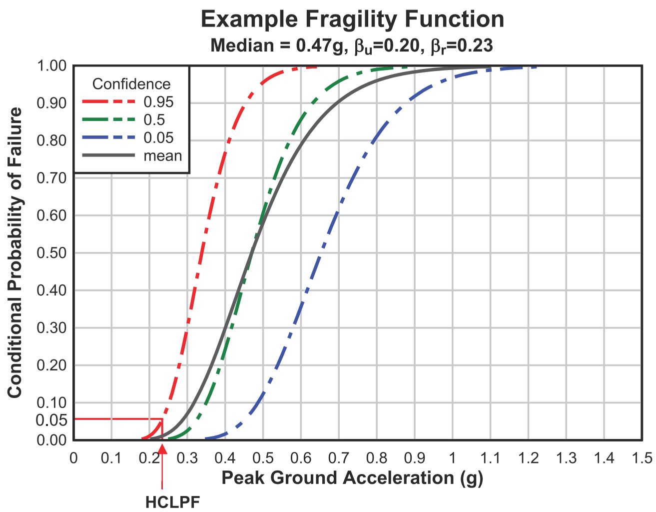

FIGURE A-2. Example Fragility Curves for a Component

along the horizontal axis is the same parameter as used in the hazard analysis, and the vertical axis is the fraction of failure (i.e., 0 to 1.0). The ground motion variable used to express fragility can be peak ground acceleration, spectral acceleration, or some other variable; however, to correctly convolve the component fragility with the seismic hazard, each curve needs to reference the same ground motion variable. Fragility curves may be developed using simulation models, existing analysis, or a combination of both. Reed and Kennedy (1994), Kennedy (1999), and Kennedy et al. (2009) provide additional information on development of seismic fragilities.

Fragility curves, as shown in Fig. A-2, are developed for each significant component in the SPRA. With the lognormal model, which is commonly used in SPRA, the result of the fragility analysis for each element is the median capacity and the logarithmic standard deviations for randomness and uncertainty. This is sufficient information to construct a family of fragility curves.

The procedure for calculating the three fragility parameters for an element involves analysis of response and capacity parameters that affect the overall capacity. Table A-1 lists the significant parameters that are typically included in fragility analysis for structures and equipment, including capacity and response variables. For equipment, both the building structure and the equipment response parameters are considered. In addition, the capacity of equipment is determined, which may be due to anchorage failure, a structural failure mode (i.e., brittle or ductile), or an equipment functionality mode (i.e., based on testing).

The bottom right box in Fig. A-1 represents the systems analysis. In this step engineers who are familiar with the plant operations and the functions that are required to shut down the plant if an accident occurs develop the SPRA event and fault trees. These logic trees relate the various components and systems required to mitigate an accident and/or shut down the plant. Fragility curves are developed for each element in the trees, and these curves are combined through probabilistic procedures to obtain core damage fragility curves.

An alternate SPRA was formulated by Smith et al. (1980, 1981) and advanced by Huang et al. (2008, 2011a, b), wherein the annual frequency of unacceptable performance is calculated in five basic steps:

- Seismic hazard analysis,
- Plant system and accident-sequence analysis,
- Component fragility curve development,
- Simulations of response of primary and secondary components and systems, and
- Consequence analysis.

This procedure differs from the more traditional approach in two ways: (1) fragility curves for primary and secondary components and systems are described in terms of response parameters such as drift (relative displacement) and floor acceleration, rather than ground motion parameters such as peak ground acceleration or spectral acceleration, and (2) the response of the primary and secondary systems and components is simulated directly by response-history analysis of soil-foundation-structure systems using sets of three-component ground motions that are consistent with the results of the seismic hazard analysis. The potential advantages of this procedure are (1) fragility curves for primary and secondary components and systems are better related to response parameters than ground motion parameters and (2) estimates of the frequency of unacceptable performance are more robust. The other three steps in the procedure are identical to those described previously. This approach of developing fragility curves using response parameters, such as drift or floor acceleration, may not be suitable for characterizing fragility for active components with several frequencies of vibration that can affect their functionality. In those cases, a ground motion parameter representative of the vibratory input to such equipment is needed. Also, data related to operator reaction during earthquakes are generally available in terms of peak ground acceleration levels.

The output from a seismic risk assessment varies depending on the stage at which the seismic event analysis is merged with other external/internal event analyses. If the seismic analysis is combined with other event analyses at the plant system and accident-sequence analysis stage, the required output consists of

Table A-1. Parameters Typically Considered in Fragility Analysis

- **Structures**
  - Capacity
    - Strength (yield or ultimate)
    - Inelastic energy absorption
  - Response
    - Ground response spectra
    - Soil-structure interaction (including vertical spatial variation and incoherence)
    - Damping
    - Frequency
    - Mode shape
    - Torsional coupling
    - Mode combination
    - Time-history simulation
    - Earthquake direction combination
- **Equipment**
  - Equipment capacity
    - Strength (yield or ultimate) or test capacity
    - Inelastic energy absorption
  - Building structural response
  - Equipment response
    - Damping
    - Frequency
    - Mode shape
    - Modal combination
    - Earthquake direction combination

seismic hazard curves, component fragilities, initiating events, modifications to event and fault trees, and containment failure analysis and quantification of fault trees. If the seismic risk is combined with the other internal/external event analyses at the consequence analysis stage, an initial output of the seismic risk assessment is a curve showing the probability density function of the annual frequency of seismically induced core melt. If the core-melt frequency is somewhat high, further computation of the release frequencies is warranted. In this case, the final output from the seismic risk assessment is a set of probability density functions of annual frequencies of the different release categories.

A.3 PURPOSE AND OVERVIEW OF SEISMIC MARGIN ASSESSMENT METHODOLOGY

The fundamental purpose of seismic margin assessments is to demonstrate sufficient margin over the design earthquake level to ensure plant safety and to find any "weak links" that might limit the plant's capability to safely shut down after a seismic event bigger than the design earthquake. The seismic margin assessment will also identify dominant contributors to risk.

Seismic margin assessments are treated as safety evaluations and not as design evaluations. Thus the criteria and approaches are designed to be as practical and economical as possible. The seismic margins methodology was initially designed to avoid the arguments associated with the seismic hazard that often proved highly contentious and unresolvable. Advancements in seismic probabilistic seismic hazards assessment made since about 1990 have largely quashed these earlier arguments, however, the seismic margin assessment approach is still very popular and is widely used to demonstrate margin over the design earthquake level to quantify plant safety. A fundamental difference between the seismic PRA and the seismic margin assessment is that the seismic margin assessment requires a review-level earthquake[1] (RLE) to be specified. The SMA approach relies heavily on the use of earthquake experience data, generic equipment qualification and fragility test data, past SPRA results, and extensive use of expert judgment and experience. Substantial use of plant walkdowns to search for weak links and to determine locations for more detailed evaluations to be performed is emphasized. The fundamental result of an SMA is the determination of the HCLPF capacity of the plant or that the plant HCLPF exceeds the review-level earthquake, by screening. Because the SMA approach uses screening rules, a chance exists that all components on the safe shutdown equipment list will be screened as having a HCLPF greater than the RLE. Only when HCLPF values are calculated explicitly can a quantitative statement be made about the plant-level HCLPF.

Two margin methodologies are available. The NRC-sponsored methodology (Budnitz et al., 1985; Prassinos et al., 1986; Amico, 1988) retains many of the aspects of an SPRA, whereas the EPRI methodology (Reed et al., 1991) is more deterministic and designed to be more fully implemented by a utility staff. Both methods have similarities but contain differences in the details. Tables 1, 3, and 4 of Kenneally and Chokshi (1991) and Kennedy et al. (1989) provide an overview of the seismic margin methodology and differences between the NRC and EPRI approaches.

[1]The terms seismic margin earthquake (SME) and review-level earthquake (RLE) are synonymous. USNRC (1991) and Kennedy et al. (1989) used the term "seismic margin earthquake" to denote an earthquake with ground motions larger than the SSE against which the plant was being reviewed. Later references use the term "review-level earthquake." These are the same earthquakes.

TABLE A-2. Summary of Conservative Deterministic Failure Margin Approach

Load combination	Normal loads + review-level earthquake (RLE)
Ground response spectrum	Defined by mean response-spectrum shape
Damping	Conservative estimate of median damping
Modeling	Best estimate (median) + uncertainty variation in frequency
Soil-structure interaction	Best estimate (median) + parameter variation
Material strength	Code-specified minimum strength or 95% exceedance actual strength if test data are available
Static capacity equations	Code ultimate strength (ACI-349, 2013), maximum strength (AISC-N690, 2010), Service Level D (ASME), or functional limits; if test data are available to demonstrate excessive conservatism of code equations, then use a value exceeded by 84% of test data for capacity equation for ductile elements
Inelastic energy absorption	For nonbrittle failure modes and linear analysis, use F_μ factors from ASCE 43-05 in capacity evaluation to account for benefits of ductility, or perform nonlinear analysis to 95% exceedance ductility levels[a]
In-structure (floor) spectra generation	Use frequency shifting rather than peak broadening to account for uncertainty plus use median damping

[a]Note that the inelastic energy absorption factor, F_μ, used in fragility analysis is different from the ductility factor, μ, used for component evaluations.

The major steps in performing an SMA are summarized in the following paragraphs. The major differences between the NRC and EPRI seismic margin methodologies as they relate to particular steps are also discussed.

1. Selection of the RLE: The review-level earthquake must exceed the design earthquake level and should be large enough to challenge the plant so that one can identify any weak links, but not at such a high level that screening of structures and components cannot be implemented in a cost-effective manner. Screening tables have been developed for 5% damped peak spectral acceleration values of 0.8 and 1.2g, which are associated with peak ground acceleration values of 0.3 and 0.5g, respectively. To define the capacities of the plant components in terms of the HCLPF level, the input motion to the structural model needs to be based on the mean uniform hazard response spectra, as shown in Table A-2.
2. Selection of the seismic margin assessment team: The SMA team consists of system engineers, seismic capability engineers, and plant operations personnel. The NRC approach requires systems analysts who are capable of developing fault trees or event trees, and the seismic capability engineers must be capable of performing fragility calculations if the fragility-analysis (FA) method is used to calculate HCLPF values.

TABLE A-3. Comparison of Seismic PRA and Seismic Margins Methodologies

Seismic PRA	NRC Seismic Margin Method (as modified by Kenneally and Chokshi, 1991)	EPRI Seismic Margin Method (per Kenneally and Chokshi, 1991)
Approach		
Probabilistic	Semiprobabilistic	Partially probabilistic
Scope of Review		
Event trees and fault trees are usually developed from the event/fault trees developed for the internal events analyses. Structures and elements where failure could impact and fail safety-related elements are added to the trees.	For pressurized water reactors, the safety functions of reactor criticality and early emergency core cooling are considered. For boiling water reactors, the safety functions of reactor subcriticality, emergency core cooling, and residual heat removal are considered. In addition, a small break loss-of-coolant accident (LOCA) is postulated to occur, and soil failure modes are considered. Potential for earthquake-induced flooding earthquake-induced fires is also considered, as are nonseismic failures and human actions.	Review includes electrical, mechanical, and nuclear steam supply system (NSSS) equipment; piping; tanks; heat exchangers; cable trays and conduit raceways; containment; and structures. In addition, leakage equivalent to a small break LOCA is postulated to occur in one success path, and soil failure modes are considered. Potential for earthquake-induced flooding and earthquake-induced fires is also considered, as are nonseismic failures and human actions.
Seismic Input		
Site-specific hazard curves, for instance, those being developed by EPRI for use on central and eastern United States plants, for peak ground acceleration and response spectra should be used.	A site-specific uniform hazard response spectrum anchored to either 0.3*g* or 0.5*g* PGA should be used. Development of new in-structure response spectra, including effects of SSI, is encouraged.	Same as the NRC seismic margin method.
Selection of Equipment		
Elements whose failure could lead to core damage (i.e., Level 1 PRA) are considered initially. Fault trees are “pruned” on the basis of systems and fragility considerations.	Elements whose failure could lead to core damage are considered initially. Fault trees are “pruned” on the basis of systems and fragility considerations.	Two separate and independent shutdown success paths are selected. One path postulates leakage equivalent to a small break LOCA.
Screening Requirements		
Screening is based on system and fragility considerations.	In general, equipment functionality is investigated on the basis of seismic experience or test data. Equipment anchorage is analyzed for each component. Caveats and guidance are provided in the criteria screening tables in NUREG/CR-4334 (USNRC, 1991) and EPRI NP-6041 (Reed et al. 1991) for three ranges of seismic input.	In general, equipment functionality is investigated on the basis of seismic experience or test data. Equipment anchorage is analyzed for each component. Caveats and guidance are provided in the criteria screening tables in EPRI NP-6041 (Reed et al. 1991) for three ranges of seismic input.
Required Experience and Training of Engineers		
The seismic PRA should be performed by experienced systems and seismic capability engineers who can perform seismic fragility analysis.	The seismic margin assessment should be performed by trained, experienced seismic capability and systems engineers. Seismic capability engineers must be capable of performing fragility analysis (FA) if this method is used.	The seismic margin assessment should be performed by trained, experienced seismic capability and system engineers.
Walkdown Procedures		
Principal elements of the walkdown are (1) seismic capacity versus seismic demand, (2) caveats based on earthquake experience and generic testing databases, (3) anchorage adequacy, and (4) seismic-spatial interaction with nearby equipment, systems, and structures. Walkdown procedures for a seismic PRA should follow the requirements contained in EPRI NP-6041 (Reed et al. 1991).	Principal elements of the walkdown are (1) seismic capacity versus seismic demand, (2) caveats based on earthquake experience and generic testing databases, (3) anchorage adequacy, and (4) seismic-spatial interaction with nearby equipment, systems, and structures. Elements not screened out are identified as outliers for further review. Potential for earthquake-induced flooding and earthquake-induced fires should be considered in the walkdown.	Same as NRC seismic margin method.
Evaluation of Component Capacity		
For elements not screened out during walkdown, calculate fragility parameter values, that is, median capacities and logarithmic standard deviations.	The capacity of components that were not screened out can be calculated using the FA or the conservative deterministic failure margin (CDFM) method.	The capacity of components that were not screened out can be calculated using the CDFM method.

3. Preparatory work prior to walkdowns: This step consists of gathering and reviewing information. In the EPRI approach, the system engineers define candidate shutdown paths and the associated frontline and support systems and components. In the NRC approach, the system analysts gather information and sort functions and then identify the appropriate functional groups. The preparatory seismic capability work should be started during this step to support the upcoming walkdowns.
4. Systems walkdown: For the EPRI approach this walkdown confirms the appropriateness of the selected primary and alternate success paths and prepares for the seismic capability walkdown. For the NRC approach the emphasis is the same except it is not limited to several success paths.
5. Seismic capability walkdown: This is the major walkdown in which components selected by the systems engineers are walked down by the seismic capability engineers. The components are screened out on the basis of the guidelines provided in the margin methodology, or data are gathered on the components that require further detailed analysis. The walkdown includes assessing the seismic ruggedness of the component, its anchorage, and seismic interaction effects.
6. Seismic margin assessment work: Components that cannot be screened out during the walkdown as having an HCLPF at or above the RLE level are evaluated as part of this step to determine the HCLPF value. Two approaches to calculate the HCLPF value are available. One is the conservative deterministic failure margin (CDFM) approach, and the other is the FA method. CDFM is a deterministic approach that uses a prescribed set of rules that can be applied without prior training in fragility-analysis methods. The fragility-analysis method describes the capacity of a component in a more probabilistic way in terms of fragility curves. Reed et al. (1991) provides more guidance on the CDFM approach. Kennedy (1999) provides guidance on recent innovations in margins and seismic PRA.
7. Documentation of results: Both the NRC and EPRI approaches provide similar guidelines for the content and format to include in a seismic margin report. The report should state the calculated plant-level HCLPF and clearly document how it was determined.

To fully comprehend the seismic margin methodology, one must understand how component capacities are estimated. The approaches are the CDFM and FA methods. ASME (2009) provides a detailed comparison of the two approaches. The EPRI SMA methodology recommends the use of the CDFM approach, but either approach can be used to estimate HCLPF seismic capacities for either the EPRI or NRC SMA methodology.

For each component the FA method defines a set of curves that expresses a probability of failure versus ground motion levels at different confidence levels. A set of typical fragility curves for a component is shown in Fig. A-2. These curves are necessary in SPRAs but lead to great difficulty in making decisions as to whether an adequate seismic margin exists. Converting the information provided by the seismic fragility curves into a single seismic margin description, i.e., the HCLPF capacity, has been found useful. This HCLPF capacity corresponds to about 95% confidence of less than about a 5% probability of failure. Fig. A-2 illustrates the location of the HCLPF on a typical set of fragility curves for a component.

The use of the FA methodology to obtain a single HCLPF capacity has several potential limitations. Several judgments and calculations have to be made, and few practitioners have experience in making seismic fragility estimates. Because of these potential drawbacks, the CDFM approach was developed to calculate an estimated HCLPF capacity using a set of deterministic guidelines (e.g., ground response spectra, damping, material strength, and ductility). Table A-2 provides a summary of the CDFM approach. This method is very similar to the design procedure followed in the industry, except that the parameter values have been liberalized. The approach summarized in Table A-2 is the same approach that is prescribed by following ASCE 43-05 and this standard, ASCE 4-16. By design, following ASCE 4-16 for response analysis, coupled with ASCE 43-05 for design, will produce an HCLPF at the design basis earthquake ground motion. The EPRI methodology, Reed et al. (1991), provides specific guidelines for the CDFM approach. Updates to this may be found in Reed and Kennedy (1994).

One should be cautious when using the CDFM approach and the hybrid method for developing fragility parameters. A key difference between the separation of variables technique and the CDFM/hybrid approach is that the separation of variables approach directly computes the median factor of safety above the RLE, while the CDFM/hybrid approach directly computes the HCLPF and then estimates the median factor of safety by assuming a range of logarithmic uncertainty values. Because the CDFM/hybrid approach calculates an HCLPF, larger values of

TABLE A-4. Advantages and Disadvantages of Seismic Margin Methodology

Advantages	Disadvantages
Most important elements of seismic PRAs are retained: plant walkdowns and an ability to identify potential plant vulnerabilities through an integrated review of plant response.	No direct risk insights are obtained.
The scope of components and systems that need to be reviewed is reduced.	Accident mitigation, accident management, and emergency planning can be addressed only to a limited extent.
A measure of plant capacity is provided that is more easily understood and appreciated by engineers. It does not require fragility calculation.	Nonseismic failures are addressed in an approximate manner.
Plant capacity estimates will be useful to judge the impact of design basis earthquake issues.	Ranking is based only on HCLPF capacities, thereby making it difficult to prioritize issues in the absence of a better risk-based ranking.
Results are not affected by seismic hazard issues.	The system-screening guideline as applied to a very old plant may require plant-specific modifications.
The level of effort required to implement is lower than that for a seismic PRA when both are done at the same level of detail.	It is more difficult for plants where the hazard is perceived so high that the review-level earthquake would be above the 0.3g and 0.5g (0.8g and 1.2g spectral acceleration) screening values.
Correlations among failures can be identified and analyzed with the NRC event/fault tree method.	

TABLE A-5. Advantages and Disadvantages of Seismic PRA Methodology

Advantages	Disadvantages
It can expand upon the event/fault trees developed for the internal events PRA analysis.	The level of effort required is higher than that for the seismic margin methodology because of the enhanced scope when done at the same level of detail.
It provides a complete risk profile and can provide all the results obtained from the seismic margin methodology. Uncertainties are explicitly accounted for.	Numerical results are often controversial because of large uncertainties and use of subjective judgment.
Decision making can be based on plant-specific risk results.	It can be used to focus on bottom line numbers, thereby introducing the tendency to make inappropriate comparisons with other initiators.
It provides a more rigorous consideration of nonseismic failures and human actions.	
Accident mitigation, accident management, and emergency planning can be addressed more systematically and with greater detail.	
Ranking based on different indices is available, for instance, core melt, frequency, and release.	
Correlations among failures can be identified and analyzed.	

uncertainty will produce larger median values and thus reduce overall probability of failures. A reasonable range of logarithmic (β_c) uncertainty values is 0.3 to 0.6 (Huang et al. 2008) for most nuclear power plant components. Therefore underestimating the beta values when using the CDFM/hybrid approach is conservative and recommended as this will yield conservative estimates of the annual probability of failure.

A.4 COMPARISON OF SEISMIC EVALUATION METHODOLOGIES

Various advantages and disadvantages are associated with the application of the seismic margin methodology (instead of a seismic PRA) in beyond design basis evaluations. Table A-3 compares the approach, scope of review, seismic input, selection of equipment, screening requirements, required experience and training of engineers, walkdown procedures, and evaluation of outliers between a seismic PRA and the NRC and EPRI seismic margin assessment methodologies. Comparisons between the NRC and EPRI seismic margin assessment methodologies vary depending on the detailed techniques employed, particularly for the NRC method. Depending on the gradations employed, the NRC method can provide results varying from a mini-level 1[2] seismic PRA to results similar to the EPRI method. The comparison assumes that the methodology enhancements described in NUREG-1407 (USNRC 1991) are included.

The user of the seismic margin methodology should examine the facility to ensure that the system assumptions and screening guidance are applicable. This is particularly vital for older facilities where, for example, the critical functions, systems, and success path logic may differ from the plants considered in the development of the seismic margin methodologies.

The advantages and disadvantages associated with the seismic margin methodology are provided in Table A-4; whereas Table A-5 provides the same information for the seismic PRA methodology.

[2]PRAs estimate three basic levels of risk: level 1, level 2, and level 3. A level 1 PRA estimates the frequency of accidents that cause damage to the nuclear reactor core. This is called core damage frequency (CDF). A level 2 PRA estimates the frequency of accidents that release radioactivity from the plant. A level 3 PRA estimates the consequences of the radioactive release in terms of injury to the public and damage to the environment.

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APPENDIX B
NONLINEAR TIME-DOMAIN SOIL-STRUCTURE INTERACTION (NONMANDATORY)

B.1 INTRODUCTION

This nonmandatory appendix provides guidance for performing nonlinear three-dimensional time-domain soil-structure interaction analysis. Nonlinear time-domain analysis involves nonlinearities in the materials and/or geometry, such as loss of contact between soil and structure and inelastic action in soil and structure. This may be useful when performing analyses for beyond design basis events (see Chapter 1), performing fragility analysis, and analyzing seismic isolation solutions. It is not anticipated to be used as the primary analysis method for new design at this time but may be used for evaluation of existing plants. This method may be used when any of the following behaviors are important to the analysis results:

- Material nonlinearity (in soil and/or structure),
- Significant uplift or sliding of the foundation,
- Static and dynamic soil pressure effects on deeply embedded structures,
- Local soil failure at the foundation-soil interface,
- Nonlinear coupling of soil and pore fluid,
- Nonlinear effects involving gaping between the structure and surrounding soil at the soil-structure interfaces, and
- Base isolation (as discussed in Chapter 12).

The analyst and reviewer must determine which of these nonlinear effects are important and model and simulate some or all as outlined in this appendix. For example, if the goal of the nonlinear analysis is capturing gaping and sliding between soil and structure, nonlinear elements (contact) should be added to capture these effects, and equivalent linear elements could be used to model the remainder. In this instance the equivalent linear elements modeled in time domain would be matched to the strain-compatible soil properties, as outlined in Chapters 2 and 5, for the frequencies of interest. The method should be verified by matching the time-domain model free field to frequency-domain free field. Rinker et al. (2006) outlines an approach when performing a time-domain analysis but matching strain-compatible soil properties.

In the context of this standard, nonlinear soil-structure interaction (SSI) can be used to provide element forces and deformations for superstructure component checking and in-structure response spectra or foundation input motions, which are the first step in a multistep analysis. This appendix does not alter prior guidance in this standard on the use of three soil columns (BE, LB, and UB) for SSI analysis or peak smoothing and broadening of in-structure response spectra.

Guidance is provided in the following subsections on

- Development of finite element meshes for analysis,
- Earthquake ground motion input,
- Nonlinear constitutive models for soils and structures,
- Analysis results and interpretation, and
- Verification and validation.

In performing a nonlinear SSI analysis, the analyst should

- Demonstrate that the soil domain modeled is sufficiently large that the predicted responses do not change significantly if the domain size is further increased;
- Account for local nonlinearities between the soil and the structure using contact algorithms or gap/frictional elements that can model possible gap opening and closing and frictional behavior (when gap is closed);
- Consider the effects of uncertainties in material parameters, properties of components, and ground motion characteristics; sources of uncertainty should be identified and their effects quantified; and
- Account for buoyancy effects for embedded structures.

Energy dissipation (damping) is captured in nonlinear SSI analysis through the development of a model that includes material nonlinear behavior (hysteretic energy dissipation), material viscous coupling behavior (pore fluid-soil and structure-fluid), Coulomb friction, and radiation damping.

When performing nonlinear analysis, unintended (numerical) damping (positive and/or negative) can arise within the numerical solution and its effect should be understood. The integration method chosen to advance the solution (e.g., Newmark and/or Hilber-Hughes-Taylor integration method; Argyris and Mlejnek 1991) may introduce nonphysical energy dissipation into the model. In addition, "stiffness proportional" viscous damping must be specified carefully, because it intrinsically increases in proportion to frequency; higher frequencies can therefore often be heavily over-damped. Over-damping can also occur if materials soften beyond their initial elastic stiffness; therefore viscous terms should be based upon instantaneous tangent stiffness, not initial stiffness.

B.2 DEVELOPMENT OF FINITE ELEMENT MESHES FOR ANALYSIS

The extent of the finite element model and the size of individual elements must be selected carefully.

The extent of the finite element model depends on the chosen method of analysis; Section B.3 provides details.

The size of the finite elements should be sufficiently small to permit adequate transmission of seismic motions up to the cutoff frequency.

In general, the mesh density depends upon the soil characteristics, the element formulation, the solution technique (implicit or explicit), and the cutoff frequency for which accurate representation is required. The analyst should demonstrate that the mesh adequately transmits the seismic motions up to the cutoff frequency. One method for doing this is using small test models with mesh densities of increasing fineness in the software used.

Some meshing considerations are

- The mesh size should be sufficiently small to capture the nonlinear behavior of the affected region.
- The mesh size should be small enough to capture the appropriate frequencies. For linear displacement interpolation elements, the longest side of each element, (Δh), is defined by Eq. (B-1). The use of larger elements can lead to excessive artificial/numerical damping (Jeremic et al. 2009, 2012).

$$\Delta h \leq \frac{v_s}{10 * f_{\max}} \tag{B-1}$$

where $f_{\max}$ = maximum frequency of interest; and v_s = smallest shear wave velocity of interest in a given area of the simulation. (The maximum mesh size should be considered for each layer because it depends on the shear wave velocity in the soil layers.)

- The time step Δt used for solving the equations of motion depends on the solution technique. Explicit solvers will automatically select a time step required for numerical stability. For implicit solvers, the time step should be limited to the smaller of (1) 10% of the smallest natural period of the system being considered or (2) the ratio of the shortest side of any element in a layer to its corresponding shear wave velocity (Jeremic et al. 2009).

$$\Delta t \leq \frac{\Delta h}{v_s} \tag{B-2}$$

where Δh = maximum grid spacing, and v_s = highest shear wave velocity.

B.3 GROUND MOTION INPUT

Seismic motions should be input into the SSI model at the boundaries of the soil domain. Three-component sets of earthquake ground motions should be applied. Section 4.7.3 should be followed for development of the ground motion. Depending on the specific issues being investigated, representing body and surface waves, including inclined waves, and the effects of lack of correlation (termed incoherence in frequency domain) may be necessary.

The type and position of the boundaries must be selected such that radiation damping (radiation of seismic waves resulting from wave reflections and oscillations/vibrations of the structure(s), systems, and components) is adequately accounted for.

Several methods are available, including

- Domain reduction method (DRM; Bielak et al. 2003) analytically replaces motions from the hypocenter with a set of time-varying forces applied on a single layer of linear finite elements encompassing the domain of interest (Fig. B-1). Such domain of interest includes soil/rock [adjacent to the nuclear power plant (NPP)], the contact zone (between foundation and soil/rock), and the structure. While the domain of interest can have arbitrary inelastic (elastic-plastic, damage, etc.) deformations (Jeremic et al. 2009, 2012), a degree of approximation still exists in the use of free-field motions for load application to the model, at the single layer of elements that are "far enough" to be counted as a free field. Jeremic et al. (2012), Chapter 14, provides information on modeling seismic motion using DRM.

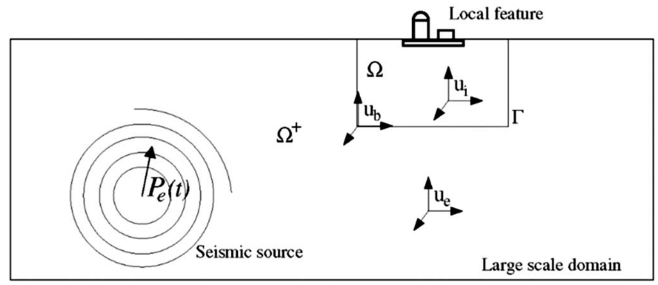

FIGURE B-1. Geometry of the Structure Foundation Structure System

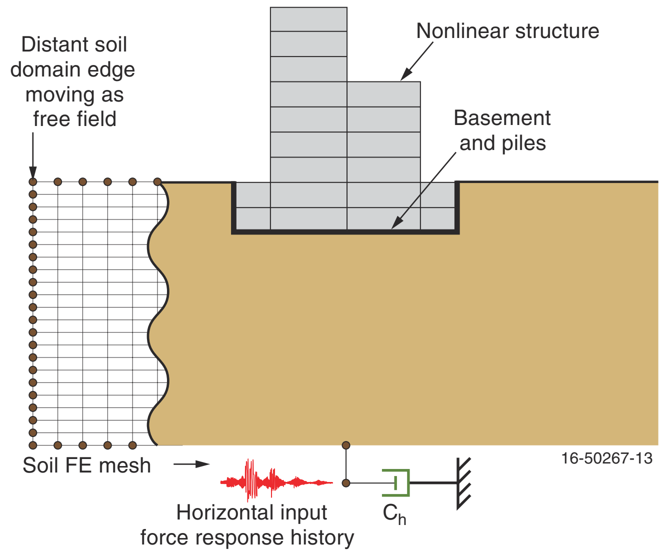

FIGURE B-2. Direct Approach

- The perfectly matched layer approach (Basu 2008), or an approach that uses infinite elements (as described in *ABAQUS*), has certain qualifications related to the linear far field. These approaches provide methods for bounded domain modeling of wave propagation on unbounded domains.
- Modeling a very large nonlinear domain with imperfect boundaries constrained to move as the (nonlinear) free (far) field. The rock outcrop ground motions are applied to viscous dampers that represent the rock in the model. The motions could be applied as force histories. This method may be necessary when significant nonlinearity in the far field is expected (Fig. B-2). This is the approach described in Willford et al. (2010).

B.4 NONLINEAR CONSTITUTIVE MODELS

Nonlinear constitutive models for soil, concrete, and other structural materials should capture appropriate nonlinear hysteretic behavior with increasing strains and during cyclic motions. The nonlinear constitutive laws and numerical procedures used to integrate constitutive equations should be verified and validated. For instance low-aspect concrete shear walls have a pinching behavior that flexural elements will not capture. Section 4.7.2 provides guidance for developing nonlinear structural constitutive and component models.

Nonlinear constitutive models provide one source of energy dissipation (damping) in time-domain SSI analysis. This nonlinear behavior (elastoplasticity, frictional dissipation, displacement proportional) results in cyclic, hysteretic energy dissipation within the material itself (solids and structures) and in contact regions (for example, contact of foundation concrete with base soil/rock; Argyris and Mlejnek 1991).

Viscous behavior can also be captured in nonlinear constitutive models by incorporating pore fluid (water usually), interaction of solids and structures with surrounding fluids (water, air, etc.), or both. This may be an important energy dissipation source to capture in the model.

Commercially available software packages such as LS-DYNA, ABAQUS, and ANSYS (Livermore Software Technology Corporation 2012; *ABAQUS*; *ANSYS*), and licensed software such as NRC ESSI Simulator (Jeremic et al. 2012) and open-source software such as OPENSEES and MASTODON provide constitutive models that can predict the nonlinear behavior of the soil.

The analyst must demonstrate that the nonlinear constitutive soil models are capturing the appropriate three-dimensional soil behavior by using verified and validated constitutive models or matching experimental results.

B.5 ANALYSIS RESULTS AND INTERPRETATION

Results from the analysis may include element forces and deformations for superstructure component checking and in-structure response spectra or development of foundation input motion. These results should be developed using the deterministic approach outlined in Chapter 2; a minimum of five sets of acceleration time series and three sets of site-specific soil profiles with the appropriate coefficient of variation (COV). The analyst should take the results as the mean for each soil profile run of five sets of acceleration time series and then envelop these. The analyses that exhibit highly nonlinear behavior will likely need more than five sets of acceleration time series. The analyst should demonstrate that an adequate number of acceleration time series have been used.

A probabilistic approach as outlined in Section 5.5 is also an acceptable method for developing results. An alternate approach involves the use of stochastic elastic-plastic finite elements (Sett et al. 2011).

B.6 VERIFICATION AND VALIDATION

Developing confidence in accurate numerical predictions of the seismic response of nuclear facilities relies heavily on verification and validation procedures. Verification and validation procedures are the primary means of assessing accuracy in modeling and computational simulations (Oberkampf et al. 2002; Roache 1998; Babuska and Oden 2004; Oden et al. 2010a, b). 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. 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.

Three nonlinear behaviors that need to be validated separately are (1) soil nonlinearity, (2) structural nonlinearity, and (3) contact interface nonlinearities (sliding and/or separation). Validation could be achieved by comparing results of the analytical model with experimental data or verification using closed form solutions (if available). Possible references for providing validation of soil, concrete, and contact nonlinearities and some experimental results are provided in International Federation for Structural Concrete (FIB 2008) and Atik Al and Sitar (2007).

Section 5.1.11 provides target validation goals that should be implemented when performing analyses in accordance with this appendix. Additional considerations for model validation are

- Sensitivity analyses should be performed on key nonlinear behaviors that significantly affect the time-domain SSI responses.
- The time-domain SSI analysis should first be validated with a representative model using low-amplitude seismic events that are expected to produce linear behavior (in soil and structure). These results should be compared with the results for similar models using procedures outlined in Chapters 2 and 5 of this standard.

The burden of proof is on the analyst to perform the necessary verification and validation for the analysis.

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