UNITED STATES
NUCLEAR REGULATORY COMMISSION
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SEP 12

MEMORANDUM FOR: Harold R. Denton, Director
Office of Nuclear Reactor Regulation

Robert B. Minogue, Director
Office of Standards Development

FROM: Thomas E. Murley, Acting Director
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SUBJECT: RESEARCH INFORMATION LETTER No.102 , "STRUCTURAL BUILDING RESPONSE REVIEW: PHASE I OF PROJECT IV OF THE SEISMIC SAFETY MARGINS RESEARCH PROGRAM"

This Research Information Letter (RIL) describes the results of a study to review the state-of-the-art in nuclear power plant structural building response computation. This study, initiated under the Seismic Safety Margins Research Program (SSMRP), is not intended to advance the art; rather, it will be used to identify analytical methods for realistically characterizing the seismic response of nuclear power plant structures. The findings of the subject study, as discussed in this RIL, should provide the staff with a basis for detailed review in the areas of analytical methods, structural modeling, uncertainty, nonlinear behavior, methods to account for interactions and nonseismic response. Because this is a RIL for the SSMRP, background information on the SSMRP is provided in addition to a descriptive summary of the structural building response review study.

1.0 Introduction

NRC has established regulations, guides, and licensing review procedures that define seismic safety criteria for nuclear power plant design. These criteria collectively constitute a seismic methodology chain. The seismic safety criteria for nuclear power plant design were developed to ensure structural as well as functional safety of buildings and equipment supported by buildings, and they depart from the conventional earthquake engineering practice in detail and complexity. The seismic methodology chain is considered sufficiently conservative to ensure safety; however, it is necessary to characterize the overall seismic safety and to improve it by establishing new criteria as may be required.

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1.1 Seismic Safety Margins Research Program

The SSMRP is developing probabilistic methods that realistically estimate the behavior of nuclear power plants during earthquakes. These probabilistic methods stand in contrast to the conservative methods appropriate to existing seismic design methodology, in which each element of the seismic methodology chain is addressed independently. Since there is uncertainty in each element, conservative assumptions are usually made, and the final result is a summation of several worst-case scenarios. For example, the strongest plausible earthquake is presumed to occur and produce the largest ground motion at the free field of the site. This motion is coupled with the bedrock and the building foundation to produce the worst possible forces and stresses. Such responses are compared to conservative estimates of the fragility of each structure or component to determine its survivability. In such a design process, the real safety issue of potential radioactive release is rarely addressed in the context of a system's assessment.

The objectives of the SSMRP are to develop an improved seismic safety design methodology and to develop a methodology to perform earthquake risk assessments of nuclear facilities. Risk will be measured by various failure probabilities and by the probability of release of radioactive materials. The SSMRP approach integrates the elements of the seismic chain, including:

Earthquake characterization

Soil-structure coupling

Structural building response

Subsystem structural response

Local failure

Systematics of how local failures could combine and lead to a release.

Each element will be characterized realistically and probabilistically, rather than conservatively and deterministically. Significant advances in technology will be required to meet the objectives. A multiphase program is underway consisting of eight projects which comprise the program. One of these projects is the Structural Building Response Project, the subject of this memorandum.

1.2 Structural Building Response Project

This project deals with the methodology to be used in the SSMRP for structural building response. The final goal is to determine structural response using state-of-the-art analysis techniques. Structural response serves two main purposes: (1) to develop input motion for the subsequent subsystem analysis and (2) to develop response for estimating structural failure. The models and methods used to determine structural response will be subject to many variables, including:

Structural material properties (modulus of elasticity, etc.)

Structural dynamic behavior (damping, nonlinear behavior, etc.)

Configuration

Idealization and modeling techniques

Methods of solution

Computer programs

As a first step in the Structural Building Response Project, a review of the state-of-the-art of structural response was performed. This review addressed several major areas of interest:

Structural modeling

Methods of dynamic analysis

Nonlinear behavior of structures and materials

Uncertainty in dynamic structural analysis

These and other issues were addressed in two reports (References 1 and 2). A summary of them comprises the remainder of this memorandum.

2.0 Descriptive Summary

This summary is intended to provide a brief description of the contents of the two reports. The reader should consult the reports for further details on the topics presented below.

2.1 Uncertainty

Two inherently different types of uncertainty were identified in the reports: (1) random variability, which is associated with such statistical variations as the natural heterogeneity in material properties; and (2) modeling uncertainty, which is a systematic type of variability related to the limited availability of information, inherent bias in certain models or predictions, consistent errors, or deviations from reality in material and structural testing.

In fact, few sources of variability can be solely attributed to either random variability or modeling uncertainty. For example, material properties are certainly a source of random variability; however, the concrete quality control requirements lead to average concrete strengths consistently greater than the nominal values. This latter type of variability is obviously a modeling or systematic type of uncertainty. Another example of combined random variability and modeling uncertainty is the damping values, which exhibit not only natural irreducible variability, but also a systematic bias in present-day calculations because the prescribed values are believed to be less than those experienced in practice, especially at high response levels.

The individual sources of uncertainty were addressed in three broad categories in Reference 2:

Constitutive properties - primarily the elastic constants and strength values for steel, concrete and reinforcing bars, but also the description of the stress-strain behavior over the entire range for use in nonlinear analyses;

Dynamic structural characteristics, include the mass, stiffness and damping characteristics, and the calculated natural frequencies and mode shapes;

Other sources of uncertainty, include modeling techniques, analytical procedures, computer software reliability and effects such as the variation in field construction practices, errors in analysis, design and fabrication and deterioration of members.

Sources of uncertainty are identified as either random variability (RV), modeling uncertainty (MU), or both, and subjective estimates of the uncertainties are provided in a summary (Table 1). This table is based on one that appears in Reference 2.

2.2 Nonlinear Behavior

Nonlinear behavior of nuclear power plant structures can result from either geometric or material nonlinearities. However, because of the size and stiffness of these structures, geometric nonlinearities due to large deformations are less likely, and the reports focus on material nonlinearities. Reference 2 discusses nonlinear material characteristics (and the attempts to treat them in analysis with simple idealized force-deformation curves) in terms of the following assumptions about non-linear behavior:

Lumped plasticity (i.e., the formation of a plastic hinge in a frame-type structure when the maximum bending moment reaches the yield moment)

Distributed plasticity (i.e., the formation of plastic regions in a shear wall)

Stiffness degradation in concrete structures due to cracking.

These sources of nonlinearity are treated in dynamic analyses in three different ways:

Detailed multiple-degree-of-freedom (MDOF) inelastic calculations

Single-degree-of-freedom (SDOF) inelastic methods

MDOF elastic analyses

The first technique is the most rigorous, but also the most time-consuming and costly. Nonlinear analysis of MDOF system response to a given time history of ground motion is carried out by step-by-step integration of the equations of motion, dividing the response history into short-time increments and assuming that the properties of the structure remain constant during each increment, but change in accordance with the deformation at the end of each increment. Thus, detailed MDOF nonlinear analysis is actually a sequence of linear analyses of a changing structure.

Because of their simplicity and low cost compared to the MDOF nonlinear analysis technique, SDOF inelastic methods are discussed in the reports. Primary among these methods is the equivalent linear approach to the analysis of simple hysteretic structures for which an equivalent linear system is developed to match the response of the nonlinear system. Reference 2 categorizes the methods for developing the equivalent linear system in terms of three types of input motion - harmonic, random and earthquake loading. The methods discussed in these categories include:

Harmonic Equivalent Linearization

Resonant Amplitude Matching

Dynamic Mass

Constant Critical Damping

Geometric Stiffness

Geometric Energy

Stationary Random Equivalent Linearization

Average Period and Damping

Average Stiffness and Energy

Reference 1 discusses approximate techniques in terms of the following methods:

Reserve Energy

Inelastic Response Spectrum

Substitute Structure

In both reports, the methods were compared to one another not to produce a "best" method, but rather, to substantiate the claim that approximate methods can be developed for the nonlinear analysis of nuclear power plant structures (if such an analysis is necessary) to avoid the costly, complex and time-consuming rigorous approach.

2.3 Structural and Component Idealization Methods and Mathematical Models

The reports discuss two major methods of discretization and their applicability to nuclear power plant structures:

Equivalent beam models, in which the mass can be considered as concentrated at a series of points and the stiffness of the overall structure approximates that of a simple cantilever beam, and

Finite element models (both two- and three-dimensional), in which various elements (shells, plates, etc.) describe the overall stiffness.

In general for excitations, axisymmetric shell structures that have a large height-to-radius ratio can be adequately modeled by the equivalent beam method. Typically, chimneys and containment vessels are modeled this way; beam properties are determined from the shell cross section and lumped masses from the dead weight of the shell. Both reports compare this method to the more complex shell (finite element) approach and find good agreement, especially when the lower modes dominate the structural behavior. It was found that for a given structure, the accuracy of the equivalent beam approach is dictated by the total number of masses chosen. Equivalent beam modeling entails two simplifying assumptions:

Plane sections remain plane after deformation

No shape distortion occurs because of the diaphragm action of the floor slabs.

The finite element approach does not require such simplifying assumptions. Various types of elements (e.g., shell, plate, beam and truss elements) describe the overall stiffness. Moreover, local behavior of a structural system can be readily incorporated in a finite element model.

2.4 Analysis Methods

Structural response can be determined by either a time-history or response-spectrum approach, both of which are addressed in the reports.

2.4.1 Time-history Techniques

Three time-history techniques were discussed:

Modal Analysis

Complex Analysis

Direct Integration

Modal analysis and direct integration are discussed for both linear and nonlinear systems.

Direct integration normally requires the use of some numerical integration technique. The reports discuss four explicit techniques in which the differential equations are converted to a set of linear algebraic equations which have state variables that are independent of one another: (1) Runge-kutta techniques, (2) Predictor-Corrector techniques, (3) Nordsieck Integration techniques and (4) Central Difference techniques. Three implicit techniques, which convert the equations to a set of linear simultaneous equations, are also discussed: (1) Newmark's Generalized Acceleration method, (2) Wilson-theta method and (3) Houbolt method.

2.4.2 Response Spectrum Techniques

In this method, the displacement response for a given node at any mass point is obtained directly from the design response spectrum through the spectral acceleration at a given frequency and damping. Shears and moments are then calculated from the displacements using the stiffness properties of the structural members. The total response is then calculated by combining these maximum modal responses, which are presumed to peak at different times. The reports discuss several statistical methods for combining the individual modal responses.

2.4.3 Random Vibration Techniques

Reference 1 discusses a third approach to dynamic seismic analysis, the random vibration method. Sometimes referred to as the power spectral density method, this statistical analysis technique uses an ensemble of possible ground-motion histories in contrast to the time-history method, which uses a deterministic time function, and the response-spectrum technique, which uses a set of smoothed response spectra.

2.5 Methods that Account for Interactions

The two reports describe the relationship between structural building response and the coupled soil-structure system. Simplifications of the structural models for soil-structure interaction analysis are discussed.

2.6 Nonseismic Response

The reports briefly discuss the state-of-the-art methods for combining seismic and nonseismic responses.

3.0 Conclusions and Recommendations

Five major areas of seismic response analysis of buildings were reviewed to assess the state-of-the-art.

Findings of this study in the areas of structural modeling, methods of structural analysis, structural damping values, nonlinear behavior of materials and structures, and the uncertainty in structural dynamic analysis will provide the staff with a basis for detailed in-depth review and, in many cases, a confirmation of the technical judgment of the staff. Following are some pertinent conclusions that would be of interest to the staff from the standpoint of licensing review.

a. In the dynamic analysis of structures, the floor slabs which support important systems or components should be modeled properly in the vertical direction of motion. Rigid floor assumption is not sufficient if the floor system is not stiff enough to justify that assumption. Since the floor usually consists of a large number of composite beams and numerous irregularities in floor geometry and thickness, detailed modeling is not usually performed. The floor systems are usually represented by sets of SDOF systems in the dynamic analysis model.

b. A proper distribution of mass and stiffness of the structure is essential in the lumped-mass-beam approach of structural modeling. Many assumptions are generally made by engineers in this approach to simplify the calculation of stiffness characteristics of structures. Close examinations of these assumptions are necessary to ensure the reasonableness of the resulting model.

c. Hydrodynamic effects developed during an earthquake cannot be ignored in the design of power plant structures in cases where the quantity of liquid is large. Due to the complicated nature of this problem, there is currently no single universally accepted code that can be utilized for computing this effect for generalized conditions.

d. Decoupling criteria for subsystems currently used in engineering practice employ two numerical ratios; mass ratio R_m, and frequency ratio R_f. In the NRC Standard Review Plan, Section 3.7.2, the definitions of these two ratios are not quite clear and may be subject to different interpretations. Reference 2 offers new definitions for these two ratios. These recommendations are one step closer to clear regulation in this area.

e. Under current practice and/or economic reasons, the complicated structural systems are treated by linear seismic analysis. For example, analysis of concrete structures are treated with both gross properties and fully-cracked properties. The cracked properties are usually estimated based on conservative forces developed for the uncracked linear models. It is usually assumed, but not demonstrated, that this analytical approach brackets the true nonlinear response. Considering the inherent difficulties and penalties associated with detailed nonlinear dynamic analysis of MDOF systems, the development of practical simplified approaches for such nonlinear response calculations is a necessity.

f. Uncertainty of structural characteristics affect virtually every aspect of analytical effort to predict the actual in-service response of nuclear power plant structures to a strong-motion earthquake. Past studies show that there is large uncertainty in predicting the structural frequencies. Calculated frequencies can deviate substantially from the test or observed structural frequencies for a whole range of excitation levels. The results of Reference 2 show that the uncertainty of structural damping values is even greater than that of structural frequencies. NRC Regulatory Guide 1.61 damping values are reasonable for design in view of the current knowledge in this area.

   One source of conservatism that has usually gone unnoticed is the current design and analysis practice in the nuclear industry. The member sizes of structures are initially set large enough to enhance nonexceedance in later design modifications. Efforts to trim down the member sizes are not emphasized since the iterative process of alternating design and analysis is not practical in reality. With so much conservatism built into the design of nuclear power plants, the actual stress level under safe shutdown earthquake and operating basis earthquake might be substantially less than the stress levels for which the damping values are originally assigned. Smaller damping values might actually be applicable for final design. Back verification is usually lacking in current practice.

The results of this state-of-the-art study in the five major areas of seismic response analysis of buildings should be reviewed by the NRR staff for use in the regulatory review process.

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Enclosures:
1. Table 1
2. Table 2

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REFERENCES

1. Sargent & Lundy Engineers, Structural Building Responses Review Report Prepared for Seismic Safety Margins Research Program, Lawrence Livermore National Laboratory, California, Chicago, IL, Report SL-3759 (Lawrence Livermore National Laboratory, Report UCRL 15183, May 1980). NUREG/CR-1423, Vol. II.

2. J. J. Healey, S. T. Wu and M. Murga, Seismic Safety Margins Research Program (Phase 1) Project IV - Structural Building Response Structural Building Response Review, Ebasco Services, Incorporated, New York City, Lawrence Livermore National Laboratory, Report UCRL 15185, May 1980. NUREG/CR-1423, Vol. I.

Enclosure 1

TABLE 1. Sources, types (random variability, RV, or modeling uncertainty, MU), and estimated magnitudes of uncertainties in the constituitive properties of concrete, concrete reinforcing bars, and structural steel, from Reference 2.

Source of Uncertainty	Type	Uncertainty Estimate
<u>CONSTITUTIVE PROPERTIES</u>		
<u>CONCRETE</u>		
COMPRESSIVE STRENGTH	RV[1],MU[2]	• MEAN, IN-PLACE STRENGTH $\bar{f}_{str} = 0.675 f'_c + 1100 \le 1.15 f'_c$ f'_c = DESIGN COMPRESSIVE STRENGTH • DISPERSION CYLINDER STRENGTH FOR $f'_c < 4000$ psi cov = 10% EXCELLENT QC cov = 15% AVERAGE QC cov = 20% POOR QC FOR $f'_c > 4000$ psi σ = 400 psi EXCELLENT QC σ = 600 psi AVERAGE QC σ = 800 psi POOR QC IN-PLACE STRENGTH $V^2_{str} = V^2_{cyl} + 0.0094$ WHERE V = cov • DISTRIBUTION FUNCTION- NORMAL FOR BOTH CYLINDER STRENGTH AND IN-PLACE STRENGTH
TENSILE STRENGTH	RV,MU	SPLITTING TENSILE STRENGTH (IN-PLACE) • MEAN $\bar{f}_r = 6.4\bar{f}_{35}^{1/2}\left[0.96(1+0.11)(\log R)\right]$ $\bar{f}_{35}$ = STRENGTH AT R=35psi/sec R= LOADING RATE • DISPERSION $V_T^2 = \frac{V_{cyl}^2}{4} + 0.0190 \ge V^2_{str}$ FLEXURAL TENSION STRENGTH (IN-PLACE) • MEAN $\bar{f}_R = 8.3\bar{f}_{35}^{1/2}\left[0.96(1+0.11)(\log R)\right]$ • DISPERSION $V_R^2 = \frac{V_{cyl}^2}{4} + 0.0421 \ge V^2_{str}$ • DISTRIBUTION FUNCTION- NORMAL FOR BOTH SPLITTING TENSILE STRENGTH AND FLEXURAL TENSION
MODULUS OF ELASTICITY	RV,MU	MODULUS IN COMPRESSION AND TENSION • MEAN- $\bar{E} = 60400\,\bar{f}_{str35}^{1/2}\,(1.16-.08\log t)$ t= LOADING DURATION (SEC) • DISPERSION $V^2 = V^2_{cyl/4} + 0.0085$ • DISTRIBUTION FUNCTION: NORMAL
CONCRETE MEMBER DIMENSION	RV,MU	SEE TABLES 2,3 AND 4
<u>CONCRETE REINFORCING BARS</u>		
YIELD STRENGTH	RV,MU	• GRADE 40 BARS MEAN f_y = 48.8 ksi cov= 10.7% • GRADE 60 BARS MEAN f_y = 71 ksi cov = 9.3% • DISTRIBUTION FUNCTION- BETA DISTRIBUTION
AREA	RV,MU	MEASURED TO NOMINAL AREA LOWER LIMIT 0.94 MEAN VALUE = 0.99 cov = 2.4% NORMAL DISTRIBUTION
ULTIMATE STRENGTH	RV,MU	ULTIMATE STRENGTH = 1.55 f_y cov = SAME AS FOR YIELD STRENGTH BETA DISTRIBUTION
MODULUS OF ELASTICITY	RV,MU	MEAN VALUE = 29200 ksi cov = 3.3% NORMAL DISTRIBUTION
<u>STRUCTURAL STEEL</u>		
YIELD STRESS	RV,MU	• FLANGES MEAN = 1.05 F_y F_y = SPECIFIED TENSILE YIELD STRESS cov = 10% • WEBS MEAN = 1.10 F_y cov = 11% • SHEAR MEAN = 0.64 F_y cov = 10% DISTRIBUTION FUNCTION: NORMAL
POISSON'S RATIO	RV,MU	MEAN = 0.30 cov = 3%
MODULUS OF ELASTICITY	RV,MU	• TENSION,COMPRESSION MEAN = 29000 ksi cov = 6% • SHEAR MEAN = 11200 ksi cov = 6% • STRAIN-HARDENING MODULUS MEAN = 600 ksi cov = 25% DISTRIBUTION FUNCTION: NORMAL

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Enclosure 2

TABLE 2. Sources, types (random variability, RV, or modeling uncertainty, MU), and estimated magnitudes of uncertainties that stem from structural dynamic characteristics and structural modeling, from Reference 2.

Source of Uncertainty	Type	Uncertainty Estimate
DYNAMIC CHARACTERISTICS		
NATURAL PERIOD	RV,MU	ALL BUILDING TYPES (STEEL,RC & COMPOSITE) • RATIO OF OBSERVED/COMPUTED PERIOD SMALL AMPLITUDE VIBRATIONS MEAN RATIO = 0.845 cov = 31.1% LARGE AMPLITDE VIBRATIONS MEAN RATIO = 1.15 cov = 30% DISTRIBUTION FUNCTION (BOTH CASES) LOG NORMAL, GAMMA
DAMPING	RV,MU	• REINFORCED CONCRETE BUILDINGS SMALL VIBRATIONS (AMPLITUDE) MEAN = 4.26% cov = 76% LARGE VIBRATIONS (AMPLITUDE) MEAN = 6.63% cov = 64% • STEEL BUILDINGS SMALL AMPLITUDE VIBRATIONS MEAN = 1.68% cov = 65% LARGE AMPLITUDE VIBRATIONS MEAN = 5.65% cov = 45% DISTRIBUTION FUNCTION (BOTH CASES) LOG-NORMAL, GAMMA • COMPOSITE BUILDINGS SMALL AMPLITUDE VIBRATIONS MEAN = 2.72% cov = 42%
	RV,MU	LARGE AMPLITUDE VIBRATIONS MEAN = 3.23% cov = 54% DISTRIBUTION FUNCTION LOG-NORMAL, GAMMA
	RV,MU	REACTOR BUILDING COMPLEX (SUBJECTIVE ESTIMATE) • CONCRETE MEAN = 5% RANGE= 3% to 15% DISTRIBUTION: UNIFORM • STEEL MEAN = 2%, RANGE 0.05% - 4%
	RV,MU	• REINFORCED-CONCRETE CONTAINMENT (SUBJECTIVE ESTIMATE) MEAN = 5% RANGE = 2% to 10% DISTRIBUTION: FIGURE 6.6
	RV,MU	• REINFORCED CONCRETE BUILDING MEAN = 5.4% RANGE = 1% to 11% • VARIOUS BUILDINGS, LOW AMPLITUDE MEAN = 3.1% RANGE = 0% to 13% • HIGH RISE BUILDINGS SHEAR WALL TYPE, MEAN = 2.34% FRAME TYPE, MEAN = 3.48% CONSERVATISM IN NOMINAL VS ACTUAL DAMPING VALUES = 20-40% cov = 20%

Source of Uncertainty	Type	Uncertainty Estimate
STRUCTURAL MODELING		
MODELING	RV,MU	• SIMPLE STRUCTURE EXPERIENCED ENGR: cov =2.5% INEXPERIENCED ENGR: cov = 5% • COMPLEX STRUCTURE EXPERIENCED ENGR: cov = 10% INEXPERIENCED ENGR:cov = 30%
	RV,MU	SUBJECTIVE ESTIMATE cov= 15%
	RV,MU	SUBJECTIVE ESTIMATE RANGE OF UNCERTAINTY = ± 20%
MASS PROPERTIES	RV,MU	SUBJECTIVE ESTIMATE RANGE OF UNCERTAINTY = ± 10%
NUMERICAL ACCURACY	MU	SUBJECTIVE ESTIMATE RANGE OF UNCERTAINTY = ± 5%

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