the following:

- (a) Less than about a 1% probability of unacceptable performance for the DBE ground motion;
- (b) Less than about a 10% probability of unacceptable performance for a ground motion equal to 150% of the DBE ground motion.

The performance goals will be met if the demand and capacity calculations are carried out to achieve the following:

- (a) Demand is determined at about the 80% non-exceedance level for the specified input motion;
- (b) Design capacity is calculated at about 98% exceedance probability.

For new NPPs (generally certified designs), during the design phase, the vendor needs to demonstrate a plant level high confidence of low probability of failure (HCLPF) of approximately 1.67 times the DBE (certified seismic design response spectra) by PRA-based seismic margin assessment procedures. This applies to nuclear island SSCs. In addition, SSCs located in the balance of plant and designed to site specific ground motions, the same principle applies, i.e., demonstration that such SSCs are designed such that the site specific plant level HCLPF is approximately 1.67 times the site specific ground motion. Finally, once the site and all site specific features have been determined, an SPRA is to be performed with the end results to demonstrate conformance with the above-stated risk metrics of CDF and LERF.

### **2.3.2. France**

The general practice for seismic design and periodic (about every other 10 years) safety assessment is based on a deterministic definition of ground motion and on a deterministic seismic analysis of safety SSCs, as follows:

- (a) The site specific design earthquake ground motion is defined from a mainly deterministic seismic hazard analysis (DSHA) (RFS2001-01) [21];
- (b) The seismic demand and capacity checks are to be performed by deterministic approaches.

The aim is to use probabilistic safety analysis in specific domains, when needed, as a complementary approach. Few specific probabilistic seismic analyses have been performed by operating organizations or the regulatory body or associated technical support organizations in recent years; some analyses have started recently and are still in progress.

For post-Fukushima checks regarding the hard-core components for the BDBE, the regulatory body has specified general conditions as follows:

- (a) The definition of the seismic ground motion is the envelope of 150% of the site specific DBE ground motion and of a probabilistic motion with a return period of 20,000 years (annual frequency of exceedance of  $5 \times 10^{-5}$ );
- (b) The use of deterministic demand and capacity checks, including specific criteria in a step-by-step approach from design to more realistic and less conservative practices, consistent with the hard-core components' functionality.

### **2.3.3. Canada**

The Canadian Nuclear Safety Commission REGDOC 2.5.2, Design of Reactor Facilities: Nuclear Power Plants [22] provides the basis for the following criteria and goals:

- (a) Safety goals: Safety analyses needs to be performed to confirm that these criteria and goals are met, to demonstrate effectiveness of measures for preventing accidents, and mitigating radiological consequences of accidents if they do occur. Safety goals are related to beyond design basis accidents and design extension conditions.
- (b) Dose acceptance criteria: The dose acceptance criteria and committed whole body dose for average members of the critical groups who are most at risk, at or beyond the site boundary, be calculated in the deterministic safety analysis for a period of 30 days after the analysed event. Dose acceptance criteria are related to the design basis. This dose has to be less than or equal to the dose acceptance criteria of:
  - (i) 0.5 millisievert (mSv) for any anticipated operational occurrence or
  - (ii) 20 mSv for any design basis accident

The values adopted for the dose acceptance criteria for anticipated operational occurrences and design basis accidents are consistent with accepted international practices and take into account the IAEA safety standards and the recommendations of the International Commission on Radiological Protection.

For practical application, quantitative safety goals have been established to achieve the intent of the qualitative safety goals.

A core damage accident results from a postulated initiating event followed by the failure of one or more safety system(s) or safety support system(s). Core damage frequency is a measure of the NPP's accident prevention capabilities.

Small release frequency and large release frequency are measures of the NPP's accident mitigation capabilities. They also represent measures of risk to society and to the environment due to the operation of an NPP.

The three quantitative safety goals are:

- (a) Core Damage Frequency: The sum of frequencies of all event sequences that can lead to significant core degradation be less than  $10^{-5}$  per reactor year;
- (b) Small release frequency: The sum of frequencies of all event sequences that can lead to a release to the environment of more than  $10^{15}$  becquerel of iodine-131 be less than  $10^{-5}$  per reactor year. A greater release may require temporary evacuation of the local population;
- (c) Large release frequency: The sum of frequencies of all event sequences that can lead to a release to the environment of more than  $10^{14}$  becquerel of cesium-137 be less than  $10^{-6}$  per reactor year. A greater release may require long term relocation of the local population.

#### **2.3.4. Japan**

The performance goals are defined deterministically. The SSC design is qualified given the DBE ground motion. However, the seismic design and structural capacity evaluations are validated for two different levels of DBE, Ss and Sd. The Sd ground motion is for elastic design; Sd demands almost linearity. The Ss ground motion allows non-linear behaviour of the structures.

For BDBE, the amplified ground motions are used to check the seismic margin of SSC based on the DBE Ss. In addition, as part of the comprehensive assessment of effectiveness of safety enhancement measures against seismic events including beyond design basis events, the results of plant specific SPRA or SMA approaches are required to report and publish periodically.

Since Japan is located in a high seismic region, the significance of elasto-plastic behaviour of structures is recognised from the early period of seismic design in Japan. Thus, design methodology based on the response analysis with lumped mass stick models (LMSMs), which can consider elasto-plastic behaviour of structures in detail, was generically developed. In addition, many large-scale experiments focusing on the ultimate capacity of structures were conducted in order to establish seismic design criteria. However, if the influence on local responses such as out-of-plane vibration of walls cannot be neglected, finite element method (FEM) models are used to evaluate such local responses.

Nonlinear modelling of soil and structures is encouraged with various levels of modelling detail. In addition, the regulatory body requires modelling from the source to NPP with various levels of modelling detail. The results of this source modelling are evaluated in light of other methods of generating ground motion for the Sd and Ss.

Three-dimensional nonlinear modelling of soil and SSCs and source modelling take advantage of currently available high-performance computing for very high-resolution modelling and analyses.

The SSI modelling procedures mentioned above are provided in Japanese seismic design standard JEAG-4601[23, 24].

Reference [25] documents the development and implementation of the seismic design standards and calculational methods in the USA and Japan over the last several decades. A step-by-step comparison of these standards and calculational methods is made to provide insight into the similarities and differences that exist.

### **2.3.5. Russian Federation**

In the Russian Federation, seismologists perform probabilistic seismic hazard analyses (PSHA), the purpose of which is to develop seismic hazard curves representing the relationship between the value of ground motion parameters and their annual probabilities of exceedance. In current Russian practice, spectral acceleration of given periods (or frequencies) in the free field are used as parameters of ground motion. In addition, seismic hazard curves for the zero-period acceleration are generated representing the PGA at annual probabilities of exceedance. The seismic hazard curves are processed to produce sets of response spectra in the free field with given annual probabilities of exceedance. Statistics of the seismic hazard curves are also generated conditional on the probability of exceedance, e.g. median, mean, 84 percentile values.

Designers choose a response spectrum for design from the sets in accordance with the requirements of the Federal design code in nuclear energy [26] with a given annual probability of exceedance (e.g. maximum design earthquake (SL2) - once in 10,000 years) and confidence level (e.g. 84%), depending on the importance level of the designed buildings and structures. Standard response spectra, scaled by the PGA defined for the NPP site, are sometimes used for preliminary calculations. Design decisions are based on deterministic calculations in which uncertainties of design model parameters (load, strength, stiffness, damping, etc.) and calculation methods are considered conservative (e.g., floor response spectra for medium soil profile are broadened in frequency range and are enveloped over the three variants of soil profiles).

Regarding NPPs, there are regulatory requirements to ensure a high level of nuclear safety for all credible internal and external natural and human-made initiating events, including BDBE. The annual safety goals for each NPP unit are:

- (a) Total probability of severe accidents is not to exceed  $10^{-5}$ ;
- (b) Total probability of a large accidental release is not to exceed  $10^{-7}$ ;
- (c) Total probability of severe accidents for nuclear fuel in existing nuclear repositories is not to exceed  $10^{-5}$ .

Compliance with these requirements needs to be proved by probabilistic safety assessment (PSA). To assess the impact of earthquakes on the safety of an NPP unit, seismic PSA is developed.

### **2.3.6. European Utility Requirements**

For standard designs of new NPPs in Europe, the European utility requirements document [27] specifies DBE ground motion comprised of broad-banded ground response spectra anchored to a PGA of 0.25g and consideration of BDBE events in the design phase. The European utility requirements document specifies that it needs to be demonstrated that a standard design achieves a plant seismic margin (HCLPF) of 1.4 times the DBE. For the standard design the preferred approach to demonstrating margin is a deterministic SMA.

## 2.4. REQUIREMENTS AND RECOMMENDATION IN IAEA SAFETY STANDARDS

The following IAEA Safety Standards are relevant to hazard definition (PSHA and DSHA), site response analyses, and SSI:

- IAEA Safety Standards Series No. SSG-9 (Rev. 1), Seismic Hazards in Site Evaluation for Nuclear Installations, Specific Safety Guide [28];
- IAEA Safety Standard Series No. NS-G-2.13, Evaluation of Seismic Safety for Existing Nuclear Installations [29];
- IAEA Safety Standards Series No. NS-G-3.6, Geotechnical Aspects of Site Evaluation and Foundations for Nuclear Power Plants [30];
- IAEA Safety Standards Series No. SSG-67, Seismic Design of Nuclear Installations [2].

These Safety Guides provide recommendations as to what is to be considered in the SHA, soil and structure modelling, and SSI models and analysis.

## 2.5. SIMPLE, SIMPLIFIED AND DETAILED METHODS, MODELS AND PARAMETERS

The objective of the seismic response analysis is to calculate the seismic demands on the soil/rock, and individual SSCs for purposes of design and assessment. The focus is on structures, including the definition of input motion to systems and components supported therein, and components supported on soil/rock where the SSI phenomena are important. Other elements that are essential to the SSI analyses are the free field ground motion, site configuration, modelling of soil material behaviour, and modelling of foundation and structure behaviour. Models and methods of analysis (linear, equivalent linear, nonlinear; one-, two-, and three-dimensions) are to be selected depending on the expected behaviour of the soil–structure system and the response quantities of interest.

A basic premise is that the selection of seismic input, models, and methods of analysis, including parameters that define these elements, need to match the expected behaviour of the soil–structure system with respect to the seismic response quantities to be calculated. The purposes of the SSI analyses are established a priori by the SSI analysts in consultation with engineers (geotechnical, structural (analysts and designers), mechanical and electrical). Whatever models are proposed or selected to be used need to be validated for the problem being addressed (see Section 9.6.5 for a discussion on validation). Simplified models introduce more uncertainties in the modelling aspects but not necessarily in the response quantities (if they can be captured by the simplified model); however, both detailed and simplified models have the same level of uncertainties in the response quantities due to uncertainties in other aspects, e.g., ground motion, soil modelling.

The meaning of the terms ‘simple’ and ‘simplified’ as used in this publication is that the expected behaviour of the soil–structure system with respect to the seismic response quantities of interest is well modelled by a simple approach or by a simplified approach derived from a more detailed model. The appropriateness of the simplification needs to be demonstrated. Appropriate justification means that analyst need to demonstrate that a simplified model does not introduce too much error in the results due to simplification. This is done by creating a more sophisticated model, then comparing results with the simplified model, for select loads, and demonstrating that the simplified model can properly predict features of interest. It is up to the analyst to decide about an error: 5%, 10% or 20%. Often, the parameters of the simple or simplified models are also simplified, but appropriately justified, for the purposes of the SSI analyses. The meaning of the term ‘detailed’ as used in this publication is that the expected behaviour of the soil–structure system with respect to the seismic response quantities of interest involves or benefits

from more detailed representations of the seismic input, site configuration, soil material modelling, modelling of foundation and structure, or other aspects of the soil-structure system, as follows:

- (a) In some cases, a ‘detailed’ model simply means a model that is needed to be more detailed to permit calculation of a further refinement of the response quantities of interest, e.g., a simple dynamic model of a structure may be adequate for calculating the overall response of the structure including SSI effects. A detailed FEM model is subsequently needed to calculate stresses in structure elements for design and assessment (including load combinations);
- (b) The term ‘detailed’ is applied to the soil–structure modelling when complex phenomena are expected. One example is modelling of saturated and partially saturated soils. If this behaviour is deemed to be important to the end result of the SSI analysis, i.e., response quantities of interest, then a more detailed model is needed.

The premise of the publication is that the basis for the selection of SSI models, methods of analysis, and input parameters to obtain the seismic response quantities of interest yield accurate results. Further, this selection relies on the expertise and experience of the SSI analysts, supplemented by experts in the areas of seismic input (free field ground motion), engineers (geotechnical, structural, mechanical), and computational experts.

Simplified models model only those aspects of the SSI problem that are deemed significant to the response quantities of interest for this phase of the dynamic analyses. Hence, simplified models/methods only introduce more uncertainties in the accurate modelling of all aspects of the SSI phenomena, which by definition need detailed modelling.

It is important to recognize that uncertainties abide in each step of the SSI modelling and analysis for simple models/methods and detailed models/methods. Generally, detailed models/methods allow more complex behaviour to be modelled, but do not necessarily reduce the uncertainties in the calculated responses of interest. In applying simplified models/methods, the elements significantly contributing more uncertainty into the response quantities of interest are excluded, because the premise of their applicability excludes such and other elements of the SSI analysis which contribute significantly more uncertainty into the calculated responses. This is a modelling uncertainty, and it is up to the analyst to decide if the different level of model sophistication (simplified versus detailed models) introduce acceptable or unacceptable additional modelling uncertainties. Analyst needs to document these uncertainties and explain why these are acceptable or unacceptable. Simple models may be used to benchmark detailed models and vice versa. Simple models may also be useful, once benchmarked, for performing sensitivity studies at a minimal cost. Detailed models need to be benchmarked with simple models that adequately represent limited implementations of the more detailed phenomena. For the purposes intended (i.e. analysing the response quantities of interest), once benchmarked, both simple and detailed models are accurate to the same degree.

### **3. ELEMENTS OF SOIL–STRUCTURE INTERACTION ANALYSIS**

The elements of SSI analysis are:

- (a) Free field ground motion – seismic input, site configuration and modelling of soil properties;
- (b) Site response analysis;
- (c) Modelling of foundation and structure;
- (d) Methods of SSI analysis;

(e) Uncertainties.

Table 1 summarizes the elements of SSI with reference to sections of this publication in which the element is addressed. Figure 3 is a flow chart showing the various steps in the process.

### 3.1. FREE FIELD GROUND MOTION

The term free field ground motion denotes the motion that would occur in soil or rock in the absence of the structure or any excavation. Describing the free field ground motion at a nuclear installation site for SSI analysis purposes entails specifying the point at which the motion is applied (the control point), the amplitude and frequency characteristics of the motion (referred to as the control motion and typically defined in terms of ground response spectra, and/or time histories), the spatial variation of the motion, and, in some cases, strong motion duration, magnitude, and other earthquake characteristics.

In terms of SSI, the variation of motion over the depth and width of the foundation is the important aspect. For surface foundations, the variation of motion on the surface of the soil is important; for embedded foundations, the variation of motion over both the embedment depth and the foundation width are important.

The free field ground motion may be defined in terms of either site-independent, or site-dependent ground response spectra:

- (a) Site independent ground motion is most often used for performing a new reference design or a certified design, which is to be placed on a number of sites with differing characteristics;
- (b) Site specific ground motion is most often developed from an SHA and is most often used for site specific design or assessments.

There are two stages in the development of the site specific free field ground motion and seismic input to the SSI analyses:

- (a) Source to neighbourhood of the site. There are four basic approaches that are used to develop ground motion models that generate ground motions in the neighbourhood of the site: empirical ground motion prediction equations (GMPEs), point source stochastic simulations, finite-fault simulations (FFS), and the hybrid empirical method (HEM). These methods are generally implemented probabilistically, and some are probabilistic by definition, e.g., point source stochastic simulations. This stage is referred to as SHA and the methods are discussed in Section 6. In the neighbourhood of the site, site specific characteristics are still not introduced into the ground motion definition, e.g., local geological or geotechnical properties, strain-dependency of soil properties, etc. In the current state-of-practice, the location of the free field ground motion in the neighbourhood, i.e., the SHA results, are most often specified at the top of grade at the site of interest or at a location within the site profile, such as on hard rock, a competent soil layer, or at an interface of soil/rock stiffness with a significant impedance contrast. U.S. NRC Standard Review Plan Section 4.7.1 [31] defines a competent soil layer as soil with shear wave velocity ( $V_s$ ) of 305 m/s or greater. The EUROCODE [32] defines a competent layer as one in which the impedance contrast exceeds a ratio of six. In the context of the ‘neighbourhood’ this location is denoted as the ‘control point’. In this latter case, a site response analysis is carried out to generate the ground motion for input to the SSI analysis.
- (b) Local site effects. Once the ground motion in the neighbourhood of the site has been defined, the analyst needs to consider the effects of local site conditions. This may be achieved through site response analysis. In the broadest sense, the purpose of site response analysis is to determine the free field ground motion at one or more locations given the motion at another location. Site response analysis is intended to take into account the wave propagation mechanism of the ground motion (usual assumption is vertically propagating



P- and S-waves; however, other wave propagation mechanisms may need to be considered) and the strain dependent material properties of the media. Either convolution or deconvolution procedures may be necessary to do so.

- (i) If the end product of the first stage of this definition process is ground motions at TOG, then deconvolution may be required to generate seismic input for SSI analyses of structures with embedded foundations. Deconvolution may also be required to generate seismic input on boundaries of a FEM model, e.g. nonlinear SSI model<sup>3</sup>;
- (ii) If the end product of the first stage of this definition process is ground motions on a hypothetical or actual outcrop at depth or an in-soil location, convolution analysis will be performed.

The output from these ‘site effects’ (or site response) analyses are the seismic input and soil material properties for the SSI analyses. There are uncertainties in both methods whether the control point is at the top of soil grade or at the bedrock. However, general practices are that the second method (convolution procedures are strongly preferred) has lesser uncertainties and is preferred in many important projects but the first method is simple and can be more suitable for some situations. The details of the free field ground motion and seismic input elements of the SSI analysis, along with the soil property definitions, are contained in Sections 4, 5, 6, and 7.

TABLE 1. SUMMARY OF ELEMENTS OF SOIL–STRUCTURE INTERACTION ANALYSIS WITH REFERENCE TO THE SECTIONS COVERING THESE TOPICS

**Free field ground motion**

Reference designs/Certified Designs (Site independent)	Section 7
Site specific information requirements interface between seismic hazard analysts and SSI analysts	Section 5
Site specific seismic hazard curves, Uniform hazard response spectrum (UHRs), deaggregated hazard, etc.	Section 3 and 5
Site effects (site response analysis) - soil properties, seismic input (control point(s), ground motion response spectra, time histories, etc.)	Sections 4, 6 and 7

**Site and soil model**

Stratigraphy, Idealized horizontal layers, 2-D, or 3-D model	Section 4
Soil material model, Strain level dependent - linear (hard rock), equivalent linear, nonlinear material	Sections 4, 7 and 8

**Structure models**

Linear, equivalent linear, nonlinear	Sections 8 and 9
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**SSI models**

Linear, equivalent linear, nonlinear	Sections 8 and 9
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<sup>3</sup> Limitations of the deconvolution process are well recognized especially when simultaneously generating equivalent linear soil properties along with ground motions at important locations in the site profile. This is discussed in more detail in Chapters 5 and 6.

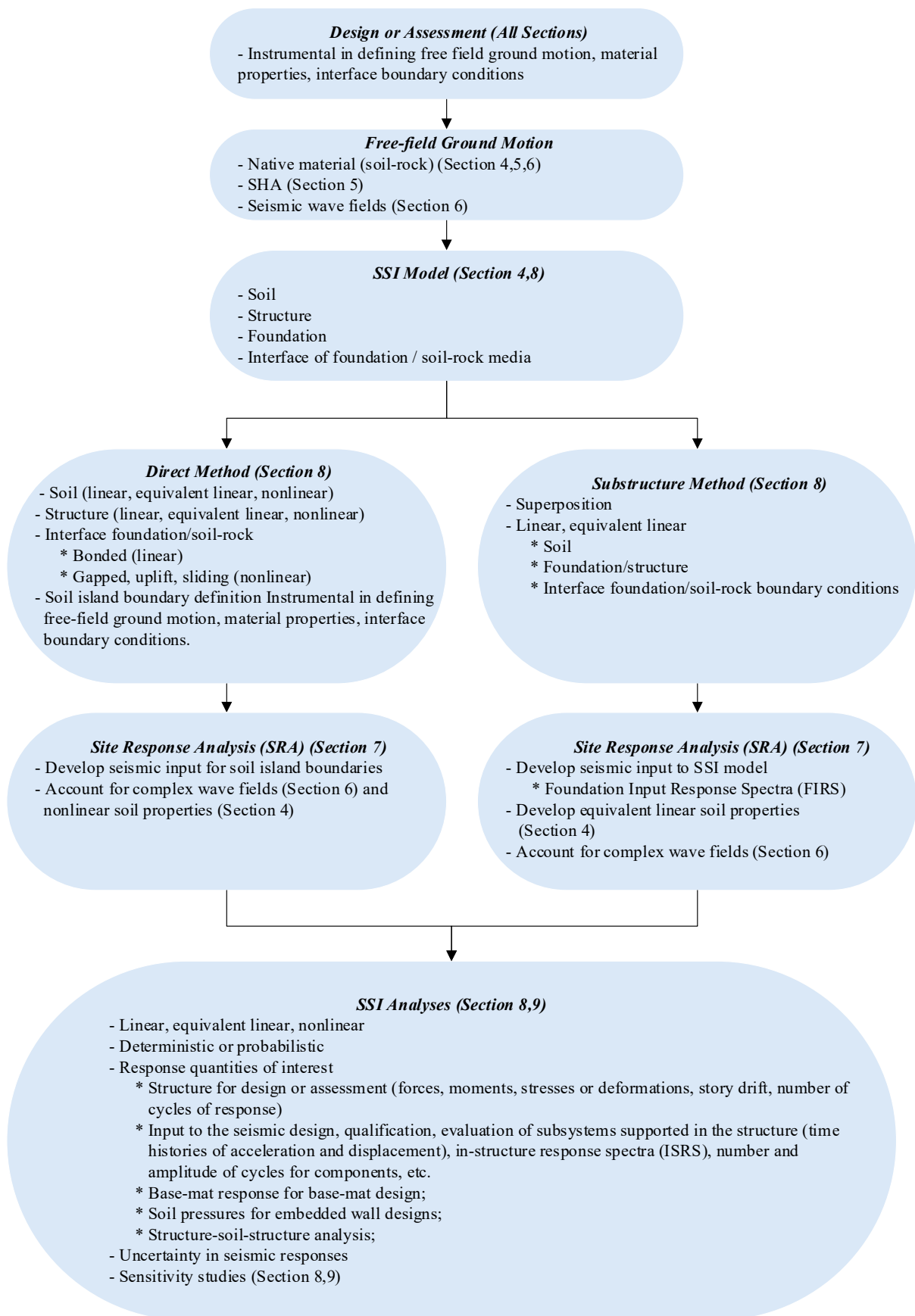


FIG. 3: Elements of soil–structure interaction analyses correlated with text.

## 3.2. MODELLING SOIL, STRUCTURES AND FOUNDATIONS

### 3.2.1. Soil for design basis and beyond design basis earthquakes

The selection of material models for in-situ soil and rock is dependent on numerous issues:

- (a) Soil characteristics – hard rock to soft soil;
- (b) Strain level;
- (c) Availability of soil material models for SSI analysis in candidate software to be used (linear, equivalent linear, nonlinear, elastic-plastic);
- (d) Laboratory tests to define material property parameters (linear and nonlinear), correlation with field investigation results for excitation levels of interest;
- (e) Phenomena to be modelled, e.g. dynamic response;
- (f) Risk importance of SSCs to be analysed.

These topics are considered extensively in Section 4.

### 3.2.2. Structures and soil–structure interaction models

In general, one can categorize seismic structure analysis, and, consequently, the foundation and structure models, into multistep methods and single step methods:

- (a) In the multi-step method, the seismic response analysis is performed in successive steps. In the first step, the overall seismic response (deformations, displacements, accelerations, and forces) of the soil-foundation-structure is determined, as follows:
  - (i) The first step of the structure model represents the overall dynamic behaviour of the structural system but need not be refined to predict stresses in structural elements.
  - (ii) The response obtained in this first step is then used as input to other models for subsequent analyses of various portions of the structure. In these subsequent analyses, detailed force distributions and other response quantities of interest are calculated.
  - (iii) Many simple and detailed sub-structuring methods assume the foundation behaves rigidly, which is a reasonable assumption taking into account the stiffening effects of structural elements supported from the foundation (base mat, shear walls and other stiff structural elements).
  - (iv) The second step analyses are performed to obtain: (i) seismic loads and stresses for the design and evaluation of portions of a structure; and (ii) seismic motions, such as time histories of acceleration and in-structure response spectra (ISRS), at various locations of the structural system, which can be used as input to seismic analyses of equipment and subsystems.
  - (v) The first step model is sufficiently detailed so that the responses calculated for input to subsequent steps or for evaluation of the first model would not change significantly if it was further refined.
  - (vi) A detailed ‘second-step’ model that represents the structural configuration in adequate detail to develop the seismic responses necessary for the seismic design of the structure or fragility evaluations is needed. Seismic responses include detailed stress distributions; detailed kinematic response, such as acceleration, velocity, and displacement time histories; and generated ISRS.
- (b) In the single step analysis, seismic responses in a structural system are determined in a single analysis. The single step analysis is conducted with a detailed second-step model as described above.

Initially, the single step analysis was most often employed for structures supported on hard rock, with a justified fixed-base foundation condition for analysis purposes. Recently, with the development of additional computing power, single step analyses are performed more frequently.

### 3.2.3. Decisions to be made in modelling soil, structures and foundations

All modelling decisions are dependent on the purpose of the analysis, i.e. for DBE design; for BDBE; or to re-evaluate and verify the system after an actual earthquake that affected the nuclear installation has taken place.

Structure modelling needs to consider the following items:

- (a) Seismic response output quantities to be calculated - multi-step vs single step analysis;
- (b) Stress level expected in the structure:
  - (i) Linear or nonlinear structure behaviour;
- (c) LMSM vs FEM. Is a lumped mass model representative of the dynamic behaviour of the structure for the purpose of the SSI analysis;
- (d) Frequency range of interest – especially high frequency considerations (50 Hz, 100 Hz).

Foundation modelling needs to consider the following items:

- (a) Multistep vs single step analysis (overall behaviour or in-structure detailed seismic response – strain level);
- (b) Mat vs spread/strip footings;
- (c) Piles and caissons;
  - (i) Boundary conditions – base mat slab retains contact with soil/separates from underlying soil;
  - (ii) Pile groups – how to model;
- (d) Behaving rigidly or flexibly;
- (e) Surface-or near surface-founded;
- (f) Embedded foundation with partially embedded structure;
- (g) Partially embedded (less than all sides);
- (h) Contact/interface zone for embedded walls and base mat (soil pressure, separation/gapping and sliding).

SSI modelling needs to consider the following items:

- (a) Direct or substructuring methods;
- (b) Purpose of the analysis – DBE design, BDBE assessment;
- (c) Strain level – equivalent linear or nonlinear soil and structure behaviour;
- (d) Irregular soil/rock profiles;
- (e) Probabilistic and deterministic methods;
- (f) Embedment conditions (partial or full);
- (g) High water table;
- (h) Structure-to-structure interaction;

Other issues may include lateral heterogeneities and horizontal variation of soil properties; azimuth of wave propagation relative to the vertical plane of the model; the presence of a nearby waterfront with free-running water (river or ocean front from which cooling water is extracted); temporal and seasonal variation of phreatic surfaces due to rain or tides; changes in material properties in the immediate vicinity of the structure that resulted from excavation and compacted fill as well as the consolidation effects caused by the excavated earth that may have been piled up next to the excavation (perturbation of in-situ material properties, which may or not match what was tested in the lab, etc.).

### 3.3. UNCERTAINTIES

#### 3.3.1. Aleatory uncertainties and epistemic uncertainties

Uncertainties exist in the definition of all elements of soil–structure interaction phenomena and their analyses.

In many cases, uncertainties can be explicitly represented by probability distributions of SSI analysis parameters, e.g., soil material properties, structure dynamic properties. At least conceptually, the uncertainties affecting an SSI model can be grouped into two distinct classes, which are aleatory uncertainties<sup>4</sup> and epistemic uncertainties<sup>5</sup>. In some cases, these uncertainties can be expressed directly in terms of underlying probability distributions, such as those that enter into the safety factors underlying the design of a structure (load and resistance factor design). In some other cases, the uncertainties in soil properties may be taken into account via sensitivity studies, where each of these is assigned a label — such as lower bound, best estimate and higher bound — uncertainties in SSI analysis elements may need to be assessed by sensitivity studies and the results entered in the analysis by combining the weighted results.

Randomness is considered to be associated with variability that cannot practically be reduced by further study, such as the source-to-site wave travel path, and the earthquake time histories occurring at the site in each direction.

Uncertainty is generally considered to be variability associated with a lack of knowledge that could be reduced with additional information, data, or models.

Aleatory uncertainties and epistemic uncertainties are often represented by probability distributions assigned to SSI parameters. These probability distributions are typically assumed to be non-negative distributions (for example lognormal, Weibull, etc.).

An input parameter to the SSI analysis may be represented by a median value ( $A_m$ ) and a double lognormal function ( $\varepsilon_R$  and  $\varepsilon_U$ ) with median values of 1.0 and variability (aleatory and epistemic uncertainty) defined by lognormal standard deviations ( $\beta_R$  and  $\beta_U$ ).

$$A = A_m \varepsilon_R \varepsilon_U \quad (1)$$

In some cases, it is advantageous to combine the randomness and modelling/data/parameter uncertainty into a “composite variability”<sup>6</sup> as defined in ASME/ANS [33].

The same functional representation of equation 1 typically defines the fragility function for SSCs in a SPRA.

Table 2 summarizes the separation of epistemic uncertainties and aleatory uncertainties.

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<sup>4</sup> Aleatory uncertainty is the uncertainty inherent in a nondeterministic (stochastic, random) phenomenon. Aleatory uncertainty is reflected by modelling the phenomenon in terms of a probabilistic model. In principle, aleatory uncertainty cannot be reduced by the accumulation of more data or additional information (Aleatory uncertainty is sometimes called “randomness.”) [33].

<sup>5</sup> Epistemic uncertainty is the uncertainty attributable to incomplete knowledge about a phenomenon that affects our ability to model it. Epistemic uncertainty is reflected in ranges of values for parameters, a range of viable models, the level of model detail, multiple expert interpretations, and statistical confidence. In principle, epistemic uncertainty can be reduced by the accumulation of additional information. (Epistemic uncertainty is sometimes also called “parametric uncertainty”) [33].

<sup>6</sup> Composite variability includes the aleatory (randomness) uncertainty ( $\beta_R$ ) and the epistemic (modelling /data / parameter) uncertainty ( $\beta_U$ ). The logarithmic standard deviation of composite variability,  $\beta_C$ , is expressed as:  $(\beta_R^2 + \beta_U^2)^{1/2}$ .

### 3.3.2. Avoiding double counting of uncertainties

There can be a tendency to unintentionally account for the same or similar uncertainties in multiple aspects of the SSI analysis process [34]. One reason for this is the multi-disciplinary nature of the process and the separation of responsibilities between disciplines and organizations: seismic hazard analysts develop the PSHA or DSHA models and results, geotechnical or civil engineers perform site response analyses, civil/structural engineers perform the SSI analyses developing the seismic demand for SSCs, mechanical, electrical, instrumentation and control and other engineering disciplines develop seismic designs and perform assessments for systems, components, equipment and distribution systems. This separation of tasks requires careful understanding of the uncertainties introduced and modelled in the ‘prior’ steps to avoid double counting. This is especially true for the seismic hazard element’s effect on all other aspects in the seismic analysis and design chain.

TABLE 2: PARTITIONING OF UNCERTAINTIES

Element	Epistemic	Aleatory
Modelling	Uncertainty about a model and the degree to which it can predict events. Model, epistemic uncertainty addresses the possibility that a model may systematically (but not necessarily predictably), overpredict or underpredict events/result of interest (i.e., deformations)	Aleatory modelling variability is the variation not explained by a model. For instance, it is variability that is attributed to elements of the physical process that are not modelled and therefore represents variability (random differences) between model predictions and observations.
Parametric	Parametric epistemic uncertainty is associated with the estimation of model parameters given available data, indirect measurements, etc.	This uncertainty is similar to aleatory modelling uncertainty. However, this is a variability that may be due to factors that are random, but have a systematic effect on model results

### 3.3.3. Treating uncertainties in the soil–structure interaction analyses: explicit inclusion and sensitivity studies

All aspects of the SSI analysis process are subject to uncertainties. The issue is how to appropriately address the issue in the context of design and assessments.

Some issues are amenable to modelling probabilistically, for example:

- Earthquake ground motion:
  - Control motion (amplitude and phase);
  - Spatial variation of motion:
    - Wave fields generating coherent ground motion;
    - Random variation of motion– high frequency incoherent ground motion.
- Physical material properties (soil, structure dynamic characteristics);
- Physical soil configurations, (e.g. thickness of soil layers);
- Water table level, including potential buoyancy effects.

Some issues are amenable to sensitivity studies to determine their importance to SSI response, for example:

- Linear vs nonlinear soil and structure material properties;
- Coupling between soil and structure(s);
- Sliding/uplift;
- Non-horizontal layering of soil;
- Use of 1-D, 2-D, or 3-D modelling of wave propagation.

## 4. SITE CONFIGURATION AND SOIL PROPERTIES

### 4.1. SITE CONFIGURATION AND CHARACTERIZATION

Requirements for site characterization are established in IAEA Safety Standards Series No. SSR-1, Site Evaluation for Nuclear Installations [35]. The size of the region to which a method for establishing the hazards associated with earthquakes is to be applied will be large enough to include all the features and areas that could be of significance for the determination of the natural induced phenomena under consideration and for the characteristics of the event. For SSI analyses, a description of the soil configuration (layering or stratigraphy) and a characterization of the dynamic (material) properties of the subsurface materials, including the uncertainties associated to them, will be investigated and a soil profile for the site, in a form suitable for design purposes, will be determined for a nuclear installation site. Lateral variations in the soil profile and uncertainties associated with them will be identified. Determining soil properties to be used in the SSI analysis is the second most uncertain element of the process, the first being specifying the ground motion.

The site for a nuclear installation will be characterized from both geological and geotechnical investigations. The incoming seismic waves and therefore the seismic response of soils and structures are not only controlled by the properties of layers in the immediate vicinity of the structures. Therefore, large scale geological investigations are needed to define the lateral and in-depth extent of the various strata, the underground topography, the possible existence of basins<sup>7</sup>, the depth to the bedrock, the elevation of the water table, etc. As described in SSG-9 (Rev. 1) [28], geological and geotechnical investigations need to be performed at four spatial scales: regional, near regional, close vicinity to the site and site area for a nuclear installation. The typical scales go from several tens of kilometres to some hundreds of metres. These data need to be compiled to form a geographical information system and produce a geological map for the site.

In addition, the geotechnical investigations will allow the assessment of the mechanical characteristics needed for the seismic studies together with the range of uncertainties and spatial variability associated with each parameter. This is accomplished with cored boreholes, field geophysical investigations, sampling of undisturbed samples for laboratory testing.

The extent of the local site investigations depends on the ground heterogeneities, dimensions of the installation and is typically much larger than for static design. Furthermore, the content will be defined in connection with the models that are used for the seismic studies and can be guided by parametric studies aiming at identifying the most influential parameters.

The site characterization phase for a nuclear installation site need to first consider to providing answer to the following questions:

- Does the geological setting exhibit significant lateral changes (e.g., a basin, a valley or rock outcrop), or do the soil properties vary significantly in the two horizontal directions?
- Does the site geometry need to be represented for the site response analyses with a 2-D or 3-D model?

To help reach answers to these questions, the geotechnical engineer may take advantage of experience from previous studies, the simplified rules proposed in the outcome of the European

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<sup>7</sup> A basin is a structural formation of rock strata formed by tectonic warping of previously flat-lying strata. Structural basins are small or large geological depressions filled with sediments and are the inverse of domes.

research project NERA [36] or in other references (e.g., [37–39]), or be guided by site instrumentation (see Section 4.6.1).

Within the framework of the NERA project [36] aggravation factors are defined as the ratios between the spectral response acceleration at the ground surface for a 2-D configuration to the same quantity for a 1-D configuration; aggravation factors different from 1.0 reveal a 2-D effect. Pitilakis et al, 2015 [40] tentatively concluded the following based on extensive numerical analyses, involving linear and nonlinear soil constitutive models:

- (a) The aggravation factors are period dependent.
- (b) The average (over the period range) aggravation factors depend on the location of the observation point relative to the edges of the basin.
- (c) The aggravation factors for locations close to the valley edges are smaller than 1.0.
- (d) The aggravation factors for location in the central part of the valley depend on the fundamental period of the valley. They are slightly larger than 1.0 if the basin period is small, typically less than 3.0s, and therefore any 2-D effect may be considered of minor importance; they may reach high values if the basin period is large.

These results, when completed with seismic instrumentation, constitute helpful guidelines to estimate if 2-D effects are likely. It is not, however, implied that the aggravation factors calculated in the NERA project can be directly used to quantify 2-D amplifications; they need to be only used for guidance.

## 4.2. SOIL BEHAVIOUR

Soil is made up of different sized particles. Sand particles are bigger in size, fairly coarse, drain out water easily and the shear strength depends on its friction. Clay is very fine-grained soil, the smallest in size but hardest, retains water and considered most difficult to work with. Its shear strength depends on cohesion. Silts possess properties in between and possess both friction and cohesion. All soils generally behave elastically at a shear strain below  $10^{-4}$ , elasto-plastically between  $10^{-4}$  to  $10^{-2}$  and fail in  $10^{-2}$  to  $10^{-1}$  cyclic shear strain ( $\gamma$ ) ranges. Highly plastic clays stay in linear range at even higher strains as compared to sand and silts. Sands are first to enter into a non-linear range when subjected to an earthquake ground motion, followed by silts as Plasticity Index controls and the higher it is, the lower is the degradation.

Soils are known to be highly nonlinear materials as evidenced from both field observations during earthquakes and laboratory experiments on samples. According to Prevost and Popescu [41], during loading, the solid particles which form the soil skeleton undergo irreversible motions such as slips at grain boundaries. In addition, Mindlin and Deresiewicz [42] show that inelastic, elastic-plastic and irreversible deformation are present for any amount of shear stress ( $\tau$ ) in particulate material (soil), although they might not be of practical significance at very low strains. When the microscopic origin of the phenomena involved are not sought, phenomenological equations are used to provide a description of the behaviour of the various phases which form the soil medium. Considering the particles essentially incompressible, deformation of the granular assembly occurs as the particles translate, slip and/or roll, and either form or break contacts with neighbouring particles to define a new microstructure. However, the high stresses that develop at points of contact between the particles may induce recoverable strains especially at low strains when sliding and rolling are still impeded by friction, i.e. particles are not totally incompressible. The result is an uneven distribution of contact forces and particle densities that manifests in the form of complex overall material behaviour such as permanent deformation, anisotropy, localized instabilities and dilatancy (change of volume



during pure shearing deformation). Although very complex when examined on the micro scale, soils, as many other materials, may be idealized at the macro scale as behaving like continua.

In most cases, the soil element is subjected to general 3-D time-varying stresses. Thus, an adequate modelling of the soil behaviour involves sophisticated constitutive relationships. It is very convenient to split the soil response to any type of loading into two distinct components: the shear behaviour and the volumetric behaviour. In shear, soil response manifests itself in terms of a stiffness reduction and an energy dissipation mechanism, which both take place from very small strains. In reality, soils behave as elastic-plastic materials. Figure 4 shows the pure shear response of soil subjected to two different shear strain levels. It is important to note that for very small shear strain cycles (see Figure 4, top) the response is almost elastic, with a very small hysteretic loop (almost no energy dissipation). On the other hand, for large shear strain cycles (see Figure 4, bottom) there is a significant loss of (tangent) stiffness, as well as a significant energy dissipation (large hysteretic loop).

One of the most important features of soil behaviour is the development of volumetric strains even under pure shear [43]. The result is that during pure shearing deformation, soil can increase in volume (dilatancy) or reduce in volume (contraction). Incorporating volume change information in soil modelling can be very important, especially under undrained behaviour. If models that are used to study soil behaviour do not allow for modelling dilatancy or contraction, potentially significant uncertainty is introduced, and results of seismic soil behaviour might be strongly inaccurate: under undrained conditions, the tendency for volume changes manifests itself in pore water pressure changes and therefore in stiffness reduction/increase and strength degradation/hardening since soil behaviour is governed by effective stresses. Effective stresses  $\sigma'$  depend on the total stress  $\sigma$  (from applied loads, self-weight, etc.) and the pore fluid pressure  $p$ :

$$\sigma' = \sigma - p\delta^8 \quad (2)$$

Where,  $\sigma$  is the total stress,  $p$  is the pore fluid pressure and  $\delta$  is the Kronecker symbol.

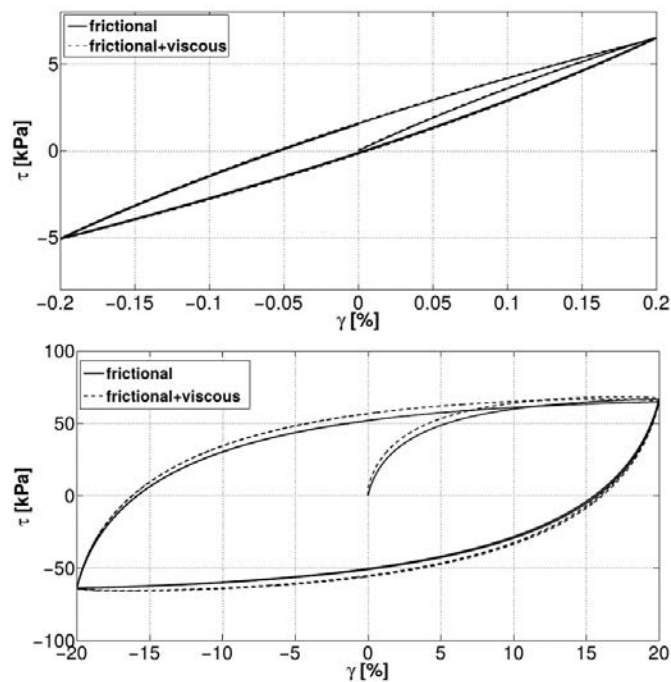


FIG. 4. Predicted pure shear response of soil at two different shear strain amplitudes.

<sup>8</sup> The sign convention of continuum mechanics is used through.

Figure 5 shows three responses for no-volume change (left), compressive (middle) and dilative (right) soil with full volume constraint, resulting in changes in stiffness for compressive (reduction in stiffness), and dilative (increase in stiffness).

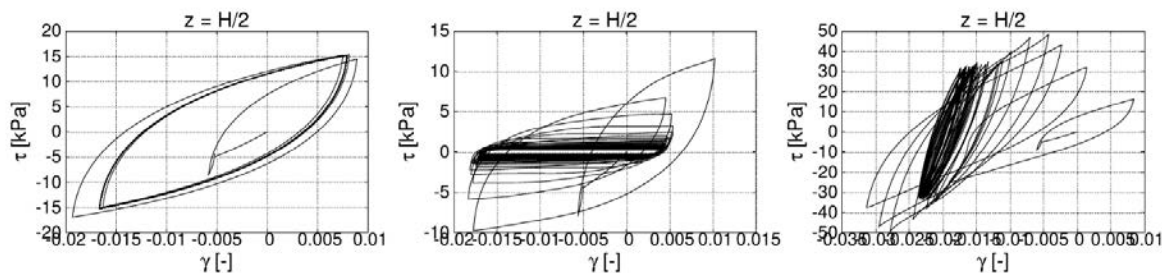


FIG. 5. Cyclic response of soils: (left) no volume change; (middle) compressive response with decrease in stiffness; (right) dilative response with increase in stiffness (reproduced from Ref. [44] with permission courtesy of Canadian Nuclear Safety Commission).

As previously said, the soil element is subjected to general 3-D time-varying stresses. However, it is frequently assumed that a 1-D situation prevails, and simpler models can then be used to describe the salient features of the soil response. Therefore, in the following, emphasis is placed on such models, which are used in practice; some indications are nevertheless provided for 3-D constitutive relationships although there is not a single model which can be said to be superior to the others. Section 8 provides further description of 3-D material models, and references to the most used ones.

#### 4.3. EXPERIMENTAL DESCRIPTION OF SOIL BEHAVIOUR

Simple 1-D models can be used to describe soil behaviour; in such cases, the material properties may depend, for example, on the total state of stress, and it is commonly assumed for site response analyses, or for SSI problems, to consider that the seismic horizontal motion is caused by the vertical propagation of horizontally polarized shear waves. Under such conditions, a soil element within the soil profile is subjected to shear stress ( $\tau$ ) cycles similar to those presented in Figure 6.

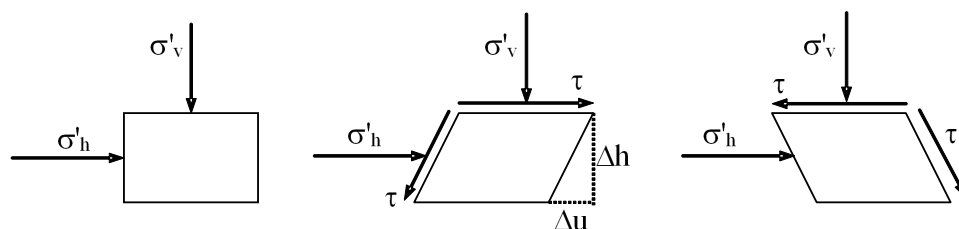


FIG. 6. Idealized stress cycle during an earthquake.

Depending on the amplitude of the induced shear strain, different types of nonlinearities take place. When the shear strain amplitude is small, typically less than  $\gamma_s = 10^{-5}$  the behaviour can be assumed to be essentially linear elastic with no evidence of significant nonlinearity. For increasing shear strain amplitudes, with a threshold typically of the order of volumetric strain,  $\gamma_v = 1 \times 10^{-4}$  to  $5 \times 10^{-4}$ , nonlinearities take place in shear while the volumetric behaviour remains essentially elastic (see Figure 7, above). It is only for strains larger than  $\gamma_v$  that significant volumetric strains take place in drained conditions (see Figure 7, below), or pore water pressures develop under undrained conditions.

These different threshold strains are summarized in Figure 8 adapted from Vucetic and Dobry [45]; similar results are presented by Ishihara [43] which indicate that the threshold strain  $\gamma_v$  depends on the plasticity index and may reach values of  $10^{-3}$  for highly plastic clays. Depending

on the anticipated strain range applicable to the studied problem, different modelling assumptions, and associated constitutive relationships, may be used (see Table 3). Accordingly, the number and complexity of constitutive parameters needed to characterize the behaviour will increase with increasing shear strains. Different soil models used in nuclear installations are explained.

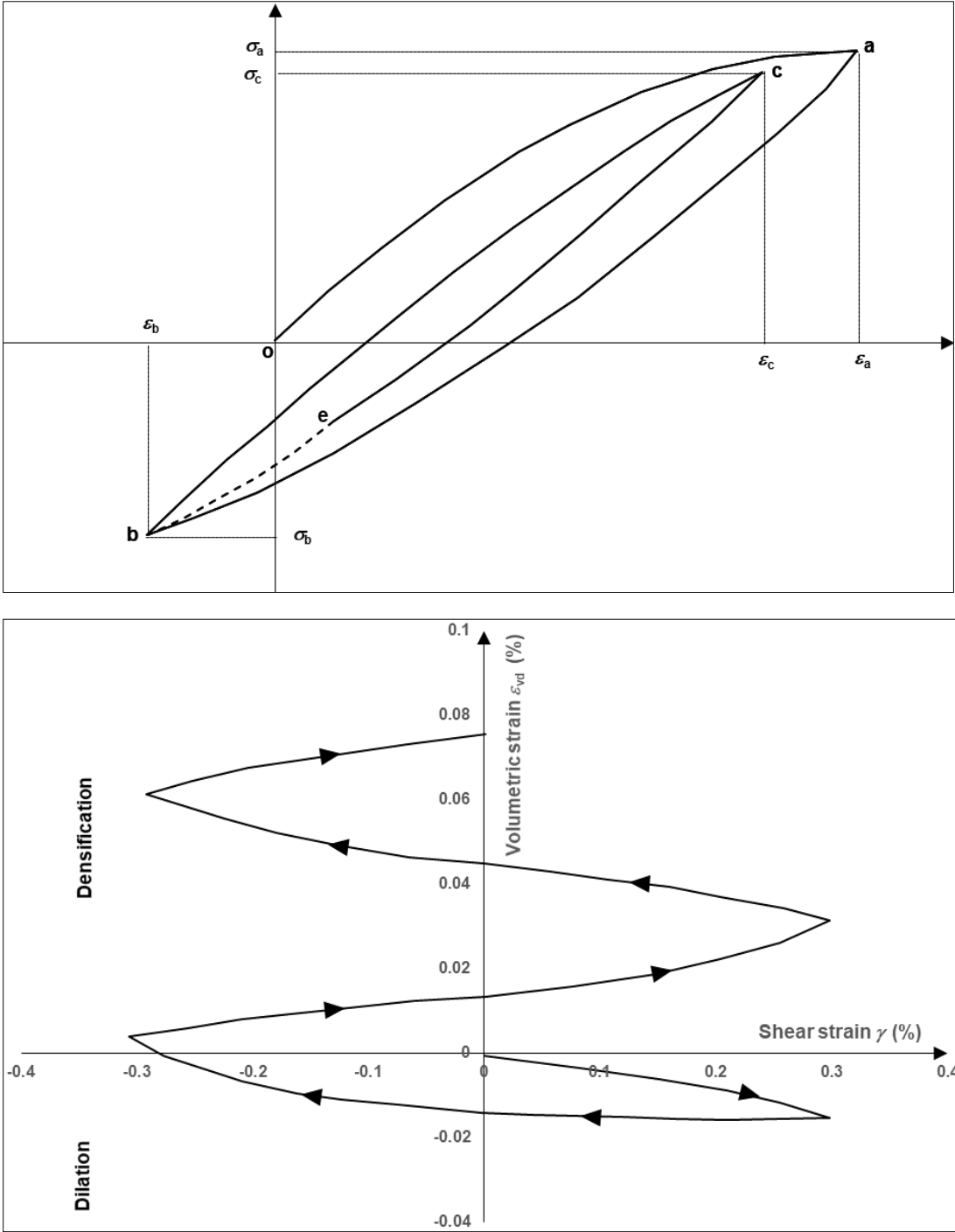


FIG. 7. Experimental stress-strain curves under cyclic loading: nonlinear shear behaviour (above); volumetric behaviour (below).

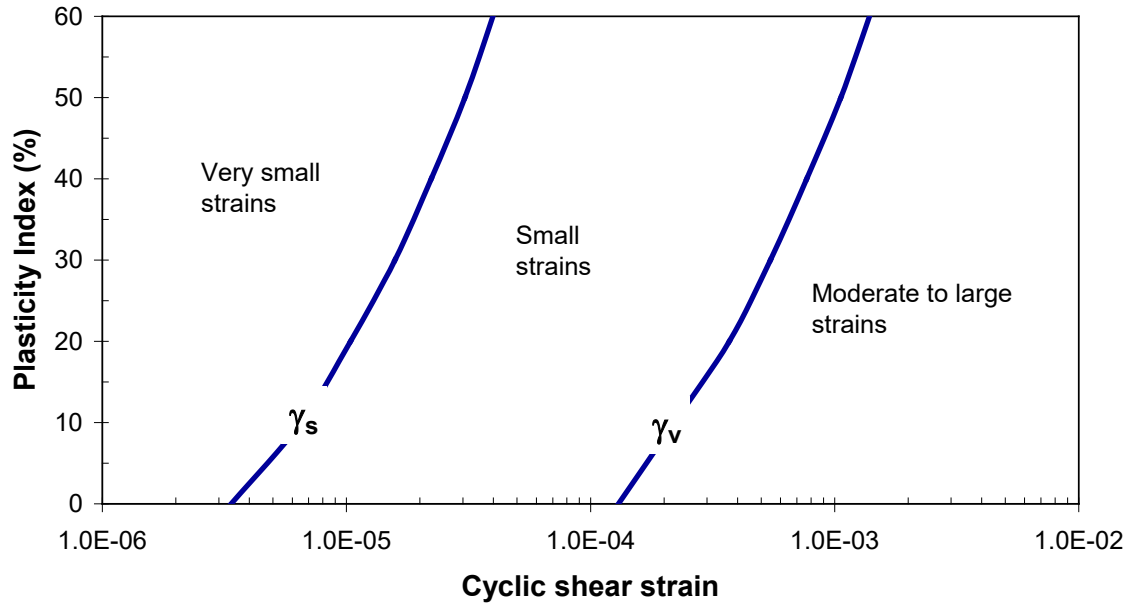


FIG. 8. Threshold values for cyclic shear strains.

TABLE 3: STRAIN THRESHOLDS AND MODELLING ASSUMPTIONS

Cyclic shear strain amplitude $\gamma$	Behaviour	Elasticity and plasticity	Cyclic degradation in saturated soils	Modelling
Very small $0 \leq \gamma \leq \gamma_s$	Practically linear	Practically elastic	Non-degradable	Linear elastic
Small $\gamma_s \leq \gamma \leq \gamma_v$	Non-linear	Moderately elasto-plastic	Practically non-degradable	Viscoelastic Equivalent linear
Moderate to large $\gamma_v \leq \gamma$	Non-linear	Elasto-plastic	Degradable	Non-linear

#### 4.3.1. Linear viscoelastic model

In the strain range  $\gamma_s \leq \gamma \leq \gamma_v$ , nonlinearities and energy dissipation become apparent in the shear-strain curve (see Figures 7 and 9). As a linear viscoelastic model (see Figure 10) exhibits under harmonic loading hysteresis loops, it is tempting to model the soil behaviour with such models. However, the viscoelastic model lends itself to an energy dissipation mechanism that is frequency dependent, in contradiction with experimental observations. Furthermore, the shape of the hysteresis loop exhibits rounded loading/unloading endings and has no sharp vertices with discontinuous slopes as the inelastic loop shown in Figure 9, which obeys Masing's law. The hysteresis loop for viscous damping is an ellipse depicted with a dotted line in Figure 9.

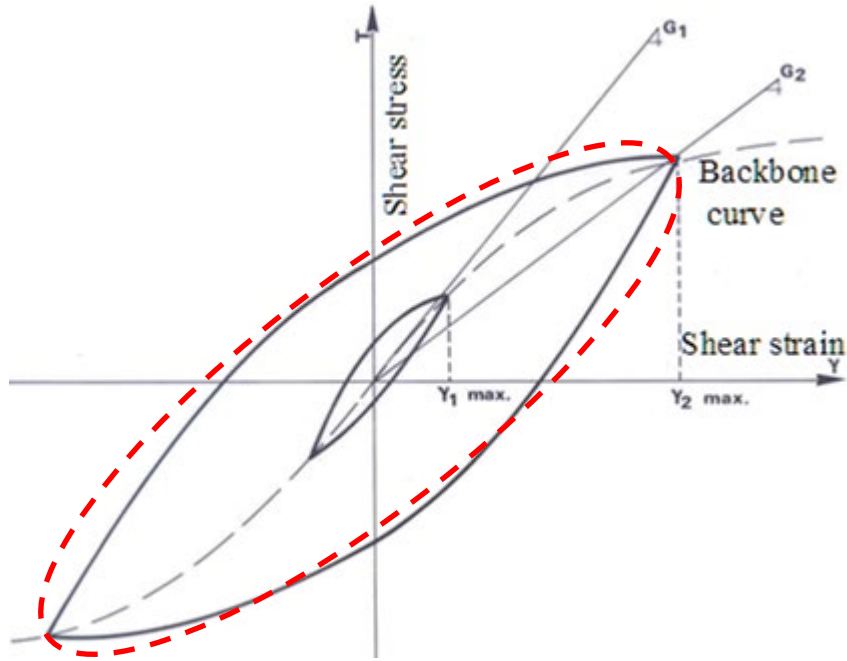


FIG. 9. Shear stress-shear strain curves for constant amplitude cyclic loading; experimental curve: solid black line; viscoelastic model: red dotted line.

For harmonic loading  $\gamma = \gamma_m e^{i\omega t}$ , the 1-D stress-strain relationship of the viscoelastic model (see Figure 10),  $\tau = G\gamma + C\dot{\gamma}$  can be written:

$$\tau_m = [G + iC\omega]\gamma_m = G \left[ 1 + i \frac{C\omega}{G} \right] \gamma_m = G^* \gamma_m \quad (3)$$

Where,  $\tau$  is the shear stress,  $G$  is the shear modulus,  $C$  is the dash-pot,  $\omega$  is angular velocity and  $\gamma$  is shear strain.

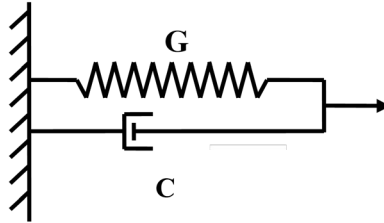


Fig. 10. One-dimensional viscoelastic model.

The energy dissipated in one cycle of loading in this model,  $\Delta W$  is equal to:

$$\Delta W = \pi C \omega \gamma_m^2 \quad (4)$$

and is frequency dependent. Normalizing by the maximum elastic stored energy in a unit volume,  $W$  which is equal to:

$$W = \frac{1}{2} \tau_m \gamma_m = \frac{1}{2} G \gamma_m^2 \quad (5)$$

One obtains:

$$\frac{\Delta W}{W} = 2\pi \frac{C}{G} \omega \quad (6)$$

Defining  $\frac{C}{G} = 2\frac{\beta}{\omega_n}$ , where  $\beta$  is the fraction of critical viscous damping and  $\omega_n$  the resonant frequency, equation (6) becomes:

$$\frac{\Delta W}{W} = 4\pi\beta \frac{\omega}{\omega_n} \quad (7)$$

At resonance,  $\frac{\Delta W}{W} = 4\pi\beta$  indicating that the critical damping ratio is a measure of the energy dissipated at resonance. Introducing the parameter  $\eta$ ,  $\eta = \frac{C\omega}{G}$ , called the loss coefficient, the energy  $\Delta W$  dissipated during one cycle of loading can be written:

$$\Delta W = \pi C \omega \gamma_m^2 = \pi G \eta \gamma_m^2 \quad (8)$$

Then, from equations (5) and (6):

$$\eta = 2\beta = \frac{1}{2\pi} \frac{\Delta W}{W} \quad (9)$$

As previously mentioned, soils do not exhibit viscous behaviour and the energy dissipated per cycle of motion is independent of frequency and depends instead on amplitude. This is because soils are characterized by material or hysteretic damping. The only way to reconcile the observed material behaviour and still insist on using a viscoelastic model is to assume that the viscosity is inversely proportional to frequency. Although from a physical point of view this is a rather inexact assumption, this is commonly done to arrive at a solid in which the energy dissipation per cycle of motion does not depend on frequency. In that case  $\beta$  is the fraction of linear hysteretic damping.

Equivalent linear viscoelastic models are defined by a constitutive relationship (for 1-D loading) of the type given by equation (3) where the complex valued  $G^*$  defined to yield the same stiffness and damping properties as the actual material. This complex modulus is defined from two experimental parameters (see Section 4.4:  $G$  and  $\eta$  or  $\beta$ ). Several models have been proposed to achieve this purpose. Their characteristics are defined in Table 4.

TABLE 4: CHARACTERISTICS OF EQUIVALENT VISCOELASTIC LINEAR CONSTITUTIVE MODELS

	Complex Modulus $G^* = \frac{\tau}{\gamma}$	Dissipated Energy in one Cycle $\Delta W$	Modulus $ G^* $
Material		$\pi G \eta \gamma_m^2$	$G$
Model1	$G = [1 + i\eta]$	$\pi G \eta \gamma_m^2$	$G\sqrt{1 + \eta^2}$
Model2	$G e^{i\theta}$ $\eta = 2\sin\left(\frac{\theta}{2}\right)$	$\pi G \eta \gamma_m^2 \sqrt{1 - \frac{\eta^2}{4}}$	$G$
Model3	$G [\sqrt{1 - \eta^2} + i\eta]$	$\pi G \eta \gamma_m^2$	$G$

In Table 4 model 1, which is the simplest one, adequately duplicates the dissipated energy but overestimates the stiffness; model 2 duplicates the stiffness but underestimates the dissipated energy and model 3 is the only one fulfilling both conditions. In standard practice, the most commonly used model is the model 2; this model is implemented in several frequency domain software programs as SHAKE, FLUSH, SASSI (see Section 9).

To account for the nonlinear shear behaviour, the parameters entering the definition of the complex modulus,  $G^*$ , are made strain dependent: the secant shear modulus (see Figure 9) and the damping ratio are plotted as function of shear strain, as depicted in Figure 11.

A common mistake in the characterization of the shear behaviour is to measure the  $G/G_{\max}$  curves in a laboratory under a given confining pressure and to consider that the same curve applies at any depth in the soil profile provided the material does not change. It is well known, however, that not only  $G_{\max}$  but also the shape of the curve depends on the confining pressure [43]. To overcome such a difficulty, and to keep the number of tests to a reasonable number, the correct representation is to normalize not only the modulus but also the strain  $G/G_{\max} = f(\gamma/\gamma_r)$ , where  $\gamma_r$  is a reference shear strain defined as the ratio of the maximum shear strength to the elastic shear modulus  $\gamma_r = \tau_{\max}/G_{\max}$  [46].

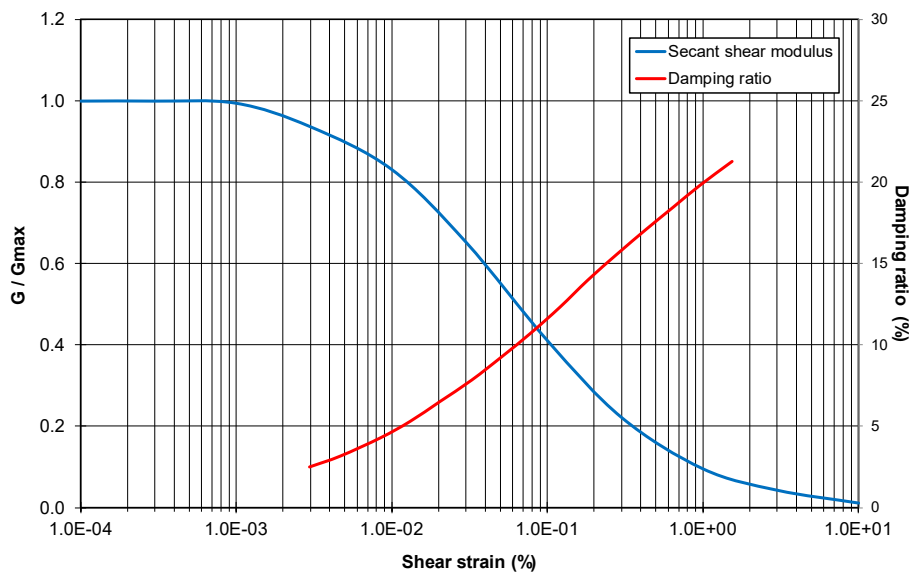


FIG. 11. Example of nonlinear characteristics of soil.

These viscoelastic models are implemented, in frequency domain solutions, with an iterative process in which the soil characteristics (secant shear modulus and equivalent damping ratio) are chosen at variance with the ‘average’ induced shear strain in order to reproduce, at least in an approximate manner, soil nonlinearities. They are typically considered valid for shear strains up to approximately  $2\gamma_r$  (which is of the order of  $1 \times 10^{-3}$  to  $5 \times 10^{-3}$  depending on the material type). Their main limitation, aside from their range of validity, is their inability to predict permanent deformations.

Extension to 3-D situations is straightforward in the framework of viscoelasticity. The constitutive relationship is simply written in terms of complex moduli<sup>9</sup>:

$$\sigma = C^*/\varepsilon \quad (10)$$

Where  $C^*$  is the tensor of elasticity expressed with complex moduli; for isotropic materials it is defined from the shear modulus  $G^*$  and bulk modulus  $B^*$  formed with the physical moduli and

<sup>9</sup> The following notations are used: tensors are in bold letters, the symbol ‘:’ is used for the double contraction of tensors (double summation on dummy indices).

the associated loss coefficients  $\eta_s$  and  $\eta_p$  for shear and volumetric strains; usually  $\eta_s$  is taken equal to  $\eta_p$ .

### 4.3.2. Nonlinear one-dimensional model

The definition of 1-D inelastic models starts with the definition of the so-called ‘backbone’, which establishes the non-linear stress–strain relationship  $\tau = f(\gamma)$  for monotonic loading. In this function, the ratio  $G_s = \tau/\gamma$  is the secant modulus.

Where  $\tau$  is shear stress,  $f(\gamma)$  is a function of shear strain,  $G$  is shear modulus.

In addition, for cyclic deformations, it is necessary to establish an unloading and re-loading rule, which defines the stress–strain path for non-monotonic loading. The most widely used rule is Masing’s rule [46] or the extended Masing’s rule [47]; such models have been initially developed by Iwan [48, 49] and further extended by Wang et al. [50] and Vucetic [51] among others. Assuming that the backbone curve is anti-symmetric with respect to the strain parameter, which is approximately true in shear when the Bauschinger effect is neglected, then the stress–strain relationship during loading-unloading sequences is defined by:

$$\frac{1}{2}(\tau - \tau_r) = f\left(\frac{1}{2}[\gamma - \gamma_r]\right) \quad (11)$$

in which  $(\tau_r, \gamma_r)$  are the coordinates of the point of last loading reversal. This formulation implies that the shape of the unloading or reloading curve is identical to the shape of the initial loading backbone curve enlarged by a factor of two. This model requires keeping track of the history of all reversal points, so that when an unloading or re-loading sequence intersects a previously taken path, that previous path is followed again as if no unloading or re-loading has taken place before. This is shown schematically in Figure 7 (above) where  $a$  is the initial loading path (backbone curve),  $ab$  the unloading path,  $bc$  the reloading path which, when reaching point  $a$ , resumes the backbone curve.

Several formulations have been proposed for the backbone curve among which the hyperbolic model [52-54] is the most popular one. In the hyperbolic model, the backbone curve is written:

$$\frac{\tau}{\tau_{\max}} = \frac{\gamma/\gamma_r}{1 + \gamma/\gamma_r} \quad (12)$$

Where  $\tau_{\max}$  represents the shear strength and  $\gamma_r$  the reference shear strain. Some modifications have recently been proposed to Eq. (11) to better fit the experimental data by raising the term  $\gamma/\gamma_r$  in the denominator to a power exponent  $\alpha$ . Such modification is not consistent with the behaviour at large strains since  $\tau/\tau_{\max} \sim (\gamma/\gamma_r)^{1-\alpha}$ ; except when  $\alpha = 1.0$  the strength tends asymptotically towards either 0 ( $\alpha > 1.0$ ) or towards infinity ( $\alpha < 1.0$ ).

The Ramberg-Osgood backbone curve is defined by the expression:

$$\gamma = \frac{\tau}{G_{\max}} \left[ 1 + \alpha \left( \frac{\tau}{\tau_y} \right)^{R-1} \right] \quad (13)$$

Where  $\alpha$  and  $R$  are dimensionless parameters and  $\tau_y$  an arbitrary reference stress.

For harmonic loading under constant amplitude strains, the secant shear modulus and loss coefficient are defined by:



- Hyperbolic model

$$\frac{G_s}{G_{\max}} = \frac{1}{1 + \frac{\gamma}{\gamma_r}} \quad , \quad \eta = \frac{4}{\pi \left(\frac{\gamma}{\gamma_r}\right)^2} \left[ \frac{\gamma}{\gamma_r} \left(2 + \frac{\gamma}{\gamma_r}\right) - 2 \left(1 + \frac{\gamma}{\gamma_r}\right) \ln \left(1 + \frac{\gamma}{\gamma_r}\right) \right] \quad (14)$$

- Ramberg-Osgood model

$$\frac{G_s}{G_{\max}} = \frac{1}{1 + \alpha \left(\frac{\tau}{\tau_y}\right)^{R-1}} \quad , \quad \eta = \frac{4 R - 1}{\pi R + 1} \frac{\alpha \left(\frac{\tau}{\tau_y}\right)^{R-1}}{1 + \alpha \left(\frac{\tau}{\tau_y}\right)^{R-1}} = \frac{4 R - 1}{\pi R + 1} \left(1 - \frac{G_s}{G_{\max}}\right) \quad (15)$$

Equations (14) and (15) establish the link with the parameters of the viscoelastic equivalent linear models and form the basis for the determination of the constitutive parameters.

Iwan [49] introduced a class of 1-D models that leads to stress–strain relations that obey Masing’s rule, for both the steady-state and non-steady-state cyclic behaviour once the initial monotonic loading behaviour is known. The concepts of the class of 1-D models are extended to 3-D and lead to a subsequent generalization of the customary concepts of the incremental theory of plasticity. These models can be conveniently simulated by means of an assembly of an arbitrary number of nonlinear elements, which can be placed (see Figure 12) either in parallel (elastoplastic springs) or in series (springs and sliders). The advantage of this model is that any experimental backbone curve can be approximated as closely as needed, and the numerical implementation is easy as compared, for instance, to the Ramberg-Osgood model: there is no need to keep track of all the load reversals.

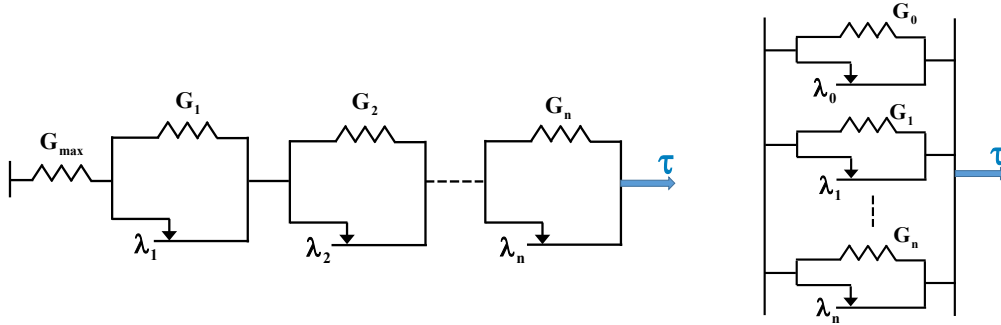


FIG. 12. Rheological models for Iwan’s nonlinear elastoplastic model.

Several software programs have implemented 1D-nonlinear constitutive models for site response analyses; the most commonly used are DEEPSOIL, DESRA, SUMDES, D-MOD, TESS. Stewart et al. [55] provides a complete description of these models.

1-D nonlinear models are convenient to describe the nonlinear shear strain–shear stress behaviour, but they cannot predict volumetric strains (settlements) that may take place even under pure shear.

#### 4.3.3. Nonlinear two and three-dimensional models

Unlike the viscoelastic models presented in Section 4.3.1, the extension of 1-D nonlinear models to general 2-D or 3-D states is not straightforward. Usually, such models are for soils, which are mostly rate-independent materials, based on the theory of incremental plasticity. Unlike in 1-D models, coupling exists between shear and volumetric strains; therefore, even in the ideal case of horizontally layered profiles subject to the vertical propagation of shear waves, elasto-plastic models will allow calculations of permanent settlements. Nonlinear models can be formulated so as to describe soil behaviour with respect to total or effective stresses. Effective stress analyses allow the modelling of the generation, redistribution, and eventual

dissipation of excess pore pressure during and after earthquake shaking. In these models, the total strain rate  $\dot{\epsilon}$  is the sum of an elastic component  $\dot{\epsilon}^e$  and of a plastic component  $\dot{\epsilon}^p$ . The incremental stress–strain constitutive equation is written:

$$\dot{\sigma} = \frac{C}{\dot{\epsilon} - \dot{\epsilon}^p} \quad (16)$$

Where  $C$  is the tensor of elasticity.

The models are fully defined with the yield surface that specifies when plastic deformations take place, the flow rule that defines the amplitude and direction of the incremental plastic deformation and the hardening/softening rule that describes the evolution of the yield surface. A large number of models of different complexity have been proposed in the literature; it is beyond the scope of this document to describe in detail these models. As noted previously, they can be viewed as extensions to 3-D states of Iwan's model. Among all the models, the most commonly used seem to be Prevost's models [56, 57] based on the concept of a multi-yield surface [58] and Wang's models [59] based on the concept of a bounding surface [60].

#### 4.4. ITERATIVE LINEAR MODEL AND ITS LIMITATIONS

In addition to their inability, already mentioned, to correctly model the soil behaviour at large strains, equivalent linear models present other drawbacks.

The frequently used program SHAKE and its derivative (see Section 10) construct its complex shear modulus using model 2 in Table 4, then it solves the linear wave equation in the frequency domain, does an inverse Fourier transform to calculate the time history of shear strain in each layer, chooses an equivalent shear strain (typically 2/3 of the maximum one) and reconstruct a complex shear modulus by picking up the secant shear modulus and damping ratio on the  $G/G_{max}$  curve and damping ratio curve, and iterates till the assumed  $G$  and damping values in the last iteration are similar to the calculated strain in the final iteration. Therefore, damping for SHAKE modelling is fully controlled by the damping curve. High frequencies are damped because the damping ratio and shear modulus are based on the strain, which is controlled by low frequencies, and the same damping is assigned to all frequencies. High frequency motions induce smaller strains and therefore need to be assigned less damping. Some attempts have been made to implement frequency dependent shear modulus and damping (e.g. Kausel and Assimaki [61]); however, these implementations have not yet received much attention in practice.

It is also well known that conventional numerical methods based on the fast Fourier transform (FFT) algorithm cannot be applied to the analysis of undamped systems, because of the singularities at the resonant frequencies of the system. In addition, those methods impose to add, at the end of the time history, a quiet zone of trailing zeroes of sufficient duration so as to damp out the free vibration terms and avoid wraparound. This duration is a function of the fundamental period of the system and the amount of damping and can be very large for lightly damped systems. For undamped systems, the free vibration terms will never decay and, therefore, the standard application of the FFT algorithm is no longer possible. A powerful general approach to obtain solutions with the FFT method for undamped or lightly damped systems is provided by the exponential window method (EWM) [62, 63].

In essence, the solution involves the following steps:

- (1) Computing the FFT of the excitation, modified by a decaying exponential window;
- (2) Calculating the transfer function of the system for complex frequencies;
- (3) Computing the inverse Fourier transform of the product of the transfer function by the FFT of the excitation;

- (4) Modifying the results by means of a rising exponential window. It is found that a quiet zone (a tail of trailing zeroes) is not needed for accurate computations, and that temporal aliasing (folding) is negligible.

This computational advantage is achieved at the expense of having to evaluate accurately the transfer functions at each frequency step, since interpolation schemes cannot be used in this method. This method originally developed in signal processing can be applied to continuous systems with an infinite number of resonant frequencies.

#### 4.5. PHYSICAL PARAMETERS

The essential parameters that need to be determined are the parameters for the constitutive models described in Section 4.2. In addition, the natural frequency of the soil profile is very important for the analyses: it represents a good indicator for the validation of the site modelling (geometry, properties), at least in the linear range, and is useful to constrain the range of possible variation of the soil's characteristics. Determination of the natural frequency is covered in Section 4.6.1.

The parameters for the constitutive relationships depend on the adopted model for the soil. For viscoelastic models, elastic (small strain) characteristics and variation of these characteristics with shear strain are needed. In view of the large uncertainties involved in soil–structure interaction problems, it is sufficiently accurate to consider an isotropic material, thereby reducing the numbers of moduli to 2: the shear and the bulk moduli, or equivalently the S-wave and P-wave velocities. Energy dissipation is represented by the loss coefficients (or damping ratios) associated to each modulus. Practically, due to the difficulty in damping measurements and to the scatter in the results, a single value of the loss coefficient is usually considered applicable to both moduli. These small strain characteristics depend mainly on the soil density and past and present state of stresses. The nonlinear shear strain–shear stress behaviour is characterized by the variation with shear strain of the secant shear modulus and loss coefficient (see Figure 11); these curves are mainly influenced by the present state of stresses and the soil plasticity index.

For 1-D nonlinear models, in addition to the previous parameters, the maximum shear strength that can be developed under simple shear loading is needed (see Eq. (17)); for dry soils or drained conditions, it can be computed from the knowledge of the strength parameters, friction angle  $\phi$  and cohesion  $c$ , and of the at rest earth pressure coefficient  $K_0$ . The latter parameter is the most difficult one to measure directly and is usually estimated from empirical correlation based on the friction angle and over-consolidation ratio of the soil.

$$\tau_{\max} = \left\{ \left[ \left( \frac{1+K_0}{2} \right) \sigma'_v \cdot \sin\phi + c \cdot \cos\phi \right]^2 - \left[ \left( \frac{1-K_0}{2} \right) \sigma'_v \right]^2 \right\}^{\frac{1}{2}} \quad (17)$$

For saturated soils under undrained conditions, the maximum shear strength is the cyclic undrained shear strength of the soil.

For the 2-D or 3-D nonlinear models the same parameters as for the 1-D nonlinear models are needed but, these models also involve a large number of additional parameters which do not all have a physical meaning; these parameters are “hidden” variables used to calibrate the constitutive model on the experimental data and depend on the formulation of the model. However, such parameters like the dilation angle and the rate of volumetric change under drained conditions, or pore pressure build up under undrained conditions, are essential physical parameters for these models. For the other model-specific parameters, tools or strategies may have been developed by the developers of the models to help their determination.

## 4.6. FIELD MEASUREMENTS AND LABORATORY MEASUREMENTS

Measurements of soil parameters are essential to classify the site geometry, to identify the soil strata and to estimate their behaviour under seismic loading. In addition, they need to provide the necessary parameters for the constitutive models that are used for the SSI analyses. These parameters can be determined from site instrumentation, field investigations and laboratory measurements.

### 4.6.1. Site instrumentation

Site instrumentation, as described in this section, is in addition to seismic instrumentation installed at the site to record the level of ground motion and in-structure response due to an earthquake. The basic motivation for this additional instrumentation is to record useful and important information to assess potential site effects and their modelling, and, consequently, reduce modelling uncertainty. Two different types of instrumentation may be implemented based on passive measurements of ambient vibrations or active measurements of seismic events.

The profile's natural frequency can be obtained with ambient vibration H/V measurements; this determination is easier when a strong impedance contrast exists between the soil layers and the substratum. As indicated in Section 4.1, this parameter is a good proxy to determine the importance of 2-D effects in the presence of a basin, even though H/V ratios do not allow the direct prediction of site amplification.

This method consists of measuring the ambient noise in continuous mode with velocity meters (not accelerometers) and then computing the ratio between the horizontal and vertical Fourier amplitude spectra [64]. Guidelines were produced by the SESAME research programme [65] to implement this technique, which is now reliable and robust. H/V measurements can provide the fundamental frequency of the studied site (but not the associated response amplitude) but can also be used to assess the depth to bedrock and its possible lateral variation when the technique is implemented along profiles. However, care needs to be taken when interpreting along the edge of basins, where the bedrock is significantly sloping, because 1-D geometry is assumed in the interpretation of measurements. The knowledge of the soil profile natural frequency is also important to validate the numerical model used for the analyses.

Ideally, active measurements of seismic events need to be made using instruments that allow recording of on-site (or in the vicinity) ground motions induced by real earthquakes. Based on these free field records, 'site to reference' transfer functions can be determined at various locations across the site. The site to reference transfer functions is useful to assess site amplification with respect to the reference and to calibrate the numerical model, at least in the linear range, provided that the 'reference' site is characterized.

### 4.6.2. Field investigations

The purpose of field investigations is to provide information on the site (stratigraphy, soil properties) at a large scale as opposed to the small scale involved in laboratory tests. They typically rely on borings, which provide information on the spatial distribution of soils (horizontally and with depth) and produce samples for laboratory analyses. However, other techniques, which do not involve borings, may provide essential information to characterize the site stratigraphy: for instance, the depth to the bedrock, which is an important parameter for site response analyses, can be assessed by high resolution seismic reflection survey techniques.

In addition to providing information on the site stratigraphy, essential dynamic properties for SSI analyses are measured from field investigations. These include the wave velocity profiles (P-wave and S-wave), which are converted into elastic, or small strain, soil parameters. Various field techniques for measuring in situ, shear and compressional, wave velocities exist [66, 67].

The most reliable and versatile techniques are invasive techniques based on in-hole measurements; these include downhole and cross hole tests, and also suspension logging tests. Use of cross hole tests with three aligned boreholes and one emitter and two receivers can be used to increase the reliability of the test interpretation: signal processing techniques can be used to identify, almost unambiguously, the time arrival of shear waves. When significant in-plane anisotropy is suspected, like in highly tectonized rock formations, cross hole tests with two receiver boreholes arranged in two orthogonal horizontal directions can be used. The invasive techniques can be advantageously complemented with non-invasive techniques like spectral analysis of surface waves (SASW), multichannel analysis of surface waves (MASW) or H/V measurements as described in the previous section [68]. Other field tests like the seismic cone test [69] may also be used (i.e. being less expensive than cross hole tests). Multiple seismic cone tests may provide information on the spatial variability of the soil properties; it is, however, important that at least one seismic cone test is calibrated against a cross hole test.

Invasive methods are considered more reliable than non-invasive ones because they are based on the interpretation of local measurements of shear wave travel times and provide good resolution. However, these methods involve drilling at least one borehole, making them quite expensive: non-invasive techniques provide more cost-efficient alternatives. In the last decades the methods based on the analysis of surface wave propagation are getting more recognition. These methods can be implemented with a low budget without impacting the nuclear installation site. However, they need processing and inversion of the experimental data, which needs to be carried out carefully. The surface wave inversion is indeed non-linear and is often an ill posed problem affected by solution non-uniqueness. This leads sometimes to very erroneous results causing a general lack of confidence in non-invasive methods in earthquake engineering.

The invasive techniques have no theoretical limitations with regard to the depth of investigation, non-invasive techniques are limited to shallow depth characterization, typically of the order of 20–50 metres. Furthermore, interpretation of MASW measurements implicitly assumes that the site is horizontally layered; therefore, they are not accurate for subsurface sloping layers. However, being less local than the invasive techniques, they can provide (at low cost) information on the spatial variation of soil parameters across the site. Furthermore, they may be very useful to constrain the variation of some parameters. For example, in the Pegasos Refinement Project [70], the dispersion curves measured in MASW tests were used to reject or keep possible alternatives of the soil velocity profiles, and thereby reduce the epistemic uncertainties in this parameter.

None of the currently available techniques are adequate for measuring the nonlinear characteristics of the soils; they are limited to the elastic domain and therefore need to be complemented with laboratory tests to allow for a complete characterization of the soil behaviour under the moderate to large strains that are applicable to seismic loading.

#### **4.6.3. Laboratory measurements**

Laboratory tests are essential to measure dynamic soil properties under various stress conditions, such as those that will prevail on the site after earthworks and construction of buildings, and to test the materials in the nonlinear strain range. They are therefore used to assess the variation with strain of the soil shear modulus and material damping. Combining field investigations and laboratory measurements is mandatory to establish a complete description of the material behaviour from very low strains to moderate and large strains. However, it is essential that these tests are carried out on truly undisturbed samples, as dynamic moduli are highly sensitive to remoulding. If sampling of fine cohesive soils can be efficiently performed, retrieving truly undisturbed samples in cohesionless uniform materials is still a challenge; some techniques however exist, like in-situ freezing or large core diameter sampling

that minimize the amount of remoulding. Freezing can disturb the sample if due care is not taken. Freezing the ground with a wave front propagating outward will avoid a big volume change: this technique is used in USA and Japan. Remoulding of samples can be qualitatively assessed by X ray diffraction; remoulding will be detected by the presence of curved shapes strata. Most laboratory tests have inherent shortcomings and therefore need to be carefully selected, performed by experienced people and preferably used in combination. These tests can be classified in three categories:

- (1) Wave propagation tests: shear wave velocities can be measured on laboratory samples using bender elements to measure the travel time of the S wave from one end to the other end of the samples. These tests are limited to elastic strains, like field tests, but unlike field tests they can be performed under various stress conditions; comparison between field measurements and bender tests is a good indicator of the quality of the samples.
- (2) Resonant tests: these tests are known as resonant column tests [71]. They are applicable to the measurements of soil properties from very small strains to moderate strains, typically of the order of  $5 \times 10^{-5}$  to  $1 \times 10^{-4}$ ; however, they cannot reach failure conditions. Depending on the vibration mode (longitudinal or torsional) Young's modulus and shear modulus can be measured. The damping ratio is calculated from the logarithmic decrement in the free vibration phase following the resonant phase. These tests can be performed under various stress conditions and are more accurate than forced vibration tests because the moduli are calculated from the knowledge of the sole resonant frequency of the specimen; no displacement or force measurements are involved.
- (3) Forced vibration tests: cyclic triaxial tests and cyclic simple shear tests belong to this category. Unlike the resonant column tests, measurements of applied force and induced displacement are required to calculate the moduli; therefore, inherent inaccuracies in both measurements immediately translate into errors in the moduli. Due to this limitation and to the classical size of samples (70 to 120mm in diameter) the tests are not accurate for strains below approximately  $10^{-4}$ . These tests can be performed under various stress conditions and can be conducted up to failure, enabling the determination of the sample strength. Damping ratio can be computed from the area of the hysteresis loop or from the phase shift between the applied force and the displacement. In simple shear tests, the shear modulus and the shear strain are directly measured, while in triaxial tests, Young's modulus  $E$  and axial strain ( $\epsilon$ ) are the measured parameters. Therefore, determination of the shear modulus from cyclic triaxial tests requires the knowledge of Poisson's ratio ( $\nu$ ):

$$\gamma = (1 + \nu)\epsilon \quad , \quad G = \frac{E}{2(1+\nu)} \quad (18)$$

Poisson's ratio is usually not directly measured but if the tests is conducted on a saturated sample under undrained conditions  $\nu = 0.5$ . However, special techniques based on the measurement of lateral strains are available for a direct measurement of Poisson's ratio [72]. In addition to the shear strain–shear stress behaviour, triaxial or simple shear tests are essential to measure the volumetric behaviour of the samples for calibration of 3-D nonlinear models or for the prediction, under undrained conditions, of the potential pore pressure build up. Cyclic triaxial tests, which are versatile enough to allow application of various stress paths to the sample, are the main tests available for full calibration of nonlinear constitutive models.

From the range of validity of each test, it appears that a complete description of the soil behaviour from very small strains up to failure can only be achieved by combining different tests. Furthermore, comparison of bender tests and resonant column tests with field tests is a good indicator of the sample representation and quality.

#### 4.6.4. Comparison of field and laboratory tests

As mentioned previously, field tests are limited to the characterization of the linear behaviour of soils while laboratory tests have the capability of characterizing their nonlinear behaviour. ASCE 4-16 [19] presents approximate ranges of applicability of tests.

Discrepancies between field-based parameters and laboratory-based ones, when measured in the same strain range (for instance resonant column tests and geophysical tests), may be observed. They may arise from laboratory tests from sample re-moulding or lack of representativeness of the sample or from field test measurement errors (erroneous detection of the wave arrival, reflection-refraction on layers of small thickness). These discrepancies be analysed and possibly reconciled. Typically, differences smaller than 50% on the shear modulus are acceptable (see Section 4.7, even with the best measurements, a COV of 0.15 is unavoidable on  $V_s$ ).

#### 4.6.5. Summary of parameters and measurement techniques

Table 5 summarizes the parameters needed for each constitutive assumption and the field and laboratory techniques needed to assess them. The list of software does not pretend to be exhaustive but simply reflects the most commonly used ones. They are limited to 2-D and 3-D software for SSI analyses and do not include codes for 1-D site response analyses.

### 4.7. CALIBRATION AND VALIDATION

Calibration and validation of the soil constitutive models is an essential step of the analysis process. These steps aim at ensuring that the experimental behaviour of materials under seismic loading is correctly accounted for by the models.

For elastic and viscoelastic linear constitutive models, calibration usually does not pose any problem; the experimental data (elastic characteristics,  $G/G_{\max}$  and damping ratio curves) measured either in situ and/or in the laboratory are directly used as input data to the models. Comparison with published results in the technical literature (e.g. Darendelli, 2001 [73], Ishibashi and Zhang, 1993 [74], EPRI, 1993 [75]) may also be useful for validation. However, these curves need to be used with caution and not replace site specific measurements. Validation will not be overlooked: results need to be critically examined since, as indicated previously, those constitutive models are only valid for strains smaller than a given threshold  $\gamma$ . If the results of analyses indicate larger strains, then the constitutive models need to be modified, and the use of nonlinear models needs to be considered.

TABLE 5: REQUIRED CONSTITUTIVE PARAMETERS

Model	Parameters			Measurement techniques		Software examples (*)
	Elastic	Yield / Failure	Dilation	Field	Laboratory	
Linear elastic				Crosshole, Downhole	Resonant column	ANSYS ABAQUS PLAXIS SAP2000 SOFISTIK RealESSI
	$G_0, B_0$	-	-	SASW MASW CPT	Bender	
Linear viscoelastic				Crosshole, Downhole	Resonant column	SASSI CLASSI MISS3D FLUSH RealESSI
	$G_0, B_0$ $\beta_p, \beta_s$	-	-	SASW MASW CPT	Bender	
Equivalent linear viscoelastic (including iterations)		$G(\gamma)$		Crosshole, Downhole	Resonant column	FLUSH SUPERFLUSH
	$G_0, B_0$ $\beta_p, \beta_s$	$B(\gamma)$ $\beta_p(\gamma)$ $\beta_s(\gamma)$	-	SASW MASW CPT	Bender Cyclic triaxial Cyclic simple shear	
Nonlinear elastoplastic (**)	$G_0, B_0$	Nonlinear shear and volumetric stress-strain curves Strength characteristics ( $C, \phi$ )	Dilation angle Dilation rate	Crosshole, Downhole SASW MASW CPT	Resonant column Bender Cyclic triaxial Cyclic simple shear	DYNAFLOW GEFDYN ABAQUS PLAXIS FLAC RealESSI LS-DYNA

(\*) See Section 10 for details on each software

(\*\*) The parameters relevant for nonlinear constitutive model are strongly dependent on the constitutive relationship; generic terms for parameters, which can range from a few to several tens are provided here. An example of complete description of a particular constitutive model is provided in Annex II. In Table 5, the following notations are used:

- $G$ : shear modulus;
- $B$ : bulk modulus;
- $\beta_s$ : damping ratio associated with S-waves;
- $\beta_p$ : damping ratio associated with P-waves; very often  $\beta_p$  is assumed to be equal to  $\beta_s$ ;
- $\gamma$ : shear strain;
- The subscript 0 is related to the elastic values (very small strains values).
- CPT: Cone Penetration Test

Calibration of nonlinear constitutive models is more fastidious and uncertain. For 1-D models, the shear strain–shear stress behaviour can be fitted to the experimental data using Eqs (14) and (15), but attention needs to be paid to the energy dissipation: elastoplastic models are known to overestimate this parameter, which means that the damping ratio in Eqs (14) and (15) is likely to exceed the experimental data.



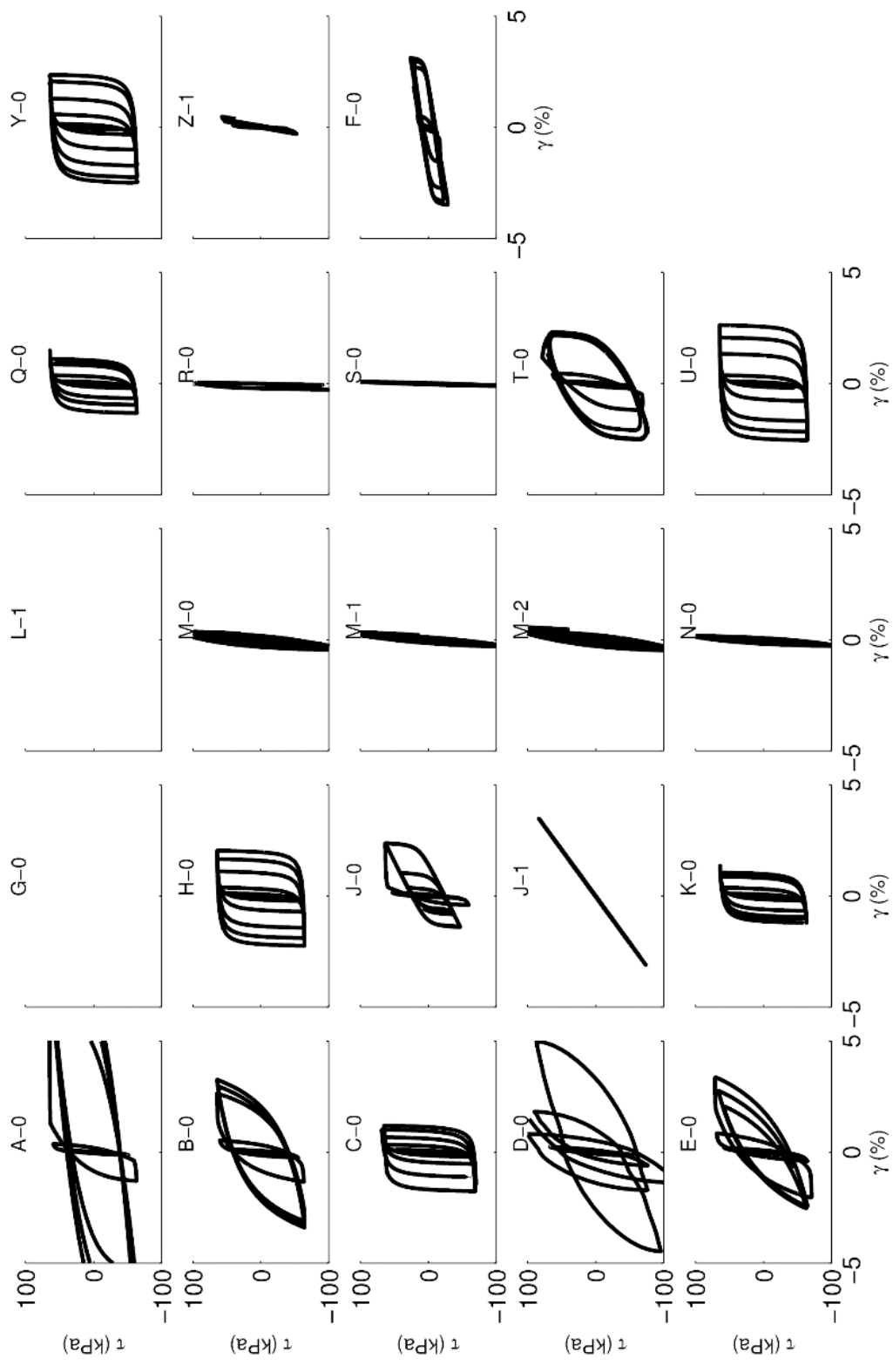
For nonlinear 3-D models, calibration is even more difficult due to the coupling between shear and volumetric strains. Calibration need not rely only on the shear stress–shear strain behaviour but also on the volumetric behaviour. Therefore, laboratory tests providing the required data are warranted. Usually, calibration is best achieved by carrying out numerical experiments duplicating the available experiments. Validation consists of reproducing additional experiments that were not used for calibration; these validations need to be performed with the same constitutive parameters and for different stress conditions and stress (or strain) paths.

Elastic plastic material models are to be used for 3-D modelling of soil need to feature rotational kinematic hardening, in order to reproduce cyclic behaviour. In addition, 3-D elastic-plastic material models need to be able to reproduce volume change, where and if dilatancy effects are deemed important (see earlier discussion in this section). For example, material models in the SaniSand and SaniClay family of models [76] are able to reproduce most of these effects, however they require extensive laboratory testing for calibration. On the other hand, models by Prevost and Pospescu [41] that are based on rotational kinematic hardening concept have also been successfully used. In addition, recent models, developed in particular to match  $G/G_{max}$  and damping curves [77] can be used, and need only 5 parameters. The most important point is that these full 3-D elastic-plastic material models are able to reproduce 1-D cyclic behaviour of soil and are defined in full 3-D and are thus able to work for general 3-D problems.

The variability in predictions of constitutive models may be very large. This is illustrated by a recent benchmark carried out within the framework of the SIGMA project [78]. A sample with a prescribed shear strength of 65kPa was subjected to a 10 cycle, quasi harmonic input motion, modulated by a linear amplitude increase, and its behaviour predicted by different models with their associated hysteresis curves, as depicted in Figure 13. The full duration of motion leads to very high strain levels (5%), and the stress–strain curves are highly variable from one computation to another. The main differences were attributed to the inability of some models to mimic the prescribed shear strength value and to differences in the way energy dissipation is accounted for by the models. One essential conclusion of the benchmark was that detailed calibration of models is essential and that, in practice, it would be advantageous to use at least two different models for the analyses.

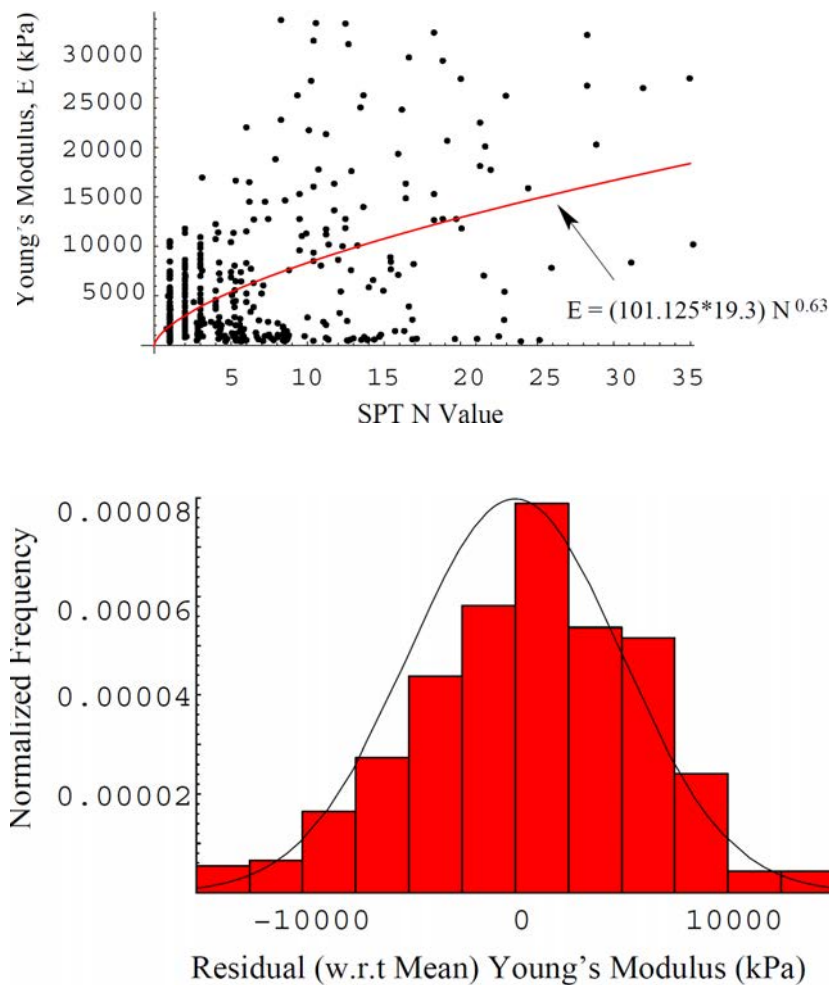
When detailed experimental data are not available for calibration, Eqs (14) and (15) can still be used for the definition of the shear stress–shear strain behaviour. Liquefaction resistance curves, like those derived from Standard Penetration Test (SPT) or CPT tests, can then be used to calibrate the volumetric behaviour [41]; however, this approach is much less accurate since only the global behaviour is predicted and not the detailed evolution of pore pressure (or volumetric strain rate).

Inelastic models, therefore, need to be used with extreme care as these lead to large discrepancies in predictions, and thus uncertainties in the results.



#### 4.8. UNCERTAINTIES

Analysis of soil properties in a homogeneous soil layer are affected by a series of uncertainties, such as inherent variability (see Section 4.9), random test errors and systematic test errors. Figure 14 shows uncertainty data from SPT tests, which is used to calibrate (curve fit) an equation for elastic modulus, with a large (think tail) residual. It is common practice to assume, in the absence of precise site specific data, that the elastic shear modulus can vary within a factor of 1.5 around the mean value [18, 45, 80, 81]. If additional testing is performed, the range could be widened to values obtained by multiplying or dividing the mean value by  $(1+COV)$  where COV is the coefficient of variation. Based on the benchmark results described below, the value of COV needs to be greater than 0.5.



In the framework of the SIGMA project [78], the Inter PACIFIC project (Inter comparison of methods for site parameter and velocity profile characterization) compares non-invasive and invasive methods in order to evaluate the reliability of the results obtained with different techniques. Three sites were chosen in order to evaluate the performance of both invasive and non-invasive techniques in three different subsoil conditions: soft soil, stiff soil and rock. Ten different teams of engineers, geologists and seismologists were invited to take part in the project in order to perform a blind test. The standard deviation of  $V_S$  values at a given depth is normally higher for non-invasive techniques ( $COV = 0.1-0.15$ ) than for invasive ones ( $COV < 0.1$ ) for the three tested sites [82]. These values are significantly less than those recommended by ASCE

[19] which for the modulus would imply that the mean value needs to be multiplied by at most  $1.15^2$  (i.e., 1.3). Ten highly specialized different teams made their own evaluation, and it can be considered that these values are minimum threshold values with no hope of achieving smaller uncertainties.

#### 4.9. SPATIAL VARIABILITY

The necessarily limited number of soil tests and their inherent lack of representation are significant sources of uncertainty in the evaluation of site response analyses, while the uncertainty associated with the accuracy of analytical or numerical models used for the analysis is generally less significant. Spatial variability may affect the soil properties but also the layer thicknesses. Soil properties in a homogeneous soil layer are affected by a series of uncertainties, for example as described in Assimaki et al, 2003 [83]. These uncertainties include: inherent spatial variability, random test errors, systematic test errors (or bias) and transformation uncertainty (from index to design soil properties). Since deterministic descriptions of this spatial variability are in general not feasible, the overall characteristics of the spatial variability and the uncertainties involved are mathematically modelled using stochastic (or random) fields. Based on field measurements and empirical correlations, both Gaussian and non-Gaussian stochastic fields are fitted for various soil properties; however, according to Popescu, 1995 [84], it is concluded that:

- (a) Most soil properties exhibit skewed, non-Gaussian distributions;
- (b) Each soil property can follow different probability distributions for various materials and sites, and therefore the statistics and the shape of the distribution function needs to be estimated for each case.

It is important to realize that the correlation distances differ widely in natural deposits between the vertical and the horizontal directions. In the vertical direction, due to the deposition process, the correlation distance is typically of the order of a metre or less, while in the horizontal direction it may reach several metres. Consequently, an accurate definition of a site specific distribution can never be achieved in view of the large number of investigation points that would be needed. For low frequency motions, a reasonable estimate can be obtained. On the other hand, a description of small scale heterogeneities is clearly not possible and one needs to rely on statistical data collected on various sites.

## 5. SEISMIC HAZARD ANALYSIS FOR NUCLEAR INSTALLATIONS

Seismic input is one of the most important parameters needed in the design of a nuclear installation at any site. This parameter is determined by carrying out detailed SHA that takes a long time and involves significant resources because a very large area around the site has to be thoroughly investigated. This assessment provides the DBE ground motion after establishing the site suitability, especially against surface rupture at the nuclear installation site. The basic concept and methodology for seismic hazard evaluation are described in SSG-9 (Rev. 1) [28], which provides recommendations on: geology, geophysics, geotechnical and seismology databases; construction of a regional seismotectonic model comprising of seismogenic structures and zones of diffused seismicity; evaluation of the ground motion hazard using both DSHA and PSHA; and the investigations necessary to determine the potential for fault displacement at the site.

Figure 15 shows the different steps of SHA. The elements of an SHA are:

- (a) Data collection and developing seismotectonic models:

- Existing geological, geophysical, and geotechnical data complemented by seismological data are collected, and placed in a database (if not already in a geographical information system database);
  - The data is reviewed, and recommendations made on obtaining additional data, if deemed necessary;
  - The results are documented.
- (b) Seismic source characterization:
- Seismic sources (faults and area sources) to be considered in the SHA are identified;
  - The faults and area sources are characterized.
- (c) Selection of GMPEs supplemented by seismic source simulations:
- The PGA, peak ground velocity, peak ground displacement and the response spectral acceleration values at specified natural frequencies (generally 5% damped) are determined;
- (d) Hazard estimates for the site are quantified by DSHA and/or PSHA methods. Any uncertainties are propagated and displayed in the final results;
- (e) Site Response Analysis (SRA), as needed for input to the soil–structure system.

The important elements with respect to SSI are the steps associated with the vibratory ground Motion Hazard Analysis, including PSHA and/or DSHA and SRA.

### 5.1. PSHA PERSPECTIVE

If the performance goal of a nuclear installation or a structure, system, or component within the installation is probabilistically defined, a basic prerequisite is the development of a site specific, probabilistically defined seismic hazard plan that is associated with the site. The seismic hazard is often termed ‘seismic hazard curve’, which represents the annual frequency (or rate) of exceedance (AFE) for different values of a selected ground motion parameter, for example the PGA or the response spectral acceleration at specified spectral frequencies. In the latter case, the PSHA process defines seismic hazard curves for response spectral accelerations over a range of spectral frequencies, for example, 5–20 discrete natural frequencies ranging from 0.5 Hz to 100 Hz, and for a specified damping value, usually 5%.

A uniform hazard response spectrum (UHRS) is constructed of spectral ordinates each of which has an equal AFE.

The natural frequencies of calculation are referred to as ‘conditioning frequencies’ in some applications. To define a UHRS at an AFE, the discrete values of acceleration (or spectral acceleration) at the requested AFE of each of the natural frequency hazard curves are selected and the values are connected by segmented lines in a log–log plot or are fitted with a curve. This becomes the UHRS at the AFE at the location of interest and form of interest (in-soil or outcrop).

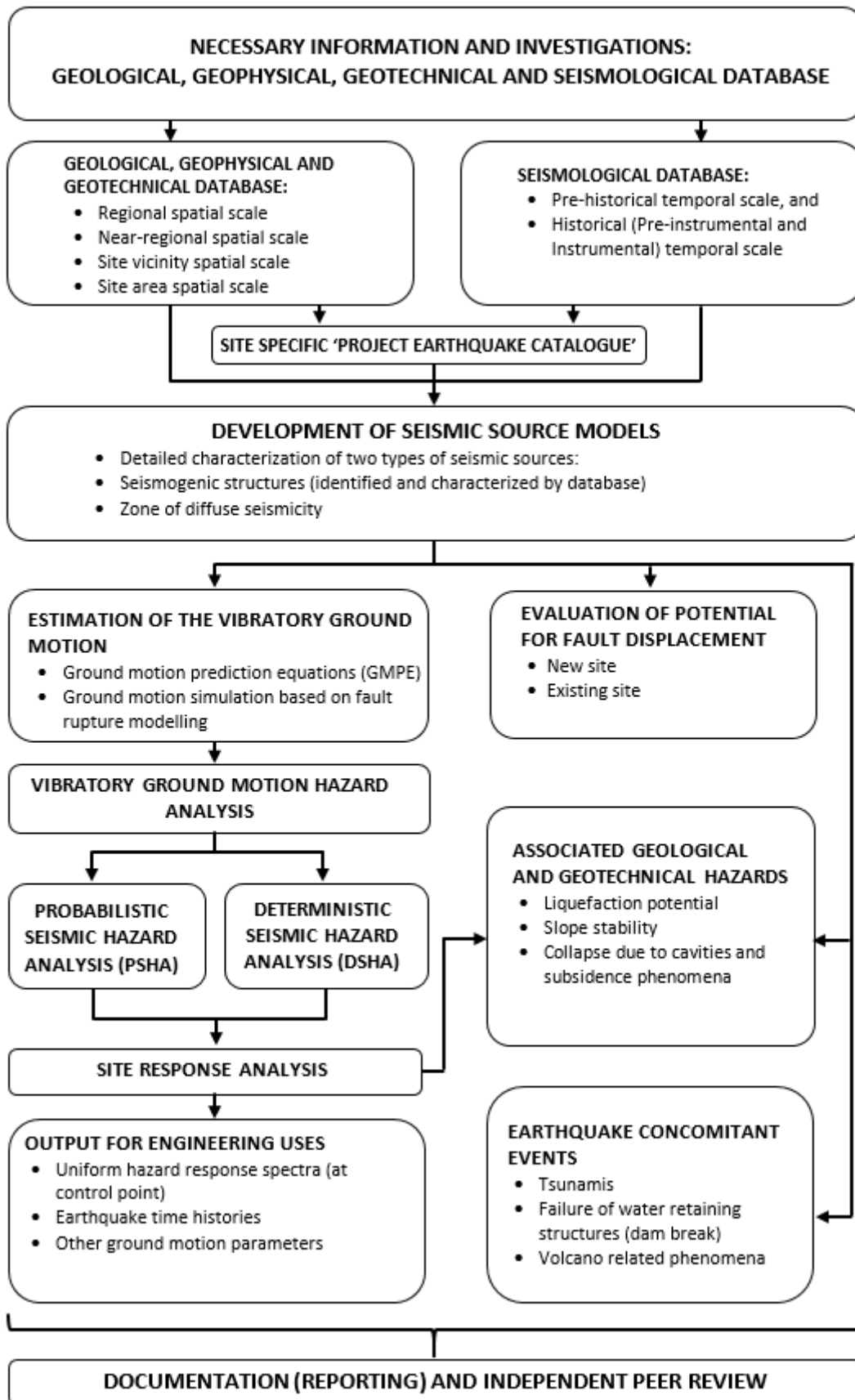


FIG. 15. Seismic Hazard Assessment Steps (reproduced from Ref. [28]).

## 5.2. DSHA PERSPECTIVE

Many of the steps in Figure 15 are equally applicable to DSHA. Recommendations on DSHA are provided in SSG-9 (Rev. 1) [28]. The differences are primarily: the probabilistic treatment of all aspects of the procedure for the PSHA, including unconditional AFE values; the inclusion of all faults and seismic zones explicitly in the PSHA; consideration of all possible locations on a fault or in a seismic zone as equally likely for the PSHA compared to conservatively selected for the DSHA; consideration of all credible GMPE for the PSHA and a selected subset for the DSHA. The results of the PSHA are fully probabilistic and are intended to be conservative through the selection of mean or higher non-exceedance probability (NEP) values (such NEP values are derived through conservative assumptions in the DSHA as part of the steps of the SHA. In general, PSHA and DSHA involve the performance of site response analysis.

Further recommendations on PSHA and DSHA are provided in SSG-9 (Rev. 1) [28].

## 5.3. INTERFACES BETWEEN THE SEISMIC HAZARD ANALYSIS AND THE SOIL-STRUCTURE INTERACTION ANALYSIS TEAMS

There are important interfaces between the SHA team and the end users of the results of the PSHA (or DSHA). The end users include:

- (a) SSI analysts responsible for SSI analyses of structures;
- (b) Geotechnical engineers responsible for soil characterization;
- (c) Civil/structural/mechanical engineers responsible for the design and assessment of soil-founded components (e.g. tanks), and buried underground systems and components (e.g. buried pipes, cable chases);
- (d) Fragility analysts (civil/structural/mechanical/geotechnical engineers) responsible for developing fragility functions or seismic capacity values (e.g. HCLPF values) for assessment of the BDBE performance of SSCs for nuclear installations;
- (e) Risk analysts responsible for risk quantification or seismic margin capacity of the nuclear installation.

The work plan to be issued to the SHA Team will specify conditions, such as those described in SSG-9 (Rev. 1) [28] and provide a typical list of PSHA output quantities. In addition, the following supplementary information could be requested:

- (a) The kappa values implemented in the hazard analysis;
- (b) Whether the seismic hazard curves (horizontal components) are in terms of geometric mean values or peak values;
- (c) A commentary on uncertainties, especially the potential issue of double-counting aleatory and/or epistemic uncertainties in the PSHA results and in the site response analyses (or other site effects analyses);
- (d) The elements included in the seismic hazard determination;
- (e) If DSHA is associated with a return period or AFE, the value and uncertainties that have been included (e.g., a return period of 20,000 years corresponding to an  $AFE=5 \times 10^{-5}$ );
- (f) The sets of time histories or response spectra (converted to random vibration theory (RVT) representations) to be used in the site response analyses, if performed;
- (g) The time histories to be used in SSI analyses if site response analyses are not performed;
- (h) If PSHA produces results at locations other than hard rock (or soft rock), are the additional results in-column or outcrop? If outcrop, will a geological model or full column method be used?
- (i) The V/H ratio at all locations within the profile from hard rock (derived seismic hazard curves - SHCs - from PSHA) to locations in the soil profile;

- (j) How to change the UHRS from 5% damping to other smaller and larger damping values.

## 6. SEISMIC WAVE FIELDS AND FREE FIELD GROUND MOTIONS

### 6.1. SEISMIC WAVE FIELDS

#### 6.1.1. Perspective and spatial variability of ground motion

Earthquake motions at the location of interest are affected by a number of factors [85–87]. Three main influences are:

- (a) **Earthquake Source:** An earthquake is caused by the release of built-up stress within the rocks along geologic faults resulting in a rapid stress drop in the crustal medium with a consequent release of energy. Part of this energy causes the rupture propagation along the fault plane. Another part of energy propagates in the crustal medium as elastic waves. From a kinematic point of view, the seismic source is described as a slip distribution starting from a nucleation point and propagating along the fault plane at a given rupture velocity. The seismic moment ( $M_0$ ) is defined as the product between the crust rigidity, the fault area, and the average slip. The moment magnitude is given by  $\log_{10}(M_0) = 1.5 M_w + 9.1$  ( $M_0$  is the moment expressed in Nm). The source mechanism is described using 3 angles: strike (orientation of the fault plane with respect to the North), dip (orientation with respect to the vertical), and rake (orientation of the slip);
- (b) **Earthquake Wave Path:** Elastic waves propagate from the fault slip zone in all directions. Some of those (body) waves travel upward toward the surface, through stiff rock at depth and, close to the surface, through soil layers. The crust is characterized by heterogeneous mechanical and rheological properties, and those heterogeneities affect the elastic wave propagation. Body waves are (P) Primary waves (compressional waves, fastest) and (S) Secondary waves (shear waves, slower). Secondary waves that feature particle movements in a vertical plane (polarized vertically) are called SV waves, while secondary waves that feature particle movements in a horizontal plane (polarized horizontally) are called SH waves;
- (c) **Shallow, surface layers response:** Seismic body waves propagating from rock and deep soil layers to the surface and interacting with the ground surface create surface seismic waves. Surface waves and shallowbody waves are responsible for SSI effects. Local site conditions (type and spatial distribution of soil near surface), local geology (basins, inclined rock layers, dykes, etc.) and local topography can have significant influence on seismic motions at the location of interest.

In general, seismic motions at surface and shallow depths consist of (shallow) body waves (P, S, SH, SV) and surface waves (Rayleigh, Love, etc.). Shallow depth is approximately one wavelength in depth where surface waves have significant amplitudes. The depth of propagation of surface waves is a function of wavelength and thus lower frequency waves propagate deeper than higher frequency waves. Since (most commonly) the stiffness of soils and rock increases with depth, surface waves of lower frequency travel faster than surface waves of higher frequency. Hence, surface waves are said to be dispersive.

Sometimes it is possible to analyse SSI effects using a 1-D wave simplification (see Section 7.3.3), however such simplification needs to be carefully assessed.



Modelling of seismic waves (P and S) is based on Snell's law of wave refraction. Seismic waves travel from great depth (many kilometres) and as they travel through horizontally layered media (rock and soil layers), where each layer features different wave velocity (stiffness) that decreases toward the surface, waves will bend toward the vertical [85]. However, even if rock and soil layers were actually horizontal, and the earthquake source was very deep, near the surface, the seismic waves will not travel along an ideal vertical path but exhibit some inclination that is typically of the order of 10-20 degrees when they reach the surface. As noted above, a change in stiffness of rock and soil layers results in seismic wave refraction, as shown in Figure 16. Thus, usually small deviations from vertical might not be important for practical purposes. However, such deviation from vertical will produce surface waves, the presence of which can have practical implications for SSI analysis. In addition, presence of valley and basin edges (local geology) will also generate surface waves.

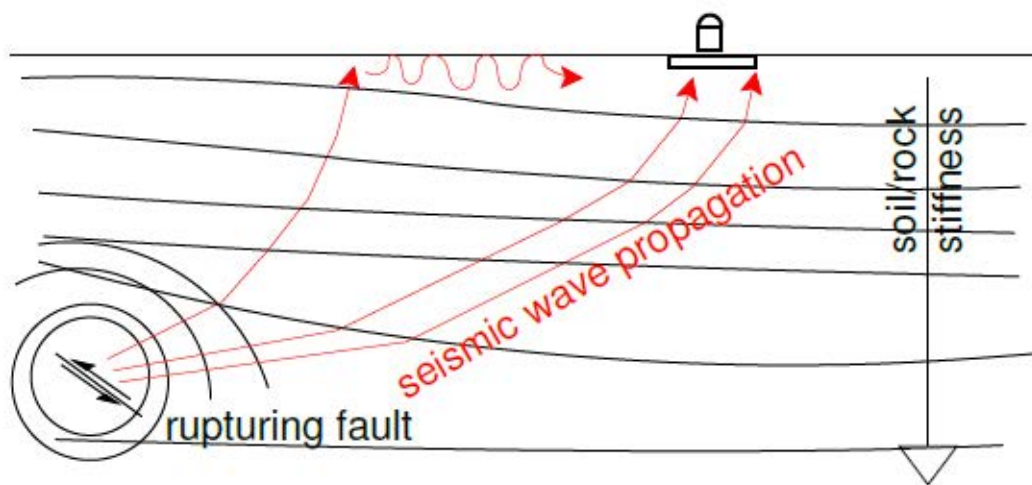


FIG. 16. Propagation of seismic waves in nearly horizontal local geology, with stiffness of soil/rock layers increasing with depth, and refraction of waves toward the vertical direction (reproduced from Ref. [44] with permission courtesy of [Canadian Nuclear Safety Commission]).

A more important source of seismic wave deviations from vertical is the fact that rock and soil layers are usually not quite horizontal. A number of different geological history effects contribute to non-horizontal distribution of layers. Figure 17 shows one such (imaginary but not unrealistic) case where inclined soil/rock layers contribute to refracting seismic wave propagating into a horizontal direction. Rock basins as well as hard rock protrusions (dykes) are also common and contribute to deviation of seismic wave propagation from vertical.

It is important to note that in both the horizontal and non-horizontal soil/rock layered cases, surface waves are created which carry most of the energy near the surface of significance for SSI effects.

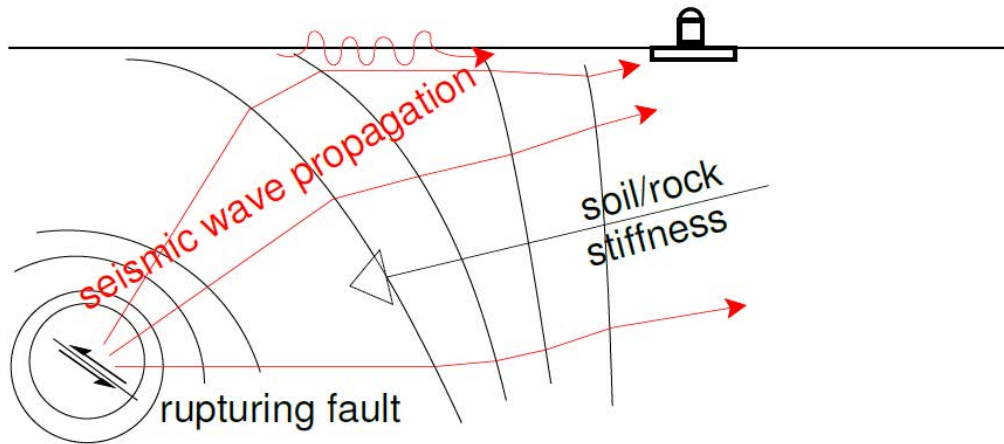


FIG. 97. Propagation of seismic waves in inclined local geology, with stiffness of soil/rock layers increasing through geologic layers, and refraction of waves away from the vertical direction. (reproduced from Ref. [44] with permission courtesy of [Canadian Nuclear Safety Commission]).

### 6.1.2. Spatial variability of ground motions

Spatial variations of ground motion refer to differences in amplitude and/or phase of ground motions with horizontal distance or depth in the free field. As introduced in Section 6.1.1, these spatial variations of ground motion are associated with different types of seismic waves and various wave propagation phenomena. Different wave propagation phenomena include reflection at the free surface, reflection and refraction at interfaces and boundaries between geological strata having different properties. Other contributing factors are diffraction and scattering induced by non-uniform subsurface geological strata and topographic effects along the propagation path of the seismic waves.

Prior to the early 1990s, scepticism existed as to the wave propagation behaviour of seismic waves in the free field and their spatial variation with depth in the soil. This scepticism arose from several sources; one of which was the lack of recorded data at shallow depths to provide recorded evidence of the variability of motion with depth in the soil profile as predicted by wave propagation theory. Since the 1980s, accumulated direct and indirect data have verified these phenomena, as follows:

- (a) Direct data are measurements of free field ground motion at depths in the soil. Johnson [88] summarises the existing data as of 2003. Substantial additional direct data has been accumulated over the intervening decade, especially from Japan with recordings from K-NET and KiK-Net<sup>10</sup>. Section 6.3 summarizes important data acquisition over the complete time period;
- (b) Indirect data are measurements of response of structures with embedded foundations demonstrating reductions of motion on the foundation compared with free field ground

<sup>10</sup> "K-NET (Kyoshin network) is a nation-wide strong-motion seismograph network, which consists of more than 1,000 observation stations distributed every 20 km uniformly covering Japan. K-NET has been operated by the National Research Institute for Earth Science and Disaster Resilience (NIED) since June 1996. At each K-NET station, a seismograph is installed on the ground surface with standardized observation facilities. KiK-net (Kiban Kyoshin network) is a strong-motion seismograph network, which consists of pairs of seismographs installed in a borehole together with high sensitivity seismographs (Hi-net) as well as on the ground surface, deployed at approximately 700 locations nationwide. The soil condition data explored at K-NET stations and the geological and geophysical data derived from drilling boreholes at KiK-net stations are also available." (Source <http://www.kyoshin.bosai.go.jp/>)

motions recorded on the free surface. These reductions are due to spatial variation of motion with depth in the soil and due to horizontal and vertical variations of frequency content due to incoherence of ground motions [88, 89]. More recent work suggests that reductions might be in part due to nonlinear effects (seismic energy dissipation) within the soil and the contact zone adjacent to structural foundations [90].

As introduced in Section 6.1.1, 1-D and 3-D wave fields consist of particle motion that demonstrates spatial variation of motion with depth in the soil or rock media. For 1-D wave propagation, a vertically incident body wave propagating in such a media will include ground motions having identical amplitudes and phase at different points on a horizontal plane. A non-vertically incident plane wave will create a horizontally propagating (surface) wave at some apparent phase velocity and will induce ground motion having identical amplitudes but with a shift in phase in the horizontal direction associated with the apparent horizontal propagation velocity of the wave. In either of these ideal cases, the ground motions are considered to be coherent, in that the acceleration time histories do not vary with location in a horizontal plane – only appearing with a time lag. Incoherence of ground motion, on the other hand, may result from wave scattering due to inhomogeneity of soil/rock media and topographic effects along the propagation path of the seismic waves. Both of these phenomena are discussed below.

In terms of the SSI phenomenon, spatial variations of the ground motion over the depth and width of the foundation (or foundations for multi-foundation systems) are an important aspect. For surface foundations, the variation of motion on the surface of the soil is important; for embedded foundations, the variation of motion on both the embedded depth and foundation width is important. Overall free field ground motion analysis is discussed next. Section 7 presents site response analyses.

## 6.2. FREE FIELD GROUND MOTION DEVELOPMENT

There are four basic modelling approaches that are used to develop ground motions: Empirical GMPEs, Point Source Stochastic Simulations, FFS, and the HEM.

### (1) Empirical GMPEs:

Empirical GMPEs are calibrated using available (regional) data, however they often need to be extrapolated beyond regions where data was collected, hence they might not be well constrained by the empirical data for, for example, short distances and large magnitudes. To expand the empirical dataset for large magnitudes and short distances, empirical GMPEs are often based on global datasets, thus implicitly assuming that the motions are statistically stationary and ergodic. Often, these assumptions may not be warranted, but are still adopted as a matter of practicality and convenience. Such GMPEs developed based on global data sets may not capture the region-specific attenuation in low to moderate seismicity regions. Corrections that are used to accommodate site specific conditions (such as  $\kappa$ ) are not straightforward and are the main contributor to the uncertainty in GMPE models.

### (2) Point Source Stochastic Simulations:

The point source stochastic model proposed by Boore [91] is the simplest numerical simulation method available based on seismological theory. Models are developed for the Fourier amplitude spectrum and the duration of shaking. Random vibration theory is then used to convert the Fourier amplitude spectrum and the duration to response spectral values.

There are six main input parameters for the point source model: earthquake magnitude, stress-drop, geometrical spreading, quality factor, crustal amplification, and high frequency attenuation ( $\kappa$ ). Region-specific models of the geometrical spreading and quality factor are often determined empirically using recordings from smaller earthquakes in the region

of interest. The duration is either computed using simple analytical models or using region-specific models based on empirical observations.

The small magnitude region-specific data does not provide constraints on the stress-drops of larger magnitude earthquakes, which is the major source of uncertainty in the application of the stochastic model. The site specific kappa value is also a key contributor to the uncertainty;

### (3) Finite-fault simulations:

FFS for large scale regional computations provide a physical basis for the extrapolation from small magnitudes to larger magnitudes by incorporating finite-fault effects [92–95]. However, they have a much larger number of input parameters and therefore need more calibration before FFS can be reliably applied to engineering applications. The FFS methods have not yet been possible for the high frequencies of interest to nuclear installations however, there are current projects that will extend modelling of frequencies up to and above 10Hz. The science behind the FFS is improving rapidly and FFS will likely be sufficiently far advanced to allow them to be included as alternative models in the next generation of seismic hazard evaluation for nuclear installations.

Currently, FFS are sometimes used to develop ground motion models as an alternative to empirical GMPEs. The FFS represent ‘technically defensible interpretations’ if all necessary input parameters can be reasonably well constrained.

The FFS per se do not require a kappa value, but the broadband simulation methods apply a kappa filter such that the simulated ground motion will match the specified target kappa. In that respect, the FFS results for high frequency remain empirically constrained.

### (4) HEM Models:

The hybrid empirical model [96] is a combination of the empirical GMPE approach and the PSSS model approach. In the HEM, point source stochastic models are developed for both the host GMPE region and the target site region capturing the region-specific parameters for both regions (stress-drop, geometrical spreading, quality factor, crustal amplification, and kappa). The stochastic model is then used to compute the response spectral scale factors from the host region to the target region for a given magnitude and distance. These factors are then applied to the host region GMPE.

A key assumption for this method is that response spectral scale factors for the point source model are applicable to the GMPE. Because response spectral scale factors at a given frequency depend on the underlying spectral shape, this assumption is only valid if the spectral shape of the GMPE is similar to the spectral shape of the point source model. To resolve this issue Vs-kappa correction need to be applied (e.g., see [97]).

Site response analyses are performed to establish the seismic input motions to the SSI analyses taking into account nonlinear behaviour of the local site properties (see Section 7).

Site response analyses are currently (usually) performed for the assumptions of 1-D wave propagation and horizontally layered soil/rock profiles. It is becoming increasingly necessary to consider 2-D or 3-D site response analysis to generate seismic input to the SSI analyses or, as a minimum, to justify the applicability of 1-D site response analysis. This justification applies to the effects of 3-D wave fields for sources close to the site and the effects of local geology/site conditions, such as non-horizontal soil layers, hard rock intrusion (dykes), basin effects, and topographic effects (presence of hills, valleys, and sloping ground) [83, 98, 99].

It is important to note that realistic seismic motions always have 3-D features. That is, seismic motions feature three translations (and three rotations, obtained from differential displacements

between closely spaced points (from few metres to a few dozen metres to few hundred metres divided by the distance between those points) at each point on the surface and shallow depth. Rotations are present in shallow soil layers due to Rayleigh and Love surface waves, which diminish with depth as a function of their wavelength [85]. As noted above, rotations appear from differential vertical (and horizontal) motions at closely spaced points on soil surface and at some depth. Seismic wave traveling effects will produce differences in vertical motions for such closely spaced point, thus producing rotational motions for stiff objects founded on surface (or shallow depth).

For probabilistic site response analysis, it is important not to count uncertainties twice [100]. This double counting of uncertainties stems from accounting for uncertainties in both free field analysis (using GMPEs for soil) and then also adding uncertainties during site response analysis for topsoil layers.

Section 7 presents site response analysis in more detail.

### 6.3. RECORDED DATA

There exist a large number of recorded earthquake motions. Most records feature data in three perpendicular directions, East-West (E-W), North-South (N-S) and Up-Down (U-D). A number of publicly available strong motion databases exist, mainly in the east and south of Asia, the west coast of North America and South America, and Europe. There are regions of the world that are not well covered with recording stations. The regions that are not covered with recording stations are quite seismically inactive. However, in some of those regions, return periods of (large) earthquakes are long, and recording of even small events would greatly help gain knowledge about tectonic activity and geology.

The development of models for predicting seismic motions based on empirical evidence (recorded motions) relies on the ergodic assumption. The ergodic assumption allows statistical data (earthquake recordings) obtained at one (or a few) worldwide location(s), over a long period of time, to be used at other locations. This assumption allows for the substitution of recordings over a large number of locations and time to be applied to the site of interest as a statistical meaningful sample.

While ergodic assumptions are frequently used, there are issues that need to be addressed when it is applied to earthquake motion records. For example, earthquake records from different geological settings are used to develop GMPEs for specific geologic settings (again, different from those where recordings were made) at locations of interest.

Current efforts focus on the development of non-ergodic, site specific models using data from the site, including measurements of very small earthquakes. It is expected that non-ergodic, site specific models will become available in near future for certain parts of the world, while one can expect non-ergodic, site specific motions to be developed for most other sites of interest soon thereafter.

#### 6.3.1. 3-D versus 1-D records/motions

Recordings of earthquakes around the world show that earthquakes are almost always featuring all three spatial components (E-W, N-S, U-D). There are very few known recorded events where one of the components was not present or is present in a much smaller magnitude. Presence of two horizontal components (E-W, N-S) of similar amplitude and appearing at about the same time is quite common. The four cardinal directions (North, East, South and West) which humans use to orient recorded motions have little to do with mechanics of earthquakes. The third direction, Up-Down, is different. Presence of vertical motions before

main horizontal motions appear, signify arrival of primary (P) waves (hence the name) or non-vertically propagating S waves (secondary, that arrive a bit later). The presence of vertical motions when horizontal motions appear, indicates the presence of inclined S waves and, more importantly, Rayleigh surface waves. If vertical motions are not present (or have very small magnitude) during horizontal motions, this indicates that Rayleigh surface waves are not present. Lack of Rayleigh (surface) waves is a very rare occurrence, where a combination of source, path and local site conditions produces a plane shear (S) wave that surfaces (almost) vertically. One such (very rare) example is from Lotung [101].

#### *6.3.1.1. Earthquake Ground Motions: Analytical Models*

There exist a number of analytic solutions for wave propagation in uniform and layered half space [102, 103]. Analytic solutions do exist for idealized geology, and linear elastic material. While geology is never ideal (uniform or horizontal, elastic layers), these analytic solutions provide very useful sets of ground motions that can be used for verification and validation. In addition, these analytic solutions can be used to make estimates of behaviour in cases where geology is close to (ideal) conditions assumed in the analytic solution process. Thus, produced motions can be used to gain better understanding of SSI response for various types of incoming ground motions/wave types (P, SH, SV, Rayleigh, Love, etc.) [104].

#### *6.3.1.2. Earthquake Ground Motions: Numerical Models*

In recent years, with the rise of high performance computing, it became possible to develop large scale models, that take into account regional geology [92–94, 98, 105–108]. Large scale regional models that encompass geology in detail can model seismic motions of up to 5Hz. There are efforts (US-DOE projects) that will extend modelling frequency to over 10Hz for large scale regions. Improvement in modelling and in ground motion predictions is predicated by fairly detailed knowledge of geology for a large scale region, and in particular for the vicinity of the location of interest. Free field ground motions obtained using large scale regional models have been validated [109–115] and are used to develop seismic free field motions for a number of large scale regions in the USA, mostly on the west coast.

Accurate modelling of ground motions in large scale regions is predicated by knowledge of regional and local geology, as well as proper (quite uncertain) modelling of seismic source. Large scale regional models make assumption of an elastic material behaviour, with a (seismic quality) quality factor representing attenuation of waves due to viscous (velocity proportional) and material (hysteretic, displacement proportional) effects. The effects of softer, surface soil layers are not well represented. In order to account for close to surface soil layers, site response analysis (linear, equivalent linear, and nonlinear) needs to be performed in 1-D or preferably in 3-D. Moreover, results from large scale regional models can also be used directly in developing seismic motions for SSI models, as described in Section 8.

### **6.3.2. Uncertainties**

Earthquakes start at the rupture zone (seismic source) and propagate through the soil layers to the surface. All three components in this process, the source, the path through the rock and the site response (soil) exhibit significant uncertainties, which contribute to the variability in ground motions. These uncertainties necessitate the use of a PSHA approach to characterizing variability in earthquake motions [116-119].

### 6.3.2.1. *Uncertain sources*

Seismic source(s) are affected by a number of uncertainties. Location(s) of the source, the magnitude of interest (associated with an annual frequency of exceedance or reference magnitude), the rupture zone, the direction of rupture, stress drop, and other source parameters need to be taken into account [86, 120, 121].

### 6.3.2.2. *Uncertain path (rock)*

Seismic waves propagate through uncertain rock (path) to surface layers. Path uncertainty is controlled by the uncertainty in crustal (deep rock) compressional and shear wave velocities, near site an elastic attenuation and crustal damping factor [120, 121]. These parameters are usually assumed to be log normal distributed and are calibrated based on available information and data, site specific measurements and regional seismic information. Both previous uncertainties can be combined into a model that accounts for free field motions [91, 122].

### 6.3.2.3. *Uncertain site (soil)*

Once such uncertain seismic motions reach surface layers (soil), they propagate through uncertain soil [123, 124]. Uncertain soil adds additional uncertainty to seismic motions response. Soil material properties can exhibit significant uncertainties and need to be carefully evaluated [125-128]. Generally, for free field motion, these uncertainties are treated in the site response analysis.

## 6.4. SEISMIC WAVE INCOHERENCE

### 6.4.1. **General consideration**

Seismic motion incoherence is a phenomenon that results in spatial variability of ground motions over small distances. Significant work has been done in researching seismic motion incoherence over the last few decades [123, 129–135] and extensively used in the study of SSI analysis of nuclear installations, especially nuclear power plants.

The main sources of incoherence [135] are:

- (a) Attenuation effects that are responsible for change in amplitude and phase of seismic motions due to the distance between observation points and losses (damping, energy dissipation) that seismic waves experience along the travel paths. This is a significant source of incoherence lack of correlation for long structures (bridges); however, for nuclear installations it is not of much significance.
- (b) Wave passage effects contribute to incoherence due to difference in recorded wave field at two points as the waves (body and surface) travel from one point to the second point.
- (c) Scattering effects are responsible for incoherence by creating a scattered wave field. Scattering is due to unknown subsurface geologic features that contribute to modifications of the wave field.
- (d) Extended source effects contribute to incoherence by creating a detailed and complex wave source field. As the (extended) fault ruptures, the rupture propagates and generates seismic sources along the rupturing fault. Seismic energy is thus emitted from different points (along the rupturing fault) and has different travel path and timing as it arrives at the observation points.

Figure 18 shows an illustration of main sources of lack of correlation.

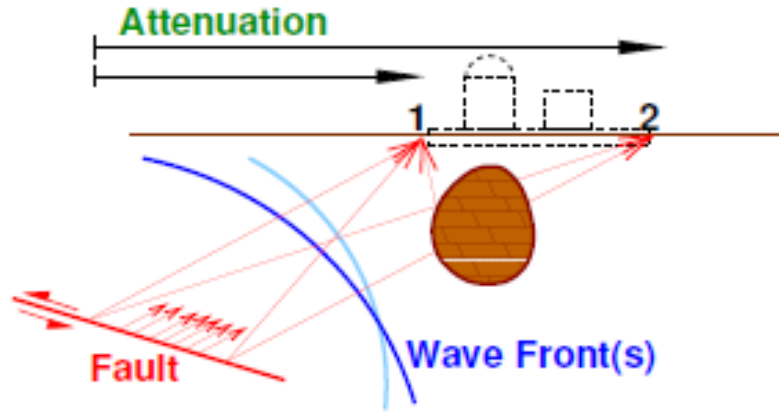


FIG. 18. Four main sources contributing to the lack of correlation of seismic waves as measured at two observation points (reproduced from Ref. [44] with permission courtesy of [Canadian Nuclear Safety Commission]).

### 6.4.2. Incoherence modelling

Early studies concluded that the correlation of motions increases as the separation distance between observation points decreases. In addition, the correlation increased with a decrease in frequency of observed motions. Most theoretical and empirical studies of spatially variable ground motions (SVGGM) have focused on the stochastic and deterministic Fourier phase variability expressed in the form of ‘lagged coherency’ and apparent wave propagation velocity, respectively. The mathematical definition of coherency (denoted  $\gamma$ ) is given as:

$$\gamma_{jk}(f) = \frac{S_{jk}(f)}{S_{jj}(f)S_{kk}(f)^{1/2}} \quad (19)$$

Where  $S_{jj}$  and  $S_{kk}$  are the power spectral density functions of stations  $j$  and  $k$ ,  $S_{jk}$  is the cross power spectral density function, and  $f$  is the frequency. The coherency is a dimensionless complex-valued function that depends on a frequency and the separation distance. This function represents variations in Fourier phase between two signals. Perfectly coherent signals have identical phase angles and a coherency of unity.

Lagged coherency is the amplitude of coherency and represents the contributions of stochastic processes only (no wave passage). Wave passage effects are typically expressed in the form of an apparent wave propagation velocity.

Lagged coherency does not remove a common wave velocity over all frequencies. Alternatively, plane wave coherency is defined as the real part of complex coherency after removing single plane-wave velocity for all frequencies. Recent simulation methods of SVGGM prefer the use of plane-wave coherency as it can be paired with a consistent wave velocity. An additional benefit is that plane-wave coherency captures random variations in plane-wave while lagged coherency does not. Zerva, (2009) [135] has called these variations as ‘arrival time perturbations’.

Most often, coherency  $\gamma$  is related to the dimensionless ratio of station separation distance  $\xi$  to wavelength  $\lambda$ . The functional form most often utilised is exponential [136–138]. The second type of functional form relates coherency  $\gamma$  to frequency and distance  $\xi$  independently, without assuming they are related through wavelength. This formulation was motivated by the study of ground motion array data from Lotung, Taiwan (SMART-1 and Large Scale Seismic Test, LSST, arrays), from which Abrahamson [130, 139] found that coherency  $\gamma$  at short distances ( $\xi < 200\text{m}$ ) is not dependent on wavelength. Wavelength dependence was found at larger



distances ( $\xi = 400$  to  $1000\text{m}$ ). For SVGGM effects over the lateral dimensions of typical structures (e.g.,  $< 200\text{m}$ ), non-wavelength dependent models are used [130, 140, 141].

Moreover, there is a strong probabilistic nature of these phenomena, as significant uncertainty is present in relation to all four sources of incoherence mentioned above. Several excellent references are available on the subject of incoherent seismic motions [123, 129–135, 142]. More detailed information on coherency is given in [63].

#### *6.4.2.1. Incoherence in 3-D*

Empirical SVGGM models are primarily developed for surface motions only. This is based on a fact that a vast majority of measured motions are surface motions, and that those motions were used for SVGGM model developments. Development of incoherent motions for 3-D soil/rock volumes creates difficulties.

A 2-D wave-field can be developed, as proposed by Abrahamson [143], by realizing that all three spatial axes (radial horizontal direction, transverse horizontal direction, and the vertical direction) do exhibit incoherence. The existence of three spatial directions of incoherence necessitates data to develop models for all three directions. Abrahamson [130] investigated incoherence of a large set of 3 component motions recorded by the Large-Scale Seismic Tests (LSST) array in Taiwan and concluded that there was little difference in the radial and transverse lagged coherency computed from the LSST array selected events. Therefore, the horizontal coherency models by Abrahamson [130] and subsequent models [144, 145] assumed the horizontal coherency model may apply to any azimuth. Coherency models using the vertical component of array data are independently developed from the horizontal.

There are limited studies of coherency effects with depth (i.e. shallow site response domain). One possible solution is to utilize the simulation method developed by Ancheta et al. [146] and the incoherence functions for the horizontal and vertical directions developed for a hard rock site by Abrahamson [145] to create a full 3-D set of incoherent strong motions. In this approach, motions at each depth are assumed independent. This assumes that incoherence functions may apply at any depth within the near surface domain ( $< 100\text{m}$ ). Therefore, by randomizing the energy at each depth, a set of full 3-D incoherent ground motions are created.

#### *6.4.2.2. Theoretical Assumptions behind SVGGM Models*

It is very important to note that the use of SVGGM models is based on the ergodic assumption. Ergodic assumptions allow statistical data obtained at one (or few) location(s) over a period of time to be used at other locations at certain times. For example, data on SVGGM obtained from the Lotung site in Taiwan over long period of time (dozens of years) is developed into a statistical model of SVGGM and then used for other locations around the world. Ergodic assumption cannot be proven to be accurate (or to hold) at all, unless more data becomes available. However, the ergodic assumption is regularly used for SVGGM models.

Very recently, several smaller and larger earthquakes in areas with good instrumentation were used to test the ergodic assumption. As an example, Parkfield, California recordings were used to test ergodic assumption for models developed using data from Lotung and Pinyon Flat measuring stations. Konakli et al. [147] shows good matching of incoherent data for Parkfield, using models developed at Lotung and Pinyon Flat for nodal separation distances up to  $100\text{m}$ . This was one of the first independent validations of family of models developed by Abrahamson et al [145]. This validation gives us confidence that assumed ergodicity of SVGGM models does hold for practical purposes of developed SVGGM models.

Jeremic et al. [148] and Jeremic [44] present detailed account of incoherence modelling.

### 6.4.2.3. Nuclear power plant – specific applications

The treatment of the effects of seismic ground motion incoherence (GMI) or SVGM on structure response for typical NPP structures was motivated in part by the development of UHRS with significant high frequency content, i.e. frequencies greater than 20 Hz. Figure 19 shows the UHRS (AFE =  $10^{-4}$ ) at one NPP rock site in the U.S. The PSHA calculated data points are shown. The UHRS is the result of curve fitting for display purposes. The peak spectral acceleration is at 25 Hz.

Efforts to evaluate the existence and treatment of GMI for conditions applicable to NPP foundations and structures were a combined effort of ground motion investigations and evaluation of the impact of implementing GMI effects on the seismic response of typical NPP structures.

For the former effort, Abrahamson [144, 145] investigated and processed recorded motions from 12 sites for 74 earthquakes. The resulting ground motion coherency functions as a function of frequency and distance between observation points were generated considering all data regardless of site conditions, earthquake characteristics, and other factors.

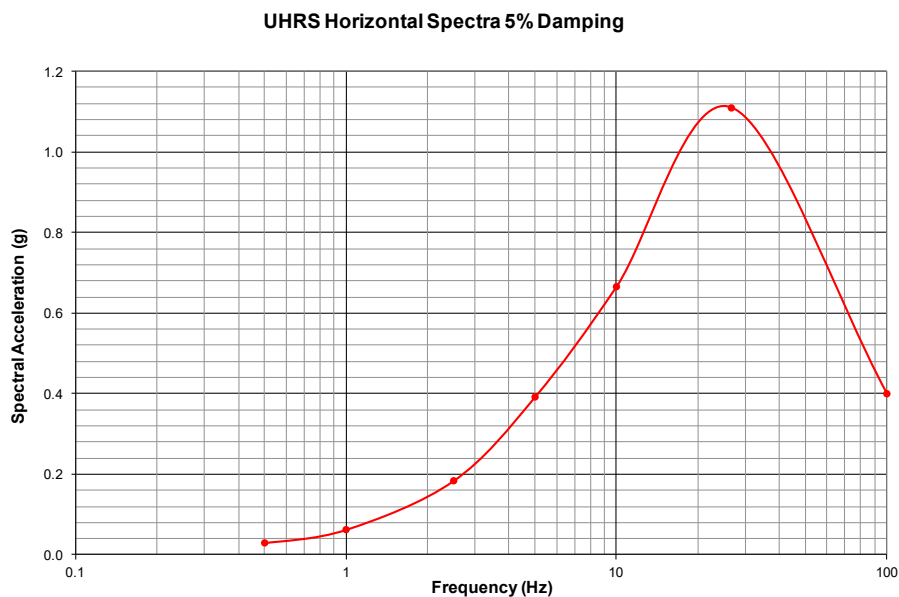


FIG. 19. Uniform Hazard Response Spectra (UHRS) – annual frequency of exceedance =  $10^{-4}$  – for a rock site in the U.S. (Courtesy of James J. Johnson and Associates).

Abrahamson [144] refined this initial effort to separate soil and rock sites. Plots of soil and hard rock ground motion coherency functions are shown for horizontal and vertical ground motion components in Figure 20.

For the latter effort, [149-151] present comparisons of in-structure responses for assumptions of coherent and incoherent ground motions for a representative NPP structure calculated using the programs CLASSI and SASSI. These references serve to benchmark and verify the treatment of incoherence of ground motion by CLASSI and SASSI. Johnson et al. [152] present the SSI analyses of the Evolutionary Power Reactor when subjected to coherent ground motions and incoherent ground motions sited on a rock site.

In general, implementing GMI into seismic response analyses has the effect of reducing translational components of excitation at frequencies above about 10 Hz, while simultaneously adding induced rotational input motions (induced rocking from vertical GMI effects and increased torsion from horizontal GMI effects). Significant reductions in ISRS in progressively higher frequency ranges are observed.

As noted above, there is an urgent need to record and process additional data to further verify GMI phenomena and its effects on structures of interest. Until such additional data is accumulated and processed, guidance on incorporating the effects of GMI on NPP structures' seismic response for design is as follows:

- (a) Seismic responses (ISRS) for assumptions of coherent ground motions and incoherent ground motions need to be calculated to permit comparisons to be made.
- (b) Currently, the following guidelines for ISRS, representing current practice in the USA (NRC) [153] are in place for design:
  - (i) For the frequency range 0–10 Hz, no reductions in ISRS are permitted;
  - (ii) For frequencies above 30 Hz, a maximum reduction in ISRS of 30% is permitted;
  - (iii) For the frequency range of 10–30 Hz, a maximum reduction based on a linear variation between 0% at 10 Hz and 30% at 30 Hz is permitted.

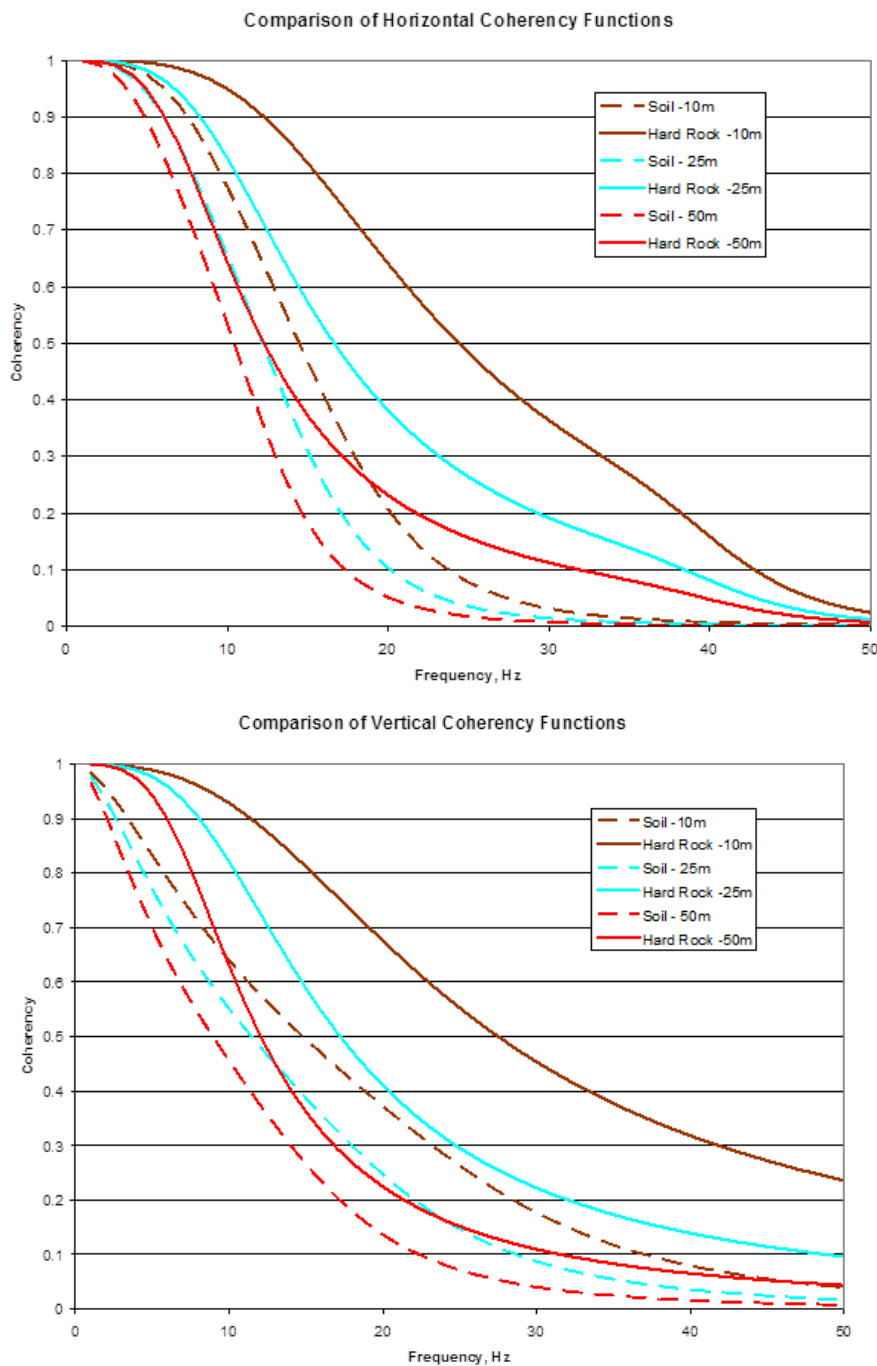


FIG. 20. Comparison of ground motion coherency functions for soil and hard rock sites.

## 7. SITE RESPONSE ANALYSIS AND SEISMIC INPUT

### 7.1. OVERVIEW

Seismic input is the earthquake ground motion that defines the seismic environment that the soil–structure system is subject to and for which the SSI analyses are performed.

The objective of the seismic analysis directly affects the approaches to be implemented for definition of the seismic input, i.e., design or assessment of the nuclear installation. The DBE ground motion may be based on standard ground response spectra (see Section 7.4) or site specific ground response spectra developed by PSHA or DSHA (see Section 5), as follows:

- (a) Assessments of the facility can be for hypothesised BDBE ground motions or for actual earthquake events that have occurred and need evaluation. BDBE is defined as the seismic ground motion (represented by acceleration time history or ground motion response spectra) corresponding to an earthquake severity higher than the one used for design derived from the hazard evaluation of the site. It is used in seismic margin assessment or seismic probabilistic safety assessment. For the assessment of nuclear installations subject to hypothesised BDBE ground motions, PSHA-defined values play an important role in seismic margin assessments and seismic probabilistic risk assessments.

The physics of the seismic phenomena dictate that, in terms of SSI, the variation of motion over the dimensional envelope of the foundation is the essential aspect, i.e. the depth and horizontal dimensions of the foundation. The detailed generation of this free field ground motion is the important factor. For surface foundations, the variation of motion over the surface plane of the soil is important; while for embedded foundations, the variation of motion over both the embedment depth and the foundation horizontal dimensions affects the seismic response;

- (b) Nonlinear effects, in the soil/rock adjacent to nuclear installation foundations and within the foundation – soil/rock contact zones play a very important role in the overall SSI response. Depending on the strength of the soil/rock and the contact zone, nonlinear effects can be significant for the DBE or BDBE. For BDBE, nonlinear effects may be very significant, even for very competent soil/rock and contact zones. The importance of nonlinear effects is in the increased stresses in the soil or rock in the neighbourhood of the foundation and structure interfaces and in the contact zone. Nonlinear effects can affect the effective input motion to the foundation/structure (kinematic interaction) of the nuclear installation and the dynamic response of the soil–structure system (inertial interaction).

This section discusses various aspects of defining the seismic input for SSI analyses for nuclear installations. Seismic input is closely coupled with the soil property definition (Section 4), free field ground motion definition (Section 6), and SSI analysis methodology (Section 8). The development of the seismic input for the SSI analysis is closely coordinated with its purpose - design and/or assessment.

Typically, three aspects of free field ground motion are needed to define the seismic input for SSI analyses: control motion; control point; and spatial variation of motion. Each of these elements contributes to the definition of seismic input for site response analyses and, subsequently, SSI analyses. These elements are discussed in the following subsections. Section 5 presents seismic hazard assessment, which is essential to defining the DBE ground motion and the considerations for the BDBE ground motions.

## 7.2. SITE RESPONSE ANALYSIS

### 7.2.1. Perspective

Site response analysis is comprised of many aspects. In the broadest sense, its purpose is to determine the free field ground motion at one or more locations given the motion at another location.

The starting point for site response analysis is the selection of the ‘control motion, and the ‘control point’. If SHA (deterministic or probabilistic) is performed, the starting point is the location at which the free field ground motion is predicted, which is dependent on the GMPEs that are implemented, as follows:

- (a) In many cases, GMPEs are associated with specific soil or rock conditions. These GMPEs may be derived for generic soil/rock conditions, perhaps defined by  $V_{s30}$  values, i.e., average shear wave velocities over the upper 30 m of soil, as follows:
  - (i) If the GMPEs fit well the native soil/rock properties up to TOG, then, these GMPEs define the seismic hazard at TOG. This is most often the case for uniform soil/rock profiles, i.e., with smoothly varying soil properties without distinct layering.
  - (ii) If the GMPEs are specified for rock or hard rock conditions, then the GMPEs may define the seismic hazard at an actual or hypothetical rock location at the nuclear installation site. This location could be at a hypothetical outcrop of natural rock located at the natural rock/soil interface in the native soil. If the GMPEs are specified for hard rock ( $V_s > 2\,800$  m/s), then the resulting seismic hazard curves or values are defined for an actual or hypothetical location where the assumption of hard rock applies.
  - (iii) If the GMPEs are specified for a suite of natural frequencies, and implemented for the suite, then the resulting seismic hazard values or curves can define a site specific ground response spectra that adequately matches the nuclear installation site. These seismic hazard values or curves could be probabilistically or deterministically defined.
  - (iv) If the GMPE of interest is only that of PGA, then values of PGA associated with selected frequencies of exceedance anchor spectral shapes, such as standard response spectra or others.
  - (v) SRA may be needed to define or provide guidance on the soil material properties to be used in the SSI analyses.
  - (vi) In case of equivalent linear soil properties taking into account the strain levels induced by the free field ground motion are used, it is common for the soil material models in the SSI analysis to treat these ‘primary’ nonlinearities, while not including the effects of ‘secondary’ nonlinearities, i.e., those induced by structure response. These equivalent linear soil properties are usually defined during the site response analysis; this is especially the case for substructuring methods.
  - (vii) With regard to nonlinear analyses, the location of boundaries of the nonlinear soil models beyond which the soil material behaviour may be treated as linear or equivalent linear are discussed in Section 8.
- (b) Site response analysis may be needed to define the seismic input to nonlinear SSI analysis. If the free field ground motion is defined by numerical source models, then one of the purposes of site response analysis is to generate the input motion from the source to the soil

island boundaries for definition of the input to the nonlinear SSI analysis. The soil island encompasses the model of the structure and the adjacent soil.

- (c) If the free field ground motion is defined by the SHA results at any of the locations described above, e.g., TOG, actual or hypothetical outcrops within the soil/rock medium, then site response analysis is needed to define the seismic input motion at soil island boundaries.
- (d) Site response analysis may be needed to define the seismic input to linear or equivalent linear SSI analysis. Computer programs may have specific requirements for the seismic input, e.g., SASSI accepts seismic input at TOG or at in-column locations, CLASSI accepts input as defined in the generation of scattering functions, which for surface foundations is TOG, and for embedded foundations, using the hybrid method, at the corresponding SASSI seismic input location. Real ESSI accepts input motions in time domain either at the surface, or at any depth, and motions can be 1C and/or 2C and/or 3C.

Site response analysis may be needed to implement the minimum ground motion check for foundation motion in the design process, as specified by Member States' requirements; this check is related to the foundation input response spectra (FIRS) (Section 7.2.2). As in the case of SSI analyses, SRA can be performed in the frequency domain or in the time domain, as follows:

- (a) Frequency domain analyses for earthquake ground motions are linear or equivalent linear. Often, strain-dependent, elastic soil material behaviour is simultaneously defined with the definition of the free field ground motion as seismic input (Section 4);
- (b) Time domain site response analyses may be linear or nonlinear. Time domain analyses are most often performed to generate the explicit definition of the seismic input motion at locations along the boundary of the linear or nonlinear SSI model (Section 8). Time domain site response analysis is almost exclusively performed as convolution analysis, i.e., the input ground motion defined by the control point and corresponding control motion are defined at depth in the soil on an actual or hypothetical rock outcrop, e.g., corresponding to the PSHA results or other definitions of the ground motion.

Investigations of the effects of irregular site profiles can be performed in the frequency domain or the time domain.

It is important to recognize that specific requirements for seismic input motion may be dependent on the SSI analysis methodology to be used. In some cases, especially for typical sub-structuring methods, there are assumptions implemented as to the wave propagation mechanism of the free field ground motion. These assumptions often are: vertically incident P and S waves; non-vertically incident P and S waves, and surface waves (Rayleigh waves, Love waves, and other surface waves).

### **7.2.2. Foundation input response spectra**

The term 'foundation input response spectra' (FIRS) was first defined in [154, 155]. Although the definition may be interpreted in a general sense, these references specifically define FIRS from the site response analysis assuming vertically propagating shear (S) and dilatational (P) waves and semi-infinite horizontal soil layers. The FIRS are defined on a hypothetical or actual outcrop at the foundation level of one or more structures.

The important point is that it is a free field ground motion input to the SSI analysis of a structure. In general, it is not equal to the foundation input motion, which is a result of kinematic interaction.

For a nuclear installation site with many structures of interest and with many different foundation depths (possibly, one foundation depth for each structure), multiple FIRS, one at each foundation depth or a single definition at a common location, such as, the free surface at

top of grade (TOG) are possible. However, there are often additional considerations, which could introduce additional analysis cases to be performed. One such consideration for design is the need to meet a specified minimum input ground motion response spectrum at foundation level.

The background to and methodologies for generation of FIRS through the geological method and full column method are contained in [154, 155]. The primary difference between the geological method and the full column method lies in the definition of the outcropping motion. Figure 21 shows a schematic of the two FIRS definitions. The geological method involves removal of the strain compatible soil layers above the foundation level and reanalysis of the soil column to extract the geologic outcrop spectrum. This reanalysis uses the strain compatible soil properties defined in the full column analyses – no additional iterations on soil properties are performed.

The full column method includes the soil layers above the foundation level where the effects of downcoming waves above the foundation level are included in the analysis and the outcrop motion (also referred to as the SHAKE outcrop) assumes that the magnitude of the upgoing and downcoming waves are equal at the elevation of the FIRS.

With reference to Figure 21, the FIRS in the full column method is  $2A_2$  with  $A_2$  and  $A'_2$  respectively the amplitudes of the upward and downward going waves; in the geological method, the FIRS is  $2A''_2$  with  $A''_2$  the amplitude of the upward (and downward) going waves. Note that  $A''_2$  is different from  $A_2$ .

For frequency domain linear or equivalent linear analyses, the full column method is simpler to implement because the outcrop motion at the level of the foundation can be extracted directly from the SRA without reanalysis of the iterated soil columns. For linear or nonlinear time domain analyses, the geological method is needed.

In terms of SSI analysis, assuming each structure is analysed independently of other structures, each structure has a defined control point and control motion. So, for a given nuclear installation site, there may be multiple control points and control motions. A different approach is that there is one control point and one control motion defined at TOG and that defines the input motion for all SSI analyses. Then (1-D) deconvolution is performed implicitly with a program like SASSI or explicitly with a program like SHAKE.

In all cases, when the DBE ground motion is defined at TOG and structures to be analysed are modelled including embedment, multiple deterministic soil profiles are used in the SSI analyses. Often, three profiles are analysed, i.e., best estimate, lower bound, and upper bound. To verify that the TOG DBE ground motion is adequately represented by the multiple soil profiles, site response analyses are performed for each of the soil profiles and the resulting envelope at the TOG is verified to be equal to or greater than the TOG DBE ground motion. If the resulting envelope does not adequately match or envelope the TOG DBE, additional soil profiles may be added, or higher FIRS may need to be considered.

Multiple different profiles are to be considered when the DBE is specified at TOG. This is also done when the DBE is specified at the outcropping bedrock. The only point that differs between both situations is the comparison of the calculated motion at TOG with the DBE, which has only to be carried out for the former situation.

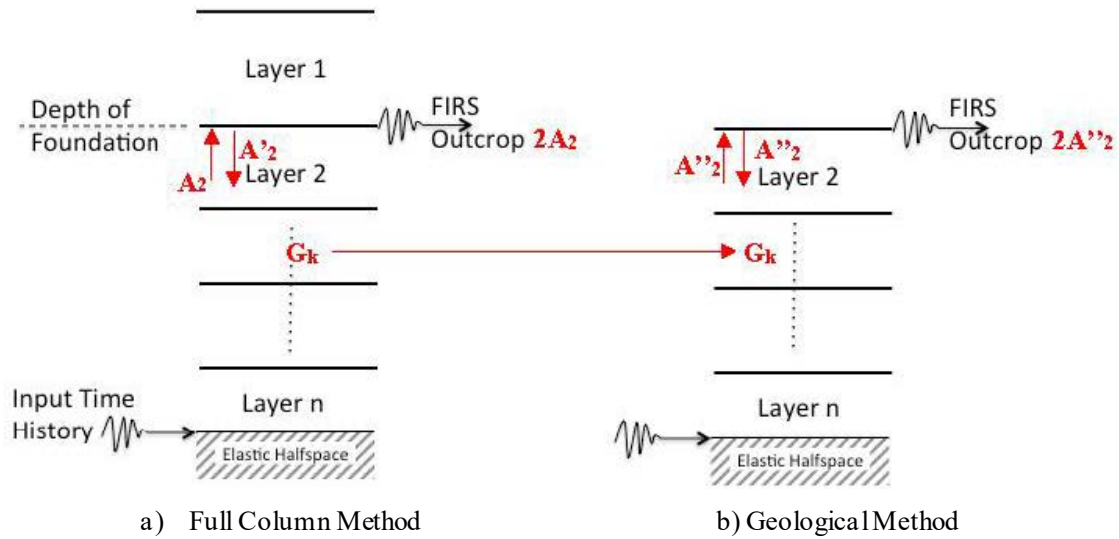


FIG. 21. Definition of FIRS for idealized site profiles.

### 7.3. SITE RESPONSE ANALYSIS APPROACHES

Characteristics of the SRA are discussed next: convolution vs deconvolution; probabilistic vs deterministic; response spectra vs a random vibration approach. Convolution analysis may be performed in the frequency domain or in the time domain. Deconvolution analysis is performed in the frequency domain.

#### 7.3.1. Idealized site profile and wave propagation mechanisms

Before proceeding to implementation approaches, it is helpful to establish the procedure of site amplification for the idealized site profile, including generating equivalent linear soil properties [86].

In principle, for the idealized assumption of the site profile being represented by semi-infinite horizontal soil layers overlying a half-space, 1-D wave propagation is assumed to be the wave propagation mechanism for horizontal motion, i.e., vertically propagating SH-waves for horizontal motion.

In general, for this case, the following approach may be taken:

- (a) The solution of the wave equation for 1-D wave propagation in a single layer for displacement, velocity, or acceleration is comprised of an upward wave and a downward wave as a function of depth in the layer and time.<sup>11</sup>

The displacement within a soil layer  $u(z, t)$  is calculated by:

$$u(z, t) = Ae^{i(\omega t + k^*z)} + Be^{i(\omega t - k^*z)} \quad (20)$$

Where,

$u$  = displacement;

$z$  = the depth within a soil layer, oriented positively downwards;

$t$  = time;

<sup>11</sup> In Figure 21, the quantities  $A_2$  and  $A'_2$  represent the upward wave and downward wave respectively.



$A$  and  $B$  = amplitude of waves traveling in the upward and downward directions, respectively;

$\omega$  = circular frequency;

$k^*$  = complex wave number [ $k^* = (\omega/V_S^*)$ ] with  $V_S^*$  the complex shear wave velocity.

- (b) The boundary conditions to be enforced in the solution of the wave equations of each layer are zero stress at free surfaces and compatibility of displacements and stresses at layer interfaces, e.g., in layer  $i$  and  $i+1$ ;
- (c) Applying the boundary conditions yields a recursive relationship for the amplitude of the upward wave and downward wave in the layers;
- (d) Within a layer, shear strain (for horizontal motions) can be calculated from the derivative of the displacement at a given location. Shear stress can be calculated from the complex shear modulus.

The SRA step is applicable to free field ground motion defined by a DSHA or PSHA. The principal difference lies in definition of the input motion calculated or specified at a given location, e.g., at a hard rock actual or hypothesized outcrop location.

As part of the process, the shear stresses (and complex shear modulus) are calculated as part of an iterative approach to converge on equivalent linear values of shear modulus and material damping to approximately account for nonlinear behaviour of soil properties. If convergence occurs, transfer functions between the responses in any two layers or boundaries can be calculated.

Given, the input motions at the specified location, the transfer functions of the previous step can then be used to calculate the motions at locations of interest within the soil profile, e.g., TOG and FIRS. These are site amplification factors (SAFs). With this background, majority of site response analyses techniques are based on the following assumptions:

- (a) Soil layer stratigraphy (semi-infinite horizontal layers overlying a uniform half-space), variability in layer thickness is modelled;
- (b) Soil material properties (one-dimensional equivalent linear viscoelastic models defined by shear modulus and material damping – median and variability); (Equation 20);
- (c) Wave propagation mechanism vertically propagating S and P waves.

The process is illustrated in Annex II using one input time history, including sensitivity studies to evaluate the effect of nonlinear material properties, incompressibility, and soil permeability. Section 9 summarises the observations and recommendations for treating these factors.

In actual applications, the input motion is not defined with a single time history, but its definition relies on the techniques designated as Approaches 1, 2A, 2B, 3, and 4; higher numbers associated with the more rigorous approaches specifically with respect to the potential sensitivity of the SAFs to magnitude and distance dependency of seismic sources, non-linearity of the soil properties, and consideration of uncertainty in the site profile and dynamic soil properties. The most rigorous of the approaches used extensively is Approach 3 (which is a simplification of Approach 4). Approaches 2B, 2A, and 1 include increasingly simplified assumptions as compared to Approach 3. Approaches 3, 2B, 2A, and 1 are briefly described in the following text.

Approach 1 uses a single response spectrum defined at hard rock (bed rock) corresponding to a specified AFE, such as  $10^{-4}$  and a shape that is consistent with the deaggregated seismic hazard for the specified AFE. This response spectrum is usually broad-banded and, when used to iterate on equivalent linear soil properties leads to unrealistic degraded values of shear moduli.

Approach 2A modifies Approach 1 by considering ‘high’ and ‘low’ frequency deaggregated seismic hazards, i.e. for AFE seismic hazard values at natural frequencies of about 1 Hz (low

frequency) and 10 Hz (high frequency), it attempts to mitigate the Approach 1 deficiencies in overestimating the reduction of soil shear moduli and increase of material damping. Probabilistic site response analyses are then performed for variations in physical characteristics of the nuclear installation site and the variability due to the high and low frequency input motions.

Approach 2B modifies Approach 2A by considering multiple response spectra to represent the deaggregated seismic hazard. This expanded set of response spectra is then used in probabilistic site response analyses performed for variations in physical characteristics of the nuclear installation site and the variability due to the high and low frequency input motions.

Approaches 2A and 2B are implemented with the goal of more realistically representing the range or major earthquake sources contributing to the seismic hazard at the AFE of interest.

Approach 3 is more rigorous in that it considers a significantly greater range of contributing seismic sources and a more complete representation of the spectral values over the natural frequency range. Approach 3 is implemented more frequently than Approaches 2A and 2B due to its perceived increased rigor and probabilistic aspects.

Approach 4 is the most computationally detailed. Approach 4 takes the results of each simulation in the PSHA process carrying it through the SRA process. This could be millions of simulations, which could be infeasible with the available computer technology.

#### *7.3.1.1. Convolution*

Site response analysis is most often considered to be convolution analysis. The intermediate output of site specific response spectra, as determined from a PSHA (UHRS) or a DSHA at a location in the nuclear installation site profile. is the starting point. This location is defined by hard rock ( $V_s > 1500\text{m/s}$  or  $2800\text{m/s}$ ), soft rock (e.g.,  $V_s \geq 800\text{m/s}$ ), or significant impedance mis-matches in the site profile. The location is dependent on the attenuation laws, or GMPEs, associated with hard rock, soft rock, or stiff underlying soil/rock comprising the significant impedance mis-match. In some cases, site condition corrections are applied for different shear wave velocities.

The end results of the PSHA process are itemized in Section 5; the most important result being the seismic hazard curves at specified natural frequencies and AFEs, which are the bases to generate UHRS for the AFEs. For a nuclear installation site, the seismic hazard curves are calculated over a range of discrete natural frequencies — usually 5-20 frequencies, such as 0.1, 1.0, 2.5, 5, 10, 25, 100 Hz or some other combination — as a function of AFEs. To define a UHRS at an AFE, the discrete values of acceleration (or spectral acceleration) at the requested AFE of each of the natural frequency hazard curves is selected and the values are connected by segmented lines in log-log space or fit with a curve. This becomes the UHRS at AFE at the location of interest and form of interest (in-soil or outcrop). These natural frequencies are referred to as ‘conditioning frequencies’ in some applications. In addition, deaggregation of the seismic hazard curves leads to the identification of earthquake events (magnitude and distance) that are major contributors to the seismic hazard curve at a specified natural frequency and AFE. In the PSHA case, the response spectra associated with these deaggregated events are used in the SRA, i.e. the free field ground motion definition at the input location is comprised of a suite of these response spectra. These response spectra are used directly [156–158] or in an RVT approach [159, 160], for which response spectra are converted to Fourier amplitude spectra supplemented by ground motion duration.

The DSHA intermediate output is ground response spectra corresponding to the parameters of the DSHA.

The next step involves the propagation of intermediate output of motion from the actual or hypothetical location (hard rock, soft rock, other) at the site to locations within the soil profile, usually, to generate FIRS at foundation levels (locations) that correspond to nuclear installation structures of interest for design or assessments. The results are site specific ground response spectra at requested locations (requested by the team performing the SSI analyses), which are then used to define the seismic input to the SSI analyses.

These site response analyses (convolution) are most often probabilistic, and consider the following variabilities:

- (a) Ground motion definition at hard rock location:
  - (i) PSHA:
    - For each natural frequency of the seismic hazard curves, identify deaggregated seismic hazard parameters of magnitude (M) and distance (R);
    - Select ground motion response spectra from data bases representing the (M, R) that also represent approximations to the UHRS or a portion thereof; these become the input motions for which the SAFs are developed.
  - Or
  - (ii) DSHA:
    - Site specific;
    - Site independent.
- (b) Base soil cases are defined from geological and geotechnical investigations and assessments; the number of base soil cases vary from one to four; each base soil case is assigned a weight; the sum of weights equals 1.0.
- (c) For each base soil case, the following parameters are defined by a probability distribution<sup>12</sup>:
  - (i) Depth to bed rock;
  - (ii) Soil layers over bed rock:
    - Thickness;
    - Low strain properties (for visco-elastic material behaviour, this includes, low strain shear modulus, Poisson's ratio, low strain material damping, etc.);
    - Coupled shear modulus degradation curves with material damping;
    - Other properties.
- (d) Construct sampling approach. This is generally a stratified sampling of the probability distributions and a Latin hypercube sample (LHS) experimental design; this could also be strict Monte Carlo sampling (MCS); LHS requires many less samples than MCS to achieve the same accuracy.
- (e) Perform simulations and generate SAFs at locations of interest between hard rock and the FIRS locations and/or TOG; loop over the natural frequencies of interest and the AFE. These simulations include iterations on soil material properties to determine equivalent linear soil properties.
- (f) Combine the results as appropriate through convolution or other approaches.
- (g) Apply SAFs to obtain seismic input at locations of interest.

Randomness and uncertainty in the soil configuration and soil material properties are then treated.<sup>13</sup>

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<sup>12</sup> The probability distribution of some parameters (depth to bedrock, layer thickness) are poorly modelled in 1-D SRA since they truly exhibit a 3-D spatial variation. The impact of such probabilistic modelling needs to be assessed with care.

<sup>13</sup> Ground motion response spectra peak and valley variability is explicitly treated in the PSHA. Consequently, the only ground motion randomness treated in the SRA and in the SSI analyses is randomness of phase.

Normally, these probabilistic analyses are performed for a series of (up to) 60 earthquake simulations.

#### *7.3.1.2. Deconvolution*

Convolution was discussed in previous sections and is a forward marching method. Deconvolution is the denoted method to solve the inverse problem to the convolution problem: What ground motions led to the specified ground motions from the PSHA, e.g., the UHRS. The PSHA results (UHRS) are generated at a site location, usually TOG, using attenuation laws or GMPEs somewhat tailored to the properties of the nuclear installation site, in some cases through a  $V_s30$  term. This case is most often implemented for relatively uniform soil properties. In this case, the UHRS can be input directly to surface-founded structures. They are the FIRS. Generally, however, some estimates or calculated soil properties adjusted for strain levels induced in the free field by the ground motion are generated for SSI analyses. These free field soil properties could be generated probabilistically or deterministically. For structures with embedded foundations, FIRS could be generated by deconvolution or, if the SSI analysis program permits exciting the SSI system with the TOG motion, then FIRS are not explicitly needed for the SSI analysis (in fact the FIRS are calculated within the program but are not visible to the user); however, the FIRS may be needed for checks against limits in the reduction of free field ground motion at foundation level, if this quantity is required by national regulations.

In the deconvolution analyses, divergence of the numerical scheme might occur if the UHRS at the TOG is incompatible with the properties of the soil deposit; this typically happens because UHRS are broad-band spectra whereas the soil deposit may exhibit some well identified resonant frequencies with sharp peaks. In that case, if the strain compatible properties are needed, or if the FIRS need to be calculated at some depth, two alternatives are possible. The first alternative consists of scaling down the ground surface motion by a factor  $\lambda$ , such that the deconvolution is essentially linear (strains smaller than  $10^{-5}$  for instance), retrieving the base outcrop motion, scaling it up by  $\lambda$  and running the convolution analyses with iterations on soil properties. The second alternative consists of running the deconvolution analyses with the elastic properties (maximum shear modulus and very small damping) without iterations on the soil characteristics, retrieving the base outcrop motion and running the convolution analyses with iterations on the soil properties. If in the first approach the scaling factor is large enough to keep the soil behaviour almost linear, both approaches yield approximately the same results.

#### **7.3.2. Non-idealized site profile and wave propagation mechanisms**

For site profiles that cannot be idealized with horizontal soil layers, that have non-horizontal layers, surface and subsurface topography, also in situations where wave propagation cannot be approximated with vertically propagating P and S waves, other methods need to be used.

One example of topographic effect is provided in the Annex II.

#### **7.3.3. Analysis models and modelling assumptions**

PSHA and DSHA are the most frequently used methods to generate the ground motion at a nuclear installation site at a location, such as top of grade (TOG), an assumed or actual outcrop, or an assumed or actual impedance mis-match. From this location, site response analyses are frequently needed to further define the motion at FIRS or at the boundary of the nonlinear soil island.

In general, GMPEs are developed from large databases of recorded motions. The ground motions comprising these records include (combine) the effects of the fault rupture, all wave propagation mechanisms, topographic effects, geological effects, and local site effects at the recording stations. These measurements are acceleration time histories from which spectral accelerations can be calculated and PGA values can be determined. These large databases may be parsed into smaller databases to permit customization of the GMPEs, e.g. site condition customization based on Vs30 values.

The important point is that all significant elements contributing to the recorded ground motion values as itemized above exist in the recorded motions, but generally they are not separable. So, it is extremely difficult, if not impossible, to determine which portion of the ground motion response spectra (GMRS) is due to topographic effects, geological effects, or wave types. One advantage of implementing SRA starting from a deep rock or soil outcrop is the possibility to introduce potentially important site specific effects for the purposes of understanding their impact on the seismic input to the soil–structure system.

The next subsections discuss 1-D and 3-D representations of the wave fields and their potential effect on SSI analyses of nuclear installation structures of interest. It is important to recognize that SSI analyses of nuclear installations are 3-D, i.e. involve 3-D soil and structure models and three spatial components of earthquake input motion. For calculation purposes, in some instances, the SSI models are analysed for each spatial direction of input motion separately. This is possible for linear elastic material assumption. However, even in this case, the 3-D response of the nuclear installation structures is determined through combining the SSI responses from each individual direction of excitation by an appropriate combination rule, e.g. algebraic sum, square root of the sum of the squares (SRSS), absolute sum, or other rules.

Development of analysis models for free field ground motions and SRA involve modelling assumptions about the treatment of 3-D or 2-D or 1-D seismic wave fields, treatment of uncertainties, material modelling for soil (see Section 4 and Section 8.3), and the possible spatial variability of seismic motions.

This section is used to address aspects of modelling assumptions. The idea is not to cover all possible modelling assumptions (simplification), rather to point to and analyse some commonly made modelling assumptions. It is assumed that the analyst will have proper expertise to address all modelling assumptions that are made and that introduce modelling uncertainty (inaccuracies) in final results.

This section addresses issues related to free field modelling assumptions. This includes a brief description of modelling in 1-D and in 3-D, and the use of 1-D seismic motions assumptions in light of full 3C (3 components) seismic motions [161]. Next, the assumption of adequate propagation of high frequencies through models (finite element mesh size/resolution) is addressed [162]. There are number of other issues that can influence results (for example, nonlinear SSI of NPPs [90]).

#### *7.3.3.1. D models*

In reality, seismic motions are always 3D, featuring body and surface waves (see Section 5.2). However, development of input, free field motions for a 3-D analysis is not easy. Recent large scale, regional models [92–94, 98, 105–115, 163–165] have shown great promise in developing realistic free field ground motions in 3-D. What is necessary for these models to be successfully used is a detailed knowledge of the deep and shallow geology as well as a local site condition (nonlinear soil properties in 3-D). Often this data is not available, however, when it is available, excellent modelling of 3-D SSI can be performed, with a possible reduction of demand due to nonlinear effects and due to use of more realistic motions. In addition, due to computational capability, local seismic response models are usually restricted to lower

frequencies (below 5Hz) while there are current projects that will extend simulations to 10Hz, for very large regions (200km x 150km x 4km). Another problem is that seismic source, fault slip models currently cannot produce high frequency motions, and stochastic high frequency motions need to be introduced.

When the data is available, a better understanding of the dynamic response of a nuclear installation can be developed. The developed nonlinear, 3-D response will not suffer from numerous modelling uncertainties (1-D vs 3-D motions, elastic vs nonlinear/inelastic soil in 3-D, soil volumetric response during shearing, influence of pore fluid, etc.). For this approach, good quality data is needed (material properties, spatial distribution of material, potential location of seismic sources, shallow and deep geology). Lack of good quality data can also introduce modelling uncertainty, which needs to be carefully considered when this approach is used. Current projects have developed a number of nonlinear, 3-D earthquake SSI procedures, that rely on full 3-D seismic wave fields (free field and site response) and it is anticipated that this trend will accelerate as the benefits of more accurate modelling become understood.

#### *7.3.3.2. 3-D/3C versus 1-D/3C versus 1-D/1C seismic models*

Seismic waves propagate in 3-D and have all three components (3C) of translations. Sometimes, full 3-D wave propagation with all three components 3C can be simplified to propagation in less than 3-D, and with less than 3C. For example, neglecting full 3-D wave propagation and replacing it with a 1-D wave propagation, while still preserving all three components (3C) of motions, can sometimes be appropriate. Such simplifying assumption need to be carefully assessed, taking into account possible intended and unintended consequences.

A brief discussion on 1C, 3 x 1C and 3C seismic wave modelling and effects on SSI is provided below:

- (a) 1-D wave propagation, with 1C modelling of seismic waves is possible if material modelling for soil is linear, equivalent linear elastic or nonlinear/inelastic. In the case of elastic or equivalent linear elastic material, 1-D/1C motions from different directions (horizontal) can be combined, as the superposition principle applies for linear elastic systems (i.e. in this case, soil). Modelling of vertical motions using a 1-D/1C approach is different, as an analysis needs to be performed to decide if the vertical wave is a compressional wave (primary, P wave) or if vertical motions are a consequence of vertical components of surface waves.
- (b) 3 x 1C modelling of seismic waves is possible, under special circumstances, described below. Most of the time, vertical motions are a result of Rayleigh surface waves; therefore, it is important to analyse vertical motions and decide if modelling as 1C is appropriate. To this end, the wavelength of surface waves plays an important role. If the Rayleigh surface wavelength (which features both horizontal and vertical components) is longer than 12 times the dimension of the object (i.e. the nuclear installation, then object rotations due to differential vertical displacements at object ends are fairly small and the object moves up and down as if excited with a vertical wave. This is shown in Figure 22, as the upper case. In this case it is appropriate to use 3x1-D modelling even with nonlinear/inelastic models. On the other hand, if the wave is shorter than 12 object dimensions, then vertical motions are gradually replaced by object rotations, while vertical motions are reduced. The lower left corner of Figure 22 shows a limiting case where the seismic wave is 4 times longer than object dimension, which results in minimal vertical motions of the object, and maximum rotations, due to differential motions of object ends. For shorter surface waves, as shown in the lower right of Figure 22, waves might not even excite any significant

dynamic behaviour of the object (except local deformation) as their wavelengths are shorter than twice the object length.

- (c) 3-D/3C modelling will capture all the body and surface wave effects for SSI analysis for NPPs [166].

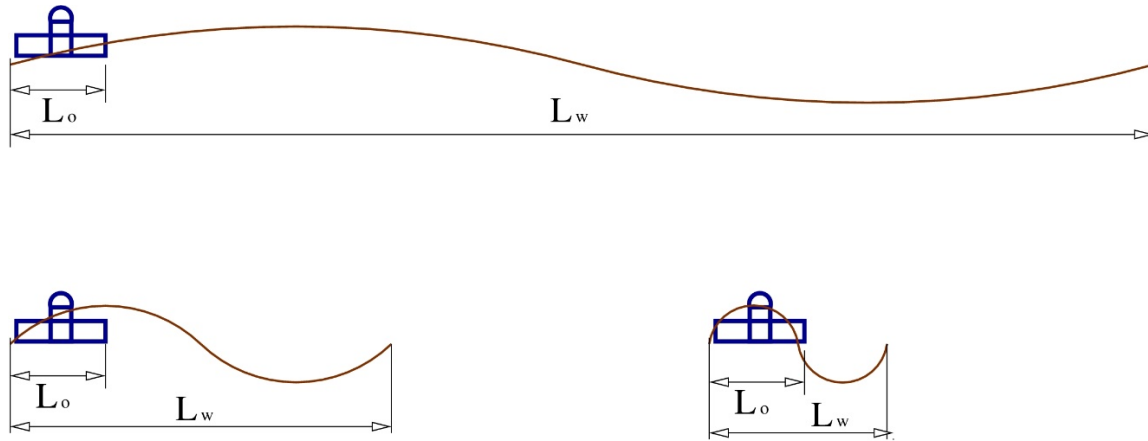


FIG. 22. Three different cases of surface wavelength. Upper case is where the surface wavelength is 12 or more times longer than the object (NPP) dimension). Lower left case is where the surface wavelength is only four times longer than the wavelength, and lower right case is where the surface wavelength is only two times longer than the object length.

#### 7.3.3.3. Propagation of higher frequency seismic motions

Seismic waves of different frequencies need to be accurately propagated through the model/mesh. This is particularly true when higher frequencies of seismic motions are to be propagated. An illustrative example is used to analyse propagation of seismic waves of different frequencies through the finite element mesh. The resulting damping of higher frequencies is clearly observable and be taken into account when finite element models are designed, and decisions about mesh quality (finite element size) are made during model development. Watanabe et al. 2016 [162] presents an in-depth analysis of wave propagation through different mesh sizes, and for elastic as well as for nonlinear (elastic-plastic) materials for SSI analysis for NPPs.

It is known that mesh size can have significant effect on propagating seismic waves [162, 167-169]. Finite element model mesh (nodes and element interpolation functions) needs to be able to approximate displacement/wave field with a certain accuracy without discarding (filtering out) higher frequencies. For a given wavelength  $\lambda$  that is modelled, it is best to have at least 10 linear interpolation finite elements ( $8 \text{ node bricks in 3-D}$ , where the representative element size is  $\Delta h^{LE} \leq \lambda/10$ ) or at least 2 quadratic interpolation finite elements ( $27 \text{ node bricks in 3-D}$ , where the representative element size is  $\Delta h^{QE} \leq \lambda/2$ ) for modelling wave propagation.

Since wavelength  $\lambda$  is directly proportional to the wave velocity  $v$  and inversely proportional to the frequency  $f$ ,  $\lambda = v/f$ , we can devise a simple rule for appropriate size of finite elements for wave propagation problems:

- For linear interpolation finite elements (1-D 2-node truss, 2-D 4-node quad, 3-D 8-node brick), the representative finite element size needs to satisfy the following condition.
- For quadratic interpolation finite elements (1-D 3-node truss, 2-D 9-node quad, 3-D 27-node brick) the representative finite element size needs to satisfy the following condition.

While the rule for number of elements (or element size  $\Delta h$ ) can be used to delineate models with proper and improper meshing, in reality having bigger finite element sizes than needed by

the above rule will not filter out higher frequencies at once, rather they will slowly degrade with increase in frequency content.

Simple analysis can be used to illustrate above rules [44]. When material becomes nonlinear (elastic-plastic), the stiffness of the material is reduced, and thus the finite element size is reduced as well. Cases with nonlinear material are described by Watanabe et al. [162].

#### *7.3.3.4. Material modelling and assumptions*

Material models that are used for site response need to be chosen to have an appropriate level of detail in order to model important aspects of response. For example, for a nuclear installation site where it is certain that 1-D waves will model all the important aspects of response, and that motions will not be large enough to excite fully nonlinear response of soil, and where volumetric response of soil is not important (soil does not feature volume change during shearing), 1C equivalent linear models can be used. On the other hand, for nuclear installation sites where full 3C wave fields are expected to provide important aspects of response (3C wave fields develop due to irregular geology, topography, seismic source characteristics/size, etc.), and where it is expected that seismic motions will trigger full nonlinear/inelastic response of soil, full 3C elastic-plastic material models need to be used. More details about material models that are used for SRA are described in Section 4.2.

#### *7.3.3.5. 1-D/3C vs 1-D/2C vs 1-D/1C material behaviour and wave propagation models*

In general, the behaviour of soil is in 3D and nonlinear/inelastic. In some cases, simplifying assumptions can be made and soil response can be modelled in 2-D or even in 1-D. Modelling soil response in 1-D makes one important assumption, that the volume of soil during shearing will not change (there will be no dilation or compression). Usually this is only possible if soil (sand) is at the so-called critical state [170] or if soil is a fully saturated clay, with low permeability, hence there is no volume change (see Section 8.4.2).

If soils will be excited to feature a full nonlinear/inelastic response, full 3-D analysis and full 3-D material models need to be used. This is true since for a full nonlinear/inelastic response it is not appropriate to perform superposition, so superimposing 3×1 D analysis, is not right.

The use of 1-D/3C models might be appropriate for seismic motions and behaviour of soil that is linear elastic. Vertical motions recorded on soil surface are usually a result of surface waves (Rayleigh). Only very early vertical motions/wave arrivals are due to compressional, primary (P) waves. Modelling of P waves as 1-D vertically propagating waves is then appropriate. However, modelling of vertical components of surface (Rayleigh) waves as vertically propagating 1-D waves is not appropriate for all frequencies, as noted above and in recent paper [166]. Elgamal also provide a description of vertical wave/motions modelling problems [171].

## 7.4. STANDARD AND SITE SPECIFIC RESPONSE SPECTRA

### **7.4.1. Introduction**

The amplitude and frequency characteristics of the free field ground motion (in 1-D) are one of the most important elements of the SSI analyses. Generally, the free field ground motion is defined by ground response spectra. The ground response spectra may be site independent, i.e. uncorrelated or weakly correlated with site specific conditions, or site specific, i.e. the end product of SHA (deterministically or probabilistically derived) as discussed in Sections 6.2 and 6.3. These cases are discussed in this section.



## 7.4.2. Standard response spectra

For nuclear power plants, depending on the vintage of the plant and the site soil conditions, the majority of the design ground response spectra have been relatively broad-banded standard spectra representing a combination of earthquakes of different magnitudes and distances from the site. Construction of such design spectra is usually based on a statistical analysis of recorded motions and frequently targeted to a 50% or 84% NEP. Three points are important relative to these broad-banded spectra. First, earthquakes of different magnitudes and distances control different frequency ranges of the spectra. For example, small magnitude earthquakes contribute more to the high frequency range than to the low frequency range. Second, it is highly unlikely that a single earthquake will have frequency content matching the design ground response spectra. Hence, a significant degree of conservatism is added when broad-banded response spectra define the control motion. Third, a single earthquake can have frequency content that exceeds the design ground response spectra in selected frequency ranges. The likelihood of the exceedance depends on the NEP of the design spectra.

Currently, standard or site independent ground response spectra are most often used in the design process for a new reference design or a certified design for a NPP. Such designs are intended to be easily licensed for a large number of sites in many different seismic environments and site conditions. Therefore, the DBE ground motion is defined by broad-banded standard ground response spectra that are site-independent or weakly site-dependent and termed certified seismic design response spectra (CSDRS).

Another application of broad-banded standard ground response spectra is the verification that the FIRS satisfy a minimum specified design basis ground motion, as follows:

- (a) A minimum of 0.1g PGA, as recommended in SSG-67 [2];
- (b) In accordance with Appendix S to 10 CFR Part 50 [172], the minimum PGA for the horizontal component of the SSE at the foundation level in the free field needs to be 0.1g or higher. The response spectrum associated with this minimum PGA needs to be a smooth broadband response spectrum (e.g., RG 1.60 [173], or other appropriate shaped spectra, if justified) and is defined as outcrop response spectra at the free field foundation level.

Figure 23 provides examples of standard ground response spectra for the horizontal direction:

- (a) U.S. NRC Regulatory Guide 1.60 (Rev. 2, 2014) [173];
- (b) U.S. NRC Regulatory Guide 1.60 enhanced in the high frequency range, which is the CSDRS for the Westinghouse AP1000 Pressurized Water Reactor;
- (c) European Utility Requirements (EUR) [174] - three design spectra corresponding to three broad site conditions, i.e., hard rock, medium soil, and soft soil;
- (d) Advanced CANDU Reactor (ACR-1000) [175] – two reference design response spectra corresponding to two broad site conditions, i.e., rock and soil.

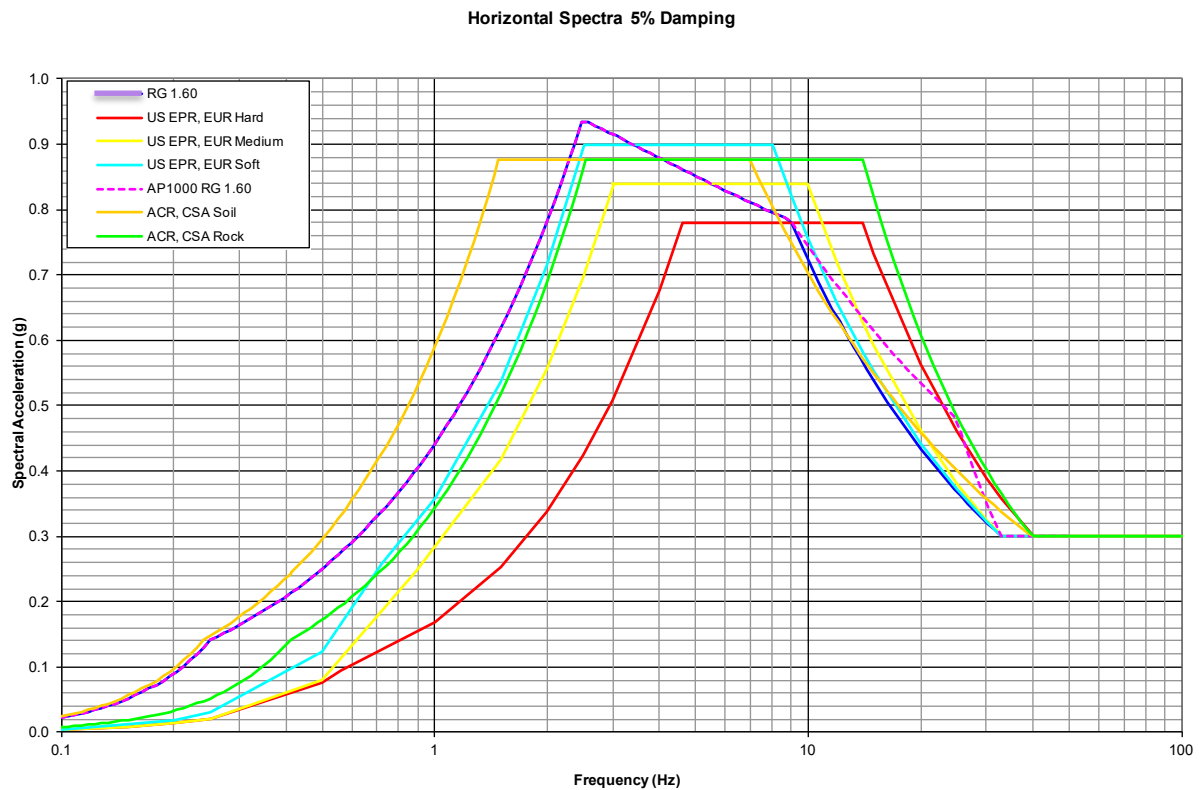


FIG. 23. Examples of site independent design basis earthquake ground motion response spectra for standard design or reference design nuclear power plants.

### 7.4.3. Site specific response spectra

For this discussion, it is assumed that a PSHA has been performed and UHRS is developed for the range of AFEs of the ground motion. This range of AFEs is from about  $1 \times 10^{-2}$  to  $1 \times 10^{-7}$ . Section 2.3.2 introduces one example of performance goals, which is based on the design of nuclear installation SSCs to achieve less than a 1% probability of unacceptable behaviour at the DBE level and less than a 10% probability of unacceptable behaviour for a ground motion equal to 150% of the DBE level. Unacceptable behaviour is tied to the Seismic Design Category (SDC) and its probabilistically defined acceptance criteria.

The concept is based on performance goals. The aim is to develop a ground motion response spectrum that, when coupled with seismic response procedures and seismic design procedures, will confidently achieve the probabilistically defined performance goal.

Given this context, ASCE 43-19 [20] and U.S. NRC Regulatory Guide 1.208 [176] establish a performance-based approach to developing the DBE (i.e. the GMRS) that achieves these goals. The concept is to assume the above performance goal (1% and 10%) is achieved for an SSC of interest (or the nuclear installation) and associate it with a lognormal probability distribution with hypothesized lognormal standard deviations (based on previously performed studies of SSC and installation performance).

For high hazard facilities like an NPP, focus on a performance goal of mean annual probability of unacceptable behaviour of  $1 \times 10^{-5}$ . Two UHRS: mean  $1 \times 10^{-4}$  and mean  $1 \times 10^{-5}$  are considered and the relationships between the individual spectral accelerations (at the discrete frequencies of the calculated seismic hazard curves) at these mean annual UHRS are developed. The scale factors to be applied to these spectral accelerations to obtain risk consistent GMRS are developed. The scale factors (less than or equal to 1) are based on previously performed studies

of the integration of seismic hazard curves over the lognormal probability distribution of performance as introduced above. Scale factors are applied to the mean  $1 \times 10^{-5}$  seismic hazard curves and the GMRS is constructed by connecting the spectral accelerations at these discrete natural frequencies.

The end result is the risk consistent definition of the DBE (GMRS) for a nuclear installation site, which is associated with the performance criteria of less than a 1% probability of unacceptable behaviour at the DBE and less than a 10% probability of unacceptable behaviour at ground motion 150% times the DBE. The GMRS concept is to be applied consistently to the free field ground motion that serves as the seismic input to the SSI analyses.

## 7.5. TIME HISTORIES

Generally, SSI analyses are performed for seismic input defined by acceleration time histories. As a minimum, three spatial components of ground motion are needed, i.e. two orthogonal horizontal components and the vertical. These three components are acceleration time histories that correspond to the response spectra described previously.

Section 7.4 describes ground motion response spectra as standard response spectra (site independent or weakly correlated to site conditions) and the response spectra calculated specifically for a given nuclear installation site at a rock outcrop, on the surface of the soil at top of grade, or at locations within the site profile. Hereafter, these are referred to as target spectra.

In general, the approach to developing acceleration time histories for use in the SHA is to select and modify recorded ground motions. Seismological characteristics play a role in the selection, i.e. parameters, such as magnitude and distance of earthquakes with major contributions to the seismic hazard curves are selected. These records contain characteristics of the contributing earthquakes, such as response spectral shape, energy content, and strong motion duration. These records are termed 'seed records'.

Two approaches have been used to modify the recorded motions for use in seismic analysis:

- (a) Scaling the selected seed time histories by constant factors over the complete record to approximately mimic the target spectra;
- (b) Implementing spectral matching software, such as RSPMatch2005 [177, 178] and its successor RSPMatch2009 [179], which modify recorded motions through the introduction of wavelets at selected frequencies to better match the target spectra. The program RSPMatch2009 provides a stable and time-efficient solution without introducing drift to the resulting velocity and displacement time series. It also allows matching records to pseudo-acceleration response spectra and ensures convergence and stability of the solution.

The first approach has been superseded by the second approach in the majority of applications.

A third approach is the generation of synthetic, simulation based time histories for the given source and site conditions. In principle, the synthetic time histories do not need scaling or spectral matching. However, as discussed in Sections 6.2 and 7.3.3, FFS currently are limited to about 3 Hz with developments in progress to increase the frequency content to about 10 Hz.

In generating acceleration time histories to represent or match the target response spectra, the following elements need to be considered [180];

- (a) Individual time histories generated to match or fit a single target response spectrum may produce in-structure responses, as calculated by SSI analyses that are not conservative;

- (b) Individual time histories generated to match or fit a single target response spectrum at a given damping factor (most often 5% damping) may produce responses that are not conservative for system damping levels other than the given damping value.

The overall goal is to achieve a mean-based fit of the single time history response spectrum or average of the multiple time history response spectra that has a tight fit to the target response spectra without deficiencies or large exceedances at any frequency. The single or average Fourier amplitude spectrum do not have gaps in the comparison for any frequency and need not be overly conservative.

For each member of the set of three spatial components of time histories, the following conditions need to be met:

- (a) The time step ( $dt$ ) of time histories needs to be small enough to capture the frequency content of interest (i.e. at a Nyquist frequency ( $f$ ), the corresponding time step is calculated as  $dt = 1/(2*f)$ ). In all cases, the time step needs to be no greater than 0.01 sec, which is a Nyquist frequency of 50 Hz.
- (b) Strong motion duration of the time histories needs to be chosen in relation to the earthquake magnitude of the earthquake scenarios determined from DSHA or assessed from deaggregation of the seismic hazard curves for PSHA. Strong motion duration is the effective duration defined as the time for Arias intensity to build up from 5% to 95% of its full value [181]. If recorded motions are the basis for the generated time histories, the Arias intensity after record modification needs to approximate the Arias intensity prior to modification.
- (c) Response spectra calculated from the time histories for comparison with the target spectrum need to be at frequency increments corresponding to a minimum of 100 points per frequency decade. If the RSPMatch software is used, a denser set of frequency points than 100 per frequency decade is preferred.
- (d) Guidelines are needed as to the number of frequency points at which the calculated response spectra may lie below the target (and how far below) and above (and how far above). These guidelines apply to a single set of time histories or to the average of the multiple sets of time histories:
- To not exceed the target by more than 30% in the frequency range of interest; if the exceedance is more than 30%, the power spectral density function of the time history of interest be calculated and verify that no gaps in energy exist.
  - To not fall below the target spectra by more than 10% at any single frequency. Similarly, a limitation on the number of adjacent frequency points at which the calculated response spectra may fall below the target spectra is to be considered.
- (e) The general relationships of (peak acceleration)-to-(peak velocity)-to-(peak displacement) for the single time history or average of the multiple time histories is to be maintained. Other indicators like the Arias intensity, cumulative absolute velocity ideally are also preserved. This holds for the standard response spectra and for the governing earthquakes for the site specific response spectra as determined from deaggregation of the seismic hazard curves.
- (f) The deaggregation of the seismic hazard curves is done at different frequencies, and the time histories have to be generated for all deaggregation scenarios of importance to the soil-structure system. As discussed above, the general relationships of (peak acceleration)-to-(peak velocity)-to-(peak displacement) for the single time history or average of the multiple time histories be maintained for each of the deaggregated scenarios.
- (g) The three components of ground motion need to be statistically independent, as determined by the directional correlation coefficients between pairs of time histories. The absolute values of the correlation coefficients need to be less than a specified amount (either 0.30 or 0.16 have been specified [182]). These criteria are easily met.

When considering multiple sets of ground motion time histories as input to SSI analyses, the number and their individual characteristics are strongly influenced by the following:

- (a) Deterministic SSI analyses where soil–structure properties are held constant for a given set of soil and structure properties, e.g. best estimate, lower bound, and upper bound soil properties, for design basis or DBBE analyses. This assumes each individual time history meets the target response spectra. Multiple time history sets serve the purpose of accounting for large variability in the calculated seismic response due to input time history variability. Two example approaches are:
  - (1) Five-time history sets are used in the SSI analyses and each set meets the above criteria. The seismic responses are averaged over the five results for use in design. Also, used in DBBE assessments where fully probabilistic SSI analyses are not performed.
  - (2) For nonlinear analyses, ASN2/01 [21] recommends to retain the mean plus a fraction  $\lambda$  of the standard deviation of the responses;  $\lambda$  is function of the number of times histories used and is based on the Student-Fischer test at a 95% confidence interval: for 5 time histories  $\lambda = 0.95$  and for 10,  $\lambda = 0.58$ .
- (b) Probabilistic SSI analyses (Section 8.5) for design basis or DBBE ground motions. The assumed ground motion variability represents the variability in the phase and directional components, but not in the frequency content. The physical properties of the soil and the structure are modelled explicitly in the probabilistic SSI analyses, as follows:
  - The results of the SSI analyses are the seismic responses for design, e.g., an 80% NEP value conditional on the ground motion definition.
  - If the results of the SSI analyses are an element in the definition of the seismic demand for a process that includes convolution of the seismic hazard with the end results of the assessment process, e.g., an SPRA, then the variability in the time histories need not include aleatory uncertainty in the time history characteristics. This is to avoid double-counting of aleatory uncertainty, since it is included in the seismic hazard curves.
  - Double counting of uncertainty in ground motion and soil properties is to be avoided.

The number of earthquake simulations to be performed is dependent on the simulation procedure used, e.g. a combination of stratified sampling of probability distributions in conjunction with a Latin hypercube experimental design involves many less simulations than a full Monte Carlo approach.

For linear or equivalent linear seismic analyses (including SSI analyses), acceleration time histories meeting these conditions are appropriate and adequate. For nonlinear seismic analyses, especially nonlinear SSI analysis, actual recorded time histories may be preferred. Before using recorded acceleration time histories directly in seismic analysis, the corresponding velocity and displacement time histories need to be verified to ensure that baseline drift is not present. If baseline drift is present, baseline corrections need to be implemented. This process is mainly to remove the linear signals trends and to use high pass filter for the time history. The final aim is to obtain velocity time histories with zero mean and zero end values, and displacement time histories without residual displacement. This is especially important for nonlinear analyses, especially nonlinear SSI analyses.

An additional approach that is becoming state-of-practice when applicable is the conditional spectrum approach [183] of partitioning the ground response spectra into contributing scenarios and selecting and/or generating time histories to use in the seismic analysis customized to the earthquake parameters of interest.

## 7.6. UNCERTAINTIES

Epistemic (E) and aleatory (A) uncertainties are to be included in the probabilistic analyses, as follows:

- (a) Ground motion – phase variability between the three spatial components (A), vertical vs horizontal components (E, A), rock seismic hazard curves/UHRS (E, A), etc.
- (b) Stratigraphy:
  - Idealized soil profile – thickness of layers (A), correlation of soil properties between layers (A), variation of soil properties in discretized layers (A), etc;
  - Non-idealized soil profile – geometry (E, A).
- (c) Soil behaviour:
  - Material model (linear/equivalent linear) stiffness;
  - Energy dissipation.
- (d) Wave propagation mechanism (E, A).

## 7.7. LIMITATIONS OF TIME AND FREQUENCY DOMAIN METHODS FOR FREE FIELD GROUND MOTIONS

The limitations of time and frequency domain methods for free field modelling are twofold. The first source of limitations is based on a (usual) lack of proper data for (i) deep and shallow geology and surface soil material, and (ii) earthquake wave fields. These limitations can be overcome by more detailed site and geologic investigation. The second source of limitations is based on the underlying formulations for both approaches. Time domain methods, with nonlinear modelling, involve the use of a sophisticated analysis program and sophistication from the analyst, including knowledge of nonlinear solutions methods, elasto-plasticity, etc. Technical limitations on what (detailed) modelling can be done are usually with the analyst and with the program that is used (different programs allow different level of modelling detail). On the other hand, frequency domain modelling requires significant mathematical sophistication by the analyst. In addition, frequency domain based methods are limited to linear elastic material behaviour (as they rely on the principle of superposition) which limits their usability for seismic events where nonlinearities are expected.

# 8. METHODS AND MODELS FOR SOIL–STRUCTURE INTERACTION ANALYSIS

## 8.1. BASIC STEPS FOR SOIL-STRUCTURE INTERACTION ANALYSIS

### 8.1.1. Preparatory activities

The first aim of SSI analysis for a nuclear installation is to identify candidate SSI models, model parameters, and analysis procedures, as follows:

- (a) Determine the purposes of the SSI analysis and define the use of results:
  - (i) Seismic response of structure for design or assessment (forces, moments, stresses or deformations, story drift, number of cycles of response);
  - (ii) Input to the seismic design, qualification, evaluation of subsystems supported in the structure (time histories of acceleration and displacement), ISRS, number and amplitude of cycles for components, etc.;

- (iii) Base-mat response for base mat design;
  - (iv) Soil pressures for embedded wall designs;
  - (v) Structure-soil-structure analysis.
- (b) Scope the problem, identify all relevant phenomena that will be simulated, e.g. seismic wave fields; linear, equivalent linear, or nonlinear/inelastic response for soil (dry - single phase) or saturated soil (effective stress, fully coupled analysis or total stress analysis); linear or nonlinear simulation of soil-structure-foundation contacts; and linear, equivalent linear, or nonlinear structure behaviour.
  - (c) Determine relative importance of phenomena to be modelled. This generally involves the ability to perform numerical experiments with numerical tools that properly model the phenomena in both a simplified and a detailed manner for comparison purposes.
  - (d) Decide on the numerical codes to be used, such as: sub structuring or direct methods for linear or equivalent linear models; direct methods for linear, equivalent linear, or nonlinear models; discretization of the soil portion of the model (either idealized stratigraphy or complex profile) i.e. using finite element, boundary element, finite difference, or some other approach.
  - (e) Confirm the availability of a verification suite for a chosen numerical code. A verification suite of the numerical code features to be used in the analyses is needed, as a minimum. This verification suite needs to be available (published) and accessible to the general user community. Estimation of numerical errors for code components (solution advancement algorithms, elements, material models, etc.) also need to be provided within the verification suite.
  - (f) Confirm availability of a validation suite for material models that are to be used in modelling and simulation.
  - (g) Perform one or more sensitivity studies for relevant modelling and simulation parameters in order to determine sensitivity of solution(s) to modelling and solution parameters, such as a sensitivity study for relevant material model parameters, or a sensitivity study evaluating simulation parameters (e.g. the values of the parameters needed for the Newmark time integration algorithm). Sensitivity studies are to be documented for the overall analysis process or specifically for the SSI analysis of interest.
  - (h) Develop a set of simplified solutions, using simplified models (with an understanding that these models do introduce modelling uncertainty, however these models are easier to manage and offer an efficient way to start a hierarchy of models, from less detailed to more detailed).
  - (i) Recognize the need for educated, knowledgeable users and experts to perform and consult on the analyses.

### **8.1.2. Site specific modelling**

For a specific nuclear installation site and structure, several choices need to be made concerning the level of detail of a model:

- (a) Determining the characteristics of the subject ground motion (seismic input motion) (see Sections 5, 6, and 7):
  - (i) Excitation level and frequency content (low vs high frequency):
    - Low frequency content (up to 10 Hz) affects structure and subsystem design/capacity;
    - high frequency content (> 10 Hz) affects operation of mechanical/electrical equipment and components;
  - (ii) Incoherence of ground motion;

- (iii) Are ground motions 3-D? Are vertical motions coming from P or S (surface) waves. If from S and surface waves, are these full 3-D motions?
- (b) Determining the characteristics of the site to be used in the SSI modelling and analyses. Section 7 results are applicable to free field response aspects of the soil, as follows:
- (i) Establishing the site profile for the free field motion from Section 7 (linear/equivalent linear soil material properties or nonlinear soil properties).
  - (ii) Determining if an idealized site profile is applicable:
    - Is a linear or equivalent linear soil material model applicable (visco-elastic model parameters assigned)?
    - Is a nonlinear (inelastic, elastic-plastic) material model necessary?
  - (iii) If a non-idealized site profile is necessary and to be developed, deciding whether to use complex site stratigraphy and/or nonlinear soil material models, as follows:
    - Linear or equivalent linear soil material model (visco-elastic model parameters assigned);
    - Nonlinear (inelastic, elastic-plastic) material model– construct nonlinear soil island model.
  - (iv) Performing sensitivity studies to clarify model requirements for site characteristics (complex site stratigraphy, inelastic modelling, etc.).
- (c) Determine the characteristics of a structure of interest at the nuclear installation (identify structures important to safety, such as structures housing safety related equipment) and large components for which SSI is important, as follows:
- (i) Identify the function(s) to be performed during and after earthquake shaking, e.g. containment (confinement of radioactive substances and radiation shielding in operational states and in accident conditions; structural support to subsystems (equipment, components, distribution systems); prevention of a failure that might cause failure of SSCs important to safety.
  - (ii) Identify load bearing systems for modelling purposes, e.g. shear wall structures, steel frame structures.
  - (iii) Assess the expected behaviour of structure (linear or nonlinear).
  - (iv) Based on initial linear model of the structure, perform preliminary seismic response analyses (response spectrum analyses) to determine stress levels in structure elements.
  - (v) If significant cracking or deformations are possible such that portions of the structure behave nonlinearly, refine the model either approximately by introducing cracked properties, or model portions of the structure with nonlinear elements.
  - (vi) For expected structure behaviour, assign material damping values.
  - (vii) Determining the frequency range of interest – especially high frequency considerations (50 Hz, 100 Hz).
- (d) Determine the foundation structure characteristics, as follows:
- (i) Effective stiffness is rigid due to base mat stiffness and added stiffness due to structure being anchored to base mat, e.g., honey-combed shear walls anchored to the base mat.
  - (ii) Effective stiffness is flexible, e.g. if additional stiffening by the structure is not enough to assume rigid behaviour; or for strip footings.



Modelling sequence needs to be established (note many of these steps of verification and validation are performed generically and only require an evaluation of applicability to the SSI analysis of interest).

- (e) Develop linear models of the various elements of the combined SSI model then slowly complete the model, as follows:
  - (i) Soil only:
    - Apply static and/or dynamic loads and free field ground motion to verify model behaviour.
  - (ii) Structure only:
    - Develop dynamic models of the components of the overall nuclear installation structure, e.g., nuclear island structures modelled independently prior to assembling a combined model – containment, internal structure, shielding buildings, and others. For example, the evolutionary power reactor has nine separate substructures.
    - Start with a fixed base model for each independent structure (and independently apply gravity in all three spatial directions to verify the model). Apply static point loads as applicable, dynamic analyses as applicable (eigensystem extraction, point loads, free field ground motions) and verify load paths and dynamic behaviour. Independently verify the models, then assemble the full structural model and repeat static and dynamic analyses as applicable to verify the fully assembled structural model.
  - (iii) Develop foundation model and repeat the benchmarking analyses as applicable.
  - (iv) Complete structure, foundation, and soil system model.
- (f) Develop equivalent linear models, and observe changes in response, to determine possible plastification effects (this is still an elastic analysis, with reduced (equivalent) linear stiffness; reduction in secant stiffness stems from plastification, although plastification is not explicitly modelled, hence an idea can be obtained of possible effects of reduction of stiffness). Care is needed in observing these effects and the focus needs to be on verification of the model (for example wave propagation through softer soil, frequencies will be damped, etc.).
- (g) In nonlinear/inelastic modelling, nonlinearities are introduced slowly and models, convergence and stability, in all the components discussed above can be tested.
- (h) Investigate sensitivities for both linear elastic and nonlinear/inelastic simulations.
- (i) Develop documentation according to project requirements and procedures on modelling choices/assumptions, uncertainties (how are they dealt with) results, etc.
- (j) Model development involves a hierarchical set of models with gradual increase in the level of detail. As hierarchy of models is developed, each model needs to be verified and be capable to (properly, accurately) model phenomena of interest.
- (k) Model verification is used to verify that mechanical features that are of interest are indeed properly modelled. In other words, model verification needs to prove that results obtained for a given (developed) model are accurately representations of the features of interest. For example, if propagation of higher frequency motions is needed, it is necessary to verify that the developed model is capable of propagating waves of certain wavelengths and frequency. Model verification is different to code and solution verification and validation (see Section 10). Model verification is to be performed for each developed model in order to gain confidence that modelling results are acceptable.
- (l) Arrange for an independent participatory peer review. ‘Participatory’ refers to continuous review through the SSI analysis process.

## 8.2. DIRECT METHODS

The direct method analyses the idealized soil–structure system in a single step. The direct method is applicable to linear and equivalent linear idealizations and is needed for nonlinear SSI analyses. This is in contrast to the substructure method that divides the SSI problem into a series of simpler problems, solves each independently, and superposes the results. The substructure method is limited to linear and equivalent linear idealizations since it relies on superposition.

### 8.2.1. Discrete methods

The mechanics of solids and structures relies on equilibrium equations of external and internal forces/stresses and/or equations of motion [184, 185] they form a basis for both the FEM and the finite difference method (FDM).

#### 8.2.1.1. Finite element method

A general formulation of the finite element method [186, 187] is used to address the SSI problem. Solid and structural finite elements with elastic and inelastic (nonlinear, elastic-plastic) material (see Section 4), are necessary to properly model the SSI problem. Seismic input is performed using seismic motions (i.e., a 3-D seismic wave field that can be simplified to 1-D or 3x1-D, see Section 7) and a number of different methods, one of which, the domain reduction method [188] is described below.

Different types of finite elements can be broadly classified as follows:

- (a) Solid elements (3-D brick, 2-D quads etc.);
- (b) Structural elements (truss, beam, plate, shell, etc.);
- (c) Special elements (contacts, etc.).

Standard single phase and two phase, elastic or inelastic finite elements are used in all instances [186, 187, 189–192]. Special elements are used for modelling contacts, base isolation and dissipation devices and other special structural and contact mechanics components of an NPP soil–structure system [193].

It is important to note that the choice of finite elements is dictated by the problem that is analysed, and by the desired level of accuracy. For example, the structure can be modelled using structural or solid finite elements.

#### 8.2.1.2. Finite difference method

Finite difference methods operate directly on dynamic equilibrium, when it is converted into dynamic equations of motion. The FDM represents differentials in a discrete form. It is best used for elasto-dynamics problems where stiffness remains constant. In addition, it works best for simple geometries [87], as finite difference method requires special treatment of boundary conditions, even for straight boundaries that are aligned with coordinate axes.

The FDM solves dynamic equations of motion directly to obtain displacements or velocities or accelerations, depending on the problem formulation. Within the context of the elasto-dynamic equations, on which FDM is based, elastic-plastic calculations are performed by changes to the stiffness matrix, in each step of the time domain solution.

### 8.2.2. Linear finite element methods

Linear finite element method has been covered in a number of books over last 50 years, starting from early books by Zienkiewicz [194] and Bathe and Wilson [195]. All the books on nonlinear finite elements do feature sections on linear finite elements.

### 8.2.3. Nonlinear finite element methods

Nonlinear problems can be separated [192, 196-198], as follows:

- (a) Geometric nonlinear problems, involving smooth nonlinearities (large strains and large displacements);
- (b) Material nonlinear problems, involving rough nonlinearities (elasto-plasticity, damage, gapping).

For SSI problems, nonlinear finite element methods dominate.

The main interest in modelling of soil–structure interaction is associated with material nonlinear problems. Geometric nonlinear problems involving large displacements and large strains are of interest for the  $P$ – $\delta$  effect, as well as for soil and structural failure.

Sometimes, contact problems where gapping occurs (opening and closing of gaps) are called geometric nonlinear problems. For the problems of interest here, namely, gap opening and closing between foundation and soil/rock, these problems are not geometrically nonlinear. They are not geometric nonlinear problems in the sense of large deformation and large strains. Problems where a (small) gap opens and closes are material nonlinear problems where material stiffness (and internal forces) vary between very small values (zeros in most formulations) when the gap is opened, and large forces when the gap is closed.

Soil can be modelled using:

- (a) Linear elastic models, where linear elastic stiffness is the initial stiffness or the equivalent elastic stiffness [86, 199, 200], as follows:
  - The initial stiffness uses the highest elastic stiffness of a soil material for modelling. It is usually used for modelling small amplitude vibrations. These models can be used for 3-D modelling.
  - Equivalent elastic models use secant stiffness for the average high estimated strain (typically 65% of maximum strain) achieved in a given layer of soil. Eventual modelling is linear elastic, with stiffness reduced from initial to approximate secant.
- (b) Nonlinear 1-D models, which comprise variants of hyperbolic models (described in Section 4.2), utilize a predefined stress–strain response in 1-D (usually shear stress  $\tau$  versus shear strain  $\gamma$ ) to produce stress for a given strain.
- (c) There are other nonlinear elastic models that define stiffness change as a function of stress and/or strain changes [199, 201-204]. These models can successfully model 1-D monotonic behaviour of soil in some cases. Special approaches with stress invariants can extend these methods to 3-D. In addition, algorithmic measures need to be used to make these models work with cyclic loads.
- (d) Elastic-Plastic material modelling can be quite successfully used for both monotonic, and cyclic loading conditions [205-210]. Elastic plastic modelling can also be used for limit analysis [211].

Material nonlinear problems for concrete can be modelled on two levels, as follows:

- (a) Solid concrete level, where concrete is modelled using solid models and 2-D or 3-D elastic-damage-plastic material models [212-215];
- (b) Beam and/or plate/wall cross section, where 1-D fibres are used to model nonlinear normal stress behaviour (concrete and reinforcement) in cross section [216-218]. In this type of modelling, influence of shear stress is neglected, and pure bending is assumed.

The interface is modelled using nonlinear inelastic constitutive law within contact elements [193].

#### **8.2.4. Inelasticity, elasto-plasticity**

Material models for elastic-plastic analysis of the SSI problem are described in detail in Section 4.

#### **8.2.5. Dynamics solution techniques**

On the global, finite element level, finite element equations are solved using time marching algorithms. Most often used are the Newmark algorithm [219] and the Hilber-Hughes-Taylor (HHT)  $\alpha$  algorithm [220]. Other algorithms exist [167, 195, 221], however they are used less frequently. Both the Newmark and HHT algorithm allow for numerical damping (through appropriate choice of the parameters,  $\beta$  and  $\gamma$  for the Newmark method, and  $\gamma$  for the HHT method) to be included in order to damp out higher frequencies that are introduced artificially into FEM models by discretization of continua into discrete finite elements.

A solution to the dynamic equations of motion can be done by either enforcing or not enforcing convergence to equilibrium (implicit versus explicit methods). Enforcing the equilibrium usually requires use of Newton or quasi Newton methods to satisfy equilibrium within some tolerance. If this tolerance is small enough, the analyst is assured that his/her solution is within the proper material response and equilibrium. Solutions without enforced equilibrium are faster, and if they are done using explicit solvers, there is a requirement of small time step, which can then slow down the solution process.

It is important to note that there are two levels of equilibrium:

- (a) Global level (mentioned above) where external forces are balanced with internal forces from elements;
- (b) Constitutive level, where the local stress state is balanced and iterated upon until it is returned to the yield surface for elastic-plastic material.

In both cases, the analyst might choose not to enforce equilibrium, for example if time steps are very small. However, it is important that a sensitivity study is performed to confirm that this approximation is appropriate.

#### **8.2.6. Energy dissipation**

Seismic energy that enters the Soil Foundation Structure (SFS) system will be dissipated in a number of ways. A part of the energy that enters the Soil Foundation Structure system can be reflected back into domain outside by:

- (a) Wave reflection from impedance boundaries, such as free surface, soil/rock layers, foundations, etc.
- (b) Structural system oscillation radiation.

While the rest of seismic energy is dissipated through one of the following mechanisms within the soil-structure domain:

- (a) Elasto-plasticity of soil, contact and the structural components.
- (b) Viscous coupling of
  - (i) porous solid with pore fluid,
  - (ii) structure with surrounding fluids.

In numerical simulations, part of the energy can be dissipated or produced by purely numerical means. That is, numerical energy dissipation (damping) or production (negative damping) needs to be carefully controlled [167, 221].

It is also important to note that plastic dissipation is not the same as plastic work [222, 223]. Proper calculation of plastic dissipation for elastic plastic material needs to be used [224].

Recent papers by Yang et al. [225] provide more details on how to calculate energy dissipation from material hysteretic response, viscous interaction and algorithmic damping.

### 8.3. SUBSTRUCTURE METHODS

#### 8.3.1. Principles

The natural progression for addressing the SSI analysis of nuclear installation structures was to implement tools developed for machine vibrations, which took into account the dynamic behaviour of the semi-infinite half-space (i.e., the force–displacement behaviour defined by impedance functions that are complex-valued and frequency dependent). For a foundation assumed to behave rigidly, these impedances are uniquely defined by 6x6 frequency-dependent matrices of complex-valued impedances relating foundation forces and moments to six rigid body degrees of freedom. For foundations that behave flexibly, a sufficient number of flexible impedance matrices (frequency-dependent and complex-valued) are developed relating forces and displacements.

The need to break the soil–structure problem into more manageable parts has led to substructuring methods.

Generally, substructuring methods applied to the SSI problem are considered to be a linear process, i.e. each step in the analysis is solved separately and then the separate parts are combined through super-position to solve the complete problem. Most substructuring methods solve the SSI problem in the frequency domain: the seismic input is defined by acceleration time histories, which are transformed into the frequency domain for the analysis. Scattering functions that define the foundation input motion are frequency dependent; impedance functions that define the force-displacement relationships between foundation node points (or the rigid foundation) are complex-valued, frequency-dependent functions; the dynamic behaviour of the structure is frequency-dependent (as demonstrated by eigen-system extraction into normal modes).

Conceptually, substructuring methods can be classified into four types depending on the methodology of treating the SSI, i.e. how the soil and structure interface degrees-of-freedom are treated. Figure 24 shows the four types. These four types are: (i) the rigid boundary method, where the term ‘rigid’ refers to the boundary between the foundation<sup>14</sup> and the soil; (ii) the

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<sup>14</sup> The term ‘foundation’ is used to denote the foundation of the nuclear installation structure (base mat) and partially embedded structure elements, such as exterior walls in contact with the soil.

flexible boundary methods; (iii) the flexible volume method or the direct method; and (iv) the substructure subtraction method.

The domain reduction method (DRM) is a direct method as it solves the (nonlinear) problem of earthquake soil structure interaction (ESSI) using a single model. However, the DRM can also be considered as a multistep method if one considers modelling of an earthquake process from the source (causative fault) to the NPP site. Regional model taking into account the fault, can be described as the first step, while ESSI modelling is then a second step. However, a large scale, regional model does not have to be used, input motions for DRM input can be developed using other, simplified methods, for example a direct deconvolution of surface motions. The DRM is described in Section 8.4.10 and in [188].

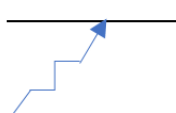
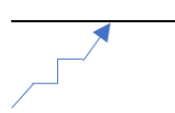

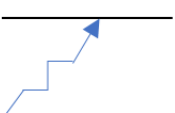
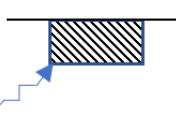
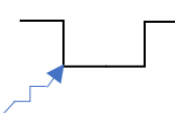
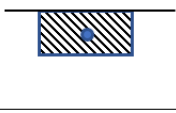
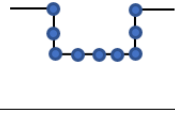
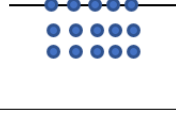
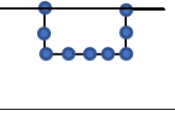
Method \ Analysis	Rigid Boundary	Flexible Boundary	Flexible Volume	Substruction
Site Response Analysis				
Scattering Analysis			None	None
Impedance Analysis				
Structural Response Analysis	Standard	Standard +	Standard +	Standard +

FIG. 24. Summary of sub structuring methods.

Unlike direct methods of analyses in which all the elements involved in a soil–structure model are gathered in a single model and the dynamic equilibrium equations (equation of motions) are solved in one step, substructuring methods replace the one-step analysis by a multi-step approach: each step in the process is analysed by the most appropriate tools available. The global model is replaced by several submodels of reduced sizes, which are solved independently and successively. To express that the individual models belong to the same global problem, compatibility conditions are enforced between the submodels: these compatibility conditions simply state that at the shared nodes of two submodels displacements are to be equal and forces needs to have the same amplitude but opposite signs. The substructuring approach is schematically depicted in Figures 25 and 26.

At nodes A and B, which are at the same location in space, the displacement vectors satisfy  $U_A = U_B$  and the force vectors  $F_A = -F_B$ . The partitioning between the two subsystems is made along the boundary separating the structure and the soil; however, other alternatives are possible. The boundary considered in FIG. Figure 26 does not need to be rigid, but the size of the problem is significantly reduced when this assumption holds.

Several approaches are proposed to handle substructuring methods. They differ essentially by the way in which the interaction between the two substructures is handled. One possibility is to define the interface between the two subsystems along the external boundary of the structure which can be either a plane (2-D model) or a surface (3-D model); the approach is known as

the (rigid or flexible) boundary method and is due to [8, 226]. Other substructuring methods have been introduced more recently; they are known as the flexible volume method [227] and the subtraction method [228].

The seismic SSI subproblems that sub structuring methods are used to solve are compared in Figure 24. As shown in Figure 24, the solution for the site response problem is needed by all methods. The analysis of the structural response problem is also necessary and involves essentially the same effort for all methods. However, the necessity and effort needed for solving the scattering and impedance problems differ significantly among the different methods. For the rigid boundary method, the scattering and impedance problems are typically solved simultaneously [229, 151] once Green's functions have been calculated or when the flexible impedance matrix is calculated. The flexible boundary model involves calculation of the scattering and impedance matrices separately. The flexible volume method and the substructure subtraction method, because of the unique substructuring technique, involve only one impedance analysis and the scattering analysis is eliminated. Furthermore, the substructuring in the subtraction method often involves a much smaller impedance analysis than the flexible volume method. The latter two methods are those implemented in SASSI. The most reliable SASSI method is the direct method which falls into the category of flexible volume method.

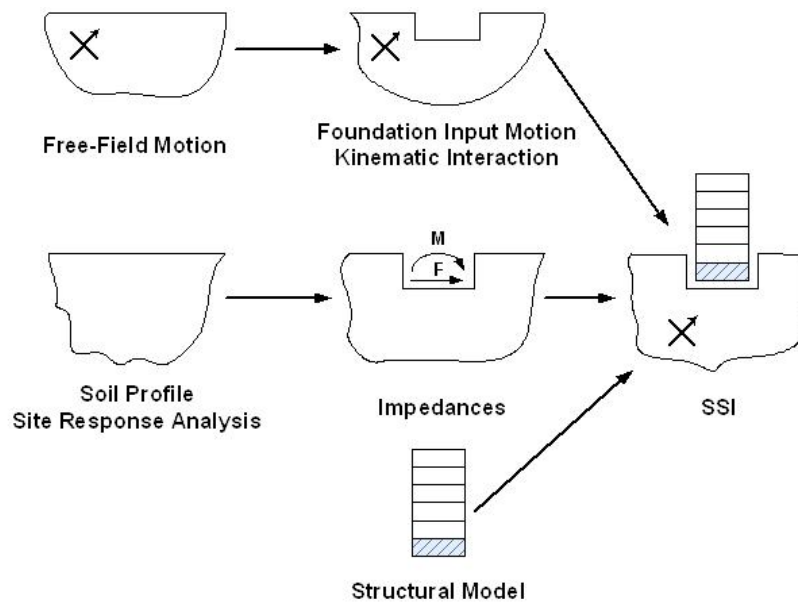


FIG. 25. Schematic representation of the elements of SSI-substructure method.

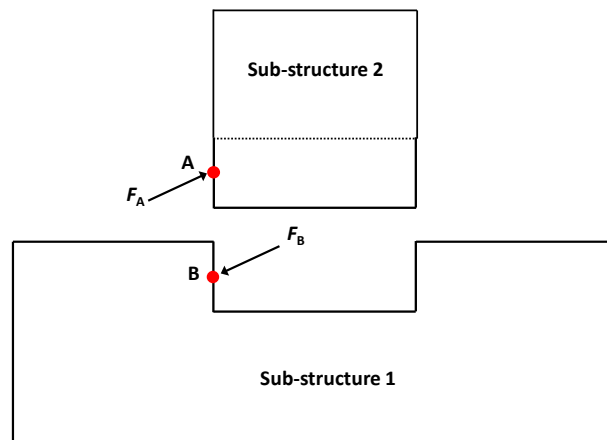


FIG. 26. Schematic representation of a substructure model.

Note also that the DRM can be viewed as a substructure method in which the substructure includes not only the structure but part of the soil. However, reference to substructure methods is usually limited in practice to one of the four methods described above.

The substructure methods apply only to linear systems. However, the nonlinear behaviour of the soil may be accounted for by using strain compatible properties derived from site response analyses carried out with the viscoelastic equivalent linear model.

### 8.3.2. Rigid or flexible boundary method

The multi-step approach involved in substructuring is based on the so-called superposition theorem established by Kausel and Roësset [8]. It is schematically depicted in Figure 25. A three-step method for SSI analysis is also presented in [230].

The solution to the global SSI problem is obtained by solving successively:

- (a) The scattering problem, which aims at determining the kinematic foundation motions, which represent at each foundation node the three (or six) components of motion of the massless foundation with its actual stiffness subjected to the incident wave field of the global SSI problem. These motions differ from the free field motions except for surface foundations subjected to coherent vertically propagating waves. In this case, scattering functions are real and equal to unity in the direction of the excitation of interest and zero in other excitation directions. For embedded foundations, scattering functions vary with the location of points on the interface between the soil and the foundation. This is due to the spatial variation of ground motion with the depth and width of the foundation and the scattering of waves by the same.  
For foundations modelled as behaving rigidly, the scattering matrix is a (6x3) matrix relating rigid body displacements and rotations to the three components of coherent free field ground motion (two horizontal and the vertical direction). See Section 7 for further discussion of the free field ground motion. The foundation input motion is defined as the result of multiplying the scattering matrix times the free field ground motion.
- (b) The impedance problem: the impedance function expresses the relationship between a unit harmonic force applied in any direction and at any location along the soil–foundation interface and the induced displacements at any node in any direction along the same interface. In calculating the impedance function, the foundation is massless and modelled with its actual rigidity, which is not necessarily infinite. The size of the impedance matrix is  $k_n$  by  $k_n$  where  $n$  is the number of nodes along the boundary and  $k$  is the number of degrees of freedom at each node. In the most general case  $k = 6$  and the stiffness matrix are full with  $6n$  by  $6n$  non-zero terms. When the foundation can be considered rigid, its kinematics can be described by the displacement of a single point<sup>15</sup>, for instance the geometric centre of the base mat, and the size of the frequency-dependent complex valued impedance matrix is 6 by 6 for each frequency of calculation for the analysis.
- (c) The structural analysis problem which establishes the response of the original structure connected to a support through the impedance functions and subjected to the kinematic foundation motions. It can be proved that, provided each of the previous steps are solved rigorously, the solution to third problem (step 3 of the analysis) is strictly equivalent to the solution of the original global SSI problem.

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<sup>15</sup> For foundations assumed to behave rigidly analysed by CLASSI, the point about which the scattering and impedance matrices are calculated is named the ‘foundation reference point’. This is also the point about which the SSI solution is generated.



Substructuring methods can be subdivided into methods that efficiently analyse nuclear installation structures whose foundation can be modelled as behaving rigidly or flexibly. This categorization is often tied to the multi-step vs single step analyses in conjunction with the required SSI analysis results.

Foundations that behave flexibly are treated by the flexible boundary method, flexible volume method (direct method), and the subtraction method – described later in this section.

Two programs that are extensively used to analyse linear elastic soil-structure systems when the foundation of the structure can be treated as behaving rigidly are CLASSI [229] and SUPELM [231]. CLASSI has evolved such that it is a staple tool in analyst's toolboxes. Section 8.3.6 describes the principal features of CLASSI. Section 8.3.6 also describes the hybrid method of CLASSI–SASSI, in which embedded foundations are modelled with SASSI and the resulting SASSI flexible impedance matrix is derived and condensed to six degrees-of-freedom for CLASSI analyses. The modules of CLASSI and SUPELM were used extensively in the SASSI verification and validation program (see Section 10.2) to benchmark individual modules of the SASSI program.

Substructuring methods are most efficiently implemented in the frequency domain, in which theoretical solutions need to be established for each frequency of the Fourier decomposition of the seismic input. In the past, this would represent a formidable task and, in practice, fewer than the total number of Fourier transform frequencies were analysed explicitly (typically 50–200 frequencies). Interpolation schemes have been developed and implemented to fill in the functions at missing frequencies. With the advent of high performance computing (HPC), this has become less of an issue.

The main steps of this implementation are described by Kausel and Roësset [8].

Once the solution is determined for a set of discrete frequencies and interpolated for the missing frequencies, an inverse Fourier transform provides the time domain solution. As superposition is involved in the frequency domain solution, the substructure methods are limited to linear systems, even though nonlinearities can partially be accounted for in the soil by using strain compatible properties.

### **8.3.3. The flexible volume method**

The flexible volume sub structuring method [227] is based on the concept of partitioning the total soil–structure system into three substructure systems. Substructure I consists of the free field site, substructure II consists of the excavated soil volume, and substructure III consists of the nuclear installation structure, of which the foundation replaces the excavated soil volume. The substructures I, II and III, when combined together, form the original SSI system. The flexible volume method presumes that the free field site and the excavated soil volume interact both at the boundary of the excavated soil volume and within its body, in addition to interaction between the substructures at the boundary of the foundation of the structure. The SASSI program implements the flexible volume method denoting it the ‘direct method’ (See Section 8.3.5 for additional information on SASSI).

### **8.3.4. The subtraction method**

The substructure subtraction method [228] is based on the same substructuring concept as the flexible volume method. The subtraction method partitions the total soil–structure system into three substructure systems as shown in Figure 27. Substructure I consist of the free field site, substructure II consists of the excavated soil volume, and substructure III consists of the nuclear

installation structure. The substructures I, II and III, when combined together, form the original SSI system. However, the subtraction method recognizes, as opposed to the flexible volume method, that SSI occurs only at the common boundary of the substructures, that is, at the boundary of the foundation of the structure. This often leads to a smaller impedance analysis than the flexible volume method.

The subtraction method in SASSI is an approximate method to the flexible volume method (direct method). Frequently, the combination of soil properties and geometry of the excavated soil have led to amplified frequencies associated with the excavated soil mass. To alleviate this anomaly, the subtraction method is modified by adding interaction nodes to the excavated soil, further restraining the free body frequencies so that any spurious results occur at frequencies above the maximum frequency of interest in the SSI problem being solved. The subtraction method with these modifications is called the extended (modified) subtraction method. When implementing any of the subtraction methods, a validation that no spurious results have been introduced is necessary [19].

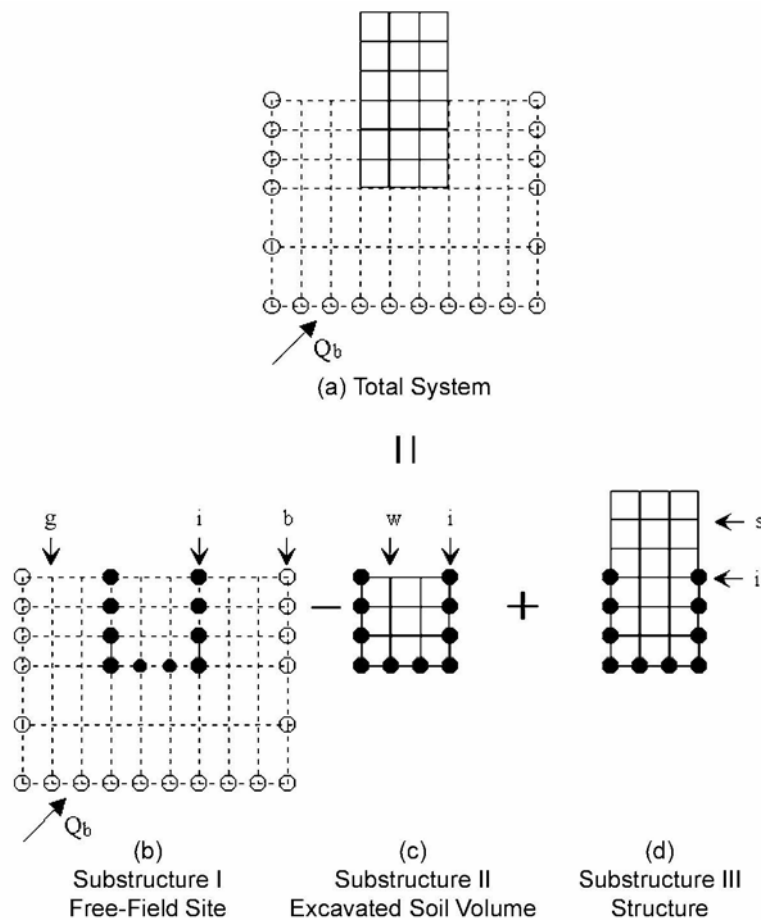


FIG. 27. Sub structuring in the subtraction method.

### 8.3.5. SASSI: System for analysis of soil–structure interaction

SASSI evaluates the dynamic response of 2-D and 3-D foundation–structure systems. SASSI is formulated using the flexible boundary method and uses linear finite element modelling and frequency domain methods. The soil is modelled as a uniform or horizontally layered, elastic or viscoelastic medium overlying a uniform half-space. The soil material model is based on complex moduli, which produces frequency-independent hysteresis damping. The structures

are modelled by 2-D or 3-D finite elements interconnected at node points. Seismic input motion is defined by acceleration time series and may be assumed to comprise vertically incident or inclined body waves or surface waves. These methods are formulated in the frequency domain.

SASSI may be used to analyse foundation–structure systems, including the flexibility of the foundation. Generally, horizontal and vertical models are analysed independently, and the results combined after the SSI analyses.

Limitations are primarily resource based, i.e. the lack of ability to analyse very detailed structural models in a timely manner and the difficulty in easily performing sensitivity studies. With the advent of HPC, this has become less of an issue.

Key elements of the SASSI approach are:

- (a) The site is modelled as semi-infinite elastic or viscoelastic horizontal layers on a rigid base or semi-infinite elastic or viscoelastic half-space.
- (b) The structures are idealized by standard 2-D or 3-D finite elements. Each nodal point may have up to six degrees of freedom.
- (c) The excavated soil zone is idealized by standard plane strain or 3-D solid elements. The finite element models of the structure and excavated soil have common nodes at the boundary.
- (d) The flexible volume method (direct method) is used extensively. Interaction between the excavated soil and semi-infinite site occurs at all excavated soil nodes in the flexible volume method.
- (e) All the interaction nodes lie on the soil layer interfaces with translational degrees of freedom.
- (f) Material damping is introduced by the use of complex moduli, which leads to effective damping ratios that are frequency independent and may vary from element to element.
- (g) The seismic environment may consist of an arbitrary 3-D superposition of the inclined body and surface waves;
- (h) The earthquake excitation is defined by time histories of acceleration. The input motion may also be specified with acceleration response spectrum using the RVT. The effects of incoherence of ground motion can be modelled with several versions of SASSI.
- (i) The control motion is applied at the control point, which may be defined on the soil free surface or at a point within the soil column;
- (j) For time series analysis, the fast Fourier transform technique is used.

The advantages of the SASSI approach are: (i) the ability to model complex foundation geometry, foundation embedment, and foundation flexibility; and (ii) the ability to calculate soil–foundation interaction parameters, such as soil pressures on embedded foundations.

The limitations of the SASSI approach are: (i) the lack of ability to analyse very detailed structural models in a timely manner (HPC is making this less of a limitation); (ii) the inability to easily perform sensitivity studies; (iii) need to run three directions of analyses separately and combine the results outside of SASSI; and (iv) only linear elastic material can be considered.

### **8.3.6. CLASSI: Soil–structure interaction - A linear continuum mechanic approach**

Key elements of the CLASSI approach [229] are:

- (a) The site is modelled as semi-infinite viscoelastic horizontal layers overlying a semi-infinite viscoelastic half space;
- (b) Complex-valued, frequency-dependent Green's functions for horizontal and vertical point loads are used in the generation of the foundation input motion (scattering functions) and the foundation impedances;

- (c) In the standard version of CLASSI, Green's functions are generated from continuum mechanics principles. In the hybrid method, the advantages of CLASSI and SASSI are combined to generate scattering functions and foundation impedances. The Green's functions are integrated over the discretized foundation subregion areas to calculate a resultant set of forces and displacements at subregion centroids;
- (d) In both cases, the constraints of rigid body motion are applied yielding the impedance and scattering matrices. In the hybrid method, any soil profile modelled in SASSI can be treated in CLASSI–SASSI;
- (e) The results consist of complex-valued, frequency-dependent scattering functions and impedances developed at a defined foundation reference point;
- (f) The structures are idealized by simple or very detailed 3-D finite element models. The dynamic characteristics of the structure are represented by the fixed-base eigen system. For the SSI analysis, these dynamic properties are projected onto the foundation, i.e. the fixed-base dynamic characteristics of the structure are represented exactly by its mode shapes, frequencies and modal damping values projected onto the foundation. An example of a detailed 3-D finite element structure model as analysed by CLASSI is shown in Figure 28. Key parameters of the model are: number of nodes = 70,366; number of degrees-of-freedom = 422,196; and number of fixed-base modes included in the SSI analyses = 3004 (frequencies ranging from 4.33 Hz to 61.05 Hz) [232];
- (g) Multiple structures on a single foundation may be modelled;
- (h) Material damping is introduced by the use of complex moduli in the soil and modal damping in the nuclear installation structure;
- (i) The seismic environment may consist of an arbitrary 3-D superposition of the inclined body and surface waves. Also, a version of CLASSI treats incoherent ground motion;
- (j) The earthquake excitation is defined by time histories of free field accelerations, called control motion. The control motion is applied at the control point which may be defined on the soil free surface or at a point within the soil column;
- (k) The FFT technique is used;
- (l) The solution of the complete SSI problem is performed in two stages: (i) the SSI response of the foundation is calculated including the effects of the structure (fixed-base modes), foundation (mass), and supporting soil (impedance functions) when subjected to the foundation input motion; and (ii) the dynamic response of the structure degrees of freedom, when subjected to the foundation SSI response, are calculated.

Figure 25 shows the steps in the CLASSI methodology.

The advantages of the CLASSI approach are: (i) the ability to include very detailed models of the nuclear installation structure in the SSI analyses and represent the structure by its fixed-base modes, including modal damping; (ii) the ability to perform 3-D analyses simultaneously; (iii) the ability to analyse the same SSI model for fixed-base conditions, coherent ground motion, and incoherent ground motion; (iv) the ability to efficiently perform sensitivity studies; (v) the ability to efficiently perform probabilistic seismic response studies; and (vi) the approach is extremely computationally efficient.

The limitations of the CLASSI approach are: (i) the inability to analyse flexible foundations; (ii) the inability to calculate soil–foundation interaction parameters, including the effects of flexible foundation and below grade embedded walls, such as soil pressures; and (iii) only linear elastic materials can be considered.

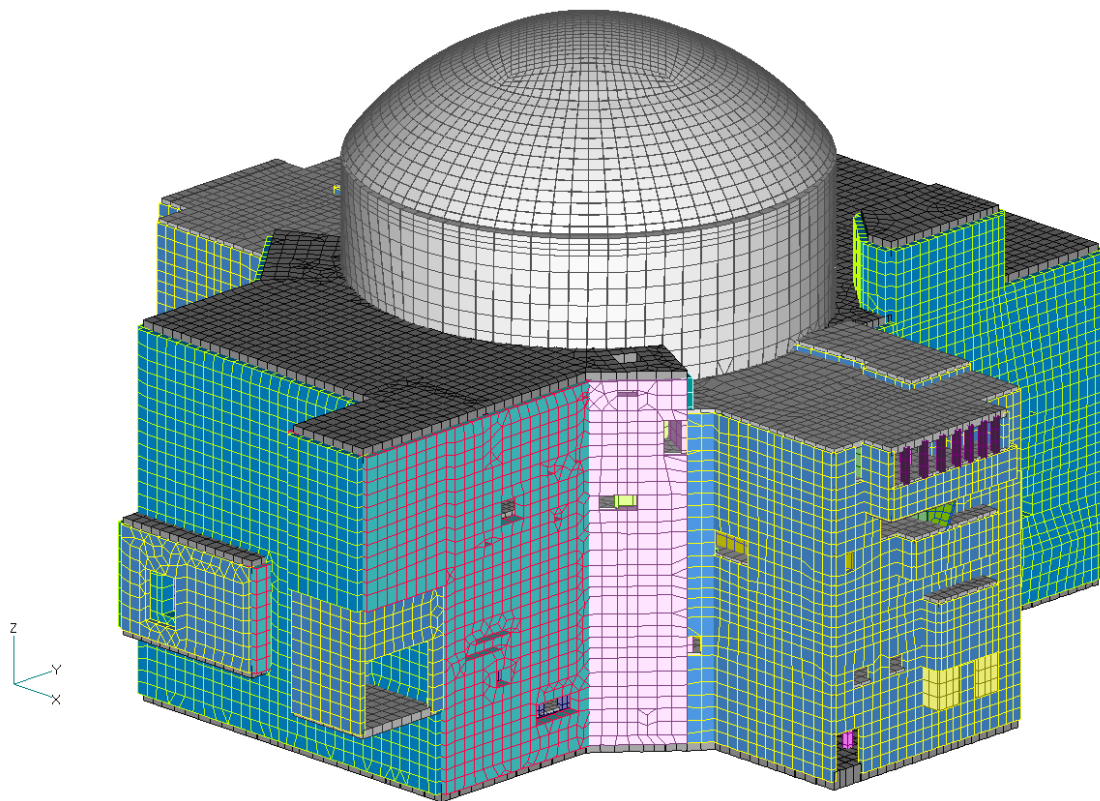


FIG. 28. AREVA Finite Element Model of the Evolutionary Power Reactor Nuclear Island (reproduced from Ref. [232] with permission courtesy of [ASME]).

#### 8.4. SOIL–STRUCTURE INTERACTION COMPUTATIONAL MODELS

Soil–structure interaction computational models are developed with a focus on three components of the problem:

- (a) Earthquake input motions, encompassing development of 1-D, 2-D or 3D motions, and their effective input in the SSI model;
- (b) Soil/rock adjacent to structural foundations, with important geological (deep) and site (shallow) conditions near the structure, contact zone between foundations and the soil/rock;
- (c) The nuclear installation structure, including structural foundations, embedded walls, and the superstructure.

It is advisable to develop models that will provide enough detail and accuracy to be able to address all the important issues. For example, for modelling higher frequencies of earthquake motions, the analyst needs to develop models that will be capable of propagating those frequencies and of documenting influence of numerical/mesh induced dissipation/damping of frequencies. Further information on effective modelling of all three components is provided in Section 8.1.

##### 8.4.1. Soil/rock linear and nonlinear modelling

Soil and rock adjacent to structural foundations can be either dry or fully (or partially) saturated [190, 233], and should be treated as follows:

- (a) Dry Soil: In the case of dry soil, without pore fluid pressures, it is appropriate to use models that are only dependent on single phase stress, that is, a stress that is obtained from applying all the loads (static and/or dynamic) without any consideration of pore fluid pressures.

- (b) **Unsaturated Soil:** For partially saturated soil, the effective stress principle needs to be also included the influence of gas (air) present in pore of soils. There are a number of different methods [191, 233]; however, computational frameworks that incorporate those methods are not yet well developed. The main approaches to modelling of soil behaviour within a partially saturated zone of soil (a zone where water rises due to capillary effects) are dependent on two main types of partial saturation of soil:
- (i) Voids fully saturated with fluid mixed with air bubbles. The water in pores is fully connected and can move and any pressure in the mixture of water and air can propagate, with reduced bulk stiffness of the water–air mixture. This type of partial saturation can be modelled using fully saturated approaches (see Section 8.4.11). The bulk modulus of the fluid–air mixture is (much) lower than that of fluid alone, and this needs to be tested. An example of the variation of the bulk modulus with the degree of saturation is shown in Figure 29. Therefore, only methods that assume fluid to be compressible are to be used ( $u - p - U, u - U$ ). In addition, permeability will be different from a case of just fluid seeping through the soil, and additional testing for permeability of the water–air mixture is warranted. Also, because this partial saturation is usually found above the water table (due to capillary rise), the hydrostatic pore pressure can be negative (i.e. suction).
  - (ii) Voids of soil are full of air, with water covering the thin contact zones between particles, creating water menisci, and contributing to the apparent cohesion of cohesionless soil material (e.g. in the case of wet sand at the beach, there is an apparent cohesion, until the sand dries). This type of partial saturation can be analysed using dry (unsaturated) modelling, where elastic-plastic material models used are extended to include additional cohesion that arises from thin water menisci connecting soil particles.
- (c) **Saturated Soil:** In the case of full saturation, the effective stress principle [234] needs to be applied. This is essential as for porous material (soil, rock, and sometimes concrete) mechanical behaviour is controlled by the effective stresses. Effective stress is obtained from the total stress acting on material ( $\sigma_{ij}$ ), with reductions due to the pore fluid pressure, as follows:

$$\sigma' = \sigma - p\delta \quad (21)$$

Where  $\delta$  is the Kronecker symbol and  $p (>0)$  is the pore fluid pressure. All the mechanical behaviour of soils and rock is a function of the effective stress  $\sigma'$ , which is affected by a full coupling with the pore fluid, through a pore fluid pressure  $p$ .

Clay particles (platelets) are so small that their interaction with water is quite different from silt, sand and gravel. Clays feature a chemically bonded water layer that surrounds clay platelets. Such water does not move freely, and stays connected to clay platelets under working loads.

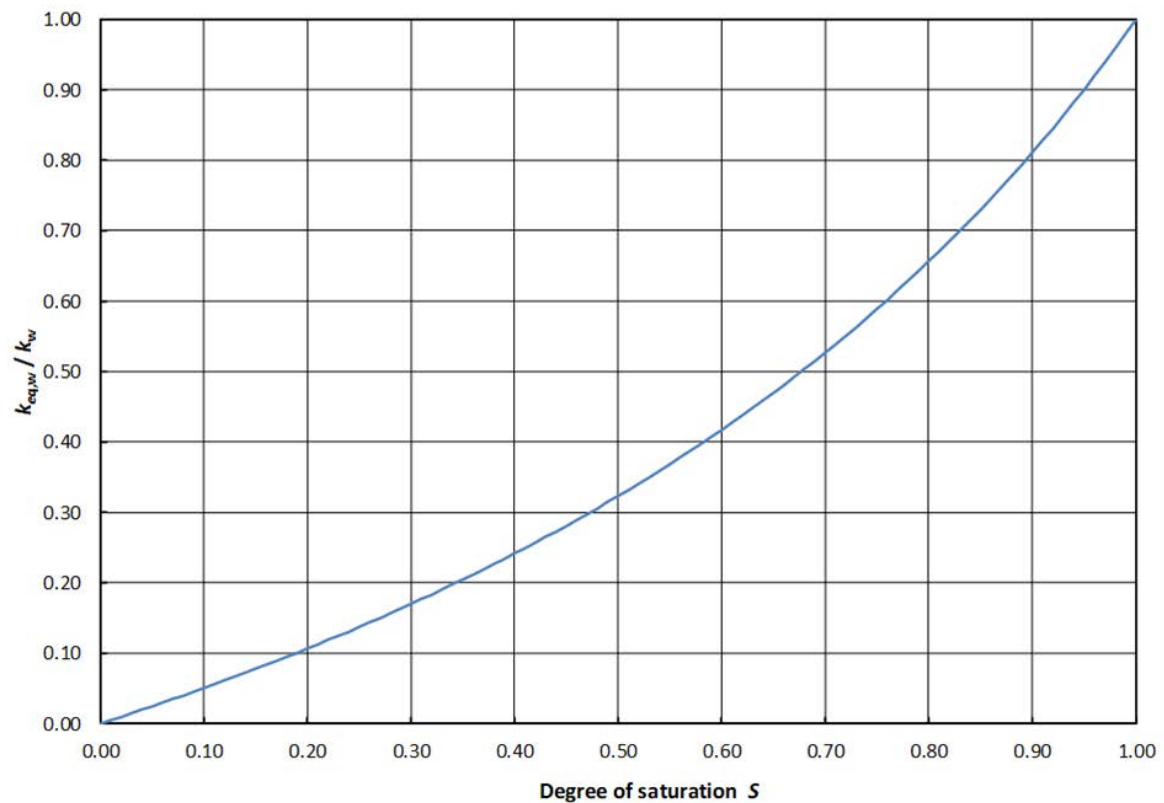


FIG. 29. Example of variation of the equivalent bulk modulus of fluid divided by the saturated fluid bulk modulus versus degree of saturation.

Usually, clays are modelled as fully saturated soil material. In addition, clays feature very small permeability, so that, while the effective stress principle applies, pore fluid pressure does not change during fast (earthquake) loading. Hence clays are analysed using total stress analysis, where the initial total stress is a stress that is obtained from an effective stress calculation that takes into account hydrostatic pore fluid pressure. In other words, clays are modelled using undrained, total stress analysis, using effective stress (total stress reduced by the pore fluid pressure) for initializing total stress at the beginning of loading.

#### 8.4.2. Drained and undrained modelling

Depending on the permeability of the soil, on relative rate of loading and seepage, and on boundary conditions [235], a decision needs to be made if analysis will be performed using drained or undrained behaviour. The permeability of soil ( $k$ ) can range from  $k > 10^{-2}$  m/s for gravel,  $10^{-2}$  m/s  $> k > 10^{-5}$  m/s for sand,  $10^{-5}$  m/s  $> k > 10^{-8}$  m/s for silt, to  $k < 10^{-8}$  m/s for clay and silty sands. If we assume a unit hydraulic gradient (reduction of pore fluid pressure/head of 1m over the seepage path length of 1m), then for a dynamic loading of 10–30 seconds (earthquake), and for a semi-permeable silt with  $k = 10^{-6}$  m/s, water can travel a few millimetres. However, pore fluid pressure will propagate (much) faster (further) and will affect the mechanical behaviour of the soil skeleton. This is due to the high bulk modulus of water ( $K_w = 2.25 \times 10^6$  kN/m<sup>2</sup>), which results in high speed pressure waves in saturated soils. Thus, a simple rule is that for earthquake loading, for gravel, sand and permeable silt, the relative rate of loading and seepage needs to be determined by the use of drained analysis. For clays, and impermeable silt (and silty sands), it might be appropriate to use (locally) undrained analysis for such short loading. Of course, if permeable layers are positioned between impermeable layers (clay or silt), then appropriate modelling for permeable and impermeable layers is needed to provide accurate results.

#### 8.4.2.1. Drained analysis

Drained analysis is performed when the permeability of soil, the rate of loading and seepage, and the boundary conditions allow for full movement of pore fluid and pore fluid pressures during loading events. As noted above, the use of the effective stress ( $\sigma'_{ij}$ ) for the analysis is essential, as is modelling of full coupling of pore fluid pressure with the mechanical behaviour of the soil skeleton. This is usually done using the theory of mixtures [189, 191, 236-238]. During loading events, pore fluid pressures will dynamically change (pore fluid and pore fluid pressures will displace) and will affect the soil skeleton, through the effective stress principle. All nonlinear (inelastic) material modelling applies to the effective stresses ( $\sigma'_{ij}$ ). Appropriate inelastic material models that are used for modelling of soil need to be used.

#### 8.4.2.2. Undrained analysis

Undrained analysis is performed when the permeability of soil, the rate of loading and seepage, and the boundary conditions do not allow movement of pore fluid and pore fluid pressures during the loading event. This is usually the case for clays and for low permeability silt. There are three main approaches to undrained analysis:

- (a) The total stress approach, where there is no generation of excess pore fluid pressure (pore fluid pressure in addition to the hydraulic pressure), and soil is practically impermeable (clays and low permeability silt). In this case, hydrostatic pore fluid (water) pressures are calculated prior to analysis, and effective stress is established for the soil. This approach assumes no change in pore fluid pressure. This usually happens for clays and low permeability silt, and due to very low permeability of such soils, a total stress analysis is warranted, using initial stress that is calculate based on an effective stress principle and known hydrostatic pore fluid pressure. Since pore fluid pressure does not affect shear strength [170], for very low permeability soils (impermeable for all practical purposes), it is convenient to perform elastic-plastic analysis using undrained shear strength ( $c_u$ ) within a total strain setup. Since only shear strength is used, and all the change in mean stress is taken by the pore fluid, material models using von Mises yield criteria can be used.
- (b) The locally undrained analysis approach, where excess pore fluid pressure (change from hydrostatic pore pressure) can be created. Excess pore fluid pressures can be created, due to compression effects on low permeability soil (usually silt). In contrast, pore fluid suction can also be created due to dilatancy effects within granular material (silt). Due to very low permeability, pore fluid and pore fluid pressure does not move during loading, and therefore pore fluid pressure increase or decrease at one location will not affect nearby locations. However, pore fluid pressure change (increase or decrease) will affect effective stress. Effective stress will change and will affect the constitutive behaviour of soil. The analysis is essentially undrained, however, pore fluid pressure can and will change locally due to compression or dilatancy effects in granular soil. Appropriate inelastic (elastic-plastic) material models that are used for modelling of soil (see Section 8.4.2) can be used, while the constitutive integration needs to take into account local undrained effects and convert any change in voids into excess pore fluid pressure change (excess pore pressure). This is still a single phase analysis, as all the pore fluid pressure changes are taken into account on a local, constitutive level, and there is no two phase material (pore fluid and porous solid) where pore fluid pressure is able to propagate.
- (c) Very low permeability soils that can, but do not have to develop excess pore fluid pressure due to constitutive level volume change of soil can also be analysed as fully drained continuum, while using very low, realistic permeability. In this case, although analysis is officially drained analysis, results will be very similar if not the same as for undrained behaviour, due to use of very low, realistic permeability. Effective stress analysis is used, with explicit modelling of pore fluid pressure and a potential for pore fluid to displace and



pore fluid pressure to move. However, due to very low permeability, and fast application of the load (earthquake), no fluid will displace, and no pore fluid pressure will propagate. This approach can be used for the total stress approach and the locally undrained approach. While this approach is actually explicitly allowing for modelling of pore fluid movement, the results for pore fluid displacement need to show no movement. As such, this approach is modelling more variables than needed, as some results are known before simulations (there will be no movement of water nor pore fluid pressure). However, this approach can be used to verify modelling using the first two undrained approaches, as it is more general.

Globally undrained problems, where for example soil is permeable, but boundary conditions prevent water from moving, need to be treated as drained problems, while appropriate boundary conditions prevent water from moving across impermeable boundaries.

### 8.4.3. Soil material modelling: linear and nonlinear elastic models

Material modelling for soil can be equivalent linear, nonlinear or elastic-plastic, as follows:

- (a) Equivalent linear models are in fact linear elastic models with adjusted elastic stiffness that represents a certain percentage of a secant stiffness of largest shear strain reached for given motions. Determining such linear elastic stiffness requires an iterative process (trial and error). This modelling approach is fairly simple, there is significant experience in professional practice, and it works well for 1-D analysis and for 1-D states of stress and strain. Features of this modelling approach are that it does not take into account soil volume change (hence it favours total stress analysis, see Section 8.4.2), and it is useable for 3-D analysis through an invariant deviatoric stress. Secant stiffness 1-D models provide the relationship between shear stress ( $\tau = \sigma_{xz}$ ) and shear strain ( $\gamma = 2\epsilon_{xz}$ ). Determination of secant shear stiffness is done iteratively, by performing 1-D wave propagation simulations, and recording the average high estimated strain (65% of maximum strain) for each level/depth. Such representative shear strain is then used to determine the reduction of stiffness using modulus reduction curves ( $G/G_{max}$ ) and the analysis is re-run. Stable secant stiffness values are usually reached after few iterations, typically 5–8. Equivalent elastic modelling is still essentially linear elastic modelling, with changed stiffness. More details are available in Section 4.2.
- (b) Nonlinear material models are used to represent 1-D stress strain response (usually in shear,  $\tau - \gamma$ ) using nonlinear functions. There are a number of nonlinear elastic models used [54, 239], as well as hyperbolic models [86], and other models. Calibrating modulus reduction and damping curves using nonlinear models is not too demanding [239]; however, there is less experience in professional practice with these types of models (see Section 4.3.2). Potential issues with this modelling approach are the same as for equivalent linear modelling, e.g. it does not take into account soil volume change as it is essentially based on a nonlinear elastic model.
- (c) Elastic-plastic material models are usually full 3-D models that can be used for 1-D or 3-D analysis. A number of models are available [41, 77, 240–242]. The use of full 3-D material models, if properly calibrated, can work well in 1-D as well as in 3-D. The main issue with these models is that calibration usually involves a number of in situ and laboratory tests. In addition, there is far less experience in professional practice with elastic-plastic modelling.

Linear and nonlinear elastic models are used for soil, rock and structural elements. Linear elastic model that are used are usually isotropic, and are controlled by two constants, the Young's modulus  $E$  and the Poisson's ratio  $\nu$ , or alternatively by the shear modulus  $G$  and the bulk modulus  $K$ .

Nonlinear elastic models are used mostly in geotechnical engineering. There are a number of models proposed over years, tend to produce initial stiffness of a soil for given confinement of over-consolidation ratio [199, 201–204].

Anisotropic material models are mostly used for modelling of usually anisotropic rock material [243, 244].

#### *8.4.3.1. Elastic-plastic models*

Elastic plastic modelling can be used in 1-D, 2-D and full 3-D analyses. A number of material models have been developed over years for both monotonic and cyclic modelling of materials. Material models for soil [205–210, 245] and rock [200, 246, 247] have been developed over last decades.

It needs to be noted that 3-D elastic-plastic modelling is the most general approach to material modelling of soils and rock. Elastic-plastic models can model simplified and detailed behaviour. However, calibration of models that can achieve such modelling sophistication requires expertise. The benefit is that important material response effects that are usually neglected if simplified models are used can be taken into account and properly modelled. As an example, soil volume change during shearing is a first order effect; however, it is not taken into account if modulus reduction curves are used.

#### **8.4.4. Structural models, linear and nonlinear: shells, plates, walls, beams, trusses, solids**

Nuclear power plants are expected to remain essentially linear during DBE ground motion. However, nonlinear structural models may be used for capacity assessments and designs of nuclear installations other than NPPs.

Significant work has been done in modelling of nonlinear effects in reinforced concrete elements [248–266].

#### **8.4.5. Contact modelling**

In all soil–structure systems, there are interfaces between the structural elements (foundations, embedded walls) of a nuclear installation and the adjacent soil and rock. There are two main modes of behaviour of these interfaces:

- (a) Normal contact where the structural elements and the adjacent soil/rock interact in a normal stress mode. This mode of interaction comprises normal compressive stress; however, it can also comprise gap opening, as it is assumed that the contact zone has zero tensile strength.
- (b) Shear and/or tangential contact where the structural elements and the adjacent soil/rock can develop frictional slip.

Modelling of contact is done using contact finite elements. The simplest contact elements are based on two node elements, the so-called joint elements which were initially developed for modelling of rock joints. Typically, normal and tangential stiffness were used to model the pressure and friction at the interfaces [44, 193, 267–268]. In addition to node to node contact elements, node to surface and surface to surface contact elements are also available (Dynaflow).

#### **8.4.6. Structures with a base isolation/dissipation system**

Base isolation systems have been used for NPPs (e.g. Cruas NPP France, Koeberg NPP South Africa). The behaviour of base isolation/dissipation systems is affected by SSI.

Base isolation systems are used to change the dynamic characteristics of seismic motions that excite structures, and also to dissipate seismic energy before it excites structures. Therefore, there are two main types of device:

- (a) Base isolators [269, 270] are usually made of low damping (energy dissipation) elastomers and are primarily meant to reduce the frequencies of motions that are transferred to the structural system. These types of isolators can also be represented by simple helical springs. They are not designed nor modelled as energy dissipators.
- (b) Base dissipators [271, 272] are developed to dissipate seismic energy before it excites structures. There two main types:
  - (i) Elastomers made of high dissipation rubber;
  - (ii) Frictional pendulum dissipators.

Both isolators and dissipators are usually developed to work in two horizontal dimensions, while motions in the vertical direction are not isolated or dissipated. This can create potential problems, and need to be carefully modelled [273, 274].

Base dissipator systems are modelled using inelastic (nonlinear) two node elements. There are three basic types of dissipator models used:

- (a) High damping rubber dissipators;
- (b) Rubber dissipators with a lead core;
- (c) Frictional pendulum (double or triple) dissipators.

#### **8.4.7. Foundation models**

Foundation modelling can be done using a variable level of detail. Early models for slabs and footings assumed rigid behaviour. This was dictated by the use of modelling methods that rely on analytic solutions, which in turn rely on simplifying assumptions in order to be solved. Soil and rock beneath and adjacent to foundations was usually assumed to be an elastic half space.

Foundation response plays an important role in the overall SSI response. Major energy dissipation occurs in soil and in the contact zone adjacent to the foundation. Buoyant forces (pressures) act on the foundation if the water table is above the lowest foundation level.

Foundations can be classified by the depth of embedment compared to plan dimensions. An embedment ratio is often used to classify embedment depth, and is defined as the embedment depth/equivalent radius (i.e.  $e/r$ ) or equivalent square side. Foundations then are designated as: surface,  $e/r < 0.3$ ; shallow embedment,  $e/r < 1.0$ ; and deep embedment,  $e/r > 1.0$  [19].

##### *8.4.7.1. Shallow and embedded foundation slabs and walls.*

Surface foundations have an embedment ratio (depth to width)  $\leq 0.3$ . Such foundations can be modelled as flexible or rigid. Their thickness can range from 3–5 metre; however, their horizontal extent can be up to 100 metres. Containment and shear wall structures, for which structural elements are connected to the foundation, significantly increase the effective stiffness of the foundation approaching rigid behaviour. For the purposes of calculating the overall response of the soil–structure system, the assumption of rigid foundation behaviour is often reasonable and justifiable. If the rigid foundation assumption is introduced, more detailed seismic responses, such as stresses in the foundation, will involve a second stage analysis with appropriate levels of modelling to design or evaluate structure capacity.

Flexible modelling of foundation slabs is best done using either shell elements (plate bending and membrane behaviour) or solids. For shell element models, it is important to bridge over

half slab or wall thickness to the adjacent soil. This is important as shell elements are geometrically representing a plane in the middle of a solid (slab or wall) with a finite thickness, so connection over half the thickness of the slab or wall is needed. It is best if shell elements with drilling degrees of freedom (out-of-plane moment) are used [275, 276], as they properly take into account all degrees of freedom (three translations, two bending rotations and a drilling rotation).

For solid element models, it is important to use the proper number of solids so that they properly represent the bending stiffness. For example, a single layer of regular 8 node bricks will overpredict the bending stiffness by over 200%. Hence, at least 4 layers of 8 node bricks are needed for proper bending stiffness. If 27 node brick elements are used, a single layer produces a predicted bending stiffness within 4% of the analytic solution.

#### 8.4.7.2. Deep foundations (piles, caissons and shaft foundations).

Deep foundations are used for nuclear installations built on compressible soils. Piles carry axial loads at the bottom end and by skin friction. Piles carry horizontal loads by soil compression and friction. Additional information of soil settlement on piles and pile group behaviour is provided in Section 9.

Piles (including pile groups) and shafts have been modelled using three main approaches:

- (a) The analytic approach [277-279], in which a main assumption is that of a linear elastic behaviour of a pile and the soil represented by a half space. The contact zone is fully connected and slip, or gap is not modelled.
- (b) P-y and t-z approaches based on the experimentally measured response of piles when subjected to loads in the lateral direction (p-y) and vertical direction (t-z). The results are used to construct linear and nonlinear springs that are then used to replace soil [280–284]. This approach is very popular with practicing engineers. However, this approach is based on site specific tests or on published results when properly correlated with site properties and pile characteristics. Moreover, in dynamic applications, the dynamics of soils surrounding piles and pile groups are poorly approximated using springs, even with additional dashpots.
- (c) Nonlinear 3-D FEM models have been developed for treatment of piles, pile groups and shafts, in both dry and liquefiable soils [285–289]. In these models, the elastic-plastic behaviour of soil is taken into account, as well as modelling the inelastic contact zone between pile and soil. Layered soils are easily modelled, while proper modelling of contact (see Section 8.4.6) resolves both horizontal and vertical shear (slip) behaviour.

In a numerical modelling of pile foundations several options are possible for the pile element:

- (a) The pile is represented by a flexural-shear beam element. This type of modelling is very convenient to retrieve the pile internal forces (bending moment, shear force) and minimizes the number of degree of freedoms attached to the pile element (6 per node). However, since the pile element has no thickness, wave diffraction by the pile is poorly represented and the pile-soil-pile interaction is only approximately considered, which may be a significant drawback for closely spaced piles. Nevertheless, for large piles group this modelling technique remains the only viable solution.
- (b) The pile element is modelled with solid elements (for solid piles). The main advantages are that the exact geometry of the pile is modelled, wave diffraction is correctly considered, and the pile-soil-pile interaction is taken into account. Significant disadvantages are the need to have a sufficient number of elements in the cross-section to properly model the bending behaviour of the pile (see Section 8.4.8), which increases the complexity of the model and the number of degrees of freedom. However, the most important drawback stems from the

difficulty in having direct access to the pile internal forces (stresses need to be integrated over the cross-section), which involves additional work.

- (c) The third possibility takes advantage of the simplicity of the first technique while tentatively preserving the rigour of the second one. The pile is modelled with a flexural-shear beam element located at the position of the central axis of the pile and connected to the soil nodes located along the periphery of the piles with radial rigid bars. At each elevation, constraints are applied to the degrees of freedom of the soil nodes along the periphery to ensure that the pile-cross section remains planar. The internal forces are directly retrieved from the beam element and the interaction between the pile and the soil takes place at the periphery of the pile. This modelling technique is important for closely spaced piles but remains difficult to implement for large pile groups.

#### *8.4.7.3. Deeply embedded foundations.*

Deeply embedded foundations have an embedment ratio greater than 0.15. The SSI for deeply embedded foundations at a nuclear installation is significantly affected by the contribution of the embedded walls, in addition to the base slab. The main issues are related to proper modelling of contact (see Sections 8.4.9), as well inelastic behaviour of soil adjacent to the slab and walls. Of particular importance for deeply embedded foundations is proper modelling of buoyant stresses (forces) as it is likely that ground water table will be above the base slab.

#### *8.4.7.4. Foundation flexibility and base isolator/dissipator systems.*

There are special cases of foundations where base isolators and dissipators are used. In this case there are two layers of foundations slabs: one at the bottom, in contact with soil; and one above the isolators/dissipators, beneath the actual structure. These two base slabs are connected with dissipators/isolators. It is important to accurately model the stiffness of both slabs, as their relative stiffness will determine how effective the isolators and dissipators will be during earthquakes.

### **8.4.8. Deeply embedded structures**

A special case of deeply embedded foundations is a deeply embedded structure (DES). Deeply embedded structures involve special considerations: an example is a Small Modular Reactor (SMR) configuration where the embedment ratio is greater than 1. Modelling and analyses issues associated with DESs are discussed below:

- (a) Seismic motions: Seismic motions will be quite variable along the depth and in all three directions. This variability of motions is a result of the mechanics of seismic wave propagation, the inherent variability and the interaction of body waves (SH, SV and P) with the surface, and the development of surface waves [85]. This results in different seismic motion wave lengths (frequencies, depending on soil/rock stiffness), propagating in a different way at the surface and at depth of a deeply embedded foundation. As a result, a deeply embedded foundation will experience very different motions at the surface, at the base and in between. Due to a number of complex issues related to seismic motions variability, as noted above, it is needed that a full wave field be developed and applied to SSI models of DESs, as follows:
  - (i) In the case of 1-D wave propagation modelling, vertically propagating shear waves are to be developed (deconvolution and/or convolution) and applied to SSI models:
  - (ii) For 3-D wave fields:

- A full seismic wave field is developed from a wave propagation or from site response modelling and analyses (Sections 6 and 7);
  - Incoherence functions, if available, are used to modify seismic wave fields accounting for randomness in the motion. This option has a limitation as incoherent functions in the vertical direction are not well developed.
- (b) Nonlinear/inelastic soil and contact: The large contact zone of the concrete walls and foundation slab of a DES, with surrounding soil, with its nonlinear/inelastic behaviour will have significant effect on dynamic response of a DES. The use of an appropriate contact model that can model frictional contact as well as possible gap opening and closing (most likely in the near surface region) is appropriate. In the case of the presence of a water table above the DES foundation base, an effective stresses approach needs to be used, as well as modelling of (possibly dynamically changing) buoyant forces as described in Section 8.4.10 and below.
- (c) Nonlinear/Inelastic Soil Behaviour: With deep embedment, the dynamic behaviour of a DES is significantly influenced by the nonlinear/inelastic behaviour of soil adjacent to adjacent SMR walls and the foundation slab. Appropriate inelastic (elastic-plastic) 3-D soil models need to be used. Of particular importance is the proper modelling of soil behaviour in 3-D as well as proper modelling of volume change due to shearing (dilatancy). One dimensional equivalent elastic models, used for 1-D wave propagation are not suitable, as they do not properly model 3-D effects and lack modelling of volume change.
- (d) Buoyant forces: With deep embedment, and (a possible) presence of underground water (i.e. a water table that is within depth of embedment), water pressure on the walls of a DES will create buoyant forces. During earthquake shaking, those forces will change dynamically due to water pumping during shaking [290]. Modelling of buoyant forces can be done using two approaches, namely static and dynamic buoyant force modelling, as described in Section 8.4.10.
- (e) Uncertainty in motions and material: Due to large contact area and significant embedment, significant uncertainty and variability (incoherence) in seismic motions will be present. Moreover, uncertainties in properties of soil material surrounding a DES will add to the uncertainty of the response. Uncertainties in seismic motions, soil configuration, and soil and material parameters can be modelled using two approaches, as described in Section 8.5. One approach is to rely on varying input motions and material parameters using Monte Carlo approach, and its variants (Latin hypercube, etc.). This approach is very computationally demanding. Another approach is to use analytic stochastic solutions for components of the full problem. For example, a stochastic FEM, with extension to stochastic elasto-plasticity with random loading. More details are given in Section 8.5.

Figure 30 illustrates modelling issues for a simple, generic DES (an SMR) FEM model (a vertical cut through the middle of a full model is shown).

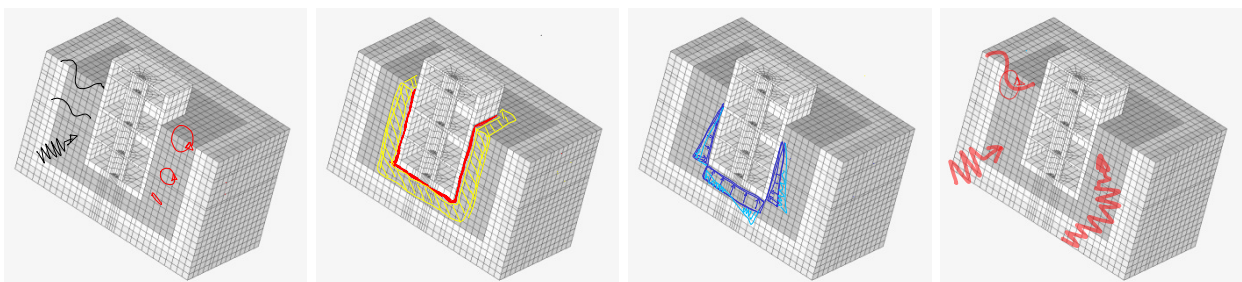


FIG. 30. Four main issues for realistic modelling of ESSI of a deeply embedded SMR: variable wave field at depth and surface, inelastic behaviour of contact and adjacent soil, dynamic buoyant forces, and uncertain seismic motions and material.

It is important to develop models with enough fidelity to address the above issues. It is possible that some of these issues will not influence results in any significant way, however the only way to determine importance (influence) of the above phenomena on seismic response of a deeply embedded SMR is through modelling.

#### **8.4.9. Buoyancy modelling**

For nuclear installation structures for which the foundation level is below the water table, there exist a buoyant pressure/forces on foundation base and all walls, creating symmetric buoyant forces. In addition, for some NPP structures, like intake structures, there exist different pore fluid pressures in the soil on different sides, creating a nonsymmetric buoyant forces. For static loads, the symmetric buoyant force  $B$  can be calculated using Archimedes' principle. Buoyant force can be applied as a single force or a small number of resultant forces directed upward around the stiff centre of the foundation. For nonsymmetric buoyant forces, static forces (pore fluid pressures) on all walls and the base slab have to be taken into account separately.

During dynamic loading, buoyant force (buoyant pressures) can dynamically change, as a result of a dynamic change of pore fluid pressures in soil adjacent to the foundation concrete. This is particularly true for soils that are dense, where shearing will lead to an increase of inter-granular void space (dilatancy), a reduction in buoyant pressures, and creation of negative pressures (suction). For soils that are loose, shearing will lead to a reduction of inter-granular void space (compression) and an increase in buoyant pressures.

For strong shaking, it also expected that gaps will initiate between soil and foundation walls and even the foundation slab. This will lead to pore water being sucked into the opening gap and pumped back into soil when gap closes. Such 'pumping' of water will lead to large, dynamic changes of buoyant pressures.

Different dynamic scenarios create conditions for dynamic, nonlinear changes in buoyant forces.

For dynamic buoyant stress/force modelling, fully coupled finite elements (u-p or u-p-U or u-U, as described in Section 8.4.2) are used for modelling saturated soil adjacent to foundation walls and the base. Modelling of contact between soil and the foundation concrete needs to take into account effects of pore fluid pressure and the buoyant stress within the contact zone in order to properly model normal stress for frictional contact.

#### **8.4.10. Domain boundaries**

One of the biggest problems in dynamic ESSI in infinite media is related to the modelling of domain boundaries. Because of limited computational resources, the computational domain needs to be kept small enough so that it can be analysed in a reasonable amount of time. By limiting the domain however an artificial boundary is introduced. As an accurate representation of the soil–structure system, this boundary needs to absorb all (or at least, most) outgoing waves and reflect no waves back into the computational domain. If Green's functions are used for the outside domain, the absorbing boundary problem is resolved analytically.

The most commonly used types of domain boundaries are as follows:

(a) Fixed or free

By fixing all degrees of freedom on the domain boundaries, any radiation of energy away from the structure is made impossible. Waves are fully reflected, and resonance frequencies can appear that do not exist in reality. The same happens if the degrees of freedom on a

boundary are left 'free', as at the surface of the soil. A combination of free and fully fixed boundaries can be chosen only if the entire model is large enough and if material damping of the soil is used to reduce wave reflection and to allow for a sufficient time window to analyse the response of the structure. When compressional and/or shear waves travel very fast, boundaries have to be further away, thus significantly increasing the size of models.

(b) Absorbing Lysmer boundaries

A possible solution to eliminate waves propagating outward from the structure is to use Lysmer boundaries. This method is relatively easy to implement in a FEM model as it consists of simply connecting dashpots to all degrees of freedom of the boundary nodes and fixing them on the other end as shown in Figure 31. Lysmer boundaries are derived for an elastic wave propagation problem in a 1-D semi-infinite bar. It can be shown that in this case, an appropriately specified dashpot has the same dynamic properties as the bar extending to infinity [102]. The damping coefficient  $C$  of the dashpot equals

$$C = A\rho c \quad (22)$$

where  $A$  is the cross section area of the bar,  $\rho$  is the mass density and  $c$  the wave velocity that needs to be selected according to the type of wave that needs to be absorbed (shear wave velocity  $c_s$  or compressional wave velocity  $c_p$ ).

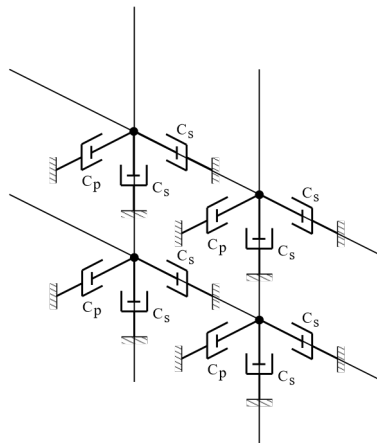


FIG. 31. Absorbing boundary consisting of dashpots connected to each degree of freedom of a boundary node.

In a 3-D or 2-D model, the angle of incidence of a wave reaching a boundary can vary from near  $0^\circ$  to near  $180^\circ$ . The Lysmer boundary is able to absorb completely only waves with an incidence angle of  $90^\circ$ . Even with this type of absorbing boundary a large number of reflected waves are still present in the domain. By increasing the size of the computational domain the angles of incidence on the boundary can be brought closer to  $90^\circ$  and the amount of energy reflected can be reduced.

- (c) Infinite elements can also be used [185]; however, their use does not guarantee full absorption of outgoing waves.
- (d) A 'perfectly matched layer' that was adopted from electromagnetic wave propagation modelling by Basu and Chopra [291] can be used for removing outgoing waves from the SSI domain.
- (e) The DRM [188] elegantly resolves the issue of outgoing waves, see Section 8.4.12.1.
- (f) More detailed boundaries can be used to model wave propagation toward infinity (boundary elements). However, the use of boundary elements for outside of FEM models destroys sparsity of the resulting stiffness matrix, and thus involves a high computational burden.



### 8.4.11. Seismic load input

Several methods are used to input seismic motions into FEM models. Most of these are based on simple intuitive approaches, rather than on rational mechanics.

Most of the widely used methods cannot properly model all three components of body waves as well as surface waves that are always present. The simplest method to input waves into the SSI model is to apply displacements or acceleration at nodes at the bottom of the SSI model. While these methods seem intuitive, it does trap waves in the SSI model; waves are not allowed to leave or radiate into half space. Other methods that are used for frequency domain modelling rely on 1C convolution or deconvolution of motions. For time domain modelling of SSI phenomena, the DRM [188] resolves many issues and provides probably the most elegant and efficient method to input seismic motions into SSI models.

#### 8.4.11.1. *Domain reduction method*

The DRM is based on rational mechanics and can model both body and surface seismic waves input into FEM models with high accuracy [188, 292]. It is a modular, two-step dynamic procedure aimed at reducing the large computational domain to a more manageable size. The method was developed with earthquake ground motions in mind, with the main idea to replace the force couples at the fault with their counterpart acting on a continuous surface surrounding the local feature of interest. The local feature can be any geologic or manmade object that constitutes a difference from the simplified large domain for which displacements and accelerations are easier to obtain.

The DRM is applicable to a wide range of problems. It is essentially a variant of the global–local set of methods and can be used for any problems where the local feature can be bounded by a continuous surface (which can be closed or not).

A large physical domain is to be analysed for dynamic behaviour. The source of disturbance is a known time history of a force field  $P_e(t)$ . The source of loading is far away from a local feature which is dynamically excited by  $P_e(t)$  (see Figure 32)

It would be beneficial not to analyse the complete system; only the behaviour of the local feature and its immediate surrounding is of interest, and the domain outside of some relatively close boundaries can be neglected. In order to do this, it is necessary to transfer the loading from the source to the immediate vicinity of the local feature. For example, the size of the domain can be reduced to a much smaller model bounded by surface  $\Gamma$  as shown in Figure 32. In doing so it needs to be ensured that the dynamic forces  $P_e(t)$  are appropriately propagated to the much smaller model boundaries  $\Gamma$ .

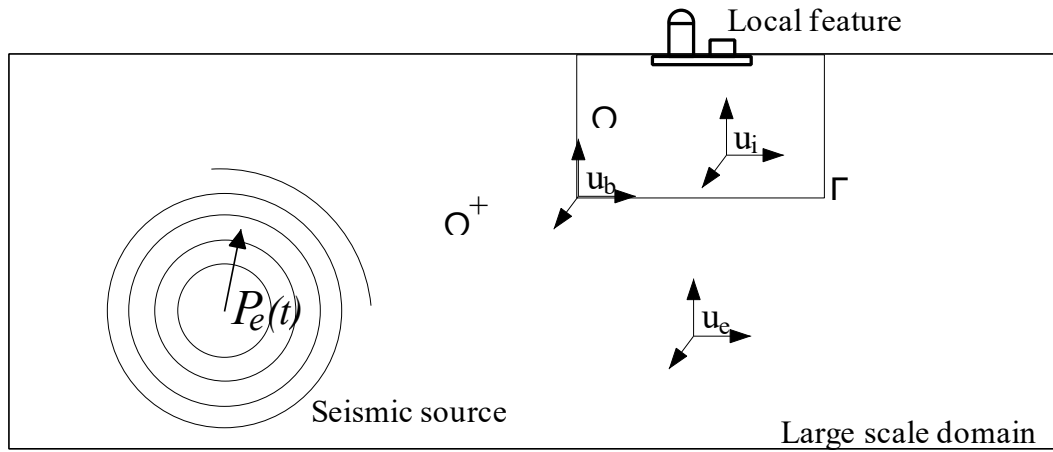


FIG. 32. Large physical domain with the source of load  $P_e(t)$  and the local feature (in this case a soil–structure system).

It can be shown [292] that the consistent dynamic replacement for the dynamic source forces  $P_e$  is a so-called effective force,  $P_e^{eff}$ :

$$P_e^{eff} = \begin{Bmatrix} P_i^{eff} \\ P_b^{eff} \\ P_e^{eff} \end{Bmatrix} = \begin{Bmatrix} 0 \\ -M_{be}^{\Omega^+} \ddot{u}_e^0 - K_{be}^{\Omega^+} u_e^0 \\ M_{eb}^{\Omega^+} \ddot{u}_b^0 + K_{eb}^{\Omega^+} u_b^0 \end{Bmatrix} \quad (23)$$

where  $M_{be}^{\Omega^+}$  and  $M_{eb}^{\Omega^+}$  are off-diagonal components of a mass matrix, connecting boundary (b) and external (e) nodes,  $K_{be}^{\Omega^+}$  and  $K_{eb}^{\Omega^+}$  are off-diagonal component of a stiffness matrix, connecting boundary (b) and external (e) nodes,  $\ddot{u}_e^0$  and  $\ddot{u}_b^0$  are free field accelerations of external (e) and boundary (b) nodes, respectively and,  $u_e^0$  and  $u_b^0$  are free field displacements of external (e) nodes, and boundary (b) nodes, respectively. The effective force  $P_e^{eff}$  consistently replace forces from the seismic source with a set of forces in a single layer of finite elements surrounding the SSI model. The DRM is quite powerful and has a number of features that makes an excellent choice for SSI modelling, as follows:

- A single layer of elements used for  $P_e^{eff}$ . Effective nodal forces  $P_e^{eff}$  involve only the sub matrices  $M_{be}$ ,  $K_{be}$ ,  $M_{eb}$ ,  $K_{eb}$ . These matrices vanish everywhere except in the single layer of finite elements in domain  $\Omega^+$  adjacent to  $\Gamma$ . The significance of this is that the only wave field (displacements and accelerations) needed to determine  $P_e^{eff}$  is that obtained from the simplified (auxiliary) problem at the nodes that lie on and between boundaries  $\Gamma$  and  $\Gamma_e$ .
- Only residual waves are outgoing. The solution to the DRM problem produces accurate seismic displacements inside and on the DRM boundary. On the other hand, the solution for the domain outside the DRM layer represents only the residual displacement field. This residual displacement field is measured relative to the reference free field displacements. The residual wave field has low energy when compared to the full seismic wave field, as it is a result of oscillations of the structure only. It is thus fairly easy to be damped out. This means that DRM can very accurately model radiation damping.
- The inside of the DRM boundary can be nonlinear/inelastic. This is a very important conclusion, based on the fact that only a change of variables was employed in DRM development, and the solution does not rely on superposition.
- All types of realistic seismic waves are modelled. Since the effective forcing  $P_e^{eff}$  consistently replaces the effects of the seismic source, all appropriate (real) seismic waves are properly (analytically) modelled, including body (SV, SH, P) and surface (Rayleigh, Love, etc.) waves.

Seismic motions (free field) that are used for input into a DRM model need to be consistent, i.e., a free field seismic wave needs to fully satisfy the equations of motion. For example, if free field motions are developed using a tool (e.g. SHAKE, EDT, SW4) using time step  $\Delta t = 0.01s$  and the analysis is run with a time step of  $\Delta t = 0.001s$ , a simple interpolation (10 additional steps for each of the original steps) might create problems. Simple linear interpolation might not satisfy wave propagation equations and if used will introduce additional, high frequency motions into the model. It is good practice to generate free field motions with the same time step as will be used in ESSI simulation.

Similar problem might occur if spatial interpolation is done, that is if the location of free field model nodes is not very close to the actual DRM nodes used in the SSI model. Spatial interpolation problems are less acute; however, it is still necessary to test the SSI model for free conditions and only then add the structure(s) on top.

#### **8.4.12. Structure–soil–structure interaction**

Structure–soil–structure interaction (SSSI) denotes the phenomenon of coupling of the dynamic response of adjacent structures through the soil.

The important potential effects of SSSI are generally:

- (a) Vibration of one structure affects the response of a second structure(s) in close proximity to the first<sup>16</sup>. That is, the amplitude and frequency content of each structure may be modified due to the vibration of others in its vicinity. This is most likely to occur when the two structures have similar masses or one structure is more massive than a second structure for which the effect may be significant.
- (b) The combined response of the adjacent structures may result in impact (pounding) of the two structures during earthquake shaking, which may impact loading conditions on the structures for design or evaluation.
- (c) Distribution systems running between structures (e.g., piping, heating, ventilation and air conditioning ducting, cable chases, conduits) may experience altered or increased relative displacements during earthquake shaking.
- (d) For two or more structures with embedded foundations and/or partially embedded structure elements (e.g. walls), the result of SSSI may be to increase the loading conditions on the embedded portion of the walls compared to treating each independently.

These situations require consideration when generating DBE loading conditions and BDBE loading conditions.

Simplified and detailed methods have been applied to these phenomena to determine their importance to the seismic response of the structures of interest.

A detailed method to account for SSSI effects is to include all structures in the same SSI model (see Section 8.4.12.1). In this approach, the interaction between the structures in all modes of vibration and the interaction among various modes of vibration are considered.

A simplified method is to compute the ground motion at the footprint of the light structure from the SSI analysis of the more massive structure and modify the input motion for SSI analysis of the light structure.

Simplified methods include substructuring approaches where a multi-stage analysis is performed. For example, overall structure response is calculated assuming interaction through the soil from one or more structures to another. Classical treatment in such fashion is reported

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<sup>16</sup> This discussion will focus on two structures, but the concept applies to multiple structures in close proximity.

in [293–297]. Then, a second stage analysis is performed on structures of interest with the input defined by the responses calculated including SSSI, e.g. foundation motion from the first stage excites the structures in the second stage. This is a common approach to address phenomena identified in (a) above. Generally, the effects of SSSI are secondary to the primary response of a structure due to direct excitation by the earthquake ground motion.

SSSI is a 3-D phenomenon: attempts to analyse it in 2-D (e.g. plane-strain analysis) introduce uncertainties of unknown magnitude and effect. In addition, the SSSI effect may be overemphasized by linear analysis. During SSSI, the soil regions in the immediate vicinity of the structures appear to behave in a highly nonlinear fashion, which may reduce the effect of the phenomenon. Tajimi [297] indicates that structure-to-structure interaction effects exist, but they are secondary effects with respect to the gross structural response. The effect on the overall structural response motions, in the case of two structures in close proximity, is also found to be secondary based on studies in [293–296]. The exception is the response of a lighter structure in close proximity to a more massive structure.

#### 8.4.12.1. *Detailed methods and models of structure–soil–structure interaction*

The simplest and most accurate approach is to develop a direct, detailed model for all (two or more) structures on subsurface soil and rock, then develop input seismic motions and analyse the results. While this approach is the most involved, it is also the most accurate, as it allows for proper modelling of all the structure, foundation and soil/rock geometries and material without making any unnecessary simplifying assumptions.

The main issue to be addressed with this approach is development of seismic motions to be used for input. A possible approach to developing seismic motions is to use incoherent motions with an appropriate separation distance. Alternatively, regional seismic wave modelling can be used to develop realistic seismic motions and use these as input through, for example, the DRM (see Section 8.4.10).

#### 8.4.12.2. *Simplified models: symmetry and anti-symmetry*

Symmetry and anti-symmetry models are sometimes used to reduce complexity of the direct model [298]. However, there are a number of concerns regarding simplifying assumptions that need to be made in order for these models to work. These models have to make an assumption of a vertically propagating shear waves and as such do not take into account input surface waves (Rayleigh, Love, etc.). Surface waves will additionally excite a nuclear installation structure for rocking and twisting motions, which will then be transferred to adjacent structures by means of additional, induced surface waves. If only vertically propagating waves are used for input (as is the case for symmetry and anti-symmetry models) the energy of the input surface waves is neglected. Depending on the surface wavelength and the distance between adjacent structures, a simple analysis can be performed to determine if particular surface waves, emitted/radiated from one structure toward the other one (and in the opposite direction) can influence adjacent structures. It is noted that the wavelength can be determined using a classical equation  $\lambda = v/f$  where  $\lambda$  is the length of the (surface) wave,  $v$  is the wave speed<sup>17</sup> and  $f$  is the wave frequency of interest. TABLE 6 gives Rayleigh wave lengths for four different wave frequencies (1, 5, 10 and 20 Hz) and for three different Rayleigh (very close to shear) wave velocities (300, 1000 and 2500 m/s):

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<sup>17</sup> For Rayleigh surface waves, a wave velocity is just slightly below the shear wave velocity (within 10%, depending on elastic properties of material), so a shear wave speed can be used for making Rayleigh wave length estimates.

TABLE 6: RAYLEIGH WAVE LENGTH AS A FUNCTION OF WAVE VELOCITY AND WAVE FREQUENCY

	1.0Hz	5.0Hz	10.0Hz	20Hz
300m/s	300m	60m	30m	15m
1000m/s	1000m	200m	100m	50m
2500m/s	2500m	500m	250m	125m

For a given separation between nuclear installation buildings, different surface wave (frequencies) will be differently transmitted with different effects. For example, for a nuclear installation building that has a basic linear dimension (length along the main rocking direction) of 100m, the low frequencies surface wave (1Hz) in soft soil ( $v_s \approx 300\text{m/s}$ ) will be able to encompass a complete building within a single wavelength, while for the same soil stiffness, the high frequency (20Hz) will produce waves that are too short to efficiently propagate through such a structure.

For making symmetric and antisymmetric assumptions (that is modelling a single building with one boundary having symmetric or antisymmetric boundary condition so as to represent a duplicate model, on the other side of such boundary) the following needs to be considered:

- (a) Symmetry: the motions of two nuclear installations are out phase and this is only achievable if the wavelength of surface wave created by one nuclear installation (radiating toward the other installation) is so large that the half wavelength encompasses both installations. This type of SSSI is illustrated in Figure 33.

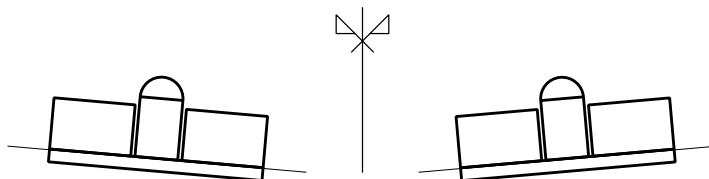


FIG. 33. Symmetric mode of deformation for two nuclear installations near each other.

- (b) Anti-symmetry: the motions of two nuclear installations are in phase. This is achievable if distance between the two installations perfectly matches the wavelengths of the radiated wave from one installation toward the other one, and if the dimension of the installation does not affect radiated waves. This type of SSSI is illustrated in Figure 34.

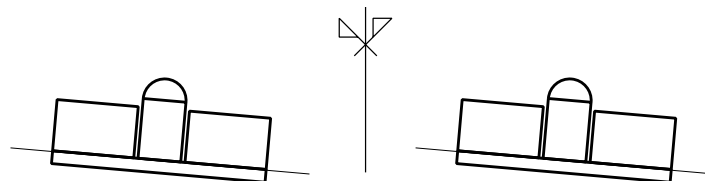


FIG. 34. Anti-symmetric mode of deformation for two nuclear installations near each other.

Both symmetry and antisymmetric assumptions place very special requirements on wave lengths that are transmitted/radiated and as such do not model general waves (various frequencies) that can be affecting adjacent nuclear installations.

### 8.4.13. Simplified models

Simplified models are used for fast prototyping and for parametric studies, as they have relatively low computational demand. Simplified modelling was addressed in some detail in the introductory part of this publication. Additional comments below provide further elaboration on simplified and more detailed modelling.

Simplified models are:

- (i) Numerical (finite element), analytical, and empirical representations of the soil–structure system and seismic input motion;
- (ii) Alternative validated models (numerical, analytical, empirical) that provide overall verification of the effect of a single parameter on results from detailed models.

Simplified models may be used to represent the soil–structure phenomena for simple structures, soil–rock situations, seismic input, etc.

Simplified models be adequate to assess the effect of the single parameter on the end results of interest. Simplified models can be used for:

- (i) Verification checks of results from a detailed model;
- (ii) Sensitivity studies, e.g. to investigate the effect of the variation of a single parameter on the end result of interest.

Significant expertise is needed to develop appropriate modelling simplifications that retain the mechanical behaviour of interest, while removing model components that are not important (for a particular analysis).

## 8.5. PROBABILISTIC RESPONSE ANALYSIS

### 8.5.1. Overview

In general, probabilistic response analysis is compatible with the definition of a performance goal for the design of nuclear installations and with the input requirements for assessments, such as SMA and SPRA. Section 2 discusses these needs in the context of performance goals defined probabilistically.

For the design of nuclear installations, one approach is to calculate the seismic demand on SSCs at 80% NEP values conditional on the DBE ground motion, as specified in [19]. The deterministic response analysis approaches specified in [19] are developed to approximate the 80% NEP response level based on sensitivity studies and judgment. The preferred approach is to perform probabilistic response analysis and generate the 80% NEP directly.

For BDBE assessments, SPRA or SMA methods are implemented, as follows:

- (a) For SPRA assessments, the full probability distribution of seismic demand conditional on the ground motion (UHRS or GMRS) is needed. The median values (50% NEP) and estimates of the aleatory and epistemic uncertainties (or estimates of composite uncertainty) at the appropriate risk important frequency of occurrence are needed. The preferred method of development is by probabilistic response analyses. An alternative is to rely on generic studies that have been performed to generate approximate values.
- (b) Two approaches to SMA assessments are used, the conservative deterministic failure margin (CDFM) method and the fragility analysis (FA) method. For the CDFM method, the

seismic demand is defined as the 80% NEP conditional on the reference level earthquake<sup>18</sup>. For the FA method, the full probability distribution is needed. In both cases, the 80% NEP values are needed to apply the screening tables of EPRI NP-6041 (1991) [298], which provide screening values for high capacity SSCs. Most applications of SMA use the CDFM method.

Soil–structure interaction includes the two most significant sources of uncertainty in the overall seismic response analysis process, i.e. the definition of the ground motion and characterization of the in-situ soil geometry and its material behaviour. Throughout this publication, aleatory and epistemic uncertainties are identified in the site response and SSI models and analyses. Implementing probabilistic response analysis permits the analysts to explicitly take into account some of these uncertainties.

To have a rational and consistent approach to applying risk-informed techniques to decision-making, the seismic demand on SSCs needs to be quantified probabilistically so that the likelihood of exceedance of a given loading condition (excitation) applied to individual SSCs is known and that the likelihood of exceedance when combined with the seismic design and fragility parameters leads to a balanced design.

Each SSC needs to be individually designed, based on how the SSC fits into the overall seismic risk profile.

### **8.5.2. Simulations of SSI phenomena**

Generally, some type of simulation is performed to calculate probabilistic responses of SSCs of nuclear installations. The seismic analysis and design methodology chain is comprised of:

- (a) Ground motion definition (amplitude, frequency content, primary earthquakes contributing to its definition, deaggregation of the seismic hazard, incoherence, location of the motion, site response analysis);
- (b) SSI phenomena;
- (c) Soil properties (stratigraphy, material properties – low strain and strain-dependent);
- (d) Structure dynamic characteristics (all aspects);
- (e) Equipment, components, distribution systems dynamic behaviour (all aspects).

An important aspect of the seismic response process is that all elements are subject to uncertainties.

#### *8.5.2.1. Seismic methodology analysis chain with statistics (SMACS)*

Probabilistic methods to analyse or reanalyse SSCs to develop seismic demands for input to SPRA evaluations were pioneered in the late 1970s. A family of computer programs called the SMACS was developed and implemented in the SPRA of the Zion nuclear power plant in the USA [299]. The method is still used in the development of seismic response of SSCs of interest to the SPRA so-called ‘seismic equipment list’ [300].

The SMACS method is based on analysing NPP SSCs using simulations of earthquakes defined by acceleration time histories at appropriate locations within the NPP site. Modelling, analysis procedures, and parameter values are treated as best estimate with uncertainty explicitly introduced. For each simulation, a new set of soil, structure, and subsystem properties are

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<sup>18</sup> The reference level earthquake is the seismic hazard realization at which the responses and capacities of the SSCs identified for the seismic safety assessment needs to be explicitly evaluated. A reference level earthquake is necessary for technical consistency in the safety evaluation, considering that several important dynamic response parameters depend on the seismic excitation level.

selected and analysed to account for variability in the dynamic properties of the soil/structure/subsystems.

For the SSI portion of the seismic analysis chain, the substructuring method is used and the basis for the SMACS SSI analysis is the CLASSI suite of computer programs [229] that implement the substructure method to SSI (see Section 8.3).

The basic steps of the SMACS method are:

(a) Seismic input:

- (i) The seismic input is defined by sets of earthquake ground motion acceleration time histories at a location in the soil profile defined in the PSHA or the SRA (Section 7). Three spatial components of motion, two horizontal and the vertical, comprise a set.
- (ii) The number of earthquake simulations to be performed for the probabilistic analyses is defined. Thirty simulations generally provide a good representation of the mean and variability of the input motion and the nuclear installation structure response. Some studies have suggested that a greater number of simulations improves the definition of the probability distributions of the response quantities of interest.
- (iii) Often, it is advantageous to use the deaggregated seismic hazard as the basis for the definition of the ground motion acceleration time histories as discussed in Section 6.

(b) Development of the median soil/rock properties and variability:

- (i) Section 4 describes site investigations to develop the low strain soil profiles – stratigraphy and other soil properties. Section 4, also, describes laboratory testing of soil samples the results of which form the basis for defining the strain-dependent soil material properties. For the idealized case (semi-infinite horizontal layers overlying a half-space), Section 6 describes site response procedures to develop an ensemble of probabilistically defined equivalent linear viscoelastic soil profiles – individual profiles from the analyses and their derived median-centred values and their variability. These viscoelastic properties are defined by equivalent linear shear moduli and material damping - and Poisson’s ratios.
- (ii) The end result of the process is used to define the best estimate and variability of the soil profiles to be sampled in the SMACS analyses.

(c) Structure model development:

- (i) The structure models are general FEM models. The input to the SMACS SSI analyses is the model geometry, mass matrix, and fixed-base eigen system. Adequate detail is included in the model to permit the generation of structure seismic forces for structure fragility evaluation and in-structure response spectra (ISRS) for fragility evaluation of supported equipment and subsystems.
- (ii) The structure model is developed to be median-centred for a reference excitation corresponding to an important seismic hazard level for the SPRA or SMA. Stiffness properties are adjusted to account for anticipated cracking in reinforced concrete elements and modal damping is selected compatible with this anticipated stress level.

(d) SSI parameters for SMACS:

- (i) Foundation input motion<sup>19</sup>: The foundation input motion differs from the free field ground motion in all cases, except for surface foundations subjected to vertically

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<sup>19</sup> The term ‘foundation input motion’ refers to the result of kinematic interaction of the foundation with the free field ground motion.



incident waves. The motions differ for two reasons. First, the free field motion varies with soil depth. Second, the soil-foundation interface scatters waves because points on the foundation are constrained to move in accordance with its geometry and stiffness. The foundation input motion is related to the free field ground motion by means of a transformation defined by a scattering matrix.

- (ii) Foundation impedances: Foundation impedances describe the force-displacement characteristics of the soil/rock. They depend on the soil/rock configuration and material behaviour, the frequency of the excitation, and the geometry of the foundation. In general, for a linear elastic or viscoelastic material and a uniform or horizontally stratified soil deposit, each element of the impedance matrix is complex-valued and frequency dependent. For a rigid foundation, the impedance matrix is  $6 \times 6$ , which relates a resultant set of forces and moments to the six rigid body degrees-of-freedom.
  - (iii) The standard CLASSI methodology is based on continuum mechanics principles and is most applicable to structural systems supported by surface foundations assumed to behave rigidly. Hybrid approaches exist to calculate the scattering matrices and impedances for embedded foundations with the computational efficiency of CLASSI. Median-centred foundation impedances and scattering functions are calculated using the embedment geometry and the median soil profile.
- (e) Statistical sampling and Latin hypercube experimental design:
- (i) The inputs to this step in the SMACS analyses are the number of simulations (N), the variability of soil/rock stiffness and damping, and the variability of the structure's dynamic behaviour (structure frequencies and damping). As presented above, the median-centred properties of the soil and structure are derived in initial pre-SMACS activities. The probability distributions of the soil stiffness, soil material damping, structure frequency, and structure damping are represented by scale factors with median values of 1.0 and associated coefficients of variation. Stratified sampling is used to sample each of the probability distributions for the defined parameters. Latin hypercube experimental design is used to create the combinations of samples for the simulations. These sets of N combinations of parameters when coupled with the ground motion time histories provide the complete probabilistic input to the SMACS analyses.
- (f) SMACS analyses:
- (i) Using the free field time histories, median-centred structural model, median-centred SSI parameters, and the experimental design obtained above, N SSI analyses are performed. For each simulation, time histories of seismic responses are calculated for quantities of interest: (i) internal forces, moments, and stresses for structure elements from which peak values are derived; and (ii) acceleration time histories at in-structure locations, which provide input for subsystem analyses, i.e., equipment, components, distribution systems, in the form of time histories, peak values, and ISRS;
  - (ii) On completion of the SMACS analysis of N simulations, the N values of a quantity of interest (e.g. peak values of an internal force, ISRS at subsystem support locations) are processed to derive their median value, i.e. the 50% NEP and other NEP values (e.g. the 84% NEP value). This then permits fitting probability distributions to these responses for combination with probability distributions of capacity, thereby creating families of fragility functions for risk quantification. The specific requirements of the SPRA, guide the identification of these probabilistically-defined in-structure response quantities based on structure and equipment fragility needs.

Example results

Nakaki et al. 2010 [300] presents the SMACS probabilistic seismic response analyses of selected NPP structures for the purposes of the SPRA, as follows:

- (i) For the NPP site, the ensemble of X-direction seismic input motions displayed as response spectra (5% damping) is presented in Figure 35. The input motion variability is apparent.
- (ii) For the selected NPP reactor building, a representative ensemble of ISRS for the X-direction at Elevation 8m (approximately 20m above the base mat) is shown in Figure 36.

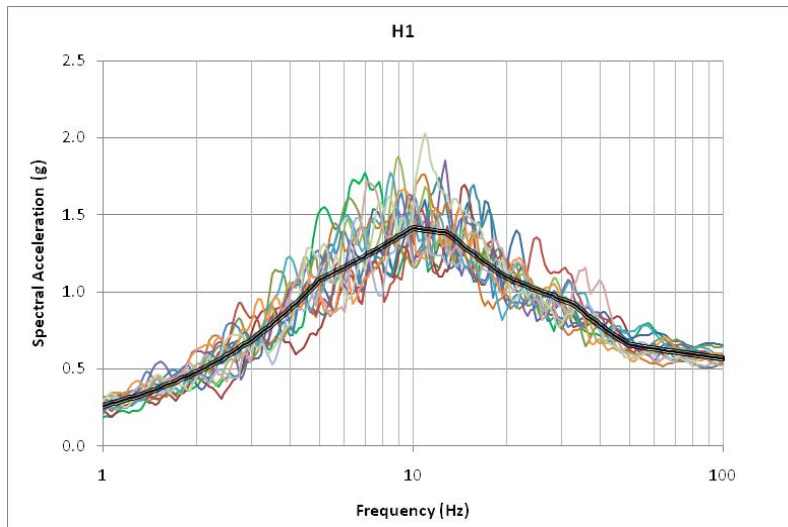


FIG. 35. Example of free field ground motion response spectra for a nuclear power plant: X-direction, 5% damping.

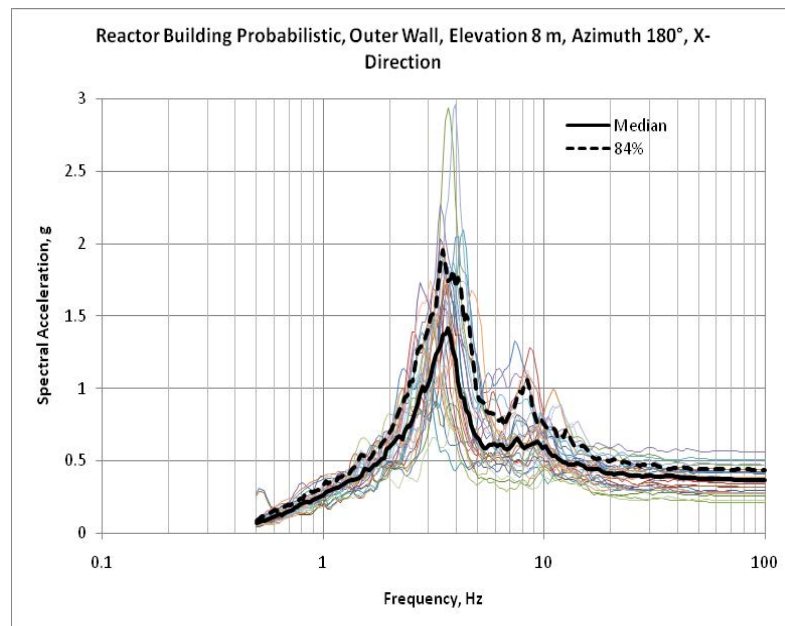


FIG. 36. Example of in-structure response spectra for a nuclear power plant: X-direction, 5% damping – median and 84% NEP.

### 8.5.2.2. Monte Carlo approach to modelling and analysis

The Monte Carlo approach is used to estimate probabilistic site response, when both input motions (rock motions at the bottom) and the material properties are uncertain. For a simplified method, using an equivalent linear (EQL) approach (strain compatible soil properties with

(viscous) damping), a large number of combinations (statistically significant) of equivalent linear (elastic) stiffness for each soil layer are analysed in a deterministic way. In addition, input loading can also be developed into a large number of (statistically significant) rock motions. A large number of results (surface motions, spectra, etc.) can then be used to develop the statistics (mean, mode, variance, sensitivity, etc.) of the nuclear installation site response. The method is fairly simple as it utilizes already existing Ella site response modelling, repeated large number of times. For proper (stable) statistics, a very large number of simulations need to be performed, which makes this method very computationally intensive.

While the Monte Carlo approach can sometimes be applied to a 1D EQL site response analysis, any use for 2-D or 3-D analysis (even linear elastic) creates an insurmountable number of simulations that cannot be performed in reasonable time even on large supercomputers. The problem becomes even more overwhelming if instead of linear elastic (equivalent linear) material models, elastic-plastic models are used, as they feature more independent (or somewhat dependent) material parameters that need to be varied using the Monte Carlo approach.

#### *8.5.2.3. Random vibration theory*

Random vibration theory is used for evaluating probabilistic site response. Instead of performing a (statistically significant) large number of deterministic simulations of site response (all still in 1-D), the RVT approach can be used [159]. RVT uses a Fourier amplitude spectrum (FAS) of rock motions to develop a FAS of surface motions that can then be used to develop peak ground acceleration and spectral acceleration at the surface. However, time histories cannot be developed, as phase angles are missing.

#### *8.5.2.4. Stochastic finite element method*

Instead of using repetitive Monte Carlo computations (with high computational cost), uncertainties in material parameters (left hand side) and the loads (right hand side), can be directly taken into account using stochastic finite element method (SFEM) [301]. Sett et al. [302] developed a Stochastic Elastic Plastic Finite Element Method (SEPFEM) that can be used for modelling of seismic wave propagation through inelastic (elastic-plastic) stochastic material (soil).

Both SFEM and SEPFEM provide very accurate results in terms of full probability density functions (PDFs) of the main unknowns (degrees of freedom) and stresses (forces). This very accurate calculation of full PDFs supplies accurate tails of PDFs, so that cumulative distribution functions (or fragilities) can be accurately obtained. However, while SFEM and SEPFEM are (can be) extremely powerful, and can provide very useful, full probabilistic results (generalized displacements, stress/forces), it involves significant analytical expertise. In addition, significant site characterization data is needed in order for uncertain (stochastic) characterization of material properties.

If such data is not available, non-site-specific data available in literature can be used [125, 126, 128]. However, use of non-site-specific data significantly increases uncertainties (the tails of material properties distributions become very ‘thick’) as data is now obtained from a number of different, non-local sites, and is averaged, a process which usually increases variability.

## 8.6. LIMITATION OF NUMERICAL MODELLING

All numerical models have inherent limitations. It is important to understand the limitations and their impact on the end results of interest. Sensitivity studies are essential for this understanding

and need to be performed to assess the sensitivity of parameters to the chosen modelling level of detail, as well as the sensitivity to modelling detail.

Extensive verification and validation of the numerical tools and models need to be used to increase confidence in the results. Sound engineering be applied to the assessment of the end results. A hierarchy of models from simplified to more detailed be used in the assessment of the results of the analyses.

## **9. SEISMIC RESPONSE ASPECTS FOR DESIGN AND ASSESSMENT OF NUCLEAR INSTALLATIONS**

### **9.1. OVERALL MODELLING DECISIONS**

All modelling decisions are dependent on the purpose of the analysis, which includes the following:

- (a) Design for DBE – realistic conservatism is to be incorporated into the seismic analyses (and design).
- (b) Assessments for BDBEs – using realistic or best estimate approaches taking into account uncertainties through probabilistic analyses or deterministic analyses incorporating appropriate variability in parameters and models in the SSI analyses. The end results may serve multiple purposes, including developing seismic demands for the nuclear installation site and for SSCs to assess realistic or conservative margins to failure.
- (c) Assessments of behaviour of nuclear installations subjected to actual earthquake ground motions (so-called ‘forensic investigations’).
- (d) Determining the following:
  - (i) The level of site specific free field ground motion (or standard values).
  - (ii) The PGA and associated frequency characteristics typically defined by response spectra.
  - (iii) Other kinematic parameters, such as ground velocity and displacement;
  - (iv) Other important factors, such as duration of strong shaking;
  - (v) Strain levels induced in the soil and structure.
- (e) Determining the requirements for modelling linear, equivalent linear, or nonlinear behaviour.
- (f) Soil modelling, which is dependent on physical characteristics of the soil and induced strain levels (covered in Section 4 and Section 9.2).
- (g) Assessing site stratigraphy and topography, including irregular soil/rock profiles, which influence decisions concerning wave propagation characteristics of the ground motion.

Decisions concerning structure modelling consider the following issues (Section 9.5.1):

- (a) Multi-step vs single step analysis – single step models are necessary when structure behaviour and seismic response output quantities are to be calculated directly from the SSI model; single step structure models need to be detailed enough to provide force and stress results for all specified load combinations. Generally, single step structure models are needed when structure behaviour is expected to be nonlinear.
- (b) The first step of a multi-step analysis involves the model and analysis of soil, foundation, and structure to adequately represent overall behaviour of the soil–structure system; subsequent steps in the analysis process will incorporate significantly more elements and detail in the analyses, e.g. interaction of flexible base mat and walls with surrounding soil, nonlinear behaviour of structure elements, flexibility of floor slabs, and other complex behaviour.

- (c) LMSMs of structures vs FEM models; typically, single step analyses involve more detailed representations of the foundation and structure using FEM. LMSMs need to adequately represent the dynamic behaviour of the structure for the purpose of the SSI analysis.
- (d) Frequency range of interest – especially high frequency considerations (50 Hz, 100 Hz).

Decisions concerning foundation modelling consider the following items:

- (a) Rigid or flexible behaviour, including accounting for stiffening effects due to structure;
- (b) elements connected to the foundation;
- (c) Mat vs spread/strip footings;
- (d) Piles and caissons; pile groups;
- (e) Boundary conditions – base mat slab retains in contact with soil/separates from underlying soil;
- (f) Surface-or near surface-founded;
- (g) Embedded foundation with partially embedded structure;
- (h) Partially embedded (less than all sides);
- (i) Contact/interface zone for embedded walls and base mat (soil pressure, separation/gapping and sliding).

## 9.2. SOIL MODELLING

As explained in Section 4, soil characterization is a complex task and, depending on the choice of the soil constitutive model used for the analyses, the number of parameters to determine may vary to a large extent and degree of complexity (see Table 5). Therefore, it is important that the level of effort put in the determination of the soil characteristics is adapted to the needs without overshadowing the essential features of soil behaviour. In any case, it is essential that soil characteristics be determined by site specific investigations including field tests and laboratory tests, which, as far as possible, yield coherent soil characteristics; any incoherence needs to be analysed and explained. Laboratory tests and field tests will be used in combination since each of them has its own merit, limitation and range of applicability [19]. Special attention needs to be paid to the characterization of human-made backfills for which the characteristics can only be measured and determined if enough specifications are available in terms of material source, identification, and compaction.

In several regions of the world, the design earthquake may represent a moderate level earthquake which will induce only small to moderate strains in the soil profile. Typically, an earthquake with PGA of the order of 0.2–0.3g may be considered as a moderate event; however, as explained below and in Section 9.3.2.1, PGA is only regarded as a rough measure for classifying the earthquake as moderate or high, and induced strains need to be definitely considered. In highly seismic regions, the DBE may represent a strong motion event. These features need to be considered when defining the soils parameters and associated investigations needed for design. In the first instance (moderate event), the most appropriate constitutive model will be the equivalent linear (EQL) model. The decision to use an EQL model is not based only on PGA; the right indicator to use is the shear strain. This model is simple enough to be amenable to the large number of parametric and sensitivity analyses necessary to take into account the variability of soil properties (see Section 9.7.2). The uncertainty on the elastic properties is not the single parameter that needs to be considered: large uncertainties exist in the determination of the nonlinear shear stress–shear strain curves (or equivalently  $G/G_{\max}$  and damping ratio curves used to define the equivalent linear model). It is important to measure these curves on undisturbed samples retrieved from the site and not to rely exclusively on published results; however, comparisons with published results are useful to define the possible variation of the curves and to assess the possible impact of such variations on the site response. It would be very uncertain to attempt to relate the domain of validity of the EQL model to some

earthquake parameter (like PGA) since the strains also strongly depend on the material: some materials are ‘more linear’ than others (e.g. highly plastic clay). For a preliminary estimate, PGA’s less than 0.2g – 0.3g may be considered as moderate earthquakes for which the EQL model is relevant. However, in general, the validity of the EQL model needs to be checked at the end of the analyses by comparing the induced shear strain to a threshold strain beyond which the constitutive model is no longer valid. As described in Section 4, the threshold strain can be set to twice the reference shear strain (see Section 4 for definition of the reference shear strain and Section 9.4.2). In a 1-D model, if the definition of the shear strain is straightforward, in a 3-D situation the shear strain, can be compared to the threshold strain, as the second invariant of the deviatoric strain tensor.

In highly seismic regions, it is most likely that the induced motions will be large enough to induce moderate to large strains in the soil profile. Therefore, the EQL model might not be appropriate to represent the soil behaviour. True nonlinear soil models are needed to analyse the SSI response. As indicated in Section 4, numerous nonlinear models exist, and the choice cannot be unique; it is important that at least two different constitutive models are used by possibly two different analysts. The models need to be validated for different stress paths, not only with respect to shear strain–shear stress behaviour but also with respect to volumetric behaviour, and their limitations need to be fully understood by the analysts. Furthermore, it is highly desirable, although not mandatory, that the models possess a limited number of parameters that are easily amenable to determination and are based on physical backgrounds. The soil response is highly sensitive to the chosen model; consequently, it is essential that uncertainty in the parameters, especially those with no physical meaning, is assessed through parametric studies.

### 9.3. FREE FIELD GROUND MOTIONS

Describing the free field ground motion at a nuclear installation site for SSI analysis purposes entails specifying the point at which the motion is applied (the control point), the amplitude and frequency characteristics of the motion (referred to as the control motion and typically defined in terms of ground response spectra, and/or time histories), the spatial variation of the motion, and, in some cases, strong motion duration, magnitude, and other earthquake characteristics.

In terms of SSI, the variation of motion over the depth and width of the foundation is the important aspect. For surface foundations, the variation of motion on the surface of the soil is important; for embedded foundations, the variation of motion over both the embedment depth and the foundation width are important.

In terms of SSI analysis, the definition of the seismic input is dependent on the SSI models to be used, for example in substructuring methods that separate the seismic input problem into kinematic interaction, often with the assumption of the foundation behaving rigidly, the seismic input is the response of a massless, rigid foundation; nonlinear SSI analysis models define a soil island and the seismic input is defined by time histories of displacements, velocities, and accelerations (or force resultants) on the boundaries of the soil island.

There are two stages in the development of the site specific free field ground motion and seismic input to the SSI analyses:

- (a) Earthquake source to the vicinity of the nuclear installation site. Several methods of modelling from the source to the vicinity of the site are described in Sections 5, 6, and 7. The most prevalent approach is DSHA or PSHA based on empirical GMPEs. The SHA results are most often derived at the TOG at the site of interest or at a location within the site profile, such as on hard rock, a competent soil layer, or at an interface of soil/rock stiffness with a significant impedance contrast. These results do not yet contain site specific characteristics

that define the spatial variation of the ground motion, strain-dependency of soil material properties, etc.

- (b) Local site effects. Given the results of the SHA, the seismic input for the SSI analysis is generated through SRA or other means to take into account local site effects. The purpose of SRA is to determine the free field ground motion at one or more locations given the motion at another location. Site response analysis is intended to take into account the wave propagation mechanism of the ground motion, the topography of the nuclear installation site and the site vicinity, the stratigraphy of the site, and the strain dependent material properties of the media. Sections 6 and 7 discuss aspects of the free field ground motion and seismic input that may be important to take into account. Later in Section 9, many of these potentially important features are summarized and addressed.

The locations of interest at the site for which SRA results are sought include seismic input motion for the SSI analysis models, e.g. motions on the soil island boundary needed for nonlinear SSI analyses, and foundation input response spectra, i.e. free field ground motion at the foundation level of structures of interest from which the SSI analysts can generate seismic input to the SSI models (kinematic interaction effects). The seismic input motions may be generated by convolution or deconvolution procedures. Convolution procedures are strongly preferred. Deconvolution needs to be used carefully especially when generating strain-dependent soil material properties.

Approaches to SRA are selected on the basis of the site specific characteristics – site stratigraphy (three-dimensional vs one dimensional soil profiles), in-situ rock/soil physical attributes, and characteristics of the free field ground motion (See Sections 9.4).

The most common assumptions applied to SRA are idealized soil profiles (semi-infinite horizontal layers over a uniform half-space), wave propagation mechanisms of vertically propagating S- and P-waves, one dimensional wave propagation theory, i.e., S- and P- waves uncoupled. Section 9.4 discusses the prevalent approaches being applied to the SRA for generating seismic input and soil properties for the SSI analyses.

#### 9.4. SITE RESPONSE ANALYSIS – APPROACHES 1, 2, 3, AND 4

Sections 6 and 7 presented many facets of SRA, including research results and methodology developments. Section 9.4 expands on the Section 7.3.1 description of current SRA approaches for the idealized conditions, as follows:

- (a) Soil layer stratigraphy (semi-infinite horizontal layers overlying a uniform half-space), variability in layer thickness is modelled;
- (b) Soil material properties (1-D equivalent linear viscoelastic models defined by shear modulus and material damping – median and variability);
- (c) Wave propagation mechanism vertically propagating S- and P-waves.

Updates to SRA to account for non-idealized soil profiles, nonlinear soil property characterization, and non-vertically incident free field ground motion are discussed in Section 9.4. These important sensitivity studies were performed for a single time history.

In actual applications, the input motion is not defined with a single time history, but its definition relies on the techniques designated as Approaches 1, 2A, 2B, 3, and 4. Higher numbers are associated with the more rigorous approaches, specifically with respect to the potential sensitivity of the SAFs to magnitude and distance dependency of seismic sources, non-linearity of the soil properties, and uncertainty in the site profile and dynamic soil properties.

Approach 4 is the most computationally detailed and takes the results of each simulation in the PSHA process carrying it through the SRA process. This could be millions of simulations, which is infeasible. Approach 3 is the second most computationally detailed. It is the most comprehensive, rigorous, and feasible approach being implemented. Approach 3 considers a significantly greater range of contributing seismic sources and a more complete representation of the spectral values over the natural frequency range compared to Approaches 1 and 2. Approach 3 is implemented more frequently than Approaches 2A and 2B due to its perceived increased rigor and probabilistic aspects. Approaches 2B, 2A, and 1 include increasingly simplified assumptions as compared to Approach 3.

The basic concept of Approach 3 is to convolve a probabilistic representation of site response with the probabilistic seismic hazard results for the reference or control point of the PSHA, usually the base rock at the nuclear installation site.

Approach 3 incorporates the SAFs into the hazard calculation through convolution of the bedrock hazard curves for each spectral frequency with the PDF for the SAFs to compute hazard curves for locations within the site profile. Convolution permits each bedrock ground motion level to contribute to the hazard for each ground motion level at the location of interest in the site profile and at each spectral frequency.

For Approaches 1, 2A, and 2B type analyses, the following steps are common:

- (a) Selecting the best estimate and variability of soil profile(s) to be analysed. In many cases there will be a single best estimate soil profile. However, in some cases (such as evaluations performed following the SPID or when the PSHA determines that epistemic uncertainty in defining the soil profile be represented by two or more soil profiles), there will be multiple soil columns each having a 'weight'. For example, the SPID specifies a weight of 0.4 for the best estimate range properties and 0.3 each for the lower and upper range estimates. Soil profiles generally include variations in depth to rock and thicknesses of layers of soil;
- (b) Selecting the degradation models ( $G/G_{MAX}$  and damping with strain level) for each soil strata. The degradation models include variability of  $G$  and damping with strain;
- (c) Constructing the UHRS spectra at the rock outcrop and its characteristics (e.g. deaggregated magnitude and distance of the principal contributors) for the AFE of interest (e.g.,  $1 \times 10^{-4}$  to  $1 \times 10^{-5}$  for DBE;  $1 \times 10^{-4}$  to  $1 \times 10^{-7}$  for BDBE considerations);
- (d) Selecting the method of analysis – time domain or RVT;
- (e) Time domain analyses - selecting seed time histories from PEER (or other) database and scale to UHRS as described below;
- (f) Randomizing soil properties and using degradation models with a depth to horizon for each soil profile to obtain 60 realizations for LHS. Sixty realizations are typically adequate to perform the SRA;
- (g) Performing 1-D response analyses (CARES, SHAKE, STRATA, etc.) to determine strain compatible properties and response of each of the realizations;
- (h) Computing statistics (mean spectra, mean and +/- 1 sigma descriptions of the calculated soil properties) for each soil profile;
- (i) When multiple profiles are defined, combining the results for each profile by weighting each of the multiple cases.

The primary difference between the three approaches is in how the input motion is defined. The differences are as follows:

### **Approach 1**

Approach 1 uses time histories fit to the UHRS spectrum at the rock outcrop to drive the soil column realizations. Alternatively, an RVT approach is used with the UHRS spectrum as input converted to FAS. Approach 1 can significantly over-drive the soil column producing larger



than realistic reductions in stiffness and larger than realistic damping values due to the broad-banded nature of the UHRS.

### **Approach 2A**

Approach 2A is intended to minimize overestimating the non-linear effects. This approach involves identifying the magnitude (M) and distance (D) for the controlling earthquake event at low frequency (1Hz) and high frequency (10Hz). SRA are then performed for the low frequency and high frequency motions separately and the results combined as discussed below.

One of two methods of defining the input motion can be used:

- (a) Recorded time histories having spectral shapes consistent with the spectral shape associated with the M and D can be selected for the low frequency and high frequency cases. These records are scaled to the UHRS at 1 and 10 Hz, respectively, and used as input to the site response process.
- (b) Target spectral shapes associated with the M and D are computed for the low frequency and high the frequency (using, for example, spectral shapes defined by spectral shape formulae in [157]). These spectra are scaled to the UHRS at 1 and 10 Hz and time histories are developed that match these target spectra. These time histories are then used as input to the site response process. Alternatively, the spectra are used in an RVT approach to perform the site response process.

If the envelope of the 1 and 10 Hz shapes fall more than 10% below the UHRS in intermediate frequencies, then an additional spectrum is selected to fill in the gap.

### **Approach 2B**

Approach 2B provides an additional level of rigor to the definition of the input motion. This approach involves identifying M and D values for multiple spectra at low frequency (1Hz) and high frequency (10Hz), such that the variability in the input M and D in the PSHA can be captured.

Response spectra are determined from the deaggregated hazard at various levels of the NEP acceleration level at the low frequency and the high frequency. The NEP levels of interest span the range of 5% to 95% if feasible. Spectra are generated for these individual M and D values following one of the two methods described in Approach 2A above. These spectra are scaled to match the input bedrock target spectrum (UHRS) and the site response process is performed for each of these events.

Mean surface amplification is calculated using the weighted mean of the results from these various input motions and the UHRS rock motion is scaled to obtain the surface (or FIRS) motion. For multiple base cases, the results are combined based on weights assigned for each of the cases, e.g., the best estimate (median) case is weighted highest and other base cases are weighted in proportion to their likelihood.

#### **9.4.1. 1-D model**

A 1-D soil column is used to develop examples, presented in Annex II, illustrating the differences between the various approaches to 1-D site response analyses. The soil profile consists of 30m of sandy gravel overlying a 20m thick layer of stiff, over consolidated, clay on top of a rock layer considered as a homogeneous half space (see Figure 37). The water table is located at a depth of 10m below the ground surface. The incident motion is imposed at an outcrop of the half space in the form of an acceleration time history. The soil constitutive models include an equivalent linear model, a nonlinear model for 1-phase medium and a nonlinear model for 2-phase (saturated) medium.

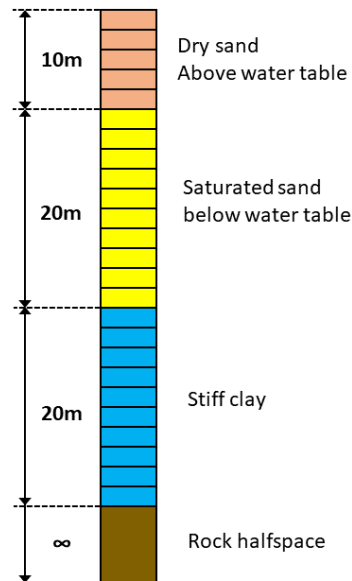


FIG. 37. Soil profile for illustrative examples on 1-D site response analyses.

Under the assumption of vertically propagating shear waves, the numerical model is a 1-D geometric model; however, to reflect the coupling between the shear strain and volumetric strains each node of the model possesses two (1-phase medium or 2-phase undrained layer) or four (2-phase medium pervious layer) degrees of freedom corresponding to the vertical and horizontal displacements (respectively vertical and horizontal translations of solid skeleton and vertical and horizontal velocities of the fluid).

The purposes of the analyses are to:

- (a) Compare equivalent linear and nonlinear constitutive models;
- (b) Show the differences between total vs effective stress analyses;
- (c) Show for a 2-phase medium the impact of the soil permeability;
- (d) Compare the predicted vertical motion assuming P-wave propagation to the motion calculated from the horizontal motion with V/H GMPEs [303].

The results are compared in terms of 5% damped ground surface response spectra, pore pressure evolution in time at mid-depth, horizontal and vertical displacements at the ground surface.

Table 7 summarizes the different analysed cases. All the nonlinear analyses are run with the software Dynaflo (see Section 10); the equivalent linear analyses are run with SHAKE.

TABLE 7: SUMMARY OF ANALYSED CASES

Case	1a – 1b	2a – 2b	3	4	5	6
Continuum	1-Phase	1-Phase 2-Phase	2-Phase	2-Phase	2-Phase	2-Phase
Model	Total stresses	Effective stresses	Effective stresses	Effective stresses	Effective stresses	Effective stresses
Constitutive relationship	Equivalent linear/nonlinear	Elastoplastic	Elastoplastic	Elastoplastic	Elastoplastic	Elastoplastic
Permeability (m/s)	-	0	$10^{-5}$	$10^{-4}$	$10^{-3}$	$10^{-2}$
Input motion	Horizontal	Horizontal	Horizontal	Horizontal	Horizontal	Horizontal
Software	Shake Dynaflow	Dynaflow	Dynaflow	Dynaflow	Dynaflow	Dynaflow
Case	6a – 6b	7a – 7b	8	9		
Continuum	1-Phase	1-Phase 2-Phase	2-Phase	2-Phase		
Model	Total stresses	Effective stresses	Effective stresses	Effective stresses		
Constitutive relationship	Equivalent linear/linear	Elastoplastic	Elastoplastic	Elastoplastic		
Permeability (m/s)	-	0	$10^{-5}$	$10^{-2}$		
Input motion	Horizontal + vertical	Horizontal + vertical	Horizontal + vertical	Horizontal + vertical		Horizontal + vertical
Software	Shake Dynaflow	Dynaflow	Dynaflow	Dynaflow		Dynaflow

#### 9.4.1.1. Comparison EQL / nonlinear (cases 1a–1b)

The example is used to point out that, beyond some level of shaking, EQL solutions are not valid. Figure 38 illustrates the comparison in terms of ground surface response spectra calculated for three increasing amplitudes of the input motion. Figure 38 and additional figures presented in Annex II show that as long as the input motion is not too strong (in this case PGA  $\sim 0.20g$ ), the EQL and nonlinear solutions do not differ significantly. For PGA = 0.25g, differences start to appear in the acceleration response spectra: high frequencies are filtered out in the EQL solution and a peak appears at 3Hz corresponding to the natural frequency of the soil column.

At PGA = 0.5g, the phenomena are amplified with a sharp peak at 2.8Hz in the EQL solution. Filtering of the high frequencies by the EQL analysis has been explained in Section 4.4: they are dumped because the damping ratio and shear modulus are based on the strain, which is controlled by low frequencies, and the same damping is assigned to all frequencies. High frequency motions induce smaller strains and therefore be assigned less damping.

The value of the PGA threshold depends on the material behaviour; the fundamental parameter to look at is the induced shear strain, or preferably the reference shear strain  $\gamma_r$ . The maximum shear strain calculated as a function of depth for each run is plotted in Figure 30. When the amplitude of the input motion is smaller than 0.20g, the maximum induced shear strain remains smaller than  $10^{-3}$ , which was indicated as the upper bound value for which equivalent linear analyses remain valid and, indeed, EQL and nonlinear analyses give similar results. When the amplitude of the input motion is equal to 0.50g, the maximum induced shear strain raises up to  $3.4 \times 10^{-3}$  at 20m depth; at that depth the reference shear strain is equal to  $10^{-3}$  (calculated from Table II.3 in Annex II) and therefore the induced shear strain is larger than two times the reference shear strain. For an input motion of 0.25g, the maximum shear strain at 20m depth is approximately equal to  $2\gamma_r$  and equivalent linear analyses and nonlinear analyses start to diverge.

#### 9.4.1.2. Total vs effective stress analyses (cases 1a–2a)

The analyses presented in Annex II show that at low level of excitation (PGA  $\leq 0.25g$ ) both solutions (total or effective stress analyses) are comparable except for the pore pressure build up which cannot be predicted by the total stress analysis. At PGA = 0.50g, differences appear in the acceleration response spectra and in the vertical displacements (see Figure 40, left). It can be concluded that effective stress analyses are not needed for low level of excitation but are important for high levels, when the excess pore pressure becomes significant.

For an impervious material the effective stress analyses carried out assuming either a 1-phase medium or a 2-phase medium (case 2a–2b) do not show any significant difference (see Figure 40, right). Based on the results of other analyses, not presented herein, this statement is true as long as the permeability is smaller than approximately  $10^{-4}$  m/s. Therefore 2-phase analyses are not necessary for those permeabilities.

The impact of the value of the permeability appears to be minor on all parameters except the excess pore pressure (see Figure 41). It is only for high permeabilities ( $10^{-2}$  m/s) and high input excitations that differences appear in the acceleration response spectrum and, to a minor extent, in the vertical displacement.

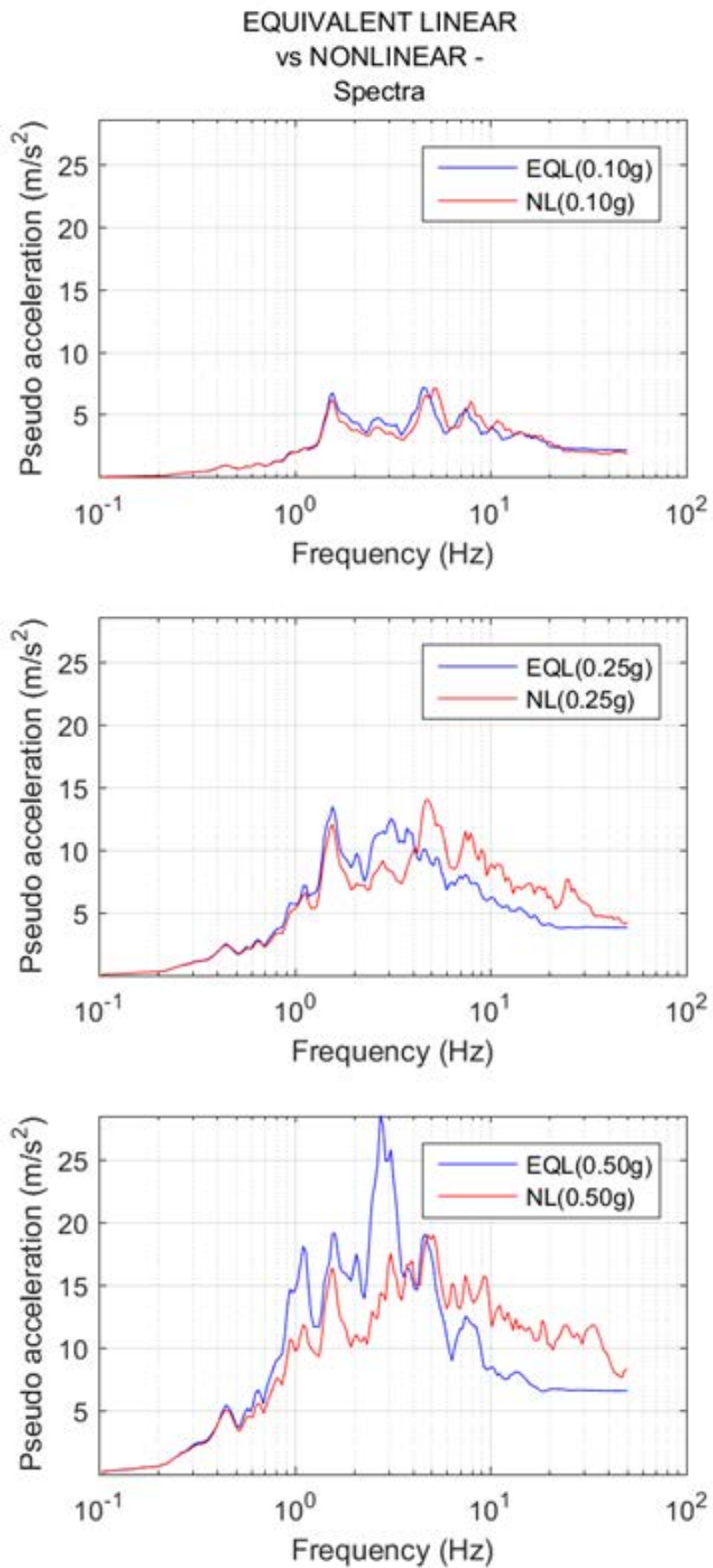


FIG. 38. Comparison of ground surface response spectra between equivalent linear (EQL) and nonlinear (NL) analyses

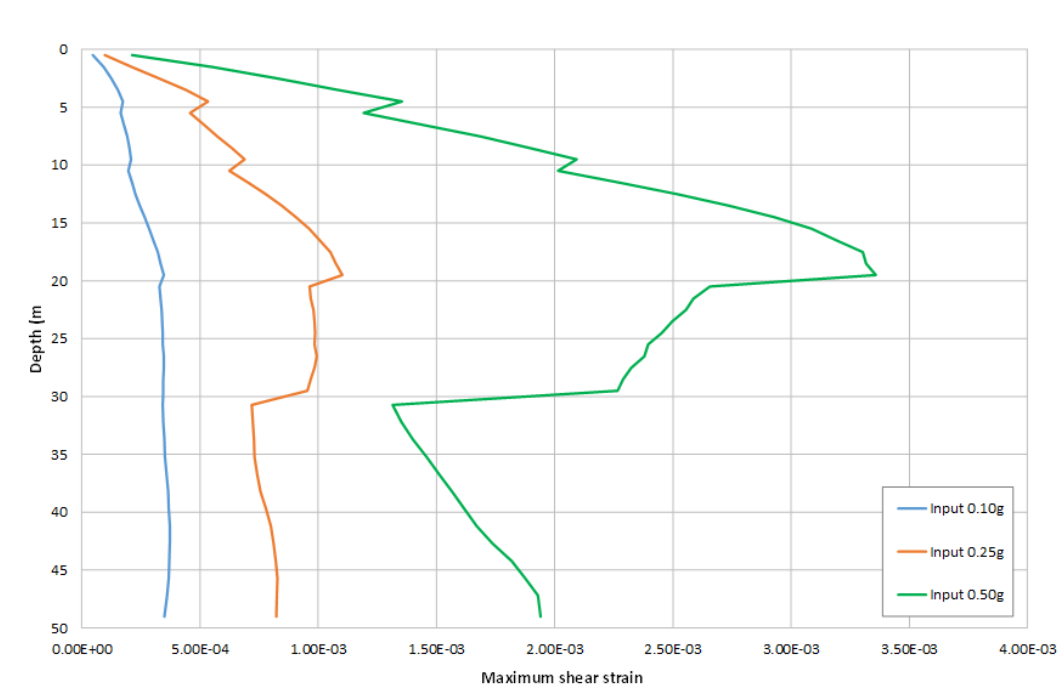


FIG. 39. Maximum shear strain versus depth for 3 input accelerations levels.

#### 9.4.1.3. Vertical motion

The previous analyses are run with a single, horizontal, component for the input motion. The vertical component of the ground motion is often assumed to be caused by the vertical propagation of P waves (Section 6.3.1); as explained in section 7.3.1 and 7.3.3 the vertical motion cannot usually be assumed to be created only by the vertical propagation of P waves.

Rayleigh waves, diffracted P–SV waves also contribute to the response. In the present analysis, the two (horizontal and vertical) components of motion are input simultaneously in the model since the material behaviour is nonlinear. From the only calculated spectra at the surface, V/H ratios have been computed and compared to the results of a Gülerce–Abrahamson prediction equation for a magnitude 6.9 event recorded at 15km from the source (Joyner–Boore distance). The results, presented in Figure 42, show that for frequencies less than 10Hz the vertical motion can be predicted assuming vertical propagation of P waves; however, for higher frequencies the vertical motion cannot be assumed to be created by the vertical propagation of P waves. Rayleigh waves, diffracted P–SV waves also contribute to the response.

#### 9.4.1.4. Lightly damped profiles

The purpose of this example is to show that for nearly elastic materials, the choice and modelling (hysteretic, Rayleigh) of damping is critical for the site response. The soil column is composed of one layer of elastic rock material (2 km thick with  $V_S = 2000\text{m/s}$ ) overlying a half space with  $V_S = 3000\text{m/s}$ . An elastic model is used for the rock because, for such a high shear wave velocity, significant nonlinear degradation of the shear modulus cannot be expected; furthermore, there is no experimental evidence nor reliable, well documented curves for the modulus degradation curve and damping curve of hard rocks. It is subjected to a real hard rock record with a duration of 60s and a maximum acceleration equal to 0.03g; although the PGA is very small it has not been scaled up since all calculations presented in this example are linear. The record, its 5% damped response spectrum, and its FAS are shown in Annex II.

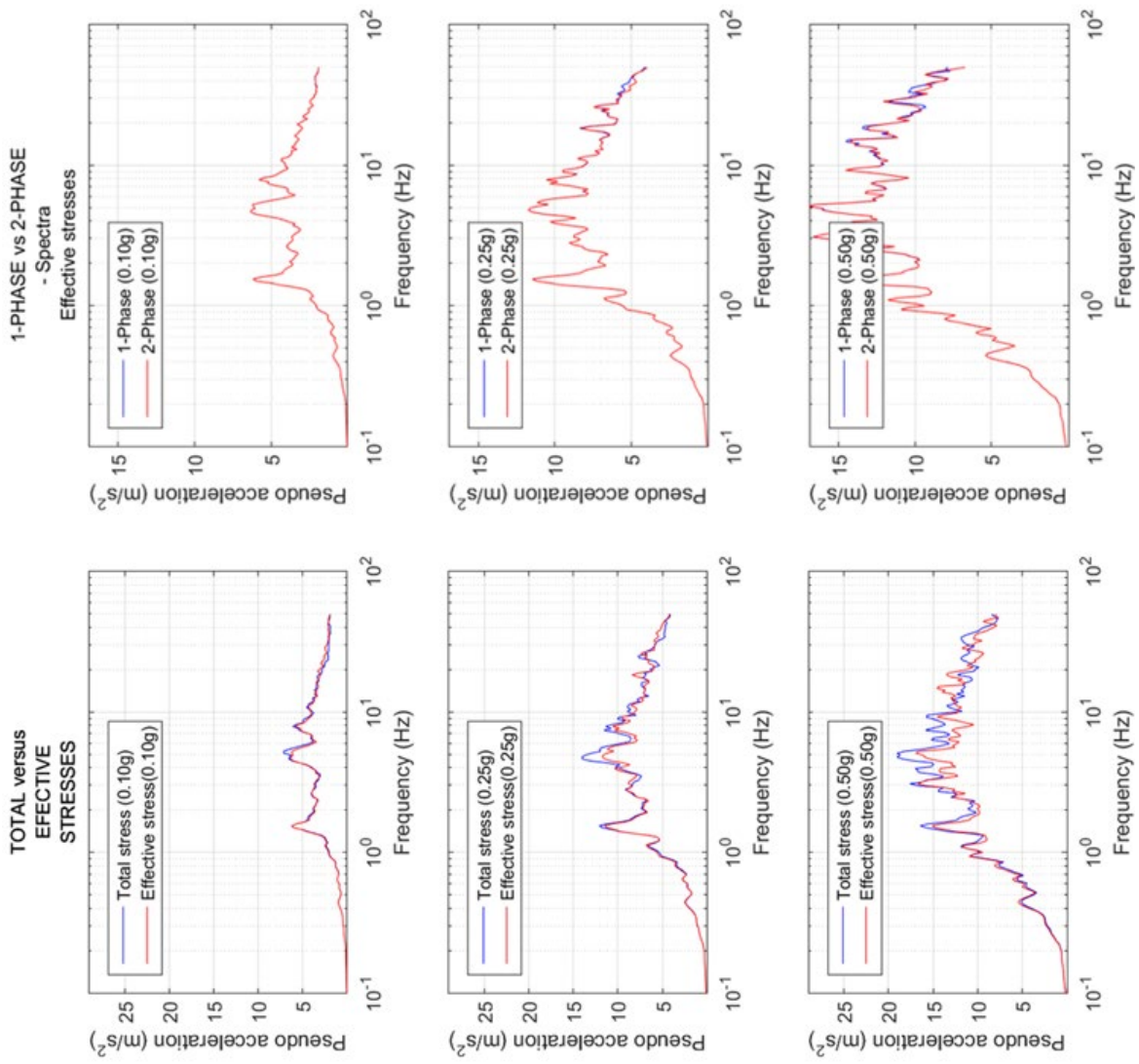
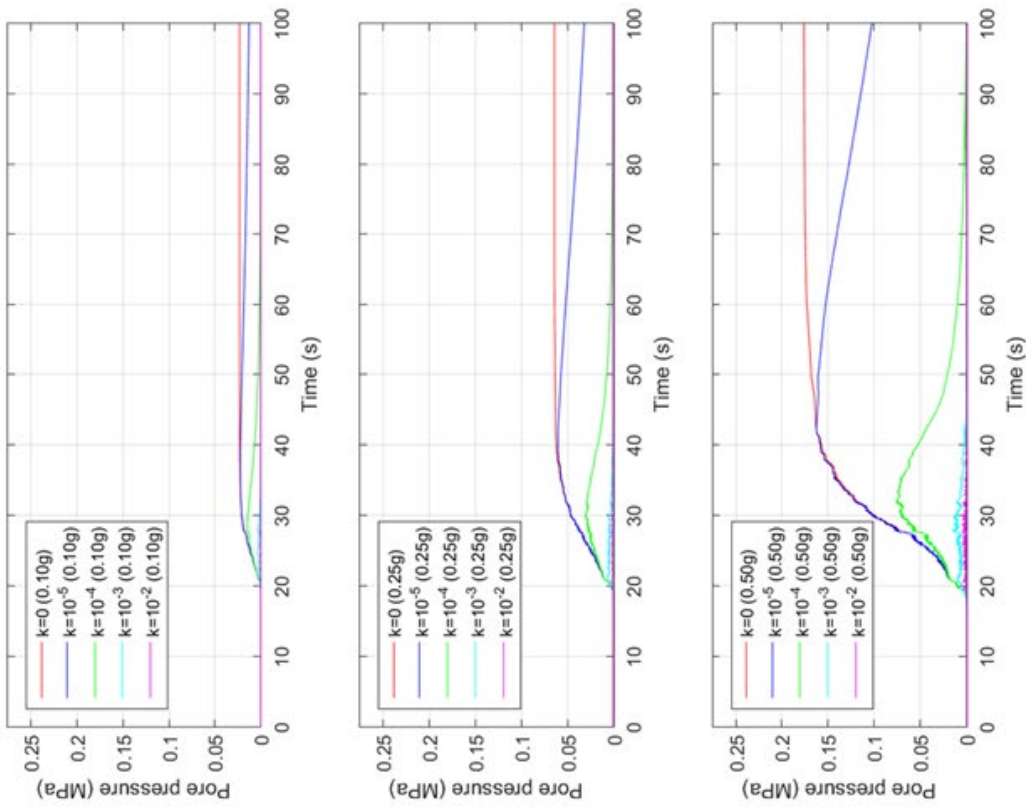


FIG. 40. Comparison of ground surface response spectra for effective and total stress analyses (left) and 1-Phase versus 2-Phase medium (right).

2-PHASE EFFECTIVE STRESSES  
Pore pressure



2-PHASE EFFECTIVE STRESSES

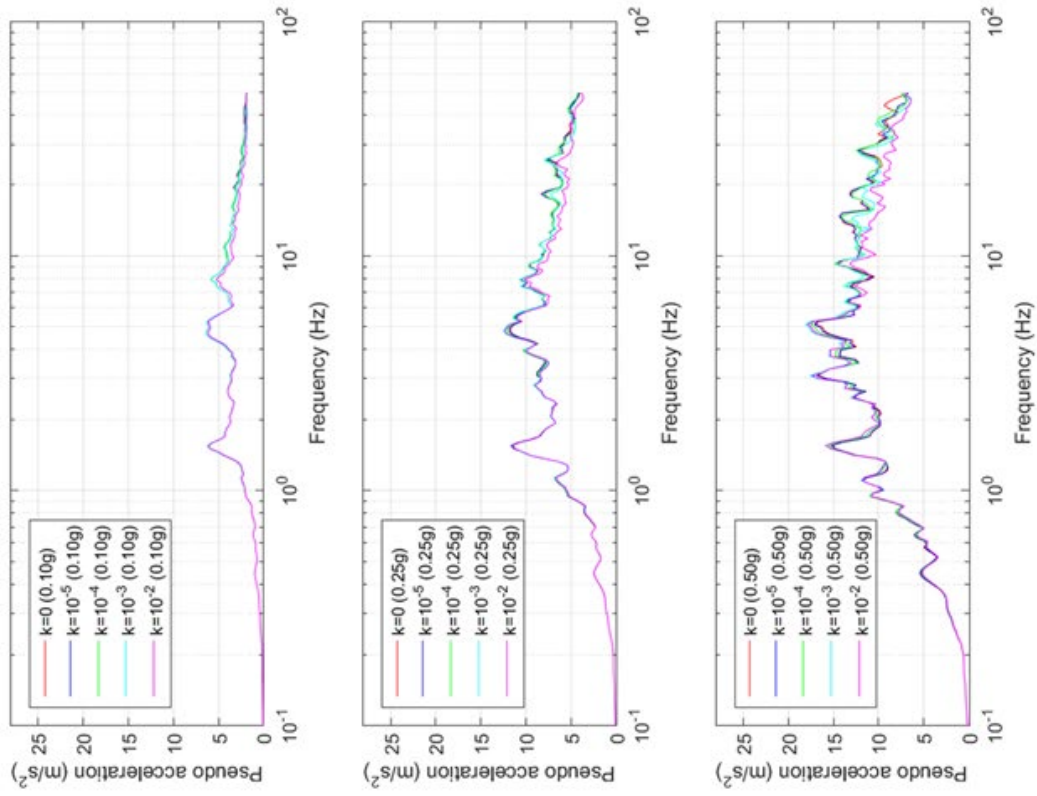


FIG. 4.1. Influence of soil permeability on pore pressure buildup (left) and ground surface response spectra (right).



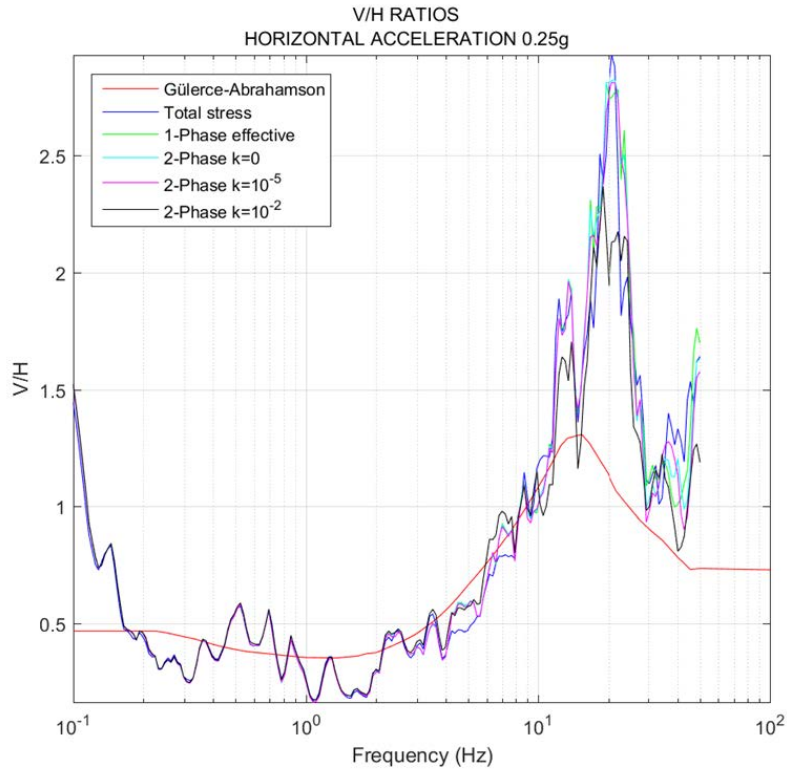


FIG. 42. Computed  $V/H$  ratios compared to statistical  $V/H$  ratios (red line).

Several methods are used for the calculations and illustrated on the rock column:

- (a) A pure elastic calculation with a time domain solution obtained with Dynaflo and two meshes: one with ten elements per wavelength (element size 5m) and one with 20 elements per wavelength (element size 2.5m) The differences between both meshes are shown to be negligible and only the results with 10 elements per wavelength are presented.
- (b) A pure elastic calculation in the frequency domain with the EWM developed by [61, 62] (see Section 4.4).
- (c) A viscoelastic calculation with 0.1% damping in the rock layer (the half-space is still undamped) with a frequency domain solution: classical FFT (SHAKE) and EWM.
- (d) A viscoelastic calculation with 1% damping in the rock layer (the half-space is still undamped) with a frequency domain solution: classical FFT (SHAKE) and EWM;
- (e) Numerical damping in the time domain analysis with Dynaflo (Newmark's parameter  $\gamma$  set equal to 0.55 instead of 0.50 for no numerical damping);
- (f) Rayleigh damping (stiffness proportional) in the time domain analysis calibrated to yield 1% damping at two times the fundamental frequency of the layer, i.e. 0.5Hz.

Results are presented in terms of 5% damped ground surface response spectra in Figure 43.

Surface motions are very sensitive to low damping values, in the range 0–0.1%. For the pure elastic calculation, either the time domain solution (without numerical damping) or the EWM is used; the agreement is good up to 25Hz; above that value they slightly differ but this may be due to filtering by the mesh in the time domain solution. For very lightly damped systems (0.1%), there is only one reliable method, the EWM. Damping cannot be controlled in the time domain solution and, in this case, the classical FFT overdamps the frequencies above 8Hz. For lightly damped systems (1%), the classical FFT and the EWM perform equally well; however, the EWM is much faster and does not require trailing zeroes to be added to the input motion.

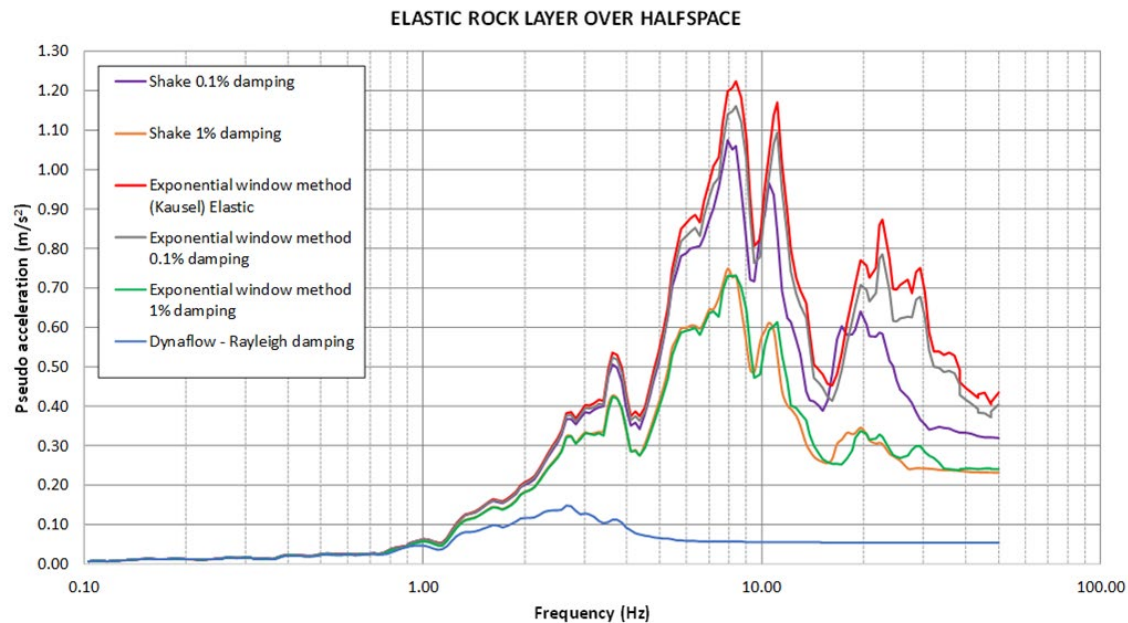


FIG. 43. Influence of damping modelling and numerical integration method on ground surface response spectra.

The duration of this quiet zone might be a cause of errors in FFT calculations if not properly chosen. Finally, Rayleigh damping is never to be used for damped systems in time domain analyses; it might even be better to rely on numerical damping, but the exact damping value implied by the choice of the Newmark integration parameter is not known because it is frequency dependent (proportional to frequency for the present analysis).

From a practical standpoint, soils or rock with very low damping represent a very critical situation because the exact amount of damping in very stiff rock (0%, 0.1%, 1%) will never be known (or measured) with sufficient accuracy and the results are very sensitive to this choice above 1Hz.

#### 9.4.2. 2-D models

This example is presented to outline the importance of topographic effect. Motions are calculated at the location in the middle of a valley (see Figure II.25 in Annex II) where a marked topography exists. Calculations are made assuming:

- (a) A 1-D model, extracted from the soil column at the examined location, and an equivalent linear constitutive model;
- (b) A 2-D model, including the whole valley shown in Figure II.25, with the strain compatible soil properties retrieved from the 1-D EQL analyses;
- (c) The same 2-D model as above but with a fully nonlinear constitutive model for the soil.

Ground surface response spectra calculated for these three assumptions are depicted in Figure 44.

The calculated surface spectra clearly indicate that the 1-D model is unable to predict the correct answer except for long periods, above 1.5s; at these periods, scattering of the incoming wave by the topography is insignificant. The main difference between the spectra arise from the geometric model rather than from the constitutive model, although the 2-D linear model needs to be treated with caution because damping is modelled as Rayleigh damping while in the other two analyses frequency independent damping is considered.

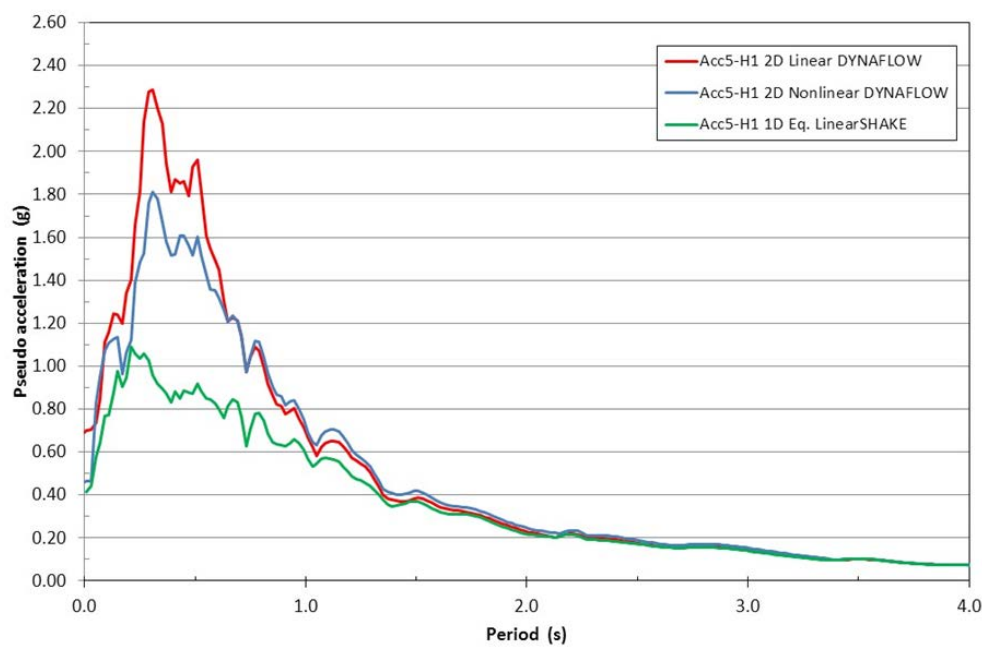


FIG. 44. Influence of surface topography on ground surface response spectra.

### 9.4.3. The 3 X 1C approach

Modelling 3 components of seismic motions can sometimes be done by modelling independently three components separately by using a 3 X 1C approach. This approach can be used if seismic wave lengths (body waves (P and S), surface waves (Rayleigh, Love, etc.) are much longer than the structure dimension. Reference [166] and Section 7.3.2.2 provide more details on this modelling approach.

### 9.4.4. Real 3C motions

Realistic three component (3C) seismic motions that are comprised of body and surface waves can be used for SSI analysis provided that the full 3C wave field of seismic motions are available. Such full wave fields can be obtained using the analytic solution for elastodynamic wave equations [304, 305], or using FEM or finite difference regional scale models, as described in [166].

## 9.5. SOIL-STRUCTURE INTERACTION MODELS

### 9.5.1. Structure

Decision-making on the types of models to be developed and used in SSI analyses are based on the following general considerations:

- (a) Determining the characteristics of a nuclear installation structure (identifying structures important to safety, such as structures housing safety related equipment) and large components for which SSI is important (see Section 8.1.2).
- (b) Determining the characteristics of the following foundation structures (Section 9.5.2):
  - (i) Conventional foundation/structure systems - surface founded and shallow embedment – embedment ratio (embedment depth/effective foundation radius) less than 0.5. The effective stiffness may be assumed as rigid, dependent on base mat stiffness reinforced

by connected structure elements, e.g., honey-combed shear walls anchored to the base mat. The effective stiffness may be assumed as flexible, e.g. if additional stiffening by the structure is not sufficient to assume rigid behaviour or for strip footings

- (ii) Deep foundation (piles);
  - (iii) Deeply embedded foundation/structure systems, e.g. a small modular reactor (SMR).
- (c) Determining the purposes of the SSI analysis and defining the use of results, for example:
- (i) Seismic response of nuclear installation structures for design or assessment (forces, moments, stresses or deformations, story drift, number of cycles of response for fatigue assessment or damage assessment);
  - (ii) Input to the seismic design, qualification, evaluation of subsystems supported in the structure (time histories of acceleration and displacement), ISRS, number and amplitude of cycles for components, etc.;
  - (iii) Base mat response for base mat design;
  - (iv) Soil pressures for base mat and embedded wall designs;
  - (v) SSSI analysis.

#### *9.5.1.1. Multi-step vs single step soil–structure interaction analyses*

In the multi-step method, the seismic response analysis is performed in successive steps. In the first step, the overall seismic responses (deformations, displacements, accelerations, and forces) of the soil-foundation-structure are calculated. The response obtained in this first step is then used as input to other models for subsequent analyses of various portions of the structure.

A detailed second step model that represents the structural configuration in adequate detail to develop the seismic responses necessary for the seismic design or assessment is needed. These subsequent analyses are performed to obtain:

- (a) Seismic loads and stresses for the design and evaluation of portions of a structure;
- (b) Seismic motions, such as accelerations, at various locations of the structural system, which can be used as input to seismic analyses of equipment and subsystems.

Typically, the structure model of the first step of the multi-step analysis represents the overall dynamic behaviour of the structural system but need not be refined to predict stresses in structural elements, e.g., an LMSM. Seismic responses include detailed stress distributions for structure design, including load combinations, and capacity evaluations for assessments. Also, detailed kinematic response, such as acceleration, velocity, and displacement time histories, and generated ISRS are usually needed.

The objectives of a single step analysis are identical to the multistep method, except that all seismic responses in a structural system are determined in a single analysis. The single step analysis is conducted using a detailed second step model introduced above.

In practice, the single step analysis is most often employed for structures supported on hard rock, with a justified fixed-base foundation condition for analysis purposes, and for structures whose dynamic behaviour is expected to be nonlinear.

#### *9.5.1.2. Structure modelling requirements*

Structure modelling need to address the following:

- (a) The model needs to accurately represent the overall dynamic behaviour of the structure.
- (b) A three dimensional model to analyse all three directions of earthquake ground motion needs to be developed (SSI analyses may be performed one direction at a time with the

results being combined appropriately at the analyses conclusion, provided phenomena, such as nonlinear behaviour, is not being modelled).

- (c) The structural mass (the total of structural elements, major components, and an appropriate portion of live load) needs to be lumped so that the total mass and the centre of gravity are preserved. Rotational inertia needs to be included if it affects response in the frequency range of interest.
- (d) The modelling of structural stiffness needs to take into account significant characteristics of the structure that affect stiffness and load paths, e.g., large floor cut-outs.
- (e) The modelling of structural stiffness needs to take into account local amplification, if expected, e.g. high frequency response (greater than 20 Hz);
- (f) The expected nonlinear behaviour at the level of excitation of the ground motion of interest needs to be modelled. This could be complete nonlinear FEM analysis of the structure or an approximate approach implementing reductions in shear stiffness and bending stiffness as a function of stress level as calculated in preliminary analyses [19].

In addition, for all models, LMSMs:

- (a) Need to be based on a sufficient number of nodal or dynamic degrees of freedom to represent significant structural modes up to structural natural frequencies of about 20 Hz in all directions (the intent of the LMSM is to represent the overall dynamic behaviour of the structure, which for nuclear installations is typically less than 20 Hz);
- (b) May be comprised of multiple sticks with appropriate connectivity at the base mat or at elevations in the structure;
- (c) Need to take into account torsional effects resulting from eccentricities between the centre of mass and the centre of rigidity at each elevation in the model;

A second analysis will typically be needed to generate all detailed response quantities of interest [19], Section 4.8.1.3.

### *9.5.1.3. Decision-making*

Based on the considerations itemized in Section 9.5, decisions as to the type and characteristics of structure models are to be made. Practical considerations also affect the decision; for example, the availability of software programs to model the phenomena judged to be important to the SSI response and expertise in their application (e.g., nonlinear behaviour of the structure when coupled with soil and foundation modelling). Uncertainties are discussed in Section 9.7.

In general, 3-D structure models are needed. However, there are situations where this is not necessary, for example, for very long structures where a judgement is made that the structure (and soil) or component (e.g. an above grade pipeline) may be modelled in two dimensions (horizontal direction perpendicular to the structures length and the vertical direction).

## **9.5.2. Foundations**

Foundation modelling is separated into conventional foundation/structure systems (surface founded and shallow embedded), deep foundation (piles), and deeply embedded foundation/structure systems.

### *9.5.2.1. Foundation modelling for conventional foundation/structure systems*

Modelling of surface foundations in a global direct SSI analysis, or using a substructure method, does not pose any difficulty provided that the analyses are carried out with the same software for all individual steps. Software such as PLAXIS, ABAQUS, GEFDYN, DYNAFLOW,

SASSI, MISS3D and Real ESSI, can take into account foundations with any stiffness. In a conventional substructure method, however, the usual assumption is to consider the foundation as infinitely stiff in order to define the foundation impedances and the foundation input motion. The question then arises of the validity of this assumption, which depends on the relative stiffness of the foundation and of the underlying soil. Stiffness ratios  $SR_v$ , for the vertical and rocking modes, and  $SR_h$  for the horizontal and torsional modes can be introduced to resolve this. These stiffness ratios depend on the foundation's characteristics (axial stiffness  $E_b S_b$  in kN/ml or bending stiffness  $E_b I_b$  in kN.m<sup>2</sup>/ml) and on the soil shear modulus  $G$  or Young's modulus  $E$ . For a circular foundation with diameter  $B$  these stiffness ratios are given by:

$$SR_v = \frac{1}{B} \sqrt[4]{\frac{E_b I_b B}{E}} \quad , \quad SR_h = \frac{1}{B} \sqrt{\frac{E_b S_b B}{G}} \quad (24)$$

The foundation can be assumed stiff with respect to the soil when:

- (i)  $SR_v > 1$  for vertical and rocking modes;
- (ii)  $SR_h > 5$  for horizontal and torsion modes.

Usually, the condition on  $SR_h$  is always satisfied. For nuclear installation buildings with numerous shear walls, the condition on  $SR_v$  is also satisfied considering the stiffening effect of the walls; for moment resisting frame buildings with a mat foundation, the condition on  $SR_v$  is hardly satisfied and either a complete analysis taking into account the raft flexibility will be carried out, or the stiffness of fictitious rigid foundations around the columns will be computed and specified at each column base, assuming that no coupling exists between the individual footings.

#### 9.5.2.2. Simplified models for conventional foundation/structure systems

The impedance functions are then introduced in the structural model as springs  $K_r$  (real part of the impedance function) and dashpot  $C$  related to the imaginary part  $K_i$  of the impedance function. Alternatively, the damping ratio of each SSI mode can be computed as:

$$\beta = \frac{K_i}{2K_r} = \frac{\omega C}{2K_r} \quad (25)$$

The usual practice is to limit the damping ratio to 30%, but higher values are allowed in some cases, if properly justified. The main difficulty with the impedance functions is their dependence on frequency, which cannot be easily be considered in time domain analyses or modal spectral analyses. Several possibilities exist to approximately take the frequency dependence into account:

- (a) To implement an iterative process which, for each SSI mode, determines the stiffness compatible with the frequency of the corresponding undamped SSI mode. The SSI mode can be identified as the mode with the maximum strain energy stored in the spring.
- (b) To develop a rheological model which takes into account the frequency dependence by addition of masses connected to the foundation with springs and dashpots [306 – 308]. The parameters of the rheological model are simply determined by curve fitting of the model response to the impedance function.

When the soil profile becomes significantly layered with sharp contrasts in rigidity between layers, the impedances functions become jagged and either of the two procedures described above may become difficult to implement; the only possibility is then to resort to frequency domain solutions.

If surface foundations exhibit two principal axes of symmetry, the coupling term between horizontal translation and rocking around the transverse horizontal axis may be neglected; this is not true for embedded foundations. In the first instance, the impedance matrix is diagonal and springs and dashpots can be assigned independently to each degree of freedom; in the second one, the impedance matrix contains off-diagonal terms, which makes the rheological model more tricky to develop: if the software does offer the possibility of adding a full stiffness matrix to the foundation, alternatives may consist in connecting the spring at a distance  $h$  from the foundation with a rigid beam element.

#### *9.5.2.3. Limitations of the substructure method*

The substructure method, on which the concept of foundation impedances is based, assumes linearity of the system. However, it is well recognized that this is a strong assumption, since non-linearities are present in the soil itself (Section 4.2) and at the soil foundation interface (sliding, uplift, Section 8.4.6). Soil non-linearities may be partly accounted for by choosing, for the calculation of the impedance matrix, reduced values of the soil properties that reflect the soil nonlinear behaviour in the free field (Section 8.4.4). This implicitly assumes that additional nonlinearities taking place at the soil foundation interface do not affect significantly the overall seismic response.

#### *9.5.2.4. Deep foundations*

As opposed to shallow, or slightly embedded, foundations, modelling of piles foundations is a more complex task because it usually involves a large number of piles and soft soil layers; a direct (3-D) analysis becomes demanding, especially for nonlinear solutions. The substructure method, in which the piles and the soil are represented through impedance functions, becomes more attractive provided the system can be assumed to remain linear; however, as opposed to shallow foundations, the impedance matrix always contains off-diagonal terms representing the coupling between the horizontal translation and rocking.

Another modelling concept has been widely used for piles foundations: the so-called Winkler models, based on the concept of (linear or nonlinear) springs and dashpots to model the effect of the soil on the piles. The springs and dashpots, distributed along the pile shaft, represent the interaction with the soil. Although conceptually the soil reaction forces are still represented by the action of springs and dashpots, unlike for the impedance matrix approach, there is no rational or scientifically sound method for the definition of these springs and dashpots. They are usually based on standards or field experiments under static conditions. Their values, but more importantly their distribution along the pile, vary with frequency; there is no unique distribution reproducing the global foundation stiffness for all degrees of freedom. Furthermore, two additional difficulties arise for piles foundations:

- (a) The choice of the springs and dashpots needs to reflect the pile group effect;
- (b) As the seismic motion varies with depth, different input motions need to be defined at all nodes shared between the piles and the soil; a separate analysis is needed for the determination of these input motions.

In view of all the uncertainties underlying the choice of their parameters, global Winkler-type models, although attractive because nonlinearities between the shaft and the soil can be approximated, are not favoured. The substructure method, with its limitations described below, is preferred or a full 3-D nonlinear model is used.

Modelling of pile foundations in a substructure method raises the following issues:

- Can full contact or full separation between the pile cap and the soil be assumed?

- Do the piles need to be considered clamped or hinged in the pile cap?

There is no definite answer to each of these questions and the situation is very likely to evolve during earthquake shaking. As full consideration of this evolution is incompatible with the substructure method which assumes linearity of the analysed system, only approximate solutions can be handled.

With regard to the contact between pile cap and soil, the contact condition may evolve during the lifetime of the nuclear installation structure due to settlement of the soft layers caused by consolidation of clayey strata, or by construction around the existing structure. It may also evolve during an earthquake due to soil compaction.

With regard to fixity at the pile cap connection, during seismic loading the connection may deteriorate and evolve, due to reduction of the connection stiffness, from perfectly clamped piles to a condition where a plastic hinge forms at the connection.

#### 9.5.2.5. *Embedded foundation*

The implementation of the substructure method for an embedded foundation is described in Section 8.3.2. The structural model used for this purpose is the deeply embedded SMR model, described in Section 9.5.2, in which the soil profile is identical to the one used for the 1-D SRA with the same input motion scaled to 0.25g, for the EQL analysis to be valid. First, the SRA is run to determine:

- The surface motion, which serves as input motion in the following steps;
- The strain compatible soil properties (shear modulus and damping ratio) used in the SSI analyses carried out with SASSI.

The site response analysis only provides the (strain compatible) shear modulus (or shear wave velocity) and not the bulk modulus (or P–wave velocity); however, both parameters are needed for a 2-D or 3-D SSI analysis. The reduction factor to be applied to the P–wave velocity be not taken equal to the reduction factor calculated for the S–wave velocity. This would be equivalent to assuming that Poisson's ratio remains constant regardless of the strain amplitude; this assumption is obviously false. The parameter which is the most likely to remain constant is the soil bulk modulus  $B$ ; in total stress analyses, like those performed with SHAKE for site response analyses, this assumption is true in saturated soils and not extremely stiff soils (like hard rock), because  $B$  is practically equal to the water bulk modulus. For unsaturated soils, this assumption is only approximate but is still reasonable and probably represents the most realistic one. With this assumption,  $V_P$  be calculated as follows:

- Calculation of the bulk modulus from the elastic S and P wave velocities  $V_{Pe}$  and  $V_{Se}$

$$B = \rho \left( V_{Pe}^2 - \frac{4}{3} V_{Se}^2 \right) \quad (26)$$

Where  $\rho$  is the soil mass density,

- Calculation of the strain compatible S–wave velocity,  $V_S$ , from the site response analysis,
- Calculation of the strain compatible P–wave velocity according to:

$$V_P = \sqrt{\frac{B}{\rho} + \frac{4}{3} V_S^2} \quad (27)$$

Note that use of eq. (4) may lead to a high Poisson's ratio, which may create numerical issues; in such cases, it is necessary to limit the ratio to a maximum value compatible with the numerical software used for the SSI analysis; typically, it ranges from 0.45 to 0.49.



With respect to the damping ratio associated with  $V_p$ , it is best to assume the same value as for  $V_s$ .

With these soil properties, three embedments for the structure are analysed: surface foundation (no embedment), 14m and 36m. Two different types of analyses are run: one with a massless structure; and one with real structure. The former analyses provide the kinematic interaction motion (see Section 8.3.2) and the latter one the global response including kinematic and inertial interaction. The most salient features of the response of each model are illustrated in Figures 45, 46 and 47.

Figure 45 compares the 5% damped response spectra on top of the structure (roof elevation) for the global response; the free field ground surface response spectrum is also shown. As expected, for the surface structure and the shallow embedded structure, the roof spectra show a marked amplification which corresponds to the fundamental SSI frequency; as the embedment increases from 0 to 14m, the peak is shifted towards higher frequencies because the stiffness of the foundation is increased. As opposed to the two previous cases, the deeply embedded structure does not show any marked amplification at a given frequency; furthermore, the spectrum is not very different from the surface motion. The structure motion is imposed by the soil displacements rather than the inertia of the structure. This behaviour is typical of underground structures.

Figures 46 and 47 present the foundation input motions (base of the structure) due to pure kinematic interaction (massless structure).

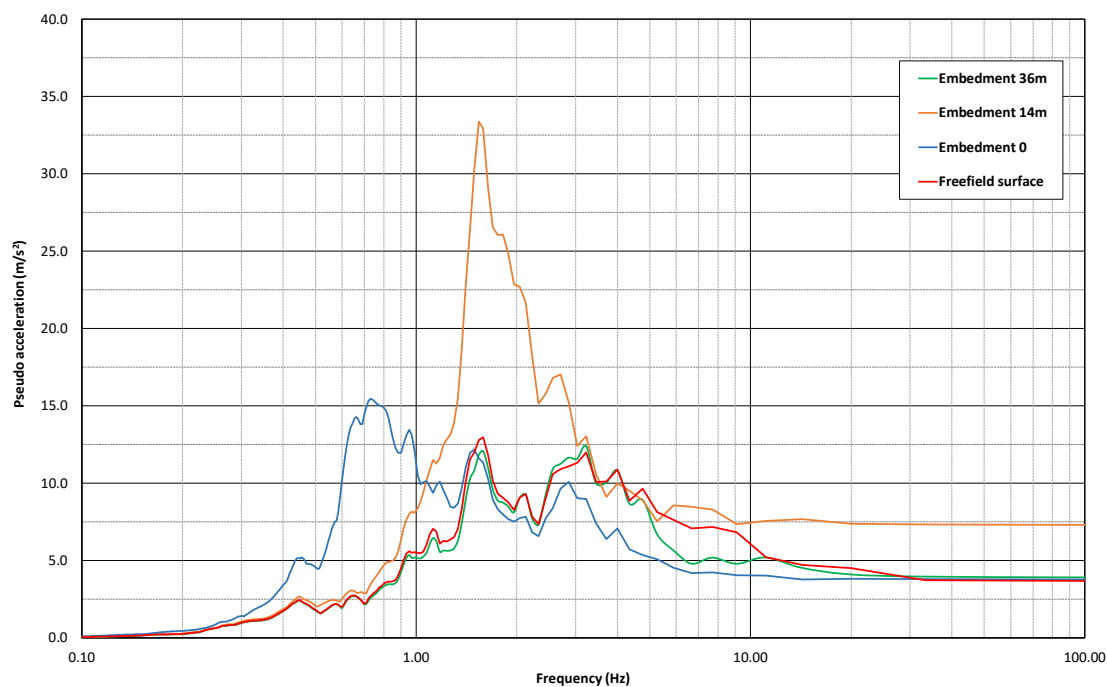


FIG. 45. 5% damped response spectra at roof elevation for three embedments.

When the structure is embedded in the ground, the foundation input motion can no longer be taken equal to the free field motion. This phenomenon is referred to as kinematic interaction; the calculated motion at the base of the structure is the foundation input motions that are used in the substructure method. Although the incoming motion consists of plane, embedment creates a rotational component of motion at the foundation and vertically propagating shear waves, and the soil profile is uniform in the horizontal direction.

For the surface foundation, kinematic interaction is totally negligible, and the rotational component of motion is nil. This is only the case for the assumptions made in the analyses, i.e. a horizontally layered soil profile subjected to vertically propagating body waves. The increase in the structure embedment has two effects:

- (a) The foundation response spectrum decreases, at all frequencies in this case, when the embedment increases;
- (b) The rotational component of motion increases, at all frequencies in this case, when the embedment increases.

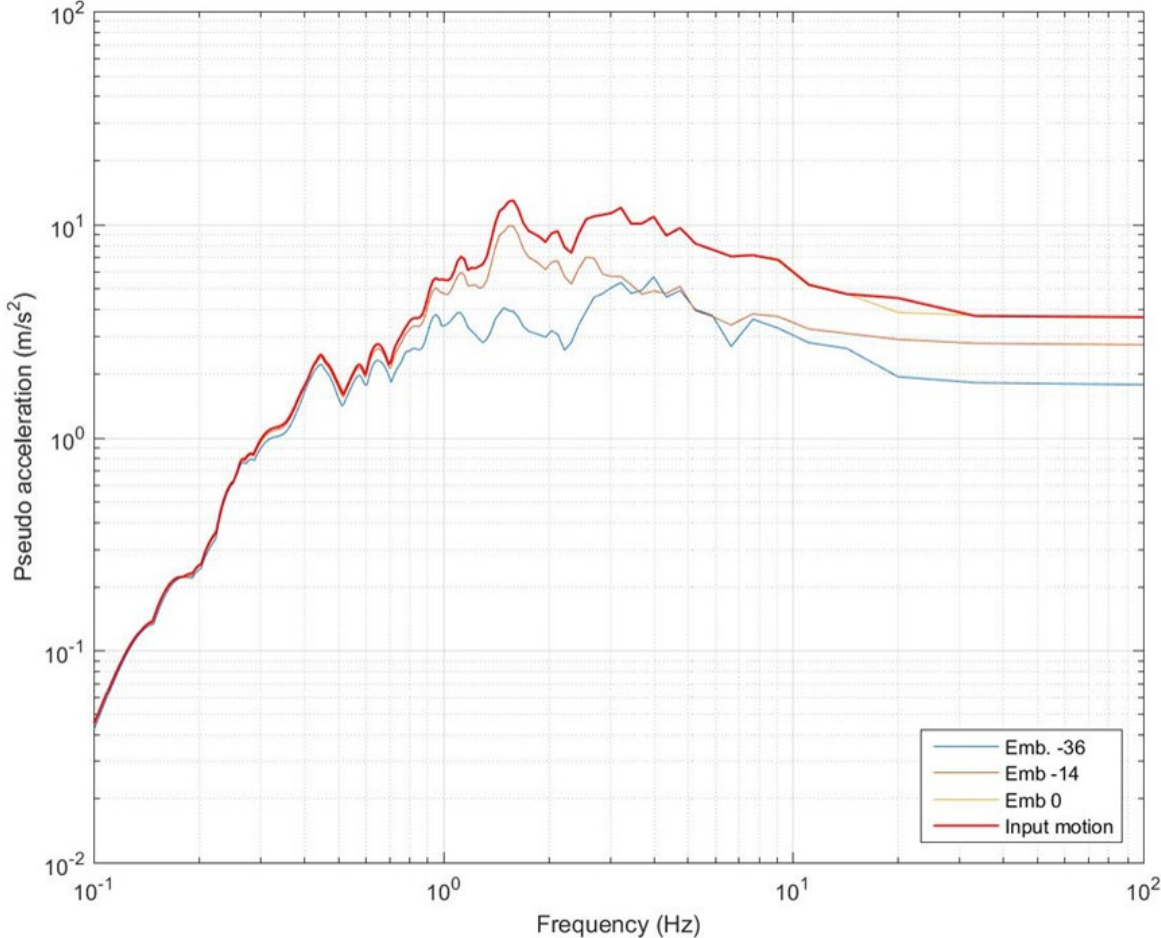


FIG. 46. Kinematic translational response spectra.

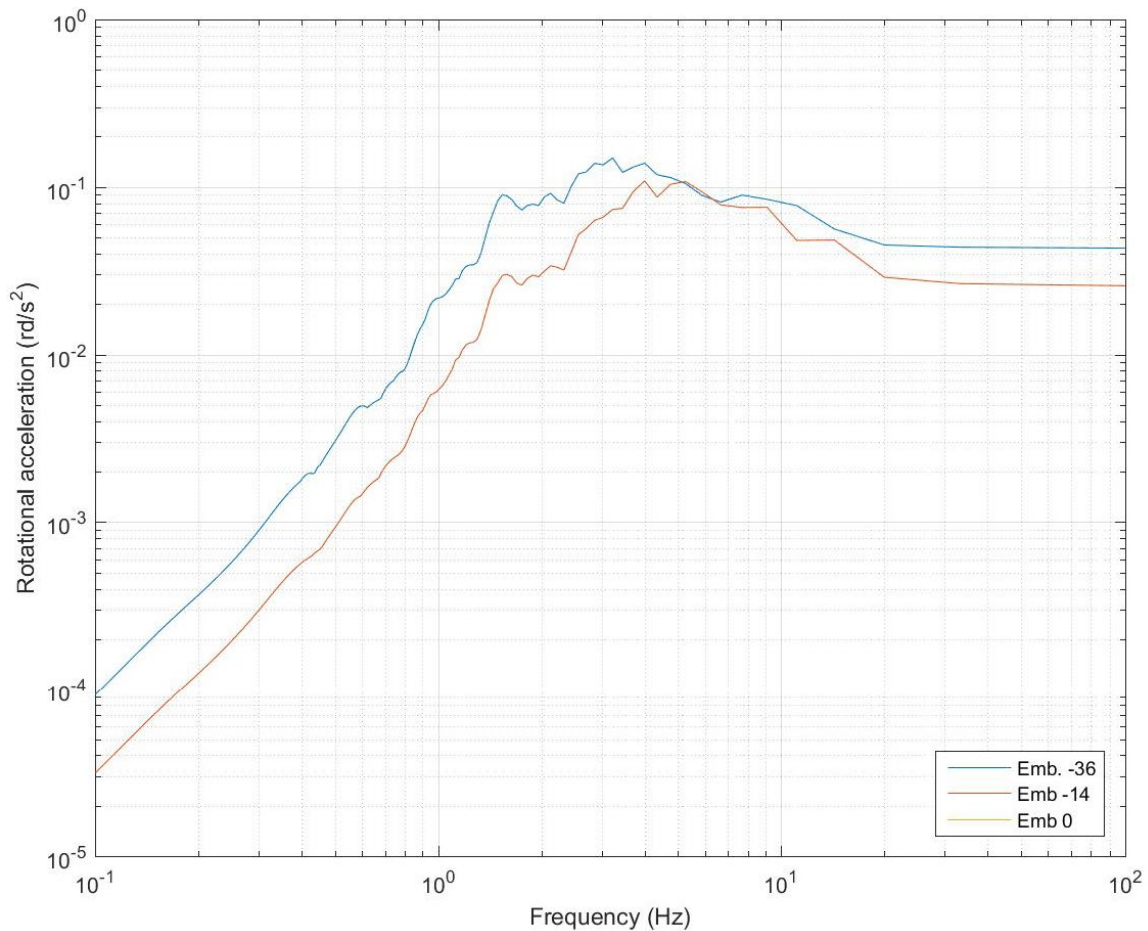


FIG. 47. Kinematic rotational response spectra.

#### 9.5.2.6. Deeply embedded foundations (SMRs)

In contrast to shallow or deep foundations, modelling and analysis of deeply embedded structures (e.g. some SMRs), are more easily achieved in a global direct time domain or frequency domain analysis. Unless the whole SSI analysis is run within a single software (like SASSI or CLASSI)<sup>20</sup>, the conventional substructure method is not well adapted, (although still theoretically possible under the assumption of linear behaviour) because of the large embedment. The embedment creates a strong kinematic interaction between the soil and the deeply embedded SMR structure, which significantly alters the free field motion and develops pressures on the lateral walls. The calculation of these two effects in a conventional substructure method is complicated and tedious:

- (a) Kinematic interaction motion, i.e., the effective foundation input motion, needs to be calculated from a model reflecting the embedment and variation of the free field motion with depth. Furthermore, the true effective input motion contains a rocking component which is not easily applied to the structural model.
- (b) There is no simple means for evaluating the earth pressures on the outside walls. Classical solutions, like the Mononobe and Okabe solution, are not valid for deeply embedded retaining structures that cannot develop an active pressure condition. Furthermore, earth

<sup>20</sup> It is recalled that SASSI uses a substructure method, but the same software and model are used for the analysis of the soil–foundation substructure and of the structure. Conventional substructure methods calculate the impedance matrix, simplify it with frequency independent springs and dashpots to be connected to the structural model, which is analysed with a different software (see Section 10).

pressures and inertia force are likely to be out of phase, without any simple solution to easily define the phase shift between both.

A rigorous consideration of these two factors, involves a global FEM model of the embedded part of the structure; the additional effort to include the structural model is then minimal.

### **9.5.3. Analysis methods**

The dynamic SSI analysis can be classified as substructure methods and direct methods.

#### *9.5.3.1. Substructure methods*

Substructure methods are only valid provided a linear elastic behaviour of all components can be assumed. Therefore, the first task before choosing the analysis method, between a direct method and a substructure method, is to assess the importance of this aspect. However, slight nonlinearities in the soil behaviour can be accepted in the substructure method and considered, at least in an approximate manner. Reduced soil characteristics can be used in the model; these reduced characteristics represent the strain compatible properties and reflect the soil nonlinearities in the free field. They are usually calculated from a (1-D or 2-D) SRA (see Section 9.4). It is further assumed in the substructure method that additional nonlinearities that develop due to the interaction between the nuclear installation structure and the soil have a second order effect on the overall response; however, they may impact the local response, like for the soil pressures developing along a pile shaft.

The substructure method has been described in Section 8.3 and the successive steps in the approach are illustrated in the flow chart of Figure. 48; the flowchart, with reference to the box numbers in brackets, is detailed below.

Two examples in the appendices illustrate some of the Steps (5) [309, 43, 53], Steps (10), (12) and (13) [51, 55] listed in the flowchart: one example is for an embedded structure and the second one for a piled foundation. They both refer to the conventional substructure method in which impedances are calculated in a first step and introduced in a structural model.

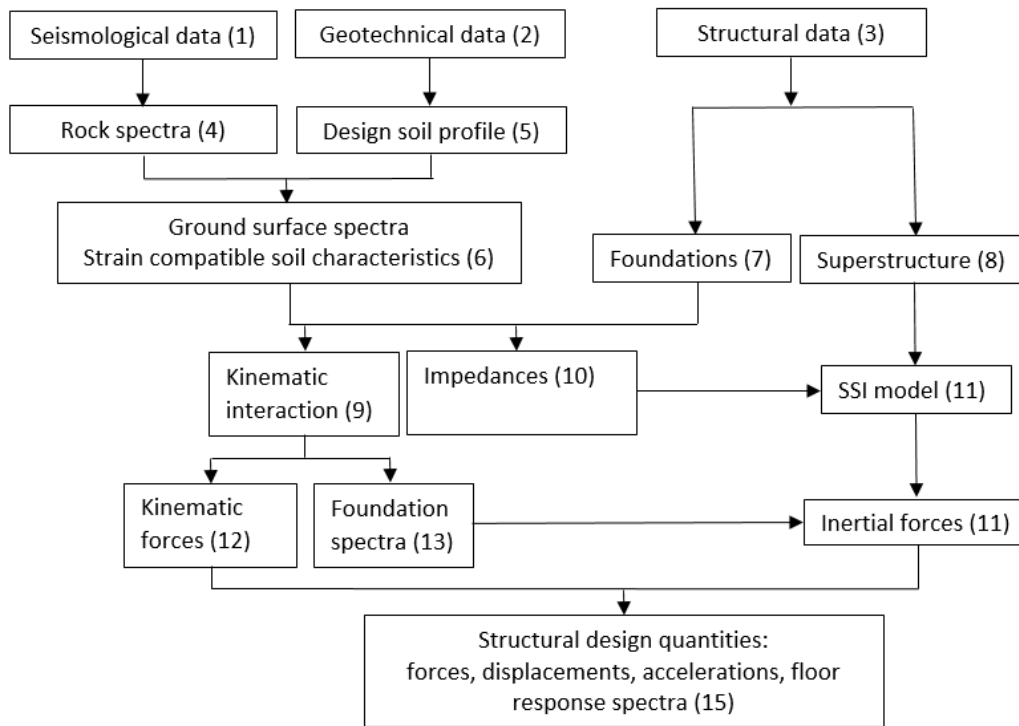


FIG. 48. The implementation of the substructure method.

One example on a deeply embedded structure is developed along the lines of the substructure method but with all the steps of the SSI analysis run with the same software (SASSI, see Section 10), thereby overcoming some of the simplifying assumptions of the conventional substructure method.

The first step of the analysis starts with the SRA to calculate the strain compatible soil characteristics and ground surface response spectra. The input data for this step are:

- (a) The geotechnical data [30] from which a design profile and a constitutive model are chosen for the site (Section 4);
- (b) The seismological data [28] from which the rock spectra (Step (4)) are established either from a probabilistic or a deterministic SHA (Section 7.4). Time histories representing the rock motion need to be defined following one of the procedures described in Section 7.5.

With these data, SRA provide the ground surface motion and the strain compatible soil characteristics [309]. Usually, they are run assuming an equivalent linear constitutive model as illustrated in the examples on embedded foundation and piles foundation in Annex III. Although nonlinear analyses are also possible, the choice of the strain compatible soil properties is less straightforward in this case and involves some judgment.

The second step corresponds to the top right boxes of the flowchart: from the formwork drawings (Step (3)), it establishes the structural model [46] and the foundation model [52]. As noted in Section 9.5, the foundation model for the shallow embedded foundation is assumed to correspond to a stiff foundation; Annex III gathers the piles and the surrounding soil, modelled as continuum media.

With the foundation model and the strain compatible soil characteristics, an impedance matrix is calculated [53]. The impedance matrix may be used in its full frequency-dependency and complex values or may be simplified to frequency independent stiffness and damping values. In the former case, an example is CLASSI; in the latter case, a conventional dynamic analysis program could be used. Section 9.5 presents two possible alternatives to define the frequency independent impedance matrix. This step produces the SSI model [54].

The same foundation model and the surface ground motion are used to calculate the kinematic response of the foundation [52]; this kinematic response is composed of the foundation input motion [47] and of the kinematic forces developed in the foundation [49].

The foundation input motion serves as the input to the structural model from which the inertial components of the response are retrieved (Step (14) [51]).

Finally, the results from the inertial response and from the kinematic response are combined [55] to yield the structural design quantities: forces, accelerations, displacements, and floor response spectra. If the kinematic response quantities  $R_K$  and the inertial response quantities  $R_I$  are obtained from time history analyses (in time or frequency domains) there is no difficulty in combining, at each time step, their contributions. The total response quantity at any time is given by:

$$R_T(t) = \pm R_I(t) \pm R_K(t) \quad (28)$$

However, in most cases the response quantities are not known as a function of time, and only the maximum inertial response quantities are retrieved from the SSI analyses (for instance when a modal spectral analysis is used). To combine both components, each of them be alternatively considered as the main action and weighted with a factor 1.0, while the other one is the accompanying action and weighted with a factor  $\lambda$ :

$$R_T = \pm \max R_I(t) \pm \lambda \max R_K(t) \text{ or } R_T = \pm \lambda \max R_I(t) \pm \max R_K(t) \quad (29)$$

The coefficient  $\lambda$  depends on how close to each other are the main frequencies leading to the maximum kinematic response quantity and the main frequency leading to the maximum inertial response quantity. The first one is controlled by the SSI mode and the second one by the soil response (fundamental frequency of the soil column). If these two frequencies are well separated, for example by 20%, both maxima are uncorrelated in time and their maximum values can be added with the SRSS rule:

$$R_T = \sqrt{|\max R_I(t)|^2 + |\max R_K(t)|^2} \quad (30)$$

If both frequencies are within 20% of each other, it is reasonable to assume that both phenomena are correlated, and the kinematic and inertial response quantities can be added algebraically:

$$R_T = |\max R_I(t)| + |\max R_K(t)| \quad (31)$$

### 9.5.3.2. Direct methods: linear, nonlinear (deeply embedded SMR, deep foundations, sliding, uplift)

The direct method analyses the idealized soil–structure system in a single step. The direct method is applicable to linear and equivalent linear idealizations and is needed for nonlinear SSI analyses. This contrasts with the substructure method that divides the SSI problem into a series of simpler problems, solves each independently, and superposes the results. The substructure method is limited to linear and equivalent linear idealizations since it relies on superposition. The direct method comprises of discrete methods and form basis for both FEM and the FDM. Linear finite element methods and nonlinear finite element methods are discussed in detail. Details are provided in Section 8.2.

#### 9.5.4. Hierarchical modelling and simulation of an inelastic soil–structure interaction system

Figure 49 shows an example of step by step, hierarchical modelling and simulation of an inelastic SSI system.

A soil–structure system is modelled in phases: each phase starts with a linear elastic material model. Material modelling is then slowly made more detailed for each component (soil, contacts, structural components, isolators/ dissipaters, etc.). The geometry of the model starts with a simple 1-D, free field soil column, with 1C motions, earthquakes and/or wavelets, being propagated. The same 1C motions are then propagated through a full 3-D free field model. The foundation is then considered, with expectations that motions at the top of foundation will be similar to the free field motions at the same location. The fixed base structure is analysed for natural modes (eigen modes) and natural frequencies. The fixed base structure is then shaken with earthquake motions derived from a 1C free field study, then with actual 1C earthquake motions (no SSI effects) and with wavelets, that will emphasize different dynamical behaviour [310].

#### 9.6. INCOHERENT MOTIONS

Seismic motion incoherence is a phenomenon that results in spatial variation of ground motion over large and small distances. Section 6.4 provides a discussion of the sources of incoherence of ground motions (i.e. complex source mechanisms producing complex wave fields), attenuation with distance, wave passage effects, and scattering effects. In the context of nuclear installations, in particular NPPs, wave passage effects and scattering effects have been revisited over several years to understand their potential impact on seismic input and structure response of NPP structures. Ground motion incoherence is horizontal spatial variation of ground motion, which is one aspect of SVGM.

GMI occurs due to:

- (a) Random spatial variation, defined as scattering of waves due to the heterogeneous nature of the soil or rock at the locations of interest and along the propagation paths of the incident wave fields (local wave scattering);
- (b) Wave passage effects, defined as systematic spatial variation due to difference in arrival times of seismic waves across a foundation.

Coherency functions that express random spatial variation as a function of frequency and separation distance have recently been developed by Abrahamson [144] and they are discussed in detail in Section 6.4. Wave passage effects have been shown to have minimal effects on the response of NPP structures mainly based on the range of apparent wave velocities (2 – 4 km/s) derived from the recorded data used to establish the coherency functions.

The treatment of the effects of GMI or SVGM on structure response for typical NPP structures was motivated in part by the development of UHRS with significant high frequency content, i.e. frequencies greater than 20 Hz. Efforts to evaluate the existence and treatment of GMI for conditions applicable to NPP foundations and structures were a combined effort of ground motion investigations and evaluation of the impact of implementing GMI effects on the seismic response of typical NPP structures.

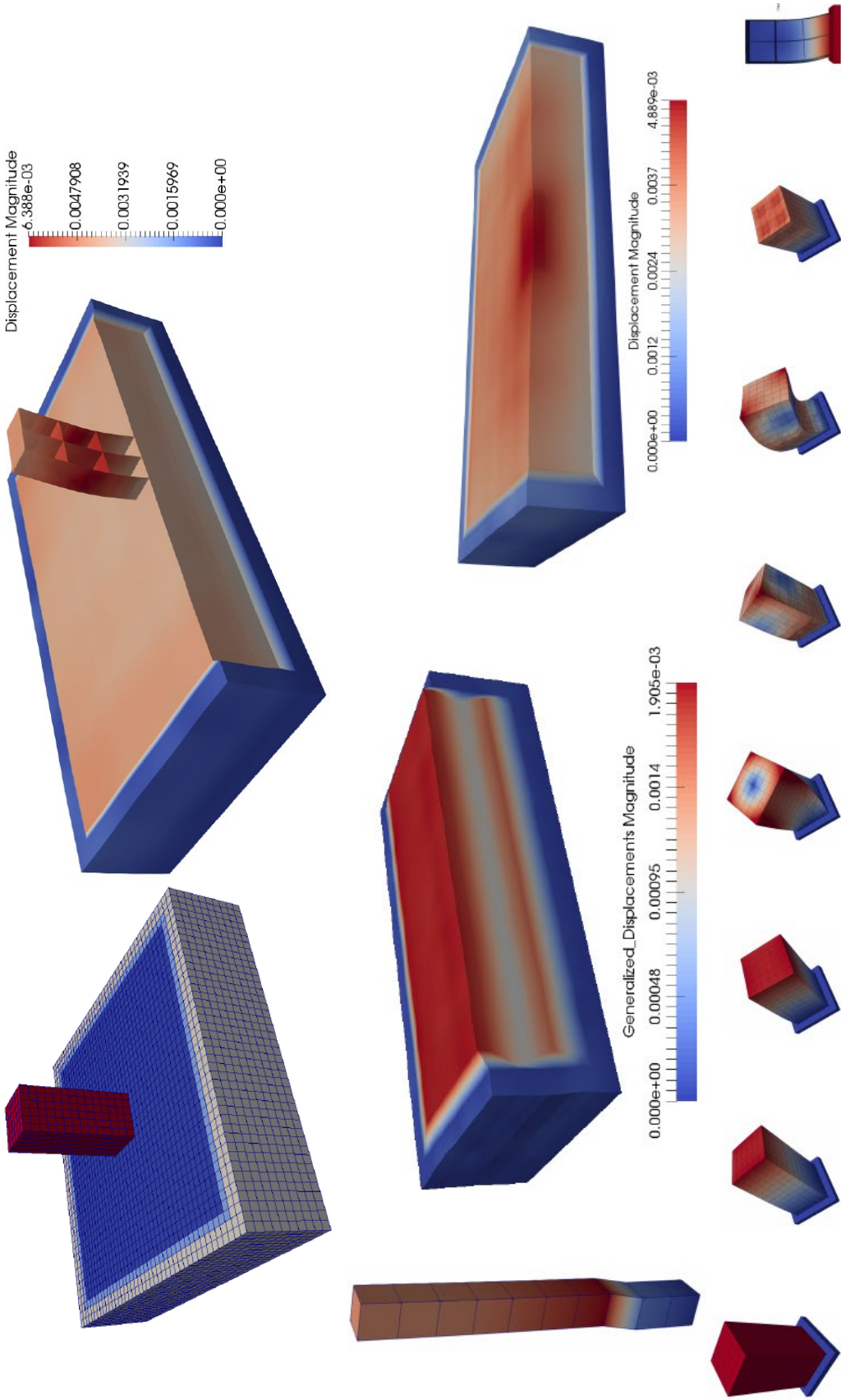


FIG. 49. Step by step, hierarchical modelling and simulation of an inelastic SSI system, modelling phases and model components.



Random spatial variation of ground motion can result in large reductions in foundation motion. Wave passage effects are typically not considered as it produces minimal further reductions, and it involves assignment of an appropriate apparent wave velocity that may be difficult to justify.

The resulting ground motion coherency functions as a function of frequency and distance between observation points were initially generated considering all data regardless of site conditions, earthquake characteristics, and other factors. Abrahamson [144] refined this initial effort to separate soil and rock sites. Plots of soil and hard rock ground motion coherency functions are shown for horizontal and vertical ground motion components in Section 6.4.

In general, implementing GMI into seismic response analyses has the effect of reducing translational components of excitation at frequencies above about 10 Hz, while simultaneously adding induced rotational input motions (induced rocking from vertical GMI effects and increased torsion from horizontal GMI effects). Significant reductions in ISRS in progressively higher frequency ranges can be observed.

### **9.6.1. Case study of the effects of ground motion incoherence on a nuclear power plant**

A case study is presented in Annex I that assesses the effects of GMI on an NPP structure.

The conclusions of this study were:

- (1) SSI (inertial and kinematic interaction) have been demonstrated to be important phenomena to take into account for generating seismic demand for design and evaluation purposes of nuclear installations. Even for hard rock sites ( $V_s = 2000$  m/s), inertial interaction contributes to determining rock-structure natural frequencies. Kinematic interaction effects (local wave scattering effects (i.e., GMI) can significantly affect the seismic demand for structures, systems, and components for design and evaluation purposes;
- (2) The effect of GMI on seismic response is a function of the ground motion frequency content, the site properties, and the nuclear installation structure;
- (3) In general, GMI has minimal effects on seismic responses in frequency ranges less than 10Hz. This is verified in the case study for broad-banded artificial time histories and recorded ground motions of high frequency, low frequency, and a mixture of the two. Coherent and incoherent SSI responses are comparable in the frequency range less than 10Hz;
- (4) In many instances, GMI has a more significant effect on vertical responses than on horizontal responses. Nuclear installation structures often have multiple vertical modes with frequencies greater than 10 Hz, each having similar degrees of importance as measured by vertical modal mass. In contrast, important horizontal modes in nuclear installation structures have frequencies less than 10 Hz. In two ways, vertical responses are more likely to be affected by GMI than horizontal responses – multiple modes with frequencies greater than 10 Hz and multiple modes with relatively close modal mass;
- (5) The importance of GMI appears to be consistently the case for nodes at varying elevations;
- (6) GMI can be important for soil sites as well as rock. However, for soil sites, the frequencies of interest of the soil–structure system in the horizontal directions are typically less than 10 Hz and often much less than 10 Hz. Thereby, GMI being a phenomenon of frequencies greater than 10Hz lacks relevance. One exception is vertical modes that may be more local than overall modes and may be affected by GMI;
- (7) The importance of GMI appears to be consistently the case for nodes at varying elevations;
- (8) GMI can be important for soil sites as well as rock. However, for soil sites, the frequencies of interest of the soil–structure system in the horizontal directions are typically less than 10 Hz and often much less than 10 Hz. Thereby, GMI being phenomena of frequencies greater

than 10 Hz lacks relevance. One exception is vertical modes that may be more local than overall modes and may be affected by GMI.

Table 8 presents seismic analyses performed for coherent and incoherent ground motion assumptions.

TABLE 8: SEISMIC ANALYSES PERFORMED FOR COHERENT AND INCOHERENT GROUND MOTION ASSUMPTIONS

	Landers	Imperial Valley	Val-des-Bois	RG 1.60	RG 1.60 enhanced	Analyses
Fixed base	X	X	X	X	X	Coherent
Rock site	X	X	X	X	X	Coherent and incoherent
Soil site	X	X	X	X	X	Coherent and incoherent
Total number of analyses	5	5	5	5	5	

### 9.7. UNCERTAINTIES AND SENSITIVITY STUDIES

Uncertainties exist in the definition of all elements of soil–structure interaction phenomena and their analyses. Randomness is considered to be associated with variability that cannot practically be reduced by further study, such as the earthquake source location and type of faulting, source-to-site wave travel path and earthquake time histories occurring at the site in each direction. Modelling uncertainty is generally considered to be variability associated with a lack of knowledge that could be reduced with additional information, data, or models.

In many cases, uncertainties can be explicitly represented by probability distributions of SSI analysis parameters, e.g. soil material properties and structure dynamic properties. In other cases, uncertainties in SSI analysis elements may need to be assessed by sensitivity studies. The results of sensitivity studies are an input to model decision-making or to provide alternative credible values that need to be considered in the design or assessment of a nuclear installation by combining the weighted results. The analyst determines the important elements to the SSI analysis results and include them appropriately.

Some issues amenable to modelling and sensitivity studies to determine their importance to SSI response are discussed in Section 3.3.3.

In general, uncertainties are categorized into aleatory uncertainties and epistemic uncertainties (see Section 3).

Aleatory uncertainties and epistemic uncertainties are often represented by probability distributions assigned to SSI parameters. These probability distributions are typically assumed to be non-negative distributions (e.g. Lognormal and Weibull.). Lognormal distributions for aleatory uncertainty and epistemic uncertainty are almost exclusively used. These parameter variations can be included explicitly in the SSI analyses.

Even if formal probabilistic analysis is not performed, defining parameter variations implicitly assigns likelihoods to the values. For example, deterministic SSI analysis for design or assessment of a nuclear installation typically considers a best estimate soil profile, a lower

bound profile, and an upper bound profile. Depending on the method of combining the SSI results, such as enveloping, averaging, or other approaches, a likelihood is assigned to each case. In addition, it is common practice to peak broaden ISRS to account for variability in natural frequencies of structures.

The hierarchy of relative uncertainty from the most uncertain to the least uncertain is: ground motion (free field motion, site specific spatial variation), soil properties (stratigraphy, spatial variation over depth and horizontal extent of site, material models – equivalent linear, nonlinear), foundation/structure behaviour.

### 9.7.1. Ground motion

Specification of design basis earthquake, beyond design basis earthquake ground motion and variabilities in the site specific ground motion needs to be considered in SSI phenomena and their analyses.

#### 9.7.1.1. Specification of design basis earthquake and beyond design basis earthquake ground motion

The largest source of uncertainty in the SSI analysis process is the definition of the ground motion. Sections 5, 6, and 7 address SHA, free field ground motion, and site response leading to seismic input to the SSI analysis process.

The DBE ground motion and the BDBE may be based on standard ground response spectra (Section 7.4) or site specific ground response spectra developed by PSHA or DSHA (Section 5).

Standard ground response spectra are often defined at the median, mean, and 84% NEP based on the development of a statistical representation of the response spectra generated for recorded earthquake motions. U.S. Regulatory Guide 1.60 [173] response spectra are targeted to about an 84% NEP based on a limited set of recorded ground motions from the 1960s and early 1970s. The logarithmic standard deviation (COV) for the 5% damped response spectra in the amplified frequency range (2.5–9 Hz) is about 0.30 conditional on a PGA of 1.0g, which represents response spectral peak to valley variability.

For design of a nuclear installation, the standard ground response spectra are scaled by PGA. Standard ground response spectra may be used for standard, or reference designs scaled to PGA = 0.3g or other values. If so, at a later stage, these standard ground response spectra are compared with the site specific response spectra to confirm their conservatism. They are also used to define the acceptable minimum ground motion for design at the foundation level.

In the assessment of the performance of a nuclear installation subjected to BDBE ground motion defined by a standard ground response spectrum, the BDBE ground motion is specified to be a factor times standard ground response spectra, e.g. a factor of 1.4–1.67 times the DBE ground motion or simply a defined standard ground response spectra anchored to a specified PGA.

In the case of the site specific seismic hazard, significant uncertainties are present in the results. Figure 50 shows an example of probabilistically generated seismic hazard curves for PGA plotted against AFE. Seismic hazard curves for the mean, median (50% NEP), 15% NEP, and 85% NEP are plotted. The variability in the individual seismic hazard curves is due to aleatory uncertainty (randomness). Variability in the NEP is due to epistemic uncertainty (modelling or parameter uncertainty). Figure 50 demonstrates the uncertainty in the seismic hazard curves for PGA.

The PSHA generates seismic hazard curves for response spectral accelerations for a large number of spectral frequencies (Hz) for a specified damping value, usually 5%. A UHRS is constructed of spectral ordinates each of which has an equal AFE and the same NEP. Essentially all PSHAs have included the response spectral peak and valley variability as part of the aleatory variability when developing seismic hazard estimates as a function of the AFE. Thus, at any AFE, the resulting UHRS already fully includes the effect of peak to valley randomness.

These hazard curves and the resulting UHRS could be at rock (e.g. at top of grade if the site is a rock site), or at a hypothetical rock outcrop at depth in the soil. In the latter case, SRA can be performed to generate SAFs, which when applied to the rock UHRS yield UHRS at various locations of interest in the site profile, e.g. FIRS.

The usual approach is to define a Reference Earthquake (RE), which is the free field ground response spectrum at a specified control point. It is intended to be representative of the most important AFE in terms of risk metrics. Typically, it is defined to be the AFE =  $1 \times 10^{-5}$  or  $1 \times 10^{-4}$  (mean or median value). Alternatively, it is defined as the GMRS, which is calculated by scaling the  $1 \times 10^{-5}$  UHRS based on the relationship between the hazard curves of AFE  $1 \times 10^{-5}$  to  $1 \times 10^{-4}$ .

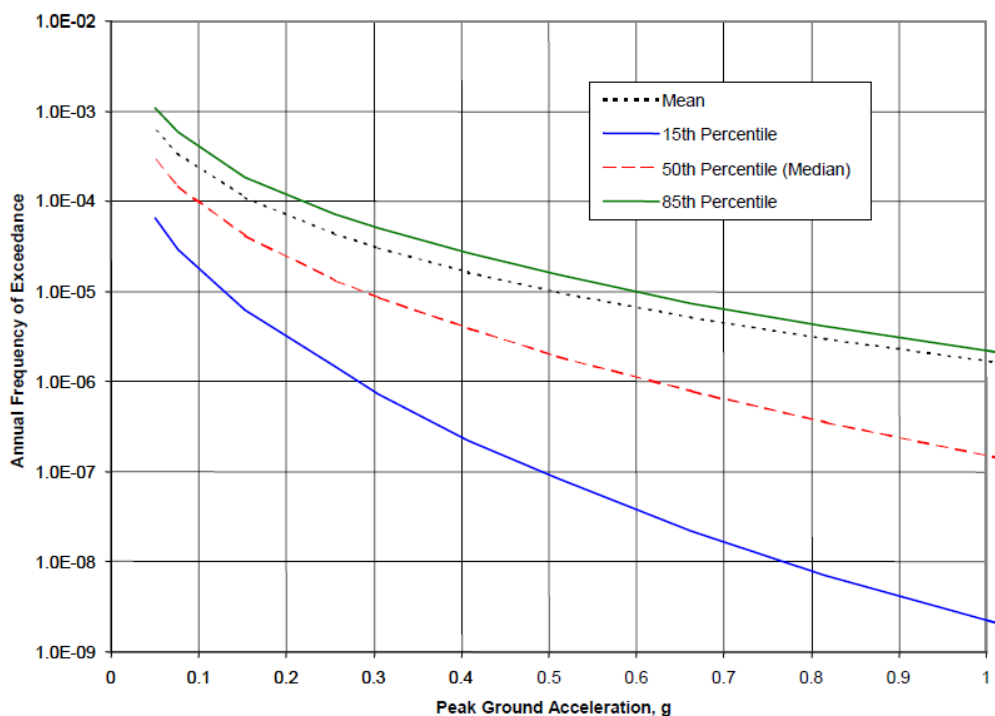


FIG. 50. Example variability in seismic hazard curves for peak ground acceleration.

The RE becomes the base response spectra for seismic response analyses, including SSI analyses.

#### 9.7.1.2. Variabilities in the site specific ground motion

Uncertainties in the site specific ground motion due to earthquake source characteristics and travel path for seismic waves from source to the vicinity of the site are contained in the seismic hazard curves and the resulting UHRS. Generally, variability in the ground response spectra is assumed to be due to randomness (aleatory uncertainty). Uncertainties due to local site effects are incorporated into the results of the site response analyses.

Given the RE, the seismic input for SSI analyses are three spatial components of acceleration time histories either as one realization or an ensemble of  $N$  sets of three spatial components. The ensemble is developed such that the response spectra of each horizontal component is closely fit to the RE target. The COV of the ensemble response spectra over the frequency range of interest is 0.2 or less.

The seismic hazard curves are based on GMPEs, which are developed for the geo-mean of the two horizontal components of recorded ground motion. Therefore, an adjustment is made that takes into account the horizontal direction random variability. The COV for the ratio of horizontal spectral acceleration in any arbitrary direction to the spectral acceleration for the geo-mean of the two horizontal components is in the range 0.16–0.21 from which the value of  $\text{COV} = 0.18$  is most often assumed.

Vertical ground motion response spectra are generated through implementation of site specific V/H ratios. The variability associated with the V/H ratios is a value of  $\text{COV} = 0.25$ .

### 9.7.2. Soil

There are large uncertainties in the soil characteristics due to the difficulty to test soils, and the inherent randomness and spatial variability of soil deposits. The uncertainty in soil characteristics is the second largest source of uncertainty (after the ground motion) in SSI analyses. Spatial variability is characterized by correlation distances of the order of one metre in the vertical direction and of a few metres in the horizontal direction; such small distances preclude a thorough characterization of the deposit. Nevertheless, when enough investigation points are available, stochastic models have been proposed to characterize the spatial variability and used in seismic analyses [83, 84, 311]. In practice, these models are seldom used, and soil uncertainties are usually handled through sensitivity analyses.

As the constitutive model becomes more complex, the effects of these uncertainties become more and more significant. For the elastic characteristics, at least three velocity profiles need to be considered, corresponding to the best estimate characteristics and to those characteristics divided or multiplied by  $(1+\text{COV})$ ; typically, the coefficient of variation (COV) on the elastic shear wave velocity needs to be less than 0.25. The uncertainty on the elastic properties is not the only parameter that needs to be considered: large uncertainties exist in the determination of the nonlinear shear stress–shear strain curves (or equivalently  $G/G_{\text{max}}$  and damping ratio curves used to define the equivalent linear model); this uncertainty stems from the difficulty to recover undisturbed samples from the ground and to test them in the laboratory. It is therefore essential to compare any measurement to published data to assess its representativeness.

With the use of nonlinear models, the number of soil parameters to define increases and so does the uncertainty in the prediction of the soil response. Furthermore, there is a large variety of nonlinear models and none of them can be considered as the best model in all cases. The choice of the constitutive model, and the control and ability of the analyst, therefore, contributes to the overall uncertainty. To cover this aspect, the use of two nonlinear constitutive models, run by two different analysts is preferable.

### 9.7.3. Structure uncertainties

The uncertainties in modelling structure behaviour are associated with the following:

- (a) Modelling of stiffness of load bearing structure elements (linear, equivalent linear, and, possibly, nonlinear).

- (b) Modelling of mass of load bearing structure elements, non-structural elements, equipment, components, and distribution systems.
- (c) Stiffness and mass characteristics are best evaluated through an intermediate step of generating their fixed base dynamic modes (natural frequencies and mode shapes). The major contributor to variability in the dynamic characteristics of the structure is the modelling of its stiffness; modelling of mass is much more precise than stiffness.
- (d) Variability of the dynamic characteristics of the structure is defined by COVs of 0.30 on the structure stiffness (0.15 on natural frequencies). This uncertainty is epistemic uncertainty.
- (e) Energy dissipation is typically incorporated into the seismic analysis through the form of viscous damping defined as a fraction of critical damping (sometimes denoted modal equivalent damping).
- (f) The damping values are defined by a lognormal distribution. Median values are a function of the seismic response level [19] in the structure or structure element. The response level is defined as a ratio of seismic demand/seismic code capacity ( $D/C$ ). Three response levels are defined:  $D/C \leq 0.5$ ;  $0.5 < D/C < 1.0$ ; and  $D/C \geq 1.0$ .

The COV for damping is assumed to be 0.35. This uncertainty is epistemic uncertainty.

Caution is needed when the dynamic response of the structure is calculated using a direct integration method where mass, stiffness, and damping matrices are implemented with the damping matrix being defined by Rayleigh damping or a variant of Rayleigh damping. In such cases, an assessment is made of the effective damping ratio over all important frequency ranges of interest, such that low frequency or high frequency ranges are not overdamped. In addition to overpredicting the damping in some frequency ranges, Rayleigh damping has some other severe drawbacks as described in [9]. Formulating the equation of motion in terms of absolute displacements, as done for nonlinear analyses, involves a rigid body motion that generates extra damping forces due to mass proportional damping as the masses are connected (for a diagonal mass matrix usually used in practice) for each degree of freedom to a fixed support. To overcome that difficulty, Hall (2006) [312] recommends eliminating the mass proportional damping contribution and bounding the stiffness-proportional damping contribution.

Other modelling alternatives to Rayleigh damping is to construct a damping matrix as the superposition of modal damping matrices each of them having the targeted modal damping ratio (Chopra, 2017) [313].

Accuracy (also, called fidelity) of the structure model is a further consideration that up to now has been assumed to be subsumed in the variability of structure frequencies. Realistically, model fidelity needs to be assessed separately, since it is highly dependent on the complexity of the structure itself and the modelling detailing.

#### **9.7.4. Verification and validation of models**

SSI models need to be verified and validated in order to increase confidence in modelling and simulation results. Sections 10.4.1–10.4.3 consider the verification and validation of analysis code. Verification and validation of SSI models needs to be performed in order to increase confidence in analysis results, particularly when inelastic analysis is performed.

## 9.8. STRUCTURAL RESPONSE QUANTITIES

### 9.8.1. Deterministic analyses

#### 9.8.1.1. Design forces, displacements, and stresses

The structural response quantities (displacements, stresses, strains, bending moments, shear forces) need to be defined in accordance with the type of analysis used to compute them. Aside from the uncertainties in the soil and structural input data, which may be taken into account by sensitivity analyses, direct step by step analyses (as opposed to modal spectral analyses) introduce another cause of uncertainty in the response. This is due to the variability of the acceleration time histories derived from response spectra (see Section 7.5). This variability is further enhanced when nonlinear step by step analyses are implemented. Some regulatory guides (e.g. ASN Guide 2/01 [81]) recognize these possible sources of variability by specifying design quantities related to the type of analyses.

If  $R_{k,i}$  represents the maximum value of any response quantity for a given input motion,  $i$  (response spectrum or time history), and for one model ( $k$ ) amongst the  $N$  models used for the sensitivity analyses,  $R_{w,i}$  is defined as a weighted average of these  $R_{k,i}$ :

$$R_{w,i} = \sum_{k=1}^N w_k R_{k,i} \quad \text{with} \quad \sum_{k=1}^N w_k = 1 \quad (32)$$

Equation 32 allows the designer to introduce any degree of conservatism in his design. The maximum value will be obtained, if model  $q$  gives the maximum response quantity, by setting  $w_q = 1$  and  $w_p = 0$  for all  $p \neq q$ . An average value, over all sensitivity analyses, will be obtained by setting  $w_k = 1/N$  for all  $k$ .

With this definition, it is suggested that the design structural response quantity,  $R_D$ , be taken equal to the following:

(a) For modal spectral analyses:

$$R_D = R_{w,1} \quad (33)$$

(b) For step by step linear time history analyses (with  $i=1$ ,  $P$  time histories) to the mean value  $\bar{m}$  of the  $R_{w,i}$ , provided  $P \geq 3$ :

$$R_D = \bar{m}(R_{w,i}) = \frac{1}{P} \sum_{i=1}^P R_{w,i} = \frac{1}{P} \sum_{i=1}^P \sum_{k=1}^N w_k R_{k,i} \quad (34)$$

(c) For step by step nonlinear time history analyses ( $i=1$ ,  $K$ ) to the mean value  $\bar{m}(R_{w,i})$  plus some fraction  $\lambda$  of the standard deviation  $\sigma(R_{w,i})$ , provided  $K > 5$ . The fraction  $\lambda(K)$  depends on the number  $K$  of simulations (time histories used for the analyses) and is based on the Student–Fisher test for a confidence interval of 95%. These values are provided in Table 9.

In summary, any design quantity is given by

$$R_D = \frac{1}{K} \sum_{i=1}^K R_{w,i} + \lambda(K) \sigma(R_{w,i}) \quad (35)$$

For modal spectral analyses,  $I=1$ , and for linear step by step analyses with  $i \geq 3$ ,  $\lambda(K) = 0$ ; for nonlinear step by step analyses with  $i \geq 5$ ,  $\lambda(K)$  is given in Table 9.

The rules detailed above are valid for deterministic analyses in the design of new nuclear installation structures. For assessment of existing nuclear installation structures,  $R_{w,i}$  represents the value calculated for the best estimate properties ( $k=1$ , no sensitivity analyses) and the same rules apply.

### 9.8.1.2. Seismic input to subsystems

The seismic input to subsystems is represented by the ISRS. ISRS are preferably calculated from the time histories of the response at the specified location; however, methods used for direct generation of ISRS are acceptable when the system remains linear. They are computed in accordance with USNRC Regulatory Guide 1.122 [314] and USNRC NUREG-800, 3.7.2, Rev. 3 [153]. Consideration needs to be given in the analysis to the effects on ISRS (e.g. peak, width) of expected variations of structural properties, damping values, soil properties, and SSI. In addition, for concrete structures, the effect of potential concrete cracking on the structural stiffness needs to be specifically addressed. To take into account these uncertainties in the structural frequencies, the computed floor response spectra from the floor time-history motions are smoothed, and peaks associated with each of the structural frequencies are broadened.

Amongst these parameters, the influence of the soil characteristics and time histories of the design earthquake are the most important. When multiple sets of time histories derived from actual earthquake records are used as the input motion to the supporting structure, the multiple sets of in-structure response spectra already account for some of the uncertainty [153] and there is no need to further broaden the peaks of the calculated ISRS.

To take into account the variability of the soil characteristics, at least three sets of velocity profiles (see Section 9.7.2) need to be used for the analyses. In addition, Ref. [81] recommends broadening the peaks of the ISRS associated with the best estimate soil properties by at least 15% on either side.

### 9.8.2. Probabilistic analyses

Probabilistic response analysis is discussed in Section 8.5, including the development of SMACS [299], which is a set of computer program modules to perform probabilistic response analyses. SMACS continues to be a viable tool to calculate probability distributions of seismic responses [300].

The SMACS methodology is based on analysing NPP<sup>21</sup> SSCs for simulations of earthquakes defined by acceleration time histories at appropriate locations within the NPP site. Modelling analysis procedures, and parameter values are treated as best estimate with uncertainty explicitly introduced. For each simulation, a new set of soil, structure, and subsystem properties are selected and analysed to account for variability in the dynamic properties of the soil, structure and subsystems. For purposes of this document, probabilistic SSI (PSSI) analysis is the subject of interest. Therefore, modelling of subsystems and generating their probabilistic seismic response is not described here except for the development of input to subsystems, ISRS, relative displacements, etc. An important aspect of the elements of the seismic response process is that all elements are subject to uncertainties.

When PSSI analysis of the NPP structures of interest is performed, the outputs are probability distributions of in-structure responses for design and capacity assessment (loads, expected cycles for fatigue evaluation, etc.) and acceleration time histories and ISRS for input to subsystems. These PSSI analyses calculate seismic responses as distributions conditional on an earthquake of a given size occurring.

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<sup>21</sup> Nuclear power plants (NPPs) are identified herein as the subject of PSSI. However, any nuclear installation structure founded on soil or rock is a candidate for the probabilistic seismic response analyses described herein.



TABLE 9: THE RELATIONSHIP BETWEEN K AND THE FRACTION  $\lambda(K)$

$K$	$\lambda(K)$
5	0.95
6	0.82
7	0.73
8	0.67
9	0.62
10	0.58
11	0.55
12	0.52
13	0.49
14	0.47
15	0.45
20	0.39
25	0.34
30	0.31
40	0.27
50	0.24

In general, probabilistic response analysis is compatible with the definition of a performance goal for design and for definitions of seismic demand for DBBE assessments, such as SMA and SPRA. However, all design procedures are deterministic. The CDFM procedure for SMA is deterministic. Input for the design procedures and the CDFM may be based on probabilistic considerations.

For design, one definition is to calculate the seismic demand on SSCs as 80% NEP values conditional on the DBE ground motion, as specified in [19]. The deterministic response analysis approaches specified in [19] are developed to approximate the 80% NEP response level. Although, the preferred approach is to perform probabilistic response analysis and generate the 80% NEP responses directly.

For DBBE assessments, SPRA or SMA methods are most often implemented. For SPRA assessments, the full probability distribution of seismic demand conditional on the ground motion (UHRS or GMRS) is needed. The median values (50% NEP) and estimates of the aleatory uncertainties and epistemic uncertainties (or estimates of composite uncertainty) at the appropriate AFE are needed. The preferred method of development is by site specific probabilistic response analyses.

Two approaches to SMA assessments are used, i.e., the CDFM method and the FA method. For the CDFM method, the seismic demand is defined as the 80% NEP conditional on the reference level earthquake. The procedures of [19] are appropriate. For the FA method, the full

probability distribution is needed. In both cases, the 80% NEP values are needed to apply the screening tables of EPRI NP-6041 [298], which provides screening values for high capacity SSCs. Most applications of SMA use the CDFM method.

With regard to an SPRA or SMA evaluation, Section 9.7 introduced the concept of an RE defined by a free field ground response spectrum at a specified control point. The RE defines the input to the seismic response analysis, which develops SSC seismic responses associated with the RE. The RE is an informed choice based on preliminary analyses to identify the range of excitations that are important to risk metrics. The RE is the seismic input for which ISRS are developed, the stress level at which structures are analysed (e.g. cracked or uncracked stiffness for concrete members and level of structure damping), and the strain level at which the underlying soil is analysed when SSI effects are important.

#### *9.8.2.1. Step-by-step probabilistic analyses*

PSSI analyses differ from many other simulations in that the dynamic excitation (earthquake ground motion) and the behaviour of the physical properties of the systems have uncertainties associated with them. Uncertainties in physical properties of the soil–structure system are described by probability distributions (generally, lognormal distributions).

PSSI analyses are based on simulations, which could be based on MCS but are much more efficiently based on LHS. For the SSI analysis, using the LHS procedures, typically, 30 simulations are developed for an adequate representation of the SSI phenomena to define the median responses and the COV of a lognormal distribution fit to the response data. For the site response analyses, using the LHS procedures, 60 simulations are generally performed.

The RE defines the amplitude and response spectral shape of the seismic input. For PSSI analyses, the seismic input is defined by an ensemble of  $N$  sets of three acceleration time histories corresponding to the three spatial directions (two orthogonal horizontal directions and the vertical direction). Section 9.7 specifies the uncertainties to be treated in the PSSI analyses:

- (a) The COV of the ensemble's response spectra over the frequency range of interest needs to be 0.2 or less, i.e., a close fit to the RE target response spectra;
- (b) Since the seismic hazard curves are based on GMPEs, which are developed for the geomean of the two horizontal components of recorded ground motion, an adjustment needs to be made to the acceleration time histories that takes into account the horizontal direction random variability. Research has established that the COV for the ratio of horizontal spectral acceleration in any arbitrary direction to the spectral acceleration for the geomean of the two horizontal components is in a range of 0.16–0.21, from which the value of COV = 0.18 is most often assumed. A scale factor  $FH$  is defined as a lognormal distribution with a median equal to 1.0 and a COV equal to 0.18. The scale factor distribution is discretized into  $N$  equal probability bins, a sample from each bin is taken, and the scale factors  $FH$  and  $(1/FH)$  are randomly applied to horizontal acceleration components 1 and 2, respectively to introduce random variability into the  $N$ th input motion;
- (c) Vertical seismic input in terms of vertical ground response spectra is usually generated by applying V/H ratios. In a similar manner to the horizontal direction variability, a scale factor  $FV$  for the vertical acceleration time histories is defined as a lognormal distribution with median equal to 1.0 and a COV equal to 0.25. The scale factor distribution is discretized into  $N$  equal probability bins, a sample from each bin is taken, and the scale factor  $FV$  is applied to the vertical acceleration time histories.

The end result of the definition of the free field ground motion (seismic input) is  $N$  sets of three component acceleration time histories matching the RE and taking into account the above factors. In addition, each of the  $N$  sets of ground motions is most often assigned to the soil

profile simultaneously developed during the site response analyses, i.e. the soil profile accounting for nonlinear behaviour of the soil.

Sections 7, 9.3 and 9.4 discuss the decisions associated with the definition of the soil profiles to be used in deterministic and probabilistic SSI analyses. There are two methods, as described below.

Method 1: Site response analyses yield site profiles that are defined probabilistically, i.e. the median values and variability of stiffness and material damping as a function of depth in the soil. Companion free field ground motions (for the RE) are defined statistically at the location and form of interest<sup>22</sup>. This location is the control point. For Method 1, N values of the stiffness properties and material damping are sampled from the probability distributions according to the stratified sampling approach for which N bins of equal probability are defined. The samples of stiffness and damping are inversely correlated, i.e. high stiffness with low damping and vice-versa. If the resulting samples can be associated with scale factors on the median values of stiffness and damping over the complete profile, this is treated easily by SMACS or other programs. If a scale factor is not applicable, then the SSI parameters of scattering (kinematic interaction) and impedances (inertial interaction) will need to be calculated for each simulation. For Method 1, the PSSI analysis proceeds by defining the RE at the control point (output of site response analyses); generating the ensemble of N free field ground motions for the SSI analyses; and then developing N samples of the properties of the soil profile, i.e. stratigraphy (layer thickness), material properties (stiffness or shear wave velocity), and material damping (correlated with material properties).

Method 2: Probabilistic site response analyses can be performed for M simulations of site response where each simulation is associated with the UHRS, or a variant of the UHRS, and the output is fully correlated individual soil profiles with site amplification factors to be applied to the UHRS to generate the RE at the location and form of interest. In this case, M soil profiles are associated with simulations of the RE at the location of the seismic input for the SSI analysis. The number of simulations in the site response analyses for this approach is typically 60.

The PSSI using stratified sampling and LHS usually implements about 30 simulations (N=30). Consequently, the 60 samples from the site response analyses are sampled to obtain 30 samples for the PSSI analyses.

For either Method 1 or 2, N sets of seismic input motions and N sets of soil profiles yield their probabilistic definition.

Sections 9.5 and 9.7 discuss structure modelling, including uncertainties. Structure models developed for multi-step or single step analyses are assumed to be median-centred and the model dynamic characteristics are well represented by the fixed-base frequencies and mode shapes.

Section 9.7 describes the identification of parameters to be treated probabilistically and the range of COVs typically considered.

#### *9.8.2.2. Epistemic uncertainty and aleatory uncertainty*

Unless otherwise identified, the uncertain parameters and the COV values presented in this publication are composite uncertainties, i.e. the combination of aleatory and epistemic uncertainties combined by the SRSS.

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<sup>22</sup> Examples of location are TOG and structure foundation level (FIRS). The form of interest is in-column or outcrop. For the direct method of SSI analyses, especially for nonlinear analyses, the form of interest is associated with the SSI analysis procedure.

The source of aleatory uncertainty is related to the ground motion. Therefore, to separate the effects of epistemic uncertainty and aleatory uncertainty, a sensitivity study is often performed.

The seismic input motion (ensemble of ground motion acceleration time histories, horizontal direction variability, horizontal and vertical direction variability) remains unchanged for the sensitivity study. All other system parameters that are defined by probability distributions, e.g. soil and structure soil properties, are treated as point estimates at their median value with no variability introduced. Calculated seismic response distributions, in particular the COVs of responses, for the full uncertainty case compared to the calculated seismic response distributions for aleatory uncertainty only, allow the analyst to estimate the epistemic uncertainties and aleatory uncertainties in each calculated response for use in decision-making.

## **10. AVAILABLE SOFTWARE FOR SOIL–STRUCTURE INTERACTION ANALYSIS**

### **10.1. EXAMPLES ON SOFTWARE FOR USE IN SOIL–STRUCTURE INTERACTION ANALYSES FOR NUCLEAR INSTALLATIONS**

Several different types SSI software (source code and/or executables) are described in this section, as follows:

- (a) Commercial software that is purchased from a commercial company, and has features and capabilities usually determined by the commercial license. Commercial programs usually only guarantee accurate working of (in the manual) provided examples. Commercial programs also usually do not provide verification and validation facilities.
- (b) Open source software that has an open source license that guarantees that the software source code and derivative source code will be always available. The programs usually do not guarantee quality to external users/developers due to legal reasons (liability). Locally, within the development team, they usually have strict quality control.
- (c) Restricted source software that is a restricted version of an open source software. The difference is that developers/owners can restrict source code distribution, so a revised open source license is used.
- (d) Open use software that is a freely available version of program executables. There are usually no guaranties of quality nor verification and validation facilities.
- (e) Public domain software that is source code and/or executables distributed with no restrictions whatsoever. The original developer/owner relinquishes all or his/her rights with respect to sources and/or executables.

#### **10.1.1. Available programs for soil–structure interaction analyses**

The following programs can perform full SSI analysis<sup>23</sup>.

- (a) Commercial software:

- ABAQUS<sup>24</sup>
- ADINA<sup>25</sup>

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<sup>23</sup> For completeness, 1D site response codes are also listed as they are also used in SSI analysis to provide input motions.

<sup>24</sup> web page: <http://www.3ds.com>

<sup>25</sup> web page: <http://www.adina.com>

- ALGOR/AutoDesk simulation<sup>26</sup>
- ANSYS<sup>27</sup>
- CLASSI<sup>28</sup>
- GT STRUDL<sup>27</sup>
- LS-DYNA<sup>28</sup>
- NASTRAN<sup>29</sup>
- FLUSH<sup>30</sup>
- SAP2000<sup>31</sup>
- SASSI 2010<sup>32</sup> and ACS SASSI<sup>33</sup> (various versions are available)
- SMACS<sup>34</sup>
- GT STRUDL<sup>35</sup>
- SOFISTIK<sup>36</sup>
- PLAXIS<sup>37</sup>
- FLAC<sup>38</sup>
- DYNAFLOW<sup>39</sup>
- Real ESSI<sup>40</sup>
- Zsoil<sup>41</sup>

(b) Open source, restricted source and open use:

- FEAP<sup>42</sup>
- DEEPSOIL<sup>43</sup>
- SIMQKE<sup>44</sup>
- OpenSees<sup>45</sup>

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<sup>26</sup> web page: <http://www.autodesk.com>

<sup>27</sup> web page: <http://www.ansys.com>

<sup>28</sup> web page: <https://www.lstc.com>

<sup>27</sup> web page: <https://hexagonppm.com>

<sup>28</sup> web page: <http://www.lstc.com>

<sup>29</sup> web page: <http://www.mssoftware.com>

<sup>30</sup> web page: <https://www.geoengineer.org>

<sup>31</sup> web page: <https://www.csiamerica.com>

<sup>32</sup> web page: <http://sassi2000.net>

<sup>33</sup> web page: <http://www.ghiocel-tech.com>

<sup>34</sup> web page: <https://www.osti.gov>

<sup>35</sup> web page: <https://ce.gatech.edu>

<sup>36</sup> web page: <http://www.sofistik.com>

<sup>37</sup> web page: <http://www.plaxis.nl>

<sup>38</sup> web page: <http://www.itascacg.com>

<sup>39</sup> web page: <https://blogs.princeton.edu>

<sup>40</sup> web page: <http://essi-consultants.com>

<sup>41</sup> web page: <http://www.zsoil.com>

<sup>42</sup> web page: <http://www.ce.berkeley.edu>

<sup>43</sup> web page: <http://deepsoil.cee.illinois.edu>

<sup>44</sup> web page: <http://nisee.berkeley.edu>

<sup>45</sup> web page: <http://opensees.berkeley.edu>

- Real ESSI<sup>46</sup>
- Code\_ASTER<sup>47</sup>
- (c) Public domain
  - SHAKE91<sup>48</sup>
  - EERA and NEERA<sup>49</sup>
  - DESRA-2
  - SUMDES
  - D-MOD
  - TESS

## 10.2. VERIFICATION AND VALIDATION OF SSI SOFTWARE

### 10.2.1. introduction

Verification and validation of SSI software is discussed in [44, 310]. It is important to define verification and validation, as follows [315]:

- (a) Verification: The process of determining that a model implementation accurately represents the developer's conceptual description and specification. It is a mathematics issue. Verification provides evidence that the model is solved correctly.
- (b) Validation: The process of determining the degree to which a model is accurate representation of the real world from the perspective of the intended uses of the model. It is a physics issue. Validation provides evidence that the correct model is solved.

The main findings are related to verification procedures that are suggested for SSI modelling and simulation, and for validation procedures (as there is a general lack of quality validation data). Verification and validation procedures are designed in time domain; for numerical analysis tools that operate in frequency domain, verification and validation procedures need to prove/demonstrate adequacy in time domain, since real ESSI behaviour takes place in time domain. The following procedures cover all the components of modelling and simulation and are applicable to the analysis of NPP systems, structures and components:

- (1) Source code verification is used to prove that the program is free of any bugs and inconsistencies that could diminish the results. For a modelling and simulation program written in any programming language (C, C++, FORTRAN, etc.) it is necessary to perform source code verification with all the necessary steps.
- (2) Verification and validation for constitutive problems is used to address issues related to material modelling and integration of constitutive integration algorithms for nonlinear/inelastic material modelling. Constitutive integration algorithms need to be verified in detail, while material modelling needs to be validated in detail. In addition, the constitutive level calculations for seismic energy dissipation need to be verified.
- (3) Verification and validation for static and dynamic FEM advancement algorithms is used to address issues related to static and dynamic incremental iterative algorithms that drive the incremental modelling process forward. These algorithms can introduce (unwanted or

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<sup>46</sup> web page: <http://real-essi.org>

<sup>47</sup> web page: [http://www.code\\_astair.org](http://www.code_astair.org)

<sup>48</sup> web page: <http://nisee.berkeley.edu>

<sup>49</sup> web page: <http://www.ce.memphis.edu>

wanted) numerical damping/energy production, and they need to be fully tested against available analytic or very accurate solutions.

- (4) Verification and validation for static and dynamic behaviour of single phase, solid elements is used to address modelling using solid finite elements. This includes the accuracy of modelling of various states of stress (uniaxial, multiaxial) and the resulting accuracy of stresses, forces and displacements for different models where very accurate or analytic solutions exist.
- (5) Verification and validation for static and dynamic behaviour of structural elements is used to assess forces and displacements for structural elements (truss, beam, shell) against very accurate and/or analytic solutions for trusses, beams and shells (plates, wall elements and combinations).
- (6) Verification and validation for static and dynamic behaviour of special elements is used to address issues with contact elements, for both dry and saturated conditions. Of particular interest here is the accuracy of modelling of axial (normal force – gap) and frictional/slipping behaviour, as these elements are known to misbehave for combination of axial and shear loads.
- (7) Verification and validation for coupled, porous solid–pore fluid problems is used to address issues with solid finite elements that model both porous solid and pore fluid, which is very important for soil and rock. In addition, these coupled elements form a basis for modelling coupled contact, where the contact zone (concrete foundation–soil/rock) is beneath the water table.
- (8) Verification and validation for seismic wave propagation problems is used to address issues of proper propagation of seismic waves of a predetermined frequency range through FEM models. In addition, this addresses the accuracy and adequacy of the seismic input to FEM models that encompasses body and surface waves.

In addition to comparison with very accurate and analytic solutions, errors tables/plots are also developed. These error table/plots are important as they are used to emphasize that numerical methods used in modelling and simulations are based on approximate methods and that all the obtained results contain uncertainties. Numerical modellers and analysts need to be aware of these uncertainties and need to address them in presenting their results.

### **10.2.2. Importance of verification and validation**

Verification and validation for SSI modelling and simulation represents a basic task, without which no results of such modelling and simulation can be presented.

With the development of advanced modelling and simulation numerical tools, there is an increased interest in verification and validation activities [316–322].

Verification and validation activities and procedures are the primary means of assessing accuracy in modelling and computational simulations. Verification and validation activities and procedures are the tools with which we build confidence and credibility in modelling and computational simulations. Without proper verification and validation, numerical modelling and simulation results cannot be used for design, licensing or any other activity that relies on those results. Errors, inconsistencies and bug in numerical modelling and simulation programs are present and need to be removed and/or documented. A well-known study by Hatton and Roberts, 1994 [323]; Hatton, 1997 [324] reveals that all the software (in engineering, databases, control, etc.) contains errors that can be removed if proper program development procedures are followed. More importantly, the first step is a realization that software/program

probably/likely has some errors, bugs and that finding those errors and bugs needs to be done before the program start being used in decision making (design, licensing, etc.). In addition, numerical modelling and simulation are based on approximations and thus approximation errors are always present in results. Those errors need to be documented and information about those errors needs to be presented to potential users of numerical modelling and simulation programs.

Reference [320] suggest that verification and validation are a prerequisite for proper numerical modelling and simulation. Results from such verified and validated modelling and simulations, are then used to gain knowledge about behaviour of infrastructure objects. Such knowledge is then used to make decisions (e.g. on design, licensing, etc.).

### 10.2.3. Detailed look at verification and validation

A diagram of verification and validation is presented in Figure 51, in which ‘Real world’ represents a highly accurate knowledge about the realistic behaviour of soil and structures to be considered for SSI. Such behaviour is represented by a conceptual model that is then used as a basis for verification. Physical testing of unit problems or small components of the complete model are used for validation.

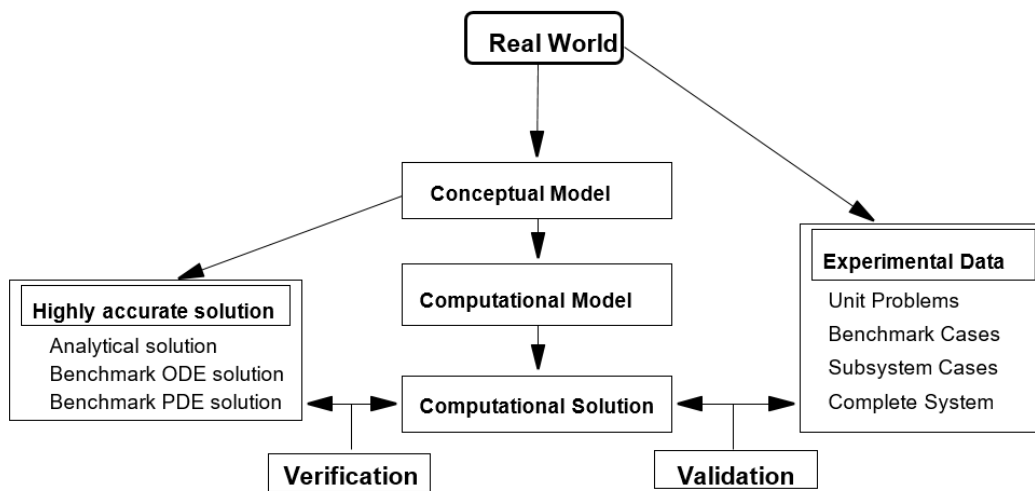


FIG. 51. Verification and validation

The main goals of verification are to:

- (a) Identify and remove errors in computer coding:
  - Numerical algorithm verification;
  - Software quality assurance practice.
- (b) Quantify numerical errors in the computed solution.

A diagram of verification is presented in Figure 52.



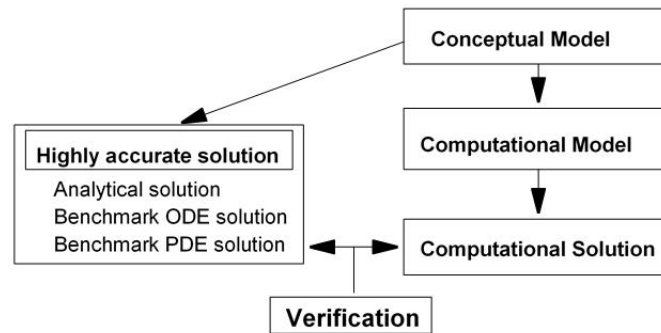


FIG. 52. Verification of software.

The main goals of validation are:

- (a) Tactical goal: To identify and minimize uncertainties and errors in the computational model.
- (b) Strategic goal: To increase confidence in the quantitative predictive capability of the computational model.

A diagram of validation is presented in Figure 53.

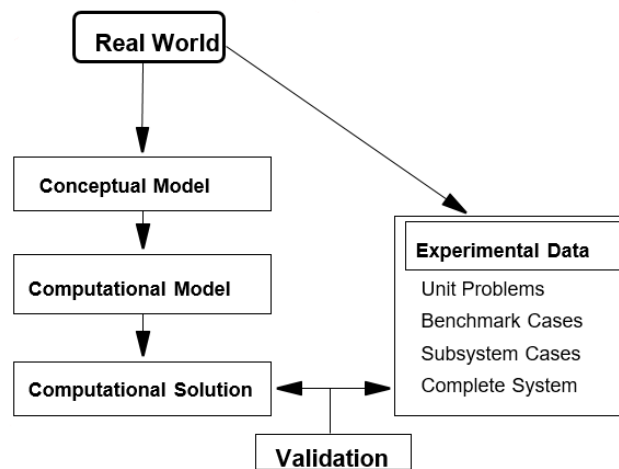


FIG. 53. Validation of software.

Numerical prediction uses a computational model to predict the state of a physical system under consideration under conditions for which the computational model has not been validated. Validation does not directly make a claim about the accuracy of a prediction because of the following:

- (a) Computational models are easily misused (unintentionally or intentionally);
- (b) The validation will depend on how closely the prediction conditions and specific cases in validation database are related;
- (c) It will depend on how well the physics of the problem is understood.

#### 10.2.4. Examples of verification and validation

A large number of verification and (not so large set of) validation examples are available in [44, 310]. Recently, several ESSI modelling, and simulation programs have published verification and validation suites, for example ACS SASSI–2020 and Real ESSI (see Section 10.1.1).

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## **ANNEXES: SUPPLEMENTARY FILE**

Examples of soil–structure interaction analysis methodologies for nuclear installations are provided as a supplementary file for this publication and can be found on the publication’s individual web page at [www.iaea.org/publications](http://www.iaea.org/publications). They are organized in 7 Annexes:

Annex I. Seismic wave incoherence: a case study

Annex II. Site response analysis

Annex III. Analysis of a pile foundation (for a bridge) by the substructure method

Annex IV. Examples of seismic response of an NPP on nonlinear soil and contact (slip and uplift)

Annex V. Nonlinear analysis of a deeply embedded small modular reactor (SMR)

Annex VI. Nonlinear, time domain, 3-D, earthquake soil–structure interaction (ESSI) analysis of NPP, analysis procedures

Annex VII. Equivalent bulk modulus for unsaturated soil.



## CONTRIBUTORS TO DRAFTING AND REVIEW

Abe, H.	Consultant, Japan
Altinyollar, A.	International Atomic Energy Agency
Bard, P.	University of Grenoble, France
Beltran, F.	International Atomic Energy Agency
Caudron, M.	Électricité de France, France
Clement, C.	Institut de radioprotection et de sûreté nucléaire, France
Coman, O.	International Atomic Energy Agency
Contri, P.	International Atomic Energy Agency
Cruz, J. R.	Comissão Nacional de Energia Nuclear, Brazil
El-Hemamy, S.	Egyptian Nuclear and Radiological Regulatory Authority, Egypt
Haddad, J.	International Atomic Energy Agency
Han, X. M.	Candu Energy, Canada
Houston, T.	Consultant, USA
Jeremic, B.	University of California, USA
Jimenez-Juan, A.	Consejo de Seguridad Nuclear, Spain
Johnson, J. J.	Consultant, USA
Kanazawa, K.	Central Research Institute of Electric Power Industry, Japan
Kausel, E.	Consultant, USA
Kim, M.	Korea Atomic Energy Research Institute, Korea
Kultsep, A.	CKTI-VIBROSEISM Ltd., Russian Federation
Kurmman, D.	Axpo Power, Switzerland
Labbe, P.	Consultant, France
Lonkhuyzen W. V.	Regulatory Authority Kernfysischer Dienst, Netherlands
Mahmood, H.	Consultant, Pakistan
Meng, C.	Shanghai Nuclear Engineering Research and Design Institute, China
Omar, R. C.	Universiti Tenaga Nasional, Malaysia
Orbovic, N.	Canadian Nuclear Safety Commission, Canada
Pecker, A.	École des Ponts ParisTech, France
Petre-Lazar, E.	Électricité de France, France
Pires, J.	Nuclear Regulatory Commission, United States of America
Potin, G.	Tractebel Eng. S.A., France



Poveda Solano, A.	International Atomic Energy Agency
Rangelow, P.	Consultant, Germany
Renault, P.	Swiss nuclear, Switzerland
Ryzhov, D.	State Scientific and Technical Center for Nuclear and Radiation Safety, Ukraine
Tardivel, J. P.	Institut de radioprotection et de sûreté nucléaire, France
Thiry, J. M.	Areva, France
Tyapin, A.	JSC Rosenergoatom, Russia
Umeki, Y.	Central Research Institute of Electric Power Industry, Japan
Volkov, Y.	JSC Rosenergoatom, Russia
Yoo, B.	The Belgian Nuclear Research Centre SCK-CEN, Belgium
Wang, F.	Commissariat à l'énergie atomique, France
Weaver, T.	Nuclear Regulatory Commission, United States of America

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