NUREG/CR-4331
UCID-20468

Simplified Seismic Probabilistic Risk Assessment: Procedures and Limitations

L. C. Shieh, J. J. Johnson, J. E. Wells,
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Prepared for
U.S. Nuclear Regulatory Commission

Lawrence Livermore National Laboratory

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Manuscript Completed: July 1985
Date Published: August 1985

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Washington, D.C. 20555
NRC FIN No. A-0126

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EXECUTIVE SUMMARY

SCOPE

At the request of the U.S. Nuclear Regulatory Commission, the Lawrence Livermore National Laboratory has developed a simplified seismic probabilistic risk assessment (PRA) methodology. The purpose of this methodology is to reduce the costs while adequately performing seismic probabilistic risk assessments of nuclear power plants. This report summarizes the development of the simplified seismic methodology and explains guidelines for applying the procedures. The development effort is part of the scope of work of the Seismic Safety Margins Research Program (SSMRP).

Our development efforts were divided into four tasks:

- Complete the development of simplified methodology for estimating seismic response (including response correlation and random and modeling uncertainties) directly from free-field ground acceleration, including to the extent necessary:
  - Methods to account for the effects of soil-structure interaction
  - Calibration of major structure response.
  - Calibration of piping system response.
- Complete the development of guidelines for event/fault trees to be used in simplified seismic PRAs.
- Analyze the Zion Nuclear Power Plant, Zion, Illinois, using the simplified methodology and compare the results with those from the SSMRP detailed seismic PRA on Zion.
- Issue a report giving procedures and guidelines for applying the simplified methodology. Limitations of these procedures should be stated.

This report completes the fourth task. A number of studies were performed in developing the simplified methodology. These studies, which emphasize the development of response factors, are summarized in the following reports:

J. J. Johnson, E. C. Schewe and O. R. Maslenikov, "SSI Response of a Typical Shear Wall Structure," Lawrence Livermore National Laboratory, Livermore, CA, UCID-20122, Vols. 1 and 2 (1984).

L. C. Shieh, N. C. Tsai and M. S. Yang, "Seismic Margin and Calibration of Piping Systems," Lawrence Livermore National Laboratory, Livermore, CA, UCID-20314, 1985.

C. Y. Kimura, "Categorization of PWR Accident Sequences and Guidelines for Fault Trees: Seismic Initiators," Lawrence Livermore National Laboratory, Livermore, CA, UCID-20211, 1984.

These reports provide supporting details for results and recommendations presented here.

BACKGROUND

The SSMRP started in 1978 with the following objective:

"To develop mathematical models that realistically predict the probability of radioactive releases from seismically induced events in nuclear power plants. These models will be used for four purposes:

- To perform sensitivity studies to determine weak links in seismic methodology.
- To estimate the probability of radioactive release for a plant.
- To estimate the conservatism in the Standard Review Plan (SRP) seismic design methodology.
- To develop an improved seismic design methodology."

The SSMRP detailed seismic PRA methodology is based on a state-of-the-art seismic and systems analysis process, and it explicitly includes the inherent uncertainties. The tools, codes, and data bases developed in the SSMRP constitute the most complete and comprehensive seismic risk methodology yet developed. It remains to be seen whether this methodology is adequately complete or comprehensive. The NRC has initiated research to validate seismic PRA methodology to address this issue.

In FY 1983 a need arose for a seismic PRA methodology that could be more widely and generally applied than the SSMRP detailed methodology. This need arose principally as a result of the utility-sponsored PRA studies on the Zion and Indian Point plants. These studies found that the seismic initiator was an important and possibly dominant contributor to the total public risk from these plants. These results led the NRC to expand its planning to include seismic and other initiators in PRA studies for programs such as the proposed Integrated Safety Assessment Program (ISAP).

It is now well-recognized that PRAs should include all initiators, the traditional internal initiators as well as external initiators such as earthquake, flood, tornado, wind, and fire. A PRA which includes internal and external initiators is called a combined PRA.

ASSUMPTIONS

The basic objectives of the SSMRP simplified seismic PRA methodology are to save time and money while adequately estimating seismic risk. Several assumptions serve as a point of departure for our development efforts:

- We assume that the seismic PRA is carried out as part of a combined PRA. This is a cost-important assumption. The incremental effort required in adding seismically initiated failures to plant logic models is relatively small if this is done at the same time that these models are developed for internal and other external initiators.

Simultaneous development of plant logic models for all initiators is also the best way to achieve consistency in the calculated risk estimates from the various initiators.

- We assume that the seismic hazard models (site-specific hazard functions and response spectra) for any eastern United States site will be available from the Seismic Hazard Characterization of the Eastern United States Project (SHCP) (Ref. 5) and comparable industry studies. We assume western sites will be treated on a case-by-case basis as part of the scope of the specific PRA.

- We assume that seismic design data is available for all structures, systems, components, and equipment.

SEISMIC PROBABILITY RISK ASSESSMENT DESCRIPTION

In the most simple general perspective of a seismic PRA, three different kinds of data are sought:

- Seismic hazard models

- Fragility/response models

- Plant logic models

The technical flow of information is that fragility/response is the interface between hazard and logic models. A calculational procedure (SEISIM in the SSMRP) assembles these data to produce estimates of the annual probability of core melt, risk to the public, and so forth.

The seismic hazard model is composed of two different kinds of information:

- seismic hazard functions

- response spectra

The seismic hazard function is a complementary cumulative probability distribution function. This function describes a relationship between the site peak acceleration at the surface in the free field and the annual probability of exceeding this acceleration. The hazard function thus describes the randomness in the occurrence of earthquakes. Accelerations in excess of the safe-shutdown earthquake (SSE) value are typically included in the analysis. The response spectra used in a seismic PRA are similar to those used in seismic design except that they are best-estimate rather than conservatively biased.

The fragility/response model describes the strength at "failure" (failure is defined as limit states such as loss of safety function, yielding significant or excessive distortion, rupture, or collapse) of individual items like structural elements, equipment, and components in the plant. Plant logic models determine the items for which a fragility function is required rather than whether the items are so-called safety-related or important to safety. The strength at failure is characterized in terms of a probabilistic model. The random variable for this model is the same as the random variable for the

hazard function. This is so the hazard and fragility functions can be integrated to obtain the annual probability of failure of the individual items. The fragility model may be developed either by using two distributions (one for response and one for strength) in the calculational procedure as in the SSMRP methodology or by a single distribution relating strength directly to the hazard function parameter as in commercial PRAs. In either case, it may be necessary to consider correlation between fragilities when the plant logic models are quantified. Note that the correlation values are not the same for the two approaches.

Plant logic models are logical descriptions of how systems could fail following individual component failures (fault trees). Logic models describe sequences of success or failure of systems (event trees) given some initiating event. The individual component failure probabilities from the fragility response functions are data to be used in the plant logic model quantification. The SEISIM code is used in the SSMRP to perform this quantification.

KEY AREAS OF SIMPLIFICATION

In the seismic hazard models, we generally eliminated the need for the development of time histories and rely instead primarily on response spectra. In some cases it may be prudent to develop time histories for limited site- and plant-specific calibration purposes.

Two distributions, one for response and one for strength, are developed in the calculational procedure for the fragility/response model in the methodology. The generic data of the equipment and component fragility functions may be applied to other plants. The site-specific fragility functions such as structural elements may be required depending on the plant logic models.

The major change in methodology is in the response models. We generally eliminated the need for detailed time history seismic response analysis and rely instead primarily on response factors to provide this information. These response factors are based on generic studies such as those performed as part of our development efforts or on a limited reanalysis of the nuclear power plant for which the seismic PRA is being performed. The studies that led to the generic response factors that we recommend formed the bulk of our efforts. These studies are discussed in more detail in Appendix A.

We have developed selected simplifications in plant logic models. However, we believe combined PRA needs will dictate requirements here. For example, we believe that the opinion that may ultimately prevail is that the plant logic models for the various initiators should be the same.

SEISMIC RESPONSE CALIBRATION

As indicated above, the primary focus of our simplification efforts is in the seismic response of structures, piping systems, components, and equipment. The SSMRP detailed seismic PRA methodology involves detailed response calculations using the SMACS code as a means to relate free-field acceleration to fragility based on local response quantities. It is thus logical to focus on response as an area to simplify.

The most important aspect of the response simplification effort is to "calibrate" seismic design data. By calibration we mean: To develop a response factor F_R that provides a relationship:

$$R_{BE} = R_D/F_R \qquad (1)$$

between seismic responses used in the plant design, R_D, and a best-estimate response, R_{BE}. This response factor is a key element in the development of response functions. R_D keys the best-estimate responses to free-field acceleration at a given earthquake level, usually the SSE. SSMRP results can then be used to relate responses at the SSE to higher levels. This is necessary since in a seismic PRA, the total annual probability of failure, core damage, and so forth, is obtained through summation or integration over all important acceleration levels as an application of the total probability theorem.

F_R is in general larger than 1.0 and in many cases it is much larger than 1.0. This is a reflection of conservatisms or margin in the calculational methods of analysis used in nuclear design practice as specified by NRC criteria. We call this conservatism or margin "calculational margin."

Methods of analysis used in nuclear design practice are generally deterministic, with some exceptions (for example, the square-root-of-the-sums-of-the-squares procedure). By deterministic we mean that a single value is sought for the seismic response R_D in Equation (1). It is also not generally recognized, at least to the extent that it should be, that there is considerable uncertainty possible in the design responses R_D. For example, if R_D is the in-structure response spectral value at some frequency, R_D can vary by as much as a factor of two to three depending only on the particular design time history that is used in the design process.

In nuclear seismic design practice the conclusion is thus that:

- o Deterministic design responses are sought.

- o Relatively large variations in the design responses can be observed from one calculation to another, even though each calculation satisfies NRC criteria.

Thus relatively large variation or uncertainty is unavoidable; it is a reflection of inherent features of the seismic design problem. This large uncertainty is also not necessarily a deficiency. It is one of the reasons the NRC introduced calculational margin into their criteria. The case for whether this uncertainty leads to a deficiency rests on whether the amounts of calculational and other margins are adequately large. As noted above, we have found cases in which the calculational margin is quite large. However, we make no claim that our findings support the conclusion that all margins are adequately large. The NRC has initiated research on the general issue of the adequacy of seismic design margins, of which calculational margins are an important element.

This discussion points out an important assumption in the SSMRP simplified seismic PRA methodology: We assume that the seismic response available from the plant seismic design data is a median value, that is, the median of the possible design values--all of which satisfy NRC criteria. This assumption will be in error in most cases. Assuming no bias, this error will be optimistic in half of the cases and pessimistic in half of the cases. This error or uncertainty is accounted for in our simplified PRA methodology.

This uncertainty points out a major limitation inherent to ours or any other simplified seismic PRA methodology (and indeed a limitation of any PRA methodology, whether seismic- or otherwise-initiated). This limitation is that the introduction of simplifications invariably leads to the need to specify the resulting random or modeling uncertainties that are a consequence of the simplifications. These uncertainties are particularly transparent in many instances in our simplified seismic PRA development efforts because we developed our simplified models from the more detailed ones available from the SSMRP. Thus we can explicitly trace how, why, and how much uncertainty is introduced. This provides analogous results to use when the available information is not detailed enough to allow explicit tracing.

This additional (relative to the SSMRP detailed seismic PRA methodology) random and modeling uncertainty has the following limitations relative to the results that would be obtained if the SSMRP detailed methodology were used:

- It increases the uncertainty in the estimates of the "figure of merit" (for example, the annual probability of core damage).
- It increases the mean figure of merit.
- It increases the median figure of merit--but probably not as much as the mean is increased.

A number of sensitivity studies were performed to obtain the response factors F_R in equation (1). These studies assessed the potential influence of many factors that might vary in the design of the various plants to which the SSMRP simplified seismic PRA methodology might be applied. The factors studied included parameters such as damping as well as alternative methods of analysis used in nuclear practice. For example, one study showed the relative unimportance of structural damping--particularly at soil sites. Another study revealed that there was surprisingly little difference in results between ten different methods of analysis of piping systems.

PLANT LOGIC MODELS

We developed guidelines for the development of event trees and faults trees (plant logic models). These guidelines focus primarily on key areas that need to be added to plant logic models when they are developed for internal initiators. The simplifications are most appropriate to Westinghouse PWRs of the Zion vintage and are less applicable to other plants. The applicability has to be determined on a plant-specific basis. The best course would be to develop logic models of the seismic initiator while developing them for other initiators.

RISK CALCULATIONS

We compared base case point estimates of core damage probability obtained using both the SSMRP detailed and simplified seismic PRA methodology. The results are as follows:

Release Category	Release probability/year Detailed	Release probability/year Simplified	Man-Rem/year Detailed	Man-Rem/year Simplified
1	2.9E-8	1.8E-7	0.2	1.0
2	1.4E-6	4.2E-6	6.5	20.0
3	5.4E-7	8.6E-8	2.9	0.5
4	0	0	0	0
5	8.3E-10	0	0	0
6	1.7E-7	9.0E-8	0	0
7	1.5E-6	8.0E-6	0	0.2
Totals	3.6E-6	1.3E-5	9.6	21.7

The base case used is our best estimate of the configuration of the Zion plant and its emergency procedures. A number of important assumptions are described below.

- It is assumed that "feed-and-bleed" emergency core cooling can be performed after an earthquake. In this procedure, which is employed if the auxiliary feedwater system has failed, the operator makes use of the emergency safety pumps to pump cooling water to the core. The resulting steam is bled off through the pressurizer relief valves.

- The identified structural failure modes are assumed to have the most serious consequences. Two structural failure modes play crucial roles.

  1. The failure of the roof of the service-water pump enclosure in the crib house is assumed to cause all six service-water pumps to fail. This results in loss of the emergency AC power diesel generators, due to lack of cooling water.

  2. The failure of the wall between the turbine building and the auxiliary building is assumed to cause loss of all electrical power and control circuits between the auxiliary building and the containment.

Both of these failures are assumed to fail the safety injection system (SIS), charging pump system (CHG), containment spray injection system (CSIS), and containment spray recirculation system (CSRS).

- Soil failure under the toe of the containment is assumed to result in rocking motions large enough to fail the SIS, CHG, CSIS, CSRS, and residual heat removal (RHR) piping between the AFT building and the reactor building.

- Failure of the vertical column supports under the steam generators and reactor coolant pumps is assumed to result in a double-ended guillotine break of the primary coolant piping equivalent to a large loss-of-coolant accident (LOCA). Two support failures in different loops are equivalent to a reactor vessel rupture.

As shown in this table, the core melt probability for the simplified methodology is about four times the base case results from the detailed methodology and the dose is twice as much. The major contributors remain to be basemat uplift, crib house, and inter-building pipes.

METHODOLOGY AND PROCEDURE

Finally, we present some summary statements on the SSMRP simplified seismic PRA methodology and procedure.

1. Plant and Site Familiarization

   The data and drawings for the reactor systems should be available from the plant owner or internal event PRA team. These data are particularly important for initiating events. The other data which are unique to seismic (for example, structural drawings) should be available through the plant owner. It is always necessary to review and inspect each plant for unique features such as weak ceilings and low clearance to ensure that such features are not overlooked. Site conditions must be closely examined. Soil properties such as soil depth, damping, shear wave velocity, and impedance contrast may strongly affect the calibration of seismic response design data for structures, piping systems, components, and equipment.

2. Seismic Hazard

   Uniform seismic hazard functions and frequency characteristics will be available for sites east of the Rocky Mountains from the Seismic Hazard Characterization Project and comparable industry studies. Specific studies will be required for all west coast sites as part of the simplified seismic PRA effort.

3. Plant Logic Models

   The same set of functional/systematic event trees will be used for internally and seismically initiated PRA. Modification of internal event fault trees is required to include passive failures and structural- or otherwise-induced secondary failures. A key effort is to identify the location of items that are basic events in the fault trees and to identify

the items that are susceptible to structurally induced failure.

4. Seismic Response

Correlation - Assign correlation values in response based on SSMRP detaile[illegible]ults on Zion.

Variab[illegible] variability in response based on SSMRP detailed resul[illegible]a from the Chiba field station. The response variability is a combination of these values and values due to the simplified procedure.

Median Response - Use design responses scaled to earthquake levels at and above the SSE. Use response factors to account for conservatism in design values. The responses corresponding to earthquake excitations above the design earthquake level are extrapolated.

5. Fragility

Site- and plant-specific fragilities should be developed for structures, equipment, and major components.

6. Plant Seismic Safety Evaluation

The SEISIM code should be used to compute the probability of core melt and releases. Uncertainty intervals should be calculated by repeated SEISIM calculations.

7. Analysis Results and Interpretation

Use the same level of detail as in the SSMRP detailed seismic PRA methodology.

CONCLUSION

The SSMRP task to simplify the detailed methodology for assessing seismic risk met its objectives by developing procedures that would assess seismic risk while saving time and money. The main area of simplification was in response models. We simplified the response model, using the response factor approach. The derivation of response factors was based on statistical results of a series of parametric studies and analysis techniques in soil-structure interaction, structure response, and subsystem response analysis. The need for time history seismic response analysis was minimized through reliance on these response factors and a limited reanalysis of the nuclear power plant for which the seismic PRA is being performed.

Some simplification in plant logic models was suggested. However, combined probabilistic risk assessment needs which acknowledge both external and internal initiators should dictate requirements here.

Further studies need to be performed in assessing the potential influence of the many factors that might vary in the design of the various plants to which the SSMRP simplified seismic risk assessment method might be applied. Other

NRC studies are continuing to broaden the application and verify the results of the simplified methodology.

PART I: OVERVIEW

1.0 INTRODUCTION

This report summarizes our development of simplified seismic probabilistic risk assessment (PRA) methodology. The objective of this methodology is to reduce seismic PRA costs and time. Our development is part of the scope of work of the NRC Seismic Safety Margins Research Program (SSMRP). The SSMRP is supported by the NRC Office of Nuclear Regulatory Research.

Our development efforts are divided into the following tasks:

- Complete the development of simplified methodology for estimating seismic response (including response correlation and random and modeling uncertainties) directly from free-field ground accelerations, including to the extent necessary:

  - Methods to account for soil-structure interaction effects.
  - Calibration (see definition below) of major structure response.
  - Calibration of piping system response.

- Complete the development of guidelines for event/fault trees to be used in simplified seismic risk analyses.

- Analyze the Zion Nuclear Power Plant, Zion, Illinois, using simplified methodology and compare results with those of the SSMRP detailed seismic PRA on Zion.

- Issue a report giving procedures and guidelines for applying the simplified methodology. Limitations of these procedures should be stated.

This report completes this last task. We also developed the following reports as part of the SSMRP simplified seismic methodology development PRA effort:

J. J. Johnson, E. C. Schewe, and O. R. Maslenikov, "SSI Response of a Typical Shear Wall Structure," Lawrence Livermore National Laboratory, Livermore, CA, UCID-20122, Vols. 1 and 2 (1984).

L. C. Shieh, [illegible] C. Tsai, [illegible] S. Yang, "Seismic [illegible] and Calibration of Piping Systems", Lawrence Livermore National Laboratory, Livermore, CA, UCID-20314, 1985.

C. Y. Kimura, "Categorization of PWR Accident Sequences and Guidelines for Fault Trees: Seismic Initiators", Lawrence Livermore National Laboratory, Livermore, CA, UCID-20211, [illegible].

These reports provide supporting details for the results and recommendations presented here.

The SSMRP started in 1978 with the following objective:

"To develop mathematical models that realistically predict the probability of radioactive releases from seismic induced events in nuclear power plants. These models will be used for four purposes:

o To perform sensitivity studies to determine weak links in seismic methodology.

o To estimate the probability of release for a plant.

o To estimate the conservatism in the Standard Review Plan (SRP) seismic design methodology.

o To develop an improved seismic design methodology."

The SSMRP detailed seismic PRA methodology is based on a state-of-the-art seismic and systems analysis process and explicitly includes the uncertainties inherent in such a process. It is apparent that the tools, codes, and data bases developed by the SSMRP constitute the most complete and comprehensive seismic PRA methodology. It remains to be seen whether this methodology is adequately complete or comprehensive. The NRC has initiated seismic PRA validation efforts to address this issue.

In FY 1983 a need arose for a seismic PRA methodology that could be more widely and generally applicable than the SSMRP detailed methodology. This need arose principally as a result of the utility-sponsored PRA studies on the Zion and Indian Point plants. These studies found that the seismic initiator was an important, if not dominant, contributor to the total public risk from these plants. PRAs should include all initiators; the traditional internal initiators as well as external initiators such as wind, tornado, fire, flood, earthquake, and seismically induced fire and flood.

The basic objectives of the SSMRP simplified seismic PRA methodology are to save time and money while adequately estimating seismic risk. The target has been to simplify the methodology so an organization could produce a seismic PRA for about $500K. The actual cost would, of course, depend on the plant studied and status of its design calculations and systems models. To determine if the objective is achievable, we made the following assumptions:

1. We assume the seismic PRA is carried out as part of a combined PRA. This is an important cost-related assumption because the incremental effort associated with adding seismic-initiated failures to plant logic models is relatively small if this is done at the same time that these models are developed for internal and other external initiators. Developing plant logic models for all initiators simultaneously is also the best way to ensure consistency in the calculated risk estimates from the various initiators.

2. We assume the seismic hazard models (site specific hazard functions and response spectra) for any eastern United States site will be available.

We assume that western sites will be treated on a case-by-case basis as part of the scope of the specific PRA.

3. We assume seismic design data is available for all structures, systems, components and equipment.

2.0 SEISMIC PROBABILITY RISK ASSESSMENT

A seismic PRA is an analytical technique for assessing the overall performance of a plant during and after an earthquake. There are two essential elements in a seismic PRA:

(1) A focus on the level of earthquake that would cause an individual item of equipment, structure, or component to fail or become incapable of performing its safety function.

(2) An assessment of which groups of failed items would cause an accident that would damage the core or lead to a release of radioactivity.

The assessment in (2) is performed using the event- and fault-tree approach of the Reactor Safety Study (RSS) (Ref. 11).

There are essential differences in (1) relative to the analogue in the RSS:

- The loss of function or failures are assumed to be caused by the earthquake rather than initiated by some unspecified random event although random event failures are included in the SSMRP.

- The failure rates are based on calculations and experimental evidence of the effects of postulated earthquakes rather than on observed data. Whereas, in the RSS, only data from experience was used.

2.1 Need for Seismic PRAs

There are several potential uses for seismic PRAs.

- One use arises as a result of the proposed NRC safety goal. If a safety goal is adopted, then there may be a need to develop estimates of the seismic induced:

- Probability of core damage
- Probability of radioactive release
- Public risk and so forth.

- Earthquakes larger than the design level event are possible, and a seismic PRA or a portion of it provide a framework for evaluating the consequences.

- A seismic PRA is helpful to establish what, if any, cost-effective changes should be made to a plant to improve its reliability.

2.2 Objectives of a Seismic PRA

Seismic PRAs serve the following purposes:

- Assist backfitting decisions, such as questions regarding the potential of low fracture toughness of the steam generator and the reactor coolant pump supports (Task Action Plan A-12) (Ref. 3).

- Assist the regulatory agency to resolve regulatory issues such as justification for proposed new regulatory requirements to the Standard Review Plan, Sections 3.7.1, 3.7.2 and 3.7.3, which are seismic design parameters, seismic system analysis, and seismic subsystem analysis, respectively (USI A-40 Value/Impact Assessment) (Ref. 4).

- Allocate resources for research activities and generic safety issues.

- Identify the dominant accident sequences which lead to core melt and radioactive release, and their frequencies of occurrence for a nuclear power plant due to a seismic event. This knowledge is valuable for improving the understanding of the design and operative weakness.

3.0 SEISMIC SAFETY MARGINS RESEARCH PROGRAM SEISMIC PROBABILISTIC RISK ASSESSMENT: OVERVIEW OF DETAILED VS. SIMPLIFIED

The overall scope and the tasks required to perform a seismic PRA for a nuclear power plant are the same for a detailed and simplified PRA. Only the level of detail and scope of some tasks differentiate a simplified from a detailed seismic PRA. The assumptions which make the goal of a simplified seismic PRA achievable are described first.

To complete a simplified seismic PRA with a reasonable cost and schedule, without significantly sacrificing the accuracy of the analysis, the following should be met:

- We assume the seismic PRA is a part of a combined PRA which consists of both internal and external events.
- We assume the data necessary to determine the seismic hazard (best estimate and bounds) for any eastern United States plant site will be available from the NRC Seismic Hazard Characterization of the Eastern United States Project and comparable industry studies. The seismic hazard curves will either be available a priori or easily developed once the site is specified.
- We assume building and subsystem seismic response design data is available through architect/engineer firms and vendors.

There are seven key elements to complete a seismic PRA (Fig. 3.1):

1. Plant and site familiarization
2. Earthquake hazard determination
3. Plant logic modeling
4. Seismic response determination
5. Fragility determination
6. Plant seismic safety analysis
7. Assessment results and interpretation

These elements are discussed in the following sections. The SSMRP detailed methodology is discussed first, followed by the recommendation for the simplified methods.

Figure 3.1 clearly shows the flow of the seismic methodology. What is not shown clearly is the interface between the systems analyst and the fragility analyst and also between the systems analysts and the subsystem, structural, and seismic hazard analysts. Much interaction takes place between these analysts as an interactive process. This interactive process adds to the efficiency in utilizing all of the resources available to the project.

3.1 Plant and Site Familiarization

Detailed

Conducting a detailed seismic PRA, requires integrating diverse sources of information and analysis. The first task is to become familiar with the plant and site. This will lay down the framework for the hazard, structure and subystem, and plant logic models to be constructed.

The data and drawings of the reactor systems, necessary to construct plant logic models and to become familiar with the plant, should be available through either the plant owner or the internal event PRA team. The other data which is unique to plant seismic analysis should be available through the plant owner/operator. It is necessary to walk-through each plant to identify unique plant features and potential weak points of structures, equipment, components, and piping systems; e.g., weak ceilings, clearance between building, etc., to see that no such features are overlooked in the analysis. Site conditions are a feature unique to each seismic PRA and must be examined closely.

Simplified

There are no significant differences between the SSMRP detailed and simplified approaches in the plant and site familiarization area.

3.2 Earthquake Hazard

Detailed

The seismic hazard is characterized in terms of seismic hazard functions, ground response spectra, and earthquake time histories. At each acceleration range of interest, typically 30 but as many as 90 sets of three components of realistic time histories of motion are specified at the surface of the soil.

The seismic hazard functions are discretized into intervals for response and systems analysis purposes. Responses are calculated as a function of earthquake acceleration levels. The probabilistic calculations are also performed for each acceleration range separately and summed.

The Zion site has a significant local site effect. This aspect of the hazard was treated explicitly in the time history response calculations which were relative to a hazard function specified at a hypothetical rock outcrop.

Simplified

The seismic hazard function and response spectra for eastern United States sites will be obtained from the NRC-sponsored Seismic Hazard Characterization Project and comparable industry studies. Western sites will require a site-specific hazard characterization as part of the PRA.

The simplified methodology is fundamentally different from the detailed. The simplified requires spectra but not time histories. Some time histories may be developed as part of the simplified seismic PRA for the purpose of

benchmarking the seismic response of structures or piping systems, but this is optional.

3.3 Plant Logic Models

Detailed

The seismically-induced initiating events are generally classified as RVR (reactor vessel rupture or any break that results in a loss of coolant at a rate that exceeds mitigation capability), LOCA (loss of coolant accident), or transient. If an initiating event occurs, safety systems are required to operate in order to either prevent a core melt or to reduce the severity of its consequences. Event trees are used to identify the possible paths that a reactor could follow during a shutdown. These paths usually involve an accident and a subsequent failure of one or more safety systems and are referred to as accident sequences. A nuclear power plant is a collection of systems. Each system consists of numerous items of equipment. During the course of a severe earthquake or abnormal conditions, failure of one or more items of equipment could prevent the system from performing its safety function (termed top event). A risk assessment identifies and quantifies the component or equipment contributing to the failure of the system. The technique used to determine the failure modes for the safety systems is a system fault tree.

Construction of a fault tree begins by identifying the immediate causes of system failure. Then each of these causes is examined for more fundamental causes, until one has constructed a downward branching tree, at the bottom of which are failures not further reducible, i.e. failures of mechanical or electrical components due to all causes, such as structural failure or human error. These lowest-order failures on the fault tree are called basic events.

The fault trees used in the Zion study contain two types of basic events. The first type is the basic event in which failure probabilities are obtained by using response/fragility curves. These basic events are, typically, failures of hardware components. The second type is the random basic event. These consist of failures, such as human errors and other events. These failure probabilities are not related to the level of seismic excitation. In our study, all random basic events are treated as independent.

The data base for random basic events is included as part of this task. A number of human-error basic events were included on the fault trees. These events represent errors committed by plant personnel, maintenance procedure, or operational procedure before, during, and after the earthquake. They include operator errors, calibration errors, and maintenance errors. The human errors have been assigned failure probabilities based on available data, with corrections made to account for earthquake-induced operator stress. For simplicity, however, we assumed that the probability is not a function of the earthquake level and, thus, these errors are modeled similar to what is done for internal event studies.

Simplified

The same set of functional/systematic event trees developed during the internal event analysis will be used as the starting point for the seismic

PRA. Fault trees from internal events are modified to include passive failures, structurally induced equipment failures, equipment locations (to assure proper modeling), and susceptible components. Guidance for constructing event and fault trees for Westinghouse plants can be found by using the dominant accident sequences found by SSMRP.

3.4 Seismic Response

Detailed

In the plant familiarization task, structures housing the safety systems and equipment, and piping systems of the safety systems were identified and reviewed. In the hazard determination task, the earthquake time histories were generated. This task generates probabilistic seismic responses for all structures, equipment, and subsystems identified in the plant logic models.

For each level of earthquake described by the seismic hazard curve, three aspects of seismic response are necessary to perform the seismic risk analysis: best-estimate response, variability of response, and correlation of responses. These seismic responses are required for all structures and components contained in the plant logic models (fault trees and event trees). The three aspects of seismic response are discussed briefly here.

- Best-estimate response - Given an earthquake occurrence, the best-estimate seismic response is required. In general, the best-estimate response differs from the design values because, in the latter case, design analysis procedures, parameter selection, and qualification procedures are conservatively biased. An additional consideration due to analyzing for the range of earthquakes is the change in properties of the soil/structure/piping systems which occurs as excitation levels increase. For example, higher excitations lead to low soil shear moduli, lower structure frequencies, and higher soil and structure damping characteristics. Such changes need to be taken into account when determining the best-estimate response.

- Variability of response - Variability in seismic response results from variations in the earthquake excitation, the physical properties of the soil/structure/piping system, and our ability to model them. This variability must be acknowledged and included in the seismic risk analysis to permit calculation of probability of component failure and core melt frequency.

- Correlation of responses - The tendency for pairs of responses to have simultaneously high or low values results from two sources: the level of the earthquake and the dynamic characteristics of the system. The level of the earthquake affects correlation because a large earthquake (large peak acceleration) may cause all responses to be large, whereas a small earthquake produces the opposite effect. The second source of correlation is due to system response itself. For example, floors within a structure may all experience high values of response simultaneously due to the dynamic characteristics of the structure. Hence, equipment supported on these floors may simultaneously have high response. The importance of response correlation on frequencies of system failure, core melt, and

radioactive release depends on correlations between fragilities and the functional characteristics of the systems.

In the SSMRP detailed methodology, the computer program SMACS was used to calculate all aspects of seismic response.

Simplified

- Use design responses scaled to earthquake levels corresponding to the SSE and above. Use response factors described below to account for conservatism in design methodology.

- Optionally, perform limited time history response calculations to benchmark and extrapolate design information.

- Assign variability in responses based on SSMRP computed values as described below.

- Assign correlation to response based on SSMRP computed values as described below.

3.5 Fragility

Detailed

The failure (fragility) of structures or structural elements may occur in one of several possible modes. If the structure provides a pressure boundary, then a failure mode is loss of pressure boundary integrity. Structures whose main purpose is to support subsystems and components fail when they no longer provide adequate support. Secondary subsystems and components fail when structural elements collapse. An example of secondary failure is collapse of the pump enclosure roof on the service-water pumps. Component failure is defined as either loss of pressure boundary integrity or loss of operability. In all cases, failure is characterized by a cumulative distribution function which describes the probability that failure has occurred given a value of loading. The loading is described by local response quantities such as spectral acceleration, local peak acceleration, or an internal force resultant such as moment, depending on the component and failure mode under consideration, rather than being related directly to the free-field peak acceleration.

Fragilities for structures and large components, such as steam generators, were developed uniquely for the Zion nuclear power plant configuration. Fragility functions for other components were treated generically. As a first step, all components identified in the fault tree analyses were grouped into 37 generic categories. Fragility functions for each generic category were developed from design analysis reports, experimental data, and an extensive expert opinion survey. Statistical methods were used to combine data from the several sources.

Simplified

There are no significant differences between the SSMRP detailed and simplified approaches in the fragility area. Fragilities will have to be developed for each seismic PRA based on the information available at the time.

3.6 Plant Seismic Safety Analysis

Detailed

To calculate the accident sequence, core damage, and radioactive release probabilities, several items are integrated in this plant seismic safety evaluation. These items are the fault tree and event tree model, the containment event tree, hazard curves, seismic response of structures and subsystems, fragility, and data for random basic event failure probability.

Boolean equations are used to specify the logical failure relationships between structural, piping, and equipment failures within the reactor systems. The common-cause failure of component groups introduced by an earthquake is explicitly included in the computation of the probability of such component failure groups. This unique feature sets the SSMRP methodology apart from other existing seismic risk assessment methods.

The scope of the safety evaluation can consist of, but is not limited to, point value estimates, importance measures analysis, sensitivity studies, uncertainty analysis, and case studies.

Simplified

There are no significant differences between the SSMRP detailed and simplified approaches in this area. Point value estimates are useful for sensitivity studies but should be used with caution as an estimator of the mean. The mean should be estimated through uncertainty analysis using recommended values for the modeling uncertainty. This will require a number of SEISIM runs (14 were done in the SSMRP detailed study).

3.7 Assessment Results and Interpretation

Detailed

The final task for a seismic PRA is to interpret and document the results produced in the preceding task. For primary objectives, the report should include the frequencies of core melt, radioactive release, and dominant accident sequences contributing to the release and public dose in release categories. Dominant hardware components and human errors should be documented in the report.

Case studies can be used to gain engineering insight to the physical meaning and importance of different assumptions. For example, the collapse of the pump enclosure roof of the crib house is assumed to cause all six service water pumps to fail.

The sensitivity and importance measure studies generate the ranking of the event or component with respect to the release category frequencies or to the risk.

Simplified

There are no significant differences between the SSMRP detailed and simplified approaches in the analysis results and interpretation area.

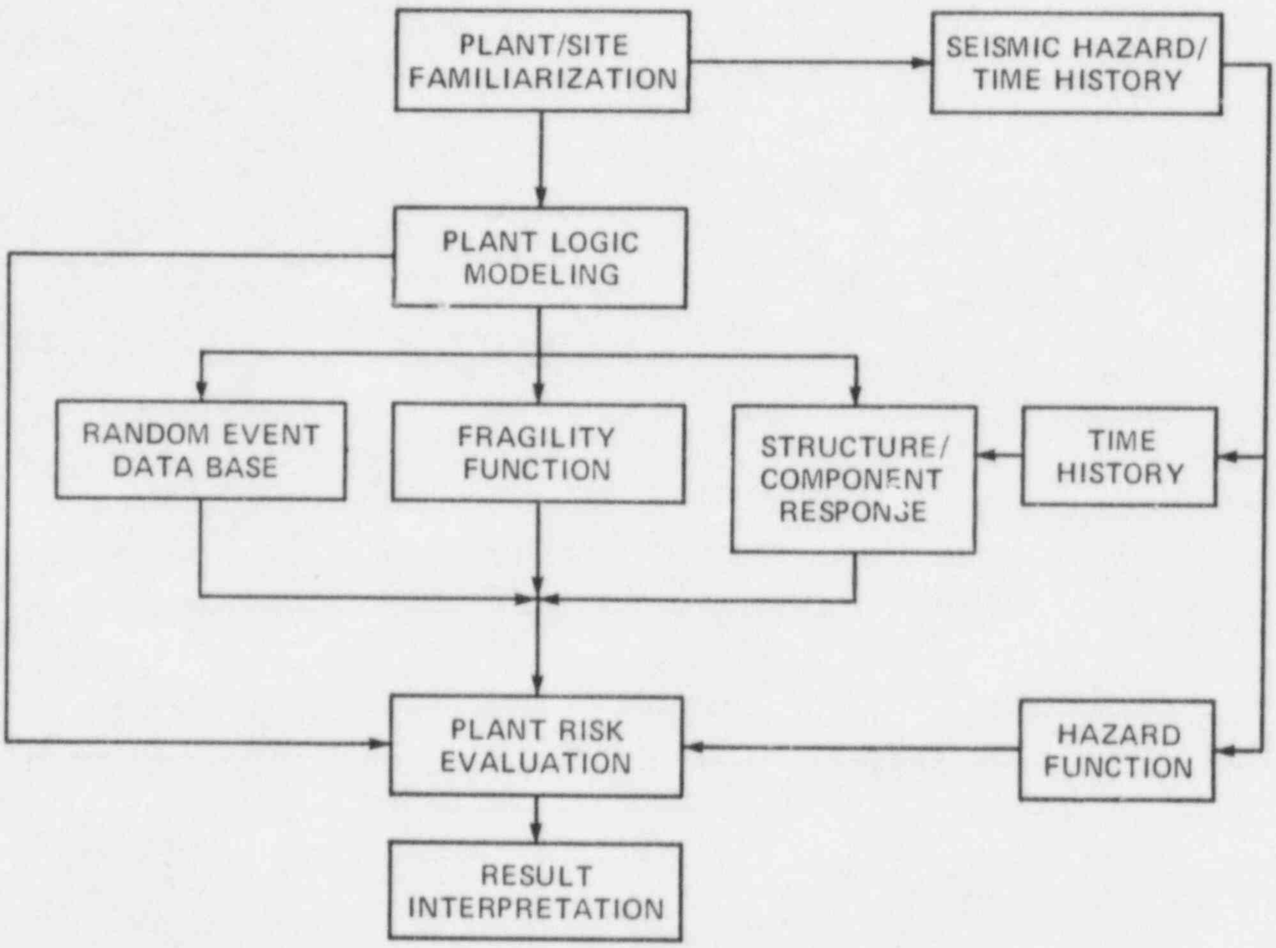


Figure 3.1. SSMRP seismic PRA methodology.

4.0 MANPOWER ESTIMATES AND SCHEDULE

Seismic PRA manpower and task estimates are presented in Tables 4.1 and 4.2. Plant familiarization is the frame work for all other tasks. This task takes two to three months and requires five to seven man-months of effort, provided event/fault trees and human factor data are available from the internal event team. Once safety-related structures, systems, and components are identified and the site conditions are examined, the seismic hazard, accident scenarios, fragility and seismic response of structures and subsystems tasks can proceed in parallel. These are followed by the plant safety evaluation which requires response data, fragility data, and fault tree quantification. Assessment result interpretation can proceed while safety evaluation continues. The documentation can get started at the completion of each task. The actual period of time required to perform the analysis depends on the manpower available and plant specific features. A typical schedule and effort estimate is given in Tables 4.1 and 4.2.

Table 4.1 PRA schedule estimates.

Task	Months 0	2	4	6	8	10	12
1. Plant and Site Familiarization							
2. Seismic Hazard							
3. Accident Sequence							
4. Seismic Response							
5. Fragility							
6. Plant Safety Evaluation							
7. Assessment Results and Interpretation							
Report							

Table 4.2 PRA manpower estimates.

Task	Manpower Estimate (Man-Month)
Plant and Site Familiarization	5 - 7
Seismic Hazard	1 - 3
Accident Sequences	3 - 5
Seismic Response of Structures and Subsystems	7 - 9
Fragility (Structures and Components)	5 - 7
Plant Safety Evaluation	7 - 9
Assessment Results, Interpretation, and Report	12 - 16
Management	10 - 14
Total:	50 - 70

PART II: PROCEDURES AND LIMITATIONS

In previous sections, an overview of seismic PRA analysis procedures is presented. The remainder of this report presents more detailed information on each task:

- Plant and Site Familiarization
- Earthquake Hazard
- Plant Logic Models
- Seismic Response
- Fragility
- Plant Seismic Safety Analysis

The purpose, assumptions, and method, and the information needs are described for each task.

5.0 PLANT AND SITE FAMILIARIZATION

5.1 Purpose

The purpose of this task is to gain an overall familiarity with the plant and site and to acquire basic detailed information for the subsequent tasks.

5.2 Information Needs

- Final Safety Analysis Report
- Piping and Instrumentation Diagrams
- General Arrangement Drawings (equipment location)
- Systems descriptions
- Soil reports

It is particularly important to have a number of meetings with the owner and architect/engineer (A/E) to clarify details. Plant visits and access to owner and A/E personnel will be necessary.

6.0 EARTHQUAKE HAZARD

6.1 Purpose

The purpose of this task is to develop seismic hazard functions and response spectra for the site in a form appropriate for use in a seismic PRA. The seismic hazard function gives the probability of exceeding a given size ground motion as a function of the ground motion parameter which is usually the peak ground acceleration.

6.2 Assumptions and Methods

The NRC has funded the Seismic Hazard Characterization (SHC) Project at LLNL to develop a consistent, uniform set of parameters (zonation, magnitude distributions, upper magnitude cutoffs, attenuation functions, and so forth) for all of the United States east of the Rocky Mountains. Seismic hazard functions and response spectra for Eastern sites will either be available a priori or easily developed once the site is identified. For western sites a specific analysis will be required as part of the simplified PRA. Seismic hazard estimations are being made by the Electric Power Research Institute (EPRI) and some individual utilities. These estimations could be useful in a seismic risk study.

Figure 6.1 shows hazard functions typical of those developed in the SHC Project. Note that 15th, 50th and 85th percentile hazard functions are shown. The simplified seismic PRA requires a number of hazard functions at other percentiles. The number of hazard functions required depends on the uncertainty analysis that is desired as part of the PRA. No rule is known for the minimum required number of functions, but at least 10 and as many as possible should be developed. (Until an acceptable rule is developed, the PRA should contain a study showing that the results (median, mean, 10th percentile, etc.) of the PRA are stable with respect to the number of functions - see below.) The necessary upper limit of acceleration of the hazard function is a function of the fragilities and logic models of the plant. For typical eastern United States plants, the results to date suggest that a limit of about 6 SSE or 1.2g, whichever is smaller, is adequate. (Whether this limit is sufficient should also be established as part of the PRA - see below.)

Figure 6.2 shows typical response spectra developed in the SHCP. (These spectra are used as part of the response calibration - see below.)

Close coordination is required between the hazard analysis and the other analyses to ensure consistent assumptions and methods in the overall uncertainty analysis.

6.3 Information Needs

- Site location (latitude and longitude)
- Required number of hazard functions (minimum of 10)
- Upper limit of hazard function (6 times SSE or 1.2g, whichever is smaller, for eastern sites is probably adequate)
- Return period for spectra

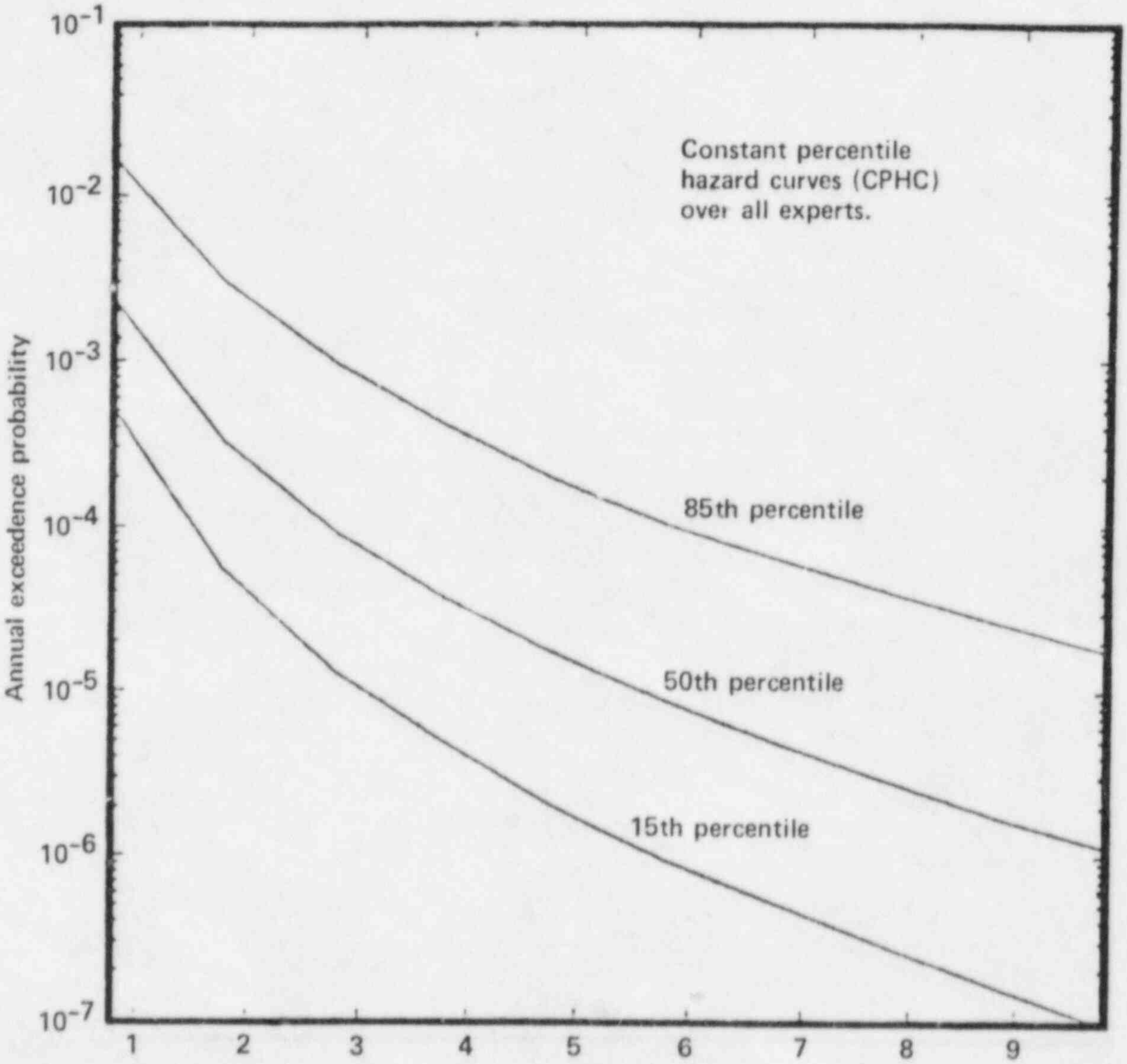


Figure 6.1. Example of hazard curves.

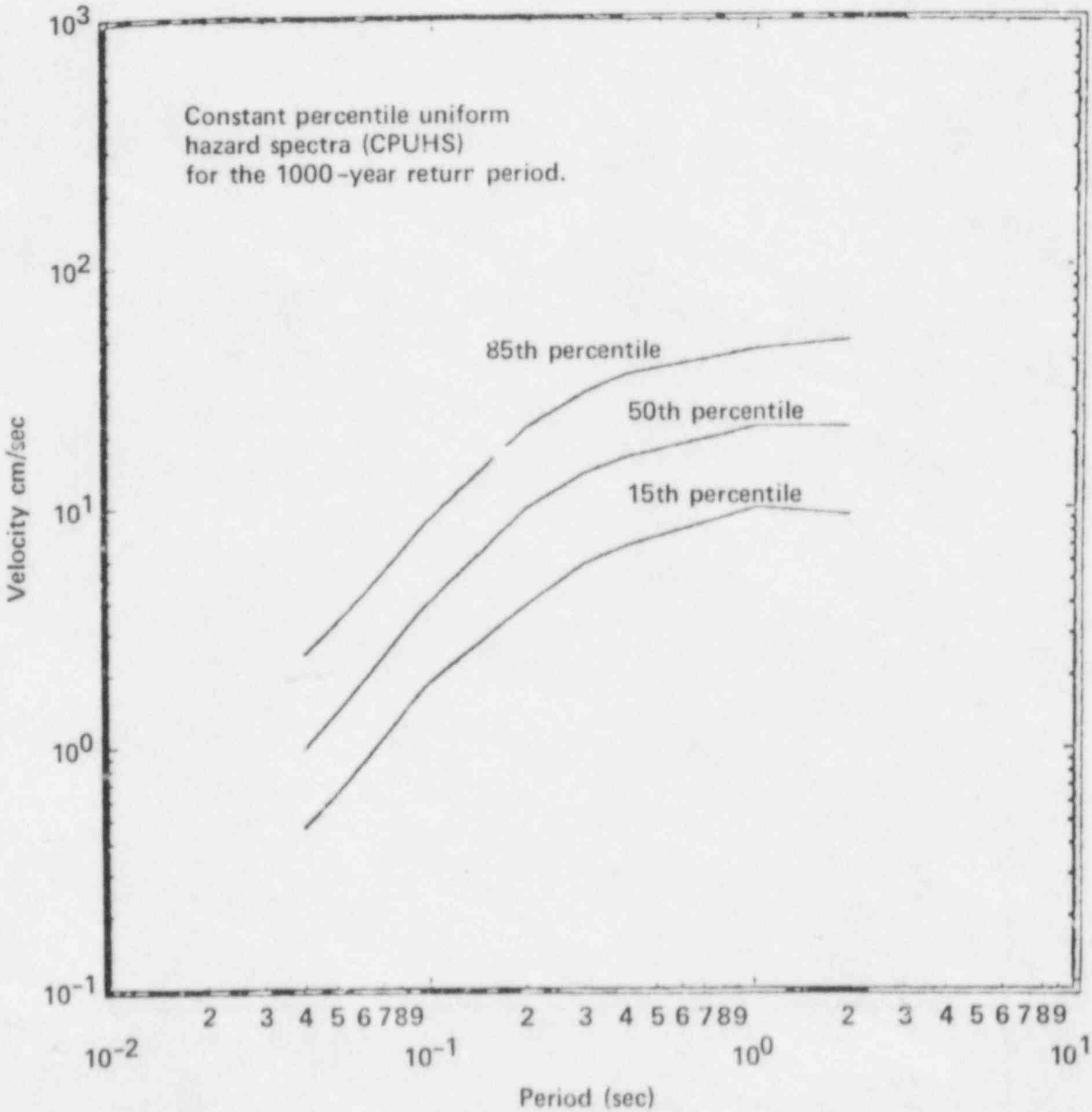
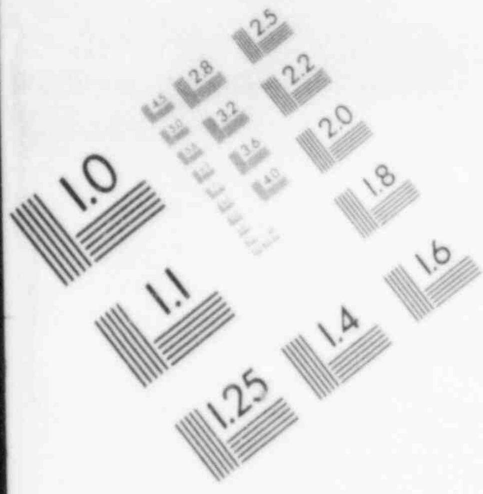
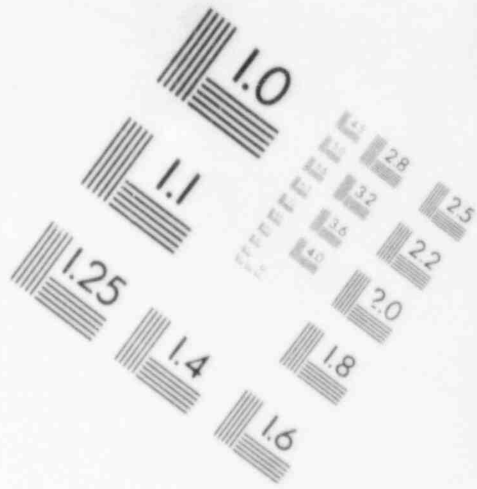
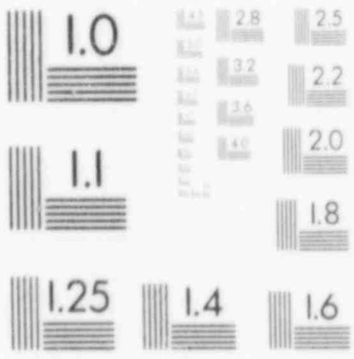


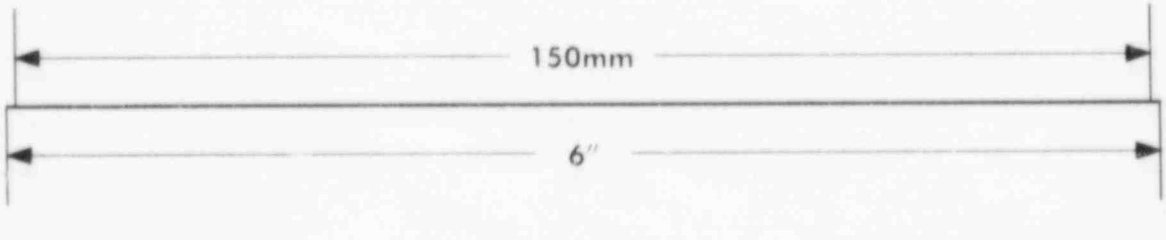
Figure 6.2. Typical response spectra.

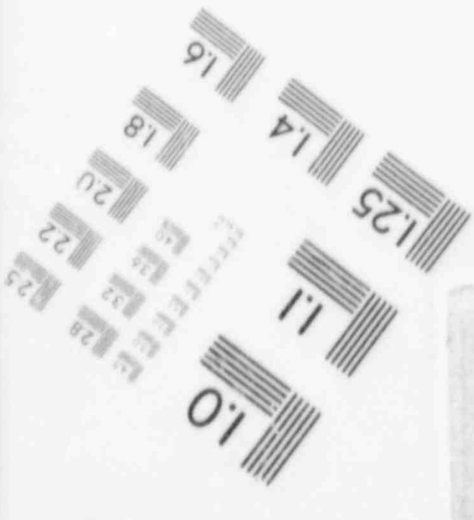
IMAGE EVALUATION
TEST TARGET (MT-3)

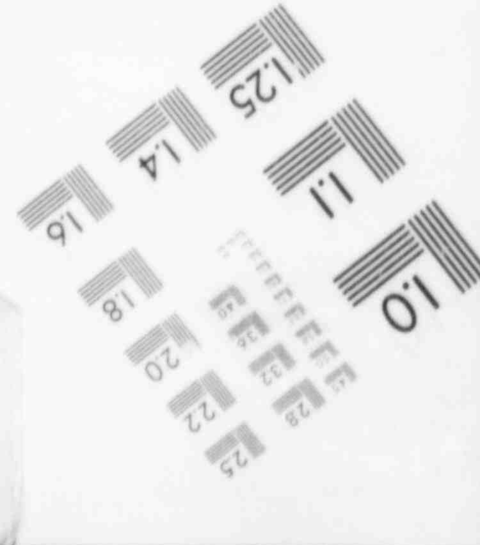












7.0 PLANT LOGIC MODELS

7.1 Purpose

In this section we describe methods for defining initiating events, event trees, and fault trees. We also identify the accident sequences found to be dominant in the SSMRP study of the Zion plant to assist the analyst in reducing the scope of work and focusing on those accident sequences that warrant the most attention. This should simplify the analysis process.

7.2 Scope

Descriptions used in this section refer only to PWR accident sequences and are not to be assumed generic. Comparisons are made between different types of PWRs: Westinghouse (W), Combustion Engineering (CE), and Babcock and Wilcox (BW). We show how the dominant accident sequences at Zion (a Westinghouse design) apply to CE and BW plants.

Because this analysis is taking a simplified approach, some areas in the logic models are not carried out in order to keep the scope reasonable. These include such things as the treatment of recovery, relay chatter, and consideration of seismic-induced fires and floods. While we agree that these could be significant contributors to risk, they are not included in the simplified approach in order to keep the scope of the approach manageable.

7.3 Assumptions and Methods

7.3.1 Assumptions

In this section, we assume that event trees and fault trees have been developed for an internal event analysis and it is necessary to convert these to apply to the seismic case. Since event trees do not contain component failures, the internal event analysis of event trees is appropriate for use in the seismic case. However, internal event fault trees are generally developed by using different criteria and are therefore subject to change when considering the seismic event. In this section, we discuss how to modify the internal event fault trees to make them compatible with the seismic issues involved. Before discussing those differences, we present a brief overview of how to develop event trees and fault trees.

7.3.2 Objective

The first step in performing any risk assessment is to review systems modeling that is required. This begins with the definition of the objectives of the study and the acquisition of substantial amounts of information on plant design and operation. The process of defining the accident includes identification of the accident initiating events, component failures, procedural faults, human errors, and dependent failure mechanisms that can cause these accident sequences to occur. Figure 7.1 shows the process of accident-sequence definition.

After defining the objectives of this study, the analyst must become familiar with the plant. Plant information needed to complete the study is described in Section 7.4. The analyst should perform a "walk-through" to view the

physical layout of equipment and to identify the components or structures that have a potential of failing during a seismic event.

7.3.3 Initiating Events

Modeling starts with definition of the initiating events and the calculation of their probability of occurrence. One general scheme for PWRs is to classify initiating events into three categories:

- Reactor vessel rupture (RVR). This is not simply a rupture of the reactor vessel, but is a break or a combination of breaks that lead to a loss of fluids that is greater than can be made up by safety systems.

- Loss of coolant accident (LOCA). This is a break or a combination of breaks that lead to a loss of fluids that can be made up by safety systems. Various sizes of LOCAs are often specified with a different event tree for each.

- Transient. This is an unscheduled shutdown of the reactor.

It is possible to define additional categories of initiating events but those above are adequate for our discussion here.

Initiating events by definition are mutually exclusive, i.e., one cannot have both a transient and a LOCA. Thus, a hierarch of precedence for the initiating events needs to be established. For example, one could have RVR, large LOCA, medium LOCA, small LOCA, and transient. To compute the initiating event probabilities, the initiating events must be defined as cut sets. SEISIM then takes the cut sets for each initiating event and computes their probability of occurrence based on the response and fragility information provided. Then, since a large LOCA cannot occur if there is an RVR, the probability of the combined event large LOCA and no RVR is computed and so on. Because it is assumed that at least one of the initiating events will occur given an earthquake, the initiating event probabilities are normalized to 1.0.

7.3.4 Event Tree Analysis

Once the initiating events are defined, the analyst must evaluate how the plant responds to each. Detailed information on the safety systems is required to identify the responses and define the criteria controlling and mitigating the accident.

The analysis describes how these systems function in relation to the initiating event. This description defines the event trees that will be used. Once the event trees have been defined, the analyst can define which systems need to be modeled. The analyst should make use of previous studies to ensure that the dominant accident sequences from those studies are at least considered in the analysis.

7.3.5 Fault Tree Analysis

The systems modeling aspect will generally use fault tree analysis techniques. The fault trees for any given system must include interfaces with the various supporting systems, such as AC power, DC power, service water, and the component cooling water. In general, these supporting systems can be developed as separate fault trees and then be input into the safety system trees. While developing the fault trees, the analyst should pay particular attention to those elements of the tree that could lead to seismic failure events.

Resolution of the fault tree is always a concern that needs to be addressed in each fault-tree-generation project. In general, fault trees are modeled down to a resolution at which applicable data exists or to a level consistent with the scope which has been determined by the analyst. It is important that sufficient detail be provided to identify key contributors. In a seismic analysis, if the analyst predetermines to study, for example, the relay chatter question in great depth, then it may be necessary to carry the resolution of the fault tree down to the contact level within the relays. However, for the simplified approach, we are recommending that the level of resolution be taken down to the component level for which fragility data are available. Also, for a seismic analysis, structural failures must be included.

Because of the dependence between events in a seismic situation, the analyst must be careful to achieve a level of resolution which will pick up any dependencies between components. Again, the analyst must use judgement in defining the level of resolution.

7.3.6 Modification of Internal Event Fault Trees

This section describes the modifications and changes necessary in a fault tree that had been developed for an internal event analysis to make it compatible with seismic concerns.

In many internal event analyses, a single passive, double active criterion is used when developing the fault tree. This means that if the fault tree has an active component, i.e., one that is required to perform some action in order to perform successfully, then failure of three components at the same time is not considered a reasonable occurrence. Therefore, a double active criterion states that only two active-type components will be allowed to fail. Three are considered to have too small a probability and therefore the fault trees are not drawn beyond that point. An example of an active component is a normally closed valve that is required to open. Cut sets containing two such valves are included, whereas cut sets containing three such valves are never generated because of the low probability of occurrence. The same analogy is true of the passive components in that a single passive is allowed but double passive components are considered to have too small a probability to include in the analysis. An example of a passive component is a pipe. Its only requirement is to stay intact. No positive action is required.

This limitation raises a problem since pipes which are close together may fail together due to correlation effects. When the single passive criterion is used, doubles and higher order events are not considered. Therefore, dominant

doubles which may become important in a seismic analysis may never show up. The same is true for active components, particularly when discussing electrical active components such as circuit breakers. The chance of a circuit breaker tripping or a relay chattering during an earthquake is high. Doubles and triples of electrical components could put the plant in a non-desirable status and may not be taken into account in internal event analysis. This omission should be studied to see what effect it may have. The analyst must go back and look at the piping and instrumention diagrams (P&IDs) to see how to insert those higher order events into the fault trees. We recommend that the original fault tree analyst perform this task since understanding the systems from the drawings could be a large task in itself. It would not be cost or time effective to perform this effort as part of the simplified seismic PRA.

Another item to be considered is an accurate location analysis to determine where components are located within the plant. Accurate location is necessary before any response calculations can take place. In many internal event analyses, location is not done unless common-cause concerns have been brought up and specifically examined. Therefore, it would be necessary to identify those components on the fault trees. Once the fault trees have been modified to take care of the single passive, double active component problem, one could take these components and locate them within the plant so that the analyst performing the subsystem calculations knows their location.

We now consider structural failures. A structural failure occurs when a structural component, be it a wall, building, or roof, collapses or otherwise damages components. This type of common-cause analysis showed up as being important in previous risk assessments. For example, if a shear wall of the auxiliary building should fail, it could damage the control wiring and electrical wiring going between the auxiliary building and containment building as was the case in the Zion analysis. Another example is if the crib house pump enclosure roof at Zion collapses and damages all the service water pumps. This combination of six failures in Zion; i.e., three pumps for Unit 1 and three pumps for Unit 2, may not be considered during an internal event analysis. Since pumps are considered active components; i.e., required to perform a function or action during an event, only doubles would be included in an internal event analysis. Therefore, items such as walls or roofs failing are types of structural failures that may not be considered in an internal analysis. To consider these, location needs to be considered. When a wall is considered failed, the analyst must look at how that wall may collapse and what components may be affected near the wall.

Another consideration is failure of a component causing the failure of another component. This was not modeled explicitly or thought to be dominant in the case of Zion; however, there is a need to look at the probability that a component, such as a steam line may break and the resulting steam environment causes the failure of another component. These types of considerations are sometimes discussed in internal event analysis, and need to be considered during a seismic risk analysis.

7.3.7 Culling of Fault Trees

Because of the size of system fault trees, a method was needed to reduce the resulting Boolean expressions. Probabilistic culling was selected.

Probabilistic culling discards cut sets that do not contribute significantly to the probability of the top event. Cut sets do not contribute significantly if their probabilities are small. Because basic events in a cut set are dependent, it is not efficient to compute bounds or approximations to cut set probabilities and cull them if the bounds or approximations satisfy some probability condition.

We require that two conditions be satisifed before we discard a cut set. If the minimum of the basic event probabilities in a cut set is sufficiently small and if the product of basic event probabilities in a cut set is sufficiently small, we discard the cut set. The first criterion, culling on the minimum probability basic events, is needed because of the common-cause aspect of the problem. If all basic events are fully correlated, then the upper bound to the cut set probability is the probability of the minimum probability basic event. The second criterion, culling on the product of the basic event probabilities in a cut set, is based on the assumption that all basic events in the cut set are independent.

7.3.8 Human Errors

It is important to consider human and maintenance errors as part of the modeling effort just as in external event analysis. These errors should be considered in terms of potential effects on the individual components and also their effect on entire systems. The following example notes the effects on components. A maintenance person tests a normally closed valve to see if it will open if required. At the end of the testing, he fails to reclose the valve. If this particular valve appears in an accident sequence, then leaving this valve open could cause failure of the system. If, however, there is another valve downstream which is also normally closed, and this valve also was left in an open position after the maintenance, then that maintenance error could cause the entire subsystem to fail. In this example several types of errors are possible. Human errors, which involve the maintenance worker not performing the actions correctly as specified, or procedural errors, which fail to tell him to reclose the valve after inspection. These types of errors are not dependent on the seismic event. Many operator errors that occur during or after a seismic event may affect the system, though to what extent they were caused directly by the seismic event is not clear.

System analysts should be familiar with the operating, maintenance, and emergency procedures for the system that they are analyzing. Human errors may then be presented on the fault trees as causes of component unavailability or failure when the error occurs before the accident sequence progresses to core damage or release of radioactive material from the facility. The potential for operator recovery from the sequence should also be considered. As indicated earlier, information relating to the state of the operator following a seismic event is not clear.

7.4 Information Needs

When starting a risk assessment of a nuclear power plant, much information is needed. Table 7.1 lists additional information that is needed for plant familiarization and accident review, event tree development, and fault tree development. In the seismic event, particular attention needs to be paid to location of equipment and possible failure modes not considered in the

internal event analysis. Information providing additional insights into these problems is needed. This information could include such things as the qualification data for pumps, valves, or other components.

It is important to obtain all the required information as soon as possible. This ensures that the analyst will be able to continue his work throughout the analysis process. Any unique features of the particular facility should be reviewed in the light of whatever additional information may be needed concerning that system or component.

7.5 Accident Sequences to be Considered

This section will provide a set of accident sequences that can be applied to many commercial PWRs.

From the Zion seismic risk study, we identified a set of 18 dominant terminal accident sequences assuming feed and bleed is possible and a set of 17 dominant terminal accident sequences assuming feed and bleed is not possible. In order to apply these sets of dominant terminal accident sequences to other PWRs, we made a safety system by safety system comparison between Zion and six other reactors. Although seven plants were PWRs of the three PWR manufacturers and operated basically the same, there were enough differences in plant systems and their functionability/operability that a determination of the applicability of the dominant terminal accident sequences of Zion to other PWRs other than of Westinghouse design could not be made.

7.5.1 Dominant Terminal Accident Sequence Development

This subsection describes the method used to identify the dominant terminal accident sequences. The results are based on the probabilistic seismic risk assessment of the Zion nuclear power plant for the event sequences assuming (1) feed and bleed operation is possible and (2) feed and bleed operation is not possible.

For the purpose of discussion, the following terms are defined. "A dominant terminal event sequence" (TES) is defined as any TES whose probability of radioactive release is greater than or equal to 1E-10/yr at any earthquake level (based on Zion Seismic Risk Study). "A dominant terminal accident sequence" is defined as those dominant TESs with common accident sequences summed over all containment failure modes. "A generic accident sequence" is defined as those dominant terminal accident sequences that can be applied to more than one PWR power plant. These generic accident sequences are discussed in more detail later in this section.

The probabilities of the dominant terminal event sequence (TES) at each earthquake level are summed over all earthquake levels to obtain the probability of radioactive release due to any seismic event. Table 7.2 lists the dominant TES based on radioactive release summed over all earthquake levels assuming feed and bleed is possible. Table 7.3 lists the dominant TES summed over all earthquake levels assuming feed and bleed is not possible. These TESs account for 87% - 92% of the total radioactive release for the respective cases.

7.5.2 Conclusion

This section developed, from the Zion seismic risk study, a set of 18 dominant terminal accident sequences assuming feed and bleed is possible and a set of 17 dominant terminal accident sequences assuming feed and bleed is not possible. In order to apply these sets of dominant terminal accident sequences to other PWRs, a safety system by safety system comparison between Zion and six other reactors was made. Although seven plants were PWRs of the three PWR manufacturers and operated basically the same, there were enough differences in plant systems and their functionability/operability that a determination of the applicability of the dominant terminal accident sequences of Zion to other PWRs other than of Westinghouse design could not be made. Fault trees for individual safety systems of Combustion Engineering (CE) and Babcock and Wilcox (BW) PWR plants should be developed and compared to determine the effect of plant system design differences, and differences in plant operations. Plant safety systems of Westinghouse designed PWRs should be compared to ensure that major differences do not exist between the plants.

In addition to the comparison mentioned, a comparison of event trees used in three other safety studies was made. The initiating event definition, the event tree itself, and the success criteria for the event contained on the event trees were compared. It is concluded that it is not possible to apply the Zion event trees to PWR plants of the Combustion Engineering and Babcock and Wilcox design. It may be possible to apply the Zion event trees, and hence, the dominant terminal accident sequences of the Zion Seismic Study, to PWRs of the Westinghouse design, particularly to those PWR plants in the 1100 MWe 4-loop class without ice condenser containments. The user is again advised to compare the safety systems of the PWR plant being analyzed with the safety systems of Zion to ensure that major differences that would necessitate the creation of the new event trees do not exist between the plants.

Table 7.1 Sources of information needed for the definition of accident sequences (PRA procedure guide).

Task	Information Sources
Plant familiarization and accident review	Operator training manuals Complete final safety analysis report (FSAR) Plant layout drawings Reviews with operating staff Emergency procedures Plant visits
Event-tree development	FSAR Chapters 6 and 15 EPRI NP-2230[a] Licensee event reports from specific plants or sister plants, plant incident reports Performance capability of the emergency core-cooling system and other systems considered in developing system-success criteria Analyses documenting system performance Plant visits
Fault-tree development	FSAR chapter on individual systems and instrumentation System descriptions Piping and instrumentation diagrams Control logic diagrams Drawings of instrumentation power supplies Piping location and routine drawings Power-source documents Drawings of the offsite and onsite power-distribution systems One-line diagrams of the electrical system Circuit diagrams and trip criteria for the electrical bus protection system Normal operating procedures for systems Emergency operating procedures for systems Chapter 16 of the FSAR (i.e., technical specifications) Testing and maintenance procedures and intervals Annunciated system parameters System-response parameters (valve opening times, pump start times) Environments for all essential sensors, detectors, and indicators under normal and accident conditions Any existing failure modes and effects analyses on plant systems Plant visits

[a] ATWS: A Reappraisal, Part 3, "Frequency of Anticipated Transients," Electric Power Research Institute, 1982.

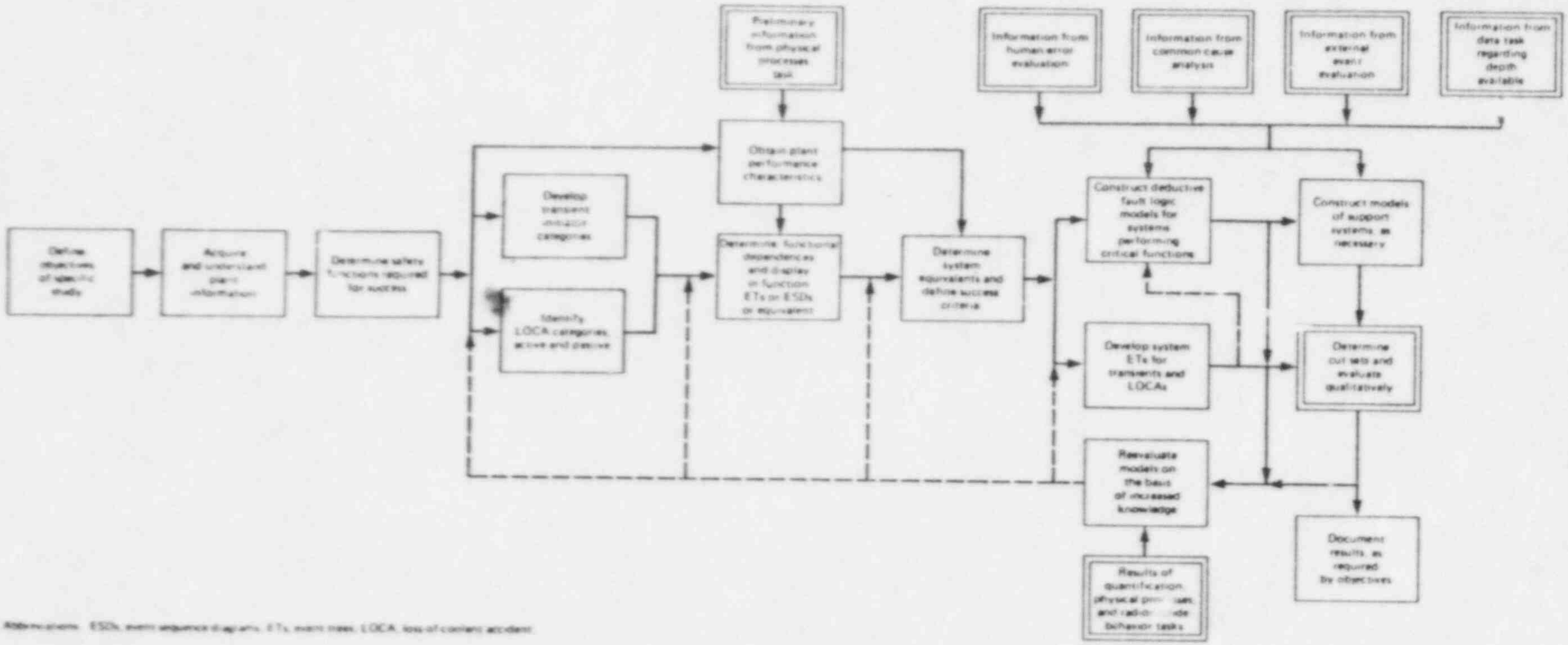
Table 7.2 Dominant terminal event sequence (TES) ranking with feed-and-bleed capability.

Rank	TES Description*
1	SLOCA-ACC21-delta
2	T2-ACC04-delta
3	RVR-ACC07-gamma
4	SSLOCA-ACC21-delta
5	T2-ACC04-gamma
6	SSLOCA-ACC35-gamma
7	MLOCA-ACC21-delta
8	SSLOCA-ACC35-delta
9	RVR-ACC06-gamma
10	T2-ACC04-alpha
11	RVR-ACC07-alpha
12	LLOCA-ACC27-gamma
13	SSLOCA-ACC34-gamma
14	T2-ACC04-epsilon
15	RVR-ACC07-delta
16	SSLOCA-ACC35-alpha
17	SLOCA-ACC21-alpha
18	SLOCA-ACC28-alpha
19	LLOCA-ACC27-delta
20	LLOCA-ACC13-alpha
21	SSLOCA-ACC34-delta
22	LLOCA-ACC27-epsilon
23	SSLOCA-ACC35-epsilon
24	LLOCA-ACC28-alpha
25	SSLOCA-ACC34-epsilon
26	SSLOCA-ACC21-alpha
28	MLOCA-ACC27-gamma
29	LLOCA-ACC13-epsilon
30	RVR-ACC01-alpha
31	SSLOCA-ACCD-gamma
32	T1-ACC07-alpha
33	MLOCA-ACC13-alpha
34	SSLOCA-ACC34-alpha
35	RVR-ACC01-epsilon

*Example: SLOCA-ACC21-delta refers to "small LOCA 21 with a delta containment failure mode." Refer to "Application of the SSMRP Methodology of the Seismic Risk at the Zion Nuclear Power Plant" (Ref. 1) for more detail. For systems involved in the accident sequence, refer to the appropriate event tree in Reference 1.

Table 7.3 Dominant terminal event sequence (TES) ranking without feed-and-bleed capability.

Rank	TES Description
1	SLOCA-ACC21-delta
2	T2-ACC04-delta
3	SSLOCA-ACC21-delta
4	RVR-ACC07-gamma
5	T2-ACC04-gamma
6	T2-ACC04-alpha
7	SSLOCA-ACC35-gamma
8	T2-ACC04-epsilon
9	MLOCA-ACC21-delta
10	SSLOCA-ACC34-gamma
11	SSLOCA-ACC35-delta
12	RVR-ACC06-gamma
13	RVR-ACC07-alpha
14	LLOCA-ACC27-gamma
15	SSLOCA-ACC34-delta
16	SSLOCA-ACC29-alpha
17	RVR-ACC07-delta
18	SSLOCA-ACC35-alpha
19	SLOCA-ACC21-alpha
20	SSLOCA-ACC34-epsilon
21	SLOCA-ACC28-alpha
22	LLOCA-ACC27-delta
23	LLOCA-ACC13-alpha
24	SSLOCA-ACC29-epsilon
25	LLOCA-ACC27-epsilon
26	SSLOCA-ACC21-alpha
27	SSLOCA-ACC34-alpha
28	SSLOCA-ACC35-epsilon
29	LLOCA-ACC28-alpha
30	RVR-ACC06-alpha
31	MLOCA-ACC27-gamma
32	LLOCA-ACC13-epsilon
33	RVR-ACC01-alpha
34	T1-ACC07-alpha
35	MLOCA-ACC013-epsilon
36	RVR-ACC01-epsilon



Abbreviations: ESDs, event sequence diagrams; ETs, event trees; LOCA, loss-of-coolant accident.

Figure 7.1. The process of accident-sequence definition (Ref. 12).

8.0 SEISMIC RESPONSE

8.1 Purpose

Seismic responses are required for all structures, equipment, and components contained in the plant logic models. Responses may be required for floor or line-mounted valves, pipes, pumps, diesel generators, circuit breakers, busses, structures, structural elements and so forth. The range of accelerations possible at the site is considered, not simply one or two accelerations such as those defined by the OBE and SSE. For each acceleration level considered in the analysis, three aspects of seismic response must be characterized: median or best estimate, variability, and correlation.

Median - Design responses are generally conservatively biased relative to median responses as a result of the criteria for analysis, parameter values, and qualification procedures. The need to analyze for the range of accelerations specified by the hazard function may lead to the need to consider changes in the properties of the soil/structure/piping systems which may occur at higher excitations such as higher soil and structure damping.

Variability - Variability in response results from variations from one earthquake to another with the same peak acceleration, variations in the physical properties of the soil/structure/piping systems, and limitations in our ability to model these systems.

Correlation - Correlation of responses (the tendency of pairs of responses to simultaneously have high or low values) arises from two sources: the acceleration level of the earthquake and the dynamic characteristics of the system at a given acceleration level. The importance of response correlation on probabilities of system failure, core damage, and release depends on correlation between fragilities and the characteristics of the systems.

In the SSMRP detailed seismic PRA methodology, these three aspects of response were determined calculationally using the program SMACS. Considerable effort is required to perform the SMACS analyses; the major portion is associated with developing and coordinating models. In the SSMRP simplified seismic PRA methodology, we use A/E and vendor responses or models to the extent possible to reduce this effort. We use SSMRP data to estimate variability and correlation.

8.2 Overview of Procedure

8.2.1 Median

The procedure is:

1. Obtain design responses from A/E and vendor data.

2. Estimate median responses for the design earthquake acceleration by applying the response factors described below to (1).

3. Extrapolate (2) to all acceleration levels of interest using SSMRP results.

As an alternative or supplement to (2), a limited best-estimate re-analysis of portions of the plant can be performed; however, this approach will not be discussed here.

Response factors, F_R relate design responses, R_D, to best estimate responses, R_{BE}, according to the following formula:

$$R_{BE} = R_D/F_R. \qquad (1)$$

The response R may be a force or moment in a structural element, peak acceleration of a floor, acceleration of a valve, moment in a piping system, and so forth. A number of factors will typically contribute to F_R. Their number and importance depend on the response of interest. There is a compounding of effects. The basic approach is to identify aspects of conservatism or non-conservatism in seismic analysis, to quantify them, and to accumulate their effects. Several assumptions are made:

- The seismic response data available for A/Es and vendors are the medians of possible design values, all of which satisfy design criteria. Analysis results differ even though the various designers use the same analysis procedures and criteria for the same structure or subsystem. We assume that the response data will be high in half of the cases and low in half of the cases. This likelihood of error is treated as an uncertainty in the PRA methodology.

- Best-estimate medians are derivable from the design values. For example, load distributions and the overall behavior characteristics of structures and piping systems are reasonably estimated by the design analyses.

- The critical stress points in the design analysis are the correct points.

- The mean of the overall response factor is derivable as the product of the means of response factors for the separate effects.

8.2.2 Variability

Variability is specified using SSMRP data. Both random and modeling uncertainty are specified for the best-estimate data. Based on the results of parametric studies, we also recommend additional modeling uncertainty that is introduced by use of the simplified procedures.

8.2.3 Correlation

Correlation is specified using SSMRP data.

8.3 Information Needs

Site Soil Conditions

- Geological data on the site
- Soil configuration
- Boring information
- Ground water data
- Static and dynamic soil properties
  - Laboratory tests
  - In-situ field test results
  - Scatter data

Structures

- Results of dynamic seismic analysis
- Dynamic design models
- Location of major equipment
- Structural drawings
- Slab and wall specifications
- Masonry wall specifications
- Steel detailing drawings
- Beam/column schedules
- Containment wall geometry
- Concrete cylinder test results
- Re-bar test results
- Field-erected tank drawings (vendor) and civil drawing showing foundation, ring girder, and anchor bolt details

Piping

- Design analysis by NSSS and A/E
- Piping area drawings
- Piping stress isometrics
- Piping fabrication isometrics
- Pipe support/hanger detail drawings
- Piping class sheets
- Valves - WT/CG data for both valve and operators

8.4 Response Procedures

8.4.1 Median

This section describes recommended procedures for estimating response in a simplified seismic PRA. However, it should be kept in mind that the objective of estimating response is to do it for components that are identified in the fault trees, that is, components for which fragility estimates are required. Thus, in Section 8.4.1.1.2 (Shear Wall Structure), the response procedure is for a structure whose fragility is desired. In Section 8.4.1.3 (Piping

Systems) the response procedure is for a piping system whose fragility is desired, but note that structural response estimat s are required as part of the process of estimating the response of the piping system.

8.4.1.1 Building

The process of calibrating structure response begins with identifying those aspects of seismic analysis that have a major influence on response prediction. We have identified three major aspects: seismic input, soil-structure interaction (SSI), and structure response analysis procedures. An additional item is the energy dissipation characteristics of the soil-structure system.

- Seismic input - In most instances, specification of the design ground motion is conservative in the sense that the design response spectra are targeted to the mean plus one standard deviation of recorded earthquake data. Best-estimate response, that is median level response conditional on an earthquake's occurring and described by a specified value of the seismic hazard function acceleration, is required. Conservatism or non-conservatism in the design seismic input must be removed.

- SSI and structure response analysis procedures - Models of SSI and structures and their parameter values are the main areas of interest. In terms of SSI, two major elements are treatment of embedment (wave scattering effects and increased energy dissipation) and treatment of the radiation damping aspects of the phenomenon. An additional area to be considered is structure models.

- Energy dissipation of the soil-structure system - In addition to the aforementioned treatment of radiation damping, the effect of increased structure damping on response is of particular interest. Before a structural member fails, energy dissipation increases and the effect of dissipation on structure response needs to be assessed.

Recall that our objective is to derive a response factor, F_R, for use in Equation (1) - see Section 8.2.1. We define a response factor for buildings, F_{RB}, as arising from a number of factors:

$$F_{RB} = F_{SI} \cdot F_{SSIFB} * F_{SSIM} * F_{SSID} * F_{SM} * F_{SD} \quad (2)$$

F_{SI} - measures the deviation from the median of the spectra used in the design of the plant (further defined below).

F_{SSIFB} - accounts for the case where a fixed base design analysis is performed but the actual physical conditions are such that a soil-structure interaction analysis could have been performed. If this is the case then F_{SSIM} and F_{SSID} are equal to 1.0.

F_{SSIM} - accounts for the case where a soil-structure interaction analysis is performed but the conservative assumption is made to locate the control motion at the elevation of the foundation of the embedded structure. There are several scenarios for placing the control motion at foundation depth. The amount of conservatism varies depending on the scenario. All were not investigated here. When a soil-structure interaction analysis is performed in design, F_{SSIFB} is equal to 1.0.

F_{SSID} - accounts for the case where SSI damping values are set so conservatively low that radiation effects are not included.

F_{SM} - accounts for the quality and refinement of the structural model used in the design analysis.

F_{SD} - accounts for conservatism or nonconservatism in the structural damping values used in the design analysis.

The factor, F_{SI}, for seismic input is obtained from a comparison of the design spectra for the plant and the best-estimate spectra obtained from the hazard analysis (Section 6). Typically, we expect the design spectra to envelope the best-estimate spectra, but there probably are exceptions. The hazard analysis focuses on horizontal motion and this dictates the same focus for the development of F_{SI}.

As exemplified in Figure 8.1, there are a number of best-estimate spectra, each corresponding to a different return period. The spectra that correspond to the return period associated with an acceleration at the SSE should be selected as a response factor.

F_{SI} is defined as the simple arithmetic average (frequency point by frequency point over the frequency range of the soil-structure system) of the quotients of the design and best-estimate horizontal spectra. The damping of the spectra should correspond to estimates of soil-structure system damping.

Local site amplification occurs at about 20 to 30 sites in the eastern United States where there are relatively shallow soil deposits overlying crystalline bedrock. These sites may have abnormally high amplifications in certain relatively narrow frequency ranges. We understand that local site amplification will be addressed in the SHC Project, which means that the best-estimate spectra will already include the local site effect.

The remaining terms in Equation (2) are defined in the following sections.

8.4.1.1.1 Containment

F_{SSIFB} is described in Table 8.2 for a variety of conditions and for peak accelerations and forces. The range is 1.16 to 3.67. The case numbers listed in Tables 8.2 - 8.4 are described in Table 8.1.

One scenario of control motion definition and control point location is quantified in the first four lines of Table 8.3. This factor was described as F_{SSIM} in Equation (2) and is multiplicative. In this case, the control motion was assumed to be the foundation input motion and all points on the foundation

experience this motion. Further work is required to develop a better estimate of this factor. Other scenarios introduce significantly more conservatism and require quantification.

In some cases, the soil-structure interaction analyses may have been performed by using two different methods and using the envelope of the two results for design. In this case, a factor defined by the quotient of the envelope and average of the two responses should be applied.

F_{SSID} is obtained from the first three lines in Table 8.4: 1.93, 2.57, and 3.46. These values are for a soil damping limit of 5%. If the damping used in design is greater than 5% then these values should be reduced. We recommend linear interpolation between 5% and 30% with a factor of 1 at 30% design damping. For a design damping value of 10%, for example, the factors 1.93, 2.57, and 3.46 become 1.75, 2.25, and 2.97 respectively. The 30% damping is assumed based on the combination of material and radiation damping in a vertical direction.

We recommend F_{SM} = 1.0 unless specific studies indicate otherwise.

F_{SD} appears to be a relatively minor factor except for the fixed-base analysis case (see Table 8.9). We recommend F_{SD} = 1.1 for all cases where the realistic damping values are larger than those used in design, F_{SD} = 0.9 where they are smaller, and F_{SD} = 1.0 otherwise. Note that if the damping factor is considered in the derivation of the fragility response factors, F_{SD} = 1.0.

8.4.1.1.2 Shear Wall Structure

F_{SSIFB} is described in Table 8.6 for a variety of conditions for accelerations and forces. The range is 0.86 to 2.73. The case numbers listed in Tables 8.6 - 8.9 for shear wall structure are described in Table 8.5.

F_{SSIM} is described in the first four lines of Table 8.7 again for one scenario. Further work is required to develop a better estimate of this factor as described above. In some cases, the soil-structure interaction analyses may have been performed by using two different methods and using the envelope of the two results for design. In this case, a factor defined by the quotient of the envelope and average of the two responses should be applied.

F_{SSID} is obtained from the first four lines of Table 8.8: 1.53, 1.86, 2.44, and 2.11. These values are for a soil damping limit of 4%. If the damping used in design is greater than 4%, then these values should be reduced. We recommend linear interpolation between 4% and 30% with a factor of 1 at 30% design damping.

We recommend F_{SM} = 1.0 unless specific studies indicate otherwise.

F_{SD} appears to be a relatively minor factor (see Table 8.9) except for the fixed-base analysis case. We recommend F_{SD} = 1.0 for all cases. If the damping factor is considered in the derivation of the fragility response factors, F_{SD} = 1.

8.4.1.2 Equipment

Recall that our objective is to derive a response factor, F_R, for use in Equation (1) (see Section 8.2.1). We define a response factor for equipment mounted on buildings, F_{RE}, as arising from a number of other factors:

$$F_{RE} = F_{SI} * F_{SSIFB} * F_{SSIM} * F_{SSID} * F_{SM} * F_{SD}$$

The procedure for F_{SI} is the same as in Section 8.4.1.1.

F_{SSIFB} is described in Tables 8.10 and 8.11 for a variety of conditions and two different structures. The range is 0.88 to 6.61. The value that should be used is the average of the values for the three frequency ranges shown. For the first line in Table 8.9, this is (0.99 + 0.93 + 1.64)/3 = 1.19, for example.

F_{SSIM} is described in the first four lines of Table 8.12 and 8.13. The range is 1.05 to 1.39. Because the range is so small, we adopt a single factor of 1.25. Further work is required to develop a better estimate of this factor. The 1.25 is believed to be quite conservative. In some case, the soil-structure interaction analyses may have been performed by using the two different methods and using the envelope of the two results for design. In this case the factor of 1.25 should be multiplied by the quotient of the envelope and average of the two responses.

F_{SSID} is obtained from the first three lines in Table 8.14 and the first four lines of Table 8.15. The range is 1.02 to 2.68. The value that should be used is the average of the values for the three frequency ranges shown.

We recommend F_{SM} = 1.0 unless specific studies indicate otherwise.

F_{SD} appears to be a relatively minor factor - see Table 8.16. We recommend F_{SD} = 1.0 for all cases.

8.4.1.3 Piping Systems

8.4.1.3.1 Inertial Response

Recall that our objective is to derive a response factor, F_R, for use in Equation (1) - see Section 8.2.1. We define a response factor for the inertial response of piping systems, F_{RPI}, as arising from a number of other factors:

$$F_{RPI} = F_{RE} * F_{PA} * F_D * F_I \qquad (3)$$

F_{RE} is a factor similar to the one for equipment except that the value to be used is the average of the values for the four frequency ranges instead of three. For the first line in Table 8.10, this is (0.99 + 0.93 + 1.64 + 1.14)/4 = 1.18, for example. F_{PA} is a factor to account for the conservatisms in the spectral methods of analysis used in the design of piping systems. F_{PA} is obtained from Table 8.17. Three different values are given for moment and

acceleration. The range is 3.9 to 6.6 for moment response, and 6.2 to 11.3 for acceleration response. The basis for deriving the F_{PA} factors is using envelope response spectra on 2% damping for the response spectrum method and 2% damping for the time history analysis technique. The method of analysis used in design determines which of the three values to use (see Appendix Table A.27).

F_D accounts for deviation from the piping system damping values used in the design analysis. F_D is obtained from Table 8.18. For example, if 2% damping is used in the design analysis but 5% is considered best-estimate, then F_D for moment is 1.5. If the damping factor is considered in the derivation of the fragility response factor, F_D = 1.0.

F_I - This factor accounts for the effect of the input spectrum specified other than the envelope spectrum. The F_I value is obtained from Table 8.19.

8.4.1.3.2 Pseudostatic Response

We define a factor, F_{RPS}, to account for the conservatisms in the methods of analysis used in design to account for the relative displacement of the support points of piping systems:

$$F_{RPS} = F_{SI} * F_{PM} \qquad (4)$$

F_{SI} is the quotient of the peak displacements of the design and the best-estimate. We recommend F_{SI} =1.0. Further work is required to derive a better estimate of this factor.

F_{PM} accounts for conservatism in the methods of analysis used for design and is obtained from Table 8.20. There are two values, 4.1 or 16.2 depending on the method of analysis used in design. Two methods were used to derive the response factor for the differential anchor movement effect. Method I moves one pipe support point at a time with the corresponding maximum support displacement calculated from the time history analysis of the building which houses the piping systems. The response from the differential anchor movement effect is obtained by combining the results from each individual support movement with the SRSS method. Method II moves all supports attached to the same structure in a given direction simultaneously and in phase. When there is more than one structure involved, the responses from each individual structure movement in the same direction are combined by the absolute sum method. The resultant pseudostatic loads are obtained by combining the three directional results with the SRSS method.

8.4.1.3.3 Combined Response

o Pipe Element Moment

The factor F_{RPI} from Section 8.4.1.3.1 is used with the peak inertial design response for a specific piping system, R_{DI}, to obtain the best-estimate inertial response, R_{BEI}, as follows:

$$R_{BEI} = R_{DI}/F_{RPI}$$

Also for moment, F_{RPS} from Section 8.4.1.3.2 is used with the peak pseudostatic design response for the same piping system, R_{DS}, to obtain the best-estimate pseudostatic response, R_{BES}, as follows:

$$R_{BES} = R_{DS}/F_{RPS}$$

These responses are combined as follows:

$$R_{BEP} = \sqrt{(R_{BEI})^2 + (R_{BES})^2}$$

Combining these responses is based on the assumption that inertially induced and pseudostatically induced moments both contribute to failures in piping systems. This is the SSMRP assumption. Determining the response factor of inertially induced moments or of the combination of inertially induced and pseudostatically induced moments depends on the failure mode of the pipe element assumed in the piping fragility generation. The response factor for piping systems thus needs to be carefully coordinated with the fragility analysis.

- o Valve Acceleration
  We set $R_{BEP} = R_{BEI}$.

8.4.1.4 Variation with Acceleration

In the previous sections we have described a procedure for estimating best-estimate responses R_{BE} .

R_D is associated with the acceleration used in the design analysis to obtain R_{BE} in Equation (1).

To compute the frequency of core melt (radioactive release) using SEISIM, it is necessary to assess the probability of failure of basic events at and above SSE level. Thus, it was necessary to estimate the response parameters (i.e., the mean vector R and variance-covariance matrixes $(S)_R$) at the acceleration levels for which these parameters are not determined directly in the calibration approach. A method to do this was developed and called "response regression models." It was discussed in detail in Appendix D of the report of Application of the SSMRP Methodology to the Seismic Risk at the Zion Nuclear Power Plant (Ref. 1). Linear extrapolation from zero through the best-estimate response at the design basis earthquake level can also be used. Linearity could be checked by calculating several responses at different earthquake levels.

8.4.2 Variability

8.4.2.1 Random Uncertainty

We obtain random uncertainty, β_R for the best-estimate response, R_{BE}, from SSMRP detailed analyses. The values are as follows:

- o Structure .28
- o Piping Moment .40
- o Valve Acceleration .34
- o Equipment mounted on building with fundamental frequencies not near soil structure frequencies .28
- o Equipment mounted on buildings with fundamental frequencies near soil structure frequencies .40

* Note, these values are for assumed peak ground acceleration intervals of approximately 0.15 to 0.25g and, hence contain this added variability.

8.4.2.2 Modeling Uncertainty

Modeling uncertainty for the best-estimate response, R_{BE}, is used in the risk calculations to describe uncertainty in the calculated results (see Section 10.2.2).

We assume that the SSMRP detailed results reflect a best estimate of modeling uncertainty (for the detailed method). These values are as follows:

- o Structure .27
- o Piping Moment .57
- o Valve Acceleration .43
- o Equipment Mounted on Buildings with Fundamental Frequencies not Near Soil-Structures Frequencies .27
- o Equipment Mounted on Buildings with Fundamental Frequencies Near Soil-Structure Frequencies .57

The simplified response procedures introduce additional modeling uncertainties due to the simplified models used. For piping moment and valve acceleration, the modeling uncertainties due to use of simplified procedures are estimated to be 0.5 and 0.4 respectively. These two values were estimated on the study of various design parameters and analysis procedures as shown in Tables A.26 through A.29.

These additional modeling uncertainties for structures and singly supported subsystems fall into the following three categories:

1) Design Analysis

In the design analysis deterministic methods are typically used. However, design responses would vary, for example, if a different time history were used. This variation would lead to a variation in the best-estimate response, R_{BE}. SSMRP treated this uncertainty as inherited and quantified this as 0.15 (Ref. 13). If an analyst wishes to treat this uncertainty as part of modeling uncertainty, he should reduce the random uncertainty by the SRSS method. For example, current structure random uncertainty is 0.28, the new value is:

$$0.24 = \sqrt{0.28^2 - 0.15^2}$$

2) Spatial Effect

The response factors are derived by averaging over spatial locations. For example, in the first line of Table A.19, the mean response factor for acceleration is shown as 1.07 with a COV of 0.261. The COV of 0.261 is a measure of the variability of the mean factor of 1.07 with respect to spatial location. Now we have assumed that the design analysis identifies the correct critical element spatially, but we have no way of knowing that the factor of 1.07, for example, applies to that critical location. This introduces an additional modeling uncertainty due to the limitations of the model we used to calibrate response. Note that this uncertainty is not additive. That is, in Equation (2) the spatial uncertainty due to F_{SSIFB} is not additive to the spatial uncertainty due to F_{SSIM}.

The amount of this uncertainty varies considerably as can be seen in the various tables in Appendix A:

Table	Range of COVs	Average COV
A.3	.099 to .641	0.17
A.5	.188 to 1.215	0.45
A.7	.324 to .400	0.36
A.9	.072 to .587	0.27
A.11	.054 to .164	0.13
A.19	.178 to .601	0.37
A.21	.244 to .432	0.36
A.23	.018 to .240	0.07

We estimate this β_U at 0.3. This applies only to structures and equipment mounted on structures.

3) Analytical Models

There is an additional uncertainty due to the quality of the analytical models used in the design analysis. If the models used in design are less than the highest detail seen in nuclear practice, assume $\beta_U = 0.3$.

Uncertainties introduced at different stages can be combined by using the SRSS method. For example, the total structure modeling uncertainty due to

simplified procedures is $(0.3^2 + 0.3^2)^{1/2} = 0.42$ (assuming no modeling uncertainty for design analysis). The combined total structure modeling uncertainty is $(0.42^2 + 0.27^2)^{1/2} = 0.50$. The first value, 0.42, is introduced by simplified procedure; the second value, 0.27, is contributed from the best-estimate.

8.4.3 Response Correlation

Correlations between seismic responses are specified based on the results of the SSMRP Phase II calculations, Tables 8.21 through 8.24. For peak ground accelerations not shown in the tables, an interpolation scheme can be employed to obtain the necessary correlation values. The values in the tables are intended for the correlation of two different response locations. When the response locations are the same, the correlation value of one should be used.

8.5 Example of the Use of Response Factors

The following were the design procedures in the original response calculations:

1) Structures.
Soil Conditions:

shear wave velocity	=	1000 fps/5000 fps
damping ratio	=	0.01
Poisson ratio	=	1/3
soil layer depth	=	100 ft.

A containment structure is embedded with embedded depth - structure radius ratio equal to 0.5. We are looking for F_{RB} where:

$$F_{RB} = F_{SI} * F_{SSIFB} * F_{SSIM} * F_{SSID} * F_{SM} * F_{SD}.$$

In the original design calculations, the fixed base of the structure was assumed. From Table 8.2, $F_{SSIFB} = 1.89$ for peak accelerations. Based on the description of Equation (2), if F_{SSIFB} accounts for the case in which a fixed base design is performed but the actual physical conditions are such that an SSI analysis could have been performed, then $F_{SSID} = F_{SSIM} = 1.0$. $F_{SM} = 1.0$ as recommended (see page 8-6); $F_{SD} = 1.0$ assuming this damping factor is considered in the derivation of the fragility response factors. $F_{SI} = 1.0$ assuming design spectrum is identical to the best-estimate spectrum.

$$F_{RB} = 1.89 \text{ for peak acceleration}$$

If the design value of a containment shell is 1.2g, the calculated best-estimate value is 0.63g.

2) Singly-supported systems.
The equation to obtain the response factor for equipment is:

$F_{RE} = F_{SI}* F_{SSIFB}* F_{SSIM}* F_{SSID}* F_{SM}* F_{SD}$
$F_{SI} = F_{SSIM} = F_{SSID} = F_{SM} = F_{SD} = 1.0$ as obtained for structures. F_{SSIFB}, which can be obtained from Table 8.10, is equal to $(1.26 + 2.34 + 3.57)/3$

= 2.39. Assuming the design value of a polar crane support is 2.5g, the calculated best-estimate value is 1.05g ($2.5^g/2.39$).

3) Piping Systems.

A response spectrum method with envelope response spectra of 2% damping was employed in the original piping inertial response calculations. The modal response combination method was the ten percent technique, and the earthquake component combination was the SRSS technique. Method II, which moves all pipe supports attached to the same structure simultaneously, was employed to calculate the piping pseudostatic responses. The design values were 90,000 ft-lb and 40,000 ft-lb for inertial and pseudostatic responses, respectively. Recall that:

$$F_{RPI} = F_{RE}{}^{*} F_{PA}{}^{*} F_{D}{}^{*} F_{I} \tag{3}$$

$$F_{RPS} = F_{SI}{}^{*} F_{PM} \tag{4}$$

From Table 8.17, $F_{PA} = 3.9$, both F_D and F_I are equal to 1.0, since 2% damping and envelope spectrum were used.

$$F_{RE} = F_{SI}{}^{*} F_{SSIFB}{}^{*} F_{SSIM}{}^{*} F_{SSID}{}^{*} F_{SM}{}^{*} F_{SD}$$

$F_{SI} = F_{SSIM} = F_{SSID} = F_{SM} = F_{SD} = 1.0$ as described earlier. F_{SSIFB}, wich is obtained from Table 8.10, is equal to $(1.26 + 2.34 + 3.57 + 2.31)/4 = 2.37$. From equation (3), $F_{RPI} = 2.37 * 3.9 * 1.0 * 1.0 = 9.2$. For pseudostatic response factor, $F_{PM} = 4.1$ and $F_{SI} = 1.0$. From equation (4), $F_{RPS} = 4.1$. Using these two response factors, we can calculate the best-estimate values, namely, 9782 ft - lb and 9756 ft - lb for inertial and pseudostatic responses. The calculated best-estimate combined piping response is $(9782^2 + 9756^2)^{1/2} = 13815$ ft - lb.

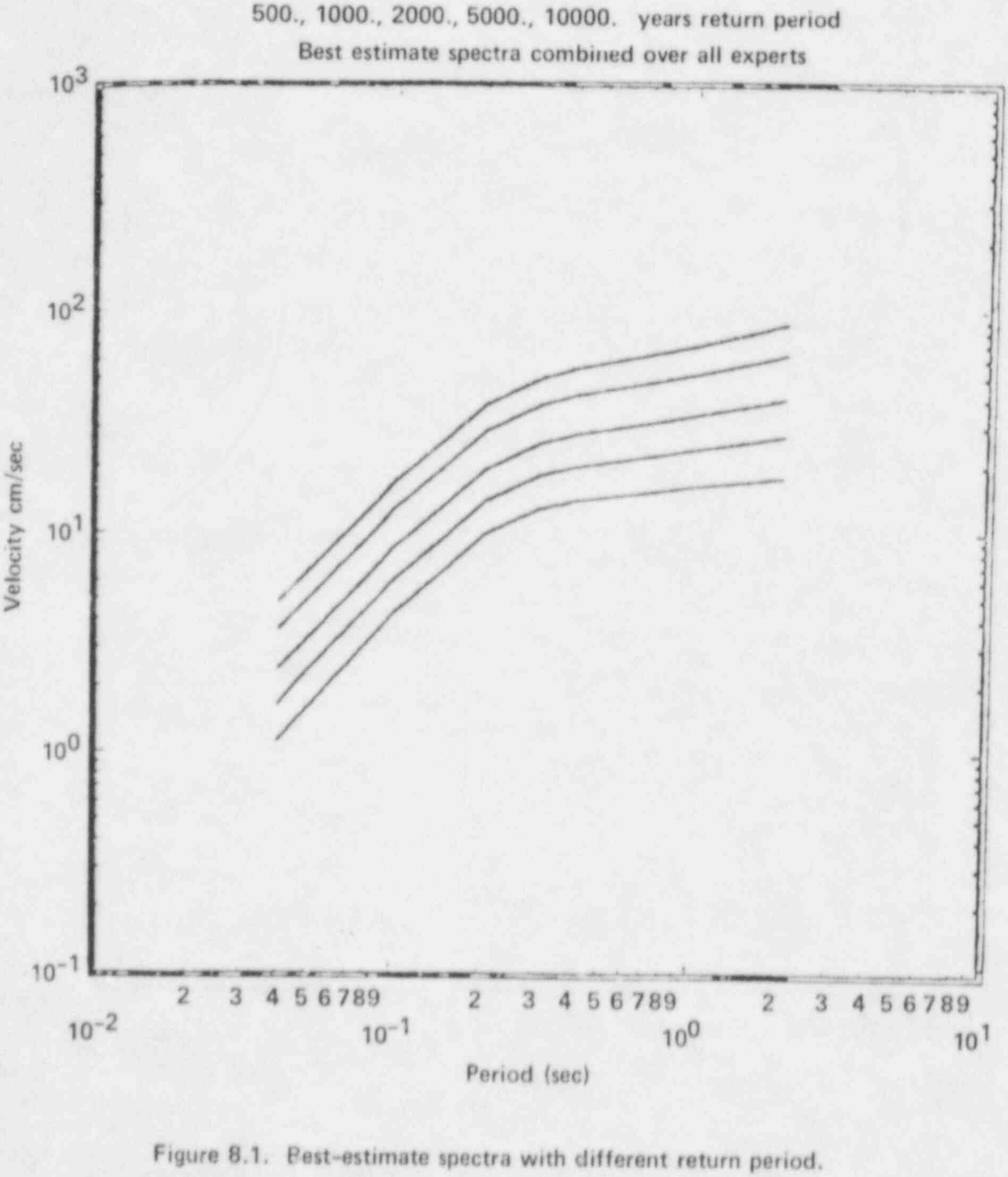


Figure 8.1. Best-estimate spectra with different return period.

Table 8.1 Identification of physical and analysis scenarios for the Containment Building.

Case No.	Soil Profile	Vs (fps)	Foundation Condition	Analysis Techniques
1	Fixed Base	Fixed Base	Surface	Best Estimate
1a	Half-Space	5000	Surface	Best Estimate
2	Half-space	3500	Surface	Best Estimate
3	Half-space	2000	Surface	Best Estimate
4	Half-space	1000	Surface	Best Estimate
5	Half-space	500	Surface	Best Estimate
6	Half-space	3500	Embedded E/R=0.46	Best Estimate
7	Half-space	2000	Embedded E/R=0.46	Best Estimate
8	Half-space	1000	Embedded E/R=0.46	Best Estimate
9	Half-space	500	Embedded E/R=0.46	Best Estimate
10	Half-space	1000	Embedded E/R=0.75	Best Estimate
11	36 ft. soil/rock	1000/5000	Surface	Best Estimate
12	36 ft. soil/rock	1000/5000	Embedded E/R=0.46	Best Estimate
13	110 ft. soil/rock	1000/5000	Surface	Best Estimate
14	110 ft. soil/rock	1000/5000	Embedded E/R=0.46	Best Estimate
15	110 ft. soil/rock	1000/5000	Embedded E/R=0.75	Best Estimate
16	250 ft. soil/rock	1000/5000	Surface	Best Estimate
17	250 ft. soil/rock	1000/5000	Embedded E/R=0.46	Best Estimate
18	250 ft. soil/rock	1000/5000	Embedded E/R=0.75	Best Estimate
19	110 ft. soil/rock	1000/9000	Surface	Best Estimate
20	110 ft. soil/rock	1000/9000	Embedded E/R=0.46	Best Estimate
21	110 ft. soil/rock	2000/5000	Surface	Best Estimate
22	110 ft. soil/rock	2000/5000	Embedded E/R=0.46	Best Estimate
23	Zion soil/rock (3 layers)	600-910-1390/9000	Embedded E/R=0.46	Best Estimate
24a	110 ft. soil (3 layers)	1600	Surface	6 Component Soil Springs
24b	110 ft. soil (3 layers)	1600	Surface	Only Rocking & Torsion Springs
25	Half-space	2000	Surface	Soil Springs
26	Half-space	1000	Surface	(6 Components)
27	Half-space	500	Surface	

Table 8.2 Response factors, F_{SSIFB} for containment building, of forces and peak accelerations for fixed-base vs. surface and embedded foundations at half-space and layered sites.

Soil Profile	Embedment* Ratio (E/R)	Characteristic Vs (fps)	Case Comparison	Peak Accelerations	Forces
Half Space	0	3500	1/2	1.20	1.16
Half Space	0	2000	1/3	1.37	1.36
Half Space	0	1000	1/4	1.82	2.02
Half Space	0	500	1/5	2.54	2.93
Half Space	0.46	3500	1/6	1.38	1.35
Half Space	0.46	2000	1/7	1.69	1.68
Half Space	0.46	1000	1/8	2.21	2.42
Half Space	0.46	500	1/9	3.11	3.67
Half Space	0.75	1000	1/10	2.37	2.61
36 Ft. Layer	0	1000/5000	1/11	1.19	1.20
36 Ft. Layer	0.46	1000/5000	1/12	1.61	1.82
110 Ft. Layer	0	1000/5000	1/13	1.54	1.60
110 Ft. Layer	0.46	1000/5000	1/14	1.89	1.99
110 Ft. Layer	0.75	1000/5000	1/15	2.26	2.39
250 Ft. Layer	0	1000/5000	1/16	1.78	1.95
250 Ft. Layer	0.46	1000/5000	1/17	2.17	2.40
250 Ft. Layer	0.75	1000/5000	1/18	2.37	2.60
110 Ft. Layer	0	1000/9000	1/19	1.50	1.56
110 Ft. Layer	0.46	1000/9000	1/20	1.84	1.96
110 Ft. Layer	0	2000/5000	1/21	1.22	1.21
110 Ft. Layer	0.46	2000/5000	1/22	1.58	1.541
110 Ft. Layer	0.46	600/910/ 1390/9000	1/23	2.00	1.75

* E/R = embedment depth/structure radius.

Table 8.3 Response factors, F_{SSIM} for containment building, of forces and peak accelerations for surface vs. embedded foundation conditions.

Soil Profile	Embedment* Ratio (E/R)	Characteristic Vs (fps)	Case Comparison	Peak Accelerations	Forces
Half Space	.46	3500	2/6	1.22	1.19
Half Space	.46	2000	3/7	1.29	1.25
Half Space	.46	1000	4/8	1.25	1.22
Half Space	.46	500	5/9	1.29	1.28
Half Space	.75	1000	4/10	1.37	1.33
36 Ft. Layer	.46	1000/5000	11/12	1.87	1.63
110 Ft. Layer	.46	1000/5000	13/14	1.30	1.26
110 Ft. Layer	.75	1000/5000	13/15	1.55	1.52
250 Ft. Layer	.46	1000/5000	16/17	1.26	1.25
250 Ft. Layer	.75	1000/5000	16/18	1.40	1.37
110 Ft. Layer	.46	1000/9000	19/20	1.30	1.27
110 Ft. Layer	.46	2000/5000	21/22	1.35	1.29

* E/R - embedment depth/structure radius.

Table 8.4 Response factors, F_{SSID} for containment building, of peak accelerations for soil springs vs. surface and embedded foundations at half-space and layered sites.

Soil Profile	Embedment* Ratio (E/R)	Characteristic Vs (fps)	Case Comparison	Peak Accelerations
Half Space	0	2000	25/3	1.93
Half Space	0	1000	26/4	2.57
Half Space	0	500	27/5	3.46
Half Space	0.46	2000	25/7	2.33
Half Space	0.46	1100	26/8	3.13
Half Space	0.46	500	27/9	3.46
Half Space	0.75	1000	26/10	3.31
36 Ft. Layer	0	1000/5000	26/11	1.56
36 Ft. Layer	0.46	1000/5000	26/12	2.54
110 Ft. Layer	0	1000/5000	26/13	2.11
110 Ft. Layer	0.46	1000/5000	26/14	2.68
110 Ft. Layer	0.75	1000/5000	26/15	3.16
250 Ft. Layer	0	1000/5000	26/16	2.51
250 Ft. Layer	0.46	1000/5000	26/17	3.08
250 Ft. Layer	0.75	1000/5000	26/18	3.27
110 Ft. Layer	0	1000/9000	26/19	2.07
110 Ft. Layer	0.46	1000/9000	26/20	2.65
110 Ft. Layer	0	2000/5000	25/21	1.69
110 Ft. Layer	0.46	2000/5000	25/22	2.17
110 Ft. Layer	0.46	600/910/ 1390/9000	24a/23	

* E/R = embedment depth/structure radius.

Table 8.5 Identification of physical and analysis scenarios for the shear wall structure.

Case No.	Soil Profile	V_s (fps)	Foundation Condition	Analysis Technique
1	Fixed-base	Fixed-base	Surface	Best estimate
2	Half-space	3500		
3		2000		
4		1000		
5		500		
6		3500	Embedded	
7		2000		
8		1000		
9		500		
10	110 ft. layer	2000	Surface	
11		1000		
12		2000	Embedded	
13		1000		
14	71 ft. layer	2000	Surface	
15		1000		
16	Half-space	3500		Soil springs
17		2000		
18		1000		
19		500		

Table 8.6 Response factors, F_{SSIFB} for shear wall structure, of forces and peak accelerations for fixed-base vs. (surface and embedded foundations) (half-space and layered sites).

Soil Profile	Embedment* Ratio (E/R)	Characteristic+ Vs (fps)	Case Comparison	Peak Accelerations	Peak Forces
Half-Space	0	3500	1/2	1.07	1.23
Surface	0	2000	1/3	1.25	1.42
Foundation	0	1000	1/4	1.47	1.64
	0	500	1/5	2.02	2.14
Half-Space	0.35	3500	1/6	1.27	1.44
Embedded	0.35	2000	1/7	1.52	1.73
Foundation	0.35	1000	1/8	1.89	2.11
	0.35	500	1/9	2.52	2.73
110 Ft. Layer	0	2000	1/10	1.07	1.23
Surface	0	1000	1/11	1.27	1.39
Foundation					
110 Ft. Layer	0.35	2000	1/12	1.39	1.60
Embedded	0.35	1000	1/13	1.67	1.90
Foundation					
71 Ft. Layer	0	2000	1/14	.86	1.03
Surface	0	1000	1/15	.99	1.09

* E/R = embedment depth/equivalent structure radius.

\+ bedrock shear wave velocity = 9000 fps.

Table 8.7 Response factors, F_{SSIM} for shear wall structure, of forces and peak accelerations for surface vs. embedded foundation conditions (half-space and layered sites).

Soil Profile	Embedment* Ratio (E/R)	Characteristic+ Vs (fps)	Case Comparison	Peak Accelerations	Peak Forces
Half-space	0.35	3500	2/6	1.25	1.17
	0.35	2000	3/7	1.26	1.22
	0.35	1000	4/8	1.31	1.29
	0.35	500	5/9	1.28	1.29
110 ft. layer	0.35	2000	10/12	1.37	1.31
	0.35	1000	11/13	1.38	1.37
71 ft. layer surface vs. 110 ft. layer embedded	0.35	2000	14/12	1.63	1.57
	0.35	1000	15/13	1.74	1.75

* E/R = embedment depth/equivalent structure radius.

+ bedrock shear wave velocity = 9000 fps.

Table 8.8 Response factors, F_{SSID} for shear wall structure, of peak accelerations for soil springs vs. surface and embedded foundations at half-space sites.

	Embedment* Ratio (E/R)	Characteristic Vs (fps)	Case Comparison	Peak Accelerations
Half-Space Surface Foundation	0	3500	16/2	1.53
	0	2000	17/3	1.86
	0	1000	18/4	2.44
	0	500	19/5	2.11
Half-Space Embedded Foundation	0.35	3500	16/6	1.84
	0.35	2000	17/7	2.26
	0.35	1000	18/8	3.12
	0.35	500	19/9	2.65

* E/R = embedment depth/equivalent structure radius.

Table 8.9 Response factor, F_{SD}, of forces and peak accelerations for surface foundation conditions at 4%, 7%, and 10% structure damping (half-space site).

Soil Profile	Characteristic Vs (fps)	Case Comparison	Peak Accelerations	Peak Forces
4% vs. 7%	Fixed Base	1	1.12	1.18
	2000	3	1.04	1.02
	1000	4	1.02	1.01
4% vs. 10%	Fixed Base	1	1.22	1.32
	2000	3	1.07	1.03
	1000	4	1.04	1.01
7% vs. 10%	Fixed Base	1	1.08	1.11
	2000	3	1.03	1.01
	1000	4	1.01	1.00

Table 8.10 Response factor, F_{SSIFB} for equipment, of spectra accelerations for fixed base vs. surface & embedded foundations, containment building, (half-space and layered sites) horizontal responses.

Soil Profile	Embedment* Ratio (E/R)	Characteristic Vs (fps)	Case Comparison	0 - 6 Hz	6 - 12 Hz	12 - 20 Hz	ZPA
Half Space	0	3500	1/2	.99	.93	1.64	1.14
Half Space	0	2000	1/3	1.02	1.14	2.36	1.26
Half Space	0	1000	1/4	1.17	2.01	3.71	1.66
Half Space	0	500	1/5	1.57	3.53	5.64	2.20
Half Space	.46	3500	1/6	1.10	1.23	1.80	1.36
Half Space	.46	2000	1/7	1.16	1.56	2.80	1.59
Half Space	.46	1000	1/8	1.39	2.51	4.33	2.08
Half Space	.46	500	1/9	1.98	3.96	6.61	2.91
Half Space	.75	1000	1/10	1.48	2.76	4.43	2.25
36 Ft. Layer	0	1000/5000	1/11	.90	1.69	2.56	1.03
36 Ft. Layer	.46	1000/5000	1/12	1.31	2.35	1.81	1.57
110 Ft. Layer	0	1000/5000	1/13	1.07	1.81	3.27	1.38
110 Ft. Layer	.46	1000/5000	1/14	1.26	2.34	3.57	2.31
110 Ft. Layer	.75	1000/5000	1/15	1.37	2.67	4.43	2.08
250 Ft. Layer	0	1000/5000	1/16	1.17	1.97	3.63	1.60
250 Ft. Layer	.46	1000/5000	1/17	1.36	2.48	4.13	2.05
250 Ft. Layer	.75	1000/5000	1/18	1.50	2.76	4.42	2.24
110 Ft. Layer	0	1000/9000	1/19	1.06	1.78	3.22	1.36
110 Ft. Layer	.46	1000/9000	1/20	1.26	2.32	3.50	1.71
110 Ft. Layer	0	2000/5000	1/21	.98	1.11	2.54	1.10
110 Ft. Layer	.46	2000/5000	1/22	1.09	1.44	2.64	1.46
110 Ft. Layer	.46	600/910 1390/9000	1/23	1.16	2.21	3.84	1.51

* E/R = embedment depth/structure radius.

Table 8.11 Response factor, F_{SSIFB} for equipment, of spectra accelerations for fixed-base vs. surface and embedded foundations shear wall structure (half-space and layered sites) over specified frequency range.

Soil Profile	Embedment* Ratio (E/R)	Characteristic+ Vs (fps)	Base Comparison	0 - 6 Hz	6 - 12 Hz	12 - 20 Hz
Half-Space	0	3500	1/2	.97	1.08	1.35
Surface	0	2000	1/3	.94	1.55	1.87
Foundation	0	1000	1/4	.94	2.70	2.77
	0	500	1/5	1.12	4.67	4.10
Half-Space	0.35	3500	1/6	1.03	1.30	1.64
Embedded	0.35	2000	1/7	1.04	1.83	2.29
Foundation	0.35	1000	1/8	1.11	3.31	3.32
	0.35	500	1/9	1.37	5.29	4.78
110 Ft. Layer	0	2000	1/10	.92	1.55	1.77
Surface	0	1000	1/11	.92	2.50	2.57
Foundation						
110 Ft. Layer	0.35	2000	1/12	1.01	1.69	2.20
Embedded	0.35	1000	1/13	1.05	3.30	2.96
Foundation						
71 Ft. Layer	0	2000	1/14	.91	1.22	1.56
Surface	0	1000	1/15	.88	2.30	2.07
Foundation						

* E/R = embedment depth/equivalent structure radius.

+ bedrock shear wave velocity = 9000 fps.

Table 8.12 Response factor, F_{SSIM} for equipment, of spectra accelerations for surface vs. embedded foundation conditions containment building (half-space and layered sites) horizontal responses.

Soil Profile	Embedment* Ratio (E/R)	Characteristic Vs (fps)	Case Comparison	0 - 6 Hz	6 - 12 Hz	12 - 20 Hz	ZPA
Half Space	.46	3500	2/6	1.13	1.35	1.14	1.20
Half Space	.46	2000	3/7	1.17	1.39	1.22	1.26
Half Space	.46	1000	4/8	1.24	1.28	1.21	1.28
Half Space	.46	500	5/9	1.30	1.13	1.24	1.36
Half Space	.75	1000	4/10	1.43	1.26	1.36	1.39
36 Ft. Layer	.46	1000/5000	11/12	1.71	1.47	.89	1.62
110 Ft. Layer	.46	1000/5000	13/14	1.26	1.31	1.16	1.29
110 Ft. Layer	.75	1000/5000	13/15	1.42	1.52	1.39	1.53
250 Ft. Layer	.46	1000/5000	16/17	1.22	1.29	1.19	1.30
250 Ft. Layer	.75	1000/5000	16/18	1.38	1.45	1.29	1.43
110 Ft. Layer	.46	1000/9000	19/20	1.27	1.32	1.15	1.29
110 Ft. Layer	.46	2000/5000	21/22	1.16	1.32	1.22	1.33

* E/R = embedment depth/structure radius.

Table 8.13 Response factor, F_{SSIM} for equipment, of spectra accelerations for surface vs. embedded foundation conditions over the specified frequency range (half-space and layered sites).

Soil Profile	Embedment* Ratio (E/R)	Characteristic+ Vs (fps)	Base Comparison	0 - 6 Hz	6 - 12 Hz	12 - 20 Hz
Half-Space	0.35	3500	2/6	1.05	1.29	1.28
	0.35	2000	3/7	1.12	1.25	1.26
	0.35	1000	4/8	1.20	1.22	1.21
	0.35	500	5/9	1.25	1.11	1.18
110 Ft. Layer	0.35	2000	10/12	1.26	1.26	1.31
	0.35	1000	11/13	1.34	1.32	1.22
71 Ft. Layer Surface vs. 110 Ft. Layer Embedded	0.35	2000	14/12	1.27	1.53	1.55
	0.35	1000	15/13	1.40	1.50	1.48

* E/R = embedment depth/equivalent structure radius.

\+ bedrock shear wave velocity = 9000 fps.

Table 8.14 Response factor, F_{SSID} for equipment, of spectra accelerations for soil srprings vs. surface & embedded foundations containment building (half-space and layered sites) horizontal responses.

Soil Profile	Embedment* Ratio (E/R)	Characteristic Vs (fps)	Case Comparison	0 - 6 Hz	6 - 12 Hz	12 - 20 Hz	ZPA
Half Space	0	2000	25/3	1.10	2.28	1.59	1.81
Half Space	0	1000	26/4	1.62	2.06	2.27	2.38
Half Space	0	500	27/5	1.91	2.34	2.55	2.68
Half Space	.46	2000	25/7	1.28	3.16	1.92	2.67
Half Space	.46	1000	26/8	2.06	2.23	2.46	3.00
Half Space	.46	500	27/9	2.55	2.63	3.11	3.56
Half Space	.75	1000	26/10	2.37	2.48	2.55	3.25
36 Ft. Layer	0	1000/5000	26/11	1.23	1.49	1.36	1.48
36 Ft. Layer	.46	1000/5000	26/12	2.22	2.20	1.15	2.30
110 Ft. Layer	0	1000/5000	26/13	1.44	1.59	1.79	1.98
110 Ft. Layer	.46	1000/5000	26/14	1.91	2.05	2.04	2.51
110 Ft. Layer	.75	1000/5000	26/15	2.22	2.41	2.48	3.02
250 Ft. Layer	0	1000/5000	26/16	1.61	1.73	2.01	2.30
250 Ft. Layer	.46	1000/5000	26/17	2.02	2.20	2.37	2.96
250 Ft. Layer	.75	1000/5000	26/18	2.41	2.47	2.55	3.24
110 Ft. Layer	0	1000/9000	26/19	1.43	1.57	1.75	1.94
110 Ft. Layer	.46	1000/9000	26/20	1.92	2.04	2.00	2.47
110 Ft. Layer	0	2000/5000	25/21	1.05	2.21	1.49	1.51
110 Ft. Layer	.46	2000/5000	25/22	1.22	2.93	1.78	2.08
110 Ft. Layer	.46	600/910 1390/9000	24a/23	1.26	4.70	2.38	2.09

* E/R = embedment depth/structure radius.

Table 8.15 Response factor, F_{SSID} for equipment, of spectra accelerations for soil springs vs. surface and embedded foundations (half-space site) over specified frequency range.

Soil Profile	Embedment* Ratio (E/R)	Characteristic Vs (fps)	Base Comparison	0 - 6 Hz	6 - 12 Hz	12 - 20 Hz
Half-Space	0	3500	16/2	1.02	1.69	1.32
Surface	0	2000	17/3	1.20	1.65	1.38
Foundation	0	1000	18/4	1.47	1.67	1.89
	0	500	19/5	1.58	1.81	1.88
Half-Space	0.35	3000	16/6	1.10	2.09	1.62
Embedded	0.35	2000	17/7	1.30	1.96	1.69
Foundation	0.35	1000	18/8	1.80	2.05	2.29
	0.35	500	19/9	1.99	2.05	2.21

* E/R = embedment depth/equivalent structure radius.

Table 8.16 Response factor, F_{SD} for equipment, of spectra accelerations for surface foundation conditions at 4%, 7%, and 10% structure damping over the specified frequency range (half-space site).

Soil Profile	Characteristic Vs (fps)	Base Case	0 - 6 Hz	6 - 12 Hz	12 - 20 Hz
4% vs. 7%	2000	3	1.00	1.05	1.09
4% vs. 7%	1000	4	1.00	1.04	1.08
4% vs. 10%	2000	3	1.00	1.09	1.17
4% vs. 10%	1000	4	1.00	1.06	1.13
7% vs. 10%	2000	3	1.00	1.04	1.06
7% vs. 10%	1000	4	1.00	1.02	1.05

Table 8.17 F_{PA}, response ratios of summary group responses vs. multiple-support time history analysis procedures.

Inertia Load	Group (1)*	Group (2)*	Group (3)*
Element Moment	4.7	3.9	6.6
Nodal Accel.	7.7	6.2	11.3

* Group (1) includes SRSS of modal combination, then HHV of directional combination (use the greater of the absolute sum of horizontal and vertical response).

Group (2) includes 10 percent modal combination, then SRSS of directional combination; double sum modal combination, then SRSS of directional combination; SRSS modal combination, then SRSS of directional combination (or vice-versa).

Group (3) includes absolute 10 percent modal combination, then SRSS of directional combination; grouping of modal combination, then SRSS of directional combination; absolute double sum of modal combination, then SRSS of directional combination; SRSS of modal combination, then 10 percent of directional combination; SRSS of modal combination, then grouping of directional combination and SRSS of modal combination, then double sum of directional combination.

Table 8.18 Adjusting Factor F_D for Damping Effects.

Piping Damping	Element Moment	Nodal Accel.
2%/1%	0.70	0.70
2%/5%	1.50	1.40

Table 8.19 Adjusting Factor F_I of envelope spectra vs. center-of-gravity and average spectra.

Spectrum Specification	Element Moment	Nodal Accel.
Ave. Spectrum	0.46	0.45
C.G. Spectrum	0.67	0.63

Table 8.20 Reference response factor, F_{PM}, for pseudostatic loads.

Method I	Method II
16.2	4.1

Table 8.21 Correlations between responses of various groups at median peak ground acceleration 0.16g.

	STR, Equipment	Piping Moment	Valve ACC
STR, Equip.	0.88	0.12	0.33
Piping Moment		0.30	0.13
Valve ACC			0.8

Table 8.22 Correlations between responses of various groups at median peak ground acceleration 0.25g.

	STR, Equipment	Piping Moment	Valve ACC
STR, Equip.	0.9	0.08	0.25
Piping Moment		0.26	0.14
Valve ACC			0.75

Table 8.23 Correlations between responses of various groups at median peak ground acceleration 0.5g.

	STR, Equipment	Piping Moment	Valve ACC
STR, Equip.	0.92	0.10	0.21
Piping Moment		0.29	0.13
Valve ACC			0.73

Table 8.24 Correlations between responses of various groups at medium peak ground acceleration 1.14g.

	STR, Equipment	Piping Moment	Valve ACC
STR, Equip.	0.93	0.10	0.25
Piping Moment		0.30	0.15
Valve ACC			0.76

9.0 FRAGILITY

9.1 Purpose

The purpose of this task is to develop the cumulative distribution functions of the seismic capacity of structures, equipment, components, and piping systems.

9.2 Assumptions and Methods

The key assumption is that the fragility functions are developed relative to local response parameters such as moment, force, peak acceleration, spectral acceleration, and so forth. The fragility functions for the SSMRP detailed and simplified methods thus have the same basic interface definition. We assume that fragility is defined in lognormal form.

There were no efforts oriented towards simplifying fragility development. This was for technical reasons:

- o Structure fragilities are dependent on the specific structure and cannot be generalized or simplified.

- o SSMRP utilized generic fragility categories for many components as described previously. We recommend using more site specific fragility categories if available.

9.3 Information Needs

Fragilities are required for all structures, equipment, components and piping systems identified in the plant logic models. Reference 14 contains some typical values. The form of the fragilities is a median value, a β_R describing random variability (typical values are 0.2 to 0.5) and a β_U describing modeling uncertainty (typical values are 0.2 to 0.7).

10.0 PLANT SEISMIC SAFETY ANALYSIS

10.1 Purpose

The purpose of this task is to perform the calculations to obtain the annual probability of core damage and release, risk in man-rems/yr. (possibly), descriptions of uncertainty on these quantities, and measures of importance and sensitivity.

10.2 Methods of Analysis

10.2.1 Basic Calculational Procedure

The calculations are performed by combining the earthquake hazard, seismic response, fragility, and plant logic models using the SEISIM computer code for specified earthquake levels. This code, designed and developed in the SSMRP, embodies a general probabilistic risk assessment methodology which can address complex systems whose failure modes are defined by many accident scenarios and for which failures of different components within the system can be highly dependent. The capacity to handle dependent failures having any degree of correlation is what sets SEISIM apart from any other existing quantitative risk assessment code.

These results are then integrated over the hazard function. This integration is performed using the SEISIM runs where each run is for a defined acceleration range in the total range for the hazard function (up to about 6 SSE for eastern sites). In the SSMRP Zion analysis, 6 integration intervals were used. If the size of these intervals is made too large, this could affect the accuracy of the integration and, more important, could prevent an accurate definition of the earthquake acceleration range that dominates the probability of core damage, release.

The calculation of radioactive release frequencies for a nuclear power plant subjected to an earthquake requires, first of all, identification of seismically induced <u>initiating events</u> which require shut-down of the reactor system. Then potential accident scenarios leading to core melt and radioactive release which could occur following an initiating event are hypothesized and characterized by <u>event trees</u>. Failure modes for the safety and auxiliary systems are identified and expressed in terms of <u>fault trees</u> for each system. Quantifications of the event and fault trees yield Boolean expressions which specify the logical relationships between failures of structures, piping, and components which could lead to core melt. These logical relationships are input in the form of minimal-cut-set expressions that define the failure modes of systems in terms of their basic events. The SEISIM code was designed to compute the probabilities of the accident sequences by computing the probabilities of the minimal cut sets that define the accident sequences.

SEISIM uses the response data and fragility functions to compute the failure probabilities of structures and components and to calculate system failure probabilities, initiating event probabilities, accident sequence probabilities, and radioactive release frequencies.

All accident sequences evaluated in SEISIM are expressed as unions of min cut sets in the form

$$P(\text{ACC SEQ}) = P(C_1C_2C_3 \text{ or } \dots \text{ or } C_iC_jC_kC_l \text{ or } \dots)$$

Each cut set is allowed to have up to 10 correlated basic events, C_i.

The cut set occurs when failure of all the components in the cut set occurs, that is, when all component responses exceed their strengths. Let

$$\{R\} = \{R_1, R_2, \dots, R_n\}$$
$$\{S\} = \{S_1, S_2, \dots, S_n\}$$

be the vectors of responses and strengths of components in a cut set of size n. Let $\{\hat{R}\}$ and $\{\hat{S}\}$ be the vectors of the corresponding medians of response and strength, respectively. Denote by the variable $\{Z\}$ the difference $\{R-S\}$. Then the probability of the cut set occurring is

$$P[Z_1 > 0, Z_2 > 0, \dots Z_n > 0] =$$

$$\int_0^\infty \int_0^\infty \cdots \int_0^\infty \int_0^\infty f_Z(Z_1, Z_2, \dots Z_n)dZ_1, dZ_2 \dots dZ_n \quad ,$$

where f_Z is the joint probability density function. Since $\{Z\}$ is assumed to be multinormal, the density is given by

$$f_Z = \frac{1}{(2\pi)^{n/2} \; \Sigma^{1/2}} \exp \left[-\frac{1}{2} \{Z - \hat{Z}\}^T \; [\Sigma]^{-1} \{Z - \hat{Z}\} \right] \quad ,$$

in which $\{\hat{Z}\} = \{\hat{R}\} - \{\hat{S}\}$ and Σ is the covariance matrix defined by

$$[\Sigma]_{ij} = COV(R_i, R_j) + COV(S_i, S_j) - COV(R_i, S_j) - COV(R_j, S_i)$$

The covariances between responses are computed in SMACS, and the covariances between strengths are user-specified.

SEISIM computes cut set probabilities given any degree of dependence among basic event failures. Each cut set probability is expressed in terms of a multinormal integral whose integrand is specified by the means, standard deviations, and correlations of the responses for the cut set basic events. Correlations between local responses are accounted for as well as correlations between component strengths (fragilities). If the local responses of two components are positively correlated, the components will tend to fail or survive together, with the frequency of both failing being higher than if their responses were uncorrelated. Correlations between local responses occur because of the nature of the seismic forcing function and types of systems being analyzed.

The initiating event probabilities are also computed from Boolean equations expressed as the union of cut sets. A hierarchy of initiating events is

defined and implemented in SEISIM, i.e., occurrence of a large LOCA implies non-occurrence of reactor pressure vessel rupture, etc.

The frequency of radioactive release in each WASH-1400 release category is defined as the sum of the frequencies of all the terminal event sequences that lead to a release in the specified category. (Category 1 is the most severe, and Category 7 is the least severe.) Terminal event sequence frequencies are defined as the products of the probabilities of the earthquake, initiating event, accident sequence, and containment failure mode. Frequency of core melt is defined as the sum of all release category frequencies.

In addition to computing the frequencies of radioactive release, SEISIM can perform sensitivity analyses. Two types of sensitivity analyses are incorporated into the SEISIM computational methodology. One is the sensitivity of outputs to changes in significant input parameters, expressed in terms of partial derivatives. The second type of sensitivity computation performed by SEISIM is called dominance analysis, in which the components, accident sequences, etc., that most influence the final results are identified.

10.2.2 Uncertainty Analysis

In seismic PRA, it is important to recognize two types of uncertainty - random uncertainty and modeling uncertainty. Random uncertainty is the inherent randomness and is irreducible; modeling uncertainty reflects incomplete knowledge of the models and calculational techniques used to quantify an event. Modeling uncertainty is reducible by improved analytical models, experiments, etc. Both types of uncertainty exist in all the key elements of seismic PRA described earlier. The separation of random and modeling uncertainty and the methodology used to propagate the modeling uncertainty through the probabilistic calculations is called "uncertainty analysis."

The goal of the uncertainty analysis is to describe the variation in the estimator of the frequency of various radioactive release categories and core-melt due to uncertainties associated with the risk analysis technique. The procedure to propagate random and modeling uncertainties is a two-loop process; the outer loop treats modeling and the inner loop treats random uncertainty. The parameters varied on the outer loop are the seismic hazard curves, the median level responses of structures and subsystems, the medians of the fragilities, and the medians of random events (human errors, corrosion, maintenance, etc.) The hazard curve shown in Figure 5.1 is an example of the kind to be used in uncertainty analysis. A number of hazard curves can be generated as desired. However, in order to adequately estimate the uncertainty interval of core-melt/mean and median, the minimum number of ten hazard curves should be used. A technique called "Latin Hypercube Experimental Design" is used to combine seismic hazard curve, seismic response, and fragility (Ref. 1).

Information and procedures for performing an uncertainty analysis are now presented.

- o The response variability due to modeling uncertainty recommended for a simplified seismic PRA is from both:

1) The SSMRP Phase II computed data

2) The variability due to using the simplified procedure.

- o The response variability due to the random uncertainty is assigned based on the SSMRP computed data as shown in Section 8.

- o Response correlations are based on the SSMRP Phase II computed data as shown in Section 7.

- o To compute the frequency of core-melt (radioactive release) using SEISIM, it is necessary to assess the probability of failure of basic events at all acceleration levels. Thus, it is necessary to estimate the response parameters (i.e., the mean vector R and variance-covariance matrix $[S]_R$) at the acceleration levels for which these parameters are not determined directly in design calculation. A method to do this was developed (Ref.1).

- o The response parameter input, along with the parameters of the fragility curve, is used in SEISIM to estimate the probability of failure for the basic events.

- o The probabilities of failure of the basic events are combined to estimate the probability of initiating events and cut sets associated with the failure of safety systems.

- o The probabilities of initiating events and cut sets are further combined in SEISIM to estimate the frequency of core-melt and radioactive release for the various release categories. These frequencies are estimated for different acceleration levels.

- o Finally, the probabilities at the different acceleration levels are integrated with the hazard curve to estimate the release frequencies.

- o The analysis is looped through the I = n design points. Thus, there are n x n estimated values for P_I[Release (n)], n =1,...7, which represents the variation in the estimated probabilities due to the uncertainties associated with the input parameters and the risk analysis methodology.

- o The n (at least 10) accumulated probabilities can then be combined to:

  1. Evaluate a median, mean of each probablilty

  2. Estimate the cumulative distribution function for the uncertainty distribution of each probability.

There are no significant differences between the SSMRP detailed and simplified approaches in this area. Point value estimates are useful for sensitivity studies, but should be used with caution as an estimator of the mean. The mean should be estimated through uncertainty analysis using recommended values for the modeling uncertainty. This will require a number of SEISIM runs (14

were done in the SSMRP study). Each run is of the order of a few minutes on a CDC 7600.

10.2.3 Importance Measures and Sensitivity Analysis

It is necessary to rank the relative importance of events in a cut set expression. The SEISIM computer code computes importance measures for components, component groups, accident sequences, response and fragility parameters. SEISIM ranks components and systems, on the basis of their importance measures. This ranking is done only for components and systems that have high-ranking importance measures.

The importance measure of components and systems calculated by SEISIM is related to the Vesely-Fussell measure. It is computed as the sum of the probabilities of cut-sets containing a component or system divided by the probability of some top event such as a release category. This is an approximation of the actual importance of independent components because the sum of cut-set probabilities is an upper bound on the probability of the union of cut-sets containing a component. It is not appropriate to include components whose failure may be dependent on other component failures in the same cut-sets.

The importance measure of response and fragility parameters computed in SEISIM is the slope of a chord obtained by dividing the change in a probability by the change in the parameter that caused the probability to change. Deviations of component and second order cut-set probabilities are calculated with respect to means and standard deviations.

For accident sequences no special importance measure computation is executed. The magnitude of an accident sequence probability compared with the total core melt probability, is a significant importance indication in itself.

Another study which is important to seismic PRA is sensitivity analysis. This analysis is to check effects of assumptions and data used in the calculations due to uncertainty and lack of knowledge on the top event. For example, it was assumed that all six service water pumps fail when the pump enclosure roof collapses in the Zion risk analysis of SSMRP. The product of this analysis is the identification of assumptions or data which, if varied, result in significant change in the core-melt frequency and the ranking of dominant components. It is also of interest to perform sensitivity studies to determine the effect of response and fragility correlations on the importance ranking of the events and on the core melt frequency.

11.0 COMPARISON OF SSMRP DETAILED AND SIMPLIFIED RESULTS

We performed a base case point estimate (best estimate) analysis using the simplified method and then compared the results with published results using the detailed method (Ref. 1).

Table 11.1 summarizes the results. The core damage probability from the simplified method is about four times that from the detailed method. The primary cause of this is that seismic responses which were used to calculate the initiating event probabilities were generally higher than the best-estimate values used in Phase II calculation. The assumption of mutual exclusion of initiating events leads to the establishment of a hierarchy of precedence for the initiating events (see Reference 1). As a result, the probability of reactor vessel rupture and large LOCA increases, and the probability of the lower hierarchial order, such as small LOCA and small-small LOCA, decreases as shown in Tables 11.2 and 11.3. Accordingly, there are more contributions to the core-melt frequency from reactor vessel rupture initiation in this simplified method than in the detailed method. The shift of contribution also affects the change of the release category frequency, and in turn, the risk of man-rem per year.

In summary, the simplified method is generally more conservative than the detailed method. However, the mean or median value is a more reliable estimator than a point estimate. The point estimate calculations as listed in Table 11.1 are useful for comparison purposes.

Table 11.1 Comparison of release frequency per year and man-rem per year between detailed and simplified analysis.

Release Category	Release Probability/Year		Man-Rem/Year	
	Detailed	Simplified	Detailed	Simplified
1	2.9E-8	1.8E-7	0.2	1.0
2	1.4E-6	4.2E-6	6.5	20.0
3	5.4E-7	8.6E-8	2.9	.5
4	0	0	0	0
5	8.3E-10	0	0	0
6	1.7E-7	9.0E-8	0	0
7	1.5E-6	8.0E-6	0	0.2
TOTALS	*3.6E-6	*1.3E-5	9.6	21.7

*These totals are equivalent to the core melt probability per year.

Table 11.2 Conditional Probabilities Per Year of Initiating Events for the Base Case (with Feed-and-Bleed and Structural Failures) -- Detailed Analysis.

Initiating Event	Earthquake acceleration level 1	2	3	4	5	6
RVR	0	7.4E-7	7.7E-3	9.7E-3	1.7E-1	5.3E-1
LLOCA	0	2.2E-5	1.8E-2	3.8E-2	1.94E-1	2.5E-1
MLOCA	1.1E-4	6.0E-5	1.1E-2	3.6E-2	5.5E-2	3.4E-2
SLOCA	2.0E-4	6.5E-4	8.7E-2	2.6E-1	3.0E-1	1.6E-1
T2	2.7E-1	8.1E-1	7.3E-1	3.7E-1	9.2E-2	2.8E-3
T1	7.3E-1	1.8E-1	6.0E-4	1.1E-6	3.5E-8	0

Table 11.3 Conditional Probabilities Per Year of Initiating Events for the Base Case (with Feed-and-Bleed and Structural Failures) -- Simplified Method.

Initiating Event	Earthquake acceleration level 1	2	3	4	5	6
RVR	3.4E-5	1.5E-3	3.3E-1	8.8E-1	1.0E+00	1.0E+00
LLOCA	3.0E-4	5.1E-3	2.2E-[illegible]	9.6E-2	2.9E-4	1.9E-5
MLOCA	9.2E-6	4.0E-5	9.3E-3	2.6E-3	1.4E-6	3.7E-8
SLOCA	4.3E-4	7.4E-4	3.9E-2	5.8E-3	1.3E-6	2.3E-8
SSLOCA	3.3E-3	1.6E-2	8.7E-3	8.9E-3	8.0E-7	5.7E-9
T2	3.3E-1	6.8E-1	3.2E-1	5.3E-3	1.0E-7	3.5E-10
T1	6.7E-1	3.0E-1	5.8E-4	2.1E-8	0 0	

12.0 LIMITATIONS

The chief limitation of using the SSMRP simplified method is the introduction of additional (relative to the SSMRP detailed method) modeling uncertainty (see Section 8.4.2.2). This additional uncertainty has the following limiting consequences relative to the results that would be obtained if the SSMRP detailed methodology were used:

- It increases the uncertainty in the estimates of the "figure of merit" (for example, the annual probability of core damage).
- It increases the mean figure of merit.
- It increases the median figure of merit--but probably less than the mean is increased.

These limitations simply state in probabilistic terms that the response calibration and associated procedures provide simplifications and economies but that they also introduce uncertainty. Similar procedures based on less detailed studies are typically used in commercial seismic PRAs. Commercial seismic PRAs thus would also have the same limitations relative to the SSMRP detailed method.

We assumed that a seismic risk analysis would be conducted as part of a combined PRA. If not, then plant logic models need to be constructed, taking guidance from this report. For Westinghouse plants, the systems need to be compared to Zion to see that the SSMRP dominant accident sequences found for Zion are appropriate. In any event, an assessment of the unique features of each plant will always be necessary.

Because this analysis is taking a simplified approach, some areas in the logic models were not carried out in order to keep the scope reasonable. These include such things as the treatment of recovery, relay chatter, and consideration of seismically induced fires and floods. While we agree that these could be significant contributors to risk, they are not included in the simplified approach explicitly in order to keep the scope of the approach manageable. If, however, treatment of recovery, for example, has been considered in the internal event analysis, then applying it in the simplified seismic approach could be considered appropriate. The only required additions to the logic models would be those factors inhibiting the recovery that were caused specifically by a seismic event. These factors could include such things as increased stress on the operator due to the seismic event or the inaccessability of some critical equipment, such as valves or pumps, due to debris falling or blocking pathways or staircases. The analyst, based on knowledge of the previous internal event analysis, could determine whether the recovery would be a major factor or not. This information could be included in the simplified seismic analysis if deemed necessary.

Under our scope of work, SSMRP did not investigate the selection of operator performance shaping factors under the conditions expected to follow seismic events. We recommend that work be undertaken to investigate and report on seismically influenced recovery, operator performance shaping factors, and the treatment of equipment recovery as well as seismically induced fires and floods. A separate project is currently studying the application of the

simplified method to a BWR, and a report if forthcoming on this study. The use of response factors to determine best-estimate responses is appropriate in seismic risk analyses. Their use for other purposes should be carefully investigated before application.

Care should be used in interpreting base case point estimates. This form of calculation takes as input the median values of the input variables and their random uncertainty, and uses one hazard curve. Modeling uncertainty is not included. Therefore, point estimate results, although useful for comparisons as in Chapter 11, do not provide a good estimate of the median or mean core melt probability or release probabilities. The point estimate results are usually lower because modeling uncertainty is not included. To come up with good estimates of the mean or median core melt probability, a number of runs (14 in SSMRP Phase II analysis) need to be made sampling from different hazard curves and incorporating modeling uncertainty (see Section 8.4.2.2 for recommended values). Fortunately, such calculations take only a short time using SEISIM on a CDC 7600.

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APPENDIX A

SUMMARY OF RESPONSE CALIBRATION STUDIES

SEISMIC RESPONSE DETERMINATION OF STRUCTURES AND SUBSYSTEMS

A.1 Purpose

To perform a seismic risk analysis, seismic responses are required for all structures and components contained in the accident scenarios (typically, fault trees and event trees). Examples of items for which responses may be required are floor- or line-mounted valves, pipes, pumps, diesel generator, electrical components such as circuit breakers and electrical busses, structures, structural elements, and so forth. A seismic risk analysis considers not simply one or two levels of earthquake (such as operating basis earthquake (OBE) and safe shutdown earthquake (SSE)) but the range of possible earthquakes at the site. Three aspects of seismic response, for each earthquake level, are necessary for seismic risk analysis: best-estimate or median level response, variability of response, and correlation of responses.

Median Level Response

The best-estimate or median level seismic response given an earthquake occurrence is required. In general, best-estimate responses differ from the design values, even for the design level earthquake, because, in the latter case, design analysis procedures, parameter selection, and qualification procedures are conservatively biased. An additional consideration in analyzing for the range of earthquake of the seismic hazard curve is the change in properties of the soil/structure/piping systems which occur as higher excitations lead to lower soil shear moduli, lower structure frequencies, and higher soil and structure damping characteristics. Such changes need to be taken into account when determining median level response.

Variability of Response

Variability in seismic response resulting from variations in the earthquake excitation, the physical properties of the soil/structure/piping system, and our ability to model them must be estimated.

Correlation of Responses

Correlation of responses, i.e., the tendency for pairs of responses to have simultaneously high or low values, results from two sources--the level of the earthquake and the dynamic characteristics of the system. The level of the earthquake affects correlation since a large earthquake (large peak acceleration) may cause all responses to be large, whereas a small earthquake produces the opposite effect. The second source of correlation is due to system response itself. For example, floors within a structure may all experience high values of response simultaneously due to the dynamic characteristics of the structure itself. Hence, equipment supported on these floors may simultaneously have high response. The importance of response correlation on probabilities of system failure, core melt, and radioactive release depends on correlations between fragilities and the functional characteristics of the systems.

For SSMRP seismic risk analysis methodology to date, these three aspects of seismic response were determined calculationally using the computer program SMACS (Seismic Methodology Analysis Chain with Statics). Considerable resources are required to perform the SMACS analysis from start to finish. The major effort is associated with developing models of soil-structure interaction (SSI), major structures, and piping systems and then coordinating the models for input to SMACS. To reduce the effort in this aspect of the seismic PRA for the SSMRP Simplified Methodology, we propose to use to the extent possible A-E and vendor-developed seismic responses and/or models. In addition, we use SSMRP Phase I and II information to define variability and correlation of the seismic responses.

A.2 Approach and Assumptions

The three aspects of seismic response (for each earthquake level) necessary for seismic risk analysis are median-level or best-estimate response, variability of response, and correlation of responses. We discuss each of these in the context of a simplified methodology.

A.2.1 Median-Level Responses

An overview of the procedure is: (i) to estimate median-level or best-estimate responses for the design level earthquake based on the A-E and vendor generated responses and to apply response factors developed from generic studies or re-analysis of the plant of interest; and (ii) to extrapolate these best-estimate values to higher excitation levels using SSMRP Phase II regression models and other appropriate information.

A major area of simplification is in determining best-estimate seismic responses. The basic premise for simplifying this aspect of the seismic PRA is to use the A-E and vendor generated seismic responses to the extent possible. At least two approaches exist for using A-E and vendor generated seismic responses in the seismic PRA. Both approaches are based on calibrating the data--estimating median level responses (wall forces and moments, in-structure response spectra, in-structure accelerations, etc.)--as a function of earthquake level, removing conservatism or non-conservatism from the A-E procedure. The two approaches are:

- o Develop a set of response factors for response quantities of interest based on comparing best-estimate results with results of a design procedure.
- o Perform limited reanalysis of the nuclear power plant for which the seismic PRA is being performed. Generate seismic responses to be used in the systems analysis and to calibrate the A-E generated data. The reanalysis would concentrate on the structures with minimal or no effort devoted to reanalyzing piping systems. In-structure response spectra could be generated to calibrate conservatism in input to piping systems. The cost of such a reanalysis could be significantly reduced by using A-E generated structural models if they were adequate and available.

The second approach, a limited reanalysis of the nuclear power plant for which the seismic PRA is being performed, will not be discussed in detail

here. However, the reanalysis should be performed with best-estimate methodology and parameter values. Uncertainties in seismic input and soil/structure/subsystem properties should be taken into account. Response factors could then be obtained by comparing best- estimate responses with the design values.

Response factors relate design calculated responses to best-estimate values.

$$R_{BE} = R_D/F_R$$

where the R of interest may be a force or moment in a structural element, peak acceleration of a floor, acceleration of a valve, moment in a piping system, etc. A number of factors will typically contribute to F_R. The number of factors and their relative importance depend on the response component of interest and where it lies relative to the seismic methodology chain. There is a compounding of effects. The basic approach is to identify aspects of conservatism or non-conservatism in the seismic analysis process, quantify them, and accumulate their effects. Several assumptions are made in this approach:

- o The seismic response data available from the plant seismic design data are the medians of possible design values, all of which satisfy the design criteria.
- o Best estimate responses are derivable from the design values and design analysis. For example, load distributions and the overall behavior characteristics of structures and piping systems (such as frequencies and mode shapes) are reasonably represented by the design analyses.
- o The median of the overall response factor is derivable as the product of the medians of response factors for the separate effects.

A.2.2 Variability of Response

Variability of response will be specified according to the SSMRP Phase I and II results. Components of random and modeling uncertainty are recommended. Random uncertainty denotes the variability in response due to variations in the earthquake itself and its induced variability in the soil/structure/subsystems properties. The SSMRP results will be supplemented by recorded field data, in particular Chiba Field Station data.

A.2.3 Correlation of Responses

Correlation of responses will be specified according to the SSMRP Phase I and II results. Calculational results of SMACS form the basis for the recommended values.

A.3 Information Needs

Site Soil Conditions

Geologic data on the site
Soil configuration

Boring information
Ground water data
Static and dynamic soil properties
- o Laboratory tests
- o In-situ field test results
- o Scatter data

Structures

Results of dynamic seismic analysis
Dynamic design models
Location of major equipment
Structural drawings
Slab and wall geometries and reinforcement schedules
Masonry wall specifications
Steel detailing drawings
Beam/column schedules
Containment wall geometry
Concrete cylinder test results
Re-bar test results
Field-erected tank drawings (vendor) and civil drawing showing foundation, ring girder, and anchor bolt details

Piping

Design analysis by NSSS and A-E
Piping area drawings
Piping stress isometrics
Piping fabrication isometrics
Pipe support/hanger detail drawings
Piping class sheets
Valves - WT/CG data for both valve and operators

A.4 Recommendations for Seismic Response Determination in a Simplified Seismic PRA

A.4.1 Median Level Response

A.4.1.1 Building Response

The process of calibrating structure response begins first by identifying those aspects of seismic analysis which have a major influence on response prediction. We have identified three major areas: seismic input, soil-structure interaction (SSI) and structure response analysis procedures, and energy dissipation of the soil-structure system.

- o Seismic input - In most instances, specification of the design ground motion is conservative; many times in the form of response spectra targeted to the mean plus one standard deviation values of recorded earthquake data conditional on a specified peak ground acceleration. Best-estimate response, that is, median-level response, conditional on an earthquake's occurring and described by a specified value of the

seismic hazard curve parameter is required. Hence, conservatism or, potentially, non-conservatism in the design seismic input must be taken into account.

- o SSI and structure response analysis procedures - Models of SSI and structures, and their parameter values are the main areas of interest. In terms of SSI, two major elements are treatment of embedment (wave scattering effects and increased energy dissipation) and treatment of the radiation damping aspects of the phenomenon. Structure models are an additional area of activity.

- o Energy dissipation of the soil-structure system - In addition to the aforementioned treatment of radiation damping, the effect of increased structure damping on response is of particular interest. Before failure of a structural member occurs, energy dissipation increases and its effect on structure response needs to be assessed.

A.4.1.1.1 Seismic Input

The F_{SI} response factor is explained in Section 8.4.1.1.

A.4.1.1.2 SSI and Structure Response

Building response factors, i.e., factors which relate best estimate or median level responses to responses calculated by selected design procedures for the design level earthquake, are presented here. The basic phenomenon of interest was soil-structure interaction (SSI) effect because significant simplifications are frequently introduced in its treatment. Also, the behavior of the structure is intimately coupled with the soil when founded on a soil site and the soil and structure must be treated as a system. Two structures were the subjects of the response factors: the Zion containment building and a typical shear wall structure derived from the Zion AFT complex. In-structure forces, peak accelerations, and response spectra were response quantities for which response factors were derived.

The basic approach to deriving response factors was to identify physical and analysis scenarios to be considered, to perform SSI analyses for the specified site and structure by best-estimate and design procedures, and to compare the responses for the derivation of response factors. All analyses assumed acceleration time histories as the seismic input. Multiple analyses were performed with response factors defined as the mean ratio of responses for the series of earthquakes. These analyses were necessary to minimize the potential spurious results due to the specific characteristics of a single- time history. Two sets of earthquake time histories were considered for the containment building--one represented a site specific definition of the input motion; the second was an ensemble of acceleration time histories targeted to NRC RG 1.60. Selected results for both are presented here. Only the latter data set was used in the analysis of the typical shear wall structure.

A.4.1.1.2.1 Containment Building

Physical Structure

The structure of interest is the Zion containment building which comprises two essentially independent structures--a containment shell and a concrete internal structure. The containment shell is a cylindrical prestressed concrete structure, 147 feet in diameter, lined with a steel liner. The cylinder is topped with an elliptical dome, a total of 211 feet above the foundation.

The internal structure denotes the nuclear system steam supply's (NSSS) reinforced concrete support structure and the NSSS itself (including the reactor pressure vessel, steam generators, piping, and coolant pumps). The internal structure extends about 50 feet above the foundation mat to the operating floor. The containment shell and internal structure interact only through the foundation, which is 157 feet in diameter and about 13 feet thick.

Figure A.1 shows the containment building.

Structure Model

The containment shell was modeled using a series of vertical beam elements, with shear and bending characteristics appropriate for a circular cylindrical shell. Masses and rotary inertias were lumped at node points. Inertias affecting both bending and torsional response of the shell were included.

The reinforced concrete internal structure was represented by a three-dimensional finite element model consisting of plate and beam elements. It included a simplified model of the NSSS. The structure model contained about 3800 structural degrees-of-freedom.

Data Set of Response Quantities

The response quantities selected for the development of response factors were in-structure accelerations, in-structure response spectra, and force quantities (moments, shears, and axial forces). For the containment shell, Fig. A.2 displays the response quantities. Those denoted by "F" in the figure are moments, shears, and axial forces at the elevation indicated. Similarly, those denoted by "A" denote accelerations. Peak values over the time histories were used for calibration purposes. At the base, all degrees-of-freedom were included. From acceleration time histories, at the node points of interest, in-structure response spectra were calculated.

Figure A.3 shows the locations in the internal structure where response quantities were calculated. For this structure, the primary quantities of interest are shear stresses and moments in wall elements and moments in floor elements. The elements were selected by subjecting the structural model to a static lateral force and picking those elements that had high stresses and were important to the integrity of the structural system. The acceleration quantities of interest associated with the internal structure are those which interface with the NSSS. The points on the NSSS where acceleration response spectra were calculated are shown in Fig. A.4. Those nodal points and elements selected in this study are listed in Table A.1.

Physical and Analysis Scenarios

Response factors were developed based on a number of physical and analysis scenarios. Table A.2 itemizes 27 scenarios which form the basis for the results presented here. Cases 1-23 represent best estimate scenarios provided the analysis scenario matches the physical characteristics of the soil, structure, and foundation. Cases 24-27 represent scenarios with simplifications in the SSI analysis procedure. Soil profiles denoted half-space are uniform half-spaces with soil properties of density of 130 pcf, Poisson's ratio of 0.4, and soil material damping of 5% of critical. Differences in stiffness characteristics are defined by differing shear wave velocities (Vs) of 500 fps, 1000 fps, 2000 fps, and 3500 fps. Soil profiles denoted by a layer thickness are shallow soil layers overlying a stiff bedrock. A single soil layer over bedrock was considered in all cases except case 23 which corresponds to the Zion nuclear power plant site and is composed of three layers over bedrock. In general, the soil layer properties were a density of 130 pcf, Poisson's ratio of 0.4, and soil material damping of 5% of critical. Soil layer stiffnesses were defined by shear wave velocities of 1000 fps, and 2000 fps. The underlying bedrock was typically assumed to have a shear wave velocity of 5000 fps, a density of 150 pcf, Poisson's ratio of 0.33, and material damping of 2% to 3% of critical. A few selected cases (cases 19, 20, and 23) assumed a bedrock shear wave velocity of 9000 fps.

Two embedment depths were considered for a number of cases--36 feet and 59 feet--which correspond to ratios of embedment depth to radius of 0.46 and 0.75, respectively.

Cases 24-27 represent scenarios with simplications in the SSI analysis procedure. These cases are denoted half-spaces; however, the procedure is typically applied as an approximation to numerous soil profiles other than a uniform half-space, i.e. assuming equivalent half-space soil properties. This simplification represents the behavior of the soil by frequency-independent soil springs. Cases 24a, 25-27 include six components for the soil springs (three translational and three rotational). Case 24b simulates a further simplification where only rocking and torsional springs are included.

Bases of Developing Response Factors

Response factors presented here are response ratios of response and coefficients of variation (COVs) for peak accelerations, peak forces, and in-structure response spectra values.

Peak acceleration and peak forces are scalar quantities which are amenable to comparison. In-structure response spectra are significantly more difficult to interpret in the context of response factors. An attempt to do so was made by separating the frequency range of the spectra into three -- 0 to to 6 Hz., 6 to 12 Hz., and 12 to 20 Hz. Response factors were developed for each frequency range, and means and COVs calculated. The utility of these response factors is limited. A more appropriate procedure is to estimate or calculate median spectra based on the design procedure and a best-estimate calculational procedure.

Surface versus Embedded Foundation

Comparisons of SSI responses of the containment building founded on the surface and embedded in the soil at depths of 36 feet (E/R = 0.46) and 59 feet (E/R = 0.75) for various site conditions are shown in Table A.3 and A.4. Table A.3 shows the response ratios of forces and peak accelerations. Table A.4a shows the mean ratios of spectral accelerations of horizontal responses while Table A.4b shows the ratio of vertical responses. For the 36-foot embedment cases, in the half-space sites, the ratios do not vary significantly as the shear wave velocity varies from 500 fps to 3500 fps. The mean response of the embedded structure is about 80% of that of the surface founded structure. As the embedment increases to 59 feet, the response reduces to 73%. For the cases of 36-foot embedment in the layered sites, the response ratios of SSI response increase from 1.26 for the site of 250-foot soil deposit to 1.30 for the site of 110-foot soil layer. The mean ratio increases to 1.87 for the site of 36-foot soil deposit. The unproportional increase of this mean ratio is due to the fact that the structural foundation is directly founded on the top of the rigid bedrock. Comparisons of the cases of deeper embedment (E/R = 0.75) in three different site conditions show that the response ratios are very close for a half-space site and 250-foot layer site (1.37 vs. 1.40). A larger mean ratio (1.55) was found for 110-foot layer site. The increase of this ratio is due to the effect of a shallow soil layer. The result comparison also shows that the soil layer effect on the SSI response of a typical PWR building may be negligible if the soil layer is more than 250 feet (i.e. the ratio of soil layer over the reactor radius is more than 3.2). The effect of increasing bedrock rigidity was examined as shown on the case comparison of 19/20. The response ratios (as well as the COVs) of forces and peak accelerations are almost identical to the case comparison of 13/14. The effect of increasing soil rigidity of the soil layer is not important as shown in the table between case comparison of 21/22 and 13/14.

Tables A.4a and A.4b contain the mean ratio of spectral accelerations for horizontal and vertical responses. In the table the mean zpa ratios were approximately taken from the mean of 24 spectral accelerations between 32 Hz and 100 Hz. As an attempt to derive response factors over a different frequency range for demonstration purposes, we arbitrarily break the span of frequency into three ranges: 0 to 6 Hz, 6 to 12 Hz and 12 to 20Hz. The variation of spectral ratio depends on the shape of the spectral acceleration of the comparison pairs. And the spectral acceleration depends on the site layer configuration, site stiffness, and the structure embedment. Thus, the peak and valley of spectral ratio varies from case to case, and the selection of frequency ranges for response factors should be case dependent. A typical mean spectral ratio and its mean plus/minus standard deviation is shown in Fig. A.5a for comparison case 13/14. The envelope of maximum and minimum ratio as well as mean ratio is shown in Fig. 9.5b. In general the shape of spectral ratios in the same category of site conditions are similar. In the category of layered half-space site, more peaks and valleys appear in the high frequency range due to the soil layer effect. A larger COV also appears for the layered site condition.

Fixed-Base versus Surface and Embedded Foundation

Many U.S. older nuclear power plants were designed by fixed-base assumption. As the SSI effects were not included in the analysis, the design

values become unrealistic and overconservative in most cases. It is important to derive a set of response factors for the ranges of typical embedments and site conditions.

Table A.5 shows the mean ratio of forces and peak accelerations for the case of fixed base versus the cases for a variety of surface and embedded foundations situated at half-space and layer sites. The ratios of peak acceleration for the fixed base against surface foundations situated on half-space sites range from 1.20 for stiff sites (Vs = 3500 fps) to 2.54 for soft sites (Vs = 500 fps). When the structure embedded to 36 feet (E/R = 0.46), the response generally decreases and the ratio increases to 1.38 for stiff sites and to 3.11 for soft sites. The ratio increases from 2.21 for embedment of 36 feet to 2.37 for embedment of 59 feet for half-space site with shear wave velocity of 1000 fps. The COVs range from 25.7% to 77.5%. The ratios of forces are in the same order of accelerations, and COVs vary from 18.5% to 38.6%. The SSI responses of the fixed-base case were also compared to the response of several cases of surface and foundation located at layered sites (Vs = 1000 fps) overlying bedrock of shear wave velocity 5000 fps. Comparing the mean ratio to the half-space cases (1/4 and 1/8), we found that the response ratios are consistently decreased as the thickness of soil layer decreases. Very close response ratios are found between the cases of half-space sites and the cases of 250-foot soil sites. Larger deviations are found on the thin soil sites. The effect of shallow soil sites is important when the response factors are applied to the fixed-base case. For the cases of surface foundation, the response ratios of peak acceleration range from 1.19 per 36-foot soil layer to 1.78 per 250-foot soil layer. For the case of embedded foundation (E/R = 0.46), the response ratios of peak acceleration range from 1.61 for 36-foot layered site to 2.17 for 250-foot layered site. As the embedment increases to 59 feet (E/R = 0.75), the response ratios of peak acceleration are 2.26 for a 110-foot layered site and 2.37 for a 250-foot layered site. The effect of increasing bedrock shear wave velocity to 9000 fps is not so important as the comparison shown between the case of 1/20 and the case of 1/14. On the other hand, when the shear wave velocity is increased from 1000 fps to 2000 fps, the mean ratio of peak acceleration decreases from 1.54 to 1.22 for the case of surface embedded foundation (E/R = 0.46). The effect of multiple-layered system for the embedded foundation is shown on the case comparison of 1/23. The response ratios of forces are in the same order of magnitude as peak acceleration. The COVs of force ratio are smaller than those of peak accelerations as shown in Table A.5.

Table A.5a compares the in-structure response spectra for horizontal response with Table A.5b, for vertical response. As stated earlier, the spectral accelerations vary from location to location in a structure, and the ratios are greatly dependent on the frequency.

Response Ratios of SSI Responses for Soil Springs vs. Surface and Embedded Foundations

The old design procedure, frequency independent soil springs method, is represented by cases 25, 26 and 27 in which the soil spring constants were developed on the basis of uniform half-space having shear wave velocities of 2000, 1000 and 500 fps, respectively. The results of each of these cases were compared to those of the corresponding cases of best estimate for surface and embedded foundations. They were also compared to those of multiple layer sites, cases 11 to 22, as shown in Table A.2.

Table A.7 shows the response ratios of peak accelerations for soil springs against the best estimate of surface and embedded foundations on different site conditions. The first data set is for uniform half-space with three different shear wave velocities. The response ratios of peak acceleration range between 1.93 and 2.78 with the COVs around 40% for the cases of surface foundations. The ratios vary from 2.33 to 3.46 with the COVs about 35% for the cases of embedded foundation (E/R = 0.46). The 110-foot layer site ratio increases from 3.13 for the case of E/R = 0.46 to 3.31 for the case of E/R = 0.75.

The second data set is for the layered sites. Most data contain the response ratios of peak acceleration for the cases of surface and embedded foundation for the sites of various thickness of soil layer with shear wave velocity of 1000 fps. For the cases of surface foundation, the mean ratio varies from 1.56 for a 36-foot layer site to 251 for a 250-foot layer site. The mean ratio increases from 2.54 to 3.08 for embedded foundation of E/R = 0.46. In two cases of deeper embedment (E/R = 0.75), the ratios increase to 3.16 for a 110-foot layer site and 3.27 for a 250-foot layer site. The effect of changing the rigidities of soil layer and bedrock is shown on the case comparisons of 26/19, 26/20, 25/21 and 25/22. The mean ratio will be reduced about 20% if the shear wave velocity of soil layer increases from 1000 fps to 2000 fps. The effect of increasing shear wave velocity of bedrock to 9000 fps is not important on the overall mean ratio of peak acceleration. The COVs mean acceleration ratios vary between 32% to 40% in this data set. The effect of a thin layer (36-foot soil) on response ratio will be discussed with special cases in the next section.

Table A.8a contains comparable data for in-structure spectral acceleration for horizontal responses. Table A.8b shows the mean ratio of spectral acceleration for vertical responses.

<u>Effect of Thin Soil Layer on SSI Responses</u>

There are about 30 U.S. nuclear power plants built on the rock sites or thin soil sites where the soil deposit is less than 40 feet thick. In most rock sites the surface material consists of thin layers of loose soil deposit and weathered rock. Foundations of nuclear power plants may be founded directly on the top of bedrock without backfill or with compacted backfill. To investigate the effect of the thin layer soil deposits on SSI response, we compared the results of the following cases.

1. Case 1a: Containment building is directly founded on the surface of a bedrock with a shear wave velocity of 5000 fps.

2. Case 11a: Containment building is founded on the surface of a 36-foot soil deposit (with a shear wave velocity of 1000 fps) overlying the bedrock with a shear wave velocity of 5000 fps. We assumed that a rock outcrop adjacent to the structure at the same level as the soil deposits and the control motions were specified at the rock outcrop.

3. Case 12a: Containment is founded on the surface of the bedrock (Vs = 5000 fps) and a 36-foot thick soil layer was backfilled around the structure. The shear velocity of the soil is 1000 fps. The adjacent rock is approximately at the same level as the backfill soil. The control motions were specified at the rock outcrop.

4. Case 11: This case is just like Case 11a, except that there is no nearby rock outcrop and the control motions are specified at the soil surface.

5. Case 12a: This case is similar to Case 12a, except that there is no nearby rock outcrop, and the control motions are specified at the ground surface.

For Cases 11a and 12a where the control motions are specified at the nearby rock outcrop, the free-field response should be considered as a complicated three-dimensional problem. However, as so many uncertainties are involved in defining the soil-rock interface and soil-rock dynamic properties, the free-field responses are usually calculated by a simple 1-D wave propagation theory for practical engineering purpose. In SSI response analysis, the calculated free-field motions at the surface of the soil deposit were used to replace the original control motion. It is clear that this is only an approximation of accounting for local soil effect on SSI response of a structure.

Using Case 1a as the numerator and comparing to the other four cases as denominators, Table A.9 shows the overall response ratios of forces and peak acceleration for the containment building subjected to the shaking of ten times histories as described before. Table A.10a shows the response ratios of spectral acceleration of horizontal motions; Table A.10b shows the comparison of vertical responses. The case comparison for the case of fixed base and the case of surface foundation on rock site at shear wave velocity of 5000 fps are shown in the tables for reference.

From Table A.9, we can see that the effect of a 36-foot thin soil layer on the SSI response is important. Higher responses are observed if the plant are built on top of the thin soil layer adjacent to a rock outcrop. The mean peak acceleration ratio of 0.72 of Case 1a/Case 11a implies that the overall response of Case 11a is approximately 1/0.72 times (or 1.39 times) that of Case 1a. However, if there is no nearby rock outcrop at the site, the effect of this thin layer is not important as the table shows the response ratio changes from 0.72 to 0.97. The effect of compacted backfill is significant for both cases (Case 12 and 12a). The peak acceleration ratio of 1.18 between Case 1a and 12a indicates that the mean response can be reduced 15% due to the compacted backfill in Case 12a. The reduction can go to 31% for Case 12. In a real situation, if a power plant is being built on a stiff rock having shear wave velocity of 5000 fps, one might consider the rigid base assumption and ignore the interaction effect in the design process. However, based on the comparisons between Case 1 and Case 1a, it should be noted that the mean response of the containment building founded on stiff rock (Vs = 5000 fps) is about 10% lower than that of the same structure designed for a fixed base. The effect of rock-structure interaction is slight.

The mean ratio of forces are in the same range of peak acceleration for both surface and embedded foundation. The coefficients of variation (COVs) of the peak acceleration ratio are approximately 50% for the cases of surface foundation. The COVs drop to 7% to 16% for the cases of embedded foundation. The COVs of the force ratio are about 30% to 35% for the cases of surface foundation and are about 8% to 21% for the cases of embedded foundation.

The response ratios of spectral acceleration for the entire frequency range for both horizontal and vertical response are shown in Table A.10a and A.10b, respectively. For the purpose of demonstration, the means and COVs of spectral ratio were shown in three frequency ranges. It can be seen from Table A.10a and A.10b, the spectral ratios for the cases of surface foundation are much smaller than those of the cases of embedded foundation for both low frequency range (<6 Hz) and high frequency range (>32 Hz). The COVs of spectral ratio for the cases of surface foundation are much larger than those of the cases of embedded foundations.

Effects of Seismic Input Motion on Response Ratios of Peak Acceleration and Forces

To investigate the effect of different input motions on the calculated response ratios of peak accelerations and forces, the first nine cases shown in Table A.2 were analyzed by two different sets of seismic input: RG 160 type and site specific type as described previously. Table A.11 shows response ratios of peak accelerations and forces for surface versus embedment foundation (E/R = 0.46) of the structure situated at half-space sites with four different shear wave velocities. It can be seen that the difference in response ratios resulting from different seismic input is not significant. The maximum differences are 5% in peak acceleration ratio and 6.5% in force ratio for the case comparison of 5 to 9. The case comparison of 1 to 2 between the cases of fixed base and the surface foundation on half-space site with shear wave velocity of 3500 fps, yields the response ratios of peak acceleration 1.20 for R.G. 160 versus 1.28 for site specific input motion. The response ratios of forces for this case are 1.16 for R.G. 160 versus 1.24 for site specific input motion. Similarly, the effect of using different seismic input on the mean ratios of peak acceleration and forces between the fixed base case and other cases is not very important.

A.4.1.1.1.2.2 Shear Wall Structure

Physical Structure

The structure of interest is a part of the Zion auxiliary/fuel-handling/turbine building (AFT) complex of the Zion nuclear power plant. Only the auxiliary, fuel-handling, and diesel generator buildings were modelled. These structures are a connected group of heavy, shear-wall buildings constructed on reinforced concrete typical of nuclear power plant structures. In the plan view, the buildings form a T-shaped unit with a plane of symmetry along the east-west axis, leading to uncoupled horizontal response along the plane of symmetry and coupled horizontal and torsional response in the perpendicular plane. Fig. A.6a shows a plan view of the Zion nuclear power plant; the portion of the AFT complex studied here is shown shaded. Fig. A.6b shows a cross-section through the structure.

The auxiliary building rests on a 5-foot-thick, soil-supported, reinforced concrete mat at an elevation of 542 feet. The reinforced concrete foundation walls are laterally supported by concrete floor slabs at elevations of 560, 579, and 592 feet. The diesel generator rooms at the north and south ends of the auxiliary building rest on walls extending to a strip footing at an elevation of 557 feet. Above grade, the lateral force resisting system is a combination of braced structural steel frames and concrete slabs and walls. The floors at

elevations 617, 630, and 642 feet are reinforced concrete slabs supported by horizontal braced steel framing. Between the auxiliary and trubine buildings is a common shear wall. It consists of vertical, braced steel frames encased in reinforced concrete. The total mass and stiffness of this wall was included in the model. The fuel-handling building also rests on a reinforced concrete mat foundation. Reinforced concrete walls rise to the roof on the three exterior sides. There are partial floor slabs at elevations 602 and 617 feet. The roof is corrugated metal decking with a concrete slab supported by steel framing.

Structure Model

The structure complex was represented by a finite element model which represents a best-estimate idealization of the structure. Thin plate and shell elements were chosen to define the stiffness of both the concrete shear wall and floor slabs. The additional stiffness due to the steel bracing frames in the common auxiliary turbine building wall was ignored. In discretizing the structure model, single elements were used to model the height of the shear walls between adjacent slabs. Whenever possible, element dimensions were chosen to give an aspect ratio close to unity. This discretization approach led to the use of 841 nodes and 1251 plate/shell elements. For the fixed-base model, there was a total of 3948 dynamic degrees-of-freedom. Two views of the model are shown in Figs. A.7 and A.8. The mass of selected equipment was included in the floor slab densities.

Data Set of Response Quantities

In-structure accelerations, in-structure response spectra, and wall forces and moments were the dynamic response quantities for which response factors were developed. Node point and element locations were selected to represent overall structural behavior and local response. Peak accelerations and in-structure response spectra were calculated at selected nodes described in Table A.12a and shown in Fig. A.9. In Fig. A.9, half of the structure has been removed to better illustrate node point locations. Wall forces and overturning moments were calculated at the roof and floor slab elevations in the major load carrying walls. Responses in the minor walls were omitted. To describe shear wall behavior, sections were taken in the selected walls, and axial load, in-plane shear factor, and overturning moment were calcuated. The wall forces and moments are a summation of the elemental forces over the wall length at the section elevation. The wall sections included in this study are tabulated in Table A.12b. The wall locations in the model are illustrated in Fig. A.10. The walls studied included the common auxiliary turbine building wall (X = 180.0), a major north-south auxiliary building wall (X = 248.0), an end wall in the fuel-handling building (X = 467.0), a major auxiliary building east-west wall (Y = 43.5), the common diesel generator auxiliary building wall (Y = 133.0), a diesel generator building wall (Y = 238.0) and two sections in major north-south walls in the auxiliary building (X = 311.0, and X = 340.0). The data base for the response factors consisted of:

o Three components of peak acceleration (two-horizontal and the vertical) at seven node points and six components of peak acceleration (three translational and three rotational) on the foundation--a total of twenty-seven.

- o Response spectra (2% damping) at seven nodes (three components) and on the foundation (six components)--a total of twenty-seven.

- o Axial load, in-plane shear, and overturning moment in eight walls- a total of thirty-five sections, and six foundation responses (three forces and three moments)--a total of 111 responses.

Physical and Analysis Scenarios

Response factors were developed based on a number of physical and analysis scenarios. Table A.13 itemizes 19 scenarios which form the basis for the results presented here. Cases 1-15 represent best-estimate scenarios provided the analysis sceanario matches the physical characteristics of the soil, structure, and foundation. Cases 16-19 represent scenarios with simplifications in the SSI analysis procedure. Soil profiles denoted half-space are uniform half-spaces with soil properties the density of 130 pcf, Possion's ratio of 0.4, and soil material damping of 5% of critical. Differences in stiffness characteristics are defined by differing shear wave velocities, V_S, of 500 fps, 1000 fps, 2000 fps, andl 3500 fps. Soil profiles denoted by a layer thickness are a single soil layer overlying a stiff bedrock. The soil layer properties are a density of 130 psf, Poisson's ratio of 0.4, and soil material damping of 5% of critical. Two soil layer stiffnesses were considered and defined by shear wave velocities of 1000 fps and 2000 fps. The material properties of the underlying bedrock in all cases were a shear wave velocity of 9000 fps, density of 130 pfs, Poisson's ratio of 0.27, and material damping of 5% of critical. Those cases whose foundation condition is denoted embedded represent an average embedment depth of 39 feet, which corresponds to a ratio of embedment depth to equivalent radius of 0.35. Cases 16-19 represent scenarios with simplications in the SSI analysis procedure. These latter cases were denoted half-spaces; however, the procedure is typically applied as an approximation to numerous soil profiles other than a uniform half-space, i.e. assuming equivalent half-space soil properties. This simplification represents the behavior of the soil by frequency independent soil springs generated for circular foundations judged to be equivalent to the actual foundation condition. In cases 16-19, only constant modal damping of 4% of critical was assumed for all modes--no attempt to include radiation damping was made.

Bases of Developing Response Factors

Response factors presented here are response ratios of response and coefficients of variation (COVs) for peak accelerations, peak forces, and in-structure response spectra values. In the categories of peak accelerations and peak forces, preliminary evaluations of the data, grouping like components, like directions, like locations, etc. led to no discernible differences in the response factors based on these groupings vs. broader groups such as all peak accelerations. Hence, the data presented here is for broader groups, namely three--peak accelerations, peak forces, and in-structure response spectra. As discussed above, multiple earthquakes were considered. In all results presented here, ten earthquakes were analyzed for each case.

Peak accelerations and peak forces are scalar quantities which are amenable to comparison. In-structure response spectra are significantly more difficult to interpret in the context of response factors. An attempt to do so was made by separating the frequency range of the spectra into three--0 to 6 Hz., 6 to 12

Hz, and 12 to 20 Hz. Response factors were developed for each frequency range and means and COVs calculated. The utility of tnese response factors is limited. A more appropriate procedure is to estimate or calculate median spectra bases on the design procedure and a best estimate calculational procedure.

Surface vs. Embedded Foundation

Tables A.14 and A.15 present response factors that relate structure responses of the typical shear wall structure founded on the surface of the soil vs. those embedded an average depth of 39 feet ($e/r = 0.35$). Table A.14 presents data for peak accelerations and peak forces. The first set of data is for uniform half-spaces of varying shear wave velocities. The effect of embedment ranges from 1.17 to 1.31. Here, and for all other ratios in Table A.14, the surface-founded structure produced higher response. The second set of data is for a 110-foot layer of soil overlying a stiff bedrock and for two soil layer stiffnesses. The effect of embedment ranges from 1.31 to 1.38 for this case. The third set of data compares a surface-founded structure on a 71-foot soil layer to a structure with an embedded foundation in a 110-foot soil layer. This case corresponds to ignoring embedment in the analysis process and artifically treating the structure as being surface--founded at the 71-foot level. For this last case, response ratios of peak accelerations and peak forces range from 1.57 to 1.75.

Table A.15 contains comparable data for in-structure response spectra. A more informative examination of in-structure spectra is contained in Fig. A.11 where the identical comparisons of Table A.15 are made for node 256 at the top of the structure. The effect of embedment is, in general, to increase the predominant frequency and lower the amplitude of response as can be seen from Fig. A.11.

Fixed-Based vs. Surface and Embedded Foundations

For many older nuclear power plants, no SSI analysis was performed; a fixed-based analysis yielded accelerations and forces for design. In general, this simplification introduces significant conservatism in the calculated values of peak accelerations and forces. Table A.16 quantifies this conservatism for Cases 2-15. The first set of data in Table A.16 is for surface-founded structures on a uniform half-space of varying stiffness characteristics. Response ratios vary from 1.07 to 2.14. One site of interest is the uniform half-space of $V_S = 3500$ fps which is typically considered to be a rock site and in many design codes, performing a fixed-base analysis is permitted. Even in this case, accounting for the supporting media reduces design loads by a mean of $0.81 = (1/1.23)$. Peak accelerations are reduced less. The second data set presents comparable data for the structure with an embedded foundation on a uniform half-space of varying stiffness. Response ratios range from 1.27 to 2.73. The third data set compares fixed-base response to that of a structure founded on the surface of a 100-foot soil layer over bedrock. Two soil layer stiffnesses were considered. Response ratios range from 1.07 to 1.39. The fourth data set compares fixed-base response to that of a structure embedded 39 feet in the 100-foot layer of soil. Two soil layer stiffnesses were considered. Response ratios range from 1.39 to 1.90. Note, this case for a shear wave velocity of 1000 fps corresponds to the ratios of response likely to exist for the Zion AFT complex since the design analysis assumed a fixed-base

condition. The final set of data compares the fixed-base case with the surface-founded structure on a 71-foot soil layer. In this case, the Response ratios are near one or slightly greater and slightly less. In these cases where the fixed-based results are less than the 71-foot layer responses, the soil layer and structure are in resonance leading to higher response at some locations especially at lower elevations. Wide scatter in all the ratios is observed.

Table A.17 contains comparable data for in-structure response spectra. Again, however, these ratios can be misleading. A more informative approach is to examine in-structure spectra themselves for the cases of interest. Fig. A.12 displays response spectra comparisons for node 256 at the top of the structure. The general trend is a frequency shift to lower values and a reduction in amplitude.

Soil Springs vs. Surface and Embedded Foundation

Simplified SSI analyses were performed for the shear wall structure and for four sets of soil springs as described earlier. These simplified analyses assumed only structure damping to act as an energy dissipation mechanism, i.e. ignoring radiation dumping. These cases simulate a surface-founded structure whose SSI characteristics are treated in a simplified fashion. The first data set of Table A.18 quantifies this effect. Response ratios of response vary from 1.53 to 2.44. The second data set of Table A.18 compares the simplified SSI results with response calculated assuming the structure to be embedded 39 feet ($e/r = 0.35$). Response ratios of response vary from 1.84 to 3.12. Uniform half-spaces are assumed for all best-estimate cases. Significant conservatism is observed.

Table A.19 compares comparable data for in-structure spectra. Again, a better evaluation of the effect of this simplified SSI procedure on in-structure spectra can be made by directly comparing the response spectra. Fig. A.13 compares response spectra at node 256 as before. Significant reductions are observed.

A.4.1.1.3 Energy Dissipation of the Soil-Structure System

The sensitivity of in-structure peak accelerations and peak forces to changes in structure damping is an important consideration. Many design procedures select conservatively biased damping values; also, as excitation levels increase and structure failure is approached, higher damping values (10% of critical or greater) can be expected. Hence, one needs to answer the question--How does in-structure response vary with increasing structure damping for soil and rock founded structures?

Table A.20 quantifies the effect of increased structure damping on peak accelerations and peak forces for the typical shear wall structure. Fixed-base and two uniform half-spaces of shear wave velocities of 2000 fps and 1000 fps were considered. Structure damping of 4%, 7%, and 10% of critical were studied; comparisons between the three were made. The results show structure damping to be important for the fixed-base condition. Whenever SSI is properly modeled and the site can be characterized as soil rather than extremely stiff rock, changes in structure damping have a minimal effect on response. This is due to the fact that one must consider the soil and structure as a system and the energy

dissipation characteristics of the supporting media dominate the system's behavior. Table A.21 presents comparable data for in-structure response spectra.

A.4.1.2 Subsystem Response

For response calibration purposes, subsystems can be grouped into at least two categories:

- Singly supported floor- or wall-mounted equipment and components, i.e. equipment whose input environment and response can be described by in-structure response spectra at the equipment support location and in two horizontal and the vertical directions.
- Multiply supported piping systems spanning different elevations in a structure and, in some cases, between buildings on separate foundations. Piping forces and moments and accelerations of line-mounted components, such as valves, are of importance.

For singly supported equipment and components, best-estimate or median-level response spectra at their support locations need to be estimated. The bases for these estimates are limited re-analysis of the plant structures of interest and experience with similar structure sited on comparable soil conditions. As mentioned in Section A.2.1, any re-analysis should be performed with best-estimate methodology and parameter values. These include seismic input definition and SSI and structure response analyses. A comparison of best-estimate or median-level response spectra with those generated during the seismic design process provides one aspect of calibration. Other aspects such as equipment or component realistic damping values and method of design and qualification must be taken into account.

The remainder of this section discusses multiply supported piping systems. The seismic response of piping systems is typically separated into two parts--the inertial response and the pseudostatic response due to the relative motion of the system supports. Various analysis procedures have been developed to calculate each portion of the response separately. The process of calibrating piping system response treats each part separately and proceeds by first identifying those aspects of seismic analysis of piping systems which may have a major influence on response prediction.

For inertial response, we have identified six areas: 1) single-support response spectrum analysis procedures vs. multiple-support time history analysis procedures; 2) modal response combination within the response spectrum analysis procedure; 3) accounting for three directional effects; 4) order between modal and directional combination; 5) equivalent static techniques vs. multiple-support time history analysis procedures; and 6) damping effects.

1) Single support response spectrum analysis procedures vs. multiple-support time history analysis - Many piping systems are analyzed by simplified response spectrum analysis techniques where all piping system supports are assumed to be excited by identical response spectra for a given direction, i.e., three response spectra are defined, one for each direction of excitation (two horizontal and the vertical) and they typically apply to all support points. This calculational margin

can be very large and is dependent on piping system size, support locations, and configuration.

2 to 4) Response spectrum analysis procedures and combination modal and directional effects - A limited survey of SAR's identified response spectrum analysis techniques used in the seismic analysis and design of piping systems. All cases identified assumed uniform support excitations as described above. Major differences arose in definition of the input response spectra (e.g. envelope, average, or center-of-gravity spectra), modal response combination, accounting for three directional effects, and the order of modal and directional response combination. Quantification of these effects leads to response response factors.

5) Equivalent static techniques vs. multiple-support time history analysis procedures - Equivalent static techniques are occasionally used to analyze piping systems. In these cases, the piping system is subjected to distributed static load equal to its mass times a factored peak acceleration. The peak acceleration is derived from in-structure response spectra at the piping system support locations.

6) Damping of piping systems -- Design procedures typically assume very low values of modal damping (1% - 3% of critical). Seismic PRA interests are two fold: predict best-estimate or median-level response which requires realistic parameter values such as damping; Predict phenomena which may occur at or near failure levels of the piping system where significantly greater energy dissipation is expected, i.e., modal damping values greater than 1% to 3% used in design. Hence, the effect of piping system damping needs to be assessed and taken into account.

For pseudostatic response, calibration concentrates on the method of analysis. Two methods of analysis were identified through a literature survey of SARs as candidates for calibration. The best-estimate methodology is a multiple support time history analysis. Both methods of analysis to be calibrated are static techniques where the load is defined by maximum structure displacements at the piping system support locations. The first method applies each support displacement independently as a static loading condition, determines response, and combines the responses by SRSS. The second method applies all support displacements in a given direction simultaneously and combines the responses due to the different directions by SRSS. For piping systems spanning more than one structure, each structure is treated independently and the responses combined by absolute sum. Each of these methods was calibrated.

A.4.1.2.1 Calibration of Inertial Response

Piping Systems Physical Description

To investigate the phenomenon and analysis techniques described above, four piping systems of the Zion nuclear power plant were analyzed by different techniques. Numerous comparisons were made to develop response factors. The best-estimate results were generated during the SSMRP phase II analyses and other sensitivity studies (Ref. 1).

The four models are listed in increasing size and complexity:

- o RHRSI-1, Residual Heat Removal System and Safety Injection System 1
- o AFWSG1A, Auxiliary Feedwater to Steam Generator 1A
- o SWTFWP, Service Water to Feedwater Pump
- o RCL, Reactor Coolant Loop

They will be denoted by RHR, AFW, SW and RCL, respectively, in the subsequent discussions. They are schematically shown in Figs. A.14 to A.17. We selected these piping systems for several reasons.

- o Size of pipe - pipe sizes range from 3 inches to 48 inches in size.
- o Types of fittings - several types of pipe fittings (such as elbows, branches, bends, reducers) and pipe supports (such as hangers, snubbers, guides, anchors) are included in the systems.
- o Elevation of systems - the systems' different elevations in the plant; two of the systems run between structures--The AFW runs between the internal structure and the containment shell, and the RHR runs between the AFT complex and the internal structure.
- o One- vs. two-building location - with the RHR and AFW piping systems running through two buildings, the effect of differential pipe support movement between two buildings can be compared with the effect of different pipe support movements within the same building.
- o Large component considerations - the RCL adds a different perspective than the other three piping systems in that its analysis model includes several large components such as steam generators, reactor pressure vessel, reactor coolant pumps, and pressurizer.

<u>Piping System Models</u>

The properties of the seismic models of the four selected piping systems are shown in the following table:

	RHR	AFW	SW	RCL
No. of Nodes	96	263	423	760
No. of Pipe Elements	71	158	322	496
No. of Modes Included (= 33 Hz)	18	36	57	140
Fundamental Frequency (Hz)	3.86	2.86	1.95	1.43
No. of Output Locations:				
(a) Moment	22	23	132	118
(b) Acceleration	2	1	13	17

The RHR, AFW, and SW models are illustrated in Fig. A.18 to A.20. Elements for moment output and nodes for acceleration output are identified. The elements selected for moment output are, in general, elbows, branches, and anchors--all locations of anticipated peak stress. The nodes selected for acceleration output represent valve locations.

Analysis Scenarios

A limited review of SARs identified a number of anaysis scenarios which have been used in the nuclear industry to analyze piping systems. Initially, we focus on response spectrum analysis procedures which assume all piping system supports to be excited by identical response spectra for a given direction. The major elements of the procedure are input definition to the piping system, modal response combination techniques, directional response combination techniques, order of modal and directional combinations, and damping of piping systems. Tables A.22, A.23 and A.24 itemize four alternative procedures for input definition, seven alternatives for modal response combination and three for directional response combination, respectively. Order of modal and directional response combination (two alternatives) and damping valves of 1%, 2%, and 5% complete the alternatives. It was not feasible nor necessary to explicitly treat all combinations of these alternatives in the quantification effort (4 X 7 X 3 X 2 X 3 = 504 alternatives per earthquake and per piping system). A first pass at reducing the number of alternatives was based on assuming envelope response spectra and 2% damping. The directional combination approach of SRSS was used for all cases but one--SRSS of modal responses in combination with HHV. Table A.25 itemizes eleven cases which were treated in detail. These eleven cases were selected based on their use in the nuclear industry. A further grouping to three summary groups is shown in Table A.25, i.e., the results of this investigation (Ref. 8) showed further grouping to be applicable. Summary results are presented here. Separate studies on the effects of input definition and damping were conducted and, again, summary results are presented here.

In all of these studies, four free-field earthquake excitations were used--three corresponding to RG 1.60 design ground response spectra and one site specific earthquake. Input to the piping systems was defined by performing SSI and structure response analyses of the Zion nuclear power plant containment building and AFT complex (Sec. A.4.1.1) for each of the four earthquakes definitions.

Modal and Directional Combination Procedures vs. SRSS-SRSS

The SRSS-SRSS technique (case B8) when applied to singly supported systems many times yields results which compare well with responses calculated by time history procedures. Hence, comparisons of cases A1 - A7 and B9 - B11 with case B8 were made to provde a benchmark. Table A.26 compares responses for the four piping systems of interest. A further grouping of the eleven techniques resulted from a detailed evaluation of each technique's characteristics. The results are displayed in Table A.26. The overall result was three summary groups of techniques as shown in Table A.25.

Multiple-Support Time History Analysis vs. Summary Groups 1,2,and 3

To quantify the effect of the simplified response spectrum analysis procedures, the three summary group responses were compared with responses calculated by the multiple-support time history analysis procedure of the SSMRP. Table A.27 summarized the results.

Envelope Spectra vs. Center-of-Gravity (CG) and Average

Table A.28 summarizes the effects of specifying input response spectra at the C.G. of the piping system or an average spectra over support point locations for each of the four piping systems. Both Tables A.27 and A.28 demonstrate that the more extensive the piping system in terms of support locations throughout the buildings, the more conservative the envelope spectrum approach.

Damping Effects

Damping effects, as measured by responses calculated by response spectrum analysis, are quantified in Table A.29.

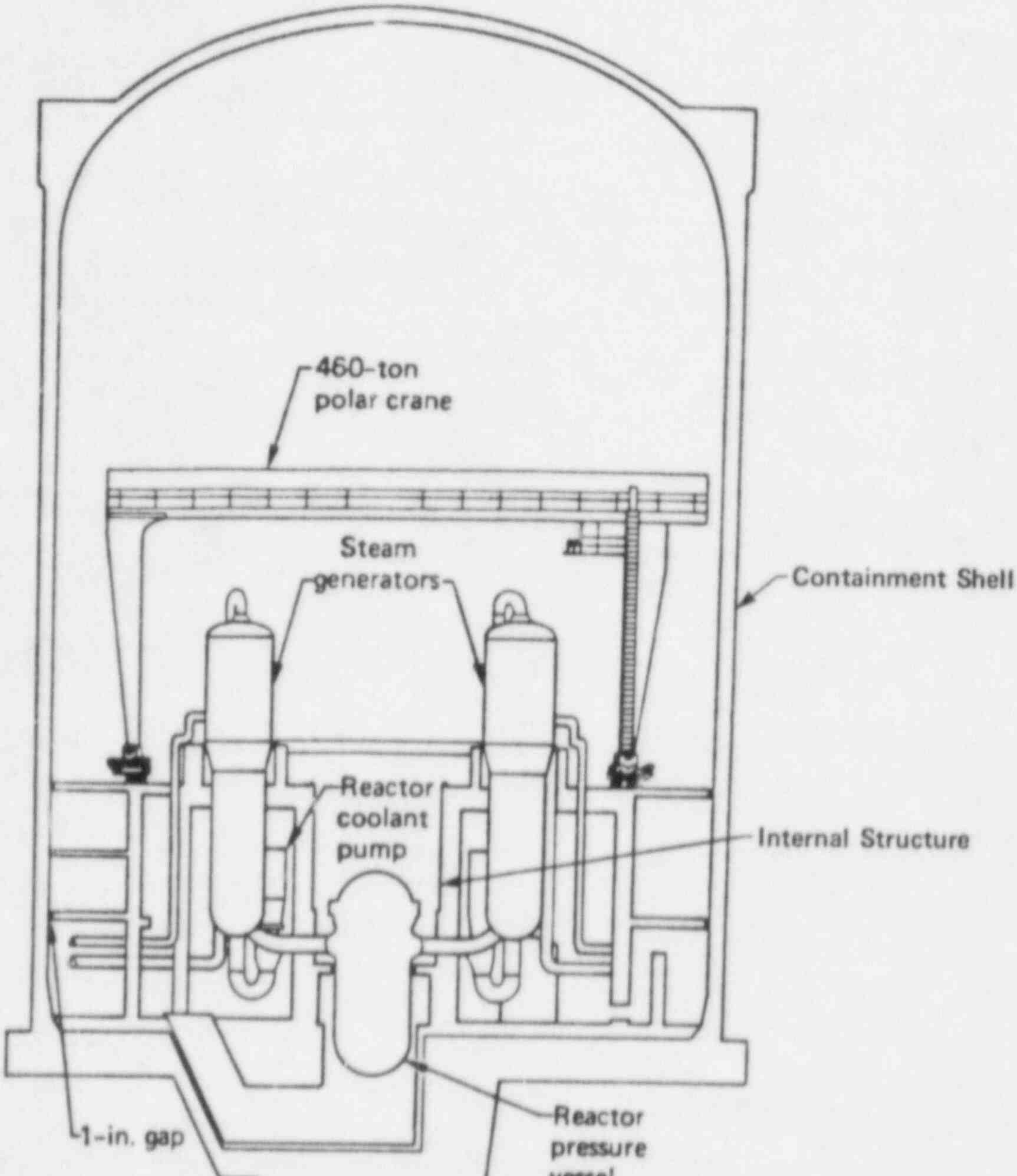


Fig. A.1 Zion containment building showing the containment shell and internal structure.

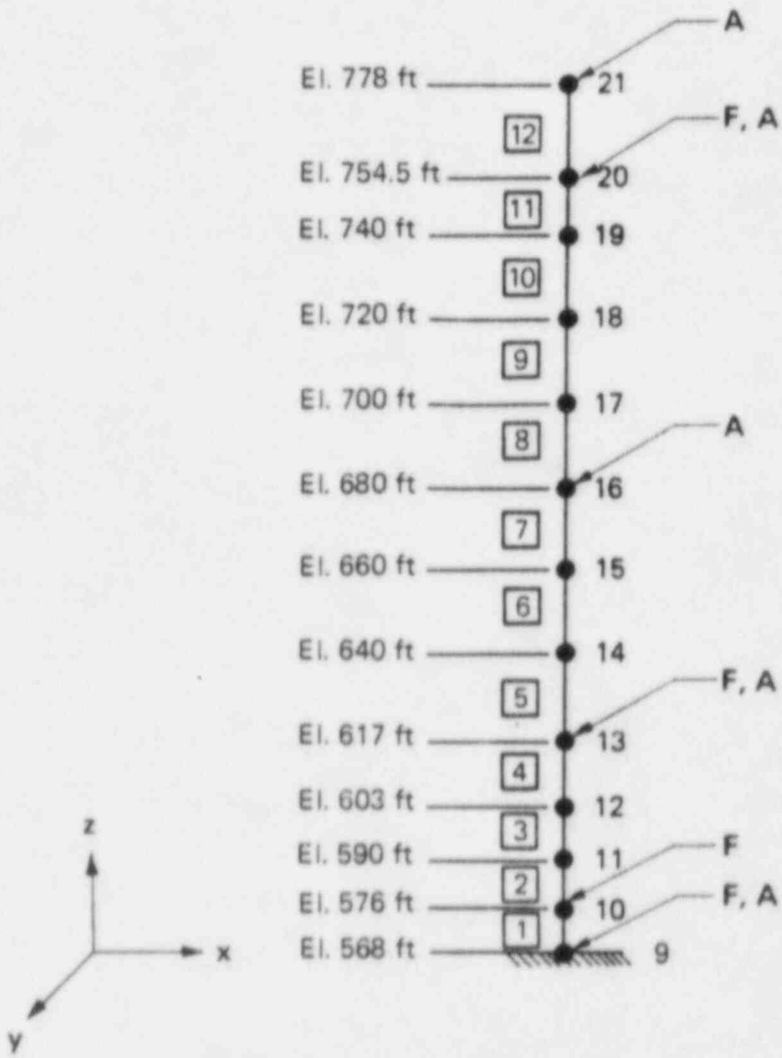


Fig. A.2 The containment shell model. A indicates nodes where acceleration responses were obtained and F indicates members where force responses were calculated.

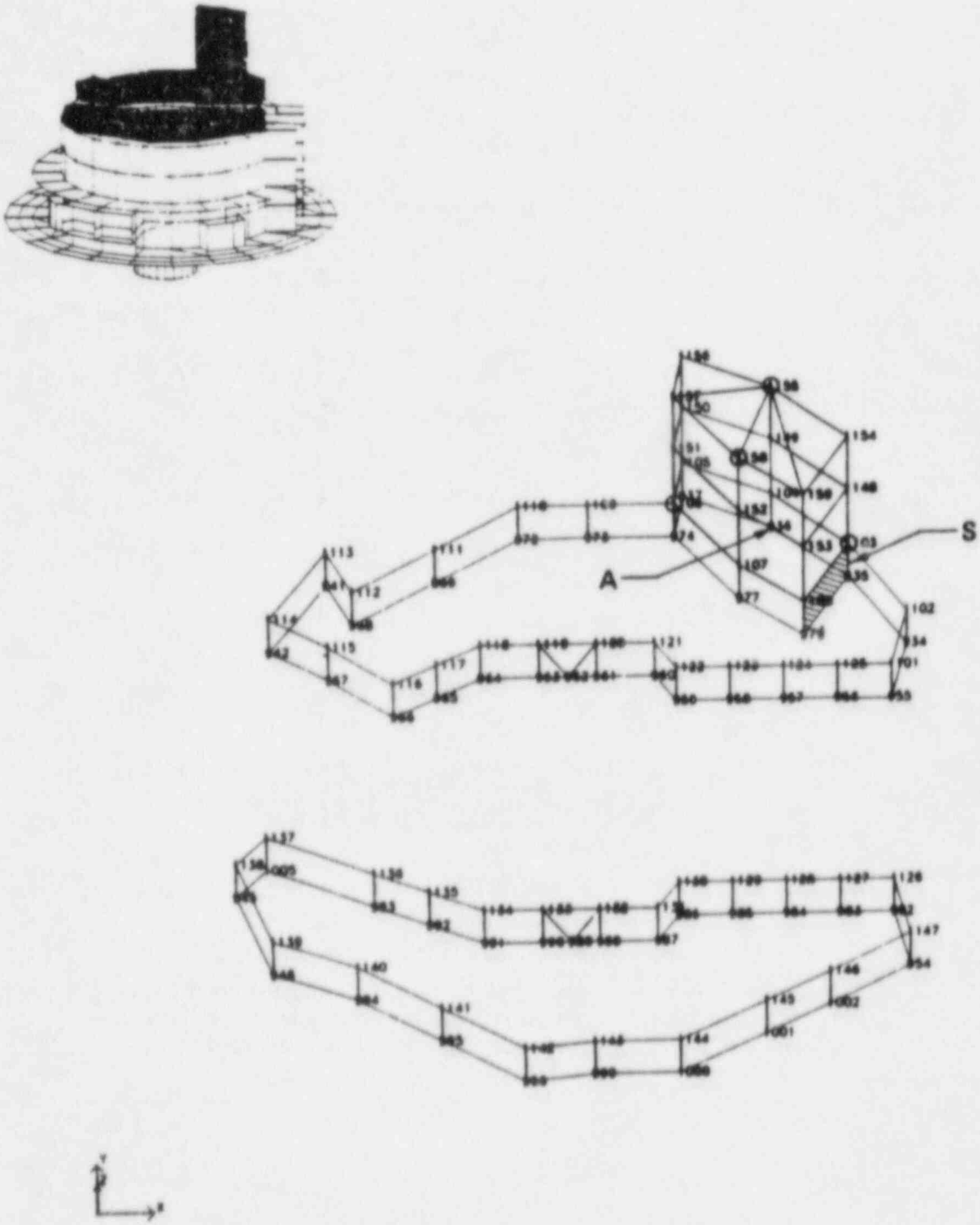


Figure A.3a Perspective view of steam generator and pressurizer compartments above El. 617 ft. "A" indicates acceleration output, "S" indicates shear stress output for shaded element.

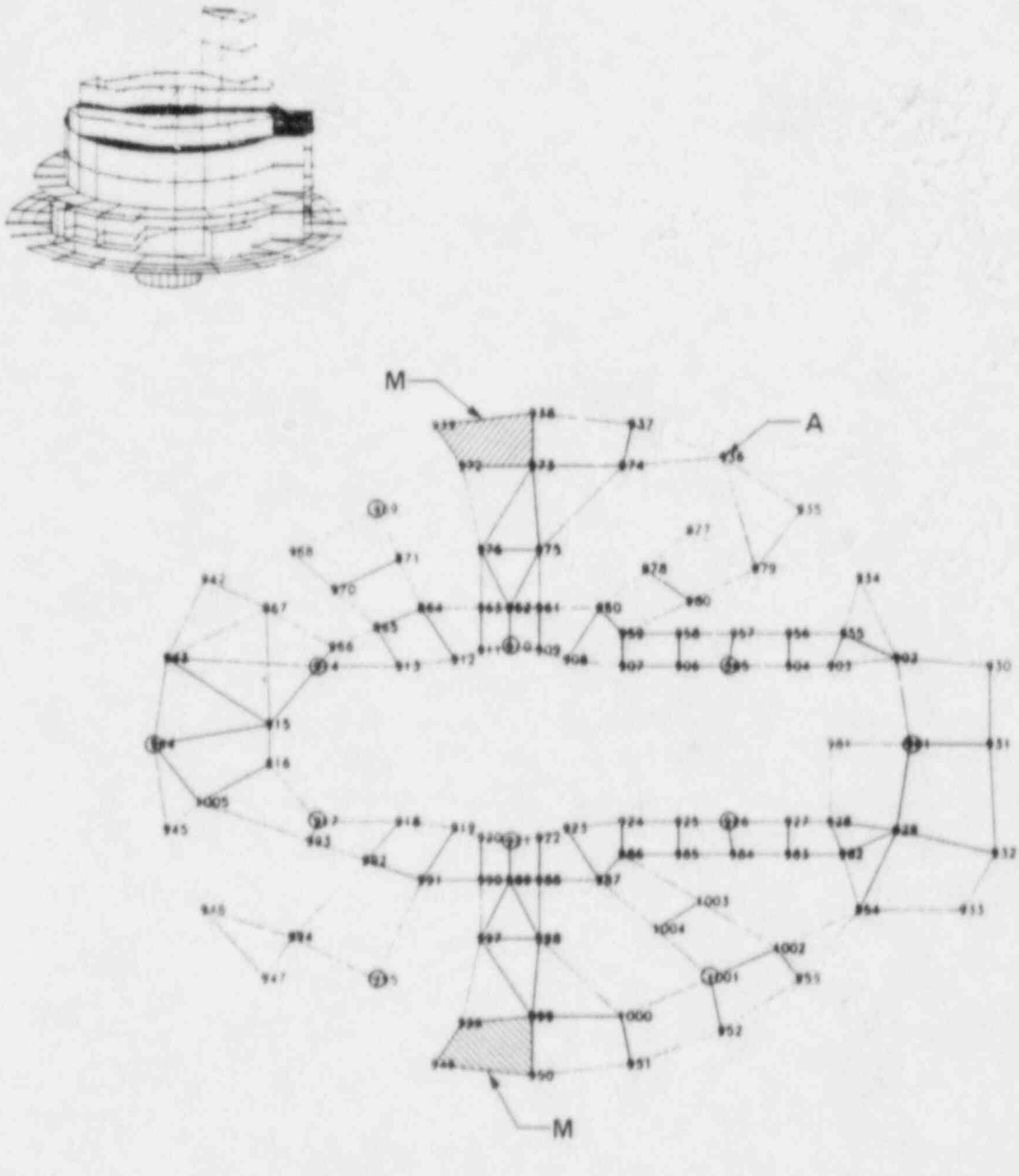


Fig. A.3b Perspective view of operating floor (El. 517 ft.)
M = Moment output, A = acceleration output.

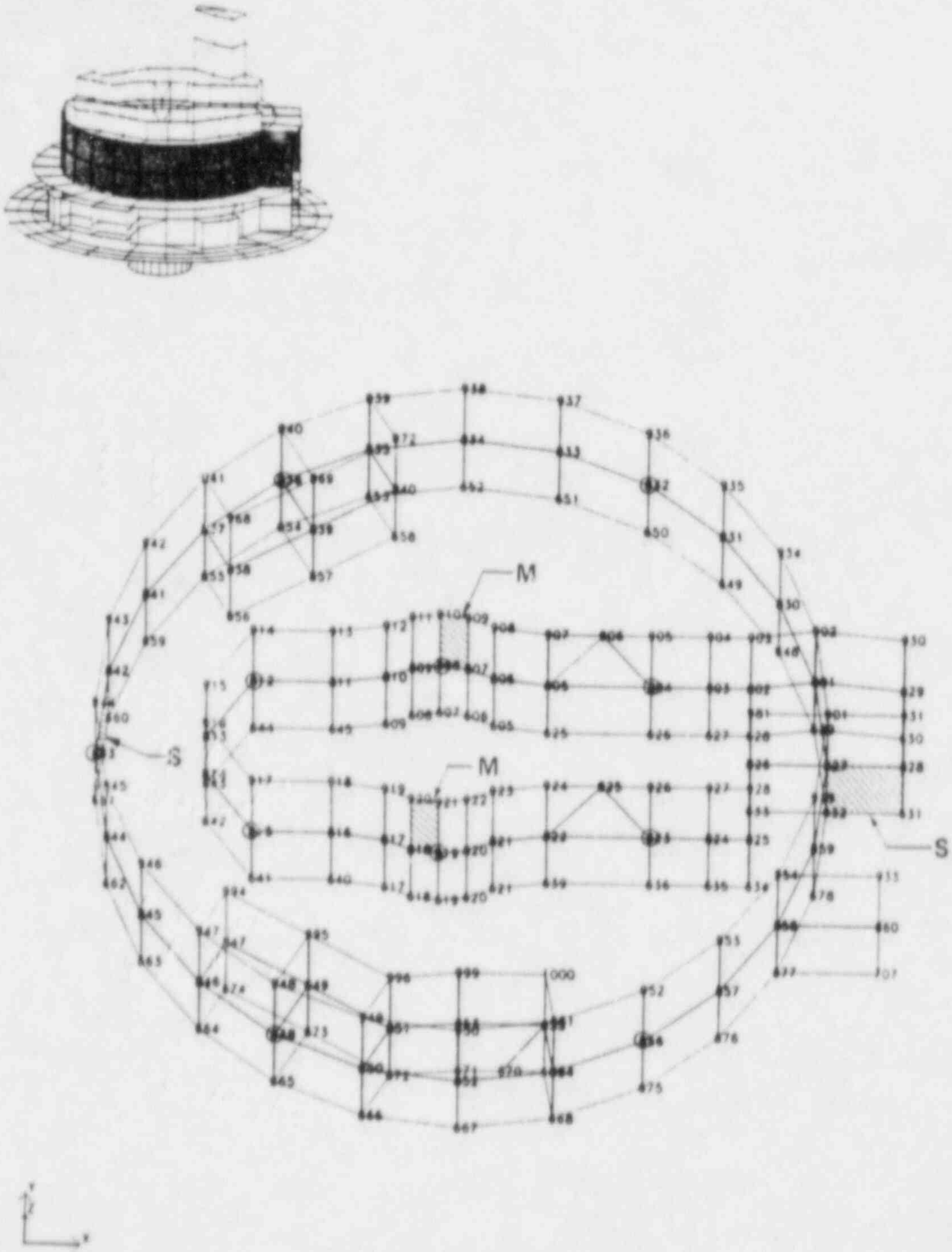
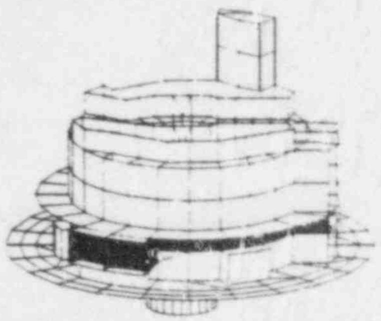


Fig. A.3c Perspective view of ring wall and fuel handling pool wall between El. 590 ft. and El. 617 ft.



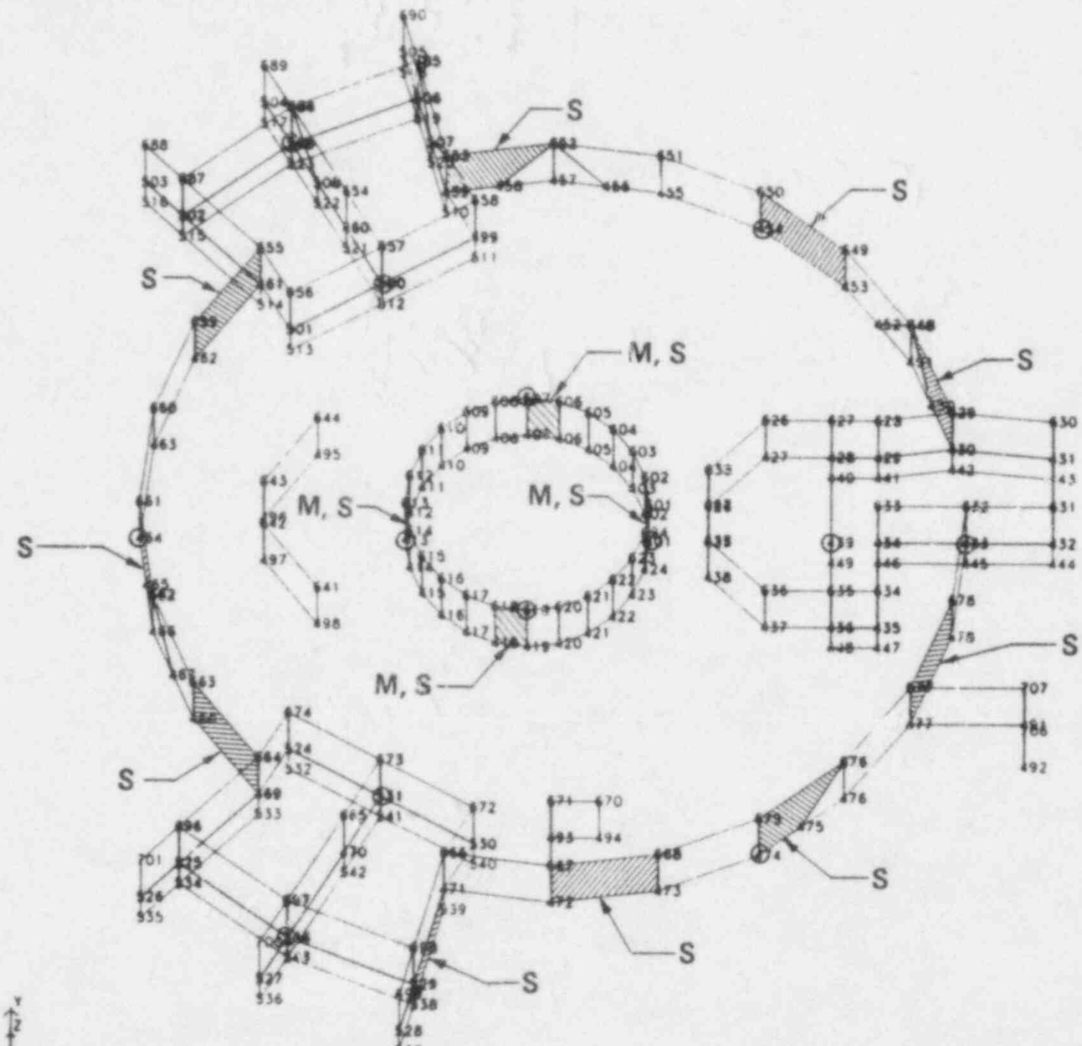


Fig. A.3d Perspective view of biological shield wall, ring wall, and fuel handling pool wall between El. 576 ft. and El. 590 ft.

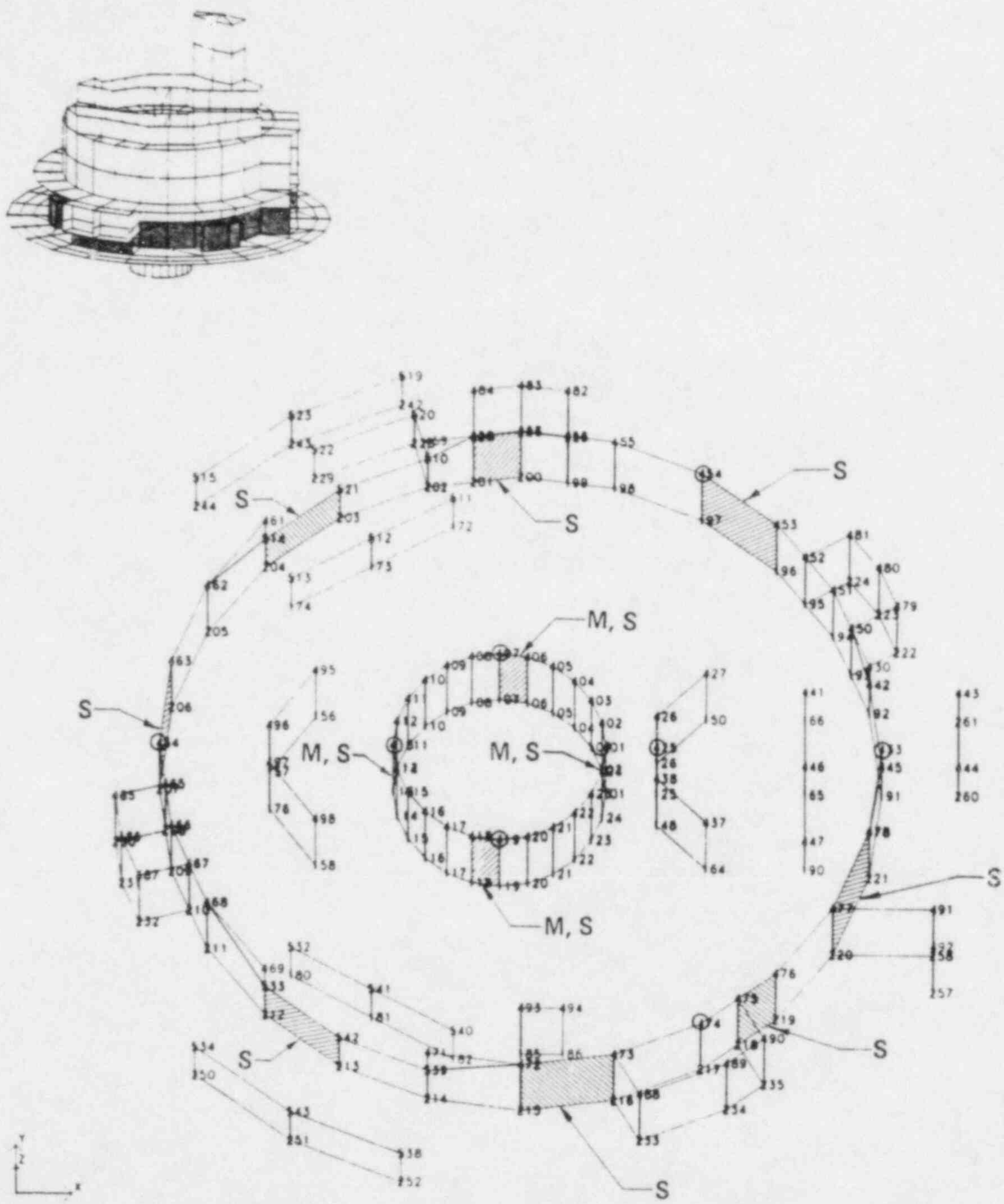


Fig. A.3e Perspective view of walls between El. 586 ft. and El. 581 ft.

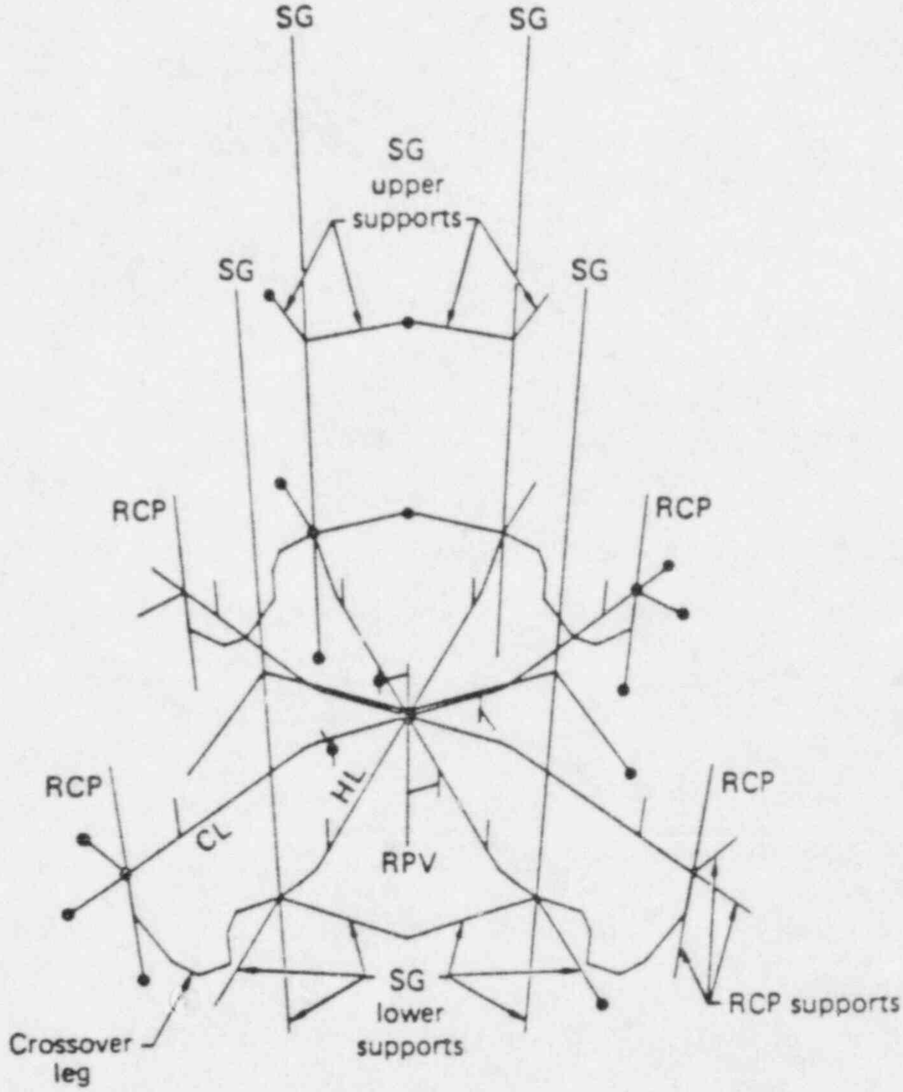


Fig. A.4. Isometric view of the reduced NSSS model.

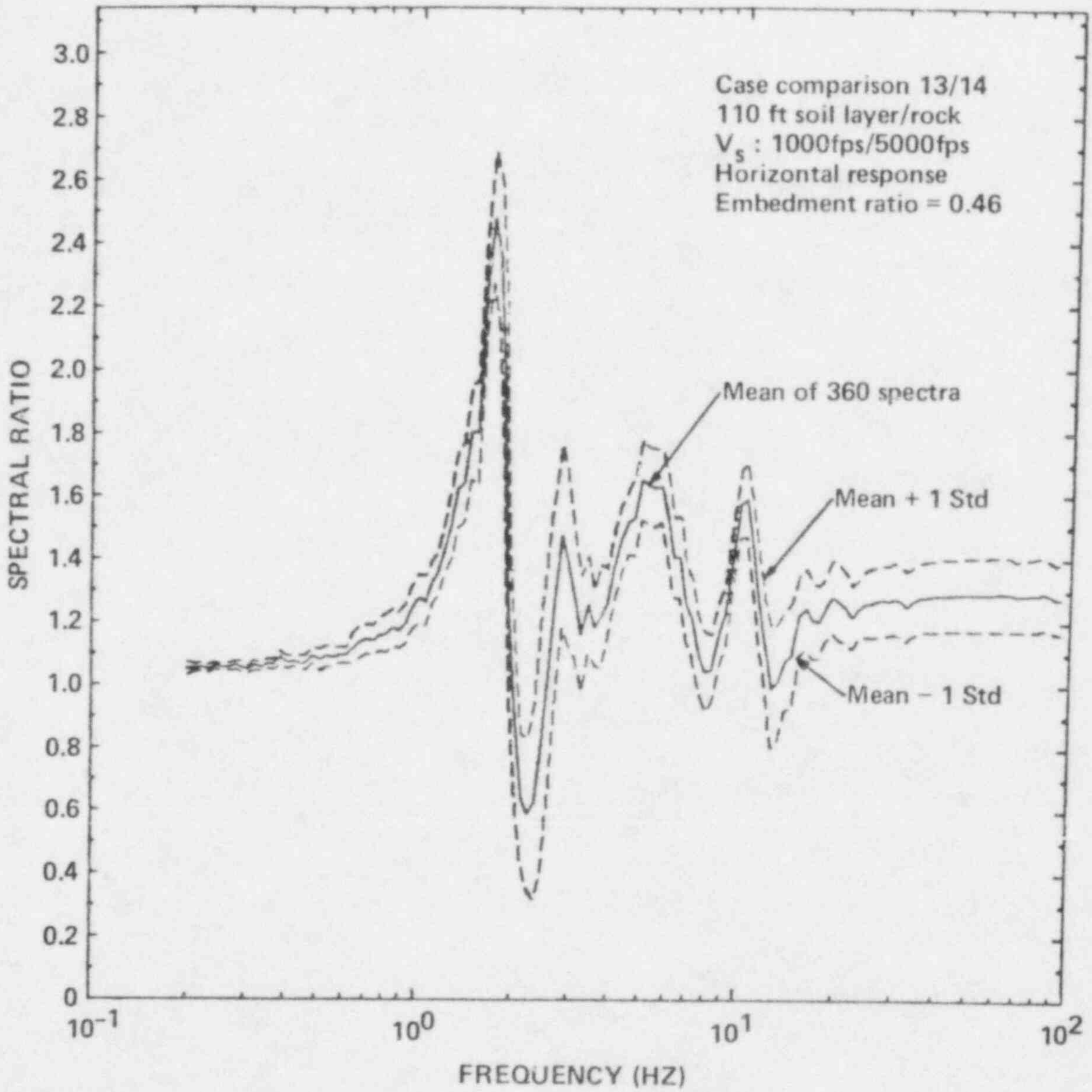


Fig. A.5a. The mean spectral ratio and the mean +/− one standard deviation for case comparison 13/14.

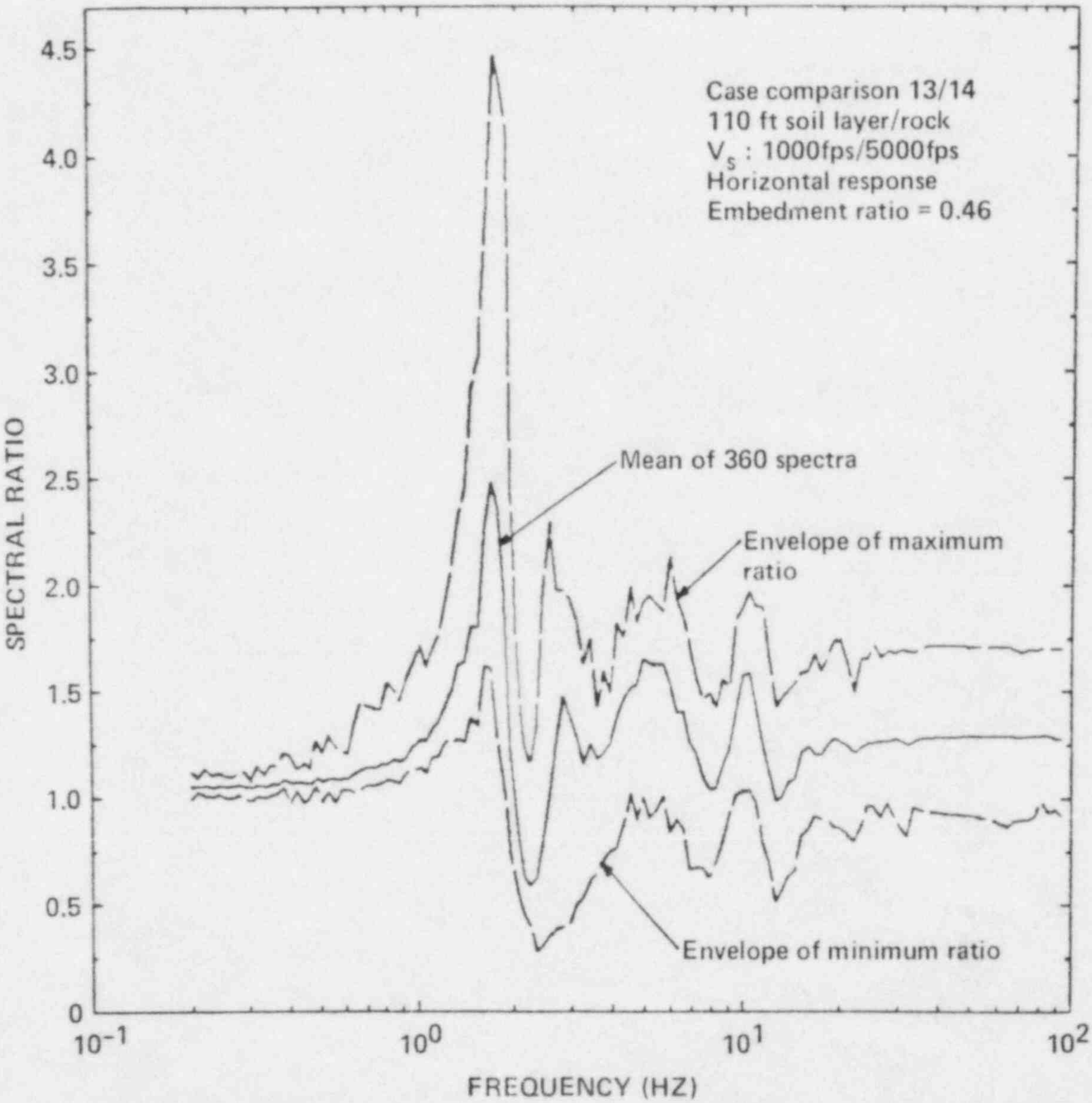


Fig. A.5b. The mean spectral ratio and the envelopes of maximum and minimum ratio for case comparison 13/14.

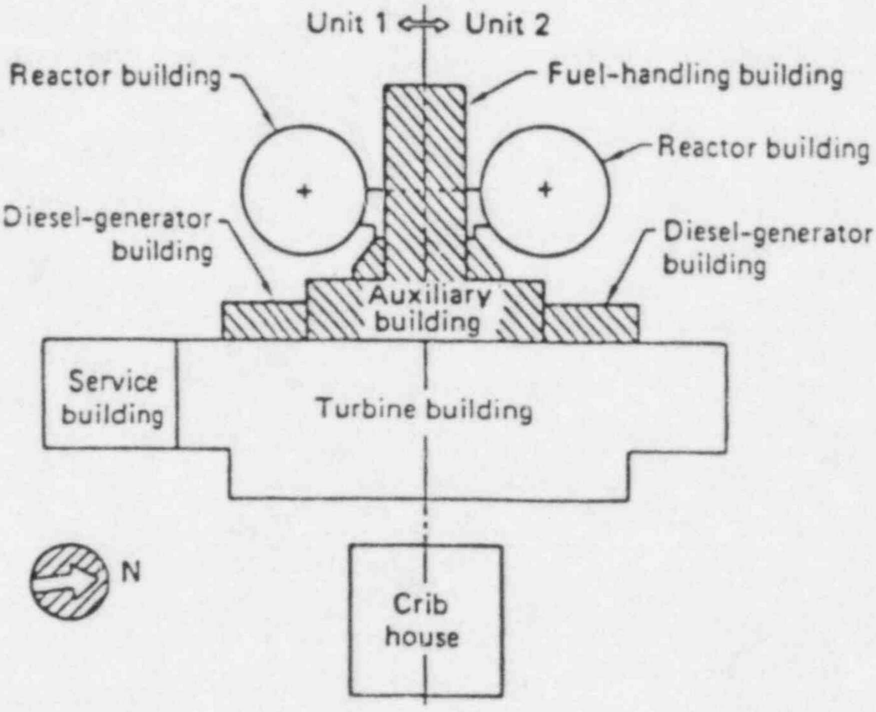


Figure A.6a Simplified plan view of the Zion, Illinois, nuclear power plant. The shaded area--comprising the auxiliary/fuel-handling/diesel-generator complex is of this study

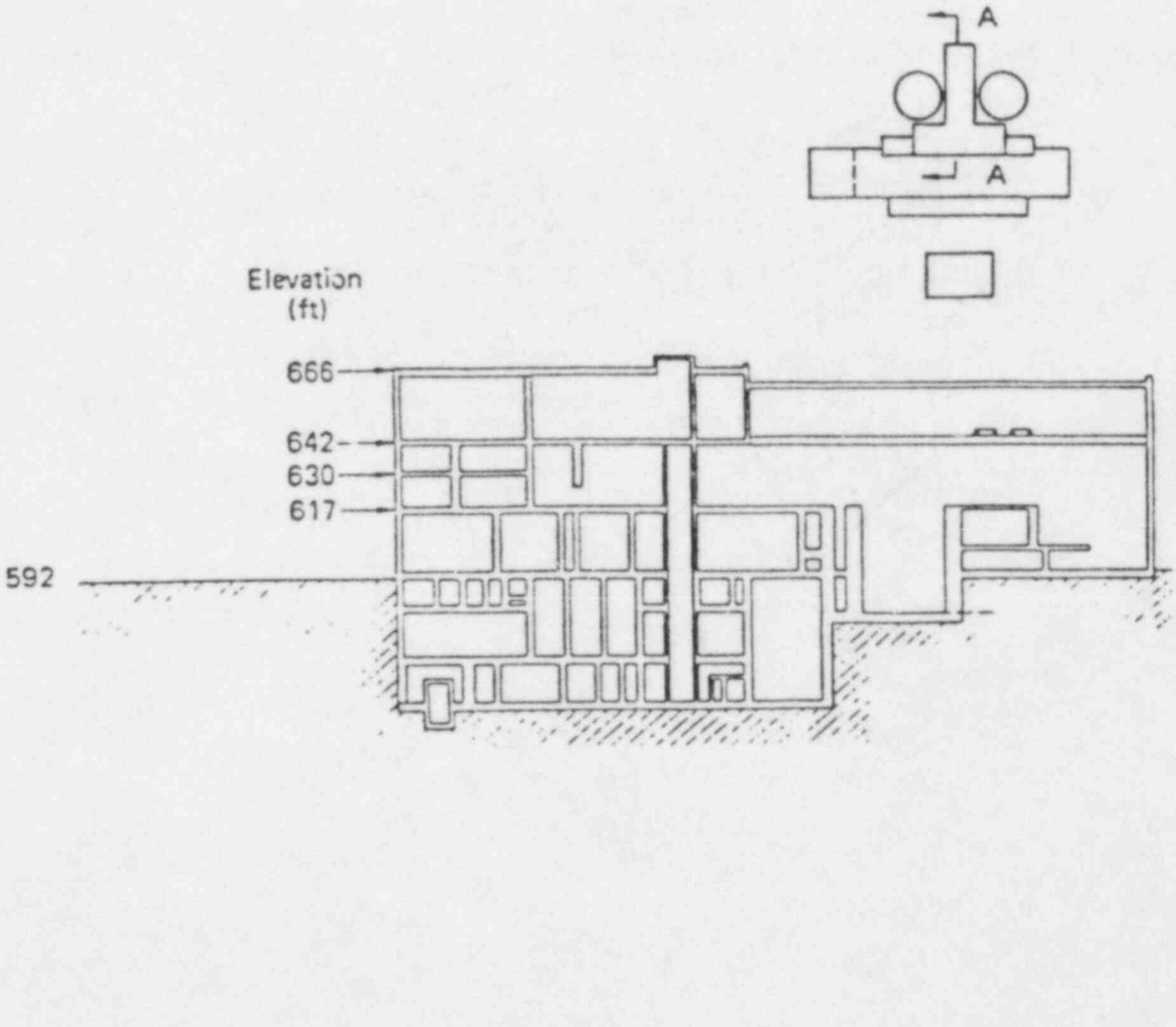


Figure A.6b Section of the auxiliary/fuel-handling/diesel- generator complex throught he plane of symmetry.

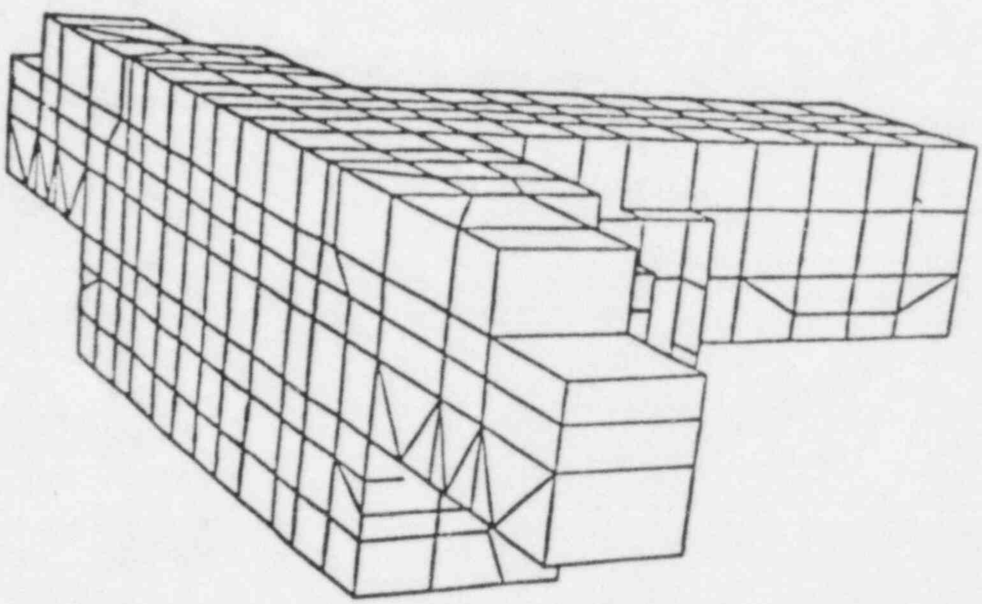


Figure A.7 Perspective view of the shear wall structure model looking southwest.

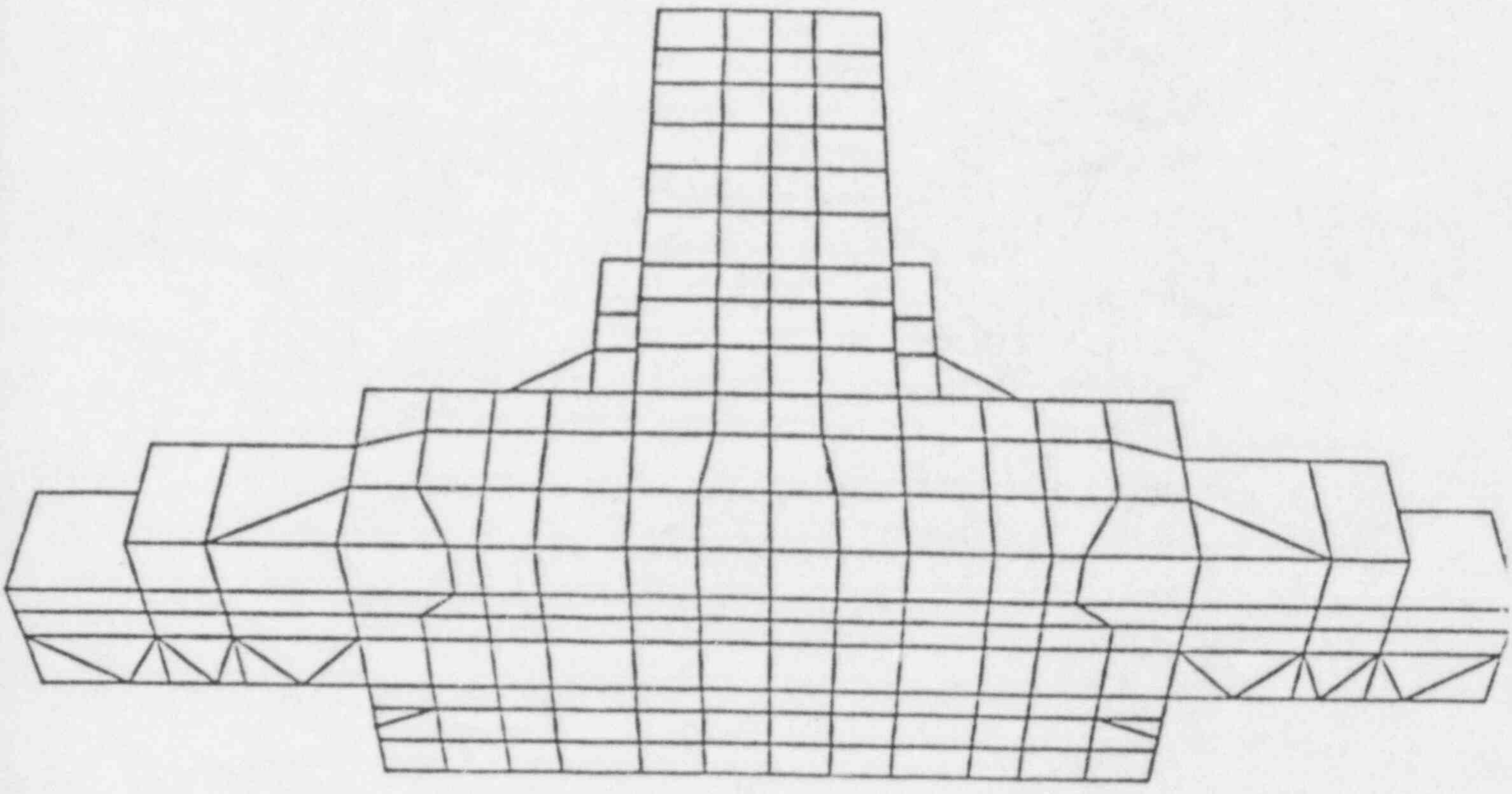


Figure A.8 Perspective view of the shear wall structure model looking west.

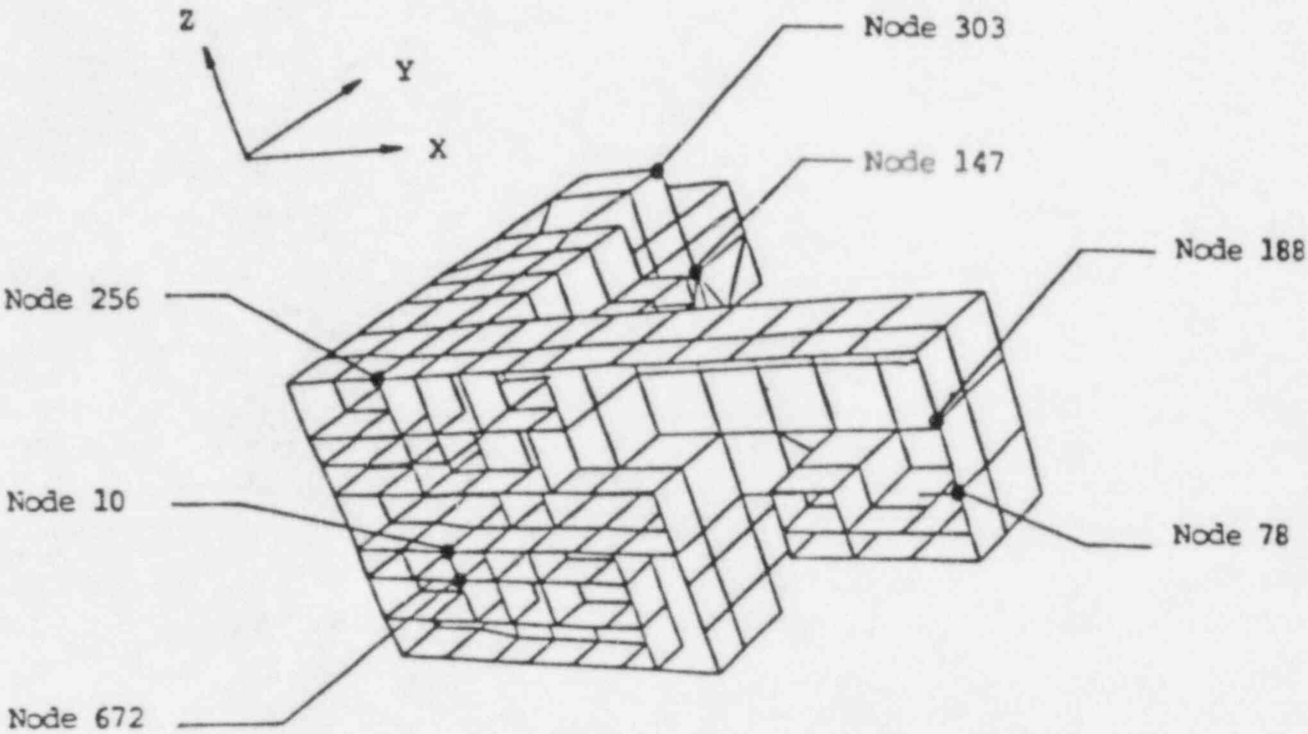


Fig. A.9 Half section of the shear wall structure model illustrating node points at which peak accelerations and in-structure response spectra were generated.

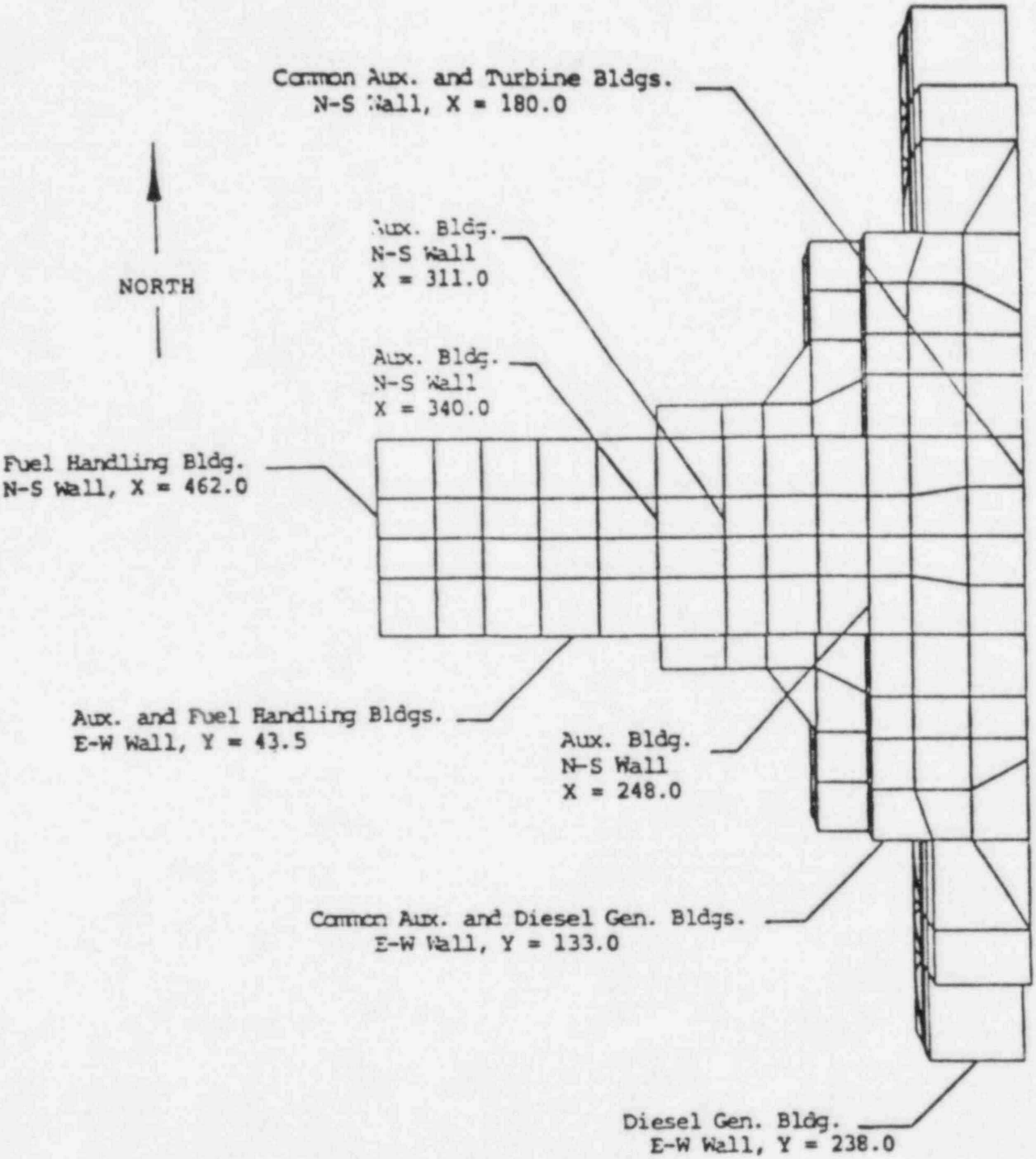
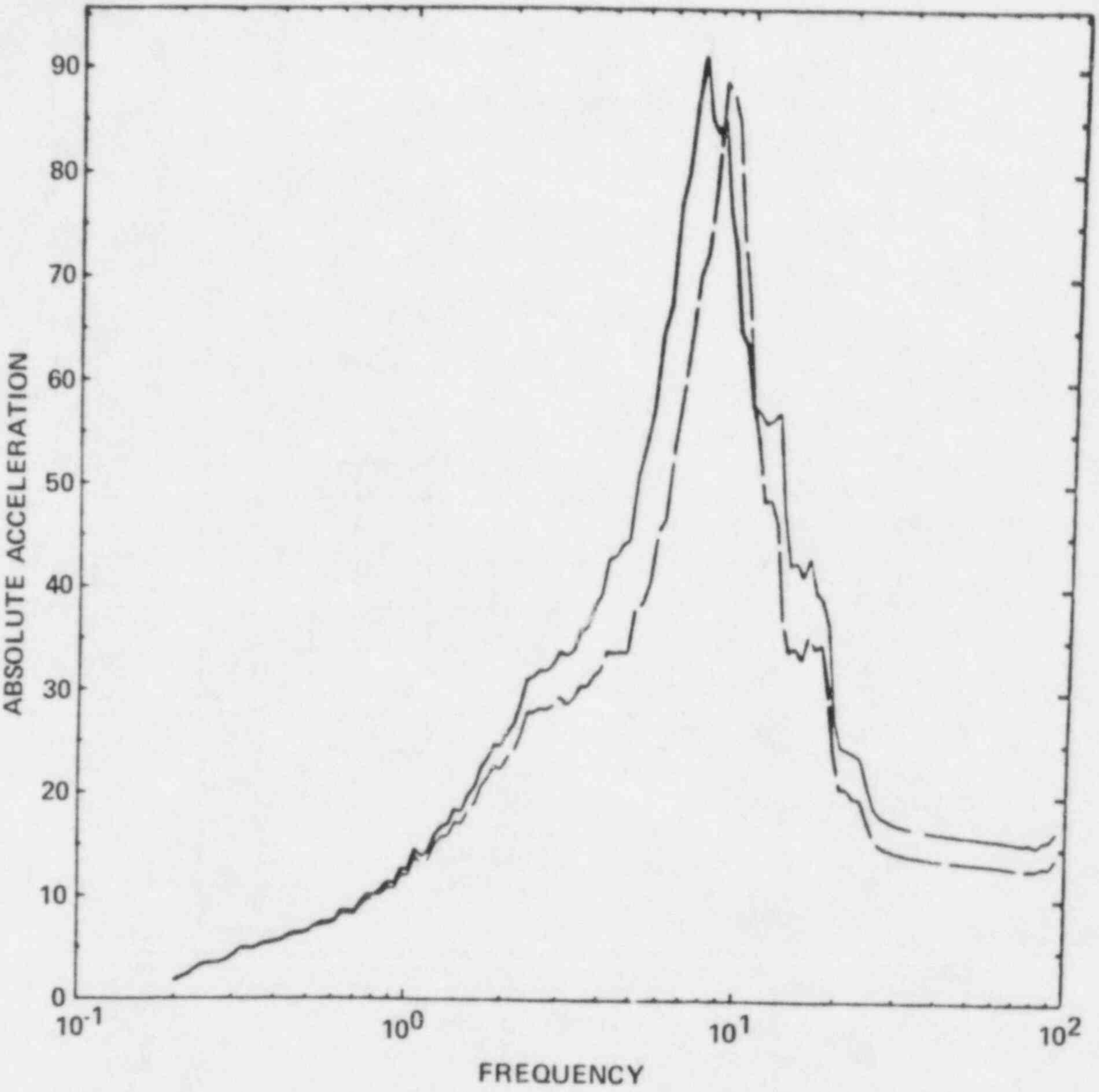
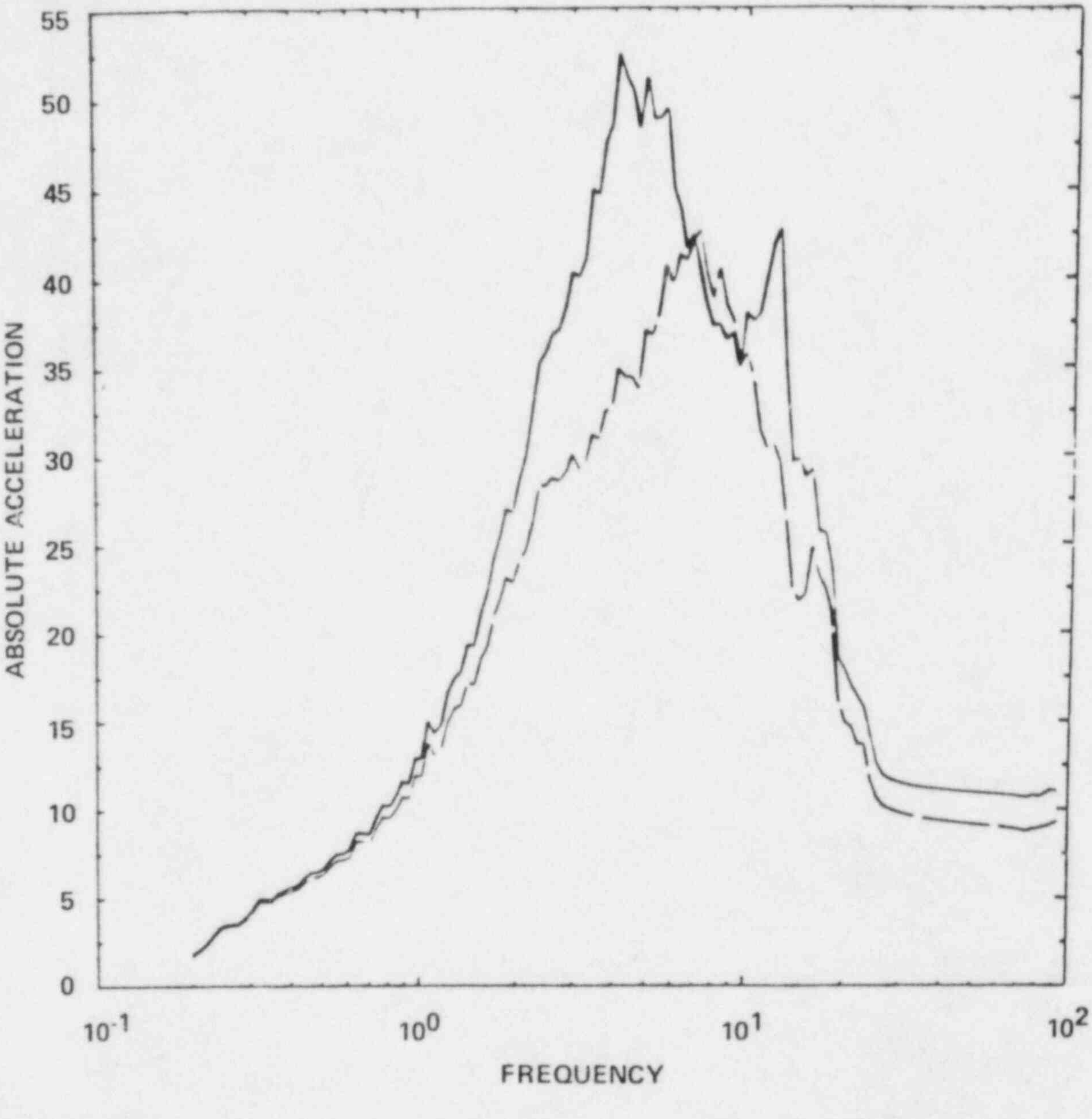


Fig. A.10 Plan view of shear wall structure model showing walls for which forces and moments were generated.



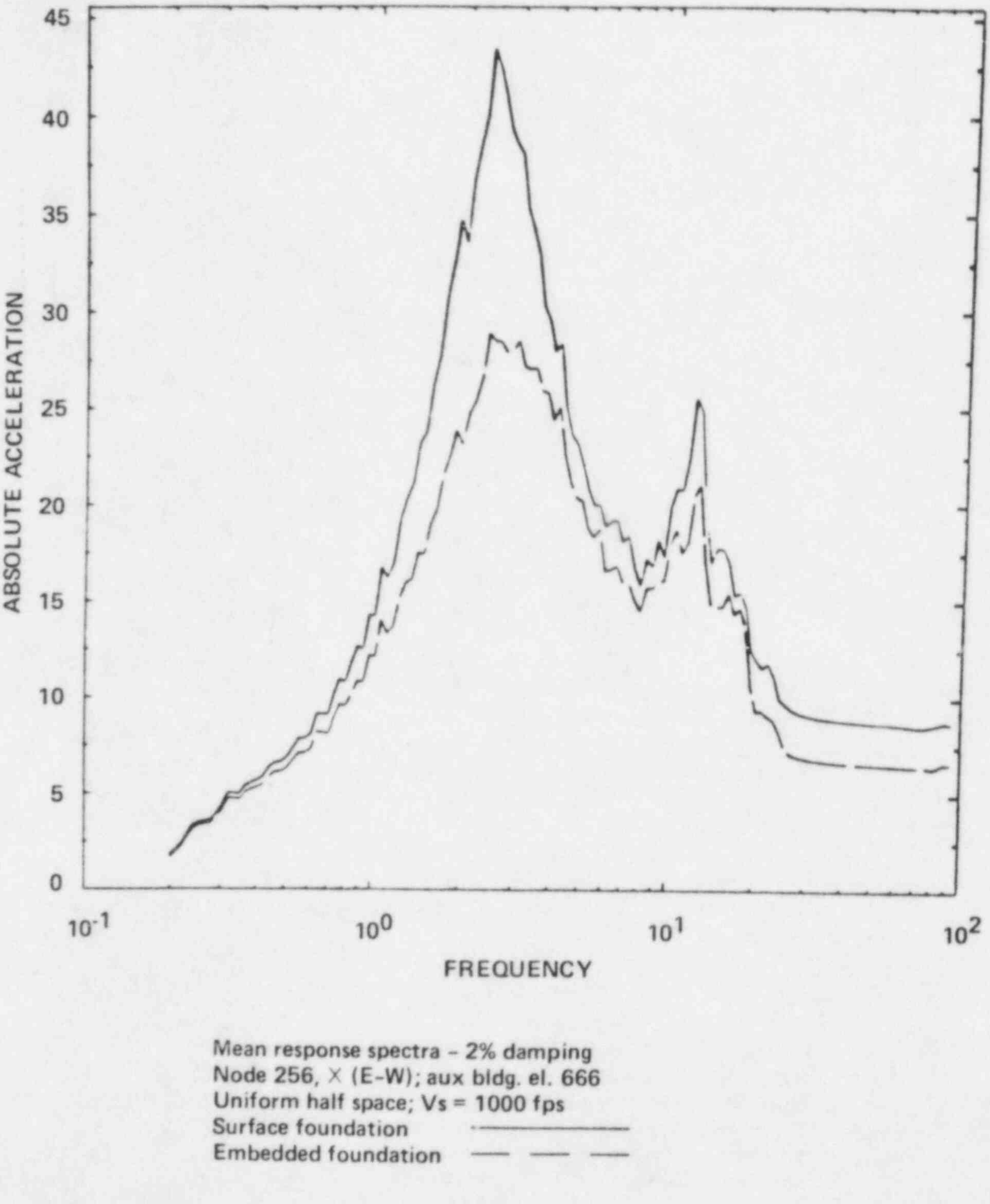
Mean response spectra - 2% damping
Node 256, X (E-W); aux bldg. el. 666
Uniform half space; Vs = 3500 fps
Surface foundation ————————
Embedded foundation — — — —

Fig. A.11a Comparison of in-structure response spectra at node 256; surfave vs. embedded foundation, shear wall structure; uniform half space, Vs = 3500 fps case 2/case 6.



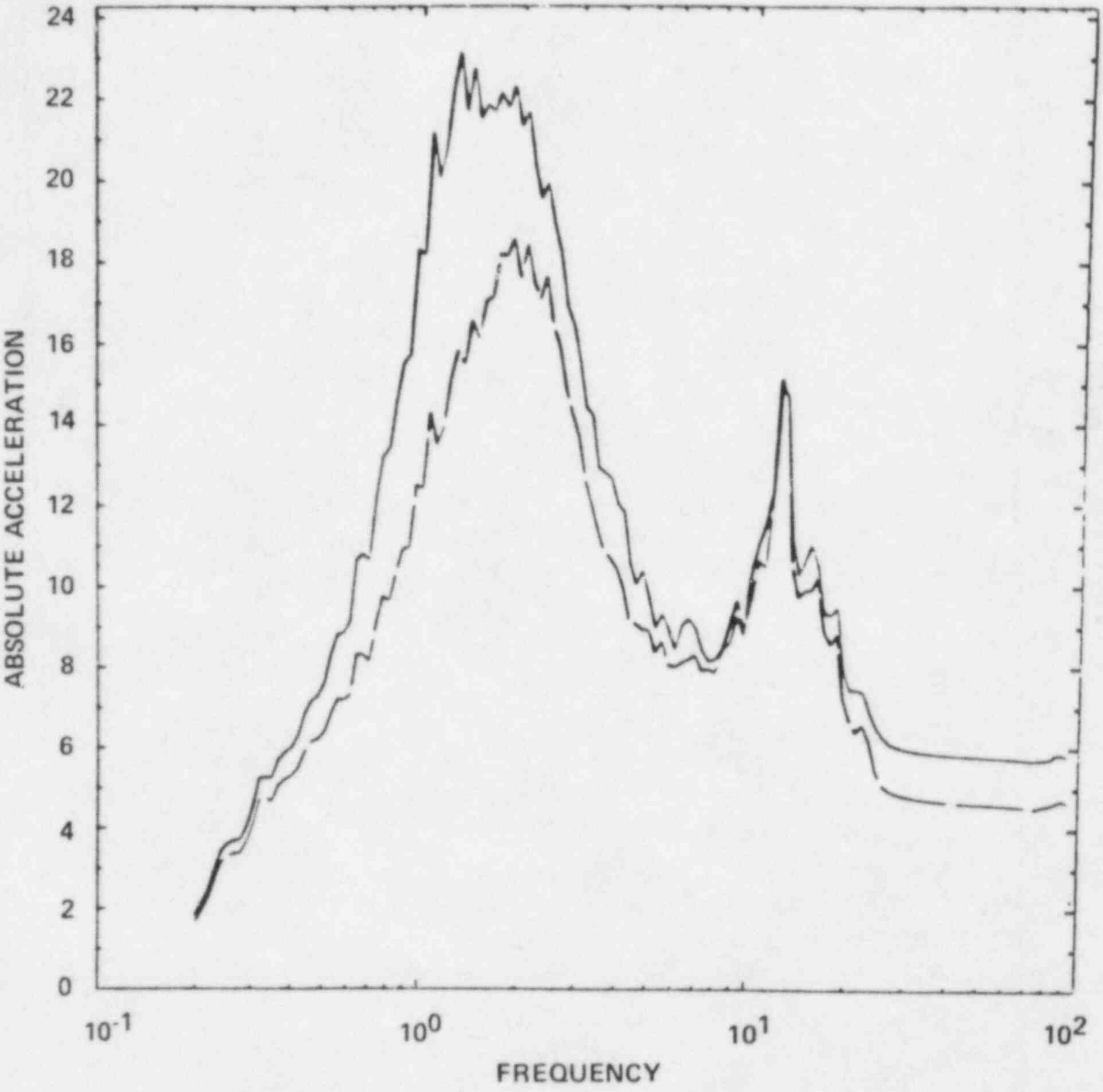
Mean response spectra - 2% damping
Node 256, X (E-W); aux bldg. el. 666
Uniform half space; Vs = 2000 fps
Surface foundation ————————
Embedded foundation —— —— ——

Fig. A.11b Comparison of in-structure response spectra at node 256; surface vs. embedded foundation, shear wall structure; uniform half space, Vs = 2000 fps case3/case 7.



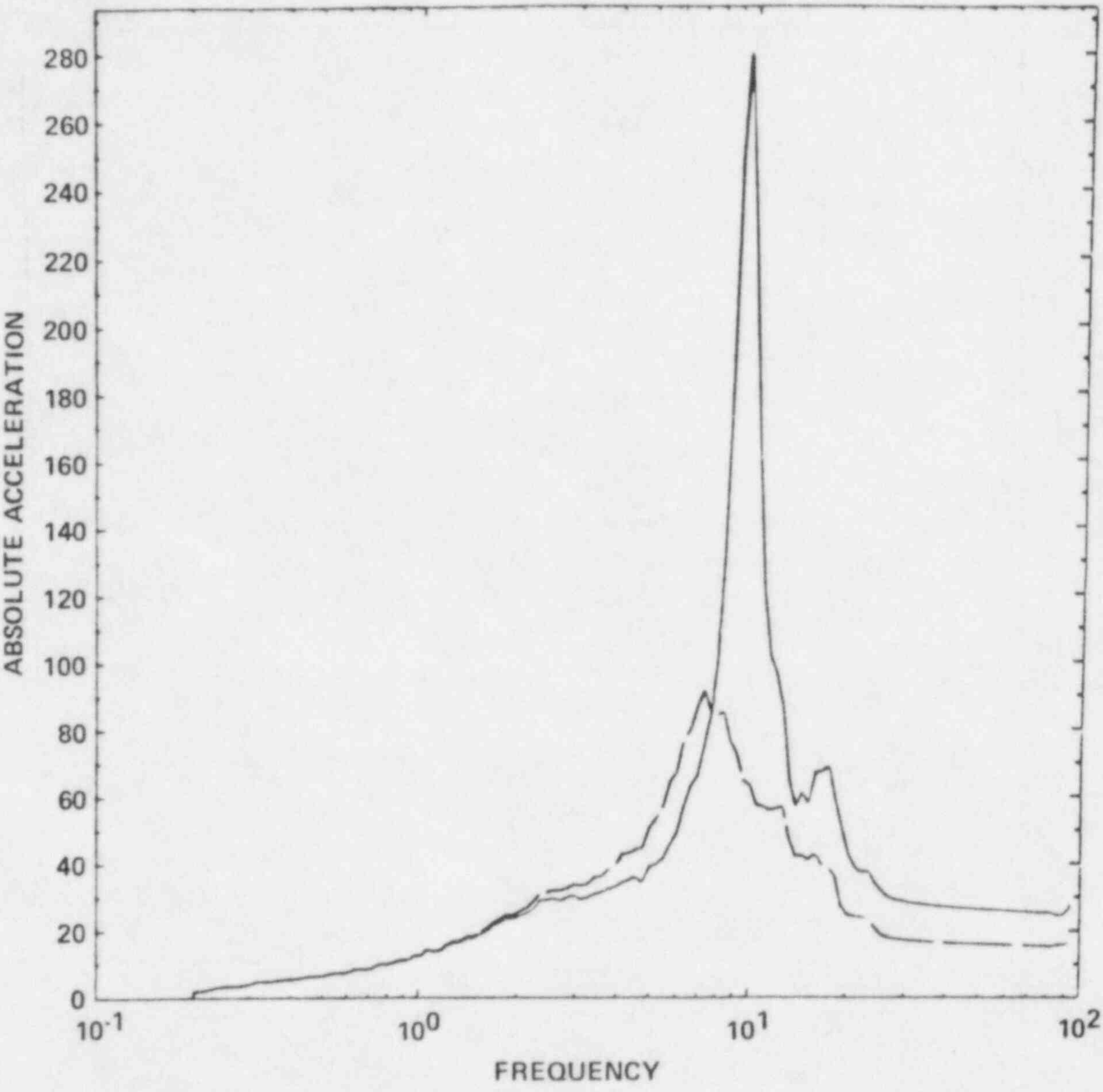
Mean response spectra – 2% damping
Node 256, X (E-W); aux bldg. el. 666
Uniform half space; Vs = 1000 fps
Surface foundation ——————
Embedded foundation — — — —

Fig. A.11c Comparison of in-structure response spectra at node 256; surface vs. embedded foundation, shear wall structure; uniform half space, Vs = 1000 fps case 4/case 8.



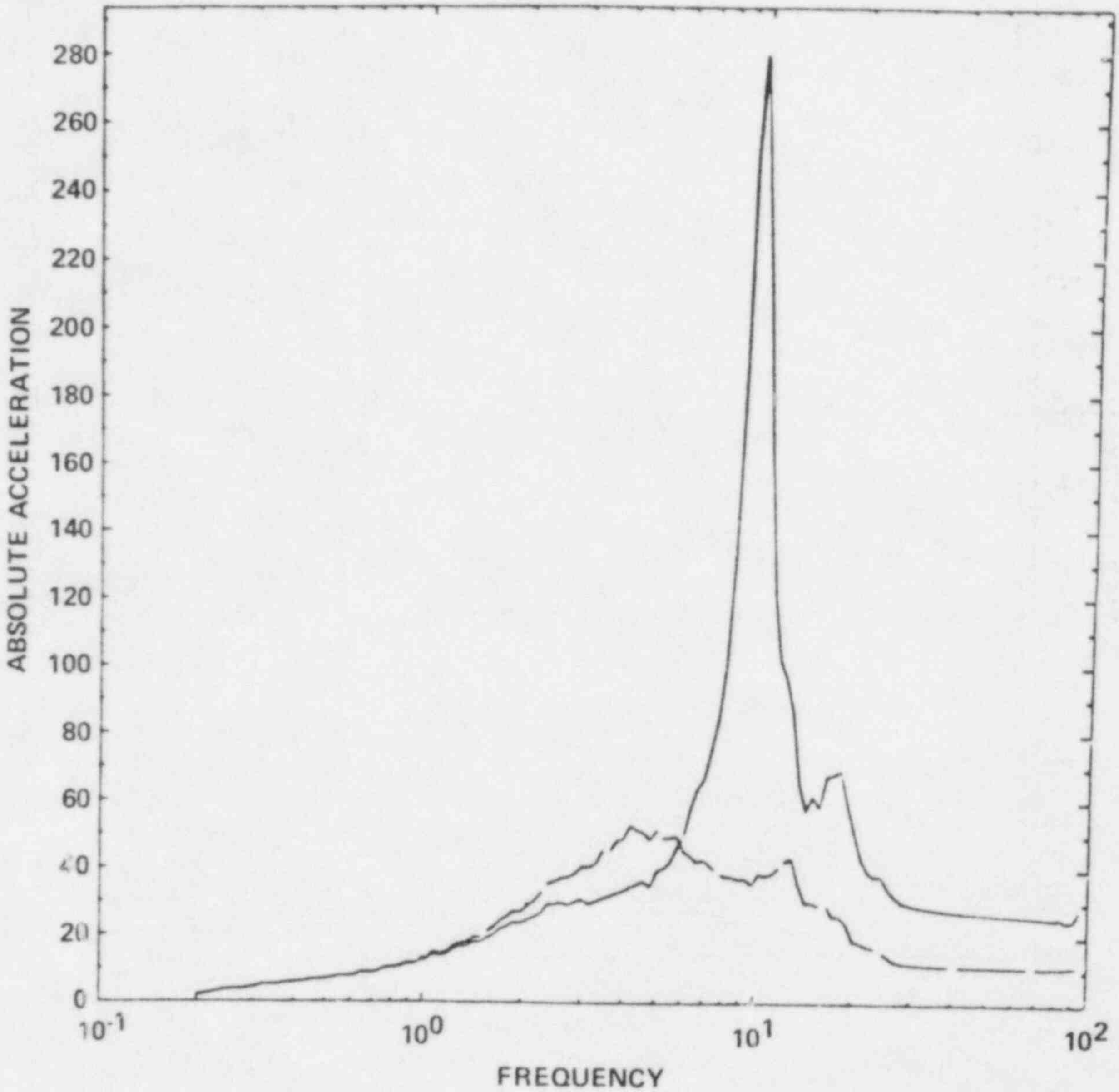
Mean response spectra – 2% damping
Node 256, × (E-W); aux bldg. el. 666
Uniform half space; Vs = 500 fps
Surface foundation ————————
Embedded foundation — — —

Fig. A.11d Comparison of in-structure response spectra at node 256; surface vs. embedded foundation, shear wall structure; uniform half space, Vs = 500 fps case 5/case 9.



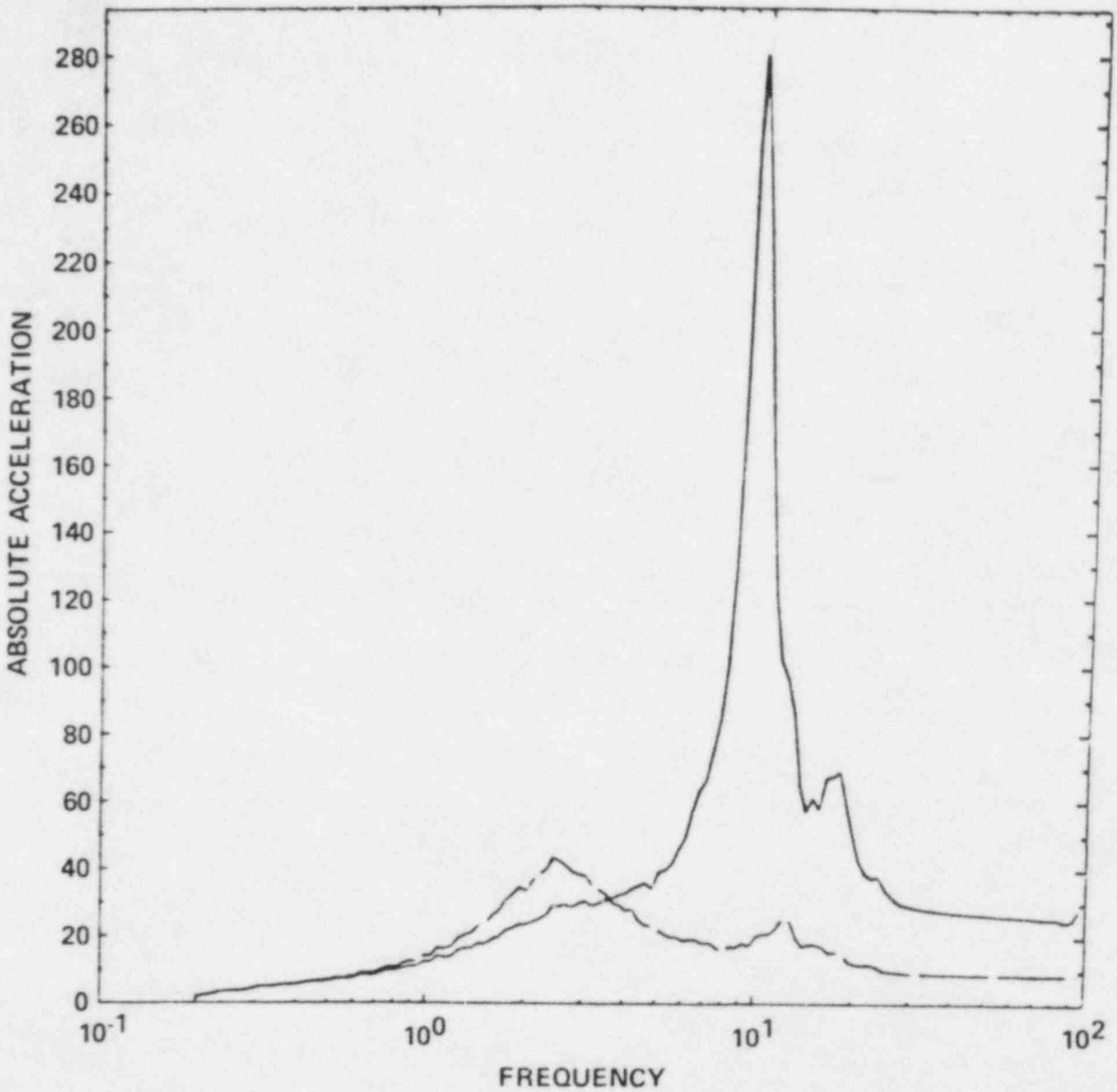
Mean response spectra - 2% damping
Node 256, X (E-W); aux bldg. el. 666
Uniform half space; Vs = 3500 fps
Fixed base ——————
Surface foundation — — —

Fig. A.12a Comparison of in-structure response spectra at node 256; fixed-base vs. surface foundation, shear wall structure; uniform half space, Vs = 3500 fps case 1/case 2.



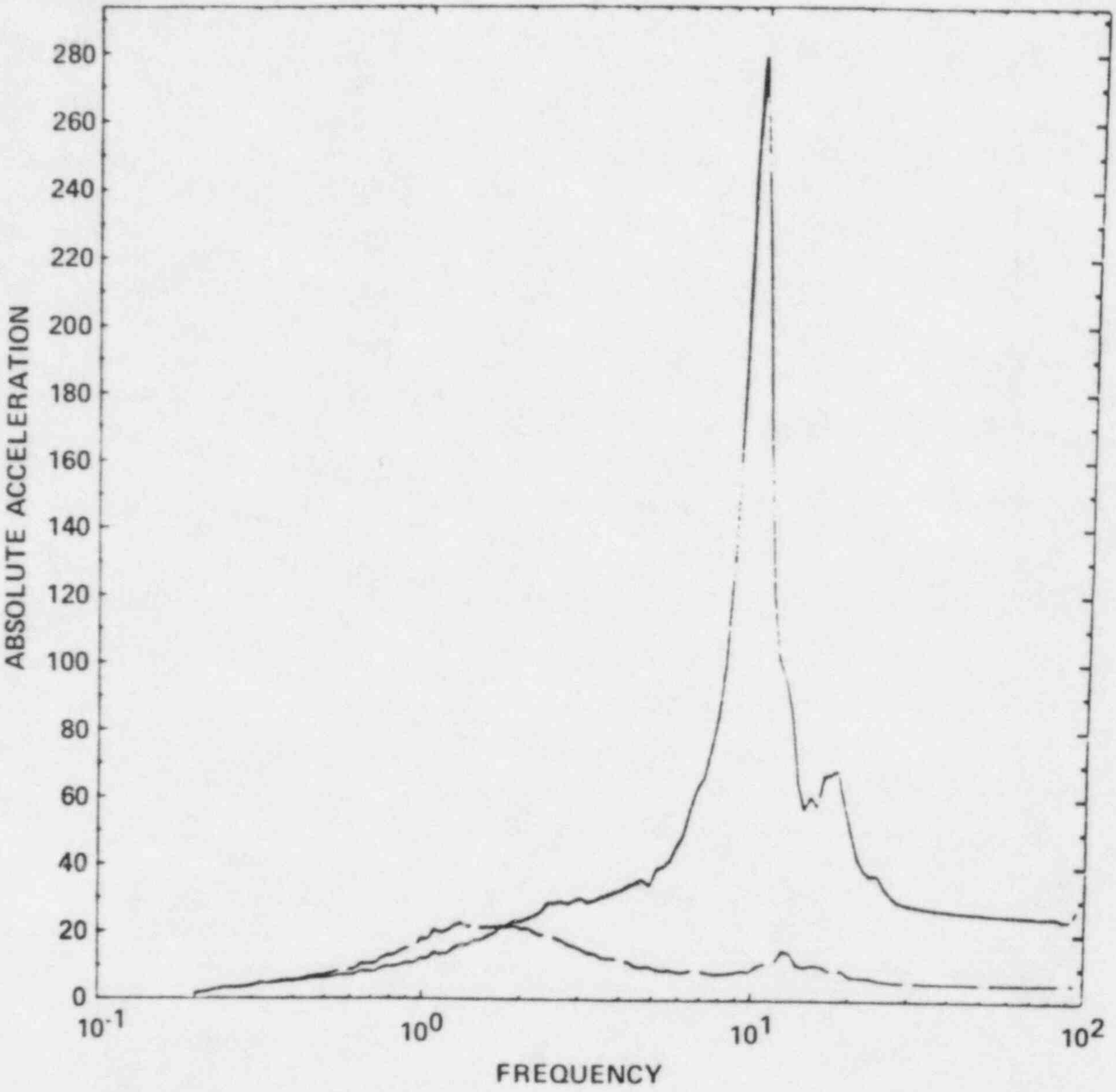
Mean response spectra - 2% damping
Node 256, X (E-W); aux bldg. el. 666
Uniform half space; Vs = 2000 fps
Fixed base ————————
Surface foundation —— —— ——

Fig. A.12b Comparison of in-structure response spectra at node 256; fixed-base vs. surface foundation, shear wall structure; uniform half space, Vs = 2000 fps case 1/case 3.



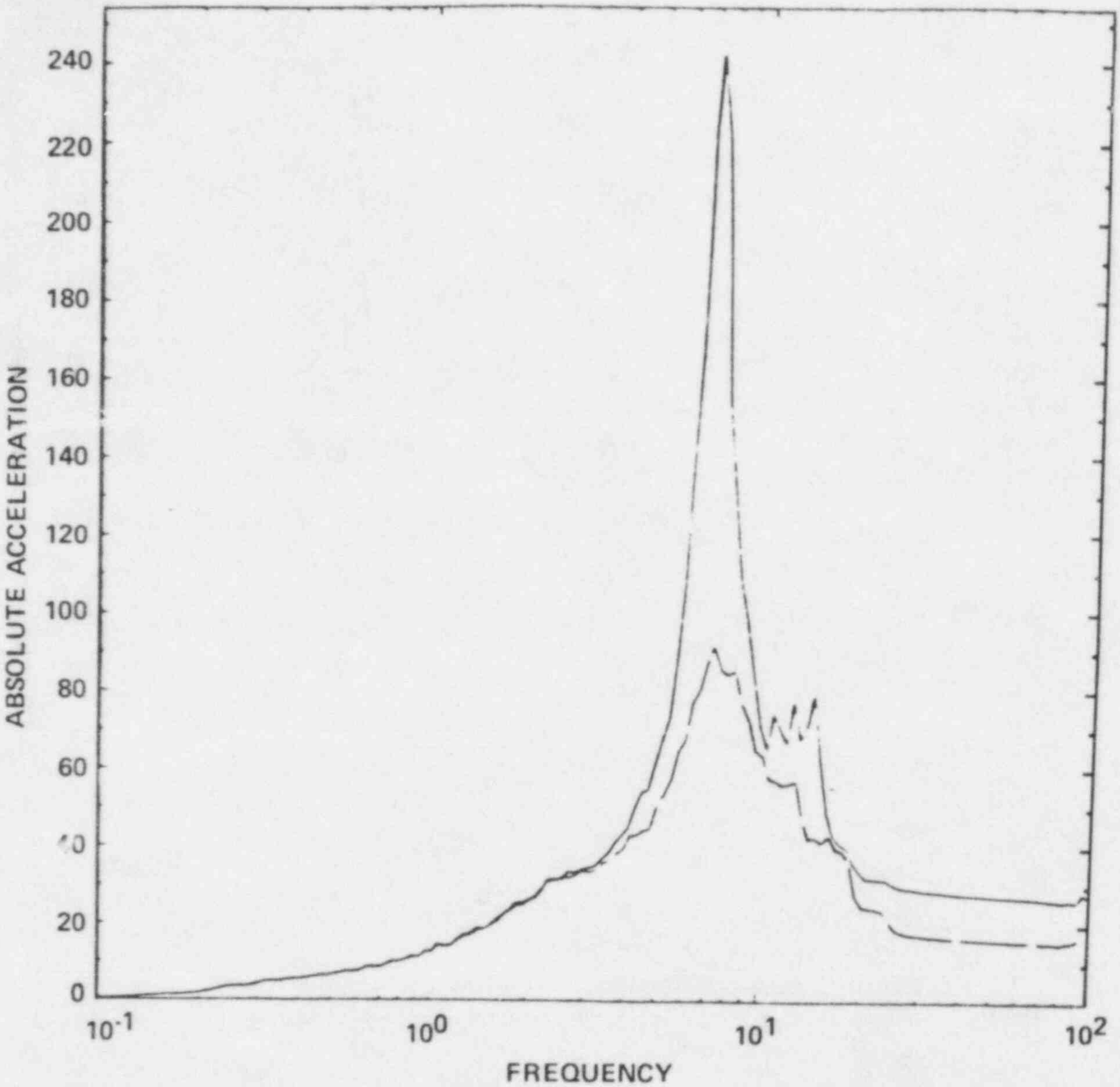
Mean response spectra - 2% damping
Node 256, X (E-W); aux bldg. el. 666
Uniform half space; Vs = 1000 fps
Fixed base ————————
Surface foundation — — —

Fig. A.12c Comparison of in-structure response spectra at node 256; fixed-base vs. surface foundation, shear wall structure; uniform half space, Vs = 1000 fps case 1/case 4.



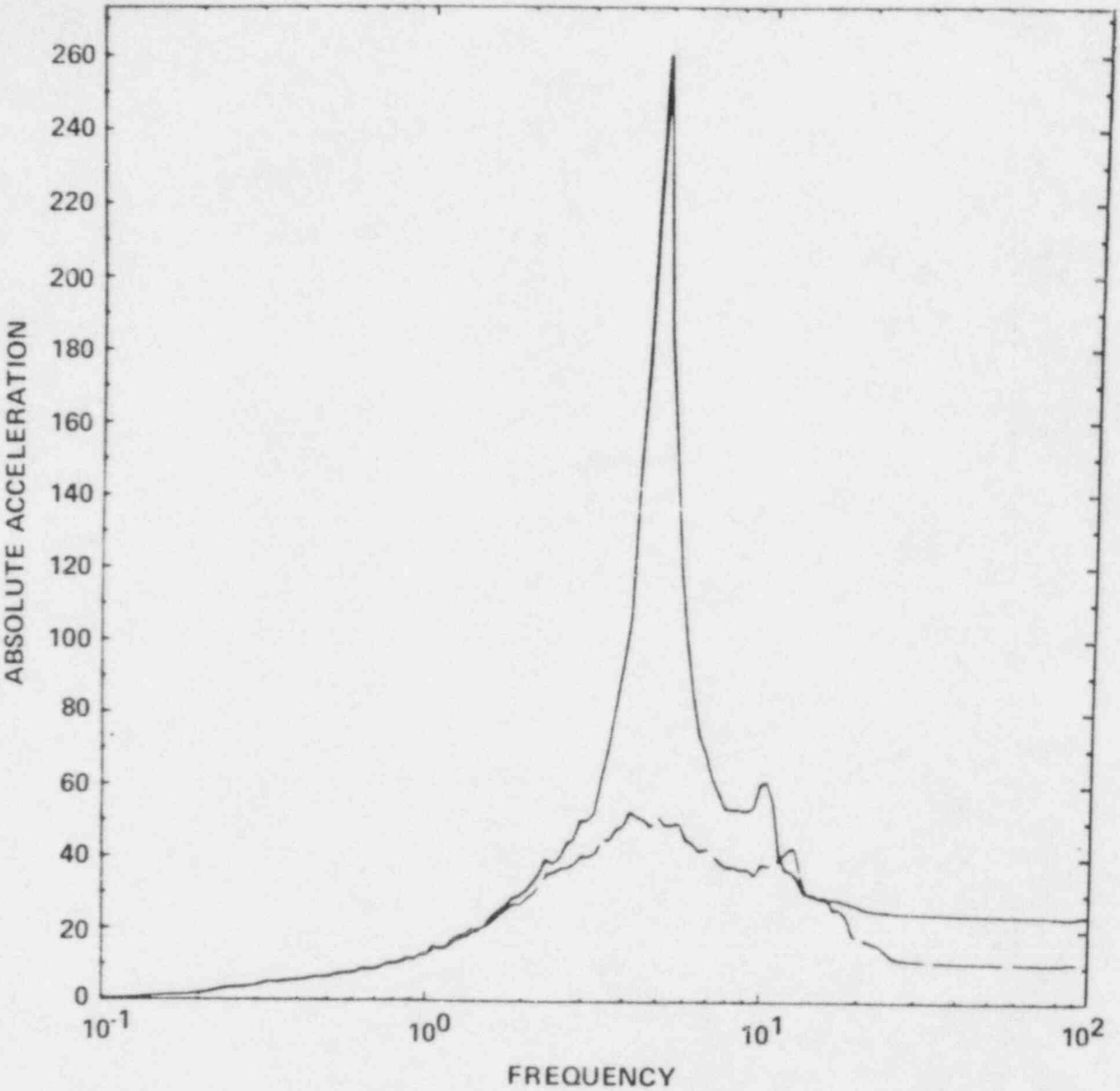
Mean response spectra – 2% damping
Node 256, X (E-W); aux bldg. el. 666
Uniform half space; Vs = 500 fps
Fixed base ————————
Surface foundation —— —— ——

Fig. A.12d Comparison of in-structure response spectra at node 256; fixed-base vs. surface foundation, shear wall structure; uniform half space, Vs = 500 fps case 1/case 5.



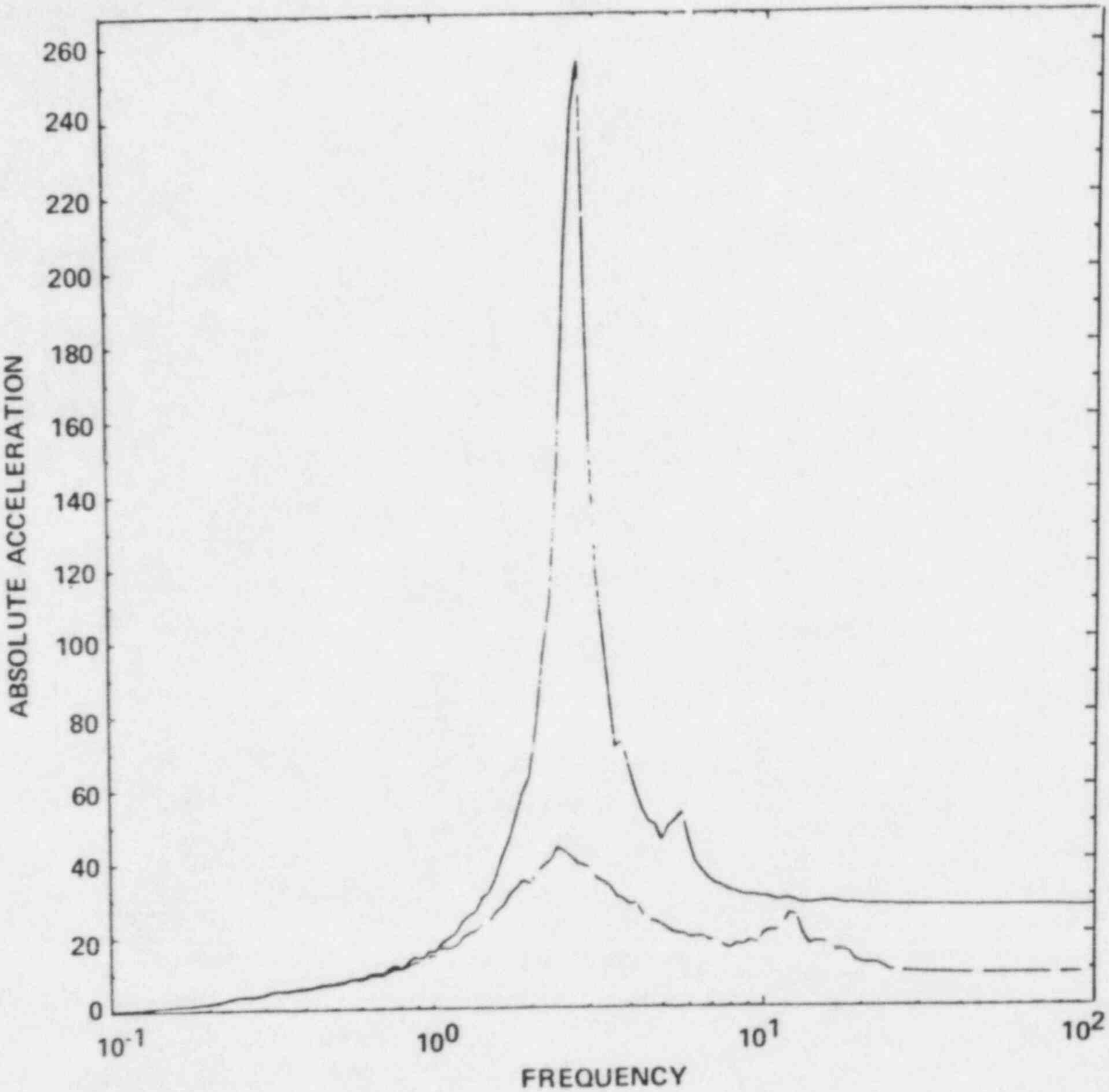
Mean response spectra - 2% damping
Node 256, X (E-W); aux bldg. el. 666
Uniform half space; Vs = 3500 fps
Soil springs ————————
Surface foundation —— —— ——

Fig. A.13a Comparison of in-structure response spectra at node 256; soil spring vs. surface foundation, shear wall structure; uniform half space, Vs = 3500 fps case 16/case 2.



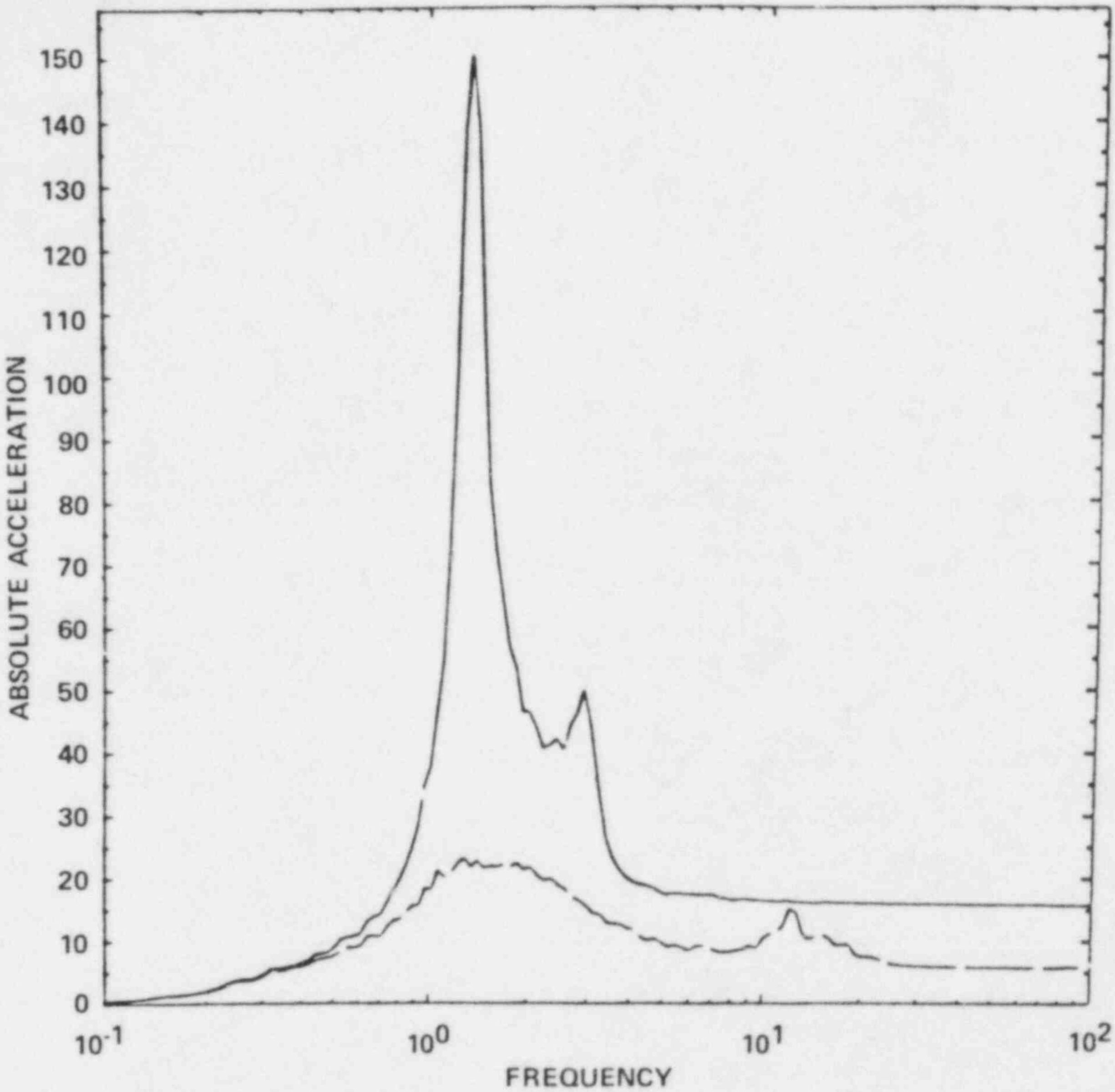
Mean response spectra - 2% damping
Node 256, X (E-W); aux bldg. el. 666
Uniform half space; Vs = 2000 fps
Soil springs ————————
Surface foundation — — —

Fig. A.13b Comparison of in-structure response spectra at node 256; soil spring vs. surface foundation, shear wall structure; uniform half space Vs = 2000 fps case 17/case 3.



Mean response spectra – 2% damping
Node 256, X (E-W); aux bldg. el. 666
Uniform half space; Vs = 1000 fps
Soil springs ————
Surface foundation — — —

Fig. A.13c Comparison of in-structure response spectra at node 256; soil spring vs. surface foundation, shear wall structure; uniform half space, V_s = 1000 fps case 18/case 4.



Mean response spectra - 2% damping
Node 256, X (E-W); aux bldg. el. 666
Uniform half space; Vs = 500 fps
Soil springs ————————
Surface foundation — — —

Fig. A.13d Comparison of in-structure response spectra at node 256; soil spring vs. surface foundation, shear wall structure; uniform half space, Vs = 500 fps case 19/case 5.

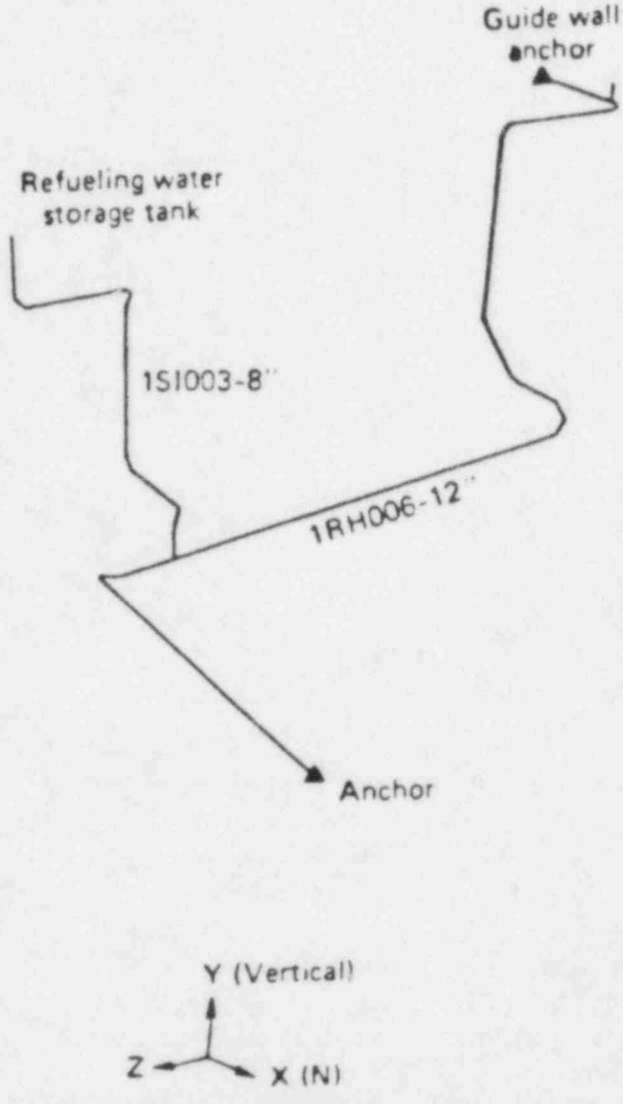


Figure A.14   Mathematical model of the RHR and the SI-1 piping configuration.

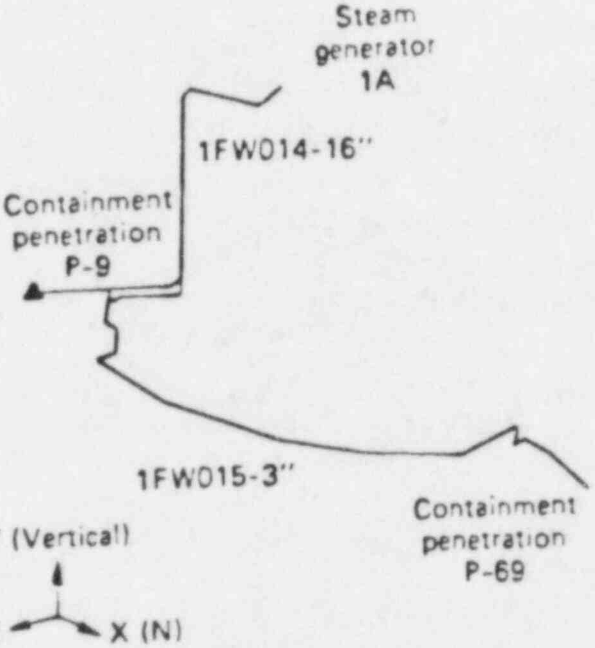


Figure A.15 Mathematical model of the AFW (inside containment) piping configuration.

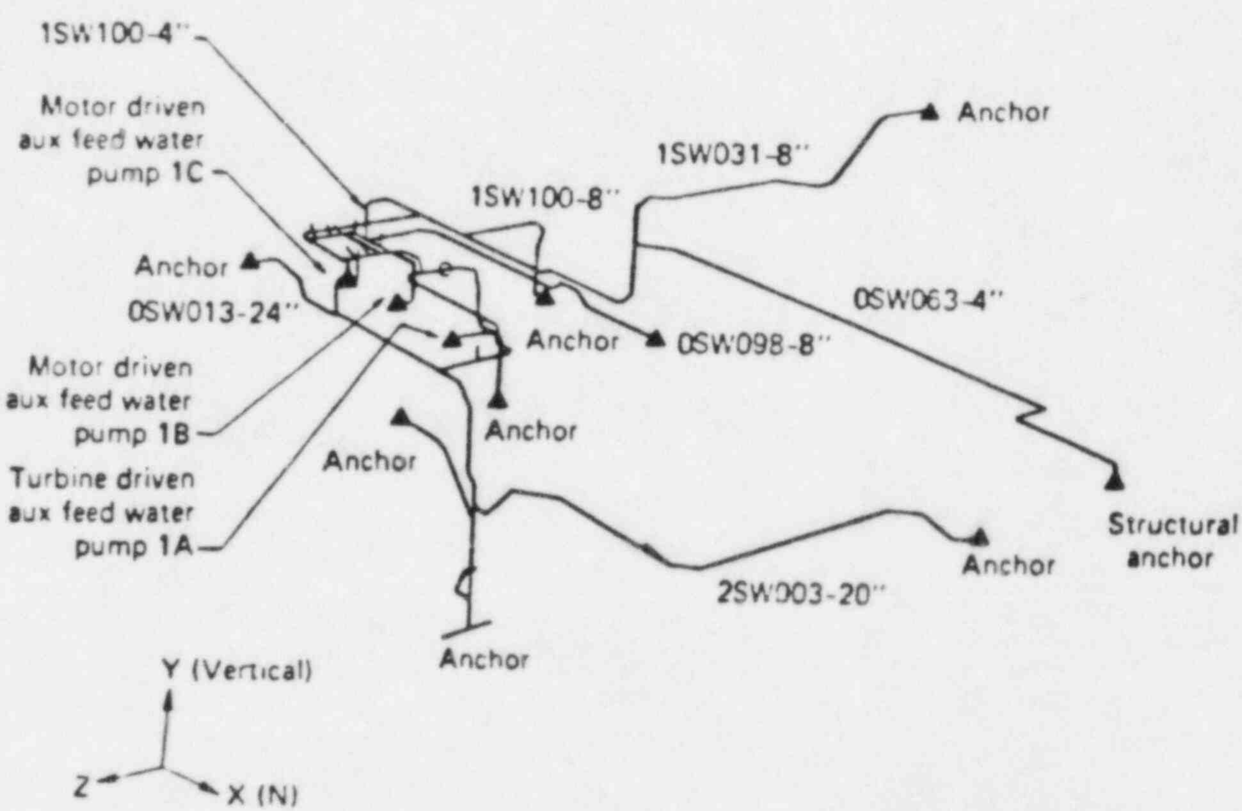


Figure A. 16 Mathematical model of the SW to the AFW pumps piping configuration.

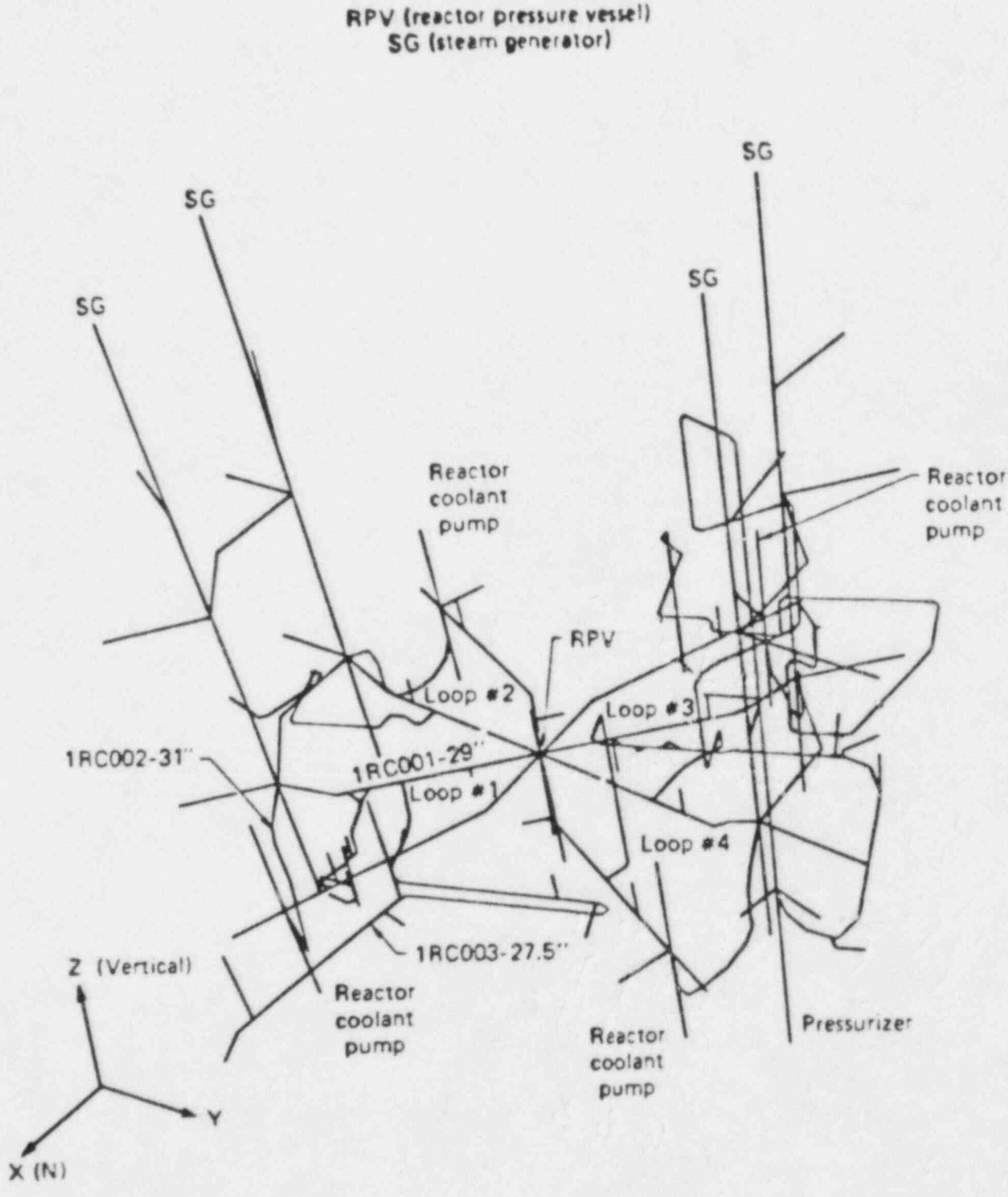


Figure A.17 Mathematical model of the reactor coolant loop piping configuration.

Table A.1a Nodes for calculation of peak accelerations and in-structure response spectra

Node Number	Location	Coordinate (ft) X	Y	Z	Output Components
13	Operating Floor	0.0	0.0	617.	x,y
16	Mid-Ht. of Shell	0.0	0.0	680.	x,y
20	Ring Girder	0.0	0.0	754.	x,y,z
21	Top of Dome	0.0	0.0	778.	z
936	Pressurizer-Op. Floor	25.6	44.3	617.	x,y,z
1212	NE Steam Generator Bottom	14.0	-42.1	588.1	x,y
1219	NE Steam Generator Bottom	14.0	-42.1	615.3	x,y
1418	SW Steam Generator Top	-3.0	34.9	615.3	x,y
1419	SW Steam Generator Top	-20.0	42.1	615.3	x,y
1410	SW Steam Generator Vert.	-15.1	30.0	568.0	z
1411	SW Steam Generator Bottom	-3.0	34.9	588.1	x,y
1412	SW Steam Generator Bottom	-20.0	42.1	588.1	x,y
1413	SW Steam Generator Cent.	-15.1	30.0	588.1	z
1367	RPV Center EL584	-3.9	0.0	584.0	x,y,z,xx,yy,zz
1462	R-V Nozzle Restraint	-3.0	9.9	581.9	x,y
1463	RPV Vertical Restraint	-6.4	8.5	579.8	z
1362	RPV Nozzle Restraint	-13.0	0.0	581.9	x,y
1363	RPV Vertical Restraint	-11.4	-3.6	579.8	z
1538	NW RCP Vertical Restraint	25.2	23.2	568.0	z
1540	NW RCP Center-Vertical	25.2	23.2	584.0	z
1541	NW RCP Tang. Restraint	30.3	18.4	584.0	x,y
1542	NW RCP Radial Restraint	30.0	28.3	584.0	x,y
1338	SE RCP Vertical Restraint	-31.2	-23.2	568.0	z
1340	SE RCP Center Vertical	-31.2	-23.2	584.0	z
1341	SE RCP Tang. Restraint	-36.3	-18.4	584.0	x,y
1342	SE RCP Radial Restraint	-36.0	-28.3	584.0	x,y

Table A.1b Elements for calculation of force, stress and moment response

Element No.	Location	Output Component
2	Shell, Near Base	1, 2, 3, 5, 6
5	Shell, Operating Floor	1, 2, 3, 5, 6
11	Shell, Ring Girder	7, 8, 9, 11, 12
6	Shield Wall-Base	3, 5
12	Shield Wall-Base	3, 5
18	Shield Wall-Base	3, 5
24	Shield Wall-Base	3, 5
41	Ring Wall-Base	3
45	Ring Wall-Base	3
52	Ring Wall-Base	3
61	Ring Wall-Base	3
71	Ring Wall-Base	3
80	Ring Wall-Base	3
87	Ring Wall-Base	3
91	Ring Wall-Base	3
124	Shield Wall-Upper	3, 5
130	Shield Wall-Upper	3, 5
136	Shield Wall-Upper	3, 5
142	Shield Wall-Upper	3, 5
166	Ring Wall-Mid	3
170	Ring Wall-Mid	3
175	Ring Wall-Mid	3
200	Ring Wall-Mid	3
203	Ring Wall-Mid	3
207	Ring Wall-Mid	3
214	Ring Wall-Mid	3
227	Ring Wall-Mid	3
231	Ring Wall-Mid	3
236	Ring Wall-Mid	3
333	Ring Wall-Upper	3
344	Ring Wall-Upper	3
377	Pool Wall	5
388	Pool Wall	5
482	Operating Floor	5
500	Operating Floor	5
529	Pressurizer Wall	3

Note: Beam Element Stress Component

1, 7 - Axial force
2, 8 - Shear force
3, 9 - Shear force
4, 10 - Torque
5, 11 - Bending moment
6, 12 - Bending moment

Plate Element Stress Component

3 - xy Stress
5 - yy Moment Resultant

Table A.2 Identification of physical and analysis scenarios for the containment building

Case No.	Soil Profile	V_s (fps)	Foundation Condition	Analysis Techniques
1	Fixed Base	Fixed Base	Surface	Best Estimate
2	Half-space	3500	Surface	Best Estimate
3	Half-space	2000	Surface	Best Estimate
4	Half-space	1000	Surface	Best Estimate
5	Half-space	500	Surface	Best Estimate
6	Half-space	3500	Embedded E/R=0.46	Best Estimate
7	Half-space	2000	Embedded E/R=0.46	Best Estimate
8	Half-space	1000	Embedded E/R=0.46	Best Estimate
9	Half-space	500	Embedded E/R=0.46	Best Estimate
10	Half-space	1000	Embedded E/R=0.75	Best Estimate
11	36 ft. soil/rock	1000/5000	Surface	Best Estimate
12	36 ft. soil/rock	1000/5000	Embedded E/R=0.46	Best Estimate
13	110 ft. soil/rock	1000/5000	Surface	Best Estimate
14	110 ft. soil/rock	1000/5000	Embedded E/R=0.46	Best Estimate
15	110 ft. soil/rock	1000/5000	Embedded E/R=0.75	Best Estimate
16	250 ft. soil/rock	1000/5000	Surface	Best Estimate
17	250 ft. soil/rock	1000/5000	Embedded E/R=0.46	Best Estimate
18	250 ft. soil/rock	1000/5000	Embedded E/R=0.75	Best Estimate
19	110 ft. soil/rock	1000/9000	Surface	Best Estimate
20	110 ft. soil/rock	1000/9000	Embedded E/R=0.46	Best Estimate
21	110 ft. soil/rock	2000/5000	Surface	Best Estimate
22	110 ft. soil/rock	2000/5000	Embedded E/R=0.46	Best Estimate
23	Zion soil/rock (3 layers)	600-910-1390/9000	Embedded E/R=0.46	Best Estimate
24a	110 ft. soil (3 layers	1600	Surface	6 Component Soil Springs
24b	110 ft. soil (3 layers)	1600	Surface	Only Rocking & Torsion Springs
25	Half-space	2000	Surface	Soil Springs (6 Components)
26	Half-space	1000	Surface	
27	Half-space	500	Surface	

Table A.3 Response ratios of forces and peak accelerations for surface vs. embedded foundation conditions

Soil Profile	Embedment Ratio (E/R)	Characteristic V_s (fps)	Case Comparison	Peak Accelerations		Forces	
				Mean	COV	Mean	COV
Half Space	.46	3500	2/6	1.22	.244	1.19	.175
Half Space	.46	2000	3/7	1.29	.202	1.25	.101
Half Space	.46	1000	4/8	1.25	.119	1.22	.107
Half Space	.46	500	5/9	1.29	.145	1.28	.120
Half Space	.75	1000	4/10	1.37	.157	1.33	.148
36 Ft Layer	.46	1000/5000	11/12	1.87	.641	1.63	.223
110 Ft Layer	.46	1000/5000	13/14	1.30	.138	1.26	.134
110 Ft Layer	.75	1000/5000	13/15	1.55	.153	1.52	.143
250 Ft Layer	.46	1000/5000	16/17	1.26	.120	1.25	.107
250 Ft Layer	.75	1000/5000	16/18	1.40	.158	1.37	.144
110 Ft Layer	.46	1000/9000	19/20	1.30	.149	1.27	.136
110 Ft Layer	.46	2000/5000	21/22	1.35	.171	1.29	.099

Table A.4a Response ratios of spectra accelerations for surface vs. embedded foundation conditions containment building; half-space and layered sites. Horizontal Responses

Soil Profile	Embedment Ratio (E/R)	Characteristic V_s (fps)	Case Comparison	0 - 6 Hz Mean	COV	6 - 12 Hz Mean	COV	12 - 20 Hz Mean	COV	ZPA Mean	COV
Half Space	.46	3500	2/6	1.13	.033	1.35	.073	1.14	.112	1.20	.079
Half Space	.46	2000	3/7	1.17	.051	1.39	.076	1.22	.107	1.26	.059
Half Space	.46	1000	4/8	1.24	.088	1.28	.085	1.21	.110	1.28	.081
Half Space	.46	500	5/9	1.30	.109	1.13	.086	1.24	.119	1.36	.099
Half Space	.75	1000	4/10	1.43	.117	1.26	.111	1.36	.140	1.39	.108
36 Ft Layer	.46	1000/5000	11/12	1.71	.139	1.47	.201	.89	.361	1.62	.185
110 Ft Layer	.46	1000/5000	13/14	1.26	.106	1.31	.118	1.16	.141	1.29	.113
110 Ft Layer	.75	1000/5000	13/15	1.42	.135	1.52	.124	1.39	.132	1.53	.112
250 Ft Layer	.46	1000/5000	16/17	1.22	.088	1.29	.086	1.19	.118	1.30	.078
250 Ft Layer	.75	1000/5000	16/18	1.38	.121	1.45	.111	1.29	.141	1.43	.102
110 Ft Layer	.46	1000/9000	19/20	1.27	.107	1.32	.121	1.15	.144	1.29	.119
110 Ft Layer	.46	2000/5000	21/22	1.16	.062	1.32	.083	1.22	.128	1.33	.062

Table A.4b Response ratios of spectra accelerations for surface vs. embedded foundation conditions containment building; half-space and layered sites. Vertical responses

Soil Profile	Embedment Ratio (E/R)	Characteristic V_s (fps)	Case Comparison	0 - 6 Hz Mean	0 - 6 Hz COV	6 - 12 Hz Mean	6 - 12 Hz COV	12 - 20 Hz Mean	12 - 20 Hz COV	ZPA Mean	ZPA COV
Half Space	.46	3500	2/6	1.04	.016	1.15	.050	1.03	.082	1.07	.076
Half Space	.46	2000	3/7	1.08	.034	1.03	.076	1.03	.099	1.10	.102
Half Space	.46	1000	4/8	1.10	.056	.95	.076	1.12	.111	1.11	.088
Half Space	.46	500	5/9	1.08	.081	1.02	.078	1.11	.093	1.09	.097
Half Space	.75	1000	4/10	1.13	.068	1.27	.092	1.21	.113	1.20	.091
36 Ft Layer	.46	1000/5000	11/12	1.23	.081	1.99	.169	1.35	.311	1.82	.183
110 Ft Layer	.46	1000/5000	13/14	1.16	.089	1.04	.119	1.24	.141	1.30	.137
110 Ft Layer	.75	1000/5000	13/15	1.27	.099	.99	.128	1.52	.145	1.51	.143
250 Ft Layer	.46	1000/5000	16/17	1.09	.054	.94	.076	1.10	.111	1.09	.093
250 Ft Layer	.75	1000/5000	16/18	1.14	.069	1.03	.095	1.25	.113	1.18	.091
110 Ft Layer	.46	1000/9000	19/20	1.17	.092	1.03	.125	1.25	.155	1.29	.142
110 Ft Layer	.46	2000/5000	21/22	1.09	.040	1.07	.117	1.04	.102	1.20	.115

Table A.5 Response ratios of forces and peak accelerations for fixed-base vs. surface and embedded foundations at half-space and layered sites

Soil Profile	Embedment Ratio (E/R)	Characteristic V_s (fps)	Case Comparison	Peak Accelerations Mean	Peak Accelerations COV	Forces Mean	Forces COV
Half Space	0	3500	1/2	1.20	.309	1.16	.205
Half Space	0	2000	1/3	1.37	.448	1.36	.277
Half Space	0	1000	1/4	1.82	.606	2.02	.341
Half Space	0	500	1/5	2.54	.775	2.93	.386
Half Space	0.46	3500	1/6	1.38	.257	1.35	.185
Half Space	0.46	2000	1/7	1.69	.479	1.68	.239
Half Space	0.46	1000	1/8	2.21	.580	2.42	.303
Half Space	0.46	500	1/9	3.11	.674	3.67	.350
Half Space	0.75	1000	1/10	2.37	.542	2.61	.286
36 Ft Layer	0	1000/5000	1/11	1.19	.809	1.20	.601
36 Ft Layer	0.46	1000/5000	1/12	1.61	.287	1.82	.237
110 Ft Layer	0	1000/5000	1/13	1.54	.683	1.60	.342
110 Ft Layer	0.46	1000/5000	1/14	1.89	.572	1.99	.300
110 Ft Layer	0.75	1000/5000	1/15	2.26	.597	2.39	.300
250 Ft Layer	0	1000/5000	1/16	1.78	.614	1.95	.345
250 Ft Layer	0.46	1000/5000	1/17	2.17	.569	2.40	.304
250 Ft Layer	0.75	1000/5000	1/18	2.37	.552	2.60	.289
110 Ft Layer	0	1000/9000	1/19	1.50	.684	1.56	.347
110 Ft Layer	0.46	1000/9000	1/20	1.84	.552	1.96	.295
110 Ft Layer	0	2000/5000	1/21	1.22	.507	1.21	.291
110 Ft Layer	0.46	2000/5000	1/22	1.58	.516	1.54	.265
110 Ft Layer	0.46	600/910/ 1390/9000	1/23	2.00	1.215	1.75	.387

Table A.6a Response ratios of spectra accelerations for fixed base vs. surface & embedded foundations containment building; half-space and layered sites. Horizontal responses.

Soil Profile	Embedment Ratio (E/R)	Characteristic V_s (fps)	Case Comparison	0 - 6 Hz Mean	0 - 6 Hz COV	6 - 12 Hz Mean	6 - 12 Hz COV	12 - 20 Hz Mean	12 - 20 Hz COV	ZPA Mean	ZPA COV
Half Space	0	3500	1/2	.99	.066	.93	.139	1.64	.235	1.14	.179
Half Space	0	2000	1/3	1.02	.135	1.14	.188	2.36	.317	1.26	.253
Half Space	0	1000	1/4	1.17	.278	2.01	.249	3.71	.410	1.66	.403
Half Space	0	500	1/5	1.57	.400	3.53	.264	5.64	.536	2.20	.505
Half Space	.46	3500	1/6	1.10	.044	1.23	.074	1.80	.161	1.36	.156
Half Space	.46	2000	1/7	1.16	.091	1.56	.131	2.80	.265	1.59	.253
Half Space	.46	1000	1/8	1.39	.184	2.51	.185	4.33	.345	2.08	.347
Half Space	.46	500	1/9	1.98	.289	3.96	.257	6.61	.431	2.91	.450
Half Space	.75	1000	1/10	1.48	.133	2.76	.155	4.43	.331	2.25	.333
36 Ft Layer	0	1000/5000	1/11	.90	.204	1.69	.220	2.56	.516	1.03	.422
36 Ft Layer	.46	1000/5000	1/12	1.31	.039	2.35	.142	1.81	.163	1.57	.169
110 Ft Layer	0	1000/5000	1/13	1.07	.242	1.81	.194	3.27	.447	1.38	.404
110 Ft Layer	.46	1000/5000	1/14	1.26	.169	2.34	.145	3.57	.357	2.31	.328
110 Ft Layer	.75	1000/5000	1/15	1.37	.129	2.67	.134	4.43	.392	2.08	.363
250 Ft Layer	0	1000/5000	1/16	1.17	.278	1.97	.239	3.63	.414	1.60	.404
250 Ft Layer	.46	1000/5000	1/17	1.36	.188	2.48	.194	4.13	.334	2.05	.351
250 Ft Layer	.75	1000/5000	1/18	1.50	.134	2.76	.160	4.42	.329	2.24	.338

Table A.6a (cont.) Response ratios of spectra accelerations for fixed base vs. surface & embedded foundations containment building; half-space and layered sites. Horizontal responses.

Soil Profile	Embedment Ratio (E/R)	Characteristic V_s (fps)	Case Comparison	0 - 6 Hz Mean	0 - 6 Hz COV	6 - 12 Hz Mean	6 - 12 Hz COV	12 - 20 Hz Mean	12 - 20 Hz COV	ZPA Mean	ZPA COV
110 Ft Layer	0	1000/9000	1/19	1.06	.238	1.78	.193	3.22	.451	1.36	.404
110 Ft Layer	.46	1000/9000	1/20	1.26	.167	2.32	.145	3.50	.357	1.71	.328
110 Ft Layer	0	2000/5000	1/21	.98	.118	1.11	.133	2.54	.346	1.10	.265
110 Ft Layer	.46	2000/5000	1/22	1.09	.074	1.44	.093	2.64	.281	1.46	.260
110 Ft Layer	.46	600/910 1390/9000	1/23	1.16	.131	2.21	.165	3.84	.483	1.51	.394

Table A.6b Response ratios of spectra accelerations for fixed base vs. surface & embedded foundations containment building; half-space and layered sites. Vertical responses.

Soil Profile	Embedment Ratio (E/R)	Characteristic V_s (fps)	Case Comparison	0 - 6 Hz Mean	0 - 6 Hz COV	6 - 12 Hz Mean	6 - 12 Hz COV	12 - 20 Hz Mean	12 - 20 Hz COV	ZPA Mean	ZPA COV
Half Space	0	3500	1/2	.97	.019	.99	.098	1.54	.223	1.28	.254
Half Space	0	2000	1/3	.95	.038	1.35	.126	1.90	.263	1.47	.373
Half Space	0	1000	1/4	.95	.064	2.04	.147	2.53	.299	1.85	.437
Half Space	0	500	1/5	1.05	.093	3.23	.186	3.75	.336	2.54	.497
Half Space	.46	3500	1/6	1.01	.013	1.12	.076	1.57	.192	1.35	.220
Half space	.46	2000	1/7	1.01	.023	1.35	.098	1.92	.248	1.56	.293
Half Space	.46	1000	1/8	1.02	.042	1.93	.132	2.80	.294	2.03	.422
Half Space	.46	500	1/9	1.10	.062	3.30	.181	4.15	.344	2.74	.486
Half Space	.75	1000	1/10	1.05	.037	2.12	.144	3.15	.294	2.22	.445
36 Ft Layer	0	1000/5000	1/11	.89	.064	1.18	.291	2.42	.488	1.08	.484
36 Ft Layer	.46	1000/5000	1/12	1.02	.010	1.45	.123	2.80	.334	1.85	.391
110 Ft Layer	0	1000/5000	1/13	.91	.079	1.90	.190	2.26	.307	1.50	.467
110 Ft Layer	.46	1000/5000	1/14	.99	.047	1.95	.131	2.74	.269	1.90	.440
110 Ft Layer	.75	1000/5000	1/15	1.06	.037	1.91	.149	3.35	.285	2.21	.446
250 Ft Layer	0	1000/5000	1/16	.95	.066	2.04	.146	2.56	.301	1.86	.444
250 Ft Layer	.46	1000/5000	1/17	1.02	.043	1.92	.130	2.77	.290	2.00	.423
250 Ft Layer	.75	1000/5000	1/18	1.05	.037	2.11	.147	3.16	.296	2.19	.446

Table A.6b (cont.) Response ratios of spectra accelerations for fixed base vs. surface & embedded foundations containment building; half-space and layered sites. Vertical responses.

Soil Profile	Embedment Ratio (E/R)	Characteristic V_s (fps)	Case Comparison	0 - 6 Hz		6 - 12 Hz		12 - 20 Hz		ZPA	
				Mean	COV	Mean	COV	Mean	COV	Mean	COV
110 Ft Layer	0	1000/9000	1/19	.90	.081	1.90	.195	2.21	.305	1.46	.470
110 Ft Layer	.46	1000/9000	1/20	.98	.047	1.95	.133	2.69	.266	1.82	.436
110 Ft Layer	0	2000/5000	1/21	.93	.038	1.39	.185	1.80	.279	1.32	.398
110 Ft Layer	.46	2000/5000	1/22	.99	.023	1.37	.117	1.86	.271	1.54	.342
110 Ft Layer	.46	600/910/ 1390/9000	1/23	.97	.048	2.13	.197	3.39	.334	1.99	.496

Table A.7 Response ratios of peak accelerations for soil springs vs. surface and embedded foundations at half-space and layered sites

Soil Profile	Embedment Ratio (E/R)	Characteristic V_s (fps)	Case Comparison	Peak Accelerations Mean	COV
Half Space	0	2000	25/3	1.93	.367
Half Space	0	1000	26/4	2.57	.391
Half Space	0	500	27/5	3.46	.369
Half Space	0.46	2000	25/7	2.33	.324
Half Space	0.46	1100	26/8	3.13	.351
Half Space	0.46	500	27/9	3.46	.369
Half Space	0.75	1000	26/10	3.31	.332
36 Ft Layer	0	1000/5000	26/11	1.56	.355
36 Ft Layer	0.46	1000/5000	26/12	2.54	.370
110 Ft Layer	0	1000/5000	26/13	2.11	.372
110 Ft Layer	0.46	1000/5000	26/14	2.68	.375
110 Ft Layer	0.75	1000/5000	26/15	3.16	.350
250 Ft Layer	0	1000/5000	26/16	2.51	.400
250 Ft Layer	0.46	1000/5000	26/17	3.08	.350
250 Ft Layer	0.75	1000/5000	26/18	3.27	.330
110 Ft Layer	0	1000/9000	26/19	2.07	.371
110 Ft Layer	0.46	1000/9000	26/20	2.65	.370
110 Ft Layer	0	2000/5000	25/21	1.69	.375
110 Ft Layer	0.46	2000/5000	25/22	2.17	.355
110 Ft Layer	0.46	600/910/ 1390/9000	24a/23		

Table A.8a Response ratios of spectra accelerations for soil springs vs. surface & embedded foundations containment building; half-space and layered sites. Horizontal responses

Soil Profile	Embedment Ratio (E/R)	Characteristic V_s (fps)	Case Comparison	0 - 6 Hz Mean	0 - 6 Hz COV	6 - 12 Hz Mean	6 - 12 Hz COV	12 - 20 Hz Mean	12 - 20 Hz COV	ZPA Mean	ZPA COV
Half Space	0	2000	25/3	1.10	.097	2.28	.355	1.59	.250	1.81	.288
Half Space	0	1000	26/4	1.62	.202	2.06	.322	2.27	.253	2.38	.317
Half Space	0	500	27/5	1.91	.288	2.34	.232	2.55	.262	2.68	.374
Half Space	.46	2000	25/7	1.28	.100	3.16	.347	1.92	.244	2.67	.293
Half Space	.46	1000	26/8	2.06	.202	2.23	.304	2.46	.244	3.00	.310
Half Space	.46	500	27/9	2.55	.286	2.63	.242	3.11	.260	3.56	.347
Half Space	.75	1000	26/10	2.37	.221	2.48	.301	2.55	.247	3.25	.313
36 Ft Layer	0	1000/5000	26/11	1.23	.209	1.49	.288	1.36	.239	1.48	.320
36 Ft Layer	.46	1000/5000	26/12	2.22	.266	2.20	.309	1.15	.330	2.30	.276
110 Ft Layer	0	1000/5000	26/13	1.44	.174	1.59	.286	1.79	.230	1.98	.309
110 Ft Layer	.46	1000/5000	26/14	1.91	.217	2.05	.284	2.04	.241	2.51	.314
110 Ft Layer	.75	1000/5000	26/15	2.22	.240	2.41	.299	2.48	.254	3.02	.339
250 Ft Layer	0	1000/5000	26/16	1.61	.204	1.73	.311	2.01	.246	2.30	.309
250 Ft Layer	.46	1000/5000	26/17	2.02	.200	2.20	.308	2.37	.254	2.96	.308
250 Ft Layer	.75	1000/5000	26/18	2.41	.219	2.47	.299	2.55	.249	3.24	.311
110 Ft Layer	0	1000/9000	26/19	1.43	.172	1.57	.285	1.75	.230	1.94	.306
110 Ft Layer	.46	1000/9000	26/20	1.92	.218	2.04	.284	2.00	.245	2.47	.314
110 Ft Layer	0	2000/5000	25/21	1.05	.077	2.21	.353	1.49	.254	1.51	.295
110 Ft Layer	.46	2000/5000	25/22	1.22	.105	2.93	.354	1.78	.236	2.08	.304
110 Ft Layer	.46	600/910 1390/9000	24a/23	1.26	.093	4.70	.405	2.38	.312	2.09	.336

Table A.8b Response ratios of spectra accelerations for soil springs vs. surface & embedded foundations containment Building; Half-Space and Layered Sites. Vertical responses.

Soil Profile	Embedment Ratio (E/R)	Characteristic V_s (fps)	Case Comparison	0 - 6 Hz Mean	0 - 6 Hz COV	6 - 12 Hz Mean	6 - 12 Hz COV	12 - 20 Hz Mean	12 - 20 Hz COV	ZPA Mean	ZPA COV
Half Space	0	2000	25/3	1.07	.078	3.59	.386	1.40	.432	2.30	.452
Half Space	0	1000	26/4	1.90	.202	2.00	.312	1.56	.486	3.19	.441
Half Space	0	500	27/5	2.06	.269	1.85	.289	1.50	.483	2.78	.430
Half Space	.46	2000	25/7	1.16	.078	3.77	.388	1.43	.412	2.44	.385
Half Space	.46	1000	26/8	2.08	.203	1.89	.308	1.72	.467	3.50	.416
Half Space	.46	500	27/9	2.22	.267	1.89	.295	1.65	.476	3.02	.418
Half Space	.75	1000	26/10	2.16	.205	2.07	.310	1.94	.470	3.80	.422
36 Ft Layer	0	1000/5000	26/11	1.60	.207	.96	.278	1.35	.381	1.83	.383
36 Ft Layer	.46	1000/5000	26/12	2.07	.203	1.48	.316	1.73	.531	3.20	.402
110 Ft Layer	0	1000/5000	26/13	1.73	.196	1.90	.304	1.39	.459	2.56	.425
110 Ft Layer	.46	1000/5000	26/14	1.91	.205	1.94	.309	1.71	.461	3.26	.420
110 Ft Layer	.75	1000/5000	26/15	2.21	.207	1.84	.313	2.07	.450	3.79	.416
250 Ft Layer	0	1000/5000	26/16	1.90	.202	2.00	.315	1.57	.486	3.20	.448
250 Ft Layer	.46	1000/5000	26/17	2.06	.202	1.88	.309	1.71	.469	3.44	.416
250 Ft Layer	.75	1000/5000	26/18	2.16	.204	2.06	.308	1.94	.468	3.75	.421
110 Ft Layer	0	1000/9000	26/19	1.68	.195	1.91	.306	1.37	.459	2.45	.431
110 Ft Layer	.46	1000/9000	26/20	1.87	.205	1.94	.310	1.69	.460	3.14	.413
110 Ft Layer	0	2000/5000	25/21	1.03	.075	3.52	.396	1.32	.425	2.06	.441
110 Ft Layer	.46	2000/5000	25/22	1.14	.078	3.50	.384	1.38	.431	2.42	.435
110 Ft Layer	.46	600/910 1390/9000	24a/23	1.08	.111	6.11	.397	2.84	.711	3.12	.453

Table A.9 Response ratios of forces and peak accelerations 36 ft. soil layer sites, containment building.

Soil Profile	Embedment Ratio (E/R)	Characteristic V_s (fps)	Case Comparison	Peak Accelerations Mean	Peak Accelerations COV	Forces Mean	Forces COV
36 Ft Layer	0	1000/5000		1a/11a*	.72	.587	.78.305
36 Ft Layer	.46	1000/5000		1a/12a†	1.18	.072	1.25.083
36 Ft Layer	0	1000/5000		1a/11	.97	.543	1.12.342
36 Ft Layer	.46	1000/5000		1a/12	1.456	.161	1.73.212
Half Space	0	5000		1/1a	1.11	.207	1.07.168

* Case conditions are the same as Case 11, except the control motion is specified at the adjacent rock outcrop.

† Case conditions are the same as Case 12 except the control motion is specified at the adjacent rock outcrop.

Table A.10a Response ratios of spectra accelerations for 36 ft. soil layer sites containment building. Horizontal responses.

Soil Profile	Embedment Ratio (E/R)	Characteristic V_s (fps)	Case Comparison	0 - 6 Hz Mean	0 - 6 Hz COV	6 - 12 Hz Mean	6 - 12 Hz COV	12 - 20 Hz Mean	12 - 20 Hz COV	ZPA* Mean	ZPA* COV
36 Ft Layer	0	1000/5000	1a/11a	.82	.214	1.05	.197	.82	.419	.66	.304
36 Ft Layer	.46	1000/5000	1a/12a	1.14	.010	1.31	.050	1.22	.039	1.19	.049
36 Ft Layer	0	1000/5000	1a/11	.91	.178	1.86	.229	1.94	.395	.94	.299
36 Ft Layer	.46	1000/5000	1a/12	1.32	.023	2.54	.148	1.41	.091	1.45	.101
Half Space	0	5000	1/2a	.99	.041	.92	.095	1.29	.170	1.08	.127

* ZPA: For frequencies above 32 HZ, spectral acceleration approximately equals peak acceleration

Table A.10b Response ratios of spectra accelerations for 36 ft. soil layer sites containment building. Vertical responses.

Soil Profile	Embedment Ratio (E/R)	Characteristic V_s (fps)	Case Comparison	0 - 6 Hz Mean	0 - 6 Hz COV	6 - 12 Hz Mean	6 - 12 Hz COV	12 - 20 Hz Mean	12 - 20 Hz COV	ZPA* Mean	ZPA* COV
36 Ft Layer	0	1000/5000	1a/11a	.88	.075	.95	.216	1.01	.284	.71	.309
36 Ft Layer	.46	1000/5000	1a/12a	1.02	.004	1.15	.020	1.21	.056	1.13	.069
36 Ft Layer	0	1000/5000	1a/11	.90	.064	1.20	.241	1.26	.384	.85	.354
36 Ft Layer	.46	1000/5000	1a/12	1.04	.007	1.56	.083	2.13	.273	1.53	.268
Half Space	0	5000	1/1a	.99	.011	.92	.072	1.32	.187	1.06	.192

* ZPA: For frequencies above 32 HZ, spectral acceleration approximately equals to peak acceleration

Table A.11 Effect of Seismic Input Motions on Response Ratios of Peak Acceleration and Forces for the Cases of Surface Versus Embedded Foundations, Containment Building

			Peak Accelerations				Forces			
			R.G. 160		Site Specific		R.G. 160		Site Specific	
Soil Profile	Characteristic vs. (fps)	Cases Comparison	Mean	COV	Mean	COV	Mean	COV	Mean	COV
Half Space	3500	2/6	1.22	.244	1.22	.277	1.19	.175	1.18	.204
Half Space	2000	3/7	1.29	.202	1.26	.199	1.25	.101	1.22	.130
Half Space	1000	4/8	1.25	.119	1.25	.152	1.22	.107	1.23	.134
Half Space	500	5/9	1.29	.145	1.23	.160	1.28	.120	1.20	.150

Table A.12a Location of shear wall sections for response calculations.

Section	Location	Elevation
1	X = 180.0 ft.	666 Ft.
2	X = 180.0 ft.	642
3	X = 180.0 ft.	630
4	X = 180.0 ft.	617
5	X = 180.0 ft.	592
6	X = 180.0 ft.	579
7	X = 180.0 ft.	560
8	X = 248.0 ft.	666
9	X = 248.0 ft.	642
10	X = 248.0 ft.	630
11	X = 248.0 ft.	617
12	X = 248.0 ft.	592
13	X = 248.0 ft.	579
14	X = 248.0 ft.	560
15	X = 462.0 ft.	666
16	X = 462.0 ft.	642
17	X = 462.0 ft.	617
18	X = 43.5 ft.	666
19	X = 43.5 ft.	642
20	X = 43.5 ft.	617
21	X = 43.5 ft.	592
22	X = 43.5 ft.	579
23	X = 43.5 ft.	560
24	X = 133.0 ft.	666
25	X = 133.0 ft.	642
26	X = 133.0 ft.	630
27	X = 133.0 ft.	617
28	X = 133.0 ft.	592
29	X = 133.0 ft.	579
30	X = 133.0 ft.	560
31	X = 238.0 ft.	642
32	X = 238.0 ft.	630
33	X = 238.0 ft.	617
34	X = 311.0 ft.	560
35	X = 340.0 ft.	560

Table A.12b Node point locations for calculation of peak accelerations and generation of in-structure response spectra

Node Number	Coordinates X	Y	Z
10	229.5 ft.	0. ft.	592.0 ft
78	462.0	0.	617.0
147	222.0	197.0	617.0
188	462.0	0.	642.0
256	229.5	0.	666.0
303	222.0	197.0	666.0
672	229.5	0.	579.0

Table A.13 Identification of physical and analysis scenarios for the shear wall structure

Case No.	Soil Profile	V_s (fps)	Foundation Condition	Analysis Technique
1	Fixed-base	Fixed-base	Surface	Best Estimate
2	Half-space	3500		
3		2000		
4		1000		
5		500		
6		3500	Embedded	
7		2000		
8		1000		
9		500		
10	110 ft. layer	2000	Surface	
11		1000		
12		2000	Embedded	
13		1000		
14	71 ft. layer	2000	Surface	
15		1000		
16	Half-space	3500		Soil Springs
17		2000		
18		1000		
19		500		

Table A.14 Mean ratios of forces and peak accelerations for surface vs. embedded foundation conditions; shear wall structure (half-space and layered sites)

	Characteristic V_s (fps)	Case Comparison	Peak Accelerations Mean	Peak Accelerations COV	Peak Forces Mean	Peak Forces COV
Half-space	3500	2/6	1.25	.164	1.17	.054
	2000	3/7	1.26	.146	1.22	.067
	1000	4/8	1.31	.159	1.29	.095
	500	5/9	1.28	.163	1.29	.122
110 ft. layer	2000	10/12	1.37	.156	1.31	.110
	1000	11/13	1.38	.148	1.37	.103
71 ft. layer surface vs. 110 ft. layer embedded	2000	14/12	1.63	.138	1.57	.139
	1000	15/13	1.74	.151	1.75	.127

Table A.15 Mean ratios of spectra accelerations for surface vs. embedded foundation conditions over the specified frequency range (half-space and layered sites)

Soil Profile	Characteristic V_s (fps)	Base Comparison	0 - 6 Hz Mean	0 - 6 Hz COV	6 - 12 Hz Mean	6 - 12 Hz COV	12 - 20 Hz Mean	12 - 20 Hz COV
Half-Space	3500	2/6	1.05	.215	1.29	.182	1.28	.171
	2000	3/7	1.12	.223	1.25	.186	1.26	.154
	1000	4/8	1.20	.217	1.22	.133	1.21	.124
	500	5/9	1.25	.197	1.11	.107	1.18	.134
110 Ft. Layer	2000	10/12	1.26	.367	1.26	.379	1.31	.135
	1000	11/13	1.34	.407	1.32	.200	1.22	.134
71 Ft. Layer Surface vs. 110 Ft. Layer Embedded	2000	14/12	1.27	.431	1.53	.334	1.55	.148
	1000	15/13	1.40	.424	1.50	.227	1.48	.139

Table A.16 Mean ratios of forces and peak accelerations for fixed-base vs. surface and embedded foundations; shear wall structure (half-space and layered sites).

Soil Profile	Characteristic V_s (fps)	Case Comparison	Peak Accelerations Mean	Peak Accelerations COV	Peak Forces Mean	Peak Forces COV
Half-Space Surface Foundation	3500	1/2	1.07	.261	1.23	.178
	2000	1/3	1.25	.389	1.42	.289
	1000	1/4	1.47	.500	1.64	.360
	500	1/5	2.02	.601	2.14	.403
Half-Space Embedded Foundation	3500	1/6	1.27	.231	1.44	.184
	2000	1/7	1.52	.373	1.73	.288
	1000	1/8	1.89	.486	2.11	.360
	500	1/9	2.52	.573	2.73	.401
110 Ft. Layer Surface Foundation	2000	1/10	1.07	.421	1.23	.284
	1000	1/11	1.27	.504	1.39	.342

Table A.16 (cont.) Mean ratios of forces and peak accelerations for fixed-base vs. surface and embedded foundations; shear wall structure (half-space and layered Sites).

Soil Profile	Characteristic V_s (fps)	Case Comparison	Peak Accelerations Mean	Peak Accelerations COV	Peak Forces Mean	Peak Forces COV
110 Ft. Layer Embedded Foundation	2000	1/12	1.39	.393	1.60	.289
	1000	1/13	1.67	.464	1.90	.353
71 Ft. Layer Surface Foundation	2000	1/14	.86	.380	1.03	.295
	1000	1/15	.99	.511	1.09	.347

Table A.17 Mean ratios of spectra accelerations for fixed-base vs. surface and embedded foundations; shear wall structure over specified frequency range (half-space and layered sites).

Soil Profile	Characteristic V_s (fps)	Base Comparison	0 - 6 Hz Mean	0 - 6 Hz COV	6 - 12 Hz Mean	6 - 12 Hz COV	12 - 20 Hz Mean	12 - 20 Hz COV
Half-Space Surface Foundation	3500	1/2	.97	.056	1.08	.456	1.35	.292
	2000	1/3	.94	.091	1.55	.624	1.87	.404
	1000	1/4	.94	.188	2.70	.738	2.77	.532
	500	1.5	1.12	.510	4.67	.790	4.10	.590
Half-Space Embedded Foundation	3500	1/6	1.03	.025	1.30	.349	1.64	.264
	2000	1/7	1.04	.053	1.83	.571	2.29	.407
	1000	1/8	1.11	.206	3.31	.710	3.32	.523
	500	1/9	1.37	.523	5.29	.759	4.78	.596
110 Ft. Layer Surface Foundation	2000	1/10	.92	.142	1.55	.747	1.77	.445
	1000	1/11	.92	.271	2.50	.643	2.57	.578

Table A.17 (cont.) Mean ratios of spectra accelerations for fixed-base vs. surface and embedded foundations shear wall structure over specified frequency range (half-space and layered sites).

Soil Profile	Characteristic V_s (fps)	Base Comparison	0 - 6 Hz Mean	0 - 6 Hz COV	6 - 12 Hz Mean	6 - 12 Hz COV	12 - 20 Hz Mean	12 - 20 Hz COV
110Ft. Layer Embedded Foundation	2000	1/12	1.01	.38	1.69	.675	2.20	.483
	1000	1/13	1.05	.201	3.30	.642	2.96	.501
71 Ft. Layer Surface Foundation	2000	1/14	.91	.172	1.22	.713	1.56	.546
	1000	1/15	.88	.262	2.30	.704	2.07	.553

Table A.18 Mean ratios of peak accelerations for soil springs vs. surface and embedded foundations; shear wall structure (half-space site).

Soil Profile	Characteristic V^s (fps)	Case Comparison	Peak Accelerations Mean	Peak Accelerations COV
Half-Space Surface Foundation	3500	16/2	1.53	.299
	2000	17/3	1.86	.354
	1000	18/4	2.44	.432
Half-Space Embedded Foundation	3500	16/1	1.84	.244
	2000	17/7	2.26	.348
	1000	18/8	3.12	.429
	500	19/9	2.65	.400

Table A.19 Mean ratios of spectra accelerations for soil springs vs. surface and embedded foundations over specified frequency range (half-space site).

Soil Profile	Characteristic V_s (fps)	Base Comparison	0 - 6 Hz Mean	0 - 6 Hz COV	6 - 12 Hz Mean	6 - 12 Hz COV	12 - 20 Hz Mean	12 - 20 Hz COV
Half-Space	3500	16/2	1.02	.081	1.69	.420	1.32	.308
Surface	2000	17/3	1.20	.523	1.65	.564	1.38	.308
Foundation	1000	18/4	1.47	.712	1.67	.359	1.89	.318
	500	19/5	1.58	.693	1.81	.313	1.88	.260
Half-Space	3000	16/6	1.10	.154	2.09	.447	1.62	.332
Embedded	2000	17/7	1.30	.677	1.96	.560	1.69	.307
Foundation	1000	18/8	1.80	.813	2.05	.392	2.29	.328
	500	19/9	1.99	.770	2.05	.350	2.21	.283

Table A.20 Mean ratios of forces and peak accelerations for surface foundation conditions at 4%, 7%, and 10% structure damping (half-space site).

Soil Profile	Characteristic V_s (fps)	Case Comparison	Peak Accelerations Mean	Peak Accelerations COV	Peak Forces Mean	Peak Forces COV
4% vs. 7%	Fixed Base	1	1.12	.102	1.18	.132
	2000	3	1.04	.063	1.02	.027
	1000	4	1.02	.046	1.01	.015
4% vs. 10%	Fixed Base	1	1.22	.172	1.32	.240
	2000	3	1.07	.108	1.03	.044
	1000	4	1.04	.071	1.01	.023
7% vs. 10%	Fixed Base	1	1.08	.070	1.11	.090
	2000	3	1.03	.041	1.01	.018
	1000	4	1.01	.024	1.00	.009

Table A.21 Mean ratios of spectra accelerations for surface foundation conditions at 4%, 7%, and 10% structure damping over the specified frequency range (half-space site).

Soil Profile	Characteristic V_s (fps)	Base Case	0 - 6 Hz Mean	0 - 6 Hz COV	6 - 12 Hz Mean	6 - 12 Hz COV	12 - 20 Hz Mean	12 - 20 Hz COV
4% vs. 7%	2000	3	1.00	.010	1.05	.089	1.09	0.96
4% vs. 7%	1000	4	1.00	.011	1.04	.081	1.08	.091
4% vs. 10%	2000	3	1.00	.017	1.09	.164	1.17	.169
4% vs. 10%	1000	4	1.00	.019	1.06	.147	1.13	.158
7% vs. 10%	2000	3	1.00	.007	1.04	.062	1.06	.068
7% vs. 10%	1000	4	1.00	.007	1.02	.053	1.05	.060

Table A.22 Input definitions for piping system analysis by response spectrum analysis techniques

- Envelope spectra -- The envelope response spectra over all piping system support locations; peak broadened; one for each direction of excitation.
- Average spectra -- The average response spectra over all piping system support locations; peak broadened, one for each direction of excitation.
- Center-of-gravity spectra -- The response spectra at the piping system support location closest to the center-of-gravity of the piping system; peak broadened; one for each direction of excitation.
- Single highest spectra -- The highest response spectra of all piping system support locations; alternatively, response spectra at the piping system support at the highest elevation in the structure; peak broadened; one for each direction of excitation.

Table A.23 Modal response combination techniques

Modal Response Combination Technique	Abbreviation	Applicable Equation	Modal Coupling Factor
1. Square Root Sum of Squares	SRSS	1 or 2	= 0
2. Ten Percent	10%	1	= 1 if w_i and w_j differ by 10% or less;
3. Absolute Ten Percent	10%	2	= 0 otherwise.
4. Grouping	GRP	2	= 1 for modes within any group bounded by frequencies less than 10% apart; = 0 otherwise.
5. Double Sum	DBS	1	See note below.
6. Absolute Double Sum	DBS	2	
7. Advanced Response	ARC		Reference

$$R = \sqrt{R_1^2 + R_2^2 + \ldots + 2\, C_{12} R_1 R_2 + \ldots + 2\, C_{ij}\, R_i\, R_j + \ldots} \tag{1}$$

$$R = \sqrt{R_1^2 + R_2^2 + \ldots + 2\, C_{12}\, R_1 R_2 + \ldots + 2\, C_{ij}\, R_i R_j + \ldots} \tag{2}$$

Notes

(1) Techniques 3,4 and 6 are the "Grouping Method", "Ten Percent method", and the "Double Sum Method" of NRC RG 1.92, respectively.

Table A.23 (Cont.)

(2)

$$c_{ij} = \frac{1}{1 + (w_i - w_j)/(B_i w_i + B_j w_j)^2}$$

$$w_j = w_j\sqrt{1 - B^2_j}$$

$$B_j = B_j + 2/w_j t_d$$

w_j, B_j = modal frequency (rad/sec) and modal damping ratio, respectively

Table A.24 Directional combination techniques

o	SRSS	Square-root-of-the-sum-of-the-squares of the response due to each excitation direction.
o	HHV	Absolute sum of the response due to the vertical excitaton with the maximum response from the two horizontal excitation.
o	ABS	Absolute sum of the response due to the three directions of excitation.

Table A.25 Summary of response spectrum analysis scenarios for which response factors were developed

Case No.	Order of Modal Directional Combination	Modal Combination	Directional Combination	Summary Group No.
A1	M	SRSS	HHV	1
A2	M	10%	SRSS	2
A3	M	10%	SRAA	3
A4	M	GRP	SRSS	3
A5	M	DBS	SRSS	2
A6	M	DBS	SRSS	3
A7	M	ARC	SRSS	2
B8	M or D	SRSS	SRSS	2
B9	D	10%	SRSS	3
B10	D	GRP	SRSS	3
B11	D	DBS	SRSS	3

Table A.26 Response spectrum analysis technique vs. SRSS-SRSS envelope spectra, 2% damping

Piping Model	Response	Response Ratio	(A) Modal-Directional Combination					(B) Directional-Modal Combination				
			SRSS - HHV (1)	10% - SRSS (2)	10%- SRSS (3)	GRP - SRSS (4)	DBS - SRSS (5)	DBS - SRSS (6)	ARC - SRSS (7)	SRSS- 10% (9)	SRSS- GRP (10)	SRSS- DBS (11)
RHR	Element Moment	Mean	1.12	1.00	1.02	1.02	1.00	1.06	.99	1.03	1.03	1.09
		COV	0.111	0.023	0.032	0.027	0.024	0.059	0.083	0.048	0.040	0.077
	Nodal Accel.	Mean	1.25	1.04	1.09	1.08	1.02	1.09	1.00	1.12	1.10	1.12
		COV	0.094	0.057	0.059	0.051	0.061	0.055	0.132	0.081	0.071	0.074
AFW	Element Moment	Mean	1.29	1.00	1.07	1.02	1.00	1.05	1.00	1.09	1.03	1.07
		COV	0.051	0.089	0.093	0.018	0.044	0.060	0.059	0.107	0.025	0.076
	Nodal Addel.	Mean	1.28	0.94	1.11	1.09	0.96	1.12	0.91	1.14	1.11	1.16
		COV	0.040	0.041	0.051	0.042	0.032	0.091	0.047	0.057	0.044	0.116
SW	Element Moment	Mean	1.20	0.98	1.08	1.07	0.98	1.11	0.99	1.09	1.08	1.13
		COV	0.093	0.075	0.096	0.091	0.064	0.096	0.136	0.107	0.096	0.112
	Nodal Accel.	Mean	1.17	0.98	1.11	1.08	0.99	1.14	1.03	1.13	1.09	1.17
		COV	0.078	0.078	0.111	0.093	0.082	0.127	0.185	0.134	0.094	0.148
RCL	Element Moment	Mean	1.12	0.90	1.57	1.49	0.91	1.44	0.83	1.77	1.64	1.61
		COV	0.122	0.248	0.184	0.215	0.205	0.156	0.232	0.225	0.268	0.195
	Nodal Accel.	Mean	1.12	0.86	1.57	1.47	0.88	1.49	0.93	1.77	1.61	1.65
		COV	0.059	0.366	0.131	0.142	0.333	0.142	0.424	0.133	0.147	0.160

Table A.27 Response ratios of summary group responses vs. multiple-support time history analysis procedures.

Inertia Load	Piping System	Group (1)*		Group (2)*		Group (3)*	
		Mean	COV	Mean	COV	Mean	COV
Element Moment	RHR	2.3	0.41	2.0	0.35	2.1	0.38
	AFW	5.0	0.23	3.9	0.25	4.1	0.24
	RCL	5.1	0.43	4.3	0.43	8.0	0.48
Nodal Accel.	RHR	7.7	0.24	6.5	0.37	6.9	0.36
	ARW	6.4	0.15	4.5	0.16	5.5	0.12
	RCL	7.8	0.30	6.3	0.42	12.1	0.42
Element Moment	ALL	4.7	0.46	3.9	0.46	6.6	0.60
Nodal Accel.	ALL	7.7	0.29	6.2	0.41	11.3	0.46

*See Table A.25 for group numbers

Table A.28 Response ratios of envelope spectra vs. center-of-gravity and average spectra.

Piping Model	Piping Response	Response Ratio	Spectrum Specification Ave./Env.	C.G./Env
RHR	Element Moment	Mean	0.84	0.83
		COV	0.060	0.083
	Nodal Accel.	Mean	0.80	0.78
		COV	0.068	0.119
AFW	Element Moment	Mean	0.55	0.55
		COV	0.241	0.240
	Nodal Accel.	Mean	0.55	0.55
		COV	0.120	0.084
SW	Element Moment	Mean	0.41	0.92
		COV	0.344	0.082
	Nodal Accel.	Mean	0.35	0.89
		COV	0.209	0.099
RCL	Element Moment	Mean	0.44	0.39
		COV	0.262	0.276
	Nodal Accel.	Mean	0.47	0.42
		COV	0.177	0.170

Table A.29 Response ratios for damping effects

Piping Model	Piping Response	Response Ratio	Damping 1% / 5%	Damping 2% /5%
RHR	Element Moment	Mean	1.78	1.38
		COV	0.147	0.103
	Nodal Accel.	Mean	1.45	1.21
		COV	0.320	0.188
AFW	Element Moment	Mean	2.04	1.56
		COV	0.138	0.078
	Nodal Accel.	Mean	2.00	1.53
		COV	0.131	0.074
SW	Element Moment	Mean	2.14	1.56
		COV	0.178	0.132
	Nodal Accel.	Mean	2.12	1.54
		COV	0.200	0.137
RCL	Element Moment	Mean	1.86	1.43
		COV	0.171	0.118
	Nodal Accel.	Mean	1.62	1.30
		COV	0.246	0.162

*U.S. GOVERNMENT PRINTING OFFICE:1985-461-721:20258

NRC FORM 335 (2-84) NRCM 1102, 3201, 3202 U.S. NUCLEAR REGULATORY COMMISSION **BIBLIOGRAPHIC DATA SHEET** SEE INSTRUCTIONS ON THE REVERSE	1. REPORT NUMBER (Assigned by TIDC, add Vol. No., if any) NUREG/CR-4331 UCID-20468
2. TITLE AND SUBTITLE Simplified Seismic Probabilistic Risk Assessment: Procedures and Limitations	3. LEAVE BLANK
	4. DATE REPORT COMPLETED MONTH: July YEAR: 1985
5. AUTHOR(S) L. C. Shieh, J. J. Johnson, J. E. Wells, J. C. Chen, P. D. Smith	6. DATE REPORT ISSUED MONTH: August YEAR: 1985
7. PERFORMING ORGANIZATION NAME AND MAILING ADDRESS (Include Zip Code) Lawrence Livermore National Laboratory Post Office Box 808 Livermore CA 94550	8. PROJECT/TASK/WORK UNIT NUMBER
	9. FIN OR GRANT NUMBER A0126
10. SPONSORING ORGANIZATION NAME AND MAILING ADDRESS (Include Zip Code) Division of Engineering Technology Office of Nuclear Regulatory Research U.S. Nuclear Reguatory Commission Washington DC 20555	11a. TYPE OF REPORT Technical
	b. PERIOD COVERED (Inclusive dates)
12. SUPPLEMENTARY NOTES	

13. ABSTRACT (200 words or less)

At the request of the U.S. Nuclear Regulatory Commission, the Lawrence Livermore National Laboratory has developed a simplified seismic probabilistic risk assessment (PRA) methodology. The purpose of this methodology is to reduce the costs while adequately performing seismic probabilistic risk assessments of nuclear power plants. This report summarizes the development of the simplified seismic methodology and explains guidelines for applying the procedures. The development effort is part of the scope of work of the Seismic Safety Margins Research Program (SSMRP).

14. DOCUMENT ANALYSIS – a. KEYWORDS/DESCRIPTORS	15. AVAILABILITY STATEMENT
seismic risk assessment nuclear power plants	Unlimited
b. IDENTIFIERS/OPEN-ENDED TERMS	16. SECURITY CLASSIFICATION (This page) Unclassified (This report) Unclassified
	17. NUMBER OF PAGES
	18. PRICE

UNITED STATES
NUCLEAR REGULATORY COMMISSION
WASHINGTON, D.C. 20555

OFFICIAL BUSINESS
PENALTY FOR PRIVATE USE, $300

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