Report to NRC on:
Investigation of Analysis Methods to Incorporate Multi-Dimensional Loading and Incoherent Ground Motions in Soil-Structure Interaction Analysis

by:

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ABSTRACT

The problem of interest is that although seismic ground motions coming into the foundation of a nuclear power plant (NPP) are quite complex, the analysis typically performed for design or for regulatory compliance make approximations by considering only coherent motions in a single direction. The responses to excitation in individual directions are analyzed separately, and then a set of "response combination rules" established in codes and regulations are used to combine them, so as to determine the input for the response calculations. This approach has been used in several analysis areas: for the site response calculations, for the SSI analysis, and also for the SSC (structural, system, and component) response analysis.

For a very long time, the broad community consensus has been that this approach, using the "response combination rules" as they have been established, will generally be conservative. However, this consensus has been increasingly called into question recently, based on research using sophisticated multi-dimensional analysis that are now feasible because of advances in computing power, computer modeling tool advances, and analysis algorithm development, and also based on some recent observations of responses to real earthquakes. The issue seems to be more important for massive structures that can experience rocking and/or torsional response as a result of their deeply embedded foundations. Other problematic responses may also occur for other classes of structures, such as rocking for tall buildings and out-of-phase input motions for long-span bridges.

There are several different but related seismic effects that are not represented well by the current standard approach: incoherent seismic inputs leading to incoherent loadings, inputs coming in at an oblique or inclined angle, multi-directional inputs arising from unusual subsurface effects, and nonlinear geotechnical site-response effects occurring in the top 100 meters of soil.

Taken together, the information that is being developed in the research community and by the most advanced SSI practitioners is calling into question the validity of the current seismic SSI analysis used by most practitioners and also used as the basis for NRC regulatory review. Work presented in this report evaluates advanced methods for SSI, and develops novel approaches that can address these shortcomings in current practice. Described is also work on advancing the state-of-the-art, performed during last year. It is important to note that many sections of this report are *DRAFT* as new project from U.S. NRC (that just started, September 2010) is continuing work on advancing the state-of-the-art in SSI.

DISCLAIMER

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Chapter 1

Introduction

This report presents work performed for the U.S. NRC from October 2009 to October 2010. Work presented here is based in part on earlier work (referenced where appropriate) performed within the Computational Geomechanics Group at University of California at Davis (UC Davis). Current report should be understood as a *living document*, that will undergo changes, improvements and additions as we continue work on a new U.S. NRC sponsored project titled: "Methods, Computational Platform, and Case Studies for Time-Domain Soil-Structure-Interaction Modeling and Simulation Incorporating Complex Seismic Loads" at UC Davis and at LBNL.

1.1 Historical Note

The history of the soil-structure interaction is quite interesting. The first written document that mentions effects of the soil structure interaction is apparently attributed to late Prof. Suyehiro (Suyehiro, 1932). Professor Kyoji Suyehiro was a member of the Japanese Imperial Academy, a Professor of Mechanical Engineering (Naval Engineering Department) at Tokyo Imperial University, a director of Mitsubishi Research Institute for Shipbuilding and later founding director of the Earthquake Research Institute at Tokyo University, (Bulletin of the Seismological Society of America, 1932; Reitherman, 2008; Trifunac, 2009). Professor Suyehiro witnessed Great Kantō earthquake in Tokyo on September 1st, 1923 (actually a series of three closely spaced earthquakes, 11:58am(7.5), 12:01pm(7.3), 12.03pm(7.2) with shaking on soft Tokyo soil lasting until 12:08pm). Professor Suyehiro apparently saw earthquake surface waves travel and buildings sway, similar to ships in the ocean. His research interest switched to Earthquake Engineering after that earthquake and he became the founding Director of the Earthquake Research Institute of the Imperial University of Tokyo, Japan. In 1931 he visited U.S.A. and gave a series of lectures at

the University of California at Berkeley, Stanford University, I the California Institute of Technology, and the Massachusetts Institute of Technology in which he presented his research in Earthquake Engineering (Suyehiro, 1932). In those lectures he outlined his views on Soil-Structure Interaction and noted that four times more damage was observed to soft wooden buildings on soft ground then same buildings on stiff soil, emphasizing one of the first (if not the first) account of earthquake-soil-structure interaction phenomena. Unfortunately. Professor Suyehiro passed away soon after his U.S.A. trip (Bulletin of the Seismological Society of America, 1932).

After those initial publications, more contributions started coming came from Japan (Sezawa and Kanai, 1935a,b,c), and the U.S.A. soon thereafter. They are nicely reviewed by Reitherman (2008).

In an excellent paper, Roesset (1998) presents a thorough overview of the available computational tools of soil structure interaction analysis, emphasizing the limitations and uncertainties of each approach. Although his paper is now 12 years old, many of the remarks are still valid today. He mentions that in the past 10 or 15 years (looking back in 1998), in spite of the continued improvements in computational capabilities, the trend in research has been towards ignoring the more rigorous methodologies already available in order to develop new, alternative, simplified procedures with different degrees of reliability.

Based on the lessons of the last 90 years an ESSI hypothesis can be postulated: the interplay of dynamic characteristic of earthquake ground motions with underlying soil/rock and the NPP structure plays major role in the accumulation of damage, or in the preventing damage. In other words, the timing and spatial location of energy dissipation within the NPPSSS determines the amount of damage in the NPP Structure, Systems and Components during an earthquake. Therefor, achieving a better understanding of the timing and spatial location of energy dissipation can add significant benefit to the safety and economy of NPPSSSs.

1.2 Mechanics of Earthquake-Soil-Structure Interaction

This section describes in some detail mechanics of Earthquake-Soil-Structure (ESS) interaction. Described is the seismic energy propagation from the source, through the rock and soil underlying a Nuclear Power Plant (NPP) to the NPP of interest. In addition to that, described are seismic energy dissipation mechanisms that are (can be) relied on to attenuate earthquake effects on NPPs. In addition to that, described is basic mechanics of ESS interaction. It is important to note that the choice to describe mechanics of ESS interaction, starting from earthquake source to the NPP of interest was not made for "academic" reasons, but rather to give the reader a description of mechanics of the problem of

Earthquake-Soil-Structure Interaction. Once such problem is defined, we proceed to describing modeling and simulation methods used and novel developments.

1.3 Mechanics of Seismic Energy Flow and Dissipation

The energy of seismic motions travels trough rock and soil underlying an NPP, interacts with NPPs on the surface and is dissipated by a number of mechanisms. Proper following of seismic energy as it propagates through the medium, and interacts with the NPP soil-structure system (NPPSSS) can contribute to gaining significant insight into soil-structure interaction (Trifunac et al., 2001). Moreover, proper modeling of seismic energy propagation can be used in designing NPPs where most of the incoming energy is dissipated at locations and in time directed by design (Jeremić, 2010). Given below is brief overview of mechanics of seismic energy propagation, including dissipation mechanism and possible modeling approaches.

Seismic energy input into NPPSSS

Earthquakes release large amounts energy at the source[1] Part of released energy is radiated as mechanical waves ($\approx 1.6 \times 10^{-5}$) and part of that energy makes it to the surface where NPPSSS is located.

Mechanical seismic wave energy enters the NPPSSS system through a closed surface Γ that encompasses soil volume as well as foundation system and the (NPP) structure (Fig. 1.1). Kinetic energy

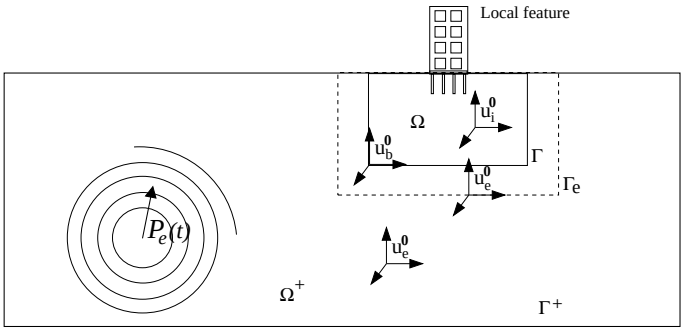

Figure 1.1: Geometry of the SSI system.

[1] for example, some of the recent large earthquake energy releases are listed: Northridge, 1994, $M_{Richter} = 6.7$, $E_r = 6.8 \times 10^{16} J$; Loma Prieta, 1989, $M_{Richter} = 6.9$, $E_r = 1.1 \times 10^{17} J$; Sumatra-Andaman, 2004, $M_{Richter} = 9.3$, $E_r = 4.8 \times 10^{20} J$; Valdivia, Chile, 1960, $M_{Richter} = 9.5$, $E_r = 7.5 \times 10^{20} J$;

flux through closed surface Γ includes both incoming and outgoing waves and can be calculated using Domain Reduction Method (Bielak et al., 2003) as:

$$E_{flux} = \left[0; -M_{be}^{\Omega+}\ddot{u}_e^0 - K_{be}^{\Omega+}u_e^0; M_{eb}^{\Omega+}\ddot{u}_b^0 + K_{eb}^{\Omega+}u_b^0\right]_i \times u_i$$

where $M_{be}^{\Omega+}$, $M_{eb}^{\Omega+}$, $K_{be}^{\Omega+}$, $K_{eb}^{\Omega+}$ are mass and stiffness matrices, respectively for a single layer of elements just outside of the boundary Γ, while $\ddot{u}_e^0$ and u_e^0 are accelerations and displacements from a free field model for nodes belonging to that layer of elements. Alternatively, energy flux can be calculated using (Aki and Richards, 2002):

$$E_{flux} = \rho A c \int_0^t \dot{u}_i^2 dt$$

Outgoing kinetic energy can be obtained from outgoing wave field w_i, (from DRM, (Bielak et al., 2003)), while the difference then represents the incoming kinetic energy that needs to be dissipated withing NPPSSS region. The DRM is described in some more details in chapter 3

Seismic energy dissipation in NPPSSS

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

- wave reflection from impedance boundaries (free surface, soil/rock layers...).
- NPP system oscillation radiation.

The rest of seismic energy is dissipated through one of the following mechanisms within NPPSSS domain:

- Elasto-plasticity of soil
- Viscous coupling of porous solid with pore fluid (air, water)
- Elasto-plasticity/damage of the foundation system
- Elasto-plasticity/damage of the structure
- viscous coupling of structure with surrounding fluids (air, water)

Energy Dissipation by Plasticity. Elastic-plastic deformation of soil, foundation and structure is probably responsible for major part of the energy dissipation for large earthquakes. This, displacement proportional dissipation is a result of dissipation of plastic work ($W = \int \sigma_{ij} d\epsilon_{ij}^{pl}$) and is present in all three components of the system (soil, foundation and the structure). Ideally, majority of the incoming energy would be dissipated in soil, before reaching foundation and structures. The possibility to direct energy dissipation to soil can be used in design by recognizing energy dissipation capacity for different soils. For example, simple elastic-plastic models of stiff and soft clay as well as dense and loose send predict different energy dissipation capacities, as shown in Figure 1.2, for single loading-unloading-reloading cycle. While Figure 1.2 shows that stiff clay and dense sand have much higher dissipation capacity,

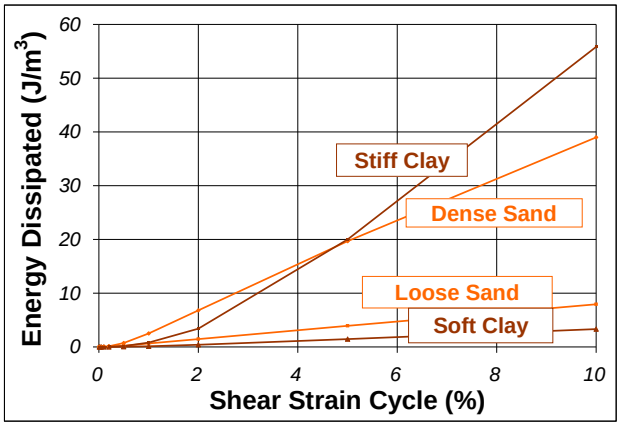

Figure 1.2: Energy dissipation capacity for one cycle at various strains for four generic soils.

it is important to note that soft/loose soils can undergo much larger deformation/strain, thus offering increased energy dissipation capacity through flexibility.

Energy Dissipation by Viscous Coupling. Viscous coupling of porous solids with the entrained pore fluid (air, water) and viscous coupling of the structure and components with their surrounding fluids (air, water) is responsible for velocity proportional energy dissipation. In particular, viscous coupling of porous solids and pore fluid results in energy loss per unit volume $E_{vc} = n^2 k^{-1} (\dot{U}_i - \dot{u}_i)^2$ (where $\dot{U}_i$ and $\dot{u}_i$ are absolute velocities of pore fluid and solid skeleton, respectively) which can contribute to significant energy dissipation. Moreover, presence of any mechanical viscous dampers within the NPP Structure, Systems and Components can add energy dissipation capacity and reduce damage.

Numerical Energy Dissipation and Production. It is important to note that numerical modeling and simulation contributes to artificial energy damping/production within the model. Numerical energy damping/production needs to be properly understood and carefully controlled if accurate results are to be obtained. For example, high frequencies, introduced into model by the discretization technique (usually finite element method, (Argyris and Mlejnek, 1991a; Hughes, 1987)) need to be filtered out in order to improve the accuracy of the model over a wide range of response frequencies (for time domain analysis). On the other hand, elastic-plastic models and simulations usually introduce additional positive (energy dissipation) and negative (energy production) damping. Both Newmark (N) (Newmark, 1959) and Hilber–Hughes–Taylor (HHT) (Hilber et al., 1977) time integration algorithms are energy preserving for linear elastic system with the proper choice of constants ($\alpha = 0.0; \beta = 0.25, \gamma = 0.5$). Both methods can also be used to dissipate higher frequency modes for linear elastic models by changing constants so that for Newmark: $\gamma \geq 0.5, \quad \beta = 0.25(\gamma + 0.5)^2$, while for HHT: $-0.3\dot{3} \leq \alpha \leq 0, \quad \gamma = 0.5(1 - 2\alpha), \quad \beta = 0.25(1 - \alpha)^2$. However, for nonlinear problems it is impossible to maintain a full and accurate accounting of the energy of the system throughout the computations (Argyris and Mlejnek, 1991a). Such modeling details have to be carefully taken into account for high fidelity modeling and simulation of NPPSSSs.

1.4 Modeling Approaches

There exist a number of NPP ESSI modeling approaches. They can be divided in two main groups.

Analytical Solutions are based on solutions of differential equations of motions in closed form. These approaches have limited application for realistic NPPSSCs, mainly due to many simplifying assumptions analyst has to make to make closed form solution possible. Simplifying assumptions are usually made for geometry of the analyzed domain as well as for the material behavior. These solutions, however are excellent for verification studies, and perhaps to offer an initial insight into ESSI problems. Due to a number of simplifying assumptions analytic solutions approaches have the potential to introduce significant modeling uncertainty into their solution.

Numerical Solutions are a preferred method of solving ESSI problems. Two main approaches currently used are based on:

- Integral Equations, or the so called Boundary Element Method (BEM), (example CLASSI, (Wong

and Luco, 1988)). The BEM has some powerful features and can treat many wave propagation problems with great rigor. They do need, however, to make simplifying assumptions, mostly in relation to material behavior (linear elastic or visco-elastic) and to boundary effects (rigid foundations, etc.).

- Finite Element Methods are by far the most popular technique used for ESSI simulations. Two most prominent techniques are the
  - Frequency domain finite elements (example SASSI, (Lymser, 1988; Lysmer et al., 1988)),
  - Direct time domain finite elements (example NRC ESSI Simulator)

Frequency and time domain finite element approaches to ESSI modeling and simulation are briefly reviewed in chapter 2. It is very important to note that for both frequency- and time- domain modeling approaches, modeler/analyst needs to have significant knowledge and experience in numerical modeling and simulation, as well as a solid knowledge of mechanics (dynamics) as applied to NPPSSS ESSI problem. Without such well educated modelers/analysts, both time- and frequency-domain approaches cannot fulfill their full potential.

Chapter 2

Overview of the available computational tools for SSI analysis

There seem to exist three camps of modelers/analysts, believers in frequency domain modeling only, believers in time domain modeling only, and those that try to use the best possible approach/tool for NPPSSS ESSI problem at hand. This Chapter provides a very brief overview of frequency and time domain approaches and focuses on addressing a small set of positive and some negative aspects of both approaches. It is possible to develop a much more in depth review of positive and negative features of both approaches, but this is not done for a simple reason. The reason is that both approaches are not served best by cross comparisons, rather they both need to be assessed based on how they satisfy verification and validation (V&V) criteria, which can then be used to assess the quality of their numerical ESSI predictions. Since V&V criteria are still in development (new project), we postpone an in depth comparison of two approaches until such V&V criteria are available.

2.1 Frequency Domain Approach

Frequency domain modeling has been used for several decades as the main modeling approach for assessing the seismic response of NPPSSSs. The approach is based on frequency domain finite element method, which relies on superposition for obtaining solution. One of the most widely used programs for frequency domain based analysis is SASSI (Lymser, 1988; Lysmer et al., 1988). A number of features of one of the versions of program SASSI (Ostadan, 2007a,b) make it very efficient for seismic analysis of NPPSSSs. Of particular benefit is the large user base, with a large amount of experience, that has

developed over years in analyzing NPPSSSs. Three dimensional seismic loading can be applied to quite sophisticated finite element models of NPPSSS. In addition, radiation damping is mimicked through use of a differential displacement field. Energy dissipation within materials (material damping) is introduced through frequency-dependent (viscous) damping (using complex moduli) (Ostadan, 2007a,b). However, it is important to note that such velocity-proportional (viscous) damping forces are not dominant in a real soil-structure systems (Argyris and Mlejnek, 1991a); Rather, displacement proportional (frictional) damping forces dominate energy dissipation. Velocity-proportional, viscous damping forces are only dominant in viscous dampers, which are sometimes used to damp dynamic energy within NPP components. It is also important to note that use of viscous damping in the frequency domain approach (SASSI for example) is used primarily because it leads to simplifying differential equations used to describe the dynamics of NPPSSS and as such allows for analytic solution (Ostadan et al., 2004). It should be noted, however, that such addition of viscous damping leads to loss of equilibrium (Wilson, 2002).

2.2 Time Domain Approach

Time domain modeling of NPPSSS for seismic events has not been used as frequently as frequency domain modeling. Relative lack of work on time domain modeling for NPPSSS can probably be attributed to a number of factors in the past. However, there is a recent increased interest in developing such modeling and simulation tools. The primary reason for increased interest in time-domain modeling is in the need to properly model material nonlinear (elastic-plastic) behavior of a number of Structures, Systems and Components of the NPPSSS. For example, modeling and simulation for NPPSSSs founded on soil/rock and for NPPSSSs that feature seismic isolation is probably best (most accurately) done using direct time domain elastic plastic finite element modeling. In addition both seismic ground motion input and radiation damping can be analytically modeled using the Domain Reduction Method, allowing for the input of realistic, three dimensional seismic waves into the models, and the efficient modeling of radiation damping that leads to a loss of energy out of the model.

It is very important to note (again) that for both frequency- and time- domain modeling approaches, modeler/analyst needs to have significant knowledge and experience in numerical modeling and simulation, as well as a solid knowledge of mechanics (dynamics) of the NPPSSS ESSI problem. Without such well educated modelers/analysts, both time- and frequency-domain approaches cannot fulfill their full potential.

Chapter 3

State of the Art in Direct Time Domain, Elastic-Plastic Soil-Structure-Interaction Modeling and Simulation

Presented in this Chapter in some detail are some modelling approaches that form a basis for the NRC ESSI Simulator program. Presented approaches and techniques represent the state-of-the-art in ESSI modeling, but are continuously being assessed and improved in order to provide the best possible ESSI modeling for NPPSSS.

3.1 Modeling Seismic Wave and Energy Propagation within NPPSSS

There exist a number o methods for inputting seismic motions into the finite element model. Many of those methods are based on various improvisations but can in general be divided into three basic approaches (Jeremić et al., 1989–2011; Wolf, 1985; Wolf and Song, 2002; Kausel, 2010):

- **No SSI**

  The ground motion is applied directly to the base of the building. Alternatively, instead of applying the ground motion directly to the base of the structure, effective earthquake forces proportional to the base acceleration can be applied to the nodes.

  This procedure is reasonable only for flexible structures on very stiff soil or rock. In this case the displacement of the ground doesn't get modified by the presence of the structure. For stiffer

structures on soil the ground motion has to be applied to the soil. The model has to incorporate propagation of the motion through the soil, its interaction with the structure and the radiation away from the structure.

- **Substructure methods**

  Substructure methods refer to the principle of superposition. The SSI-system is generally subdivided into a structure part and a soil part. Both substructures can be analyzed separately and the total displacement can be obtained by adding the contributions at the nodes on the interface.

  This method reduces the size of the problem considerably. As the time needed for an analysis doesn't increase linearly with an increasing number of equations the substructure method is much faster than the direct method. The biggest drawback of the method however is the fact that linearity is assumed. For nonlinear systems the substructure method cannot be used.

- **Direct methods**

  Direct methods treat the SSI-system as a whole. The numerical model incorporates the spatial discretization of the structure and the soil. The analysis of the entire system is carried out in one step. This method provides most generality as it is capable of incorporating all nonlinear behavior of the structure, the soil and also the interface between those two (sliding, uplift).

For the direct methods, of interest here, different levels of sophistication are possible:

- **Foundation Stiffness Approach**

  The behavior of the soil is accounted for by simple mechanical elements such as springs, masses and dash pots. Different configurations of the subsoil can be taken into account by connecting several springs, masses and dash pots whose parameters have been determined by a curve fitting procedure (Wolf, 1994). This approach is very popular among structural engineers as it is relatively easy to be integrated in a commonly used finite element code.

  Other methods use frequency dependent springs and dash pots and therefore require an analysis in frequency domain. Relatively complex configurations of layered subsoil and embedded foundations can be modeled with good accuracy by replacing the (elastic) soil with a sequence of conical rods (Wolf and Song, 2002; Wolf and Preisig, 2003).

- **p-y Methods**

Attempts have been made to apply the static p-y approach for evaluating lateral loading on pile foundations to dynamic problems. Mostafa and El Naggar (2002) lists several references and provides a parametric study of single piles and pile groups in different soil types under simplified loading cases. Even if p-y curves are widely used for estimating lateral loading on piles they are rarely used in full dynamic soil-structure interaction analysis.

- **Direct, Full 3D**

  Full nonlinear three-dimensional modeling of dynamic soil-foundation-structure interaction can be regarded as the 'brute force' approach. Displacements and forces can be obtained not only for the structure as in the above mentioned methods but also for the soil. In spite of the computational resources and modeling effort required for an analysis it is the only method that remains valid for all kinds of problems involving material nonlinearities, contact problems, different loading cases and complex geometries.

One of the main problems with most currently used methods is the inability to consistently simulate incoming and outgoing waves (Basu and Chopra, 2003; Bielak et al., 2003). Various improvisations are used, some of which even seem to properly model the mechanics of seismic wave propagation into and out of the finite element model.

Treatment of input seismic motions seems to be simpler and various methods can be used, ranging from simple prescription of displacements/accelerations at some nodes close to boundary, the outgoing motions present a real problem. Two main approaches are possible to modeling the seismic motions traveling (leaving) the FEM model and going into unbound media (half space) (Basu and Chopra, 2003):

- Use of rigorous, non-local boundaries and
- Use of approximate, local boundaries.

Rigorous, non-local methods are highly accurate but they significantly increase the size of the problem matrices as they are based on the boundary element method. In addition to that, such closed form solutions, used for boundary elements might not be available for all problem geometries. On the other hand, local, approximate methods, while simple to implement, require large boundary domains in order to dissipate outgoing waves, and are only partially accurate in dissipating such outgoing waves.

Most of the problems with proper modeling of incoming and outgoing seismic waves seem to be properly and elegantly solved using the so called Domain Reduction Method (DRM). The DRM was

developed recently 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 local feature of interest. The local feature can be any geologic or man made object that constitutes a difference from the simplified large domain (free field) for which displacements and accelerations are easier to obtain. The DRM is applicable to a much wider 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). The local feature in general can represent a soil–foundation–structure system (NPPSSS).

3.1.1 The Domain Reduction Method

A large physical domain is to be analyzed for dynamic behavior. 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 3.1).

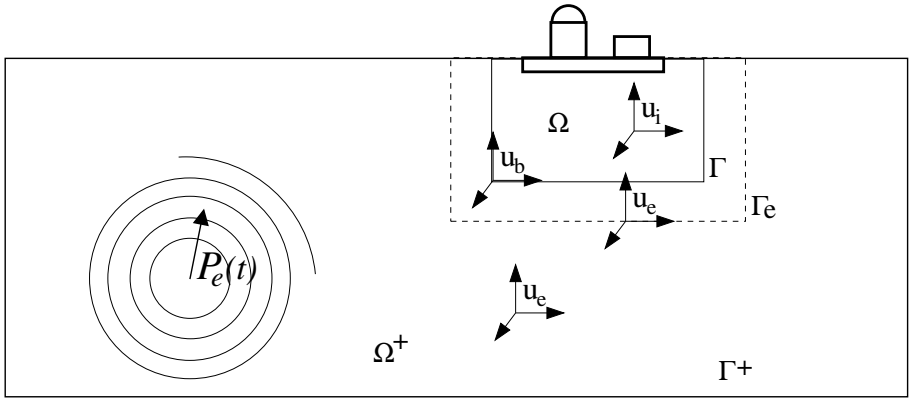

Figure 3.1: Large physical domain with the source of load $P_e(t)$ and the NPPSSS.

The system can be quite large, for example earthquake hypocenter can be many kilometers away from the local feature of interest. It would be beneficial not to analyze the complete system, as we are only interested in the behavior 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 somehow 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 Γ as shown in Figure

3.1. In doing so we must ensure that the dynamic forces $P_e(t)$ are appropriately propagated to the much smaller model boundaries Γ.

Method Formulation

In order to appropriately propagate dynamic forces $P_e(t)$ one actually has to solve the large scale problem which will include the effects of the local feature. Most of the time this is impossible as it involves all the complexities of large scale computations and relatively small local feature. Besides, the main goal of presented developments is to somehow reduce the large scale domain as to be able to analyze in details behavior of the local feature.

In order to propagate consistently the dynamic forces $P_e(t)$ we will make a simplification in that we will replace a local feature with a simpler domain that is much easer to be analyzed. That is, we replace the local feature (NPPSSS) with a much simpler geometry and material. For example, Figure 3.2 shows a simplified model, without a foundation–building system. The idea is to simplify the model so that it is

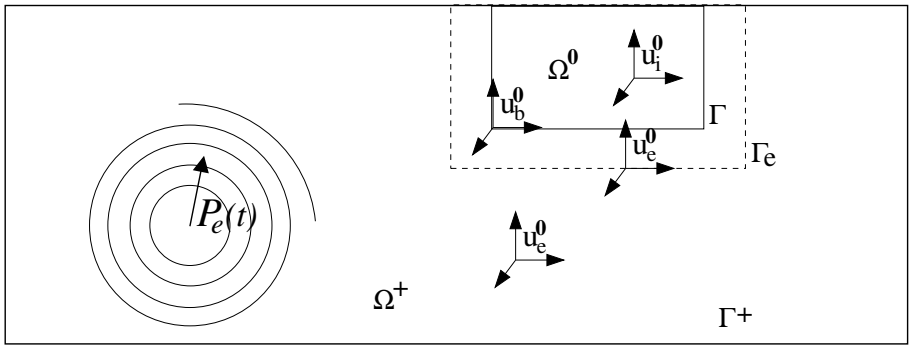

Figure 3.2: Simplified large physical domain with the source of load $P_e(t)$ and **without** the NPPSSS. Instead of the local feature, the model is simplified so that it is possible to analyze it and simulate the dynamic response as to consistently propagate the dynamic forces $P_e(t)$.

much easier to consistently propagate the dynamic forces to the boundary Γ. The notion that it is much easier to propagate those dynamic forces is of course relative. This is still a very complex problem, but at least the influence of local feature is temporarily taken out.

It is convenient to name different parts of domain. For example, the domain inside the boundary Γ is named Ω_0. The rest of the large scale domain, outside boundary Γ, is then named Ω^+. The outside domain Ω^+ is still the same as in the original model, while the change, simplification, is done on the domain inside boundary Γ. The displacement fields for exterior, boundary and interior of the boundary

Γ are u_e, u_b and u_i, on the original domain.

The equations of motions for the complete system can be written as

$$\left[\ M\ \right]\left\{\ \ddot{u}\ \right\}+\left[\ K\ \right]\left\{\ u\ \right\}=\left\{\ P_e\ \right\} \tag{3.1}$$

or if written for each domain (interior, boundary and exterior of Γ) separately, the equations obtain the following form:

$$\begin{bmatrix} M_{ii}^{\Omega} & M_{ib}^{\Omega} & 0 \\ M_{bi}^{\Omega} & M_{bb}^{\Omega}+M_{bb}^{\Omega+} & M_{be}^{\Omega+} \\ 0 & M_{eb}^{\Omega+} & M_{ee}^{\Omega+} \end{bmatrix} \begin{Bmatrix} \ddot{u}_i \\ \ddot{u}_b \\ \ddot{u}_e \end{Bmatrix} + \begin{bmatrix} K_{ii}^{\Omega} & K_{ib}^{\Omega} & 0 \\ K_{bi}^{\Omega} & K_{bb}^{\Omega}+K_{bb}^{\Omega+} & K_{be}^{\Omega+} \\ 0 & K_{eb}^{\Omega+} & K_{ee}^{\Omega+} \end{bmatrix} \begin{Bmatrix} u_i \\ u_b \\ u_e \end{Bmatrix} = \begin{Bmatrix} 0 \\ 0 \\ P_e \end{Bmatrix} \tag{3.2}$$

In these equations, the matrices $\mathbf{M}$ and $\mathbf{K}$ denote mass and stiffness matrices respectively; the subscripts i, e, and b refer to nodes in either the interior or exterior domain or on their common boundary; and the superscripts Ω and Ω^+ refer to the domains over which the various matrices are defined.

The previous equation can be separated provided that we maintain the compatibility of displacements and equilibrium. The resulting two equations of motion are

$$\begin{bmatrix} M_{ii}^{\Omega} & M_{ib}^{\Omega} \\ M_{bi}^{\Omega} & M_{bb}^{\Omega} \end{bmatrix} \begin{Bmatrix} \ddot{u}_i \\ \ddot{u}_b \end{Bmatrix} + \begin{bmatrix} K_{ii}^{\Omega} & K_{ib}^{\Omega} \\ K_{bi}^{\Omega} & K_{bb}^{\Omega} \end{bmatrix} \begin{Bmatrix} u_i \\ u_b \end{Bmatrix} = \begin{Bmatrix} 0 \\ P_b \end{Bmatrix}, \quad \text{in}\Omega \tag{3.3}$$

and

$$\begin{bmatrix} M_{bb}^{\Omega+} & M_{be}^{\Omega+} \\ M_{eb}^{\Omega+} & M_{ee}^{\Omega+} \end{bmatrix} \begin{Bmatrix} \ddot{u}_b \\ \ddot{u}_e \end{Bmatrix} + \begin{bmatrix} K_{bb}^{\Omega+} & K_{be}^{\Omega+} \\ K_{eb}^{\Omega+} & K_{ee}^{\Omega+} \end{bmatrix} \begin{Bmatrix} u_b \\ u_e \end{Bmatrix} = \begin{Bmatrix} -P_b \\ P_e \end{Bmatrix}, \quad \text{in}\Omega^+ \tag{3.4}$$

Compatibility of displacements is maintained automatically since both equations contain boundary displacements u_b (on boundary Γ), while the equilibrium is maintained through action–reaction forces P_b.

In order to simplify the problem, the local feature is removed from the interior domain. Thus, the interior domain is significantly simplified. In other words, the exterior region and the material therein are identical to those of the original problem as the dynamic force source. On the other hand, the interior domain (denoted as Ω_0), is simplified, the localized features is removed (as seen in figure 3.2).

For this simplified model, the displacement field (interior, boundary and exterior, respectively) and action–reaction forces are denoted by u_i^0, u_b^0, u_e^0 and P_b^0. The entire simplified domain Ω_0 and Ω^+ is now easier to analyze.

The equations of motion in Ω^+ for the auxiliary problem can now be written as:

$$\begin{bmatrix} M_{bb}^{\Omega+} & M_{be}^{\Omega+} \\ M_{eb}^{\Omega+} & M_{ee}^{\Omega+} \end{bmatrix} \begin{Bmatrix} \ddot{u}_b^0 \\ \ddot{u}_e^0 \end{Bmatrix} + \begin{bmatrix} K_{bb}^{\Omega+} & K_{be}^{\Omega+} \\ K_{eb}^{\Omega+} & K_{ee}^{\Omega+} \end{bmatrix} \begin{Bmatrix} u_b^0 \\ u_e^0 \end{Bmatrix} = \begin{Bmatrix} -P_b^0 \\ P_e \end{Bmatrix} \tag{3.5}$$

Since there was no change to the exterior domain Ω^+ (material, geometry and the dynamic source are still the same) the mass and stiffness matrices and the nodal force P_e are the same as in Equations (3.3) and (3.4).

Second part of previous equation (3.5) can be used to obtain the dynamic force P_e as

$$P_e = M_{eb}^{\Omega+}\ddot{u}_b^0 + M_{ee}^{\Omega+}\ddot{u}_e^0 + K_{eb}^{\Omega+}u_b^0 + K_{ee}^{\Omega+}u_e^0 \tag{3.6}$$

The total displacement, u_e, can be expressed as the sum of the free field u_e^0 (from the background, simplified model) and the residual field w_e (coming from the local feature) as following:

$$u_e = u_e^0 + w_e \tag{3.7}$$

It is important to note that this is just a change of variables and not an application of the principle of superposition. The residual displacement field, w_e is measured relative to the reference free field u_0^e.

By substituting Equation (3.7) in Equation (3.2) one obtains:

$$\begin{bmatrix} M_{ii}^{\Omega} & M_{ib}^{\Omega} & 0 \\ M_{bi}^{\Omega} & M_{bb}^{\Omega} + M_{bb}^{\Omega+} & M_{be}^{\Omega+} \\ 0 & M_{eb}^{\Omega+} & M_{ee}^{\Omega+} \end{bmatrix} \begin{Bmatrix} \ddot{u}_i \\ \ddot{u}_b \\ \ddot{u}_e^0 + \ddot{w}_e \end{Bmatrix} + \begin{bmatrix} K_{ii}^{\Omega} & K_{ib}^{\Omega} & 0 \\ K_{bi}^{\Omega} & K_{bb}^{\Omega} + K_{bb}^{\Omega+} & K_{be}^{\Omega+} \\ 0 & K_{eb}^{\Omega+} & K_{ee}^{\Omega+} \end{bmatrix} \begin{Bmatrix} u_i \\ u_b \\ u_e^0 + w_e \end{Bmatrix} = \begin{Bmatrix} 0 \\ 0 \\ P_e \end{Bmatrix} \tag{3.8}$$

which, after moving the free field motions u_e^0 to the right hand side, becomes

$$\begin{bmatrix} M_{ii}^{\Omega} & M_{ib}^{\Omega} & 0 \\ M_{bi}^{\Omega} & M_{bb}^{\Omega} + M_{bb}^{\Omega+} & M_{be}^{\Omega+} \\ 0 & M_{eb}^{\Omega+} & M_{ee}^{\Omega+} \end{bmatrix} \begin{Bmatrix} \ddot{u}_i \\ \ddot{u}_b \\ \ddot{w}_e \end{Bmatrix} + \begin{bmatrix} K_{ii}^{\Omega} & K_{ib}^{\Omega} & 0 \\ K_{bi}^{\Omega} & K_{bb}^{\Omega} + K_{bb}^{\Omega+} & K_{be}^{\Omega+} \\ 0 & K_{eb}^{\Omega+} & K_{ee}^{\Omega+} \end{bmatrix} \begin{Bmatrix} u_i \\ u_b \\ w_e \end{Bmatrix} = \begin{Bmatrix} 0 \\ -M_{be}^{\Omega+}\ddot{u}_e^0 - K_{be}^{\Omega+}u_e^0 \\ -M_{ee}^{\Omega+}\ddot{u}_e^0 - K_{ee}^{\Omega+}u_e^0 + P_e \end{Bmatrix} \tag{3.9}$$

By substituting Equation (3.6) in previous Equation (3.9), the right hand side can now be written as

$$\begin{bmatrix} M_{ii}^{\Omega} & M_{ib}^{\Omega} & 0 \\ M_{bi}^{\Omega} & M_{bb}^{\Omega}+M_{bb}^{\Omega+} & M_{be}^{\Omega+} \\ 0 & M_{eb}^{\Omega+} & M_{ee}^{\Omega+} \end{bmatrix} \begin{Bmatrix} \ddot{u}_i \\ \ddot{u}_b \\ \ddot{w}_e \end{Bmatrix} + \begin{bmatrix} K_{ii}^{\Omega} & K_{ib}^{\Omega} & 0 \\ K_{bi}^{\Omega} & K_{bb}^{\Omega}+K_{bb}^{\Omega+} & K_{be}^{\Omega+} \\ 0 & K_{eb}^{\Omega+} & K_{ee}^{\Omega+} \end{bmatrix} \begin{Bmatrix} u_i \\ u_b \\ w_e \end{Bmatrix} = \begin{Bmatrix} 0 \\ -M_{be}^{\Omega+}\ddot{u}_e^0 - K_{be}^{\Omega+}u_e^0 \\ M_{eb}^{\Omega+}\ddot{u}_b^0 + K_{eb}^{\Omega+}u_b^0 \end{Bmatrix} \tag{3.10}$$

The right hand side of equation (3.10) is the dynamically consistent replacement force (so called effective force, P^{eff} for the dynamic source forces P_e. In other words, the dynamic force P_e was consistently replaced by the effective force P^{eff}:

$$P^{eff} = \begin{Bmatrix} P_i^{eff} \\ P_b^{eff} \\ P_e^{eff} \end{Bmatrix} = \begin{Bmatrix} 0 \\ -M_{be}^{\Omega+}\ddot{u}_e^0 - K_{be}^{\Omega+}u_e^0 \\ M_{eb}^{\Omega+}\ddot{u}_b^0 + K_{eb}^{\Omega+}u_b^0 \end{Bmatrix} \tag{3.11}$$

Method Discussion

Single Layer of Elements used for P^{eff}. The Equation (3.11) shows that the effective nodal forces P^{eff} involve only the sub-matrices M_{be}, K_{be}, M_{eb}, K_{eb}. These matrices vanish everywhere except the single layer of finite elements in domain Ω^+ adjacent to Γ. The significance of this is that the only wave-field (displacements and accelerations) needed to determine effective forces P^{eff} is that obtained from the simplified (auxiliary) problem at the nodes that lie on and between boundaries Γ and Γ_e, as shown in Figure 3.1.

Only residual waves outgoing. Another interesting observation is that the solution to problem described in Equation (3.10) comprises full unknowns (displacements and accelerations) inside and on the boundary Γ ($\mathbf{u}_i$ and $\mathbf{u}_b$ respectively). On the other hand, the solution for the domain outside single layer of finite elements (outside Γ_e) is obtained for the residual unknown (displacement and accelerations) field, w_e only. This residual unknown field is measured relative to the reference free field of unknowns (see comments on page 22). That effectively means that the solution to the equation Equation (3.10) outside the boundary Γ_e will only contain additional waves field resulting from the presence of a local feature. This in turn means that if the interest is in behavior of local feature and the surrounding media (all within boundary Γ) one can neglect the behavior of the full model (outside Γ_e in Ω^+) and provide

appropriate supports (including fixity and damping) at some distance from the boundary Γ_e into region Ω^+. This is significant for a number of reasons:

- large models can be reduced in size to encompass just a few layers of elements outside boundary Γ_e (significant reduction for, say earthquake problems where the size of a local feature is orders of magnitudes smaller then the distance to the dynamic source force P_e (earthquake hypocenter).
- the residual unknown field can be monitored and analyzed for information about the dynamic characteristics of the local feature. Since the residual wave field is w_e is measured relative to the reference free field u_0^e, the solution for w_e has all the characteristics of the additional wave field stemming from the local feature.

Inside domain Ω can be inelastic. In all the derivations in section 3.1.1 no restriction was made on the type of material inside the plastic bowl (inside Γ_e). That is, the assumption that the material inside is linear elastic is not necessary as the DRM is not relying on principle of superposition. The Equation 3.7 was only describing the change of variables, and clearly there was no use of the principle of superposition, which is only valid for linear elastic solids and structures. It is therefor possible to assume that the derivations will still be valid with any type of material (linear or nonlinear, elastic or inelastic) inside Γ_e. With this in mind, the DRM becomes a very powerful method for analysis of soil–foundation–structure systems.

3.1.2 Seismic Wave Lack of Correlation (Incoherence)

Seismic motion incoherence (as it is called for frequency domain analysis, for time domain analysis it is called the lack of correlation) is a phenomena that results in spatial variability of ground motions over small distances. A significant work has been done in researching seismic motion incoherence in last few decades. Lack of spatial correlation is due to the following effects (Zerva, 2009)

- Attenuation effects,
- Wave passage effects,
- Scattering effects,
- Extended source effects

Figure 3.3 shows an illustration of main sources of lack of correlation.

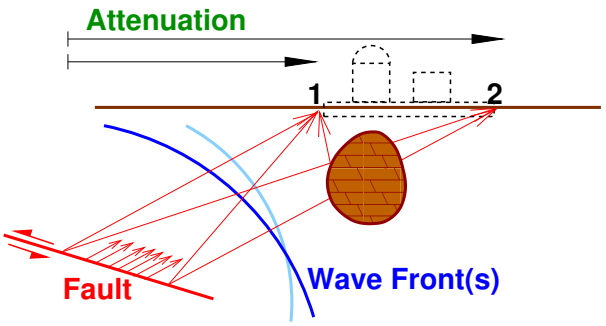

Figure 3.3: Four main sources contributing to the lack of correlation of seismic waves as measured at two observation points.

Attenuation effects are responsible for the change in amplitude and phase of seismic motions due to the distance between observation points and losses (damping, energy dissipation) that seismic wave experiences between observation points. This is a significant source of lack of correlation for long structures (bridges), however for NPPSSS it is not of much significance.

Wave passage effects are due to inclined wave front arriving at observation stations at different times thus creating lack of correlation. Such traveling surface wave(s) can be composed of Rayleigh and Love waves.

Scattering effects are due to (unknown) underground geologic features that scatter seismic waves. Such scattered seismic waves travel and reach observation points at different times and with different intensities, thus creating a lack of correlation.

Extended source effects are due to seismic motions radiating from an extended source, the fault rupturing, with the rupture traveling along the fault. Fault rupturing along certain length (and not at the single point, as is sometimes assumed) creates a complex wave source field. Seismic energy is thus emitted from different points (along the rupturing fault) and will have different travel path and timing as it makes it observation points.

Early studies concluded that the correlation of motions increases as the separation distance between observation points decreases, that is, the closer the observation points are, the seismic motions are more correlated. In addition to that, correlation increased for decrease in frequency of observed motions, that is, lower frequencies see more correlated ground motions, while lack of correlations grows with

increase in frequency. Moreover, there is a strong probabilistic nature of this phenomena, as significant uncertainty is present in relation to all four sources of lack of correlation, mentioned above. A number of excellent references are available on the subject of incoherent (or lacking correlation) seismic motions (Abrahamson et al., 1991; Roblee et al., 1996; Abrahamson, 1992a, 2005, 1992b; Zerva and Zervas, 2002; Liao and Zerva, 2006; Zerva, 2009)

It is very important to note that most current models for modeling incoherent seismic motions make an ergodic assumption. Most models assume that a variability of seismic motions at a single site-source combination will be the same as variability in the ground motions from a data set that was collected over different site and source locations (Walling, 2009). There exist some important consequences of making such ergodic assumption. For example, incoherence models that are developed for one location (based on seismic measurements at one location) might not be transferable to other regions where such measurements were not made.

Currently available incoherence model are developed in frequency domain. To use them for direct time domain simulations, a technique developed by Abrahamson (1992b) is suggested.

3.2 Time Domain Earthquake-Soil-Structure Modelling Details

This section presents some important details of the direct time domain ESSI modeling. Focus is on a small number of practical modeling details, and the section will be expanded as need arises during NRC Staff Capacity Building work that is part of new NRC project.

Numerical Modeling Details: Element Size. In order to represent a traveling wave of a given frequency accurately about 10 nodes per wavelength λ are required (Bathe and Wilson, 1976; Hughes, 1987; Argyris and Mlejnek, 1991a). Fewer than 10 nodes can lead to numerical damping as the discretization misses certain peaks of the wave. In order to determine the appropriate maximum grid spacing the highest relevant frequency f_{max} that is present in the model needs to be found by performing a Fourier analysis of the input motion. Typically, for seismic analysis f_{max} is about 10 Hz. By choosing the wavelength $\lambda_{min} = v/f_{max}$, where v is the wave velocity, to be represented by 10 nodes the smallest wavelength that can still be captured partially is $\lambda = 2\Delta h$, corresponding to a frequency of $5\, f_{max}$.

The maximum grid spacing should not exceed

$$\Delta h \leq \frac{\lambda}{10} = \frac{v}{10\, f_{max}} \tag{3.12}$$

where v is the lowest wave velocity that is of interest in the simulation. Generally this is the shear wave velocity.

Time Step Length Δt. The time step Δt used for numerically solving nonlinear vibration or wave propagation problems has to be limited for two reasons. The stability requirement depends on the numerical procedure in use and is usually formulated in the form $\Delta t/T_n <$ value. T_n denotes the smallest fundamental period of the system. Similar to the spatial discretization T_n needs to be represented by about 10 time steps. While the accuracy requirement provides a measure on which higher modes of vibration are represented with sufficient accuracy, the stability criterion needs to be satisfied for *all* modes. If the stability criterion is not satisfied for all modes of vibration, then the solution may diverge. In many cases it is necessary to provide an upper bound to the frequencies that are present in a system by including frequency dependent damping to the model.

The second stability criterion results from the nature of the finite element method. As a wave front progresses in space it reaches one point after the other. If the time step in the finite element analysis is too large the wave front can reach two consecutive elements at the same moment. This would violate a fundamental property of wave propagation and can lead to instability. The time step therefore needs to be limited to

$$\Delta t \; < \; \frac{\Delta h}{v} \tag{3.13}$$

where v is the highest wave velocity.

Influence of Nonlinear Material Models. If nonlinear material models are used the considerations for stability and accuracy as stated above don't necessarily remain valid. Especially modal considerations need to be examined further for these cases. It is however save to assume that the natural frequencies decrease as plastic deformations occur. The minimum time step required to represent the natural frequencies of the dynamic system can therefore taken to be the same as in an elastic analysis.

A high frequency component can be introduced due to plastic slip and counter balancing of the resulting displacement. This is especially true if a linear algorithm with no iterations within one time step is used (Jeremić et al., 1989–2011).

The frequencies corresponding to these peaks are typically of the order of $1\,/(\text{a few}\,\Delta t)$. Normally the time step is small enough so that these frequencies don't interfere with the input motion. They can be prevented from propagating through the model by an appropriate choice of algorithmic or material damping.

For stability the time step used in a nonlinear analysis needs to be smaller than in a linear elastic analysis. By how much it has to be reduced is difficult to predict as this depends on many factors such as the material model, the applied loading or the numerical method itself. Argyris and Mlejnek (1991b) suggest the time step to be reduced by 60% or more compared to the time step used in an elastic analysis. The best way to determine whether the time step is appropriate for a given analysis consists in running a second analysis with a reduced time step.

Numerical Modeling Details: Boundary Conditions One of the biggest problems in dynamic ESSI in semi-infinite media is related to the modeling 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 has to absorb all outgoing waves and reflect no waves back into the computational domain. The most commonly used domain boundaries (for ESSI simulations) are presented below

- 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 don't exist in reality. The same happens if the degrees of freedom on a boundary are left 'free', as at the surface of the soil.

  A combination of free and fully fixed boundaries should be chosen only if the entire model is large enough and if material damping of the soil prevents reflected waves to propagate back to the structure.

- Absorbing Lysmer Boundaries

  A way 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 dash pots to all degrees of freedom of the boundary nodes and fixing them on the other end (Figure 3.4). Lysmer boundaries are developed for an elastic wave propagation problem in a one-dimensional semi-infinite bar. It can be shown that in this case a dash pot specified appropriately has the same dynamic properties as the bar extending to infinity (Wolf, 1988). The damping coefficient C of the dash pot is $C = A\,\rho\,c$ where A is the section of the bar, ρ 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).

In a 3D or 2D model the angle of incidence of a wave reaching a boundary can vary from almost $0°$ up to nearly $180°$. The Lysmer boundary is able to absorb completely only those waves under an angle of incidence 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.

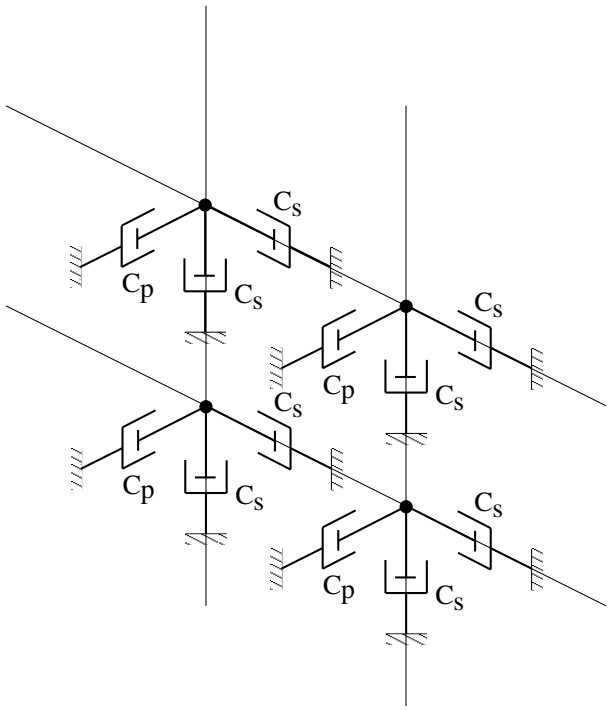

Figure 3.4: Absorbing boundary consisting of dash pots connected to each degree of freedom of a boundary node

- More sophisticated boundaries modeling wave propagation toward infinity

  For a spherical cavity involving only waves propagating in radial direction a closed form solution for radiation toward infinity, analogous to the Lysmer boundary for wave propagation in a prismatic rod, exists (Sections 3.1.2 and 3.1.3 in Wolf (1988)). Since this solution, in contrast to the Lysmer boundary, includes radiation damping it can be thought of as an efficient way of eliminating reflections on a semi-spherical boundary surrounding the computational domain.

More generality in terms of absorption properties and geometry of the boundary are provided by the various boundary element methods (BEM) available in the literature.

- Perfectly matched layer (PML)

  The recently developed PML method, (Basu and Chopra, 2003; Basu, 2009) can also be used to damp out outgoing waves. This method can be thought of as a generalization of Lysmer boundary dashpots and can be quite powerful absorbing multiple frequencies and non-tangential angles-of-incident waves.

- The Domain Reduction Method (DRM)

  The recently developed DRM (Bielak et al., 2003; Yoshimura et al., 2003)), has some excellent features that makes it an almost prefect method for defining boundary conditions for ESSI analysis. This method is described in some more detail in section 3.1.1.

Chapter 4

Computer System for Direct Time Domain, Elastic-Plastic Earthquake-Soil-Structure-Interaction Modeling and Simulation

4.1 NRC Earthquake-Soil-Structure Interaction Simulator (NRC ESSI Simulator)

This section describes a simulation system (software and hardware) for undertaking direct, time-domain finite element Earthquake Soil Structure Interaction simulations. The NRC Earthquake-Soil-Structure-Interaction Simulator (NRC ESSI Simulator) is comprised of two components, the NRC ESSI Simulator Program and the NRC ESSI Simulator Computer. The pronunciation of *ESSI* should be similar to *easy*, as the design, implementation and documentation efforts have ease of use as one of the primary goals.

4.1.1 NRC ESSI Simulator Program

The NRC ESSI Simulator Program is designed to be flexible, modular, portable and efficient. It is organized on a number of functional levels and is based on a number of public domain libraries. The NRC ESSI Simulator Program is following library centric design (Bloch, 2005; Veldhuizen, 2005; Stroustrup, 2005; Ramey, 2005; Niebler, 2005) and as such is easy to expand, verify and maintain. Significant effort

is made to extensively verify the source code, theory behind formulations and the implementation. In addition to that, significant effort is made to develop best possible documentation (theory, verification examples, educational examples, practical examples, etc.). Moreover the Application Programming Interface (API) is developed for a number of functional levels, which will contribute to transparent and easy management of the code. The source code consists of a number of public domain libraries:

- Modified OpenSees Services (MOSS) library that was developed from a trimmed OpenSees core (version of 2005) (Jeremić et al., 1989–2011; Jeremić and Jie, 2007, 2008; McKenna, 1997). In particular, finite element model classes from (class abstractions Node, Element, Constraint, Load, Domain and set of Analysis classes) are used. The domain and analysis classes were significantly modified to improve parallel performance (see PDD description in section 4.1.1), and to remove some known drawbacks of the OpenSees framework. A parallel processing simulation model (desirable for full 3D NPPSSS modeling) is based on the Plastic Domain Decomposition (PDD) method (Jeremić and Jie, 2007, 2008),

- Element and material libraries from the UCD CompGeoMech toolset (Jeremić et al., 1989–2011) were used to supply element and material modeling components. For example Template3Dep numerical libraries (Jeremić and Yang, 2002) are used for constitutive level modeling and FEMtools element libraries from are used to supply single and multi-phase finite elements.

- Numerical utility libraries (blas, lapack, ndarray, matrix...) are use to manipulate tensors, matrices and vectors (Jeremić and Sture, 1998).

- The solution of system of equations is provided by public domain solvers, PETSc (Balay et al., 2001, 2004, 1997)), or UMFPACK (Davis and Duff, 1997a,b, 1999; Davis, 2004a,b).

- Graph libraries used for parallel computations are based on ParMETIS libraries (Karypis et al., 1998)).

- Domain Specific Language (DSL, compiled, interpreted) is being developed for a number of different functional levels (Mernik et al., 2005; Fowler, 2005; Spinellis, 2001; Knuth, 1984).

The NRC ESSI Simulator Program is managed through a well documented API. It is currently hosted on `http://code.google.com`, however, the development will be moved to LBNL in near future. In addition to the source code management, a significant effort is underway to developed a repository of verification, educational and practical (real) NPPSSS example libraries. Initial management of the

source code, documentation and example libraries is done by Prof. Jeremić, however, it is envisioned that at least part of the management will be handed over to NRC Staff, who will act as library editors. The idea is to have a very transparent management style and to develop NRC ESSI Simulator as a two sided platform. A two-sided software platform (Evans et al., 2005, 2006; Boudreau and Hagiu, 2008), connects developers and users through a positive feedback and, if developed properly, can greatly benefit development and use of NRC ESSI Simulator.

High Performance Parallel Algorithm for NRC ESSI Simulator: The Plastic Domain Decomposition

In this section we describe the high performance parallel algorithm used for the NRC ESSI Simulator Program, the so called Plastic Domain Decomposition (PDD).

Domain Decomposition (DD) approach is one of the most popular methods that is used to implement and perform parallel finite element simulations. The underlying idea is to divide the problem domain into subdomains so that finite element calculations will be performed on each individual subdomain in parallel. The DD can be overlapping or non-overlapping. The overlapping domain decomposition method divides the problem domain into several slightly overlapping subdomains. Non-overlapping domain decomposition is extensively used in continuum finite element modeling due to the relative ease to program and organize computations and is the one that will be examined in this work. In general, a good non-overlapping decomposition algorithm should be able to (a) handle irregular mesh of arbitrarily shaped domain, and (b) minimize the interface problem size by delivering minimum boundary conductivities, which will help reducing the communication overheads. Elastic–plastic computations introduce a number of additional requirements for parallel computing. Those requirements are described below.

The Elastic–Plastic Parallel Finite Element Computational Problem

The distinct feature of elastic-plastic finite element computations is the presence of two iteration levels. In a standard displacement based finite element implementation, constitutive driver at each integration (Gauss) point iterates in stress and internal variable space, computes the updated stress state, constitutive stiffness tensor and delivers them to the finite element functions. Finite element functions then use the updated stresses and stiffness tensors to integrate new (internal) nodal forces and element stiffness matrix. Then, on global level, nonlinear equations are iterated on until equilibrium between internal and

external forces is satisfied within some tolerance.

Elastic Computations. In the case of elastic computations, constitutive driver has a simple task of computing increment in stresses ($\Delta\sigma_{ij}$) for a given deformation increment ($\Delta\epsilon_{kl}$), through a closed form equation ($\Delta\sigma_{ij} = E_{ijkl}\Delta\epsilon_{kl}$). It is important to note that in this case the amount of work per Gauss point is known in advance and is the same for every integration point. If we assume the same number of integration points per element, it follows that the amount of computational work is the same for each element and it is known in advance.

Elastic-Plastic Computations. For elastic-plastic problems, for a given incremental deformation the constitutive driver iterate in stress and internal variable space until consistency condition is satisfied. Number of implicit constitutive iterations is not known in advance. Similarly, if explicit constitutive computations are done, the amount of work at each Gauss point is much higher that it was for elastic step. Initially, all Gauss points are in elastic state, but as the incremental loads are applied, the elastic–plastic zones develops. For integration points still in elastic range, computational load is light. However, for Gauss points that are elastic–plastic, the computational load increases significantly (more so for implicit computations than for explicit ones). This computational load increase depends on the complexity of material model. For example, constitutive level integration algorithms for soils, concrete, rocks, foams and other granular materials can be very computationally demanding. More than 70% of wall clock time during an elastic-plastic finite element analysis might be spent in constitutive level iterations. This is in sharp contrast with elastic computations where the dominant part is solving the system of equations which consumes about 90% of run time. The extent of additional, constitutive level computations is not known before the actual computations are over. In other words, the extent of elastic-plastic domain is not known ahead of time.

The traditional pre–processing type of Domain Decomposition method (also known as topological DD) splits domain based on the initial geometry and mesh connectivity and assigns roughly the same number of elements to every computational node while minimizing the size of subdomain boundaries. This approach might result in serious computational load imbalance for elastic–plastic problems. For example, one subdomain might be assigned all of the elastic–plastic elements and spend large amount of time in constitutive level computations. The other subdomain might have elements in elastic state and thus spend far less computational time in computing stress increments. This results in program having to wait for the slowest subdomain (the one with large number of elastic-plastic finite elements)

to complete constitutive level iterations and only proceed with global system iterations after that.

The two main challenges with computational load balancing for elastic–plastic computations are that they need to be:

- Adaptive, dynamically load balancing computations, as the extent of elastic and elastic-plastic domains changes dynamically and unpredictably during the course of the computation.
- Multi-phase computations, as elastic-plastic computations follow up the elastic computations and there is a synchronization phase between those two. The existence of the synchronization step between the two phases of the computation requires that each phase be individually load balanced.

PDD Algorithm The Plastic Domain Decomposition algorithm (PDD) provides for computational load balanced subdomains, minimizes subdomain boundaries and minimizes the cost of data redistribution during dynamic load balancing. The PDD optimization algorithm is based on dynamically monitoring both data redistribution and analysis model regeneration costs during program execution in addition to collecting information about the cost of constitutive level iterations within each finite element. A static domain decomposition is used to create initial partitioning, which is used for the first load increment. Computational load (re–)balancing will (might) be triggered if, during elastic–plastic computations in parallel, one computations on one of compute nodes (one of subdomains) becomes slower than the others compute nodes (other subdomains). Algorithm for computational load balancing (CLB) is triggered if the performance gain resulting from CLB offsets the extra cost associated with the repartitioning. The decision on whether to trigger repartitioning or not is based on an algorithm described in some details below.

We define the global overhead associated with load balancing operation to consist of two parts, data communication cost T_{comm} and finite element model regeneration cost T_{regen},

$$T_{overhead} := T_{comm} + T_{regen} \tag{4.1}$$

Performance counters are setup within the program to follow both. Data communication patterns characterizing the network configuration can be readily measured (T_{comm}) as the program runs the initial partitioning. Initial (static) domain decomposition is performed for the first load step. The cost of networking is inherently changing as the network condition might vary as simulation progresses, so whenever data redistribution happens, the metric is automatically updated to reflect the most current network conditions. Model regeneration cost (T_{regen}) is a result of a need to regenerate the analysis

model whenever elements (and nodes) are moved between computational nodes (CPUs). It is important to note that model (re–) generation also happens initially when the fist data distribution is done (from the static DD). Such initial DD phase provides excellent initial estimate of the model regeneration cost on any specific hardware configurations. This ability to measure realistic compute costs allows developed algorithm (PDD) to be used on multiple generation parallel computers.

For the load balancing operations to be effective, the $T_{overhead}$ has to be offset by the performance gain T_{gain}. Finite element mesh for the given model is represented by a graph, where each finite element is represented by a graph vertex. The computational load imposed by each finite element (FE) is represented by the associated vertex weight $vwgt[i]$. If the summation SUM operation is applied on every single processing node, the exact computational distribution among processors can be obtained as total wall clock time for each CPU

$$T_j := \sum_{i=1}^{n} vwgt[i],\ j = 1, 2, \ldots, np \tag{4.2}$$

where n is the number of elements on each processing domain and np is the number of CPUs. The wall clock time is controlled by T_{max}, defined as

$$T_{sum} := sum(T_j),\ T_{max} := max(T_j), \text{ and } T_{min} := min(T_j),\ j = 1, 2, \ldots, np \tag{4.3}$$

Minimizing T_{max} becomes here the main objective. Computational load balancing operations comprises delivering evenly distributed computational loads among processors. Theoretically, the best execution time is,

$$T_{best} := T_{sum}/np, \text{ and } T_j \equiv T_{best},\ j = 1, 2, \ldots, np \tag{4.4}$$

if the perfect load balance is to be achieved.

Based on definitions above, the best performance gain T_{gain} that can be obtained from computational load balancing operation as,

$$T_{gain} := T_{max} - T_{best} \tag{4.5}$$

Finally, the load balancing operation will be beneficial iff

$$T_{gain} \geq T_{overhead} = T_{comm} + T_{regen} \tag{4.6}$$

Previous equation is used in deciding if the re–partitioning is triggered in current incremental step. It is important to note that PDD will always outperform static DD as static DD represents the first decomposition of the computational domain. If such decomposition becomes computationally unbalanced and efficiency can be gained by repartitioning, PDD be triggered and the T_{max} will be minimized.

Solution of the System of Equations A number of different algorithms and implementations exists for solving unsymmetric[1] systems of equations in parallel. In presented work, use was made of both iterative and direct solvers as available through the PETSc interface (Balay et al., 2001, 2004, 1997). Direct solvers, including MUMPS, SPOOLES, SuperLU, PLAPACK have been tested and used for performance assessment. In addition to that, iterative solvers, including GMRES, as well as preconditioning techniques (Jacobi, inconsistency LU decomposition and approximate inverse preconditioners) for Krylov methods have been also used and their performance assessed.

Some of the conclusions drawn are that, for our specific finite element models (non–symmetric, using penalty method for some connections, possible softening behavior), direct solvers outperform the iterative solver significantly. As expected direct solver were not as scalable as iterative solvers, however, specifics of our finite element models (dealing with soil–structure interaction) resulted in poor initial performance of iterative solvers, that, even with excellent performance scaling, could not catch up with the efficiency of direct solvers. IT is also important to note that parallel direct solvers, such as MUMPS and SPOOLES provided the best performance and would be recommended for use with finite element models that, as ours did, feature non–symmetry, are poorly conditioned (they are ill–posed due to use of penalty method) and can be negative definite (for softening materials).

PDD Scalability Study The developed PDD method was tested on a number of static and dynamic examples. Presented here are scalability (speed–up) results for a series of soil–foundation–structure model runs. A hierarchy of models, was used in scaling study for dynamic runs. Finite element models were subjected to recorded earthquakes (two of them, using DRM for seismic load application (Bielak et al., 2003; Yoshimura et al., 2003).

Total wall clock time has been recorded and used to analyze the parallel scalability of PDD, presented in Figure 4.1. There is a number of interesting observations about the performance scaling results shown in Figure 4.1:

- the scalability is quite linear for small number of DOFs (elements),
- there exists a relation between number of DOFs (elements) and number of CPUs which governs the parallel efficiency. In other words, there exists certain ratio of the number of DOFs to number of CPUs after which the communication overhead starts to be significant. For example for a

[1] Non-associated elasto–plasticity results in a non-symmetric stiffness tensors, which result in non–symmetric system of finite element equations. Non–symmetry can also result from the use of consistent stiffness operators as described by Jeremić and Sture (1997).

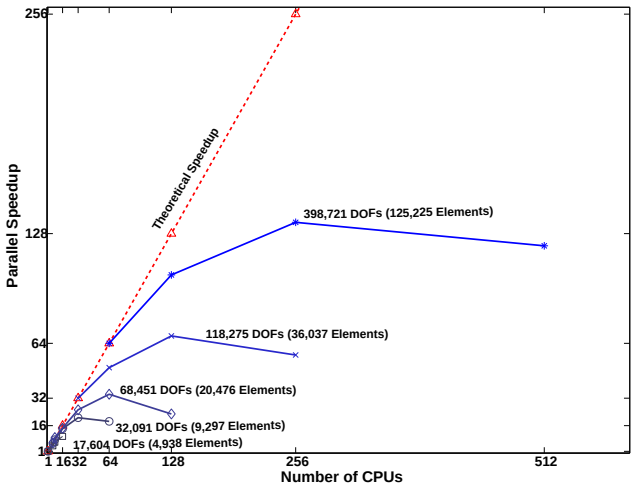

Figure 4.1: Scalability Study on a large ESSI Models, DRM Earthquake Loading, Transient Analysis, ITR=1e-3, Imbal Tol 5%.

models with 484,104 DOFs in Figure 4.1, the computations with 256 CPUs are more efficient that those with 512 CPUs. This means that for the same number of DOFs (elements) doubling the number of CPUs does not help, rather it is detrimental as there is far more communication between CPUs which destroys the efficiency of the parallel computation. Similar trend is observable for the large model with 1,655,559 DOFs, where 1024 CPUs will still help (barely) increase the parallel efficiency. Such efficiency bound can be removed by using a very faster computer network for the NRC ESSI Simulator Computer.

- Another interesting observation has to do with the relative computational balance of local, element level computations (local equilibrium iterations) and the system of equations solver. PDD scales very nicely as its main efficiency objective is to have equal computational load for element level computations. However, the efficiency of the system of equations solver becomes more prominent when element level computations are less prominent (if they have been significantly optimized with a large efficiency gain). For example, for the model with 56,481 DOFs it is observed that for sequential case (1 CPU), elemental computation amount for approx. 70 % of wall clock time. For the same model, in parallel with 8 CPUs, element level computation accounts for approx. 40% of wall clock time, while for 32 CPUs the element level computation account for only 10% of total wall clock time. In other words, as the number of CPUs increase, the element level computations

are becoming very efficient and the main efficiency gain can then be made with the system of equations solver. However, it is important to note that parallel direct solver are not scalable up to large number of CPUs (Demmel et al., 1999) while parallel iterative solver are much more scalable but difficult to guarantee convergence. This observation can be used in fine tuning of parallel computing efficiency, even if it clearly points to a number of possible problems.

4.1.2 NRC ESSI Simulator Parallel Computer

The NRC ESSI Simulator consists of both software (program) and hardware (parallel computer) components. The hardware component, the NRC ESSI Simulator Computer, is a distributed memory parallel computer, with a number of compute nodes (which feature multiple processors and cores). Connectivity is provided by a fast ethernet network (Gigabit or higher, although other high bandwidth / low latency network can be used as well). While the new NRC ESSI Simulator Computer is being designed and acquired (in near future), the GeoWulf parallel computer is currently used at UCD. GeoWulf computer (see Figure 4.2) features design similar to the new NRC ESSI Simulator Computer, and currently has 36 compute cores (multiple compute cores per compute node), one Nvidia TESLA1060 with 240 GPGPU cores, over 53GB of distributed memory and over 8TB of distributed disk space.

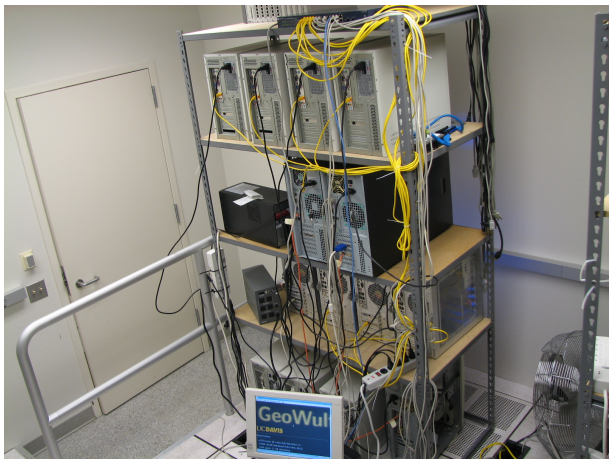

Figure 4.2: GeoWulf computer used for ESSI simulations.

Figure 4.3 shows GeoWulf architecture. Distributed memory parallel computers are very cost effective, affordable, have high availability (they are local, you control them directly) and have fairly simple, replicable design that can be used by professional practice companies. Distributed memory parallel computer architecture is very similar to the architecture of large parallel supercomputers (SDSC, TACC,

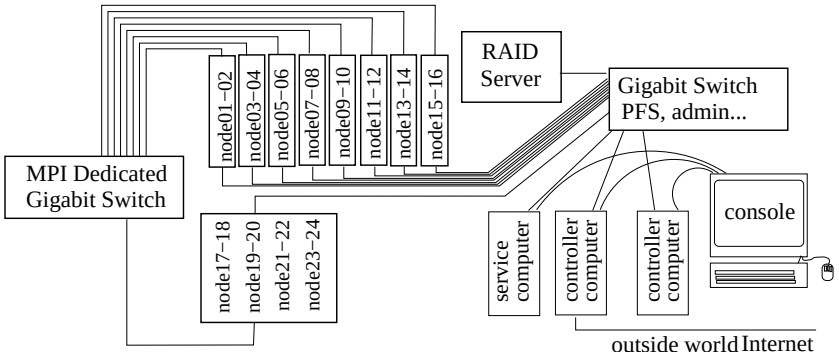

Figure 4.3: Distributed Memory Parallel Computer GeoWulf architecture. Similar architecture will be applied to NRC ESSI Simulator, just scaled up (in number of compute nodes, memory and disk space) by about an order of magnitude.

EarthSimulator, etc.), which means that NRC ESSI Simulator Program is source code compatible with any of these (or any other) distributed parallel computers. More details about the GeoWulf parallel computer are available at `http://sokocalo.engr.ucdavis.edu/~jeremic/GeoWulf/`. Similar web site will be developed once NRC ESSI Simulator Computer is installed at UCD (most likely by end of 2010). Such web site, with all the design and component details of the NRC ESSI Simulator Computer will help NRC ESSI Simulator Program and computer user, other than NRC staff and LBNL and UCD project members, to copy and further develop similar simulation platform for use with the NRC ESSI Simulator Program.

4.2 Verification, Validation, Documentation, Licensing

Much of what is being presented in this section is heavily influenced by an excellent paper by Oberkampf et al. (2002), a short course by Oberkampf (2003) and by a book by Roache (1998). A number of other publications were found very useful too (Mróz, 1988; Arulanandan and Scott, 1993; Babuška and Oden, 2004; Ladevéze et al., 2005; Schwer, 2007a,b; Lucas et al., 2008; Chamberland et al., 2010).

Verification and Validation. Developing a strong assurance of accurate numerical predictions of the seismic response of NPPSSS rely heavily on **Verification and Validation** (V&V) procedures. Verification and validation procedures are the primary means of assessing accuracy in modeling and computational simulations (Roache, 1998; Oberkampf et al., 2002; Oberkampf, 2003). Verification is the process of determining that a model implementation accurately represents the developer's conceptual description

and specification. Verification provides evidence that the model is solved correctly. It is essentially a mathematics issue. One of the main goals of verification is to identify and remove errors in computer coding (perform numerical algorithm verification, and assure software quality) and also to quantify the numerical errors in computed solution. Validation is the process of determining the degree to which a model is an accurate representation of the real world from the perspective of the intended uses of the model. Validation provides evidence that the correct model is solved. It is essentially a physics issue. Tactical goal of validation is to identify and minimize uncertainties and errors in the computational model. Strategic goal of validation is to increase confidence in the quantitative predictive capability of the computational model. In addition to that, validation strategy should seek to reduce as much as possible (a) computational model uncertainties and errors; (b) random (precision) errors and bias (systematic) errors in the experiments; and (c) incomplete physical characterization of the experiment. V&V procedures are the tools with which we build confidence and credibility in numerical predictions resulting from modeling and computational simulations. Figure 4.4 shows role of verification and validation in modeling and simulating real world NPPSSSs. It is very important to note that a tight

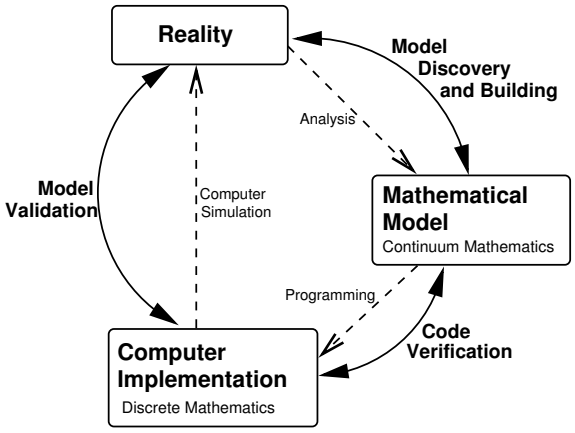

Figure 4.4: Schematic representation of role of verification and validation in relation to real model.

coupling exists between each of the phases of modeling and simulation process shown in Figure 4.4. Figure 4.5 shows a more detailed view of relationship of verification and validation to the developed computational solution. It is also important to note that verification procedures include software quality assurance practices, which are based on an extensive (fully and semi–) automatic test suite (eg. regression testing, white and gray box testing, static and dynamic testing, etc.).

The main motivation for performing verification stems from the simple question: "how much can you trust model implementations?" Validation, on the other hand can be motivated by the question:

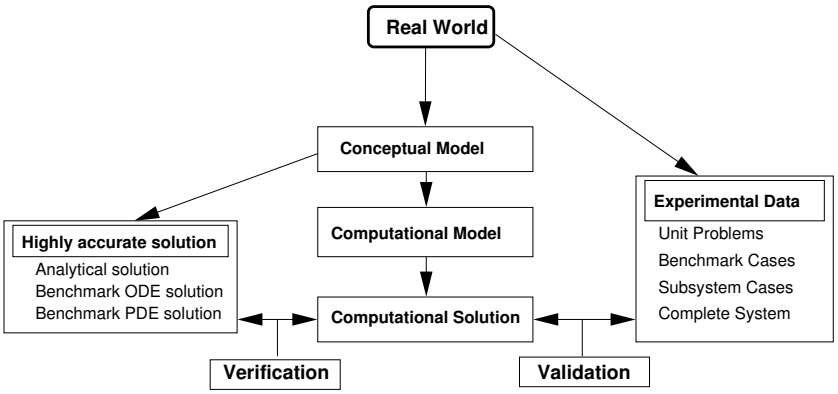

Figure 4.5: Schematic representation of relationship between verification and validation.

"how much can your trust numerical simulations?" Both questions can be summarized by the question: "how good are our numerical predictions in simulating real world NPPSSSs?" In relation to numerical prediction of seismic performance of NPPSSSs, the question can also be rephrased: "can numerical simulation tools (NRC ESSI Simulator for example) be used for assessing public safety?".

Prediction Numerical prediction comprises use of computational model to foretell the state of an object (NPPSSS in this case) under conditions for which the computational model has not been fully validated, typically due to its complexity. It is important to note that validation does not directly make a claim about the accuracy of a prediction since computational models can be easily misused (unintentionally or intentionally). Accuracy of prediction is closely related to how closely related the conditions of the prediction and the specific cases in the validation database are; and how well is physics of the problem understood. Validation and application domains rarely have any significant overlap. Figure 4.6 shows two such cases, left side showing partial overlap and the right side showing complete lack of overlap. For seismic behavior of NPPSSSs, partial overlap is potentially available for small seismic

Figure 4.6: Schematic representation of partial or no overlap of verification and application domains.

events (somewhat frequent) where the response of a complete NPPSSS can be recorded (depending on the extent of installed instrumentation). In this case, numerical predictions could be validated against such recordings. However, in the case of numerical simulations for predicting NPPSSS behavior during significant (rare, catastrophic) seismic events, overlap with validation cases is non-existent and application domain behavior has to be inferred through numerical simulations. Such inference needs to be based on rational mechanics of all important phenomena that affect NPPSSS behavior. It must also account for uncertain nature of material parameters, seismic loading and other components of NPPSSS. Ever present uncertainty needs to be properly estimated and modeled for numerical predictions. It is important to identify all sources of uncertainty, create mathematical representation of individual sources, and propagate those uncertainties through modeling and simulation process.

Licensing The NRC ESSI Simulator will be placed in the public domain using one of open-source licences (Weber, 2004). Current candidate licenses are GNU Public License (GPL) and Lesser GNU Public License. Actual license for NRC ESSI Simulator will be finally decided in collaboration with NRC Staff.

Chapter 5

Summary

Presented in this report, in some detail, was work performed for the U.S. NRC from October 2009 through October 2010 on a project titled: "Investigation of Analysis Methods to Incorporate Multi-Dimensional Loading and Incoherent Ground Motions in Soil-Structure Interaction Analysis". A number of previous developments from within the Computational Geomechanics Group at University of California at Davis (UC Davis) were used as well. This report (a *living document*) is continuously updated, changed and improved under new U.S. NRC sponsored project "Methods, Computational Platform, and Case Studies for Time-Domain Soil-Structure-Interaction Modeling and Simulation Incorporating Complex Seismic Loads" at UC Davis and at LBNL. Moreover, this report will form a basis for the NRC ESSI Simulator documentation, main focus of which is on NRC staff capacity building.

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