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AN EVALUATION AND ASSESSMENT OF NUCLEAR POWER PLANT SEISMIC METHODOLOGY

D. Bernreuter, F. Tokarz, L. Wight,
P. Smith, J. Wells, and R. Earlow

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AN EVALUATION AND ASSESSMENT OF NUCLEAR POWER PLANT SEISMIC METHODOLOGY

Abstract

The major emphasis of this study is to develop a methodology that can be used to assess the current methods used for assuring the seismic safety of nuclear power plants. The proposed

methodology makes use of system-analysis techniques and Monte Carlo schemes. Also, in this study, we evaluate previous assessments of the current seismic-design methodology.

Section 1 INTRODUCTION

Seismic considerations play a major role in nuclear power-station design. Present seismic criteria — as set forth in Nuclear Regulatory Commission (NRC) regulations, guides, standards, and licensing review procedures (the Standard Review Plan) — establish a sequence of methodology steps that, together, define a seismic methodology chain (SMC) for power-station design.

It is generally agreed that a detailed assessment of the seismic design of nuclear power plants is needed. However, there are several schools of thought as to the shortcomings of the current SMC. On the one hand, industry and the NRC both agree that the current methodology is too conservative; yet, groups of interveners have seriously questioned the adequacy of the seismic design of such plants as Seabrook, Indian Point, and Diablo Canyon.

Review of the literature indicates that the currently regulated methodology for the seismic design of nuclear power plants has three major weaknesses:

- Conservatism is introduced at various methodology steps to account for uncertainties without adequate consideration for total design process. It is generally felt that this buildup results in designs with unwarranted conservatism.
- It does not always reflect state-of-the-art considerations.
- Existing regulations, because of the way they are written, produce considerable variations of methodology results.

Presently, there exists no comprehensive way to consider the impact these weaknesses have on the total

seismic design process. The goal of this study is to provide NRC with a comprehensive approach/work plan that can be used to develop an accurate technical basis for evaluating and assessing both the current and proposed future changes to the seismic methodology regulations for nuclear power plant design including:

- Quantification of the variability in the current regulated approach.
- Quantification of the conservatism in the current regulated approach.
- Quantification of the relative impact that each of the important methodology steps have on the total conservatism.
- Identification of areas that will provide the most benefit from improved methodology.

The quantification of the variability in the current approach can be easily carried out. On the other hand, it must be recognized that any assessment of the conservatism, other than quantification obtained by full-scale testing of a number of prototype plants, will be controversial. However, adequate full-scale testing is prohibitively expensive because a large number of tests on a variety of structures would be required. Thus an alternative methodology - primarily computational - must be developed.

One of the main advantages of full-scale testing is that when the entire system is used, all important interactions are included. This allows determination of what is and is not important. Thus, a major purpose of this study is to propose a computational methodology that interrelates each important step of the SMC and to provide a basis for judging the decisions made at each step of the SMC as well as the final results of the analysis. To carry out this goal, we

- Critically reviewed other recent assessments of the adequacy of the current SMC (Section 2).
- Reviewed and analyzed the current SMC, including its technical bases, assumptions and judgments required, and identified the main shortcomings of the current SMC (Section 3).
- Developed programs to overcome the identified shortcomings of the SMC (Sections 6-9).
- Developed a program to link all steps of the SMC together (Sections 4-5).
- Developed a program to make the required overall assessments (Sections 4-5).
- Identified those areas most needing developmental work in the near future to provide the basis for needed guides/regulations (Section 11).

Section 2

REVIEW OF PREVIOUS ASSESSMENTS OF SMC

INTRODUCTION

A number of assessments of the SMC have been made, including,

- NRC's presentations to the Advisory Commission for Reactor Safety (ACRS).^{1,2}
- Study funded by Electric Power Research Institute (EPRI), "Seismic Design of Nuclear Power Plants - An Assessment."³
- Several studies by Newmark.^{4,5}
- Draft study by General Electric.⁶
- WASH-1400.⁷

Each of these will be briefly reviewed and assessed below relative to the goals of this study as outlined in Section 1. Typically, the main shortcomings of most of the studies are: (1) they are cursory and lack an adequate technical basis for overall conclusions, and (2) individual steps of the SMC are typically studied without an adequate overall assessment of the impact that each step has on the final outcome of the process.

NRC'S ASSESSMENTS

NRC has made several formal presentations to the ACRS identifying why NRC believes that the current SMC is conservative.^{1,2} The appropriate

technical staff at the NRC has stated that major examples of the significant conservatism inherent do exist in the seismic design basis required for all Category I structures. Briefly, they are as follows:

- An elastic dynamic analysis is performed using, as input, wide-band response spectra with conservative amplification factors. The use of wide-band spectra is conservative since the response spectra of recorded earthquake motions do not normally exhibit such wide-band characteristics. The amplification factors used are generally greater by one standard deviation than the mean-amplification factors of actually recorded strong-motion-earthquake spectra.
- Damping values used in seismic analysis are considerably smaller than those indicated by currently available test results. This tends to increase seismic responses significantly and thus provide a large margin of safety.
- Use of state-of-the-art soil structure interaction analysis procedures accounts for the deformability and compliance of plant foundation media, and

provides conservative system seismic response. In the soil-structure interaction analysis, variations of soil properties are considered to assure conservative responses.

- Seismic responses are determined by accounting for three conservatively defined input components (i.e., the maximum acceleration amplitude is used for both horizontal components and two-thirds of the maximum horizontal acceleration amplitude is assumed for the vertical component, at near rigid frequencies).
- Either the time-history analysis method or the response-spectrum method of analysis is used for seismic analysis of Category I structures, systems and components, and accounts for all significant modal responses. This is significantly more conservative than the essentially single-mode response and one-directional input consideration usually employed in the design of conventional structures.
- The allowable stresses and strains used in design of all safety-related structures, systems and components are selected such that the structures, systems and components are capable of

withstanding, without failure, the design loadings with margin beyond the specified loads.

- Additional conservatism for the design of mechanical and electrical equipment is introduced by the criteria that the peaks of the generated floor-response spectra must be broadened. In addition, redundant systems that must meet rigid qualification criteria are required.

At the present time, NRC is in the process of developing a basis that can be used to judge how much actual conservatism is introduced by each of these requirements.⁸ However, its proposed work plan is a conventional piece-meal approach.⁸ Assessments that are made are relative. In addition, NRC's proposed work plan is not comprehensive because it fails to include all of the important steps of the seismic methodology chain. For example, neither the methodology chain leading to the selection of the SSE (Safe Shutdown Earthquake) nor a study relative to the impact of site-response calculations on the design-response spectra is included. Other areas such as soil structure interaction are only peripherally included. Lastly, its work plan does not account for the accumulation of step-to-step conservatism.

NRC's assessment is useful in that it points to those areas most likely contributing to unwarranted conservatism. However, it will be difficult to assess how much overall conservatism exists, how it builds up, how an equally safe but less costly SMC can be evolved, or how to develop a basis to use as comparison.

EPRI's Assessment

EPRI funded a study entitled "Seismic Design of Nuclear Power Plants - An Assessment." This report contains a review and evaluation of the analytical methods, design methods, and design standards used in the seismic design of nuclear power plants. Three major areas were investigated: (a) soils, siting and seismic ground motion specification; (b) soil-structure interaction; and (c) the response of major nuclear-power-plant structures and components. The purpose of this study was to prepare an independent assessment of the state-of-the-art of the seismic design of nuclear power plants and to identify seismic analysis and design research areas meriting support by the various organizations comprising the nuclear power industry.

Three major areas of review are found in this study. The first area reviewed was the international experience in the dynamic testing of nuclear-power-plant structures and

components, and related experience with scaled and model tests.

The second area involved the review of current literature, focusing primarily on publications dated later than 1970. This review included the results of foreign studies, and concentrated on Japanese studies and those presented in recent international conferences.

The third review approach obtained the opinions and recommendations of technically knowledgeable individuals in the nuclear industry.

The major conclusions of this report include the necessity of improving knowledge of seismicity and soil dynamic parameters, improved specification of seismic inputs used for design Safe Shutdown Earthquake (SSE) and Operating Bases Earthquake (OBE), and validation of models and methods used for soil-structure interaction calculations. Also considered important are the acquisition of additional experimental and test data on equipment and structures; certain changes to Regulations and Standards; and the development of simplified design methods suitable for industry-wide application. In addition, the report concludes that substantial conservatism in seismic designs appears to be present in many areas; in others the reliability and uncertainty of data need quantification. There is industry concern for

the lack of skilled manpower with seismic-design experience. In responding to a questionnaire, nearly everyone agreed that the need was urgent for preselected, preapproved sites with pre-established seismic design criteria for nuclear power plants:

Although no comparisons or any attempt to develop a basis for comparisons were made in this report, the investigators reviewed available empirical data relative to the verification of various analyses (SSI, structural, etc.) currently performed. From this data, they concluded:

"There exists virtually no data on the response of nuclear power plants to even modest ground excitation. Limited dynamic response data does exist as a result of various dynamic tests conducted during the past seven years at nuclear power stations, but are generally limited to low response levels for both equipment and structures."

"Currently available comparisons between analytical dynamic response predictions and the experimentally determined reality are quite limited. The preponderance of comparisons apply to semi-scale models of equipment or structures (particularly for studies in soil-structure interaction)."

"As might be expected, comparisons between experiment and analysis present

examples of both good and poor correlation. The correlation quality is in part a function of the particular analyst and assumptions employed in reducing large degree-of-freedom problems to their essentials. The quality also is a function of analytical tools generally available to the engineer and the fundamental limitations of alternative approaches."

"The nonlinear response characteristics of soils, structures, and equipment present a major source of potential error when using currently accepted analytical approaches (linear models). As discussed, nonlinear effects may result in either conservative or unconservative designs, depending upon the magnitude of various phenomena. Experimental evidence of nonlinear dynamic behavior exists for both soil-structure systems and for equipment; however, data and analytical comparison are quite limited and inconclusive."

While this report is quite useful, it contains no systematic study or quantification of the SMC. It makes no attempt to link the various steps of the SMC together to see how conservatism may be accumulating and, most importantly, to develop a basis that can be used to judge the results from the current SMC.

Newmark's Studies

N. M. Newmark has written several papers commenting on the conservatism in earthquake resistant design. One paper is entitled "Overview of Seismic Design Margins,"⁴ and the other "Probability of Predicted Seismic Damage in Relation to Nuclear Reactor Facility Design."⁵ Both of these papers take a heuristic approach. No attempt is made to model the current SMC. A very simple probabilistic model is set up and most of the factors entering into the model are based on judgment rather than analysis, with no actual comparisons made. Although these papers present a very cursory review of the problem, they give some insights into the possible compounding of conservatism, and indicate the power of a probabilistic approach.

General Electric Study

L. K. Liu of General Electric's Nuclear Energy Division provided us with a draft copy of a report prepared under his direction but never released.⁶ Although it is a long personal communication, we include an overview of this study, as it is the most detailed attempt to assess the seismic safety of nuclear power plants available. A somewhat incomplete SMC is defined and various aspects of the SMC are examined. (It should be noted that this report was written in

1973, before the Standard Review Plan (SRP) was available to define the SMC better.)

The interconnecting of the various methodology steps was not handled in a consistent manner. The various steps were examined individually, and the linking of the various conservatisms was handled in a heuristic manner. Some attempt was made to provide a basis to compare the results with but the approach used was inadequate. For example, there are such statements as "From this definition of the Safe Shutdown Earthquake, most experts would agree that the probability that the maximum ground acceleration a_{max} exceed the SSE is in the range of 0.01 to 0.0001." However, it should be noted that the study was not aimed at detailed assessment of the conservatism with the goal of removing unwarranted conservatism but only to show that the present criteria are adequately conservative.

The report concludes that the safety factors involved in each of the steps of the design process are multiplicative, i.e., the steps are in series and the conservatisms are compounded. The report also concludes that the basis of the seismic criteria and analysis procedures has been established with adequate conservatism.

This study serves as a first step towards the development of an acceptable methodology to assess the

SMC. Most of the elements required are included in the study in at least a rudimentary form.

WASH-1400

The AEC funded a study entitled "Reactor Safety - An Assessment of Accident Risks in U.S. Commercial Nuclear Power Plants." This report assessed the consequences of having a major accident in a nuclear power plant and quantified the probability of that accident occurring from a variety of causes. One cause, namely the earthquake risk, is of particular interest to us. With respect to the seismic issue, the study was concerned with the probability of an earthquake causing a release of radioactive material to the atmosphere. To obtain this information, the probabilities of ground acceleration were calculated for various magnitudes in both the eastern and western U.S., and factors were obtained for the attenuation of earthquake forces relative to the distance from the earthquake to epicenter. This information was needed to answer the question, "What's the probability of an earthquake producing a core melt accident?" This was answered by combining the probability of having a given earthquake with the probability that earthquakes will produce enough damage to cause potential accidents involving core melt. The basis used

for predicting damage from a given earthquake was obtained from Ref. 5. As discussed above, Newmark's approach was heuristic and the probabilities presented are based largely on his engineering judgment. WASH-1400 does state that "the safety factor for reactor systems is about 20 for the SSE." While this statement does relate directly to our conservatism problem, the report's earthquake data is not detailed enough to obtain a good value for plant conservation. WASH-1400 did not attempt to develop a standard from which the current SMC could be evaluated. Although WASH-1400 is a very commendable study, it was not its purpose to evaluate the conservatism in seismic design.

CONCLUSIONS

One of the major shortcomings of the evaluations of the current SMC discussed above, was the lack of a developed, meaningful basis to use in comparing the results obtained via the use of the current SMC. As noted in the EPRI Study, it is difficult to specify a basis because of the lack of suitable empirical data. This is compounded by another major but common shortcoming; namely, the lack of detailed approaches that interrelate each step of the current SMC.

For several reasons, we feel that it is essential to develop a complex interrelated (Systems) model to

evaluate the conservatism in the current SMC. First, the overall problem is highly nonlinear when a soil site is involved. The final basemat design response spectra is a very sensitive function of soil conditions, peak g level, and spectral content of the ground motion. In addition, any reasonable assessment of the stress-analysis methodology will require the introduction of nonlinear methods of analysis. Thus, the final resultant stress will also be a sensitive function of the duration of the strong motion. Because of the nonlinear nature of the problem, the assessment should be carried out as a connected whole. Secondly, for each region of the country, the seismic hazards differ. The important regional variations of hazards in terms of earthquake magnitudes, number of earthquakes, and level and duration of ground motion must be included in the analysis if a meaningful assessment is to be made. Thirdly, for any meaningful assessments, the overall final effect on the stress levels at the critical locations should be the criteria used. Because of fabrication problems, the size of most structural members are controlled by stress levels at certain critical locations. Elsewhere in these members, the stresses are lower. In

addition, the SSI analysis and the structural analysis act as narrow band-pass filters on the critical ground motion. Thus, only certain very limited ranges affect the final resultant critical stress levels. Because of the complex nature of problems, it is difficult to develop a way to assess each methodology step "piece meal."

A third major problem not addressed by any of the studies is how to make the assessment generic. Regional and site factors are extremely important. In addition, each plant is usually custom designed. Although the introduction of standardized plants should somewhat alleviate this problem, each vendor is different. In fact, there is always room for considerable differences in each standardized plant.

The above review identifies a number of "piece meal" conservatisms that have been introduced. However (as discussed later), these conservatisms may be counterbalanced by factors not appropriately accounted for in the current SMC and not identified nor included in the assessments reviewed above. Thus, one cannot make definite conclusions. Clearly, if the above assessments are to be improved, the above major shortcomings must be overcome.

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Section 3

DEFINITION AND INITIAL ASSESSMENT OF THE SEISMIC METHODOLOGY CHAIN (SMC)

INTRODUCTION

In this Section, we define the basic methodology chain (SMC) currently used to analyze nuclear power plants. The current SMC defined in this section is based on careful review of the SRP,¹ 10CFR² (Code of Federal Regulations) and numerous discussions with the staff of the Geology/Seismology and Structural Engineering Branches in the Office of Nuclear Reactor Regulation. We then carry out a very limited assessment of the main stream of the methodology to determine those factors and judgments that appear to have the greatest impact or introduce the greatest uncertainty associated with the specification of the design loads/stresses. Because few clear cut decisions can be judged independently of the entire process, the task of assessing the SMC is very difficult. How to do this is the subject of Sections 4-9. The purpose of the assessment in this section is primarily to identify the major weaknesses of the current methodology. Then we can develop programs to overcome these weaknesses in a form that can be incorporated into an overall assessment of the seismic methodology chain outlined in detail in Section 4.

To avoid getting "bogged" down in the details, we have referenced what we have determined to be the key decisions. Hopefully, this will help the reader to understand what is truly important relative to the specification of the design basis. Because an elastic structural analysis is used, one can easily determine the key assumptions relative to the specification of the seismic hazard having the greatest impact on the design. At the same time, they introduce the largest uncertainty in the process. These assumptions are:

- That the design response spectra typically used are in Regulatory Guide 1.6.
- That the response spectra scales linearly with peak g value.
- That the peak g value is a function only of M or I .

In reality, both I vs g or M vs g , and both spectral shape and amplitude are only weak functions of each other. Thus, considerable uncertainty exists that overwhelms all other aspects of the analysis, and the true meaning of the effect of earthquake magnitude or maximum I is lost.

Other key assumptions are that the free-field motion of foundation level

can be computed by using, for example, the computer program SHAKE; and that in the East the earthquake is random (within the tectonic province of the site), and its maximum intensity is the largest that has occurred in the sites' province during the historical record. This introduces the problem of where the earthquake should be located relative to the site and because little consideration is given to earthquake occurrence models, no attempt is made to set the level of seismic hazard which the choice of SSE represents.

DEFINITION OF METHODOLOGY

A concise overview of the SMC is shown in Fig. 3-1. Each of the boxes represent a complex methodology chain and the main interfaces between various methodologies.

Figure 3-2 is a flow chart of the important steps typically followed to arrive at a definition of the ground motion. The final output of the entire process is an elastic response spectra and a time history that envelopes the spectra to be used as input into the site response/SSI chain. The primary source outlining the basic criteria defining this methodology chain is Appendix A of 10CFR Part 100. This sets the basic approach that is used. The SRP¹ fills in the details of the generic approach outlined in Appendix A.

Figure 3-3 is a flow chart of the important steps typically followed to develop a basemat ground motion for the structural response analysis given the prescribed free-field motion. The need for such analysis is noted in Appendix A; however, the basic criteria etc., defining this methodology chain are primarily contained in the SRP.

Figure 3-4 is a flow chart of the important steps comprising the methodology chain used to develop structural member forces and/or floor spectra given a basemat ground excitation. Very general criteria are given in 10CFR Part 50 and the Appendices to Part 50. However, most of the definition comes from the SRP.

The methodology used to calculate earthquake stresses given member loads or floor spectra is not outlined in any detail. For the most part, various applicable codes (ASME, ASCE, etc.) are followed. For this reason, Office of Standards Development (OSD) indicated that at this time we should not assess in detail these various codes nor attempt to quantify the conservatism possibly present in the nonseismic loads. In fact, as discussed in Section 4, one of the important results of the proposed analysis will allow NRC to determine if nonseismic loads are sufficiently important relative to the overall seismic design to warrant a detailed study of the possible conservatism.

Fig. 3-1. Seismic methodology chain breakdown.

Fig. 3-2. Seismic input methodology chain.

Fig. 3-3. Methodology chain for developing basemat ground motion for structural response analysis.

Fig. 3-4. Methodology chain for developing structural member forces and/or floor spectra.

DETAILED ASSESSMENT OF SMC

The above discussion has identified the main decisions and assumptions of the currently used methodology. In the following paragraphs, we will proceed in an orderly fashion to assess the methodology steps shown in Figs. 3-2 to 3-4.

SEISMIC INPUT

As shown in Fig. 3-2, this leg of the SMC is complex. The output of the seismic input chain is a free-field response spectra and a time history that envelopes the predicted response spectra. This is then used as input to site response/SSI analysis. The final resultant basemat motion is used as input into the seismic-stress analysis which is primarily an elastic analysis.

The current approach for establishing the ground motion at a site is centered around defining the Safe Shutdown Earthquake (SSE). This in turn fixes the design g value used to scale the spectra. The SSE is defined in Appendix A as, "that earthquake which is based upon an evaluation of the maximum earthquake potential and seismology and specific characteristics of local subsurface material. It is that earthquake which produces the maximum vibratory ground motion for which certain structures, systems, and components are designed to remain functional."

It should be noted that the definition of the SSE allows for considerable judgment to enter the process. The key word is evaluation. In fact it is specifically noted in Section 100.2 of Title 10, that "In the application of these criteria which are deliberately flexible, the safeguards provided — either site isolation or engineered features — should reflect the lack of certainty that only experience can provide."

These remarks set the stage for a detailed review of the Decision Chain leading to the definition of the SSE at any given site. Our discussion is keyed to Fig. 3-2. Briefly, the main steps that result in the choice of the SSE " g value" for a given site are (1) determination of either magnitude or intensity of possible earthquakes; (2) determination of the location of earthquakes relative to site and (3) determination of possible local site amplification factors.

The first step in the chain represents the investigations required to develop the basic data needed to make the evaluations required. Clearly, a number of important decisions must be made during the geological investigation of a site; e.g., number of borings to make, number of samples required, extent of geologic mapping required, etc. Typically, such decisions are site-related and are basically judgment-type decisions;

hence, not amenable to the type of assessment being carried out here. Therefore; no attempt is made to assess this aspect of SMC.

From the seismological point-of-view, the first major decision is to determine if a correlation can be made between tectonic structure and observed, historical earthquake activity. Thus, determination is extremely important as it fixes both the methodology by which the largest SSE is determined and the methodology used to locate the SSE relative to the site. Several cases occur, and the simplest is when no correlation can be made, and the earthquake activity is assumed random. This is the typical case east of the Rocky Mountains. For this case, Appendix A requires that "... (ii) Where epicenters or locations of highest intensity of historically reported earthquakes cannot be reasonably related to tectonic structures but are identified pursuant to the requirements of paragraph (a) (6) of Section IV with tectonic provinces in which the site is located, the accelerations at the site shall be determined assuming that these earthquakes occur at the site; (iii) Where epicenters or locations of the highest intensity of historically reported earthquakes cannot be reasonably related to tectonic structures but are identified pursuant to the requirements of paragraph (a) (6)

of Section IV with tectonic provinces in which the site is not located, the accelerations at the site shall be determined assuming that the epicenters or locations of highest intensity of these earthquakes are at the closest point to the site on the boundary of the tectonic province."

Sufficient seismic coverage or complete correlation between earthquake activity and tectonic structures exists in only a few places in California. Sites are typically located in regions at the edges of seismic networks and where the seismicity is low. Thus the epicentral location of earthquakes is so poor that a pattern of diffuse seismicity exists. While the major earthquakes can be assigned to tectonic structure, the smaller ones must be often assumed random.

Little guidance exists either in Appendix A or in the literature as to what is an acceptable correlation between earthquake activity and tectonic structure. In practice, the choice has hinged on the determination of whether or not capable faulting exists in the vicinity of the site. Appendix A defines a capable fault as "a fault which has exhibited one or more of the following characteristics.

(1) Movement at or near the ground surface at least once within the past 35,000 years or movement of a recurring nature within the past 500,000 years.

(2) Macro-seismicity instrumentally determined with records of sufficient precision to demonstrate a direct relationship with the fault.

(3) A structural relationship to a capable fault according to characteristics (1) or (2) of this paragraph such that movement on one could be reasonably expected to be accompanied by movement on the other.

"In some cases, the geologic evidence of past activity at or near the ground surface along a particular fault may be obscured at a particular site. This might occur, for example, at a site having a deep overburden. For these cases, evidence may exist elsewhere along the fault from which an evaluation of its characteristics in the vicinity of the site can be reasonably based. Such evidence shall be used in determining whether the fault is a capable fault within this definition."

"Notwithstanding the foregoing paragraphs III(g)(1), (2) and (3), structural association of a fault with geologic structural features which are geologically old (at least pre-Quaternary) such as many of those found in the eastern region of the United States shall, in the absence of conflicting evidence, demonstrate that the fault is not a capable fault within this definition."

The definition of a capable fault is related to a fault's capability to

cause ground rupture. In some cases - particularly east of the Rocky Mountains - there are faults [and other tectonic structures such as massive intrusives] not capable in the above sense, but which can be considered active in that earthquakes are generated on them. For example, the Geology/Seismology Staff at NRC has proposed³ that the high rate of seismicity in New England correlates with the massive intrusives in the White Mountains.

As yet, no criteria exist as to what constitutes a reasonable association of earthquakes with tectonic structure in the absence of observable surface faulting. In eastern regions where the major earthquakes have occurred, there is no evidence of any recent (Quaternary) movement of any of the fault systems in the epicentral region.⁴⁻⁸ In fact, the date of the last movement goes back many millions of years (excluding growth faulting and regional subsidence).

In the Northwest, many of the earthquakes are on very deep faults, 60 to 100 km in depth. Large earthquakes occurring on these deep faults did not cause surface faulting. Because the level of seismicity is low enough and the past seismic network used to locate current earthquakes was poor; it is not possible to define these major fault systems.

These fault systems form yet another class of "capable faults" in the sense that large earthquakes with surface movement are associated with them. A third type of faulting that exists is the massive growth faults in the Gulf Coastal Region.⁶ Although many of these faults are still active they seem to give rise to only small earthquakes. Because of the lack of major seismic activity associated with this type of faulting, we will exclude them from our discussion.

If one of the above cases do not apply, then earthquakes are associated with tectonic provinces. Appendix A defines a tectonic province as "a region of the North American continent characterized by a relative consistency of the geologic structural features contained therein." Appendix A defines a tectonic structure "as a large scale dislocation or distortion within the earth's crust. Its extent is measured in miles."

The SRP gives very little additional criteria as to what constitutes an acceptable definition of a tectonic province or what is an adequate correlation of instrumentally recorded earthquakes with geologic structure. Considerable judgment is required.

Let us consider typical eastern sites where the geologic structure is old and the last movement on fault systems is generally assumed to have been millions of years ago. This

should be contrasted with the criteria of the date of the last movement of 500,000 years used in defining capable faults. In the East, tectonic provinces are basically formulated on the basis of these old structures.⁴⁻⁶ Yet, the earthquakes are reacting to present-day tectonic stresses. Thus, the utility of using such geological definitions is not clear from the point-of-view of present-day tectonics. Little confidence can be placed in the rationale of attempting to relate current-day seismicity with a very old geologic structure not formed from current tectonic activity. The underlying framework of Appendix A is clearly drawn from the older "static" geologic models rather than the current, dynamic plate-tectonic models.

Little is known about the causes of earthquakes east of the Rocky Mountains. The plate-tectonic model is extremely useful in explaining the major tectonic features observed as well as the current seismicity in the West.^{3,9} A number of models can be offered to explain midplate seismicity, but none of the models have been generally accepted. The difficulty in developing a meaning model is two-fold. First, as noted earlier, none of known fault systems located within the epicentral regions of the major earthquakes has exhibited any recent

movement. In fact, the date of last movement was many millions of years ago.⁴⁻⁸ As noted above, the recent growth faulting in the Gulf Coastal regions is not seismic and, hence, is not included in the above. It should be noted that recent United States Geological Survey (U.S.G.S.) studies³ in the Southeast may have mapped several small faults with recent movement on them. However, these findings are tentative since the movement could well be from other causes. Secondly, only a few fault-plane solutions exist.^{3,9} Some of these solutions³ suggest that the current seismic activity is associated with faulting and stress which are "cross-cutting" the major trends of faulting. As of yet, it is not possible to identify what faults have moved or the tectonic causes of the movement.

Several investigators³ suggest that the major earthquakes activity in the East can be correlated with large gravity and geomagnetic anomalies associated with regions with massive granitic intrusives such as those that exist in the White Mountains in New England.

Walper⁹ in his review concluded that, "midplate seismicity - the earthquake of the stable continental interior - is more random and results from high stress in regions of

unhealed fault zones formed during earlier episodes of continental rifting and collision that may have overlapped or show no coherent pattern.

The seismicity of the eastern and western United States is different, and this difference appears to be the result of crustal properties, structures, and stress conditions.

The maximum principal compressive-stress distribution of the central and eastern United States contrasts with that of the West in one major particular. The region extending from west of the Appalachian Mountains throughout the central United States is characterized by compressive stress that trends east to northeast; whereas, with the exception of the Colorado Plateau, the western United States is characterized by extension, with the least compressive stress trending east to southeast."

Considering the state of knowledge, it is evident that considerable disagreement exists among various investigators. For example, Hadley and Devine⁶ concluded that, "In a few places, such as central Virginia, central New England, and the vicinity of Charleston, S.C., seismic activity of intermediate or even high levels cannot, in the light of present information, be considered localized along known structures." Elsewhere in their report they conclude relative to the

Charleston, S.C., earthquake that, "the lithologic character and structure of the basement rocks in the region are, however, poorly known and there is very little evidence to indicate what kind of structure was involved." However, based on a different set of judgments, NRC has chosen to assume that the geologic structure in the Charleston, S.C. area is sufficiently different from the rest of the Atlantic Coastal plain as to form a small, local tectonic province. Clearly, with present state-of-the-art, there is no way to directly assess the current approaches; thus, judgment plays a key role. However, as a practical matter, the importance and possible drawbacks of the current approach can be provisionally assessed. We can examine the consequences of the decisions to accept the definition of many different tectonic provinces and to limit the largest possible future earthquake in each of these provinces to the largest historical earthquake observed in the different provinces.

There are a number of tectonic provinces in the eastern U.S. Although these vary between various investigators, for our purposes here, the provinces defined by King⁴ are adequate as they correspond roughly to what NRC has accepted in past licensing decisions. These are listed below.

1) Central stable region:

- Laurentian shield (part of Precambrian Canadian shield).
- Interior lowlands (bordering platforms covered by younger rocks).

2) Appalachian (orogenic) system or tectonic province:

- Fold belt (roughly the Valley and Ridge physiographic province).
- Blue Ridge belt (Blue Ridge physiographic province).
- Piedmont belt (Piedmont province and part of the Blue Ridge physiographic province).
- New England — maritime belt.

3) Coastal Plains:

- Post-orogenic deposits overlapping the Appalachian system (of subordinate tectonic significance except in Mississippi embayment).

For our purposes here, we need not consider all of the tectonic provinces in the East. It will suffice to consider only 2) and restricting 3) only to the region of the Atlantic Seaboard.

In the past, NRC has accepted MMVII as the maximum intensity in the New England Maritime belt, and MMVIII in the White Mountain area. The exact definition of these provinces is under study as it affects both the *Indian Point* site in New York and the

Pilgrim site in New England. The current SSE at both of these sites is MMVII. The Blue Ridge, Piedmont belt and part of the Coastal plains have been assigned MVII as the maximum intensity. The Valley and Ridge fold belt has typically been assigned as MMVIII. It has been assumed by NRC that the Charleston earthquake (1886) is fixed to some structure in the Charleston area and has not moved.

There are several approaches we can use to assess the consequences of the above decisions and to determine if the above definition of the seismic hazard is consistent with available data. For example, McGuire¹⁰ has combined regional attenuation of intensity laws with commonly accepted relationship between the number and intensity of earthquakes:

$$\log N = a + b \log I,$$

where

$N(I)$ = Relative number of events
of given intensity I in a
given tectonic province
(a, b , constants), and

I = modified Mercalli Intensity

for various subdivisions of observed historical data assuming that in each subdivision the earthquake is random. It should be noted that the subdivisions used are different than what would correspond to tectonic provinces provisionally used by NRC.

To carry out the analysis, it is also necessary to assume how the earthquake process unfolds in time and what the maximum earthquake should be. This is the most controversial part of the analysis. First, it must be assumed that the short seismic history is adequate to make reliable predictions. Then it is necessary to make assumptions about the historical time-series of earthquakes. Typically, it is assumed that the series is stationary. McGuire,¹⁰ also assumes that the series can be represented as a Poisson Process.

Allen¹¹ points out several important examples where the seismic history is (for the 2400-yr record at least) nonstationary. Allen emphasizes that one should not put too great of reliance on statistical methods that can lead to a grossly inadequate assessment of the seismic risk for a given region, but should use a variety of approaches "well laced" with good judgment and detailed geological exploration.

The currently used methodology has all of the defects of the risk-analysis approach. For example, it assumes the earthquake process is stationary and also that the largest event has occurred and the future events are not related to past events, i.e., more or less likely to occur because a previous earthquake of given size did occur at a certain

location. However, the current approach does assume that the historical seismicity is a good guide to what will happen in the future.

Figure 3-5 shows the annual risk at two sites (Florida and Boston). It also shows the entire region considered as a single province. Other commonly accepted approaches and subdivisions of data lead to very

similar results. Figure 3-6, also from McGuire,¹⁰ notes the 10,000-yr intensity for various locations, as well as the single-province risk. The sites in the vicinity of the Baltimore-Washington area are typically assigned MMVII as the SSE, and (as noted before) in the New York area the MMVII has been used as the intensity of the SSE. Clearly, the

Fig. 3-5. Probability of selected sites experiencing a given intensity earthquake.

Fig. 3-6. 10,000-year intensity for random I_{max} .

SSE typically assigned does not provide a uniform level of protection at all sites.

It is of some comfort to note that various subdivisions of historical data lead to very similar results. On the other hand, the various assumptions that can be made relative to the largest earthquake that can occur in a given subdivision introduce greater variability.

It is not possible at this stage to say if the choice of maximum intensity for the projected SSE provides adequate protection because the manner in which the estimated site intensity is translated into design

ground motion has a profound influence on the actual seismic hazard that a given design affords. Thus, from the design point of view, the above assessment is relatively meaningless. The SRP provides guidance as to what is acceptable in Subsection 2.5.2.6. In Section 2.5.2.6, it is suggested that, "Numerous correlations between intensity and acceleration are given in the literature (Refs. 12-16)*; several of them are considered acceptable by the staff. The correlation used is accepted if

*Reference numbers have been changed to agree with references listed at the end of this Section.

it is conservative when compared to the actual observational data. Acceptance is based on an analysis of the site's seismic energy transmission properties (Ref. 17*, or equivalent). Conservatism should be assessed based on consideration of the amplification analysis and in comparison with the actual published data. The staff will generally accept an acceleration for seismic design as being conservative if, when applied at the ground surface, it results in a value at the foundation free-field level as large as would be obtained, secondly, from the empirical relation of the mean of the intensity-acceleration values in Ref. 16.*"

The difficulty with this approach is that little correlation exists between intensity and acceleration. Figure 3-7 taken from Ref. 18 graphically illustrates the lack of correlation between intensity and acceleration. There are numerous reasons for this lack of correlation:

- The quality of construction varies widely.
- The frequency content of the ground motion relative to the natural periods of the structures affected causes considerably different structural

amplification factors over the peak ground acceleration.

- Large and small earthquakes are intermixed.
- A wide range of epicentral distances are used in the correlations.

The first two factors noted are very difficult to assess from available data. However, the last two factors are easily assessed. For example, Ambraseys¹⁵ correlation between intensity and acceleration is much higher than determined by most investigators. One of the main reasons for this is that most of Ambraseys¹⁵ data is obtained within 20 km of the epicenter for small earthquakes; whereas, most other correlations are based on records taken at much greater epicentral distances.¹⁹ O'Brien et al.¹⁹ also found that the peak g level observed for a given intensity is a strong function of epicentral distance.

If the requirement of Appendix A that the earthquake be located at the site is taken somewhat literally, then we would expect that accelerations greater than the mean should be used; or alternatively, the correlation between I and g should be made by using only near field records. Figure 3-8 illustrates the difference between Ambraseys' results and more general correlations¹⁹ used by NRC.

*Reference numbers have been changed to agree with references at the end of this Section.

Fig. 3-7. Correspondence of worldwide intensity (MM) and acceleration.

Fig. 3-8. Comparison of the intensity acceleration relationship obtained from the results by Ambraseys' (1974), maximum horizontal component.

Although the intensity of an earthquake is not directly related to the magnitude of an earthquake, the correlation between the two is reasonably good. For our purposes here, we can relate intensity to magnitude. Hanks and Johnson²⁰ show that there is little correlation between magnitude and peak g level recorded within 10 km of the epicenter. Small magnitude (low intensity) events can and often do have large peak g values. Several recent studies show that the stress drop is the parameter that governs peak acceleration; whereas, the magnitude of an earthquake is governed by both the stress-drop and the area of faulting involved.²⁰⁻²³

If eastern earthquakes are short-fault-length, high-stress-drop earthquakes, we would then expect high g values in the near field.²¹

The relationship between the surface ground motion and what is projected to occur at foundation level is complex and is discussed in detail later.

It is not evident from the above assessment that the current specification of the SSE is manifestly conservative. Since little is known about the likely stress-drop for eastern earthquakes, it is only possible to speculate that the probability of exceeding the SSE response spectra at typical sites is uncomfortably high. However, this does not imply that the resultant design is inadequate. For example, as discussed in Section 2, many conservatism are added in the design process. Thus, the fact that the elastic response of some structural component exceeds the design response spectra does not imply failure. In addition, there is the "filtering" effect imposed on the real free-field spectra by the SSI and structural analysis. It is thus unlikely that the peak of the spectral will align with the narrow windows of the "filters". Because of the above considerations, it is not possible to determine if the SSE as formulated by the current SMC provides an adequate

level of protection. This can only be determined via a detailed analysis which appropriately models the input as well as conservatism afforded by the total design methodology.

Some consideration of the inherent conservatism of the SMC is included in the SRP, where it is noted in Subsection 2.5.2.6:

"In some cases, the peak acceleration may not be as significant for engineering design purposes as a sustained acceleration at a lower level. One situation where the sustained acceleration level may differ from the peak acceleration is in proximity to the causative fault of the earthquake. It is appropriate in such cases to define the "reference acceleration for seismic design" as representative of the level of sustained acceleration. The "reference acceleration for seismic design" determined in this section of the applicant's SAR is taken to be the high frequency asymptote to the design response spectrum defined in R.G. 1.60. At this time, the staff is not aware of any published relations between earthquake intensity or magnitude and sustained acceleration. Such relations could be developed from analyses of the response spectra of accelerograph time histories in those areas where magnitude and

intensity measurements are also available. In lieu of such studies, the peak accelerations are considered to represent conservative reference accelerations for seismic design. Lower levels of reference acceleration may be justified on a site-specific basis."

Note, once again, the total reliance on judgmental process.

Clearly, judgment of both the applicant and reviewer plays a far-too-important role in the current SMC. It is impossible, at least at the current state-of-the-art, to remove judgment completely. However, it is possible to reformulate the methodology used to arrive at the SSE in a manner that allows for a reasonable assessment of the protection afforded by the overall design process. One such formulation is given in Sections 4-10 of this report, particularly in Section 6.

The above assessment was carried out primarily for eastern sites. Less speculation is required for sites located in the West because some ground motion data are available. However, the problem of assigning a location to the SSE as well as its magnitude often occurs in the West as well as in the East because typical western sites are not located in the most active areas. Typically, the sites are often located in regions

where the epicenter location of local earthquakes is poor. In addition, only a short seismic history usually exists, and only the larger earthquakes have been recorded. If we could assume that all major earthquakes cause surface rupture then the careful geological investigations required would pinpoint these faults and greatly augment the historical record. However, the relationship between the likelihood of the surface-rupture magnitude of the earthquake, fault length on which movement occurred, depth of rupture, average fault displacement and stress-drop is not known. For example, Iida²⁴ studied a number of earthquakes and noted that of about 140 destructive earthquakes with epicenters on land (Japan) since 1840, only twelve shocks are known to have been accompanied by surface faulting. Allen¹¹ notes, however, that too often past, geological field work has been insufficient in searching for and documenting such displacements. Judgment must play an important role in this aspect of SSE determination. If sufficient overburden exists in the site vicinity along with a diffuse pattern of seismicity, then the likelihood of a smaller magnitude earthquake occurring near the site and not causing surface faulting must be considered. As discussed above, this causes problems because these earthquakes can have

high g values, depending upon the estimated local stress-drop.

Several problems exist relative to the nearby capable faults. The first problem is how to determine the maximum magnitude earthquake that can occur on a given fault. The only guidance that the SRP gives is in Subsection 2.5.2.4, which states, "where an earthquake is associated with geologic structure, the maximum earthquake which could occur on that structure should be evaluated, taking into account such factors as the type of faulting, fault length, fault displacement and earthquake history," e.g., Brune,²⁵ Tocher.²⁶

In the past, it has been assumed that a relation exists between earthquake magnitudes and fault-rupture length. Recent studies^{21,23} (as noted before) show that both fault length and stress-drop must be used to determine the largest earthquake possible on a given fault. In addition, there is the problem of the maximum length of the fault that should be assumed to rupture. Albee and Smith,²⁷ on the basis of very limited data, suggest that the maximum length is one-half the fault length. However, Allen¹¹ notes several cases in Japan where virtually the entire fault ruptured. Allen further notes that in Japan some of the largest earthquakes have occurred on seemingly innocuous faults. (What

about high stress-drop, short-length events?)

When we consider the uncertainty about choosing the maximum earthquake that should be associated with a given fault (i.e., we need to know the stress-drop and length of fault likely to rupture), how important is it to know the magnitude? Under the current approach, it is important because Subsection 2.5.2.6 states "the acceleration should be determined using a relation between acceleration, magnitude or fault length and distance from the fault." (See Refs. 28 and 26.) This (as discussed above) is not the case, at least in the near field. Increasing the fault dimensions does have considerable effect on the Fourier spectra and on the attenuation of the ground motion. However, it is the low-frequency part of the spectra that is altered. Figure 3-9 illustrates the effect of increasing fault dimension on the Fourier spectra.²¹ Nuclear power stations are stiff, higher-frequency structures and components. Thus, after a certain fault length is reached, larger earthquakes (with the same stress-drop) have little influence on the design of structures. The added duration may be important relative to the foundation analysis, however.

Site factors are also extremely important relative to both the peak g

Fig. 3-9. Magnitude effect on spectra by increasing fault length (with fixed stress-drop).

value observed at a site and the shape of response spectra. This is discussed in detail in the next section and in Ref. 21.

The above discussion shows that the approach of defining a single SSE, as is currently done, is an exercise in futility. Judgment must play an overwhelming role. Unfortunately, this approach, when coupled with shortcomings of the rest of the analysis (discussed below), forces NRC to rely upon expert judgment to argue that the design is adequately safe.

SITE RESPONSE/SOIL-STRUCTURE INTERACTION

In this section we will review the critical assumptions and procedures contained in the site response (SR)/

Soil-Structure Interaction (SSI) leg of the design chain. Our review is designed to supplement the many existing reviews of SR/SSI, and our intent here is to cover only the major areas of concern in the procedures — those that have the most impact on design. Detailed reviews of many other concerns can be found in such reports as the EPRI assessment of seismic design,³⁰ various final research reports prepared for the NRC,^{31,32} ASCE assessments,³³ and Whitman's state-of-the-art review.³⁴

Note that the flow chart in Fig. 3-3 for SR/SSI contains one significant option: the choice between soil spring and finite element methods to evaluate SSI. Our evaluation in this section will be limited to only the finite element method (FEM) approach to SSI because of its general versatility. We have documented the basis for this preference in a recent final report to the NRC.³⁵

Below, we review and critically appraise the SR/SSI current procedures at two levels: the philosophic and the very detailed. Our systems analysis proposal responds to both levels of this review.

Philosophically, we perceive three general questions that can be raised regarding the current SR/SSI procedures:

- Do we know enough about SR/SSI to perform a deterministic SSI design calculation?
- What is the effect of a + 10³ surface spectrum on the site response?
- What is the engineering significance of vertically propagating SH waves?

SSI Design Calculations

Statistical uncertainty in the seismic design chain is presently handled in one of two ways. In one case, a proper statistical analysis is performed and a statistical level of conservatism required for the parameter is selected. Such is the case with the seismic input spectrum, R.G. 1.60. The other case treats the situation quite differently by attempting to perform bounding calculations. Examples include evaluation of the effects of uncertain soil properties and structural stiffnesses. This accounts approximately for the uncertainty in some parameters within the SR/SSI calculations. Our concern is with the possibility of other parameters whose uncertainty is presently unaccounted for. The overall result is a design with an unknown margin of safety.

Surface Spectrum Effect

The philosophic questions here is whether to prescribe a surface spectrum to obtain the response for a specific site at lower levels when that spectrum already contains (in a statistical sense) the effects of many sites. The deconvolution will inevitably result in notches in the spectrum of lower levels which reduce the statistical + 1 σ conservatism desired in the specification of the surface spectrum. This point can be seen more clearly by considering a specific site. First, observe that the + 1 σ spectrum has averaged out the specific site peaks and valleys in the spectrum. Although this results in a + 1 σ spectrum regarding an ensemble of sites, it has very little statistical significance regarding the specific site having spectral peaks at the specific site periods. This is shown in Ref. 21 for three sites that each had a number of earthquakes recorded. The nature of this difference is shown in Fig. 3-10. The problem is compounded when R.G. 1.60 is deconvolved to obtain spectra at lower levels. The generic effect of this deconvolution is also shown in Fig. 3-10 where we can observe that a probable net effect of the current procedure is a lack of conservatism at certain frequencies. The NRC has acknowledged this short-

coming by requiring that the foundation-level spectrum not be less than 60% of the surface spectrum, but this additional constraint does not address the overall problem in a satisfactory manner.

Vertically Propagating SH Waves

Current techniques assume that all the earthquake energy is in the SH wave. For calculations this is appealing because it simplifies the input to, and the methods of, the computations. Physically, however, it has no basis. The earthquake energy is partitioned between many modes, each with its own propagation characteristics and region of influence. The concept of vertical incidence is particularly crucial, for

Fig. 3-10. Effect of site periodicities on the deconvolved free-field spectrum.

nonvertical incidence can result in diffraction from foundations and traveling wave effects. Both these effects have tremendous impact on the seismic design.

On a more detailed scale, we have identified four major areas of concern in the current SR/SSI procedures. We discuss each of these below.

The first concern relates to the soil measurements and modeling. The input required from an analyst is shear modulus (G) and damping factor (Δ) as a function of average strain (Fig. 3-11), density, and Poisson's ratio for each significant layer above bedrock. A great deal of information can be obtained from logs in boreholes, but the only way G and Δ can be determined presently is through laboratory-core tests. There are three problems with this procedure. First, it is our experience that the required lab experiments result in very uncertain estimates of average shear strain throughout the core. Second, there are potentially large errors due to core-sample disturbance and sample-size effects. In support of this, are the experiments conducted by Terra Tek.³⁶ Finally, the strain-dependent soil properties used in current calculations are symmetric in tension and compression. This precludes tensile failures in the soil which could likely occur along the site of the

Fig. 3-11. Variation of soil damping and shear modulus with strain.

structure. Thus, the mathematical formulation not only allows the soil to push the structures, but to pull them also. Perhaps in recognition of some of these problems, the SR/SSI leg includes the stipulation that the foundation level spectrum be derived from three separate SR calculations: one with the best estimate of the soil properties, and two with minimum and maximum possible variations. It is interesting to note that the consequences of such a policy, in effect, increase the surface-design spectrum³⁷ (Fig. 3-12). The computed spectrum is a result of a convolution up to the surface of the envelope foundation-level spectrum. Note, that the calculated spectrum exceeds the design spectrum everywhere, especially at the fundamental site period.

Fig. 3-12. Comparison of design spectrum and computed-surface spectrum (5% damping).

A second problem concerns the manner in which the site is modeled. The modeling is based on data from borehole logs that delineate the stratigraphy. With no guidelines as to what constitutes a significant impedance mismatch, several equivalent models could be developed from the same data. Our SHAKE calculations have shown that the final design parameters are fairly sensitive to this choice of site models. The final design is particularly sensitive to another aspect of site modeling.

Because there are no criteria specifying the characteristics of bedrock, the analyst has the freedom to put the base of the model at a convenient location, albeit stiff. Our experience indicates that the location of bedrock in a SR/SSE calculation has significant effect on the final design parameters.

A third major problem concerns the conversion of seismic-response spectra into acceleration-time histories. This is done twice in the current procedures — once for the

R.G. 1.60 surface spectrum and again at the foundation level with the envelope spectrum (Fig. 3-3). There are many different computer programs available for the spectra-time history conversion, but they all follow the same calculational procedure (Fig. 3-13). There are two key assumptions in this procedure that appear to have major impact on the SSI results. The first is the association of a 0.1% damped-response spectrum with a Fourier spectrum. The relation is tenuous. For a zero-damped response spectrum, the assumption may be conservative, but

the effects of using a nonzero-damped spectrum are not at all understood. A second, and more important, assumption is that the spectral amplitudes control the SSI system response and that the phasing of the individual frequencies is not an important parameter. It is clear that there are an infinite set of time histories that envelop a target spectrum, each with a different phase spectrum. If phase was deemed an important parameter, it would be necessary to know the sensitivity of the SSI system response. However, present regulations do not consider this sensitivity.

Fig. 3-13. Main steps in developing an artificial time-history to match a given spectrum.

Recent calculations by Donovan, et al.³⁸ have shown that the response of multinode systems is very sensitive to the particular phase spectrum used. Their results show that the response can differ by 50%, depending on the phase spectrum chosen.

The fourth major problem area in the current procedure is the equivalent linear method (ELM), and our concern is derived from two observations. First, the ELM has never been properly tested against data, largely because of the absence of good borehole earthquake data. Validity is often touted on the basis of some comparisons with very low-level surface accelerations. These comparisons are inappropriate because the low-level excitation does not really test the ELM and further there was, in each comparison, large uncertainty regarding the bedrock excitations. Other indirect evidence against the ELM method has been published in the form of comparison of response between SHAKE and direct integration codes that employ nonlinear hysteretic equations-of-state.³⁹⁻⁴¹ These comparisons generally indicate that the ELM introduces nonconservative errors in the high frequency range. Our second concern relates to the basis of the damping formulation in the ELM. Hudson⁴² has shown that the damping in a hysteretic system for low-amplitude sinusoidal excitations

can be approximately represented by an equivalent viscous damping. This is the formulation used in the equivalent linear method, and we think that its use may be inappropriate for strong motion earthquake records.

STRUCTURAL RESPONSE ANALYSIS/ STRESS ANALYSIS

The really important decisions in this methodology chain are the decisions to:

- Use elastic analysis.
- Add peak nonseismic accident loads to the seismic loads to determine the stress level.
- Use failure criteria that are very conservatively determined from the load combination equations.
- Use the damping values defined in R.G. 1.61.
- Use single-time history (3 components).

The difficulty in any analysis is to relate the calculated loads and stresses to the dynamic failure of a complex structure. Numerous studies confirm that properly designed structures can carry dynamic loads far in excess of the elastic design load. The use of elastic methods of analysis most likely introduces considerable conservatism into the SMC; however, this conservatism is used to justify the depression of the design of value

representing the SSE because (as mentioned before) neither the size of the SSE nor the peak g value associated with the projected SSE can be considered conservative. This coupling of the selection of the input to the choice of analysis methodology requires, for reasons discussed in detail in Section 2, a complex assessment of both factors.

In addition, one can only effectively evaluate the method of analysis that is used relative to the type of structures involved and to the total loading conditions imposed on the structures. In many cases the non-seismic accident loads are large; thus, the interaction of the different loadings is important. The load combinations can only be assessed when the nonseismic loads are known.

At this stage we can add little else to the assessments reviewed in Section 2. They cover, in some detail, the various fine points of the analysis and critical assumptions made. In addition, a complete overview is provided in Ref. 43.

Various assumptions made during the analysis process³⁰ can introduce differences on the order of 20%. The magnitude of these differences is small compared to the typical range of results obtained between the use of the FEM (LUSH) of SSI or soil-spring method.³⁵

Available data indicate considerable uncertainty in modeling and the estimation of damping, as well as nonlinear response of both convention structures and nuclear power plants to seismic loading. Little data exist for nuclear power plants; however, for convention structures the effect is extremely important. For example, Fig. 3-14 taken from Ref. 44 depicts a measure of the degree of nonlinear behavior induced during an earthquake in real structures. Although it is not possible to extrapolate these data to nuclear power plants, available data for nuclear power indicate similar nonlinear dynamic response occurs.

The point of the above discussion is that little is to be gained from one more conventional assessment of the structural-response methodology chain. The various refinements in analysis and modeling are lost in the sea of uncertainty surrounding the estimation of the basic structural parameters such as the amount of mass "lumped" at each node, determination stiffness, various complex structural members, the action of nonstructural members not included in the analysis, etc. While detailed requirements are given in the SRP relative to the number of nodes required, other substantial reviews are lacking. For example, how in fact are the masses lumped at each node determined or the

Fig. 3-14. Histogram of ratios of earthquake to pre-earthquake period determinations for buildings subjected to the San Francisco earthquake.

assumptions made in arriving at estimates of stiffness of various complex substructures? Although many of the interacting factors may be selfcompensating and have little effect on critical load, a complex probabilistic study carefully based on real data as outlined in Sections 4-10 of this report is needed. Such a study can yield a meaningful assessment of current

SMC. Various studies, as found in Refs. 45 and 46, help to show both that such an analysis is possible and the possible variation in results over typical deterministic analysis. The main difficulty with these analyses is that no attempt is made to model the entire SMC. For the most part, subjective probability distribution functions were used.

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Section 4

SYSTEMS ANALYSIS METHODOLOGY

INTRODUCTION

To achieve the goals outlined in Section 1, we have, as discussed in Sections 2 and 3, several major problems to solve. The most difficult of these problems is to develop a basis of comparison for judging the results obtained by using the current SMC. As noted earlier, we would like to use test data from prototype systems subjected to design loadings. Although reasonable simulation of seismic conditions can be obtained via field-testing prototype or model reactors at the Nevada Test Site (NTS), this procedure is expensive. In addition, even if a testing program were undertaken, it would necessitate an overall calculation approach that could be used to extend the results of a limited number of tests to generic conditions. Thus, an alternative approach is to develop a computational approach that simulates all the major interdependencies of the overall system.

In this section, we discuss how such a practical computational approach can be developed. We also outline the type of analysis that can be performed by using such an approach. In the next section, we give the details of how such an

analysis can be used to develop a reasonable basis for judging the current SMC.

Realistically, it is not yet possible to model every detail of a large complex system such as a nuclear power station. The proposed assessment of conservatism is not directed towards the fine details of modeling each structural joint, pipe hanger, etc., but is primarily a global assessment of the loads imposed on various subsystems, and the stresses in the major structural elements. Much of our effort is directed towards interlinking the entire analysis so that important interactions are both included and interconnected. To do this requires the use of a systems approach, and the reasons why were detailed in Section 2. This approach would involve developing a model for the system and then analyzing the model. The model of the system will characterize the entire SMC associated with representative nuclear power plant structures, subsystems, piping, and equipment. As discussed in Section 3, Fig. 3-1 illustrates a generic breakdown of the methodology chain we have extracted from various sources. Primarily, this breakdown is for convenience of discussion and for

implementation of work assignments although the divisions do, in fact, reflect reasonably definable methodology interfaces. In the systems model, we characterize the different individual methodology steps as probability distributions reflecting the possible uncertainty and conservatism introduced at that step. The model will also reflect the interactions and dependencies of the different methodology steps.

The analysis will employ Monte Carlo techniques. Such analysis will permit the development of probability density functions (PDF) for the critical structural parameters (e.g., stress or strain) that characterizes the integrity of the structure, subsystem, piping or equipment being considered. From integration of the PDF's, quantified assessments of the total conservatism in the SMC can be obtained. The analysis will also include a sensitivity study which permits quantified assessments of the relative impact of the different methodology steps on the total conservatism. Such results provide a sound technical basis for recommending research and/or modifications to existing regulations.

The probabilistic approach also offers other advantages in overcoming the difficulties discussed in Section 5. Clearly, the state-of-the-art is not yet far enough advanced to provide

a fully realistic calculation. The many shortcomings and difficulties of doing this will be discussed in detail later. However, a meaningful basis can be developed by incorporating the most advanced state-of-the-art, and "ad hoc" correction factors as part of the system model, and recording the uncertainty of each aspect of the overall calculational scheme.

The credibility of the analytical results will depend largely on the comprehensive development of PDF's for each step of the SMC which will entail a major effort. We envision a case-by-case evaluation with the technical basis developed for the more difficult PDF's. This is discussed in some detail in Sections 6-10.

Details of the probabilistic approach are given below, the main analytical techniques broken down into five parts: (1) probabilistic analysis of the SMC, (2) importance sampling, (3) statistical characteristic of input parameters, (4) time series for seismic computer codes and development of interpolation functions, and (5) sensitivity analysis.

PROBABILISTIC ANALYSIS OF SMC

The "systems model" links all the independent variables and their associated PDF's with the dependent variables of interest. For the most part, the relation between dependent

and independent variables are expressed in complex computer programs such as SAPIV, LUSH, etc. The independent variables are all those describing earthquake ground motion, site characteristics, and structural characteristics. The dependent variables are calculated structural loads, stresses, and floor response spectra, etc. In a Monte Carlo simulation, a trial consists of selecting a random value from the distributions of each independent variable. These random values are weighted to fit the specified input distributions. A large number of trials then produce a precise estimate of the probable system behavior for the range of variability specified for the independent variables.

The probabilistic approach is valuable in that it provides more information than the deterministic approach. A distribution for the system stress will be obtained, giving not only the average stress but illustrating the possible deviations from the average.

IMPORTANCE SAMPLING

Importance sampling concepts will play an important role in the analysis because when low probability values are involved, a large number of Monte Carlo trials are required. Importance sampling allows us to direct these random choices to the

tail of the distribution and thereby provides a reduction in the number of Monte Carlo trials needed. The details of this technique are discussed below.

To perform this importance sampling procedure, a weighting function, which may be converted into a distribution function is needed. Let's first illustrate the single variable case.¹ Assume that the distribution containing our random variable X is known, and we want to generate an estimate of the probability that $X < X_1$. Using normal Monte Carlo procedures, assign a value of 1 to each trial where $X < X_1$, and a value of 0 otherwise. Note that if the failure probability of concern is on the order of 10^{-3} , then on the average, the Monte Carlo run would assign the value 1 to only one of every 1000 trials generated. This, of course, would require a large number of trials to obtain the desired accuracy. To reduce the number of trials required, importance sampling techniques will be employed. As mentioned earlier, this requires applying a weighting function to the uniformly distributed random numbers generated to use when simulating the appropriate distributions. To illustrate, introduce a weighting function $f_2 [F_1(x)]$ and let it be uniformly distributed over $0 \leq F_1(x) < 0.001$, and then $f_2 [F_1(x)]$ is set equal to 1000 for all values

of $F_1(x)$ in this range. By integrating the weighting function's cumulative distribution function (CDF), $F_2[F_1(x)]$ is generated. Again obtain uniformly distributed random numbers and assign them to $F_2[F_1(x)]$, then choose the corresponding $F_1(x)$ values and the corresponding X values. For all X values that fall below x_1 , assign a weight of $1/f_2[F_1(x)]$ instead of a value of 1. For our example,

$1/f_2[F_1(x)] = 1/1000 = 0.001$. Thus, the probability of $X < x_1$ is estimated correctly as 0.001 on every trial.

For real problems, determining the appropriate weighting function or even the distribution of the variable of concern can present problems.

However, the amount of computer time saved, particularly for this seismic problem, outweighs the effort required in generating the weighting function. To illustrate this point, we present the following example.² Assume we have a probability density function, PDF, that is exponential:

$$p(x) = 0.01 \exp(-0.01x) \quad 0 \leq x < \infty. \quad (1)$$

Therefore,

$$p(x < 1) = 1 - \exp(-0.01) = 0.00995. \quad (2)$$

Using normal Monte Carlo procedures, hundreds of trials would be required

to obtain the result. Therefore, the density function can be distorted by

$$p^*(x) = \exp(-x). \quad (3)$$

Thus, we get

$$p^*(x < 1) = 1 - \exp(-1) = 0.63. \quad (4)$$

Now start the Monte Carlo and assume the first value of x generated is 3. The likelihood of generating this value in the undistorted case is

$$p(3) = 0.01 \exp(-0.03) = 0.009704. \quad (5)$$

For the distorted case,

$$p^*(3) = \exp^{-3} = 0.049787. \quad (6)$$

Therefore, the ratio of the likelihoods is

$$p(3)/p^*(3) = 0.009704/0.049787 = 0.194910. \quad (7)$$

This says that the undistorted sample requires approximately five times as many trials to obtain the same number of samples in the range between 3 and $3 + dx$. Clark² generated the likelihood ratio for values other than $x = 3$. The results are shown in Table 4-1.

Therefore, we have shown that importance sampling techniques will play an important role in helping to reduce significantly the required number of Monte Carlo trials.

Table 4-1. Weights related to the use of $p^*(x)$ as a distortion of $p(x)$.²

x	$p(x)$	$p^*(x)$	$p(x)/p^*(x)$
0.1	0.0099900	0.90484	0.0110
0.2	0.0099800	0.81873	0.0122
0.3	0.0099700	0.74082	0.0135
0.4	0.0099600	0.67032	0.0149
0.5	0.0099501	0.60653	0.0164
0.6	0.0099402	0.54881	0.0181
0.7	0.0099302	0.49659	0.0200
0.8	0.0099203	0.44933	0.0221
0.9	0.0099104	0.40657	0.0244
1.0	0.0099005	0.36780	0.0269
2.0	0.0098020	0.13534	0.0724
3.0	0.0097045	0.049787	0.1949
4.0	0.0096080	0.018316	0.5246
5.0	0.0095123	0.0067379	1.4120
6.0	0.0094176	0.0024788	3.7990
7.0	0.0093239	0.00091188	10.2200
8.0	0.0092311	0.00033546	27.5200
9.0	0.0091393	0.00012341	74.5600
10.0	0.0090484	0.000045400	199.3000

STATISTICAL CHARACTERIZATION OF INPUT PARAMETER

Initial efforts should be directed towards identifying the significant input parameters such as peak g level of ground acceleration, total duration of strong motion, phase angle, soil-layer modulators of elasticity, etc. These input parameters will be considered as random variables for the purposes of the study. Appropriate statistical information must be obtained

to characterize these parameters as statistical distributions. The credibility of this characterization will depend largely on the amount of available data. Two cases will be considered: the no-data case where engineering judgment will be used as the basis of specifying the appropriate distribution in this case; and the some-data case where some data are available. In the latter case, engineering judgment can be combined

with the sample observations by using Bayesian inference to specify the distribution. To illustrate these procedures, the problem of specifying an appropriate distribution for peak of values of ground acceleration will be studied.

In the first case, i.e., no-data case, an effort must be made to characterize the distribution type. From the data analysis of peak acceleration values in O'Brien, Murphy and Lahud,³ the lognormal distribution for peak acceleration values will be assumed appropriate. If Y is peak acceleration, then $X = \log Y$ is normal with mean θ and variance σ^2 and Y has mean $\exp(\theta + 1/2 \sigma^2)$ and standard deviation,

$$\text{Var}(Y) = e^{\theta} W(W-1), \quad (8)$$

where

$$W = \exp(\sigma^2).$$

For illustrative purposes, we shall determine the distribution of Y for an eastern site. This requires the use of engineering judgment to specify two points on the lognormal distribution curve. For example,* assume

*This example is made simply for illustrative purposes. Sections 6 and 7 give in detail a more appropriate way to characterize the input. However, the detailed approach outlined in Sections 6-7 is much more complicated and not suited for illustrative purposes.

$$P[Y > 0.1g] \sim 10^{-3}, \text{ and}$$

$$P[Y > 0.3g] \sim 10^{-4}.$$

This determines two linear relationships between θ and σ ; namely,

$$\log(0.1) - \theta = 3.0905 \sigma, \text{ and}$$

$$\log(0.3) - \theta = 3.71912 \sigma,$$

so that $\sigma = 1.7476$ and $\theta = -7.7036$. It follows that Y has mean 0.0021 and standard deviation 0.0093 g.

For a western site, assume

$$P(Y > 0.1g) \sim 2 \times 10^{-1}, \text{ and}$$

$$P(Y > 0.3g) \sim 2 \times 10^{-2}.$$

Then as before,

$$\log(0.1) - \theta = 0.8414 \sigma, \text{ and}$$

$$\log(0.3) - \theta = 2.0541 \sigma,$$

and $\theta = -2.9931$ and $\sigma = 0.9059$. It follows that Y has mean 0.0765 g and standard deviation 0.0852 g in this case.

For the case where data are available, use (as an example) the sixteen peak acceleration values for El Centro, California shown in Table 4-2. Analysis of these data provides us with a distribution of Y , the peak acceleration value, given that an earthquake has occurred, and accomplished as described below.

A total time on test plot of the data in Table 4-2 indicates that a lognormal probability distribution model would be appropriate. Letting Y be peak acceleration, it follows that Y has density,

$$\frac{1}{y\sigma\sqrt{2\pi}} e^{-1/2\left(\frac{\log y - \theta}{\sigma}\right)^2}, \quad (9)$$

and the likelihood, given observations, $\underline{Y} = (Y_1, Y_2, \dots, Y_N)$ is

$$L(\theta, \sigma | \underline{Y}) = \frac{e^{-1/2 \sum_{i=1}^N \left(\frac{\log Y_i - \theta}{\sigma}\right)^2}}{(2\pi)^{N/2} \sigma^N \prod_{i=1}^N Y_i}. \quad (10)$$

Table 4-2. Peak ground accelerations from El Centro, California.

Time	g
December 1934	0.12
April 1938	0.048
June 1938	0.03
June 1938	0.012
May 1940	0.314
October 1942	0.06
January 1951	0.03
January 1953	0.035
November 1954	0.027
December 1955	0.03
December 1955	0.007
December 1955	0.07
February 1956	0.05
February 1956	0.015
August 1966	0.015
April 1968	0.13

Assuming a noninformation prior density for (θ, σ) , we find that the posterior density of (θ, σ) is

$$\Pi(\theta, \sigma | \underline{Y}) = \frac{L(\theta, \sigma | \underline{Y})}{\int_{-\infty}^{\infty} \int_0^{\infty} L(\theta, \sigma | \underline{Y}) d\theta d\sigma}, \quad (11)$$

for $-\infty < \theta < \infty$ and $\sigma > 0$. The unconditional density of Y , then, is

$$-\int_{-\infty}^{\infty} \int_0^{\infty} \frac{1}{\sigma y \sqrt{2\pi}} e^{-1/2\left(\frac{\log y - \theta}{\sigma}\right)^2} \Pi(\theta, \sigma | \underline{Y}) d\theta d\sigma. \quad (12)$$

For our Monte Carlo study, we would simulate peak g values by using the unconditional density of peak g values computed in this way.

The sample mean of logarithmic values is $\bar{X} = -2.98$, and the sample variance is 1.19. The corresponding mean, $E[Y/\text{Earthquake occurs}]$, is 0.092 g , and standard deviation is 0.139 g conditional on earthquake occurrence for the lognormal distribution. Sixteen earthquakes* were recorded in 34 years so that $\lambda = 16/34 = 0.471$. Assuming a Poisson distribution for the number of earthquakes per year, we have $1 - 3^{-\lambda} = 0.376$ probability of at least one earthquake

*For purpose of illustration, only the earthquakes which tripped the strong motion accelerograph were used in the analysis. More correctly, the historical record should be researched to ensure that all earthquakes were used.

in any given year. To compare our sample data with our subjective probability distribution for eastern and western sites, we need the mean and standard deviation of Y.

Now,

$$EY = \int_0^{\infty} y P[Y > y | \text{earthquake occurs}] dy P[\text{earthquake occurs}], \text{ and}$$

$$EY = (0.092) (0.376) = 0.035.$$

Likewise,

$$EY^2 = E[Y^2 | \text{earthquake}] P[\text{earthquake}],$$

$$EY^2 = [\text{Var}(Y | \text{earthquake}) + E^2(Y | \text{earthquake})] (0.376),$$

and

$$EY^2 = 0.010.$$

Therefore, the unconditional variance of Y is

$$EY^2 - (EY)^2 = 0.0087 \text{ g},$$

and the unconditional standard deviation is 0.093 g. The unconditional mean and standard deviations for both eastern and western sites are shown in Table 4-3.

TIME SERIES MODELING FOR SEISMIC COMPUTER CODE

The Monte Carlo simulation will be performed on a systems model that incorporates the most computationally time-consuming subsystems, e.g., site

Table 4-3. Peak ground acceleration statistical information.

Site	Mean, g	Standard deviation, g
Eastern	0.002	0.009
Western	0.076	0.085
El Centro	0.035	0.093

response, soil-structure interaction, and structural-response subsystems. Computer time on the complex systems model could thus become prohibitive. To offset this problem, we intend to develop sample relationships for the seismic computer codes that will be required. These relationships will be developed by using a time series approach and by developing interpolation functions that employ multiple regression theory.

The input and output functions of the SHAKE and FLUSH computer codes are, in large part, accelerogram-time histories. The codes are essentially linear-transfer functions operating on time histories. We propose to approximate these computer-code transfer functions, independent of input-time histories, by parsimonious linear-transfer functions using white-noise input and Box-Jenkins statistical techniques.⁵ The coefficients of our parsimonious approximating model will be functions of the INPUT PARAMETERS (e.g., soil density,

shear modulus, etc.). Using multivariate regression techniques, we hope to express the model coefficients as linear functions of the INPUT PARAMETERS.

The first benefit of the approach is that we will be able to assess the sensitivity of the computer codes to variations in the INPUT PARAMETERS.

The second benefit is that we will be able to simulate quickly the computer-code output accelerograms for arbitrary input accelerograms (including nonstationary stochastic processes).

An example using the SHAKE computer code would now be appropriate. The input to the computer code SHAKE consists of the following INPUT PARAMETERS: 1) shear modulus, 2) critical damping ratio, 3) soil density and 4) thickness of each layer. To run SHAKE, we must also input a free-field ground-acceleration time history. The output from SHAKE is the bedrock-acceleration time history. Since SHAKE is a complex linear-transfer function of the input time history, it seems reasonable to try to approximate the output of SHAKE by a parsimonious (i.e., minimum number of parameters) linear-transfer function.⁵

We propose to do this by using the Box and Jenkins scheme for fitting an arbitrary time history by an autoregressive, integrated, moving average

model. The idea is to input a white-noise process to SHAKE for a given INPUT PARAMETER configuration. The white-noise process will be denoted by $(a_t)_t^N = 1$. SHAKE typically uses $N = 800$ observations. Denote the output process by $(Z_t)_t^N = 1$. Using the Box and Jenkins procedure, we will identify models like the following:

$$Z_t = \phi_1 Z_{t-1} \dots - \phi_p Z_{t-p} \\ = a_t - \theta_1 a_{t-1} \dots - \theta_q a_{t-q}, \quad (13)$$

where $(\phi_1, \phi_2, \dots, \phi_p, \theta_1, \dots, \theta_q)$ are the model coefficients. An attempt will be made to obtain an adequate fit (with respect to residual mean square error) by using, at most, four non-zero coefficient values. The adequacy of the model fit will be determined by diagnostic checking of the residuals.

Let $\underline{\alpha}$ be the vector of input parameters. Then clearly the model coefficients will be functions of the input parameter vector $\underline{\alpha}$, i.e.:

$$(\phi_1(\underline{\alpha}), \dots, \phi_p(\underline{\alpha}), \theta_1(\underline{\alpha}), \dots, \theta_q(\underline{\alpha})). \quad (14)$$

For each choice of $\underline{\alpha}$, the model coefficients will be obtained by using Box and Jenkins statistical techniques. We will attempt to fit a linear regression model to each mapping: $\underline{\alpha} \rightarrow \phi_i(\underline{\alpha})$, $i = 1, 2, \dots, p$ and $\underline{\alpha} \rightarrow \theta_j(\underline{\alpha})$, $j = 1, 2, \dots, q$ by using the

interpolation function approach.⁶

Basically, interpolation functions are simple relationships that represent complex and time-consuming computer codes. These interpolation functions can be obtained by using a multiple regression theory that allows the significant input parameters and their interactions and dependencies to be taken in all combinations. Linear-regression models for each combination are formed and regressed against the data by using a procedure called combinational analysis. The interpolation function is then chosen among these regression models.

The SHAKE computer program output process will then be replaced by the stochastic process model,

$$Z_t - \phi_1(\alpha)Z_{t-1} - \dots - \phi_p(\alpha)Z_{t-p} = a_t - \theta_1(\alpha)a_{t-1} - \dots - \theta_q(\alpha)a_{t-q}, \quad (15)$$

where $(a_t)_t^N = 1$ is a white-noise process. Define the backward shift operator, B , by $BZ_t = Z_{t-1}$. Define

$$\phi_1(B) = 1 - \phi_{11}(\alpha)B - \dots - \phi_{1p}(\alpha)B^p, \text{ and} \quad (16)$$

$$\theta_1(B) = 1 - \theta_{11}(\alpha)B - \dots - \theta_{1q}(\alpha)B^q. \quad (17)$$

Then (15) can be rewritten as

$$\phi_1(B)Z_t = \theta_1(B)a_t, \quad (18)$$

or

$$Z_t = \phi_1^{-1}(B)\theta_1(B)a_t. \quad (19)$$

For arbitrary input ground motion $(X_t)_t^N = 1$, we get corresponding SHAKE output by using Eq. (19),

$$y_t = \phi_1^{-1}(B)\theta_1(B)X_t, \quad (20)$$

with X_t replacing white noise $(a_t)_t^N = 1$. In principle, we can model any ground-motion time history by an autoregressive, integrated, moving average (ARIMA) model. Sample ground-motion time histories can then be generated at will by simulating white noise and using the white-noise process to "drive" the ARIMA model.

To estimate our error in approximating computer codes by ARIMA transfer function models, we will want to use slightly more complicated versions of the above models. If $(X_t)_t^N = 1$ is a stochastically specified ground motion (i.e., we know its ARIMA model), then we may want to consider the model,

$$\phi_1(B)Z_t = \theta_1(B)X_t + N_t(1), \quad (21)$$

where Z_t is the computer-output process and $N_t(1)$ is a noise process (not necessarily white noise). The variance of the noise process, $N_t(1)$, would measure our error in approximating the computer program by the

transfer function model $\phi_1^{-1}(B)\theta_1(B)$. The noise process, $N_t(1)$, would also, in general, have to be modeled by an ARIMA model.⁵

Let the initial input ground motion be denoted by X_t . Then,

$$\phi_0(B)X_t = \phi_0(B)a_t^0 \quad (22)$$

is an ARIMA model for the X_t process. By simulating the white-noise process (a_t^0), we can simulate initial ground-motion time histories at will.

Let Y_t denote the output process from SHAKE. Then,

$$\phi_1(B)Y_t = \theta_1(B)X_t + N_t(1) \quad (23)$$

is our general transfer function model for SHAKE, and $\sigma_1 = \sqrt{\text{Variance } (N_t(1))}$ is our error in approximating SHAKE by a parsimonious linear-transfer function model.

Let W_t denote the output process from FLUSH. Then,

$$\phi_2(B)W_t = \theta_2(B)Y_t + N_t(2) \quad (24)$$

is our general transfer function model for FLUSH and $\sigma_2 = \sqrt{\text{Variance } N_t(2)}$ is our error.

It is now clear that we can simulate output from FLUSH in terms of the input ground motion to SHAKE as follows. Since

$$Y_t = \phi_1^{-1}(B)\theta_1(B)X_t + \phi_1^{-1}(B)N_t(1), \quad (25)$$

and

$$W_t = \phi_2^{-1}(B)\theta_2(B)Y_t + \phi_2^{-1}(B)N_t(2), \quad (26)$$

we have

$$\begin{aligned} W_t = & \phi_2^{-1}(B)\theta_2(B)\phi_1^{-1}(B)\theta_1(B)X_t \\ & + \phi_2^{-1}(B)\theta_2(B)\phi_1^{-1}(B)N_t(1) \\ & + \phi_2^{-1}(B)N_t(2). \end{aligned} \quad (27)$$

The FLUSH output process W_t can thus be directly simulated from the input process X_t . For simulation purposes, we can rewrite the above as

$$\begin{aligned} \phi_1(B)\phi_2(B)W_t = & \theta_1(B)\theta_2(B)X_t \\ & + \theta_2(B)N_t(1) \\ & + \phi_1(B)N_t(2). \end{aligned} \quad (28)$$

This procedure is illustrated in Fig. 4-1.

SENSITIVITY ANALYSIS

A sensitivity analysis will be performed to determine the effects of the input parameter values and distributional assumptions for these values on the systems PDF's. This sensitivity analysis will be performed during the normal Monte Carlo computer runs in order to save computer time and to make use of all the information generated during that regular Monte Carlo run. This is done by evaluating

Fig. 4-1. Time series modeling program.

the changes in output with respect to the input values selected during the simulation procedure. These changes would then have to be weighted to

reflect the appropriate parameter of concern. This provides a measure for the relative effect of each input parameter on the system variation.

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Section 5

APPLICATION OF SYSTEMS ANALYSIS METHODOLOGY TO THE SMC

INTRODUCTION

In this Section, we show how the systems analysis discussed in Section 4 can be applied to develop a basis of comparisons for judging the current SMC. In addition, because the current SMC is an extremely complex interconnected methodology chain (as discussed in detail in Sections 2 and 3), it is necessary to "quantify" the current SMC so meaningful comparisons and assessments can be made.

To solve the problem of developing a basis that can be used to assess the results obtained from the current SMC, as well as establish a meaningful representation of the output of the current SMC relative to its possible variability discussed above, we propose the development of two SMC models. One model would represent the current SMC and the other would represent an improved SMC (discussed in detail below*). A probabilistic analysis (discussed in Section 4) would be performed on both. The results of this analysis will include both the most likely value of each,

selected, critical structural-integrity parameter and its expected variability. Results from the current SMC will be compared with those from the improved SMC.

The improved SMC will be developed largely from the studies discussed in Sections 6-10 which focus on individual steps in the SMC. These results will form the basis of a new SMC representing, to the limits of state-of-the-art, a better approximation for each step of the SMC. Note that the improved chain is not being suggested as new regulations because a few of the "better estimates" may be minor and unduly complex. However, we feel only through complete analysis of the entire system can one determine the relative importance of the different methodology steps and the appropriate level of complexity. Only then, after understanding the true impact of a proposed change, would it be possible to recommend practical improvements.

Major project emphasis is directed toward developing the improved SMC. The systems-analysis results will provide the real technical basis for assessing and evaluating the current methodology chain, developing quantified assessments of the total conservatism in the existing chain, and

*Refer to the section on Improved SMC in this Section and to Sections 6-10 for the details of the rationale used to develop the improved SMC.

making recommendations for improved regulations and/or needed research.

The number of variables and combinations in the SMC is considerable. It is not necessary to run all cases or consider all possible structures to achieve our objectives because we are dealing with a predictive methodology. Thus, the initial efforts focus first on the most important factors; then the analysis is easily extended to include extremes (i.e., rock and soft soil sites, to different structures), with conclusions drawn after each analysis.

CURRENTLY REGULATED SMC

Figure 5-1 is a flow chart of SMC we propose for use in characterizing the important, currently regulated methodology.

The current chain is intended to include all important methodology steps. These steps start with the specification of seismic design ground motion, proceed through the calculation of seismic stresses or strains, combine seismic and nonseismic stresses or strains, and then make comparisons with code-defined allowables.

It should be remembered that the primary purpose of characterizing the current SMC is to have the ability to compare systems analysis results with the improved SMC. Our basic intent is to focus on the important sources

of conservatism and possible variation. Therefore, rather than include all of the many alternative methodologies acceptable to applicants at the different steps, we propose using either what we judge the "best" currently acceptable approach and/or the methodology commonly used. For example, rather than including both a soil-spring lumped-mass half-space approach and one that incorporates finite-element techniques for soil-structure-interaction (SSI) analysis, we suggest using only a LUSH-like (finite-element) approach. In the structural response area, rather than considering several response analysis techniques (i.e., time history, response spectra, etc.), we suggest that only a time-history analysis type be used. A "package study"* is included to address the sensitivity of the different acceptable methodologies on the total methodology process (see Section 10).

In developing the SMC shown in Fig. 5-1, we have tried to maintain a

*In planning the overall flow of information to maximize useful results, we have found that it is preferable to "break-out" different issues into a separate study that can be ongoing while the main system models are being developed. These are referred to as "package studies." They will supply both useful and timely information to NRC as well as important input into the systems model. These "packages" are discussed in detail in Sections 6-10.

*Procedures reflect eastern U.S. site location.

†Soil profile and properties will reflect existing sites (soil and rock).

Fig. 5-1. Seismic methodology chain reflecting current regulations.

reasonably generic methodology chain to give the systems analysis results a larger impact. However, as pointed out earlier, the total number of possible combinations is large. Thus, for practical purposes, we propose that the initial effort be simplified so that the methodology chain is directed at an existing nuclear power plant site located in the eastern U.S. on a soil site. Furthermore, the reactor plant is assumed to be a BWR type, which is characterized by the GESSAR balance of plant concepts. The eastern U.S. selection seems reasonable since it reflects the location where the majority of nuclear reactors are and will be located. We also chose soil site over rock as more representative. There are a number of criteria that could be used to select the detail definition [the site (layering), depth to bedrock, etc.]. We propose that, for the first pass, one should keep as close as possible to a real site with data. In doing so, one can get a realistic measure of the possible variability in the definition of site parameters used in the analysis. GESSAR is tentatively selected because it incorporates the latest BWR containment considerations based on the past-year experiences. It should also be fairly representative of future BWR designs. In addition, General Electric has informally agreed

to make the necessary detailed structural information available.

After our initial systems analyses are performed by use of the current SMC as shown in Fig. 5-1, we will have the insight needed to choose the next step. Each pass through the system analysis is a lengthy process, mainly because of the SSI calculation and the complex structural model required. Thus, it is not really practical to allow all parameters of the model to take on all possible variations. As discussed above, our approach is a bootstrap process, i.e., at each step one draws conclusions and determines which case should be considered next.

Seismic Input Methodology

For the typical eastern U.S. reactor location, the current methodology for defining the free-field surface ground motion is quite straightforward. For a selected region or geologic province, a peak ground acceleration is specified simply to scale the design response spectra defined by R.G. 1.60. We suggest the use of peak ground-motion values ranging from 0.05 to 0.30 g. This range should cover both SSE and OBE peak ground motion values for all eastern U.S. locations. PDF's are not required as both the peak g value and, typically, the spectral shape

are regulated, these should be held fixed. That is, for the first complete Monte Carlo simulation, we suggest that the site g value be equal to 0.2 g , which is consistent with typical soil sites in the East.

Site Response Methodology

The site response methodology as discussed in Section 3 currently involves first the generation of an accelerogram, which will adequately represent the scaled surface-response spectra. Next, properties and a soil profile for the specified site must be defined. Finally, SHAKE-like calculations are performed to develop a bedrock accelerogram for subsequent SSI calculations.

We propose the use of a random-phase-like analysis technique to develop the surface accelerogram. Because of the repetitive nature of this process (i.e., many accelerograms are reasonably suitably within current guidelines), one can express the variability of the analysis input parameters as PDF's. Included will be such parameters as phase angle, total duration and duration of strong motion. These PDF's will be obtained by using a stochastic process of parameter selection with the random-phase-like analysis technique to develop acceptable response spectra within currently specified guidelines.

An alternative is to use a suite of surface accelerograms rather than expressing the variability as PDF's. Each accelerogram would be an acceptable representation of the response spectra. In the suite, we could include accelerograms currently used by industry (e.g., Bechtel, Stone-Webster, General Electric, etc.).

Relative to the soil profile and site properties, we propose that all of the necessary SHAKE-like input parameters be expressed as PDF's to reflect possible variations. These soil profile parameters will include shear modulus and damping as a function of strain level. These PDF's can easily be extracted from studying available soil-boring data and laboratory data from sites with similar characteristics. Preliminary and Final Safety Analyses Reports (PSAR and FSAR) will form the primary source for this information. While some arbitrariness is required in selecting the layering and the PDF's that gives the weighed range of the thickness for each layer, the basis will be related to real sites.

The newly published SRP has a new requirement that the spectra in the free-field at foundation level must be at least 60% of the surface value. Since there are several possible ways to achieve this, we suggest that NRC and industry be contacted to determine

what particular approach, if any, most applicants plan to follow, and then use it. However, where considerable variability among applicants exists this is inserted into the chain. That is, one of the different approaches typically used would be randomly chosen on each trial of the Monte Carlo simulation.

SSI Methodology

This methodology involves the generation of a basemat or foundation-level ground motion for subsequent structural-response calculations. This will first involve the selection of an SSI model, then the identification of a soil profile including soil properties and, finally, the performance of response calculations using LUSH-like analytical techniques.

As discussed above, we propose the use of GESSAR as the basis for developing or selecting a calculational model. Although several different approaches are currently used to develop a model of a structure for a finite-element SSI calculation, presently we plan to use only one model. Our judgment is that a sufficiently credible model accounting for possible SSI effects is adequate, i.e., other parameters are much more important. However, subsequent efforts may consider a suite of acceptable models to evaluate total system-results sensitivity to model

variability. Soil properties and soil-profile definitions will be based on those developed for the site-response methodology.

In addition, subsystem analysis studies including the seismic input, site response, and terminating with the SSI methodology can be used to evaluate and assess the current regulations relative to foundation integrity and liquefaction potential. The relevant LUSH-like analysis results will be soil stresses and deformation beneath the reactor. We can evaluate the variability of these parameters in the same manner we propose to treat those related to structural integrity. Soil strengths and deformation allowable will be used to assess the possible degree of conservatism.

Structural Response Methodology

We consider the current structural-response methodology in several steps. First, a calculational model that simulates the response characteristics of the structure, subsystem, piping, or equipment to be investigated is developed. Then member forces and/or floor-level loading are calculated by using linearly elastic dynamic analysis techniques. Basemat motion derived from the SSI analysis is used as the exciting motion. The member forces are used to calculate structural parameters such as stress

or strain that characterize member strength or deformation integrity. The floor-level loading, usually expressed as an accelerogram, is used to develop floor response spectra for subsequent subsystem, piping, and/or equipment analysis. The floor spectra are used for input loading into dynamic analysis. They are also used for load definition if experimental testing is used to qualify seismically equipment integrity.

Note that both member forces and floor-level loading are sometimes obtained directly from the SSI calculations. Nevertheless, we feel the approach described above is more representative.

Our initial systems of the current SMC will address the GESSAR reactor building. Subsequent systems studies, as discussed earlier, will address other representative structures, subsystems, equipment, and piping.

For our system studies on the GESSAR reactor building, we plan to use one calculational model, and the GESSAR document will be used as the primary basis for the model. Damping values selected will reflect R.G.

1.61. The possible variability of structural dynamic characteristics such as mass, stiffness, composite-damping treatment will be expressed as PDF's. The PDF's will be developed from studying the PSAR's and FSAR's of other similar structures and from

discussion with NRC staff and industry. They may be supplemented with some theoretical studies as well.

Stress Analysis Methodology

This methodology chain involves, first, the calculation of a strength or deformation parameter that defines the structural integrity of the structure, subsystem, piping, or equipment being investigated. Stress or strain quantities appear reasonable choices. Next, it involves an estimate of a nonseismic stress or strain and how the two will be combined. Finally, an allowable stress or strain will be selected and compared.

One should follow SRP guidelines for combining different earthquake loading direction, when combining seismic and nonseismic effects and when selecting allowable stress or strain values. We propose the use of GESSAR methodology as the basis for calculating stress or strain values from previously calculated member forces and for providing an estimate of the nonseismic stress.

Seismic stresses considered will include those caused by both OBE and SSE loading conditions. These will be combined with nonseismic stresses resulting from normal and accident loads. Both seismic and nonseismic stresses will reflect directional

considerations. Since systems studies will involve time-history analysis rather than response spectra, it will not be necessary to address the effects of closely-spaced modes. To limit the project scope, the NRC staff (OSD) suggested we not include the study of variation, uncertainties, and conservatism contribution embedded in the SRP load-combination formulae, the nonseismic stress or strain estimates, and the code-allowable stress or strain.

IMPROVED SMC

The major difficulties that must be addressed in any creditable computational to serve as a basis for assessing the conservatism afforded by the current SMC are:

- Proper modeling of a large and complex system so that all important details and interactions are included.
- Specification of the input.
- Carrying out a realistic nonlinear analysis by using "correct" constitutive laws for the many materials/components included in the model.
- Translating the results of the analysis into "failure" of elements/components.

- Translating the calculated "failure" into release of off-site radioactivity.

In Section 4, we discussed how the "systems" approach appropriately modeled the necessary interactions of the various components and the consequences of the assumptions made during the analysis.

In Sections 6-8, we discuss how to specify the seismic input in a realistic manner consistent with our proposed analysis and goals. In Section 9, we discuss how a realistic nonlinear analysis can be formulated. Because the last two items listed above (per discussion with NRC-OSD) are not addressed in the same detail as the first three items, the results of the proposed analysis are somewhat limited.

Our basic approach is to incorporate technology presently available into the current SMC, and thus provide an improved methodology chain and more realistic results. The approach that we have outlined to better assess the "true" value of critical design parameters includes identifying what we consider the most important shortcomings of the current SMC (discussed in some detail in Section 3). We have also reviewed the most current state-of-the-art and selected methods of analysis which,

relative to the identified restrictions, are more general. In addition, because no one approach, yet available, has removed all the identified restrictions, we will outline how some "ad hoc" correction factors can be developed to combine several different calculational schemes. The probabilistic approach allows such correction schemes to be incorporated realistically into the analysis. While this approach is certainly not the "last word," it will give the best assessment possible within the state-of-the-art. In addition, because each important factor is included, it is possible to identify what parts of the SMC may or may not require improvement even though some parts may seem extremely crude.

Improved Seismic Input Methodology

Under the current procedure for sites located in the East discussed in Section 3, the design response spectra is based on the intensity of the largest earthquake that has occurred in the historical record for the site's province. The peak acceleration is arbitrarily scaled from an intensity-peak acceleration correlation, and the response spectra is averaged (1-sigma) from many earthquakes that do not appropriately model the SSE. The improvements to be incorporated into the SMC are as follows:

- An appropriate earthquake-occurrence model will be used to account for local seismicity.
- A maximum earthquake will be based on local seismicity data by extreme value methods.
- The response spectra (shape and level) and corresponding time-history for each step of the Monte Carlo scheme will be consistently chosen to reflect appropriately the magnitude of the individual earthquake and its location relative to the site-soil conditions.

The details of how each of these factors will be obtained is given in Section 6. However, in summary, we can determine the appropriate earthquake-occurrence model for the region where the site is located. Thus, each trial of the systems analysis (Monte Carlo) starts by a choice of earthquake magnitude based on the earthquake-occurrence statistics. The earthquake is then located randomly relative to the site. (Note: for some regions we may have reason to locate earthquakes more frequently along some tectonic structure, but this is easily handled by a weighting scheme.) Using the earthquake magnitude, location, and regional attenuation characteristics and site soil conditions, we obtain

the correct spectral shape, spectral level and corresponding time-history at the site by using the appropriate set of data as defined from the results of Section 6.

Improved Site Response/ SSI Methodology

As discussed in Section 3, the current methodology in the site response/SSI area is centered around the SHAKE/LUSH-like analysis technique which lacks the following:

- Attention to nonvertical seismic motion (including the effect of wave passage on the response of massive foundations).
- Better accounting of nonlinear soil properties.
- Better reflect *in situ* soil characteristics.
- Consideration of 3-D effects.

The improved chain consists of a better accounting of the above factors by the development of two approaches (Sections 6-8). The primary way involves using new analysis techniques already available (or will be soon) and less restrictive than the current SHAKE/LUSH approach. However, as it is not possible to develop a general method of analysis that removes all the above mentioned restrictions, we will also incorporate correction factors into the systems analysis chain as described below.

In particular, if the results of the studies outlined in Section 7 find it necessary to replace SHAKE by a nonlinear code (compatible with the code used for SSI), the latter then will better approximate true nonlinear soil behavior. The soil-test data obtained from typical laboratory testing procedures should be corrected to better approximate *in situ* behavior in the manner discussed in Section 7.

It is not possible at this time to incorporate fully a proper division of seismic energy into the various wave types. In Section 7, we have outlined an approach that will allow the development of approximate correction factors to account for non-vertically propagating wave energy, surface waves, etc. However, if such corrections are not significant, they should not be applied. We envision this correction applied to the initial surface-input motion so that the deconvolved time history at the foundation level in the free-field would better approximate the "correct" motion at this level. This correction will be a function of the relative location of the earthquake to the site.

The results of the study may not be sufficient to allow for development of a reliable correction factor to apply to the time history at the surface. In this case, we also outlined in Section 7 how one could

develop an approximate criteria to fix how much alteration of the surface spectra should be allowed in the free-field at the foundation level. This would correspond to the NRC 60% criteria currently used.

In addition to the need to correct for nonlinear soil behavior, *in situ* soil properties, and division of seismic energy (among the various wave types discussed above), two other effects can occur in the SSI calculation that are distinct from the site response area. The first is the 3-D effect of the foundation and, in particular, how energy is radiated away from the foundation. This effect is discussed in detail in Section 8. The second is referred to as traveling waves. For structures with large basemat foundation, it can be very important since much more than simple wave passages are tied up in this effect (Section 8). However, it appears possible to lump much of this together by use of a simple traveling wave. If this effect is important, it will be included in the 2-D analysis. If 3-D effects are so important that a 3-D analysis must be used, then it will not be feasible to include a traveling wave in the 3-D analysis at this time. However, there are several possible ways of getting traveling waves in the 3-D analysis, and the best way would be to correct the output-basemat time

history in an approximate fashion by simple averaging process discussed in Section 8. This "improved" time history would then be used in the analysis.

As indicated in Sections 7-8, a dual assessment (both in site response and SSI areas) will be made for the need to replace the equivalent linear assumption with a full nonlinear approach. Although the equivalent linear method might be adequate for site response, the addition and complication of the very rigid structures might require a full nonlinear analysis. As noted above, if a nonlinear analysis is required, the approach used to generate the input for the SSI analysis will be compatible with the nonlinear equations of state used in the SSI code.

The systems analysis for the improved chain will proceed in a manner very similar to that provided for the current chain. The same sites, layering, etc., will be used, and the nonlinear soil properties will be compatible with the equivalent linear soil properties. The only real differences will be that the improved codes and correction factors will be included. Also, the requirement that the free-field spectra at basemat level be at least 60% of the surface spectra will be dropped or replaced by the criteria discussed above,

Improved Structural Response Methodology

Current structural response methodology is centered around elastic dynamic analyses using the SAPIV-like analysis technique. Our proposed plan is to replace the elastic analysis with one that incorporates inelastic-dynamic-response analysis techniques. This is not a simple process. Although inelastic-dynamic-response analysis methodology has received considerable study, the adaptation to reactor design is difficult. One source of difficulty is the nonseismic loads and the manner in which current codes combine nonseismic loads with the seismic. Clearly, for an inelastic analysis to be meaningful, all loads — both seismic and nonseismic — must be included in the analysis. How this should be done is part of the package study outlined in Section 9.

Another difficulty is the assignment of failure criteria. In a conventional building this can be done relative to collapse. However, a containment building could be considered "failed" if a vent path is opened up to allow radioactivity to be released, which makes it a much more difficult task to define "allowables." These considerations are part of a package study, discussed in Section 9, to select the best techniques and models.

Although a linear analysis is generally recognized as too conservative, there is considerable reluctance to use inelastic analysis for the difficulties discussed above. For this reason, we also propose that an elastic analysis very similar to that described above in the current SMC be carried out each trial (defined in Section 4). However, for the improved chain, the damping should be allowed to take on higher values that current data support. The manner in which these values can be obtained is given in Section 10. They should be entered as a PDF. However, because damping is nonlinear (i.e., a function of strain), the choice of damping values should be weighed relative to the appropriate level of spectra acceleration. Thus, if the earthquake ground motion is such that the basemat spectra has a high level of response (relative to the design OBE) at the first natural frequency of the structure, a higher damping value would have a much higher probability of being used than if the response (in frequency band of interest) was low. The rest of the variation in structural parameters would remain the same as for the elastic analysis.

Stress Analysis Methodology

We do not propose to incorporate any additional improvements in this leg of the improved SMC. As discussed

above in the section on structural response methodology, we proposed that both an elastic and inelastic analysis be carried out. The elastic-analysis case can proceed exactly as described above in the current SMC, but the inelastic analysis case is much more complex. As indicated above, both the seismic and non-seismic loads as well as the appropriate load factors should be included in the analysis. Thus, the stresses computed will include both seismic and nonseismic stresses. For these structures for which the nonseismic loads are small then, a direct comparison can be made between the elastic and inelastic analysis. For these cases where the nonseismic loads are important, it is somewhat more difficult to insure that all loads are being handled in a consistent manner. As already noted, this is one of the objectives of a package on inelastic analysis.

EXPECTED SYSTEM STUDY RESULTS

Systems studies will be performed on both the current and improved SMC's. Previous sections discuss the SMC's in detail and the type of analyses to be conducted. To develop study results that meet project objectives, it is necessary that the results be as generic as possible. To accomplish this, the initial

analysis results described above should be extended to consider:

- A peak free-field ground acceleration values range of 0.05 to 0.70 g.
- Soil and rock site.
- Eastern and western U.S. site locations.
- BWR and PWR reactor plants.
- Selected representative structures, subsystems, piping or equipment.
- OBE and SSE loading conditions.

A peak g range of 0.05 to 0.30 is adequate to cover all eastern U.S. reactor sites for both OBE and SSE relative to the current SMC. Larger g values may occur in the improved SMC. For western sites, one may consider increases to 0.7 g for the current chain while the improved SMC could have higher g values occur. The soil and rock site variation will permit an evaluation of the importance of SSI. SSI should have substantial influence on the total system result for soil sites, but not much for rock sites. Procedures for seismic input methodology must be accounted for because they differ considerably for eastern and western locations. Representative structures, subsystems, equipment and piping from both BWR

and PWR reactor plants must be considered because of their wide variations in their dynamic structural-response characteristics, the type of design analysis, nonseismic load contribution, and selection of allowable stresses. OBE and SSE loading conditions will reflect different choices of load-combination formula and selection of allowable stresses.

For each of the cases indicated above, system studies results will be centered around:

- The development of probability-density functions (PDF's) of some selected parameter characterizing the structural integrity of structure, subsystem, piping or equipment being studied.
- The relative effect that the different methodology parameters have on the total system results (i.e., the PDF's).

Figure 5-2 illustrates a typical PDF resulting from a system study using either of the SMC's and one combination of possible cases cited above. The selected parameter will probably be stress or strain. The PDF expresses the variability of the selected parameter, and the shape of this distribution is directly related to the PDF's that were assigned to

Fig. 5-2. Typical PDF from systems analysis.

characterize the different significant methodology steps. It also depends on the interrelations and dependencies of the different methodology parameters through the procedures and analysis techniques. Results will include a mean and a standard deviation, and these results allow us to meet our first main objective of quantifying the variation in the current SMC. Ideally, current regulations would yield a distribution that is peaked around the mean with small standard deviations. If this does not happen, then the sensitivity results will indicate what steps of the current SMC need to be better regulated and what must be done to improve the regulation.

By using only the seismic input, site response and SSI methodologies, one can also develop similar results

for foundation integrity and liquefaction potential. Soil stresses and deformation beneath the reactor would be the selected parameter.

The sensitivities studies will permit an assessment of the relative impact that the parameters of the different methodologies have on the total methodology chain. For both the current and the improved SMC's, we anticipate in getting the sensitivity of the PDF's from variations due to:

- Generated surface accelerograms (total duration, phase angle, duration of strong shaking).
- Site properties (moduli, damping).
- Site profile (layer thickness, location).
- Structural response properties (mass, stiffness, damping).
- Peak free-field ground acceleration
- Site conditions (soil versus rock).
- Location (eastern versus western U.S.).
- Structure type (BWR versus PWR: structures, subsystems, piping, and equipment).

In addition, for the improved SMC, we will get sensitivity of the PDF's due to:

- Significant parameters associated with an improved seismic-input methodology chain (i.e., allowing the spectral shape to be governed by the earthquake magnitude and location relative to the site rather than just peak g level).
- Restrictive assumptions in the SHAKE/LUSH analysis techniques.
- Elastic and inelastic structural-analysis techniques.

Figures 5-3 and 5-4 help to achieve the objective of assessing the conservatism in the current SMC. Figure 5-3 shows a typical PDF from the systems calculation with a superimposed allowable (A) stress or strain

Fig. 5-3. Typical PDF from system studies with allowable and nonseismic contribution.

Fig. 5-4. Typical PDF's from current and improved SMC system studies.

and an estimated envelope of nonseismic (non-E) stress or strain. The allowable stress or strain will reflect code value based on current regulations. The nonseismic stress or strain will be estimated based on reviews of PSA's and discussions with industry and the NRC staff. Both the allowable value and the nonseismic stress will be treated as a deterministic quantity. No attempt will be made to assess their possible variability or degree of conservatism. The probability of the likelihood of the nonseismic stress or strain actually occurring at the same time as the seismic stress (E) will be considered.

From Fig. 5-3, we can make some interesting observations. For example, we can get a quantitative assessment of the probability of a calculated seismic stress being greater than the mean stress, allowable stress, and a stress equivalent to the allowable less the nonseismic.

This is merely the area under the PDF curve and to the right of E_m , A, and A-non-E.

Furthermore, if we express the margin of safety (MS) of the critical selected parameter (characterizing the structural, subsystem, etc.) being studied as

$$MS = \frac{A}{E + \text{non } E} - 1,$$

it is possible to make some additional observations. First, the probability of E exceeding A would reflect the probability of having a margin of safety less than zero for the case where the nonseismic stress or strain contribution is zero or smaller. Second, the probability of E being greater than A-non E-values reflects the probability of having a margin of safety less than zero because case non-E is known or can be estimated.

When the above evaluation of the systems results is carried out for the improved SMC, it is possible to assess a lower bound for the conservatism. Even with the improved SMC, the allowable (A), the nonseismic stress (non-E), and load-combination formulae contain conservatism for which we have not accounted.

Figure 5-4 illustrates a comparison of the PDF's developed from system analysis of the current and improved SMC's. Such comparison will

Appendix A

Feedwater Line Material and Water Properties at Elevated Temperatures

FEEDWATER LINE MATERIAL PROPERTIES

As mentioned in Table 1 (Section 4), our typical feedwater pipe is made of SA-106, Class C steel. A complete stress-strain curve for this material at elevated temperatures was unavailable. Data that we were able to obtain on the behavior of this material at elevated temperatures are given in Table A-1. In addition, in 1971, personnel at the Lawrence Livermore Laboratory (LLL) performed room temperature tensile tests of SA-106, Class B steel.³² The stress-strain curve resulting from their test is given in Fig. A-1. Based on these properties, we have constructed an estimate of the stress-strain curve for SA-106, Class C steel at 300°F as shown in Fig. A-2. The ideal representation as used in the PTA calculations is also given.

Table A-1. Property variation of SA106 Class C steel with temperature.

	Temperature, °F					
	80	200	300	400	500	600
Elastic modulus (10^6 psi) ³⁰	29.9	29.5	29.0	28.3	27.4	26.7
Poisson's ratio	0.30	-	-	-	-	-
Yield strength (10^3 psi) ³⁰	40.	36.5	35.4	34.3	32.3	29.6
(Ref. 31)	42.0	-	38.0	-	34.0	-
Tensile strength (10^3 psi) ³¹	72.0	-	80.5	-	75.5	-
Elongation (%) ³¹	31.0	-	24.0	-	22.0	-

WATER PROPERTIES

A knowledge of the properties of water at elevated temperatures and pressures is necessary to provide proper input to the computer codes (described in Section 6). Extensive water property data were obtained,³³ and polynomial functions were fit to it. The data for density and sound velocity are shown in Fig. A-3. The data cover a temperature range from 50 to 500°F and a pressure range from 14.7 to 1500 psia. These properties, particularly density, are relatively insensitive to pressure. We are thus justified in neglecting pressure

- X First systems analysis computer run attempt.
 ○ Final report submitted to NRC.
 * These refer to tasks associated with packages described in Appendices.

Fig. 5-5. Schedule, manpower and computer requirements.

Manpower requirements for the project are estimated at 8.0 FTE's the first year; an average of 5.5 FTE's, the second; and 3.0 FTE's for the first 6 mo of the third year. Their effort would be directed at both the systems and package studies simultaneously.

Computer-time requirements are significant. We estimate approximately 300 hr of CDC-7600 time will be required to complete both the specific and generic system studies of the current SMC. An estimated 350 hr will be required for the systems studies on the improved SMC. We also estimate another 60 hr of computer time will be required for the package studies and the generation of PDF's for the systems analysis. Figure 5-5 indicates when during the calendar years the time will be expended.

Needed Interactions with NRC, Industry, AIF, and ASCE/ASME

To achieve the objectives of this project in an efficient and timely fashion, it is necessary to have considerable input from both the regulators and those performing the design analysis and writing industrial design codes. In the body of the proposed work plan, we have explicitly indicated some of the direct input required from either NRC or industry. The actual need for advice/criticism

goes far beyond what we have explicitly identified as we are proposing an approach to evaluate a controversial methodology. Since this evaluation cannot be made by appealing to carefully designed verification experiments, it must be based on a Bayesian approach. Such approaches are good provided that the input to the Bayesian model is good and comes from both industry and the regulators.

Two types of input are needed: straightforward and general. Examples of straightforward input are: (1) structural response models (masses, stiffness geometry) of various Category I structures for use in the current SMC, (2) detailed information on the nonseismic loads for inclusion in the inelastic analysis, and (3) approximate dynamic characteristics of all Category I structures, systems, components, etc. (to determine what structures must be considered, and what floor spectra must be calculated, etc.). Examples of general information are: (1) suggestion on the development of credible inelastic models including such items as modeling, identification of possible failure criteria, etc., (2) suggestions as to how applicants will meet the criteria on the enveloping of the free-field foundation spectra for the SSI analysis, (3) suggestions and comments regarding our proposed improved SMC, and (4) what cases must

be considered to draw generic conclusions.

We suggest that the program be carried out by the investigators in the following fashion. First, solicit comments and active participation through both formal and informal presentations to the AIF and to both the American Society of Mechanical Engineers (ASME) and American Society of Civil Engineers (ASCE) Subcommittees on seismic design. We have had considerable success with this approach from such meetings previously. We already have informal promises of cooperation to provide us the structural-analysis information needed.

In addition, we feel that it would be very helpful if each of the appropriate branches of the NRR would each assign a key person as a contact. Such "contacts" could help obtain the detailed information that will be needed and provide the branch's views of the analysis. This would allow for an easy and continuous flow of invaluable information and ensure that the improved SMC explicitly addresses the important concerns of the responsible Regulatory branches. Also, it would ensure that the current SMC is truly representative of what is, in fact, currently accepted.

Section 6

SEISMIC INPUT PACKAGE

INTRODUCTION

Most reactor sites are located in regions where the seismicity level is so low that the seismic activity cannot be correlated with observable tectonic structure. Thus, the earthquakes are assumed to be random. For these sites (Section 3), the major problems with the current procedure of assigning a design-response spectra in the free-field are:

- A single-spectrum shape is used, scaled on the basis of peak g value [Regulatory Guide (R.G.) 1.60].
- Acceleration is scaled from some correlation between intensity and peak acceleration.
- Generic site factors are not taken into account (i.e., except for an extremely "soft" site, no site corrections are applied to surface spectra).

Because the correlation between acceleration and intensity is so weak, there is no rational basis for choosing any particular value. This is then compounded by the fact that the spectra used are not related to earthquake-source parameters and local site conditions but rather are an "across-the-board" mixture of all

these factors.¹ Thus, it is not possible to assign any meaning to the statistics associated with spectral shape or earthquake probabilities for a given site. Currently, the seismicity statistics for a specific site do not consistently enter into the assignment of the SSE. Thus, typically, the design-response spectra used do not represent a uniform level of protection in the various regions of the country.

OBJECTIVE

The objective of this chapter is two-fold. First, a program is outlined that, if followed, will allow the generic development of regionally/site-dependent spectra to replace R.G. 1.60 spectra. These new design-response spectra are directly related to the seismic risk of the region within which a given site is located, and to possible ground motion from all earthquakes that could affect a given site (based on a given probability of "exceedance"), corrected for important generic site conditions. Thus, the spectra level and shape will differ in various regions of the nation (a site in Florida will have a much different spectral shape and level than one in New England). A

site on hard rock will have a different spectral level and shape than one on soil. The spectra will also have a correction factor for any prominent topographic features of a specific site.

The second objective of this section is to develop all the correlations required as input to the general systems analysis.

TANGIBLE PRODUCT

The new response spectra will represent a "uniform" level of protection relative to the seismic hazard and will ensure that designs are appropriate for given sites. The new approach will alleviate current difficulties arising between the NRC and applicants or intervenors because of arbitrarily fixed I_{max} and such weakly correlatable factors as intensity and acceleration.

In addition, this effort will help to develop a complete understanding of the factors governing both the shape of the response spectra and the spectral level. This will allow the development of alternate methods of scaling response spectra, which will result in a better (less uncertain) definition of possible design-response spectra.

APPROACH

Ground motion will be based primarily on available seismic data,

and appropriate statistical correlations will be made. However, source theories will be used to shape spectra, and other calculational approaches will be invoked to fill out our results. Three main factors will be considered:

- Seismicity of the region.
- Appropriate ranges of basic parameters.
- Generic correction for site factors.

To do this, our approach is broken up into tasks and detailed below. Briefly, Task A.1 defines appropriate regions that have a uniform seismic risk associated with them and evaluates the seismic statistics of these regions: i.e., earthquake-frequency distribution, attenuation characteristics, etc. Task A.2 defines the statistics of the ground motion as a function of the important variables used in Task A.1. Task A.3 assembles the results of Tasks A.1 and A.2 in a Monte Carlo scheme to develop generic design spectra for each region. Lastly, Task A.4 determines which site/source conditions require additional corrections factors to the generic spectra/time histories of Task A.2 and develops either generic correction factors or procedures that can be applied to particular classes of sites.

Task A.1: Definition of Regional Seismicity

To achieve the objectives of this task, it is necessary first to define regions with such uniform seismicity as uniform earthquake-frequency distribution, attenuation characteristics, and relations between magnitude and intensity. Because intensity is not directly related to the basic earthquake parameters, we suggest that magnitude be used as one of the variables. Magnitude estimates can be made by using a felt-area basis as developed by Nuttli and Zollweg² rather than a direct correlation between magnitude and intensity. This allows a reasonably correct assessment of smaller, shallow earthquakes that have a high intensity but are of small magnitude.

Much of the basic work has been carried out by Bollinger,³ Chinnery and Rogers,⁴ Nuttli,^{5,6} Nuttli and Zollweg,² Brazee,⁷ McGuire,⁸ and others now working at U.S.G.S. under the direction of Algermissen. The major efforts of this task will be to correlate these data with the definitions of the tectonic framework currently used by NRC. The regions defined will be based primarily on currently defined regions as outlined by past-licensing activities. These regions may be modified if such factors as the earthquake frequency and attenuation characteristics can

be shown to be significantly different for the modified regions.

These possible alternative regions can be developed from a literature review and from visual trends in the seismicity patterns as correlated with tectonic structures. It is not intended that these regions will serve as a definitive basis for a seismic zoning of the East but simply as a basis for developing generic spectra for large areas of the country. The proposed method can also be used to develop spectra for sites near the boundaries of complex regions, but on a site-by-site basis because the site location relative to the boundary is an important but nongeneric variable.

There are several different occurrence models that can be used to develop the earthquake-frequency distribution required.^{9,10} Typically used are the Poisson and Markov models, and the combined simple-magnitude-frequency relation of Gutenberg and Richter.¹¹ The implications of these will be explored relative to the final enveloped spectra obtained in Task A.3 to determine whether the statistical model used has any impact on design. If the design were based on a "likely occurrence" (i.e., with large probability of exceedance), then the effect of different models would most likely show up. However, since we are dealing primarily with the "tails" of the

distributions, then the choice of statistical model is less important.

The effect of limiting the maximum magnitude of earthquakes that can occur in a given region based on extreme-value methods¹⁰ should also be included. Other arbitrary limits, such as those currently used by NRC, should also be included.

The earthquake-frequency distribution for each region is based on the entire region, but it needs to be corrected. This distribution should allow for the fact that a much smaller number of earthquakes will actually occur close enough to any given site within the region to affect the design of the plant. This will be discussed below under Task A.3.

Task A.2: Spectra

In Task A.2, we outline a method that can be used to develop the appropriate response spectra-time history as a function of I , $I_{\max}$ location of earthquake relative to the site and magnitude. The main difficulties with using this for eastern sites are:

- The attenuation of intensity with distance is much less in the East.
- The high-frequency part of the response spectra attenuates much more strongly with distance than

the low-frequency (1 Hz and below) part.

- We will be mostly interested in near-field smaller earthquakes in the East.
- "Smaller"* source dimensions in the East make the attenuation of intensity and the high-frequency responses even more important.
- No data exist in the East other than intensity, attenuation of intensity and some small amounts of far-field seismometer data.

Because of the absence of eastern data and the large differences in attenuation, it is necessary to extrapolate western data for use as eastern data. To do this, we suggest the use of a dual approach to develop spectral level and shape. The primary approach will be to partition the data appropriately and run correlations on each partition, thus "choosing" spectra for a given earthquake from the appropriate set of data. This assumes that similar data for the East, properly partitioned, would statistically resemble that of the West. However,

*Smaller, for example, when compared to the Parkfield earthquake with over 30 km of fault at a magnitude of only about 5.4. No such earthquakes are known to have occurred in the East. Lack of surface faulting suggests that the source dimensions are "smaller."

because even the western-data set is much less complete than desired - particularly, for small near-field earthquakes - the second approach will be to develop reasonable correction factors for attenuation effects and thus transform western data into "eastern" data. The two approaches can then be compared to determine which is best. If they are very different, they will have to be re-evaluated to determine why.

The data base will consist of the uniformly processed records of Cal Tech, all additional records processed by U.S.G.S., and any available worldwide data.

In the first approach, the effect of scaling for distance must be replaced by comparing data correlated with intensity-magnitude in the West with that in the East. As discussed in Task A.1, the primary relation to the definition of the earthquakes will be magnitude. However, site intensity relative to given magnitude will also be used, by partitioning the data base into groups.

The first partitioning will be into groups $G_1(I_{\max})$, $G_2(I_2 = I_{\max} - 1)$, $G_3(I_3 = I_{\max} - 2)$, etc. Group G_1 , for example, will contain only those records associated with $I_{\max}$ of the earthquake. Group G_2 will contain only those records associated with an intensity less than $I_{\max}$, etc. While probably not subdividing any finer

than G_3 , group G_1 should be subdivided further based on epicentral distance, source fault dimension (where possible), and site conditions. Site conditions can be accounted for by use of the simple site-classification scheme of Trifunac and Brady.¹² As noted, the source dimension is important in that some smaller magnitude earthquakes in the West have large fault-rupture lengths that significantly affect the spectral shape. Such earthquakes apparently do not occur in the East. Appropriate regression analysis (form and choice of variables based partly on earthquake-source models) should be run on each group to develop seismic spectra for each group as a function of magnitude, $I_{\max}$, I_{local} , $A_{\max}$, and $V_{\max}$ for each spectral frequency. As noted, because of the strong influence of magnitude on both spectral shape and on the higher frequency-amplification factors of the spectra, we distinguish between two, time histories both with the same $I_{\max}$ but with different magnitude. Care should be taken to ensure that this partitioning is consistent with what is done in Task A.

To define the final response spectra, we suggest that two approaches be explored:

- Normalizing the spectra to peak values of acceleration and peak

velocity and using separate correlations on $A_{\max}$ and $A_{\max}$ versus $V_{\max}$.

- Simply allowing the variation of $A_{\max}$ to occur as part of the variation in spectral shape.

The two approaches lead to different results.¹³ Because the data base will be somewhat more extensive than that used by Trifunac,¹⁴ a regression analysis should be run on the entire data base. Our basic approach is different than that outlined by Trifunac.¹⁴

As discussed in Ref. 1, the attenuation of seismic energy is typically assumed to be a function of geometric spreading as well as material/nonlinear characteristics. Although there is no reason to believe that the geometric spreading is any different in the East than in the West, the other factors are different. The differences material/nonlinear attenuation between East and West should be more important in the high-frequency end of the spectra (also discussed in Ref. 1).

In determining if any significant differences in the data (if available) exist between the East and the West, one would attempt to account for differences in attenuation and thereby transform the western data into "eastern data." For the same partitions outlined above, a regression analysis

should be run to determine attenuation characteristics as a function of epicentral distance, magnitude, $I_{\max}$, I_{local} , $A_{\max}$, and $V_{\max}$ for each spectral frequency in the West. By using the approach and results suggested in Ref. 1 provisional corrections for attenuation are possible.

It is also possible to develop a provisional correction another way. For western data, it is known that the attenuation of $A_{\max}$ and $V_{\max}$ with R is approximately the same as the rate of attenuation of the high-frequency part of the spectra. For the near-field, there is no reason to assume that this result would be different in the East although the average rates of attenuation would be. With the data base of intensity vs site-acceleration data developed by Computer Science Corporation for NRC, the appropriate correlation of intensity can now be found in the West as a function of $A_{\max}$, $V_{\max}$, and the high-frequency part of the spectra. This correlation can then be combined with the correlation attenuation of I with R and with studies of attenuation of seismic waves.^{1,5,15} Combined, the attenuation of I with R and attenuation coefficients for seismic waves are used to determine an approximate relation for scaling (in the near-field) western spectra to the eastern attenuation.

By comparing the results of these two approaches, some reasonable averaging process can be used to get an "averaged" frequency-dependent correction factor that accounts for the differences in attenuation between the two regions.

A time history is required for use in the SSI calculation in the improved SMC. Time histories can be obtained by two basic means. The simplest is to choose randomly for each trial of the Monte Carlo system simulation one of the time histories from the appropriate partition. The main disadvantage of this approach is that some of the partitions (of most interest) will only have a few earthquakes in them. Thus, it would be preferable to use the second basic approach which is mainly developing a time history from the shape of the spectra. To do this, one can use the Fourier spectra rather than the relative-velocity spectra. While obtaining meaningful Fourier amplitude spectra is no problem, phase spectra are also needed for a time history. It is not known if this is a problem or not. Clearly, from the data one can develop phase spectra in the same fashion as amplitude spectra, but it is not clear that the resultant time history made up from the two spectra is acceptable. Although the problem of developing a time history to envelope a given spectra has received some study in

the literature, the phase-spectra problem seems to have received none. However, for these studies, no attempt was made to make the time history representative of a given restricted class of earthquakes. Thus, the role of the phase spectra in these studies was very arbitrary.

To determine if the time histories generated from both the Fourier amplitude and phase spectra are meaningful, we suggest the following approach. For each of the partitions,

- Visually compare generated records.
- Statistically compare the two sets of records (real, and generated) on the basis of energy, duration, times of the acceleration, number of cycles of strong motion and other tests.

If there is a significant difference, a randomly selected, different real-time history of the correct partition each trial of the Monte Carlo simulation should be used instead of an "artificial" time history.

We feel that the most important reason that the time histories generated must be "real" is their impact on liquefaction and SSI-type analysis. Soils are so nonlinear that both their strength and response are very sensitive to level of motion, duration of motion, and number of cycles of each level of strong motion. Thus,

the acceptability of generated time histories will be made primarily on this basis. However, any possible anomalous impact of the generated time-histories on the structural analysis should also be included. Typically, the structural response analysis is much less sensitive to many of these factors than the soil-type analysis.

Task A.3: Development of Design Spectra

The objective of this task is to couple Tasks A.1 and A.2 and develop regional, design response spectra based on all possible earthquakes that could affect a site within a region with a given probability of not being exceeded.

In its simplest form, the design-response spectra will be developed in the following way. The "output" of Task A.1 is regional earthquake-magnitude frequency-occurrence relations and attenuation relations. For each of the regions, the extreme value analysis will define the largest earthquake that can occur. Using this earthquake and the regional attenuation characteristics, one can determine the maximum area around the site that must be considered. Since the earthquakes are assumed to be random, it is possible to simply "ratio" the area of the region to the area around the site that must be considered, i.e.,

earthquakes occurring out of this area will not affect the site. The area will be divided into small, equal-area grids. Then, in the Monte Carlo scheme, the appropriate earthquake magnitude will be chosen as dictated by the earthquake-frequency distribution.* The earthquakes will be located at one of the grids randomly selected (other important effects can be obtained by weighting the choice of earthquake location). The earthquake magnitude, location relative to the site, and the regional attenuation curve will then determine the appropriate spectra (Task A.2) to use for representing the earthquake effects at the site.

This is repeated enough times to determine the ground-motion distribution at the site. Then, the design spectra are defined by contouring the data for given probabilities of not being exceeded. Note that the process is easily adapted to the other choices outlined in Task A.2; e.g., as discussed above in Task A.2, the spectra in each of the data partitions will be scaled by the peak g level of the record (and possibly other factors). Then, in the Monte Carlo scheme, we will include the step of going from site intensity to peak g level; also,

*At this stage, the effect of various occurrence models, mentioned above, can be tested.

various ways to scale the response spectra should be explored to determine which is the most appropriate for the design of nuclear power plants. The final choice of a "best" method must be determined by a complete systems analysis of the entire design-methodology chain.

The development of the computer program to carry out the analysis is straightforward, as is the process of carrying out the analysis. The difficult tasks are development of the distributions and relations between variables in Tasks A.1 and A.2.

Task A.4: Site Correction Factors

In Task A.2, simple site-correction factors were obtained via a simple site-classification scheme. The objectives of this task are to determine under what conditions additional site-correction factors are required and to develop the required generic correction factors. To determine whether additional site-correction factors are required, we suggest first that the data from those sites having several recorded earthquakes should be examined. In particular, the El Centro, Ferndale, and Hollister sites have each had a number of earthquakes,¹ and the data from these sites should be examined in an approach similar to that used by Trifunac and Idriss.¹⁶ However, their approach, which is primarily visual, is not

very sensitive nor does it allow for quantification. Thus the data at these sites should also be examined by using the approach suggested by McGuire.¹³ His approach is to perform regression analyses on the logarithm of the velocity spectra for various periods, using an equation of the form,

$$\log S = a + bM - c \log R + 25)$$

$$+ \sum_{i=1}^k d_i X_i,$$

where S is the spectral level for the natural period being studied; M and R are the magnitude and hypocentral distance of the record; and a, b, and c are least-squares parameters estimated by the regression. For the study of site correlation, k is the number of sites with more than one earthquake record in the data set; variables X_i are zero-one variables set to one if the observation is at the i^{th} site (where i is one of the multiple-record sites) and zero otherwise; and coefficients d_i are parameters estimated by the regression analysis. For the study of correlation of records from the same earthquake, k is the number of earthquakes with multiple records; variables X_i are zero-one variables set to one if the observation is of the i^{th} earthquake (where i is one of the multiple records earthquakes) and zero

otherwise. Parameters d_1 again are estimated by regression.

In effect, the coefficients d_1 calculated by the regression analysis give us an indication of how much knowledge is gained about the response spectrum of the motion (corrected for magnitude and distance effects) by knowing at what site (or during which earthquake) it was recorded. If the value of a particular d_1 is near zero, there is very little knowledge gained; if d_1 has a large positive or negative value, we would conclude qualitatively that the spectra of motion from that site (or during that earthquake) would lie above or below (respectively) the predicted spectrum for an earthquake of the same magnitude and distance, based on all records in the data set. The estimators of the coefficients d_1 are of course random variables since they are computed from random data; hence, the important parameter in concluding whether there is any real site or source-mechanism effect in the data is the t statistic, the ratio of the estimated d_1 to its standard error. Because we are primarily interested in the higher frequency part of the spectra and the velocity spectra is not very sensitive to high frequency, we will use acceleration spectra as well as the velocity spectra. The main differences between what McGuire did and what we

plan are in the sites considered, numbers of earthquakes at the sites, number of periods considered (we will consider more) and, finally, in our examination (comparison of) relative to several different partitions of the data. If McGuire's very preliminary results hold up, then the conclusion for the sites considered will be that the site effect is small.

Because only a few sites have several records, no general inference can be made. By generalizing, we suggest also that a regression analysis be developed for the spectra shape of several earthquakes including San Fernando, Parkfield, Lytle Creek, and San Francisco (1957) for which several-to-many records were obtained as a function of epicentral distance. Once the records that differ significantly from predicted values are determined, they should then define the sites for which additional site corrections must be developed. By examining these sites in detail, it should be possible to identify under what conditions the results from Task A.3 should be modified.

In addition to the above assessments, similar assessments for other sites and ground motion will be carried out as part of the site response package.

Of particular interest in these analyses is whether the "spikes" and

"valleys" typically observed in response spectra are primarily site-related or source/travel-path-related. If they are site-related, then it is necessary to adjust the generic-predicted spectra from Task A.2 for the local site factors so that the final deconvolved record is correct. The data from underground testing should be most useful. For these cases, the source is the same, and by proper-grouping the travel path should be the same. Thus, at the same site we would have cases with several different travel paths and also several cases with the same travel path. If, for example, all spectra are very similar, then this can be attributed to site factors. If they all are significantly different, then we can conclude that small changes in travel path is extremely important. If travel-path/source variations are significant enough, then considering the spikes and valleys of the spectra as random should be adequate. Thus, the spectra obtained from Task A.2 would not need further correction.

It is also possible that the data show that spectral spikes and valleys

are random, but the calculational scheme of deconvolution may over-correct for site conditions. This will also be checked for by using the mass of calculations carried out on the site response package. If this is the case, then an iterative loop would have to be built in to adjust the predicted time history for local site factors.

Lastly, the input spectra may need correcting because the site response calculation and SSI calculation do not correctly partition the seismic energy into the appropriate wave types. The analysis carried out in the site response package and in the traveling-wave section of the SSI package will indicate what, if any, corrections should be applied to the surface spectra to adjust for these effects. This aspect must carefully be coordinated with the SSI calculational approach to ensure that all corrections are compatible and that a double correction is not being applied. At the present time, it appears that it is possible to lump all of this correction into the traveling wave in the SSI calculation.

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Section 7

SITE RESPONSE METHODOLOGY PACKAGE

INTRODUCTION

One of the steps leading to specification of the seismic input for design of nuclear power plants involves modifying the SSE/OBE accelerogram for site effects. The SHAKE code is one method that is acceptable to NRC¹ to account for these effects. Despite the absence of absolute verification, SHAKE has been deemed acceptable because it represents the state-of-the-art in deconvolution techniques. However, we believe that three recent events require a reassessment:

- Additional site-response data from several sources have become available, and it appears to be straightforward to field experiments for additional data.
- The calculational state-of-the-art has advanced beyond SHAKE.
- Recent analyses indicate that the validity of the synthetic seismogram concept in relation to SHAKE is questionable.

On the following pages, we outline an approach to this assessment of SHAKE that would assist NRC in taking a regulatory position on site response, point the way to fruitful research, and provide input to the systems project described earlier.

SHAKE is the last in a series of site response codes developed by Prof. Seed and his coworkers at the University of California (Berkeley). SHAKE and its derivatives all solve the equations of motion for a horizontally layered viscoelastic half-space. The soil nonlinearities are accounted for by iterating to a consistent average strain (equivalent linear method), and the excitation is assumed to be vertically propagating SH waves. Each of these fundamental assumptions in SHAKE has been questioned by various investigators. For example, assuming the layers to be viscoelastic is to assume that all the damping is viscous; whereas, we know that soil damping is primarily hysteretic. Under some circumstances, this may be a reasonable assumption,² but for strong motion earthquake excitation, it is probably a very poor assumption.³ The basis of the equivalent linear method (ELM) assumption is a series of calculations between an early concept of SHAKE (based on the lumped-mass approach) and a program that employs a bilinear stress-strain law.⁴ Although the comparison was acceptable, we cannot extrapolate to the conclusion that the ELM sufficiently captures real-soil nonlinearities, because real soils are not

bilinear elastic. Additional evidence against the credibility of the ELM has recently been presented by Joyner and Chen⁵ and others.^{6,7} These comparisons were between the response of a wave, physically reasonable, soil model (Ramberg-Osgood; Davidenkov, Iwan) and the SHAKE ELM model. The response was in each case different and, more disturbing, sometimes indicated the SHAKE results were nonconservative.

Interesting results were recently obtained by Martin⁸ in a detailed comparison between SHAKE and two nonlinear calculative approaches. We feel that the comparison, which included six case studies, added to the uncertainty in SHAKE. The results showed that SHAKE was generally nonconservative, particularly in the higher frequencies.

If SHAKE were always conservative, the results could be of considerable engineering value, but the variability referenced above suggest that the results should be used cautiously. Unfortunately, the above comparisons were not sufficiently comprehensive to provide a clear indication of the specific problem area in the ELM or SHAKE. It's possible, for example, further investigation will show that a simple modification of the ELM would yield a superior and valuable site-analysis approach.

The fundamental assumption regarding the one-dimensional motion was clearly made for calculational convenience with the hopes that it would be conservative. This assumption was believed to be conservative because (in conjunction with Regulatory Guide 1.60) it is equivalent to the assumption that all the horizontal earthquake motion is in the SH mode, and SH waves were believed to be the most damaging. Recently, however, calculations⁹ and observations¹⁰ have suggested that the traveling wave adds at least one further important aspect to the response. This evidence indicates that the response of structures can be appreciably influenced by the phase differences in the excitation across the basemat. Whether the phase difference is due to diffraction or nonvertical incidence, the phenomenon is referred to as the traveling wave. The effects of the traveling wave cannot be captured in the current methodologies.

It is important to appreciate the consequences of the assumptions in SHAKE, but it is equally important to understand the relationship of the SHAKE method to the SMC. This has been covered extensively elsewhere in this proposal, but in this section, we will focus on one crucial aspect of this relationship: the synthetic seismogram. Current regulations provide sufficient guidance to specify

a free-field surface-response spectrum at a site. This spectrum, however, does not in itself uniquely specify the dynamic environment at the surface because the phase spectrum is not specified. Because SHAKE (and all other deterministic methodologies) require a unique specification of the seismic input and because no guidelines exist for specifying the phase, the actual seismic input to SHAKE can vary greatly, depending on how the phase spectrum is chosen. This will certainly result in a variability of the SHAKE output for the same input response spectrum. This potential variability has been graphically illustrated by Donovan and Valera.¹¹ They show that synthetic seismograms with the same response spectrum can result in structural responses differing by 100%.

OBJECTIVE AND TANGIBLE PRODUCT

The overall objective of this package is to determine the suitability of SHAKE for site response calculations. We technically assess its basic assumptions, as described above, and evaluate its role relative to the synthetic seismogram concept.

The product of the effort will be a quantitative assessment of the validity of SHAKE and then a determination of the sensitivity of SHAKE results to the various basic assumptions discussed above. The results

will be expressed as probability distribution functions (Fig. 7-1) and will, therefore, convey in summary form the overall variability of SHAKE results and the sensitivity of the SHAKE results to each of the assumptions. The results could provide possible interim "correction factors" for SHAKE for use in licensing and will point the way to future required research. Finally, and most important, many of the results are required for input to the SMC systems analysis.

APPROACH

The assessment of SHAKE can be conveniently divided into five topics:

- Comparison of SHAKE with data.
- Evaluation of the equivalent linear method (ELM).
- Evaluation of the credibility of vertically propagating SH waves.
- Evaluation of the sensitivity of results to the selection of a synthetic seismogram.
- Determination of the differences between *in situ* and laboratory soil properties.

Each of these contribute uniquely to the objective of this package in the manner described below.

Comparison of SHAKE with Data

The result of this topic is an overall evaluation of SHAKE expressed

Fig. 7-1. Various probability distributions to be used to evaluate the overall variability of the SHAKE program.

as a PDF (Fig. 7-1a). The general approach is to attempt to reproduce recorded data with SHAKE given an input motion and soil properties. Of the several sources of data suitable for an evaluation, the most promising sources seem to be:

- Some relatively unknown, Japanese borehole data and borehole data from nuclear explosions at the ERDA's Nevada Test Site.
- Abundant surface site-response data recorded at and around NTS.
- Earthquake data at Parkfield, in conjunction with the newly acquired soil properties.

Japanese Data

The Japanese data were reported by Kanai¹² and represents, to our knowledge, the only borehole earthquake

data available. The data have tremendous potential as there is information for four boreholes with several earthquake records for each (8, 5, 5, and 2, respectively). Although soil properties are lacking, one should be able to obtain good estimates from the known geology, or through the assistance of the Japanese Earthquake Research Institute. The approach would be to input the soil properties and a borehole accelerogram and to calculate a surface accelerogram. We suggest the use of several methods to evaluate the results. First, if this calculated response spectrum were normalized to the observed response spectrum at each frequency and this process were repeated for other earthquakes recorded in the same borehole, one could superpose all the normalized surface response spectra (Fig. 7-2a).

Fig. 7-2. Proposed methodology to determine the required probability distribution.

We would then be able to perform such statistics on these spectra as calculating the PDF at each frequency (Fig. 7-2), thus yielding the mean and variability of SHAKE for that borehole as a function of frequency. A second method is to use a standard significance test to determine if the surface motion calculated via SHAKE is significantly different (level and frequency content) than actually observed. Interesting statistics may possibly be obtained also by performing the above superposition for all the boreholes together, and this will be explored.

The borehole data from NTS, while from a different source, would be operated upon by the same process. We would carefully preselect the NTS data to guarantee that the borehole was outside the spall zone (a zone

defined by tensile failure of the soil). We have shown elsewhere¹³ that for this situation the ground motion from a nuclear explosion is much like an earthquake.

The above results will be exceedingly valuable in a preliminary assessment of SHAKE. A flaw in the approach does exist if the data were not acquired specifically for nuclear power plant site-response studies, and the motion was either of a very low-level (Japanese) or a moderately high-level (NTS). We feel that this approach of comparing data to SHAKE calculations has such merit that additional data should be obtained. We therefore propose to add on a series of experiments at NTS, designed such that the peak g acceleration would be $\approx 0.3 g$ and that the experimental sites would be representative of

typical eastern sites. These data would supplement the existing data to broaden the statistical base of the conclusions, and would provide the opportunity to study other related phenomena such as the effects of topography on surface motion. Finally, the experiments would be inexpensive to conduct. At this time we simply suggest the compatibility of an experiment as this with a SHAKE assessment. We will supply a detailed proposal on this subject on request.

NTS Data. There is also a great deal of data readily available from surface-motion measurements at and around NTS.¹⁴ We would use these data in two ways: first, qualitatively to assess the significance of the site response relative to the source and travel path effects; and second, to develop additional SHAKE - data comparisons.

In assessing the significance of the site response, the records from a variety of sources would be statistically analyzed for fundamental site periods. We would expect to see the best possible reproducibility of site periods in this search because the excitation will be low-level and the nonlinear frequency shifts will, therefore, be small. The results of this analysis coupled with Task A (Section 6, Seismic Input Package) would either substantiate or refute Trifunac's conclusion^{15,16} that the

motion at a site is dominated by source effects. Therefore, it is pointless to attempt to calculate site response with such precision. We suggest developing additional SHAKE - data comparisons at these sites by deriving the bedrock motion for each event at each site by using nearby rock motions and state-of-the-art attenuation laws. This bedrock motion and soil properties would provide the input for SHAKE to calculate the surface motion. The results would be displayed as PDF's as in Fig. 7-2. We feel that these comparisons are very important in that the basis for SHAKE was originally established from very similar calculations.⁴

California Data

Finally, we feel that the same techniques as described above should be used for earthquake data from Parkfield, California. There are substantial earthquake data recorded at this site, and the soil properties have recently been determined.¹⁷ These comparisons would further add to SHAKE statistics.

The result of the effort in this topic (Comparison of SHAKE with Data) would be extensive statistics on the ratio of the SHAKE-calculated spectral acceleration to the data spectral acceleration as well as significance tests on the two sets of data, i.e., recorded vs calculated. In addition,

we would gain significant insight into the relative roles of source, travel-path, and site response. Of the several possible conclusions that can result, one would be that SHAKE is adequate in all cases. Considering the range of data, this would strongly support the overall adequacies of SHAKE. If SHAKE were found inadequate in all cases, we then would make use of the results from the other tasks in this package to determine if the inadequacy is caused by a combination of shortcomings of SHAKE or primarily because source and travel-path effects dominate over site effects. A third possible conclusion is some cases are adequate and some are not. To make this assessment, both the information developed in Task A of the Seismic Input Package and the other tasks of this package will be used.

Evaluation of the Equivalent Linear Method

The purpose of this topic is to determine the variability of the SHAKE results due only to this one assumption. This approach capitalizes on the availability of fully nonlinear, soil-response codes by comparing SHAKE results to nonlinear results for the same input. There are several nonlinear codes available for this comparison, such as, Streeter,¹⁸ Joyner⁵ and Seed.⁸ We suggest evaluation of these candidate codes to

determine which is most suitable for a SHAKE comparison. The parameters to be exercised in the SHAKE comparison will include:

- Peak ground acceleration.
- Response-spectrum shape.
- Sice thickness.
- Site stiffness.

The results will be expressed in a PDF plot as shown in Fig. 7-2b and will represent the sensitivity of the SHAKE results to the ELM. These results also can be used qualitatively to determine if the comparisons between SHAKE calculations and recorded data can be improved by making a nonlinear analysis. Without added soil-test data, we feel that little can be gained by an extensive after-the-fact calculational effort. The actual form of any nonlinear-constitutive equations used will be determined in the soil-structure-interaction package. However, the finally selected form of site-deconvolution calculations will be compatible with the requirements of the SSI calculational scheme.

Evaluation of the Credibility of Vertically Propagating SH Waves

The product of this topic is an assessment of an underlying assumption in SHAKE: it is conservative to consider all the horizontal earthquake

motion in the SH mode. We suggest approaching this assessment through the use of several analytical dislocation solutions.^{19,20} These solutions are for the class of problems depicted in Fig. 7-3a, where the line dislocation represents a simplified earthquake source and the solution yields time-history response to the dislocation of desired locations. The response is, of course, a superposition of all the modes of the half-space, including surface waves. In spite of their simplicity, these solutions are interesting because they provide the opportunity to address the validity of vertically propagating shear waves without introducing the effects of other assumptions such as the ELM. The procedure would be

Fig. 7-3. Proposed models for the calculation of surface and sub-surface ground motion to be used to evaluate SHAKE.

to input the half-space properties and one component of the horizontal motion at some depth into SHAKE and compare the SHAKE calculated motion in the same component at the surface with the exact half-space solution. This would be done for a variety of half-space properties, for different site ranges, and for dislocations of different complexity in order to define the sensitivity of the comparison over a range of situations.

We also suggest approaching the solution for thrust and normal faults numerically with one of our large two-dimensional finite-difference wave-propagation codes. The situation for this approach is shown in Fig. 7-3b. The numerical approach, while more complex and expensive, would allow us to solve for the response of more complex dislocation mechanisms and more complex geologies.

The result for either approach, however, would be the same: a PDF representing the variability of the SHAKE-calculated spectral accelerations with respect to spectral accelerations calculated from the half-space solution.

Evaluation of the Sensitivity of Results to the Selection of the Synthetic Seismogram

In this topic, we address the important question of whether different synthetic seismograms, all with the same response spectrum, can produce

different SHAKE results. The evidence available indicates that they probably do,¹¹ but the extent of the variability has not been quantified.

The approach would be to generate an ensemble of synthetic seismograms, all of which have the R.G. 1.60 spectrum, and to calculate the bedrock response spectrum at a typical site for each of them. To extend the validity of our conclusions, the process would be repeated for several sites of different thicknesses and stiffnesses. The statistical properties of the bedrock spectral accelerations for each site would provide clear evidence of the sensitivity of results to the synthetic seismogram as a function of site type. This will provide excellent guidance to OSD in formulating new site-response policies.

Determination of the Differences Between In Situ and Laboratory Soil Properties

This is a somewhat separate but very important topic. Interest in this topic develops from the fact

that all site response and soil-structure-interaction calculations require soil-property data, and most frequently unmodified laboratory data is used. There are several reasons why laboratory data might not accurately reflect the *in situ* soil properties. The most significant is that the samples are disturbed when removed and transported to the laboratory. Another aspect is the size effect; it is possible that a 1-in. circular core might respond quite differently than, say, a 1-ft or 1-yd sample. If this is the case, it is important to assess these differences.

We suggest approaching the topic principally by reviewing the literature for all available data comparing *in situ* with laboratory soil properties. We also suggest consulting with various soil firms to obtain additional unpublished data. These data will be analyzed by correlation techniques to determine the significant parameters. The data will then be presented as a function of the significant parameter(s).

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Section 8

SOIL-STRUCTURE INTERACTION (SSI) METHODOLOGY PACKAGE

TASK C.1. TWO- VS THREE-DIMENSIONAL RESPONSE

The present state-of-the-art of soil-structure interaction (SSI) analysis may be roughly divided into three methods: plane strain, axisymmetric, and quasi-three-dimensional. The L'SH or FLUSH computer programs can provide capabilities in the first two cases and the FLUSH code in the last. None of the above three methods used in practice provides a true three-dimensional capability because of an economic reason rather than safety.

There are sound reasons to expect the effect of spatial mathematical model dimension to be physically significant in design. For example, this dimension has a significant effect on the way energy is radiated away from the foundation of the structure; that is, the way interaction occurs. In one dimension these radiated waves do not decay, in two dimensions they decay as $1/\sqrt{r}$, and in three dimensions they decay as $1/r$, where r is the distance from the foundation. This means that, in general, two-dimensional models will exhibit more interaction than three-dimensional ones, but this does not

mean two-dimensional models are therefore conservative. Three reasons for this conclusion are the uncertainties in phasing and structure-to-structure interaction, and the higher material damping in the two-dimensional case.

A second example concerns the site cross sections used in the present two-dimensional methods. Typically, "N-S" and "E-W" cross sections are used, but there is no basis to believe these cross sections are more conservative than "NE-SW" and "NW-SE," or any other. The present two-dimensional methods can only inadequately address this concern.

A third example concerns the interactions between the various Category I and other structures in a typical power-block unit layout: that is, structure-to-structure interaction where the present two-dimensional methods are completely inadequate.

Fourth, the present methods cannot provide a rational torsional input for structural design purposes.

The final example includes the foundation design which is more difficult than determining base reaction response because stresses are the prime quantities of interest. In the analogous problem in structural design, detailed

structural models are used, rather than the coarse SSI model. Coupling this with the primitive state-of-the-art of soil properties creates uncertainties on the state-of-the-art of foundation design. For example, compressional effects can be significant in the region local to the basemat even when only the horizontal component of earthquake motion is considered and the adequacy of the vertical propagating shear-wave is assumed. As discussed above, in general, the differences in the way that this compressional energy is radiated from the basemat in the two- and three-dimensional cases may be significant, and there are uncertainties whether the present two-dimensional method is conservative. Again, basemat flexibility may be relatively insignificant for structural purposes, but at the same time it can be critical for foundation design. In general, two-dimensional basemat flexibility modeling can only be considered approximate. These concerns are reinforced considering the uncertainties in the present two-dimensional method of combining three components of earthquakes. All these concerns can result in questions about the adequacy of foundation design in the areas of liquefaction or other soil-cyclic strength or performance evaluations considering two-dimensional and three-dimensional differences.

The concerns expressed above would be quantified by comparisons of the two- vs three-dimensional response of a SSI system.

Our general approach is to (1) identify and evaluate candidate analytical and numerical methods, (2) develop a basis for deciding whether solution methods will be linear or nonlinear, and (3) use the approach selected to evaluate two- and three-dimensional response of SSI system. The results would be used for deciding the approach used in improved SMC.

Objectives

- Preliminary evaluation of present two-dimensional SMC.
- Selection of method appropriate for improved SMC.
- Early identification of additional unanticipated work for improved SMC.

Tangible Product

- Early formal report on the direction that an improved systems model is taking.
- Updated projected costs and schedule.
- Early input for NRC Standards decisions.

Approach

In this section, the general approach described above will be

described in more detail. Because of the second objective listed above, careful consideration must be given to the solution techniques adopted in the area of true, overall efficiency. This is necessary not only to keep computer costs as low as possible but also because, ultimately, inefficient solution techniques lower the applied technological level otherwise attainable and thus dilute or frustrate major objectives.

Task C.1.1 of the proposed effort is to identify and evaluate candidate solution methods that appear to be more analytical than numerical. An example of one such method is given in Wong¹ or Wong and Trifunac.²

Task C.1.2 is to identify and evaluate candidate numerical-solution methods. Examples of methods that will be examined are LUSH, LUSH extended to 3-D, FLUSH, SAPIV, and explicit finite element/finite difference methods such as found in HONDO and LLL codes, as well as advanced alternating implicit/explicit methods such as those used in hopscotch codes.

Task C.1.3 is to develop a basis for deciding whether linear or nonlinear methods are required in this effort. The decision has been made that nonlinear methods would be used in the improved SMC.

Task C.1.4 is to use the method selected to obtain two-dimensional

SSI response. Results to be evaluated are floor-response spectra and basemat and foundation response. The effects of a flexible basemat would be included. If linear analytical methods are used, the input may be sinusoidal. If nonlinear methods are used, the input would be realistic time-histories similar to those used in the system's conservatism evaluation.

Task C.1.5 is to use the method selected and obtain three-dimensional SSI response. The same solution methods and inputs as in Task C.1.4 would be used.

Task C.1.6 is to perform analyses similar to those in Task C.1.5, except that "NE-SW" and "NW-SE" input rather than "N-S" and "E-W" would be used.

Task C.1.7 is to compare the two- and three-dimensional SSI responses. The areas of primary interest are the floor-response spectra, basemat, foundation, and effect of torsion.

Task C.1.8 is to select the method(s) appropriate for the improved SMC. Because of the increased difficulty of the foundation analysis, more than one method may be selected - one method for the basemat response and a more sophisticated one for the foundation response.

TASK C.2. TRAVELING-WAVE EFFECTS IN SSI

The present state-of-the-art (SHAKE/LUSH/FLUSH) of seismic-design

methods for nuclear power plants have, as a fundamental assumption, the vertically propagating shear-wave theory. While the effects of vertically propagating shear waves may be dominant in many cases, the inclusion of other modes of propagating seismic energy has not been investigated sufficiently to determine if they are of engineering significance.

One could perform analytical SSI analyses with and without the vertically propagating shear-wave assumption. The results of these analyses could be compared and the engineering significance of the vertically propagating shear-wave assumption evaluated.

Our proposed approach would be to

- (1) identify and evaluate candidate numerical methods and approaches,
- (2) develop a basis for deciding whether solution methods should be linear or nonlinear and
- (3) use the approach selected to evaluate response of SSI system with and without traveling wave. The traveling wave to be used herein would step the earthquake time-history in time across the base-rock according to the rock-wave speed. (As used here, the traveling-wave and vertically propagating-wave theories are assumed to be mutually exclusive.)

One could statistically review existing strong-motion data and use these results to decide on approach to be used in improved SMC.

Objectives

- Preliminary evaluation of the present SMC.
- Identification of methods important for an improved SMC.
- Early identification of additional unanticipated work for an improved SMC.

Tangible Product

- Early formal report on the direction that an improved SMC is taking.
- Updated projected project costs and schedule.
- Early input for NRC Standards decisions.

Approach

In this section the general approach described above will be described in more detail. Because of the second objective above, the results of this effort could significantly affect the conclusions of the improved SMC.

Task C.2.1 is to identify and evaluate candidate solution methods. It is believed that the evaluation proposed herein can be performed by using two-dimensional mathematical models. This is not meant to imply that two-dimensional methods are adequate for design; that question is addressed elsewhere in this proposal. What is meant is that if the effect

of traveling waves is significant, it should be apparent from two-dimensional results. Task C.2.1 also includes a decision as to whether to use linear or nonlinear methods. A linear frequency domain program (TRAVEL) does exist and is a modification of LUSH. Since the solution techniques used in these programs are effectively implicit, they propagate energy at infinite speed unless excessively small time steps and/or a large number of "modes" are used; that is, they essentially do not propagate energy at the correct speed. It is doubtful whether any significant differences would be seen from a LUSH or TRAVEL approach unless very small time steps were used (see Marcuson and Krinitzky³). Appropriate computer programs do exist, such as HONDO and LLL codes. These codes are explicit and can propagate the energy properly when it is necessary to do so as in the case under consideration here. The use of a linear method would preclude shock formation during the analysis and, therefore, the decision would probably be to use nonlinear methods.

Task C.2.2 is to perform identical analyses with and without traveling waves. Responses parameters to be evaluated are floor-response spectra and basemat and foundation response.

Task C.2.3 is a decision on the methods to be used, traveling wave or not, in the improved SMC.

It should be clearly understood that the effort proposed above is not addressed toward identifying propagation mechanisms, but rather it is an engineering assessment of the significance of one such mechanism. The conclusions to be drawn from such a study must also be clearly understood. First, if the effects of traveling waves are found to be significant for seismic design, as a result of this study, then the legitimate conclusion is that they may be significant in design. Second, if these effects are not found to be significant, then their effect in design is unknown. This uncertainty is due to two sources: the evaluation is two-dimensional, but more important the true nature of propagation mechanisms is unknown and thus its significance in design is unknown. Third, if these effects are found to be significant, the degree of their significance is unknown because of the unknowns in propagation mechanisms.

Task C.2.4 is to review statistically the strong-motion data by comparing data obtained in large buildings with those obtained in small instrument buildings. The comparisons made in Task C.2.3 would

indicate the type of effects that should be observable in these data. This would provide an additional basis for comparing the simplified calculational model with actual data to assure that the calculational approach is adequate.

TASK C.3: EVALUATION OF IMPROVED SSI APPROACH USING LARGE-SCALE EVENTS

The present state-of-the-art field evaluation of SSI predictions is that it is largely absent. Certain events have been evaluated — such as the June 7, 1975, earthquake at the Humboldt Nuclear Power Plant and the effects of the Project Rulison detonation on Vega, Harvey Gap, and Rifle Gap dams — but the results are unconvincing if they are interpreted to imply the adequacy of the present seismic design bases for nuclear power plants.

The NRC has adopted a definitive design method for the design of nuclear power plants for earthquakes in the absence of a demonstration of the adequacy of this design basis. The conservatism in the methods adopted have not been quantified by the comparison of design results with the actual performance in the field during large-scale events.

The adequacy of the present methods and the methods proposed in the improved SMC should be comprehensively evaluated by comparison

with the data obtained from large-scale field events. This evaluation would establish the credibility of the improved SMC described elsewhere in this proposal.

General Approach

One-dimensional evaluations of the adequacy of present and proposed methods and soil properties are described elsewhere in this proposal. The objective of the tasks described herein is to perform this evaluation by using two- and three-dimensional methods. To do this, past and future tests at the Nevada Test Site (NTS) should be evaluated as well as real earthquake events such as the 1975 earthquake at the Humboldt Nuclear Power Plant. These evaluations would provide significant benchmarks for the proposed improved SMC and soil properties.

Objectives

- Preliminary evaluation of the conservatisms in the present SSI SMC.
- Evaluation of the adequacy of the methods used in the improved SMC.
- Comparison of analytical predictions with data from real events.
- Identification of additional work required for improved SMC.

Tangible Product

- Formal reports on evaluation of large-scale field events;
- Updated projected project costs and schedule.
- Early input for NRC Standards decisions.

Detailed Approach

The general approach described above is presented here in more detail. Because of the second objective listed above, careful consideration must be given to whether the methods and soil properties adopted reflect reality, and/or the limits (e.g., frequency range) over which realistic predictions may be obtained. Careful consideration must also be given to the mathematical models and solution techniques used to perform this evaluation, as well as the soil-constitutive models adopted. These tasks are described elsewhere in this proposal.

Task C.3.1 of the proposed effort is to identify candidate large-scale events* that may be appropriate for

*Future tests at NTS. The concept here is to use LLL knowledge of future tests at NTS and to "piggy-back" SSI tests on time. This would include placing large-scale but not necessarily full-size buildings at the NTS. Tests of this type were recently proposed by General Electric/LLL to EPRI. These tests are not proposed to be carried out under the work suggested herein but will be the subject of a future separate proposal.

further evaluation. Events already identified are:

- 1976 earthquake at Humboldt Nuclear Power Plant.
- Project Gasbuggy (December 10, 1967): Navajo Dam.
- Project Rulison (September 10, 1969): Vega, Harvey Gap, and Rifle Gap Dams.
- Events at the Hollywood Storage Building.
- Events in Japan and elsewhere in the world.

Task C.3.2 is to evaluate the candidate events and select events appropriate for detailed analytical evaluation. The idea here is to provide some sense of a technological benefit/cost ratio in order to select the most appropriate events. Some of the factors that will be considered are whether two- or three-dimensional mathematical modeling would be required, whether additional field and/or laboratory testing is required, the adequacy of the existing acceleration data, the appropriateness of the site type for SSI purposes, any other significant costs and the potential payoff. Because of the significance of the event, the 1975 Humboldt Power Plant earthquake should definitely be evaluated.

Task C.3.3 is to provide a detailed comprehensive evaluation of the

events selected in Task C.3.2. The present methodology (SHAKE/LUSH/FLUSH) as well as the methods proposed for the conservatism evaluation will be used. This evaluation will emphasize comparing both the analytical results with available results from the recorded event. This is the critical point in the entire conservatism evaluation portion of this proposal. These comparisons would establish the credibility of the conservatism figures. If the agreement between the predicted results from the analyses and the results from the real large-scale event is inadequate, parametric studies should be performed to identify potential sources of error. These parametric studies will be strictly identified as such. These potential sources for the error will be evaluated in the light of the latest results in other portions of the project such as the significance of source mechanisms, soil-constitutive properties, etc. Also, a rational basis will be attempted to justify the variations required to provide good agreement. These results would be used to suggest future large-scale tests specifically designed to test the correctness of the potential error sources identified and the modifications to parameters not otherwise justified.

TASK C.4. SOIL PROPERTIES/FOUNDATION DESIGN

Present State-of-the Art

This short review concentrates on two aspects of soil properties: their dynamic behavior during strong-motion excitation when the soil does not fail, and the failure or performance of soil. In addition, the development of a philosophy of nuclear power plant foundation design is discussed.

Elastic Moduli and Damping

Strain-Controlled-Laboratory Triaxial Tests - Secant-shear modulus and hysteretic damping is obtained as a function of strain. Isotropy is assumed, and there is little basis for the selection of the bulk modulus.

Simple-Shear Laboratory Tests - Generally, the tests are not used except in research. The equipment is not common and more care is required.

Torsional-Shear Laboratory Tests - Some of this testing is widely used for low strain properties. The torsus-like equipment is used much as simple shear. Since torsus equipment is not widely available, more care is required, and tests are more applicable to remolded samples.

In Situ Tests - The current testing of this type obtains low-strain propagation speeds/moduli. Recently, new *in situ* tests have been developed,⁴

but at this time, a complete evaluation apparently is not available. Real events provide the most credible *in situ* testing, but events of this type have yet to receive an appropriate evaluation.

Strength and Performance

Stress-Controlled-Laboratory

Triaxial Tests = Shear-strength reduction and/or residual deformation is obtained as a function of cyclic-shear-stress and the number of cycles. Pore-water pressure buildup is also obtained, thus tests are more common for cohesionless soils (liquefaction, cyclic mobility).

Simple-Shear Laboratory Tests =

Generally, the tests are not used except in research. The equipment is not common and more care is required.

Torsional-Shear Laboratory Tests =

Various types of torus-like tests are performed but, again, usually for research. Tests are more applicable to remolded samples.

In Situ Tests =

Some blasting is performed but is not widely used. Real events like Niigata provide some results, but generalizations to design events are questionable.

Foundation Design = Static and dynamic analyses are performed (SAPIV/ISBUILD/LUSH/FLUSH) by using static and dynamic moduli. Strength and performance evaluations are performed by using results from these analyses.

Typically, dynamic stresses are used to obtain local "factors of safety" based on cyclic strengths. Any effects soil-strength reductions or residual deformations have on the properties used in dynamic analyses are not used to perform the dynamic analyses.

Weaknesses of the Present Approach

Soil Properties: General =

Little or no modern continuum mechanics are applied. Little correlation exists with actual strong-motion field performance. The correlations made are useful, but extrapolation to other sites and events is questionable because the correlations are based on unproved analytical methods. Therefore, there is little confidence in laboratory-soil properties.

Soil Properties: Cyclic-Triaxial

Laboratory Tests = The imposed stress or strain state is not representative of field states. The combination of three earthquake components is generally ignored except for liquefaction shaker-table tests. In the latter, the conclusions are questionable because they are not based on the varying ways that field-stress components may combine. Primary emphasis is on shear modulus because of the vertically propagating shear-wave assumption. Although this energy-transfer mode may be dominant, this does not mean the compressional

modulus is unimportant in design. No testing is performed to obtain this modulus, and assumed values are often used.

There is much discussion about equivalent average-stress history for liquefaction but little analogous discussion on a similar topic for moduli. Isotropy is assumed, but there is no evidence that soil in the field behaves isotropically. Considering the requirement of three components and the strain-dependent nature of soil moduli, it is easily seen that soil in the field behaves somewhat anisotropically. There is concern about the effect of pore-pressure buildup on liquefaction strength or strain, but little analogous discussion on the effect of pore-pressure buildup on moduli. In liquefaction and soil moduli in saturated soils, pore pressure is a physically significant state variable that is treated only in an ad hoc way.

Soil Properties: Simple-Shear Tests — The weaknesses of the stress or strain state imposed on the soil sample in triaxial testing have been recognized and criticized, resulting in various simple-shear and other shear tests. These shear tests are an attempt to replicate field conditions. Many triaxial tests, especially liquefaction, have been benchmarked against simple-shear tests.

Nevertheless, the question of actual field performance remains.

Foundation Design — The uncertainties in the present approach abound, and a general comment should help set the stage. One of the fundamental problems in this area is philosophical. That is, because the SSE is or is supposed to be the largest credible event at the site, credit is taken in an implicit way in the design methods for the low probability of it ever occurring. The point that seems to be missed is that once a design-basis event has been adopted, its credibility is not subject to question. What it is, is precisely what it says: it is an event for which the adequacy of a design must be demonstrated. This single point has a significant effect on the level of technology judged adequate for design methods. For example, another way of defining a design-basis event such as an SSE is that this event is larger than one which can reasonably be expected to occur during the life of the plant. This basic-design objective has tremendous consequences. What it means, among other things, is that the present civil engineering design methods for Category I foundations are, by and large, bereft of the traditional historical final check on their adequacy; that is, failures. This is a major way that

the present design methods have improved over their predecessors as the recent dam failure in Idaho indicates. Another way is through testing. Both of these methods are, in general, not available to nuclear power plant design. The implication is then, that something radically different in design method is required, and it does not appear that it is being used. In such cases, conservative designs may be an alternative. In present practice, the differences in the allowable "factors of safety" do not seem to be an adequate assessment of the true uncertainties comparing steel and foundation design. A great deal of the required technology does exist and the remainder could be developed, but the decision has just not been made that it is required. The present state-of-the-art simply has not progressed beyond the art stage. Expert opinions are still required and the experts basically agree on only one thing, they cannot write down how to evaluate the adequacy of a given design so that someone else can perform a similar evaluation and reach the same conclusion.

Foundation stresses are a major objective in foundation design. This is a more difficult problem than structural response, yet detailed analyses are performed on the structures but not on the foundation. The use of soil properties in dynamic analyses

is done in a curious way -- variations of soil properties analyses are performed for structural response but not for foundation stresses. Soil stresses computed by using R. 1.60 time histories are questionable as to their significance and meaning. In other cases, soil stresses are computed by using realistic time histories. The use of more than one such time-history is uncommon in SSI analyses because of the cost, which means the conservatism in R.G. 1.60 may not be present.

SSE plus aftershocks called for in 10 CFR 100 Appendix A are generally ignored, as is postearthquake pore-pressure migration. Both of these could have a significant effect on liquefaction evaluations.

Present cyclic-strength reductions and factors of safety are based on local "factors of safety." These local safety factors may or may not have anything to do with foundation failure, depending on the propagation of failure and/or the foundation performance and its effect on the structures. This is analogous to the use of elastic design in ductile-steel structures. Successful development of static slope-failure-design rules in soil engineering, simple plastic design in the design of steel structures, and the work in pressure vessels⁵ suggest that designs are better based on total or overall

strength or overall performance rather than on localized extreme values.

Comment on Plans - Some confidence must be obtained that the elastic-soil properties used in design provide even an approximate guide to the behavior of soil in the field. This is the objective of the large-scale test effort described elsewhere in this report. In this task, the development of appropriate soil-constitutive models is the main topic of interest in this area. These models would be evaluated and redefined if necessary and possible by using these large-scale tests. This evaluation will determine the soil models to be used in the improved SMC. In addition to this, work is also required to evaluate foundation failure and performance. The soil-constitutive models will be of some importance, but since failure is more difficult to predict than response, the use of existing large-scale events at which failure occurred is not proposed at this time. Rather, potential candidate events will be identified for a future proposal. New methods are proposed for the foundation evaluation in the improved SMC, and a foundation design philosophy is suggested.

Objectives

- Identification of candidate soil-constitutive models.

- Development of improved foundation-evaluation methods.
- Development of foundation-design philosophy.
- Early identification of additional work for improved SMC.

Tangible Product

- Formal report on direction improved SMC is going.
- Updated projected project costs and schedule.
- Early input for NRC Standards decisions.

Approach

Task C.4.1 is to identify candidate soil-constitutive models, including new ones based on modern continuum mechanics. Models already identified are Ramberg-Osgood hysteretic, which is an advancement over the present secant-modulus approach; and models based on Biot's pioneering work on continuum mixtures where the soil-pore-water pressure is modeled directly. As discussed above, in saturated soils the pore-water pressure is a physically important state variable which is not modeled as such in the present soil-constitutive models. Recent advances have been made on Biot's basic idea in the directions required for nuclear power plant applications.

Task C.4.2 is to evaluate the candidate soil models identified in Task C.4.1. Among the factors to be considered in this evaluation are the cost implications in solution techniques. For example, in the Ramberg-Osgood model, the cost of determining the material property value to be used in the next time step is relatively high. In the present method, this cost is zero once the iteration is complete because a linear solution is used. As another example, models based on Biot's idea would have an additional cost for a similar reason as in the case of Ramberg-Osgood and also because an additional variable has been introduced into the solution. Another factor is the physical reasonableness of the scheme for obtaining the material properties. For example, the intricate schemes used to obtain material properties from the present Ramberg-Osgood model raise doubts as to the quality of the model. In these schemes the soil must "remember" its entire strong-motion loading history. Physically, one would think the soil would "forget" after some time. Another factor is the practicality of obtaining adequate laboratory tests results. For example, tests have been performed comparing the present liquefaction model and a model based on Biot's idea. These tests, as well as any other similar

ones available at the time this effort is performed, should be evaluated. New laboratory tests are not proposed herein; if such a recommendation is judged meaningful, it will be the subject of a future proposal.

Task C.4.3 is to identify candidate field failures for future analytical evaluation. As discussed above, the evaluation of field failures is not proposed at this time as it is (see Task C.3) in the case of field response. The evaluation of field failures will be the subject of a future proposal.

Task C.4.4 is to develop analytical methods for the evaluation of foundation adequacy. The improved SSI methodology would be used to obtain the overall basemat and foundation response. Before these methods are finalized for the Monte Carlo runs, the analytical alternatives for the foundation would be evaluated. Some alternatives are discussed below.

In the selected two-dimensional SSI analyses, the soil foundation mesh near the basemat will be refined to the extent necessary for good results. Since explicit nonlinear numerical methods would be used, efficient computer usage would require the use of a different integration, time step in this refined region but this presents no real difficulty. In the normal SSI analyses, a different

time step is already anticipated in the soil and structure and, perhaps, in different parts of the soil for different reasons than the above. The results of these selected, refined SSI analyses would then be compared with alternative methods for obtaining good results. One such alternative is to use the basemat kinematic response at various times as input to the static analyses of the foundation near the basemat. If the inertial effects in the soil near the structure are not significant, then it may be possible to obtain reasonable agreement between the refined SSI analyses and the simplified static ones.

Another alternative is to proceed similarly but use dynamic rather than static analyses. Another possibility is to use the substructure approach for the SSI analyses as proposed for the design of the floating nuclear power plants by Offshore Power Systems. In these detailed studies, the sensitivity of the results to the bulk modulus or equivalent will be obtained. If these results indicate sensitivity to this quantity, and if the soil model used is not adequate in this regard, then additional testing would be proposed.

It seems clear that some combination of the above or other methods could produce better results than are presently obtained. The important practical question is whether these

methods are more economical than direct refinement of the SSI analyses. Intuitively, the latter is not attractive; particularly, if the results of other portions of this proposal indicate that three-dimensional SSI analyses are required.

Task C.4.5 is to develop a foundation-design philosophy. As discussed above, one area to be investigated includes the advantages and disadvantages of designing the soil foundation for the OBE as well as the SSE loading condition.

Task C.4.6 is to develop functional requirements for the soil foundation. For example, the functional requirements on the foundation would be expected to be different for the OBE and SSE loading conditions. In addition, methods will be developed for defining when localized "factors of safety" are significant and when they are not. For example, in a given soil foundation, it may be much more realistic to be concerned about the overall factor of safety against gross failure than local "failure." The methods will also be developed towards addressing performance limits rather than factors of safety. For example, considering liquefaction evaluations, it may be more realistic to be concerned about the deformations that may result rather than loss of strength. This is particularly true in liquefaction where cyclic mobility

rather than complete loss of strength is the realistic possibility. SSE and OBE analyses would be performed, and the comparison of alternate allowable stresses and performance would be evaluated.

Task C.4.7 is to identify candidate large-scale field events where

failures occurred. This task will also include a preliminary evaluation of these events as to their appropriateness for analytical evaluation. If events are identified as appropriate, the effort required to evaluate them will be the subject of a future proposal.

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Section 9

INELASTIC ANALYSIS PACKAGE

Introduction

There is abundant evidence that the nonlinear response of structures is extremely important. First, it is well known that many structures were loaded well past their design limit by the recent San Fernando earthquake, yet no observable damage was recorded.¹ It has also been demonstrated that, even at reasonably low response levels, the nonlinear response of reactor facilities is important.² In addition, NRC has often emphasized that one of the major sources of conservatism in the entire SMC is that a linear elastic analysis is required.^{3,4}

The literature is replete with proposed nonlinear methods of analysis. These range from a very simple elastic-plastic response spectrum approach to extremely complex 3-D finite-element computer programs. But as is with all engineering analysis, it is not possible to model adequately all aspects of a complex structures behavior by any single computer program. Thus, the method of analysis used must be chosen primarily because it does the "best" job of modeling the important phenomena that characterize the dynamic response and "failure" of a particular class of complex structures.

The problem of inelastic analysis of nuclear power plants is more

complex than that of conventional structures. First, the important structural and safety systems are much more complex and massive. Secondly, and extremely important, the nonseismic loads (including thermal stresses) must be properly incorporated into an inelastic analysis. Finally, it is difficult to define what constitutes failure. For conventional structures, collapse is usually an adequate definition of failure for an event as remote as the SSE. However, for nuclear power plants, failure is the release of any significant radioactivity. Thus, the opening of possible vent paths would constitute "failure."

Objectives

The efforts in this proposed study are devoted to overcoming the above difficulties. In particular, the objectives are to:

- Select the most appropriate inelastic analysis techniques for use in the improved SMC.
- Determine what nonseismic loads must be included in the analysis, their magnitude, and how best to incorporate them into the analysis.
- For the various structures and systems, define appropriate

"failure criteria." This includes both the inelastic behavior and "ultimate" failure.

- Compare results obtained, using selected inelastic methods with available experimental data.
- Assess the impact of different choices of "failure criteria" and applied nonseismic loads on the overall assessment of seismic safety.

Approach

Task D.1 - The first task is to determine the important Category I structures to be included in systems studies. In addition to gaining and understanding what will be required of the inelastic approach to capture the important dynamic response of the structure, it will also identify the possible physical "failure" modes. Some simple bounding calculations may be required to determine the most critical cases.

For complex structures, it will be necessary also to identify the general nature of the intended constitutive-material properties to ensure the capture of any hysteric damping that could be developed in the inelastic response of the structure. This is potentially a crucial factor that must be adequately modeled in the choice of analytical technique.

Task D.2 - The second task is to identify the nonseismic loads that are applied to the different structures identified in Task D.1, and any extreme environmental conditions that are applied so sources of thermal stresses as well as temperature effects on strength can be identified. Not only is the level of nonseismic load needed but also their time histories. This information can be obtained from FSAR's and from discussions with both industry and NRC. Definition of the loads will be difficult without full cooperation from both industry and NRC. Because some of these loads are transient and unknown relative to the transient earthquake loading, some reasonable assumptions will have to be made. As suggested by OSD, our proposed plan does not attempt to evaluate the "most likely" case for these nonseismic loads but simply follows the currently used conservative approach. When the total analysis is complete, it will be possible to determine the importance of these nonseismic loads and whether it is worthwhile to evaluate them in a similar fashion to the approach used for evaluating the SMC.

Task D.3 - The results of Tasks D.1 and D.2 have defined what must be modeled, the type and magnitude of external loading, what transients and thermal stresses must be included,

and what failure modes must be analyzed. With this information, it is now possible to review the recent state-of-the-art on inelastic-structure response and choose the most appropriate technique to use. The choices should be based on judgment (i.e., which approach best models the phenomena of interest), complexity (i.e., a faster and simpler-to-use approach would be chosen over a slightly better but slower and more difficult approach), and comparisons with experimental data (see Task D.5).

Task D.4 -- This task will develop failure criteria consistent with the method of analysis and modes of failure identified. Clearly, this is one of the most difficult tasks. For each of the failure modes identified, the literature must be researched to develop failure criteria. These criteria should be entered into the analysis in the form of a PDF. Most, if not all of these PDF's, will have to be obtained by use of a Bayesian approach using the little available data. Even if the failure criteria are poor, they do not directly enter into the calculation of the loads. Thus, just comparing the loads calculated by the inelastic methods as compared to the current SMC will certainly allow one to determine whether it is worthwhile to use an inelastic analysis and, if so, what added experimental data are required.

In addition to failure criteria, it is necessary to choose both the appropriate constitutive laws consistent with the general requirements developed in Task D.1 and the required values of the material constants to use in the analysis. Once again, it will be necessary to use a Bayesian approach to define both the values and the likely variation for these values. These do enter in the calculation of the loads and, thus, are extremely important. However, more data are available to provide a better basis for defining these parameters.

Task D.5 -- Several dynamic tests, into the nonlinear range, have been performed on a number of reactors. Many of these are reviewed by Howard et al.⁵ Using this as a data base, as well as careful review of the testing currently being performed in Japan, will provide a number of cases to test the adequacy of the techniques chosen in Task D.2 and constitutive laws in Task D.4.

The available data should be reviewed and cases should be chosen for which meaningful comparisons can be made to test the approach adopted. If these results will be limited, they should give some confidence in both the analysis technique and, more importantly, in the definition of the nonlinear behavior of the material including overall damping effects.

Task D.6 - A number of somewhat arbitrary assumptions were made in the above tasks: in particular, the values used for the nonseismic loads, failure criteria, and constitutive relations. Eventually, each of these factors will be appropriately varied in the systems analysis. However, as discussed in the sections on the systems analysis, it is not practical to vary all factors. Thus, this task will be to determine which factors are important: i.e., have sufficient impact on the final assessment of the adequacy of design to insure that these factors are appropriately included in the systems analysis.

The initial systems simulation of both the current and improved chain will allow the choice of the critical cases that should be considered. For these cases, inelastic analysis should be performed where the nonseismic

loads, failure criteria, and material properties are varied. At this time, it is not known whether this will require a Monte Carlo analysis or can be determined from the analysis of a few bounding cases. The choice will be based on how well one can define the nonseismic loads, and how important they are relative to the earthquake loads. If it is felt that one can easily develop meaningful PDS's for the nonseismic loads, then we suggest the use of a simple Monte Carlo approach.

Once these bounding calculations are carried out, it is possible to make an overall assessment of the advantages and disadvantages of using an inelastic analysis. It will also define those areas requiring either added data or better definition of loads and loading combinations.

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Section 10

STRUCTURAL PARAMETERS SENSITIVITY STUDIES

This section covers three different items which we feel must be evaluated in the systems context as part of the sensitivity analysis. They are:

- (1) Evaluation of the conservatism in the approaches used to develop floor response spectra.
- (2) Evaluation of adequacy of the mode-combination procedures currently used.
- (3) Evaluation of both the damping values used and the manner in which these values are reflected in the analysis.

The approach for each of the above will differ somewhat because different parameters are involved. However, while doing the Monte Carlo analysis, the appropriate intermediate variables can be saved. Then, by using these data, the effect of various factors can be determined.

TASK: E.1. GENERATION OF FLOOR RESPONSE SPECTRA

Objectives

The objectives of this effort are to: assess the conservatism that may be regulated into the current approach of using a single time-history that envelopes the broad-band R.G. 1.60 spectra as input in the free-field;

then to arbitrarily broaden the calculated floor response spectra to account for unknowns in the structural analysis, (2) determine if approximate methods of generating floor spectra need to be improved so that they are an acceptable alternative.

Approach

It is necessary to address these objectives in the whole context of the systems analysis because:

- (1) The level and shape of the basemat response spectra are considerably different in a nonpredictable way, depending upon the time history used and peak g value, and type of soil conditions, depth to bedrock, depth of embedment, etc.
- (2) The basemat response spectra are considerably altered, depending on the dynamic response characteristics of the different main structures, and location of the points where the floor response spectra are to be calculated.
- (3) The uncertainty introduced in the analysis due to the calculation of stiffnesses, estimation of the mass distribution,

and estimation of material and damping properties is also a function of structure type and complexity.

- (4) Only the narrow band of natural frequencies of the mounted equipment is important.

Because of this successive filtering (1 and 2), what might seem an important variation at one step may not in fact be important for the dynamic response of the mounted equipment.

Task E.1.1 - First, it is necessary to determine the approximate dynamic characteristics of all important Category I equipment classes and the relative location of this equipment in the various buildings and piping systems, etc., and the approximate dynamic characteristics of these buildings and systems. This information should be developed in conjunction with a similar effort described in Section 9. From this matrix and a few building SSI calculations, one can select which buildings and systems should be analyzed to ensure that we have covered a sufficient range of cases. Clearly, the important parameters are the filtering effects of the main structure and the relative dynamic characteristics of the mounted equipment.

Having identified what class of structures must be considered and at

what relative locations floor spectra should be generated, it is an easy process to save the calculated time histories of these locations from the Monte Carlo runs of both the current and improved SMC's.

Task E.1.2 - From these time histories, it is possible to compute floor-response spectra and make the following comparisons:

(1) For the current SMC, one can from the population of floor spectra (for each location) determine the most "likely" spectra and the possible variations and probability of getting a given variation. These variations can be compared to the current guidelines used to broaden the mean spectra to account for all the factors which were varied in the systems analysis. These comparisons will be made, in particular, in the frequency bands of typical mounted equipment. (Note the probability of getting a given variation does not include the probability of occurrence of a R.G. 1.60 spectra.) From this comparison, one can determine if the current guidelines used to broaden the spectra are adequate or if for certain classes of equipment they can be relaxed.

(2) For the improved SMC, proceed as in (1); however, in this case, because all earthquakes that could affect the site are included, we do not want the "most likely" but the

envelope spectra that has a given probability of not being exceeded. This envelope spectra can be compared with the spectra in (1) to determine the conservatism of the current method. The comparison will also be weighed in the frequency bands of typical mounted equipment.

The spectra developed in (2) are the best estimate of what would occur at the site for a given period of time based on the probability level chosen. If comparisons (1) and (2) turn out significantly different, then one should determine what factors mainly contributed to the difference. We expect the main contributor to the difference to come from the choice of time-histories used; i.e., in (1), an envelope R.G. 1.60 time history was used and in (2), real earthquakes related to the seismic characteristics of the region, corrected for site conditions, were used. Traveling waves and structural damping may also be important, as may inelastic behavior. Once the main contributions to the difference(s) are identified, one can then determine what additional system analysis are required so that the results can be generalized for purposes of recommending improvements in the current procedure.

Task E.1.3 - The second objective of the effort is to assess and suggest improvements, if necessary, on the

approximate methods of generation of floor spectra. The two most widely used methods are the ones suggested by Biggs¹ and by Kapur and Shao.² Biggs' method can be improved as it is based on the enveloping of results to obtain an "empirical procedure." Biggs used only a very limited number of cases and handled SSI by soil springs. The extensive calculations carried out in getting (2) provide a much better "data base" to use to extend Biggs' method. We envision that the factors used in the method would vary regionally and with site conditions. Comparisons of the proposed improvements of Biggs' method to the result of (2) and other bounding calculations should be made to determine the adequacy of the method and to assess its conservatism.

The method of Kapur and Shao is based on theoretical considerations and therefore not easily extended in the same way as Biggs' method. However, if the results in the above analysis [(1) and (2)] indicate, for example, that traveling waves are important, one can extend both methods to account in a simple fashion for these factors by using an approach suggested by Scanlan.³ The "improved" site-response design spectra from the package on seismic input can also be used. If inelastic response is extremely important, then approximate

correction factors can be developed for both Biggs' approach and Kapur and Shao's approach.

TASK: E.2. COMBINATION OF MODES

Because of the assumptions that must be made in deriving the mode combination rules given in R.G. 1.92, final verification of the rules must be obtained by comparison of the results of a number of time-history analyses to the results using the accepted mode-combination rules. To date, these rules have not been tested over a spectrum of possible time histories which envelope R.G. 1.60, nor have the comparisons adequately included the important effects on the response spectra caused by SSI. Finally, they have not been compared to the sets of real earthquakes that could occur at various sites in different regions of the country. This is particularly true of the references cited in R.G. 1.92 for verification of the acceptable mode combination rules.

Objectives

The objectives of the effort will be to:

- Determine if the mode combination rules given in R.G. 1.92 are adequate and to assess the relative conservatism of using the spectral analysis approach vs a time-history approach.

- Assess the added conservatism that is introduced by using an envelope response spectra in the free-field rich in all frequency components.

Approach

To improve existing assessments, it is necessary to:

(1) Ensure that the important dynamic characteristics of all important classes of Category I structures, systems, equipment, etc., have been included. Of particular interest are closely spaced modes.

(2) Ensure that a statistically adequate sample of possible time histories representing R.G. 1.60, real earthquakes, and the biasing effect that could occur because of the different levels in seismic activity in various regions have been included.

(3) Ensure that a range of sites accounting for the strong "filtering" effect induced by the SSI calculational step has been adequately included.

The system analysis carried out for the current SMC should include a statistically adequate sample of time histories. Also, the analysis should be carried out for both a representative and several "bounding" sites.

The systems analysis of the improved SMC will provide the needed data

covering the range of real earthquakes that can occur at a given site.

Task E.2.1 - The dynamic characteristics, etc., of all important Category I items should be obtained from the effort described above in the section on the development of floor response spectra. For each structure included in the systems analysis, it is necessary to determine at what elevations and locations comparisons should be made. These should be chosen by using several criteria. The most important are those locations contributing mostly to the design controlling stress. Each structure used in the system model should be examined to determine, according to either structural configuration and/or dynamic characteristics of the structure at which additional locations comparisons should be made.

Task E.2.2 - The analysis scheme outlined for both the current and improved SMC is based on the time-history approach. However, the information needed to carry out a modal analysis is part of the output of the dynamic analysis program used to analyze the structures. Thus, it is a simple problem to combine these results with the spectra developed above and compare the dynamic response quantities computed via the mode-combination rules with those calculated by the time-history method.

The assessments of the above comparisons should be made in the following manner. For each pass through the systems analysis, calculate the response (at each location) by using both the time-history analysis and a spectral analysis with the appropriate mode-combination rule. This gives two sets of data. Then one can determine by use of standard "significance" tests whether there is a significant difference between the two methods. This can be done both by considering each data set separately and by testing the two sets as an ordered pair (i.e., each pass gives a time-history value and a modal value). If these tests show a significant difference, then further significance testing should be performed for subgroups of the data. Such testing would determine which (if any) conditions might consistently lead to a significant difference between the two methods; such parameters as closely spaced modes, odd structural configuration, real earthquakes, time histories, etc., should be used in the cross-correlations. From this analysis one can determine if the mode combination method is or is not adequate under all cases, or what are (both type and value) the conditions for which the method is "inadequate." Clearly, if the mode combination method significantly underestimates

the dynamic quantities of interest, the method would be deemed inadequate. However, if it overestimates the dynamic quantities, it is more difficult to determine if the approach is "inadequate." In either case, the only meaningful test of the significance of any differences that occur between the two methods is the impact on the final calculated stresses relative to the variation that can occur in these calculated values caused by all the other uncertainties existing in the overall SMC.

The necessity of adding special cases to fill out the data set depends on the results of the somewhat restricted systems analysis of both the current and improved SMC. One can then determine whether meaningful conclusions can be reached without the necessity of carrying out the detailed system analysis over an extended range of sites and conditions.

The above analysis will allow one to assess the adequacy of the mode-combination rules and their relative conservatism vs the time-history method. An important question is the conservatism that occurs because an envelope spectra used [R.G. 1.60] in the free-field rich in all frequency components is compared to real earthquakes which have many frequencies suppressed. The answer to this question also falls out of the above analysis. If the use of an envelope

spectra is conservative then the significance test will show (presumably) that the mode-combination results are significantly higher than those obtained by the time-history method for the current SMC as compared to the same comparison for the improved SMC. It should be noted that we are proposing only comparing the differences between a time-history calculation and a modal calculation. Thus, it is possible to conclude that there was very little difference between the current SMC and the improved SMC regarding the stress differences between the time-history method and the modal method; and yet elsewhere conclude that there was considerable difference between the level of response between the two SMC's.

TASK: E.3. DAMPING VALUES AND THEIR IMPACT ON SEISMIC LOADS

This task deals with two different aspects of damping. The first does not need to be evaluated in the context of a systems analysis. It determines what damping values should be used in the linear analysis that is part of improved SMC. It also reviews adequacy of the current guidelines given in R.G. 1.61. The second aspect is concerned with the impact that any given choice of damping values has on the final loads. This evaluation must be carried out in the systems context.

Of particular importance is the damping model employed. In actual structures, the energy dissipation is caused by following phenomena:

- Viscous damping due to molecular viscous action of materials.
- Hysteretic damping due to imperfect elasticity of structural members.
- Slip motion at joints between structural members.
- Radiation damping due to soil/structures interaction.
- Collision between structural components.
- Compaction of crushable material such as insulation on pipes.
- Sloshing of liquids in tanks and vessels.

In most dynamic analyses, the damping mechanism of a structure has been evaluated as viscous-damping proportional to the velocity of the structure. Experimental data confirm that energy dissipation is not linear. Nevertheless, simplicity of analysis causes most studies to use equivalent viscous damping. The tabulation by Newmark, or NRC in R.C. 1.61, are most often referred to. These dampings, given as fraction of critical damping, are specified for various types of materials, construction and assembly methods, and stress levels. Note

that these dampings are generally used as modal damping, a different value being assigned to each mode of vibration of the structure.

This formulation is then used to evaluate the damping coefficients from the experimental data. However, because the model used to evaluate "damping" for a given test is such a poor representation of the actual phenomena, it is difficult to properly extrapolate the test to some other problem.

To evaluate damping fully, it is necessary to include, in addition to the above factors, the effect of the response of the structure. Depending upon the nature of the excitation, the role that damping plays changes. For example, for piping systems, etc., which are assumed to be very lightly damped, it makes considerable difference if the earthquake is of very short duration (shock-like) or many cycles of near-peak motion. A time-history enveloping a broad-band input spectra will generally have many cycles of near-peak motion; whereas, it may well be that none of the earthquakes that a given site might experience would be of this nature. Thus, importance and impact of various choices of damping values could also be regionally dependent. Also, the damping role of the soil may be very important. The really major effect of different levels of damping occurs

near resonant conditions. Thus, the filtering effect of SSI analysis could have considerable impact on the effects of different damping values and the final loads.

Objectives

The objectives of this package are:

- To develop a definitive source book on damping factors for use in design of nuclear power plants and define what added empirical-damping data are needed either to assess the currently used values and/or change them.
- To assess the impact on the final loads/floor spectra of various choices of damping relative to both the current SMC and to a series of real earthquakes as given in the improved SMC.

Task E.3.1 - This task is to gather all damping data in the literature and to assess for each test the nature and level of excitation, the stiffness of the test structures, possible sources of nonlinearity, main sources of the damping, and the approach used to estimate damping values from the test data.

Task E.3.2 - This task is to review the way damping is entered into typical calculational approaches for analysis of nuclear power plants. In

light of Task E.3.1 and relative to the stiffness of structures, and piping loops,

- a) What typical ways of estimating damping values from experimental data are applicable?
- b) What class of nonnuclear structures can we use damping data from to extrapolate for use in nuclear power plant analysis?
- c) What effect will the difference between seismic excitation and test excitation have on damping values, and when do nonlinear effects become important, etc?

Task E.3.3 - This task is to use the results of Task E.3.2 to extract the useful data from Task E.3.1. In some cases, the data for given tests obtained in Task E.3.1 will have to be reevaluated in light of Task E.3.2. With this improved data set, and in conjunction with the assessments discussed, it is possible to cast the damping values into probabilistic form suitable for use in the improved SMC by using a Bayesian approach. At this stage, it is then possible to make a simple assessment of current guidelines.

Task E.3.4 - If this task is to achieve the second objective above, it is necessary to use the output of the systems analysis. It should be recalled that, in the systems analysis

of the current SMC, the damping values were restricted to the current regulations. Thus, it will be necessary to extend some of the structural analysis in the current chain by using the variation in damping developed in Task E.3.3.

The results of the systems analysis of both the current and improved chains, plus the added data developed above, will allow one to directly determine how structural loads vary relative to damping as a function of

the type of excitation (real earthquakes vs R.G. 1.6 earthquakes) and structure type. The added filtering effect of the SSI calculation can also be calculated. From this, one will be able to determine if increasing the damping values is significant and if it is important to have well-defined damping values and distributions.

The effect of damping on floor spectra can be made by using the results of the study on floor spectra.

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Section 11

MOST NEEDED FUTURE WORK

In the preceding sections, we have analyzed the current SMC in some detail and have presented many alternative approaches to phases of the current SMC. Rather than attempt to summarize the large amount of material, we pinpoint what we judge to be the most pressing items having the greatest impact on the overall design/assessment process.

Currently, we judge that the most severe problem is the proper relation between the specification of possible earthquake ground motion that could be recorded by an accelerometer at the surface of a given site to the design response spectra, considering that an elastic analysis is used with a conservative "failure criteria." As pointed out in Section 3, this problem has been handled by arbitrarily suppressing the design g value relative to the definition of the seismic hazard defined in Appendix A. While the current seismic design of nuclear power plants undoubtedly offers considerable protection relative to earthquakes, it is impossible to judge what this level of protection is and if it is acceptable. The program/plan we outlined above would, of course, provide this information and much more. In our judgment, this

problem is so pressing that it should be addressed at once, even if an "ad hoc" approach is used.

Alternatively, nonlinear methods of analysis could be introduced and more realistic failure criteria used. "Failure" should not be defined in terms of the failure of any given element but factored into a fault-tree analysis such as that developed in WASH-1400. It would then be possible to define the earthquake in a more realistic manner, and the true weak points of the design could then be properly identified. The net result would be a better designed system with improved safety for all conditions.

The SR/SSI analysis methodology profoundly influences the design spectra, yet there is little-to-no evidence that the methods used to predict the effects of analysis are realistic, conservative, or unconservative. Qualification of the present quasilinear or newly proposed nonlinear methods by large-scale testing and analysis of real large-scale events such as earthquakes and nuclear weapons tests is badly needed. The problem is rendered more complex by the suggestion (discussed above) that traveling waves are extremely important. All of these factors can be

reasonably verified by carefully assessing existing data and methodology, as discussed in Sections 6-8, and a properly designed testing program. The relation of the spectral shape to local site factors and deconvolution must also be resolved so that test data to generic sites can be extrapolated.

Many other important items have been identified elsewhere along with proposed programs. In some cases, it is possible to perform the work without reference to the overall

system, e.g., the relation of the spectra level and shape to the earthquake-source parameters, relation of spectral shape to site factors/travel/and source effects, etc. But, as discussed in detail, considerable uncertainty will be associated with the results of these studies. Hence, to factor these "improvements" into the SMC, requires that a methodology similar to that outlined in this report be used to set appropriate levels, etc.

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