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The Use of In-Situ Procedures for Seismic Qualification of Equipment in Currently Operating Plants

Prepared by S. Sadik, J. G. Arendts, B. W. Dixon, T. E. Rahl, M. J. Russell

EG&G Idaho, Inc.

Prepared for
U.S. Nuclear Regulatory
Commission

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ABSTRACT

This project supports Nuclear Regulatory Commission (NRC) Unresolved Safety Issue (USI) A-46 for "Seismic Qualification of Equipment in Operating Plants." The project was performed in four distinct efforts which are discussed in separate sections of the report. The first effort (Part A) identified the basic technical approaches for using in-situ test procedures as a tool in alternate methods for the seismic qualification of equipment in operating plants. For most applications the full potential is achieved when structural dynamics analysis methods are used in conjunction with in-situ procedures. Thus, the basis and applications for the combined use of in-situ procedures and analysis methods was developed. To provide cost effective applications, improved methods of analysis are required. Part B concentrated on the development of improved methods of developing structural models using the results of in-situ procedures, and predicting structural response during seismic events using methods of random vibrations. Thorough technical justification for these methods of analysis was provided to support the related guidance and acceptance criteria presented in Part C. Also, new developments in the area of in-situ procedures were reviewed. Part C developed guidance and acceptance criteria, presented in the format of an engineering standard, for the performance of in-situ testing, the use of structural analysis methods, the development of structural models, and combinations thereof. Finally, Part D developed a cost estimate for using the various alternative methods for seismic qualification of equipment. The NRC will combine the results of this work with input from other contractors to resolve USI A-46.

EXECUTIVE SUMMARY

This project provided technical assistance to the United States Nuclear Regulatory Commission (USNRC) toward the resolution of Unresolved Safety Issue (USI) A-46, "Seismic Qualification of Equipment in Operating Plants." This work was performed during fiscal years 1982, 1983, and 1984 at the Idaho National Engineering Laboratory under contract to the USNRC. During the period since licensing of older plants the qualification criteria and methods, and the Seismic Category 1 categorization of equipment have been modified. USI A-46 addresses questions concerning the existing and required levels of dynamic qualification of equipment for these currently operating plants. This work addresses a portion of those questions.

The body of this report is divided into 4 parts, each of which addresses a specific area.

In Part A the current technologies of equipment qualification and in-situ testing procedures are reviewed. The potential uses and limitations of in-situ procedures in qualifying equipment are presented. Technology was found (with minor exceptions) to be lacking in the area of in-situ methods which could be used as the sole method of determining the level of equipment qualification. One link in the qualification process for equipment in operating plants is determining the design basis environment. For equipment located in equipment supporting structures such as cabinets, panels, and racks the supporting structure can strongly modify the environment on the equipment. The most important future application of in-situ procedures will be their use in streamlining the process for determining the design basis environment (the required seismic capacity). Another link in qualifying equipment in operating plants is determining the seismic capacity of equipment. This link can be filled by comparing to a similar piece of tested equipment. In-situ testing can be a portion of the test for similarity by providing frequencies and mode shapes for the subject equipment. Recommendations on the use of in-situ procedures and structural analysis methods for operating plant equipment qualification are presented.

Part B develops and presents methodology improvements which will substantially reduce the analysis effort associated with the prediction of seismic environments. A method of seismic structural dynamics analysis based on the principles of random vibrations and the mean square integrated structural response is presented. The detailed justification of this methodology for seismic analysis is presented. This method of analysis will radically streamline the analysis process and represents an improvement on methods for transferring response spectra. Improvements to methods for constructing an analytical model using the results of in-situ procedures are developed. This method allows the development of the structural dynamics model knowing only the significant mode shapes and the associated natural frequencies which are readily determined using in-situ procedures. Using the performance of equipment in real earthquakes to estimate the seismic capacity of similar equipment in nuclear power plants is a concept which is under evaluation in USI A-46. Floor spectra experienced by equipment in real earthquakes must be conservatively estimated to develop a seismic experience data base. It was found that estimated ground spectra could be used as a lower bound for estimating floor spectra under specific conditions. In-situ procedures currently under development with potential future application to seismic equipment qualification are also reviewed. Finally it is pointed out that studies which confirm the accuracy of the combined use of in-situ procedures and analysis methods to predict the dynamic environment of equipment do not exist. The validity of the methods is based on theoretical considerations, engineering judgement, and appropriate margins of safety.

Part C develops guidance and acceptance criteria for using seismic experience data, in-situ testing, and structural dynamics analysis methods to qualify operating plant equipment. Seventeen (17) technical areas which require specific guidance and acceptance criteria are defined. These 17 technical areas are individually examined, and guidance and acceptance criteria are reported. These guidance and acceptance criteria have been presented in a form similar to an engineering standard so that they can be readily employed.

Part D estimates the cost to qualify various equipment types. Table 1 in Part D lists these estimates.

FOREWORD

Beyond the Abstract and Executive Summary this report is divided into four parts, each of which is capable of standing alone as a separate report. The tasks associated with the different parts were performed sequentially except for Part D which was addressed in parallel. Different authors were associated with the different parts as cited in the Acknowledgment. These parts were originally intended to be separate informal reports but subsequently were requested to be combined into a single report.

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ACKNOWLEDGMENTS

Part A was authored by S. Sadik and B.W. Dixon and originally issued as an EG&G informal report. Parts B and C were authored by S. Sadik and originally issued as letter reports for the US Nuclear Regulatory Commission. Part D was originally issued as an EG&G informal report authored by J. G. Arendts, T. E. Rahl, and M. J. Russell.

The assistance and advice of Professor M. P. Singh (Virginia Polytechnical Institute), and G. B. Patrick and J. R. Henricks (Farwell and Hendricks, Inc.) was greatly appreciated by S. Sadik. These individuals provided their technical expertise on a goodwill basis and their contributions extend beyond the extent to which they are credited in the references.

PART A: PRELIMINARY STUDY OF THE USE OF IN-SITU PROCEDURES FOR SEISMIC EQUIPMENT QUALIFICATION IN CURRENTLY OPERATING PLANTS

SUMMARY

The Nuclear Regulatory Commission (NRC) has designated seismic and dynamic equipment qualification of safety related equipment in currently licensed and operating nuclear power plants as Unresolved Safety Issue A-46 (USI A-46). During the period since licensing of older plants, qualification criteria, qualification methods, and safety classification of equipment have been modified. Thus various questions concerning the existing and the required level of dynamic qualification for these currently operating plants are being addressed by USI A-46.

EG&G Idaho, Inc. has provided technical assistance toward the resolution of USI A-46 by examining the potential uses of in-situ testing in operating plant equipment qualification. The efforts included a limited review of in-situ procedures. The potential applications and limitations of in-situ testing to equipment qualification were examined. Alternate qualification criteria and methods have been considered and an alternate methodology is proposed. The effective use of in-situ procedures requires the use of associated analysis methods and these methods have been examined or developed, as required. These efforts are summarized in the following paragraphs along with the recommendations derived from the studies.

Potential applications exist for in-situ procedures, especially when used in conjunction with analysis procedures. A review aimed at finding developed technology or technology which is near full development was performed. This review has not uncovered any practical and widely applicable in-situ methods which can be employed as the sole means of qualifying or for determining the relative level of equipment qualification. In-situ procedures performed at low excitation levels can be employed to determine dynamic natural frequencies and mode shapes of equipment supporting structures. The majority of equipment qualified by testing are mounted in such equipment supporting structures. These structural quantities can then be employed in combination with analysis procedures to estimate the design basis dynamic environment for equipment.

Several detailed routes are discussed in the report to achieve this end. Thus in-situ procedures will be most useful in determining the required seismic capacity for equipment.

For the majority of active safety related equipment, a seismic qualification chain can be defined. The chain consists of qualifying the equipment supporting structure anchorage, the equipment supporting structure dynamic response during the earthquake, the mounting of equipment to the equipment supporting structure, and the functional operability of equipment during (if required) and after the seismic event. Recommendations on equipment supporting structure response and mounting adequacy have been developed and are presented.

Alternate qualification criteria and procedures have been considered. No further alternatives are required for estimating required seismic capacity. Since the missing link in the qualification chain is estimating the seismic capacity of equipment, an alternate method based on similarity between equipment which has been tested and the equipment in question is presented. The basis of the method is a categorization of failure modes into four types. Basically, a critical failure mode is established, a tested piece of equipment with less than or equal seismic capacity is identified, and a conservative seismic capacity for the item of interest is inferred from the tested item. The method is most applicable for simpler pieces of equipment where a design evaluation can provide the justification for similarity.

Analysis procedures are employed in combination with parameters determined from in-situ testing to predict the required seismic capacity of equipment. Seismic analysis procedures based on linear modal superposition require knowledge of the frequencies of significant modes, the associated mode shapes, damping, and the mass distribution. In-situ procedures provide frequencies and mode shapes, and damping is specified in NRC regulatory guidance. Methods for determination of mass distribution, or alternately the modal participation factor of seismic structural analysis, have not been extensively discussed in the literature. A relatively straightforward, verifiable technique which rewards accurate determination of the significant mode shapes is presented in detail in the report. Other methods are also discussed.

Seismic inputs and outputs are commonly described by means of response spectra. In performing seismic analysis it is necessary to transfer response spectra through structures such as the reactor base mat to a building floor and then to a specific location in a support device. The commonly employed process involves the generation of synthetic time history inputs followed by a time history analysis. Direct methods of response spectra transfer would combine the systems mechanical characteristics directly with input response spectra to yield output response spectra. No intermediate time history generation or analysis is required. Direct methods would provide a substantial gain for operating plant qualification because the analysis procedures are algebraic thus providing considerable streamlining of the current analysis procedures. However no validated method for direct response spectra transfer could be established in this phase of the program. The difficulty occurs in determining the response spectra when the spectral (or oscillator) frequency is very near one of the structural natural frequencies. Appendix A presents a method for estimating the response spectra at frequencies remote from the natural frequencies.[a]

Specific recommendations for qualifying equipment in operating plants have been developed and are discussed in more detail in section 5 of this report. In-situ procedures have been recommended as an acceptable procedure for determining structural mode shapes and natural frequencies. The combined use of analysis and in-situ procedures for determining required seismic capacity without the development of a finite element model is described. The modal participation factor is calculated from a verifiable procedure which is described. This method is the recommended method for the direct use of in-situ parameters for determination of required seismic capacity. If the required seismic capacity is calculated

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a. It must be noted that in Part B the response spectra transfer issue is completely resolved and supersedes the results presented in Part A.

using a finite element model then it is recommended the model be validated by showing close correspondence between model and in-situ determined frequencies and mode shapes of significant modes. Seismic qualification is achieved if prior testing has shown the equipments' capacity to exceed the required capacity.

A procedure for establishing similarity of seismic capacity between two pieces of equipment has been recommended. Successful use of the procedure would yield an estimate of seismic capacity in situations where data for the equipment in question is not available. Finally recommendaticns for two considerations unique to older currently operating plants have been made. One recommendation is to experimentally (in-situ) determine the fundamental natural frequencies of all support devices containing safety related equipment to identify if they align with the amplified region in the floor response spectra. The final recommendation is that all mountings for safety related equipment be screened for potential shortcomings. The recommended screening procedure is a plant walk-through.

PRELIMINARY STUDY OF THE USE OF IN-SITU PROCEDURES FOR SEISMIC EQUIPMENT QUALIFICATION IN CURRENTLY OPERATING PLANTS

1. INTRODUCTION

The growth of the nuclear power industry during the 1960s and 1970s coincided with increasing emphasis on safety issues inherent in commercial nuclear facilities. As a matter of public safety the industry is federally regulated, requiring standby safety systems capable of controlling and stabilizing a facility in the event of environmental transients or equipment failures.

These safety related systems are categorized into passive and active groups where active safety related equipment must perform some operation in fulfilling its safety related function. They are subject to design control measures[1] whereby the design must be qualified to specific criteria established by the Nuclear Regulatory Commission (NRC). In the field of seismic safety the movement of the state-of-the-art and the accompanying regulatory stance has resulted in qualification criteria where newer plants and plants currently undergoing licensing review are seismicly qualified to a greater degree than older plants. The NRC therefore has implemented Unresolved Safety Issue-A46 (USI-A46) whose focus is restricted to seismic qualification of equipment in operating plants. Several contractors are active in developing technical assistance to USI-A46. Generally speaking the technical assistance is concentrating on practical methods for evaluating the seismic qualification of older facilities, assessments of the level of qualification required for public safety, and the development of procedures which will expedite the industry's achievement of these qualification criteria.

1.1 The Qualification Process

While the first nuclear power plant (NPP) designs were based more or less on conservative engineering judgment, recent advances have provided enhanced methodology for seismic design. Initiated by requirements found in Chapter 10 of the Code of Federal Regulations as well as a recognition of need within the major professional engineering associations, design and testing criteria have evolved over a period of time. These criteria are contained in foundation documents such as the IEEE and ASME publications which are endorsed by the NRC via NRC Regulatory Guides. Additional guidance and data are presented in NRC NUREGs and professional papers. These documents outline acceptable seismic qualification methods and criteria through the use of analysis, testing, the combination of the two, and finally similarity to previously qualified equipment. Testing is the preferred qualification procedure for active equipment.

1.2 Introduction to Task

Many currently operating nuclear plants were designed, licensed and placed on line prior to adoption of the current seismic qualification criteria. These criteria implement recent developments in experimental and analytical methods. As operating plant equipment may not meet the current criteria, there is a need to consider the amount and level of requalification needed to ensure structural integrity and operability of the safety related equipment in these facilities. Due to the character of operating plants, application of current qualification criteria may result in substantial impact on these plants. Excessive plant downtime, shipment of irradiated components to test labs, and extended manhours in contaminated areas are but some potential concerns.

EG&G, Idaho is assisting the NRC by providing technical assistance to the resolution of USI-A46. Our task has been to consider the methods by which in-situ procedures can be applied to qualifying equipment in operating plants. Toward this end a review of in-situ testing practices has been performed. This review has consisted of examining technical literature as well as personal contacts with professionals active in the field. Analysis procedures are inherent to the utilization of data derived from in-situ measurements. Thus a limited review of potentially applicable analysis procedures has also been conducted. The focus has been primarily on well developed methods. However the relative lack of literature has necessitated independent developments as well. The combined use of analysis procedures and modal parameters determined by in-situ procedures has been outlined.

One goal of USI-A46 is to develop alternate qualification criteria for currently operating plants. The use of in-situ procedures as the basis for major alternatives to current criteria and procedures has, therefore, also been examined. The results of this examination led to a broader study resulting in a definition of failure mode categories. Evaluating a design for each failure mode provides a basis for seismic similarity between two non-identical pieces of equipment that can be used as a qualification tool. Aging degradation has been examined from the standpoint of in-situ testing and also failure modes.

1.3 Report Scope

This report covers interim progress during the period 4-15-82 to 11-1-82. Pertinent topics covered by this report include the following:

- A limited discussion of the current qualification process is presented in Section 2. Intent, requirements, and approved procedures are discussed consistent with the limited examination necessary for this program. Current qualification procedures for active equipment are emphasized.

- Section 3 discusses the use of in-situ procedures in qualifying equipment. The discussion is general and identifies uses for which no technical basis exists. Its potential uses are also discussed.

- Section 3 discusses alternate qualification methods which are not necessarily dependent upon in-situ testing. These considerations have been limited to methods which are strongly aligned to current qualification criteria. Probabilistic techniques, for example, are not employed. The result is a proposed basis for establishing similarity of seismic capability between nonidentical components. Section 3 also addresses other considerations affecting seismic equipment qualification in operating plants. These are the effects of aging degradation on seismic capacity, equipment mounting evaluations, and cabinet dynamic response.

- Section 4 discusses the use of analysis procedures in conjunction with in-situ testing. An analysis procedure is presented which directly employs modal parameters (quantities determined by in-situ procedures) to predict the design environment on equipment contained in support devices (cabinets, racks, etc.). Dynamic response within support devices is very important because they contain the bulk of active safety related equipment. The use of in-situ procedures in conjunction with standard finite element methods is also discussed.

2. FUNDAMENTALS OF QUALIFICATION

Developing and understanding guidance related to seismic requalification of operating plants requires a prerequisite knowledge of the current qualification process. This chapter is designed to provide necessary background information while introducing many of the issues to be examined later. The chapter is divided into four sections containing, in order, a description of the current safety philosophy, a discussion of seismic events and their simulation, an outline of the current qualification criteria, and a summary of the application of the criteria in the qualification of components in plants applying for operating licenses. The discussions provided on each topic are not intended to be exhaustive. The knowledgeable reader may wish to concentrate on the final section concerning current criteria application.

2.1 The Safety Philosophy

The philosophy utilized to assure integrity of nuclear facility safety systems is a combination of redundancy and separation. Redundancy minimizes the impact of the random failure whose source is usually traced to less than adequate quality in a particular item. The separation and isolation of redundant systems eliminates many of the common mode failures usually associated with loading extremes or insufficiency of design. In the seismic arena the common mode failure is of the greater importance as separation cannot be assured, leaving the facilities' safety systems open to a failure mode which attacks several redundant systems simultaneously. A major portion of seismic qualification involves verification of design by proof test to ensure sufficient seismic capacity of components with safety related functions against these common failure modes.

2.2 Seismic Phenomena

2.2.1 Seismic Events

In the nuclear industry seismic events include any natural events which produces a vibratory ground motion. An earthquake will produce pressure and shear waves, the properties of which depend on earthquake magnitude, distance from the site and the intervening structures of rock and soil. These waves will produce motion along all three axes of any reference coordinate system, an important property for design being the statistical independence of the relative motions.

Seismic events are recorded and categorized by a variety of methods. Information that is considered for plant design include the location of historic epicenters and hypocenters and the potential ground accelerations at the plant site if a similar event were to occur during the plant operating lifetime. Recorded data can be used to determine site properties such as soil damping, filtering, or possibly amplification caused by the rock and soil substructure. By considering the site specific properties in conjunction with recorded time histories from natural or induced events the effect of potential events can be predicted at the plant foundations. These effects are reduced into response spectrum form.

2.2.2 Seismic Design Loads

The response spectrum graph (Figure 1) is the main descriptor of a seismic event currently used in the design and qualification process. A number of response spectra from actual events are overlaid and a smooth curve is drawn enveloping all peaks. This curve is the "required response spectrum" (RRS) used in the determination of design loads.

If a response spectrum is developed for a specific plant site based on local geology, it is referred to as a site specific RRS. The NRC has developed a generic RRS which, while usually more conservative in shape, can be used at most sites without modification. The design earthquake spectra is based on this generic curve scaled to the maximum or zero period acceleration (ZPA).

The response spectrum has properties which limit its use to certain analysis techniques. It does not indicate the duration, exact shape, or phasing of the exciting waveform. Without this information the exact response of a particular piece of equipment cannot be determined. For this reason all testing and some analysis requires that a synthetic time history be developed.

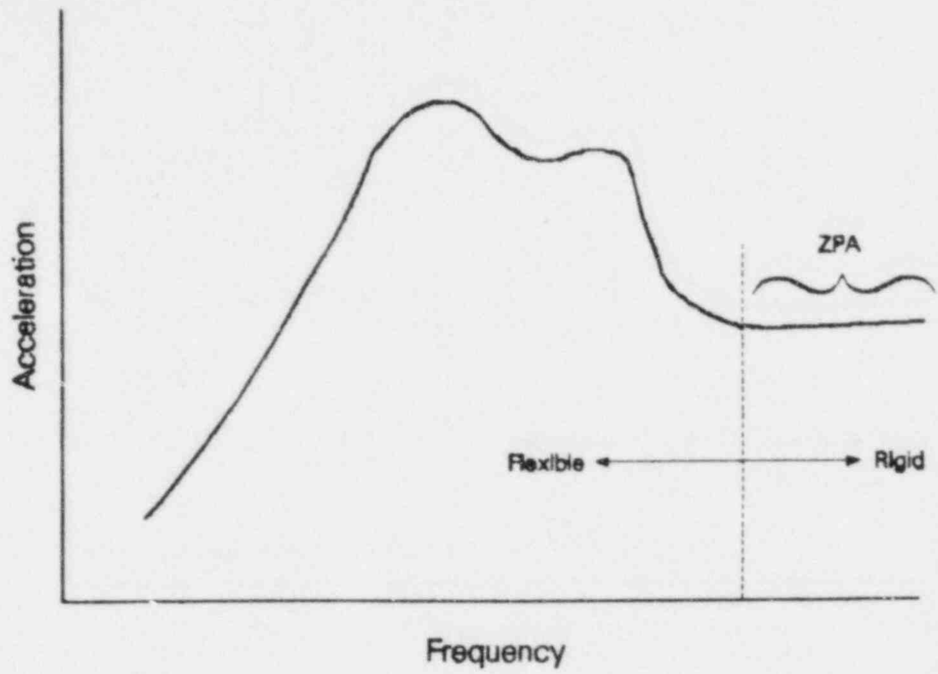

Figure 1. Typical response spectrum

The requirements for enveloping, frequency content, strong motion duration and multiple axis excitation will be discussed in the next section.

2.3 Qualification Criteria

Qualification criteria have undergone considerable evolution during the last decade. Plants designed in the 1960s for the most part had no official criteria other than the Uniform Building Code[2]. Initial criteria were published in the early 1970s and subsequently revised a few years later. In the intervening period a large amount of feedback was received and reviewed. The present criteria reflect technical refinement and recognition of testing and analysis limitations derived from these reviews.

The present criteria are based on the directive of Chapter 10 of the Code of Federal Regulations, primarily 10 CFR 50 Appendix A (see Figure 2). This appendix establishes principal design, testing and performance requirements for safety systems and components to "provide

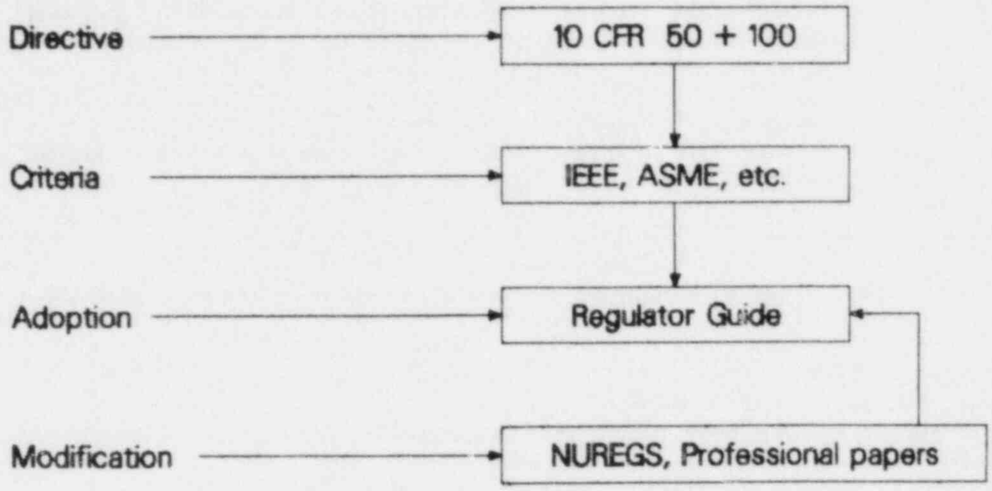

Figure 2. Structure of criteria instigation

reasonable assurance that the facility can be operated without undue risk to the health and safety of the public." Criteria 2 of Appendix A addresses the method of risk mitigation: "structures, systems and components important to safety shall be designed to withstand the effects of natural phenomena--without loss of capability to perform their safety functions." Thus the thrust of qualification is truly public safety.

In response to the need for specific design and testing standards to meet the 10 CFR directives professional societies have published documents for industry use. Many of these documents, such as the ASME codes, address materials and structural design criteria. The scope of this discussion will be limited to criteria for equipment with operability requirements.

2.3.1 IEEE Standards

In 1968 IEEE-279[3] was first presented to the industry. This standard, revised in 1971, gives general design criteria for plant safety systems. Section 4.4 addresses equipment qualification as follows:

"Type test data or reasonable engineering extrapolation based on test data shall be available to verify that protection system equipment shall meet, on a continuing basis, the performance requirements determined to be necessary for achieving the system requirements."

IEEE-308[4] publication followed IEE-279, with the original version released in 1970. This standard specifically addresses criteria for safety related electrical equipment. While this document is limited only to electrical equipment, it addresses the problem of functionality of components with operability requirements and so has been used as a guide for the design scope of pumps, valves and motors which also have these requirements. IEEE-603-1980[5] addresses the same safety related electrical components as IEEE-308 as well as mechanical equipment; however the approach is from the system view rather than the component view.

The historic lead document for qualification criteria, IEEE-323-1974,[6] is again specifically limited in scope to electrical equipment but is used as the standard for all equipment qualification. This document presents the specific types of qualification (by test, by experience and similarity, and by analysis) as well as the scope of the qualification process (loads, interfaces, etc.). IEEE-627-1980[7] addresses all components, both electrical and mechanical, from a generic view. While IEEE-627 has a broadened scope compared to IEEE-323, it does not contain th .me depth of information when subject matter overlaps.

IEEE-323-1974 was the first document to significantly address the problem of equipme aging. Aging tends to induce or assist common mode failures; therefore the development of some method of simulating and incorporating aging into the qualification procedure was required.

IEEE 344-1975,[8] specifically treats seismic qualification of electrical components. This standard provides a brief description of earthquakes and then examines the simulation of earthquakes in detail.

The frequency range of concern in an earthquake is stated as typically 1 to 33 Hz. An approximation used in the earthquake description is that the magnitude of the vertical component of excitation will be between 67 and 100% of the horizontal magnitude below the frequency of 3.5 Hz and equal to the horizontal above 3.5 Hz.

Three methods of seismic simulation allowed by the standard are the time history, response spectrum and power spectral density (PSD) function.

Two methods of damping value determination are endorsed. These are the decay rate method and the resonant peaks method, also referred to as the bandwidth method. The first involves measurement of the decay rate of a particular "pure" mode of vibration while the second is based on measurements of the width of the resonance peaks for different vibration modes when the equipment's response is frequency plotted. Other justified methods of damping determination are also acceptable.

Three primary methods of qualification are described in detail in the document:

- o Predict the equipment's performance by analysis
- o Test the equipment under simulated seismic conditions
- o Qualify by combined analysis and test.

The following summary of qualification by analysis is taken from the standard's text:

> "The general procedure is to first study the equipment to assess the dynamic characteristics; second, to determine the response using one or more of the several methods described in Section 5 of the text; third, to analyze the stresses which result from the response; and, finally, to determine if the design is adequate."

In Section 6 of the document proof testing and fragility testing are discussed. Mounting for either test must simulate the intended service mounting. This simulation must account for electrical lines, conduits, etc., as well as mounting bolts and brackets.

The following is a list of the considerations involved in testing:

- o Frequency bandwidth of the RRS compared to that of the TRS and equipment characteristics and responses
- o Duration of the test compared to the design seismic event
- o Peak acceleration of the test input motion and the amplification observed
- o Natural frequencies and modes of equipment vibration
- o Typical equipment damping
- o Fragility levels
- o Number of test cycles and fatigue failure simulation.

The basic criteria for the number of tests required is five Operating Basis Earthquakes (OBEs) followed by one Safe Shutdown Earthquake (SSE). The duration of each test must equal or exceed the strong motion portion of the original time history used in the development of the RRS for the SSE. Single axis tests will be allowed if they are conservative or if cross-axis coupling is zero or very low; otherwise multiaxis testing is required.

Combined analysis and testing can be utilized in qualification of over-large equipment by exciting equipment to SSE levels using analysis to perform the excitation, and validating the mathematical model for analysis by favorable comparison with low excitation test results. A second use of combined methods is in the qualification of equipment based on extrapolation of test results for similar equipment using analysis techniques. A third use, related to the second, is for extrapolation from test loads to a (different) required loading for the same equipment.

2.3.2 Regulatory Guides

The IEEE standards are endorsed by the NRC through the use of Regulatory Guides. These documents present the basis for the requirements (10 CFR and others) and then comment on the standard to be endorsed. Exceptions in the endorsement and additional criteria are presented. Exceptions and additional criteria are normally concerned with minor details. A partial list of the IEEE standards and the endorsing regulatory guides is shown in Table 1.

Some regulatory guides are designed to supply guidance on a particular issue and are not associated with any particular industry standard. Regulatory Guide 1.60[18] presents a generic ground response spectrum which may be utilized and has the advantage that it is easily defined compared to site specific spectra. Regulatory Guide 1.61[19] details conservative damping values to be used in design. The values are categorized by structure and stress level. Regulatory Guide 1.92[20] treats the combination of loads from different vibration modes.

2.3.3 Additional Input

The integration of ongoing research in the qualification process is achieved by guidance from professional papers and NRC supported publications. The NRC reports recent findings and recommendations which are used as the basis for the development of rules in the Code of Federal Regulations and are also used as a guide in the actual design and qualification process.

2.4 Qualification for Plants with Construction Permits

The current application of seismic qualification criteria for plants seeking operating licenses is a process of comparison and adaptation. Qualification must include proper conservative enveloping of design loads and boundary conditions as well as conservatively accounting for minor design differences within a component type to be both effective and affordable.

TABLE 1 INDUSTRY STANDARDS AND ASSOCIATED REGULATORY GUIDES

IEEE-334	Regulatory Guide 1.40[12]
IEEE-382[9]	Regulatory Guide 1.73[13]
IEEE-384[10]	Regulatory Guide 1.75[14]
IEEE-323	Regulatory Guide 1.89[15]
IEEE-344	Regulatory Guide 1.100[16]
ANSI N278.1[11]	Regulatory Guide 1.148[17]

2.4.1 Approach

Qualification is achieved through two basic approaches--analysis and testing. These methods are often combined for best results.

Analysis. Analysis is utilized most often to confirm structural integrity of a component or its support. Analytic models are utilized to represent a structure and the dynamic properties of the structure are derived. The design load is coupled with these properties and the response is determined. Typically qualification is then a matter of maximum stress determinations; although allowable deflections are also often a consideration, especially in equipment alignment or interference situations.

Another major use of analysis is for very large items. The two forms here are operability determination for equipment too large to test and load transfer characterization of building structures and large supports and mounts.

Testing. Testing is used at full load levels for direct qualification and at lower load levels for dynamic system characterization. Full level tests are almost always utilized to qualify components with operability requirements due to difficulties in analyzing this equipment type. Full load tests are also used to define the response of complex support systems such as electrical cabinets which cannot easily be modeled with sufficient accuracy.

Low level testing is used primarily for determining dynamic characteristics of a system or component and not for direct qualification itself. Often low level exploratory tests are conducted prior to high level testing to determine fundamental frequencies in the range of interest. Similarly low level testing, often in-situ, can be used to find mode shapes, frequencies and damping values for equipment qualified by analysis. Here the testing is utilized to verify the accuracy of the analytical model.

2.4.2 Load Types

A major factor in the present qualification process is proper determination of design loads. In equipment qualification the SSE magnitude is considered to be known but the actual loading seen by a component must be derived.

Information required to determine component loads are:

- o Loading seen by support system
- o Stiffness, damping and mass properties of the support system
- o Verification that design is adequate to maintain linear response during an SSE.
- o Potential sources of high frequency loads

In simple problems the effects of the last two items are often trivial.

Form. Two loading forms are commonly used in conjunction with analysis in the current qualification process. For many problems the response spectrum is used directly as the load model. In situations where the output of analysis is an output required response spectra a time history is required and is synthesized from the input required response spectrum. Currently a major use of time history analysis is the determination of large structure response An example is the modeling of a reactor building with a time history forcing function input at the foundations for determining the response of upper floors which are then converted into a floor response spectra.

Load forms for testing are of four types--static loads and three dynamic load forms; the simple waveform such as a sine wave, complex waveforms intended to represent a response spectra and the waveform produced by an impactive or explosive device.

Static load use in qualification is limited to components whose failure modes are structural. Thus a static force is applied and the component is examined for yielding or relative interferences. This form of testing is simple to apply but may only be used in special cases such as valve operator shaft clearance qualification.

Simple waveforms have been used for full level qualification testing under special conditions. If the design load is of a highly filtered type such as might be found for components supported by piping systems then a sine beat, sine dwell or decaying sine at the major frequency of the RRS may sufficiently envelope the magnitude and shape of the RRS.

Artificial time histories and other synthesized waveforms are the most utilized loadings for full level tests. The waveforms can be modified so as to produce a TRS with the basic shape of almost any RRS, no matter how skewed. A common method used to develop a complex artificial waveform is to submit a random multifrequency waveform or a group of decaying sine waves of different frequencies to a series of narrow band filters. These filters, spaced at 1/3 or 1/6 octave intervals, are used in shaping the resultant waveform so as to meet the RRS enveloping requirements while not producing an excessive ZPA.

Waveforms produced via impactive or explosive sources are utilized almost exclusively for low level loading in-situ to determine damping and transfer characteristics. Explosive charges are infrequently used, (primarily in research activities to excite a building) while instrumented impact hammers are used more often to excite smaller structures and components. An advantage of impact hammers and portable shakers is the physical incorporation of the actual mounting conditions.

Direction. The ideal qualifying load form would be applied in all directions simultaneously. This is now a technical possibility but in earlier years only single axis tests were possible.

In older practice the specimen is repeatedly tested at full level and rotated so as to expose all three axes to testing. IEEE 344-1975 states that single axis tests are only to be used when it can be demonstrated that no cross-axis coupling is present in the dynamic properties of the specimen. In biaxial tests if the inputs in the different axes are not independent, then rotations about vertical axis three times with 90° rotation each time are performed so as to examine both positive and negative inputs in one axis relative to positive input in another axis. Presently only two independent triaxial test tables exist. Triaxial machines may become more common mainly due to the ability to perform the biaxial test series without physical rotation of the specimen between tests, producing both a savings on table time and a consistent mounting stiffness. Actual triaxial tests have the asset of requiring only one full level test, thus reducing the possibility of fatigue failures; however extra effort is involved in developing three independent time histories which all produce enveloping TRSs. These time histories cannot necessarily be synthesized separately due to cross coupling in the test machine.

2.4.3 Test Types

Three types of full level equipment tests can be utilized for qualification. These types, proof, generic, and fragility, vary in philosophy and severity.

Proof testing is used to "prove" a component to be sufficient for a particular application. In this type of test a RRS is developed for an individual component to be mounted in a particular manner at a particular location in a plant. The proof test is most often used for a one-of-a-kind situation or equipment changeout.

The generic test is used to qualify a component type to a generic RRS. This component type can then be placed anywhere in the plant where the actual RRS is enveloped properly by the generic RRS. The generic test does require a particular mounting configuration and the individual components placed in the facility must be nearly identical to the one tested. The generic test is used often to qualify a large number of items simultaneously by choosing as the generic RRS the envelope of all the actual RRSs for the items.

The fragility test involves determination of the maximum loads a component type can withstand. There is no specific RRS for such a test. Specimens are tested at increasing loads until failure occurs. The fragility TRS is the maximum TRS that did not cause a failure for any particular mounting orientation and method. The application of this information involves determining the actual mounting method and RRS. If this RRS is enveloped by the fragility TRS for the particular mounting method, the component is qualified for the particular use. Fragility testing is expensive and not always definitive. The main use is by equipment vendors, who then can supply a "qualified" component to a utility with the utility's only effort being determination of the RRS.

2.4.4 Testing Equipment

There is a wide diversity of dynamic testing apparatus available for both in-situ and laboratory programs, the main qualifier for in-situ equipment being mobility. The main types of in-situ equipment include portable hydraulic or electromagnetic shakers and impact hammers. The hydraulic shaker is limited by its size and weight, which includes a reactive mass. The portable shakers can produce relatively large loads on small structures, potentially approaching full qualification levels. Most are capable of a wide range of waveforms including random time histories. The impact hammer consists of a mallet with an instrumented replaceable head. By using hammer heads with different stiffnesses , the waveform produced by the mallet impact can be modified for frequency content and relative magnitude.

The most common laboratory test machine is the independent biaxial shake table. There now exist two independent triaxial machines. When large deflections are needed a single axis long stroke machine may be used. A particularly heavy specimen may exceed the forcing ability of any of these dynamic simulators, just as an excessive RRS may not be duplicable. Most simulators can produce frequency content throughout the seismic range. One difficulty with lab tests for operability during an event is when extraneous supplies are required. An example would be water for a large pump qualification. When technically feasible testing is not possible, qualification must necessarily be established by analysis.

2.4.5 Procedure

Up to this point the main factors in the current qualification process have been noted. The consolidation of these factors into an actual qualification program is now discussed.

A typical procedure for qualification is outlined in Figure 3. Adoption of the best qualification method requires knowledge of the loads to be applied as well as the equipment response to be monitored. For many situations a multiple qualification method will be chosen based on this information. For example, a large electrical cabinet might be qualified in two steps, the main structure analyzed and the subcomponents tested, because the number of variables to be monitored are beyond the capacity of the laboratory equipment or the number of state changes (operability checks) to be verified would take an excessive amount of time.

Once a design's adequacy has been confirmed, a detailed documentation of all factors is needed. This documentation is required for licensing and also aids any modifications or retrofits proposed later in the system. Documentation should include not only the design loads but also the higher test loads. This will aid in requalification without retesting if more stringent criteria are implemented later.

As a part of the licensing process a seismic qualification review is conducted. At this time NRC contractors inspect the accuracy and scope of the qualification records. This review includes inspection of the installed configurations of components and their boundary conditions to verify that the correct situation was qualified.

All qualification programs should include maintenance and surveillance of the installed equipment during the years of plant operation. An ongoing record of component conditions is the most reliable method of excessive aging detection.

At present the requirements for maintaining qualification, as well as requalification, are not explicit. The development of procedures in these

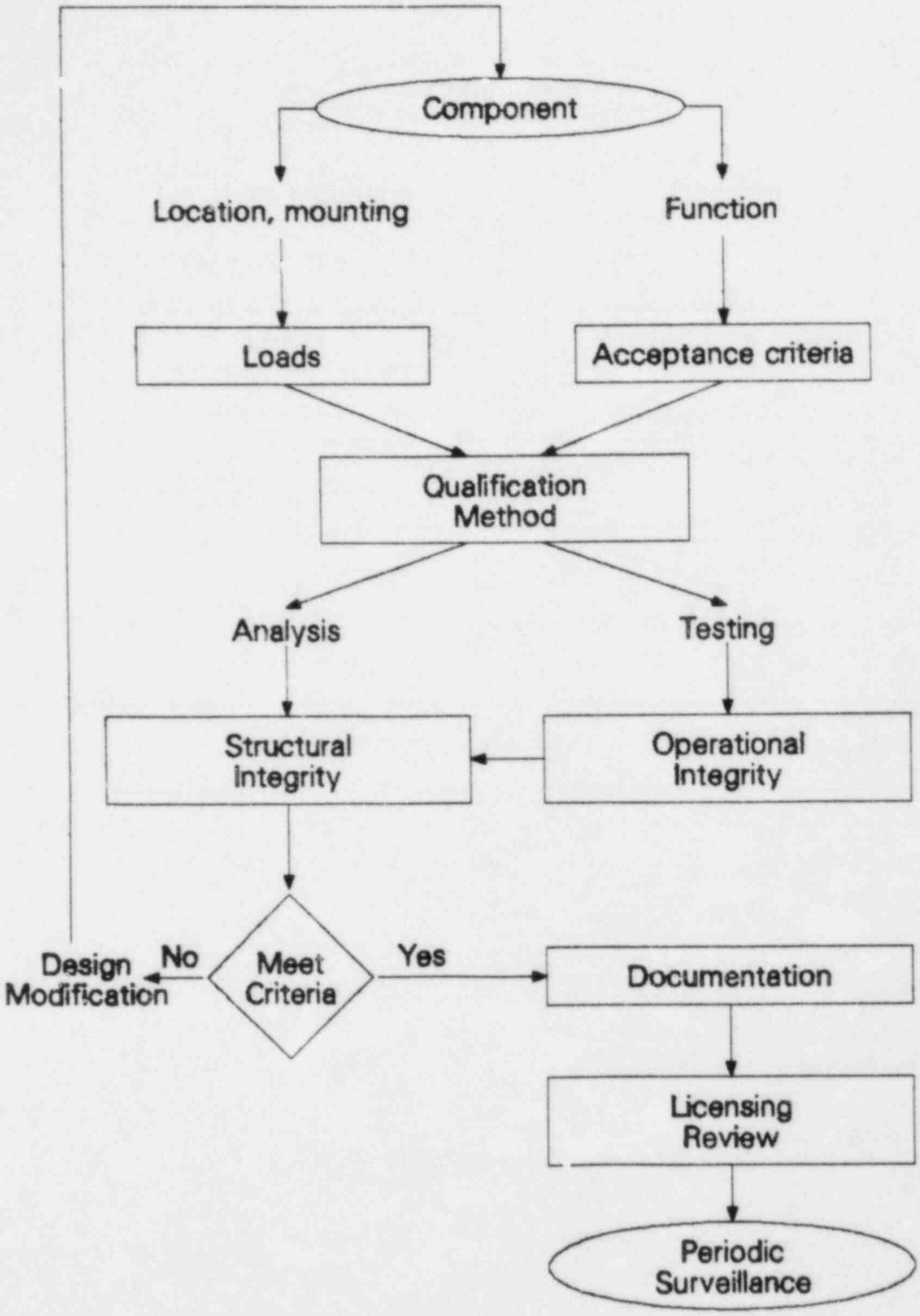

Figure 3. The qualification procedure.

areas implies a role for in-situ qualification methods. The next chapter examines some aspects of equipment qualification for operating plants and demonstrates the potential for in-situ tests in this process.

3. ASSESSMENT OF QUALIFICATION PROCEDURES FOR OPERATING PLANTS

3.1 Technical Inputs to USI-A46

One aspect of current NPP licensing requirements is to verify the design of active safety related equipment to the design basis seismic environment. Current practice is that operability of safety related equipment must be verified by testing when such testing is within the state-of-the-art. The testing chain for new plant equipment is very specific and was discussed in Section 2. If currently operating plants are required to demonstrate seismic performance via current criteria and procedures the cost impact will be large. Thus alternative approaches which can be used to satisfy the intent of equipment qualification are being examined in USI A-46.

Several studies currently in progress will provide information helpful to resolving qualification issues associated with operating plants. These include studies to examine the effect on plant probabilistic risk arising from changes in the qualification status and/or the seismic capacity of equipment. Other studies include evaluating the use of seismic experience in nonnuclear power plants to establish minimum seismic capacity levels. EG&G, Idaho is considering the manner in which in-situ procedures can be applied to equipment qualification. The first two studies may involve significant departures from the qualification chain described in Section 2.

The use of in-situ procedures is geared more toward a modification of current qualification procedures and criteria. Later in this section those applications are discussed in detail. Our considerations with in-situ testing have also addressed whether these procedures provide a basis for more diverse qualification criteria. No useful relationship was found. However the same investigation did identify an approach for estimating seismic capability based on similarity. As discussed in Section 3.3 this entails an analysis of specific equipment failure modes, test data, and similarity. The most immediate use would be for simpler types of equipment such as pumps or valves.

The goal in the present effort is to examine the most important uses of in-situ testing on the assumption that some level of substantial requalification of safety related equipment will be required. This assumption does not indicate a predisposition but rather an assumption from which to proceed.

3.2 In-Situ Testing Procedures

Alternate qualification procedures are sought which will yield procedures in lieu of shaker table qualification testing. One set of potential methods involves performing dynamic tests with equipment in-place in the plant.

In-situ test procedures could potentially be applied in the following techniques:

1. Testing at full load level with equipment in-place
2. Low load level testing, especially on equipment supporting structures which position and support safety related equipment.
3. Periodic intermediate or low load level testing to support a continuing surveillance data base.

Method 3 could in principle be useful for identifying aging degradation. However for the types of equipment of interest in this program no potential applications are apparent. This is because changes significant to operability of safety related equipment (particularly in a seismic environment) cannot generally be detected by in-situ procedures.

3.2.1 Full Level In-Situ Testing

This process allows self-standing qualification of a given component design. If it can be justified that no significant mechanical aging degradation has occurred during testing, then the component can be employed in service for its nominal useful lifetime. However full load level testing with equipment in place is not a developed technology.[21,22] Our literature review has uncovered no examples of this type of testing for the purpose of qualification. Dynamic testing has been proposed for commercial facilities but at less than design loads. The major goal is to validate computer models used in structural design. In fact, this type of testing has not been performed to date on a nuclear power plant in the United States. Evaluating operability in this type of test is useful but does not qualify equipment to design basis environments. It is also possible to consider removing equipment and testing this equipment on portable shaker units at full load levels. This appears to have little advantage over shipping the equipment to a testing laboratory for testing.

Some conditions may exist where it is possible to load the mounting position of a piece of equipment and result in a motion equivalent to the required response spectra. Required conditions are that

- o The support structure motion which occurs during the test must not excessively load the support device, appurtenances, or other components mounted on or in the vicinity of the support device

- o Sufficient access must exist in order to load the equipment mounting

- o No damage occurs local to areas where load is applied.

Again, no substantial mechanical aging should occur during testing. This is a special set of conditions which severely limits the usefulness of full load level in-situ tests. Valve operators are one equipment type that have been dynamically qualified in-situ by using a static load to perform an interference evaluation. However, the potential for performing full load level in-situ testing is so limited that it is not considered further in this report.

3.2.2 Low Load Level In-Situ Testing

Structural systems can be subjected to low level in-situ testing where small loads are applied to the structure. Typically the mechanical system is excited by a hammer, electromagnetic, or hydraulic type exciter. The input force and the output, normally acceleration, are recorded on a computer's memory as loads are applied at various positions. The recorded quantities are converted from time histories to a frequency representation by use of the Fourier transform. Using the frequency representation, transfer functions are calculated between the points of interests. These calculations are typically performed with minicomputers which are part of the modal analyzer system. Software internal to these computers then identifies natural frequencies and mode shapes. The mode shapes encompass points on the structure where data was recorded.

By combining the dynamic characteristics of a system with a load description the elements for predicting the dynamic response are complete. The dynamic characteristics of a linear structural system are its mode shapes, natural frequencies, mass distribution, and damping. In-situ procedures identify the natural frequencies and mode shapes. In certain cases the mass distribution can also be estimated (alternate methods for determining the mass distribution are discussed in Section 4). A characterization of viscous damping is also possible by using in-situ test data which represents the damping which actually occurred during the test. Since damping may depend on response level[23], values obtained from low level in-situ tests may not necessarily be valid and current NRC guidance should be followed. Thus the basic product of low load level in-situ procedures is a structural description. A final note is that the mass distribution, while represented in in-situ testing, is normally not directly available. Estimation procedures which use the results of in-situ testing have been suggested but have not been adequately justified.[a] Consequently a method is described in Section 4 which is not dependent on the in-situ measurements but is readily verified.

One basic use of low load level in-situ testing in operating plants will be to determine the required seismic capacity of equipment. That is, in establishing the required seismic capacity of equipment, equipment supporting structures and mountings. Even on the same floor of a plant the

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a. Part B describes a method for calculating the modal participation factors (i.e., the effective mass distribution) from the results of in-situ testing.

environment experienced by components varies from one equipment supporting structure to the next and from component to component within a equipment supporting structure. To determine the design basis environment for equipment the SSE floor motions are used as input to the equipment supporting structure. For new plants, shaker table testing is used to determine the environment for contained equipment. For operating plants the alternative is to use the modal parameters from in-situ procedures in determining the design basis environment. This environment, represented in the form of a RRS, is thus determined by a process where shaker table testing has been replaced by low level in-situ testing.

Several methods which use in-situ modal parameters are potentially available for determining RRS's. One approach is to develop a finite element computer model of the support system and mounted equipment. A computer program analyzes the modeled system and calculates the natural frequencies, important mode shapes, and modal participation factors (MPF's). These quantities are then used in determining the response of individual modes (see Section 4.1) which are superimposed to determine the total response. It is felt that the basic procedure is potentially unreliable because of system complexity and unreliability of boundary condition modeling. Consequently, it can only be used if the equipment is already installed and in-situ procedures are used to verify the calculated modal parameters. In practical application the finite element model may require modification to achieve acceptable correspondence with the results of in-situ procedures. A major disadvantage of the approach is that it is relatively costly because of the cost associated with developing a finite element model. An advantage is that if minor equipment modifications are made at a later date the model can be updated and a new set of RRS's calculated. The procedure is discussed in more detail in Section 4.2.

Above it was mentioned that in-situ procedures could be employed to help develop and verify the finite element model of a structure used in predicting RRS of devices located within the structure. It is also possible to develop an equivalent model by using in-situ modal parameters directly. In this type of model the mode shapes and frequencies used in calculations are those determined by in-situ procedures. As with the finite element approach, the responses of individual modes are calculated and then superimposed for the total response. No development of a finite element model is necessary thus substantially reducing the cost. The accuracy of this approach, as described in Section 4.3, depends on an accurate spacial resolution of the mode shapes. In practice, in-situ testing is accomplished rapidly (i.e., after initial set-up and pre-test activities) so that accurate spacial resolution of mode shapes does not substantially increase cost.

In the typical situation, equipment is mounted in a support device. A safety related system may consist of:

1. A support device which houses the safety related equipment

2. The anchorage of the support device to the building

3. The mounting of equipment to the support device

4. The equipment which must be qualified to operate

5. Various appurtenances which affect equipment operability.

Item 4 is the most basic qualification requirement. Once the design basis environment has been determined, the final qualification step consists of comparing this RRS with the seismic capacity of the existing equipment. The equipment's seismic capacity must be based on full load level testing. In-situ testing provides no help in this regard. Qualification tests of identical designs are the preferred type of data. There are some indications that much of the data may exist, scattered throughout the industry. Other forms of useful data include dynamic tests of very similar designs, as well as field experience during earthquakes. At any rate, an assessment of seismic capacity based on test experience is required to complete the qualification chain.

The other items above constitute a lesser share of the qualification burden. Certain considerations pertinent to items 1, 2, and 3 are discussed further in Section 3.6. Note that with any method of determining the RRS for equipment the acceleration throughout the structure will be available. This information should be useful in evaluating anchorage loads. The same statement is true of mountings. In qualifying equipment support structures structural integrity is the primary consideration. The commonly used models for stress analysis include beams and plates which employ rotational degrees of freedom. Currently, rotational degrees of freedom are not developed using in-situ procedures. However stress analysis using in-situ data is being investigated by several private organizations and methods may be proposed in the future.

Current qualification criteria stipulate that equipment can be qualified by using test data from equipment which is similar. To justify complete similarity (by using in-situ procedures) between several items of safety related equipment may be a complicated task requiring verification of similarity of many detailed features. However, it is foreseeable that situations will arise where it will be important to know that two structures or equipment items are dynamically similar. In these situations dynamic characteristics (natural frequencies and mode shapes) determined through the use of in-situ procedures can provide a basis for evaluating dynamic similarity.

The most important uses of in-situ testing have been discussed. In-situ procedures lend themselves toward situations where a substantial level of requalification is desireable (an exception is discussed in Section 3.6). These procedures can be used in predicting the required seismic capacity of a piece of equipment. The seismic capacity of the equipment must also be assessed using experimental data to complete the chain. Recapping, the recommended qualification strategy is to

- o Determine the dynamic characteristics from in-situ procedures
- o Complete the model required for analysis procedures
- o Subject the model to the input response spectra
- o Determine the equipment RRS by using analysis procedures

- o Evaluate the seismic capacity of the existing equipment
- o Evaluate adequacy by comparing seismic capacity and the RRS.

3.3 Alternate Qualification Criteria and Procedures

Qualification by test is the highest possible level of qualification. Such a level of qualification for all Class 1 equipment may not be appropriate for operating plants in view of a potentially low value/impact ratio. Thus consideration has been directed at defining alternate qualification criteria and procedures. An imposed ground rule has been that the intent of qualification as currently implemented by the NRC be maintained. This intent is interpreted as meaning that each safety related component be qualified to perform its safety related function for SSEs. This approach precludes the broader value/impact and probabilistic risk assessment avenues which could be used in developing alternate criteria. Thus the alternatives sought in this program are methods which can be applied at lower impact to equipment which must be qualified in some way.

As discussed in Section 3.2.2 the required seismic capacity of equipment in operating plants can be determined. The actual seismic capacity of equipment is the final link in the qualification chain. If test data specific to a given piece of equipment is not available then alternate methods for estimating seismic capacity will be beneficial. Alternatives based on in-situ testing have been considered. These considerations have revealed no applicable criteria or procedures. However another concept based on operability failure modes may be a useful basis for alternate criteria under certain circumstances.

3.3.1 Failure Modes

With the substantial qualification testing which has occurred in the recent past, the evaluation of seismic capacity using test data from similar equipment may be feasible. To develop such methods a systematic treatment of operability and inoperability is necessary. The failure modes which result in inoperability are an essential ingredient to these methods. In this section, operability and inoperability are defined. The failure modes which cause inoperability are defined and discussed. Since this categorization is new it should be critically reviewed. The procedure for conservatively estimating seismic capacity is discussed along with the circumstances which facilitate its use.

Operability Failures. Operability failures are defined here as any action or interacting of component parts or interfaces which prevent a component from performing an active operation or maintaining a state continuously. Equipment with operability requirements are distinguished by the need for a controlled state:

- Parameters are monitored which are coupled with the equipment state
- State change is initiated when the parameter enters or exits a preset range
- The state transition must occur within applicable performance limits.

Inoperability can result from:

- Inability to monitor the control parameter
- Inability to change states when so directed by the monitor
- Inability to maintain the current state when no state change is directed.

It is suggested that inoperability during dynamic environments occurs through the following failure modes:

a. Structural integrity-stress limits are exceeded, permanent deformation occurs, flaw initiation or extension occurs.

b. Operability loss due to temporary or permanent reconfiguration-vibratory elastic motion results in a state change or prevents a state change from occurring.

c. Structural interference-excessive relative motion results in a tolerance mismatch.

d. Nonstructural changes in state-piezoelectric effects, effects of dynamics on contact resistance, and others. Anywhere a fundamental nonstructural response is affected by vibration or stress.

Violation of structural integrity yields a system which is measurably changed as a result of the dynamic environment. Its ability to maintain or to change state are no longer assured. Loss of separation is also a potential consequence. Aging degradation can impact structural integrity when susceptable subcomponents exist along load paths. Dimensional changes resulting from aging are a consideration if they can affect operability. In many systems qualification testing has demonstrated that structural integrity is not an active failure mode.

If structural integrity is eliminated as a failure mode then permanent structural reconfiguration can only occur if some portion of the design is inherently unstable to large deflections, or "unstable in the large." For example see Figure 4 which shows a switch contact which is inherently unstable in the large because excessive relative motion causes a loss of restoring force. Temporary reconfiguration is a potential failure mode if the equipment has a safety function during the earthquake. This is the situation where vibratory motion results in a change of state. The prototypical example is a switch inadvertently breaking or making contact. This failure mode is certainly the most complicated of the modes. The design aspects controlling the configuration during dynamic events must be evaluated thoroughly to justify using test data from non-identical equipment. Of course, if the equipment is not active during the seismic environment then temporary reconfiguration is not an issue and qualification is more readily achieved.

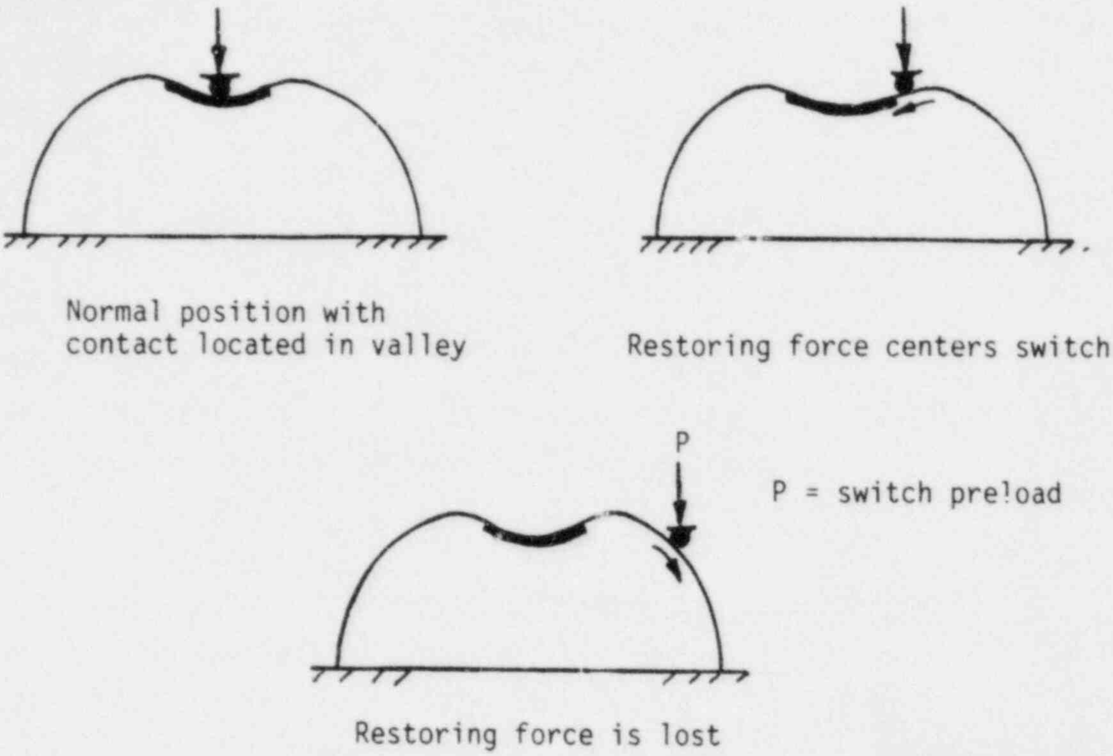

Figure 4. Instability in the large.

In the absence of structure integrity failures structural interference is a mechanical mode of failure and can exist only during the seismic environment . Structural interference is of particular importance in valves, valve operators, and rotating equipment. Structural interference could for example seize an operating motor or prevent a valve operator from functioning on demand. This qualification is often performed by analysis. Identifying the design features controlling interference is the crucial step to establishing equivalence between two pieces of equipment in this failure mode.

Many safety related components employ nonstructural phenomena, perhaps electromagnetic, in their basic operation. Nonstructural failure modes occur when motion or stress affects a basic operability function. For example contact resistance in degraded contacts can be increased by vibratory reduction of preload across contacts. Piezo-electric devices are affected by stress. These types of effects must be considered in evaluating the seismic capacity of equipment. If their effect can be significant then equipment similarity is based partially on similarity in these non-structural phenomena.

3.3.2 Alternate Criteria Based on Failure Modes

Alternate qualification criteria based on similarity of seismic capacity can now be considered. The four failure modes described earlier are the starting point for these alternate criteria. By justifying qualification in each mode total qualification is justified. Similarity between two equipment designs can be defined as similarity in potential failure modes. The basic premise involves two pieces of non-identical equipment having a common critical failure mode. The first piece has been qualification proof tested and its controlling design features are either identically or inherently more fragile than the equipment in question. In that case qualifying the first amounts to qualifying the other to the same environment. This process is facilitated if the equipment being compared have strong physical similarity in the design features which control failure and seismic capacity.

The following procedure is suggested for establishing seismic capacity based on similarity:

- o Specify operability requirements: take into account whether equipment is required to operate and/or maintain a continuous state during earthquakes. If there are no requirements during the earthquake then certain failure modes will be eliminated and qualification is simplified.

- o Identify the design features/subcomponents which affect operability. The procedure will be impractical if there are to many.

- o Identify potential failure modes and the critical failure mode if possible. Past qualification testing of other equipment will in many cases facilitate identification of the critical failure mode. Analysis procedures or a design review may also be useful in this regard.

- o Identify similar pieces of equipment, i.e., equipment with nominally the same or less seismic capacity in the potential failure mode(s). Some form of design evaluation/comparison will be required in making this assessment. Equipment used for comparison must be of known seismic capacity.

These pieces of equipment are similar because they have the same failure mode and because a design evaluation has shown that the seismic capacities are related. Now the seismic capacity of the equipment in question is conservatively taken to be that of the similar article.

Clearly the design evaluation and similarity analysis described above will not always be practical. If two pieces of equipment are nearly identical in all features affecting operability then establishing similarity may be practical for moderately complicated systems. However the most potential exists with simple systems where operability is a simple process and failure modes are readily identified. Another assest is a large qualified seismic capacity. In this case equipment and tests useful for comparison are more readily identified and justified. Finally, it will be helpful if the equipment belongs to an equipment group which has been extensively tested or analyzed.

Examining the application of this process to any specific equipment type is beyond the program scope. However, application to equipment such as pumps, valves, and motors appears to be one practical option. Identifying classes of equipment which are inherently hard seismicly and therefore requiring minimal qualification is another potential application. The methodology may be useful in conjunction with the Seismic Qualification Utilities Group program[24] by providing a formal design control measure.[1] This program is gathering nonnuclear power plant service experience data during seismic events. Finally, it is foreseeable that rationalizing seismic capacity will have benefits in both seismic (and other) qualification and design of equipment.

3.4 Environmental Aging Consideration

The environmental history of a piece of equipment can produce changes in properties and dimensions which affect its seismic capacity. An assessment of all potential property changes and the integration of property variances in equipment dynamic capacity is a part of the current NPP qualification process.[25] Addressing the total environmental qualification of equipment in operating plants is impractical. An approach based on the interaction of aging with dynamic capacity is adopted here. Such an approach suggests that since some aging mechanisms will not affect seismic capacity these cases need not be considered in seismic qualification.

The use of in-situ testing in evaluating the affects of aging on seismic qualification has been considered. However no well developed technologies were identified. Consequently aging has been examined in a broader context where:

- o The consequences of aging degradation are examined. This allows the relationship between dynamic qualification and aging degradation to be organized in a fashion which more clearly demonstrates the interaction.

- o Alternate criteria based on the failure mode and similarity analysis of the last section are discussed. This provides both an organized aging assessment procedure and a method for using test data from "similar" equipment.

- o Equipment without specific operability requirements during seismic events have been identified as less vulnerable to aging.

3.4.1 The Effect of Aging on Seismic Capacity

The effect of aging on seismic capacity is illustrated in Figure 5 First, if it can be demonstrated that no significant aging can occur then no potential problem exists. Routine maintenance programs, where subcomponents susceptable to aging are replaced and can be examined, and in-service experience (earthquake experience) can provide a data base for this assessment. For components where environmental aging is anticipated, the first branch (Figure 5) depends upon whether or not the dynamic response is affected.

Situations where the dynamic response is affected by aging will be discussed first. For operating plant equipment the observation of an interaction is based on reviewing equipment design and finding that aging degradation exists on an active load path. Inadequate seismic design cannot be discounted. Since every failure mode may be affected, the condition is potentially serious. If the effect on seismic capacity cannot be shown to be benign or supported by test data on similar systems then qualification to current criteria is recommended. However the dynamic response of many components can be shown to be unaffected by aging degradation and thus the problem may arise infrequently.

If dynamic response is shown to be unaffected by expected environmental aging then the remaining branch in Figure 5 applies. Inoperability results directly from non-structural aging degradation. It is assumed that degradation has not been so extensive as to render this equipment inoperative in normal environments. This level of degradation should be addressed by routine in-service surveillance. If structural integrity can be assured, operability after the event is also assured. However it is necessary to qualify that any potential degradation effect was temporary and associated with the dynamic response. At this point such an assumption seems reasonable. If this form of degradation is anticipated and the equipment has a safety function during the seismic event, then a more thorough evaluation is required.

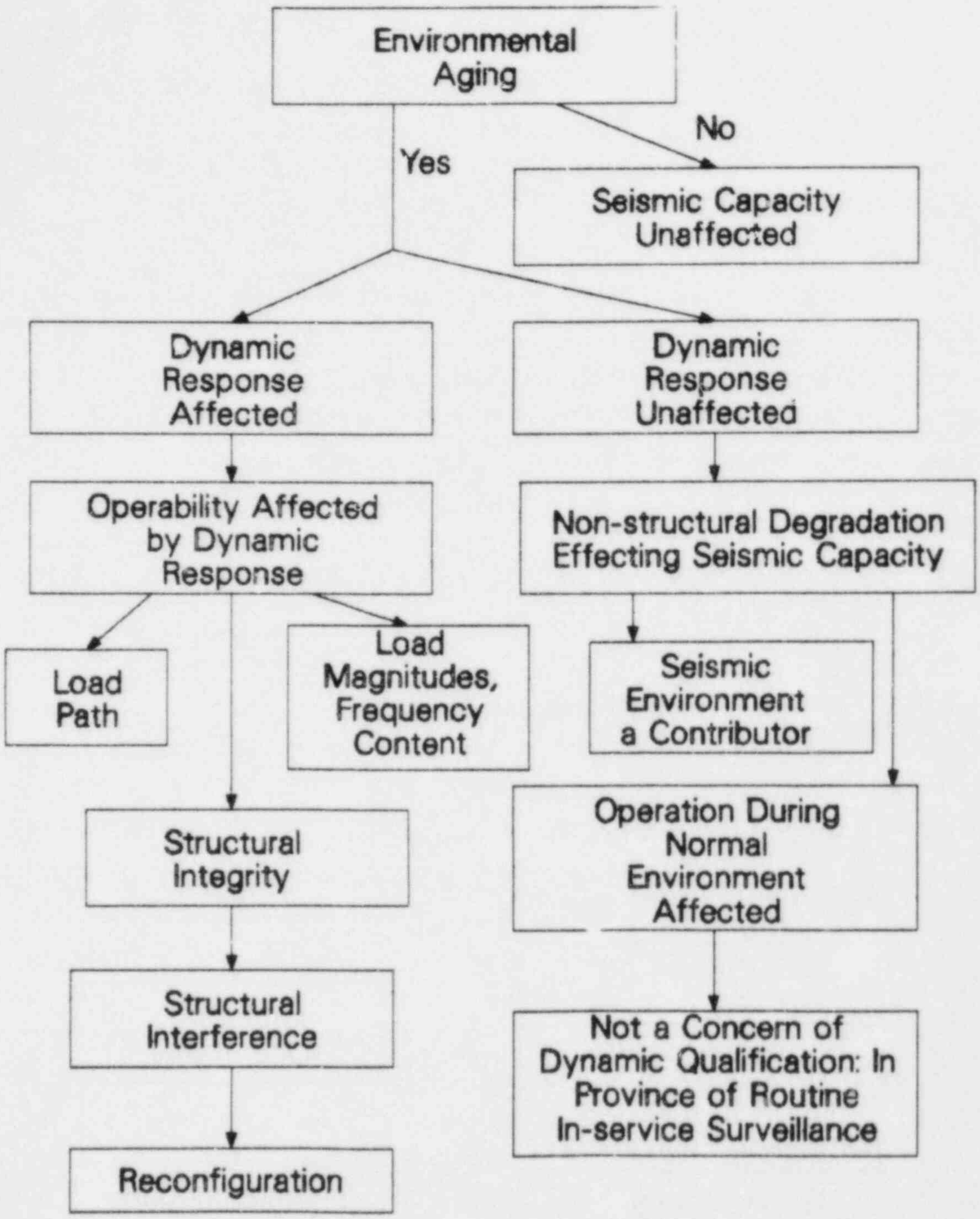

Figure 5. Effect of aging on seismic capacity

3.4.2 Qualification Considerations

A systematic basis for evaluating aging degradation is provided by the failure mode analysis of Section 3.3 and the procedures embodied in Figure 5. Again this methodology will be most readily applicable to simple equipment. The method is now discussed.

First, a determination of any aging effects produced by the design basis environments should be conducted. This involves listing all vulnerable materials and examining environmental data for each. Presently such data is only available for some materials. Those components demonstrating no environmental aging require no further examination.

For components containing materials affected by the design environments the aging mechanisms should be defined and categorized as follows:

- o Category I aging includes all aging mechanisms which modify the dynamic response. The changes in dynamic response can affect all four failure modes: structural integrity, system reconfiguration, structural interference, and nonstructural effects. Each failure mode must be examined in light of the anticipated degradation. If it cannot be established that no significant change in seismic capacity occurs then the critical failure mode(s) should be established. A similar system with a known aged seismic capacity may provide data on which to base the aged seismic capacity. Realistically, equipment designed for dynamic environments should not be susceptible to this type of aging and the problem may be infrequent. Otherwise, adversely affected items should be qualified to current criteria.

- o Category II aging is any aging mechanism which could affect the operability of safety equipment when combined with the predicted seismic loads. It is assumed that the dynamic response has not been affected. This is a type of aging mechanism which impacts only the nonstructural effects. It need only be examined if a known aging effect exists in a component. Again seismic capacity can be inferred from tests on similar equipment. However the requirements on similarity are somewhat more stringent in this case. Any loss of seismic capacity will be due to degradation combined with local structural dynamics. Thus similarity requires that both be simulated.

- Category III aging mechanisms are those identified mechanisms which have no effect on seismic qualification.[26] For a typical component many mechanisms would typically fall in Category III.

The application of the above approach would probably be most economical if conducted in stages. Initially all equipment would have a cursory examination for a) no aging, b) some aging, though with no effect on seismic capacity, c) aging with a potential effect on seismic capacity, or d) too complex to determine easily. For situations where further consideration is warranted the steps are similar to those of Section 3.3 The failure modes are used to establish similarity and data from similar equipment is transferred to the equipment in question. The important factor is that much equipment will exhibit no significant seismic aging interaction of concern and thus screening can narrow the field effectively without overlooking substantial aging degradation.

Currently, limited qualification research is being conducted in the Category III aging effeccs.[27] The expected future result of this effort is the identification of a Class 1E equipment subclass showing no seismic aging interaction. Such preliminary work will develop a data base also useful for qualifying equipment in operating plants.

Finally, a specific and potentially useful class of equipment can be identified. This is the equipment which has no safety related function during the seismic event. If structural integrity for the earthquake environment is validated then one can be reasonably assured it will operate after a SSE. Minor checks on the adequacy of design for permanent reconfiguration and dynamic effects on nonstructural aging degradation are required. These should be straight forward if equipment is not overly complicated.

3.5 Equipment Supporting Structure Response and Mountings

The level of equipment supporting structure response during a seismic event can be related to the corresponding floor response spectra. The design floor response spectra will generally contain a region with significantly amplified magnitude. The center of this amplified region will generally lie between 2 and 10 hertz and coincides with the fundamental frequency of the building. The motion of the equipment supporting structure is reckoned as a combination of its free vibration modes whose maximum values are determined from the floor response spectra. Generally the first mode has the largest modal participation factor (MPF) and is the most important. Knowing the first mode frequency and its MPF the maximum response is estimated readily from the floor response spectra.

Tuning of the equipment supporting structure and the building containing the equipment occur when a natural modal frequency of the equipment supporting structure coincides with the fundamental building modal frequency. As an example, cabinet frequencies between 5-15 hertz are typical so that tuning is possible. In case tuning occurs the floor response spectra dictates a response level 2-5 times a non-tuned response. A complicating factor is that the lowest natural frequency of a equipment support structure depends upon how it is attached to the floor as well as its physical properties. For instance a welded mounting will result in a higher frequency than a mounting with a minimum number of bolts. Thus for operating plants uncertainties relating to equipment supporting structures include both physical properties and the mounting boundary condition.

Hence equipment design environment will depend heavily on the relationship between support device and building fundamental frequencies. It is clear that most of the safety related systems were not intentionally designed to function in highly amplified dynamic environments (i.e., tuned conditions). Systems that may be subject to these loads should be identified by in-situ procedures. Here an abbreviated process can be followed where all support device natural frequencies below 15 hertz are experimentally determined. Mode shape determination is not required. A modal analysis crew should be able to check a number of cabinets in a single day so cost is not an overwhelming burden. Currently operating plants are mainly located in regions of low seismicity and this utilization of in-situ procedures insures that actual response loads are as low as generally perceived.

Where amplified equipment supporting structure response is identified two options are recommended. Regardless of the criteria applied to other equipment in operating plants, this equipment should be qualified vigorously. The first option is to determine the design basis environment (see Sections 3.2 and 4.2) and qualify equipment to that environment. The second option is to modify the support device by either altering its mounting or stiffening the device, depending upon which is appropriate. That a lower response is assured should be verified by in-situ procedures.

If one analyzes an equipment supporting structure, verifies its structural integrity, and provides evidence that all mounted components have seismic resistance exceeding their RRSs, it still remains to qualify the mounting design. Review of proprietary qualification documents indicated that mounting inadequacy has been a major cause of retrofit and retest in qualification programs. The current qualification process essentially qualifies mountings during shake table testing. For operating plants several options are available. Analysis procedures using data from in-situ testing can predict the maximum acceleration of equipment. Thus the loads that mountings must transmit can be predicted. It should be a straight forward process to assess existing designs. The main distraction is the large number of mountings that exist. Enveloping the maximum acceleration could be an approach to reducing this work load.

Examining mountings on a theoretical basis may not address some (perhaps the major) problems. There is some feeling that quality of installation or use of problem prone designs may be a stronger influence on mounting adequacy than strength considerations. To address these concerns a physical mounting review by practitioners experienced in seismic qualification testing as well as current mounting design practice would be an effective design control measure. This process would be enhanced if the reviewers were supplied with an equipment table identifying an enveloping acceleration, equipment weight, and a simple description of the mounting. The plant walk-down would then screen mountings for those requiring in-depth review or retrofit. The effectiveness of this process is that it screens out items which are clearly adequate and concentrates more costly review on questionable items.

4. ANALYSIS PROCEDURES

It has appeared reasonable that knowledge of a structure's linear dynamic characteristics along with a dynamic input description are sufficient to define the resulting environment anywhere in the structure. Toward this end, a review of analysis procedures has found that several procedures can be used. Part of the dynamic description required can be determined from in-situ procedures, the natural frequencies and the mode shapes. Damping is a third dynamic property and should be based on current NRC guidance. The dynamic input is reckoned via the ground or floor response spectra. The methods for using in-situ generated dynamic characteristics in determining response are described in this section.

The primary purpose of these procedures is to develop response spectra within support devices. The predicted response spectra then act as the required response spectra for component qualification. The analysis procedures can be divided into methods which use the parameters determined by in-situ procedures directly in the analysis, and methods which use the in-situ results to validate a computer model.

4.1 Basic Theory

It is assumed that all structures transmitting inputs act linearly. The structure is considered as an "n" degree of freedom system and represented by matrix equations as:

$$[M]\{\ddot{X}_r + \ddot{Y}_b\} + [K]\{X_r\} = 0 \qquad (1)$$

or

$$[M]\{\ddot{X}_r\} + [K]\{X_r\} = -[M]\{\ddot{Y}_b\} = -\{M_i\}\ddot{Y}_b \qquad (2)$$

Damping will be ignored in these developments. However it can be incorporated into the modal equations of motion at any convenient time. To keep the equations as simple as possible they are written for a system experiencing earthquake base motion in one direction and which has sufficient physical symmetry to respond only in that direction.

$[M]$ = n x n diagonal mass matrix

$[K]$ = n x n stiffness matrix

$-\{M_i\}\ddot{Y}_b$ = n x 1 load vector due to base motion

$\{X_r\}$ = n x 1 solution vector.

Next the use of the modes of free vibration is incorporated.

$$\{X_r\} = [\phi]\{\alpha\} \tag{3}$$

$[\phi]$ = consists of columns of free vibration modes; $\{\phi\}_1, \ldots \{\phi\}_n$

$\{\alpha\}$ = consists of 'n' time varying functions $\alpha_i(t)$.

The free modes of vibration satisfy several relationships including

$$[\phi]^T[M][\phi] = [I]$$

$$[\phi]^T[K][\phi] = [\omega_n^2] \tag{4}$$

Now by using Equation (3) above in Equation (2) and premultiplying bv $[\phi]^T$ we have

$$[\phi]^T[M][\phi]\{\ddot{\alpha}\} + [\phi]^T[K][\phi]\{\alpha\} = -[\phi]^T\{M_i\}\ \ddot{Y}_b \tag{5}$$

Because of the diagonal nature of the matrices in Equation (4) we see the equations in Equation (4) are effectively uncoupled.

$$\{\ddot{\alpha}\} + [\omega_n^2]\{\alpha\} = -[\phi]^T\{M_i\}\ddot{Y}_b \tag{6}$$

The quantity $[\phi]^T\{M_i\}$ is a constant vector $\{\Gamma_i\}$ with

$$\Gamma_i = \{\phi\}^T{}_i\{M_i\} \tag{7}$$

and is called the modal participation factor.

Thus for a given mode "i" we have

$$\ddot{\alpha}_i + \omega_i^2 \alpha_i = -\{\phi\}_i^T\{M_i\}\ddot{Y}_b \quad . \tag{8}$$

The maximum values for α_i can then be interpreted from the ground response spectra. The ground response spectra provides the solution for the equation

$$\ddot{g} + \omega^2 g = -\ddot{Y}_b \tag{9}$$

for a specified range in ω. By identifying the ground response spectra value at structural (i.e., free vibration) frequencies and multiplying by the modal participation factor (MPF) it is evident that the solution to Equation (6) has been determined. One proceeds on this basis for all "n" structural modes, finding the maximum values of $\ddot{\alpha}_i$ and α_i. Now,

$$\alpha_i = \Gamma_i \; g_i(t)$$

$$(\alpha_i)_{max} = \Gamma_i \; (g_i)_{max}$$

$$\Gamma_i \; (\ddot{g}_i + \omega_i^2 \; g_i = -\ddot{Y}b)$$

$$\Gamma_i \; \ddot{g}_i + \omega_i^2 \; g_i \; \Gamma_i = -\Gamma_i \ddot{Y}_b$$

and $\ddot{\alpha}_i + \omega_i^2 \, \alpha_i = -\Gamma_i \ddot{Y}_b$

so the desired equation is recovered. The final step concerns combination of the modes. For modes whose motion is statistically independent of one another the "square-root-sum of the squares" (SRSS) is used to determine maximum values. These values are called "most likely maximum values" and are purported to have that statistical property.

Consequently one can see the natural correspondence of the ground response spectra with the structure's equations of motion when they are rewritten in the modal degrees of freedom.

4.2 Model Validation

The response of equipment supporting structures during design basis dynamic events is central to equipment qualification because a large portion of the equipment qualified by test is mounted in these devices. Furthermore each equipment supporting structure may contain many pieces of equipment. While it is possible to estimate the dynamic response of these systems using computer models this procedure has not been widely used for equipment qualification. It has been considered that the most reliable procedure is to subject the support device system to testing thus simulating design basis events. The support device may contain instrumented masses instead of components in which case the required qualification environment is recorded or it may contain prototype components in which case the entire system is qualified.

Specific in-plant situations have occurred where some feature of the installation was not compatible with the qualification testing performed. In some of these situations finite element analysis has been performed to predict the dynamic response during a design basis event.[28] To validate the adequacy of the computer model in-situ tests are performed which identify the fundamental natural frequencies and associated mode shapes. The experimentally based parameters are compared with the same parameters computed from the model. If required, the model and its boundary conditions are adjusted until an adequate correspondence is achieved. The final computer model is used to determine both the RRS at specific points in the support device and stresses within the support device.

The analysis procedures involved here are those of the typical time history method. In this process, 1) a synthetic time history is developed from a specified floor response, 2) the modes, frequencies, and modal participation factors are calculated from the model, 3) a time history analysis is performed on each significant mode, 4) the modes are algebraically combined to determine total time histories, and 5) the time histories are converted to RRS for the components of interest. This process requires the development of a finite element model which in the writer's assessment is the dominant expense in the process. This process can be directly applied to equipment in operating plants. It has the advantage that once a finite element model is developed and validated, this same model can be used to evaluate the qualification of future changes to the system. Reiterating, the use of in-situ procedures is to validate a finite element structural model.

4.3 Analysis Using Modal Parameters Directly

It is possible to perform analyses yielding support device motion and RRSs without developing a finite element model. A note of caution is that no detailed theoretical discussion or case studies have been found in the literature. However the writer knows of several organizations currently active in developing methodology. The process involves using the frequencies and mode shapes determined from in-situ procedures directly in constructing a numerical solution. By contrast in Section 4.2 these parameters were determined from a finite element computer model. Analysis procedures based on the direct use of modal parameters is now discussed.

As a starting point refer to the linear equations of motion (damping neglected) written using the free vibration mode shapes and frequencies.

$$\{\ddot{\alpha}_n\} + [\omega_n{}^2]\,\{\alpha_n\} \equiv -[\phi]^T\{m_i\}\ddot{Y}_b(t)\ ,$$

$$\{X_r\} = [\phi]\,\{\alpha\}$$

$$\{Y\} = \{X_r\} + Y_b\,\{1\} \tag{10}$$

These equations are (3) and (6) repeated from Section 4.1. Note that equation (8) for a particular mode is

$$\ddot{\alpha}_i + \omega_i^2\ \alpha_i = -(\Sigma\phi_{ji}M_j)\ddot{Y}_b = -\Gamma_i\ddot{Y}_b$$

To completely specify this equation (the equation for the "i^{th}" mode) it is necessary to know the natural frequency, mode shape, and the modal participation factor for the "i^{th}" mode. Then, since Y_b is known, a time history analysis can be performed to determine α_i (t`. Once the time histories of all significant modes have been calculated then equations (6) and (10) are used to construct the complete response.

To proceed, it is assumed that in-situ testing procedures have identified a given set of mode shapes and frequencies accurately. The number of points (refered to as 'n') at which measurements were taken is of central importance. It represents the number of points used in describing a mode shape, the maximum number of natural frequencies, and the maximum number of mode shapes which can be determined from a particular test. In situations where highly resolved (large 'n') mode shapes are sought it is not practical to determine all 'n' mode shapes and frequencies. Thus an incomplete set of accurately known modal parameters is determined from in-situ testing. This set is quite adequate provided it contains all significant modes.

The final step required is to determine the MPF's for the significant modes. If a complete set of accurate modes were available the MPF's could be determined directly using the complete modal matrix. The procedure is discussed later in this section. For the situation in which an incomplete set of modes is known, the writer is aware of several proposed schemes [29,30,31] for estimating the MPF's. Currently these procedures are proposed resolutions whose limitations and validity have not been verified. Thus it is not possible to recommend their use today.

Fortunately a method of determining modal masses and MPF's is available and is developed below. This method estimates the MPF to the same accuracy level as the mode shapes. Although the method is straight forward it has not been previously suggested in the literature. Consider the equation for the exact modal participation factor[a] based on the exact continuous "i^{th}" mode shape where a one dimensional system is considered for simplicity.

$$(MPF)_i = \int_a^b \phi_i(s) \frac{dm(s)}{ds} ds \tag{11}$$

s = independent coordinate

m(s) = mass distribution along coordinate "s"

$\phi_i(s)$ = continuous mode shape.

The quantity dm/ds is evaluated from the actual, existing mass distribution and thus can be evaluated to any desired degree of accuracy. Since the mode shape is estimated at discrete points, the approximations in (11) are inherently governed by the estimate of $\phi_i(s)$.

The discrete approximation to (11) is

$$(MPF)_i \simeq \sum_{j=1}^{n} \phi_{ji} \Delta M_j \tag{12}$$

$$WM_j = \frac{\int_a^b \phi_i(s) \frac{dm(s)}{ds} ds}{\phi_{ji}} \tag{13}$$

The equations above clearly indicate the modal participation factor can be more accurately predicted by increased resolution of the discrete mode

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a. If the modes are not mass-orthonormalized then equations (11) and (12) must be modified by the factor

$$\left[\int_o^s \phi_i^2(s) \frac{dm(s)}{ds} ds^{-1}\right]$$

shape. Equation (13) also shows that if $\phi_i(s)$ is relatively constant over a span then ΔM_j will be nearly the mass in the span. Estimating the continuous mode shape allows for calculating ΔM_j directly from equation (13). Note that generally it will not be precisely the mass in the interval. This is one recommended procedure for calculating nodal masses and the MPF. It is recommended because it is theoretically sound and verifiable, it does not penalize accurate description of modes, and it can be performed in a straightforward fashion. A minor drawback is that the distribution of mass in the system must be described. Figure 6 illustrates the flow diagram for the proposed analysis procedure.

A method has also been proposed (see Reference 31) in which a complete set of modes is always generated. This is accomplished by using a number of nodes equal to the number of significant modes from which the solution will be constructed. In this case it is possible to invert the pseudo-modal matrix and predict the pseudo-MPF factor directly as follows (the word "pseudo" is used to identify quantities which are not mass-orthonormalized)

$$[\psi]^T[M][\psi]\{\ddot{q}\} + [\psi]^T[K][\psi]\{q\} = -[\psi]^T[M]\ \{1\}\ \ddot{Y}_b$$

and $[\psi]$ = psuedo-modes, i.e., modes which have not been mass-orthonormalized

$$[\psi]^T[M][\psi] = [M_e]$$

$$[\psi]^T[M][\psi] = [M_e]\ [\omega_n^2]$$

and $\{\ddot{q}\} + [\omega_n^2]\{q\} = -[M_e]^{-1}[\psi]^T[M]\{1\}\ \ddot{Y}_b = -\Gamma_i^*\ddot{Y}_b$

and $[M_e][\psi]^T[M]\{1\} = [\psi]^{-1}\{1\}$

which is the psuedo-modal participation factor and can be readily determined. While this process is very straightforward it employs a relatively crude discretization of the system. The limitations and conditions where it can be accurately employed have not been determined, and it also cannot be recommended at the current time.

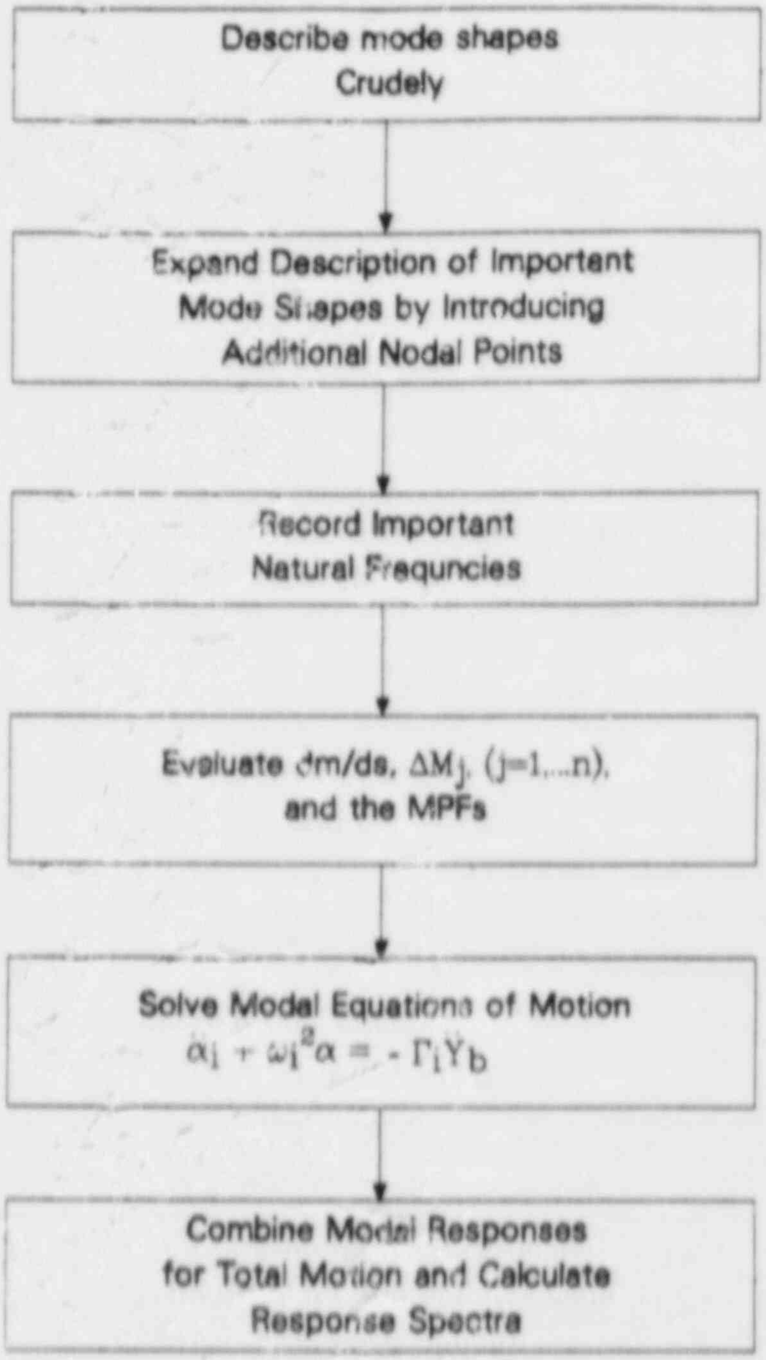

Figure 6. Proposed analysis procedure

Finally there are several notes of caution to be mentioned. For seismic analysis it is felt that higher modes, or modes with several antinodes will result in low or negligible MPFs. Consequently accurate calculation of only the lower mode shapes will probably be necessary. The situation must be checked for every individual case. The second comment concerns closely spaced modes. The decomposition of the total frequency response into modal frequency response functions is one step in the development of the mode shapes. Closely spaced mode shapes (i.e., two modes with nearly equal frequencies) increase the difficulty and reduce the accuracy with which the modal frequency response functions are calculated from the experimental transfer functions. It is anticipated that this situation will occur infrequently in which case the alternative of Section 4.2 can be used to determine RRSs.

A final comment is that the advantage of the direct use of modal parameters is that the modal parameters are relatively inexpensive to generate experimentally. Generation of modal parameters by the finite element method will require substantially more cost. A consideration in the total cost of developing a finite element model is the cost for validating the model accuracy using in-situ procedures. Consequently analysis procedures which use experimentally determined modal parameters are the prime candidate for predicting RRSs in operating plants.

4.4 Direct Response Spectra Transfer

Sections 4.2 and 4.3 discussed several procedures for predicting the RRS of equipment located in support devices. Both of the procedures employed variations of time history analysis where a synthetic time history is used to define the load. Using these procedures an input response spectra can be transferred to an output location yielding an output response spectra. Since the input is initially specified by a response spectra, the use of time history analysis in transferring response spectra is essentially artificial and the output response spectra is not uniquely defined by the input spectra. Methods for transferring the input response spectra in a unique, more meaningful, and less costly way are preferable.

Direct methods for response spectra transfer have been sought by various investigators.[32--38] A direct method uses the input or floor response spectra in combination with the modal parameters and modal participation factor to determine output response spectra. The associated analytic procedures are algebraic. The initial motivation for developing these methods was to reduce the effort involved in generating floor response spectra for buildings. Any direct methods will eliminate the time history analysis portions of the transfer process. In addition by using mode shapes and frequencies determined from in-situ procedures the need for a finite element model can be eliminated, yielding a very cost effective method. However, more recently another equally important motivation has arisen.

Response spectra transferred by the time history method are dependent on the synthetic time history used as base input. Ideally the transferred response spectra would depend only upon the input response spectra and the dynamic characteristics (mode shapes, natural frequencies, damping, and MPFs) of the system. But large variations have been reported when transferring spectra consistent synthetic base time histories. The variations, or response spectra dispersion, are an inherent aspect of the time history process. The large variation possible in the amplified region of the response spectra is an inherent weakness of the time history method of transfer. A direct method of transfer, identifying a consistent or average transferred spectra would eliminate the arbitariness associated with time history transfer.

Some aspects of response specta transfer are presented in more detail in Appendix A. Two distinct modes of dispersion, i.e., the features by which the transferred response spectra become non-unique, seem to exist. In areas where the spectral frequency is not near one of the structures natural frequencies, Equation A.13 shows the dispersion is a result of arbitary modal combination. The SRSS rule for modal combination allows the prediction of a "most likely response spectra" as in Equation A.13. Thus the correct transfer in these areas of the response spectra curve is resolved. In areas where the spectral frequency is near one of the structures frequencies, i.e., tuned conditions, the problem is more complicated. The explanation for dispersion in this area has not been found in the literature. One potential explanation is motivated by observing that frequency content in the structure's motion near the tuned frequency is the dominant contributor to the oscillator's motion. In this frequency range/band the mode with the corresponding natural frequency dominates the structure's response, i.e., the other modes can be neglected in these arguments. This motion (one mode shape with a narrow frequency band) then acts as input to the tuned oscillator. However, the frequency response function of the oscillator shows that phase angle changes depend strongly on the exact frequency within the band of interest (see Figure A-1). For low damping, large variations in phase angle change occur within a narrow frequency band. Consider two different input spectra consistent time histories for a structure which has an attached light oscillator. The oscillator can achieve significantly different peak motions because of the phasing changes within the dominant frequency band as the structure's motion is transferred into the oscillator.

The acceptance of a method for direct response spectra transfer awaits a firm resolution to predicting response at tuned conditions. Several methods have been proposed, but none have received total recognition. It is the writer's assessment that development of an acceptable procedure will be a major benefit in equipment qualification because only knowledge of the input spectra and the dynamic mechanical properties are necessary. No time history analysis or finite element model is required and the calculated response spectra is not subject to dispersion. The RRSs can probably be determined while a modal analysis crew is actually conducting in-situ experiments.

5. RECOMMENDATIONS

The following recommendations concerning equipment qualification in operating plants have been developed in the course of these studies.

1. It is recommended that in-situ procedures be accepted as a method for determining dynamic structural mode shapes and natural frequencies. A standard or preferred format should be evolved for presenting test procedures and results to assist in validating the data reduction and analysis procedures used for construction of mode shapes.

2. It is recommended that the application of analysis procedures combined with in-situ derived dynamic properties (discussed in Section 4.3) be accepted as a method for determining the RRS of components mounted in support devices. The dynamic characteristics are the mode shapes and natural frequencies. The modal participation factor required for analysis may be calculated by any justifiable method; one such approach was described in Section 4.3. Use of the above parameters with the time history method is one acceptable analysis procedure for transferring the floor response spectra to a mounting position in a support device.

3. It is recommended that the seismic qualification requirements for retrofitted equipment be based on a RRS that has been either confirmed by in-situ testing or developed using in-situ dynamic characteristics. In-situ procedures may be employed to validate the finite element model used in developing a component RRS. Validation is achieved by showing close correspondence in the frequencies and mode shapes of significant modes. On the other hand, as described in recommendation 2, the dynamic characteristics derived from in-situ procedures may be used with analysis procedures to predict the RRS. In either case, seismic qualification for the retrofitted equipment is achieved if prior testing has successfully enveloped this RRS.

4. It is recommended that all support devices containing safety related equipment be subjected to an in-situ frequency evaluation, and that a comparison of these natural frequencies with floor response spectra be

performed as a screening technique to identify highly loaded systems. It must be insured that the natural frequencies of buildings are sufficiently removed from the as-installed support device natural frequencies. The alignment of these frequencies will result in substantially larger support device motion. In such cases modifications to the support device which alter its natural frequency are required. An alternative is to qualify the equipment and support device to the higher load levels.

5. It is recommended that all mountings attaching safety related equipment to equipment supporting structures be subjected to a walk-through examination. Suspect mountings should be retrofitted to current practice. The examination should be performed by someone experienced in seismic qualification testing as well as current mounting design practice in the nuclear industry.

6. It is recommended that design evaluations based on the failure mode analysis of Section 3.3 be accepted as one method of establishing seismic similarity between different pieces of equipment. The seismic qualification of one piece of equipment implies the qualification of similar designs in operating plants.

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APPENDIX A

STRUCTURE WITH APPENDAGE

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STRUCTURE WITH APPENDAGE

A.1 Response at Untuned Conditions.

If an oscillator is attached to a structure with 'n' degrees of freedom, the combined system takes the form of an 'n + 1' degree of freedom system.

The oscillator's frequency is identified as

$$\omega_o{}^2 = K_o/M_o \tag{A.1}$$

and if the oscillators mass is small compared to any in the structure then the natural frequencies of the total system are made up of the frequencies near the structures 'n' frequencies and the oscillator's frequency (ω_o). Here the factors required to transfer the ground input response spectra to any point on the support device are sought.

Assuming the coupling between oscillator and structure is weak, the equations are organized such that the first 'n' mode shapes and frequencies are those associated with the structure and the structural frequencies. The 'n + 1' modal component in each mode is the oscillator's motion relative to the moving base in each mode. We can solve for that modal component directly from the eigenproblem equations:

$$\left[\begin{array}{c:c} K^*-\omega_k^2 M^* & \begin{matrix} 0 \\ \vdots \\ 0 \\ -K_o \end{matrix} \\ \hdashline 0 \;\cdots\cdots\; 0 \;\; -k_o & (k_o - \omega_k{}^2 M_o) \end{array}\right] \begin{Bmatrix} \phi_1 \\ \vdots \\ \phi_n \\ \phi_{n+1} \end{Bmatrix}_k = 0 \tag{A.2}$$

where ω_k is one of the structures frequencies.

The final equation provides that

$$\frac{\phi_{n+1}}{\phi_n} = \frac{K_o}{K_o - \omega_K^2 M_o} = \frac{\omega_o^2}{\omega_o^2 - \omega_K^2} \quad . \tag{A.3}$$

Since the mass 'm_o' is taken to be very small compared to $M_1 \ldots M_N$, the first N frequencies are very close to those of the structure alone, then by partitioning the eigenproblem equations to eliminate the last two equations

$$\left[\begin{array}{ccc:cc} & & & K_{1n} & 0 \\ & & & K_{2n} & \vdots \\ & K' - \omega_K^2 M' & & \vdots & \vdots \\ & & & \vdots & \vdots \\ & & & K_{n-1,\,n} & 0 \\ \hdashline K_n & \cdots\cdots & K_{n,\,n-1} & (K_o + K_k - \omega_k^2 M_n) & -K_o \\ 0 & \cdots\cdots & 0 & -K_o & (K_o - \omega_k^2 M_o) \end{array}\right] \begin{Bmatrix} \phi_1 \\ \vdots \\ \vdots \\ \vdots \\ \vdots \\ \phi_n \\ \phi_{n+1} \end{Bmatrix}_K = 0 \tag{A.4}$$

the first 'n - 1' equations become

$$\left[K' - \omega_K^2 M'\right] \begin{Bmatrix} \phi_1 \\ \vdots \\ \phi_{n-1} \end{Bmatrix} = \phi_n \begin{Bmatrix} K_{in} \\ \vdots \\ K_{n-1,\,n} \end{Bmatrix} \tag{A.5}$$

These are exactly the equations solved to determine the first 'n - 1' components of the structural mode shape at any of its natural frequencies. So that for the first 'n' mode shapes

$$\left\{\phi\right\}_k = \left\{\begin{matrix} \phi_1 \\ \phi_2 \\ \\ \phi_n \\ \left[\omega_o^2/\left(\omega_o^2 - \omega_K^2\right)\right] \phi_n \end{matrix}\right\}_k \tag{A.6}$$

where $\phi_1 \ldots \phi_n$ is the structure only mode. Provided that the mass M_o is very small the eigenvector need not be re-mass-orthonormalized.

The final frequency of the total system is very near $\omega_{n+1} \approx \omega_o$, the oscillator frequency Again we view the last equation from the eigenvalue problem

$$\frac{\phi_n}{\phi_{n+1}} = \frac{\omega_o{}^2 - \omega_{n+1}^2}{\omega_o{}^2} \sim 0 \tag{A.7}$$

It is expected that the other 'n - 1' modal components are also negligible. To motivate this examine the eigenvalue equations after partitioning and rearranging

$$\left[K_s - \omega_o{}^2 M_s\right]\left\{\phi\right\}_{n+1}^{n} = \phi_{n+1,n+1} \left\{\begin{matrix} 0 \\ \vdots \\ 0 \\ \vdots \\ K_o \end{matrix}\right\} \tag{A.8}$$

$\{\phi\}_{n+1}^{n}$ = first 'n' modal components of 'n + 1' mode shape

$\left[K_s\right]$ = structure's stiffness matrix

$\left[M_s\right]$ = structure's mass matrix.

It will be shown that $\phi_{n+1,n+1} \sim 1/\sqrt{m_o}$ so that

$$K_o{}^*\phi_{n+1,n+1} = \frac{K_o}{\sqrt{m_o}} = \sqrt{m_o}\ \omega_o{}^2 \tag{A.9}$$

and thus the right hand side is a very small number.
Since ω_o is <u>not</u> an eigenvalue of the structure

$$\left[K_s - \omega_o{}^2 M_s\right]\left\{\phi^*\right\}_{n+1}^{n} = 0 \tag{A.10}$$

yields that

$$\{\phi^*\}_{n+1}^{n} = \{0\}$$

identically.

The right hand side of Equation (A.9) is small and thus (not proved here) the first 'n' modal components will be small in each component. Thus the final mode shape is

$$\{\phi\}_{n+1} = \begin{Bmatrix} 0 \\ \cdot \\ \cdot \\ \cdot \\ 0 \\ \phi_o \end{Bmatrix}$$

and since

$$\sum_{j=1}^{n+1} m_j \phi_j^2 = 1$$

we obtain $\phi_o = 1/\sqrt{m_o}$. In examining the final mode shape further we see that

$$\Gamma_{n+1} = \{\phi\}_{n+1}^{T} * \{M_i\} = \sqrt{m_o} \tag{A.11}$$

and

$$\alpha_{n+1} + \omega_o^2 \alpha_{n+1} = -\sqrt{m_o} Y_B \tag{A.12}$$

thus $\alpha_{n+1} = \sqrt{m_o} * g(t,\omega_o)$

where $g(t,\omega_o)$ is the solution to the SDOF oscillator whose maximum response is represented on the ground response spectra. Thus the oscillator's motion in this mode is

$$\begin{Bmatrix} 0 \\ \cdot \\ \cdot \\ \cdot \\ \cdot \\ 0 \\ X_o \end{Bmatrix} = \begin{Bmatrix} 0 \\ \cdot \\ \cdot \\ \cdot \\ \cdot \\ 0 \\ 1/\sqrt{m_o} \end{Bmatrix} \sqrt{m_o}\, g(t,\omega_o) = g(t,\omega_o)$$

Now it is a direct matter to employ the ground response spectra to predict the maximum values of the motion in each mode and combine these motions. For modes not close to one another that are also less than 33 Hz [or below the frequency of the zero period acceleration (ZPA) of the ground response spectra] the Square Root Sum of Squares (SRSS) will be the appropriate summation to employ. If more than one important structural mode has frequency greater than 33 Hz then these two modes are combined by an algebraic rule that maintains their correct relationship relative to one another. This total maximum value can then be combined by SRSS directly with the other structural modes. If there is only one structural mode above 33 Hz, it is combined as usual using SRSS. Thus we can construct certain portions of our in-structure response spectra.

For 'γ' important modes we write

$$\text{ACC. MAX. } (\omega) = \sum_{i=1}^{\gamma}\left(\left[\Gamma_i \phi_{n+1,i}\ S_A(\omega_i)\right]^2 + S_A(\omega_o)^2\right)^{1/2} \tag{A.13}$$

It is noted that the response spectra is constructed except near the structures natural frequencies. Note that the frequency on the abscissa is the oscillator frequency ω_o. Examining the modal matrix in Equation (A.8) makes it evident that the situation is singular at the points where $\omega_n = \omega_o$. A special treatment is required near these conditions.

A.2 Response at Tuned Conditions

In developing an in-equipment response spectra (or a floor spectra for that matter) one imagines placing a small mass supported by a variable spring in the position for which the response spectra description of the motion is sought. The structure must then undergo the same time history a number of times while the variable spring is taken through a range of values. The maximum oscillator acceleration is recorded and plotted versus the oscillator frequency. As the oscillator spring frequency is varied it will at some time be near to one of the support devices natural frequencies. When this occurs we can say we have tuning

($\omega_o \sim \omega_n$). The equations provided earlier degenerate when they are close to these conditions. It is necessary to examine these conditions further to determine the special modal responses which occur with light appendages and tuning simultaneously.

At the current time, in-structure (i.e., floor) response spectra are often determined by a time history method. As discussed later this is a procedure which can lead to variable results. However, it does point out the central feature of response when the oscillator frequency, at tuned conditions, aligns with structural frequency. That feature is a substantially amplified oscillator motion. Over a period of time various investigators have attempted to rationalize the response for tuned systems (i.e., $\omega_o \sim \omega_n$) that dominate the peaks in these time history transferred response spectra. Some of the analysis are ad hoc and depend on arbitrary amplification to drive their proposed methods. Others use numerical calculations that themselves may be somewhat suspect due to the highly singular nature of the response when the oscillator is tuned to the support device.

Unpublished numerical results have indicated that when several synthetic time histories are developed from a single input response spectra, the transferred response spectra can show wide variations at the tuned frequencies. The source of these variations has not been discussed in the literature. This delima must be understood both for understanding the inconsistencies of time history transfer and developing a direct method of transfer. The complete answer does not appear to be available at the current time. However a proposal for the underlying mechanism is presented below.

Recall that the response spectra at a specific point in a structure is not the structure's motion but rather a description of the motion experienced by an oscillator mounted in that position. To proceed, the structure's motion is decomposed into components in its free vibration modes. Of interest is the structural motion in the mode whose frequency is equal to that of the oscillator (bear in mind the motion of the oscillator in the non-tuned modes is characterized by the equations in Appendix A.1).

In order to proceed the base input is decomposed into a series of trignometric functions (or alternately a Fourier integral). The total response of the tuned mode is the superposition of the response for each term in this series. For an earthquake the load duration is of sufficient length to consider the response as steady state in each of these trignometric components. The steady state response to a trignometric input component occurs at the same frequency and can be written using the "frequency resonse function" (FRF). The FRF[39] is the solution to

$$\ddot{x} + 2\omega_o \xi x + \omega_o^2 x = \sin \omega t.$$

The FRF for displacement and acceleration are

$$FRF_d = (1/\omega_n^2) \Bigg/ \left[\left(1 - \left(\frac{\omega}{\omega_n}\right)^2\right)^2 + \left(2\xi\left(\frac{\omega}{\omega_n}\right)\right)^2\right]^{1/2}$$

$$FRF_a = (\omega^2/\omega_n^2) \Bigg/ \left[\left(1 - \left(\frac{\omega}{\omega_n}\right)^2\right)^2 + \left(2\xi\left(\frac{1}{\omega_n}\right)\right)^2\right]^{1/2}$$

The forced steady state solution is

$$x(t) = FRF_d * \sin(\omega t - \phi).$$

The FRF shows that inputs near the natural frequency are enhanced (amplified) and others are either filtered out or transmitted without amplification. Thus the motion of the tuned mode is richer in frequency content banded around its natural frequency ω_o. Now exactly the same enhancement occurs once again as the signal is transmitted through the oscillator and the motion of the oscillator is especially rich in frequency content near ω_o. Thus the response of the oscillator to the structure's motion in the tuned mode is dominated by inputs associated with frequency content in a tight band around ω_o.

Consider the oscillator response for several different base input time histories with the same response spectra. Since the input spectras are the same the maximum response of the structure's tuned mode will be the same for the several time histories. The time histories of the structural motion, however are not identical. To see the potential effect of non-unique time histories the FRF and phase angle for the single degree of freedom equation are taken from Reference 39 and shown in Figure A.1.

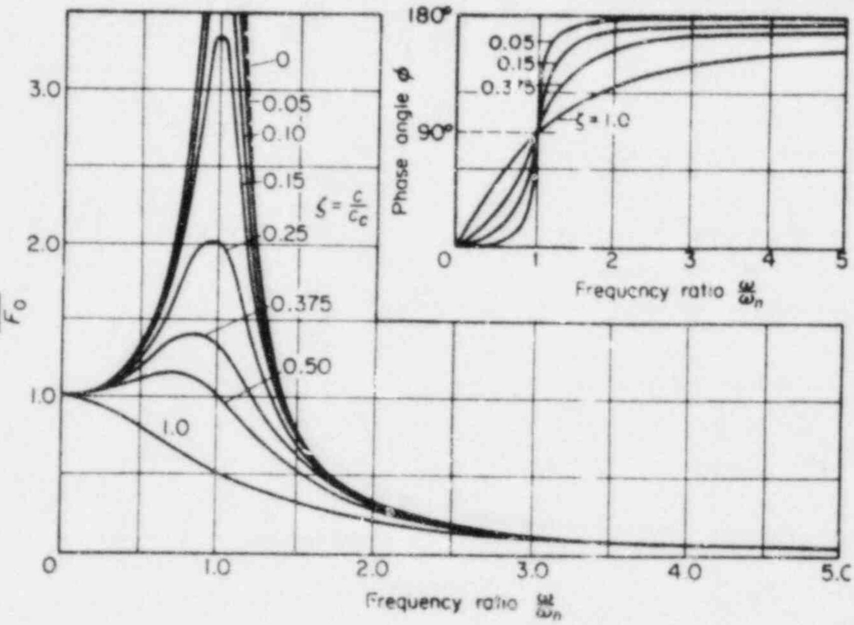

Figure A.1. Frequency response function and phase angle at various damping values.

The structural motions that act as an input to the oscillator are the result of different time histories and thus each has a unique frequency content and phasing in the frequency band which dominants the oscillator response. The several inputs to the structure were such that the phasing in the structure's motion yielded a common maximum value in the tuned mode. The oscillator response in that frequency band is determined by applying the FRF amplification (slowing varying) throughout the band as well as the phase angle change. As seen in Figure A.1 the phase angle is modified in a variable fashion over the frequency band. Thus the phasing in the oscillator and the structure are not similar. If several time histories are considered the oscillator's components within the frequency band need not combine to yield a unique maximum value. This appears to be the fundamental mechanism for variations near tuned conditions, i.e., rapidly varying phase angle changes.

PART B: IMPROVED IN-SITU PROCEDURES AND ANALYSIS METHODS FOR SEISMIC EQUIPMENT QUALIFICATION IN CURRENTLY OPERATING NUCLEAR POWER PLANTS

SUMMARY

The Nuclear Regulatory Commission (NRC) has designated seismic equipment qualification of active safety related equipment in currently licensed and operating nuclear power plants as Unresolved Safety Issue (USI) A-46. Alternate methods for seismic qualification of equipment are being developed as part of the resolution to USI A-46. EG&G Idaho is providing technical support to the NRC by developing and justifying technical procedures which facilitate the implementation of these alternate seismic qualification methods. This has been an on-going effort and progress for the period 1-83 to 9-83 is reported.

EG&G Idaho has been providing technical assistance in three areas: structural analysis methods, in-situ procedures, and the use of seismic experience data. In-situ procedures and analysis methods can be used together in estimating seismic environments. The combined use of in-situ procedures and methods of analysis which are currently accepted may be impractical. Other methods of analysis which make this application feasible are described, and technical justification to support their use is provided. The experience data base is composed of equipment of which the majority is located near the building foundation elevation. Justification is provided for using the estimated conservative ground spectra to estimate floor spectra less than 40 ft above the foundation. In the field of in-situ procedures, improvements which may impact equipment qualification in the future are discussed.

A structural model suitable for seismic uniform base excitation analysis must include the natural frequencies, mode shapes, damping, and modal participation factors of the system's significant dynamic modes. The need exists for an improved method of estimating the modal participation factors (MPFs). A method is presented which determines the optimized MPF given an incomplete set of mode shapes. By writing the equations of motion in standard form it is revealed that the force vector is always a specified vector made up of 1's and 0's. The role of the exact MPF is to reconstruct this force vector using a linear combination of the complete set of modes.

For practical analysis the limited number of significant modes, an incomplete set, is used so that the force vector is approximately reconstructed. The procedure described uses mode shapes determined from in-situ procedures. The known force vector is approximated by a linear combination of these modes and the coefficients (the MPFs) which provide the most accurate approximation are sought. An error vector based on the difference between the exact and approximate force vector is defined and minimizing the length of this vector yields an optimized estimate of the MPFs. This method requires less effort than the method recommended in the past and provides an optimized estimate for the given set of mode shapes.

To evaluate the seismic qualification of equipment it is necessary to compare the equipments' qualified seismic capacity with the Safe Shutdown Earthquake dynamic environment. This environment is usually described by a required response spectra. If the equipment is located in a supporting structure then the dynamic environment can be predicted by applying a suitable seismic floor input to a structural model and predicting the resulting time history and associated response spectra at positions where equipment is located. This process is described in Regulatory Guide 1.92. An alternative to the currently accepted time history method is presented which will substantially reduce the prediction phase of performing an analysis. This method of prediction is based on random vibration theory. The theory and equations are presented in order to technically justify this method. By referencing existing studies of earthquake characteristics it is shown that earthquakes are well described in the frequency domain, have frequency components whose phasing is random, have a Gaussian probability distribution, and can be considered as a segment from a stationary random process. The response spectrum of a motion is then related to the mean square integrated response and the power spectral density function (which is the spectral density of the mean square integrated response). Using a structural model the theory of steady state vibrations allows the power spectral density (PSD) and the mean square response to be predicted throughout the structure. These predicted mean square responses are then related to the output response spectra by using so-called peak value factors referenced from existing literature. Two peak value factors are recommended; one is based on the mean expected peak value and the second is based on a probability of exceedance. Finally, the theory for determining an input PSD from a known input response spectra is presented. A practical inversion method is recommended from the existing literature. Thus the justification for all the elements necessary to consistently transfer an input response spectrum from the base of a supporting structure to positions where equipment is located has been presented. Since no input time histories must be synthesized or employed in the analysis, the method represents a considerable reduction in effort. An additional benefit is that the non-uniqueness and dispersion in the time history method of transferring response spectra has been eliminated.

The Seismic Qualification Utilities Group is compiling seismic experience data. They have proposed that the estimated ground response spectra be used as estimates of the floor response spectra. It has been verified that this will be an adequate estimate for floors less than 40 ft from the building foundation. This determination was made by reviewing several existing analyses of power plant type structures. The use of ground spectra has been recommended as an acceptable alternative in lieu of using structural analyses to predict low elevation floor response spectra.

Finally, the field of in-situ procedures has been monitored. Studies which experimentally confirm the accuracy of the combined use of in-situ procedures and analysis methods to predict equipment required response spectra were sought. No studies which could fulfill this need were identified. The validity of this method continues to be based on theoretical principles, engineering judgment, and the sound application of margin. In-situ procedures continue to evolve. In-situ procedures using multiple point random excitation are the main area experiencing considerable progress and could substantially impact nuclear applications in the future. Here, broadband frequency excitation is applied at several points simultaneously. Dependence of the measured frequency response functions on excitation point or load level is reduced and results with improved consistency are obtained. The method appears to be useful with larger structures where it can be difficult to obtain representative frequency response functions.

EG&G Idaho has recently proposed guidance and acceptance criteria for the use of in-situ procedures, analysis methods, and certain aspects of the application of experience data. This information is reported elsewhere. In cases where substantial technical justification is required to support these guidance and acceptance criteria the justification has been presented in this document. A method for determining the modal participation factor directly from the significant modes (determined from in-situ procedures) has been recommended as an acceptable procedure. A method for transferring base input response spectra through support devices has been presented and justified. When this method is used with modal characteristics determined from in-situ procedures, the least total analysis effort is necessary. This method is recommended as an acceptable alternative to the time history method described in Regulatory Guide 1.92. Based on our review the use of ground spectra to estimate floor spectra is acceptable at least for floors less than 40 ft above the foundation. These results substantially enhance the methodology for applying the alternate qualification methods of USI A-46 and are reflected in the guidance and acceptance criteria reported elsewhere.

IMPROVED IN-SITU PROCEDURES AND ANALYSIS METHODS FOR SEISMIC EQUIPMENT QUALIFICATION IN CURRENTLY OPERATING NUCLEAR POWER PLANTS

INTRODUCTION

Significant changes in seismic qualification criteria have occurred in the period since the first commercial nuclear power plants were licensed. The analytical and experimental methods used to seismically qualify equipment have also evolved. The margins of safety provided in existing nuclear power plant equipment to resist seismically induced loads and perform their intended safety functions may vary, and may not meet current seismic qualification criteria. Therefore the Nuclear Regulatory Commission (NRC) has recognized[1] the need to re-assess the seismic qualification of equipment in operating plants.

Also recognized by the NRC is that seismic equipment qualification using current criteria and methods may not be practical in operating plants, in part, because of excessive plant down time, difficulties in shipping irradiated equipment to a test laboratory, and in acquiring identical old vintage equipment for laboratory testing. In December 1980, the Nuclear Regulatory Commission designated "Seismic Qualification of Equipment in Operating Plants" as Unresolved Safety Issue (USI) A-46. The objective of USI A-46 is to develop alternate seismic qualification methods and associated acceptance criteria that can be used to assess the capability of mechanical and electrical equipment in operating nuclear power plants to perform their intended safety functions.

EG&G Idaho is providing technical assistance to the NRC by developing and justifying technical procedures which facilitate the application of these alternate seismic qualification methods. This has been an on-going effort and progress for the period 1-83 to 9-83 is reported here. Technical assistance in the development and justification of procedures has been provided in three areas: in-situ procedures, structural analysis methods, and the use of seismic experience data. The determination of dynamic structural characteristics requires in-situ testing plus data processing and the analysis of processed data; thus the overall processes are referred to as in-situ procedures rather than in-situ testing.

Past progress by other contractors supporting USI A-46 has verified that a major component of the alternate methods of qualification is historical evidence of equipment seismic capacity.[2,3] This evidence comes from seismic experience data and possibly also from prior qualification testing. Past technical progress by EG&G Idaho[4] has shown that there are uses for both in-situ testing, and for analysis methods. This study found that in-situ procedures could provide the dynamic characteristics of structures. The dynamic characteristics, in certain cases, are useful for establishing dynamic similarity of systems. More important is the combined use of in-situ procedures and analysis methods for the prediction of seismic environments. One can evaluate the seismic adequacy of equipment with knowledge of the design basis seismic environment and the seismic capacity from seismic experience (or other) data. Furthermore, these methods for predicting seismic environment can be employed for determining the seismic capacity of experience data base equipment.

Since past progress in USI A-46 has defined the pertinent qualification alternatives, recent emphasis has been to complete the technical developments necessary for effective implementation of the alternative qualification methods, and to provide the guidance and acceptance criteria for all the technical procedures involved. Guidance and acceptance criteria applying to in-situ procedures, analysis methods, and certain portions of the determination of a qualified seismic capacity from seismic experience data have been proposed and are reported elsewhere.[5] This document and Reference 5 are closely coupled in that technical justification for certain guidelines and acceptance criteria is contained here.

Certain alternative analysis methods have been judged to be valuable to the practical application of the alternate qualification methods. The analysis methods are a new procedure for estimating the modal participation factor (a parameter of the structural model analyzed to define the seismic environment) and the use of special random vibration methods for predicting seismic environments. The theoretical and empirical bases for these two methods are developed thus providing the technical justification for the associated guidance and acceptance criteria (see Part C of this report).

Seismic experience data is being drawn from non-nuclear power generating or distributing facilities and the equipment in the experience data base is mainly located near the elevation of the building foundation. For this equipment it is desirable to use the conservative ground response spectra estimates as estimates for the floor response spectra. The adequacy of this approximation is considered and recommendations are presented.

The methods for predicting seismic environments of equipment located in supporting structures (cabinets, racks, etc.) using in-situ procedures and analysis methods in combination are relatively new and experience is lacking. The technical justification for these procedures is based primarily upon similarity with approved analysis procedures, theoretical principles, engineering judgment, and sound application of margin. Natural frequencies and mode shapes are determined at low levels of excitation using in-situ procedures and verification of the applicability of these structural parameters to Safe Shutdown Earthquake excitation levels was sought. Verification of the total method was also sought. Finally, upcoming developments in in-situ procedures which might have future use in seismic equipment qualification were reviewed.

MODAL PARTICIPATION FACTOR

The modal participation factor (MPF) is required in all analysis procedures predicting required response spectra (RRS) for positions within support structures experiencing a uniform base motion excitation. In Section 4.3 of Reference 4, "Analysis Using Modal Parameters Directly," one method for determining the MPFs was described. In this section an alternative method of determining the MPFs is presented. This method for determining the MPFs is based on reconstructing the force vector using the significant modes of the structure.

The absolute displacement vector $\{x\}$ is defined by

$$\{x\} = \{I_x\} x_b + \{I_y\} y_b + \{I_z\} Z_b + \{U_r\} \quad (1)$$

where

$\{U_r\}$ = relative displacement vector

x_b, y_b, z_b = x, y, z motion of base

$\{I_x\}, \{I_y\}, \{I_z\}$ = x, y, and z selector vectors.

The equation of motion is

$$[M] \{\ddot{U}_r\} + [C] \{\dot{U}_r\} + [K] \{U_r\} = - [M] \{I_x\} \ddot{x}_b - [M] \{I_y\} \ddot{y}_b - [M] \{I_z\} \ddot{z}_b \quad (2)$$

where

[M], [C], and [K] = mass, damping, and stiffness matrices.

As usual, the modal decomposition

$$\{U_r\} = [\phi] \{\alpha\} \tag{3}$$

can be introduced to yield the equations of motion in modal coordinates

$$\{\ddot{\alpha}\} + [2\zeta_n\omega_n] \{\dot{\alpha}\} + [\omega_n^2] \{\alpha\} = -\{\Gamma_x\} \ddot{x}_b - \{\Gamma_y\} \ddot{y}_b - \{\Gamma_z\} \ddot{z}_b \tag{4}$$

where

ζ	=	% of critical damping
$[\phi]$	=	matrix whose columns are the free vibration modes
$\{\alpha\}$	=	vector of modal coordinates
$\{\Gamma_x\}$, $\{\Gamma_y\}$, $\{\Gamma_z\}$	=	modal participation vectors for x, y, and z base excitations.
$[\omega_n^2]$	=	diagonal matrix of natural frequencies squared
$[2\zeta_n\omega_n]$	=	diagonal damping matrix involving natural frequencies and modal damping.

The normality relations

$$[\phi]^T [M] [\phi] = [M_{rr}]$$

$$[\phi]^T [C] [\phi] = [M_{rr}] [2\zeta_n\omega_n] \tag{5}$$

$$[\phi]^T [K] [\phi] = [M_{rr}] [\omega_n^2]$$

have been used to yield the uncoupled form of Equation (4) where $[M_{rr}]$ is a diagonal matrix containing the equivalent modal masses.

The three modal participation factor vectors are

$$\{\Gamma_x\} = [M_{rr}]^{-1} [\phi]^T [M] \{I_x\} = \text{vector of x direction MPFs}$$
$$\{\Gamma_y\} = [M_{rr}]^{-1} [\phi]^T [M] \{I_y\} = \text{vector of y direction MPFs} \quad (6)$$
$$\{\Gamma_z\} = [M_{rr}]^{-1} [\phi]^T [M] \{I_z\} = \text{vector of z direction MPFs}$$

To motivate the method to be proposed the original equations of motion are rewritten in standard form

$$\{\ddot{U}_r\} + [M]^{-1} [C] \{\dot{U}_r\} + [M]^{-1} [K] \{U_r\} = -\{I_x\} \ddot{x}_b - \{I_y\} \ddot{y}_b - \{I_z\} \ddot{z}_b \quad (7)$$

The form of the physical forcing function should be noted. It is observed the physical force is uniformly distributed across all the nodes (and hence the loading can be categorized as a body force loading). Recall that

$$[\phi] \{\Gamma_x\} = \{I_x\} \quad (8)$$

$$[\phi] \{\Gamma_y\} = \{I_y\} \quad (9)$$

$$[\phi] \{\Gamma_z\} = \{I_z\} \quad (10)$$

and the right hand sides of Equations (8), (9), and (10) are proportional to the force vector in the x, y, and z directions. Therefore the role of the MPF vectors is to construct the load vectors in Equation (7), essentially three vectors whose components are 1s and 0s, from the mode shapes of the system. Of course, for the application discussed here the available mode shapes are approximations to the real mode shapes and these approximations are produced from in-situ procedures. If a complete set of mode shapes was available then Equations (8), (9), and (10), could be inverted to yield the MPFs, viz.,

$$\{\Gamma_x\} = [\phi]^{-1} \{I_x\} \quad (11)$$

$$\{\Gamma_y\} = [\phi]^{-1} \{I_y\} \tag{12}$$

$$\{\Gamma_z\} = [\phi]^{-1} \{I_z\} \tag{13}$$

The complete set of modes will not be determined using in-situ procedures. The force vector cannot be exactly constructed as in equations (11), (12), and (13) because this requires the complete set of modes. However, with an incomplete set of modes the physical force vector can be approximately constructed as

$$\{I_i\} = \text{force vector} = [\phi'] \{\Gamma_i\} + \{R_i\} \tag{14}$$

where

$[\phi']$	=	rectangular mode shape matrix containing m experimentally estimated mode shapes: matrix is n x m
$[\Gamma_i]$	=	m x 1 modal participation factor vector
$[R_i]$	=	n x 1 error vector
i	=	x, y, or z.

The error measure, E, is defined as the standard vector norm

$$E_i = \{R_i\}^T \{R_i\} = \{I_i\}^T \{I_i\} - 2 \{\Gamma_i\}^T [\phi']^T \{I_i\} + \{\Gamma_i\}^T [\phi']^T [\phi'] \{\Gamma_i\} \tag{15}$$

This error depends only upon the unknown MPFs and the error can be minimized by imposing

$$\frac{\partial E}{\partial \{\Gamma_i\}} = 0 \tag{16}$$

yielding

$$\{\Gamma_x\} = ([\phi']^T [\phi'])^{-1} [\phi']^T \{I_x\} \tag{17}$$

$$\{\Gamma_y\} = ([\phi']^T [\phi'])^{-1} [\phi']^T \{I_y\} \tag{18}$$

$$\{\Gamma_z\} = ([\phi']^T [\phi'])^{-1} [\phi']^T \{I_z\} \tag{19}$$

Note that these optimized MPFs should provide an improvement over the exact MPFs (assuming the use of a limited number of modes) because the latter do not minimize the error between the physical force vector and the force vector constructed from the estimated modes in $[\phi']$. Consequently these MPFs are the very best estimates to use in combination with the incomplete set of estimated modes.

Reiterating, the MPFs should be determined via Equations (17), (18), and (19). The role of the MPF vector is to construct the force vector of Equation (7) in terms of the significant modes of vibration. The significant modes are those required to accurately reconstruct the force vector. By examining the error vector, or its magnitude squared ($<R_x> \{R_x\}$) it can be determined if all significant modes have in fact been included. Using these significant modes, Equations (17), (18), and (19) provide the best possible fit to the force vector. Furthermore note that estimated equivalent modal masses and equivalent modal stiffness have not been employed in estimating the MPFs. Consequently errors in the estimates of these quantities do not effect the accuracy of the estimated MPFs.

Several other schemes have been suggested for estimating the MPFs.[5,6] The first scheme eliminates nodes from the experimental modal model (thus shortening the length of the mode shape vector) and develops a truncated (m x m) square mode shape matrix. The MPF vector is related to the inverse of this matrix [see Equation (11)]. The truncation will normally be severe (a typical situation might be a reduction from

100 degrees of freedom to 10 degrees of freedom). Many different predictions can be obtained by altering the nodes eliminated from the mode shapes. Definitive rules for the truncation process are not available and the resulting ambiguity is unacceptable.

The second method utilizes the orthonormality condition

$$\{\phi\}_j \, [M] \, \{\phi\}_i = 0, \ i \neq j$$

$$\{\phi\}_j \, [M] \, \{\phi\}_j = m_{jj}$$

$$i, j = 1, \ . \ . \ . \ N$$

where N is the total number of degrees of freedom. The actual mass matrix is assumed to be diagonal. If all n exact mode shapes were available these equations could be solved to yield the mass matrix (ignoring the fact that the real mass matrix is probably full). However only estimates of a limited number of mode shapes are available and thus the actual mass matrix will not satisfy the orthonormality equations when the approximate mode shapes are substituted. The method is further aggravated because the equations can only be satisfied approximately by minimizing the error. Further validation would be required before either of the latter two methods could be confidently employed in predicting the MPFs. The method presented at the beginning of this section is judged to provide the best possible estimate and is recommended.

RESPONSE SPECTRA TRANSFER

Preliminary Remarks

Response spectra transfer using the time history method was discussed in Reference 4. This guidance is essentially identical to the combined guidance in Regulatory Guides 1.92 and 1.122. The requirements on time histories were not discussed in that section. More recent work has indicated that, for both broadband and narrow band inputs to equipment supporting structures, requirements on frequency content and phasing of frequency components should be imposed. All input motions should contain essentially continuous frequency content (or at least many frequency components) and phasing should be random. A correlary requirement is that no concentrated frequency content, i.e., a pure sinusoid over part or all of the excitation duration, be allowed. The factors dictating these recommendations should become more evident later (see pages 15-23 for a more complete discussion).

The methods of response spectra transfer to be discussed herein do not involve time history analysis since they are performed in the frequency domain. These methods do not require the generation of synthetic time histories, and changes in the position of floor spectra peaks and structural resonances are readily incorporated. Consequently, multiple calculations to incorporate frequency shifts for the purpose of maintaining margin are not an extraordinary hardship. As a consequence the analyses are performed more readily and with less expense.

These methods are based on the application of random vibration theory. When applied to seismic environments this normally implies that the mean square response is used as the basis for predicting peak response values. In fact, the square root sum of squares method of modal combination for determination of peak values is based on the fact that under specific conditions the total mean square response can be estimated using this method of modal combination.[7] There are a number of conditions when it does not provide an accurate estimate and improved methods of modal calculation have been proposed. However, the point here is that the existing basic methods in seismic analysis are based substantially on random vibration theory.

The purpose of this section is to provide the equations and theory behind response spectra transfer. However, even more fundamental is to provide a technical justification for the use of these methods. To perform this function the discussion below must address several areas including general characteristics of earthquakes, the power spectral density (PSD) function, application of steady state vibration theory in the frequency domain, the calculation of mean square response for light oscillators attached to structures, and the peak value factor.

Earthquake Characteristics

The application of random vibration theory to a particular process is simplified if the process is Gaussian, zero mean, and stationary. This is because the process mean square integrated spectral density, or power spectral density function, completely defines the process under these restrictions. The requirement for stationarity cannot be met for structural response to earthquakes because structures do not experience entirely steady state response to the various frequency components of an earthquake. However earthquakes may be considered as a finite duration segment in a stationary process and corrections can be applied to structural response for the non-stationary (non-steady) effect of duration. The fact that seismic motions are zero mean is obvious. As will be discussed next, earthquakes are Gaussian in character because of broad frequency content and the random phasing of the frequency components.[8]

It is necessary to describe earthquakes in terms of the frequency content of their strong motion. The frequency content for G(t) is determined from the Fourier transform

$$g(\omega) = \frac{1}{2\pi} \int_{-\infty}^{\infty} G(t)e^{-i\omega t}\, dt \qquad (20)$$

[the earthquake time history is repeated in order to define a non-zero function $G(t)$ over the limits of the integral]. Typical earthquakes have frequency content that is continuous over a range of frequencies. In these frequency ranges the phase angle [determined by comparing real and imaginary components of $g(\omega)$] is found to vary randomly.[9,10,11] Clough and Penzien (see Reference 8) have shown that random phasing in a process with many frequency components implies the process is Gaussian. Consequently it is reasonable to idealize earthquakes as random, zero mean, Gaussian processes.

For the methods under consideration structural response is predicted using the principles of linear steady state vibrations. One useful manner in which to visualize the situation is by imagining the earthquake as a long process made up of many superimposed trigonometric signatures whose phasing is random. Any linear structure must eventually assume a steady state response to each frequency component of the input. This steady state response is at precisely the exciting frequency and will have an associated phase angle. It is apparent the output is composed of many (and the same) frequency components which are still randomly phased. Consequently the output is random, zero mean, Gaussian, and stationary. For this situation all statistical properties of the output can, in theory, be inferred from the input using the properties (natural frequencies, mode shapes, modal participation factors, and modal dampings) of the intervening structure. For a Gaussian, zero mean process the basic statistical properties are the mean value and the autocorrelation function. However the power spectral density (PSD) function can be substituted directly for the autocorrelation since the two quantities are Fourier transforms of one another. In other situations it is convenient to use the mean square response (autocorrelation with zero time difference) in combination with a peak value factor to describe the necessary statistics. The peak value factor associates the maximum value of a response with the mean square integral value of that response. The peak value factor is determined from the statistics of the process and can be based on the mean or a probability of exceedance. These theoretical considerations are fairly well developed within the theory of random processes.

Thus seismic input motions are well suited for description by their frequency content and thus their statistical characteristics are also well defined. A difficulty in seismic analysis is due to the structural motion starting from zero initial conditions. By assuming a sufficiently long duration it is reasonable that free vibrations will be completely damped from the response and steady state conditions will be achieved for the frequency range of interest. However, while these motions are being damped from the response they contaminate the response, affecting the stationarity, Gaussianism, and effective peak value factor. The development of the steady state response occurs simultaneously with the complete damping of the free vibration response. The magnitude of the response generally builds as the free vibration components are dissipated and peak values do not occur until after the free vibration component can be neglected (see Reference 8). For low frequency systems with high damping a greater period of time is required to achieve steady state conditions. Thus it is necessary to account for these effects of duration. The developed procedures use the methods of steady state vibrations but employ artificial values of damping and a corrected duration to correct the response.[12,13]

(To clarify this situation consider the difference between the real situation with a transition through zero start conditions, and an ideal situation where motion has been ongoing for long enough so that all free vibrations have been damped away. For a given duration it is clear that the second case is more likely to achieve the larger peak value, that this case is more consistent with the assumptions of random vibration theory, and finally that the response is more accurately predicted by uncorrected random vibration theory.)

Mean Square Response and the Power Spectral Density Function

In the last subsection it was argued that real earthquakes (and composite earthquakes such as embodied in Regulatory Guide 1.60) are random processes which are well modeled as zero mean, Gaussian, and stationary. Since earthquakes are of finite duration while random processes must be of long duration, it is entirely consistent to think of an earthquake as a segment of a stationary process. It was also acknowledged that structural response would not be steady state vibration and therefore would not be stationary during the initial portion of the response (this is sometimes referred to as a duration effect because its' significance is related to the total duration of motion with longer duration motion being less affected). The analysis methods to be discussed below are based on steady state response (this is true for predicting the resulting motion as well as the associated statistics of the motion) and mention was made that correction factors proposed in the literature must be used to correct for the differences between steady state response and response from realistic initial conditions.

An important quantity in describing the statistical characteristics of motion is the mean square response value. Thus attention is focused on calculating the mean square response

$$\sigma^2(x) = \frac{1}{T} \int_0^t x(t)x(t)dt \tag{21}$$

or the standard deviation, which is the square root of the mean square response. The mean square response, for the types of random motions under consideration, is directly related to maximum values of response. It will be necessary to deal with the mean square response of the excitation and the response of oscillators attached to the structure at points for which a response spectrum is sought.

Considerations attendant to determining these mean square values will now be discussed. The peak value factor required to transform the mean square response into an estimate of the maximum response value will be discussed in a subsequent section.

The mean square response is rewritten in the frequency domain as,

$$\sigma^2 = \int_{-\infty}^{\infty} \Phi(\omega)d\omega \tag{22}$$

where Φ is the power spectral density (PSD) function of the process $x(t)$. It is possible to interpret the PSD in several ways including:

- The spectral density can be discrete (concentrated frequency content) or continuous and there is no appreciable difference if the discrete representations contain a sufficient number of components with nearly equal magnitude

- $\Phi(\omega) * (\omega_2 - \omega_1) = \Phi(\omega) \bullet \Delta\omega$, is the mean square response due to frequency components in the frequency range $\Delta\omega$ such that $\omega_1 \leq \omega \leq \omega_2$

- The autocorrelation and the PSD are a Fourier transform pair

$$\Phi(\omega) = \int_{-\infty}^{\infty} R(\tau)\, e^{-i\omega\tau d\tau} \tag{23}$$

$$R(\tau) = \frac{1}{2\pi} \int_{-\infty}^{\infty} \Phi(\omega) e^{i\omega t} d\tau$$

where $R(\tau)$ is the autocorrelation function for the process $x(t)$

- The PSD can be represented in terms of the Fourier Transform as

$$\Phi(\omega) = \lim_{T\to\infty} \frac{|X(\omega)|^2}{T} \tag{24}$$

where $X(\omega)$ is the Fourier Transform.

$$X(\omega) = \int_{-\infty}^{\infty} x(t) e^{-i\omega t} dt$$

Thus if the frequency content of a process is known or the frequency content of the mean square response is known then the PSD is known. Likewise if the PSD of a motion is known then the mean square value of the motion can be evaluated by integration of the PSD with respect to frequency.

The PSD at an excitation position can be transferred to an output position using the results of steady state vibration theory. These steps are probably familiar to most readers and are presented in a brief form. First of all the transfer function for the modal coordinates is determined as shown below. The equation of motion for a single base input is

$$[M]\{\ddot{x}_a\} + [C]\{\dot{x}_a\} + [K]\{x_a\} = [C]\{\dot{y}_b\} + [K]\{y_b\} \tag{25}$$

where

$\{x_a\}$

$\{\dot{x}_a\}$

$\{\ddot{x}_a\}$

are the total physical displacement, total physical velocity, and total physical acceleration. The vectors $\{y_b\}$ and $\{y_b\}$ are vectors whose components are the base input displacement time history and the base input velocity time history, respectively. In modal coordinates the equation becomes

$$\begin{aligned} \{\ddot{\alpha}_a\} + [2\zeta_n\omega_n]\{\dot{\alpha}_a\} + [\omega_n^2]\{\alpha_a\} & \qquad (26) \\ = [2\zeta_n\omega_n][\phi]^{-1}\{1\}\dot{y}_b + [\omega_n^2][\phi]^{-1}\{1\}y_b & \\ = \Big[[2\zeta_n\omega_n]\dot{y}_b + [\omega_n^2]y_b\Big]\{\Gamma\} & \end{aligned}$$

where Γ is the modal participation factor and

$$\{x_a\} = [\phi]\{\alpha_a\}$$

An equation for the nth mode is

$$\ddot{\alpha}_{an} + 2\zeta_n\omega_n\dot{\alpha}_{an} + \omega_n^2\alpha_{an} = (2\zeta_n\omega_n\dot{y}_b + \omega_n^2 y_b)\,\Gamma_n \quad . \tag{27}$$

The modal transfer function for this modal equation is determined (Γ is normalized to 1) as

$$y_b = e^{i\omega t}$$

$$\alpha_{an} = Ae^{i\omega t}$$

$$h_n(\omega) = \frac{\alpha_{an}}{y_b} = \frac{\ddot{\alpha}_{an}}{\ddot{y}_b}$$

$$h_n(\omega) = \frac{(\omega_n^2 + i2\zeta_n\omega_n\omega)}{\omega_n^2 - \omega^2 + i2\zeta_n\omega_n\omega} \tag{28}$$

where $h_n(\omega)$ is the modal transfer function for the nth mode and i is the imaginary unit such that $i^2 = -1$.

The transfer functions for the physical variables are

$$H_{rx}(\omega) = \sum_{n=1}^{N} \phi_{rn}\,\{\Gamma_n\}^T\,\{\delta_x\}\,h_n(\omega)$$

$$H_{ry}(\omega) = \sum_{n=1}^{N} \phi_{rn}\,\{\Gamma_n\}^T\,\{\delta_y\}\,h_n(\omega) \tag{29}$$

$$H_{rz}(\omega) = \sum_{n=1}^{N} \phi_{rn} \{\Gamma_n\}^T \{\delta_z\} h_n(\omega)$$

where r is the degree of freedom (3 degrees of freedom at each unrestrained mode), {X, Y, Z} are the three directions of base excitation, N is the total number of significant modes, ϕ_{rn} is the modal coefficient for degree of freedom r for the nth mode, $h_{rx}(\omega)$ is the transfer function for the rth node associated with base input in the X direction while $H_{ry}(\omega)$ is the transfer function for the rth node associated with base input in the Y direction and so forth, and $\{\Gamma_n\}$ provides the three modal participation factors for the nth mode.

Earlier it was stated that the mean square response of y(t) was related to its PSD and Fourier Transform $Y(\omega)$ by

$$\frac{1}{T} \int_{o}^{T} y(t)y(t)dt = \int_{-\infty}^{\infty} \Phi_{yy} \, d\omega = \int_{-\infty}^{\infty} \frac{Y(\omega)Y^*(\omega)}{T} \, d\omega$$

If y(t) is the input, $Y(\omega)$ is the Fourier transform (FT) of y(t), x(t) is the response, $X(\omega)$ is the FT of x(t), and $H(\omega)$ is the transfer function between these quantities then

$$H(\omega) = \frac{X(\omega)}{Y(\omega)}$$

and $X(\omega) = H(\omega) \, {}^*Y(\omega)$

so that

$$\frac{1}{T} \int_{o}^{T} x^2(t)dt = \int_{-\infty}^{\infty} \frac{X(\omega)X^*(\omega)}{T} \, d\omega = \int_{-\infty}^{\infty} \frac{Y(\omega)Y^*(\omega)}{T} \, H(\omega) \, H^*(\omega)d\omega$$

$$= \int_{-\infty}^{\infty} \frac{Y(\omega)Y^*(\omega)}{T} \; H(\omega)^{\,2} d\omega = \int_{-\infty}^{\infty} \Phi_{yy} \; H(\omega)^{\,2} \, d\omega \tag{30}$$

In Equation (30) above the complex conjugate is indicated by the star superscript (*). Equation (30) provides the mean square response of x(t) as a function of the input PSD and the transfer function.

In order to transfer a response spectra from the base of an equipment support structure to a degree of freedom within the structure it is necessary to calculate the mean square response of an oscillator attached to that degree of freedom on the structure. The transfer functions for the oscillator response due to X, Y, and Z base inputs are

$$H^{o}_{rx}(\omega) = \sum_{n=1}^{N} \phi_{rn} \{\Gamma_n\}^T \{\delta_x\} h_n(\omega) h_o(\omega)$$

$$H^{o}_{ry}(\omega) = \sum_{n=1}^{N} \phi_{rn} \{\Gamma_n\}^T \{\delta_y\} h_n(\omega) h_o(\omega) \tag{31}$$

$$H^{o}_{rz}(\omega) = \sum_{n=1}^{N} \phi_{rn} \{\Gamma_n\}^T \{\delta_z\} h_n(\omega) h_o(\omega)$$

where $h_o(\omega)$ is the absolute displacement or absolute acceleration transfer function for the oscillator given by

$$h_o(\omega) = \frac{\omega_o^2 + i2\zeta_n\omega_o\omega}{\omega_o^2 - \omega^2 + i2\zeta_n\omega_o\omega} \tag{32}$$

The mean square response for a single base input (say in the X direction) is

$$\sigma_a^2 (r,\omega_o) = \int_{-\infty}^{\infty} \Phi_{xx} H^{o}_{rx}(\omega) H^{o*}_{rx} (\omega) d\omega \tag{33}$$

$$= \int_{-\infty}^{\infty} \Phi_{xx} \left[\sum_{i=1}^{N} \phi_{ri} \Gamma_{ix} h_i(\omega) h_o(\omega)\right]\left[\sum_{k=1}^{N} \phi_{rk} \Gamma_{kx} h_k(\omega) h_o(\omega) \quad d\omega\right]$$

where Φ_{xx} is the PDS for base excitation in the x direction.
Rearranging the summation and integrals

$$\sigma_a^2 \ (r,\omega_o) = \sum_{j=1}^{N} \sum_{k=1}^{N} \phi_{rj} \ \phi_{rk} \ \Gamma_{jx} \ \Gamma_{kx} \int_{-\infty}^{\infty} \Phi_{xx}(\omega) \tag{34}$$

$$\left[h_j(\omega) \ h_k^*(\omega) \ h_o(\omega) \ h_o^*(\omega)\right] d\omega$$

where

$$h_j(\omega) = \frac{\omega_j^2 + i2\zeta_j\omega_j\omega}{\omega_j^2 - \omega^2 + i2\zeta_j\omega_j\omega}$$

and

$$h_o(\omega) = \frac{\omega_o^2 + i2\zeta_o\omega_o\omega}{\omega_o^2 - \omega^2 + i2\zeta_o\omega_o\omega}$$

For simultaneous x, y, and z base inputs

$$\sigma_a^2 \ (r,\omega_o) = \sum_{j=1}^{N} \sum_{k=1}^{N} \phi_{rj}\phi_{rk} \int_{-\infty}^{\infty}\Big[\Gamma_{jx}\Gamma_{kx}\Phi_{xx}(\omega) \tag{35}$$

$$+ \Gamma_{jy}\Gamma_{ky}\Phi_{yy}(\omega) + \Gamma_{jz}\Gamma_{kz}\Phi_{zz}(\omega)\Big] * \left|h_o(\omega)\right|^2 h_j(\omega)h_k^*(\omega)d\omega$$

It is observed that the equation above involves a double sum over the significant mode shapes. The transfer functions $h_j(\omega)$ are complex numbers as are the cross modal products $h_j(\omega)h_k^*(\omega)$. The star superscript (*) indicates the complex conjugate. Real numbers are attained by factoring the combination $(h_j(\omega)h_k^*(\omega) + h_k(\omega)h_j^*(\omega))$ from the summation. The cross terms do not necessarily yield a positive contribution to the mean square response,[14] i.e., they can achieve a negative value. The single summation terms (i.e., involving $\left|h_j(\omega)\right|^2\left|h_o(\omega)\right|^2$), are inherently positive. The combined single sum terms yield the equivalent of the square root sum-of-squares modal combination. As the cross modal terms in the mean square response increase in significance (compared to the single sum terms) then the SRSS modal combination becomes less accurate.

To evaluate the mean square response properly one should understand the character of the transfer function products in the integral. Recall that the transfer function has,

$$\begin{aligned} h_j(\omega) &\sim 1 \text{ for } \omega \ll \omega_j \\ h_j(\omega) &\sim 0 \text{ for } \omega \gg \omega_j \\ h_j(\omega) &\sim \frac{1}{2\zeta_j} \text{ for } \omega \approx \omega_j \end{aligned} \tag{36}$$

Note that the total transfer function products in Equation (35) are.

$$h_o(\omega)h_o^*(\omega)[h_j(\omega)h_k^*(\omega) + h_k(\omega)h_j^*(\omega)] \tag{37}$$

or

$$h_o(\omega)\ h_o^*(\omega)\ [h_j(\omega)\ h_j^*(\omega)] \tag{38}$$

and that for non-tuned conditions ($\omega_o \neq \omega_j, \omega_k$) the peaks in the total product do not occur at a common frequency. In addition for non-tuned conditions the single sum terms, Equation (38), differ from the cross product terms, Equation (37), only by the fact that the single sum terms provide two pronounced peaks in the integrand of Equation (35) while the cross modal products provide a peak only at the oscillator natural frequency. It is apparent the cross product terms are important in evaluating the mean square oscillator response whenever there is a significant mode with a frequency greater than that of the oscillator. Consequently the SRSS method described in Appendix A Reference 4 is not a valid method of predicting the transferred response spectra. It is necessary to compute the mean square oscillator response using the complete double summation over significant modes.

An interesting observation can be made concerning the use of the SRSS combination for calculation of peak values of structural quantities when modes are well spaced. Consider a situation where one or more significant modes occur at frequencies above the frequency range of interest. In this case the integrals associated with the corresponding single sum terms have zero PSD at the peaks of $h_j(\omega)h_j(\omega)$ and the important range of integration is $\omega<\omega_j$. Here again the cross modal terms are not negligible and should be accounted for in the mean square response. In addition there appears to be no basis for combining the high frequency modes into a single response and then combining this quasistatic response with the amplified modes using SRSS. This method has been suggested[15] as an appropriate method of combination for these situations but apparently requires a more detailed examination.

Thus Equation (35) can be used to calculate the mean square response of an oscillator attached to the structure of interest. Since the maximum value of oscillator response is related to the mean square response it only remains to determine this relationship which is discussed in the next section. Variations in the method of calculation of the mean square response are discussed in the literature. M. P. Singh[16,17,18,19] and his colleagues, in particular, have been very active in introducing and developing these applications of random vibration methodology to response spectra transfer.

Peak Value Factor

It was noted earlier that the response spectrum resulting from a limited segment of a random process is a random variable, i.e., it must be described statistically. As the duration of the segment is increased the response spectrum shows less statistical variation (see Reference 8, page 549). The numerical studies by Kana, et al., show that for structural response to earthquake time histories, the structural responses have response spectrum which approximate the mean response spectrum calculated from the random vibration methods. The implication is that for the time histories studied (typical earthquakes) the duration was long enough so that only moderate statistical variation occurs. Thus while the relationship between the oscillator mean square response and the peak value (i.e., spectrum value) is not deterministic, it also does not show excessive statistical dispersion (see Reference 12, Figures 6.2-3, 6.2-4, and 6.2-9). The probability distribution for maximum response in a limited segment of a Gaussian motion varies between a Gaussian distribution for broadband frequency content and a Rayleigh distribution for narrow band frequency content (see Referenc 8). Thus a basis exists for predicting the peak value (i.e., response spectrum value) from a knowledge of the mean square response.

A quantity called the peak value factor, F_o, can be employed to relate the root mean square value of the oscillator acceleration to the maximum value of this quantity. The analysis procedures for transferring a response spectra use peak value factors at two steps. Once in determining the input PSD for a given input response spectra and later for the inverse process of determining the output response spectra from an output PSD. Sensitivity studies for floor spectra have indicated that response spectra transfer through buildings is relatively insensitive to the peak value factors employed (see Reference 17). For this case the input is broad band and the output is relatively narrow band. For the problem of transferring a floor input motion through an equipment supporting structure, the input can be broad band (lower floors) or narrow band with several distinct bands containing the dominant frequency content. It is likely this process also will be insensitive to the peak factor chosen. However, since no confirming studies have been performed the question remains open and confirmatory studies would be useful.

Several approaches for calculating the peak factor are popular in the literature.[20-25] One is

$$F_o = \left[2\ln\left(\frac{\dot{\sigma}_o t_d}{\sigma_o \pi}\right) + 0.577 \right]^{1/2} \Bigg/ \left[2\ln\left(\frac{\dot{\sigma}_o t_d}{\sigma_o \pi}\right) \right]^{1/2} \tag{39}$$

which is based on $R(\omega_o)$ (the response spectrum value) representing the mean of the maximum response probability distribution. Here σ_o and $\dot{\sigma}_o$ are the standard deviation of the response and the time derivative of response, and t_d is the earthquake duration. The other relationship commonly employed is

$$F_o = \left\{ 2\ln \left[-\frac{1}{\pi} \frac{\dot{\sigma}_o}{\sigma_o} \frac{t_d}{\ln(1-p)} \right] \right\}^{1/2} \tag{40}$$

and is based on the actual maximum value having a probability, p, of exceeding the response spectrum value. Neither of these approaches has been reviewed in detail although it is apparent that they were developed specifically for relating maximum values to the mean square value of Gaussian stationary processes.

For the interim it is judged acceptable to transfer response spectra using either type of mean peak value factor. This assumes that the input and output processes are sufficiently similar in their statistical structure to warrant use of the same peak value factor. The studies by M. P. Singh (see Reference 17) on transferring broad band response spectra through buildings for the generation of floor spectra confirmed this assumption. However the process of response spectra transfer from floors through support structures is sufficiently different to warrant a separate evaluation.

Evaluation of Input PSD

In order to evaluate Equation 35 for the oscillator mean square response it is necessary to possess an input PSD. The seismic input is prescribed by a response spectrum so that the input PSD must be derived from this response spectrum. It will be shown below that under reasonable assumption a unique PSD can be determined from a response spectrum.

As discussed in the last several subsections the response spectrum is described in terms of the mean square response of the light oscillator with frequency ω_o, i.e.,

$$R^2(\omega_o) = F_o^2\sigma_o^2(\omega_o) = F_o^2 \int_{-\infty}^{\infty} \Phi(\omega) \frac{(\omega_o^4 + 4\omega_o^2\zeta_o^2\omega^2)d\omega}{(\omega_o^2 - \omega^2)^2 + 4\omega_o^2\zeta_o^2\omega^2} \tag{41}$$

Consistent with the known frequency content of earthquakes, it is necessary to deal only with frequencies less than 33 Hz. In order to simplify the inversion of Equation (41) we assume the oscillator damping is low

$$\zeta_o \sim 1\%$$

$$\zeta_o^2 \sim 10^{-4}$$

and $(4\omega_o^2\zeta_o^2\omega^2)$ is very small compared to ω_o^4 for $\omega \sim \omega_o$.

It is thus possible to a high order of accuracy to write

$$R^2(\omega_o) = F_o^2\sigma_o^2(\omega_o) \approx \int_{\infty}^{\infty} \frac{\Phi(\omega)\omega_o^4 d\omega}{(\omega_o^2 - \omega^2)^2 + 4\omega_o^2\zeta_o^2\omega^2} \tag{42}$$

and $$\left|H_o(\omega)\right|^2 \approx \frac{\omega_o^4}{(\omega_o^2 - \omega^2)^2 + 4\omega_o^2\zeta_o^2\omega^2}$$

where $H_o(\omega)$ is a frequency response function linking output acceleration to input acceleration. This frequency response function has the property that $H_o(\omega)^{\ 2} \approx 0$ for $\omega >> \omega_o$. In addition the transfer function has a very large peak at $\omega \sim \omega_o$ so that in the frequency range of interest the mean square value has a contribution from two frequency ranges. One frequency range is near $\omega \sim \omega_o$ and the other frequency range is $0 < \omega < \omega^* < \omega_o$. Thus

$$\sigma^2(\omega_o) = \int_o^{\omega^*} \frac{\Phi(\omega)\omega_o^4 d\omega}{(\omega_o^2 - \omega^2)^2 + 4\omega_o^2\zeta_o^2\omega^2} \tag{43}$$

$$+ \int_{\omega^*}^{\omega_o + e(\omega)} \frac{\Phi(\omega)\omega_o^4 d\omega}{(\omega_o^2 - \omega^2)^2 + 4\omega_o^2\zeta_o^2\omega^2}$$

and in the 2nd integral

$$\int_{\omega^*}^{\omega_o + e(\omega)} \frac{\Phi(\omega)\omega_o^4 d\omega}{(\omega_o{}^2 - \omega^2)^2 + 4\omega_o^2\zeta_o^2\omega^2} \simeq \Phi(\omega_o) \int_{\omega^*}^{\omega_o + e(\omega)} \frac{\omega_o^4}{(\omega_o{}^2 - \omega^2)^2 + 4\omega_o^2\zeta_o^2\omega^2} \, d\omega \tag{44}$$

The inversion process is started with an oscillator whose frequency is near zero. For $\omega_o \sim$ small only the frequency response function peak and the frequency content near the origin can contribute to the mean square response. In this region the PSD is

$$\Phi(\omega_o) \sim \frac{R^2(\omega_o)}{F_o{}^2} \int_o^{\omega_o + e(\omega)} \frac{\omega_o^4}{(\omega_o{}^2 - \omega^2)^2 + 4\omega_o^2\zeta_o^2\omega^2} \, d\omega \tag{45}$$

and $\varepsilon(\omega)$ is a value of frequency sufficient to have allowed the frequency response function to reach a negligible value. As one progresses to higher values of oscillator frequency the first integral in Equation (43) is numerically evaluated using the portion of the PSD already calculated. This integral represents the background (Singh from Reference 18) mean square response. Solution of Equation (45) using Equation (43) thus allows the PSD to be determined for progressively higher oscillator frequencies. Consequently if the frequency content varies slowly a unique PSD can be inferred from the response spectrum.

Unruh and Kana[26] have proposed a simple iterative procedure for determining a spectra consistent PSD. The iterative method is essentially consistent with the decomposition of the mean square response into two integrals as in equation (43). It is expected that the method will rapidly converge even for floor response spectra where the frequency content varies much more rapidly than for seismic ground motion. Unruh and Kana, and Kana et al., have also examined the statistical variation of the process by computing the response spectrum and PSD from a realistic time history (i.e., continuous slowly varying frequency content with random phasing). If the inversion above is used to independently predict the PSD from the RS and this PSD matches a separate PSD derived independently from the time history, then it provides an indication that all spectrum consistent PSDs fall within a reasonable band. The process was repeated for several earthquakes and good correspondence was achieved. Also of significance is that the duration corrections of Rosenblueth and Elorduy (see Reference 12) was employed

$$\zeta_{eff} = \zeta_o + 2/(\omega_o T) \tag{46}$$

where ω_o is the oscillator frequency in radians/sec and T is the earthquake duration in seconds.

This tends to validate that oscillator damping can be corrected to approximately account for the effects of duration.

Thus, the PSD can be determined from a specified response spectrum. This PSD should be evaluated using the corrected damping to approximately account for the effects limited duration.

Concluding Remarks on Response Spectra Transfer

Reference has been made to properties of realistic seismic time histories as having no concentrated frequency content, slowly varying spectral density, and random phasing. Earlier studies conducted in this program lead to the examination of response spectra dispersion wherein several response spectra consistent time histories lead to transferred response spectra with widely varying peaks. These peaks are most pronounced when the oscillator frequency coincides with a structural frequency, i.e., tuned conditions. An explanation was proposed based on structural motion composed of discrete frequency components near the tuned frequency. Only frequency content near the tuned frequency need be considered because the frequency response function is very nearly ($\omega_o{}^4/2\zeta_o$) near the tuning frequency and negligible elsewhere. The oscillator motion is made up of the same frequency components as the structure but the magnitudes are amplified by the frequency response functions. However, due to the dependence of oscillator phase angle change on frequency, phasing of these concentrated frequency components in the oscillator and the structure is not similar. Thus if several spectrum consistent time histories are considered, the oscillator's components within the dominant frequency range are not phased in the same manner as the structure and thus need not combine to yield approximately the same peak response value. It should be noted that this explanation is based on time histories with concentrated frequency content and thus do not meet the proposed criteria for realistic time histories. The dispersion mechanism stated above clearly does not apply to time histories with continuous frequency content and random phasing because the phasing is random in both the structural response and the oscillator response. For the idealized situation only a moderate amount of statistical dispersion is inherent. It would be very worthwhile to study/verify this situation (numerical sensitivity studies) since knowledge that response spectra transfer is a relatively stable process for realistic time histories would enhance the rational use of the response spectra in design and testing.

Mention should be made of the direct response spectra transfer methods described by M. P. Singh and his colleagues (see References 16 through 19). In this method a PSD is not required because all terms in the mean square response equation are approximated using the relative displacement, pseudo-velocity, and pseudo-acceleration response spectra. This method has been fully developed and verified for generating floor spectra in buildings. They have also developed other direct methods for special situations (significant high frequency modes, floors near ground level, etc.) where non-standard response spectra quantities are employed, viz., total acceleration spectra and relative velocity spectra. The extension of these methods to generating response spectra in support structures has not been verified although future research may rectify this situation. Therefore it is not possible to recommend a direct method today.

The recommended procedure is to develop a response spectrum consistent PSD using an appropriate correction for duration, calculating the output PSD including the effects of all cross modal terms and multiple directions of excitation, integrating this PSD to determine the mean square response and finally determining the response spectrum value from the root mean square response and an appropriate peak value factor.

FLOOR RESPONSE SPECTRA FOR EXPERIENCE DATA BASE

The Seismic Qualification Utilities Group (SQUG) has proposed that the estimated ground response spectra (GRS) be used as the estimate of the floor response spectra. The geologic conditions at the data base plants are such that significant soil-structure interaction is not expected. The vast majority of equipment entered into the SQUG data base is located less than 40 ft above the building foundation. Thus only minor modification of the ground motion may occur on the floors of interest. Equipment located at a higher level will be identified as such in the SQUG data base. Since the peaks and zero period acceleration of floor response spectra are normally elevated above the corresponding values in the GRS, SQUG has suggested the FRS will always envelope the GRS. This method would provide an alternative to analysis for estimating floor response spectra less than 40 ft above the foundation in the current data base plants.

This premise has been examined on a straight forward basis. It cannot be justified in general because it is possible for the frequency content of the building motion to be substantially different than the input frequency content. This can lead to the FRS crossing below the GRS in a frequency range where the floor has diminished frequency content (see Figure 1). However for the data base plants where the soil-structure interaction is negligible and attention is placed on elevations less than 40 ft above foundation the frequency content is judged to undergo only minor changes. In this case modification in response spectra from the ground level to the floor level is reduced and the proposed approximation should be sufficiently accurate.

Additional information to evaluate this assumption was found in three existing building analysis.[27,28,29] One building analysis was for a non-nuclear power plant (El Centro, California). The results show that calculated floor response spectra for floors near foundation level envelope the ground response spectrum. These studies included soil-structure interaction efforts. The soil-structure interaction did not cause the floor response spectrum and the ground response spectrum to cross one another (Figure 1). In addition, the floor response spectra for the floors to about elevation ~40 ft were compared against the foundation response spectrum. Here the elevated floors have response spectrum only slightly different than the foundation spectra and these spectra envelope the foundation spectra. Thus these practical examples verify the adequacy of employing ground response spectra to approximate floor response spectra where the floor is less than 40 ft above the foundation and there is negligible soil-structure interaction.

NEW DEVELOPMENTS IN IN-SITU PROCEDURES OR THEIR APPLICATIONS

Reference 4 (also see Part A of this report) discusses the possible uses of in-situ procedures for qualifying equipment in currently operating plants. These assessments were made prior to December 1982 but the field of in-situ procedures has been continually monitored in the hope of identifying additional methods of application. No new applications have been identified and the original assessments continue to reflect the feasible applications of in-situ procedures.

There is considerable activity in the area of improved in-situ testing methodology.[30-33] These improved methods are currently in the development and validation stage but are judged to have potential use for nuclear power plant application. The improvements result from providing excitation loads simultaneously at several points on a structure. The character of the applied excitation is random so that the entire frequency domain of interest can be evaluated at one time. Modal extraction tasks must be performed with recently developed modal extraction software.

As the number of joints and/or connections along active load paths increases and as the size of a structure increases it becomes increasingly more difficult to obtain reliable test data. A threshold in load level must be exceeded to exercise the system along it's universal load paths. Test data representative of the true natural frequencies and mode shapes can only be developed if the load paths are properly exercised. As well, single point excitation may not load remote load paths at a level comparable to local load paths and consequently elevated levels of loading are required. However, exciting a structure through a single point at the required load levels may be undesirable. Many structures will not have any areas on which the desired load can be applied without the possibility of local damage. By applying random excitation simultaneously at several locations the maximum levels of loading can be substantially reduced while uniformly exercising the significant load paths. Favorable results have been obtained in testing aircraft and missile type structures. For these structures a problem area is the transmission of load through joints/connections and these tests indicate the new methods are considerably more effective. The reported studies show that natural frequencies and mode shapes stabilize at low load levels (compared to single point excitation), and that closely spaced modes are more easily (readily, in fact) identified and extracted. Furthermore, there is indication that in some cases this method requires less total testing and modal extraction effort.

Modal extraction using data from multi-point random excitation testing has only become possible recently. The modal extraction software are described in recent literature (see References 30 and 31). In the future several versions of the necessary software will probably be available on a commercial basis. In the case that adequate validation has been provided there is no reason these methods cannot be used for modal testing of appropriate systems in nuclear power plants. If a situation arises where single point excitation cannot meet the testing acceptance criteria proposed by the NRC, then multiple point random excitation may provide an acceptable alternative. If experience shows the method requires less total testing and analysis effort then the method probably will become popular.

The results of in-situ testing must be combined with analysis procedures in order to predict the required response spectrum for equipment located within support structures. These technical areas have already been discussed in detail in earlier subsections of this report as well as Reference 4. The combined use of modal properties (damping excluded) extracted from in-situ testing and seismic analysis methods to transfer input response spectra is a new concept and relatively untried. Consequently no studies have been identified which are adequate to illustrate experimental verification of the methodology. The studies which are the most closely related have been performed by Southwest Research Institute for the NRC (see Reference 6). However unexplained anomalies in the test results render it impossible to determine whether or not the tests verify the basic response spectra transfer methodology discussed in this report. A need continues to exist for an combined experimental/analytical study directed explicitly at testing the methodology.

CONCLUSIONS

A method for predicting the modal participation factor directly from an incomplete set of structural mode shapes has been presented. The method is suited for alternate qualification methods using in-situ procedures and analysis methods in combination to predict the required response spectra for equipment located in support structures. It is recommended as an acceptable procedure for determining this quantity. A method of analysis for predicting required response spectra for equipment mounted within support structures has been presented and justified. This method is based on treating earthquakes and their consequences as random vibrations. Since the method must transfer an input power spectral density function a method for determining the input power spectral density has been recommended. The method is well suited for predicting the following response spectra within support structures: either required response spectra, response spectra experienced by equipment in real earthquakes, or response spectra experienced by equipment during qualification testing. It is recommended that this method of analysis for transferring response spectra through support structures be accepted as an alternative to the time history method as defined in Regulatory Guide 1.92.

The adequacy of using ground response spectra to estimate floor response spectra in experience data base plants has been reviewed. It has been concluded that this approximation is adequate for floors less than 40 ft from the foundation. The conclusions and recommendations above have been incorporated into proposed guidance and acceptance criteria (reported elsewhere) for use of combined in-situ and analysis procedures in operating plant equipment qualification.

Studies verifying the combined in-situ and analysis methodology have been sought. A major requirement in any such study is determination of mode shapes and natural frequencies at low levels of excitation typical of in-situ procedures and also at levels of excitation associated with Safe Shutdown Earthquakes. It has been concluded that no studies providing a suitable evaluation of the combined use of in-situ and analysis methods currently exists. A review of upcoming developments in in-situ procedures has identified that multiple point random excitation procedures will be a useful adjunct to the current single point excitation methods. The method will be useful in the future in many cases where single point excitation is inadequate.

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PART C: GUIDANCE AND ACCEPTANCE CRITERIA FOR APPLICATION OF COMBINED IN-SITU AND ANALYSIS PROCEDURES IN OPERATING PLANT EQUIPMENT QUALIFICATION

SUMMARY

The Nuclear Regulatory Commission (NRC) has designated seismic equipment qualification of active safety related equipment in currently licensed and operating nuclear power plants as unresolved safety issue A-46 (USI A-46). Alternate methods for seismic qualification of equipment are being developed as part of the resolution to USI A-46 and are described in other documents. These alternatives may employ the use of experience data for establishing seismic capacity, and the combined use of in-situ procedures and analysis methods for estimating seismic environments. Several technical procedures are involved in applying these tools to the alternate methods of qualification, and guidance and acceptance criteria governing the use of these procedures are required. These guidance and acceptance criteria are presented in this report with one exception. Guidance and acceptance criteria for physical similarity of equipment are being developed for USI A-46 in a separate effort and are not reported here.

A detailed analysis of the procedures which will be used in applying the alternate qualification methods yielded 17 technical areas which require the development of associated guidance and acceptance criteria. Each of these technical areas deals with one or more of the following qualification issues: (a) seismic dynamic environment, (b) in-situ testing procedures, (c) seismic capacity, (d) functional requirements, and (e) floor input motion. Each technical area is defined by a title and a short description. This description of the technical areas is provided in tabular form and the reader is referred to Table 1 in the report.

Each technical area is introduced and discussed separately. The discussion is brief and limited primarily to a statement of the guidance and acceptance criteria. For technical areas where the stated guidance requires extensive technical justification, the justification is provided elsewhere. The guidance and acceptance criteria are organized in a format similar to a technical standard. They cover the following general technical subjects:

o Performance of in-situ testing and modal extraction

- Analysis methods
- Functional requirements
- Experience floor spectra
- Margin
- Support structure linearity
- Structural integrity of mountings
- Enveloping criteria.

Since the discussion of most of the 17 technical areas is brief it is not practical to summarize the guidance and acceptance criteria for each area and the reader is referred to the report for this information.

The guidance and acceptance criteria have been organized into the form of a technical standard covering and facilitating the use of in-situ procedures, analysis methods, and experience data in seismically qualifying equipment in operating plants. Thus, they also can be used as a portion of the basis for reviewing qualification submittals using the alternate qualification methods being developed in USI A-46.

GUIDANCE AND ACCEPTANCE CRITERIA FOR APPLICATION OF COMBINED IN-SITU AND ANALYSIS PROCEDURES IN OPERATING PLANT EQUIPMENT QUALIFICATION

INTRODUCTION

Seismic qualification of active Class 1 equipment requires equipment specific considerations of the functional requirements, the Safe Shutdown Earthquake (SSE) dynamic environment, and seismic capacity. To avoid confusion, note that the qualified seismic capacity generally represents a lower limit to the actual seismic capacity. When "seismic capacity" is indicated in this report, it always refers to the qualified seismic capacity. Proof that the qualified seismic capacity exceeds the SSE dynamic environment provides a basis for seismic qualification.

Alternate methods for seismic qualification of equipment in operating plants are being developed as part of the Nuclear Regulatory Commission's (NRC's) Unresolved Safety Issue (USI) A-46.[1] The alternatives under consideration include the use of experience data for establishing seismic capacity, and the use of combined in-situ test and analysis procedures for estimating seismic environments and establishing dynamic similarity. Past work has indicated that both steps are feasible. The current need is for a detailed evaluation of the procedures which will be used in applying the alternate qualification methods. The final evaluations are organized into the form of a standard which provides guidance and acceptance criteria necessary to consistently apply the alternate qualification methods.

The next section identifies the technical areas for which guidance and acceptance criteria are necessary. The final section discusses the technical areas and associated guidance.

TECHNICAL AREAS REQUIRING GUIDANCE AND ACCEPTANCE CRITERIA

For the alternative qualification methods being developed, various steps or technical areas require the development of associated guidance and acceptance criteria. Table 1 identifies and defines the pertinent technical areas with the most important items listed first. This table addresses technical areas associated with both the use of combined in-situ and analysis procedures, and the use of experience data for qualifying equipment in operating plants. The first column in the table is the number assigned to the technical area. The second column gives a short title. The third column identifies which of the following qualification issues are impacted:

(I) = Seismic Dynamic Environment

(II) = Seismic Capacity

(III) = Functional Requirements

(IV) = Floor Motion Input

(V) = Testing Guidelines and Acceptance Criteria.

The final column is a short description of the technical area.

Discussion of Technical Areas Listed in Table 1

1. Dynamic Parameters from Tests

   Natural frequencies and mode shapes determined from low load level in-situ tests will be used in combination with analysis procedures to determine required response spectra of equipment mounted in support devices. Using a similar process the experience response spectra for equipment in nonnuclear facilities subjected to earthquakes can be

TABLE 1. TECHNICAL AREAS REQUIRING GUIDANCE AND ACCEPTANCE CRITERIA

Number	Title	*Impacted Areas	Description
1	Dynamic Parameters from Tests	(I) and (II)	Determination of natural frequencies and mode shapes using in-situ methods.
2	Analytically Determined Dynamic Parameters	(I) and (II)	Determination of natural frequencies and mode shapes using analysis methods.
3	Analysis Methods for Generating Device Required Response Spectra.	(I) and (II)	Use of analysis methods for prediction of Required Response Spectra (RRS) at device locations and Experience Response Spectra (ERS) from experience data.
4	Functional Requirements and Functional Similarity	(III)	Specification of safety related functional requirements for nuclear power plant application. Specification of functional requirements exercised in experience data at nonnuclear facilities.
5	Experience Floor Spectra	(II) and (IV)	Estimation of conservative experience floor response spectra based solely on estimated ground response spectra.
6	Damping	(II)	Damping to be employed in determination of experience data TRS.
7	Modal Participation Factor	(I) and (II)	Modal participation factors to be used in analysis procedures.
8	Fundamental Equipment Supporting Structure Frequency	(I) and (II)	Special requirements to insure lowest equipment supporting structure frequencies are determined.
9	Frequency Margin	(I)	Modifications to fundamental frequencies to maintain margin.
10	Equipment Supporting Structure Linearity	(I)	Justification of equipment supporting structure linearity for prediction of RRS.
11	Enveloping Criteria	(I)	Acceptable procedures for envelopi equipment RRS.

TABLE 1. (continued)

Number	Title	*Impacted Areas	Description
12	Mounting Structural Integrity	(II)	Evaluation of equipment mounting structural integrity.
13	Calibration/ Certification of Equipment, Instrumentation, and Computer Software	(V)	Reporting and certification requirements on test instrumentation and software.
14	Pretest Evaluations	(V)	Requirements pertinent to pretest evaluations.
15	Data Collection	(V)	Reporting requirements on data collection parameters.
16	Calculation of FRFs from Recorded Data	(V)	Certification of software used in generating FRFs.
17	Modal Extraction	(V)	Certification of software used in modal extraction.

* (I) = Seismic Dynamic Environment
(II) = Seismic Capacity
(III) = Functional Requirements
(IV) = Floor Motion Input.
(V) = Testing Guidelines and Acceptance Criteria

estimated. The mode shapes and frequencies used should comply with the following criteria to maintain accuracy and conservatism in predicted results.

All points at which significant masses are attached (>5% of total system mass) should be included as node points of the mode shape description. The node points of the model must also assure adequate spatial resolution of the modes. These measures will enhance the accuracy with which the modal participation factor is estimated. If no less than four points are used in describing the mode shape between the modal antinodes for the significant mode with the largest natural frequency, then the mode shape resolution should be adequate. Thus a structure with five amplified modes will require greater spatial resolution than a stiffer structure with fewer amplified modes. The requirement for spatial resolution holds in all directions, and thus for a cabinet type structure resolution must be considered in both directions in the cabinet plane.

Errors in frequencies appear to be more important than errors in mode shapes. Relatively small changes or errors in frequency estimates can lead to major changes in response level. Thus frequency shifting and/or broadening will be required to maintain margin in view of the potential uncertainties in the frequency estimates. This is discussed in Item 9 and other frequency related considerations are discussed in Items 5 and 8.

A special situation occurs when closely spaced modes exist. A preliminary examination indicated that methods for separating closely spaced modes can be developed but the frequency of occurrence was judged not to warrant an in-depth study. Thus detailed acceptance criteria have not been developed. If closely spaced modes occur the use of in-situ procedures is not precluded. However, both the methods used for determining, and the accuracy of, the estimated closely spaced mode shapes must be justified.

Guidelines to insure against non-linear response are discussed in Item 10.

Guidance is required on the number and position of nodal points for mode shape description. Node points are to be located at all significant masses, and there should be no less than four node points between modal antinodes for the significant mode with the largest natural frequency.

Assurance must be provided that all modes in the frequency range of interest have been determined. Additional guidance concerning natural frequencies is included in Items 8 and 14.

2. Analytically Determined Dynamic Parameters

The natural frequencies and mode shapes can also be estimated using analysis methods. However, experimentally determined dynamic parameters are preferable because the actual base boundary conditions are reflected. Typical base boundary conditions will always allow some relative motion which tends to reduce overall stiffness (compared to an analytical model), resulting in a lower fundamental frequency. Again, because relatively small changes in frequency can result in major changes in response, the frequencies should be experimentally verified.

No evidence has appeared to suggest analytically determined mode shapes require verification. Modal participation factors computed analytically should be sufficiently accurate.

Guidance relating to analytically determined support structure models is that these models are to be verified by comparing computed and experimentally determined natural frequencies. The analytic and experimental frequencies must correlate to a reasonable tolerance--say 10% for frequencies in the range of interest (0.2 Hz to 34 Hz).

3. Analysis Methods

Analysis methods are required to predict the dynamic environment of equipment mounted in support structures. The procedure which is currently accepted involves applying orthogonal time history components to the support structure base, calculating time history responses at selected positions as described in Regulatory Guide 1.92, and then calculating the associated response spectra. The required synthetic floor time histories are probably not available and would need to be generated. Since this method is consistent with current criteria no additional guidance on its use is necessary. However, the cost for analysis may be significant and a simpler method is desirable.

Direct methods for transferring response spectra were studied in FY 82.[2] No basis for transferring the response spectra of general time histories was derived. In fact, it appeared that response spectra transfer is a non-unique process, where two spectra consistent time histories could yield substantially differing transferred response spectra.

An explanation for this occurrence was discussed (see Reference 2). More recent progress has added to the description of the conditions under which substantial dispersion in transferred response spectra are likely.[3] However the properties of earthquake signatures are such that the expected statistical dispersion is moderate. These considerations are discussed in more detail (see Reference 3).

The guidance presented here is in the form of several statements. For additional details and technical justification the reader is referred to Reference 3.

a. Random vibration methods of response spectra transfer are acceptable for predicting RRS of equipment in support structures. The structures natural frequencies and mode shapes may be determined from in-situ procedures. The analysis methods employed should be based on the complete mean square response. The calculation of mean square response requires that the power spectral density (PSD) consistent with the input response spectra (RS) be used in predictions. The theory, associated references, and justification for transferring response spectra using the mean square response approach are presented in Reference 3.

b. The mean peak value factor or an exceedance probability based peak value factor relates the maximum oscillator response (response spectrum value) to the root mean square value of oscillator motion. The peak factor relationship used in analysis must be justified. Insufficient information was generated to provide specific guidance on determination of a peak factor response for the specific class of motions of interest. Preliminary technical evaluation of the problem, however indicates it is tractable and progress to date is discussed in Reference 3.

The time history analysis method is currently accepted (Regulatory Guide 1.92) and the same guidance should be applied to operating plant application. Response spectra transfer using random vibration methods is acceptable; the complete mean square response must be employed, the peak value factors must be justified, the modal participation factors employed must meet the criteria in Item 7, and all significant modes must be included in the structural model. Additional details are available in Reference 3.

4. Functional Similarity and Functional Requirements

Documentation and verification of functional similarity is necessary during the process of using experience data to qualify safety-related equipment. The experience data base must identify the specific functional roles which were exercised during and/or after the seismic event. The utilities will be supplying the entries to the experience data base including itemization of the functions performed during and/or after the seismic event. Procedural guidelines are required so these functional assessments can be audited or reviewed by the NRC.

The approach suggested is to divide equipment into several groups and develop a comprehensive list of potential functional requirements for each group. When experience data is entered into the experience data base it should identify the functional requirements from the list that were adequately evaluated during and/or after the real seismic event. As the SQUG methodology for determining the evaluated functional requirements evolves and is accepted, these developments can be reflected in the details of implementing requirements on functional requirements.

<u>Guidance on functional requirements--Experience data must identify the functional requirements evaluated during and/or after the real seismic events. Claims that these functional requirements were performed during and/or after the seismic event must be supported by documentation of the methods used to establish these claims</u>.

5. Experience Data Floor Spectra

The SQUG program has proposed that the estimated ground response spectra (GRS) be used as the estimate of the experience floor response spectra (FRS). Since the peaks and zero period acceleration of floor response spectra are normally elevated above the GRS it was suggested the FRS would always envelope the GRS. This method would provide an alternative to analysis.

The premise has been examined on a straightforward basis. It cannot be justified in general because it is possible for the frequency content of the building motion to be substantially different than the input frequency content. This can lead to the FRS crossing below the GRS in a frequency range where the floor motion has diminished frequency content. (See Figure 1.) However, much of the experience data equipment is located at elevations near the ground level. In this case modification in response spectra from the ground level to the floor level is reduced and the proposed approximation may be sufficiently accurate.

Several existing building analyses (see Reference 3) have been examined. The results show that calculated floor response spectra for floors near ground level envelope the ground response spectrum. These studies included soil-structure interaction effects. The soil-structure interaction did not cause the floor response spectrum and the ground response spectrum to cross one another (Figure 1). It is reasonable to conclude that the experience data base plants will also have floor response spectra enveloping the ground response spectrum.

The NRC staff has reviewed the Seismic Qualification Utilities Group estimates of the data base plant ground response spectra and accepted these estimates as a conservative approximation.

<u>Providing that ground response spectrum estimates for data base plants have been reviewed and approved by NRC staff, then it is acceptable to use them for estimating floor response spectra for equipment at elevations less than 40 ft from the foundation</u>.

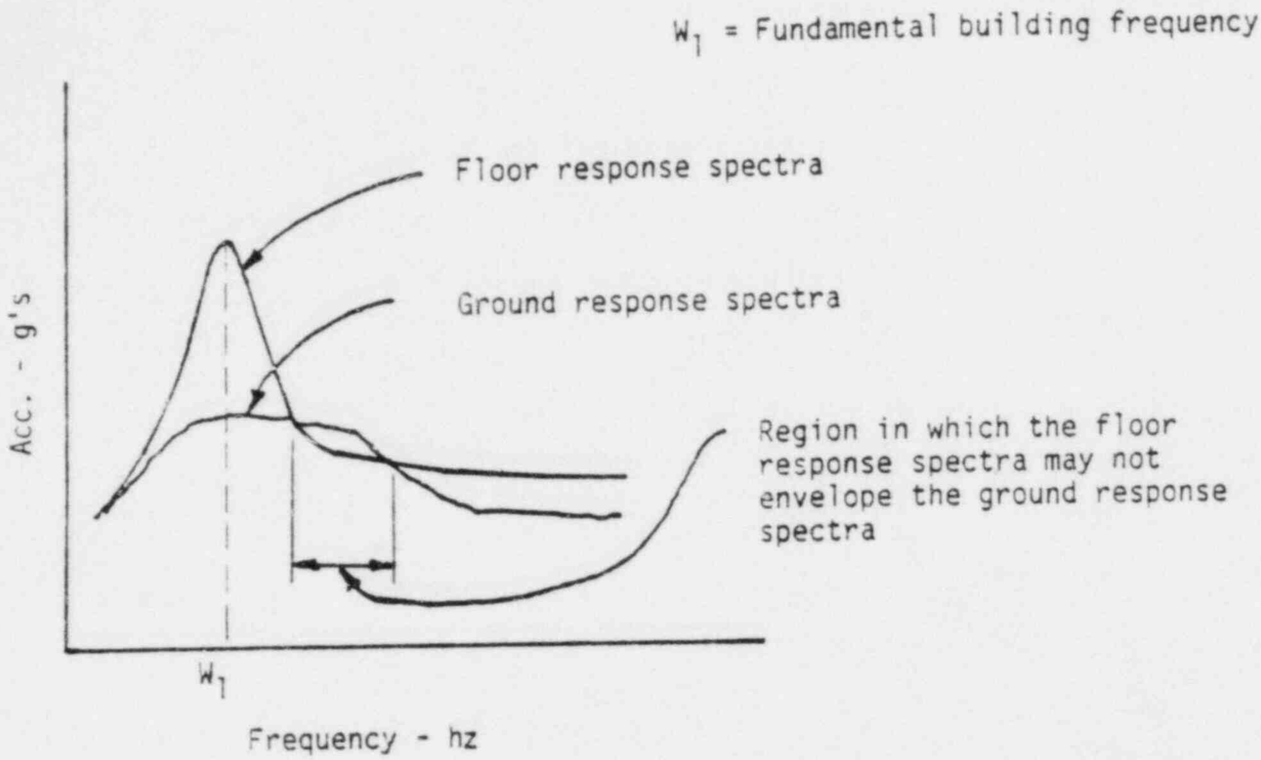

Figure 1. Comparison of floor and ground response spectra.

6. Damping

Damping must be specified to estimate component specific experience response spectra (ERS) from the experience data base. One condition necessary for conservatism in estimating ERS is that the damping not be underestimated because underestimation of damping results in an overestimation of the actual ERS.

The recommended approach is to predict the ERS at several values of damping. The actual ERS used will depend on (1) the type of equipment supporting structure associated with the experience data, (2) the type of equipment supporting structure the nuclear power plant (NPP) equipment is attached to, (3) the damping used to predict the RRS for the safety related equipment and (4) the correlation of damping between the two (NPP and experience data) equipment supporting structures.

Consider a case where 2% damping is used in predicting a component RRS in an equipment supporting structure with known low damping. Experience data is available for the same component with the same functional requirements but mounted in an equipment supporting structure which will likely experience somewhat greater damping. An ERS is then predicted based on a damping of 3-4% to assure a conservative estimate of seismic capacity.

This comparative approach has the benefit that damping need not be estimated with high precision. This is an important factor since damping values seem to vary over a large range. A common condition may be that the equipment for which the comparison is made are located in physically similar support structures. Such similarity (i.e., built to a common industry standard by a common manufacturer) is adequate justification for use of a common damping value for ERS and RRS estimates.

Guidance relating to damping is that experience data for equipment in support devices be estimated based on a range of damping ratio values (3%, 5%, and 10%). The same damping value should be employed for RRS and ERS predictions when the support structures are physically similar and are anchored to the plant floor in a similar fashion.

7. Modal Participation Factor

The modal participation factor (MPF) is used in estimating the required response spectra for equipment. Accurate estimates are not normally available directly from in-situ test results and special estimation methods are required.

One acceptable method is to calculate the MPF from a mass matrix developed using a physical description of the equipment supporting structure and attached equipment. This method assures that the system's total mass is exactly duplicated in the analysis and that this mass is properly distributed to the node points of the model. Locating node points at the center of gravity of equipment with significant mass will assist in achieving this proper mass distribution. This method is inherently sound and requires no further justification. The information necessary to reconstruct the MPF should be retained in the permanent qualification documentation.

A second method of determining the MPF is also acceptable. The justification for this method is discussed in Reference 3. The method is based on reconstructing the load vector from an incomplete set of mode shapes. The load vector for the equation of motion in standard form is (one degree of freedom at each node and base input in one direction).

$$\{I\}\ \ddot{y}_b(t)$$

where $\{I\}$ is a vector whose components are 1s and $\ddot{y}_b(t)$ is the base acceleration time history.

Basically the incomplete modes $[\phi]^*$ must reconstruct the load vector whose components are all ones, i.e.,

$$[\phi]^*\ \{\Gamma\}^* \equiv \{I\} \text{ or } [\phi]^*\ \{\Gamma\}^* - \{I\} = \{R\}$$

where $\{\Gamma\}$ is the vector of MPFs and $\{R\}$ is an error vector. Minimizing the usual measure of the vector $\{R\}$ achieves a MPF vector which reconstructs the load in an optimal fashion. The equation for the MPF is

$$\{\Gamma\}^* = ([\Phi]^{*T} [\Phi]^*)^{-1} [\Phi]^{*T} \{I\}$$

Proposed guidance is to determine the mass matrix ([M]) from physical characteristics of the system and calculate MPF according to the following equation

$$\frac{\{\phi\}_i^T [M] \{I\}}{\{\phi\}_i^T [M] \{\phi\}_i} = MPF_i$$

An alternative method is to use the equation

$$\{\Gamma\}^* = ([\phi]^{*T} [\phi]^*)^{-1} [\phi]^{*T} \{I\}$$

and verify that the body force load is well simulated, i.e.,

$$\left|\{R\}\right| / \left|\{I\}\right| \leq 0.05.$$

Other methods for approximating the MPF must be justified and will be evaluated on a case by case basis.

8. Fundamental Frequency Determination

Elevated floor response spectra in nuclear power plants normally contain a strong peak at the building's fundamental frequency. Since this peak is often below 5 hertz it is important to accurately estimate any equipment supporting structure natural frequencies in the low frequency range. Obtaining accurate transfer function resolution at low frequencies can sometimes be a problem when performing in-situ tests. A factor is the difficulty of applying sufficient low frequency input with hammer blows. Therefore special attention should be paid to accurately establishing the lowest natural frequency. Many support devices will have a fundamental frequency outside the highly amplified region of the floor response spectra. Here again it is important to verify that testing in the low frequency range was adequate to insure there are no corresponding modes.

If the low frequency components of experimentally measured signals have insufficient strength then poor noise to signal ratios and inaccurate estimates of frequency response functions result. The coherence function reflects this effect of noise in measurements. The coherence is 1.0 for no noise and decreases as the contribution of noise increases. A coherence of 0.8 should be maintained in determining the transfer functions in the low frequency range (2 to 8 hertz). Another acceptable check on acceptability of data is to plot the magnitude and phase of acceleration/force (a/f) or displacement/force (d/f) driving point frequency response functions. These FRFs have a well defined character at frequencies less than the first fundamental frequency of the structure.[4] Excessive noise to signal ratio will not allow these FRFs to maintain these features. The identifying features are that the d/f magnitude is constant and a/f varies with frequency squared until the first fundamental frequency is reached. In addition, both FRFs have constant phase angle until the first fundamental frequency is reached. Upon reaching the first frequency rapid phase angle and frequency response magnitude changes occur. Experimental data clearly illustrating this behavior provides an adequate definition of the first natural frequency.

Often a natural frequency will not occur in the low frequency range where it is most difficult to obtain data adequate for modal extraction tasks. Verifying that the first natural frequency does not occur below a given frequency provides the basis for analyzing data only at larger frequencies. This approach may prove to be practical because the signal/noise ratio improves dramatically near the natural frequencies. The overall FRF may have a poor noise/signal ratio yet the data may be capable of identifying the position of the fundamental natural frequency. If this type of approach is taken then adequate validation of the test methodology must be supplied.

The equipment supporting structure frequencies are acceptable if the transfer function in the frequency range of interest are determined from data maintaining a coherence of 0.8 or greater at the natural frequencies.

Another acceptable approach is to document that the magnitude and phase angle of the driving point FRF follow rules consistent with the absence of a natural frequency.

Other methods of establishing the low frequency range containing no natural frequencies will be evaluated on a case by case basis until experience warrants the development of general guidelines.

9. Margin

When modal parameters are combined with analysis procedures the exact value of the fundamental equipment supporting structure frequency plays a primary role in determining the resulting shape of device RRS. The significance of the fundamental frequency is even more pronounced whenever a natural frequency is located near a floor response spectra peak where small errors in the in-situ frequency estimates can result in significant errors in the calculated RRS. There are potential sources of uncertainty in the frequency estimate and the introduction of margin may be required to assure conservative results.

The methods for incorporating margin defined below deal with the uncertainty in building structural models and equipment structural models simultaneously. The approach discussed below incorporates an uncertainty of ±10% in the natural frequencies determined using in-situ procedures. In this guidance a time history or PSD consistent with an unbroadened floor response spectrum is employed. The philosophy of Regulatory Guide 1.122 where ±15% margin is applied on building natural frequencies is adopted in treating the seismic floor input uncertainty. However the floor response spectrum used is unbroadened and margin is applied by considering several realistically shaped input response spectra within the bounds of the uncertainty.

Use is made of the fact that Brookhaven National Laboratory in the results of Task 2 of the Task Action Plan for Unresolved Safety Issue A-46 has included one discernible peak in their development of so-called generic floor spectra. If it is desired to account for frequency content in other frequency ranges then modifications to guidance presented herein may be required.

In Figure 2 several frequency regions are defined on a line graph. If ω_s is the best estimate building fundamental frequency and ω_c is the best estimate support structure frequency, then Region 1 is $0.85\ \omega_s \leq \omega \leq 1.15\ \omega_s$, Region 2 is $0.9\ \omega_c \leq \omega \leq 1.1\ \omega_c$ and ΔD is the distance, measured in frequency (Hz) between the two regions as shown in Figure 2. If $\Delta D > 0.1\ \omega_c$ then the two regions are considered to be well spaced (i.e., uncoupled), otherwise they are considered to be coupled. One set of guidance applies if the regions are well spaced and a separate set applies to coupled regions. As noted earlier all guidance presented herein is based on unbroadened floor response.

For well spaced frequencies, either time history or mean square response (i.e., using PSD function) analysis procedures may be used (see Reference 3). The input to the support structure is consistent with the unbroadened response spectra with peak at ω_s. The structure for which an in-structure response spectrum is sought is modeled with its best estimate modal properties. These estimates must

ω_s = building frequency

ω_c = support structure frequency

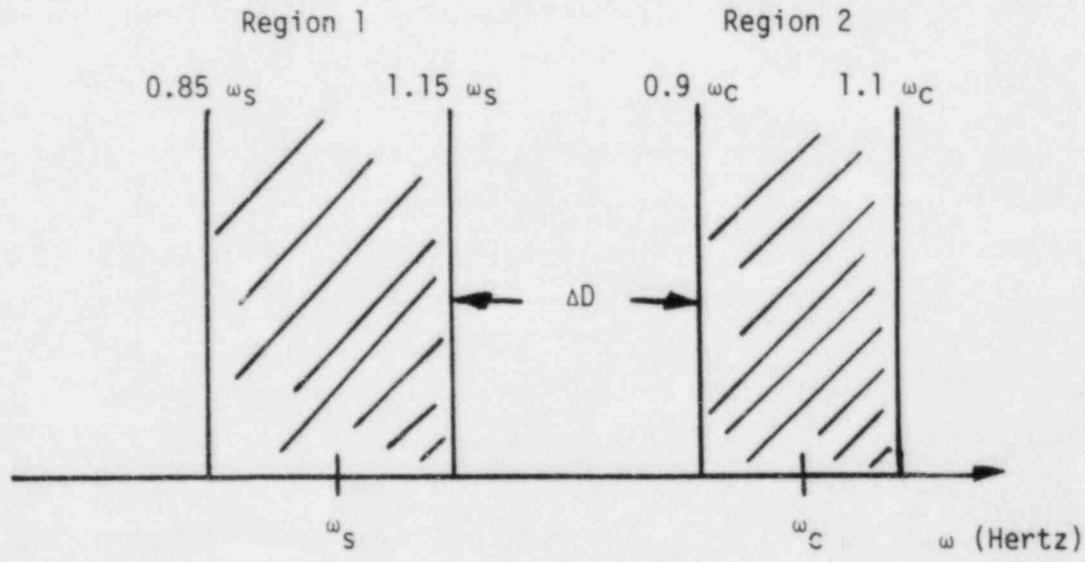

Figure 2. Line graph definition of Region 1, Region 2, and frequency separation ΔD.

be consistent with guidelines presented elsewhere in this document or in existing Regulatory Guidelines. The required in-structure responses are predicted using time history or root mean square procedures. Figure 3 shows the expected features of the in-structure response spectrum. The response spectrum peaks are horizontally extended across Region 1 and Region 2 to apply margin and the remainder of the spectrum is formed in conformance with NRC Regulatory Guide 1.122.

For the situation where Region 1 and Region 2 couple the procedure is somewhat different. Time history methods are not practical because three separate spectra consistent floor time histories are required to use the procedures to be described. Coupling or tuning of building and support structure is not expected to occur frequently. SQUG experience data investigations show support structures natural frequencies above 6 Hz to be the typical situation. This is significant because incorporating margins for building modal parameters and support structure modal parameters is relatively more complicated for the condition where Region 1 and Region 2 couple.

The methodology for estimating secondary response spectra with the incorporation of margin on support structure frequency is now described. Three floor response spectra are defined. These response spectra have peaks at ω_s, $0.85\ \omega_s$, and $1.15\ \omega_s$, respectively. A spectrum consistent PSD is calculated (see Section C of this report) for each response spectrum. Several versions of the support structure modal model are generated. The mode shapes are not modified. One modal model has a set of natural frequencies in which the first mode frequency is $0.90\ \omega_c$. A second model employs a first mode natural frequency of $1.1\ \omega_c$. If Region 1 and Region 2 do not overlap no other support structure models need be considered. The condition where Region 1 and Region 2 do overlap is treated separately, later. The floor input PSD for $0.85\ \omega_s$ is combined with the support device structural model using $0.90\ \omega_c$ as its fundamental frequency

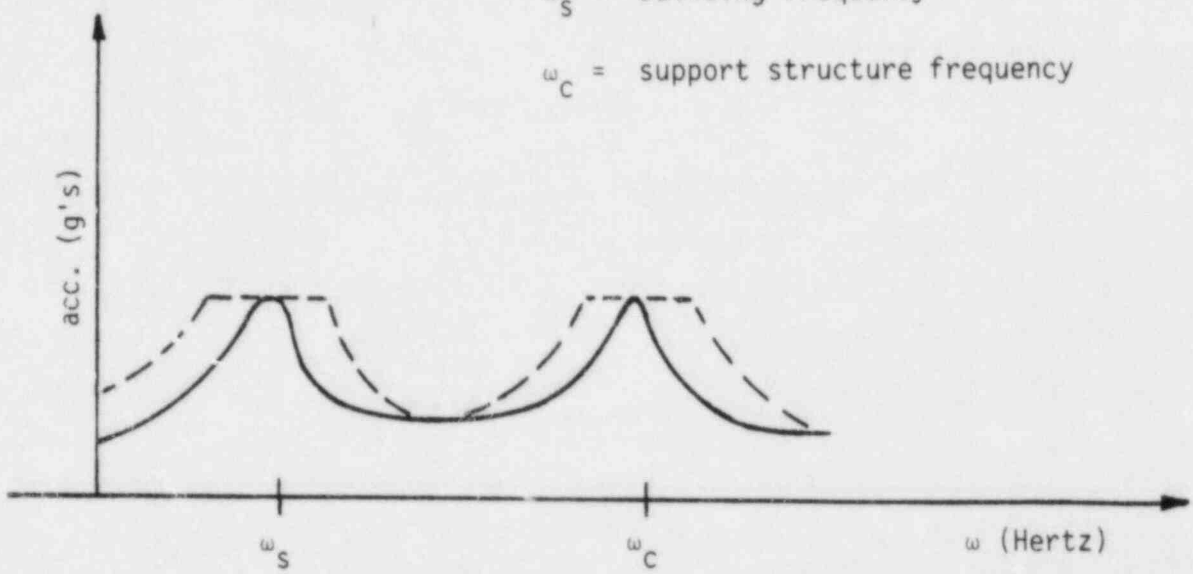

Figure 3. Best estimate in-structure response spectra and broadened response spectra

and a RS is generated using the root mean square approach. A second in-structure response spectrum using a PSD for 1.15 ω_s and fundamental support structure frequency of 1.1 ω_c is constructed. A third response spectra must be determined bv combining the worst combination of building and equipment supporting structure frequency. The modal model frequency is chosen on the boundary to Region 2 (Figure 2) closest to Region 1. The building frequency is chosen on the boundary of Region 1 closest to Region 2. This combination of input and support structure modal properties will yield the response spectra with the largest peak. Finally, a combined response spectrum enveloping these three response spectra is formed and this response spectrum incorporates margin on both building properties and support structure properties.

If Region 1 and Region 2 overlap then the overall procedure is slightly modified. The first two response spectra described above are not modified. A third response spectra is required. An input PSD is generated for a floor response spectra whose peak is at the smaller of 1.15 ω_s or 1.1 ω_c. This input PSD is applied to a structural model with a fundamental frequency also set at the smaller of 1.15 ω_s or 1.1 ω_c. As before the RS are superimposed and an envelope is formed.

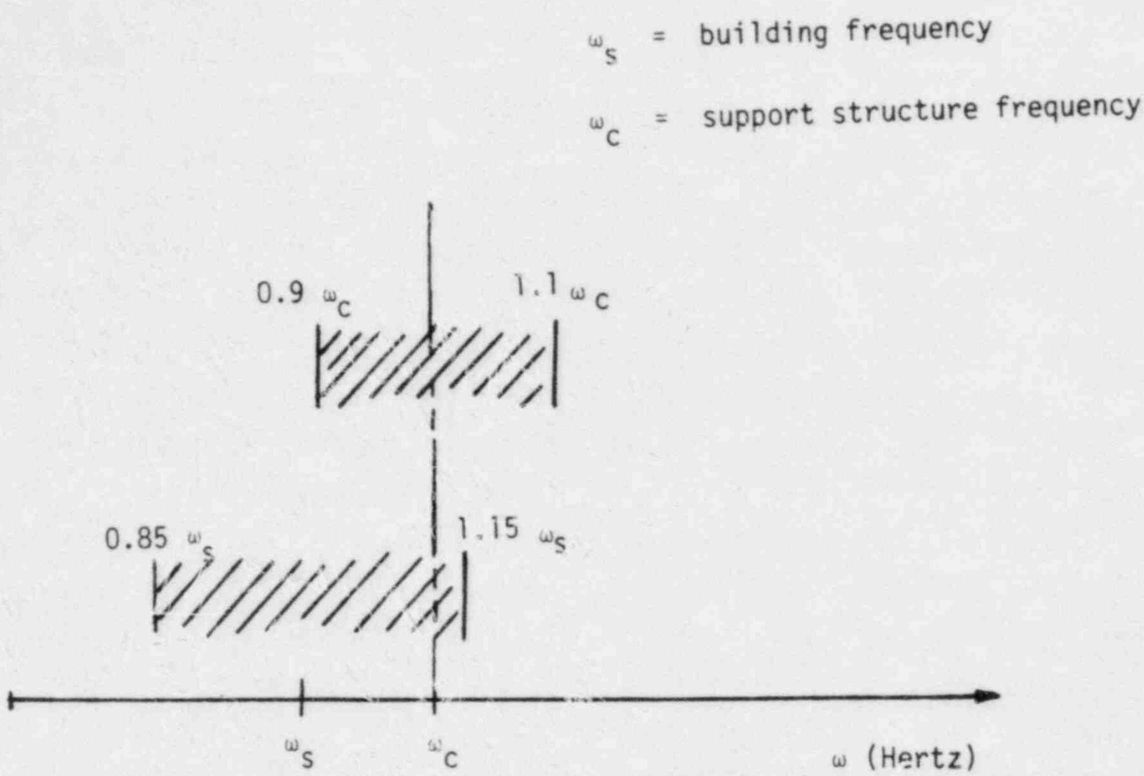

Figure 4. Coupled building and support structure natural frequencies.

Margin against uncertainties in building and support structure natural frequencies should be accounted for using the detailed guidance provided above.

10. Support Device Linearity

Assurance of equipment supporting structure linearity is required to justify use of combined in-situ and analysis procedures. Experience data and prior qualification testing provide assurance of linear material behavior in physically similar systems. Linearity of response is also affected by the base attachment boundary condition. Guidelines which will assure that the base boundary condition is independent of dynamic load level are required to assure stable, i.e., independent of load level, natural frequencies and mode shapes.

The implications are that assurance must be provided the boundary condition is stable and that the fundamental frequency should be determined using in-situ methods. Separate guidance will be required for welded and bolted anchorages. Bolted attachments require acceptance criteria on bolt preload (this is over and above structural integrity requirements). Assurance should be provided that compressive preload is maintained throughout the SSE loading.

Assurance that the equipment supporting structure anchorage is stable must be provided. Welded anchorages should inherently be stable and require no additional considerations provided that structural integrity for the SSE environment has been demonstrated. For bolted anchorages assurance should be provided that installation preloads are not reduced by greater than 70% during the SSE environment. Justification for structural integrity should be provided for all anchorages of equipment supporting structures.

If the existing anchorage design makes this difficult then additional margin should be applied by lowering of natural frequencies with increasing excitation level. A margin of -25%, +10% is suggested for this case.

<u>Support devices attached to the floor using bolt attachments must justify that installation preloads are not reduced by greater than 70% during the SSE environment</u>.

11. Enveloping Criteria

For specific applications it may be advantageous and justifiable to define a frequency range of interest which is narrower than 33 hertz. Normally this would mean ignoring some portion of the low frequency range in the RRS. An advantage for operating plant equipment qualification is that the effect of the low frequency peak in the floor spectra can then be ignored.

For equipment whose rigidity has been established by analysis, current procedures allow one to qualify to the zero period acceleration (ZPA) of the RRS. For operating plant equipment qualification one would compare the required ZPA with the ZPA from experience data on similar equipment. Qualification of equipment supporting structure structural integrity using experience data is another example where defining a frequency range of interest can be of potential usefulness. In this case physical similarity with experience data units is established by similarity of fundamental frequency and mode shape, and a general physical correlation (i.e., manufacturer and model number) between the two equipment supporting structures. The frequency range of interest is defined such that the lower limit corresponds to the equipment support structure fundamental frequency. Provided the experience data spectra envelopes the FRS in the range of interest, the equipment support structure structural integrity is qualified to the experience data spectra.

It may also be useful to define frequency ranges for equipment mounted in equipment supporting structures. Some nuclear power plant equipment may be subjected to greater low frequency input than similar equipment in the experience data base. This NPP equipment may have RRS which exceed data base spectra in the low frequency range (see Figure 5). Again, if it can be verified that equipment operability is insensitive to the low frequency content, then enveloping can be based on a limited frequency range of interest. It is not clear if there is sufficient need for guidance in this area, or whether justifiable guidelines can be developed.

As with current criteria, the experience response spectra for rigid equipment must envelope the RRS at the ZPA. Envelopment at lower frequencies is not essential. For equipment supporting structure structural integrity, envelopment is required only at frequencies greater than the support device fundamental frequency (with 15% margin on frequency).

If justification can be provided that equipment is not specifically sensitive to low frequency inputs (i.e., so that the input does not have to be rich in low frequency content to perform a qualification test) then envelopment can be restricted to the remaining frequency range.

12. Component Mounting Structural Integrity

Combined in-situ and analysis procedures can be employed to evaluate structural integrity of equipment mountings within equipment supporting structures. The floor response spectra is used to predict the maximum motion in each significant mode of the equipment supporting structure. These maximum values are combined by the "square-root-sum of squares" rule to determine an estimated maximum acceleration. The maximum load transmitted through the mounting is predicted by using the equipment mass and the maximum acceleration.

Loads on component mounting can be calculated using dynamic parameters developed from in-situ procedures. An acceptable maximum acceleration is calculated using the peak broadened FRS, the modal parameters, and the analysis methods discussed in Regulatory Guide 1.92. The mass is taken as the sum of the component and mounting fixture masses..

Reporting requirements, and guidance and acceptance criteria for the performance of low load level in-situ tests and for the extraction of natural frequencies and mode shapes from the resulting data are described below. Basically, all information required to audit an in-situ investigation should be maintained by the utility. If the contractor performing testing is aware of information beyond that described below

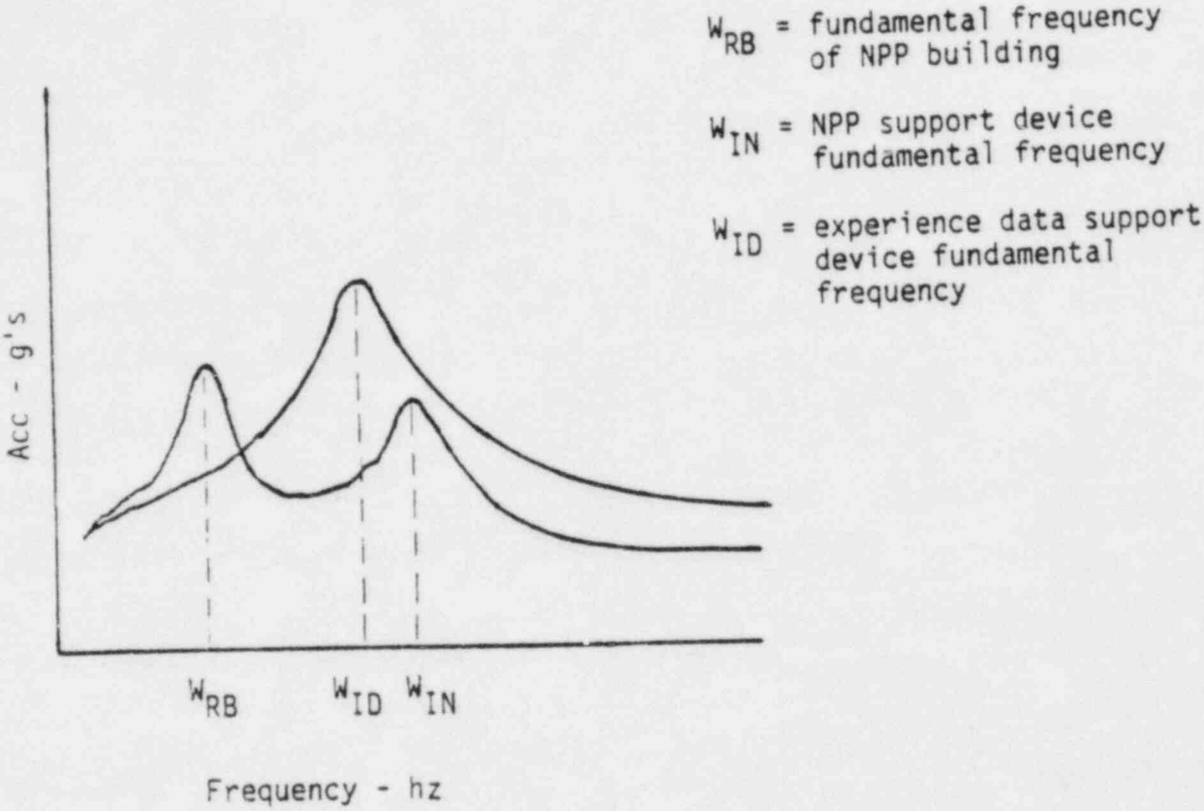

Figure 5. Comparison of envelopment.

which may be essential to auditing his test program then this information should be retained in the testing documentation. An in-situ investigation must pass through five sequential steps. These will be identified as technical Areas 13 through 17 and guidance and acceptance criteria for these steps are presented below. The steps are (1) calibration/

certification of equipment, instrumentation, and software, (2) pretest evaluations, (3) testing and data collection, (4) determination of frequency response functions (FRFs), and (5) modal parameter extraction.

13. Calibration/certification of equipment, instrumentation, and computer software.

    The performance of in-situ testing applied to the determination of natural frequencies and mode shapes requires that the excitation force to the structure and the response (generally acceleration) of the structure be accurately measured. The actual measurements are electrical signals which must be scaled by the instrument calibration curves to yield the desired physical quantities. The method or standards by which the calibration levels are determined as well as the calibration level themselves should be documented. In some situations there will be intervening components between the actual measuring instrument and the physical response of interest (for example the load history applied by a hammer impact may be measured behind the hammer head rather than at the impacting interface). For these cases the method for determining the system calibration should be included with the qualification documentation.

    A verification that the test instruments were stable during the performance of testing should be performed. This can be accomplished by reproducing a driving point frequency response function measured during testing again at the end of testing. The two measured frequency response functions should compare closely if the instrument calibrations and the structural characteristics of the system have been constant during testing.

    Manufacturer's specifications for instruments (accelerometers) used in measuring structural response should be included in test documentation. In particular, this includes the weight of the accelerometers. The calibrated and rated range for instrumentation should be recorded.

A sketch of the system tested illustrating the equipment support structure and the locations of safety related components should be maintained with the qualification documentation. Complete details of anchorage should be documented including method of anchorage and size of fasteners (if used).

Documentation certifying accuracy of modal extraction software should be maintained by the utility for each in-situ test contractor providing these services to the utility.

Guidance with respect to calibration of equipment and instruments is that the calibration procedures used must be recorded and included with the test documentation. These procedures should be referenced to an applicable testing standard if possible. The methods of calibration (system or component), the instrument calibrations and the calibrated range, and manufacturer's specifications for calibration should be included in test documentation. Manufacturer's specifications for instruments (including weight and rated operating range) and equipment should be included with test documentation. A driving point frequency response function measured during the initial stages of testing should be repeated at the completion of testing. These two measurements of the same driving point frequency response function must compare within acceptable limits to verify stability of measurements. The modal extraction software employed should have been certified by the solution of a standard problem. Software certification is discussed further in Item 17. A sketch of the system tested showing overall dimensions, location of Seismic Category 1 equipment, instrumented positions, and detailing of anchorage must be included with documentation.

14. Pretest Evaluations.

Pretest evaluations refer to work performed during test setup. During the pretest phase it is necessary to

- Determine a suitable method of excitation,

- Determine load application points and directions,

- Establish linearity of response and reciprocity of measurements,

- Check the coherence at several active/moving locations remote from exciter,

- Define the nodal points (per guidance in Item 1) for the modal model,

- Approximately determine all natural frequencies in the frequency range of interest by evaluating driving point frequency response functions at several points across structure,

- Determine the lowest natural frequency (covered separately in Item 8).

The major item to be resolved during pretest evaluations is identifying the appropriate method, locations, and directions for exciting the structure. To ensure that all natural frequencies have been determined it is required that excitation be applied at a minimum of three positions for each principle horizontal direction. The driving point frequency response functions at these points should provide the complete set of natural frequencies.

The excitor location to be used in generating the complete set of FRFs should maintain an acceptable value of coherence over the frequency range of interest (0.8 or greater). A coherence check at the natural frequencies between the input point and a remote accelerometer position is also required. In this case it is expected that the coherence will be lower in frequency ranges where the FRF indicates an antinode (a small modal coefficient for a given mode). Over the remainder of the frequency range of interest, the coherence must meet the same standard as the standard imposed at the driving point.

The reciprocity (output at 1 for an input at 2 versus output at 2 for an input at 1) between excitation location and a remote point should be verified. The comparison between FRFs should be sufficiently close to indicate that the same load paths are operating for both cases. Finally, the most representative driving point frequency response function should be evaluated at several levels of loading. The purpose is to demonstrate, in combination with the reciprocity check, that the natural frequencies and mode shapes will remain relatively invariant with excitation level.

15. Data Collection

Data collection procedures are somewhat standard throughout the industry (Reference 5 discusses accepted data collection procedures) and compliance with those standards should be maintained. The qualification documentation should record the following information: (a) total number of data points in sample (b) number of samples used to develop FRFs (c) anti-aliasing filter employed (d) windowing (if used) to prevent leakage in data, and (e) the sampling frequency.

16. Calculation of FRFs from Recorded Data

Discussions with test contractors has indicated that determination of FRFs is a standard operation. It is considered that no special guidance or acceptance criteria is necessary. A requirement to develop FRFs for a standard set of data could be imposed if the NRC staff felt that this level of certification is necessary. If the NRC staff felt certification of software was necessary then a one time requirement for development of accurate FRFs from a standard set of data could be imposed.

17. Modal Extraction

Modal extraction is the process by which experimental FRFs are analyzed in order to determine the modal parameters: natural frequencies, mode shapes, modal damping, equivalent modal masses, and equivalent stiffness. This analysis is performed via specialized computer programs. In fact several approaches will be available within a given software package.

The contractor should identify the developer of the software and the basis for choosing the modal extraction process used.

The major item in auditability of the modal extraction process is validation of the software used in modal extraction. The theory of steady state linear vibrations, Fourier transforms, linear algebra, etc. provides the common basis for modal extraction. However, numerous details are involved in developing computer software for application to modal extraction. Hence a direct check on software accuracy is desirable. In-situ test contractors should certify their software to one or more standard problems. This certification should be maintained by the utility for each such contractor retained for performance of in-situ investigations. Furthermore, it is recommended that the standard problem use data recorded during testing of an equipment supporting structure typical to those found in nuclear power plants.

CONCLUSIONS

The following are some final remarks on the proposed guidance and acceptance criteria presented above. Note that Items 1, 2, 7, and 8 relate to the methods used for determining the dynamic parameters. Items 8 and 13 through 17 are specific to the testing tasks. Item 10 relates to ensuring the usefulness of these dynamic parameters. Many of the other items cover the use of analysis or the application of experience data.

Alternate qualification methods which combine the use of in-situ procedures and analysis methods, or employ seismic experience data have been evaluated to define the procedures requiring specific guidance and acceptance criteria. The result is that 17 technical areas have been defined (see Table 1). The guidance required for these areas has been addressed, and guidance and acceptance criteria were presented in the preceding section as underlined text.

These final evaluations have been organized into the form of a standard which provides guidance and acceptance criteria necessary to consistently apply the alternate qualification methods.

It is recommended the guidance and acceptance criteria above be used in the NRC's regulatory review of operating plant qualification submittals when these submittals employ alternate qualification methods based on combined in-situ and analysis procedures and/or experience data.

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5. "DATM, User Manual," Structural Dynamics Research Corporation, Cincinnati, Ohio, June 1981.

Part D: SEISMIC QUALIFICATION
COST ESTIMATING TASK

LIST OF ABBREVIATIONS

BWR Boiling Water Reactor

CRO Cathode Ray Oscilloscope

DA Data Aquisition

DMM Digital Multi-Meter

DCC Depreciated Capital Cost

DOF Degrees of Freedom

EH Electro Hydraulic

EM Electro Mechanical

F.L.&T. Food, Lodging, and Travel

gpm gallons per minute

k x 1000

m-wk man-weeks

mo month

PWR Pressurized Water Reactor

SQRT Seismic Qualification Review Team

1. INTRODUCTION

In December of 1980 the NRC started a task to address the concern of seismic safety margin in operating plant-equipment. The objective of this task was to develop alternate methods, guidelines, and acceptance criteria for seismic equipment qualification in operating plants. A number of alternate procedures were proposed by the Nuclear Regulatory Commission (NRC). As an aid to the NRC in decision making, EG&G Idaho Inc. made a cost comparison study of the alternate procedures.

The following sections of this report contain explanations of these alternate procedures and the cost estimate associated with each.

2. DISCUSSION

Figure 1 shows the alternate seismic qualification procedures as proposed by the NRC. The following discusses the nodes of the alternate paths. The results of this study consist primarily of a table summarization of the cost estimates. A discussion of the table and background information as to the source of the numbers can be found in the following section titled Results.

NOTE 1: Beyond the scope of this work.

NOTE 2: An estimate was made of the cost of determining equipment/support dynamic characteristics via in-situ testing. Supports are typically either included in the qualification of equipment (e.g. diesel generator skid) or qualified as separate equipment (e.g., panels, racks, cabinets).

NOTE 3: An estimate was made for the cost of comparing dynamic and functional characteristics of equipment in plant and that in the data base.

NOTE 4: The cost of comparing spectra is negligible in comparison with the cost of obtaining the spectral data, therefore, no estimate was made for comparison of spectra.

NOTE 5: A cost estimate of simple support modifications to obtain similarity with the data base was made.

NOTE 6: An estimate of replacement cost was made.

NOTE 7: An estimate of the cost of comparison between qualification methodologies was not made because of the strong dependence of the estimate on plant-specific data. However, an estimate was made for qualifying equipment by analysis only.

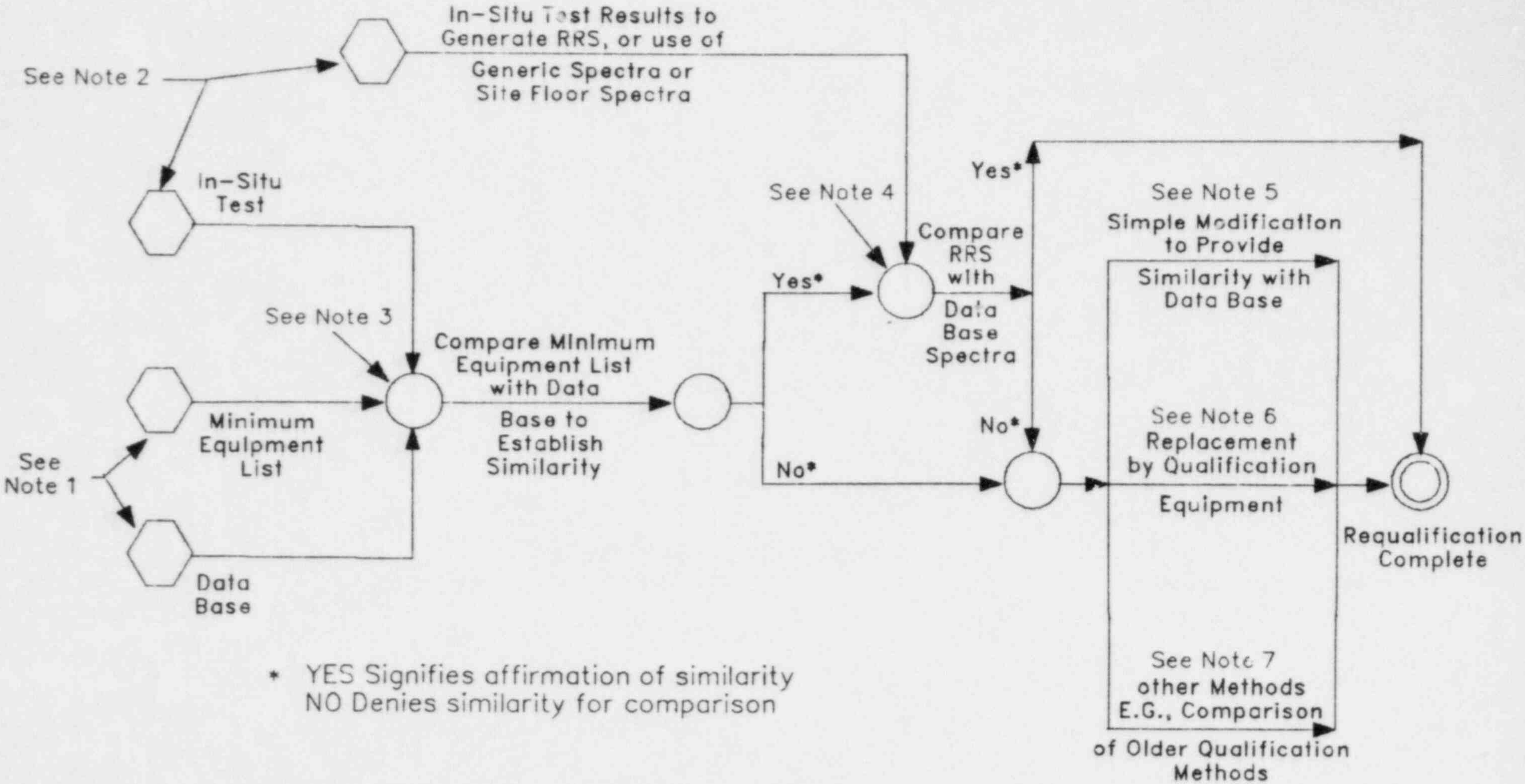

Figure 1. Alternative seismic qualification procedure

3. RESULTS

The results of this study are summarized in Table 1. The following subsections provide background information for the major column headings. Sections are numbered in the same order as the headings appear in the table. All numbers are in dollars.

3.1 Equipment List

The equipment list used in Table 1 was obtained by modifying the list offered in Reference 1. The modifications resulted from a comparison with two complete lists of safety-related equipment for two new plants--one PWR, one BWR.

3.2 Analysis

The "Analysis" cost estimates were based on the author's experience in estimating analysis jobs and on reviews of such analysis performed during SQRT audits. Equipment which has no estimate for analysis is not suitable for qualification by this technique.

3.3 Test & Analysis

The numbers under "Test & Analysis" represent the cost to determine equipment/support dynamic characteristics via in-situ testing. These numbers were based primarily on the information contained in the Appendix, "In-Situ Structural Characterization Test Cost Estimates." The estimate in the attachment was compared to some actual cost data from the private sector and shown to be high. This was attributed to two factors: First, the estimate was based on a single test per trip (i.e., travel to plant, perform test on single piece of equipment, travel home), while the actual data involved multiple tests per trip. Second, the estimate was based on a full reduction of data (as opposed to simply running the test), which yields full mass and stiffness matrices in addition to the natural frequency, mode shape, and damping data actually obtained. The numbers in the estimate were reduced by a constant multiplier to account for these

TABLE 1. COST ESTIMATES

Equipment Type	Analysis High	Analysis Low	Analysis Average	Test and Analysis High	Test and Analysis Low	Test and Analysis Average	Replacement High	Replacement Low	Replacement Average	Comparison High	Comparison Low	Comparison Average	Support Modification High	Support Modification Low	Support Modification Average
Air Circ Fan/Motor	10,000	6,000	8,000	44,500	9,900	15,300	75,000	3,500	13,500						
Air Cond Unit	200,000	75,000	100,000	118,000	26,200	40,600	260,000	28,000	115,000	600	100	200	7,000	1,300	2,600
Cabinet[b]	13,000	7,000	9,000	44,500	9,900	15,300	4,500	1,000	2,500	1,600	400	800	15,000	2,400	7,000
Circuit Board	--	--	--	--	--	--	600	90	400	600	100	200	850	350	500
CRDM[c]	--	--	--	44,500	9,900	15,300	32,580K	2,450K	27,000K	600	100	200	350	230	275
Diesel Generator	200,000	75,000	100,000	118,000	26,200	40,600	750,000	250,000	500,000	600	100	200	33,700	5,800	13,800
Inverter	--	--	--	--	--	--	1,300	200	900	2,000	400	1,200	88,600	24,800	49,400
MSIV	18,000	12,000	15,000	53,600	11,900	18,400	350,000	140,000	200,000	600	100	200	370	240	300
Panels	13,000	7,000	9,000	44,500	9,900	15,300	30,000	1,000	7,000	600	100	200	37,400	13,100	21,600
Small Horiz Pump/ Motor	23,000	14,000	17,000	44,500	9,900	15,300	95,000	6,000	54,000	600	100	200	1,870	360	710
										1,200	200	400	8,100	1,460	4,400
Medium Horiz Pump/ Motor	23,000	14,000	17,000	44,500	9,900	15,300	160,000	17,000	78,000	1,200	200	400	16,800	3,400	8,400
Large Horiz Pump/ Motor	23,000	14,000	17,000	44,500	9,900	15,300	245,000	31,000	125,000	1,200	200	400	25,200	5,200	12,800
Small Vert Pump/ Motor	26,000	17,500	22,000	44,500	9,900	15,300	42,000	7,000	24,000	900	100	300	12,100	3,040	6,300
Medium Vert Pump/ Motor	26,000	17,500	22,000	44,500	9,900	15,300	87,000	30,000	59,000	900	100	300	18,900	5,200	10,200
Large Vert Pump/ Motor	26,000	17,500	22,000	44,500	9,900	15,300	160,000	50,000	100,000	900	100	300	31,800	8,500	16,800
Racks (Instr.)	13,000	7,000	9,000	44,500	9,900	15,300	3,300	750	1,900	600	100	200	800	350	510
Racks (Bat.)	13,000	7,000	9,000	44,500	9,900	15,300	5,000	1,100	2,800	600	100	200	870	360	540
Strip Chart Rec.	--	--	--	--	--	--	7,500	800	3,400	600	100	200	970	350	570
Relays	--	--	--	--	--	--	800	130	560	600	100	200	350	230	280
Metal Clad Switchgear	--	--	--	53,600	11,900	18,400	73,000	12,000	42,500	600	100	200	9,000	2,140	4,800
Voltage Switchgear	--	--	--	--	--	--	7,100	300[d]	3,200	600	100	200	680	230	430
Motor Control Center	--	--	--	--	--	--	10,700	350[e]	3,650	600	100	200	1,270	270	410
Transducer	--	--	--	--	--	--	1,300	500	1,000	600	100	200	370	250	300
Transformer	--	--	--	27,400	6,100	9,400	8,500	1,500	5,500	600	100	200	1,530	500	920
Check Valve	6,000	2,000	4,000	27,400	6,100	9,400	9,000	150	4,800	600	100	200	1,150	350	700
Small Instr. Valve	6,400	3,200	4,800	26,800	[illegible],000	9,200	300	90	125	600	100	200	330	230	260
Small Relief Valve	13,000	8,500	11,000	44,500	9,900	15,300	15,000	1,300	8,000	600	100	200	1,150	340	700
Large Relief Valve	13,000	8,500	11,000	53,600	11,900	18,400	45,000	5,200	25,500	600	100	200	3,400	760	1,920
Small Safety Valve	11,000	6,500	9,000	44,500	9,900	15,300	6,000	2,800	4,500	600	100	200	1,030	460	670
Large Safety Valve	11,000	6,500	9,000	53,600	11,900	18,400	35,000	6,000	14,000	600	100	200	2,500	660	1,200

a. Equipment with no estimate for a particular method is not suitable for qualification by that method.

b. Cabinet only. Contents of cabinet not included.

c. K = X 1,000

d. 15 Amp-240 Volt Ac 3-pole circuit breaker.

e. 600 V 3 Phase Ac @ 2 HP motor starter.

factors. Numbers in the "Low" column were obtained by a multiplier that yielded an estimate within 5% of the actual cost for a test contract involving 17 tests in a single trip. Numbers in the "High" column were obtained with a multiplier to account for the more complete data reduction included in the estimate. The numbers in the "Average" column were obtained with a multiplier to account for the more complete data reduction and to adjust the estimate to a 5 test per trip basis.

3.4 Replacement

"Replacement" is the cost incurred to replace equipment with qualified equipment. This includes purchase of the equipment with qualification documentation and installation. It does not include freight charges. Estimates are primarily based on "Process Plant Construction Estimating Standards," by Richardson Engineering Services, Inc.[2] Two editions of the standard were used, one dated 1975 and the other 1981. Estimates taken from the 1975 edition were increased by 30% to account for inflation. Two components on the list (Main Stem Isolation Valve and Control Rod Drive Mechanism) were not covered by the standard. Estimates for these two were obtained by contact with vendors.

Qualification documentation was assumed to cost 150% of the cost of the unqualified components for all but three of the components--small instrument valves, transducers, and relays. These components are produced in large quantities and required in large quantities in typical plants. Their qualification documentation is assumed to be less costly--50% of the cost of the unqualified component. The 50% and 150% numbers were based on engineering judgment of the authors.

3.5 Comparison

The "Comparison" estimate is the cost of comparing dynamic and functional characteristics of equipment in the plant with characteristics of equipment in the data base. The estimate is based on the authors judgment and the assumption that necessary data is readily available. Therefore, no costs resulting from analysis or in-situ testing have been included.

3.6 Support Modification

These numbers represent the cost of providing simple support modifications to obtain similarity with the data base equipment. They were calculated using the following formula:

$$\text{Cost} = (1.5\ L_i \times W) + 0.1\ C_i + 200$$

where

L_i = the number of manhours required for installation of a new piece of equipment (the "average" L_i is twice the "low" L_i and one half the "high" L_i)

W = hourly wage for installation labor ($20/hr was used)

C_i = base cost of a new piece of equipment.

The first term of the equation ($1.5\ L_i \times W$) represents the labor cost to make the modification. The second term ($0.1\ C_i$) is the material cost. the third term (200) represents four hours of an engineers time @ $50/hr. This formula was derived to provide a consistent and reasonable way of estimating modification costs based on the complexity of the installation and the base cost of the equipment.

4. REFERENCES

1. Kana, D. D.; Simonis, J. C.; Pomerening, D. J.: Survey of Methods for Seismic Qualification of Nuclear Plant Equipment And Components, Report No. SWRI 6582-001-01, October 15, 1982.

2. Richardson Engineering Services Inc., The Richardson Rapid Construction Cost Estimating System, 1975 and 1981 Editions.

APPENDIX

In-Situ Structural Characterization
Test Cost Estimate

Estimate Notes:

1. Equipment is categorized by a 3 x 4 matrix relating complexity of experimental model (labor intensive costs) to size of equipment/test technique (equipment intensive costs).

2. Cost assumptions are listed under "Base Costs."

3. Intermediate cost workups are contained on "Worksheets."

   a. Labor estimates are based on a "Med-Small Model."

   b. Multiplication factors for other size categories are applied to these to account for increased time required for larger or smaller experimental models.

   c. Depreciated capital costs are based on 1 month. Multiplication factors are applied to account for longer or shorter equipment usage time.

4. Cost summary figures are based on worksheet figures with appropriate multiplication factors applied.

COST SUMMARY

Structural Complexity	Very Large (>50,000 lb) Multiple E.H. Actuators 40-60 gpm ($K)	Large (5000-50,000 lb) Single E.H. Actuator 20 gpm ($K)	Medium Small (100-5000 lb) E.M. Actuator and/or Hammer ($K)	Very Small (<100 lb) Hammer/Mini Accelerometer ($K)
	Size/Test Technique			
Complex (>100 DOF)				
Personnel				
Labor	57.5	48.6	44.2	48.6
Travel	18.2	15.4	14.0	15.4
Equipment				
D.C.C.	30.6	21.9	21.3	18.8
Fixed	15.0	10.0	5.0	5.0
Trans.	9.8	5.3	3.3	2.3
Total	131.1	101.2	87.8	90.1
Moderate (10-100 DOF)				
Personnel				
Labor	27.7	23.4	21.3	23.4
Travel	9.1	7.7	7.0	7.7
Equipment				
D.C.C.	18.4	13.1	12.8	11.3
Fixed	15.0	10.0	5.0	5.0
Trans.	9.8	5.3	3.3	2.3
Total	80.0	59.5	49.4	49.7

COST SUMMARY (continued)

Structural Complexity	Very Large (>50,000 lb) Multiple E.H. Actuators 40-60 gpm ($K)	Large (5000-50,000 lb) Single E.H. Actuator 20 gpm ($K)	Medium Small (100-5000 lb) E.M. Actuator and/or Hammer ($K)	Very Small (<100 lb) Hammer/Mini Accelerometer ($K)
	Size/Test Technique			
Simple (<10 DOF)				
Personnel				
Labor	13.3	11.2	10.2	11.2
Travel	4.4	3.8	3.4	3.8
Equipment				
D.C.C.	12.3	8.8	8.5	7.5
Fixed	14.0	10.0	5.0	5.0
Trans.	9.8	5.3	3.3	2.3
Total	54.8	39.1	30.4	29.8

WORKSHEET 1. LABOR

Structural Complexity		Formula[a]
Complex (>100 DOF)	Testing (4 persons)	
	Setup/Transportation	2 wk
	Test	3 wk
		20 m-wk
	Test plan	3 m-wk
	Data Reduced, Report	3 m-wk
		26 m-wk @ $1.7 k/m-wk = $44.2K
Moderate (10-100 DOF)	Testing (3 persons)	
	Setup/Transportation	1 wk
	Test	2 wk
		9 m-wk
	Test plan	2 m-wk
	Data Reduced, Report	1.5 m-wk
		12.5 m-wk @ $1.7 k/m-wk = $21.3K
Simple (<10 DOF)	Testing (2 persons)	
	Setup/Transportation	1 wk
	Test	1 wk
		4 m-wk
	Test plan	1 m-wk
	Data Reduced, Report	1 m-wk
		6 m-wk @ $1.7 k/m-wk = $10.2K

a. Multiplication Factor for various Sizes/Test Techniques

Size/Test Technique	Multiplication Factor
Very Large (>50,000 lb.) Multiple E.H. Actuators 40-60 gpm	1.3
Large (5000-50,000 lb.) Single E.H. Actuator 20 gpm	1.1
Medium Small (100-5000 lb.) E.M. Actuator and/or Hammer	1.0
Very Small (<100 lb.) Hammer/Mini Accelerometer	1.1

WORKSHEET 2. PERSONNEL TRAVEL

Structural Complexity	Formula[a]			
Complex (>100 DOF)	Airfare	0.5K/person @ 4 persons	=	2K
	F.L. and T.	0.6K/person/wk @ 20 m-wk	=	12K
				$14K
Moderate (10-100 DOF)	Airfare	0.5K/person @ 3 persons	=	2K
	F.L. and T.	0.6K/person/wk @ 9 m-wk	=	5K
				$ 7K
Simple (<10 DOF)	Airfare	0.5K/person @ 2 persons	=	1.0K
	F.L. and T.	0.6K/person/wk @ 4 m-wk	=	2.4K
				$3.4K

a Multiplication Factor for various Sizes/Test Techniques.

Size/Test Technique	Multiplication Factor
Very Large (>50,000 lb.) Multiple E.H. Actuators 40-60 gpm	1.3
Large (5000-50,000 lb.) Single E.H. Actuator 20 gpm	1.1
Medium Small (100-5000 lb.) E.M. Actuator and/or Hammer	1.0
Very Small (<100 lb.) Hammer/Mini Accelerometer	1.1

WORKSHEET 3. DEPRECIATED CAPITAL COSTS (D.C.C)

Size	Test Technique	Formula	Total ($)
Very Large	(50,000 lb) Multiple E.H. Actuators 40-60 gpm	D.C.C.	
		Actuators 0.5K/Actuators/mo @ 3 actuators and 1 mo	1.5K
		Power Supply 0.1K/gpm/mo @ 50 gpm and 1 mo	5.0K
		D.A. System 10K/mo @ 1 mo	10.0K
		Software 5K/mo @ 1 mo	5.0K
		Miscellaneous Equipment and Transportation 3K/mo @ 1 mo	3.0K
			24.5K
		Fixed Cost	
		(Fixtures, Cables, etc.)	15.0K
Large	(5000-50,000 lb) Single E.H. Actuator 20 gpm	D.C.C.	
		Actuators 0.5K/Actuators/mo @ 3 actuators and 1 mo	0.5K
		Power Supply 0.1K/gpm/mo @ 50 gpm and 1 mo	2.0K
		D.A. System 10K/mo	10.0K
		Software 2K/mo	2.0K
		Miscellaneous Equipment and Transportation 3K/mo @ 1 mo	3.0K
			17.5K
		Fixed Cost	
		(Fixtures, Cables, etc.)	10.0K
Medium Small	(100-5000 lb) E.M. Actuator and/or Hammer	D.C.C.	
		E.M. Actuators/Amp 2K/mo @ 1 mo	2.0K
		D.A. System 10K/mo @ 1 mo	10.0K
		Software 2K/mo	2.0K
		Miscellaneous Equipment and Transportation	3.0K
			17.0K
		Fixed Cost	
		(Fixtures, Cables, etc.)	5.0K

DEPRECIATED CAPITAL COSTS (continued)

Size	Test Technique	Formula	Total ($)
Very Small	(100 lb.) Hammer/Mini Acceleromete	D.C.C.	
		D.A. System 10K/mo	10.0K
		Software 2K/mo	2.0K
		Miscellaneous Equipment and Transportation	3.0K
			15.0K
		Fixed Cost	
		(Fixtures, Cables, etc.)	5.0K

a. Multiplication factor for various levels of structural complexity.

Structural Complexity	Multiplication Factor
Complex (100 DOF)	1.25
Moderate (10-100 DOF)	0.75
Simple (10 DOF)	0.50

WORKSHEET 4. EQUIPMENT TRANSPORTATION

Size	Test Technique	Formula	Total ($)
Very Large	(50,000 lb) Multiple E.H. Actuators 40-60 gpm	Actuators 0.25K/Actuators @ 3 actuators	0.75K
		Power Supply 0.05K/gpm @ 50 gpm	2.5 K
		D.A. System, Transportation, Miscellaneous Equipment	0.5 K
		Fixtures, Cables, etc.	1.0 K
		Pack, Unpack (Manufacturing and Labor)	5.0 K
			9.8 K
Large	(5000-50,000 lb) Single E.H. Actuator 20 gpm	Actuators 0.25K @ 1 actuators	0.25K
		Power Supply 0.05K @ 20 gpm	1.0 K
		D.A. System, Transportation, Miscellaneous Equipment	0.5 K
		Fixtures, Cables, etc.	0.5 K
		Pack, Unpack	3.0 K
			5.3 K
Medium Small	(100-5000 lb) E.M. Actuator and/or Hammer	E.M. Actuators/Amp	0.25K
		D.A. System, Transportation, Miscellaneous Equipment	0.5 K
		Fixtures, Cables, etc.	0.5 K
		Pack, Unpack	2.0 K
			3.3 K
Very Small	(100 lb) Hammer/Mini-Accelerometer	D.A. System, Transportation, Miscellaneous Equipment	0.5 K
		Fixtures, Cables, etc.	0.25K
		Pack, Unpack	1.5 K
			2.3 K

a. Equipment transportation cost is the same for the various levels of structural complexity.

BASE COSTS

Personnel	
Labor	\$1.7K/person/wk
Travel	
Airfare	\$500
F and C	\$500/person/wk
Transportation	\$100/person/wk
Equipment	
Depreciated Capital Costs (DCC)	
H. Actuator (3K capital)	\$5000 @ 10%/mo \$0.5K/actuator/mo
H. Power Supply (3000 psi)	\$1000/gpm @ 10%/mo \$0.1K/gpm/mo
Data Acquisition System (8 ch. G.R.)	\$100K @ 10%/mo \$10K/mo
Software	
Multi-actuator	\$50K @ 10%/mo \$5K/mo
Single-actuator	\$20K @ 10%/mo \$2K/mo
E.M. Actuator/Amp (800W-100 lb)	20K% @ 10%/mo \$2K/mo
Miscellaneous Equipment (Signal Generator, CRO, DMM, etc.)	\$20K @ 10%/mo \$2K/mo
Transducers/Power Supply 20 @ \$500)	\$10K @ 10%/mo \$1K/mo

BASE COSTS (continued)

Transportation	
H. Actuator ($3K capital)	500 lb @ $50/c[a] $0.25K/actuator
H. Power Supply (3000 psi)	100 lb/gpm @ $50/c $0.05K/gpm
Data Acquisition System/ Transportation/Miscellaneous Equipment	1000lb @ $50/c $0.5K
E.M. Actuator/Amp	500 lb @ $50/c $0.25K

a. 1c = 100 lb.

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This report supports the Nuclear Regulatory Commission (NRC) Unresolved Safety Issue A-46, "Seismic Qualification of Equipment in Operating Plants." The report is divided into four distinct sections. Part A identifies the basic technical approaches for using in-situ test procedures as a tool in alternate methods for the seismic qualification of equipment in operating plants. Part B includes the development of improved methods of developing structural models using the results of in-situ procedures, and predicting structural response during seismic events using methods of random vibrations. Thorough technical justification for these methods of analysis is provided to support the related guidance and acceptance criteria presented in Part C. Part D contains a cost estimate for using the various alternative methods for seismic qualification of equipment.

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