IAEA TECDOC

On
Seismic Soil Structure Interaction for Design and Assessment of Nuclear Installations

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1. **INTRODUCTION**

1.1 **BACKGROUND**

The response of a structure during an earthquake depends on the characteristics of the ground motion, the surrounding soil, and the structure itself. In general, the foundation motion differs from the free-field motion due to soil-structure interaction (SSI).

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.

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, for engineering purposes, the foundation motion and the ground motion in the free-field can be assumed to be the same. That is, SSI is not an important phenomenon. In these cases, a fixed-base analysis can be justified. However, care must be exercised in so doing especially for structures subjected to ground motion with high frequency content, i.e., greater than about 20 Hz.

Figure 1-1 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 (Vs = 1830 m/s) to soft soil (Vs = 152 m/s). The input ground motions are acceleration time histories in the two horizontal directions and the vertical direction (U.S. NRC RG 1.60 ground response spectra shapes anchored to PGA = 1 m/s²). It is clear from Figure 1-1 that ignoring SSI in the dynamic response of the structure misrepresents the seismic response of the structure and the seismic input to equipment, components, distribution systems, and supporting sub-structures.
Figure 1-1. Effect of soil stiffness on structure response of a typical nuclear power plant structure; rock ($V_s = 1830$ m/s), stiff soil ($V_s = 762$ m/s), medium soil ($V_s = 305$ m/s), and soft soil ($V_s = 152$ m/s) (Courtesy of James J. Johnson and Associates)

SSI analysis has evolved significantly over the past five decades.

- 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.

- In the 1970s and early 1980s, research activities were initiated. Research centers sprouted up around academic institutions – University of California, Berkeley (Profs. Seed and Lysmer group), University of California, San Diego (Prof. Luco)/University of Southern California (Dr. Wong), Massachusetts Institute of Technology (Profs. Roesset and Kausel, and Dr. Christian), Prof. Wolf (Swiss Federal Institute of Technology).
In addition, 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 the earthquake problem was attempted with mixed results.

As one element of the U.S. Nuclear Regulatory Commission’s sponsored Seismic Safety Margin Research Program (SSMRP), the state of knowledge of SSI as of 1980 was well documented in a compendium Johnson [1-1] of contributions from key researchers (Luco; Roesset and Kausel; Seed, and Lysmer) and drew upon other researchers and practitioners as well (Veletsos, Chopra). This reference provided a framework for SSI over the 1980s and 1990s.

SSI 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 by Luco and colleagues (University of California, San Diego) and Roesset, Kausel and colleagues (Massachusetts Institute of Technology). The University of California, Berkeley Team (Seed, Lysmer and colleagues) developed several direct approaches to performing the SSI analyses (LUSH, ALUSH, FLUSH).

- 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 was the Electric Power Research Institute (EPRI), in cooperation with Taiwan Power Company (TPC), constructed two scale-model reinforced concrete nuclear reactor containment buildings (one quarter and one twelfth scale) within an array of strong motion instruments (SMART-I, Strong Motion Array Taiwan, Number 1) in Lotung, Taiwan. The SMART-I array was sponsored by the U.S. National Science Foundation and maintained by the Institute of Earth Sciences of Academia Sciences of Taiwan. The structures were instrumented to complement the free-field motion instruments of SMART-I.

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 Lotung site.
Additional recorded data in Japan and the U.S. served to demonstrate important aspects of the free-field motion and SSI phenomena [1-3]. 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, i.e., 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, which continues to be in use today.

- In the 1980s, skepticism 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 requirement implemented by the U.S. NRC that SSI analysis should be performed by “lumped parameter” and finite element methods and the results enveloped for design.

- In the 1990s, some clarity of the issues was obtained. In conjunction with data acquisition and observations, the statements by experts, researchers, and engineering practitioners that the two methods yield the same results for problems that are defined consistently finally prevailed. The EPRI/TPC efforts 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, 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 nuclear power plants worldwide. Events affecting plants in high-seismic-hazard areas, such as Japan, have been documented in International Atomic Energy Agency (IAEA) Safety Reports Series No. 66, “Earthquake Preparedness and Response for Nuclear Power Plants” (2011) [1-2]. In some cases, 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) [1-4]. Figure 1-2 shows the recorded responses of the free-field top of grade measured motion (blue curve) compared to the motions recorded at the Unit 7 Reactor Building basement (red curve) and the third floor (black curve). 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 soil-structure interaction behaviour.

Figure 1-2. 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 (IAEA, KARISMA program [1-4])
1.2 OBJECTIVES

This TECDOC provides a detailed treatise on SSI phenomena and analysis methods specifically for nuclear safety related facilities. It is motivated by the perceived need for guidance on the selection and use of the available soil-structure interaction methodologies for the design and assessment of nuclear safety-related structures.

The purpose of the task is to review and critically assess the state-of-the-practice and near future (1 to 5 years) regarding soil-structure interaction modelling and simulation methodology. The emphasis is on the engineering practice, while noted are also methods that are currently available (or will be available in the very near future, next few years), and are making their way into engineering practice. The final goal is to provide practical guidance to the engineering teams performing this kind of analyses in present and in the near future. The objectives are to:

- Describe the physical aspects of site, structure, and earthquake ground motion that lead to important SSI effects on the behaviour of structures, systems, and components (SSCs).
- Describe the modelling of elements of SSI analysis that are relevant to calculating the behaviour of SSCs subjected to earthquake ground motion.
- Identify the uncertainty associated with the elements of SSI analysis and quantify, if feasible.
- 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.
- Provide guidance on the selection and use of available SSI analysis methodologies for design and assessment purposes.
- 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)
- Provide a near future view of the SSI analysis field looking to the next one to five year period.
- Document the observations, recommendations, and conclusions.

This TECDOC is intended for use by SSI analysis practitioners and reviewers (Peer Reviewers and others). This TECDOC 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 programmes.
1.3 SCOPE OF THE TECDOC

This TECDOC provides a detailed treatise on SSI phenomena and analysis methods specifically for nuclear facilities\(^1\) or nuclear installations\(^2\). Herein, these terms are used interchangeably along with the term nuclear power plant (NPP). The context is facility design, and assessment.

- Design includes new nuclear power facility design, such as Reference Designs or Certified Designs of nuclear power plants (NPPs) and the design/qualification of modifications, replacements, upgrades, etc. to an existing facility.
- Assessments encompass evaluations for Beyond Design Basis Earthquake (BDBE)\(^3\) ground motions typically Seismic Margin Assessments (SMAs) or Seismic Probabilistic Risk Assessments (SPRAs) for new and existing nuclear installations and existing nuclear installations. The results of assessments may lead to design changes or modifications.
- Assessments for the forensic analysis of facilities that experience significant earthquake ground motions.

1.3.1 Design Considerations: General Framework

Design requires a certain amount of conservatism to be introduced into the process. The amount of conservatism is dependent on a performance goal to be established. Performance goals are established dependent on the critical nature of the structures, systems and components (SSC) and the consequences of “failure” to personnel (on-site and public) and the environment.

Safety objectives and performance goals:
- May be defined probabilistically or deterministically;
- May be specified at the individual structure, system, and component (SSC) level;
- May be specified in terms of overall facility behaviour;
- May encompass both design level and beyond design basis performance goals.

In general, safety objectives and performance goals need to be tiered down to the level of SSC design and qualification procedures that when implemented yield the overall objectives and goals. An example of overall risk based performance goals is in the U.S., which is described in Section 1.3.2.

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\(^1\) Nuclear facility is “A facility and its associated land, buildings, and equipment in which radioactive materials are produced, used, handled, stored or disposed of on such a scale that consideration of safety is required.”[1-11]

\(^2\) Nuclear installation is “A nuclear fuel fabrication plant, research reactor (including subcritical and critical assemblies), nuclear power plant, spent fuel storage facility, enrichment plant or reprocessing facility.”[1-11]

\(^3\) “For external events that exceed the design basis, derived from the site evaluation, i.e. the magnitude for which the safety systems are designed to remain functional both during and after the external event, the term ‘Beyond Design Basis External Event’ (BDBEE) is proposed and used in this publication.”[1-12]
In France, the performance goals are defined deterministically and at the SSC design, qualification, and evaluation level. This approach is described in Section 1.3.3.

In general, the process of establishing a comprehensive approach to defining and implementing the overall performance goals and, subsequently, tier down these performance goals to SSI (and other elements of the seismic analysis, design, and evaluation process) is 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. Partition achievement of the performance goal into elements, including SSI.

c. Develop guidance for SSI modelling and analysis to achieve the performance goal.

Considering that the subject of this TECDOC is SSI, an important element in this process is to establish the levels of conservatism in the current design and evaluation approaches to SSI.

1.3.2 U.S. Practice

In the U.S., performance goals are established at the highest level of overall nuclear facility performance.

- For nuclear power plants (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 for CDF are CDF ≤ 1x10^-4 and LERF ≤ 1x10^-6.

- For U.S. Department of Energy (DOE) nuclear facilities, overall performance goals are that confinement of nuclear material should be ensured at a mean annual frequency of failure of 10^-4 to 10^-5 (DOE Order 420.1C and Implementation Guide 420.1-2).

ASCE 4-16[1-5], ASCE 43-05[1-6], and U.S. DOE Standards define specific performance goals of structures, systems, and components (SSCs) in terms of design (DBE) and in combination with beyond design basis earthquakes (BDBEs).

Given the seismic design basis earthquake (DBE), the goal of ASCE/SEI 4-16 is to develop seismic responses with 80% probability of non-exceedance. For probabilistic seismic analyses, the response with 80% probability of non-exceedance is selected.

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

i. Less than about a 1% probability of unacceptable performance for the Design Basis Earthquake Ground Motion, and

ii. Less than about a 10% probability of unacceptable performance for a ground motion equal to 150% of the Design Basis Earthquake Ground Motion.

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

i. Demand is determined at about the 80% non-exceedance level for the specified input motion.

ii. Design capacity is calculated at about 98% exceedance probability.

For new NPPs (generally Certified Designs), during the design phase, the vendor should 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 (BOP) 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, a seismic probabilistic risk assessment (SPRA) is to be performed with the end results to demonstrate conformance with the above-stated risk metrics of CDF and LERF.

1.3.3 French Practice

In the past and up to now, the general French 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:

i. The site specific design earthquake ground motion is defined from a mainly deterministic seismic hazard analysis (RFS2001-01);

ii. The seismic demand and capacity checks are required to be performed by deterministic approaches (ASN2/01);

On the other hand, French practice aims to use probabilistic safety analysis in specific domains, when needed, as a complementary approach. Few specific probabilistic seismic analyses have been performed by operators and/or the regulator TSO in the last years; some others have started recently and are still in progress.
For post Fukushima checks regarding the hard-core components for the Beyond Design Basis Earthquake, so-called Seism Noyau Dur, (SND), the Regulator has specified general conditions as follows:

- The definition of the seismic ground motion is the envelope of 150% of the site specific Design Basis Earthquake motion and of a probabilistic motion with a return period of 20,000 years (annual frequency of exceedance of $5 \times 10^{-5}$);
- 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.

1.3.4 Canada Practice

The NSCA (National Safety and Control Act) and the REGDOC 2.5.2 provide the basis for the following criteria and goals:

1. Safety goals

Safety analyses shall 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.

2. 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, shall be calculated in the deterministic safety analysis for a period of 30 days after the analyzed event. Dose Acceptance Criteria are related to the Design Basis.

This dose shall be less than or equal to the dose acceptance criteria of:

a. 0.5 millisievert (mSv) for any AOO or
b. 20 mSv for any DBA

The values adopted for the dose acceptance criteria for Anticipated Operational Occurrences (AOOs) and Design Basis Accidents (DBA) are consistent with accepted international practices, and take into account the recommendations of the IAEA and the International Commission on Radiological Protection.
Quantitative application of the safety goals

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

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

Small release frequency and large release frequency are measures of the plant’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 shall 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} becquerels of iodine-131 shall 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} becquerels of cesium-137 shall be less than 10^{-6} per reactor year. A greater release may require long term relocation of the local population.

1.3.5 Japan

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

For Beyond Design Basis Earthquake, 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 the high seismic region, the significance of elasto-plastic behaviour of structures is recognized 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 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 wall cannot be neglected, FEM models are used to evaluate such local responses.

Nonlinear modelling of soil and structures is encouraged with various levels of modeling detail. In addition, the regulator requires modelling from source to NPP with various levels of modeling 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 fidelity modelling and analyses. This use of high performance computing will evolve quickly over the next five years.

The SSI modelling procedures mentioned above are provided in Japanese seismic design standard JEAG-4601[1-13], [1-14].

Reference [1-15] documents the development and implementation of the U.S. and Japan seismic design standards and calculational methods over the last several decades. A comparison of U.S. and Japan standards and calculational methods is made step-by-step of the various elements to provide insight into the similarities and differences that exist. Post-Fukushima-2011 efforts in Japan and the U.S. are covered briefly.

1.3.6 Russia

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 (ZPA) are generated representing the peak ground acceleration (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 spectra for design from the sets in accordance with the requirements of the Federal design codes in nuclear energy with 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).

With regard to NPPs there are requirements to ensure a high level of nuclear safety for all credible internal and external natural and human-made initiating events, including beyond design basis earthquakes. Safety goals for each NPP unit in the interval of one year are:

- total probability of severe accidents shall not to exceed value $10^{-5}$;
- total probability of a large accidental release shall not to exceed value $10^{-7}$;
- total probability of severe accidents for nuclear fuel in the existing nuclear repositories shall not to exceed values of $10^{-5}$.

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

1.3.7 EU practice

As an aside, for new NPPs, the European Utility Requirements document (EUR) specifies requirements for standard designs in Europe: design basis earthquake (DBE) ground motion comprised of broad-banded ground response spectra anchored to a PGA of 0.25g and consideration of beyond design basis earthquake events in the design phase. The EUR specifies that it should 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.

1.3.8 IAEA Guidance

Four documents are relevant to hazard definition (PSHA and DSHA), site response analyses, and SSI:


These references provide overall guidance as to what should be considered in the SHA, soil and structure modelling, and SSI models and analysis. The present TECDOC complies with the guidance provided in the references. Note, updates of SSG-9, NS-G-3.6, and NS-G-1.6 are in progress as of 2018.

1.4 STRUCTURE OF TECDOC

Final step.

1.5 REFERENCES


2 ELEMENTS OF SSI ANALYSIS

Chapter 2 presents an overview of the elements of SSI and directs the reader to other chapters in this TECDOC for more in-depth and complete discussion.

The elements of SSI analysis are:

- Free-field ground motion – seismic input
- Site configuration and modelling of soil properties
- Site response analysis
- Modelling of foundation and structure
- Methods of SSI analysis
- Uncertainties

Table 2-1 summarizes the elements of SSI with reference to chapters of the TECDOC where the element is addressed. Figure 2-1 is a flow chart showing the various steps in the process.

2.1 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 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 is important.

Free-field ground motion may be defined by site independent or site dependent ground response spectra.
- Site independent ground motion is most often used for performing a new reference design or a Certified Design (CD), which is to be placed on a number of sites with differing characteristics.
- Site specific ground motion is most often developed from a seismic hazard analysis (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:

- **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 seismic hazard analysis (SHA). These methods are discussed in Chapter 5.

In the neighbourhood, means in the site vicinity, but not yet having site specific characteristics 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 derived at to the top of grade (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. U.S. NRC SRP Section 3.7.1 defines a competent soil layer as soil with shear wave velocity (Vs) of 305 m/s or greater. The EUROCODE defines a significant impedance contrast as a ratio of six for the shear modulus.

In the context of the “neighbourhood” this location is denoted the “control point”.

In this latter case, a site response analysis is performed to generate the ground motion for input to the SSI analysis.

- **Local site effects.** Given the free-field ground motion in the neighbourhood of the site, the next stage in defining the seismic input to the SSI analyses is to incorporate local site effects. 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.
- If the end product of the first stage of this definition process is ground motions at top of grade (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 finite element SSI model, e.g., nonlinear SSI model.  

- 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.

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 Chapters 3, 4, 5, and 6.

2.2 MODELING SOIL, STRUCTURES, FOUNDATIONS

2.2.1 Modelling of soil for DBE and BDBE

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

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

These topics are considered extensively in Chapter 3.

2.2.2 Modelling structures and SSI models

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

\footnote{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.}
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.

- The structure model of the first step of the multistep analysis represents the overall dynamic behaviour of the structural system but need not be refined to predict stresses in structural elements.
- 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.
- 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).
- 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.
- 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.
- 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.

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 introduced 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.

2.2.3 Modelling decisions to be made

All modelling decisions are dependent on the purpose of the analysis, i.e., for DBE design or for assessments (BDBE, DEC), or actual earthquake occurs and affects the nuclear installation.
Decisions concerning structure modelling should consider the following items:

- Multi-step vs. single step analysis – seismic response output quantities to be calculated
- Stress level expected in the structure
  - Linear or nonlinear structure behaviour.
- Lumped mass vs. finite element models. Is a lumped mass model representative of the dynamic behaviour of the structure for the purpose of the SSI analysis?
- Frequency range of interest – especially high frequency considerations (50 Hz, 100 Hz).

Decisions concerning foundation modelling should consider the following items:

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

Decisions concerning SSI modelling should consider the following items:

- Direct or sub-structuring methods
- Purpose of the analysis – DBE design, BDBE assessment
- Strain level – equivalent linear or nonlinear soil and structure behaviour.
- Irregular soil/rock profiles.
- Probabilistic/deterministic.
- Embedment conditions (partial or full).
- High water table.
- Structure-to-structure interaction.
- Other issues.

Chapter 7 Methods and Models for SSI Analysis and Chapter 8 Seismic Response Aspects for Design and Assessment address these and other issues.
2.3 UNCERTAINTIES

2.3.1 Aleatory 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. In other cases, 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.

In general, uncertainties are categorized into aleatory uncertainty and epistemic uncertainty (ASME/ANS [2-5]) with the following definitions:

“aleatory uncertainty: 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.”)”

“epistemic uncertainty: 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”).

Randomness is considered to be associated with variabilities that cannot practically be reduced by further study, such as the source-to-site wave travel path, 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 and epistemic uncertainties are often represented by probability distributions assigned to SSI parameters. Further, 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 $\beta_U$).

$$A = A_m \varepsilon_R \varepsilon_U$$

(2.1)

In some cases, it is advantageous to combine the randomness and modelling/data/parameter uncertainty into a “composite variability” as defined in ASME/ANS, 2013):

“composite variability: the composite variability includes the aleatory (randomness) uncertainty ($\beta_R$) and the epistemic (modeling/data/parameter) uncertainty ($\beta_U$). The logarithmic standard deviation of composite variability, $\beta_C$, is expressed as:

$$(\beta_R^2 + \beta_U^2)^{1/2}$$

The same functional representation of Eqn. 2.1 typically defines the fragility function for SSCs in a SPRA.

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

2.3.2 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.[2-6] 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/I&C 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 of such. This is especially true for the seismic hazard element’s affect on all other aspects in the seismic analysis and design chain.

2.3.3 Treating uncertainties in the SSI 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:

- 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.
• Others.

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

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

This is discussed in Chapter 8.

2.5 REFERENCES


Table 2-1  Summary of elements of SSI analysis with reference to chapters covering topics

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</thead>
<tbody>
<tr>
<td>Linear, equivalent linear, nonlinear</td>
<td>Chapter 7, 8</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>SSI Models</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Linear, equivalent linear, nonlinear</td>
<td>Chapter 7, 8</td>
</tr>
</tbody>
</table>
Table 2-2  Partitioning of uncertainties

<table>
<thead>
<tr>
<th>Element</th>
<th>Epistemic</th>
<th>Aleatory</th>
</tr>
</thead>
<tbody>
<tr>
<td>Modeling</td>
<td>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), over- or under-predict events/results of interest (i.e., deformations).</td>
<td>Aleatory modeling 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 modeled and, therefore, represents variability (random differences) between model predictions and observations.</td>
</tr>
<tr>
<td>Parametric</td>
<td>Parametric epistemic uncertainty is associated with the estimate of model parameters given available data, indirect measurements, etc.</td>
<td>This uncertainty is similar to aleatory modeling uncertainty. However, this is variability that may be due to factors that are random, but have a systematic effect on model results.</td>
</tr>
</tbody>
</table>
Figure 2-1  Flow chart of elements of SSI analyses correlated with text
Box 4
Direct Method (Chapter 7)
- Soil (linear, equivalent linear, nonlinear)
- Structure (linear, equivalent linear, nonlinear)
- Interface foundation/soil-rock
  - Bonded (linear)
  - Gapped, uplift, sliding (nonlinear)
- Soil island boundary definition
  Instrumental in defining free-field ground motion, material properties, interface boundary conditions

Box 5
Substructure Method (Chapter 7)
- Superposition
- Linear, equivalent linear
  - Soil
  - Foundation/structure
  - Interface foundation/soil-rock boundary conditions

Box 6
Site Response Analysis (SRA) (Chapter 6)
- Develop seismic input for soil island boundaries
- Account for complex wave fields (Chapter 5) and nonlinear soil properties (Chapter 3)

Box 7
Site Response Analysis (SRA) (Chapter 6)
- Develop seismic input to SSI model
  - Foundation Input Response Spectra (FIRS)
- Develop equivalent linear soil properties (Chapter 3)
- Account for complex wave fields (Chapter 5)
- Input consistent with SSI analysis software (Chapter 7)

Figure 2-1 Flow chart of elements of SSI analyses correlated with text (cont.)
<table>
<thead>
<tr>
<th>Box 8</th>
<th>SSI Analyses (Chapters 7,8)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Linear, equivalent linear, nonlinear</strong></td>
<td></td>
</tr>
<tr>
<td><strong>Deterministic or probabilistic</strong></td>
<td></td>
</tr>
<tr>
<td><strong>Response quantities of interest</strong></td>
<td></td>
</tr>
<tr>
<td>- Structure for design or assessment (forces, moments, stresses or deformations, story drift, number of cycles of response);</td>
<td></td>
</tr>
<tr>
<td>- Input to the seismic design, qualification, evaluation of subsystems supported in the structure (time histories of acceleration and displacement), in-structure response spectra (ISRS), number and amplitude of cycles for components, etc.;</td>
<td></td>
</tr>
<tr>
<td>- Base-mat response for base-mat design;</td>
<td></td>
</tr>
<tr>
<td>- Soil pressures for embedded wall designs;</td>
<td></td>
</tr>
<tr>
<td>- Structure-soil-structure analysis;</td>
<td></td>
</tr>
<tr>
<td><strong>Uncertainty in seismic responses</strong></td>
<td></td>
</tr>
<tr>
<td><strong>Sensitivity studies (Chapter 7,8)</strong></td>
<td></td>
</tr>
</tbody>
</table>

Figure 2-1  Flow chart of elements of SSI analyses correlated with text (cont.)
3 SITE CONFIGURATION AND SOIL PROPERTIES

3.1 SITE CONFIGURATION AND CHARACTERIZATION

The general requirements for site characterization are defined in IAEA Safety Standard NS-R-3 [1]. The size of the region to which a method for establishing the hazards associated with earthquakes is to be applied shall 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 soil-structure interaction (SSI) analyses, description of the soil configuration (layering or stratigraphy) and characterization of the dynamic (material) properties of the subsurface materials, including the uncertainties associated to them, shall be investigated and a soil profile for the site, in a form suitable for design purposes, shall be determined. Lateral variations in the soil profile and uncertainties associated with them shall 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 shall be characterized from both geological and geotechnical investigations. Attention is drawn on the fact that the incoming seismic waves and therefore the seismic response of soils and structures are not controlled only by the properties of layers in the immediate vicinity of the structures. Therefore, large scale geological investigations are required to define the lateral and in-depth extent of the various strata, the underground topography, the possible existence of basins (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), the depth to the bedrock, the elevation of the water table, etc. As stated in IAEA SSG-9 [2], geological and geotechnical investigations should be performed at four spatial scales: regional, near regional, close vicinity to the site and site area. The typical scales go from several tens of kilometres to some hundreds of metres. These data should be compiled to form a geographical information system (GIS) and should produce a geological map for the site.

In addition, the geotechnical investigations shall allow assessing the mechanical characteristics required 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 shall 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 should allow addressing the following aspects for SSI analyses: is the geological setting hosting the site characterized by significant lateral changes (e.g., presence
of a small-scale basin or a valley, vicinity of basin border? does the site geometry need to be represented for the site response analyses with a 2D or 3D model? To help reaching such a decision, the geotechnical engineer may take advantage of his/her own experience on previous studies, simplified rules proposed in the outcome of the European research project NERA (2014) [3] or other references (e.g. Bard and Bouchon, 1985; Moczo et al 1996, Meza-Fajardo et al, 2016) or be guided by site instrumentation (Section 3.5.1). Whatasoever the justification, the designer shall be able to document and justify his/her choice.

Within the framework of the NERA project [3], aggravation factors are defined as the ratios between the spectral acceleration at the ground surface for a 2D configuration to the same quantity for a 1D configuration; aggravation factors different from 1.0 reveal a 2D effect. Pitilakis et al, (2015) [4] tentatively concluded the following based on extensive numerical analyses, involving linear and nonlinear soil constitutive models:

- The aggravation factors are period dependent,
- The average (over the period range) aggravation factors depend on the geographical location of the site with respect to the basin edges,
- The aggravation factors for locations close to the valley edges are smaller than 1.0,
- 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 2D effect may be considered of minor importance; they may reach high values if the basin period is large.

These results with more precise recommendations, still to be published, combined with seismic instrumentation, constitute helpful guidelines to estimate the potential for 2D effects. It is not implied by the above statements that the Aggravation Factors calculated in the NERA project be directly used to quantify 2D amplifications; they should rather be used as a guide to identify situations where 2D effects are significant.

### 3.2 SOIL BEHAVIOR

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 (1996), during loading, the solid particles which form the soil skeleton undergo irreversible motions such as slips at grain boundaries. In addition, Mindlin and Deresiewicz (1953) show that inelastic, elastic-plastic, irreversible deformation is present for any amount of shear stress 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 neighboring particles to define a new microstructure. 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 microscale, soils, as many other materials, may be idealized at the macroscale as behaving like continua.

In most cases, the soil element is subjected to general three-dimensional state of time-varying stresses. Thus, an adequate modelling of the soil behaviour requires 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 3-1 shows pure shear response of soil subjected to two different shear strain levels. It is important to note that for very small shear strain cycles (Figure 3-1, top) response is almost elastic, with a very small hysteretic loop (almost no energy dissipation). On the other hand, for large shear strain cycles (Figure 3-1, bottom) there is a significant loss of (tangent) stiffness, as well as a significant energy dissipation (large hysteretic loop).

![Figure 3-1. Predicted pure shear response of soil at two different shear strain amplitudes, from Pisano and Jeremic (2014).](image-url)
One of the most important features of soil behaviour is the development of volumetric strains even under pure shear, Ishihara (1996). 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 model 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 $p$: $\sigma' = \sigma + p\delta$ where $\delta$ is the Kronecker symbol and $p$ is the pore fluid pressure.

Figure 3-2 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).

As previously said, the soil element is subjected to general three-dimensional state of time-varying stresses. However, it is frequently assumed that a one-dimensional 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 three-dimensional constitutive models although there is not a single model which can be said to be superior to the others, many of which being still under development today. Chapter 7 provides further description of 3D material models, and references to most commonly used models.

---

5 The sign convention of continuum mechanics is used throughout the document with tensile stresses and pore water pressures positive
3.3 EXPERIMENTAL DESCRIPTION OF SOIL BEHAVIOUR

As mentioned previously simple one-dimensional models can be used to describe soil behaviour; for instance, it is commonly admitted for site response analyses, or for soil structure interaction problems, to consider that the seismic horizontal motion is caused by the vertical propagation of horizontally polarized shear waves. Under those conditions, a soil element within the soil profile is subjected to shear stress ($\tau$) cycles similar to those presented in Figure 3-3.

\[ \gamma = \frac{\Delta u}{\Delta h} \]

Depending on the amplitude of the induced shear strain, defined as $\gamma = \Delta u/\Delta h$, different types of nonlinearities take place. When the shear strain amplitude is small, typically less than $\gamma_s = 10^{-5}$ the behavior 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 $\gamma_v = 1 \times 10^{-4}$ to $5 \times 10^{-4}$, nonlinearities take place in shear while the volumetric behaviour remains essentially elastic (Figure 3-4, left). It is only for larger than $\gamma_v$ strains that significant volumetric strains take place in drained conditions (Figure 3-4, right), or pore water pressures develop under undrained conditions.

These different threshold strains are summarized in Figure 3–5 adapted from Vucetic and Dobry (1991) [8]; similar results are presented by Ishihara (1996) [7] 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 (Table 3-1). Accordingly, the number and complexity of constitutive parameters needed to characterize the behaviour will increase with increasing shear strains.
Figure 3-4. Experimental stress-strain curves under cyclic loading: nonlinear shear behaviour (left); volumetric behaviour (right)

Figure 3-5. Threshold values for cyclic shear strains

<table>
<thead>
<tr>
<th>CYCLIC SHEAR STRAIN AMPLITUDE</th>
<th>BEHAVIOUR</th>
<th>ELASTICITY and PLASTICITY</th>
<th>CYCLIC DEGRADATION in SATURATED SOILS</th>
<th>MODELLING</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very small ( \gamma \leq \gamma_s )</td>
<td>Practically linear</td>
<td>Practically elastic</td>
<td>Non degradable</td>
<td>Linear elastic</td>
</tr>
<tr>
<td>Small ( \gamma_s \leq \gamma \leq \gamma_v )</td>
<td>Non-linear</td>
<td>Moderately elasto-plastic</td>
<td>Practically non-degradable</td>
<td>Viscoelastic Equivalent linear</td>
</tr>
<tr>
<td>Moderate to large ( \gamma \geq \gamma_v )</td>
<td>Non-linear</td>
<td>Elasto-plastic</td>
<td>Degradable</td>
<td>Non-linear</td>
</tr>
</tbody>
</table>
3.3.1 Linear viscoelastic model

In the strain range $\gamma \leq \gamma_v$, more or less pronounced nonlinearities and energy dissipation become apparent in the shear-strain curve (Figure 3-4 and Figure 3-6).

As a linear viscoelastic model (Figure 3-7) 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 observation.

![Figure 3-6. Shear stress-shear strain curves for constant amplitude cyclic loading](image)

For harmonic loading, $\gamma = \gamma_m e^{i\omega t}$, the one-dimensional stress-strain relationship $\tau = G\gamma + C &\&$ (Figure 3-7) can be written:

$$\tau_m = G\left[1 + \frac{C_0}{G}\right]\gamma_m = G^* \gamma_m$$  \hspace{1cm} (3.1)

![Figure 3-7. One-dimensional viscoelastic model](image)

In equation (3.1), $G^*$ is a complex valued modulus formed from the elastic (shear) modulus $G$ and $\eta = C_0/G$, called the loss coefficient; this parameter is related to the energy $\Delta W$ dissipated during one cycle of loading.

$$\Delta W = \pi C_0 \gamma_m^2 = \pi G\eta \gamma_m^2$$  \hspace{1cm} (3.2)
The maximum elastic energy that can be stored in a unit volume of a viscoelastic body is given by:

\[ W = \frac{1}{2} \tau_m \gamma_m = \frac{1}{2} G\gamma_m^2 \]  

(3.3)

Then, from equations (3.2) and (3.3):

\[ \eta = 2\beta = \frac{1}{2\pi} \frac{\Delta W}{W} \]  

(3.4)

where \( \beta \) is known as the equivalent damping ratio.

Equivalent linear viscoelastic models are defined by a constitutive relationship (for one dimensional loading) of the type given by equation (3.1) where the complex valued \( G^* \) must be defined to yield the same stiffness and damping properties as the actual material. This complex modulus is defined from two experimental parameters (see Section 3.4: \( G \) and \( \eta \) or \( \beta \)). Several models have been proposed to achieve that purpose. Their characteristics are defined in Table 3-2.

TABLE 3-2: CHARACTERISTICS OF EQUIVALENT VISCOELASTIC LINEAR CONSTITUTIVE MODELS

| MATERIAL | COMPLEX MODULUS \( G^* = \tau/\gamma \) | DISSIPATED ENERGY IN ONE CYCLE \( \Delta W \) | MODULUS \( |G^*| \) |
|----------|-------------------------------------|-------------------------------------|-----------------|
| MODEL 1  | \( G = [1 + i\eta] \)             | \( \pi G \eta \gamma_m^2 \)          | \( G \)         |
| MODEL 2  | \[ G e^{i\theta} \eta = 2\sin(\theta/2) \] | \( \pi G \eta \gamma_m^2 \sqrt{1 - \frac{\eta^2}{4}} \) | \( G \)         |
| MODEL 3  | \( G \left[ \sqrt{1 - \eta^2} + i\eta \right] \) | \( \pi G \eta \gamma_m^2 \)          | \( G \)         |

The first two models were developed by Seed and his co-workers, Seed et al. (1970) [9]; the third one is due to Dormieux (1990) [10]. Examination of Table 3-2 shows that the first model, which is the simplest one, adequately duplicates the dissipated energy but overestimates the stiffness; the second one duplicates the stiffness but underestimates the dissipated energy and the third one is the only one fulfilling both conditions. In standard practice, the most commonly
used model is the second one; this model is implemented in several frequency domain software programs as SHAKE, FLUSH, SASSI (Section 8).

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 (Figure 3-6) and the damping ratio are plotted as function of shear strain as depicted in Figure 3-8.

![Figure 3-8. Example of nonlinear characteristics of soil](image)

A common mistake in the characterization of the shear behaviour is to measure the $G/G_{\text{max}}$ curves in the lab 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 $G_{\text{max}}$ but also the shape of the curve depends on the confining pressure (Ishihara, 1996) [7]). 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 $G/G_{\text{max}}=f(\gamma/\gamma_r)$, where $\gamma_r$ is a reference shear strain defined as the ratio of the maximum shear strength to the elastic shear modulus $\gamma_r=\tau_{\text{max}}/G_{\text{max}}$, (Hardin & Drenvich, 1972) [11].

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 $1\times10^{-3}$ to $5\times10^{-3}$ depending on the material type). Their main limitation, aside from their range of validity, is their inability to predict permanent deformations.
Extension to three-dimensional situations is straightforward in the framework of viscoelasticity. The constitutive relationship is simply written in terms of complex moduli and writes\(^6\):

\[
\sigma = C^* : \varepsilon
\]  

(3.5)

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

### 3.3.2 Nonlinear one-dimensional model

The definition of one-dimensional 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. 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 is the so-called Masing’s rule (1926) \[12\] or extended Masing’s rule (Pyke 1979 \[13\]; Wang et al 1980 \[14\]; Vucetic 1990) \[15\]. Assuming that the backbone curve is anti-symmetric with respect to the strain parameter, which is approximately true in shear when Bauschinger effect is neglected, then the stress-strain relationship during loading-unloading sequences is defined by:

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

(3.6)

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 3-4 (left) where 0a 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 (Konder 1963 \[16\], Hardin and Drnevich 1972 \[17\]) and Ramberg-Osgood (1943) \[17\] are the most popular ones. In the hyperbolic model, the backbone curve is written:

\[^6\] 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).
where $\tau_{\text{max}}$ represents the shear strength and $\gamma_r$ the reference shear strain. Some modifications have recently been proposed to eq. (3.7) to better fit the experimental data by raising the term $\gamma/\gamma_r$ in the denominator to a power exponent $\alpha$. It must be pointed out that such modification is not consistent with the behaviour at large strains since $\tau/\tau_{\text{max}} \sim (\gamma/\gamma_r)^{1-\alpha}$; 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:

$$\gamma = \frac{\tau}{G_{\text{max}}} \left[ 1 + \alpha \left( \frac{\tau}{\tau_y} \right)^{\frac{R-1}{\alpha}} \right]$$  (3.8)

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

For harmonic loading under constant amplitude strains (Error! Reference source not found.), the secant shear modulus and loss coefficient are defined by:

- **Hyperbolic model**

$$\frac{G_s}{G_{\text{max}}} = \frac{1}{1 + \gamma/\gamma_r}, \quad \eta = \frac{4}{\pi (\gamma/\gamma_r)^2} \left[ \frac{\gamma}{\gamma_r} \left( 2 + \frac{\gamma}{\gamma_r} \right) - 2 \left( 1 + \frac{\gamma}{\gamma_r} \right) \ln \left( 1 + \frac{\gamma}{\gamma_r} \right) \right]$$  (3.9)

- **Ramberg-Osgood model**

$$\frac{G_s}{G_{\text{max}}} = \frac{1}{1 + \alpha \left( \tau/\tau_y \right)^{\frac{R-1}{\alpha}}}, \quad \eta = \frac{4}{\pi R + 1} \frac{\alpha \left( \tau/\tau_y \right)^{\frac{R-1}{\alpha}}}{1 + \alpha \left( \tau/\tau_y \right)^{\frac{R-1}{\alpha}}}$$.  (3.10)

Equations (3.9) and (3.10) establish the link with the parameters of the viscoelastic equivalent linear models and form the basis for the determination of the constitutive parameters.

Iwan (1967) [18] introduced a class of 1D-models that leads to stress-strain relations which 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 one-dimensional class of models are extended to three-dimensions 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 (Figure 3-9) 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.

![Figure 3-9. Rheological models for Iwan’s nonlinear elastoplastic model](image)

Several software programs have implemented 1D-nonlinear constitutive models for site response analyses; the most commonly used are DEEPSOIL, DESRA, SUMDES, D-MOD, TESS. One can refer to Stewart et al (2008) [19] for a complete description of these models.

1D-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.

### 3.3.3 Nonlinear two and three-dimensional models

Unlike the viscoelastic models presented in Section 3.3.1, the extension of 1D-nonlinear models to general 2D or 3D states is not straightforward. Usually, such models are based for soils, which are mostly rate-independent materials, on the theory of incremental plasticity. Unlike in 1D-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, elastoplastic 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 $\dot{\varepsilon}$ is the sum of an elastic component $\dot{\varepsilon}^e$ and of a plastic component $\dot{\varepsilon}^p$. The incremental stress-strain constitutive equation is written:

$$\dot{\sigma} = C : (\dot{\varepsilon} - \dot{\varepsilon}^p) \quad (3.11)$$

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 three dimensional states of Iwan’s model. Among all models the commonly used seem to be Prevost’s models (1977) [20], (1985) [21], based on the concept of multiyield surfaces (Mroz, 1967) [22], and Wang et al. [15] (1990) models based on the concept of bounding surface (Dafalias, 1986) [23].

3.4 LIMITATIONS OF EQUIVALENT LINEAR MODELS

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

Frequently used program SHAKE (and its derivative) construct its complex shear modulus with the second model described in Table 3-2, 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/Gmax curve and damping ratio curve, and iterates. Therefore, damping for SHAKE modelling is fully controlled by the damping curve. It is noted that 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 should be assigned less damping. More details and one example are provided in annex 1. Some attempts have been made to implement frequency dependent shear modulus and damping (Kausel and Assimaki, (2002); 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) (Kausel and Roësset 1992, Kausel, 2017).

In essence, the solution involves: (1) computing the Fast Fourier Transform 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; and (3) 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. An example of the efficiency of the method is presented in the appendix.

3.5 PHYSICAL PARAMETERS

The essential parameters that need to be determined are the parameters entering the constitutive models described in Section 3.2. In addition, the natural frequency of the soil profile proves to be 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 soils characteristics. Determination of the natural frequency is covered in section 3.5.1.

The parameters entering 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 required. 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 (Figure 3-8); these curves are mainly influenced by the present state of stresses and the soil plasticity index.

For 1D-nonlinear models, in addition to the previous parameters, the maximum shear strength that can be developed under simple shear loading is required (equation (3.12)); 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 overconsolidation ratio of the soil.

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

(3.12)

For saturated soils under undrained conditions, the maximum shear strength is the cyclic undrained shear strength of the soil.
For the 2D or 3D-nonlinear models the same parameters as for the 1D-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 ease their determination.

3.6 FIELD 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 should provide the necessary parameters to enter the constitutive models that are used for the soil structure interaction analyses. These parameters can be determined from site instrumentation, field tests and laboratory measurements.

3.6.1 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 occurrence. The basic motivation for this additional instrumentation is to record very 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 (AVM); this determination is easier when a strong impedance contrast exists between the soil layers and the substratum. As indicated in Section 3.1, this parameter is a good proxy to decide of the importance of 2D effects in presence of a basin. It should however be reminded that H/V ratios do not allow to predict directly site amplification.

This method consists in 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 (Nakamura, 1989) [24]. Guidelines were produced by the SESAME research programme (SESAME 2004) [25] 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, in this case, care should 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.

Active measurement of seismic events is recommended with installation of instruments that allow recording, 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 are 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.

### 3.6.2 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 from High Resolution Seismic Reflection Survey (HRSRS) 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 (ISSMFE, 1994 [26]; Pecker, 2007 [27]). The most reliable and versatile techniques are invasive techniques based on in-hole measurements; these include the downhole, and crosshole tests but also the suspension logging tests. Use of crosshole tests with three aligned boreholes, one emitter and two receivers, is highly recommended 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, crosshole tests with two receiver boreholes arranged in two orthogonal horizontal directions are recommended. The invasive techniques can be advantageously complemented with non-invasive techniques like the MASW tests (Multichannel Analysis of Surface Waves) or H/V measurements as described in the previous section (Foti et al., 2014 [28]). Other field tests like the seismic cone test (Campanella and Stewart, 1992) may also be used; being less expensive than crosshole tests multiple seismic cone tests may provide indication on the spatial variability of the soil properties; it is however recommended that at least one seismic cone test be calibrated against a crosshole 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 require drilling of at least one borehole, making them quite expensive.
Non-invasive techniques provide cost efficient alternatives. In the last decades the methods based on the analysis of surface wave propagation are getting more and more recognition. These methods can be implemented with a low budget without impacting the site. However, they need processing and inversion of the experimental data, which should be carried out carefully. The surface-wave inversion is indeed non-linear, ill-posed and it is affected by solution non-uniqueness. This leads sometimes to strongly erroneous results causing a general lack of confidence in non-invasive methods in the earthquake engineering community.

The invasive techniques have no theoretical limitations with regards to the depth of investigation; the non-invasive techniques are limited to shallow depth characterization, typically of the order of 20m to 50m. 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 site heterogeneity and spatial variation of the 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 (Renault, 2009 [29]), the dispersion curves measured in MASW tests were used to reject or keep possible alternatives of the soil velocity profiles and thereby allowing to reduce the epistemic uncertainties in this parameter.

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

### 3.6.3 Laboratory investigations

Laboratory tests are essential to measure dynamic soil properties under various stress conditions, such as those that will prevail on 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 tests and laboratory tests is mandatory to establish a complete description of the material behaviour from the very low strains to the moderate and large strains. However, it is essential that these tests be 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. Remoulding of samples can be qualitatively assessed by submitting the samples to X-Ray diffraction; remoulding will be detected by the presence of curved shapes strata. Laboratory tests can be classified in three categories:

- 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 those 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.

- Resonant tests: these tests are known as resonant column tests (Wood, 1998 [30]). 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.

- Forced vibration tests: cyclic triaxial tests and simple shear tests belong to that 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 $\varepsilon$ are the measured parameters. Therefore, determination of the shear modulus from cyclic triaxial tests requires the knowledge of Poisson’s ratio ($\nu$):

$$\gamma = (1 + \nu) \varepsilon, \quad G = E/2(1 + \nu)$$  \hspace{1cm} (3.13)

Poisson’s ratio is usually not directly measured but if the tests is conducted on a saturated sample under undrained conditions, $\nu = 0.5$. 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 3D 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 the very small strains up to failure can only be achieved by combining different tests. Furthermore, as previously noted, comparison of bender tests and resonant column tests with field tests is a good indicator of the sample representativity and quality.
3.6.4 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. Figure 3-10 (from ASCE 4-16 [31]) 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 for laboratory tests from sample remoulding or lack of representativeness of the sample or for field tests from measurement errors (erroneous detection of the wave arrival, reflexion-refraction on layers of small thickness). These discrepancies should be analysed and possibly reconciled. Typically, differences smaller than 50% on the shear modulus are acceptable.
(see section 3.7 and figure 3-13 indicating that even with the best measurements a COV of 0.15 is unavoidable on Vs).

### 3.6.5 Summary of required parameters and measurements techniques

Table 3-3 summarizes the parameters required 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 2D/3D software for soil structure interaction analyses and do not include codes for 1D site response analyses.

#### Table 3-3: Required Constitutive Parameters

<table>
<thead>
<tr>
<th>MODEL</th>
<th>PARAMETERS</th>
<th>MEASUREMENT TECHNIQUES</th>
<th>SOFTWARE EXAMPLES (*)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Elastic Yield / Failure Dilation</td>
<td>Crosshole, Downhole MASW CPT</td>
<td>Resonant column Bender</td>
<td>ANSYS ABAQUS PLAXIS SAP2000 SOFISTIK Real ESSI</td>
</tr>
<tr>
<td>Linear elastic ( G_0, B_0 )</td>
<td>-</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>Linear viscoelastic ( G_0, B_0 ), ( \beta_r, \beta_s )</td>
<td>-</td>
<td>-</td>
<td>SASSI CLASSI MISS3D FLUSH Real ESSI</td>
</tr>
<tr>
<td>Equivalent linear viscoelastic (including iterations) ( G_0, B_0 ), ( \beta_r, \beta_s )</td>
<td>( G(\gamma) ), ( B(\gamma) ), ( \beta_r(\gamma) ), ( \beta_s(\gamma) )</td>
<td>-</td>
<td>FLUSH SUPERFLUSH</td>
</tr>
<tr>
<td>Nonlinear elastoplastic (**) ( G_0, B_0 )</td>
<td>Nonlinear shear and volumetric stress-strain curves Strength characteristics ( (C, \phi) )</td>
<td>Dilation angle Dilation rate</td>
<td>Resonant column Bender Cyclic triaxial Cyclic simple shear</td>
</tr>
</tbody>
</table>

(*) See section 9 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 few to several tens are provided here; an example of complete description of a particular constitutive model is provided in the appendix. In the previous table 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)

### 3.7 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, $G/G_{\text{max}}$ 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), Ishibashi and Zhang (1993), EPRI (1993)) may also be useful for validation. However, these curves should be used with caution and should not replace site specific measurements. Validation shall not be overlooked: results should be critically examined since, as indicated previously, those constitutive models are only valid for strains smaller than a given threshold $\gamma$. If results of analyses indicate larger strains, then the constitutive models should be modified and nonlinear models should be advocated.

Calibration of nonlinear constitutive models is more fastidious and uncertain. For 1D models, the shear strain-shear stress behaviour can be fitted to the experimental data using equations (3.9) and (3.10), but attention must be paid to the energy dissipation: elastoplastic models are known to overestimate this parameter, which means that the damping ratio in equations (3.9) and (3.10) is likely to exceed the experimental data.

For nonlinear 3D models, calibration is even more difficult due to the coupling between shear and volumetric strains. It is stressed that calibration should 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 in reproducing additional experiments that were not used for calibration; these validations should 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 3D modelling of soil need to feature rotational kinematic hardening, in order to reproduce cyclic behaviour. In addition, 3D 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 chapter). For example, material models in the SaniSand and SaniClay family of models (Dafalias et al.) are able to reproduce most of these effects, however they require extensive laboratory testing for calibration. On the other hand, models by Prevost (Prevost 1996) that are based on rotational kinematic hardening concept have also been successfully used. In addition, recently models, developed in particular to match G/Gmax and damping curves (Pisano and Jeremic 2015) can be used, and require only 5 parameters. The most important point is that these full 3D elastic plastic material models are able to reproduce 1D cyclic behaviour of soil, and are defined in full 3D and are thus able to work for general 3D problems.

It is important to note that 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 (Pecker et al, 2016 [32]). 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 3-11. 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.

![Figure 3-11. Cyclic stress-strain loops for a soil element with shear strength 65 kPa subjected to a sinusoidal input seismic motion of 10 s. Predictions produced by the different constitutive models.](image)
When detailed experimental data are not available for calibration, equations (3.9) and (3.10) can still be used for the definition of the shear stress-shear strain behaviour; liquefaction resistance curves, like those derived from SPT or CPT tests, can then be used to calibrate the volumetric behaviour (Prevost & Popescu, 1996 [5]) but, obviously, 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).

3.8 UNCERTAINTIES

Soil properties in a homogeneous soil layer are affected by a series of uncertainties, such as inherent variability (see Section 3.8), random test errors, systematic test errors, etc. Figure 3-12 above shows uncertain data from SPT tests, that 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 (ASCE 41-13 [33], ASN2/01 [34], ASCE 4-16 [31]). If additional testing is performed the range could be wider to values obtained by multiplying or dividing the mean value by \((1+\text{COV})\) where \(\text{COV}\) is the coefficient of variation; based on the benchmark results described below, it is recommended that, however, under no conditions \(\text{COV}\) be taken smaller than 0.5.

In the framework of the SIGMA project (Pecker et al, 2016 [32]), the InterPACIFIC project (Intercomparison of methods for site parameter and velocity profile characterization) compares the 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, by and large, higher for non-invasive techniques - coefficients of variation (COV=0.1 to 0.15) than for invasive ones (COV<0.1) as shown in Figure 3-13 for the three tested sites (Garofalo et al, 2015 [35]). These values are significantly less than those recommended by ASCE which would for the modulus imply that the mean value be multiplied by at most $1.15^2=1.3$; however, it should be considered that 10 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.

![Figure 3-13: Comparison of invasive and non-invasive $V_S$ COV values as a function of depth at Mirandola (MIR; a,b), Grenoble (GRE; c) and Cadarache (CAD;d)](image)

### 3.9 SPATIAL VARIABILITY

The necessarily limited number of soil tests and their inherent lack of representability are significant sources of uncertainty in the evaluation of site response analyses, while the uncertainty on the accuracy of analytical or numerical models used for the analysis is in general 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, such as (Assimaki et al, 2003 [36]): inherent spatial variability, random test errors, systematic test errors (or bias), transformation uncertainty (from index to design soil properties) etc. 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) [37], it is concluded that: (1) most soil properties exhibit skewed, non-Gaussian distributions, and (2) each soil property can follow different probability distributions for various materials and sites, and therefore the statistics and the shape of the distribution function have to be estimated for each case. It is important to realize that the correlation distances widely differ in natural deposits between the vertical and the horizontal directions: typically, in the vertical direction, due to the deposition process the correlation distance is typically of the order of a meter or less, while in the horizontal direction it may reach several meters. Consequently, accurate definition of a site specific distribution can never be achieved in view of the large number of investigation points that would be needed; if for low frequency motions, long wave lengths, a reasonable estimate can be done, description of small scale heterogeneities is clearly out of reach and one has to rely on statistical data collected on various sites.

3.10 REFERENCES


[34] American Society of Civil Engineers (ASCE), “Seismic Analysis of Safety-Related Nuclear Structures and Commentary,” Standard ASCE 4-98, 1998. (ASCE 4-16 currently in publication)


4 SEISMIC HAZARD ANALYSIS (SHA)

4.1 OVERVIEW

The basic concept and methodology for seismic hazard evaluation are described in IAEA SSG-9 “Seismic Hazards in Site Evaluation for Nuclear Installations (2010)” [4-1]. IAEA SSG-9 describes Deterministic Seismic Hazard Analysis (DSHA) and Probabilistic Seismic Hazard Analysis (PSHA). Figure 4-1 shows the various steps in the seismic hazard analysis (assessment) process.

The elements of a Seismic Hazard Analysis (assessment) SHA are:

- Data collection and developing seismo-tectonic models;
  - Existing geological, geophysical, and geotechnical (GGG) data complemented by seismological data are collected, and implemented in a data base (if not already in a GIS database);
  - Review data and recommend obtaining additional data, if deemed essential;
  - Document;
- Seismic source characterization:
  - Identify seismic sources to be considered in the SHA – faults and area sources;
  - Characterize the faults and area sources accordingly;
- Selection of ground motion prediction equations (GMPEs) supplemented by seismic source simulations:
  - Peak ground acceleration (PGA), peak ground velocity (PGV), peak ground displacement (PGD), response spectral acceleration values at specified natural frequencies (generally 5% damped);
- Quantification of hazard estimates for the site by DSHA and/or PSHA methods. Uncertainties propagated and displayed in the final results;
- Site Response Analyses (SRAs) as required for defining input to 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 Site Response Analyses.

PSHA Perspective. If the performance goal of a facility or structure, system, or component is probabilistically defined, a basic prerequisite is the development of the site specific probabilistically defined seismic hazard associated with that 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, e.g., peak ground acceleration.

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7 As of this writing (2018), SSG-9 is in the revision process. All references to SSG-9 in the present document refer to the existing version (2010) unless otherwise stated.
(PGA) or 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, e.g., 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)\(^8\) 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 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 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).

**DSHA Perspective.** Many of the steps in Figure 4-1 are equally applicable to the DSHA. TABLE 4-1 itemizes the considerations for DSHA as given in IAEA SSG-9 [4-1]. 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 sub-set for the DSHA; etc. The results of the PSHA are fully probabilistic (TABLE 4-2 and TABLE 4-3). The results of the DSHA are intended to be conservative through selection of mean or higher non-exceedance probability results (note, such NEP values are derived through assumptions in the conservative selections in the DSHA implementation of the steps of the SHA. In general, PSHA and DSHA require the performance of site response analysis.

### 4.2 INTERFACES BETWEEN SHA TEAM AND SSI ANALYSTS TEAM

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

- SSI analysts responsible for SSI analyses of structures;
- Geotechnical engineers responsible for soil characterization;
- 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);
- 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 NPP structures, systems, and components (SSCs);

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\(^8\) The terms Uniform Hazard Response Spectrum (UHRS) and Uniform Hazard Spectrum (UHS) are often used interchangeably.

60
- Risk analysts responsible for risk quantification or NPP seismic margin capacity.

The specification (termed Work Plan in SSG-9) to be issued to the SHA Team from the procurer of the services will specify conditions, such as those of IAEA SSG-9 paras. 11.15 and 11.16 reproduced in TABLE 4-2. TABLE 4-3 (SSG-9, Annex) provides a typical list of PSHA output quantities. In addition, the following supplementary request list should be considered:

A. Kappa values implemented in hazard analysis;
B. Specify whether the seismic hazard curves (horizontal components) are in terms of geomean values or peak values;
C. Commentary on uncertainties, especially the potential issue of "double-counting" aleatory and/or epistemic uncertainties in the PSHA results and then again in the site response analyses (or other site effects analyses); what elements have been included in the seismic hazard determination?
D. If the DSHA is associated with a return period or AFE, what is the value and which uncertainties have been included (e.g., a return period of 20,000 years corresponding to an AFE = 5 x 10^-5);
E. Recommendation for sets of time histories or response spectra (converted to RVT representations) to be used in the site response analyses, if performed;
F. Recommendation for time histories to be used in SSI analyses if site response analyses are not performed;
G. If PSHA produces results at locations other than hard rock (or soft rock), are the additional results in-column or outcrop? If outcrop, geological model or full column method?
H. Recommendation for 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;
I. Recommendations on how to change the UHRS from 5% damping to other smaller and larger damping values;
TABLE 4-1 ELEMENTS OF DETERMINISTIC SEISMIC HAZARD ANALYSIS (DSHA)  
(IAEA SSG-9 [4-1])

7. DETERMINISTIC SEISMIC HAZARD ANALYSIS

7.1. The assessment of seismic hazard by deterministic methods should include:
(1) Evaluation of the seismotectonic model for the site region in terms of the defined seismic sources identified on the basis of tectonic characteristics, the rate of earthquake occurrence and the type of magnitude–frequency relationship;
(2) For each seismic source, evaluation of the maximum potential magnitude;
(3) Selection of the attenuation relationships for the site region and assessment of the mean and variability of the ground motion as a function of earthquake magnitude and seismic source to site distance;
(4) Performing the hazard calculation as follows:
   (i) For each seismogenic structure, the maximum potential magnitude should be assumed to occur at the point of the structure closest to the site area of the nuclear power plant, with account taken of the physical dimensions of the seismic source. When the site is within the boundaries of a seismogenic structure, the maximum potential magnitude should be assumed to occur beneath the site. In this case, special care should be taken to demonstrate that the seismogenic structure is not capable (see Section 8).
   (ii) The maximum potential magnitude in a zone of diffuse seismicity that includes the site of the nuclear power plant should be assumed to occur at some identified specific horizontal distance from the site. This distance should be determined on the basis of detailed seismological, geological and geophysical investigations (both onshore and offshore) with the goal of showing the absence of faulting at or near the site, or, if faults are present, of describing the direction, extent, history and rate of movements on these faults as well as the age of the most recent movement. If the absence of faulting in the area is confirmed, it can be assumed that the probability of earthquake occurrence in this area is negligibly low. This investigation is typically for the range of a few kilometres to a maximum of about ten kilometres. The actual distance used in the attenuation relationships will depend on the best estimate of the focal depths and on the physical dimensions of the potential earthquake ruptures for earthquakes expected to occur in the seismotectonic province.
   (iii) The maximum potential magnitude associated with zones of diffuse seismicity in each adjoining seismotectonic province should be assumed to occur at the point of the province boundary closest to the site.
   (iv) Several appropriate ground motion prediction equations (attenuation relationships or, in some cases, seismic source simulations) should be used to determine the ground motion that each of these earthquakes would cause at the site, with account taken of the variability of the relationship, the source model simulation and the local conditions at the site.
   (v) Ground motion characteristics should be obtained by using the recommendations given in the relevant paragraphs of Section 5.
(5) Taking account appropriately of both aleatory and epistemic uncertainties at each step of the evaluation, with the consideration that the conservative procedure described in (4) has already been introduced to cover uncertainties, and double counting should be avoided.
(6) Incorporation of the site response (see para. 9.3).
TABLE 4-2 TYPICAL OUTPUT SPECIFICATION OF PROBABILISTIC SEISMIC HAZARD ANALYSES [4-1]

<table>
<thead>
<tr>
<th>ENGINEERING USES AND OUTPUT SPECIFICATION</th>
</tr>
</thead>
<tbody>
<tr>
<td>11.15. A seismic hazard analysis is usually conducted for purposes of seismic design and/or seismic probabilistic safety assessment. The work plan for the seismic hazard analysis should identify the intended engineering uses and objectives of the study, and should incorporate an output specification for the seismic hazard analysis that describes all specific results necessary to fulfil the intended engineering uses and objectives of the study, in addition to the general requirements identified.</td>
</tr>
</tbody>
</table>

11.16. To the extent possible, the output specification for the seismic hazard analysis should be comprehensive. The output specification may be updated, as necessary, to accommodate additional results, to alter the prescription of the results, and/or to reduce the scope of the results. Elements that should be considered in the output specification include (but are not limited to):

- **Ground motion parameters.** Specified ground motion parameters should be sufficient to develop the recommended results and any additional outputs required for engineering use (see the annex for typical outputs of a probabilistic seismic hazard analysis).

- **Vibration frequencies.** In addition to specific client requirements, the range and density of specified vibration frequencies for the uniform hazard spectra should be sufficient to adequately represent the input for all safety relevant structures, systems and components.

- **Damping.** Specified damping values should be sufficient to adequately represent input for, and effects on, responses of all safety relevant structures, systems and components.

- **Ground motion components.** Provision for the output of both vertical and horizontal motions should be specified.

- **The reference subsurface rock site condition.** For studies where site response analysis is performed, the output specification should include definition of the rock site condition (usually for a depth significantly greater than 30 m, corresponding to a specified value of the shear wave velocity, VS, consistent with firm rock). Rock hazard results to be developed should correspond to this reference rock site condition.

- **Control point(s).** The output specification should specify the control points (e.g. depths at the site) for which near surface hazard results are obtained. Usually, the control points include the ground surface and key embedment depths (e.g. foundation levels) for structures and components. The specified control points should be sufficient to develop adequate input(s) for soil–structure interaction analyses.
## TABLE 4-3  TYPICAL OUTPUT OF PROBABILISTIC SEISMIC HAZARD ANALYSES

<table>
<thead>
<tr>
<th>Output</th>
<th>Description</th>
<th>Format</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mean hazard curves</td>
<td>Mean annual frequency of exceedance for each ground motion level of interest associated with the suite of epistemic hazard curves generated in the probabilistic seismic hazard analysis.</td>
<td>Mean hazard curves should be reported for each ground motion parameter of interest in tabular as well as graphic format.</td>
</tr>
<tr>
<td>Fractile hazard curves</td>
<td>Fractile annual frequency of exceedance for each ground motion level of interest associated with the suite of epistemic hazard curves generated in the probabilistic seismic hazard analysis.</td>
<td>Fractile hazard curves should be reported for each ground motion parameter of interest in tabular as well as graphic format. Unless otherwise specified in the work plan, fractile levels of 0.05, 0.16, 0.50, 0.84 and 0.95 should be reported.</td>
</tr>
<tr>
<td>Uniform hazard response spectra</td>
<td>Response spectra whose ordinates have an equal probability of being exceeded, as derived from seismic hazard curves.</td>
<td>Mean and fractile uniform hazard response spectra should be reported in tabular as well as graphic format. Unless otherwise specified in the work plan, the uniform hazard response spectra should be reported for annual frequencies of exceedance of $10^{-2}$, $10^{-3}$, $10^{-4}$, $10^{-5}$ and $10^{-6}$ and for fractile levels of 0.05, 0.16, 0.50, 0.84 and 0.95.</td>
</tr>
<tr>
<td>Magnitude–distance deaggregation</td>
<td>A magnitude–distance (M–D) deaggregation quantifies the relative contribution to the total mean hazard of earthquakes that occur in specified magnitude–distance ranges (i.e. bins).</td>
<td>The M–D deaggregation should be presented for ground motion levels corresponding to selected annual frequencies of exceedance for each ground motion parameter considered in the probabilistic seismic hazard analysis. The deaggregation should be performed for the mean hazard and for the annual frequencies of exceedance to be used in the evaluation or design.</td>
</tr>
</tbody>
</table>
### Table 4-3 Typical output of probabilistic seismic hazard analyses [4-1] (CONTINUED)

| Mean and modal magnitude and distance | The M–D deaggregation results provide the relative contribution to the site hazard of earthquakes of different sizes and at different distances. From these distributions, the mean and/or modal magnitudes and the mean and/or modal distances of earthquakes that contribute to the hazard can be determined. | The mean and modal magnitudes and distances should be reported for each ground motion parameter and level for which the M–D deaggregated hazard results are given. Unless otherwise specified in the work plan, these results should be reported for response spectral frequencies of 1, 2.5, 5 and 10 Hz. |
| Seismic source deaggregation | The seismic hazard at a site is a combination of the hazard from individual seismic sources modelled in the probabilistic seismic hazard analysis. A deaggregation on the basis of seismic sources provides an insight into the possible location and type of future earthquake occurrences. | The seismic source deaggregation should be reported for ground motion levels corresponding to each ground motion parameter considered in the probabilistic seismic hazard analysis. The deaggregation should be performed for the mean hazard and presented as a series of seismic hazard curves. |
| Aggregated hazard curves | In a probabilistic seismic hazard analysis, often thousands to millions of hazard curves are generated to account for epistemic uncertainty. For use in certain applications (e.g. a seismic probabilistic safety assessment), a smaller, more manageable set of curves is required. Aggregation methods are used to combine like curves that preserve the diversity in shape of the original curves as well as the essential properties of the original set (e.g. the mean hazard). | A group of aggregated discrete hazard curves, each with an assigned probability weight, should be reported in tabular as well as graphic format. |
| Earthquake time histories | For the purposes of engineering analysis, time histories may be required that are consistent with the results of the probabilistic seismic hazard analysis. The criteria for selecting and/or generating a time history may be specified in the work plan. Example criteria include the selection of time histories that are consistent with the mean and modal magnitudes and distances for a specified ground motion or annual frequency of exceedance. | The format for presenting earthquake time histories will generally be defined in the work plan. |
Figure 4-1 Seismic Hazard Assessment Steps (IAEA, SSG-9[4-1] - FIG. 1, of Draft Safety Guide No. DS 507, as of this writing, Step 8 “Soliciting comments by Member States” revision in progress)
4.3 REFERENCES

5 SESIMIC WAVE FIELDS AND FREE FIELD GROUND MOTIONS

5.1 SEISMIC WAVE FIELDS

5.1.1 Perspective and spatial variability of ground motion

Earthquake motions at the location of interest are affected by a number of factors (Aki and Richards [5-1]; Kramer [5-2]; Semblat and Pecker [5-3]).

Three main influences are:

- Earthquake Source: An earthquake is produced by 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).

- 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 surface, soil layers. The crust is characterised 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.

- 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 shallow body waves are responsible for soil-structure interaction 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, SH, SV) and surface waves (Rayleigh, Love, etc.). Shallow depth is approximately one wave length in depth where surface waves have significant amplitudes. It is noted that depth of propagation of surface waves is a function of wave length (as noted above) and thus lower frequency waves propagate deeper than higher frequency waves. Since (most commonly) 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 dissipative waves.

Sometimes it is possible to analyze SSI effects using a 1D wave simplification (see more detailed discussion in section 6.3.3), however such simplification needs to be carefully assessed.

A usual assumption about propagation of seismic waves (P and S) is based on Snell’s law of wave refraction. Seismic waves travel from great depth (many kilometers) 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 vertical (Aki and Richards [5-1]). However, even if rock and soil layers are ideally horizontal, and if the earthquake source is very deep, seismic waves will be few degrees off vertical, depending on layering (usually 10-20 degrees off vertical) when they reach the surface. As noted above, change in stiffness of rock and soil layers results in seismic wave refraction, as shown in Figure 5-1. 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.

![Figure 5-1 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. Figure from Jeremic (2018).](image-url)
A more important source of seismic wave deviations from vertical is the fact that rock and soil layers are usually not horizontal. A number of different geologic history effects contribute to non-horizontal distribution of layers. Figure 5-2 shows one such (imaginary but not unrealistic) case where inclined soil/rock layers contribute to bending seismic wave propagation to horizontal direction. Rock basins as well as hard rock protrusions (dykes) are also common and contribute to deviation of seismic wave propagation from vertical.

![Figure 5-2](image)

Figure 5-2 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. Figure from Jeremic (2018).

It is important to note that in both the horizontal and non-horizontal soil/rock layered case, surface waves are created and propagate/transmit most of the earthquake energy at the surface that is responsible for SSI effects.

5.1.2 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 5.1.1, 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 nonuniform subsurface geological strata and topographic effects along the propagation path of the seismic waves.
Prior to the early 1990s, skepticism existed in some quarters as to the wave propagation behavior of seismic waves in the free-field and their spatial variation with depth in the soil. This skepticism 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. In the 1980s, 1990s, and continuing today, accumulated direct and indirect data verifies the phenomena.

- **Direct data** are measurements of free-field ground motion at depths in the soil. Johnson [5-4] summarizes 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\(^9\). Section 5.3 summarizes important data acquisition over the complete time period.
- **Indirect data** are measurements of response of structures with embedded foundations demonstrating reductions of motion on the foundation compared with free-field ground 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 (Johnson [5-4]; Kim and Stewart [5-5]). It should be noted that more recent work suggests that reductions might be in part due to nonlinear effects (seismic energy dissipation) within soil and the contact zone adjacent to structural foundations (Orbovic et al 2015 [5-6]).

As introduced in Section 5.1.1, one-dimensional and three-dimensional wave fields consist of particle motion that demonstrate spatial variation of motion with depth in the soil or rock media. For one-dimensional 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

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\(^9\) “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/)
horizontal plane – only appearing with a time lag. Incoherence of ground motion, on the other hand, may result from wave scattering due to inhomogeneities 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 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 on both the embedded depth and foundation width is important. Overall free-field ground motion analysis is discussed next. Chapter 6 presents site response analyses.

5.2 FREE FIELD GROUND MOTION DEVELOPMENT

There are four basic modelling approaches that are used to develop ground motions: Empirical Ground Motion Prediction Equations (GMPEs), Point Source Stochastic Simulations (PSSS), Finite-Fault Simulations (FFS), and the Hybrid Empirical Method (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 making an implicit ergodic assumption, which might not be appropriate. 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. PSSS:

The point source stochastic model (Boore 2003) 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 (Q), crustal amplification, and high frequency attenuation (kappa). Region-specific models of the geometrical spreading and Q 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 do 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. FFS:

Finite-fault simulations (FFS) for Large Scale Regional (LSR) computations provide a physical basis for the extrapolation from small magnitudes to larger magnitudes by incorporating finite-fault effects (Bao et al., 1998; Bielak et al., 1999; Taborda and Bielak, 2011; Rodgers and Pitarka, etc.). However, they have a much larger number of input parameters and therefore require greater calibration before the FFS can be reliably applied to engineering applications. The FFS methods have not yet been possible for the high frequencies of interest to nuclear power plants, 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 the seismic hazard evaluation.

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 (Campbell, 2003a) is a combination of the empirical GMPE approach and the PSSS model approach. In the HEM method, 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, Q, 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. Al Atik et al. 2014).

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 Chapter 6).

Site response analyses are currently (usually) performed for the assumptions of one-dimensional wave propagation and horizontally layered soil/rock profiles. It is becoming increasingly necessary to consider two- or three dimensional site response analysis to generate seismic input to the SSI analyses or, as a minimum, to justify the applicability of one-dimensional site response analysis. This justification applies to the effects of three-dimensional 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) (Bielak et al., 2000; Assimaki and Kausel, 2007; Assimaki et al., 2003).

It is important to note that realistic seismic motions always have three-dimensional features. That is, seismic motions feature three translations (and three rotations, obtained from differential displacements between closely spaced points (from few meters to few dozen meters to few hundred meters divided by 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 wave length (Aki and Richards, 2002). 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 (Abrahamson, 2010). 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 top soil layers.

Chapter 6 presents site response analysis in more detail.
5.3 RECORDED DATA

There exist a large number of recorded earthquake motions. Most (all) records feature data in three perpendicular directions, East-West (E-W), North-South (N-S) and Up-Down (U-D). Number of recorded strong motions, is (much) smaller. A number of strong motion databases (publicly available) exist, mainly in the east and south of Asia, west coast of North and South America, and Europe. There are regions of the world that are not well covered with recording stations. These regions that are not covered with recording stations also happen to be fairly 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.

**Ergodic Assumption.** 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 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 2019 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.

5.3.1 3D versus 1D 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 (that arrive a bit later, Secondary). On the other hand, the presence of vertical motions at about the same time when horizontal motions appear, indicates presence of inclined S
waves and, more importantly, Rayleigh surface waves. If vertical motions are not present (or have very small magnitude) during occurrence of horizontal motions 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) waves that surfaces (almost) vertically. One such (very rare) example is a recording LSST07 from Lotung recording array in Taiwan (Tseng et al., 1991). Figure 5-3 shows three directional recording of earthquake LSST07 that occurred on May 20th, 1986, at the SMART-1 Array at Lotung, Taiwan.

Note almost complete lack of vertical motions at around the time of occurrence of two components of horizontal motions, indicating absence of Rayleigh surface waves. In other words, a plane shear wave front was propagating vertically and surfaced as a plane shear wave front. Other recordings, at locations FA15 and FA35 for event LSST07 reveal almost identical earthquake shear wave front surfacing at the same time (Tseng et al., 1991).
Figure 5-3  Acceleration time history LSST07 recorded at SMART-1 Array at Lotung, Taiwan, on May 20th, 1986. This recording was at location FA25. Note the (almost complete) absence of vertical motions, signifying absence of Rayleigh waves. Figure from Tseng et al. (1991).

On the other hand, recording at the very same location, for a different earthquake (different source, different path) (LSST12, occurring on July 30th 1986) reveals quite different wave field at the surface, as shown in Figure 5-4. Significant vertical and horizontal motions, at the same time, do occur, indicating presence of surface waves.
It is important to note that this motion was recorded at the same location as the previous one, and yet it features all three components. One possible conclusion would be that local geology alone does not control the type of motions that are to be expected. In this case, it is known that at the Lotung SMART-1 array, geology is layered and deep. It would be thus assumed that for a deep soil geology, that is very much layered, and based on Snell’s law, motions would (should) be vertically propagating. Case presented above shows that that is not guaranteed.

5.3.2 Analytical and numerical (synthetic) earthquake models

In addition to a significant number of recorded earthquakes (in regions with frequent earthquakes and good (dense) instrumentation, analytic and numerical modeling offers another source of high quality seismic motions.

Earthquake Ground Motions: Analytical Models

There exist a number of analytic solutions for wave propagation in uniform and layered half space (Wolf, 1988; Kausel, 2006). 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 soil structure interaction response for various types of incoming ground motions/wave types (Liang et al., 2013) (P, SH, SV, Rayleigh, Love, etc.).

**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 (Bao et al., 1998; Bielak et al., 1999; Taborda and Bielak, 2011; Cui et al., 2009; Bielak et al., 2010, 2000; Restrepo and Bielak, 2014; Bao et al., 1996; Xu et al., 2002). Large scale regional models that encompass geology in detail, are currently able to model seismic motions of up to 5Hz. There are currently new efforts (US-DOE projects) that will extend modeling frequency to over 10Hz for large scale regions. Improvement in modeling and in ground motion predictions is predicated by fairly detailed knowledge of geology for 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 (Taborda and Bielak, 2013; Dreger et al., 2015; Rodgers et al., 2008; Aagaard et al., 2010; Pitarka et al., 2013, 2015, 2016) and are currently used to develop seismic free field motions for a number of large scale regions in the USA, mostly on the west coast.

It is important to note, again, that 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 anelastic material behaviour, with (seismic quality) factor Q representing attenuation of waves due to viscous (velocity proportional) and material (hysteretic, displacement proportional) effects. With that in mind, 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) has to be performed in 1D or better yet in 3D. Moreover, results from large scale regional models can also be used directly in developing seismic motions for SSI models, as described in some details in Chapter 7.

**5.3.3 Uncertainties**

Earthquakes start at the rupture zone (seismic source), propagates through the rock to the surface soils layers. All three components in this process, the source, the path through the rock and the site response (soil) feature significant uncertainties, which contribute to the uncertainty of ground motions. These uncertainties require use of Probabilistic Seismic Hazard Analysis approach to characterizing uncertainty in
earthquake motions (Hanks and Cornell; Bazzurro and Cornell, 2004; Budnitz et al., 1998; Stepp et al., 2001).

**Uncertain Sources.** Seismic source(s) feature 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 (Kramer, 1996, Silva, 1993; Toro et al., 1997).

**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 anelastic attenuation and crustal damping factor (Silva, 1993; Toro et al., 1997). These parameters are usually assumed to be log normal distributed and are calibrated based on available information/data, site specific measurements and regional seismic information. Both previous uncertainties can be combined into a model that accounts for free field motions (Boore, 2003; Boore et al., 1978).

**Uncertain Site (Soil).** Once such uncertain seismic motions reach surface layers (soil), they propagate through uncertain soil (Roblee et al., 1996; Silva et al., 1996). Uncertain soil adds additional uncertainty to seismic motions response. Soil material properties can exhibit significant uncertainties and need to be carefully evaluated (Phoon and Kulhawy, 1999a,b; DeGroot and Baecher, 1993; Baecher and Christian, 2003). Generally, for free-field motion, these uncertainties are treated in the site response analysis.
5.4 SEISMIC WAVE INCOHERENCE

5.4.1 Introduction

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 (Abrahamson et al., 1991; Roblee et al., 1996; Abrahamson, 1992a, 2005, 1992b; Zerva and Zervas, 2002; Liao and Zerva, 2006; Zerva, 2009)

The main sources of incoherence (Zerva, 2009) are:

- 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 NPPs it is not of much significance.

- 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.

- 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.

- Extended source effects contribute to incoherence by creating a detailed wave source field. As the (extended) fault ruptures, rupture propagates and generate 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 makes it to observation points.

Figure 5-5 shows an illustration of main sources of lack of correlation.
Figure 5-5 Four main sources contributing to the lack of correlation of seismic waves as measured at two observation points. Figure from Jeremic (2016).
5.4.2 Incoherence modelling

Early studies concluded that the correlation of motions increases as the separation distance between observation points decreases. In addition to that, correlation increased for decrease in frequency of observed motions. Most theoretical and empirical studies of spatially variable ground motions (SVGM) 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:

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

Coherency is a dimensionless complex-valued function, that depends on a frequency and 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 SVGM 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) has called these variations "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 utilized is exponential (Loh and Yeh, 1988; Oliveira et al., 1991; Harichandran and Vanmarcke, 1986). 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 (1985, 1992a) found that coherency $\gamma$ at short distances ($\xi < 200m$) is not dependent on wavelength. Wavelength-dependence was found at larger distances ($\xi = 400$ to $1000m$). For SVGM effects over the lateral dimensions of typical structures (e.g., $< 200m$), non-wavelength dependent models (Abrahamson, 1992a, 2007; Ancheta, 2010) are used.

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. A number of excellent references are available on the subject of incoherent seismic

**Incoherence in 3D.** Empirical SVGM 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 SVGM model developments. Development of incoherent motions for three dimensional soil/rock volumes creates difficulties.

A 2 dimensional wave-field can be developed, as proposed by Abrahamson (1993), by realizing that all three spatial axes (radial horizontal direction, transverse horizontal direction, and the vertical direction) do exhibit incoherence. Existence of three spatial directions of incoherence requires existence of data in order to develop models for all three directions. Abrahamson (1992a) 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 (1992a) and subsequent models (Abrahamson, 2006 [5-7], 2007[5-8]) 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.

Currently, 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. (2013) and the incoherence functions for the horizontal and vertical directions developed for a hard rock site by Abrahamson [5-8] 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 (< 100m). Therefore, by randomizing the energy at each depth, a set of full 3-D incoherent ground motions are created.

**Theoretical Assumptions behind SVGM Models.** It is very important to note that the use of SVGM 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 SVGM obtained from a Lotung site in Taiwan, over long period of time (dozens of years) is developed into a statistical model of SVGM 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, ergodic assumption is regularly used for SVGM models. Very recently, a number of smaller and larger earthquakes in the 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. (2013) shows good
matching of incoherent data for Parkfield, using models developed at Lotung and Pinyon Flat for nodal separation distances up to 100m. This was one of the first independent validations of family of models developed by Abrahamson and co-workers. This validation gives us confidence that assumed ergodicity of SVGM models does hold for practical purposes of developed SVGM models.

Recent reports by Jeremić, et al. (2011); Jeremić (2016) present detailed account of incoherence modeling.

**Nuclear Power Plant (NPP) – specific applications.** In the context of the above technical discussion, the treatment of the effects of seismic ground motion incoherence (GMI) or SVGM on structure response for typical nuclear power plant (NPP) structures was motivated in part by the development of Uniform Hazard Response Spectra (UHRS) with significant high frequency content, i.e., frequencies greater than 20 Hz. Figure 5-6 shows the UHRS (annual frequency of exceedance = 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 [5-7, 5-12] investigated and processed recorded motions from 12 sites as listed in TABLE 5-1 for 74 earthquakes as listed in TABLE 5-2. 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.

Abrahamson [5-8] 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 5-7.
For the latter effort, Short et al. [5-9, 5-13] and Johnson et al. [5-10] 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. [5-11] present the SSI analyses of the EPR when subjected to coherent 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 in-structure response spectra (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.

- Seismic responses (ISRS) for assumptions of coherent and incoherent ground motions are required to be calculated to permit comparisons to be made.
- Currently, the following guidelines for ISRS, representing current USA practice (NRC) are in place for design:
  (i) For the frequency range 0 to 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 to 30 Hz, a maximum reduction based on a linear variation between 0% at 10 Hz and 30% at 30 Hz is permitted.
### TABLE 5-1  ARRAYS USED TO DEVELOP THE COHERENCY MODELS [5-7]

<table>
<thead>
<tr>
<th>Array</th>
<th>Location</th>
<th>Site Class</th>
<th>Topography</th>
<th>Number of Stations</th>
<th>Station Separation (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>EPRI LSST</td>
<td>Taiwan</td>
<td>Soil</td>
<td>Flat</td>
<td>15</td>
<td>3 – 85</td>
</tr>
<tr>
<td>EPRI Parkfield</td>
<td>CA</td>
<td>Soft Rock</td>
<td>Flat</td>
<td>13</td>
<td>10 – 191</td>
</tr>
<tr>
<td>Chiba</td>
<td>Japan</td>
<td>Soil</td>
<td>Flat</td>
<td>15</td>
<td>5 – 319</td>
</tr>
<tr>
<td>USGS Parkfield</td>
<td>CA</td>
<td>Soft Rock</td>
<td>Ridge Tops</td>
<td>14</td>
<td>25 – 952</td>
</tr>
<tr>
<td>Imperial Valley Differential</td>
<td>CA</td>
<td>Soil</td>
<td>Flat</td>
<td>5</td>
<td>18 – 213</td>
</tr>
<tr>
<td>Hollister Differential</td>
<td>CA</td>
<td>Soil</td>
<td>Flat</td>
<td>4</td>
<td>61 – 256</td>
</tr>
<tr>
<td>Stanford (Temp)</td>
<td>CA</td>
<td>Soil</td>
<td>Flat</td>
<td>4</td>
<td>32 – 185</td>
</tr>
<tr>
<td>Coalinga (Temp)</td>
<td>CA</td>
<td>Soft-Rock</td>
<td>Flat</td>
<td>7</td>
<td>48 – 313</td>
</tr>
<tr>
<td>UCSC ZIYA (Temp)</td>
<td>CA</td>
<td>Soft-Rock</td>
<td>Mountains</td>
<td>6</td>
<td>25 – 300</td>
</tr>
<tr>
<td>Pinyon Flat (Temp)</td>
<td>CA</td>
<td>Hard-Rock</td>
<td>Flat</td>
<td>58</td>
<td>7 – 340</td>
</tr>
<tr>
<td>SMART-1</td>
<td>Taiwan</td>
<td>Soil</td>
<td>Flat</td>
<td>39</td>
<td>100 – 4,000</td>
</tr>
<tr>
<td>SMART-2</td>
<td>Taiwan</td>
<td>Soil</td>
<td>Flat</td>
<td>8</td>
<td>200 – 750</td>
</tr>
</tbody>
</table>

### TABLE 5-2  EARTHQUAKES IN THE ARRAY DATA SETS [5-7]

<table>
<thead>
<tr>
<th>Array</th>
<th>No. of Earthquakes</th>
<th>Magnitudes</th>
<th>Epicentral Distances (km)</th>
<th>Maximum PGA (g)</th>
</tr>
</thead>
<tbody>
<tr>
<td>EPRI LSST</td>
<td>13</td>
<td>3.0 – 7.8</td>
<td>5-113</td>
<td>0.26</td>
</tr>
<tr>
<td>EPRI Parkfield</td>
<td>2</td>
<td>3.0 – 3.9</td>
<td>13-15</td>
<td>0.04</td>
</tr>
<tr>
<td>Chiba</td>
<td>9</td>
<td>4.8 – 6.7</td>
<td>61-105</td>
<td>0.41</td>
</tr>
<tr>
<td>USGS Parkfield</td>
<td>9</td>
<td>2.2 – 3.5</td>
<td>18-45</td>
<td>0.04</td>
</tr>
<tr>
<td>Imperial Valley Differential</td>
<td>2</td>
<td>5.1 – 6.5</td>
<td></td>
<td>0.89</td>
</tr>
<tr>
<td>Hollister Differential</td>
<td>1</td>
<td>5.3</td>
<td>17</td>
<td>0.20</td>
</tr>
<tr>
<td>Stanford (Temp)</td>
<td>4</td>
<td>3.0 – 4.0</td>
<td>40</td>
<td>0.007</td>
</tr>
<tr>
<td>Coalinga (Temp)</td>
<td>4</td>
<td>3.2-5.2</td>
<td>1-5</td>
<td>0.21</td>
</tr>
<tr>
<td>UCSC ZIYA (Temp)</td>
<td>3</td>
<td>2.3 – 3.0</td>
<td>9-19</td>
<td></td>
</tr>
<tr>
<td>Pinyon Flat (Temp)</td>
<td>5</td>
<td>2.0 – 3.6</td>
<td>14-39</td>
<td></td>
</tr>
<tr>
<td>SMART-1</td>
<td>20</td>
<td>4.0 – 7.8</td>
<td>5-80</td>
<td>0.33</td>
</tr>
<tr>
<td>SMART-2</td>
<td>2</td>
<td>4.0 – 5.5</td>
<td>15-60</td>
<td>0.06</td>
</tr>
</tbody>
</table>
Figure 5-6  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)
Figure 5-7 Comparison of ground motion coherency functions for soil [5-7] and hard rock [5-8] sites
5.5 REFERENCES


6 SITE RESPONSE ANALYSIS AND SEISMIC INPUT

6.1 OVERVIEW

Seismic input is the earthquake ground motion that defines the seismic environment to which the soil-structure system is subjected and for which the SSI analyses are performed. Chapter 2 introduces the concept of free-field ground motion, i.e., the motion that would occur in soil or rock in the absence of the structure or any excavation. Chapters 3, 4 and 5 discuss free-field ground motion in more detail.

Important points that need to be noted are:

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 facility. The Design Basis Earthquake (DBE) ground motion may be based on standard ground response spectra (Section 6.4) or site specific ground response spectra developed by probabilistic seismic hazard analysis (PSHA) or deterministic seismic hazard analysis (DSHA) (Chapter 4).

- Assessments of the facility can be for hypothesized Beyond Design Basis Earthquake (BDBE) ground motions or for actual earthquake events that have occurred and require evaluation. For assessments of the facility subjected to hypothesized BDBE ground motions, PSHA-defined values play an important role, i.e., for 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 detail of 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; for embedded foundations, the variation of motion over both the embedment depth and the foundation horizontal dimensions is to be defined.

- Nonlinear effects, in the soil/rock adjacent to nuclear installation foundations and within the foundation – soil/rock contact zone 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) and the dynamic response of the soil-structure system (inertial interaction).

This chapter discusses various aspects of defining the seismic input for SSI analyses. Seismic input is closely coupled with soil property definition (Chapter 3), free-field ground motion definition (Chapter 5), and SSI analysis methodology (Chapter 7). 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. Chapter 4 presents seismic hazard assessment, which is essential to defining the Design Basis Earthquake (DBE) ground motion and the considerations for the Beyond Design Basis Earthquake (BDBE) ground motions.

6.2 SITE RESPONSE ANALYSIS

6.2.1 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 seismic hazard analysis (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 Ground Motion Prediction Equations (GMPEs) that are implemented.

- 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 Vs30 values, i.e., average shear wave velocities over the upper 30 m of soil.
  - If the GMPEs fit well the native soil/rock properties up to top of grade (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.
  - 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 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 ($V_s \geq 2,800$ 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.

- 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 site of interest. These seismic hazard values or curves could be probabilistically or deterministically defined.

- The GMPE of interest may be only that of peak ground acceleration (PGA). Then, values of PGA associated with selected frequencies of exceedance anchor spectral shapes, such as standard response spectra or others.

- Site response analysis may be needed to define or provide guidance on the soil material properties to be used in the SSI analyses:

  - Equivalent linear soil properties taking into account the strain levels induced by the free-field ground motion; 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, those induced by structure response; these equivalent linear soil properties are, usually, defined during the site response analysis; this is especially the case for sub-structuring methods;

  - Nonlinear analyses – location of boundaries of the nonlinear soil models beyond which the soil material behaviour may be treated as linear or equivalent linear; refer to Chapter 7;

- 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.

- If the free-field ground motion is defined by the seismic hazard analysis 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;

- 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 IAEA and Member States’ requirements; this check is related to the Foundation Input Response Spectra (FIRS) (Section 6.2.2). As in the case of SSI analyses, site response analyses can be performed in the frequency domain or in the time domain.

- Frequency domain analyses for earthquake ground motions are linear or equivalent linear. Often, strain-dependent, elastic soil material behavior is simultaneously defined with the definition of the free-field ground motion as seismic input (Chapter 3).

- 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 (Chapter 7).

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-structure 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, etc.).

### 6.2.2 Foundation Input Response Spectra (FIRS)

The term Foundation Input Response Spectra (FIRS) was first defined in Refs. [6-1, 6-2]. Although the definition may be interpreted in a general sense, Refs. [6-1, 6-2] 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 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, as discussed later in this chapter, there are often additional requirements to which one must adhere, which could introduce additional analysis cases to be performed. One such requirement for design is the requirement to meet a specified minimum input ground motion response spectrum at foundation level.

Background and methodologies for generation of FIRS through the geologic method and full column method are contained in Refs. [6-1, 6-2]. The primary difference between the geologic method and the full column method lies in the definition of the outcropping motion. Figure 6-1 shows a schematic of the two FIRS definitions. The geologic method requires 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 down-coming 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 up-going and down-coming waves are equal at the elevation of the FIRS.

With reference to Figure 6-1, 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 geologic 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 site response analysis without reanalysis of the iterated soil columns. For linear or nonlinear time domain analyses, the geological method is required.

In terms of SSI analysis, assuming each structure is analyzed independently of other structures, each structure has a defined Control Point and Control Motion. So for a given site, there may be multiple Control Points and Control Motions. A different point of view 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 (1D) 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 analyzed are modeled including embedment, multiple deterministic soil profiles are used in the SSI analyses. Often, three profiles are analyzed, 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.

As noted above, 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.

![Diagram of site response analysis](image)

*Figure 6-1 Definition of FIRS for idealized site profiles*

### 6.3 SITE RESPONSE ANALYSIS APPROACHES

Characteristics of site response analysis (SRA) are discussed next: convolution vs. deconvolution; probabilistic vs. deterministic; response spectra vs. 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.
6.3.1 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 (Kramer [6-3]).

In principle, for the idealized assumption of the site profile being represented by semi-infinite horizontal soil layers overlying a half-space, one-dimensional 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.

- The solution of the wave equation for one-dimensional 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.\(^{10}\)

The displacement within a soil layer \(u(z,t)\) is computed by:

\[
    u(z, t) = A e^{i(\omega t + k^* z)} + B e^{i(\omega t - k^* z)} \tag{6-1}
\]

where,

- \(u\) = displacement;
- \(z\) = the depth within a soil layer, oriented positively downwards;
- \(t\) = time;
- \(A\) and \(B\) = amplitude of waves traveling in the upward and downward directions, respectively;
- \(\omega\) = circular frequency; and
- \(k^*\) = complex wave number \(k^* = (\omega/V_s^*)\) with \(V_s^*\) the complex shear wave velocity.

- 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 \(i+1\).
- Applying the boundary conditions yields a recursive relationship for the amplitude of the upward wave and downward wave in the layers.
- 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.

\(^{10}\) In Figure 6-1, the quantities \(A_2\) and \(A'_2\) represent the upward wave and downward wave respectively.
The SRA step is applicable to free-field ground motion defined by a DSHA or PSHA. The principle 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).

Given this background, the majority of site response analyses over the last ten years implement techniques based on the following assumptions:

- Soil layer stratigraphy (semi-infinite horizontal layers overlying a uniform half-space), variability in layer thickness is modelled;
- Soil material properties (one-dimensional equivalent linear viscoelastic models defined by shear modulus and material damping – median and variability); (Equation 6-1);
- Wave propagation mechanism vertically propagating S- and P-waves.

The process is illustrated in appendix 1 using one input time history, including sensitivity studies to evaluate the effect of nonlinear material properties, incompressibility, soil permeability. Chapter 8 summarizes 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 site amplification functions (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 more 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 annual frequency of exceedance (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 over estimating 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 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 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. Appendix 1 provides a detailed flow chart of one implementation of Approach 3. Approach 3 is being 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. In current implementation of the PSHA analyses, this could be millions of simulations, which in current technology would be infeasible.

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 locations in the site profile is the starting point. This location is defined by hard rock (Vs >1500m/s or 2800m/s), soft rock (e.g., Vs > 800m/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.

In the PSHA process, the end results are itemized in Chapter 4; 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. 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 annual frequencies of exceedance (AFE). To define a UHRS at 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 (McGuire et al. [6-4, 6-5]) or in a Random Vibration Theory (RVT) approach (Rathe et al. [6-6, 6-7]), 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 then is to propagate that 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 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 accounting for the following variabilities:

- Ground motion definition at hard rock location
  - 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
  - DSHA
    - Site specific
    - Site independent

- 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;
• For each base soil case, the following parameters are defined by a probability distribution¹¹:
  - Depth to bed rock;
  - 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
• Construct sampling approach – generally a stratified sampling of the probability distributions and a Latin Hypercube Sample (LHS) experimental design; this could also be strict Monte Carlo (MC) sampling; LHS requires many less samples than MC to achieve the same accuracy;
• 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;
• Combine the results as appropriate through convolution or other approaches;
• Apply SAFs to obtain seismic input at locations of interest.

Randomness and uncertainty in the soil configuration and soil material properties are treated.¹²

Normally, at least over the last five or more years, these probabilistic analyses are performed for a series of (up to) 60 earthquake simulations.

**Deconvolution.** In other cases, the PSHA results (UHRS) are generated at a site location, usually TOG, using attenuation laws or GMPEs somewhat tailored to the site properties, in some cases through a Vs30 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 required for the SSI analysis (in fact the FIRS are calculated within the program, but are not

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¹¹ It must be stressed that the probability distribution of some parameters (depth to bedrock, layer thickness) are poorly modelled in 1D site response analyses since they truly exhibit a 3D spatial variation. The impact of such probabilistic modelling must be assessed with care.

¹² Ground motion response spectra peak and valley variability is explicitly treated in the PSHA. Consequently, the only ground motion randomness treated in the site response analyses and in the SSI analyses is randomness of phase.
transparent for the user); however, the FIRS may be required for checks against limits in the reduction of free-field ground motion at foundation level, if this quantity is required by the regulations.

In the deconvolution analyses, it may happen that divergence of the numerical scheme occurs 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.

### 6.3.2 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 appendix 1.

### 6.3.3 Analysis Models and Modelling Assumptions

#### 6.3.3.1 Perspective

Four basic methods of developing ground motions were introduced in Section 5.2: Empirical Ground Motion Prediction Equations (GMPEs), Point Source Stochastic Simulations (PSSS), Finite-Fault Simulations (FFS), and the Hybrid Empirical Method (HEM).

Currently, the Probabilistic Seismic Hazard Analysis (PSHA) and Deterministic Seismic Hazard Analysis (DSHA) are the most frequently used methods to generate the ground motion at the 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 peak ground acceleration (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 GMRS is due to topographic effects, geological effects, or wave types. One advantage of implementing site response analysis 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 sub-sections discuss one-dimensional and three-dimensional representations of the wave fields and their potential effect on SSI analyses of structures of interest. It is important to recognize that SSI analyses of nuclear installations are three-dimensional, i.e., three-dimensional soil and structure models and three spatial components of earthquake input motion. For calculation purposes, in some instances, the SSI models are analyzed for each spatial direction of input motion separately. This is possible for linear elastic material assumption. However, even in this case, the three dimensional response of the 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 site response analysis require an analyst to make modelling assumptions. Assumptions must be made about the treatment of 3D or 2D or 1D seismic wave field, treatment of uncertainties, material modelling for soil (see chapter 3 and section 7.3), 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 analyze 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.

Addressed in this section are issues related to free field modeling assumptions. Firstly, a brief description is given of modelling in 1D and in 3D. Then, addressed is the use of 1D seismic motions assumptions, in light of full 3C (3 Components) seismic motions (Abell et al., 2016). Next, an assumption of adequate propagation of high frequencies through models (finite element mesh size/resolution) is addressed as well (Watanabe et al., 2016). There are number of other issues that can influence results (for example, nonlinear SSI of NPPs (Orbovic et al. (2015)) however they will not be elaborated in much detail here, rather they will be addressed in appendix through select examples.
6.3.3.2 3D Models.

In reality, seismic motions are always three directional, featuring body and surface waves (see more in section 4.2). However, development of input, free field motions for a 3D analysis is not an easy task. Recent large scale, regional models (Bao et al., 1998; Bielak et al., 1999; Taborda and Bielak, 2011; Cui et al., 2009; Bielak et al., 2010, 2000; Restrepo and Bielak, 2014; Bao et al., 1996; Xu et al., 2002; Taborda and Bielak, 2013; Dreger et al., 2015; Rodgers et al., 2008; Aagaard et al., 2010; Pitarka et al., 2013, 2015, 2016, Maufroy et al. (2015, 2016) have shown great promise in developing (very) realistic free field ground motions in 3D. What is necessary for these models to be successfully used is the detailed knowledge about the deep and shallow geology as well as a local site conditions (nonlinear soil properties in 3D). Often this data is not available, however, when this data is available, excellent modelling of 3D SSI can be performed, with possible reduction of demand due to nonlinear effects and due to use of more realistic motions. In addition, it should be noted, that due to computational requirements, large scale regional models are usually restricted to lower frequencies (below 5Hz) while there are current projects (US-DOE) 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.

In addition, when the data is available, a better understanding of dynamic response of an NPP can be developed. Developed nonlinear, 3D response will not suffer from numerous modeling uncertainties (1D vs. 3D motions, elastic vs. nonlinear/inelastic soil in 3D, soil volumetric response during shearing, influence of pore fluid, etc.). It is important to note (emphasize) that 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, and that has to be carefully taken into consideration when this approach is used. Recent US-NRC, CNSC and US-DOE projects have developed and are developing a number of nonlinear, 3D earthquake soil structure interaction procedures, that rely on full 3D seismic wave fields (free field and site response) and it is anticipated that this trend will only accelerate as benefits of (a more) accurate modelling become understood.

3D/3C versus 1D/3C versus 1D/1C Seismic Models

It is noted, that seismic waves propagate in 3 directions (or 3 dimensions, 3D) and have all three components (3C) of translations. Sometimes, full 3D wave propagation with all three components 3C can be simplified to propagation in less than 3D, and with less than 3C. For examples, an assumption of neglecting full 3D wave propagation and replacing it with a 1D wave propagation, while still preserving all three components (3C) of motions, can sometimes be appropriate. It is also very important to note that, such simplifying assumption should 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:

- 1D wave propagation, with 1C modelling of seismic waves is possible if material modelling for soil is linear, equivalent linear elastic or nonlinear/inelastic. In case of elastic or equivalent linear elastic material, 1D/1C motions from different directions (horizontal) can be combined, as superposition principle applies for linear elastic systems (soil in this case). Modelling of vertical motions using 1D/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.

- 3 x 1C modelling of seismic waves is possible, under special circumstances, described below. Since most of the time, vertical motions are a results (consequence) of Rayleigh surface waves, it is important to analyze vertical motions and decide if modelling as 1C is appropriate. To this end, a wave length of surface wave plays an important role. If the Rayleigh surface wave length (which features both horizontal and vertical components) is longer than 12 times the dimension of the object (NPP), than object rotations, due to differential vertical displacements at object ends, are indeed fairly small and object does move up and down as if excited with a vertical wave. This is shown in Figure 6.2 below, as the upper case. In this case it is appropriate to use 3x1D 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. Case in the lower left corner of Figure 6.2 shows a limiting case where 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 Figure 6.2, lower right case, waves might not even be exciting any significant dynamic behaviour of the object (except local deformation) as their wave lengths are shorter than twice object length.

- 3D/3C modelling, will capture all the body and surface wave effects for SSI analysis of NPPs.
An example is developed in the appendix, that illustrates differences between use 3D/3C seismic motions, 1D/3C and 1D/1C seismic motions. Example is based on a recent paper by Abell et al, (2017).

6.3.3.3 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 it is required that higher frequencies of seismic motions be propagated. An illustrative example is used to analyse propagation of seismic waves of different frequencies through the finite element mesh. Resulting damping of higher frequencies is clearly observable, and should be taken into account when finite element models are designed, and decisions about mesh quality (finite element size) are made during model development. Recent paper by Watanabe et al. 2016 presents an in depth analysis of wave propagation through different mesh sizes, and for elastic as well as for nonlinear (elastic-plastic) materials.

It is known that mesh size can have significant effect on propagating seismic waves (Argyris and Mlejnek, 1991; Lysmer and Kuhlemeyer, 1969; Watanabe et al., 2016; Watanabe, 2016). Finite element model mesh (nodes and element interpolation functions) needs to be able to approximate displacement/wave field with certain required accuracy without discarding (filtering out) higher frequencies. For a given wave length $\lambda$ that is modeled, it is suggested/required to have at least 10 linear interpolation finite elements (8 node bricks in 3D, where representative element size is $\Delta h^{LE} \leq \lambda/10$) or at least 2 quadratic interpolation finite elements (27 node bricks in 3D, where representative element size is $\Delta h^{QE} \leq \lambda/2$) for modeling wave propagation.

Since wave length $\lambda$ 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:
• Linear interpolation finite elements (1D 2-node truss, 2D 4-node quad, 3D 8-node brick) the representative finite element size needs to satisfy the following condition:

\[ h_{LE}^{max} \leq \frac{v}{10 f_{max}} \]

• Quadratic interpolation finite elements (1D 3-node truss, 2D 9-node quad, 3D 27-node brick) the representative finite element size needs to satisfy the following condition:

\[ h_{QE}^{max} \leq \frac{v}{2 f_{max}} \]

It is noted that while the rule for number of elements (or element size ∆h) can be used to delineate models with proper and improper meshing, in reality having bigger finite element sizes than required 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 (Jeremic, 2016). Example analysis is presented in the appendix for linear elastic material model. When material becomes nonlinear (elastic-plastic), 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. (2016).

6.3.3.4 Material Modelling and Assumptions

Material models that are used for site response need to be chosen to have appropriate level of detail in order to model important aspects of response. For example, for site where it is certain that 1D waves will model all 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 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 site response analysis are described in some detail in section 3.2 above.

6.3.3.5 3D/3C vs 1D/3C vs 1D/1C Material Behaviour and Wave Propagation Models

In general behaviour of soil is three dimensional (3D) and nonlinear/inelastic. In some cases, simplifying assumptions can be made and soil response can be modelled in 2D or even in 1D. Modelling soil response in 1D makes one important assumption, that 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 (Muir Wood, 1990) or if soil is a fully saturated clay,
with low permeability, hence there is no volume change (see more about modeling such soil in section 6.4.2).

If soils will be excited to feature a full nonlinear/inelastic response, full 3D analysis and full 3D 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 3×1D analysis, is not right.

Recent increase in use of 1D/3C models (1D material models for 3C (3 components) wave propagation) requires further comments. Such models might be appropriate for seismic motions and behaviour of soil that is linear elastic. In addition, it should be noted that 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 1D vertically propagating waves is then appropriate. However, modelling of vertical components of surface (Rayleigh) waves as vertically propagating 1D waves is not appropriate for all frequencies, as noted above and in recent paper by Abell et al. (2017) (see also example in the appendix). Paper by (Elgamal and He, 2004) also provides nice description of vertical wave/motions modelling problems.

6.4 STANDARD AND SITE SPECIFIC RESPONSE SPECTRA

6.4.1 Introduction

The amplitude and frequency characteristics of the free-field ground motion (in 1D) 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 a seismic hazard analysis (deterministically or probabilistically derived) as discussed in Sections 5.2 and 5.3. These cases are discussed in this section.

6.4.2 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% non-exceedance probability (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. Small magnitude earthquakes contribute more to the high frequency range than to the low frequency range and so forth. 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. Such designs are intended to be easily licensed for a large number of sites in many different seismic environments and site conditions. Therefore, the Design Basis Earthquake (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 Foundation Input Response Spectra (FIRS) satisfies a minimum specified design basis ground motion:


- Appendix S to 10 CFR Part 50, the minimum PGA for the horizontal component of the SSE at the foundation level in the free-field should be 0.1g or higher. The response spectrum associated with this minimum PGA should be a smooth broadband response spectrum (e.g., RG 1.60, or other appropriate shaped spectra, if justified) and is defined as outcrop response spectra at the free-field foundation level.

Figure 6-3 provides examples of standard ground response spectra for the horizontal direction:

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

6.4.3 Site specific response spectra

For this discussion, assume a PSHA has been performed and UHRS are developed for the range of annual frequencies of exceedance of the ground motion. This range of annual frequencies of exceedance is from about 1e-02 to 1e-07. Section 1.3.2 introduces one example of performance goals, which is based on the design of 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. 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 and U.S. NRC Regulatory Guide 1.208 establish a performance-based approach to developing the DBE (termed the ground motion response spectra - 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 plant) and associate it with a lognormal probability distribution with hypothesized lognormal standard deviations (based on previously performed studies of SSC and plant performance).

For high hazard facilities, focus on a performance goal of mean annual probability of unacceptable behaviour of Pf = 1e-05. Consider two UHRS – mean 1e-04 and mean 1e-05. Develop the relationships between the individual spectral accelerations (at the discrete frequencies of the calculated seismic hazard curves) at mean annual UHRS of 1e-04 and 1e-05. Develop scale factors to be applied to the spectral accelerations (SAs) of the mean 1e-05 to obtain risk consistent GMRS. The scale factors (less than or equal to 1.0) 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 1e-05 seismic hazard curves and the GMRS is constructed by connecting the SAs at these discrete natural frequencies.

The end result is the risk consistent definition of the site specific DBE (GMRS) 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.
FIG. 6-3 Examples of site independent Design Basis Earthquake (DBE) ground motion response spectra for standard design or reference design nuclear power plants
6.5 TIME HISTORIES

Throughout Chapter 6, the discussion has been focused on ground motion response spectra and scalar descriptors of the ground motion – peak ground acceleration, velocity, and displacement. Generally, SSI analyses are performed for seismic input defined by acceleration time histories. As a minimum, three spatial components of ground motion are required, 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 6.4 describes ground motion response spectra as standard response spectra (site independent or weakly correlated to site conditions) and site specific response spectra calculated specifically for a given site at a rock outcrop, on the surface of the soil at top of grade, or at locations within the site profile. Herein, these are referred to as target spectra.

In general, the approach to developing acceleration time histories for use in the seismic analysis 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:

- Scale the selected seed time histories by constant factors over the complete record to approximately mimic the target spectra.
- Implement spectral matching software, such as RSPMatch2005 [6-8, 6-9] and its successor derivative RSPMatch2009 [6-10], which modify recorded motions through 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.

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, Sections 5.2 and 6.3.3 point out that finite-fault simulations (FFS) currently are limited in the frequency content that can be directly
generated, i.e., limited to about 3 Hz with developments in progress to increase the frequency content to about 10 Hz.

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

In generating acceleration time histories to represent or match the target response spectra, the following elements should be considered (Houston et al. [6-11]):

- 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 unconservative.
- 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 unconservative 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 should not have gaps in the comparison for any frequency and should not be overly conservative.

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

- Time step (dt) of time histories should be small enough to capture 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 should be no greater than 0.01 sec, which is a Nyquist frequency of 50 Hz.
- Strong motion duration of the time histories should be chosen in relation with the earthquake magnitude of the earthquakes 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.[6-12] If recorded motions are the basis for the generated time histories, the Arias intensity after record modification should approximate the Arias intensity prior to modification.
- Response spectra calculated from the time histories for comparison with the target spectrum should 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.

- Guidelines 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.
  - Should 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 should be calculated and verify that no gaps in energy exist;
  - Shall 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;

- The general relationships of (peak acceleration)-to-(peak velocity)-to-(peak displacement) for the single time history or average of the multiple time histories should be maintained. Other indicators like Arias intensity, Cumulative Absolute Velocity should also ideally be preserved. This holds for the standard response spectra based on published guidance and for the governing earthquakes for the site specific response spectra as determined from deaggregation of the seismic hazard curves.

- 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 should be maintained for each of the deaggregated scenarios.

- The three components of ground motion should be statistically independent as determined by the directional correlation coefficients between pairs of time histories. The absolute values of the correlation coefficients should be less than a specified amount (either 0.30 or 0.16 have been specified). 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:

- 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 beyond design basis earthquake 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:

- 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 beyond design basis earthquake assessments where fully probabilistic SSI analyses are not performed.

- The French ASN/2/01 guide recommends for nonlinear analyses 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 95% confidence interval: for 5 time histories $\lambda = 0.95$ and for 10, $\lambda = 0.58$.

- Probabilistic SSI analyses (Section 7.X) for design basis or beyond design basis earthquake ground motions. Assume ground motion variability represents variability in phase and directional components, but not in frequency content and physical properties of soil and structure are modeled explicitly in the probabilistic SSI analyses.

  - Results of the SSI analyses are the seismic responses for design, e.g., a 80% non-exceedance probability (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., a SPRA, then the variability in the time histories should 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 should 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 requires many less simulations that 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 that baseline drift is not present. If baseline drift is present, baseline corrections should be implemented. This process is mainly to remove the signals linear trends, and high pass
filter the time history. The final aim is to obtain velocity time histories with zero mean and zero end value, 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 (CS) approach [6-13] of partitioning the ground response spectra into contributing scenarios and selecting/generating time histories to use in the seismic analysis customized to the earthquake parameters of interest.

6.6 UNCERTAINTIES

Epistemic (E) and aleatoric (A) uncertainties are to be included in the probabilistic analyses.

- Ground motion – phase variability between the three spatial components (A), vertical vs. horizontal components (E, A), rock seismic hazard curves/UHRS (E, A), etc.
- 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)
- Soil behaviour
  ✓ Material model (linear/equivalent linear) Stiffness
  ✓ Energy dissipation
- Wave propagation mechanism (E, A)

6.7 LIMITATIONS OF TIME AND FREQUENCY DOMAIN METHODS FOR FREE FIELD GROUND MOTIONS

Limitations of time and frequency domain methods for Free Field modelling are twofold. First source of limitations is based on (usual) lack of proper data for (a) deep and shallow geology, surface soil material, and (b) earthquake wave fields. This source of 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, require sophisticated analysis program that is used, 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 sophistication on the analyst side in mathematics. 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.

6.8 REFERENCES


7 METHODS AND MODELS FOR SSI ANALYSIS

7.1 BASIC STEPS FOR SSI ANALYSIS

7.1.1 Preparatory activities

Identify candidate SSI models, model parameters, and analysis procedures:

A. Determine the purposes of the SSI analysis and define the use of results:
   - Seismic response of structure for design or assessment (forces, moments, stresses or deformations, story drift, number of cycles of response);
   - Input to the seismic design, qualification, evaluation of subsystems supported in the structure (time histories of acceleration and displacement), in-structure response spectra (ISRS), number and amplitude of cycles for components, etc.;
   - Base-mat response for base-mat design;
   - Soil pressures for embedded wall designs;
   - Structure-soil-structure analysis;

B. Scoping 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 behavior.

Determine relative importance of phenomena to be modelled. This generally requires the ability to perform numerical experiments with numerical tools that properly model the phenomena in a simplified and detailed manner for comparison purposes.

C. Determine numerical codes to be used: substructuring or direct methods for linear or equivalent linear models; direct methods for linear, equivalent linear, or nonlinear models; discretization of soil portion of model (either idealized stratigraphy or complex profile) (finite element, boundary element, finite difference, or some other approach).

D. Confirm availability of a verification suite for a chosen numerical code. A verification suite of the numerical code features considered to be used in the analyses is required, as a minimum. This verification suite should be available (published) and accessible to the general user community. Estimation of numerical errors for code components (solution advancement algorithms,
elements, material models, etc.) should also be provided within the verification suite.

E. Confirm availability of a validation suite for material models that are to be used in modelling and simulation.

F. Perform one or more sensitivity studies for relevant modelling and simulation parameters in order to determine sensitivity of solution(s) to modeling and solution parameters, e.g., (i) a sensitivity study for relevant material model parameters; (ii) sensitivity study evaluating simulation parameters, e.g., values of $\beta$ and $\gamma$ parameters for the Newmark time integration algorithm. Sensitivity studies should be documented for the overall analysis process or specifically for the SSI analysis of interest.

G. Develop a set of simplified solutions, using simplified models (with 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.

H. Recognize the need for educated, knowledgeable users/experts to perform or consult on the analyses.

7.1.2 Site specific modelling

For a specific site and structure, a number of activities are advised in order to decide level of detail of a model.

1. Determine the characteristics of the subject ground motion (seismic input motion) (see Chapters 4, 5, and 6).
   a. 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;
   b. Incoherence of ground motion;
   c. Are ground motions 3D? Are vertical motions coming from P or S (surface) waves. If from S and surface waves, full 3D motions.

2. Determine the characteristics of the site to be used in the SSI modelling and analyses. Chapter 6 results are applicable to free-field response aspects of the soil.
a. Establish the site profile for the free-field motion from Chapter 6 (linear/equivalent linear soil material properties or nonlinear soil properties);

b. Idealized site profile is applicable:
   i. Linear or equivalent linear soil material model applicable (visco-elastic model parameters assigned);
   ii. Nonlinear (inelastic, elastic-plastic) material model necessary?

c. Non-idealized site profile is necessary and to be developed, complex site stratigraphy and/or nonlinear soil material models:
   i. Linear or equivalent linear soil material model applicable (visco-elastic model parameters assigned);
   ii. Nonlinear (inelastic, elastic-plastic) material model necessary – construct nonlinear soil island model;

d. Sensitivity studies to be performed to clarify model requirements for site characteristics (complex site stratigraphy, inelastic modelling, etc.)

3. Determine the characteristics of a structure of interest (identify safety related structures, such as structures housing safety related equipment) and large components located in the yard for which SSI is important:
   a. Identify function to be performed during and after earthquake shaking, e.g., leak tightness, i.e., pressure or liquid containment; structural support to subsystems (equipment, components, distribution systems); prevention of failure causing failure of safety related SSCs;
   b. Identify load bearing systems for modelling purposes, e.g., shear wall structures, steel frame structures;
   c. Expected behaviour of structure (linear or nonlinear);
   d. Based on initial linear model of the structure, perform preliminary seismic response analyses (response spectrum analyses) to determine stress levels in structure elements;
   e. If significant cracking or deformations possible (occur) such that portions of the structure behave nonlinearly, refine model either approximately introducing cracked properties or model portions of the structure with nonlinear elements;
   f. For expected structure behaviour, assign material damping values;

4. Determine the foundation characteristics:
   a. 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 base-mat;
   b. Effective stiffness is flexible, e.g. if additional stiffening by the structure is not enough to assume rigid behavior; or for strip footings;
Modelling sequence should be (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):

5. Develop linear models of the various elements of the combined SSI model first then slowly complete the model:
   - Soil only
     - Apply static and/or dynamic loads and free-field ground motion to verify model behaviour;
   - Structure only
     Develop dynamic models of the components of the overall structure, e.g., nuclear island structures modelled independently prior to assembling a combined model – containment, internal structure, shield buildings, and others. For example, the EPR has nine separate sub-structures.
     
     Start with a fixed base model for each independent structure (independently apply gravity in all three spatial directions to verify model); apply static point loads as applicable; dynamic analyses as applicable (eigensystem extraction, point loads, free field ground motions); verify load paths and dynamic behaviour. Independent models verified, then assemble the full structural model and repeat static and dynamic analyses as applicable to verify full assembled structural model.
     - Develop foundation model – repeat benchmarking analyses as applicable.
     - Complete structure, foundation, and soil system model;

6. Equivalent linear modelling, and observe changes in response, to determine possible plastification effects. It is very important to note that it is still an elastic analysis, with reduced (equivalent) linear stiffness. Reduction in secant stiffness really stems from plastification, although plastification is not explicitly modeled, hence an idea can be obtained of possible effects of reduction of stiffness.

   One has to be very careful with observing these effects, and focus more on verification of model (for example wave propagation through softer soil, frequencies will be damped, etc...).

7. Nonlinear/inelastic modelling, slowly introduce nonlinearities to test models, convergence and stability, in all the components as above.

8. Investigate sensitivities for both linear elastic and nonlinear/inelastic simulations!
9. Develop documentation according to project requirements and procedures on modelling, choices/assumptions, uncertainties (how are they dealt with) results, etc.

10. Model development requires 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.

11. Model verification is used to verify that mechanical features that are of interest are indeed properly modelled. In other words, model verification is required to prove that results obtained for a given (developed) model are accurately modeling features of interest. For example, if propagation of higher frequency motions is required (analyzed), it is necessary to verify that developed (used) model is capable of propagating waves of certain wave lengths and frequency. Model verification is different than code and solution verification and validation, described in some details in Chapter 9. Model verification should to be performed for each developed model in order to gain confidence that modelling results are acceptable.

12. Independent participatory peer review should be required and performed. “Participatory” refers to continuous review through the SSI analysis process.

7.2 DIRECT METHODS

The direct method analyzes the idealized soil-structure system in a single step. The direct method is applicable to linear and equivalent linear idealizations and required 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.

7.2.1 Discrete methods

Mechanics of solids and structures relies on equilibrium equations of external and internal forces/stresses and/or equations of motion (Bathe (1996) and Zienkiewicz and Taylor (1991); they form a basis for both Finite Element Method (FEM) and Finite Difference Method (FDM).
Finite Element Method

General formulation of the finite element method, as found in Zienkiewicz and Taylor (1991a,b) is used to address the SSI problem. Solid and structural finite elements (as described below) with elastic and inelastic (nonlinear, elastic-plastic) material (as described in chapter 3, are necessary to properly model the SSI problem. Seismic input is performed using seismic motions (3D seismic wave field that can be simplified to 1D or 3x1D, see Chapter 6) and a number of different methods, one of which, the Domain Reduction Method (Bielak et al 2003) is described below.

There exist different types of finite elements. They can be broadly classified into:

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

Standard single phase and two phase, elastic or inelastic finite elements are used in all instances (Zienkiewicz and Taylor, 1991a,b; Zienkiewicz and Shiomi (1984); Zienkiewicz et al. (1990, 1999) Bathe, 1996a).

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 (Wriggers, 2002).

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

Finite Difference Method

Finite different methods (FDM) operate directly on dynamic equilibrium (Zienkiecz and Taylor, Bathe), 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 (Semblat and Pecker, 2009), 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.
7.2.2 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 (1965) and Bathe and Wilson (1976). All the books on nonlinear finite elements do feature sections on linear finite elements.

7.2.3 Nonlinear finite element methods

Nonlinear problems can be separated into (Felippa, 1993; Crisfield, 1991, 1997; Bathe, 1996a)

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

For soil structure interaction problems, nonlinear finite element methods dominate.

Main interest in modelling of soil structure interaction is with material nonlinear problems. Geometric nonlinear problems involve large displacements and large strains are of interest for P–δ effect as well as for soil and structural failure.

It should be noted that sometimes contact problems where gapping occurs (opening and closing of gaps) are called geometric nonlinear problems. For 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 (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:

- Linear elastic models, where linear elastic stiffness is the initial stiffness or the equivalent elastic stiffness (Kramer, 1996; Semblat and Pecker, 2009; Lade, 1988; Lade and Kim, 1995).
  - Initial stiffness uses highest elastic stiffness of a soil material for modelling. It is usually used for modelling small amplitude vibrations. These models can be used for 3D 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.

- Nonlinear 1D models, that comprise variants of hyperbolic models (described in section 3.2), utilize a predefined stress-strain response in 1D (usually shear stress $\tau$ versus shear strain $\gamma$) to produce stress for a given strain.

There are other nonlinear elastic models also, that define stiffness change as a function of stress and/or strain changes (Janbu, 1963; Duncan and Chang, 1970; Hardin, 1978; Lade and Nelson, 1987; Lade, 1988)

These models can successfully model 1D monotonic behaviour of soil in some cases. Special approaches with stress invariants can extend these methods to 3D. In addition, algorithmic measures must be used to make these models work with cyclic loads.

- Elastic-Plastic material modelling can be quite successfully used for both monotonic, and cyclic loading conditions (Manzari and Dafalias, 1997; Taiebat and Dafalias, 2008; Papadimitriou et al., 2001; Dafalias et al., 2006; Lade, 1990; Pestana and Whittle, 1995). Elastic plastic modelling can also be used for limit analysis (de Borst and Vermeer, 1984).

Material nonlinear problems for concrete can be modelled on two levels

A. Solid concrete level, where concrete is modelled using solid models and 2D or 3D elastic-damage-plastic material models (Willam et al. de Borst et al, Fenstra et al., Fenves and Kim, etc.)

B. Beam and/or plate/wall cross section, where 1D fibers are used to model nonlinear normal stress behaviour (concrete and reinforcement) in cross section (Fillipou et al, Spacone et al. Scott et al., etc.). It is important to note that in this type of modelling influence of shear stress is neglected, that is pure bending is assumed.

Interface is modelled using nonlinear inelastic constitutive law within contact elements (Wriggers 2002).

7.2.4 Inelasticity, Elasto-Plasticity

Material models for elastic-plastic analysis of the SSI problem are described in detail in Chapter 3.

7.2.5 Dynamics Solution Techniques:

On the global, finite element level, finite element equations are solved using time marching algorithms. Most often used are Newmark algorithm (Newmark, 1959) and Hilber-Hughes-Taylor (HHT) $\alpha$ algorithm (Hilber et al., 1977). Other algorithms (Wilson
θ, l’Hermite, etc.) also do exist Argyris and Mlejnek (1991); Hughes (1987); Bathe and Wilson (1976), however they are used less frequently. Both Newmark and HHT algorithm allow for numerical damping (through appropriate choice of parameters, β and γ for Newmark method and γ for 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.

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, analyst is assured that his/her solution is within 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:

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

In both cases, analyst might choose not to enforce equilibrium, for example if time steps are very small. However, it is recommended that a sensitivity study be performed in order to determine that this approximation (not enforcing equilibrium) is indeed appropriate.

### 7.2.6 Energy dissipation

Seismic energy that enters the SFS system will be dissipated in a number of ways. Part of the energy that enters SFS system can be reflected back into domain outside by:

1. Wave reflection from impedance boundaries, such as free surface, soil/rock layers, foundations, etc.
2. Structural system oscillation radiation.

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

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

It is also important to note that in numerical simulations (advocated and used in this work), part of the energy can be dissipated or produced by purely numerical means. That is, numerical energy dissipation (damping) or production (negative damping) has to be carefully controlled (Argyris and Mlejnek, 1991), (Hughes, 1987).

It is also important to note that plastic dissipation is not the same as plastic work (Farren and Taylor 1925, Taylor and Quinney 1934). Proper calculation of plastic dissipation for elastic plastic material needs to be used (Han et al. 2017). Recent papers by Yang et al. 2018 (a,b) provide more details on how to calculate energy dissipation from material hysteretic response, viscous interaction and algorithmic damping.

7.3 SUB-STRUCTURING METHODS

7.3.1 Principles

As described in Section 1.1, the natural progression for addressing soil-structure interaction (SSI) of structures, such as nuclear power plant structures, was to implement tools developed for machine vibrations. This was an initial attempt at sub-structuring methods. It 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 desire to break the complicated soil-structure problem into more manageable parts has led to the sub-structure methods.

Generally, sub-structuring 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 sub-structuring 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; and the dynamic behavior of the structure is frequency-dependent (as demonstrated by eigen-system extraction into normal modes).

Conceptually, sub-structuring methods can be classified into four types depending on the methodology of treating the interface between the soil and structure, i.e., how the soil and structure interface degrees-of-freedom are treated. Figure 7-1 shows the four types. These four types are: 1) the rigid boundary method, where the term “rigid” refers to the boundary between the foundation\(^{13}\) and the soil; 2) the flexible boundary methods; 3) the flexible volume method or the direct method; and 4) the substructure subtraction method.

It is noted that the Domain Reduction Method (DRM), described in section 7.4.10, 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 modeling 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, large scale, regional model does not have to be modeled, 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 some detail in section 7.4.10, while original reference is by Bielak et al. (2003).

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, sub-structuring methods replace the one-step analysis by a multi-step approach: each step in the process is analyzed by the most appropriate tools available. The global model is replaced by several sub-models 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 sub-models: these compatibility conditions simply state that at the shared nodes of two sub-models displacements must be equal and forces must have the same amplitude but opposite signs. The sub-structure approach is schematically depicted in Figure 7-3.

\(^{13}\) The term “foundation” is used herein to denote the foundation of the structure (base mat) and partially embedded structure elements, such as exterior walls in contact with the soil.
<table>
<thead>
<tr>
<th>Method Analysis</th>
<th>Rigid Boundary</th>
<th>Flexible Boundary</th>
<th>Flexible Volume</th>
<th>Subtraction</th>
</tr>
</thead>
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<tr>
<td>Site Response Analysis (a)</td>
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<tr>
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<td>Impedance Analysis (c)</td>
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<td>Structural Response Analysis (d)</td>
<td>Standard</td>
<td>Standard +</td>
<td>Standard +</td>
<td>Standard +</td>
</tr>
</tbody>
</table>

*Figure 7-1: Summary of substructuring methods*

*Figure 7-2 Schematic representation of the elements of soil-structure interaction – sub-structure method*
At nodes A and B, which are at the same location in space, the displacement vectors satisfy $\mathbf{U}_A = \mathbf{U}_B$ and the force vectors $\mathbf{F}_A = -\mathbf{F}_B$. Note that here the partitioning between the two subsystems is made along the boundary separating the structure and the soil; however, as it will be briefly mentioned below, other alternatives are possible. The boundary considered in Figure 7-3 does not need to be rigid but the size of the problem to handle is significantly reduced when this assumption holds.

Several approaches are proposed to handle the sub-structure methods. They differ essentially by the way in which the interaction between the two sub-structures is handled. One possibility is to define the interface between the two sub-systems along the external boundary of the structure which can be either a plane (2D model) or a surface (3D model); the approach is known as the (rigid or flexible) boundary method and is due to Kausel and Roësset (1974) and Luco and Wong (1971). Other sub-structuring methods have been introduced more recently; they are known as the flexible volume method (Tabatabaie-Raissi, 1982) and the subtraction method (Chin, 1998)$^{14}$.

The seismic SSI sub-problems that these three types of substructuring methods are required to solve to obtain the final solution are compared in Figure 7-1 (SASSI manual). As shown in this figure, the solution for the site response problem is required by all four methods. This is, therefore, common to all methods. The analysis of the structural

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$^{14}$ For the subtraction method, spurious results could exist, which require a work-around including introducing additional interaction nodes which constrains the frequency characteristics of the excavated soil. This modified subtraction method is termed "extended (modified) subtraction method (see section 7.4.3)."
response problem is also required and involves essentially the same effort for all methods. The necessity and effort required for solving the scattering and impedance problems, however, differ significantly among the different methods. For the rigid boundary method, the scattering and impedance problems are typically solved simultaneously (Luco and Wong (1971), Johnson et al. (2007) once Green’s functions have been calculated or when the flexible impedance matrix is calculated. The flexible boundary model requires calculation of the scattering and impedance matrices separately. The flexible volume method and the substructure subtraction method, because of the unique substructuring technique, require only one impedance analysis and the scattering analysis is eliminated. Furthermore, the substructuring in the subtraction method often requires 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.

Note also that the domain reduction method (DRM) described in Section 7.4.10 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.

Actually, 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.

7.3.2 Rigid or flexible boundary method

The multi-step approach involved in sub-structuring is based on the so-called superposition theorem established by Kausel and Roësset (1974). It is schematically depicted in Figure 7-2 and Figure 7-4 (Lysmer, 1978) and can be described as follows. Note that Figure 7-4 has been drawn for the special case described below of a 2D problem and a rigid foundation.

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

- 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. Note, 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 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 Chapter 6 for further discussion of the free-field ground motion). Foundation input motion (FIM) is defined as the result of multiplying the scattering matrix times the free-field ground motion.

- 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 kn by kn where n is the number of nodes along the boundary and k the number of degrees of freedom at each node. In the most general case k=6 and the stiffness matrix is 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\(^\text{15}\), for instance the geometric centre of the basemat, and the size of the frequency-dependent complex valued impedance matrix is 6 by 6 for each frequency of calculation for the analysis.

- 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.

\(^{15}\) For foundations assumed to behave rigidly analyzed by CLASSI, the point about which the scattering and impedance matrices are calculated is named the “Foundation Reference Point (FRP).” This also the point about which the SSI solution is generated.
Sub-structure methods can be sub-divided into methods that efficiently analyze structures whose foundation can be modeled 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 chapter.

**Foundations behaving rigidly.** Two programs that are extensively used to analyze linear elastic soil-structure systems when the foundation of the structure can be treated as behaving rigidly are CLASSI (Soil-Structure Interaction: A Linear Continuum Mechanics Approach) (Luco and Wong, 1980) and SUPELM (Kausel, xxxx). In the same spirit as the evolution of SASSI, CLASSI has evolved over the last decades such that it is a staple tool in analyst’s tool boxes. Section 7.3.6 describes the principle features of CLASSI. Section 7.3.6 also describes the hybrid method of CLASSI-SASSI, in which embedded foundations are modeled with SASSI and the resulting SASSI flexible impedance matrix is derived and condensed to six degrees-of-freedom for CLASSI analyses. Appendix CC contains the derivation of the CLASSI methodology and important equations. The modules of CLASSI and SUPELM were used extensively in the SASSI V&V program to benchmark individual modules of the SASSI program.

As introduced above, sub-structuring methods are most efficiently implemented in the frequency domain, which requires that theoretically solutions must 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, only less than the total number of Fourier transform frequencies are analyzed explicitly (typically, 50 – 200 frequencies). Interpolation schemes have been developed and implemented to fill in the functions at missing frequencies, e.g., Tajirian, 1981. 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 (1974) and Appendix YY.

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. It must again be stressed that, 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.

7.3.3 The flexible volume method

The flexible volume substructuring method (Tabatabaie-Raissi, 1982) 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 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 7.3.5 and Appendix ZZ for additional information on SASSI.

7.3.4 The subtraction method

The substructure subtraction method (Chin, 1998) is basically based on the same substructuring concept as the flexible volume method. The subtraction method partitions 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 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 soil structure interaction 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.
Note that 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 spurious results (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 additions is called the extended (modified) subtraction method. When implementing any of the subtraction methods, validation that no spurious results have been introduced is necessary. (ASCE 4-16)

Figure 7-5: Substructuring in the subtraction method.
7.3.5 SASSI: System for Analysis of Soil-Structure Interaction

SASSI evaluates the dynamic response of two- and three-dimensional foundation-structure systems. As introduced above, SASSI is formulated using the flexible boundary method and uses linear finite element modeling and frequency domain methods. The soil is modeled 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 modeled by two- or three-dimensional 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 treat foundation/structure systems, including flexibility of the foundation. Generally, horizontal and vertical models are analyzed independently, and the results combined post SSI analyses.

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

Key elements of the SASSI approach are:

1. The site is modeled as semi-infinite elastic or viscoelastic horizontal layers on a rigid base or semi-infinite elastic or viscoelastic half-space.

2. The structures are idealized by standard two- or three-dimensional finite elements. Each nodal point may have up to six degrees of freedom.

3. The excavated soil zone is idealized by standard plane strain or three-dimensional solid elements. The finite element models of the structure and excavated soil have common nodes at the boundary.

4. The flexible volume method (Direct Method) is generally recommended and used extensively. Interaction between the excavated soil and semi-infinite site occurs at all excavated soil nodes in the flexible volume method.

5. All the interaction nodes lie on the soil layer interfaces with translational degrees of freedom.
6. 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.

7. The seismic environment may consist of an arbitrary three-dimensional superposition of inclined body and surface waves.

8. The earthquake excitation is defined by time histories of acceleration. The input motion may also be specified with acceleration response spectrum using the random vibration theory or RVT. The effects of incoherence of ground motion can be modeled with several versions of SASSI.

9. The control motion is applied at the control point that may be defined on the soil free surface or at a point within the soil column.

10. For time series analysis, the fast Fourier transform technique is used.

### 7.3.6 CLASSI: Soil-Structure Interaction - A Linear Continuum Mechanics Approach

Key elements of the CLASSI approach (Luco and Wong, 1980) are:

The site is modeled as semi-infinite viscoelastic horizontal layers overlying a semi-infinite viscoelastic halfspace.

1. 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.

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 for generation of scattering functions and foundation impedances. The Green’s functions are integrated over the discretized foundation sub-region areas to calculate a resultant set of forces and displacements at sub-region centroids.

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.

The results consist of complex-valued, frequency-dependent scattering functions and impedances developed at a defined foundation reference point (FRP).
2. The structures are idealized by simple or very detailed three-dimensional finite element models. The dynamic characteristics of the structure are represented by the fixed-base eigensystem. 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 three-dimensional finite element structure model as analyzed by CLASSI is shown in Figure 7-6. 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) (Johnson et al., 2010).

Multiple structures on a single foundation may be modeled.

3. Material damping is introduced by the use of complex moduli in the soil and modal damping in the structure.

4. The seismic environment may consist of an arbitrary three-dimensional superposition of inclined body and surface waves. Also, a version of CLASSI treats incoherent ground motion.

5. 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.

6. The Fast Fourier Transform technique is used.

7. The solution of the complete SSI problem is performed in 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 7-2 shows schematically the steps in the CLASSI methodology. Derivation of important equations is provided in Appendix CC.

7.3.7 Attributes of two programs SASSI and CLASSI for consideration

SASSI.
Advantages: (i) ability to model complex foundation geometry, foundation embedment, and foundation flexibility; (ii) ability to calculate soil-foundation interaction parameters, such as soil pressures on embedded foundations;

Limitations: (i) lack of ability to analyze very detailed structural models in a timely manner (HPC is positively affecting this limitation); (ii) inability to easily perform sensitivity studies; (iii) need to run three directions of analyses separately and combine the results outside of SASSI. (iv) linear elastic material only!

CLASSI.

Advantages: (i) ability to include very detailed structure models in the SSI analyses and represent the structure by its fixed-base modes, including modal damping; (ii) ability to perform three-dimensional analyses simultaneously; (iii) ability to analyze the same SSI model for fixed-base conditions, coherent ground motion, and incoherent ground motion; (iv) ability to efficiently perform sensitivity studies; (v) efficiently perform probabilistic seismic response; and (vi) extremely computationally efficient.

Limitations: (i) inability to analyze flexible foundations; (ii) inability to calculate soil-foundation interaction parameters including the effects of flexible foundation and below grade embedded walls, such as soil pressures; (iii) linear elastic material only.
Figure 7-6  AREVA Finite Element Model of the EPR™ Nuclear Island (Johnson et al., 2010)
7.4 SSI COMPUTATIONAL MODELS

7.4.1 Introduction

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

- Earthquake input motions, encompassing development of 1D or 2D or 3D motions, and their effective input in the SSI model,
- Soil/rock adjacent to structural foundations, with important geological (deep) and site (shallow) conditions near structure, contact zone between foundations and the soil/rock, and
- 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. Modelling steps described in section 7.1 present good guidance for effective modelling of all three components above.

7.4.2 Soil/Rock Linear and Nonlinear Modelling

Effective and Total Stress Analysis

Soil and rock adjacent to structural foundations can be either dry or fully (or partially) saturated (Zienkiewicz et al., 1990; Lu and Likos, 2004).

1. **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.

2. **Unsaturated Soil:** For partially saturated soil, the effective stress principle (see equation 7.4 below) must also include the influence of gas (air) present in pore of soils. There are a number of different methods to do that (Zienkiewicz et al., 1999; Lu and Likos, 2004), however computational frameworks that incorporate those methods are not yet well developed.

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:

- voids fully saturated with fluid mixed with air bubbles, water in pores is fully connected and can move and pressure in the mixture of water and
air can propagate, with reduced bulk stiffness of water-air mixture. This type of partial saturation can be modelled using fully saturated approaches, noted in section 7.4.11 below. It is noted that bulk modulus of fluid-air mixture is (much) lower than that of fluid alone, and to be tested for. A full derivation of the equivalent bulk modulus of a partially saturated soil is provided in appendix I.15 and an example of the variation of the bulk modulus with the degree of saturation is shown in Figure 7-7. Therefore, only methods that assume fluid to be compressible should be used \((u - p - U, u - U_i)\). In addition, permeability will change from a case of just fluid seeping through the soil, and additional testing for permeability of water-air mixture is warranted. It is also noted, that since this partial saturation is usually found above water table, (capillary rise), hydrostatic pore pressure can be suction.

- Voids of soil are full of air, with water covering thin contact zone between particles, creating water menisci, and contributing to the apparent cohesion of cohesionless soil material (think of wet sand at the beach, there is an apparent cohesion, until sand dries up). This type of partial saturation can be modelled 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.

![Figure 7-7 Example of variation of the equivalent bulk modulus of fluid divided by the saturated fluid bulk modulus versus degree of saturation](image-url)
3. **Saturated Soil.** In the case of full saturation, the effective stress principle (Terzaghi et al., 1946) has 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 total stress acting on material ($\sigma_{ij}$), with reductions due to the pore fluid pressure.

$$\sigma' = \sigma + p\delta$$

where $\delta$ is the Kronecker symbol and $p$ ($>0$) is the pore fluid pressure and will follow with this definition although it does change the convention for pore fluid pressure sign.

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 $p$.

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

Usually, clays are modelled as fully saturated soil material. In addition, clays feature very small permeability, so that, while the effective stress principle (from Equation 7.4) applies, pore fluid pressure does not change during fast (earthquake) loading. Hence clays should be analyzed 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.

### 7.4.3 Drained and Undrained Modelling

Depending on the permeability of the soil, on relative rate of loading and seepage, and on boundary conditions (Atkinson, 1993), a decision needs to be made if analysis will be performed using drained or undrained behaviour. Permeability of soil ($k$) can range from $k > 10^{-2} \text{m/s}$ for gravel, $10^{-2} \text{m/s} > k > 10^{-5} \text{m/s}$ for sand, $10^{-5} \text{m/s} > k > 10^{-8} \text{m/s}$ for silt, to $k < 10^{-8} \text{m/s}$ for clay and even this low for 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 $k = 10^{-6} \text{m/s}$, water can travel few millimetres. However, pore fluid pressure will propagate (much) faster (further) and will affect mechanical behaviour of soil skeleton. This is due to high bulk modulus of water ($K_w = 2.25 \times 10^6 \text{ kN/m}^2$), which results in high speed of pressure waves in saturated soils. Thus a simple rule is that for
earthquake loading, for gravel, sand and permeable silt, relative rate of loading and seepage requires 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 should provide accurate results.

**Drained Analysis:** Drained analysis is performed when permeability of soil, rate of loading and seepage, and boundary conditions allow for full movement of pore fluid and pore fluid pressures during loading event. As noted above, 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 soil skeleton. This is usually done using theory of mixtures (Green and Naghdi, 1965; Eringen and Ingram, 1965; Ingram and Eringen, 1967; Zienkiewicz and Shiomi, 1984; Zienkiewicz et al., 1999) During loading events, pore fluid pressures will dynamically change (pore fluid and pore fluid pressures will displace) and will affect the soil skeleton, through 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 should be used.

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

- **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 (Muir Wood, 1990), 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.
- Locally undrained analysis 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). On the other hand, 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 constitutive behaviour of soil. Analysis is essentially undrained, however, pore fluid pressure can and will change locally due to compression or dilatancy effects in granular soil. 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 (as noted in section 6.4.2) should be used, while the constitutive integration should 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 local, constitutive level, and there is no two phase material (pore fluid and porous solid) where pore fluid pressure is able to propagate.

- 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 analyzed 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 (one of two approaches above) 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 load (earthquake) no fluid will displace and no pore fluid pressure will propagate.

This approach can be used for both cases noted above (total stress approach and locally undrained approach). While this approach is actually explicitly allowing for modelling of pore fluid movement, results for pore fluid displacement should show no movement. In that sense, 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.

It is noted that globally undrained problems, where for example soil is permeable, but boundary conditions prevent water from moving, should be treated as drained problems, while appropriate boundary conditions should prevent water from moving across impermeable boundaries.

Examples of undrained and drained site response analyses, with different permeabilities, are provided in appendix 1.

7.4.4 Linear and Nonlinear Elastic Models

Material modelling for soil can be equivalent linear, nonlinear or elastic-plastic.

- 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 1D analysis and for 1D states of stress and strain. Potential issues with this modelling approach are that it is not taking into account soil volume change (hence it favours total stress analysis, see details about total stress analysis in section 7.4.2), and it is useable for 3D analysis through deviatoric stress invariant. Secant stiffness 1D models provide relationship between shear stress ($\tau = \sigma_{xz}$) and shear strain ($\gamma = 2\varepsilon_{xz}$). Determination of secant shear stiffness is done iteratively, by performing 1D wave propagation simulations, and recording average high estimated strain (65% of maximum strain) for each level/depth. Such representative shear strain is then used to determine reduction of stiffness using modulus reduction curves ($G/G_{\text{max}}$ and the analysis is re-run. Stable secant stiffness values are usually reached after few iterations, typically 5-8. It is important to emphasize that equivalent elastic modelling is still essentially linear elastic modelling, with changed stiffness. More details are available in sections 3.2

- Nonlinear material models are representing 1D stress strain (usually in shear, $\tau - \gamma$) response using nonlinear functions. There are a number of nonlinear elastic models used, for example Ramberg-Osgood (Ueng and Chen, 1992), Hyperbolic (Kramer, 1996), and others. Calibrating modulus reduction and damping curves using nonlinear models is not too demanding (Ueng and Chen, 1992), however there is less experience in professional practice with these types of models (see section 3.3.2). Potential issues with this modelling approach are the same as for
equivalent linear modelling, e.g. it is not taking into account soil volume change as it is essentially based on a nonlinear elastic model,

- Elastic-plastic material models are usually full 3D models that can be used for 1D or 3D analysis. A number of models are available (Prevost and Popescu, 1996; Mróz et al., 1979; Elgamal et al., 2002; Pisanó and Jeremic, 2014; Dafalias and Manzari, 2004). Use of full 3D material models, if properly calibrated, can work well in 1D as well as in 3D. Potential issues with these models is that calibration usually requires 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 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 (OCR) (Janbu, 1963; Duncan and Chang, 1970; Hardin, 1978; Lade and Nelson, 1987; Lade, 1988).

Anisotropic material models are mostly used for modelling of usually anisotropic rock material (Amadei and Goodman, 1982; Amadei, 1983).

**Elastic-Plastic Models**

Elastic plastic modelling can be used in 1D, 2D and full 3D analyses. A number of material models have been developed over years for both monotonic and cyclic modelling of materials. Material models for soil (Manzari and Dafalias, 1997; Taiebat and Dafalias, 2008; Papadimitriou et al., 2001; Dafalias et al., 2006; Lade, 1990; Pestana and Whittle, 1995; Prevost and Popescu, 1996; Mróz and Norris, 1982), rock (Lade and Kim, 1995; Hoek et al., 2002; Vorobiev, 2008) have been developed over last decades.

It should be noted that 3D elastic plastic modelling is the most general approach to material modelling of soils and rock. Elastic plastic models can model simplified and detailed behavior. However, calibration of models that can achieve such modelling sophistication requires expertise. The payoff 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.
7.4.5 **Structural models, linear and nonlinear: shells, plates, walls, beams, trusses, solids**

Nonlinear structural models may be used for capacity assessments and designs of nuclear installations (NI) other than Nuclear Power Plants (NPP). One of the main reasons is that NPPs are required to remain essentially elastic during design basis earthquake ground motion.

Significant work has been done in modelling of nonlinear effects in reinforced concrete elements (Feenstra, 1993; Feenstra and de Borst, 1995; de Borst and Feenstra, 1990; de Borst, 1987, 1986; de Borst, 1987; de Borst et al., 1993; Bicanic et al., 1993; Kang and Willam, 1996; Rizzi et al., 1996; Menetrey and Willam, 1995; Carol and Willam, 1997; Willam, 1989; Willam and Warnke, 1974; Etse and Willam, 1993; Scott et al., 2004, 2008; Spacone et al., 1996a,b; Scott and Fenves, 2006).

7.4.6 **Contact Modelling**

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

- **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.
- **Shear**, tangential, contact where the structural elements and the adjacent soil/rock can develop frictional slip.

Modelling of contact is done using contact finite elements. 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 interface (Wriggers, 2002; Haraldsson and Wriggers, 2000; Desai and Siriwardane, 1984), Jeremic (2016). In addition to node to node contact elements, node to surface and surface to surface contact elements are also available (Dynaflow).

7.4.7 **Structures with a base isolation/dissipation systems**

Base isolation systems have been used (Cruas NPP France, Koeberg NPP South Africa) and are being developed for NIs (ITER, RJH France). Base isolation/dissipation systems behaviour is affected by SSI.
Base isolation systems are used to change dynamic characteristics of seismic motions that excite structure and also to dissipate seismic energy before it excites structure. Therefore, there are two main types of devices:

- **Base Isolators** (Kelly, 1991a,b) are usually made of low damping (energy dissipation) elastomers and are primarily meant to change (reduce) 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.

- **Base Dissipators** (Kelly and Hodder (1982); Fadi and Constantinou (2010)) are developed to dissipate seismic energy before it excites the structure. There are two main types of such dissipators:
  - Elastomers made of high dissipation rubber, and
  - Frictional pendulum dissipators

Both isolators and dissipators are usually developed to work in two horizontal dimensions, while motions in vertical direction are not isolated or dissipated. This can create potential problems, and need to be carefully modelled Hijikata et al. (2012); Araki et al. (2009).

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

- High damping rubber dissipators
- Rubber dissipators with lead core
- Frictional pendulum (double or triple) dissipators

### 7.4.8 Foundation models

Foundation modelling can be done using 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 overall soil-structure interaction (SSI) response. Major energy dissipation happens in soil and 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 defined as (embedment
depth/equivalent radius \( e/r \) or equivalent square side); foundations then are designated as: surface \( e/r<0.3 \); shallow embedment \( e/r<1.0 \); and deep embedment \( e/r>1.0 \); ASCE 4-16, Section 5.4.2.4 has surface so defined.

**Shallow and embedded foundation slabs and walls.** Embedded foundations have an embedment ratio (depth to width) less or equal than 0.15. Such foundations can be modelled as flexible or rigid. Their thickness can range from 3 – 5 meters, however, their horizontal extent can be up to 100 meters. Containment and shear wall structures, for which structure elements are connected to the foundation, significantly increase the effective stiffness of the foundation approaching rigid behavior. Depending on the stiffness of structure and the slab systems and of the objective of the SSI analysis, the foundations can be modelled as rigid systems; however, influence of such modelling needs to be analyzed. For purposes of calculating overall response of the soil-structure system, the assumption of rigid foundation behavior is often reasonable and justifiable. If the rigid foundation assumption is introduced, more detailed seismic responses, such as stresses in the foundation, require a second stage analysis with appropriate levels of modeling to design or evaluate structure capacity.

Flexible modelling of foundation slabs is best done using either shell elements (plate bending and membrane behavior) 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 plane in the middle of a solid (slab or wall) with a finite thickness, so connection over half 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 (Ibrahimbegović et al., 1990; Militello and Felippa, 1991), 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 represent properly the bending stiffness. For example, a single layer of regular 8 node bricks will over-predict bending stiffness over 2 times (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 is predicting bending stiffness within 4% of analytic solution.

**Deep foundations (piles, caissons and shaft foundations).** Deep foundations are used for nuclear facilities 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 8.

Piles (including pile groups) and shafts have been modelled using three main approaches:
• Analytic approach (Sanchez-Salinero et al., 1982, 1983; Mylonakis and Gazetas, 1999) where 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.

• p-y and t-z approaches are based on 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 (Stevens and Audibert, 1979; Brown et al., 1988; 1999; Tower Wang and Reese, 1998; Reese et al., 2000; Georgiadis, 1983). 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, dynamics of soils surrounding piles and piles groups is poorly approximated using springs, even with additional dashpots.

• Nonlinear 3D finite element models have been developed for treatment of piles, pile groups and shafts, in both dry and liquefiable soils (Brown and Shie, 1990, 1991; Yang and Jeremic, 2003, 2005a,b). In these models, 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 7.4.6) resolves both horizontal and vertical shear (slip) behaviour.

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

• 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 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.

• The pile element is modelled with solid elements (for solid piles). The main advantages are: the exact geometry of the pile is modelled, wave diffraction is correctly considered and the pile-soil-pile interaction is taken into account. The significant disadvantages are the requirement of having a sufficient number of elements in the cross section to properly model the bending behavior of the pile (see 7.4.8), which increases the complexity of the model and the number of degrees of freedom. However, the most important drawback stems from the
difficulty of having direct access to the pile internal forces (stresses should be integrated over the cross section), which requires additional work.

- The third possibility takes advantage of the simplicity of the first technique while tentatively preserving the rigor 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 express that pile cross section remains plane. 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.

Deeply Embedded Foundations. Deeply embedded foundations have an embedment ratio greater than 0.15. The SSI for deeply embedded foundations is significantly affected by the contribution of the embedded walls, in addition to the base slab. Main issues are related to proper modelling of contact (see more in sections 7.4.9), 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 base slab.

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 isolators/dissipators, beneath the actual structure. Those two base slabs are connected with dissipators/isolators. It is important to properly (accurately) model stiffness of both slabs as their relative stiffness will control how effective will isolators and dissipators be during earthquakes.

7.4.9 Deeply Embedded Structures

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

1. Seismic Motions: Seismic motions will be quite variable along the depth and in all three directions. This variability of motions is a result of mechanics of seismic wave propagation, inherent variability and the interaction of body waves (SH, SV and P) with the surface, and the development of surface waves (Aki and Richards, 2002). 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 recommended that a full wave field be developed and applied to SSI models of DESs.

- In the case of 1D wave propagation modelling, vertically propagating shear waves are to be developed (deconvolution and/or convolution) and applied to SSI models.

- For 3D wave fields,
  * develop a full seismic wave field from a wave propagation /site response modelling and analyses (chapters 5 & 6).
  * use incoherence functions if available 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.

2. Nonlinear/Inelastic Soil and Contact: Large contact zone of a DES concrete walls and foundation slab, with surrounding soil, with its nonlinear/inelastic behaviour will have significant effect on dynamic response of a DES.

Use of appropriate contact model that can model frictional contact as well as possible gap opening and closing (most likely in the near surface region) is recommended. In the case of presence of water table above DES foundation base, effective stresses approach needs to be used, as well as modelling of (possibly dynamically changing) buoyant forces, as described in section 7.4.10 and below.

Nonlinear/Inelastic Soil Behaviour: With deep embedment, dynamic behaviour of a DES is significantly influenced by the nonlinear/inelastic behaviour of soil adjacent to adjacent SMR walls and foundation slab.

Use of appropriate inelastic (elastic-plastic) 3D soil models is recommended. Of particular importance is proper modelling of soil behaviour in 3D as well as proper modelling of volume change due to shearing (dilatancy). One dimensional equivalent elastic models, used for 1D wave propagation are not recommended for use, as they do not model properly 3D effects and lack modelling of volume change.

3. Buoyant Forces: With deep embedment, and (a possible) presence of underground water (water table that is within depth of embedment), water
pressure on walls of DES will create buoyant forces. During earthquake shaking, those forces will change dynamically due to water pumping during shaking (Allmond and Kutter, 2014).

Modelling of buoyant forces can be done using two approaches, namely static and dynamic buoyant force modelling, as described in Section 7.4.10.

4. 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 DES will add to uncertainty of the response.

Uncertainty in seismic motions, soil configuration, and soil and material parameters can be modelled using two approaches, as described in section 7.5. 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, stochastic finite element method, with extension to stochastic elasto-plasticity with random loading. More details are given in section 7.5.

Figure 7–8 illustrates modelling issues on a simple, generic DES (an SMR) finite element model (vertical cut through middle of a full model is shown). Illustrative example using this SMR is provided in the appendix.

![Figure 7–8: Four main issues for realistic modelling of Earthquake Soil Structure Interaction of SMRs: variable wave field at depth and surface, inelastic behaviour of contact and adjacent soil, dynamic buoyant forces, and uncertain seismic motions and material.](image)

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

A modelling and simulation example for an SMR is presented in the appendix and is used to illustrate challenges and modelling and simulation methodology.
7.4.10 Buoyancy Modelling

For NPPs structures for which 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 NPPs related structures, like intake structures, there exist different pore fluid pressures in the soil on different sides, creating a nonsymmetric buoyant forces. For static loads, symmetric buoyant force \( B \) can be calculated using Archimede’s principle. Buoyant force can be applied as a single force or a small number of resultant forces directed upward around the stiff centre of foundation. For nonsymmetric buoyant forces, static forces on all (pore fluid pressures) walls and 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 increase of inter-granular void space (dilatancy), and reduction in buoyant pressures and creation of negative pressures (suction); for soils that are loose, where shearing will lead to reduction of inter-granular void space (compression) and increase in buoyant pressures.

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

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

Dynamic Buoyant Stress/Force Modelling. Fully coupled finite elements (u-p or u-p-U or u-U, as described in section 7.4.2) are used for modelling saturated soil adjacent to foundation walls and base. Modelling of contact between soil and the foundation concrete needs to take into account effects of pore fluid pressure – buoyant stress within the contact zone, in order to properly model normal stress for frictional contact.

7.4.11 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 analyzed 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 should absorb all/most outgoing waves and reflect no waves back into the computational.
functions are used for the outside domain, absorbing boundary problem is resolved analytically.

The most commonly used types of domain boundaries are presented in the following:

- **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.

  Combination of free and fully fixed boundaries should 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 placed far enough, thus significantly increasing the size of models.

- **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 finite element code as it consists of simply connecting dashpots to all degrees of freedom of the boundary nodes and fixing them on the other end (Figure 7-9).

  Lysmer boundaries are derived for an elastic wave propagation problem in a one-dimensional semi-infinite bar. It can be shown that in this case a dashpot specified appropriately has the same dynamic properties as the bar extending to infinity (Wolf, 1988). The damping coefficient \( C \) of the dashpot equals

  \[
  C = A \rho c
  \]

  where \( A \) is the cross section area of the bar, \( \rho \) is the mass density and \( c \) the wave velocity that has to be selected according to the type of wave that has to be absorbed (shear wave velocity \( c_s \) or compressional wave velocity \( c_p \)).
Figure 79 Absorbing boundary consisting of dash pots connected to each degree of freedom of a boundary node

In a 3D or 2D model the angle of incidence of a wave reaching a boundary can vary from near 0° to near 180°. The Lysmer boundary is able to absorb completely only waves with an incidence angle of 90°. 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° and the amount of energy reflected can be reduced.

- Infinite elements can also be used (Zienkiewicz and Taylor, 1991) however their use does not guarantee full absorption of outgoing waves.

- Perfectly Matched Layer (PML) that was recently adopted from electromagnetic wave propagation modeling by Basu and Chopra (2003) can be used for removing outgoing waves from the SSI domain.

- Domain Reduction Method (DRM), developed recently by Bielak et al, elegantly resolves the issue of outgoing waves, see details in section 7.4.12.1 on DRM below.

More detailed boundaries model wave propagation toward infinity (boundary elements). However, use of boundary elements for outside FEM models destroys sparsity of the resulting stiffness matrix a full matrix, and thus places high computational burden on FEM solvers.

7.4.12 Seismic Load Input

A number of methods are used to input seismic motions into finite element model. Most of them are based on simple intuitive approaches, and as such are not based on rational mechanics.
Most of those currently still widely used methods cannot properly model all three components of body waves as well as always present surface waves. Simplest method to input waves into the SSI model is apply displacements or acceleration at nodes at the bottom of the SSI model. While these method seems intuitive, it does trap waves in the SSI model, waves are not allowed to leave, radiate into half space. Other methods that are used for frequency domain modeling rely on 1C convolution or deconvolution of motions. For time domain modeling of SSI phenomena, the Domain Reduction Method (DRM, Bielak et al.) resolves many issues and provides probably the most elegant and efficient method to input seismic motions into SSI model.

**Domain Reduction Method**

There exists a method that is based on rational mechanics and can model both body and surface seismic waves input into finite element models with high accuracy. That method is called the Domain Reduction Method (DRM) and was developed by Bielak et al. (2003), Yoshimura et al. (2003). 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 global–local set of methods and as formulated can be used for any problems where the local feature can be bounded by a continuous surface (that can be closed or not).

A large physical domain is to be analyzed for dynamic behaviour. The source of disturbance is a known time history of a force field \( P_e(t) \). That source of loading is far away from a local feature which is dynamically excited by \( P_e(t) \) (see Figure 7-10).

It would be beneficial not to analyze the complete system, as we are only interested in the behaviour of the local feature and its immediate surrounding, and can almost neglect the domain outside of some relatively close boundaries. In order to do this, we need to transfer the loading from the source to the immediate vicinity of the local feature. For example, we can try to reduce the size of the domain to a much smaller model bounded by surface \( \Gamma \) as shown in Figure 7-10. In doing so we must ensure that the dynamic forces \( P_e(t) \) are appropriately propagated to the much smaller model boundaries \( \Gamma \).
It can be shown (Bielak et al., 2003) that the consistent dynamic replacement for the dynamic source forces \( P_e \) is a so-called effective force, \( P_{\text{eff}} \):

\[
P_{\text{eff}} = \begin{bmatrix}
P_{\text{eff},e} \\
P_{\text{eff},b} \\
P_{\text{eff},e}
\end{bmatrix} = \begin{bmatrix}
0 \\
-M_{\Omega^+}^{\text{be}} \ddot{u}_e - K_{\Omega^+}^{\text{be}} \dot{u}_e \\
M_{\Omega^+}^{\text{eb}} \ddot{u}_b + K_{\Omega^+}^{\text{eb}} \dot{u}_b
\end{bmatrix}
\]  

(7.10)

where \( M_{\text{be}}^{\Omega^+} \) and \( M_{\text{eb}}^{\Omega^+} \) are off-diagonal components of a mass matrix, connecting boundary (b) and external (e) nodes, \( K_{\text{be}}^{\Omega^+} \) and \( K_{\text{eb}}^{\Omega^+} \) are off-diagonal component of a stiffness matrix, connecting boundary (b) and external (e) nodes, \( \ddot{u}_e \) and \( \ddot{u}_b \) are free field accelerations of external (e) and boundary (b) nodes, respectively and, \( \dot{u}_e \) and \( \dot{u}_b \) are free field displacements of external (e) nodes, and boundary (b) nodes, respectively. The effective force \( P_{\text{eff}} \) 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 modeling:

- Single Layer of Elements used for \( P_{\text{eff}} \). Effective nodal forces \( P_{\text{eff}} \) involve only the submatrices \( M_{\text{be}}, K_{\text{be}}, M_{\text{eb}}, K_{\text{eb}} \). 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 effective forces \( P_{\text{eff}} \) is that obtained from the simplified (auxiliary) problem at the nodes that lie on and between boundaries \( \Gamma \) and \( \Gamma_e \).

- Only residual waves outgoing. 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. 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.

- Inside of DRM boundary can be nonlinear/inelastic. This is a very important conclusion, based on a fact that only change of variables was employed in DRM development, and solution does not rely on superposition.

- All types of realistic seismic waves are modelled. Since the effective forcing $P_{\text{eff}}$ 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.

A Note on Free Field Input Motions for DRM. Seismic motions (free field) that are used for input into a DRM model need to be consistent. In other words, a free field seismic wave that is used needs to fully satisfy equations of motion. For example, if free field motions are developed using a tool (SHAKE, or EDT or SW4, or fk, &c.) using time step $\Delta t = 0.01$s and then you decide that you want to run your analysis with a time step of $\Delta t = 0.001$s, simple interpolation (10 additional steps for each of the original steps) might create problems. Simple linear interpolation actually might (will) not satisfy wave propagation equations and if used will introduce additional, high frequency motions into the model. It is a very good idea to generate free field motions with the same time step as it will be used in ESSI simulation.

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

7.4.13 Structure-Soil-Structure Interaction

Introduction

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:

- Vibration of one structure affects the response of a second structure(s) in close proximity to the first\textsuperscript{16}. That is, the amplitude and frequency content of each structure may be modified due to the vibration of others in its neighborhood. This is most likely to occur when the two structures have similar masses or one

\textsuperscript{16} This discussion will focus on two structures, but the concept applies to multiple structures in close proximity.
structure is more massive than a second structure for which the effect may be significant.

- The combined response of the adjacent structures may result in impact (pounding) of the two structures during the earthquake shaking, which may impact loading conditions on the structures for design or evaluation.
- Distribution systems running between structures, e.g., piping, HVAC duct, cable chases, conduit, may experience altered or increased relative displacements during the earthquake shaking.
- 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 design basis earthquake loading conditions and beyond design basis earthquake 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 sub-section below). In this approach, the interaction between the structures in all modes of vibration and the interaction among various modes of vibration are considered.

A second method (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 sub-structuring 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 in Refs. [1] – [4]. 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 bullet #1 above. Generally, the effects of SSSI are secondary to the primary response of a structure due to direct excitation by the earthquake ground motion.

A second simplified method is to compute the ground motion at the footprint of the lighter structure from the SSI analysis of the more massive structure and modify the input motion for SSI analysis of the lighter structure appropriately.
Important considerations in the SSSI analyses are: First, SSSI is a three-dimensional phenomenon. Attempts to analyze it in two dimensions, e.g., plane-strain analysis, introduce uncertainties of unknown magnitude and effect. Second, the SSSI effect may be overemphasized by linear analysis. During SSSI, the soil regions in the immediate neighborhood of the structures appear to behave in a highly nonlinear fashion, which may reduce the effect of the phenomenon. Tajimi [5] indicates that structure-to-structure interaction effects exist, but they are secondary 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 reported in Luco and Contesse [1], Wong and Luco [2,3], and Ostadan et al. [4]. The exception is the response of a lighter structure in close proximity of a more massive structure.

**Detailed methods/Models of SSSI**

Global Models. The simplest and most accurate approach is to develop a direct, detailed model for all (two or more) structures on subsurface soil and rock, develop input seismic motions and analyze 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. Possible approach to developing seismic motions is to use incoherent motions with appropriate separation distance. Alternatively, regional seismic wave modelling can be used to develop realistic seismic motions and use those as input through, for example, the Domain Reduction Method (see Section 7.4.10).

**Simplified Models: Symmetry and Anti-symmetry**

Symmetry and Anti-Symmetry modeling commentary. These models are sometimes used in order to reduce complexity of the direct model (see recent paper by Roy et al. (2013) for example). 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 NPP for rocking and twisting motions, which will then be transferred to adjacent NPP by means of additional, induced surface waves. If only vertically propagating waves are used for input (as is the case for symmetry and antisymmetry models) energy of input surface waves
is neglected. It is noted that depending on the surface wave length 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 wave length can be determined using a classical equation \( \lambda = \frac{v}{f} \) where \( \lambda \) is the length of the (surface) wave, \( v \) is the wave speed\(^{17} \) and \( f \) is the wave frequency of interest. Table 7-1 below gives Rayleigh wave lengths for four different wave frequencies (1, 5, 10, 20 Hz) and for three different Rayleigh (very close to shear) wave velocities (300, 1000, 2500 m/s):

<table>
<thead>
<tr>
<th></th>
<th>1.0Hz</th>
<th>5.0Hz</th>
<th>10.0Hz</th>
<th>20Hz</th>
</tr>
</thead>
<tbody>
<tr>
<td>300m/s</td>
<td>300m</td>
<td>60m</td>
<td>30m</td>
<td>15m</td>
</tr>
<tr>
<td>1000m/s</td>
<td>1000m</td>
<td>200m</td>
<td>100m</td>
<td>50m</td>
</tr>
<tr>
<td>2500m/s</td>
<td>2500m</td>
<td>500m</td>
<td>250m</td>
<td>125m</td>
</tr>
</tbody>
</table>

It is apparent that for given separation between NPP buildings, different surface wave (frequencies) will be differently transmitted with different effects. For example, for an NPP building that has a basic linear dimension (length along the main rocking direction) of 100m, the low frequencies surface wave (1Hz) in soft soil (\( v_s \approx 300 \) m/s) will be able to encompass a complete building within a single wave length, while for the same soil stiffness, the high frequency (20Hz) will produce waves that are too short to efficiently propagate through such NPP structure. On the other hand, for higher rock stiffness (\( v_s \approx 2500 \) m/s), waves with frequencies up to approximately 5Hz, can easily affect a building with 100m dimension.

Further comments on are in order 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):

\(^{17}\) 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.
Symmetry: motions of two NPPs are out phase and this is only achievable, if the wave length of surface wave created by one NPP (radiating toward the other NPP) is so large that half wave length encompasses both NPPs. This type of motions (symmetry) is illustrated in Figure 7-9 below.

Figure -11 Symmetric mode of deformation for two NPPs near each other.

Antisymmetry: motions of two NPPs are in phase. This is achievable if distance between two NPPs is perfectly matching wave lengths of the radiated wave from one NPP toward the other one, and if the dimension of NPPs is not affecting radiated waves. This type of SSSSI is illustrated in Figure 7-11 below.

Figure 7-12 Anti-symmetric mode of deformation for two NPPs near each other.

Both symmetry and antisymmetry 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 NPPs.

References


### 7.4.14 Simplified models

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

Simplified models are:

- Numerical (finite element, ), analytical, and empirical representations of the soil-structure system and seismic input motion;
- 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 should be adequate to assess the effect of the single parameter on the end results of interest. Simplified models can be used for:

- Sanity checks of results from detailed model;
- Sensitivity studies, e.g., investigate the effect of the variation of a single parameter on the end result of interest;

It is very important to note that a significant expertise is required from analyst in order to develop appropriate modelling simplifications that retain mechanical behavior of interest, while simplifying out model components that are not important (for particular analysis).
7.5 PROBABILISTIC RESPONSE ANALYSIS

7.5.1 Overview

In general, probabilistic response analysis is compatible with the definition of a performance goal for design and input requirements for assessments, such as seismic margin assessment (SMA) and seismic probabilistic risk analysis (SPRA). Section 1.3 discusses these needs in the context of performance goals defined probabilistically.

For design, one definition is to calculate seismic demand on structures, systems, and components (SSCs) as 80% non-exceedance probability (NEP) values conditional on the design basis earthquake ground motion (DBE), as specified in ASCE 4. Deterministic response analysis approaches specified in ASCE 4 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.

- For SPRA assessments, the full probability distribution of seismic demand conditional on the ground motion (UHRS or GMRS) is required. 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 as described in succeeding sections. An alternative is to rely on generic studies that have been performed to generate approximate values.

- Two approaches to SMA assessments are used, i.e., the conservative deterministic failure margin (CDFM) method and the fragility analysis (FA) method. For the CDFM, the seismic demand is defined as the 80% NEP conditional on the Review Level Earthquake (RLE). The procedures of ASCE 4 are acceptable to do so. For the FA method, the full probability distribution is required. In both cases, the 80% NEP values are needed to apply the screening tables of EPRI NP-6041 (1991), 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 document, 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.
**Commentary.** In addition to the above described applications of probabilistic analysis methods, to have a rational consistent approach to applying risk-informed techniques to decision-making, requires that the seismic demand on SSCs be quantified probabilistically so that the industry (Licensee and Regulator) knows the likelihood of exceedance of a given loading condition (excitation) applied to individual SSCs and that the likelihood of exceedance when combined with the seismic design and fragility parameters leads to a balanced design.

In today’s world, the framework requires that each SSC be individually designed to meet the code, with nothing in the way of cognizance of how that SSC fits into the overall seismic risk profile, beyond the overall classification of an SSC as either “safety-related” or not. Finally, because the design codes have been developed by different code committees (ASME, ASCE, IEEE, ANS, ACI, etc.) at different times using different ideas, there is little in the way of consistency in the margins that these design codes introduce at or above the design basis, and little in the way of “working together” to produce a rational performance-based design.

### 7.5.2 Simulations of the SSI Phenomena

Generally, some type of simulation is performed to calculate probabilistic responses of SSCs. The seismic analysis and design methodology chain is comprised of: (i) 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); (ii) SSI phenomena; (iii) soil properties (stratigraphy, material properties – low strain and strain-dependent; (iv) structure dynamic characteristics (all aspects); and (v) equipment, components, distribution systems dynamic behaviour (all aspects).

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

**SMACS**

Probabilistic methods to analyze or reanalyze SSCs to develop seismic demands for input to SPRA evaluations were pioneered in the late 1970s in the U.S. NRC Seismic Safety Margin Research Program (SSMRP). A family of computer programs called the Seismic Methodology Analysis Chain with Statistics (SMACS) was developed and implemented in the SPRA of the Zion nuclear power plant in the U.S. (Johnson et al., 1981). This method is often called the “SSMRP” method. The method is still used in the development of seismic response of SSCs of interest to the SPRA so-called Seismic Equipment List (SEL) items (Nakaki et al., 2010).
The SMACS methodology is based on analyzing NPP 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 analyzed to account for variability in the dynamic properties of the soil/structure/subsystems.

For the SSI portion of the seismic analysis chain, the substructure approach is used and the basis for the SMACS SSI analysis is the CLASSI (Continuum Linear Analysis for Soil Structure Interaction) family of computer programs (Luco and Wong, 1980). CLASSI implements the substructure approach to SSI (see Section 7.3).

The basic steps of the SMACS methodology are:

- **Seismic Input**
  - 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 site response analyses (Chapter 6). Three spatial components of motion, two horizontal and the vertical, comprise a set.
  - 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 structure response. Although recent studies have suggested that a greater number of simulations improves the definition of the probability distributions of the response quantities of interest.
  - 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 Chapter 5.

- **Development of the median soil/rock properties and variability.**
  - Chapter 3 describes site investigations to develop the low strain soil profiles – stratigraphy and other soil properties. Chapter 3, 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), Chapter 5 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-centered values and their variability. These viscoelastic properties are defined by
equivalent linear shear moduli and material damping - and Poisson’s ratios.

- 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.

- **Structure model development.**

The structure models are general finite element 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.

The structure model is developed to be median-centered 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.

- **SSI parameters for SMACS:**

  - Foundation input motion. The term “foundation input motion” refers to the result of kinematic interaction of the foundation with the free-field ground motion. The foundation input motion differs from the free-field ground motion in all cases, except for surface foundations subjected to vertically 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 according to 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.

  - 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 x 6, which relates a resultant set of forces and moments to the six rigid body degrees-of-freedom.
• 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-centered foundation impedances and scattering functions are calculated using the embedment geometry and the median soil profile.

• **Statistical sampling and Latin Hypercube experimental design.**

Input 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-centered 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.

• **SMACS analyses.**

Using the free-field time histories, median-centered structural model, median-centered SSI parameters, and experimental design obtained above, perform N SSI analyses. 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; (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 in-structure response spectra (ISRS).

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., 50% non-exceedance probability (NEP), and other NEP values, e.g., the 84% NEP. 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) present the SMACS probabilistic seismic response analyses of selected Nuclear Power Plant structures for the purposes of the SPRA.

- For the NPP site, the ensemble of X-direction seismic input motions displayed as response spectra (5% damping) is presented in Figure 7-11. The input motion variability is apparent.
- 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 7-12.

![Figure 7-8 Nuclear Power Plant – Ensemble of free-field ground motion response spectra – X-direction, 5% damping](image)
Monte Carlo approach to modelling and analysis

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 approach, using 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 analyzed in a deterministic way. In addition, input loading can also be developed into large number (statistically significant) rock motions. Large number of result surface motions, spectra, etc. can then be used to develop statistics (mean, mode, variance, sensitivity, etc.) of site response. Methodology is fairly simple as it utilizes already existing EqL site response modeling, repeated large number of times. There lies a problem, actually, as for proper (stable) statistics, a very large number of simulations need to be performed, which makes this method very computationally intensive.

While Monte Carlo method can sometimes be applied to a 1D EqL site response analysis, any use for 2D or 3D analysis (even linear elastic) creates an insurmountable number of (now more involved, not 1D any more) of simulations that cannot be performed in reasonable time even on large national supercomputers. Problems 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 Monte Carlo approach.

**Random Vibration Theory**

Random Vibration Theory (RVT) is used for evaluating probabilistic site response. Instead of performing a (statistically significant) large number of deterministic simulations of site response (all still in 1D), as described above, the RVT approach can be used (Rathje and Kottke (2008)). RVT uses Fourier Amplitude Spectrum (FAS) of rock motions to develop FAS of surface motions. Developed FAS of surface motions 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.

**Stochastic Finite Element Method**

Instead of using Monte Carlo repetitive 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) (Ghanem and Spanos, 1991). Recently, (Sett et al., 2011a) developed 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 main unknowns (Degrees of Freedom, DoFs) and stress (forces). Of particular importance is the very accurate calculation of full PDF, which supplies accurate tails of PDF, so that Cumulative Distribution Functions (CDFs, 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 requires significant expertise from the analyst. In addition, significant site characterization data is needed in order for uncertain (stochastic) characterization of material properties.

If such data is not available, one can of course resort to (non-site specific) data available in literature (Baecher and Christian, 2003; Phoon and Kulhawy, 1999a,b). However, use of non-site specific data significantly increases uncertainties (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.

### 7.6 Limitations 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. Sensitivity studies should be performed for sensitivity of parameters for chosen modeling level of detail, as well as for sensitivity to modeling detail.

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

REFERENCES for section 7 (also added to bibliography section)

8 Seismic response aspects for design and assessment

8.1 Introduction (Jim)

The objective of the seismic response analysis is to calculate the seismic demands on individual SSCs for purposes of design and assessment. The subject of the current document is SSI response, which means 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.

Chapter 8 identifies the salient features of SSI modelling and analysis and the modelling decisions to be made. It draws extensively from Chapters 1-7 and the Appendices.

All modelling decisions are dependent on the purpose of the analysis:
- Design for design basis earthquake (DBE) – realistic conservatism is to be incorporated into the seismic analyses (and design);
- Assessments for beyond design basis earthquakes (BDBEs) – 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; end results may serve multiple purposes including developing seismic demands for the site and structures, systems, and components (SSCs) to assess realistic or conservative margins to failure;
- Assessments of behavior of nuclear installations subjected to actual earthquake ground motions, so-called forensic investigations.

Level of site specific free-field ground motion (or standard values)
- Peak ground acceleration (PGA) and associated frequency characteristics typically defined by response spectra;
- Other kinematic parameters, such as ground velocity and displacement;
- Other important factors, such as duration of strong shaking;

Strain levels induced in the soil and structure
- Determines requirements for modeling linear, equivalent linear, or nonlinear behavior;

Soil modeling, which is dependent on physical characteristics of the soil and induced strain levels (covered in Chapter 3 and Section 8.3);

Site stratigraphy and topography, including irregular soil/rock profiles, which influence decisions concerning wave propagation characteristics of the ground motion;

Decisions concerning structure modeling consider the following issues (Section 8.4.1):
- Multi-step vs. single step analysis – single step models are necessary when structure behavior and seismic response output quantities are to be calculated directly from the SSI model; single step structure models are required to be detailed enough to provide force and stress results for all required load combinations; generally, single step structure models are required when structure behavior is expected to be nonlinear;
8.1 Introduction

The first step of a multi-step analysis requires the model and analysis of soil, foundation, and structure to adequately represent overall behavior 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 behavior of structure elements, flexibility of floor slabs, and other complex behavior;

Lumped mass stick models (LMSMs) of structures vs. finite element models; typically, single step analyses require more detailed representations of the foundation and structure using finite element methods; LMSMs need to adequately represent the dynamic behaviour of the structure for the purpose of the SSI analysis;

Frequency range of interest – especially high frequency considerations (50 Hz, 100 Hz).

Decisions concerning foundation modelling consider the following items:
- Rigid or flexible behavior, including accounting for stiffening effects due to structure elements connected to the foundation;
- Mat vs. spread/strip footings;
- Piles and caissons; pile groups;
- Boundary conditions – base mat slab retains in contact with soil/separates from underlying soil;
- Surface-or near surface-founded;
- Embedded foundation with partially embedded structure;
- Partially embedded (less than all sides);
- Contact/interface zone for embedded walls and base mat (soil pressure, separation/gapping and sliding).
8.2 Soil Modelling

As explained in chapter 3 of the document 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 3.3). Therefore, it is important that the level of efforts put in the determination of the soil characteristics be adapted to the needs without overshadowing the essential features of soil behavior. In any case, it is essential that soil characteristics be determined by site specific investigations including field tests and laboratory tests, which should, as far as possible, yield coherent soil characteristics; any incoherence should be analyzed and explained. Laboratory tests and field tests shall not be opposed to each other but used in combination since each of them has its own merit, limitation and range of applicability (see Figure 3.10). Special attention must be paid to the characterization of manmade backfills for which the characteristics can only be measured and determined provided 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 later in section 8.3.2.1, PGA should only be regarded as a rough proxy for classifying the earthquake as moderate or high and induced strains should definitely be considered. In others, highly seismic, regions the design earthquake may represent a strong motion event. These features should be considered when defining the soils parameters and associated investigations needed for design. In the first instance (moderate event), and as indicated in section 3.3, the most appropriate constitutive model will be the equivalent viscoelastic linear model (EQL); as pointed out above the decision to use an EQL model should not be based only on PGA and the right indicator to use is the shear strain. This model, which represents the state of practice, is simple enough to be amenable to the large number of parametric and sensitivity analyses required to account for the variability of soil properties (see section 8.6.2). It must be remembered that 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 $G/G_{\text{max}}$ and damping ratio curves used to define the equivalent linear model). It is strongly recommended 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). However, for a preliminary estimate, PGA’s less than 0.2 –0.3g may be considered as moderate earthquakes for which the EQL model is relevant. However, in general, validity of the equivalent linear model has 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. Chapter 3 has proposed to fix that threshold strain to twice the
8.2 Soil Modelling

reference shear strain (see chapter 3 for definition of the reference shear strain) and one example in the appendix on site response illustrates this aspect (see also section 8.3.2); note also that, if in a 1D model the definition of the shear strain is straightforward, in a 3D situation it is proposed to define the “shear strain”, to be compared to the threshold strain, as the second invariant of the deviatoric strain tensor.

In highly seismic areas, 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 may not be appropriate to represent the soil behaviour. True nonlinear soil models are required to analyse soil structure interaction response. As indicated in chapter 3, numerous nonlinear models exist and the choice cannot be unique; it is strongly recommended that at least two different constitutive models be used by possibly two different analysts. The models should 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 should be fully understood by the analysts. Furthermore, it is highly desirable, although not mandatory, that the models possess a limited number of parameters easily amenable to determination and be based on physical backgrounds. As it is well known that soil response is highly sensitive to the chosen model, it is essential that uncertainty in the parameters, especially those with no physical meaning, be assessed through parametric studies. An example of a nonlinear constitutive model is described in the appendix along with the examples on site response analyses.
8.3 Free-field ground motions

8.3 Free field ground motions (Jim)

8.3.1 Overview

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 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 is important.

In terms of SSI analysis, the definition of the seismic input is dependent on the SSI models to be used and their specific requirements, e.g., sub-structuring methods that separate the seismic input problem into kinematic interaction, often with the assumption of the foundation behaving rigidly, require the seismic input to be 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:

Earthquake source to neighbourhood of the site. Several methods of modelling from source to neighbourhood of the site are described in Chapters 4, 5, and 6. The most prevalent approach in the state-of-practice is deterministic seismic hazard analysis (DSHA) or probabilistic seismic hazard analysis (PSHA) based on empirical ground motion prediction equations (GMPEs). The SHA results are most often derived at the top of grade (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.

Local site effects. Given the results of the SHA, the seismic input for the SSI analysis is generated through site response analyses (SRA) or other means to take into account local site effects. 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, the topography of the site and site vicinity, the stratigraphy of the site, and the strain dependent material properties of the media.
8.3 Free-field ground motions

Chapters 5 and 6 discuss aspects of the free-field ground motion and seismic input that may be important to take into account. Later in Chapter 8, 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, Foundation Input Response Spectra (FIRS), 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 should be used carefully especially when generating strain-dependent soil material properties.

Approaches to SRA should be selected based on the basis of the site specific characteristics - site stratigraphy (Section 8.3.3 two-dimensional vs. one dimensional soil profiles), in-situ rock/soil physical attributes (Section 8.3.2), and characteristics of the free-field ground motion (Sections 8.3.4 and 8.3.5).

In the current state-of-practice, 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 8.3.2 discusses the prevalent approaches being applied to the SRA for generating seismic input and soil properties for the SSI analyses.

8.3.2 Site Response Analysis - Approaches 1, 2, 3 (Jim)

The Introduction summarized the current state-of-practice. Chapters 5 and 6 presented many facets of site response analysis (SRA), including anticipated near future research results and methodology developments. Section 8.3.2 expands on the Section 6.3.1 description of current site response analysis approaches for the idealized conditions as follows:

- Soil layer stratigraphy (semi-infinite horizontal layers overlying a uniform half-space), variability in layer thickness is modelled;
- Soil material properties (one-dimensional equivalent linear viscoelastic models defined by shear modulus and material damping – median and variability); (Equation 6-1);
- 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 later in Section 8.3. 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 site amplification functions (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.

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. In current implementation of the PSHA analyses, this could be millions of simulations, which in current technology would be infeasible. Approach 3 is the second most computationally detailed. It is the most comprehensive, rigorous, and feasible approach being implemented today. 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 being 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 site.

Approach 3 incorporates the site amplification functions into the hazard calculation through convolution of the bedrock hazard curves for each spectral frequency with the probability density function for the site amplification functions 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. Appendix 2f discusses Approach 3 in more detail, including a presentation of the flow of calculations to achieve the desired end results. [Need input from Tom Houston on Approach 3 for Appendix 2f]

For Approaches 1, 2A, and 2B type analyses, the following steps are common:

Select best estimate and variability of soil profile(s) to be analyzed. 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 should 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. Select degradation models (G/GMAX and damping with strain level) for each soil strata. The degradation models include variability of G and damping with strain.

Construct the UHRS spectra at the rock outcrop and its characteristics (e.g. deaggregated magnitude and distance of the principal contributors) for the Annual Frequency of Exceedance (AFE) of interest (e.g., 1x10⁻⁴ to 1x10⁻³ for DBE; 1x10⁻⁴ to 1x10⁻⁷ for BDBE considerations).

Select method of analysis – time domain or random vibration theory (RVT).
8.3 Free-field ground motions

Time domain analyses - select seed time histories from PEER (or other) database and scale to UHRS as described below.

Randomize soil properties, degradation models used, depth to horizon for each soil profile to obtain 60 realizations for Latin Hypercube Simulations (LHS). Sixty realizations is typically adequate to perform the SRA.

Perform 1D response analyses (CARES, SHAKE, STRATA, etc) to determine strain compatible properties and response of each of the realizations.

Compute statistics (mean spectra, mean and +/- 1 sigma descriptions of the calculated soil properties) for each soil profile.

When multiple profiles are defined, combine 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 Fourier amplitude spectra. 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:

- Recorded time histories having spectral shapes consistent with the spectral shape associated with the M and D can be selected for the low 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.
- Target spectral shapes associated with the M and D are computed for the low and high frequencies (using, for example, spectral shapes defined by spectral shape formulae in NUREG/CR-6728). 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 spectra is selected to “fill in the gap”.

**Approach 2B**
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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 each of the low and 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 1 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 proportionately to their likelihood.
8.3 Free-field ground motions

8.3.3 1D model

A 1D soil column is used to develop examples, presented in appendix 1, illustrating the differences between the various approaches to 1D site response analyses. The soil profile consists of 30.0m of sandy gravel overlying a 20m thick layer of stiff, overconsolidated, clay on top of a rock layer considered as a homogeneous halfspace (Figure 8-1). The water table is located at a depth of 10.0m below the ground surface. The incident motion is imposed at an outcrop of the halfspace 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.

Under the assumption of vertically propagating shear waves, the numerical model is a one-dimensional 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:

- Compare equivalent linear and nonlinear constitutive models;
- Show the differences between total vs effective stress analyses;
- Show for a 2–phase medium the impact of the soil permeability;
- Compare the predicted vertical motion assuming P-wave propagation to the motion calculated from the horizontal motion with V/H GMPEs (Gülerce & Abrahamson).

Figure 8-1: Soil profile for illustrative examples on 1D site response analyses
8.3 Free-field ground motions

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.

The following Table 8–1 summarizes the different analysed cases. All the nonlinear analyses are run with the software Dynaflow; the equivalent linear analyses are run with SHAKE.

<table>
<thead>
<tr>
<th>Case</th>
<th>1a – 1b</th>
<th>2a – 2b</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
</tr>
</thead>
<tbody>
<tr>
<td>Continuum</td>
<td>1-Phase</td>
<td>1-Phase</td>
<td>2-Phase</td>
<td>2-Phase</td>
<td>2-Phase</td>
<td>2-Phase</td>
</tr>
<tr>
<td>Model</td>
<td>Total stresses</td>
<td>Effective stresses</td>
<td>Effective stresses</td>
<td>Effective stresses</td>
<td>Effective stresses</td>
<td>Effective stresses</td>
</tr>
<tr>
<td>Constitutive relationship</td>
<td>Equivalent linear/ nonlinear</td>
<td>Elastoplastic</td>
<td>Elastoplastic</td>
<td>Elastoplastic</td>
<td>Elastoplastic</td>
<td>Elastoplastic</td>
</tr>
<tr>
<td>Permeability (m/s)</td>
<td>-</td>
<td>0</td>
<td>10^-5</td>
<td>10^-4</td>
<td>10^-3</td>
<td>10^-2</td>
</tr>
<tr>
<td>Input motion</td>
<td>Horizontal</td>
<td>Horizontal</td>
<td>Horizontal</td>
<td>Horizontal</td>
<td>Horizontal</td>
<td>Horizontal</td>
</tr>
<tr>
<td>Software</td>
<td>Shake Dynaflow</td>
<td>Dynaflow</td>
<td>Dynaflow</td>
<td>Dynaflow</td>
<td>Dynaflow</td>
<td>Dynaflow</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Case</th>
<th>6a – 6b</th>
<th>7a – 7b</th>
<th>8</th>
<th>9</th>
</tr>
</thead>
<tbody>
<tr>
<td>Continuum</td>
<td>1-Phase</td>
<td>1-Phase</td>
<td>2-Phase</td>
<td>2-Phase</td>
</tr>
<tr>
<td>Model</td>
<td>Total stresses</td>
<td>Effective stresses</td>
<td>Effective stresses</td>
<td>Effective stresses</td>
</tr>
<tr>
<td>Constitutive relationship</td>
<td>Equivalent linear/ linear</td>
<td>Elastoplastic</td>
<td>Elastoplastic</td>
<td>Elastoplastic</td>
</tr>
<tr>
<td>Permeability (m/s)</td>
<td>-</td>
<td>0</td>
<td>10^-5</td>
<td>10^-2</td>
</tr>
<tr>
<td>Input motion</td>
<td>Horizontal + vertical</td>
<td>Horizontal + vertical</td>
<td>Horizontal + vertical</td>
<td>Horizontal + vertical</td>
</tr>
<tr>
<td>Software</td>
<td>Shake Dynaflow</td>
<td>Dynaflow</td>
<td>Dynaflow</td>
<td>Dynaflow</td>
</tr>
</tbody>
</table>

Table 8-1: Summary of analysed cases

8.3.3.1 Comparison EQL / NL (cases 1a–1b)

The example is used to point out that, beyond some level of shaking, EQL solutions are not valid. Figure 8-2 illustrates the comparison in terms of ground surface response spectra calculated for 3 increasing amplitudes of the input motion. This figure and additional figures presented in the appendix show that, as long as the input motion is not too strong (here pga ~ 0.20g), the EQL and NL solutions do not differ significantly. For 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 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 chapter 3.4: they are dumped because the damping ratio and shear modulus are based on the strain, which is
8.3 Free-field ground motions

controlled by low frequencies, and the same damping is assigned to all frequencies. High frequency motions induce smaller strains and therefore should be assigned less damping.

The value of the pga threshold should not be considered as a universal value: it depends on the material behaviour; as explained in the main document (section 3.3.1), the fundamental parameter to look at is the induced shear strain, or better the reference shear strain $\gamma$. The maximum shear strain calculated as a function of depth for each run is plotted in Figure 8-3 below. 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, equivalent linear 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 3 in the appendix) 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 $2\gamma$ and equivalent linear analyses and nonlinear analyses start to diverge.
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8.3.3.2 Total vs effective stress analyses (cases 1a–2a)

The analyses presented in the appendix show that at low level of excitation (pga ≤0.25g) both solutions (total or effective stress analyses) are comparable except of course 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 (Figure 8-4, 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 (Figure 8-4, right). Based on the results of other analyses, not presented herein, this statement holds as long as the permeability is smaller than approximately 10⁻⁴ m/s. Therefore 2–phase analyses are not required for those permeabilities.

The impact of the value of the permeability appears to be minor on all parameters except the excess pore pressure (Figure 8-5). It is only for high permeabilities (10⁻²m/s) and high input excitations that differences appear in the acceleration response spectrum and, to a minor extent, in the vertical displacement.
Figure 8-4: Comparison of ground surface response spectra for effective and total stress analyses (left) and 1-Phase versus 2-Phase medium (right)
8.3 Free-field ground motions

8.3.3.3 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 5.3.1); as explained in section 6.3.1 and 6.3.3 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 calculated spectra at the surface, $V/H$ ratios have been computed and compared to Gülerce–Abrahamson prediction equation for a magnitude 6.9 event recorded at 15km from the source (Joyner–Boore distance). Results, presented in Figure 8-6 clear 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 only by the vertical propagation of P waves: Rayleigh waves, diffracted P–SV waves also contribute to the response.
8.3 Free-field ground motions

8.3.3.4 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 (2km thick with $V_S = 2000\text{m/s}$) overlying a halfspace 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 and well documented curves for the modulus degradation curve and damping curve of hard rocks. It is subjected to a real hard rock record recorded in Canada and provided by Gail Atkinson: record OT012-HNG with a duration of 60s and a maximum acceleration equal to $0.03\text{g}$; although 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 Fourier amplitude spectrum are shown in the appendix.

Several methods are used for the calculations and illustrated on the rock column:

Pure elastic calculation with a time domain solution obtained with Dynaflow and two meshes: one with ten elements per wave length (element size 5m) and one with 20 elements per wave length (element size 2.5m); differences between both meshes are shown to be negligible and only the results with 10 elements per wave length are presented;

Pure elastic calculation in the frequency domain with the Exponential Window Method (EWM) developed by Eduardo Kausel and introduced in the TECDOC (section 3.4);

Viscoelastic calculation with 0.1% damping in the rock layer (the halfspace is still undamped) with a frequency domain solution: classical FFT (SHAKE) and EWM;
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Viscoelastic calculation with 1% damping in the rock layer (the halfspace is still undamped) with a frequency domain solution: classical FFT (SHAKE) and EWM; Numerical damping in the time domain analysis with Dynaflow (Newmark’s parameter \( \gamma \) set equal to 0.55 instead of 0.50 for no numerical damping); 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 (Figure 8-7).

![ELASTIC ROCK LAYER OVER HALFSPACE](image)

**Figure 8-7: Influence of damping modelling and numerical integration method on ground surface response spectra**

Surface motions are very sensitive to low damping values, in the range 0% to 0.1%. For pure elastic calculation either the time domain solution (without numerical damping) or the EWM should be 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 the classical FFT overdamps the frequencies above 8Hz (in that case). 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; the duration of this quiet zone might be a cause of errors in FFT calculations if not properly chosen. Finally, Rayleigh damping should never 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).

Last but not least, 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.
8.3 Free-field ground motions

8.3.4 2D 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 in the appendix) where a marked topography exists. Calculations are made assuming

a/ a 1D model, extracted from the soil column at the examined location, and an equivalent linear constitutive model,

b/ a 2D model, including the whole valley shown in the figure, with the strain compatible soil properties retrieved from the 1D–EQL analyses,

c/ the same 2D 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 8-8.

![Figure 8-8: Influence of surface topography on ground surface response spectra](image)

The calculated surface spectra clearly indicate that the 1D 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. They also indicate that the main difference between the spectra arise from the geometric model rather than from the constitutive model, although the 2D linear model should be regarded with caution because damping is modelled as Rayleigh damping while in the 2 other analyses frequency independent damping is considered.
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8.3.5 3 X 1C

Modeling 3 components of seismic motions can sometimes be done by modeling independently three components separately by using 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 object/structure dimension. Recent paper by Abell et al. (2018) as well as section 6.3.2.2 provide more details on this modeling approach.

8.3.6 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 analytic solution for elastodynamic wave equations (Thomson 1950, Haskel 1953), or using finite element or finite difference regional scale models, as described by Abell et al. (2018).
8.4 SSI models

8.4.1 Structure (Jim)

8.4.1.1 General considerations

Decision-making on the types of models to be developed and used in SSI analyses are based on the following general considerations.

Determine the characteristics of a structure of interest (identify safety related structures, such as structures housing safety related equipment) and large components located in the yard for which SSI is important:

a. Identify function to be performed during and after earthquake shaking, e.g., leak tightness, i.e., pressure or liquid containment; structural support to subsystems (equipment, components, distribution systems); prevention of failure causing failure of safety related SSCs;

b. Identify load bearing systems for modelling purposes, e.g., shear wall structures, steel frame structures;

c. Expected behaviour of structure (linear or nonlinear);

d. Based on initial linear model of the structure, perform preliminary seismic response analyses (response spectrum analyses) to determine stress levels in structure elements;

e. If significant cracking or deformations are expected, such that portions of the structure behave nonlinearly, refine model either approximately introducing cracked properties or model portions of the structure with nonlinear elements;

f. For expected structure behaviour, assign material damping values;

g. Frequency range of interest – especially high frequency considerations (50 Hz, 100 Hz).

Determine the foundation characteristics (Section 8.4.2):

a. Conventional foundation/structure systems - surface founded and shallow embedment – embedment ratio (embedment depth/effective foundation radius) less than 0.5. Effective stiffness is rigid dependent on base mat stiffness reinforced by connected structure elements, e.g., honey-combed shear walls anchored to the base mat. Effective stiffness is flexible, e.g., if additional stiffening by the structure is not sufficient to assume rigid behavior or for strip footings.

b. Deep foundation (piles).

h. Deeply embedded foundation/structure systems, e.g., small modular reactor (SMR) systems.

Determine the purposes of the SSI analysis and define the use of results:

a. Seismic response of structure for design or assessment (forces, moments, stresses or deformations, story drift, number of cycles of response for fatigue assessment or damage assessment);
b. Input to the seismic design, qualification, evaluation of subsystems supported in the structure (time histories of acceleration and displacement), in-structure response spectra (ISRS), number and amplitude of cycles for components, etc.;

c. Base mat response for base mat design;

d. Soil pressures for base mat and embedded wall designs;

e. Structure-soil-structure interaction analysis;

8.4.1.2 Multi-step vs. single step SSI 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. The subsequent 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 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 multistep analysis represents the overall dynamic behavior of the structural system but need not be refined to predict stresses in structural elements. 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. A lumped-mass stick model (LMSM) may be used for the first-step model provided that the requirements of Section 8.4.1.3 are met.

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: (i) seismic loads and stresses for the design and evaluation of portions of a structure; (ii) 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 behavior of the structural system but need not be refined to predict stresses in structural elements, e.g., a 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 in-structure response spectra (ISRS) are usually needed.

The objectives of one-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 with a detailed “second-step” model introduced above.

Practically speaking, 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 behavior is expected to be nonlinear.
8.4 SSI models

8.4.1.3 Structure modelling requirements

All structure models should satisfy the following requirements: (i) accurately represent the overall dynamic behaviour of the structure; (ii) a three dimensional model to analyze all three directions of earthquake ground motion should 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 modeled); (iii) structural mass (total of structural elements, major components, and appropriate portion of live load) should be lumped so that the total mass, as well as the center of gravity, is preserved; rotational inertia should be included if it affects response in the frequency range of interest; (iv) structural stiffness should be modeled accounting for significant structure characteristics affecting stiffness and load paths, e.g., large floor cutouts; (v) structural stiffness should be modeled accounting for local amplification if expected, e.g., high frequency response (greater than 20 Hz); (vi) model expected nonlinear behavior at the level of excitation of the ground motion of interest; this could be complete nonlinear finite element 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 (ASCE 4-16).

In addition to these requirements for all models, LMSMs: (i) should 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 (recall 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); (ii) may be comprised of multiple sticks with appropriate connectivity at the base mat or at elevations in the structure; (iii) torsional effects resulting from eccentricities between the center of mass and the center of rigidity at each elevation in the model; (iv) a second step analysis will typically be required to generate all detailed response quantities of interest. (ASCE 4-16, Section 3.8.1.3) and/or the requirements of EPRI, 2013a, SPID Section 6.3.1).

8.4.1.4 Decision-making

Based on the considerations itemized in Section 8.4.1.1, decisions as to the type and characteristics of structure models are to be made. In addition to Sections 8.4.1.1 and 8.4.1.3 considerations, practical considerations affect the decision: 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 8.6.4.

In general, three dimensional structure models are required. However, there are situations where this requirement may be eased, i.e., for very long structures where the judgement is made that the structure (and soil) or component (e.g., above grade pipeline) may be modelled in two dimensions (horizontal direction perpendicular to the structures length and the vertical direction).
8.4 SSI models

8.4.2 Foundations

Foundation modelling is separated into conventional foundation/structure systems (surface founded and shallow embedded), deep foundation (piles), and deeply embedded foundation/structure systems.

8.4.2.1 Foundation modelling for conventional foundation/structure systems

Modelling of surface foundations in a global direct SSI analysis, or in substructure analyses provided the analyses are carried out with the same software for all individual steps, does not pose any difficulty: software like PLAXIS, ABAQUS, GEFDYN, DYNAFLOW, SASSI, MISS3D, Real ESSI, etc., can account for foundations with any stiffness. In a conventional substructure approach, however, the usual assumption is to consider the foundation as infinitely stiff 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 this end. These stiffness ratios depend on the foundations characteristics (axial stiffness $E_bS_b$ in kN/ml or bending stiffness $E_bI_b$ in kN.m$^2$/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

$$SR_v = \frac{1}{B} \sqrt{\frac{E_bI_bB}{E}}$$

$$SR_h = \frac{1}{B} \sqrt{\frac{E_bS_bB}{G}}$$

(1)

The foundation can be assumed stiff with respect to the soil when:

- $SR_v > 1$ for vertical and rocking modes
- $SR_h > 5$ for horizontal and torsional modes

Usually the condition on $SR_h$ is always satisfied. For nuclear reactors and 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 shall be carried out, or the stiffness of fictitious rigid foundations around the columns shall be computed and specified at each column base, assuming that no coupling exists between the individual footings.

8.4.2.1.1 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

$$\beta = \frac{K_i}{2K_r} = \frac{\omega C}{2K_r}$$

(2)
The usual practice limits the damping ratio to 30%, but some standards allow for higher values, if properly justified. The main difficulty with the impedance functions is their dependence on frequency, which cannot be easily considered in time domain analyses or modal spectral analyses. Several possibilities exist to approximately take the frequency dependence into account:

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.

To develop a rheological model which accounts for the frequency dependence by addition of masses connected to the foundation with springs and dashpots (De Barros & Luco 1990, Wolf 1994, Saitoh 2012). The parameters of the rheological model are simply determined by curve fitting of the model response to the impedance function. An example of such a model is shown in Figure 8-9 (Pecker 2006).

It should be noted that 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.

Finally, if surface foundations exhibit 2 principal axis of symmetry, the coupling term between horizontal translation and rocking around the transverse horizontal axis may be neglected; this 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 (see for instance Kolias et al. 2012).
8.4.2.1.2 Limitations of the substructure approach

The substructure approach, 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 3.2) and at the soil foundation interface (sliding, uplift, section 7.4.6). 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 7.4.4). This implicitly assumes that additional nonlinearities taking place at the soil foundation interface do not affect significantly the overall seismic response.

8.4.2.2 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 (3D) analysis becomes demanding, especially for nonlinear solutions. The substructure approach, in which the piles and the soil are represented through impedance functions, becomes more attractive provided the system can be assumed to remain linear; note however that, as opposed to shallow foundations, the impedance matrix always contains off–diagonal terms representing the coupling between the horizontal translation and rocking. An example of the substructuring approach is shown in Figure 8-10 for the case of a bridge pier (Kolias et al., 2012).
Another modelling concept has been traditionally 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. It is illustrated in Figure 8-11 for a bridge pier (Kolias et al., 2012). 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:

the choice of the springs and dashpots should reflect the pile group effect and,
as the seismic motion varies with depth, different input motions should be defined at all nodes shared between the piles and the soil; one should resort to a separate analysis for the determination of these input motions.

Therefore, 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, should not be favoured. The substructure approach, with its limitations described below, should be preferred or a full 3D nonlinear model should be used.
Modelling of pile foundations in a substructure method raises several issues:

Can full contact or full separation between the pile cap and the soil be assumed? Should the piles be considered clamped or hinged in the pile cap?

There is no definite answer to each of this question and the situation is very likely to evolve during earthquake shaking. As full consideration of this evolution is incompatible with the substructure approach which assumes linearity of the analysed system, only approximate solutions can be handled.

Contact between pile cap and soil: the contact condition may evolve during the lifetime of the structure due to settlement of the soft layers caused by consolidation of clayey strata, construction around the existing structure, …. It may also evolve during the earthquake due to soil compaction. Results presented in the appendix for horizontal sway, vertical and rocking impedances show that the impact is negligible for vertical and rocking impedances and is only marginally important for the horizontal impedance. These conclusions apply to pile group with a large number of piles and may be different when few piles are considered. Given the insignificant difference between the two assumptions, it is recommended for design purpose to retain the no contact condition, which is the most likely situation in soft soils.

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. The results presented in the appendix compare the dynamic impedances for both extreme conditions: the clamped condition affects the horizontal stiffness but does not affect the vertical or rocking ones. Note however that the reduction in the horizontal stiffness, almost a factor of 2, is certainly much larger than the reduction that would be obtained under the formation of a plastic hinge which, as opposed to the hinged condition, exhibits a residual moment capacity. Regarding the kinematic
interaction forces, if the maximum values are not significantly affected, the distribution of the forces along the pile is totally different with maximum values occurring at deeper locations along the pile when the stiffness deteriorates. Based on the obtained results recommendation would be either to run both types of analyses (clamped and hinged connection) and to take the envelope of both conditions, or for a more conservative approach, to take the maximum forces from the hinged analysis and to extend the maximum values upwards to the pile cap connection.

### 8.4.2.3 Embedded foundation

An example is provided in the appendix illustrating the implementation of the substructure method for an embedded foundation (indeed similar to the implementation for a surface foundation) but which details all the intermediate steps and, more importantly, illustrates how the embedment can affect the foundation input motion due to kinematic interaction. These steps have been described in details in section 7.3.2. The structural model used for this purpose is the Small Modular Reactor model, described in section 8.4.2.4; the soil profile is identical to the one used for the 1D site response analysis with the same input motion scaled to 0.25g, for the EQ analysis to be valid. First, the site response analysis 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.

Note that 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 2D or 3D SSI analysis. The reduction factor to be applied to the P-wave velocity should not be 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, \(V_p\) should be calculated as follows:

\[
B = \rho \left( V_{pe}^2 - \frac{4}{3} V_{se}^2 \right)
\]

where \(\rho\) is the soil mass density,

Calculation of the bulk modulus from the elastic S and P wave velocities \(V_{pe}\) and \(V_{se}\)

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:

\[
B = \rho \left( V_{pe}^2 - \frac{4}{3} V_{se}^2 \right)
\]
8.4 SSI models

\[ V_p = \sqrt{\frac{B}{\rho}} - \frac{4}{3}V_s^2 \]  

(4)

Note that use of eq.(4) may lead to high Poisson's ratio which may create numerical issues; in that case, it is recommended to limit the value to a maximum value compatible with the numerical software used for the SSI analysis; typically it ranges from 0.45 to 0.49. In the example provided in the annex it was limited to 0.48.

With respect to the damping ratio associated with \( V_p \) there is no better assumption than assuming 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 and 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 7.3.2) 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 Figure 8-12 to Figure 8-14.

![Figure 8-12: 5% damped response spectra at roof elevation for 3 embedments](image)

Figure 8-12 compares the 5% damped response spectra on top of the structure (roof elevation) for the global response; the freefield 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.

Figure 8-13: Kinematic translational response spectra

Figure 8-13 and Figure 8-14 presents the foundation input motions (base of the structure) due to pure kinematic interaction (massless structure).

As pointed out in section 7.3.2, when the structure is embedded in the ground, the foundation input motion can no longer be taken equal to the freefield motion. This phenomenon is referred to as kinematic interaction; the calculated motion at the base of the structure are the foundation input motions that should be used in the substructure approach (figure 7-4). It has also been pointed out (section 7.3.2) that embedment creates a rotational component of motion at the foundation, although the incoming motion consists of plane, vertically propagating, shear waves and the soil profile is uniform in the horizontal direction.

As expected, for the case of the surface foundation, kinematic interaction is totally negligible and the rotational component of motion is nil. Be careful that only holds under the assumptions made in the analyses: horizontally layered soil profile subjected to vertically propagating body waves. The increase in the structure embedment has two effects:

- The foundation response spectrum decreases, at all frequencies in this case, when the embedment increases;
- The rotational component of motion increases, at all frequencies in this case, when the embedment increases.
8.4.2.4 Deeply embedded foundations (SMR)

As opposed to shallow or deep foundations, modelling and analysis of deeply embedded structures, like 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)\(^\text{18}\), the conventional substructure approach 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 structure which significantly alters the freefield motion and develops pressures on the lateral walls. Calculation of these two effects in a conventional substructure approach is complicated and tedious:

\(^{18}\) It is recalled that SASSI uses a substructure approach, but the same software and model are used for the analysis of the soil–foundation substructure and of the structure. Conventional substructure approaches calculate the impedance matrix, simplify it with frequency independent springs and dashpots to be connected to the structural model, which is analyzed with a different software (see section 8.4.2.1.1 for instance).
8.4 SSI models

Kinematic interaction motion, i.e. the effective foundation input motion, needs to be calculated from a model reflecting the embedment and variation of the freefield motion with depth; furthermore, the true effective input motion contains a rocking component which is not easily applied to the structural model;

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 pressures and inertia force are likely to be out-of-phase, without any simple solution to easily define the phase shift between both.

Rigorous consideration of these two factors, requires a global finite element model of the embedded part of the structure, and the additional amount of effort to include the structural model is then minimal.

8.4.3 Analyses methods

8.4.3.1 Dynamic analyses

8.4.3.1.1 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 approach and considered, at least in an approximate manner: as indicated in several instances in the TECDOC, reduced soil characteristics can be used in the model; these reduced characteristics represent the strain compatible properties and reflect the soil nonlinearities in the freefield. They are usually calculated from a (1D or 2D) site response analysis (section 8.3.2 and 8.3.3). It is further assumed in the substructure approach that additional nonlinearities that develop due to the interaction between the 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 approach has been described in section 7.3 and the successive steps in the approach are illustrated in the flow chart of Figure 8-15; the flowchart, with reference to the boxes numbers in brackets, is detailed below.
8.4 SSI models

Two examples in the appendices illustrate some of the steps ([5], [6], [7], [10], [12], [13], [14], [15]) 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 approach in which impedances are calculated in a first step and introduced in a structural model.

One example on a deeply embedded structure is developed along the lines of the substructure approach but with all the steps of the SSI analysis run with the same software (SASSI, see note in section 8.4.2.3); therefore, some of the simplifying assumptions of the conventional substructure approach are overcome in this example.

The first step of the analysis starts with the site response analysis to calculate the strain compatible soil characteristics and ground surface response spectra. Site response analyses have been detailed in section 6.3 and 8.3. The input data for this step are:

- the geotechnical data ([1]) from which a design profile and a constitutive model are chosen for the site (section 3);
- the seismological data ([2]) from which the rock spectra ([3]) are established either from a probabilistic, or a deterministic, seismic hazard analysis (section 6.4). Time histories representing the rock motion need to be defined following one of the procedures described in section 6.5.
With these data, site response analyses provide the ground surface motion and the strain compatible soil characteristics ([16]). Usually, they are run assuming an equivalent linear constitutive model as illustrated in the examples on embedded foundation and piles foundation in the appendix. Although nonlinear analyses are also possible, the choice of the strain compatible soil properties is less straightforward in this case and requires some amount of judgment.

The second step corresponds to the top right boxes of the flowchart: it establishes from the formwork drawings ([3]) the structural model ([8]) and the foundation model ([9]). As noted in section 8.4.2, the foundation model for the shallow embedded foundation is assumed to correspond to a stiff foundation; the one for the piles foundation of the appendix 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 ([10]). 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 8.4.2.1.1 presented two possible alternatives to define the frequency independent impedance matrix. This step produces the SSI model ([11]).

The same foundation model and the surface ground motion are used to calculate the kinematic response of the foundation ([9]); this kinematic response is composed of the foundation input motion ([13]) and of the kinematic forces developed in the foundation ([12]).

The foundation input motion serves as the input to the structural model from which the inertial components of the response are retrieved ([14]).

Finally, the results from the inertial response and from the kinematic response are combined ([15]) to yield the structural design quantities: forces, accelerations, displacements, 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:

\[
R_T(t) = \pm R_I(t) \pm R_k(t) \quad (5)
\]

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 should 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 \( \lambda \):
8.4 SSI models

\[ R_T = \pm \max_i R_i(t) \pm \lambda \max_i R_k(t) \quad \text{or} \quad R_T = \pm \lambda \max_i R_i(t) \pm \max_i R_k(t) \] (6)

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, let’s say by 20\%, both maxima are uncorrelated in time and their maximum values can be added with the SRSS rule.

\[ R_T = \sqrt{\left(\max_i R_i(t)\right)^2 + \left(\max_i R_k(t)\right)^2} \] (7)

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 should be added algebraically:

\[ R_T = \max_i R_i(t) + \max_i R_k(t) \] (8)

8.4.3.1.2 Direct methods: linear, nonlinear (SMR, deep foundations, sliding, uplift

A detailed example of a deeply founded structure, a small modular reactor is provided in section I.11.
8.5 Incoherent motions

8.5 Incoherent motions (Jim)

8.5.1 Overview

Seismic motion incoherence is a phenomenon that results in spatial variation of ground motion over large and small distances. Section 5.4 provides a discussion of the sources of incoherence of ground motions – 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 the last 15 years to understand their potential impact on seismic input and structure response of NPP structures. Ground motion incoherence (GMI) is horizontal spatial variation of ground motion, which is one aspect of spatially variable ground motions (SVGM).

GMI occurs due to:

Random spatial variation
- Scattering of waves due to 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).

Wave passage effects
- 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 and they are discussed in detail in Section 5.4. 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 (2km/s to 4km/s) derived from the recorded data used to establish the coherency functions. In the context of the above technical discussion, the treatment of the effects of GMI or SVGM on structure response for typical nuclear power plant (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.

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 requires assignment of an appropriate apparent wave velocity that may be controversial and difficult to defend.

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). Abrahamson refined
8.5 Incoherent motions

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 5.4.

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 in-structure response spectra (ISRS) in progressively higher frequency ranges can be observed.

As noted in Section 5.4, 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:

Seismic responses (ISRS) for assumptions of coherent and incoherent ground motions are required to be calculated to permit comparisons to be made.

Currently, the following guidelines for ISRS, representing current USA practice (NRC) are in place for design:

(i) For the frequency range 0 to 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 to 30 Hz, a maximum reduction based on a linear variation between 0% at 10 Hz and 30% at 30 Hz is permitted.

For BDBE assessments, no such limitations are currently implemented.

8.5.2 Case study

Appendix I.3 presents a case study assessing the effects of GMI on a NPP structure. The subject structure is the Kashiwazaki-Kariwa Nuclear Power Plant (KKNPP) Unit 7 Advanced Boiling Water Reactor (ABWR) reactor building.

The parameters of the study are:

a. The subject structure is the KKNPP Unit 7 reactor building, which experienced the Niigataken-chuetsu-oki (NCO) earthquake (16 July 2007) and was studied extensively post-NCOE (IAEA, 2013). The structure is shown in Figure 8-16.

A detailed finite element dynamic model was developed for the U.S. NRC (Johnson et al., 2015) as part of the KARISMA project (IAEA, 2013) and is used for this case study. The fixed-base model has 480 fixed-base modes with validated horizontal fundamental frequencies: 4.85Hz (43.7% mass)/6.19Hz (24.2% mass) in the X-direction; and 5.64Hz (62.1% mass)/7.43Hz (9.25%) in the Y-direction. Vertical frequencies are not clearly distinguishable, i.e., no single mode or two modes dominate the vertical dynamic response.
8.5 Incoherent motions

b. For simplicity, the foundation is assumed to be surface founded, square in shape (58.0m), and assumed to behave rigidly. The actual foundation/structure configuration is deeply embedded as observed in Figure 7-28. The actual plan dimensions of the structure are: 56.6m (NS) x 59.6m (EW). The coordinate system used in the case study is the same as identified in Figure 7-28 with the bottom of the base mat located at elevation -13.7m (TMSL), +X (N), +Y (W), +Z(up).

c. The site conditions analyzed in the case study are: fixed-base and uniform half-spaces with shear wave velocities (Vs) (Vs = 305m/s; Vs = 2,000m/s).

d. Five sets of earthquake ground motions (acceleration time histories) are analyzed, each comprised of three spatial components (two orthogonal horizontal directions and the vertical direction). Three sets of recorded motions and two sets of artificially generated time histories are used. The sets of recorded motions were selected such that one set is dominated by low frequency motion, one dominated by high frequency, and the third with intermediate frequency content. The sets of artificially generated time histories match the broad-banded response spectra of the U.S. NRC Regulatory Guide 1.60 (RG 1.60) design response spectra and an enhanced RG 1.60 design response spectra, enhanced in the high frequency range.

e. The recorded ground motion excitations are:
   (i) Landers Earthquake M=7.3; June 28, 1992; recording at San Gabriel, E. Grand Ave. Station (Figure 8-17);
   (ii) Imperial Valley Earthquake M=6.9; May 18, 1940; El Centro (Figure 8-18);
   (iii) Val-des-Bois Earthquake, M=5.0, June 23, 2010, Sta OT012 (Figure 8-19);
   All input motions have three spatial components of recorded motion.

f. Abrahamson hard rock ground motion coherency functions (horizontal and vertical) specify local wave scattering for the rock site. Abrahamson soil ground motion coherency functions (horizontal and vertical) specify local wave scattering for the soil site. Figure 5-7 shows all ground motion coherency functions for hard rock and soil at representative observer distances apart.

g. Comparisons are in-structure response spectra (ISRS) for fixed-base, coherent SSI (Vs = 305 m/s and Vs = 2000 m/s), at six locations/three directions each; incoherent SSI (Vs = 305 m/s and Vs = 2000 m/s), at six locations/three directions each. The locations at which response spectra are calculated are:
   (i) Free-field;
   (ii) Base mat center, denoted foundation reference point; (free-field and base mat center are identical for fixed-base case);
   (iii) ISRS are at calculated node points (NPs):
       NP 1640, 1st Fl, FP1 (-18.50,13.75,12.30);
       NP 1694, 1st Fl, CP1 (0.00,-15.50,12.30);
       NP 2419, 3rd Fl, FP1 (18.50,13.75,23.50);
       NP 6594, 3rd Fl, FP2 (20.50,17.00,23.50);
       NP 2559, 4th Fl, FP1 (-18.50,13.75,31.70);
       NP 2621, 4th Fl, FP2 (0.00,-26.50,31.70).
8.5 Incoherent motions

Response spectra were calculated at 151 frequencies between 0.1 and 100 Hz for 5% damping.

The conclusions of Appendix I.3 are:

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 (Vs = 2000m/s), inertial interaction contributes to determining rock-structure natural frequencies. Kinematic interaction effects (local wave scattering effects, i.e., ground motion incoherency (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 structure.

3. In general, GMI has minimal effects on seismic responses in frequency ranges less than 10Hz. This is verified in the current 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 than horizontal responses. Nuclear structures often have multiple vertical modes with frequencies greater than 10Hz each having similar degrees of importance as measured by vertical modal mass. Whereas, important horizontal modes in nuclear structures are low frequency modes, i.e., frequencies less than 10Hz. In two ways, vertical responses are more likely to be affected by GMI than horizontal responses – multiple modes with frequencies greater than 10Hz and multiple modes with relatively close modal mass.

5. The trend of GMI incoherence being important 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 10Hz and often much less than 10Hz. Thereby, GMI being phenomena 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.
### Table 8-2: Seismic analyses performed for coherent and incoherent ground motion assumptions

<table>
<thead>
<tr>
<th>Site Type</th>
<th>Location</th>
<th>Val-des-Bois</th>
<th>RG 1.60</th>
<th>RG 1.60 enhanced</th>
<th>Analyses</th>
</tr>
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<td>FB</td>
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<td>X</td>
<td>X</td>
<td>X</td>
<td>Coherent</td>
</tr>
<tr>
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<td>Imperial Valley</td>
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<td>X</td>
<td>X</td>
<td>Coherent and incoherent</td>
</tr>
<tr>
<td>Soil site</td>
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<td>X</td>
<td>X</td>
<td>X</td>
<td>Coherent and incoherent</td>
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<tr>
<td>Total number of analyses</td>
<td></td>
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<td>5</td>
<td>5</td>
<td>5</td>
</tr>
</tbody>
</table>
8.5 Incoherent motions

Figure 8-16: KKNPP Unit 7, reactor building cross section in N-S direction (IAEA, 2013)
8.5 Incoherent motions

Figure 8-17: Landers Earthquake $M=7.3$; June 28, 1992
8.5 Incoherent motions

Figure 8-18: Imperial Valley Earthquake M=6.9; May 18, 1940
8.5 Incoherent motions

Figure 8-19: Val-des-Bois earthquake, $M=5.0$, June 23, 2010
8.5 Incoherent motions
8.6 Uncertainties and sensitivity studies

8.6.1 Overview

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. In other cases, uncertainties in SSI analysis elements may need to be assessed by sensitivity studies. The results of sensitivity studies are input for model decision-making or to provide alternative credible values that should be considered in the design or assessment by combining the weighted results. The analyst should determine the important elements to the SSI analysis results and include them appropriately.

Some issues amenable to modelling probabilistically are:

- 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.

Others.

Other issues are amenable to sensitivity studies to determine their importance to SSI response:

- One, two, or three dimensional modelling of wave propagation;
- Irregular stratigraphy and topography; non-horizontal layering of soil;
- Linear vs nonlinear soil and structure material properties;
- Coupling between soil and structure(s);
- Sliding/uplift;

In general, uncertainties are categorized into aleatory uncertainty and epistemic uncertainty (Chapter 2).

“aleatory uncertainty: 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.”)”

“epistemic uncertainty: 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 “modelling uncertainty” or “parametric uncertainty”).

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

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

Aleatory 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.). 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, the analyst, by defining parameter variations, implicitly assigns likelihoods to the values, e.g., deterministic SSI analysis for design or assessment, 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, the analyst has assigned a likelihood to each case. In addition, it is common practice to peak broaden ISRS to account for variability in natural frequencies of structures.

It is generally acknowledged that 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.

8.6.2 Ground motion (Jim)

8.6.2.1 Specification of DBE and BDBE ground motion

The largest source of “uncertainty” in the SSI analysis process is definition of the ground motion. Chapters 4, 5, and 6 address seismic hazard assessment, free-field ground motion, and site response leading to seismic input to the SSI analysis process.

The Design Basis Earthquake (DBE) ground motion and the Beyond Design Basis Earthquake (BDBE) may be based on standard ground response spectra (Section 6.4) or site specific ground response spectra developed by probabilistic seismic hazard analysis (PSHA) or deterministic seismic hazard analysis (DSHA) (Chapter 4).
8.6 Uncertainties and sensitivity studies

Standard ground response spectra are often defined at the median, mean, and 84%NEP based on development of a statistical representation of the response spectra generated for recorded earthquake motions. U.S. Regulatory Guide 1.60 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 Hz to 9 Hz) is about 0.30 conditional on a PGA of 1.0g, which represents response spectral peak to valley variability.

In an application for design, 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 spectra, the BDBE ground motion is specified to be a factor times standard ground response spectra, e.g., a factor of 1.4 to 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 8-20 shows an example of probabilistically generated seismic hazard curves for PGA plotted against annual frequency of exceedance (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 8-20 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 uniform hazard response spectrum (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, site response analyses can be performed to generate site amplification functions, which when applied to the rock UHRS yield UHRS at various locations of interest in the site profile, e.g., Foundation Input Response Spectra (FIRS).
Recently, the trend 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 = 1x10^{-5} or 1x10^{-4} (mean or median value). Alternatively, it is defined as the Ground Motion Response Spectra (GMRS), which is calculated by scaling the 1x10^{-5} UHRS based on the relationship between the hazard curves of AFE 1x10^{-5} to 1x10^{-4}.

The RE becomes the base response spectra for seismic response analyses, including SSI analyses.

8.6.2.2 Variabilities in the site specific ground motion

Uncertainties in the site specific ground motion due to earthquake source characteristics and seismic waves travel path from source to neighbourhood 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 ground motion prediction equations (GMPEs), which are developed for the geomean of the two horizontal components of recorded ground motion. Therefore, an adjustment should be made 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 to 0.21 from which the value of 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 COV = 0.25.
8.6 Uncertainties and sensitivity studies

Figure 8-20  Example variability in seismic hazard curves for peak ground acceleration
8.6 Uncertainties and sensitivity studies

8.6.3 Soil

It has been pointed in several instances throughout the document that great uncertainties prevail in the soil characteristics due to the difficulty to test soils, to the inherent randomness and spatial variability of soil deposits; uncertainty in soil characteristics is the second, after the ground motion, largest source of uncertainty in SSI analyses. Spatial variability is characterized by correlation distances of the order of a meter in the vertical direction and of some meters in the horizontal one; 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 (Popescu 1995, Popescu et al. 1995, Assimaki et al. 2003). These models remain however seldom used in practice 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, it has been recommended to consider at least three velocity profiles corresponding to the best estimate characteristics and to those characteristics divided or multiplied by (1+COV); typically, the coefficient of variation (COV) on the elastic shear wave velocity should not be taken less than 0.25. However, 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 $G/G_{\text{max}}$ 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 therefore so does the uncertainty in the prediction of the soil response. Furthermore, there is a large variety of nonlinear models in the technical literature and none of them can be considered as the best model; 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 preferably 2 nonlinear constitutive models, run by 2 different analysts has been recommended.

8.6.4 Structure (Jim) (Uncertainties)

Section 8.4.1 discusses structure modelling. Uncertainties in modelling structure behaviour are:

Modeling of stiffness of load bearing structure elements (linear, equivalent linear, and, possibly, nonlinear).

Modeling of mass of load bearing structure elements, non-structural elements, equipment, components, and distribution systems.
8.6 Uncertainties and sensitivity studies

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 modeling of its stiffness; modeling of mass is much more precise than stiffness.

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.

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).

The damping values are defined by a lognormal distribution. Median values are a function of the seismic response level (ASCE 4-16) 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.

A further important point on damping is that caution should be taken 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 this case, an assessment should be made of the effective damping ratio over all important frequency ranges of interest, such that low frequency or high frequency ranges are not over damped. In addition to overpredicting the damping in some frequency ranges, Rayleigh damping has some other severe drawbacks as pointed out in reference [1]. 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) recommends eliminating the mass proportional damping contribution and bound 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).

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 should be assessed separately, since it is highly dependent on the complexity of the structure itself and the modeling detailing.
8.6.5 Verification and validation of models

SSI models need to be verified and validated in order to increase confidence in modeling and simulation results. Sections 9.4.1 – 9.4.3 provide detailed look at verification and validation (V&V) of analysis code. V&V of SSI models needs to be performed in order to increase confidence in analysis results, particularly when inelastic analysis is performed. Figure 8-21 shows an example of step by step, hierarchical modeling and simulation of an inelastic SSI system.

![Figure 8-21: Step by step, hierarchical modeling and simulation of a n inelastic SSI system, modeling phases and model components.](image)

A soil structure system is modeled in phases. Each phase starts with a linear elastic material model. Material modeling is then made more detailed, slowly, for each component (soil, contacts, structural components, isolators/dissipators, etc.). Geometry of the model starts with a simple 1D, free field soil column, with 1C motions, earthquakes and/or wavelets, being propagated. Same 1C motions are then propagated through a full 3D free field model. Foundation is then added, with expectations that motions at the top of foundation will be...
somewhat similar to the free field motions at the same location. Fixed base structure is analyzed for natural modes (eigen modes) and natural frequencies. Fixed base structure is then shaken with earthquake motions from 1C free field study, then with actual 1C earthquake motions (no SSI effects) and also with wavelets, that will emphasize different dynamical behavior (Ricker, Ormsby wavelets, etc.) (Jeremic et al. 2018).
8.6.6 Structural response quantities

8.6.6.1 Deterministic analyses

Design forces, displacements and stresses

The structural response quantities (displacements, stresses, strains, bending moments, shear forces...) should 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 accounted for by sensitivity analyses, it must be realized that 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 6.5). This variability is further enhanced when nonlinear step by step analyses are implemented. Some regulations (e.g. ASN Guide 2/01) 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 $R_{k,i}$:

$$R_{w,i} = \sum_{k=1}^{N} w_k R_{k,i} \quad \text{with} \quad \sum_{k=1}^{N} w_k = 1$$  \hspace{1cm} (9)

Note that the previous equation 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:

- For modal spectral analyses to

  $$R_D = R_{w,i}$$  \hspace{1cm} (10)

- For step by step linear time history analyses (with $i=1, P$ time histories) to the mean value $\overline{m}$ of the $R_{w,i}$, provided $P \geq 3$;

  $$R_D = m(R_{w,i}) = \frac{1}{P} \sum_{i=1}^{P} R_{w,i} = \frac{1}{P} \sum_{i=1}^{P} \sum_{k=1}^{N} w_k R_{k,i}$$  \hspace{1cm} (11)

- For step by step nonlinear time history analyses ($i=1, K$) to the mean value $m(R_{w,i})$ 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 the following table.
### 8.7 Structural response quantities

<table>
<thead>
<tr>
<th>$K$</th>
<th>$\lambda(K)$</th>
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</thead>
<tbody>
<tr>
<td>5</td>
<td>0.95</td>
</tr>
<tr>
<td>6</td>
<td>0.82</td>
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<tr>
<td>7</td>
<td>0.73</td>
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<tr>
<td>8</td>
<td>0.67</td>
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<tr>
<td>10</td>
<td>0.58</td>
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<td>11</td>
<td>0.55</td>
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<tr>
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<td>13</td>
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<td>40</td>
<td>0.27</td>
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<tr>
<td>50</td>
<td>0.24</td>
</tr>
</tbody>
</table>

In summary, any design quantity is given by

$$R_D = \frac{1}{K} \sum_{i=1}^{K} R_{w,i} + \lambda(K) \sigma(R_{w,i})$$

(12)

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 the previous table.

The rules detailed above are valid for design of new structures in deterministic analyses. For assessment of existing structures, $R_{w,i}$ represents the value calculated for the best estimate properties ($k=1$, no sensitivity analyses) and the same rules apply.

#### 8.6.1.2 Seismic input to sub-systems

The seismic input to sub-systems is represented by the In–Structure Response Spectra (ISRS). ISRS are preferably calculated from the time histories of the response at the required location; however, methods used for direct generation of in-structure response spectra are acceptable when the system remains linear. They shall be computed in accordance with USNRC Regulatory Guide 1.122 and USNRC NUREG-800, 3.7.2, Rev. 3. Consideration should be given in the analysis to the effects on in–structure response spectra (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 should be specifically addressed. To account for these uncertainties in the structural frequencies the computed floor response spectra from the floor time-history motions should be smoothed, and peaks associated with each of the structural frequencies should be 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
8.7 Structural response quantities

the uncertainty (NUREG-800, 3.7.2) and there is no need to further broaden the peaks of the calculated ISRS.

To account for the variability of the soil characteristics, at least three sets of velocity profiles should be used (section 8.6.2) for the analyses. In addition, the ASN2/01 guide recommends broadening by at least 15%, on either side, the peaks of the ISRS associated with the best estimate soil properties.

8.6.7 Probabilistic analyses (Jim)

8.6.7.1 Overview

Probabilistic response analysis is discussed in detail in Section 7.5, including the development of SMACS (Johnson et al., 1981), which is a set of computer program modules to perform probabilistic response analyses. SMACS was initially funded by U.S. NRC. SMACS continues to be a viable tool to calculate probability distributions of seismic responses, e.g., Section 7.5.2 (Nakaki et al., 2010). Various upgrades have been implemented over the ensuing decades.

The SMACS methodology is based on analyzing NPP19 SSCs for simulations of earthquakes defined by acceleration time histories at appropriate locations within the NPP site. Modeling, 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 analyzed to account for variability in the dynamic properties of the soil/structure/subsystems. For purposes of this document, probabilistic SSI (PSSI) analysis is the subject of interest. Therefore, modeling of subsystems and generating their probabilistic seismic response is not included herein except for the development of input to subsystems – in-structure response spectra (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 from the SSI analyses 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 occurring of a given size.

In general, probabilistic response analysis is compatible with the definition of a performance goal for design and for definitions of seismic demand for BDBE assessments, such as seismic margin assessment (SMA) and seismic probabilistic risk analysis (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.

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19 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.
For design, one definition is to calculate seismic demand on structures, systems, and components (SSCs) as 80% non-exceedance probability (NEP) values conditional on the design basis earthquake ground motion (DBE), as specified in ASCE 4-16 (ASCE, 2017). Deterministic response analysis approaches specified in ASCE 4-16 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 BDBE 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 required. The median values (50% NEP) and estimates of the aleatory and epistemic uncertainties (or estimates of composite uncertainty) at the appropriate risk important annual frequency of exceedance are needed. The preferred method of development is by site specific probabilistic response analyses (EPRI, 1994, 2002, 2009, 2013a, 2013b) (ASME/ANS, 2013).

Two approaches to SMA assessments are used, i.e., the conservative deterministic failure margin (CDFM) method and the fragility analysis (FA) method. For the CDFM, the seismic demand is defined as the 80% NEP conditional on the Review Level Earthquake (RLE). The procedures of ASCE 4 are acceptable to do so. For the FA method, the full probability distribution is required. In both cases, the 80% NEP values are needed to apply the screening tables of EPRI NP-6041 (1991), which provides screening values for high capacity SSCs. Most applications of SMA use the CDFM method.

In a SPRA or SMA evaluation, Section 8.6.2.1 introduced the concept of a Reference Earthquake (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 to be 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 analyzed (e.g., cracked or uncracked stiffness for concrete members and level of structure damping), and the strain level at which the underlying soil is analyzed when SSI effects are important.

8.6.7.2 Step-by-step

PSSI analyses differ from many other simulations in that the dynamic excitation (earthquake ground motion) and the behavior of the physical properties of the systems have uncertainty associated with them. Uncertainty in physical properties of the soil-structure system are described by probability distributions (generally, lognormal distributions). Note, Section 8.7.2 is correlated with Section 8.6 uncertainties.

PSSI analyses are based on simulations, which could be based on Monte Carlo sampling (MCS), but are much more efficiently based on Latin Hypercube Simulations (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.

**Free-field ground motion.** 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 8.6.2.2 specifies the uncertainties to be treated in the PSSI analyses:

The COV of the ensemble’s response spectra over the frequency range of interest should be 0.2 or less, i.e., a close fit to the RE target response spectra.

Since the seismic hazard curves are based on ground motion prediction equations (GMPEs), which are developed for the geomean of the two horizontal components of recorded ground motion, an adjustment should 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 to 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 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 Nth input motion.

Vertical seismic input in terms of vertical ground response spectra are 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 accounting for 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.

**Soil profile – stratigraphy, material properties, strain dependent.** Chapter 6 and Sections 8.2 and 8.3 discuss the decisions associated with the definition of the soil profiles to be used in deterministic and probabilistic SSI analyses.

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 (RE) are defined statistically
8.7 Structural response quantities

at the location and form of interest\textsuperscript{20}. 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); generate the ensemble of N free-field ground motions for the SSI analyses; develop N samples of the properties of the soil profile – 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 the Latin Hypercube Simulations (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.

Structure modelling. Sections 8.4.1 and 8.6.4 discuss structure modelling, including uncertainties. In summary, structure models developed for multi-step or single step analyses are assumed to be median-centered and the model dynamic characteristics are well represented by the fixed-base frequencies and mode shapes.

The reader is referred to Section 8.6.4 for identification of parameters to be treated probabilistically and the range of COVs typically considered.

\textsuperscript{20} Examples of location are: top of grade and structure foundation level (FIRS). 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.
8.7 Structural response quantities

8.6.7.3 Epistemic uncertainty – aleatory uncertainty

In summary, unless otherwise identified, the uncertain parameters and the COV values presented herein are composite uncertainties, i.e., the combination of aleatory and epistemic uncertainties combined by the square-root-of-the-sum-of-the-squares (SRSS).

It is generally recognized that 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-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 and aleatory uncertainties in the each calculated response for use in decision-making.
8.7 Appendices: Examples

Site response analyses:
Develop approaches 1, 2, 3
SSI analyses
Incoherence
9 Available Software

Budnitz and Mieler (2016)

9.1 Examples of Commercially Available, Open Source, Open Use and Public Domain, etc.

A number of different licenses that are used to distribute software (source code and/or executables) are described:

- **Commercial Software:** Purchased from a commercial company, features and capabilities usually determined by the commercial license. Commercial programs usually only guarantees accurate working of (in the manual) provided examples. Commercial programs also usually do not provide verification and validation (V&V) suites.

- **Open Source:** An open source license (OSL) (for example General Public License (GPL), or Lesser General Public License (LGPL), or Creative Commons (CC)) is controlling software source code distribution. The OSL guarantees that software source code and derivative source code will be always available through similar OSL. The OSL programs usually do not guarantee quality to external users/developers due to legal reasons (liability). Locally, within development team, they usually have strict quality control.

- **Restricted Source:** A restricted version of an open source license is usually used. The difference is that developers/owners can restrict source code distribution, so a revised OSL license is used (usually a revised version of CC license).

- **Open Use:** A freely available version of program executables are available. There are usually no guaranties of quality, not V&V. Open use

- **Public Domain:** Source code and/or executables are distributed with no restrictions whatsoever. Original developer/owner relinquishes all or his/her rights with respect to sources and/or executables.

9.1.1 Available Software

Listed are programs that can perform full SSI analysis. For completeness, we also list 1D site response codes as they are also used in SSI analysis to provide input motions.
• Commercial Software:
  - ABAQUS
    (web page: http://www.3ds.com/products-services/simulia/products/abaqus/)
  - ADINA
    (web page: http://www.adina.com/)
  - ALGOR/AutoDesk Simulation
    (web page: http://www.autodesk.com/products/simulation/overview)
  - ANSYS
    (web page: http://www.ansys.com/)
  - CLASSI
    (SGH/JJJ and Associates)
  - GT STRUDL
    (web page: http://www.intergraph.com/products/ppm/gt_strudl/default.aspx)
  - LS-DYNA
    (web page: http://www.lstc.com/products/ls-dyna)
  - NASTRAN
  - RIGID
    (SGH/JJJ and Associates)
  - SAP2000
  - SASSI (various versions available (?!))
    * SASSI 2010
      (web page: http://sassi2000.net/)
    * ACS SASSI
      (web page: http://www.ghiocel-tech.com/engineering-tools)
  - SMACS
    (SGH/JJJ and Associates)
  - STARDYNE
STARDYNE.txt
- SOFISTIK
  (web page: http://www.sofistik.com/en/)
- PLAXIS
  (web page: http://www.plaxis.nl/)
- FLAC
  (web page: http://www.itascacg.com/software/flac)
- DYNAFLOW
  (web page: https://blogs.princeton.edu/prevost/dynaflow/)
- Real ESSI on Amazon Web Services
  (web page: http://essi-consultants.com/)
- Zsoil
  (web page: http://www.zsoil.com)

Real ESSI on

  - Open Source, Restricted Source and Open Use:
    - FEAP
      (web page: http://www.ce.berkeley.edu/projects/feap/)
    - DEEPSOIL
      (web page: http://deepsoil.cee.illinois.edu/)
    - SIMQKE1
      (web page: http://nisee.berkeley.edu/elibrary/getpkg?id=SIMQKE1)
    - OpenSees
      (web page: http://opensees.berkeley.edu/)
    - Real ESSI
      (web page:
        http://real-essi.info;
        http://sokocalo.engr.ucdavis.edu/~jeremic/Real_ESSI_Simulator/)
    - Code_ASTER
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(web page: http://www.code_astair.org)

- Public Domain
  - SHAKE91
    (web page: http://nisee.berkeley.edu/elibrary/getpkg?id=SHAKE91)
  - EERA and NEERA
    (web page: http://www.ce.memphis.edu/7137/eera.htm)
  - DESRA-2
    (web page: )
  - SUMDES
    (web page: )
  - D-MOD
    (web page: )
  - TESS
    (web page: )

A bit of details about each, that is distinguish between (a) Linear elastic and equivalent linear elastic, and (b) Incrementally nonlinear, and incrementally inelastic (elastic-plastic).

9.2 Software support

A very important component of each software system is the documentation

9.2.1 Documentation

Documentation can consist of

- Theory Manuals
- Implementation Manuals
- Example Manuals
- Educational Material

9.2 Online Support

- Email Support
- News Groups
9.3 Education and Training

Educational and Training Documentation
Short Courses, in person
Short Courses, online
Refresher Courses (Just in Time, JiT)

9.4 Quality Assurance

ASME NQA-1
ISO9003
V&V

9.4.1 Verification & Validation (V&V) Introduction

Verification and Validation description is based on Jeremic (2016) and Jeremic et al. (1989-2018).

A number of verification activities are recommended.

The main findings are related to verification procedures that are recommended (and they should probably be mandatory!) for modeling and simulation, and for validation procedures that are recommended (as there is a general lack of quality validation data!). A list of procedures is provided below that cover all the components of modeling and simulation and is applicable any numerical analysis of NPP systems, structures and components. It is noted that verification and validation procedures are designed in time domain domain, and that for numerical analysis tools that operate in frequency domain, it is required that V&V procedures need to prove/demonstrate adequacy in time domain, since real earthquake soil structure interaction behavior takes place in time domain.

1. Source code verification has to be provided in order to prove that the program is free of any bugs and inconsistencies that can diminish results. Modeling and simulation program, written in any programming language (C, C++, FORTRAN, etc/) need to perform source code verification with all the necessary steps.
2. Verification and validation for constitutive problems addresses issues related to material modeling and integration of constitutive integration algorithms for nonlinear/inelastic material modeling. Constitutive integration algorithms need to be verified in detail, while material modeling needs to be validated in detail. In addition, seismic energy dissipation is verified at constitutive level calculations.

3. Verification and validation for static and dynamic finite element level solution advancement algorithms address issues related to static and dynamic incremental iterative algorithms that advance (drive) the incremental modeling process forward. These algorithms can introduce (unwanted or wanted) numerical damping/energy production, and as such need to be fully tested against available analytic or very accurate solutions.

4. Verification and validation for static and dynamic behavior of single phase, solid elements addresses modeling using solid finite elements. Addressed is accuracy of modeling of various states of stress (uniaxial, multiaxial) and 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 behavior of structural elements addresses similar set of issues as previous activity, where forces and displacements for structural elements (truss, beam, shell) are verified 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 Behavior of Special Elements addresses issues with contact elements, for both dry and saturated conditions. Of particular interest here is the accuracy of modeling of axial (normal force – gap) and frictional/slipping behavior, as these element are known to misbehave for combination of axial and shear loads.

7. Verification and Validation for Coupled, Porous Solid – Pore Fluid Problems addresses issues with solid finite element that model both porous solid and pore fluid, as is very important for soil and rock. In addition, these coupled elements form a basis for modeling coupled contact, where the contact zone (concrete foundation – soil/rock beneath) is beneath water table.

8. Verification and Validation for Seismic Wave Propagation Problems address issues of proper propagation of seismic waves of predetermined frequency range through finite element models. In addition this activity addresses accuracy and adequacy of seismic input, that encompasses body and surface waves, into finite element models.
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 modeling and simulations are based on approximate methods and that all the obtained results do contain errors. Numerical modelers and analysts need to be aware of these errors and need to address them in presenting their results.

9.4.2 Introduction to Verification and Validation

Verification and validation (V&V) for numerical modeling and simulation represents a basic development task without which no results of such modeling and simulation should be presented. It is important to set the definitions for V&V (Oberkampf et al., 2002):

- **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.

- **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.

With the development of advanced modeling and simulation numerical tools, there is an increased interest in V&V activities, (Roache, 1998; Oberkampf et al., 2002; Oberkampf, 2003; Oden et al., 2005; Babuška and Oden, 2004; Oden et al., 2010a,b; Roy and Oberkampf, 2011)

Importance of V&V activities cannot be overstated! V&V activities and procedures are the primary means of assessing accuracy in modeling and computational simulations. V&V activities and procedures are the tools with which we build confidence and credibility in modeling and computational simulations. Without proper V&V, numerical modeling and simulation results cannot/should not be used for design, licensing or any other activity that relies on those results. Errors, inconsistencies and bug in numerical modeling and simulation programs are present and need to be removed and/or documented. A well known study by Hatton and Roberts (1994); Hatton (1997) 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.).
addition, numerical modeling 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 modeling and simulation programs.

The role of V&V activities can be explained by simple graphs. For example graph in Figure 9.1 (developed after Oberkampf et al. (2002)) shows that mathematical models and computer implementation try to mimic reality.

![Figure 9-1 Role of Verification and Validation per Oberkampf et al. (2002).](image)

Slightly different view V&V activities is presented by Oden et al. (2010a). In this view, V&V must be available as it is a prerequisite for proper numerical modeling and simulation. Results from such V&V-ed modeling and simulations, are then used to gain knowledge about behavior of infrastructure objects. Such knowledge is then used to make (design, licensing, etc.) decisions.

### 9.4.3 Detailed Look at Verification and Validation

A detailed view of V&V is presented in Figure 8.3. It is important to note that the "Real World" is
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Figure 9-2  Detailed view of V&V.

(where ODE stands for Ordinary Differential Equations and PDE stands for Partial Differential Equations) meant to represent a high fidelity knowledge about the realistic behavior of our infrastructure objects. Such behavior is represented by 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.

Verification. The process of determining that a model implementation accurately represents the developer’s conceptual description and specification.

Main goals of verification are to:

- Identify and remove errors in computer coding
  - Numerical algorithm verification
  - Software quality assurance practice

- Quantification of the numerical errors in computed solution
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.

Main goals of validation are to:

- Tactical goal: Identification and minimization of uncertainties and errors in the computational model
- Strategic goal: Increase confidence in the quantitative predictive capability of the computational model

Prediction

Numerical prediction then uses computational model to foretell 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 as:

- Computational models are easily misused (unintentionally or intentionally),
- It will depend on how closely related are the conditions of the prediction and specific cases in validation database, and
- How well is physics of the problem understood.

Verification and Validation Examples

A large number of verification and (not so large set of) validation examples are available in Jeremi´c (2016) and Jeremi´c et al. (1989-2016).
9.4.4 Examples of V&V

Recently, a number of Earthquake Soil Structure Interaction modeling and simulation programs have published Verification and Validation suites. We note two such examples:

- SASSI V&V Project Need references, ALSO which version of SASSI? is it SASSI2000?.

- Real ESSI V&V Suite is a set of scripts and routines that document stability, accuracy and approximation accuracy (errors) of the Real ESSI Simulator (Jeremić et al., 2016).
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