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Seismic Review of the Robert E. Ginna Nuclear Power Plant as Part of the Systematic Evaluation Program

R. C. Murray, T. A. Nelson, D. S. Ng, C. Y. Liaw, J. D. Stevenson

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U.S. Nuclear Regulatory Commission

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Prepared by
R. C. Murray, T. A. Nelson, D. S. Ng, C. Y. Liaw, (EG&G/San Ramon Operations),
J. D. Stevenson (Structural Mechanics Associates)

Lawrence Livermore Laboratory
7000 East Avenue
Livermore, CA 94550

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FOREWORD

The U.S. Nuclear Regulatory Commission (NRC) is conducting the Systematic Evaluation Program (SEP), which consists of a plant-by-plant limited reassessment of the safety of eleven operating nuclear reactors that received construction permits between 1956 and 1967. Because many safety criteria have changed since these plants were initially licensed, the purpose of the SEP is to develop a current documented basis for the safety of these older facilities.

The eleven SEP plants were categorized into two groups based upon the extent to which seismic design was originally considered and the quantity of available seismic design documentation. The Robert E. Ginna Nuclear Power Plant, the subject of this report, was categorized under Group 1 on the assumption that enough documentation existed to perform the SEP review.

A detailed evaluation of plant structures and the hundreds of individual components within each Group 1 plant has not been performed. Rather, the evaluations rely upon limited analysis of selected structures and sampling of representative components from generic groups of equipment. The component sample was augmented by walk-through inspections of the facilities to select additional components based upon their potential seismic fragility.

This limited assessment of the Ginna facility relied in large part upon the guidance, procedures, and recommendations of recognized seismic design experts. Accordingly, a Senior Seismic Review Team (SSRT) under the direction of N. M. Newmark was established. Members of the SSRT and their affiliations are

Nathan M. Newmark, Chairman
Nathan M. Newmark Consulting Engineering Services
Urbana, Ill.

William J. Hall
Nathan M. Newmark Consulting Engineering Services
Urbana, Ill.

Robert P. Kennedy
Structural Mechanics Associates, Inc.
Newport Beach, Calif.

John D. Stevenson
Structural Mechanics Associates, Inc.
Cleveland, Ohio

Frank J. Tokarz (member until September 30, 1980)
Lawrence Livermore National Laboratory
Livermore, Calif.

The SSRT was charged with the following responsibilities:

- To develop the general philosophy of review, setting forth seismic design criteria and evaluation concepts applicable to the review of older nuclear plants, and to develop an efficient, yet comprehensive review process for NRC staff use in subsequent evaluations.

- To assess the safety of selected older nuclear power plants relative to those designed under current standards, criteria, and procedures, and to recommend generally the nature and extent of retrofitting to bring these plants to acceptable levels of capability if they are not already at such levels.

The SSRT developed its general philosophy and presented it in the first SEP report, which reviews Unit 2 of the Dresden Nuclear Power Station.[1] The assessment of Ginna reported here is the second in the series of SEP seismic reviews of Group 1 plants.

This report provides partial input into the SEP seismic evaluation of the Robert E. Ginna Nuclear Power Plant. The results of the seismic evaluation will be documented in a Safety Assessment Report prepared by the NRC staff that will address the capability of the Ginna systems to respond to seismic events or to mitigate the consequences of such events.

A limited peer review of this report was conducted by the SSRT to ensure consistency with the review philosophy established during the SSRT's review of Dresden Unit 2 and to review the results of the limited reanalyses of plant structures and the component sample.

Safety for seismic excitation implies that certain elements and components of an entire system must continue to function under normal operating and test loads. The SSRT did not review all aspects of the plant's operation and the safety margins available to assure that those elements and components needed for seismic safety would not be impaired beyond the point for which they can be counted on for seismic resistance because of unusual operating conditions, sabotage, operator error, or other causes. These aspects will have been studied by others. However, where unacceptable risks of essential elements not being able to function properly to resist seismic events were noted or inferred, greater margins of safety or provision for redundancy in the design of these elements are considered by the SSRT to be necessary.

The authors wish to thank T. M. Cheng, technical monitor of this work at the NRC, for his continuing support. We also wish to thank T. R. Weis, Rochester Gas and Electric Corp., and C. Chen, Gilbert Associates, Inc., and their colleagues for help and cooperation. Finally, we wish to thank R. K. Johnson of EG&G/San Ramon Operations for publications support.

ABSTRACT

A limited seismic reassessment of the Robert E. Ginna Nuclear Power Plant was performed by the Lawrence Livermore National Laboratory (LLNL) for the U.S. Nuclear Regulatory Commission (NRC) as part of the Systematic Evaluation Program. The reassessment focused generally on the reactor coolant pressure boundary and on those systems and components necessary to shut down the reactor safely and to maintain it in a safe shutdown condition following a postulated earthquake characterized by a peak horizontal ground acceleration of 0.2 g. Unlike a comprehensive design analysis, the reassessment was limited to structures and components deemed representative of generic classes. Conclusions and recommendations about the ability of selected structures and equipment to withstand the postulated earthquake are presented.

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CHAPTER 1: INTRODUCTION

This report describes work at the Lawrence Livermore National Laboratory (LLNL) to reassess the seismic design of the Robert E. Ginna Nuclear Power Plant. This limited reassessment includes a review of the original seismic design of selected structures, equipment, and components, and seismic analyses of selected items using current modeling and analysis methods.

The LLNL work is being performed for the U.S. Nuclear Regulatory Commission (NRC) as part of the Systematic Evaluation Program (SEP). The purpose of the SEP is to develop a current documented basis for the safety of 11 older operating nuclear reactors, including Ginna. The primary objective of the SEP seismic review program is to make an overall seismic safety assessment of the plants and, where necessary, recommend backfitting in accordance with the Code of Federal Regulations (10 CFR 50.109, Ref. 2). The important SEP review concept is to determine whether or not a given plant meets the "intent" of current licensing criteria as defined by the Standard Review Plan (Ref. 3)--not to the letter, but, rather, to the general level of safety that these criteria dictate. Additional background information about the SEP can be found in Refs. 4 and 5.

1.1 SCOPE AND DEPTH OF REVIEW

This review of Ginna is considerably different in scope and depth from current reviews for construction permits and operating licenses. Its focus is limited to identifying safety issues and to providing an integrated, balanced approach to backfit considerations in accordance with 10 CFR 50.109, which specifies that backfitting will be required only if substantial, additional protection can be demonstrated for the public health and safety. Such a finding requires an assessment of broad safety issues by considering the interactions of various systems in the context of overall plant safety.

Because individual criteria do not generally control broad safety issues, this review is not based on demonstrating compliance with specific criteria in the Standard Review Plan or Regulatory Guides. However, current licensing criteria do establish baselines against which to measure relative safety factors to support the broad integrated assessment. Therefore, we compare the seismic resistance of the Ginna facility in a qualitative fashion to that dictated by the intent of today's licensing criteria in order to determine acceptable levels of safety and reliability.

References in this report to load ratios and safety factors do not refer in an absolute sense to acceptable minimums, but to design-based levels thought to be realistic in light of current knowledge. In general, original levels do not represent maximum levels because such unclaimed factors as low stress and a structure's ability to respond inelastically contribute to seismic resistance. In particular, resistance to seismic motions does not mean the complete absence of permanent deformation. Structures and equipment may deform into the inelastic range, and some elements and components may even be permitted to suffer damage, provided that the entire system can continue to function and to maintain a safe shutdown condition.

This seismic reevaluation of Ginna centers on:

- An assessment of the integrity of the reactor coolant pressure boundary, that is, major components that contain coolant for the core and piping or any component not isolatable (usually by a double valve) from the core.

- A general evaluation of the capability of essential structures, systems, and components to shut down the reactor safely and maintain it in a safe shutdown condition, including removal of residual heat, during and after a postulated Safe Shutdown Earthquake (SSE). The assessment of this subgroup of equipment can be used to infer the capability of such other safety-related systems as the Emergency Core Cooling System.

The owner supplied a list of mechanical and electrical equipment necessary to ensure integrity of the reactor coolant pressure boundary and to safely shut down the reactor and maintain it in a safe shutdown condition during and after a postulated seismic event.[6] The licensee also listed the bases that it considered appropriate for evaluating the seismic classification of Ginna structures, systems, and components. These bases and the list of equipment are given in Appendix A. They reflect plant-specific requirements, not the more general light-water reactor (LWR) standards now in effect.

Not all equipment was examined as part of this reassessment. Mechanical and electrical equipment representative of items installed in the reactor coolant system and safe shutdown systems at the Ginna facility were examined for structural integrity and electrical and mechanical functional operability. To develop a basis for estimating the lower-bound seismic capacity of mechanical and electrical components and distribution systems, components that potentially have a high degree of seismic fragility were identified for review during a site visit by representatives of the NRC, the SSRT, LLNL and its subcontractors. The methods of selection of the representative equipment for this limited assessment are described in detail in Chapter 5.

To review the selected systems, an evaluation was made of the interconnected building complex, the containment building, and the containment internal structures to demonstrate structural adequacy and to obtain seismic input to equipment. For the structural evaluation, a peak horizontal ground acceleration of 0.2 g was used along with a Regulatory Guide (R.G.) 1.60 response spectrum.[7]

Because of the nature and extent of original documentation, the models developed for the interconnected building complex, the containment building, and the containment internal structures to evaluate critical structures and generate in-structure response spectra for equipment reevaluation were much more detailed than those used to review other Group 1 plants. Moreover, because there were no original force calculations, seismic stresses for the structures were evaluated at many locations. Note however, that the complexity of the models and the calculations of seismic stresses does not imply that this SEP review is intended to be a design analysis; the scope is limited to that explained in this section.

The Safe Shutdown Earthquake (SSE) is the only earthquake level considered because it represents the limiting seismic loading to which the plant must respond safely. Present licensing criteria sometimes result in the Operating Basis Earthquake (OBE), which is usually 1/2 the SSE, controlling the design of structures, systems, and components for which operation, not safety, is at issue. Because a plant designed to shut down safely following an SSE will be safe for a lesser earthquake, investigation of the effects of the OBE was deemed unnecessary.

Safety for seismic excitation implies that certain elements and components of an entire system must continue to function under normal operating and test loads. The seismic review team did not review all aspects of the plant's operation and the safety margins available to assure that vital elements and components would withstand unusual operating conditions, sabotage, operator error, or other nonseismic events.

The report addresses structures, systems, and components in the as-built condition and considers those modifications since the issuance of the operating license that have been made to all seismic Category I components. Information about structures, systems, and components was primarily obtained from the Ginna docket (Docket 50244) maintained by the NRC in Bethesda, Md. Additional information was supplied by the utility and the architect-engineer either through correspondence or during site visits.

Additional information about the general nature of SEP reassessments is provided in Appendix B.

1.2 PLANT DESCRIPTION

Owned and operated by the Rochester Gas and Electric Corporation (RG&E), the Robert E. Ginna Nuclear Power Plant is located on the south shore of Lake Ontario, 16 mi east of Rochester, N. Y. (Fig. 1). The plant is a pressurized light-water moderated and cooled nuclear reactor, commonly designated as a PWR. The plant was designed to produce 1300 MW of heat and 420 MW of gross electrical power.

Westinghouse Electric Corporation was the prime contractor for the plant. Westinghouse engaged Gilbert Associates, Incorporated, of Reading, Penn., as the architect-engineer responsible for the plant design. Gilbert Associates also prepared the specifications for construction, which was done by Bechtel Corporation.

The Atomic Energy Commission issued Construction Permit No. PPR-14 to RG&E on April 25, 1966. Provisional Operating License No. DPR-18 was issued on Sept. 19, 1969. RG&E filed for a full-term operating permit on Aug. 9, 1972.

FIG. 1. Aerial photograph of the Robert E. Ginna Nuclear Power Plant, located on the south shore of Lake Ontario, about 16 mi east of Rochester, N. Y.

1.2.1 Seismic Categorization

According to Sec. 5.1.2.4 of Ref. 8, the plant equipment and structures were categorized in one of three seismic classes, based on Refs. 9 and 10, as follows:

Class I. Those structures and components, including instruments and controls, the failure of which might cause a loss-of-coolant accident (LOCA), increase the severity of a LOCA, or result in an uncontrolled release of excessive amounts of radioactivity. Also, those structures and components vital to safe shutdown and isolation of the reactor.

Class II. Those structures and components that are important to reactor operation but not essential to safe shutdown and isolation of the reactor.

Their failure could not result in the release of substantial amounts of radioactivity.

Class III. Those structures and components that are unrelated to reactor operation or containment.

Note that these classifications differ from those in Regulatory Guide 1.29 (Ref. 11), which was issued after the design of Ginna.

1.2.2 Principal Structures

A complex of interconnected buildings surrounds the containment building (Fig. 2). Though contiguous, these buildings are structurally independent of the containment building (Fig. 3). However, several Class I structures are

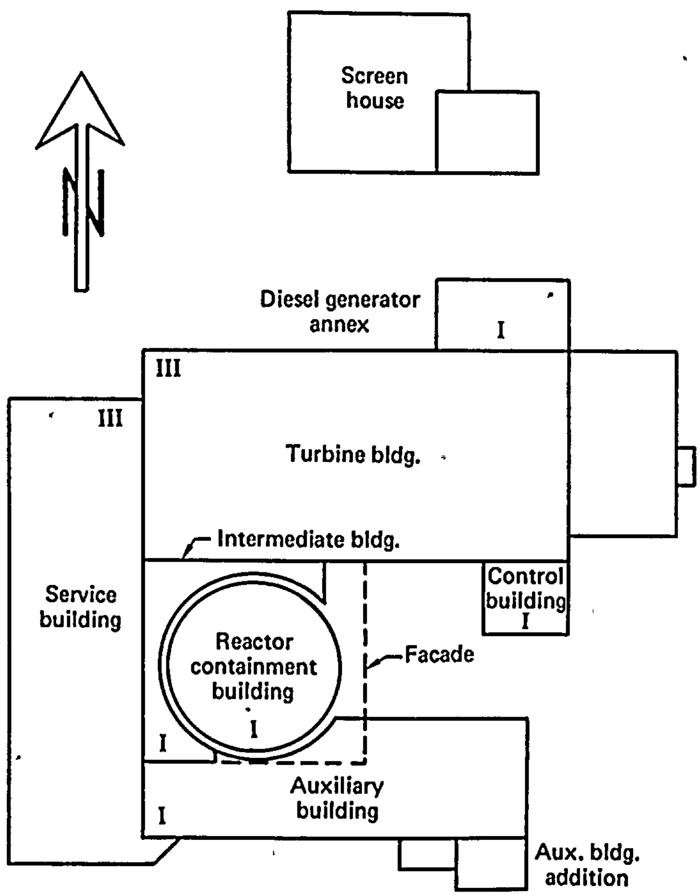

FIG. 2. Schematic plan view of the major Ginna structures shows the structurally independent containment building and the complex of interconnected seismic Class I and Class III structures.

FIG. 3. A 1-in. gap (arrow) separates the floors and roofs of the intermediate building from the containment building. Note that the intermediate building floors are supported by columns and are not cantilevered from the containment structure.

connected to Class III structures. The auxiliary building (Class I) is contiguous with the service building (Class III) on the west side. The intermediate building (Class I) adjoins the service building (Class III) to the west, the turbine building (Class III) to the north, and the auxiliary building to the south. The turbine building adjoins the diesel generator annex (Class I) to the north and the control building (Class I) to the south. The facade--a cosmetic rectangular structure that encloses the containment building--has all four sides partly or totally in common with the auxiliary and intermediate buildings (Fig. 4). These structures are described in greater detail in Appendix C.

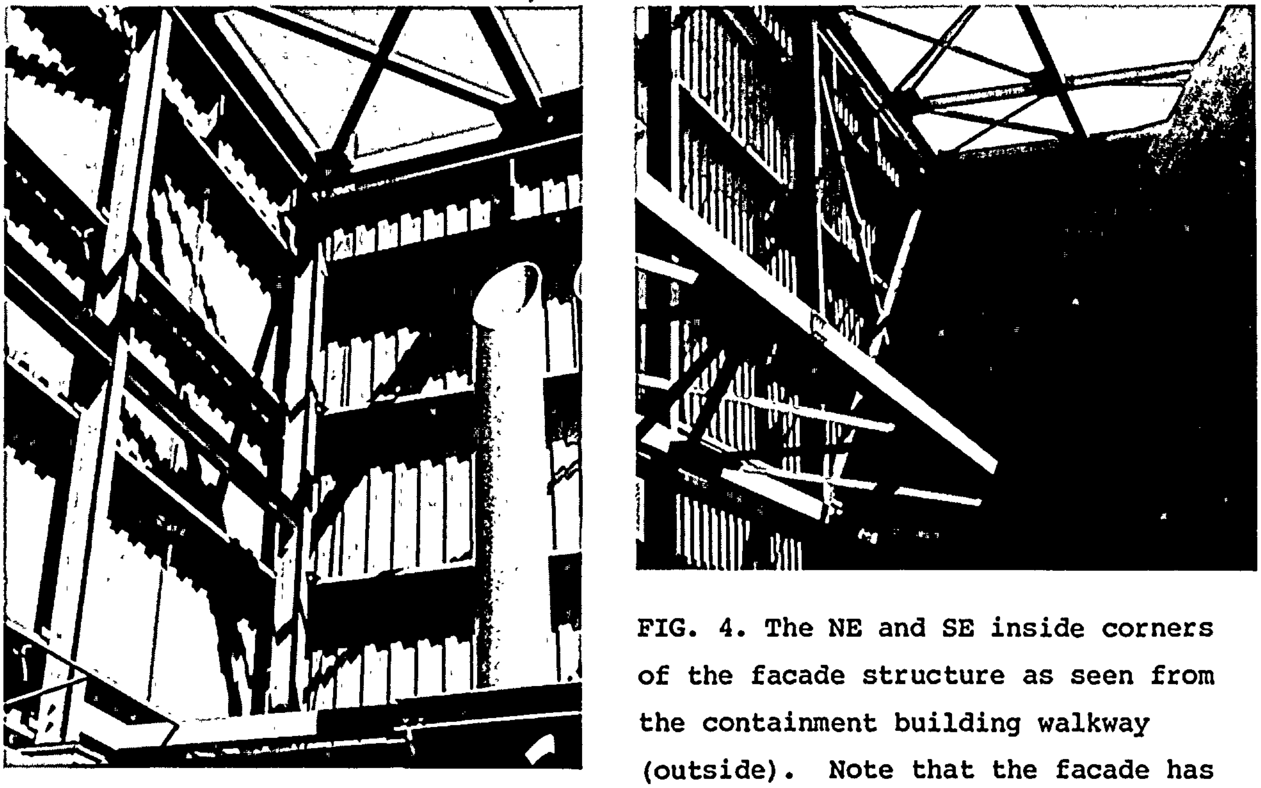

FIG. 4. The NE and SE inside corners of the facade structure as seen from the containment building walkway (outside). Note that the facade has no roof, only a horizontal truss connecting the four sides to provide out-of-plane support.

The reactor containment building is a vertical, cylindrical reinforced concrete structure (Fig. 5). It has prestressed tendons in the cylindrical wall (vertical direction only), a reinforced concrete ring anchored to bedrock, and a reinforced hemispherical dome, all designed to withstand the pressure of a LOCA. Two closed reactor coolant loops connected in parallel to the reactor vessel comprise the Reactor Coolant System (RCS). Each loop has a reactor coolant pump and a steam generator, and one has an electrically heated pressurizer (Fig. 6). The containment building is described in more detail in Sec. 4.5.

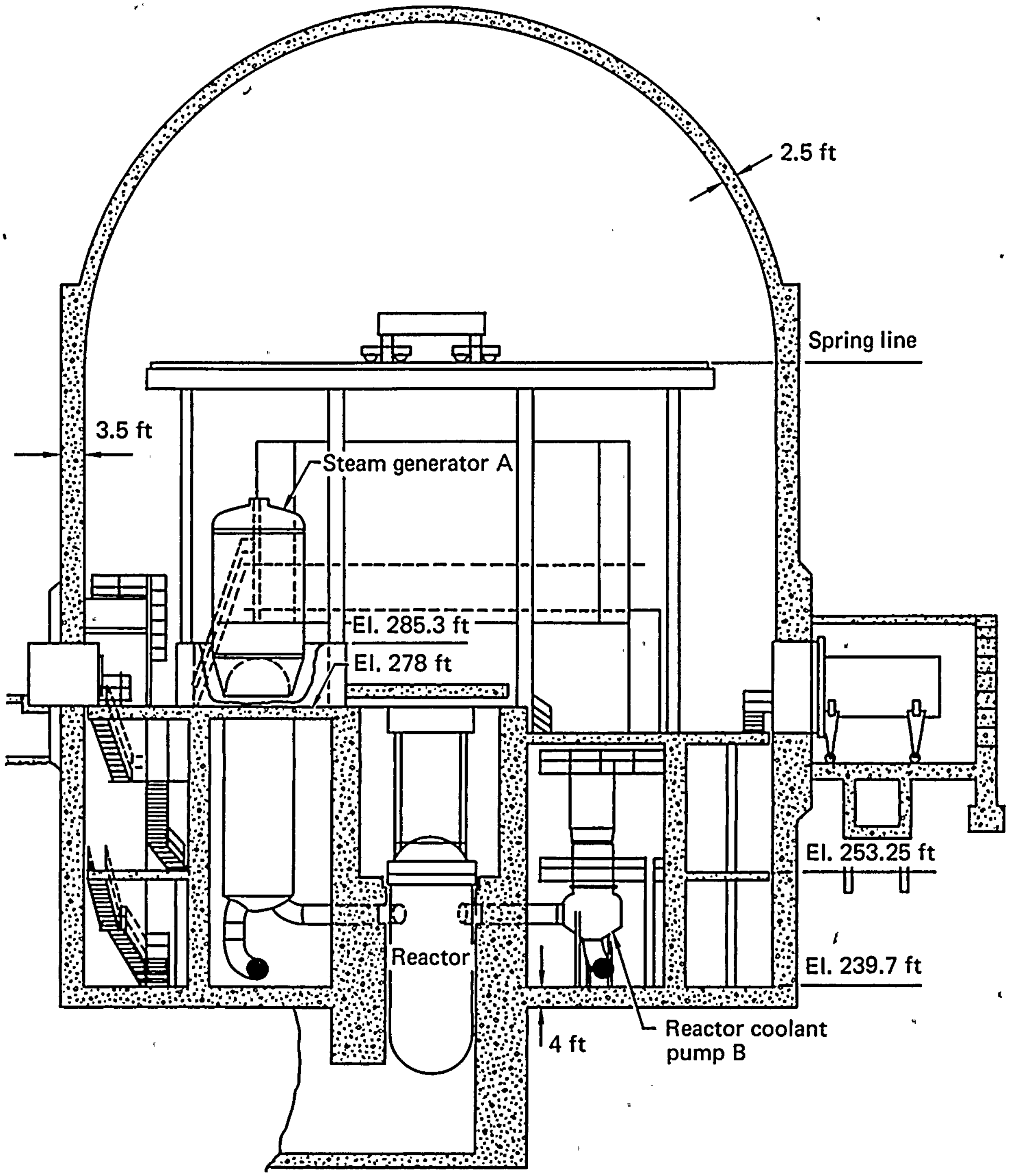

FIG. 5. The reactor containment building is a vertical, cylindrical reinforced concrete structure that has prestressed tendons in the cylindrical wall (vertical direction only) and a reinforced hemispherical dome.

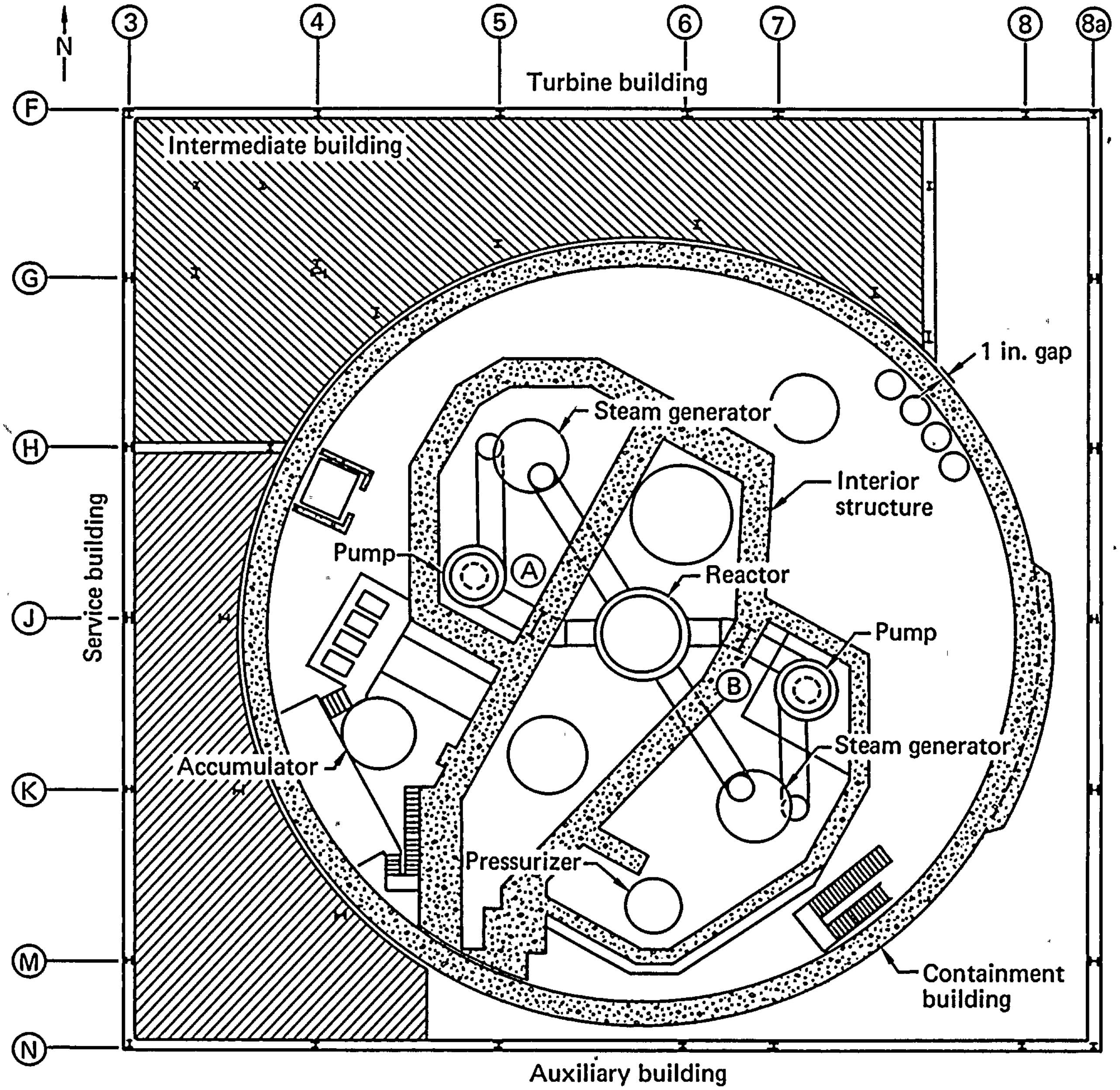

FIG. 6. Plan view of the facade structure and containment. Note: discontinuity of intermediate building floors at column line H; 1-in. gap surrounding the containment building; another gap separating the internal structure from the containment wall; and two closed reactor coolant loops (A and B) connected in parallel to the reactor vessel.

1.3 ORGANIZATION OF REPORT

The report has five chapters. Chapter 2 is a summary of the overall assessment of the ability of Ginna to resist the stipulated SSE event. Included is an evaluation of the significance of any identified deficiencies or areas that may require further study. Chapter 3 is a review of the original facility seismic design methods and criteria for structures, equipment, and piping. Where available, original calculated seismic responses and acceptance criteria are summarized. Chapter 4 presents our analysis of the interconnected building complex, the containment building, and the containment internal structures to estimate structural adequacy and to generate seismic input to equipment. In Chapter 5, the in-structure response spectra presented in Chapter 4 and other available information are used to evaluate the ability of selected mechanical and electrical equipment and fluid- and electrical-distribution systems to resist seismic loads and to perform their necessary safety functions.

CHAPTER 2: SUMMARY AND CONCLUSIONS

Within the limited scope of this reevaluation (see Chapter 1), we examined typical structures, equipment, components, and systems individually, to:

- Assess the adequacy of the existing plant to function properly during and following an SSE.
- Qualitatively judge the overall margin of safety with regard to seismic resistance.
- Make specific recommendations on upgrading or retrofitting, as appropriate.

We evaluated the containment building and its internal structures and the interconnected building complex to demonstrate structural adequacy and to obtain seismic input to equipment. For the SSE structural evaluation, a peak horizontal ground acceleration of 0.2 g was used along with Regulatory Guide (R.G.) 1.60 response spectra.

2.1 STRUCTURES

A structure was generally judged to be adequate without the need for additional evaluation if one of the following three criteria was met:

A. Reassessment loads were less than original design loads.

B. Reassessment seismic stresses were low compared to the yield stress of steel or the compressive strength of concrete.

C. Reassessment seismic stresses exceeded the steel yield stress or the concrete compressive strength, but estimated reserve capacity (or ductility) of the structures was such that we expect inelastic deformation without failure.

2.1.1 CONTAINMENT BUILDING AND INTERIOR STRUCTURES

The containment building is the only structure with enough information about the original seismic design and analysis to evaluate on the basis of criterion A above. The original design loads were derived from an equivalent static analysis and checked by a response spectrum analysis using Housner spectra. Our reanalysis gave seismic loads higher than those of the original Housner response spectrum analysis, but lower than the original equivalent static design loads.

Evaluated on the basis of criterion B above, estimated reassessment seismic stresses of interior structures--including concrete shield walls, steel and concrete columns, and crane support structures--are low.

The containment building and its interior structures are, therefore, considered able to withstand the 0.2 g SSE.

2.1.2 INTERCONNECTED BUILDING COMPLEX

No original seismic design calculations were available for comparison with current analysis techniques and criteria. Therefore, a detailed three-dimensional beam model was developed to capture the effects resulting from torsion and the complex interconnections between Class I and Class III buildings. The model was analyzed according to the procedure shown in Fig. 17 of Sec. 4.

Stress calculations indicate that there are three highly stressed areas in the braced steel frames of these buildings--the east end of the auxiliary building, the south wall of the turbine building, and the west facade structure. This finding of high seismic stresses in portions of the bracing is not surprising since the systems were originally designed on the basis of a two-dimensional wind load analysis, and the eccentricity of mass and stiffness distributions was not considered. These weak areas and our recommendations are discussed below.

2.1.2.1 Auxiliary Building

The N-S steel braced frame above the operating floor that supports the northeast corner of the low roof has a safety factor (defined as f_y/f) of about 0.8. There is only one other lateral load-resisting system for the auxiliary building superstructure in the N-S direction (the bracing between the high and low roofs), and its stress is close to yield. Therefore, we recommend upgrading at the east end of the auxiliary building to provide adequate lateral load resistance (Fig. 7).

2.1.2.2 Turbine Building

The lateral load-resisting system for turbine building floors has stresses below yield. However, stresses in the cross bracings above the operating floor in the south, north, and west walls exceed yield. The bracings right above the control building superwall have the lowest safety factor (0.7). These bracings sustain high loads because of the relatively high stiffnesses of the superwall and the control building compared to the turbine building frames.

We recommend upgrading of the turbine building south wall near the superwall to improve lateral load-resisting capacity (Fig. 7). Upgrading should also help that part of the turbine frame above the operating floor where redundant resistance to lateral loads is low.

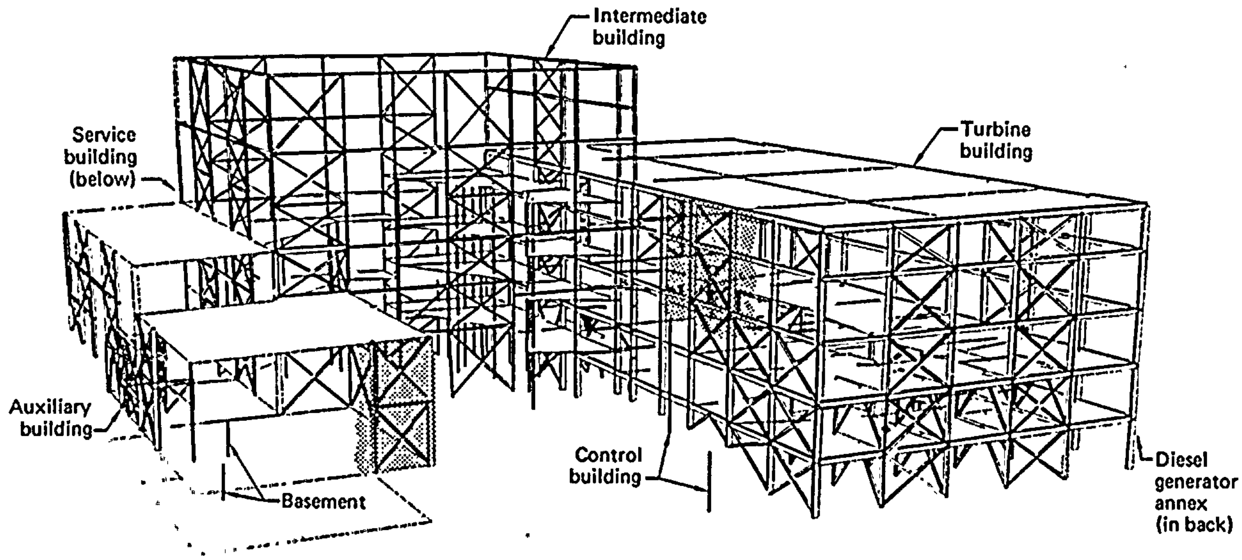

FIG. 7. Three-dimensional representation of the interconnected building complex shows areas (shaded) where upgrading is recommended to improve lateral load-resisting capacity.

2.1.2.3 Intermediate Building and Facade Structure

The braced frames in the low portion of the west facade have stresses at or a little over yield (safety factor of 0.9). However, the lateral load-resisting systems have more reserve capacity than do the braced steel frames of the auxiliary building discussed above. For example, the interior columns supporting the floors and nonstructural members, such as stairway structures between floors and sidings, provide reserve lateral support. We believe that enough strength and ductility of the structure can be mobilized to withstand the 0.2 g SSE.

2.1.2.4 Control Building and Diesel Generator Building

Both the control and diesel generator buildings are believed to have enough strength and ductility to resist the seismic shear forces resulting from the 0.2 g SSE.

2.2 MECHANICAL AND ELECTRICAL EQUIPMENT, AND FLUID- AND ELECTRICAL-DISTRIBUTION SYSTEMS

As discussed in Chapter 5, typical mechanical and electrical equipment components were selected for review in large part on the basis of the judgment and experience of an SEP seismic review team comprised of the authors plus certain SSRT and NRC staff members. The documentation that exists regarding the original specifications applicable to procurement of equipment, as well as documentation concerning qualification of the equipment, varies greatly. In some cases the qualification for an item of equipment is quite specific, whereas in other cases the qualification is of a generic sense with regard to a class of equipment and not the specific item in the Ginna facility.

Based upon the design review and independent calculations made for this reassessment for the SEP seismic load condition, we recommend design modifications or reanalysis be undertaken by the licensee for the following mechanical and electrical components to demonstrate their capability to withstand the 0.2 g SSE without loss of structural integrity and required safety function (specific recommendations are presented in Chapter 5):

1. Essential service water pump
2. Motor-operated valves
3. Component cooling surge tank
4. Refueling water storage tank
5. Battery racks.

Because we lacked essential seismic design/qualification data, as discussed in Table 10 of this report, our review of the seismic design adequacy of mechanical and electrical equipment is incomplete. Additional data in the form of analysis or test results must be developed before the status of the seismic design adequacy of equipment can be definitively determined.

2.3 CONCLUDING REMARKS

Based on the combined experience and judgment of the members of the SEP seismic review team, the reviews of the original design analyses, and comparisons with similar items of equipment and components in other more recently designed reactors, we conclude that:

1) Structures and structural elements of the Ginna facility are adequate to resist an earthquake with a peak horizontal ground acceleration of 0.2 g, subject to the condition that structural upgrading recommended in Sec. 2.1.2 is implemented. In designing upgrades, consideration should be given to the effects of the modifications on the rest of the structural system. Note also that concrete block portions of Ginna were not addressed in this study. This material may perform poorly when subjected to seismic loading, and the NRC is studying this generic issue. We recommend that Ginna be included in these studies. The tall block wall between the intermediate and turbine buildings is of particular concern.

2) In view of the limited amount of both analysis and test documentation, no definitive statement can be made about the overall seismic design adequacy of mechanical and electrical equipment. More data must be developed before equipment seismic design adequacy can be determined in accordance with evaluation criteria in this report.

The SEP seismic review team recommends that:

- Specific mechanical and electrical equipment items listed in Sec. 2.2 that were found to be inadequate be upgraded as recommended in Chapter 5.

- All safety-related electrical equipment in the plant be checked for adequate engineered anchorage; that is, the anchorage should be found to be adequate on the basis of analysis or tests employing design procedures (load, stress and deformation limits, materials, fabrication procedures, and quality acceptance) in accordance with a recognized structural design code.

- A general reconnaissance of the plant be made to identify items that are (1) overhead or suspended, (2) on rollers, or (3) capable of sliding or overturning. All such items, whether permanently installed or not, that could dislodge, fall, or displace during an earthquake and impair the capability of the plant to shut down safely should be upgraded so that they no longer jeopardize the plant.

Although not within the scope of this reassessment, we also recommend that the calculated moment caused by the original operating and pressure loads for the containment building (Figs. 13 and 14) should be checked. There is no moment shown at the cylinder-sphere interface as would be expected. We recognize that the relative stiffness between the partially prestressed cylinder and the nonprestressed dome may reduce the moment at the interface. Nevertheless, our concern is whether adequate moment capacity exists at this interface under the 60 psi internal pressure loading.

CHAPTER 3: PREVIOUS SEISMIC ANALYSES

3.1 INTRODUCTION

This chapter presents the original seismic design methods, results, and criteria for Ginna. The seismic loadings for seismic Class I structures, equipment, and piping are defined, and the seismic responses and allowable stress criteria for critical structures are outlined. Information presented in this chapter is intended to define the design basis for comparison with current seismic design methods and criteria in Chapter 4. The information was pieced together from many sources, primarily a search of the Ginna Docket (Ref. 12) and letter reports from RG&E (Refs. 6 and 13) in response to NRC questions. Few original documents concerning seismic design other than the FDSAR (Ref. 8) were on the docket or available from the utility.

3.2 DESIGN EARTHQUAKE MOTION

Ginna was designed for an OBE characterized by a peak horizontal ground acceleration (A_{max}) of 0.08 g and reviewed for an SSE with an A_{max} of 0.2 g. Peak horizontal and vertical accelerations were assumed to be the same. Response spectra used were those developed by Housner (Fig. 8).[10]

3.3 SEISMIC ANALYSIS

3.3.1 Methods of Analysis

Most seismic Class I structures and equipment were analyzed by the equivalent static method. The maximum response acceleration of a structure or equipment item was read from the response spectrum for selected values of damping and a fundamental natural frequency. The frequency was

- Calculated from a mathematical model,
- Measured from a plastic model (the case of the reactor coolant system),
- Estimated by experience, or
- Selected to be conservative (the peak of the spectrum was used).

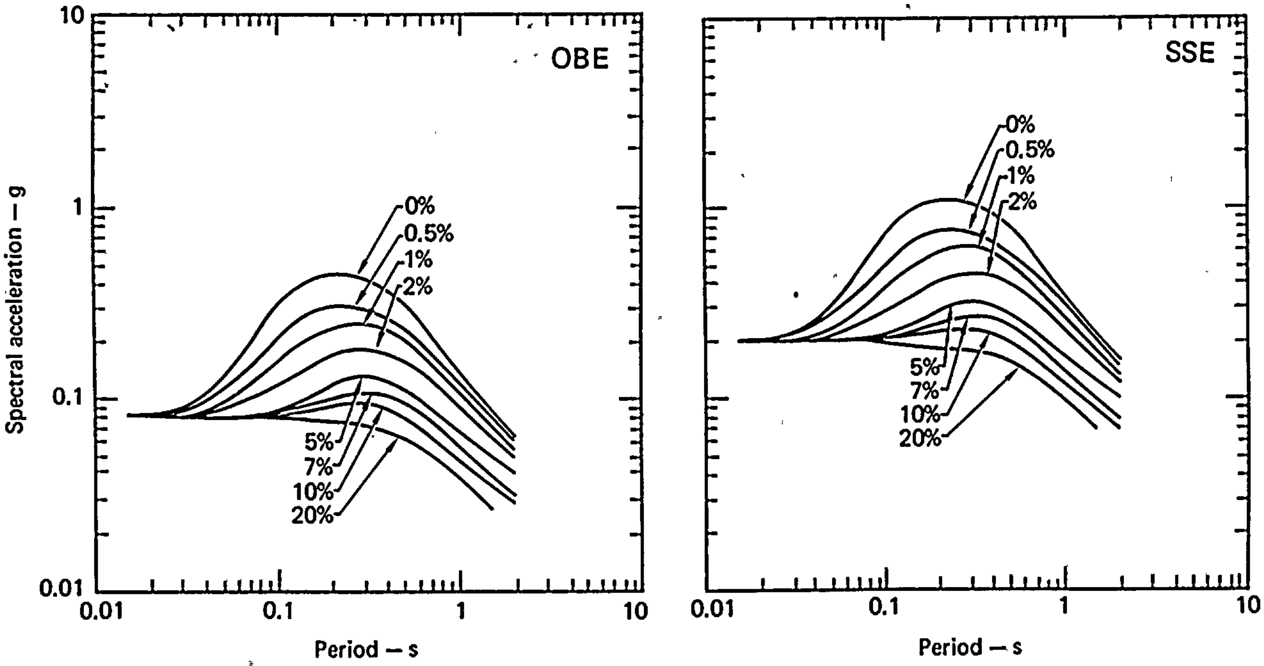

FIG. 8. Housner seismic response spectra for the OBE analysis (A_{max} = 0.08 g) and the SSE analysis (A_{max} = 0.2 g) for various levels of damping, from Ref. 8.

From the mass of the structure or equipment and the maximum response acceleration, the equivalent static force was obtained. The equivalent static force, which represents the total dynamic effect, was then distributed along the system according to a selected shape (an inverted triangle for the containment vessel) or according to the mass distribution. The static response to this equivalent static force was taken to be the seismic response of the system. Responses to horizontal and vertical ground accelerations were calculated separately, then combined by direct addition in most cases.

The containment vessel and the residual heat removal system (RHRS) pipe line from the reactor coolant system (RCS) loop to containment were analyzed by both the equivalent static and the response spectrum methods.

3.3.2 Damping

Damping values, where available, are given for each structure or system component reviewed below. General damping values are compared to those now recommended in Sec. 4.2 of this report.

3.3.3 Soil-Structure Interaction

Soil-structure interaction was not considered in the design of Ginna. The effect of neglecting this interaction is discussed in Sec. 4.3.1 of this report.

3.4 STRESS CRITERIA

According to the FDSAR (Ref. 8, Secs. 5.1.2.4 and 7.2), all seismic Class I components, systems, and structures were designed to meet the following criteria:

- Primary steady-state stresses, when combined with the seismic stress from simultaneous 0.08-g peak horizontal and vertical ground accelerations, are maintained within the allowable working stress limits accepted as good practice and, where applicable, set forth in the appropriate design standards (ASME Boiler and Pressure Vessel Code, USAS B31.1 Code for Pressure Piping, ACI 318 Building Code Requirements for Reinforced Concrete, and AISC Specifications for the Design and Erection of Structural Steel for Buildings).

- Primary steady-state stresses, when combined with the seismic stress from simultaneous 0.2-g peak horizontal and vertical ground accelerations, are limited in such a way that the safe-shutdown function of the component, system, or structure is unimpaired.

There are no Class II structures at Ginna. The Class III structures (see Fig. 2) were designed to meet the 1961 State Building Construction Code for the state of New York.

3.5 SEISMIC ANALYSIS OF STRUCTURES

The original analyses of seismic Class I structures are summarized in this section. Construction details of many of the structures are presented in Chapter 4 and Appendix C.

3.5.1 Containment Building

A seismic Class I structure, the containment building was modeled for the equivalent static design analysis as a fixed-base cantilever beam (Ref. 8, Sec. 5.1.2.). Damping was 2%; bending stiffness was based on a Young's modulus of 4.1×10^6 psi; shear stiffness was based on a shear modulus of 1.8×10^6 psi. Vertical response was assumed to be unamplified because of the high axial stiffness.

The period of the first harmonic was calculated to be 0.22 s for horizontal motion and 0.07 s for vertical motion. From the SSE response spectrum for 2% damping at 0.22 s, the maximum horizontal spectral acceleration was found to be 0.46 g. The resultant shear load was assumed to be distributed in the form of an inverted triangle extending the full height of the vessel. The resulting peak shear forces are shown in Fig. 9.

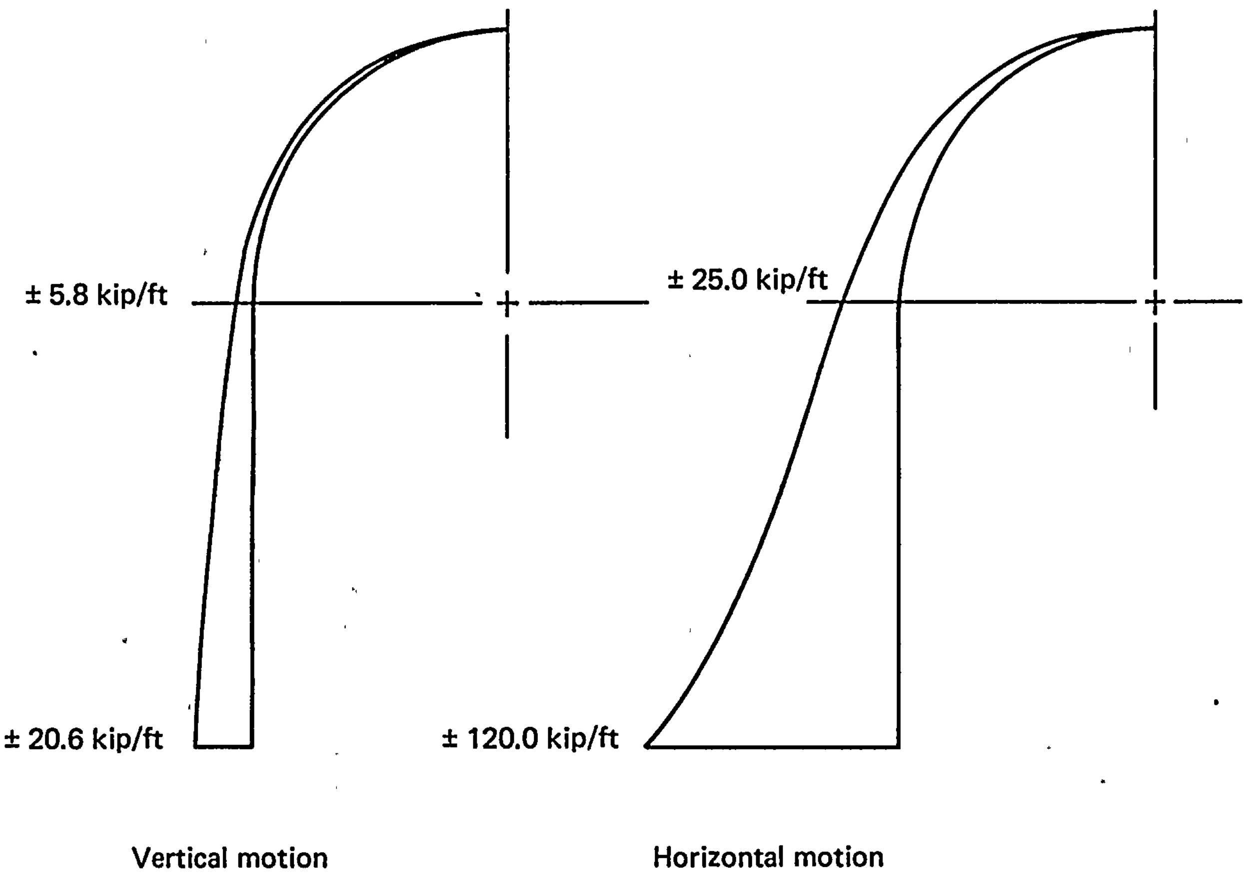

FIG. 9. Peak shear forces acting on the containment structure, from the equivalent static design analysis in Ref. 8.

As a check on the static analysis, the containment vessel was modeled as a fixed-base system of lumped masses connected by weightless springs (Fig. 10) and analyzed by the response spectrum technique for 2% of critical damping. The normal mode values were calculated by the computer program SAND (a modified version of a Jet Propulsion Laboratory code). Table 1 shows the periods, effective masses, and response accelerations for the six modes. Figure 11 shows the first three mode shapes. Responses of each mode were summed on a "root-mean-square basis" (Square-root-of-the-sum-of-the-squares basis was probably the intended meaning) by program SPECTA. The resulting shear forces and moments are shown in Fig. 12.

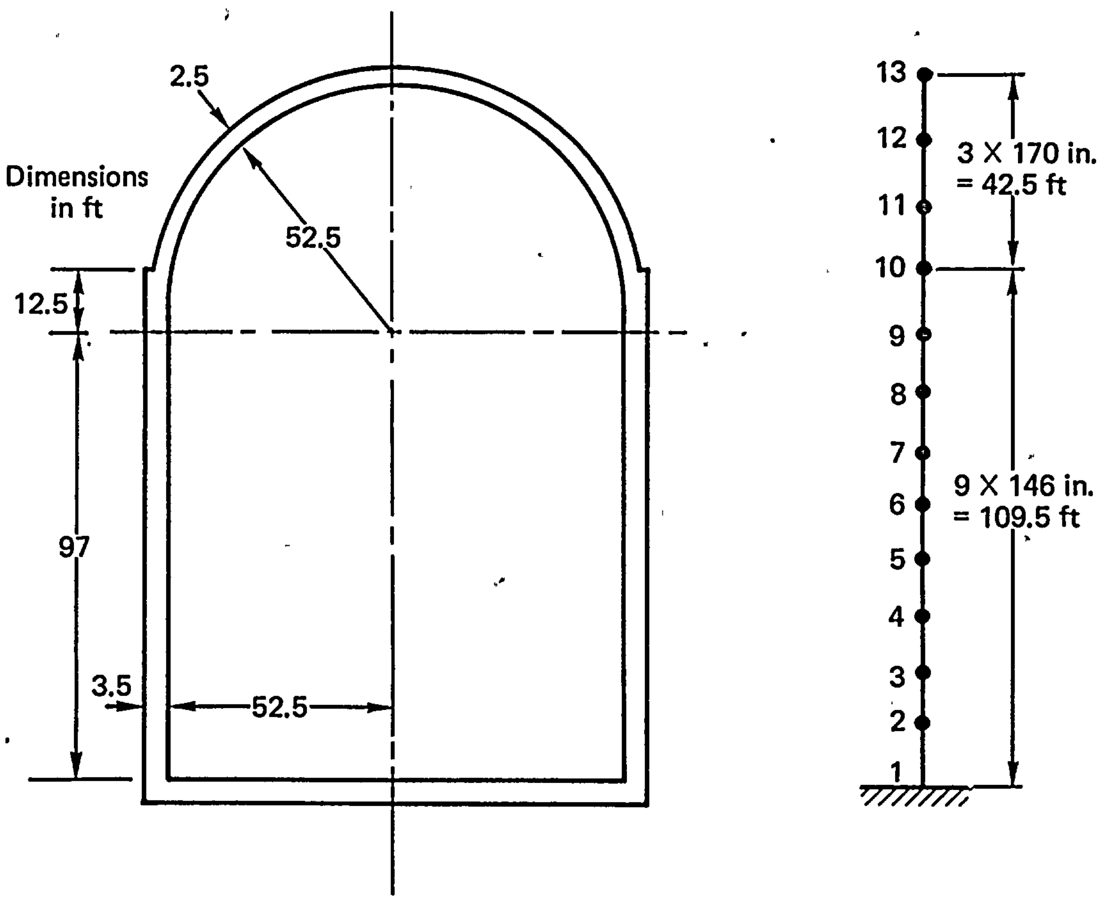

FIG. 10. Cross section and mathematical model of the containment structure for the response spectrum analysis conducted to check the design analysis, from Ref. 8.

TABLE 1. Periods, effective masses, and response accelerations for the response spectrum analysis of the containment vessel, from Ref. 8, Sec. 5.1.2.

Mode no.	Period, s	Effective mass[a], 10^6 lb	Response acceleration, g OBE	SSE
1	0.144	18.46	0.14	0.36
2	0.052	4.78	0.09	0.22
3	0.029	0.30	0.08	0.20
4	0.026	0.92	0.08	0.20
5	0.018	0.51	0.08	0.20
6	0.015	0.05	0.08	0.20
	Total	25.02		

[a]This mass is assumed to be the square of the modal participation factor divided by the generalized mass.

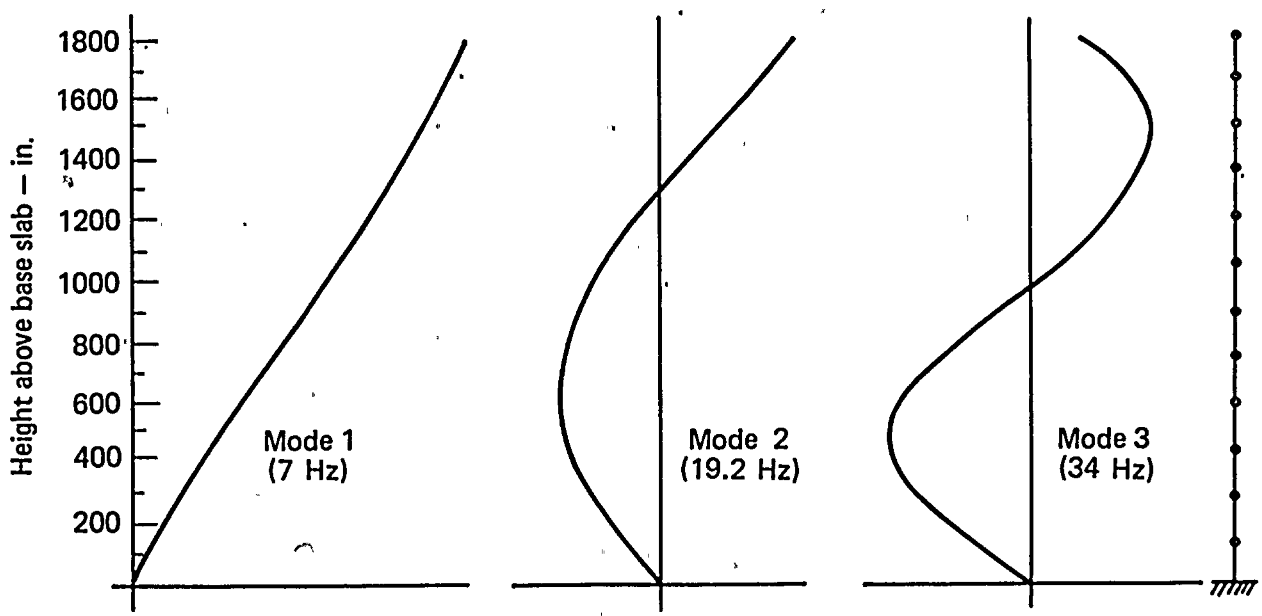

FIG. 11. First three mode shapes for the containment structure model of Fig. 10, from Ref. 8.

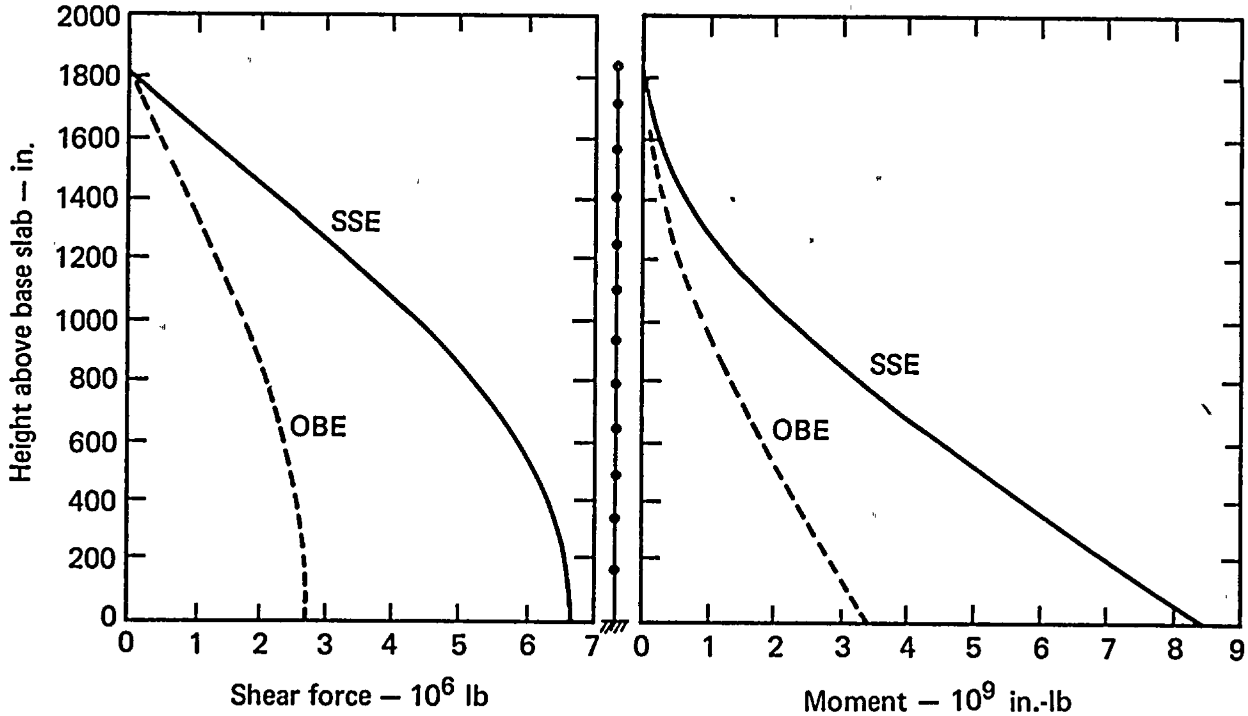

FIG. 12. Shear force and moment calculated for the containment structure by the response spectrum method using 2% of critical damping, from Ref. 8.

The effect of a flexible foundation was considered in a second dynamic analysis by assuming the foundation rock to have elastic properties similar to those of concrete ($E = 3 \times 10^6$ psi; $\nu = 0.2$). Calculated deflections, accelerations, shear forces, and moments differed from the rigid-base values by less than 5%.

To determine the required limiting capacity of any structural element of the containment vessel, three load combinations were considered (Ref. 8, Sec. 5.1.2.3):

(a) $C = 0.95\ D + 1.5\ P + 1.0\ T$

(b) $C = 0.95\ D + 1.25\ P + 1.0\ T' + 1.25\ E$

(c) $C = 0.95\ D + 1.0\ P + 1.0\ \underline{T} + 1.0\ E'$

where

C = required load capacity of section

D = dead load of structure

P = accident pressure load (60 psig)

T = thermal loads based upon the temperature transient associated with 1.5 times the accident pressure

T' = thermal loads based upon the temperature transient associated with 1.25 times the accident pressure

$\underline{T}$ = thermal loads based upon temperature transient associated with the accident pressure

E = OBE-based seismic load

E' = SSE-based seismic load.

Results for load combinations (b) and (c) are presented in Figs. 13 and 14, respectively.

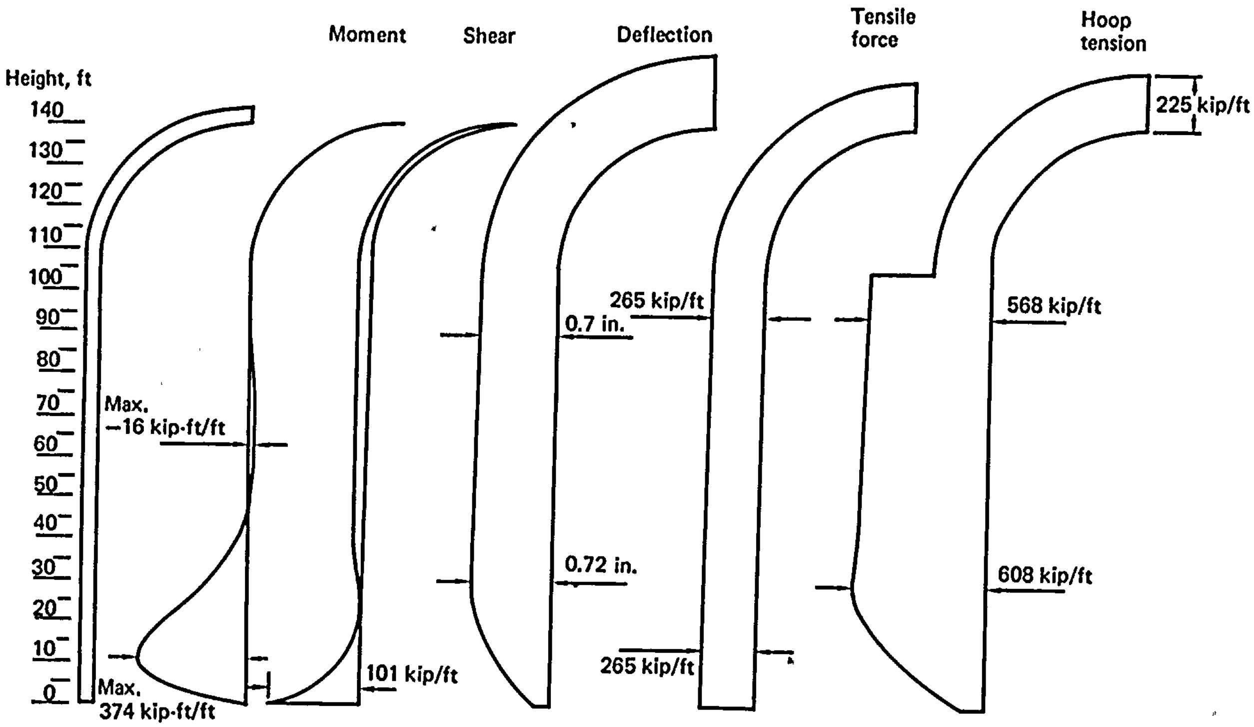

FIG. 13. Operating loads plus incident loads plus OBE-based loads (0.95D + 1.25P + T' + 1.25E) from the response spectrum analysis of the containment vessel, from Ref. 8, Sec. 5.1.2.

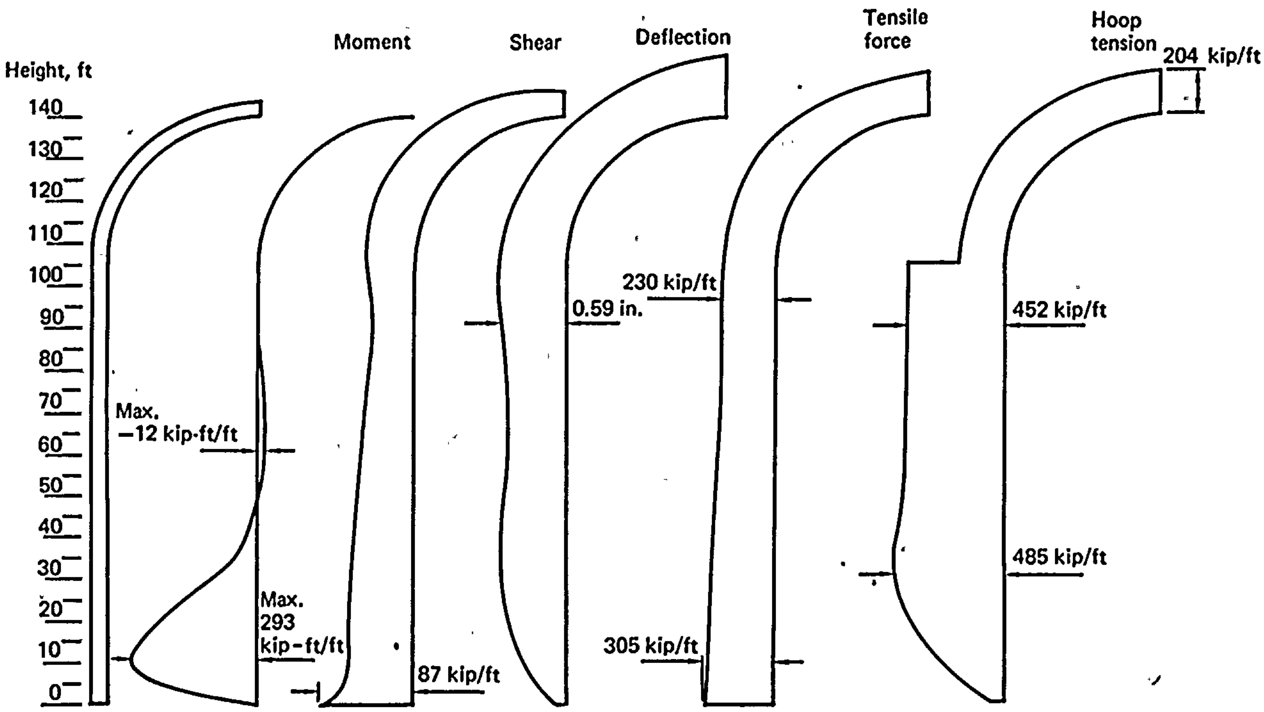

FIG. 14. Operating loads plus incident loads plus SSE-based loads (0.95D + P + $\underline{T}$ + E') from the response spectrum analysis of the containment vessel, from Ref. 8, Sec. 5.1.2.

3.5.2 Containment Building Internal Structures

Reinforced concrete structures inside the reactor containment building were reportedly modeled as simple cantilever beams with all mass lumped at the center of gravity (c.g.).[6] Analysis was by the equivalent static method as follows:

- The fundamental period was calculated based on the assumption that the structure is a simple harmonic oscillator.
- The response acceleration was taken from the appropriate response spectrum (Fig. 8).
- This acceleration times the total mass acting at the c.g. gave the shear force and overturning moment at the base.

- The shears and moments were distributed throughout the model in proportion to structural stiffness, which was based on the flexural properties of the wall systems.
- Structural element design capacity was evaluated.

Walls and floor slabs were designed for the concentrated seismic reactions of the attached major components.

Overhead crane support structures within the containment building were reportedly evaluated for natural periods of simple harmonic motion in the two horizontal directions. Equivalent horizontal seismic forces were then obtained by applying the corresponding acceleration from the seismic response spectra to the mass of the crane. Vertical response of the crane and crane support structure was taken as the peak of the response spectra. Vertical forces were obtained by applying the peak acceleration to the mass of the crane, crane support structure, and lifted load.

No other details or original documentation of these analyses exist or were made available.

3.5.3 Auxiliary Building

RG&E reported (Ref. 6) that steel superstructures above El. 271 ft had been evaluated for equivalent horizontal seismic loads based upon either the maximum spectral response or the spectrum value corresponding to the first harmonic frequency of the structure. No details of the analysis were provided. RG&E then submitted original calculations showing this superstructure to have been designed originally to withstand a wind loading of 18 lb/ft^2 (Ref. 13). RG&E also submitted a 1979 analysis by Gilbert Associates, Inc., that evaluated the structure for seismic resistance by the equivalent static method. Stiffness and mass properties from the original design analyses were used in the 1979 analysis.

3.5.4 Control Building

RG&E reported (Ref. 6) that the original seismic design of the control building was based on the OBE as follows:

- Structural steel columns were designed for flexural moments resulting from a horizontal load equivalent to 10% of the axial load applied at the mid-span of the column.

- Concrete walls above grade were subjected to a horizontal reaction normal to the wall and applied at mid-span. The wall was treated as a fixed-base cantilevered beam. The equivalent seismic load was 10% of the wall weight.

No other details or original documentation of these analyses were made available. However, engineering worksheets were provided in Ref. 13 showing the control building model developed in 1979 by Gilbert Asociates, Inc., for generating in-structure response spectra.

3.5.5 Intermediate Building

The bracing system of the intermediate building is common to the turbine, service, and auxiliary buildings and the facade structure. The bracing was reportedly checked to demonstrate that it could resist equivalent seismic load components from the above structures.[6] No details or original documentation of this check were made available.

3.5.6 Diesel Generator Building

The diesel generator building has concrete shear walls and steel framed roof structures. The seismic design of the concrete shear walls reportedly considered both in-plane and normal equivalent static loads.[6] Seismic accelerations were taken as the peak of the seismic response spectra (Fig. 8) for 5% of critical damping. The steel roof framing was reportedly designed for a horizontal equivalent SSE seismic load, taken as the mass of the roof structure and superimposed loads times the peak seismic response for 2.5% damping. Column foundations were designed for an additional 20% of axial load to account for seismic effects. No other details or original documentation of these analyses were made available.

3.5.7 Turbine Building and Service Building

The turbine and service buildings are non-Category I structures that are connected to Category I structures. For purposes of the original seismic design, coupling between the two classes of structures was not considered.

3.6 SEISMIC ANALYSIS OF PIPING

Most original piping systems were analyzed by static methods, primarily the equivalent static method. Seismic input for these analyses was based on the Housner ground response spectra (Fig. 8), not on in-structure response spectra as is done today. Peak spectral accelerations were taken from the curves for those components for which the natural frequency was estimated. If natural frequencies were unknown (vendor-designed equipment, for example) the maxima of the curves were used.

Exceptions to the static analysis approach include the analysis of

- The residual heat removal system (RHRS) line from the reactor coolant system loop A to the containment penetration.
- The main steam line from steam generator B to the containment penetration.
- The reactor coolant system.

Two response spectrum analyses of the RHRS line were performed.[14] One analysis used Fig. 8 as input; the other used a response spectrum that was a modification of the 0.5%-damping spectrum in Fig. 8 to account for building effects at the steam-line elevation. No details are available on the construction of this modified response spectrum (Fig. 2, Ref. 14).

Both static and dynamic analyses were performed on the main steam line of loop B inside the containment (Fig. 3, Ref. 14). The modified response spectrum used for the RHRS line analysis was also used for this dynamic analysis.

The reactor coolant system was qualified by tests using a plastic model.[6] Input was a sinusoidal wave for the vertical direction and each of the two horizontal directions, independently. The plastic model output--mode shapes and frequencies--was then used as input (along with the Housner spectrum, Fig. 8) to a three-dimensional mathematical model of the primary coolant loop. Stresses, deflections, support reactions, and equipment nozzle reactions were then calculated. Key results are shown in Table 2.

Table 3 summarizes the original seismic analyses of Class I piping at Ginna. A summary of modifications to the piping systems is provided in Table 2c-2 of Ref. 6. Because the piping design is now under review by RG&E, it was not reassessed as part of this review.

3.7 SEISMIC ANALYSIS OF EQUIPMENT

As was the case with piping, no in-structure response spectra were developed for the analysis of equipment. Instead, seismic Class I items were qualified on an individual and often generic basis. Qualification of major equipment items--the steam generator, control rod drive mechanisms, reactor internals, reactor vessel, and pressurizer--are summarized in Ref. 6, answer to question 2d. Appendix A provides a list of items and the basis of seismic qualification for Ginna equipment. The original seismic analyses of those equipment items selected for review are summarized by item in Chapter 5.

Seismic design requirements for Class I instrumentation and controls were reportedly specified in equipment specifications as follows:

a) Control Room
The racks have been assembled and the mounting and wiring of all components has been designed such that the functions of the circuits or equipment will perform in accordance with prescribed limits when subjected to seismic accelerations of 0.21 g in the horizontal direction and in the vertical direction simultaneously. In addition, the mounting and wiring of all components has been done such that simultaneous accelerations of 0.52 g in the horizontal and vertical planes will not dislodge, cause relative movement or result in any loss or change of function of circuits or equipment.

b) Containment and Auxiliary Building
The mounting and wiring of all components has been designed such that simultaneous accelerations of 0.52 g in the horizontal and vertical planes will not dislodge, cause relative movement, or result in any loss or change of function of circuits or equipment.

TABLE 2. Predominant frequencies and computed stresses for Class I piping systems and equipment, from Ref. 6, answer to question 2g.

Item	Frequencies, Hz	Stress, psi	Comment
Reactor coolant system[a]	1.2 1.6 3.58	2880 7680 790	
Main steam piping system	3.06	19,500	Snubbers recommended
Feedwater piping system	3.42	19,300	Snubbers recommended
Residual heat removal system piping	4.94	80,600	Snubbers recommended to minimize high stress
Reactor vessel	46	---	Rigidity confirmed
Accumulator	23.7	---	
Pressurizer	3.04	---	

[a]Motion primarily from the steam generator and the reactor coolant pump.

TABLE 3. Summary of Class I piping seismic analyses. Note: damping was 0.5% of critical.

Item	Analysis method	Input	Load combination method	Acceptance criteria	Reference no.
Reactor coolant system (RCS) piping	Equivalent static (ES)	Fig. 8 (Housner spectra)	E^a (or E'^b) + D^c + P^d	1.2 S^e (B31.1, 1956)	14 (Sec. 2.1)
Residual heat removal line from RCS loop A to containment	Response spectrum (RS) Two analyses	Fig. 8, 0.5% damping. A modified spectrum.	$I^f(E' + D) + P$	1.2 S	14 (Sec. 2.2) 6 (answer to question 2g)
Main steam line from steam generator B to containment	ES and RS	Modified spectrum	E' + normal load	1.2 S	14 (Sec. 2.2)
Safety injection piping	ES	Maximum of Fig. 8 (SSE)	---	18,750 psi	8 (Sec. 6.2)
Class I piping diam $\geq$2.5 in.	ES	Maximum of Fig. 8 (0.8 g)	E' + normal load	1.2 S^g	14 (Sec. 2.2)
Class I piping diam $\leq$2 in.	Simple beam static	Inertial load (0.8 g)	---	Code allowable	14 (Sec. 2.3)

[a]OBE-based seismic load.

[b]SSE-based seismic load.

[c]Dead load.

[d]Pressure load.

[e]Temporary overload stress from ASA B31.1, 1955 Nuclear Code Cases N-7 and N-10.

[f]Stress intensification factor.

[g]Temporary overload stress from USAS B31.1.1.0-1967, paragraph 119.6.4.

CHAPTER 4: REASSESSMENT OF SELECTED STRUCTURES

4.1 INTRODUCTION

In this chapter, seismic loads and responses derived from current analysis techniques are developed and compared with those loads and seismic responses for which Ginna structures were originally designed (see Chapter 3). Many Ginna structures have no original seismic analysis. In those cases, the derived loads are compared with current seismic criteria. These comparisons are made to identify those regions of the plant that would essentially meet current seismic design criteria and those regions of the plant that might exhibit low margins compared to current criteria and need to be investigated further.

Seismic loadings and responses are examined for the complex of interconnected buildings comprising the control, auxiliary, intermediate, turbine, service, and diesel generator buildings and for the reactor containment building and its internal structures. In addition, seismic input motions (in-structure response spectra) are developed based on current design practice for locations throughout the buildings where seismic Category I equipment and piping are supported. These response spectra are used to reassess equipment in Chapter 5.

4.2 DESIGN EARTHQUAKE MOTION

This section presents the ground motion parameters used in the reassessment of Ginna structures and compares them with those originally used for design.

As discussed in Chapter 3, Ginna was designed for an equivalent OBE peak horizontal ground acceleration (A_{max}) of 0.08 g and an SSE A_{max} of 0.2 g. A simultaneous vertical component of earthquake motion equal to the horizontal component was considered in the plant design. For this reassessment, an SSE A_{max} of 0.2 g was also used for the horizontal component, and 2/3 of this value was used for the vertical component.

In addition to specifying A_{max}, either a design time history (or histories) or ground response spectra are also needed to define the design earthquake. Most of the original equivalent static design analyses involved response spectra developed by Housner (Fig. 8) based on the records from four events available at the time.

Typical current practice is to specify either site-dependent spectra, or, as is more often the case, averaged ground response spectra like those in R.G. 1.60 (Ref. 7). These spectra are based on the mean plus one standard deviation of spectra generated from a series of strong-motion earthquake records that include horizontal and vertical components for both rock and soil sites. We used R.G. 1.60 spectra in our reassessment.

Rather than compare response spectra directly for equal damping values, it is more informative to include the damping used in the design of Ginna. Table 4 lists the damping values used for Ginna together with those from R.G. 1.61 (Ref. 15) for the SSE and those values recommended in NUREG/CR-0098 (Ref. 4) for structures at or below the yield point. The damping values used in the design of Ginna are lower than current design levels. One reason is that the design damping values were used for the OBE, and the design loads were increased for the SSE evaluation in direct proportion to the ratio of the two values of A_{max}. Because higher response and, consequently, increased damping are expected for the SSE, a significant degree of conservatism was typically introduced over current practice.

TABLE 4. Original and currently recommended damping values.

Structure or component	Percent of critical damping		
	Ginna	R.G. 1.61[a] (SSE)	NUREG/CR-0098[b] (Yield levels)
Prestressed concrete	2	5	5 to 7
Reinforced concrete	5	7	7 to 10
Steel frame	1 or 2.5	4 or 7	10 to 15
Welded assemblies	1	4	5 to 7
Bolted and riveted assemblies	2.5	7	10 to 15
Vital piping	0.5	2 or 3	2 to 3

[a]Ref. 15.
[b]Ref. 4.

A comparison of the response spectrum developed by Housner for 2% damping with the 7% spectrum from R.G. 1.60 indicates the relative magnitudes of the response of bolted steel structures and equipment designed to Ginna versus current criteria. Similarly, the 0.5% spectrum for the original design and the 3% spectrum from R.G. 1.60 may be used to compare expected levels of response for base-level-mounted large piping for the two criteria. Figure 15 shows these comparisons.

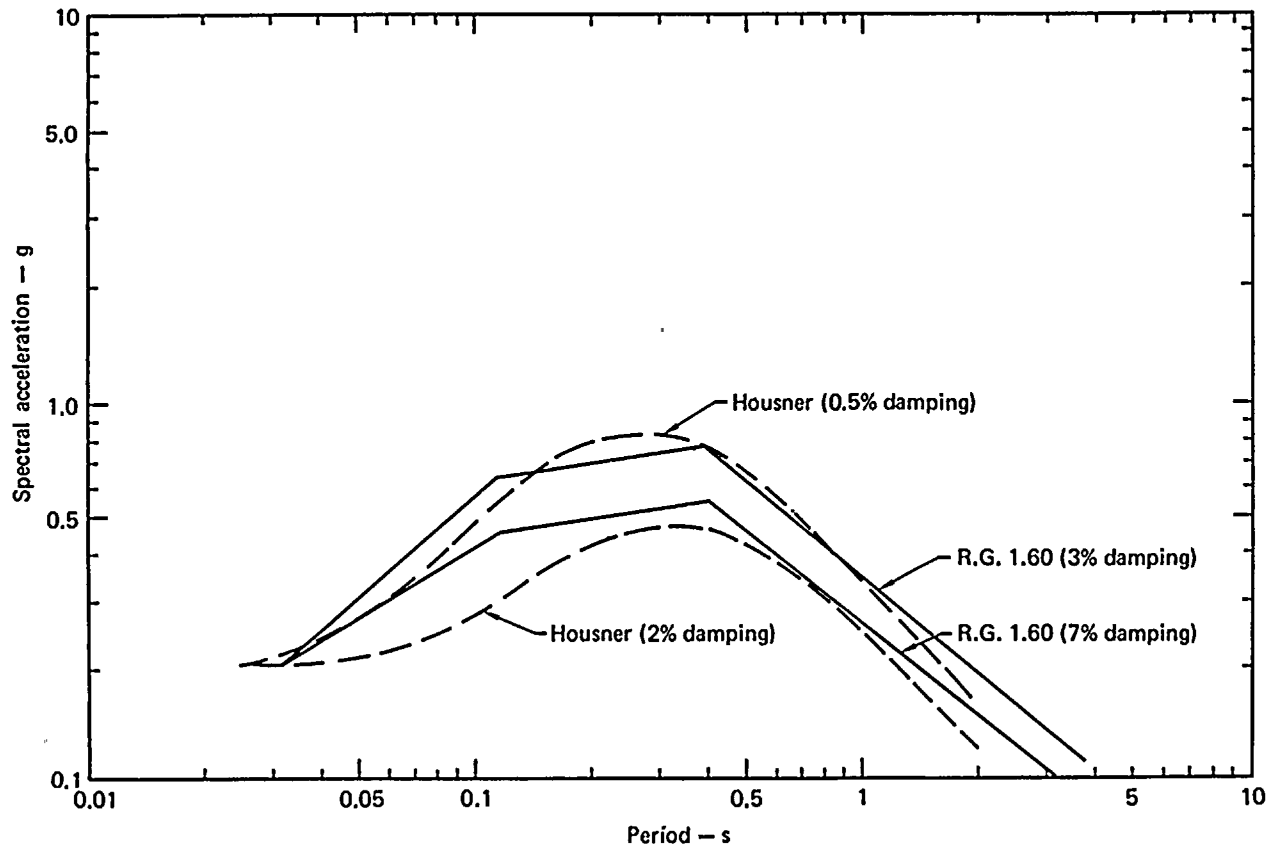

FIG. 15. Comparison of the Housner response spectrum for 2% of critical damping with the 7% R.G. 1.60 spectrum indicates the relative magnitude of the response of bolted steel structures and equipment designed to Ginna criteria to that from current criteria. Similarly, expected levels of response for base-level-mounted large piping for the two criteria can be made by comparing the 0.5% Housner spectrum and the 3% R.G. 1.60 spectrum.

4.3 SEISMIC DESIGN METHODS

Seismic analysis methods have changed greatly since the design of Ginna. The original seismic analysis was primarily by the equivalent static method based on an estimated fundamental frequency of the structure. Response spectra were used primarily to predict the peak acceleration of the fundamental mode. The check of the static design analysis of the containment building was the only analysis that involved a multi-mode system.

Current analytical techniques and computer models have increased considerably the sophistication and level of detail that can be treated. A complete dynamic analysis of complicated structural systems such as the interconnected building complex can now be done conveniently and inexpensively.

Current licensing requirements would typically require load combinations resulting from transients other than those considered when Ginna was designed. This reevaluation concentrates on the original design combinations with primary attention devoted to the seismic margins. Other current assumptions and criteria are discussed below in comparison with those used in the design and analysis of Ginna.

4.3.1 Soil-structure Interaction

Sophisticated methods of treating soil-structure interaction exist today. However, for structures that are founded on competent rock, as is Ginna, the effects of soil-structure interaction are probably relatively small. There is little radiation damping, and consideration of rock foundation compliance results in only slight increases in the periods of response of a structure when compared with the fixed-base case. We expect any variation in load that results from neglecting soil-structure interaction to be well within the accuracy of the calculations. This is especially true for the containment structure, in which the walls are attached to the foundation rock by rock anchors.

4.3.2 Combination of Earthquake Directional Components

The design of Ginna structures involved the combination of a vertical and horizontal load, usually on an absolute basis. Current recommended practice is to combine the responses for the three principal simultaneous earthquake directions by the square root of the sum of the squares (SRSS) as described in R.G. 1.92 (Ref. 16). There is only a small difference between the two combination methods for circular plant structures like the containment building, which is the only structure for which a dynamic analysis was originally performed.

4.3.3 Combinations of Earthquake and Other Loads

The design and analysis of Ginna used the load combinations for the containment structure shown in Sec. 3.5.1. Load combinations are now specified in applicable design codes and standards such as ASME Sec. III, Div. 2, and ACI-349 (Refs. 17 and 18). These codes, which describe the load combination procedures and cases to be considered, tend to be system dependent. The NRC has endorsed these load combinations with some exceptions as noted in Sec. 3.8 of the Standard Review Plan.[3]

Because stresses resulting from load cases and combinations of loads from these more recent criteria are not available, the reevaluation of the containment building concentrates on the effects of variations of seismic criteria on the stresses developed for the original design load combinations. In the other cases, for which no original seismic analysis results are available, conservative estimates of stresses from other loads are made.

4.4 ANALYSIS OF THE INTERCONNECTED AUXILIARY, INTERMEDIATE, TURBINE, CONTROL, SERVICE AND DIESEL GENERATOR BUILDINGS

The auxiliary, intermediate, control, and diesel generator buildings are Class I structures, and the turbine and service buildings are Class III structures (see Fig. 2). In the original analysis, each Class I structure was treated independently. The seismic review team for the SEP believed that the interconnected nature of the buildings was an important feature, especially in view of the lack of detailed original seismic design information. Hence, both Class I and Class III buildings are included in the reanalysis model. Note that Gilbert Associates, Inc., developed separate models for the auxiliary and control buildings in 1979. The basic assumptions and model properties for these two buildings were adopted and incorporated into this analysis.

The auxiliary, intermediate, turbine, control, diesel generator, and service buildings form an interconnected U-shaped building complex (Fig. 16) that is mainly a steel frame structural system supported by concrete foundations or concrete basement structures. A typical steel frame is made of vertical continuous steel columns with horizontal beams and cross bracing. The connections are typically bolted. The braced frames serve as the major lateral load-resisting system. Several such steel frames connect various parts of different buildings, which makes the building complex a complicated three-dimensional structural system. The compositions and interrelationships of the buildings in the complex are described in Appendix C.

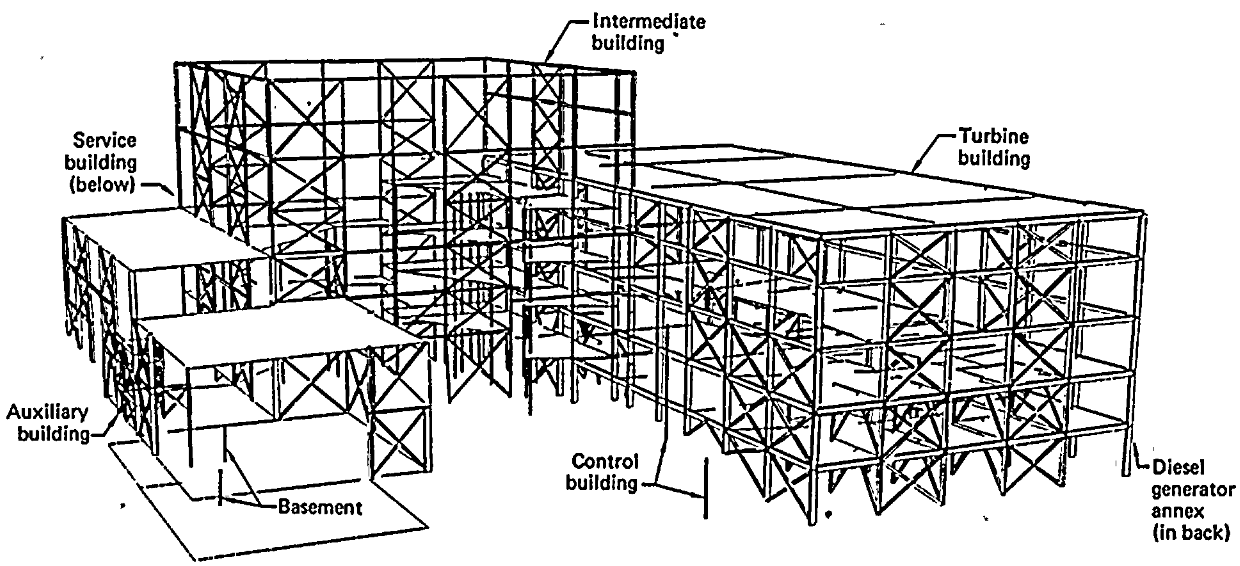

FIG. 16. Three-dimensional representation of the interconnected building complex shows details of the framing and floor and roof levels.

4.4.1 Mathematical Model

As described above, the principal lateral force-resisting systems of the interconnected building complex are the braced frames. Several such systems tie all buildings together to act as one three-dimensional structural system. It was, therefore, necessary to model these buildings in a single three-dimensional model to properly simulate interaction effects. The model was developed based on the following assumptions.

1. Rigid Foundation

All buildings except the control building are founded on solid sandstone rock and are assumed to have rigid foundations; thus, no soil-structure interaction effects are considered. The control building foundation, a concrete mat supported by soil, is modeled by six linear elastic springs.

2. Uncoupled Horizontal
   and Vertical Responses

There is no coupling betwen horizontal and vertical responses (i.e., only horizontal responses result from horizontal loadings and only vertical responses from vertical loadings). This is a reasonable assumption for this type of medium-height building that has regular frames and floors.

3. Only Horizontal Ground
   Motion in the Dynamic Analysis

For the dynamic analysis, the mathematical model is designed to have only horizontal responses because the major concern is the capacity of the lateral force-resisting system. Vertical response is calculated assuming no dynamic amplification. Because the structures were originally designed for vertical loads, such as dead and live loads, they are relatively stiff in the vertical direction and, in most cases, are not considered to have significant dynamic amplification during vertical excitation. There is no need to simulate both vertical and horizontal behavior simultaneously.

4. Rigid Floors and Roofs

All floors and roofs are assumed to be rigid in-plane because of the high stiffness for horizontal loads of the in-plane steel girders and concrete slabs. Each floor or roof has three degrees of freedom--two in horizontal translation and one in vertical (torsional) rotation. All points on a floor or roof move as a rigid body. If the motion of any point on a rigid plane is known completely (all three degrees of freedom are known), the motion of any other point on that plane can be found from a rigid-body transformation. Therefore, any point on the plane can be selected as representative. In this analysis, the center of gravity of each rigid floor or roof is selected as the representative node.

5. Lumped Masses

All structural and equipment masses are assumed to be lumped at the floor or roof elevations, then transformed to the centers of gravity of each rigid floor or roof.

6. Hinge Connections

Most bolted joints that connect bracing and beams to columns (and columns to base supports) are treated as pin or hinge connections based on reviews of pertinent drawings. The few exceptions are described in the discussion of the model for each building.

7. Buckled and Unbuckled
Bracing Systems

Cross bracing members, which are the primary elements of the lateral load-resisting system, are expected to buckle during compression cycles because of their large slenderness ratios. After a member buckles, it has zero or very small stiffness, but it regains its capacity under tension. Such nonlinear behavior was approximately accounted for by considering two linear models--a half-area model that simulates buckled bracing and a full-area model that simulates unbuckled bracing.

In the half-area model, it is assumed that both cross-bracing members have only half the actual member cross-sectional area and can take both compression and tension during earthquake excitation. In terms of the stiffness of the bracing, this approximation may be on the low side because the compression member may not buckle most of the time (i.e., when the seismic load is low), and it may still provide some stiffness even after buckling occurs . The full-area model is based on the assumption that bracings with the full cross-sectional area are effective in both compression and tension.

8. Stick Model for Concrete Wall Structures

The control building, which has concrete walls and roof that are much stiffer than the other structures, is modeled as an equivalent beam. The two-story concrete substructure in the basement of the auxiliary building is treated similarly.

9. Stiffness and Mass Effects of the Diesel Generator and Service Buildings

The one-story diesel generator building has four shear walls that have significant stiffness but minimal mass (only the roof mass needs to be considered; the other masses are on the rigid foundation). Therefore, the four shear walls are modeled as four elastic springs that have the equivalent stiffnesses of the shear walls. In contrast, the service building is a relatively flexible steel frame structure, and only its mass is included.

10. Damping

A uniform damping of 10% of critical is assumed for the whole structural system based on the suggestion of NUREG/CR-0098 for bolt-connected steel structures under SSE loading.

Details of the model and additional assumptions about individual parts of the overall model are discussed in Appendix C.

The three-dimensional mathematical model for the building complex was prepared for the computer program SAP4 (Ref. 19). All steel frames are modeled by beam elements. The model's rigid diaphragms for all roofs and floors are represented by the rigid restraint (also called the master-slave restraint) option of SAP4. In this representation, the stiffnesses of all structural members connected to the floor or roof are mathematically transformed to a master node, which we selected to be the floor or roof center of gravity. Such a stiffness transformation, which requires no additional members or computational effort, is mathematically equivalent to the more common approach of placing infinitely rigid beams between the master node and the corresponding slave nodes of the structural members. There are 17 such rigid diaphragms in the model that are treated this way. Use of the master-slave option together with the rigid-floor assumption significantly reduces the number of degrees of freedom in the mathematical model without sacrificing its completeness.

The two-story concrete substructure of the auxiliary building and the control building are modeled by equivalent beams. The four shear walls of the diesel generator building are represented by four elastic springs attached to the north frame of the turbine building at the diesel generator building roof. The masses of the service building roof are lumped to the turbine and intermediate buildings. All other masses are lumped to the centers of gravity of floors or roofs.

The complete model has 686 nodal points, 44 dynamic degrees of freedom, 1213 beam elements, and 10 elastic springs. Further details of the model are described in Appendix C.

This model may appear to be far too complex for the level of reassessment of the SEP. However, this level of detail was needed in view of the lack of original seismic design information, especially if the model was to be complete enough to capture the effects resulting from the complex interconnections and torsion. Reduction techniques allowed us to limit the computational complexity. Thus, we had a model that captured all the important effects plus a relatively economical computational procedure.

4.4.2 Method of Analysis

Figure 17 is a flow chart of the analytical procedure. The total global stiffness of the structural system is obtained by assembling the stiffnesses of all members. The total stiffness matrix has 1624 static degrees of freedom. The lumped-mass matrix is similarly obtained; however, only 44 dynamic degrees of freedom were identified as having nonzero mass.

The frequencies and mode shapes of the structural system were obtained by the subspace iteration method provided in SAP4. Since there are only 44 nonzero-mass dynamic degrees of freedom, the structural system has only 44 independent modes. By requesting solutions for all 44 modes, the subspace iteration method reduces to the standard Guyan reduction, and the iteration process converges in the first step. The frequencies and the ten largest modal participation factors are listed in Table 5. Representative mode shapes are shown in Figs. 18 through 21.

After the frequencies and mode shapes were obtained, the structural responses were computed by the response spectrum method. The seismic input was defined by the horizontal spectral curve of the SSE specified in R.G. 1.60 for 10% structural damping and 0.2 g peak ground acceleration.

Two structural models were analyzed, one with half the bracing area (half-area model), one with the full bracing area (full-area model). For each model, two analyses were performed, one with the input excitation in the N-S direction, the other in the E-W direction. In each analysis, 44 response modes were included, and for each direction the modal responses were combined by the square-root-of-the-sum-of-the-squares (SRSS) method. Responses to N-S and E-W excitations were also combined by the SRSS method. Vertical responses were obtained by taking 13% (0.2 g x 2/3) of the dead load responses.

4.4.3 In-Structure Spectra

A direct method was applied to generate seismic input spectra for equipment at various locations in the structure.[20,21] This method treats earthquake input motions and the response motions as random processes. The response spectrum at any location in the structure can be derived from the frequency response function of an oscillator, the frequency response function

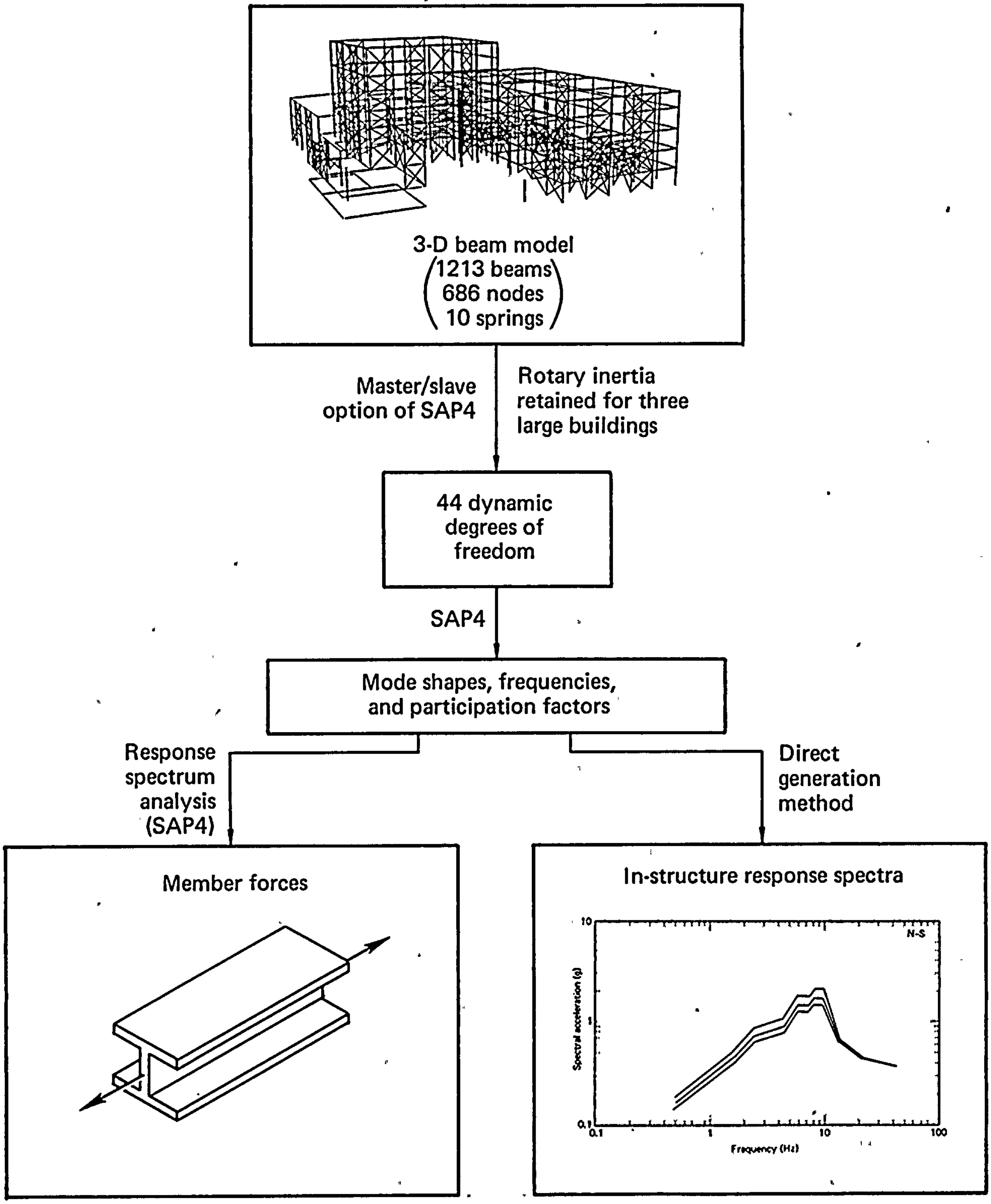

FIG. 17. Flow chart of the analysis of the interconnected building complex.

TABLE 5. Modal frequencies of the interconnected building model. Note: Numbers in parantheses are the ten largest modal participation factors in the E-W and N-S directions, respectively.

Mode no.	Frequency, Hz Half-area model	Full-area model
1	1.8 (3.4, 12.9)	2.3 (7.4, 12.6)
2	2.0 (10.2, 0.2)	2.4 (8.5, 4.7)
3	2.1	2.8
4	2.4	3.1
5	2.6	3.2 (7.4, 0.6)
6	2.8	3.4
7	2.9	3.4
8	3.3	3.6
9	3.4	3.9
10	3.6	4.0 (6.3, 1.4)
11	4.0	4.3
12	4.2	4.3
13	4.2	4.6
14	4.4	4.6
15	4.7	5.4
16	5.6	6.7
17	6.1	6.9 (12.7, 6.4)
18	6.5 (6.4, 4.5)	7.0
19	6.6	7.3
20	6.7 (8.4, 8.5)	7.4
21	6.9 (10.3, 7.2)	7.5
22	7.0	8.0
23	7.8	9.7 (5.1, 8.3)
24	9.3	10.4
25	9.5 (5.4, 8.4)	10.6
26	10.4	10.9
27	10.8	11.1
28	11.1	11.7
29	11.2	12.1
30	12.2	12.8
31	13.5	14.0
32	13.8	16.4
33	16.4	16.7
34	17.8 (2.4, 6.6)	17.8 (2.3, 6.5)
35	18.5	18.6
36	19.3	19.5
37	21.1 (0.1, 27.1)	21.2 (0.1, 27.1)
38	22.9 (26.9, 0.1)	22.9 (26.9, 0.1)
39	27.0	27.2
40	33.5	33.6
41	41.2	41.2
42	45.1	45.7
43	57.8	57.8
44	60.4 (6.7, 0.0)	60.4 (6.7, 0.0)

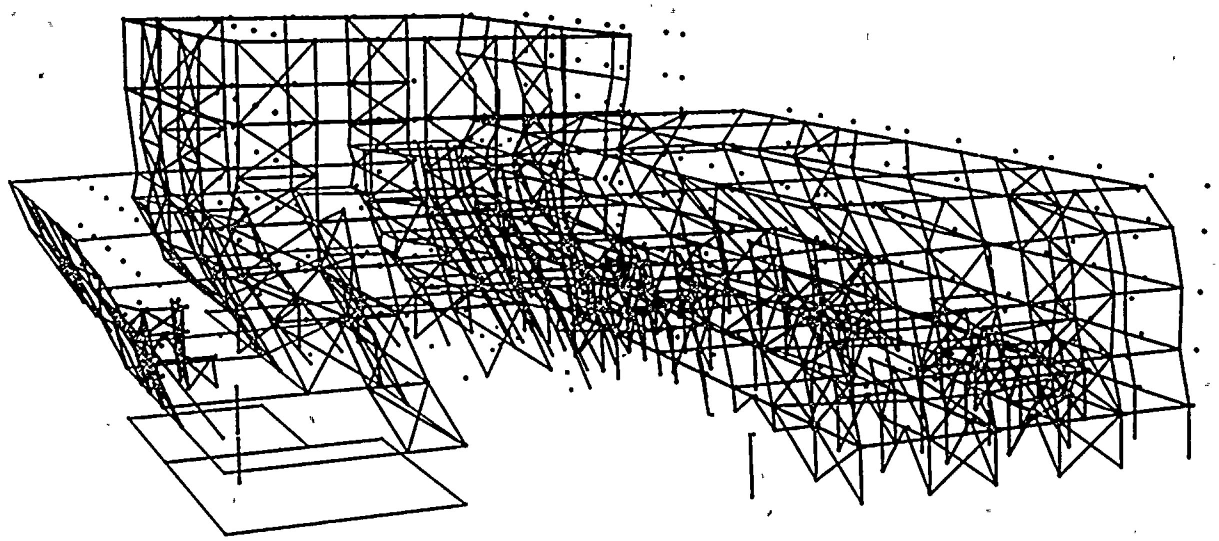

FIG. 18. The shape of Mode No. 1 (half-area model, frequency = 1.8 Hz). Note that distortion from the original (dotted shape) is exaggerated for clarity.

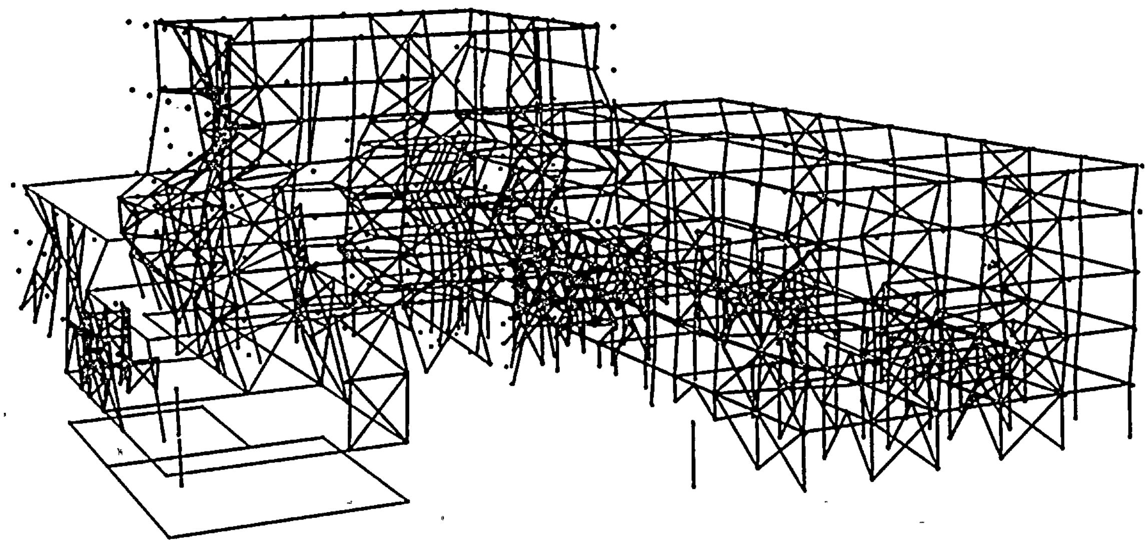

FIG. 19. The shape of Mode No. 21 (half-area model, frequency = 6.9 Hz). Note that distortion from the original (dotted shape) is exaggerated for clarity.

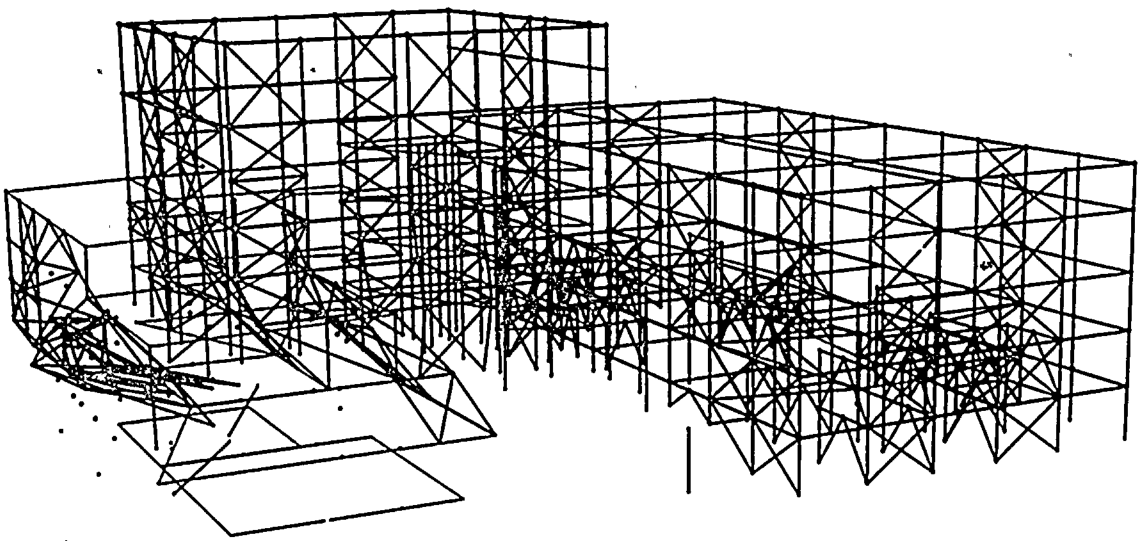

FIG. 20. The shape of Mode No. 37 (half-area model, frequency = 21.1 Hz). Note that distortion from the original (dotted shape) is exaggerated for clarity.

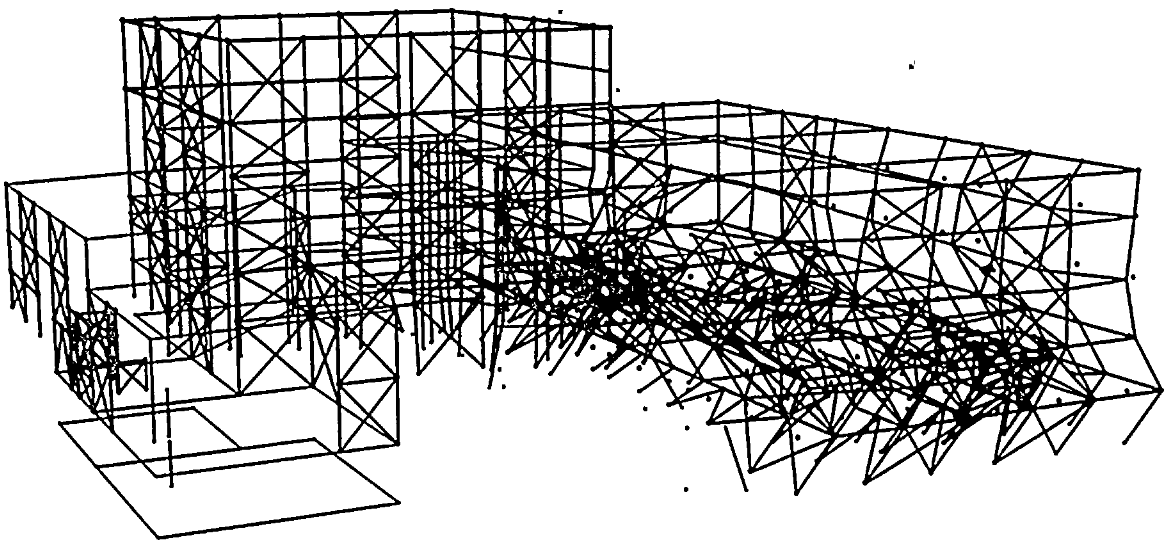

FIG. 21. The shape of Mode No. 39 (half-area model, frequency = 27 Hz). Note that distortion from the original (dotted shape) is exaggerated for clarity.

of the structure at that location, and the input ground response spectrum. This method avoids the troublesome task in the time-history approach of selecting the proper corresponding time-history input for the specified response spectrum. Other direct methods, such as those suggested in Refs. 22 and 23, can also generate in-structure spectra from the input ground spectrum and the structural modal properties, but they require some semi-empirical formulas for dynamic amplification factors.

The in-structure spectra generated from the half-area and full-area models were enveloped to give the final spectra (Fig. 22). If peaks were still obvious at structural frequencies, spectrum-widening techniques in accordance with current practice were then applied to ensure $\pm 15\%$ broadening to account for modeling and material uncertainties. The spectra generated in

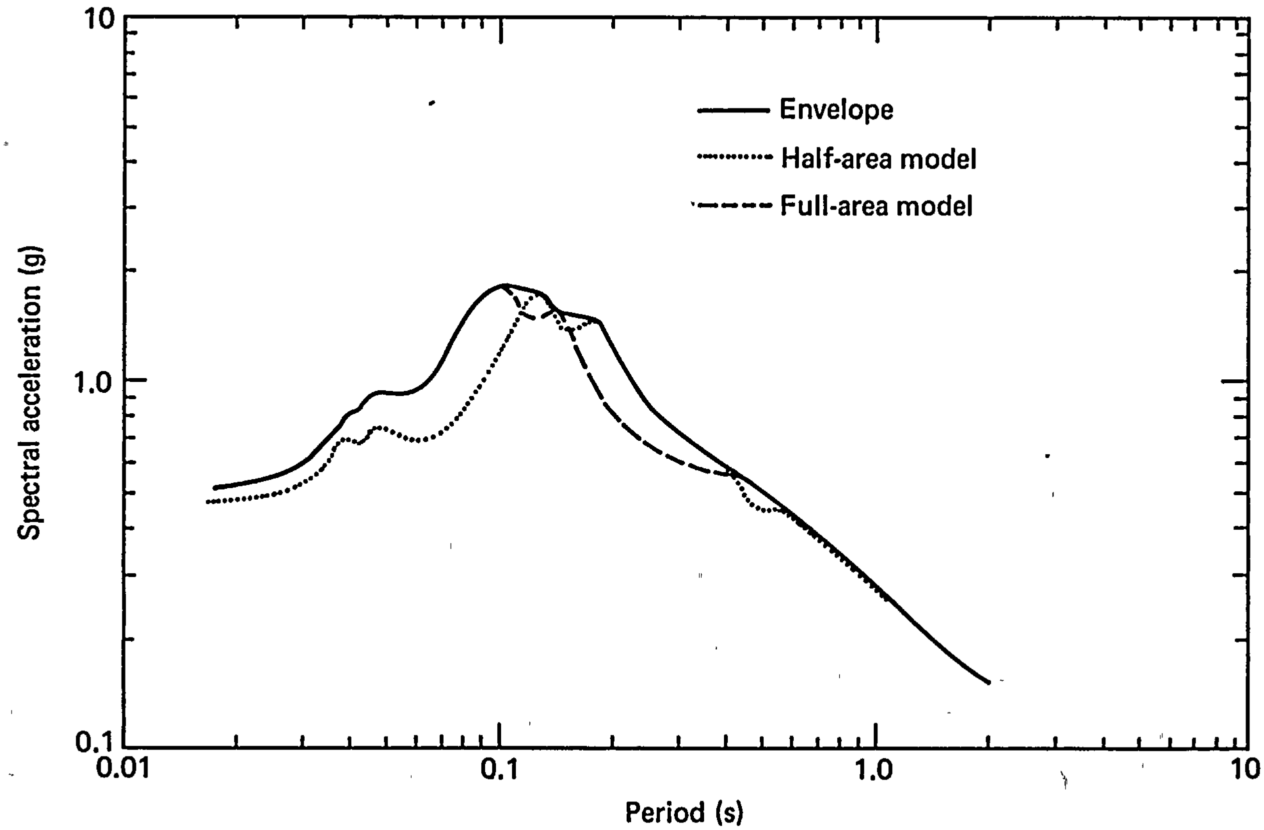

FIG. 22. In-structure response spectra for the interconnected building complex were generated by a direct method for two models that bracket the behavior of the braced frames. A half-area model simulates buckled bracing, and a full-area model simulates unbuckled bracing. Spectra for the two models were enveloped to produce the recommended spectrum for equipment reevaluation.

this way were checked to some extent against spectra generated by Gilbert Associates, Inc., for their piping analysis.

4.4.4 Analysis Results

The frequencies and the ten largest modal participation factors for all 44 modes of the full-area and half-area models are listed in Table 5. The first mode is a steel structure mode involving the braced frames of all buildings. The 18th mode of the half-area model (6.50 Hz) and the 17th mode of the full-area model (6.92 Hz) involve primarily the intermediate building and partially the turbine building. The 34th mode of both models is a north facade and south turbine building wall mode. The 37th mode (Fig. 20) involves the auxiliary building floors. The 39th mode (Fig. 21) is mainly a turbine-control building mode.

In comparing the frequencies and modal participation factors, we found, as expected, that modes with low frequencies are those dominated by steel parts of the structural system (i.e., the framing system) and that high-frequency modes are dominated by the concrete structures (i.e., the control building and the basement structures of the auxiliary building). Also, as expected, the difference between the half- and full-area models is apparent only at low frequencies or for steel structure modes. High-frequency modes are almost identical for both models.

Several high-frequency modes have significant modal particpation factors. In fact, the modes having the highest factors in the N-S and E-W directions are the 37th and 38th modes, respectively (see Table 5). Inclusion of the high-frequency modes is therefore necessary, especially in computing the in-structure response spectra.

Comparisons of member forces between the two models show that bracing forces are generally lower in the half-area model than those in the full-area model, but the reverse is true for column forces.

Euler's buckling loads for each bracing member based on the full length and hinged ends condition were calculated and compared with the computed peak member forces. Most bracing members, especially those at low elevations, have peak forces higher than Euler loads, which suggests that buckling may commonly occur for most bracing members during an SSE. Note also that bracing members are generally designed to buckle under compression. Therefore, the comparison with Euler loads is not to determine buckling capacities but rather to provide some information regarding the behavior of the braced frame during an SSE.

To evaluate the capacities of bracing members, the peak member forces are compared with the yield loads (yield stress = 36 ksi). The peak force for a cross bracing member equals the absolute sum of the computed tensile and compressive member forces (or roughly twice the member force). This is because the member is assumed to take both tension and compression. In reality, once one cross bracing member buckles, the other member must take almost the full lateral load.

The ratios of the peak forces to the yield forces are computed for all bracing members. A member is considered to be loaded beyond yielding if the ratio is greater than 1.

For the auxiliary building, the cross braces at the N-E corner of the operating floor have the highest yield load ratio, about 1.3 (see Fig. 7). The difference between the half- and full-area models is small in this case. The highest yield load ratio in the east facade structure (Fig. 6) is at the bottom brace between column lines J and H (0.97 for the half-area model and 1.23 for the full-area model). In the west facade structure, the highest ratio is in the cross brace above floor El. 271 ft and between column lines K and M (1.1 for the half-area model and 1.7 for the full-area model).

In the turbine building, the cross brace at the south wall between column lines 10 and 11 and above El. 307 ft has the highest ratio (1.6 for the half-area model and 2.2 for the full-area model). This high yield load ratio, however, is not typical; the other turbine building bracing members have ratios below 1.3. This anomalous cross brace is close to the control building and its relatively stiff superwall, which could account for the lateral load concentration in this part of the braced frame.

The member forces and stresses in the horizontal beams are low and all well below yield levels.

To evaluate the stresses in columns, dead load stresses must be included. The same model was used, and all vertical degrees of freedom were released. In this case, the stresses in beams and bracings are small, and differences between the half- and full-area models are negligible. To include the effect of the vertical SSE component, the dead load responses scaled to 0.13 g are used, and the SRSS combination of the responses to the three earthquake components are then added to the dead load responses. Maximum column stresses calculated from the total responses are compared to the yield stress (36 ksi). Most columns have maximum stresses below the yield stress. Only column line H of the west facade structure, interior column H3e of the intermediate building, and column lines 8 and 8a of the south turbine wall have stresses exceeding the yield level. The highest maximum stress to yield stress ratio (about 2.0) is at the west facade column. However, for all column sections in which the maximum stress in the section exceeds the yield stress, the bending moments are much less than the ultimate section moment capacities (or plastic hinge moment).

The stresses in the concrete structure of the auxiliary building are low--the maximum shear stress is less than 50 psi. The stresses in the concrete control building are higher; for example, the maximum shear stress due to lateral shear force is 170 psi, and the maximum shear due to torsion is 20 psi. The maximum shear stress induced in the concrete shear walls of the diesel generator building is about 120 psi. The peak concrete stresses cited above are computed from the maximum responses of the half- and full-area models.

The maximum shear stress in the steel pressurization walls is about 2 ksi.

4.5 ANALYSIS OF THE CONTAINMENT BUILDING AND ITS INTERIOR STRUCTURES

The containment building is surrounded by the auxiliary, intermediate, and turbine buildings (Fig. 6). Since there are no structural connections between the containment building and the other buildings, the containment building and its interior structures were modeled and analyzed independently.

The containment building is a vertical right cylinder with a flat base and a hemispherical dome (Fig. 5). The building is 99 ft high to the spring line of the dome and has an inside diameter of 105 ft. The cylindrical concrete wall, which is prestressed vertically and reinforced circumferentially with mild steel deformed bars, is 3.5-ft thick. The concrete dome is a reinforced concrete shell 2.5-ft thick. The base is a 2-ft-thick reinforced concrete slab. The containment cylinder is founded on rock by means of post-tensioned rock anchors. A welded steel liner attached to the inside face of the concrete shell is 3/8-in. thick in the cylinder and dome and 1/4-in. thick in the base. An additional 2 ft of concrete fill covers the bottom liner plate.

The basement floor of the containment building is at El. 235.66 ft, while the surrounding ground surface is at about El. 270 ft. The design provided for no backfill against the containment wall (Ref. 8, Sec. 5.1.2).

The containment interior structures include the concrete reactor vessel support, concrete floors (at Els. 245, 253.25, and 278.33 ft), concrete shield walls, the steel overhead crane support structures, the NSSS, and other auxiliary equipment (see Fig. 5). The only connection between the containment building and its interior structures (other than the common basement floor) is at the top of the crane rail, where the rail top may bear on the concrete shell at four locations of neoprene pads.

4.5.1 Mathematical Models

Two separate mathematical models were used in the reassessment analyses. The first, a model for the containment shell only, is similiar to the fixed-base cantilever beam model with 12 lumped masses shown in Fig. 10. Mass and section properties are uniform up to El. 232.66 ft. The remaining shell wall and the dome are modeled by four equivalent beam elements, each with a different uniform section.

The second model includes the interior structures, the NSSS, and the crane structure and is based on a model developed for the utility by Gilbert Associates, Inc., in 1979.[24] The following assumptions were made in modeling the containment building and its interior structures:

- The containment has a rigid foundation at the basement floor (El. 235.66 ft) and has no lateral support from the surrounding soil above that elevation.
- Since the concrete containment shell is much stiffer than the steel crane structure, the constraints from the crane structure can be neglected in modeling the containment shell. However, the model for the interior structures and crane supports has to include the constraint effect from the containment shell at the crane top.
- The interior structures are assumed to have rigid diaphragms at Els. 245, 253.25, 267.25 and 278.33 ft. Masses of all concrete floors and walls are lumped to the centers of gravity of the diaphragms. Major NSSS equipment items--including steam generators, coolant pumps, and the reactor vessel--are modeled as lumped-mass systems.

- The crane structure is assumed to have two lumped masses located at the center of the crane structure at Els. 329.66 and 311 ft.
- Based on the recommendation in NUREG/CR-0098, damping is assumed to be 7% of critical damping for the steel-and-prestressed-concrete part of the structures and 10% for the concrete part.

The interior structures model, which was prepared for the computer program STARDYNE, includes plate elements for the concrete shield walls and rigid beams for the rigid floors (Fig. 23). The concrete-and-steel columns are represented by elastic beam elements. The NSSS and the neoprene pads at the crane top are included as equivalent stiffness matrices. A cantilever beam model that has 7 lumped masses represents the containment shell. The total mass of each floor is lumped to the center of gravity of the floor, and rotational inertia is acounted for. Equipment masses are represented by lumped masses at the corresponding nodes. There are 99 nonzero-mass degrees of freedom in the model. Use of the Guyan reduction technique reduced the 99 to 45--those associated with the interior structure floor centers of gravity and containment shell nodes.

4.5.2 Method of Analysis

We analyzed both models (Figs. 10 and 23) by the response spectrum method in the horizontal and vertical directions. The spectral curves of R.G. 1.60 were scaled to 0.2 g peak acceleration for the horizontal component and 0.13 g for the vertical component and input as the base excitations. Modal responses and responses to horizontal and vertical excitations were both combined by the SRSS method.

A time-history method was used to generate in-structure response spectra for the interior structures. Only horizontal excitations were included in the analysis. The input base excitation was a synthetic time-history acceleration record for which the corresponding response spectra were compatible with the 0.2 g R.G. 1.60 spectra. Response spectra associated with two orthogonal horizontal base excitations were generated independently at equipment locations and then combined by the SRSS method. Peaks of the spectra were broadened ±15% in accordance with current practice.

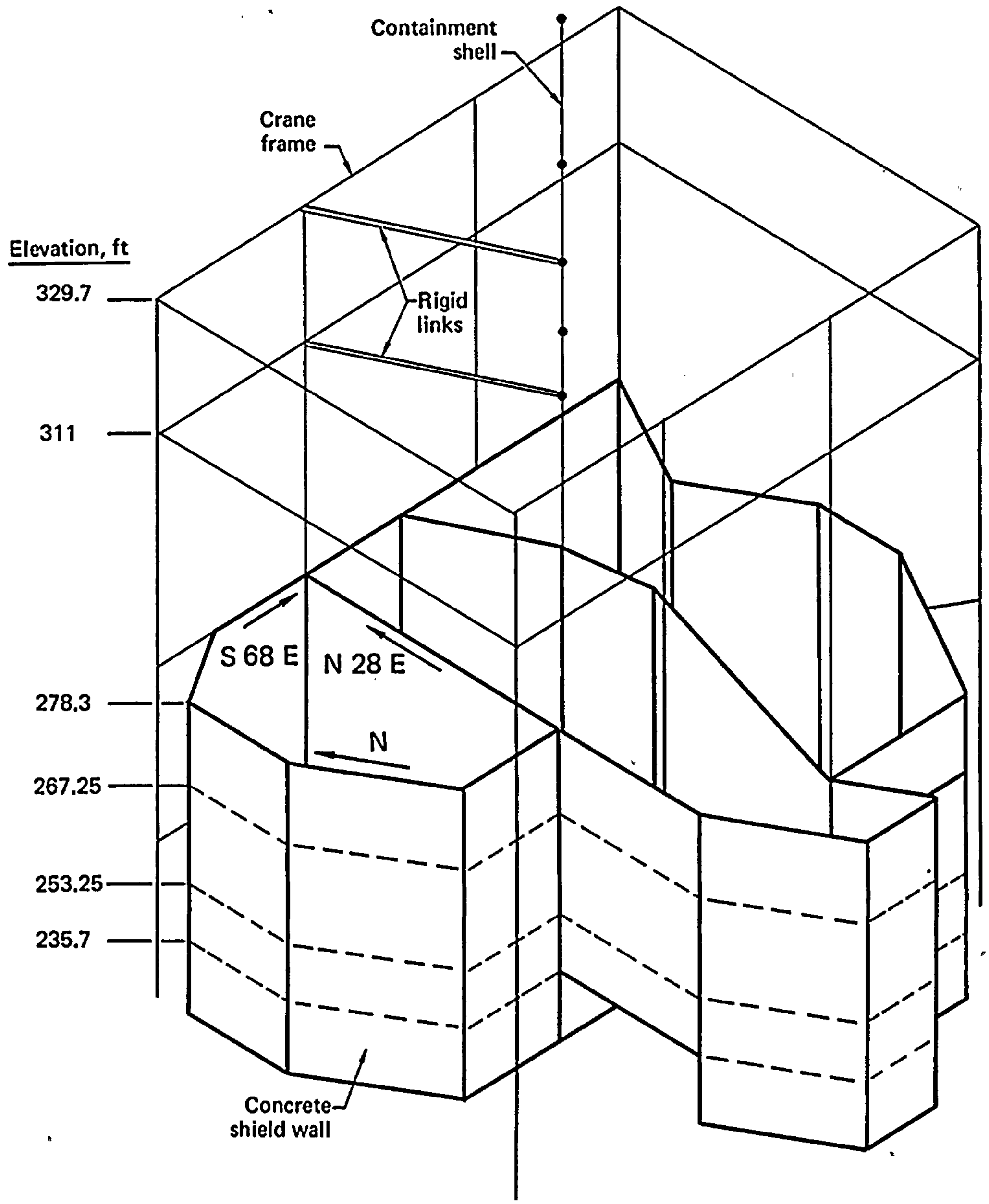

FIG. 23. The interior structures model, which was prepared for the computer program STARDYNE, includes plate elements for the concrete shield walls, rigid beams for the rigid floors, and elastic beam elements for the concrete-and-steel columns. A cantilever beam model that has 7 lumped masses represents the containment shell. There are 99 nonzero-mass degrees of freedom in the model.

4.5.3 Results

The containment shell model analysis included the first ten modes of the model. The natural frequencies ranged from 6.97 Hz for the first mode to 92.38 Hz for the tenth mode. The calculated earthquake forces are shown in Fig. 24 together with the original design forces (Fig. 9) and the original modal analysis results. Note that the reanalysis (R.G. 1.60, 7% damping) gives higher forces than those from the original modal analysis (Housner, 2% damping). However, the reassessment results are still lower than the original design seismic forces, which were based on the equivalent static analysis.

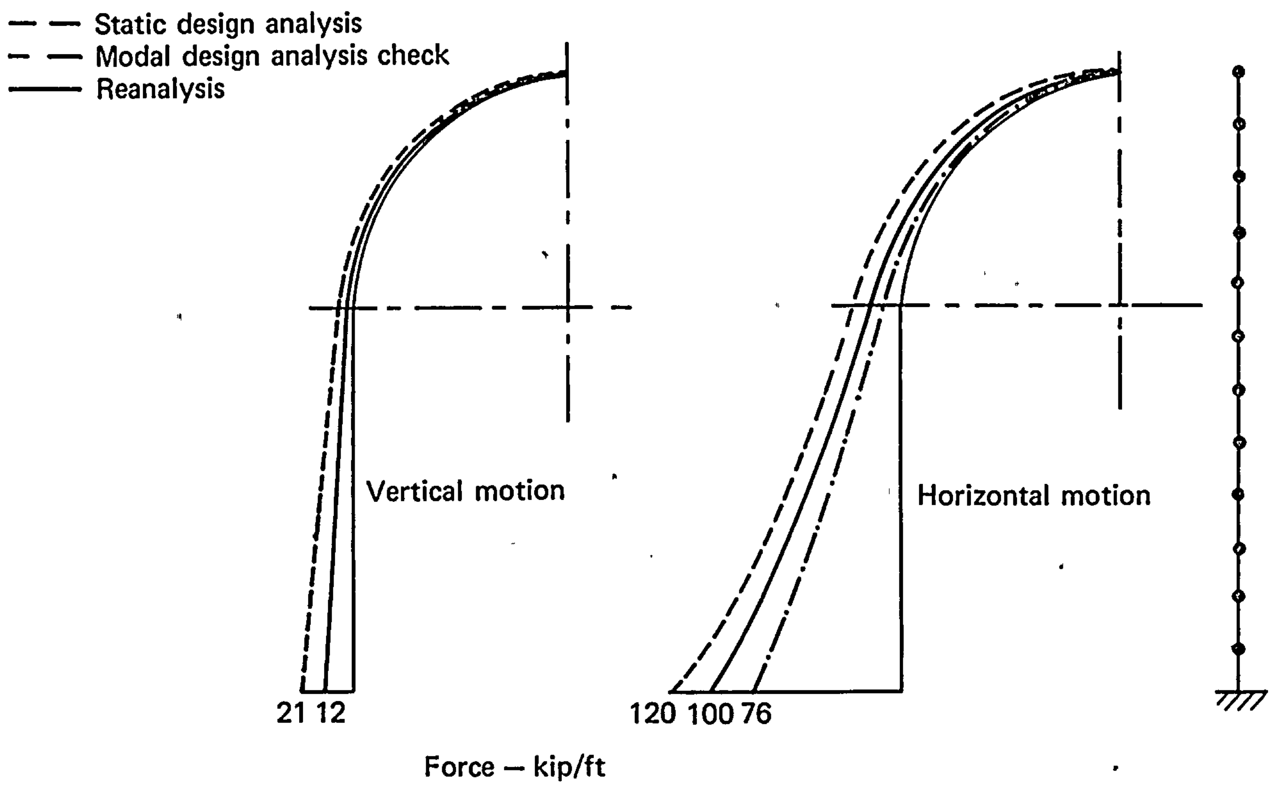

FIG. 24. Calculated earthquake forces from the containment shell modal reanalysis (R.G. 1.60, 7% damping) are higher than those from the original modal analysis (Housner, 2% damping) but lower than the original design seismic forces, which were based on the equivalent static analysis.

Twenty structural modes with frequencies ranging from 5.57 to 87.48 Hz were included in the analysis of the interior structures model. The calculated member stresses of the concrete shield wall were low--peak normal stress was less than 50 psi, and maximum shear stress was less than 40 psi. The maximum stress is less than 70 psi in the concrete columns and less than 600 psi in the steel columns. The stresses in the steel crane supporting columns are higher, but the maximum stress is still less than 4 ksi.

4.6 EVALUATION OF CRITICAL STRUCTURES

The seismic capability of critical structures was evaluated using loads developed in the reanalysis. A structure was generally judged to be adequate without the need for additional evaluation for the following two cases:

- Where loads resulting from the reanalysis were less than those used in the original design.

- Where loads resulting from the reanalysis exceeded the original loads (or where there was insufficient information about the original seismic analysis for a comparison) but the resulting stresses were low compared to the yield stress of steel or the compressive strength of concrete.

For cases in which the seismic loads from the reanalysis were not low and exceeded the steel yield stress, or the concrete compressive strength, conclusions were reached on the basis of the estimated reserve capacity (or ductility) of the structures; that is, the capability of structures to deform inelastically without failure.

4.6.1 Containment Building

The containment building is the only structure with enough information about the original seismic design and analysis to make a comparison. The original analysis was an equivalent static analysis, which was checked by a response spectrum analysis using Housner spectra. The seismic design loads were based on the equivalent static analysis. The reanalysis gave seismic loads higher than those of the original Housner response spectrum analysis, but lower than the seismic design loads from the equivalent static analysis (Fig. 24). The containment building can, therefore, be considered to be acceptable in light of current criteria, if the structure meets the original design criteria.

4.6.2 Containment Interior Structures

Results from the reanalysis show that the estimated seismic stresses of interior structures--including concrete shield walls, steel and concrete columns, and crane support structures--are low. No further evaluation is necessary.

4.6.3 Auxiliary Building

Based on the stresses calculated in the reanalysis, the concrete structure apparently has adequate load margins to withstand seismic loads. However, the braced steel frames of the superstructure are more critical. The bracings in the E-W direction have stresses below yield, but the N-S bracings are near or exceed yield. The bracing at the NE corner of the low roof has a safety factor (defined as fy/f) of about 0.8. Alone this may be considered marginal, but this bracing is one of only two lateral load-resisting systems for the auxiliary building superstructure in the N-S direction. The other one is the bracing between the high and low roofs, and its stress is close to yield. Therefore, it may be necessary to increase the lateral load-resisting capacity of the superstructures.

4.6.4 Intermediate Building and Facade Structures

The braced frames in the low portion of the east and west facades are the relatively weak areas of the intermediate building and facade structures. The stresses in the cross bracings are at or a little over yield (safety factor of 0.9). The lateral load-resisting systems have more reserve capacity than do the braced steel frames of the auxiliary building discussed above. The vertical columns of the floors and nonstructural members, such as stairway structures between floors and sidings, provide additional lateral support to the structure.

As discussed in Appendix C, a special characteristic of the west facade is that the horizontal floor or roof girders are connected not to the bracing joints but somewhere between joints. The reanalysis indicates that the columns supporting intermediate floors may yield locally at locations where floors at different elevations meet at mid-points between joints. However, those columns still have sufficient moment-resisting capacity, and the column systems can be considered marginally acceptable.

4.6.5 Turbine Building

The lateral load-resisting system for turbine building floors has stresses below yield. The cross bracings above the operating floor in the south, north, and west walls have stresses that exceed yield. The bracings right above the superwalls have the lowest safety factor (0.7).

The bracings above the superwalls sustain high loads because of the relatively higher stiffnesses of the superwalls and the control building compared to the turbine building frames. Strengthening those bracings or adding bracings to adjacent frames to spread the loads can improve lateral load-resisting capacity. Such changes would also help that part of the turbine frame above the operating floor where redundant resistance to lateral loads is low.

4.6.6 Control Building

Excluding stress concentration effects, the maximum shear stress in the reinforced concrete walls of the control building is approximately 200 psi. Because the walls have No. 5 reinforcing steel bars (5/8-in. diameter) at 12-in. spacing (in both horizontal and vertical directions), the structure is considered to be adequate for resisting shear.

Because the type of floor connections between the control and turbine buildings eliminates any possibility of overturning or bending of the control building by itself, such possibilities were not considered.

4.7 SEISMIC INPUT MOTION FOR EQUIPMENT

Seismic input motion for equipment is typically defined by means of in-structure (or floor) response spectra for items that have relatively small mass. Floor response spectra can be generated either by means of time-history analyses as discussed in Sec. 4.5.2 or by a direct method as discussed in Sec. 4.4.3. The spectra are normally smoothed and the peaks broadened to account for modeling and material uncertainties.

For the purpose of equipment evaluation (Chapter 5), we generated in-structure response spectra for both the interconnected building complex and the containment building. In both cases, in-structure spectral curves were smoothed, and the peaks were widened ±15% in accordance with current practice. As described in Sec. 4.4.3, two mathematical models of the interconnected building complex were analyzed to bracket the behavior of the braced frames--a half-area model that simulates buckled bracing and a full-area model that simulates unbuckled bracing. Envelopes of spectra generated from the two models by the direct method were used for reanalysis of equipment. In-structure response spectra for the containment interior structures were generated from time-history analyses of the mathematical model (Sec 4.5.2).

Response spectra were generated at the equipment locations and floor centers of gravity indicated in Table 6 and shown in Fig. 25 . At each location, two orthogonal horizontal spectral components were computed at three different equipment damping ratios--3%, 5% and 7%. Since the vertical dynamic amplification was judged to be negligible, all vertical floor spectra were considered to be the same as the ground input spectra with 0.13 g peak acceleration.

The in-structure response spectra generated for equipment review are shown in Figs. 26 through 42. Note that horizontal in-structure spectra of the containment interior structure are oriented in the directions of S62E and N28E shown in Fig. 23. Spectra outside the containment building are in the NS and EW directions.

TABLE 6. Equipment items and locations where in-structure spectra were generated.

Building	Equipment	Elevation, ft
Containment interior structures	Pressurizer PR-1	253
	Control rod drive	253 and 278
	Steam generator SG-1A	250 and 278
	Steam generator SG-1B	250 and 278
	Coolant pump RP-1A	247
	Coolant pump RP-1B	247
Auxiliary building	Platform center of gravity (c.g.)	281.5
	Heat exchanger (35)	281.5
	Surge tank (34)	281.5
	Boric acid tank (40 B)	271
	Operating floor c.g.	271
Control building	Basement floor c.g.	250
	Relay room floor c.g.	269.75
	Control room floor c.g.	289.75

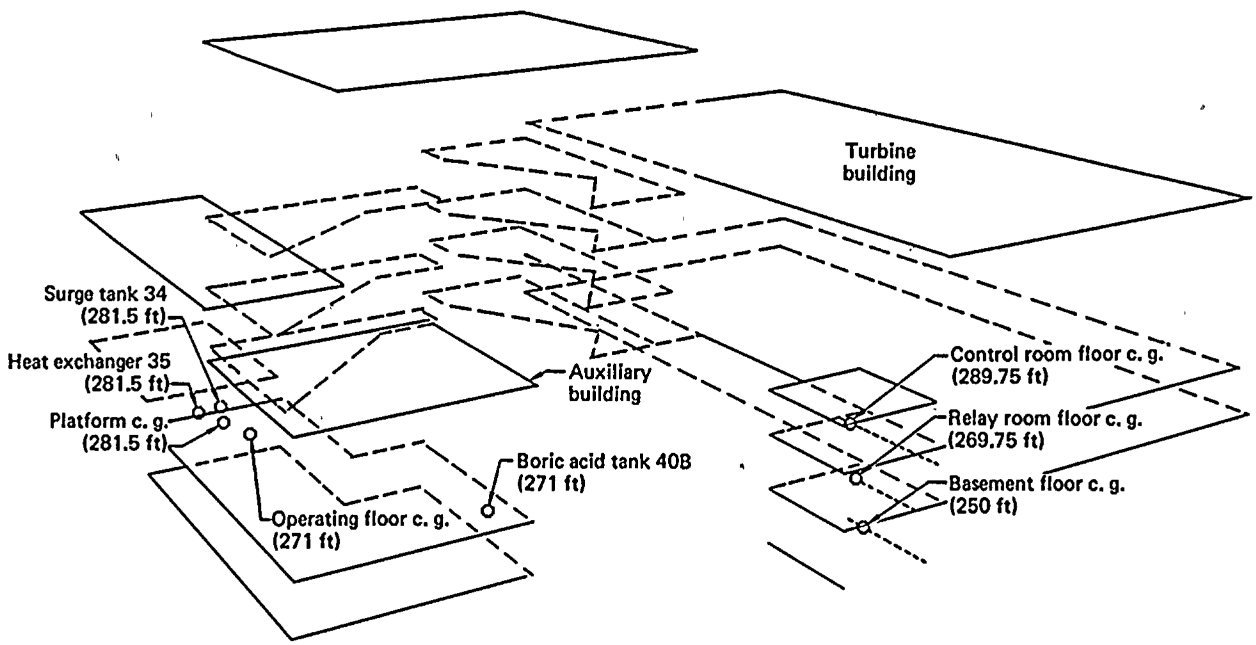

FIG. 25. Floor locations within the interconnected building complex at which in-structure response spectra were generated.

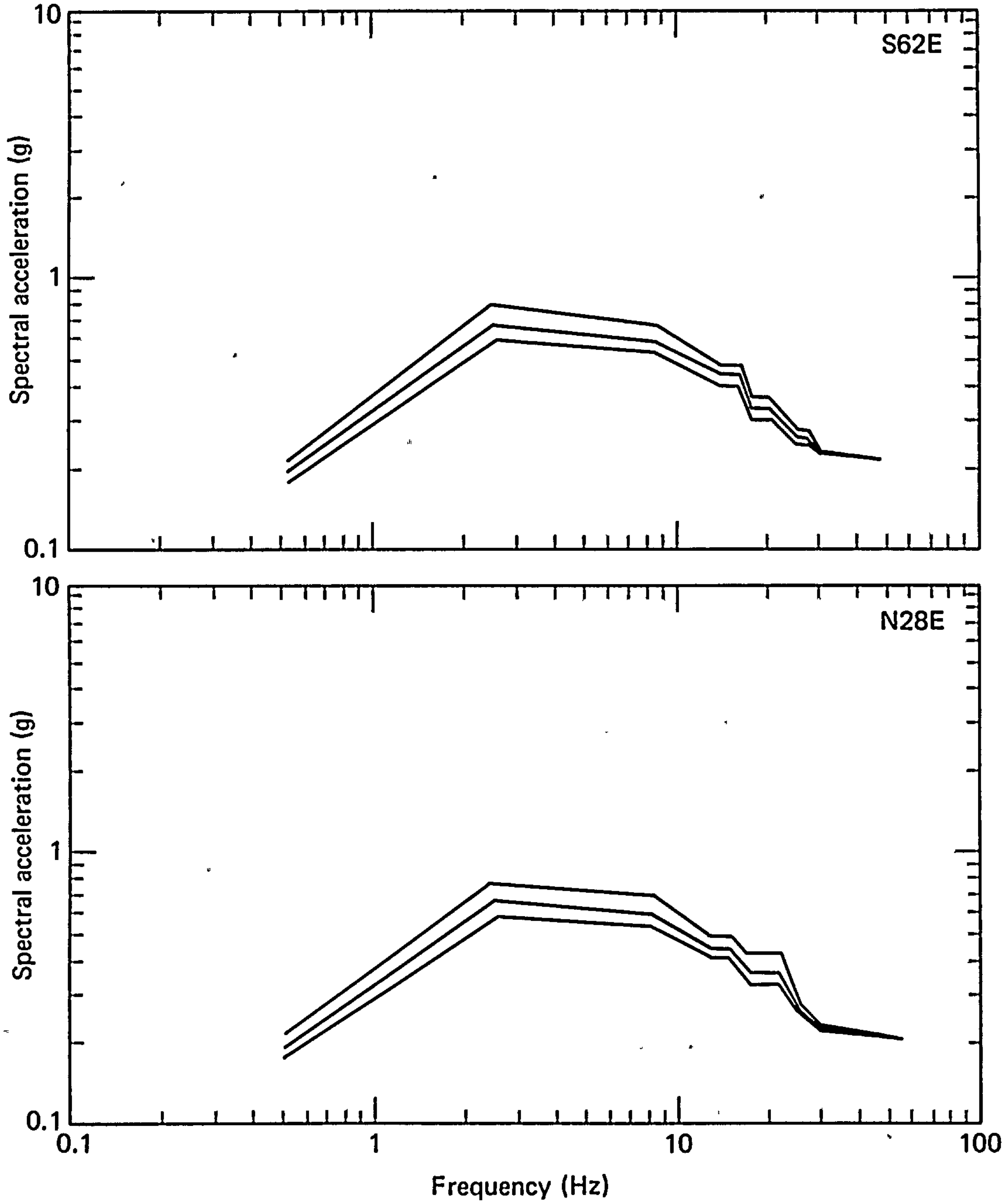

FIG. 26. In-structure response spectra for use in the reevaluation of pressurizer PR-1 (containment building El. 253) were generated for 3, 5 and 7% of critical damping by a time-history method.

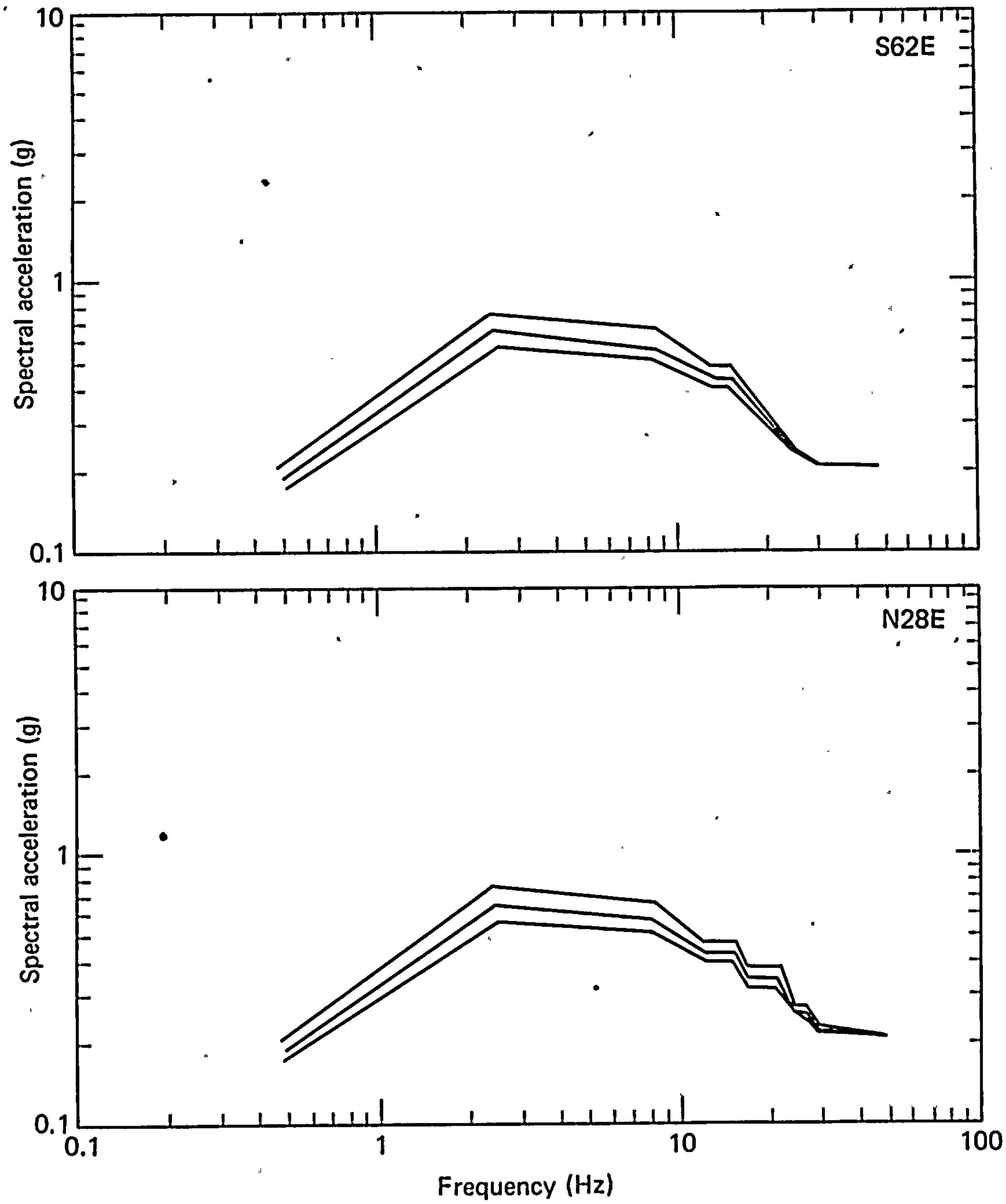

FIG. 27. In-structure response spectra for use in the reevaluation of the control rod drive (containment building El. 253) were generated for 3, 5 and 7% of critical damping by a time-history method.

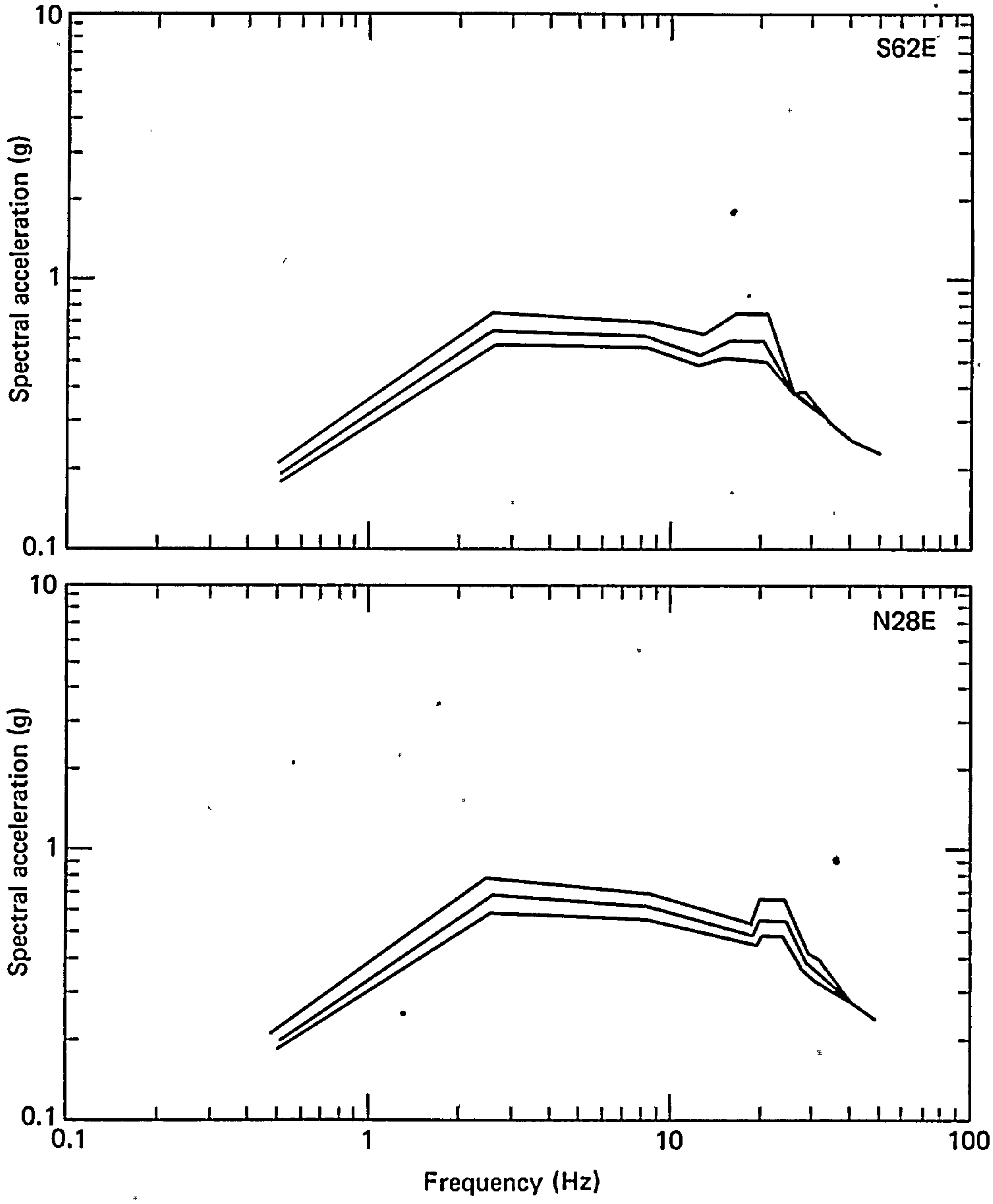

FIG. 28. In-structure response spectra for use in the reevaluation of the control rod drive (containment building El. 278) were generated for 3, 5 and 7% of critical damping by a time-history method.

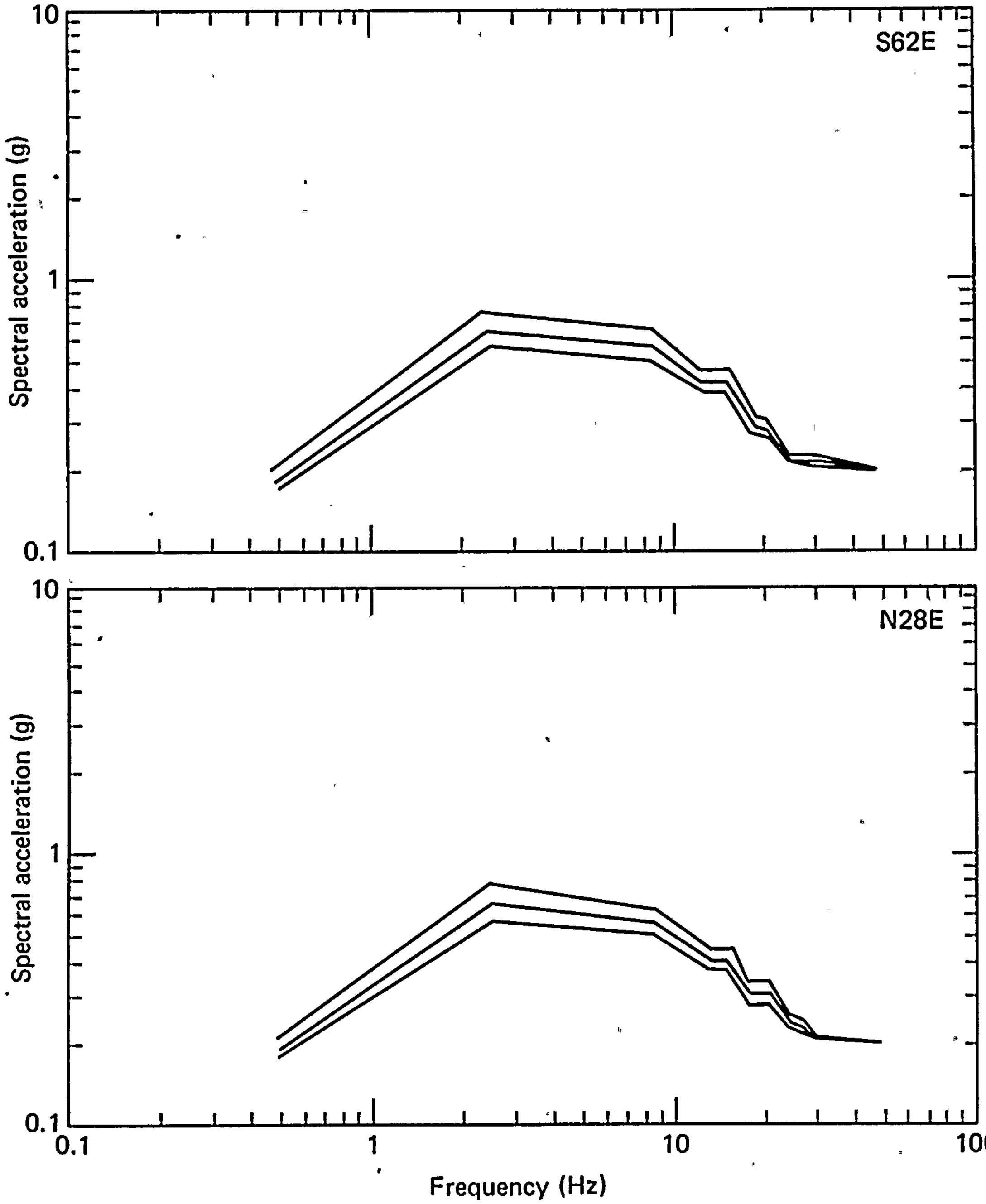

FIG. 29. In-structure response spectra for use in the reevaluation of steam generator SG-1A (containment building El. 250) were generated for 3, 5 and 7% of critical damping by a time-history method.

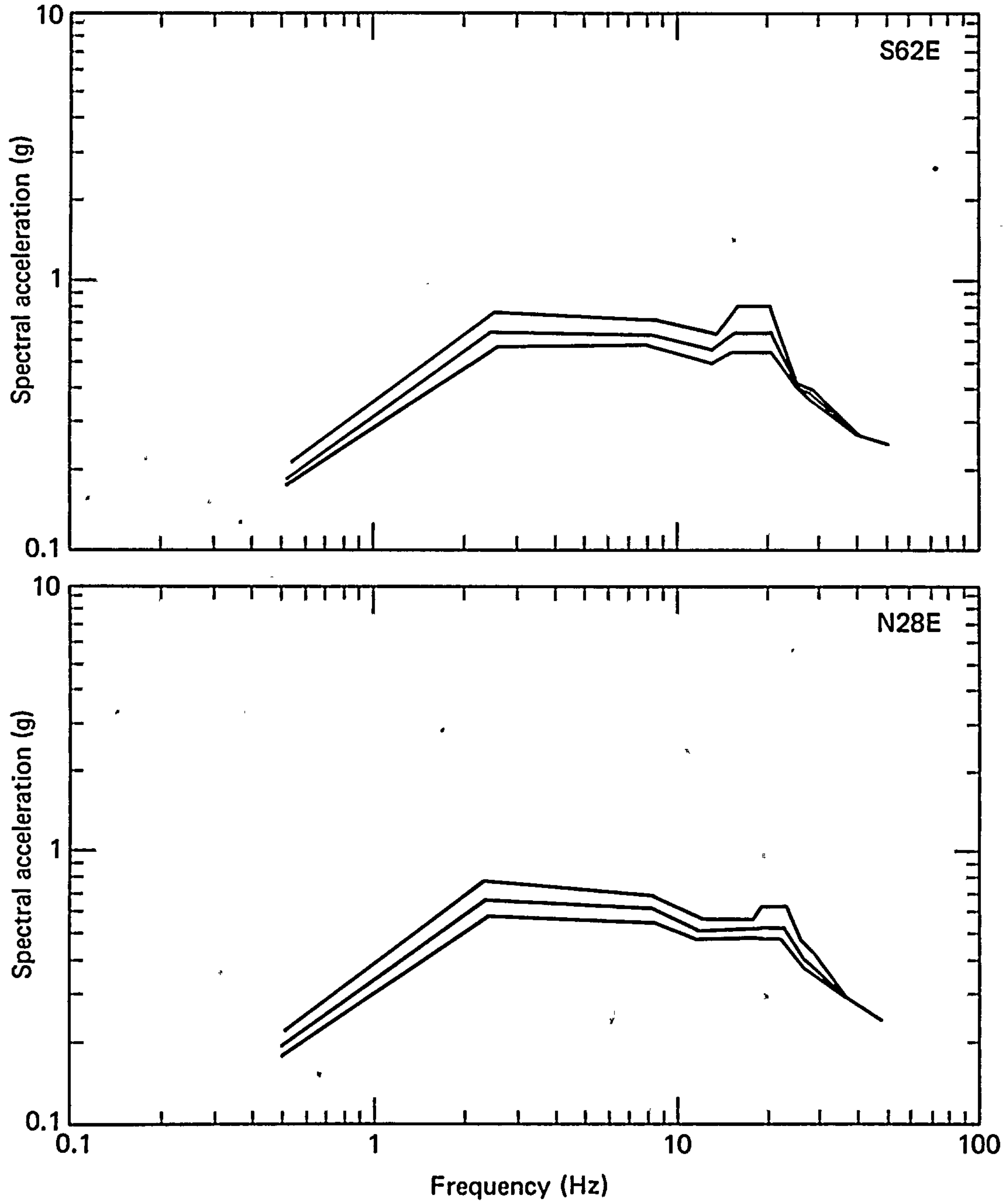

FIG. 30. In-structure response spectra for use in the reevaluation of steam generator SG-1A (containment building El. 278) were generated for 3, 5 and 7% of critical damping by a time-history method.

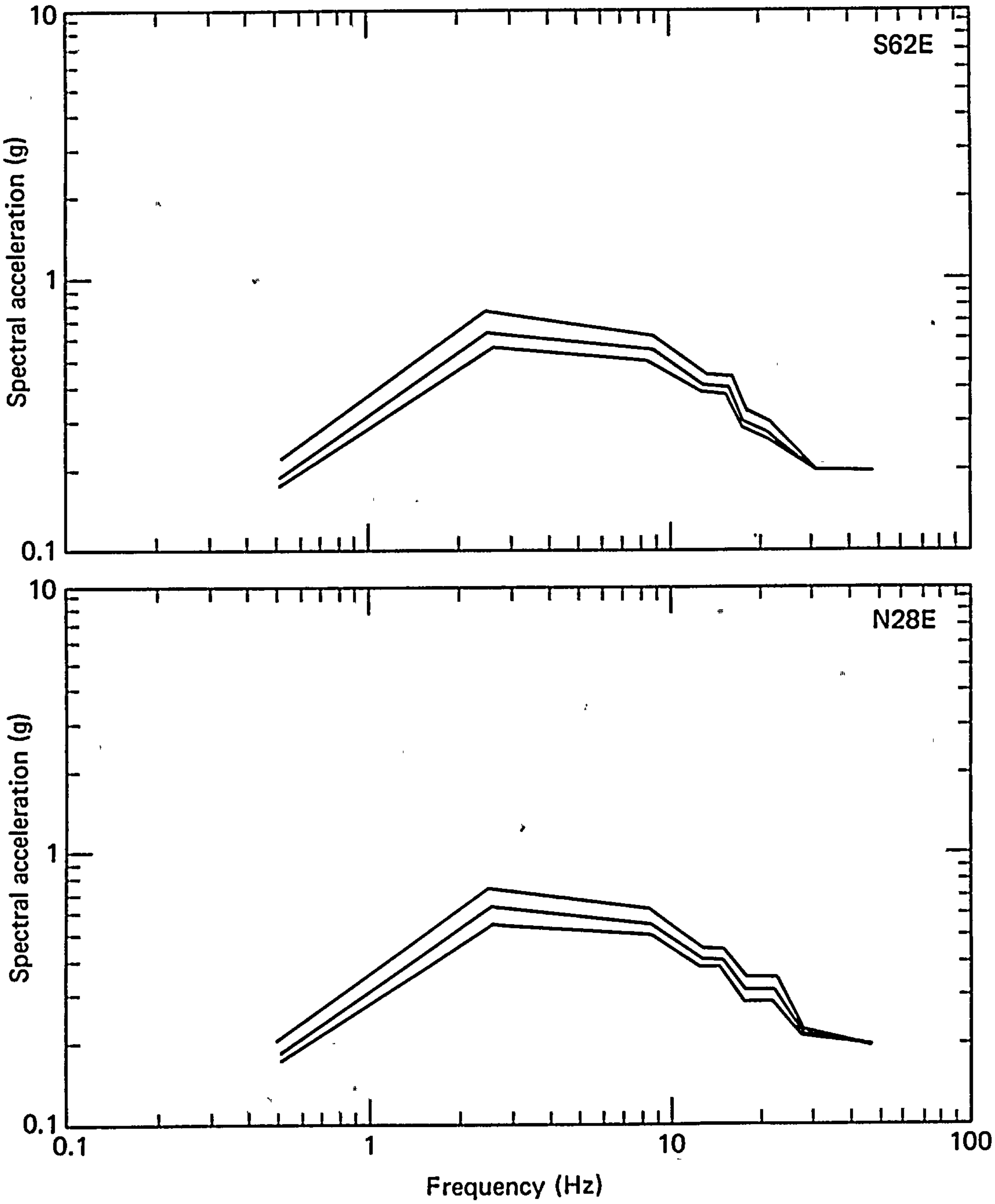

FIG. 31. In-structure response spectra for use in the reevaluation of steam generator SG-1B (containment building El. 250) were generated for 3, 5 and 7% of critical damping by a time-history method.

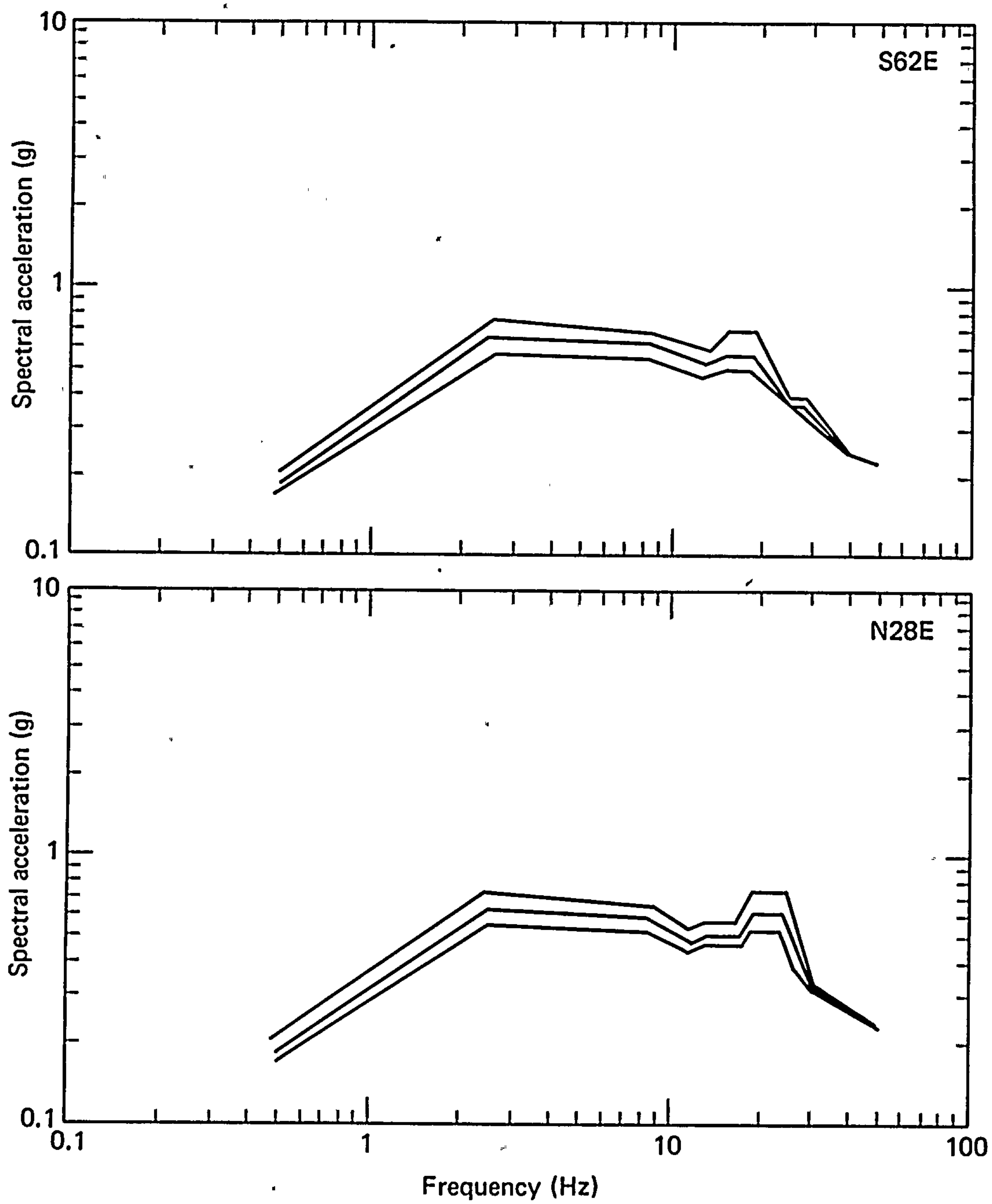

FIG. 32. In-structure response spectra for use in the reevaluation of steam generator SG-1B (containment building El. 278) were generated for 3, 5 and 7% of critical damping by a time-history method.

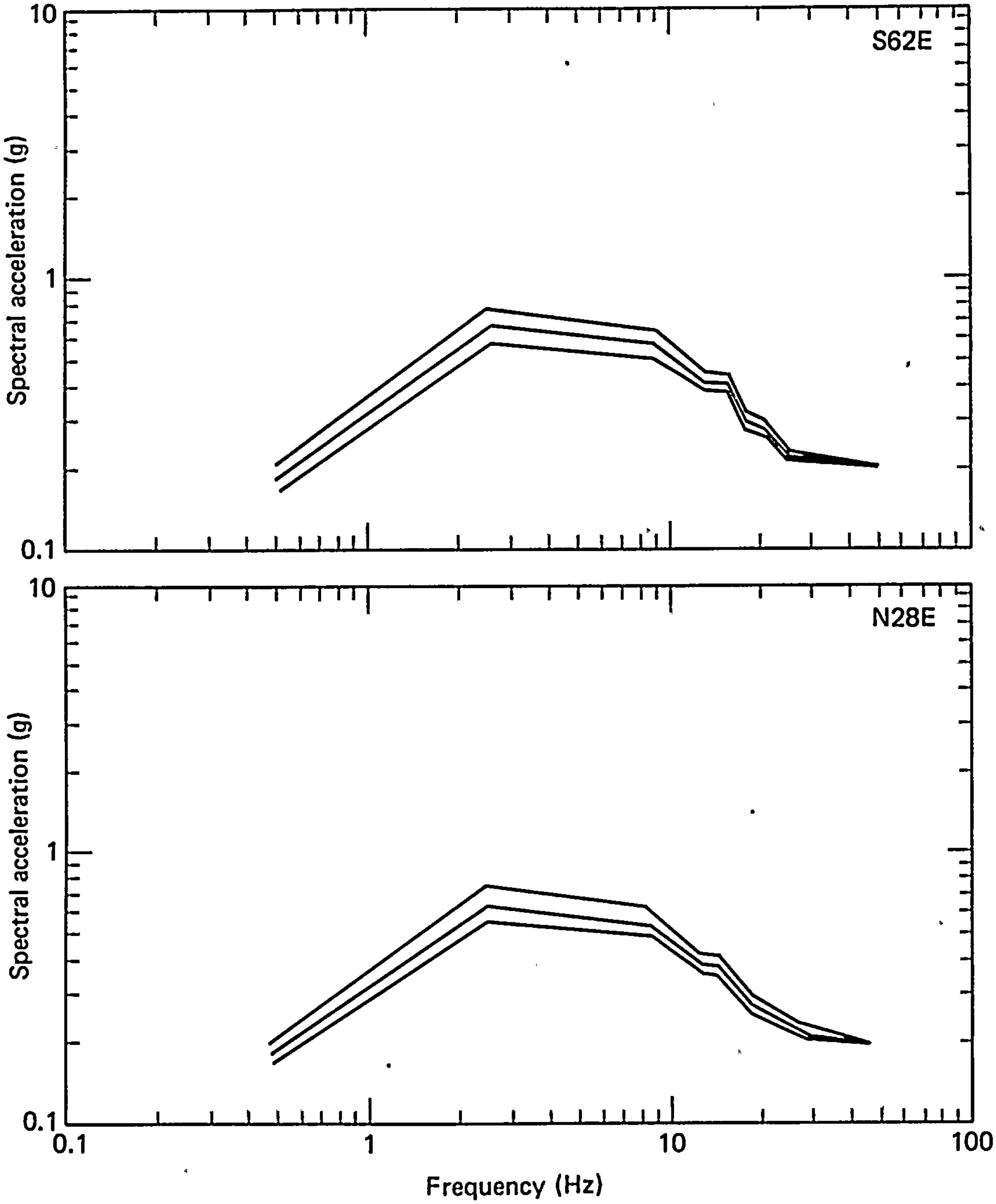

FIG. 33. In-structure response spectra for use in the reevaluation of reactor coolant pump RP-1A (containment building El. 247) were generated for 3, 5 and 7% of critical damping by a time-history method.

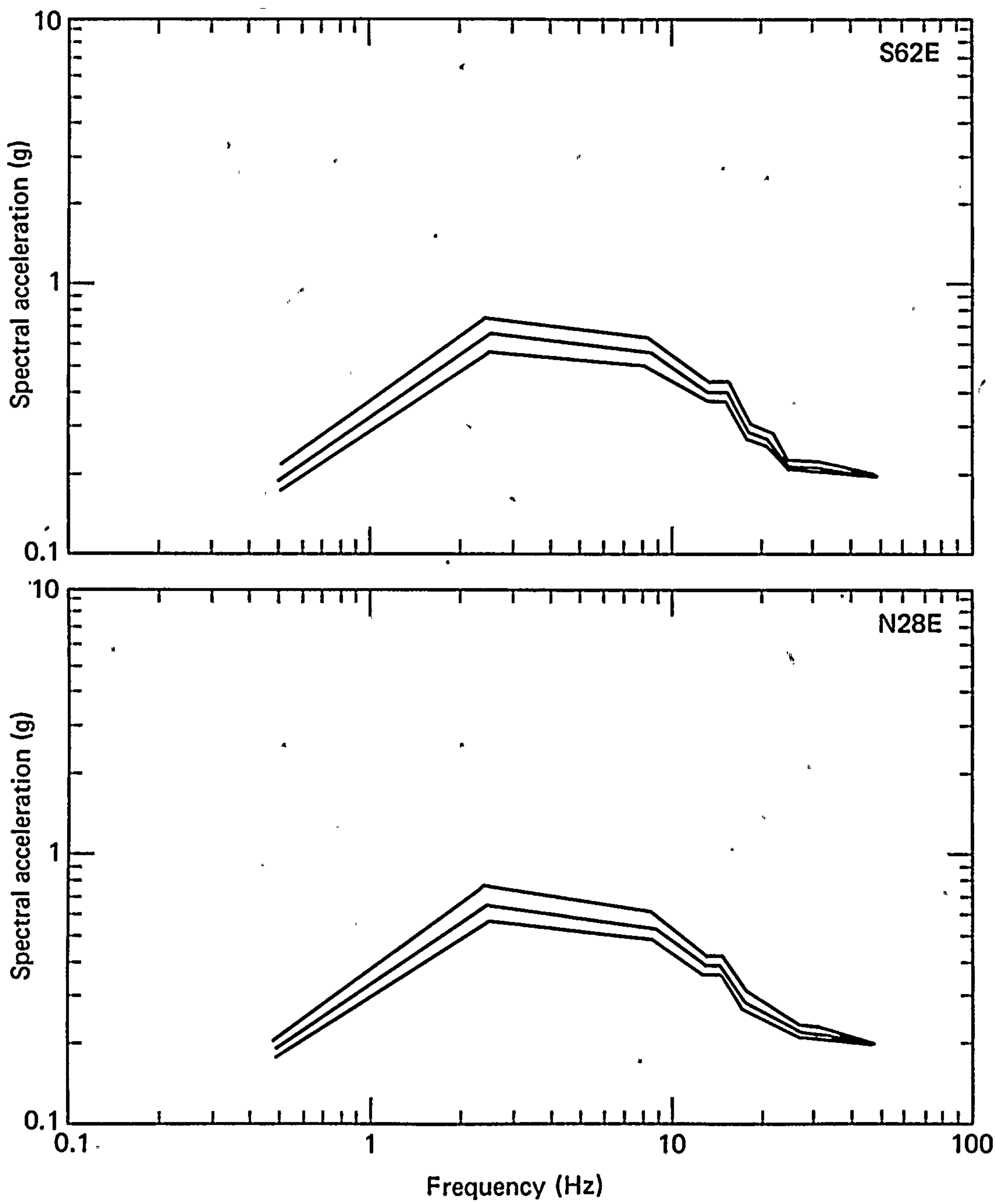

FIG. 34. In-structure response spectra for use in the reevaluation of reactor coolant pump RP-1B (containment building El. 247) were generated for 3, 5 and 7% of critical damping by a time-history method.

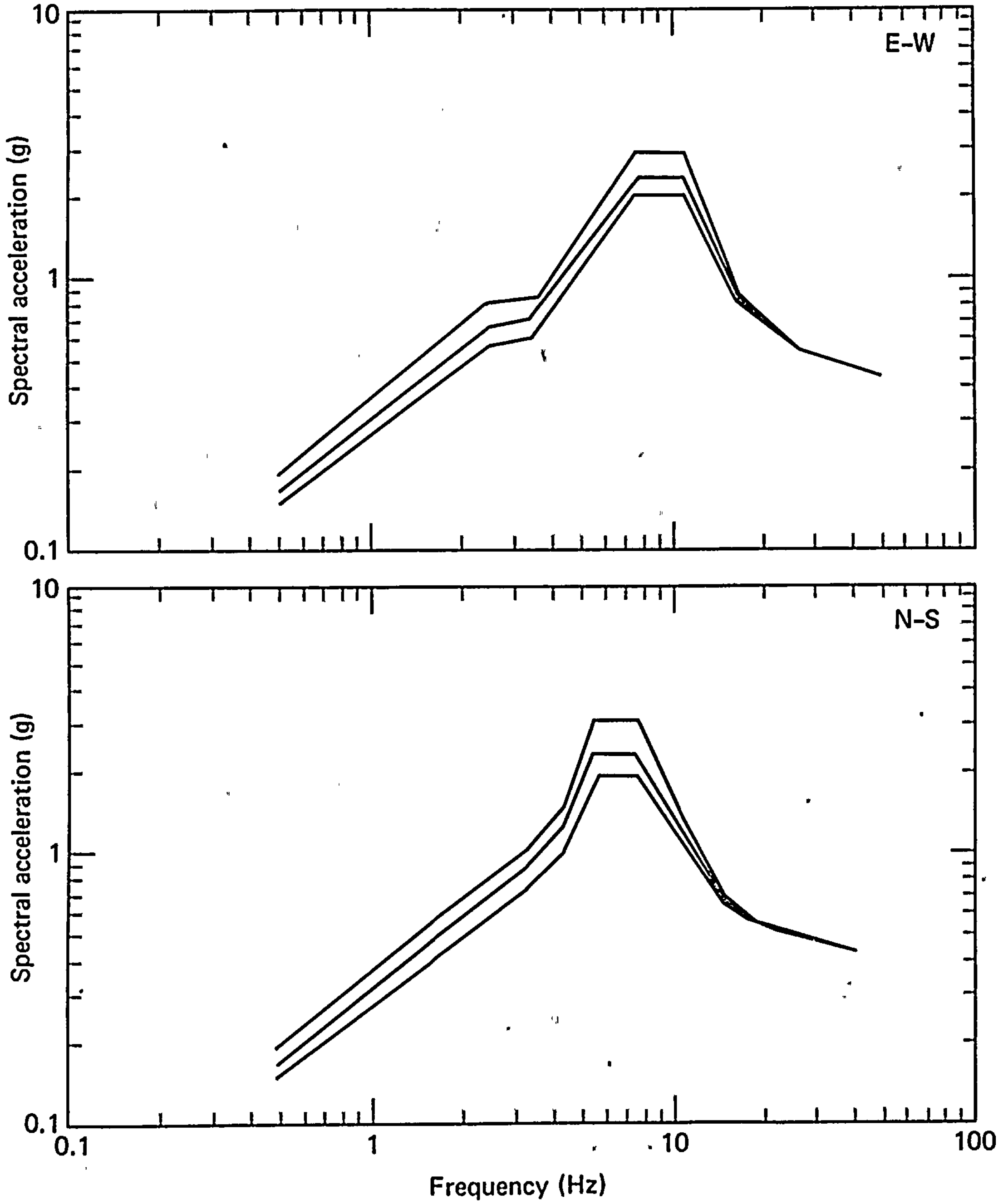

FIG. 35. In-structure response spectra for equipment reevaluation were generated for 3, 5 and 7% of critical damping by a direct method for the interconnected building complex at the auxiliary building platform center of gravity (El. 281.5).

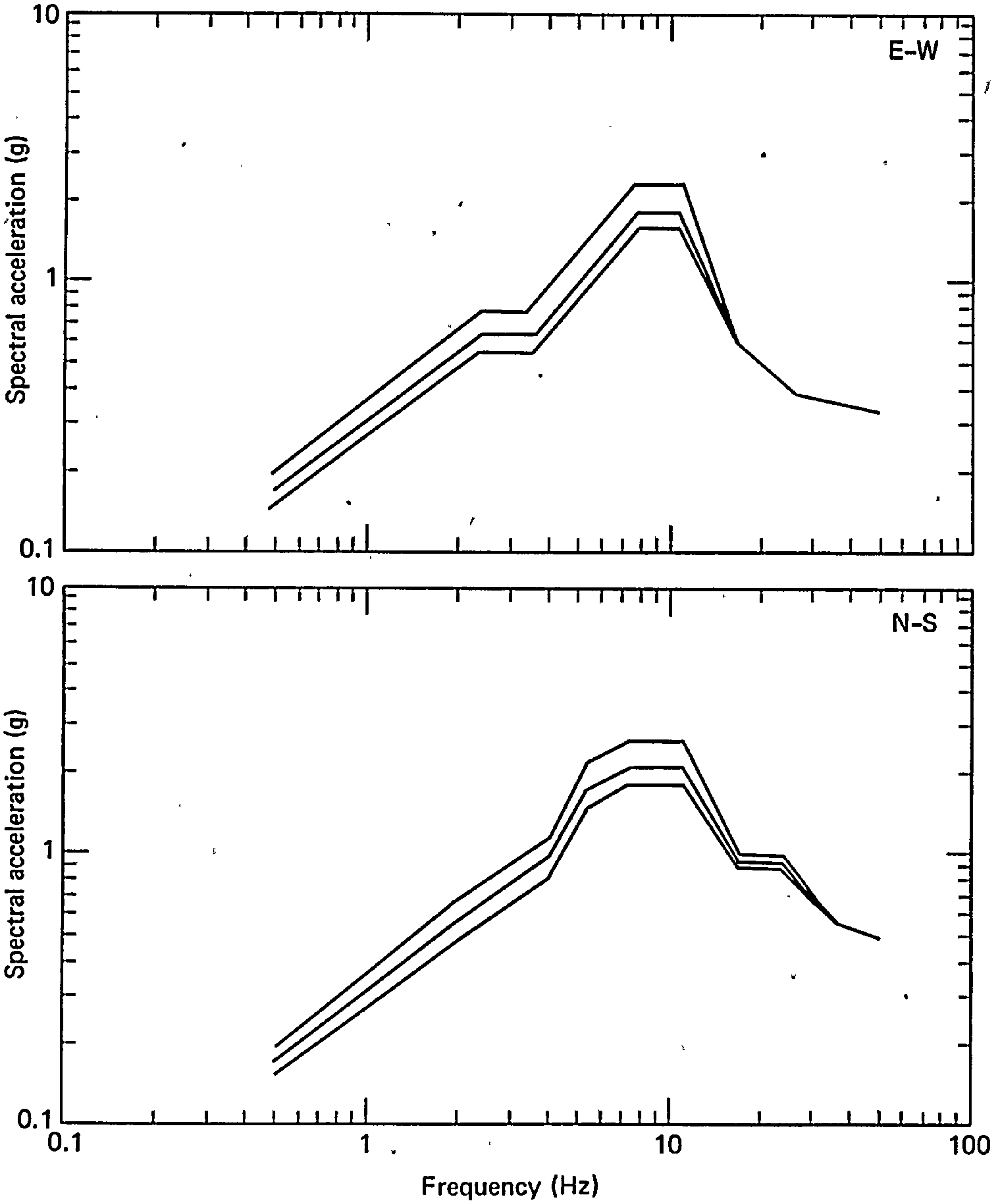

FIG. 36. In-structure response spectra for equipment reevaluation were generated for 3, 5 and 7% of critical damping by a direct method for the interconnected building complex at the auxiliary building heat exchanger 35 (El. 281.5).

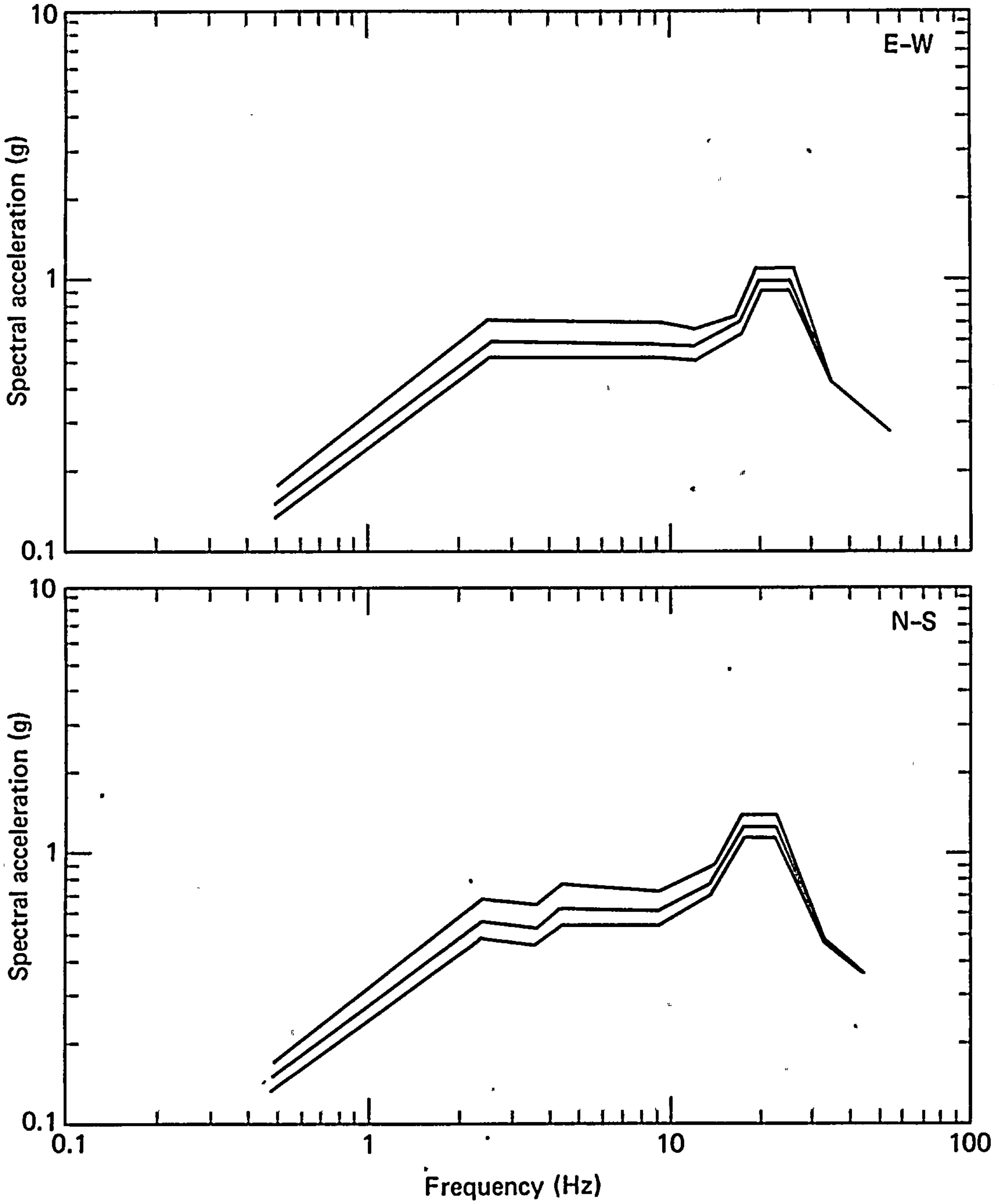

FIG. 38. In-structure response spectra for equipment reevaluation were generated for 3, 5 and 7% of critical damping by a direct method for the interconnected building complex at the auxiliary building boric acid tank 34 (El. 271).

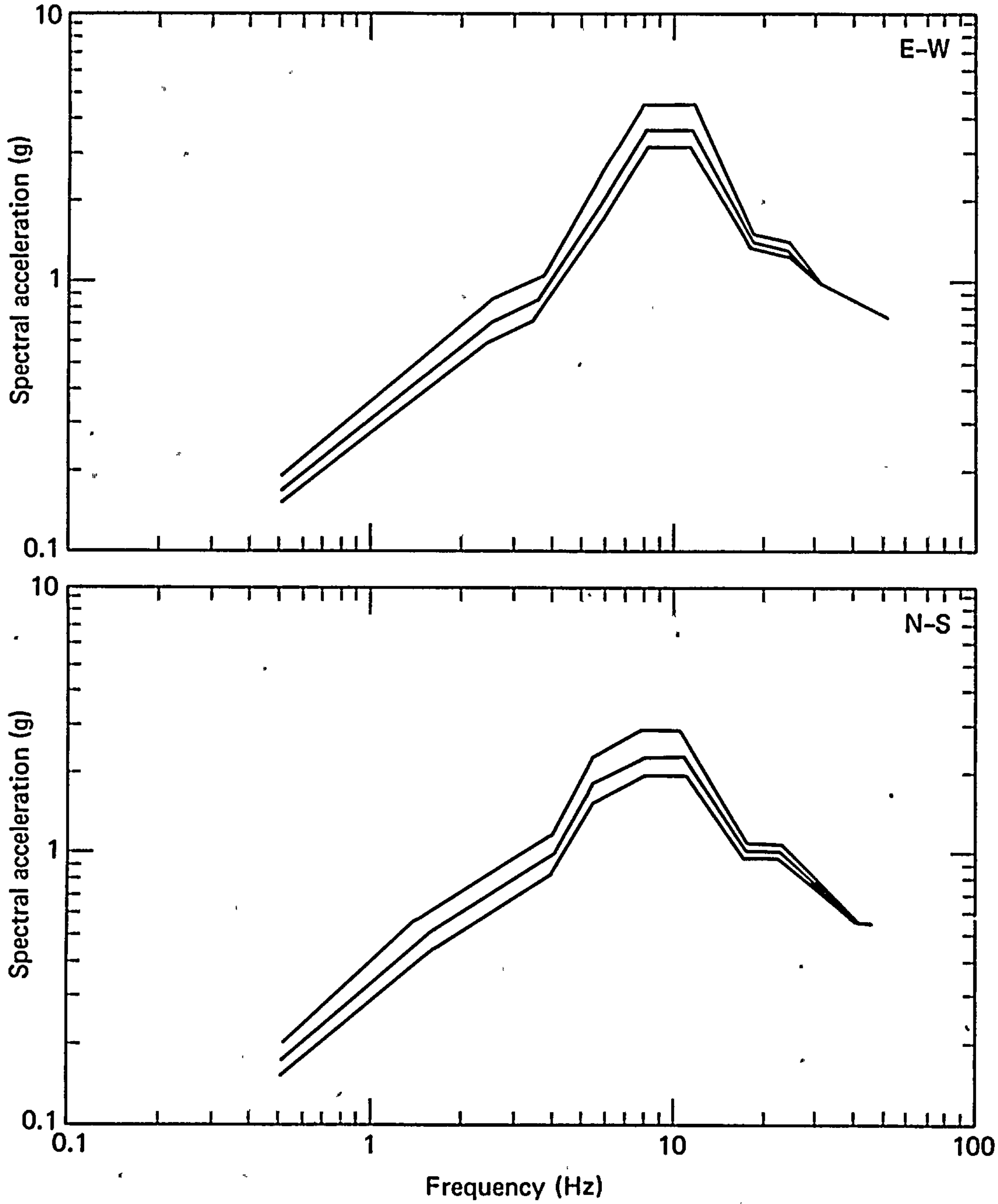

FIG. 37. In-structure response spectra for equipment reevaluation were generated for 3, 5 and 7% of critical damping by a direct method for the interconnected building complex at the auxiliary building surge tank 34 (El. 281.5).

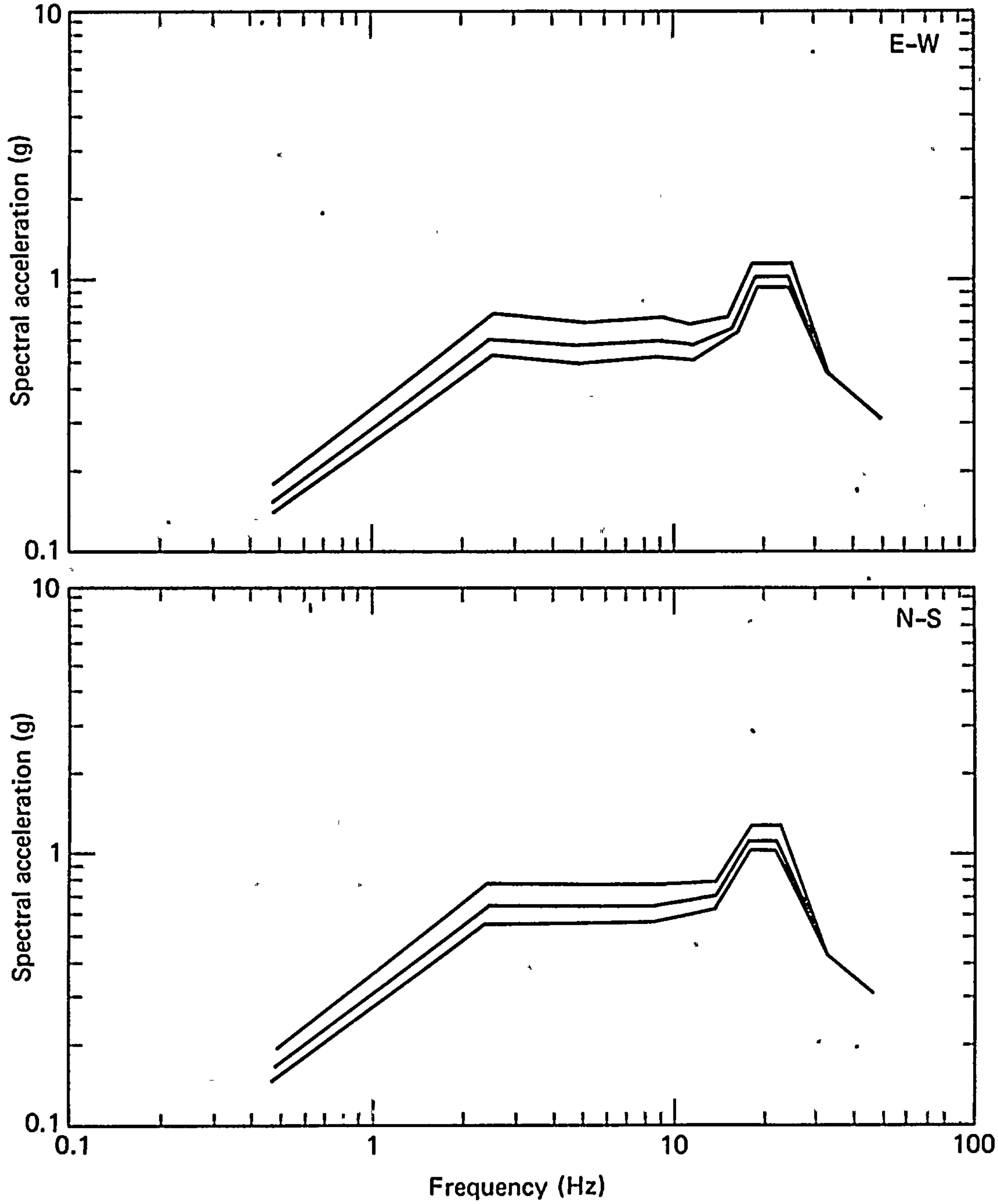

FIG. 39. In-structure response spectra for equipment reevaluation were generated for 3, 5 and 7% of critical damping by a direct method for the interconnected building complex at the auxiliary building operating floor center of gravity (El. 271).

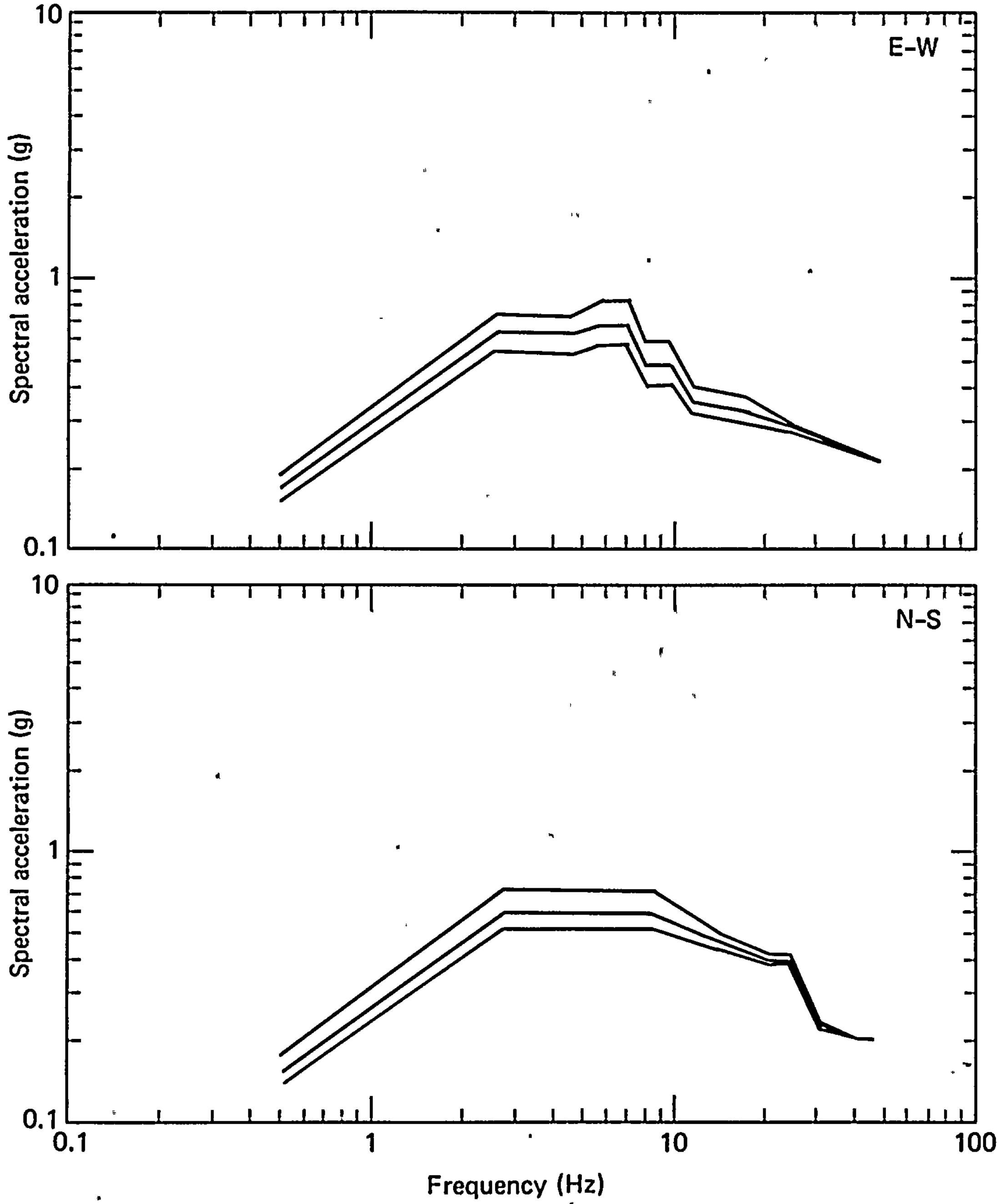

FIG. 40. In-structure response spectra for equipment reevaluation were generated for 3, 5 and 7% of critical damping by a direct method for the interconnected building complex at the control building basement floor center of gravity (El. 250).

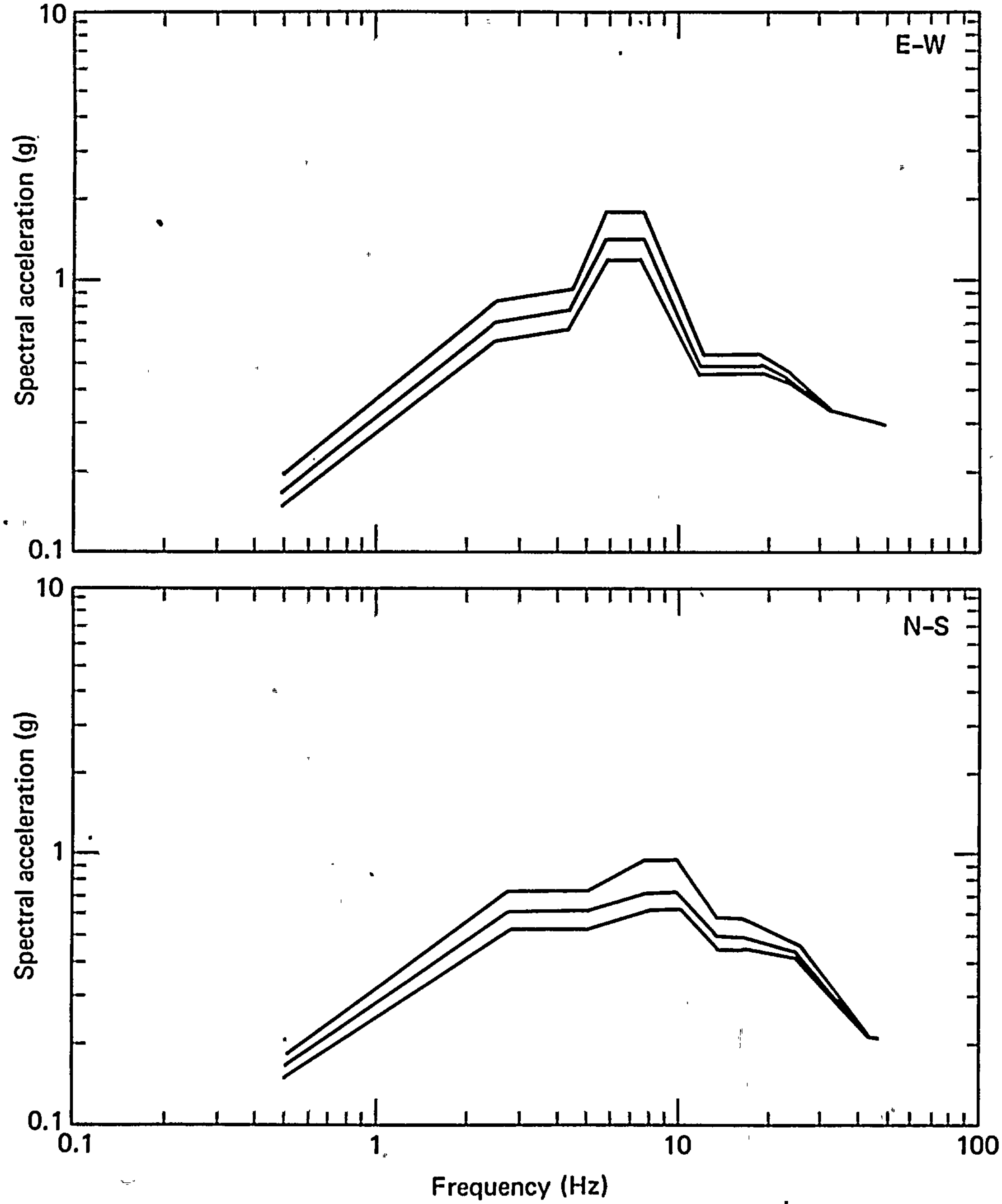

FIG. 41. In-structure response spectra for equipment reevaluation were generated for 3, 5 and 7% of critical damping by a direct method for the interconnected building complex at the control building relay room floor center of gravity (El. 269.75).

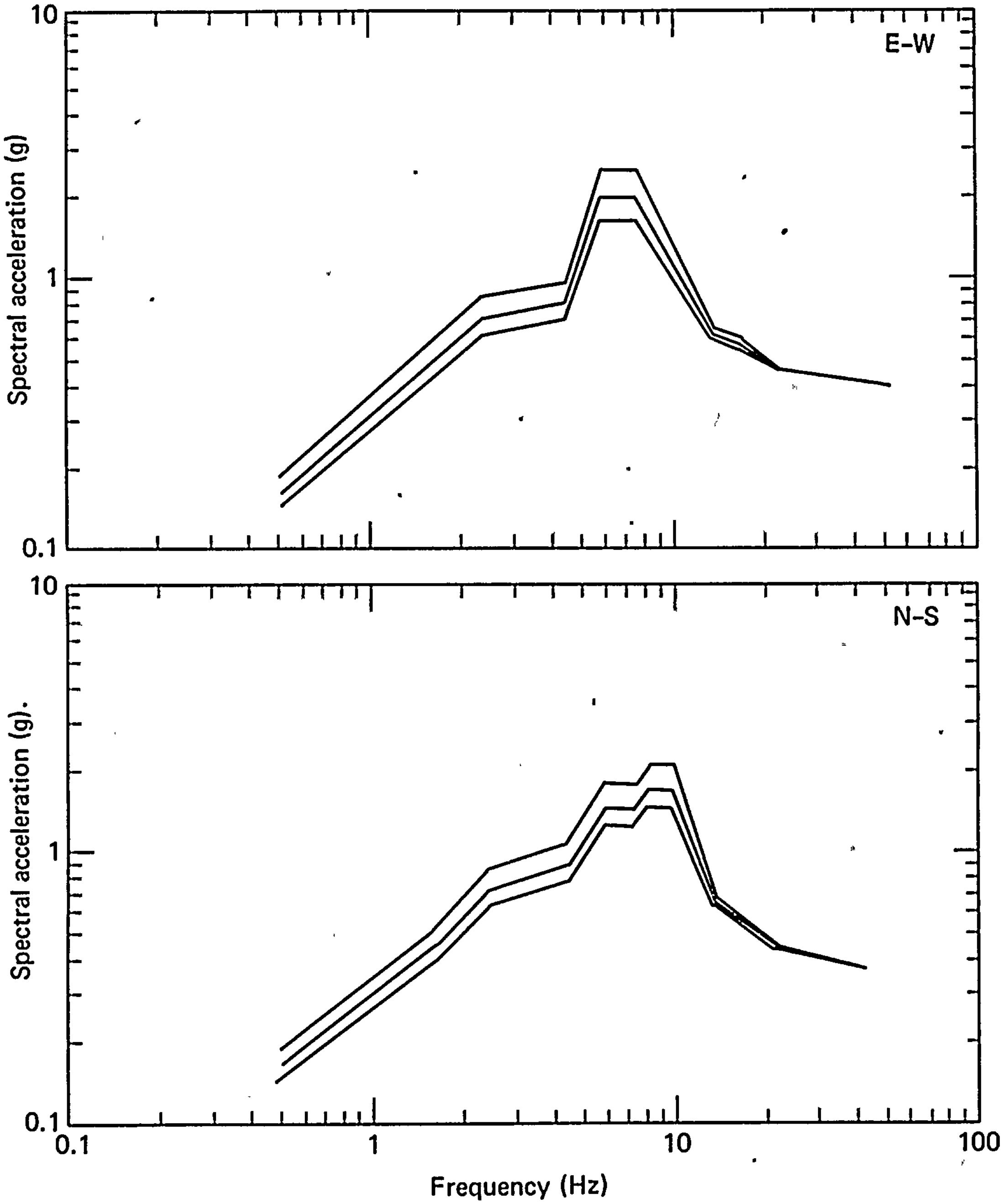

FIG. 42. In-structure response spectra for equipment reevaluation were generated for 3, 5 and 7% of critical damping by a direct method for the interconnected building complex at the control room floor center of gravity (El. 289.75).

CHAPTER 5: SEISMIC EVALUATION OF MECHANICAL AND ELECTRICAL EQUIPMENT AND FLUID AND ELECTRICAL DISTRIBUTION SYSTEMS

5.1 INTRODUCTION

5.1.1 Purpose and Scope

In this chapter we review selected seismic evaluation data that were developed to qualify certain mechanical and electrical equipment and fluid- and electrical-distribution systems at Ginna. Based on that review, we also evaluate the ability of the reactor to be safely shut down and remain in a safe shutdown condition in the event of an SSE. Note that the SEP senior review team purposely identified those components that are expected to have a high degree of seismic fragility. Moreover, the team believes that these components are representative not only of those installed in the safe shutdown systems, but of other seismic Category I systems, such as engineered safeguards, as well. Thus, evaluation of these components establishes an estimated lower-bound seismic capability for the mechanical and electrical components and the distribution systems of Ginna.

Considered in terms of seismic design adequacy, nuclear power plant equipment and distribution systems fall into two main categories and two subcategories. The two main categories are active and passive, and the two subcategories, under both the active and passive designations, are rigid and flexible.

As discussed in R.G. 1.48 (Ref. 25) and Sec. 3.9.3 of the Standard Review Plan[3], active components are those that must perform a mechanical motion to accomplish a system safety function. For the purpose of this report, this definition is expanded to include electrical or mechanical components, required for safe shutdown, that must change state (move) during or after a seismic event to perform their design safety function. Typically found in the active category are

- Pumps
- Valves
- Motors and associated motor-control centers
- Switch gear.

Seismic design adequacy of active components--which should demonstrate functional as well as structural integrity--may be demonstrated either by analysis or, preferably, by testing. However, because of size or weight restrictions or difficulty in monitoring function, many active components are seismically evaluated by analysis. To assure active component function by analysis, deformations must be limited and predictable. Therefore, total stresses in such components are normally limited to the elastic linear range of 0.5 to 0.9 times the yield stress of the material. Typically, the higher allowable stress limits are used with components constructed to meet what are generally considered to be the more rigorous requirements of the ASME Code (Ref. 17). The higher stress limits also tend to be used with austenitic type materials. Other manufacturing or construction codes and standards usually have less rigorous fabrication, inspection, and test requirements than those in Ref. 17. Hence, components manufactured to such other codes and standards tend to be qualified for lower allowable stress limits.

Passive components considered in this report are those components, required for safe shutdown, for which the only safety functions are to maintain leak-tight or structural integrity during or following the SSE. Typically found in the passive category are

- Pressure vessels
- Heat exchangers
- Tanks
- Piping and other fluid-distribution systems
- Transformers
- Electrical-distribution systems.

In determining seismic design adequacy by analysis, the most important distinction between active and passive components is the stress level that the component is allowed to reach in response to the SSE excitation. For passive components, higher total stress limits, which range from 1.0 times yield to 0.7 times ultimate strength of the material, are permitted by current design procedures and codes.[17]

The designation of flexible or rigid, as it relates to components and distribution systems, is important in developing the magnitude of seismic input for component evaluation.

Designation of rigid or flexible components for Ginna is complicated by the fact that many components are supported in the auxiliary and reactor buildings by concrete structures, which have high fundamental frequencies between 15 and 25 Hz as typically shown in Fig. 28, while other components are supported by steel superstructures, which have fundamental frequencies between 6 and 11 Hz as typically shown in Fig. 35. Note also that equipment supported at or near grade will be subject to nearly the ground response, which has peak response acceleration in the 2 to 9 Hz range as shown typically in Fig. 31. Therefore, components that have fundamental frequencies greater than 20 Hz and are located on grade or supported by structural steel could be considered rigid since there is little amplification in this region of the applicable response spectra. Similar components supported by concrete structures would be at or near building resonance and are considered flexible. For flexible components whose fundamental frequencies are less than twice the dominant building frequencies, the seismic inertial accelerations are typically 5 to 15 times the SSE peak ground acceleration, depending on:

- Potential resonance with the supporting building structure
- Structure and equipment damping levels
- Equipment support elevations.

The designation of rigid or flexible may also depend on a component's support. Many otherwise rigid components must be evaluated as flexible because of their support flexibility.

For this reassessment, components are grouped as active or passive, and rigid or flexible. Then, a representative sample of each group is evaluated to establish that group's seismic design factor of safety or degree of adequacy. In this way, seismic design factors within groups of similar components are established without the detailed reevaluation of hundreds of individual componets within each group.

A representative sample of components was selected for review by one of two methods:

- Selection based on a walk-through inspection of the Ginna facility by the SEP seismic review team. Based on their experience, team members selected components as to the potential degree of seismic fragility for that component's category. Particular attention was paid to the component's support structure.

- Categorization of the safe shutdown components into generic groups such as horizontal tanks, heat exchangers, and pumps; vertical tanks, heat exchangers, and pumps; motor control centers and motors.

The licensee was asked to provide seismic qualification data on the selected components from each group. Where available, such information is referenced.

In the rest of this chapter, we review the seismic capacity of the selected components and recommend, if necessary, additional analysis or hardware changes to qualify them for the SSE defined in this report. Based on the detailed review of the seismic design adequacy of the representative components discussed above, conclusions are developed as to the overall seismic design adequacy of seismic Category I equipment installed in Ginna. Table 10 and Sec. 5.4 summarize these conclusions.

5.1.2 Description of Components Selected for Review

Table 7 lists and describes those components that the SEP seismic review team selected based on its plant walk through as well as components that are representative of the generic groups of safety related components. Evaluation input was solicited from the licensee for the components listed. Table 7 also gives the basis for each selection.

Note that the review in this chapter emphasizes what are normally listed as auxiliary components. Such components are typically supplied by manufacturers who--unlike the nuclear steam supply system vendors and particularly when this plant was under construction--may not have routinely designed and fabricated components for the nuclear power industry. Therefore, if there is a reduction in seismic design adequacy, it would tend to be found in the auxiliary equipment, rather than in the major nuclear components. However, because of its importance to safety, the seismic design adequacy of the reactor coolant system support structures is also evaluated in this report.

TABLE 7. Mechanical and electrical components selected by the SEP seismic review team for seismic evaluation and the bases for selection.

Item No.	Description	Reason for selection
	<u>Mechanical components</u>	
1	Essential service water pump	This item has a long vertical unsupported intake section which was originally statically analyzed for seismic effects.
2	Component cooling heat exchanger	This item is supported on what appears to be a relatively flexible structural steel framing and by two saddles that do not appear to be seismically restrained. Concern was expressed about the saddles' ability to carry required seismic loads, particularly in the longitudinal direction.
3	Component cooling surge tank	Same as Item 2.
4	Diesel generator air tanks	This item is a skirt-supported vertical tank.
5	Boric acid storage tank	This item is a column-supported vertical tank.
6	Refueling water storage tank	Anchor-bolt systems for in-structure flat-bottom tanks that are flexible may be overstressed if tank and fluid contents were assumed rigid in the original analysis.
7	Motor-operated valves	A general concern with respect to motor-operated valves, particularly for lines 4 in. or less in diameter, is that the relatively large eccentric mass of the motor will cause excessive stresses in the attached piping if the valves are not externally supported.

TABLE 7. (Cont.)

Item No.	Description	Reason for selection
	Mechanical components	
8	Steam generators	Items are particularly critical to ensure reactor coolant system integrity.
9	Reactor coolant pumps	Same as Item 8.
10	Pressurizer	Same as Item 8.
11	Control rod drive mechanism	Same as Item 8.
12	Reactor coolant system supports	Same as Item 8.
	Electrical components	
13	Battery racks	The bracing required to develop lateral load capacity may not be sufficient to carry the seismic load.
14	Motor control centers	Typical seismically qualified electrical equipment. Functional design adequacy may not have been demonstrated. In addition, anchorage to floor structure may be inadequate.
15	Switchgear	Same as Item 14.
16	Control room electrical panels	The control panels appear to be adequately anchored at the base. However, there appear to be many components cantilevered off of the front panel, and the lack of front panel stiffness may permit significant seismic response of the panel, resulting in high acceleration of the attached components.
17	Electrical cable raceways	The cable tray support systems do not appear to have positive lateral restraint and load carrying capacity.

5.2 SEISMIC INPUT AND ANALYTICAL PROCEDURES

5.2.1 Original Seismic Input and Behavior Criteria

For seismic Category I mechanical equipment, all components and systems originally classified as Class I (Sec. 1.2.1) were reportedly designed in accordance with the criteria described in Sec. 3.4. All components of the reactor coolant system and associated systems were designed to the standards of the applicable ASME or USAS Codes. The loading combinations and behavior criteria not otherwise defined by the USAS and ASME Codes in use at the time of the original design which were employed by Westinghouse in the design of the components of these systems, i.e., vessels, piping, supports, vessel internals and other applicable components, are given in Table 8. Table 8 also indicates the stress limits which were used in the design of the equipment for the various loading combinations. In addition, the supports for the reactor coolant system were designed to limit the stresses in the pipes and vessels to the stress limits given in Table 8.

For seismic Category I electrical equipment, all components and systems originally classified as Class I (Sec. 1.2.1) were reportedly designed in accordance with the criteria described in Sec. 3.7.

5.2.2 Current Seismic Input

Current seismic input requirements for determining the seismic design adequacy of mechanical and electrical equipment and distribution systems are normally based on in-structure (floor) response spectra for the elevations at which the equipment is supported. The floor spectra used in this reassessment, which are based on R.G. 1.60 spectra, are shown in Figs. 26 through 42. For details about how the spectra were generated, see Secs. 4.7, 4.5.2, and 4.4.3.

For mechanical and electrical equipment, a composite 7% equipment damping is used in the evaluation for the 0.2 g SSE. For piping evaluation, the equipment damping associated with the SSE is limited to 3%. These values also are consistent with a recent summary of data presented to define damping as a function of stress level.[26] For cable trays, recent tests seem to indicate that the damping levels to be used in design depend greatly on the tray and

TABLE 8. Loading combinations and stress limits used by the prime contractor when Ginna was designed.

Loading combinations	Vessels and reactor internals	Piping	Supports
1. Normal + Design Earthquake Loads	$P_m \leq S_m$ $P_L + P_B \leq 1.5\ S_m$	$P_m \leq 1.2\ S$ $P_L + P_B \leq 1.2\ S$	Working stresses
2. Normal + Maximum Potential Earthquake Loads	$P_m \leq 1.2\ S$ $P_L + P_B \leq 1.2\ (1.5\ S_m)$	$P_m \leq 1.2\ S$ $P_L = P_B \leq 1.2\ (1.5\ S)$	Within yield after load redistribution
3. Normal + Pipe Rupture Loads	$P_m \leq 1.2\ S_m$ $P_L + P_B \leq 1.2\ (1.5\ S_m)$	$P_m \leq 1.2\ S$ $P_L + P_B \leq 1.2\ (1.5\ S)$	Within yield after load redistribution

Where P_m = primary general membrane stress; or stress intensity

P_L = primary local membrane stress; or stress intensity

P_B = primary bending stress; or stress intensity

S_m = stress intensity value from ASME B&PV Code, Section III

S = allowable stress from USAS B31.1 Code for Pressure Piping support construction and the manner in which the cables are placed in the trays. Damping may be as high as 20% of critical damping.[27]

5.2.3 SEP Acceptance Criteria

Seismic Category I components that are designed to remain leak tight or retain structural integrity in the event of an SSE are now typically designed to the ASME Section III Code (ASME III), Class 1, 2 or 3 stress limits for Service Condition D. The stress limits for supports for ASME leak-tight components are limited as shown in Appendix F or Appendix XVII to ASME III.[17]

When qualified by analysis, active ASME III components that must perform a mechanical motion to accomplish their safety functions typically must meet ASME III Class 1, 2, or 3 stress limits for Service Condition B. Supports for these components are also typically restricted to Service Condition B limits to ensure elastic low deformation behavior.

For other passive and active equipment, which are not designed to ASME III requirements, and for which the design, material, fabrication, and examination requirements are typically less rigorous than ASME III requirements, the allowable stresses for passive components are limited to yield values and to normal working stress (typically 0.5 to 0.67 yield) for active components. The current behavior criteria used in various equipment and distribution systems for Ginna passive components are given in Table 9. For active electrical components such as switches, relays, etc., functional adequacy should be demonstrated by test.

Experience in the design of such pressure retaining components as vessels, pumps, and valves to the ASME III requirements, at 0.2 g zero period ground acceleration, indicates that stresses induced by earthquakes seldom exceed 10% of the dead weight and pressure-induced stresses in the component body.[28] Therefore, design adequacy of such equipment is seldom dictated by seismic design considerations.

Seismically induced stresses in nonpressurized mechanical and electrical equipment, in fluid- and electrical-distribution systems, and in all component supports may be significant in determining design adequacy. Note that SSE loadings seldom control the design of piping systems. For primary stresses, the OBE normally controls design.

TABLE 9. SEP structural behavior criteria for determining seismic design adequacy of passive mechanical and electrical equipment and distribution systems.

Components	SEP criteria (SSE)	
Vessels, pumps, and valves	$S_{m_{all}} \leq 0.7\ S_u$ and $1.6\ S_y$	ASME III Class 1 (Table F 1322.2.1)
	$S_{m_{all}} \leq 0.67\ S_u$ and $1.33\ S_y$	ASME III Class 2 (NC 3217)
	$\sigma_{m_{all}} \leq 0.5\ S_u$ and $1.25\ S_y$	ASME III Class 2 (NC 3321)
	$\sigma_{m_{all}} \leq 0.5\ S_u$ and $1.25\ S_y$	ASME III Class 3 (ND 3321)
Piping	$S_{m_{all}} \leq 1.0\ S_u$ and $2.0\ S_y$	ASME III Class 1 (Table F 1322.2.1)
	$S_h \leq 0.6\ S_u$ and $1.5\ S_y$	ASME III Class 2 and Class 3 (NC 3611.2)
Tanks	No ASME III Class 1	
	$\sigma_{m_{all}} \leq 0.5\ S_u$ and $1.25\ S_y$	ASME III Class 2 and Class 3 (NC 3821)
Electric equipment	$S_{all} \leq 1.0\ S_y$	
Cable trays	$S_{all} \leq 1.0\ S_y$	
ASME supports	$S_{all} \leq 1.2\ S_y$ and $0.7\ S_u$	ASME III Appendices XVII, F for Class 1, 2 and 3
Other supports	$S_{all} \leq 1.6\ S$	Normal AISC S allowable increased by 1.6 consistent with NRC Standard Review Plan, Sec. 3.8
Bolting	$S_{all} \leq 1.4\ S$	ASME Section III Appendix XVII for bolting where S is the allowable stress for design loads

5.3 EVALUATION OF SELECTED COMPONENTS FOR SEISMIC DESIGN ADEQUACY

5.3.1 Mechanical Equipment

5.3.1.1 Essential Service Water Pump

The essential service water (ESW) pump-and-motor unit is oriented vertically in the screen house and supported at El. 253.5 ft. As shown on Worthington Corporation drawing DEN-19353, the intake portion of the pump extends downward from the discharge head and pump base a distance of 36.5 ft.

The previous seismic analysis was performed for equivalent static loads of 0.32 g acting simultaneously in one horizontal and the vertical direction.[29]

The pump-motor unit is located at grade; therefore, the seismic input used in our reassessment is essentially the R.G. 1.60 ground response spectrum for 7% of critical damping (Fig. 15). The pump was evaluated for an inertial acceleration value considering peak response of 0.52 g horizontal acceleration and 0.35 g vertical acceleration.[30] Overturning tensile and shear stresses in the pump base anchor bolts were determined as were stresses at the attachment of the intake column pipe to the discharge head.

Because the intake portion of the pump is oriented vertically as a cantilever beam, the dynamic characteristic of the intake suction pipe was determined. The intake suction pipe was found to have a fundamental frequency of 1.6 Hz based on a weight distribution that includes water in the shaft. Because of this natural frequency, the spectral acceleration used was the peak of Fig. 14 (0.52 g). The stress calculated at the flange connecting the discharge head to the intake column pipe is 54,000 psi. Even if the use of cast iron material were acceptable, this stress level is clearly unacceptable, as the equivalent ANSI B31.1 allowable stresses for a Condition D service limit on a typical cast iron piece (A48Gr.40) would be 9600 psi. If a lateral brace is installed on the intake column pipe approximately 24 ft from the intake head, the stress would drop to 7600 psi. With the brace, the stresses at the bolts would be 15,700 psi in tension and 7,000 psi in shear, which yield a minimum factor of safety in shear of 2.29 for ASME Condition D stress limits for an assumed A307 bolt material. Note that without the column brace, the tensile stress at the anchor bolts (91,300 psi) would exceed the ultimate tensile strength (55,000 psi) of A307 bolts.

We also are concerned about the intake shaft seismic design adequacy. In the calculations shown in Ref. 29, it was assumed that the 2-3/16 in. shaft deformed as did the 14-in. pipe. The span of the shaft between support bearings within the pipe is not defined. In general, this span will have a much greater effect on the stresses in the shaft than will the deformation of the supporting pipe. Shaft bearing capacities are required as well. Thus, additional information is needed to evaluate the shaft design adequacy.

We believe that the ESW pump-motor unit designed as a passive component to Condition D stress levels will not withstand a 0.2 g SSE unless a support brace is added to the pipe column at approximately El. 229.5 ft. To meet active component design limits (ASME III, Condition B), the cast iron discharge head would require replacement. In addition, the intake shaft requires further evaluation.

5.3.1.2 Component Cooling Heat Exchanger

The component cooling heat exchanger (CCHX) is a horizontal heat exchanger located in the auxiliary building and supported by two saddles at El. 281.5 ft. One saddle is slotted in the longitudinal direction to permit thermal expansion. The heat exchanger is shown on Atlas Industrial Manufacturing Company Drawing D-1260-7. The previous seismic qualification of the heat exchanger is described in Ref. 31.

We reviewed the previous analysis and independently evaluated the dynamic response characteristics of the heat exchanger and its saddle support system using the response spectra for 7% damping shown in Fig. 36.[32] The review indicates that the system is relatively rigid and has no response frequencies below 33 Hz. Thus, SSE input horizontal seismic accelerations in the orthogonal directions in Fig. 36 are 0.36 g and 0.60 g.

Note that both the CCHX and the component cooling surge tank (CCST) discussed in Sec. 5.3.1.3 are supported by a complex structural steel framework. Evaluation of the fundamental frequencies of both the CCHX and CCST have not considered any flexibility of the structural steel support framing. It has been assumed that the dynamic characteristics of this structural steel framing are included in the response spectra.

In addition to evaluating the CCHX saddle and anchor bolt support system, the seismic stresses induced in its tubes and shell were determined, combined with other applicable loads, and compared to code allowables.[32] The safety factor determined for the heat exchanger tube is 33.9, and that for the shell is 11.0. Both factors are controlled by hoop stress caused by internal pressure. Note that no evaluation has been made of nozzle loads in the heat exchanger since they were determined from the attached piping system analysis, which is not currently available for evaluation. In general, such piping loads, which can be a limiting load to the nozzle, seldom significantly affect the heat exchanger support loads.

The seismic accelerations were simultaneously applied to the heat exchanger, and the resulting anchor bolt stresses were determined. The analysis established a factor of safety with respect to ASME Code-allowable stress limits of 1.41 for the anchor bolts. Therefore, we believe that the component cooling heat exchanger will withstand a 0.2 g SSE, without loss of structural integrity, based on:

- Review of the original seismic analysis (Ref. 31)
- Evaluation of the dynamic characteristics of the tank-support system and supplemental analysis given in Ref. 32
- Experience in reviewing similar saddle-supported tanks.

5.3.1.3 Component Cooling Surge Tank

The component cooling surge tank is a horizontal component located in the auxiliary building and supported by two saddles at El. 281.5 ft. The surge tank is shown on Westinghouse Electric Corporation Drawing 684-J-700, Sheets 1 and 2. The previous seismic qualification of the surge tank is described in Ref. 33.

We reviewed the previous analysis and independently evaluated the structural characteristics of the surge tank and its support system using the response spectra for 7% damping shown in Fig. 37.[34] In the transverse (E-W) direction, the tank-support system is rigid. However, since the tank is restrained only by two slotted holes in both saddles in the longitudinal direction, it is not positively anchored against sliding. Lateral stability of the tank in the longitudinal direction is developed only by friction. In our opinion, all seismically qualified components should be positively anchored against earthquake forces unless the component is analyzed considering friction forces and the potential for nonlinear response associated with rocking and sliding, including potential impact effects on the anchor bolts. In the absence of such an analysis, we recommend that the tank saddle supports be modified to provide positive lateral restraint in the longitudinal direction in one saddle and thermal expansion movement on the other saddle.

The seismic forces in the transverse (E-W) direction developed from a 0.75 g in-structure spectral acceleration (the rigid value of Fig. 37) were applied to the surge tank, and the resulting tank, saddle, and anchor bolt stresses were determined.[34] Factors of safety for the tank, saddle, and anchor bolts--loaded seismically in the tranverse and vertical directions--are 125.5, 57.7, and 5.08, respectively. As was the case for the CCHX, no attempt has been made to evaluate nozzle loads on the tank since they are unavailable and are seldom large enough to affect support integrity.

5.3.1.4 Diesel Generator Air Tanks

The diesel generator air tanks (shown on the ALCO Products, Inc., Drawing 49-C-73137 and anchored as shown on Gilbert Associates, Inc., Drawing SS-581-112) are oriented vertically in the diesel generator building and supported at grade elevation in a rock-supported structure. The previous analysis to seismically qualify the tanks for a 0.2 g SSE ground response spectrum is presented in Ref. 35.

In view of the support, the seismic input used for reassessment is the R.G. 1.60 ground response spectrum for 7% of critical damping (Fig. 15). The tanks are supported by a skirt structure, and the combined tank-support system was found to have a fundamental frequency of 33 Hz.[36] Therefore, the tank-support system may be considered rigid, and the input acceleration is 0.2 g. Considering two independent horizontal seismic components, maximum stress in the anchor bolts is approximately 0.28 ksi in shear, which yields a safety factor of 61.3 for A307 bolt material. The minimum safety factors in the tank body (for which hoop stress controls) and skirt support are 4.43 and 3968, respectively. As was the case for the CCHX and the CCST, no stress evaluations at attached piping nozzles were made because loadings are unavailable. We believe that the diesel generator tanks, including supports, will withstand a 0.2 g SSE without loss of structural integrity based on:

- Review of the analysis of the diesel generator air tanks supplied by the licensee (Ref. 35)

- Evaluation of the dynamic characteristic of the tank-support system and supplemental analysis performed in connection with this report (Ref. 36)
- Experience in reviewing similar vertical tanks.

5.3.1.5 Boric Acid Storage Tank

The boric acid storage tank is a column-supported tank as shown on Westinghouse Electric Corp. Drawing 684-J-809, Sheets 1 and 2. The previous seismic qualification of the tank-support system is described in Ref. 37.

We reviewed the tank, its support legs, and its anchors to determine seismic design adequacy.[38] The tank, which is supported at El. 271 ft., was evaluated using the in-structure response spectra shown in Fig. 38. The dynamic analysis considered the effective impulsive and convective response of the contained fluid. The fundamental response frequencies for the tank were calculated to be 17.2 Hz for tank-support system bending and shear deformation under impulsive loading (7% damping) and 0.56 Hz under convective loading (1/2% damping). The analysis determined gross dynamic characteristics of the tank and established a minimum factors of safety of approximately 41.7 for membrane stress in the tank, 6.20 for compressive stresses in the tank legs, and 4.65 for compressive stresses in the anchor bolts. As is the case for other components with attached piping, we did not evaluate nozzle capacities because piping loads are unavailable. We believe that the boric acid storage tank will withstand the 0.2 g SSE without loss of structural integrity, based on:

- Review of the stress analysis of the boric acid storage tank support supplied by the licensee (Ref. 37)
- Check on the dynamic characteristics of the tank and an additional evaluation of tank, support leg, and anchor bolt stresses performed in connection with this report (Ref. 38)
- Experience in reviewing similar tanks.

5.3.1.6 Refueling Water Storage Tank

The refueling water storage tank is a vertical vessel that is 81 ft high to the top of the cylindrical portion and 26.5 ft in diameter (the tank is shown on Pittsburgh-Des Moines Steel Company Drawing 5606-02-5A, the anchorage on Gilbert Associates, Incorporated, Drawing SS-581-111). The anchorage consists of 30, 2.5-in.-diam A36 bolts. The tank was qualified according to TID-7024 assuming an SSE ground acceleration of 0.2 g (Ref. 39).

The tank, which is supported at the ground floor (El. 236 ft) of the auxiliary building, was reevaluated as shown in Ref. 40 for R.G. 1.60 response spectra normalized to 0.2 g. The dynamic analysis considered the effective convective and impulsive response of the contained fluid and determined fundamental response frequencies for the tank--0.34 Hz under convective loading (1/2% damping) and 2.3 Hz for tank bending and shear deformation under impulsive loading (7% damping). Therefore, the tank can be considered flexible for the impulsive moment effect.

The analysis to determine overall dynamic characteristics of the tank-fluid system revealed that the anchor bolts will fail and that the tank shell will buckle. For ASME III, Appendix XVII allowable bolt stresses for A36 bolt material, the factor of safety for combined tension and shear stresses is 0.86. The safety factor is 0.47 for compressive stress in the tank wall from combined seismic overturning and deadweight stresses. Therefore, we believe that the refueling water storage tank will potentially fail under 0.2 g SSE loading based on:

- Evaluation of the flexible characteristics of the tank
- Very high stresses developed in the anchor bolts and tank wall
- Experience in reviewing similar tanks.

5.3.1.7 Motor-Operated Valves

It has been our experience that for lines smaller than 4 to 6 inches in diameter, the eccentricity of motor-operated valves not otherwise externally supported may cause additional significant piping stresses (in excess of 10% of code allowable) that should be considered in the computation of total pipe stresses. The applicable 10% stress levels we have considered are ASME III, Class 2, Condition B for active valves and ASME III, Class 2, Condition D when only pressure boundary integrity is required. This tendency to develop significant stresses increases as the diameter of the line decreases.

Our seismic evaluation of valves for lines 4 in. in diameter and smaller is described in Ref. 41. Calculations performed on randomly selected motor-operated valves (2-in., 3-in. and 4-in. diameter) in the Ginna plant demonstrate that stress levels are well in excess of the above mentioned 10%, regardless of service condition.

For a typical ferritic piping material (S_h = 15,000 psi) the Condition B and D stress limits would be 18,000 psi and 36,000 psi, respectively. Preliminary calculations indicate that the following stress levels would be reached if a peak acceleration of 3 g, as determined from the auxiliary building platform spectra (Fig. 35), is applied to the valves:

Diam, in.	Stress, psi	% of Condition B	% of Condition D
4	5,700	32%	16%
3	14,300	79.5%	40%
2	36,300	202%	101%

Based on this evaluation, it is recommended that the licensee evaluate the seismic stresses induced by motor-operated valves in supporting pipe that is 4 in. in diameter and smaller and show that stresses resulting from motor operator eccentricity are less than 10% of the service Condition B code allowable stresses. If not, the total stresses at motor-operated valve locations should be calculated to determine that they are within Condition B code allowable levels. For passive valves, Condition D service levels would apply.

Alternatively, for all motor-operated valves supported by pipes that are 4 in. in diameter and smaller, we recommend that a requirement to support the valve operator externally be developed and implemented.

In addition, we have reviewed the seismic function qualification test report provided for Limitorque Valve Actuators in Ref. 42. We cannot agree that the test as performed meets the qualification requirements of IEEE 344-1975 in that

a. The input acceleration wave form is not defined. It appears to be single frequency and could be sinusoidal or sine-beat form. The limitations of the use of single frequency testing given in Sec. 6.6.2 of IEEE 344-1975 do not appear to have been addressed.

b. Qualification testing at high acceleration ($\geq$ 3.2 g) was performed only at 33 Hz. The test input acceleration from 5 to 10 Hz, which is the resonant region for most piping, ranges from 0.13 to 0.45 g, well below the input in-structure spectral acceleration values for Ginna in the rigid region.

c. The mounting of the valve and valve-operator during the test is not described sufficiently to determine if it is representative of field installation.

Based on the information supplied to date, we cannot conclude that the seismic functional adequacy of the Limitorque has been demonstrated for Ginna.

5.3.1.8 Steam Generators

The stress analyses provided for the steam generator, reactor coolant pump, pressurizer, and control rod drive mechanism were summaries of the stress resultants determined from detailed analyses of the components (Refs. 43 to 46. Such stress resultants depend on the analytical assumptions, procedures, models and boundary conditions used in the analyses. Too little information was supplied for us to be able to review the original analyses; therefore, we cannot comment on whether the stress summaries reflect the use of the state of the art or generally acceptable analytical methods and assumptions. Given the limited information provided by the licensee, we cannot comment on the actual design adequacy of the steam generator, reactor coolant pump, pressurizer, and control rod drive mechanism.

In this section, however, we do report the changes in the stress summaries provided given the changes in seismic input associated with the use of the in-structure response spectra shown in Figs. 29 through 32 for 7% of critical damping and the use of two rather than one horizontal component of earthquake motion.

A summary of the stress analysis of the steam generators is given in Ref. 43. In 1975, a generic stress report was written which contained updated analyses of most areas of the steam generator that are subject to external loads, i.e., primary nozzles, feedwater nozzle, steam nozzle and lower support pads. The updated stress report also contains an analyses of the tubes, swirl vanes and feedwater ring. Calculated stress intensities were compared with the ASME III design condition allowable levels for an OBE and the Emergency Condition allowable levels for an SSE.

Since the fundamental frequency of the steam generator is below 10 Hz, the peak acceleration in both the N-S and E-W directions is 0.60 g (Figs. 29 through 32), and the SRSS value for two horizontal components is 0.85 g. Since the original horizontal response spectra used for the design of the steam generator give a minimum spectral acceleration of 2.0 g for the SSE condition, the seismic stresses resulting from use of the Ginna reassessment response spectra would be less than the 1975 stress report values.

5.3.1.9 Reactor Coolant Pumps

A summary of the stress analysis of the reactor coolant pumps is given in Ref. 44. In 1968, a static seismic load stress analysis was performed for the pumps. The SSE analysis used 0.8 g horizontally and 0.54 g vertically. The stresses and deformations resulting from these loads were then combined with the dead weight and other normal operating loads to determine the total stresses in the motor, support stand cylinder, flange welds, support stand bolts, and main flange bolts. The 1968 analysis also contains evaluations of the pump support feet, primary nozzles and casing for seismic plus normal operating loads. The stresses calculated in these analyses were compared with ASME III allowables.

The reevaulation in-structure response spectra for the reactor coolant pumps are given in Figs. 33 and 34. By using the peak spectral acceleration of 0.55 g for both the N-S and E-W directions, the SRSS value is 0.78 g, and the ratio of this value to the original design value of 0.8 g is 0.97 (Note that SRSS is used in this context to designate a vector sum, which is mathematically equivalent). Thus, the pump input acceleration is less than that considered in the 1968 SSE analysis, and the pump is adequate to the extent that the 1968 generic analysis verifies design adequacy.

5.3.10 Pressurizer

The pressurizer is a vertical cylindrical vessel with a skirt type support attached to the lower head. The lower part of the skirt terminates in a bolting flange where 24, 1.5-in. bolts secure the vessel to its foundation.

The stress analysis of the pressurizer is summarized in Ref. 45. In 1969, a generic seismic analysis of the pressurizer shell, support skirt, support skirt flange, and pressurizer support bolts was performed. The weight of the largest pressurizer (1800 cu ft) was used instead of the actual operating weight of the Ginna pressurizer (800 cu ft). In the SSE evaluation accelerations were applied statically at the center of gravity of the 1800-cu-ft model--0.48 g in the horizontal direction and 0.32 g in the vertical direction. ASME III Upset Condition allowable levels were used for SSE load cases.

The pressurizer heaters were qualified generically for the 51 Series Pressurizer. The heaters in the 800-cu-ft pressurizer are shorter than those qualified above, but are otherwise identical. The qualification procedure used an equivalent static load of 37.5 g for the SSE condition. The fundamental frequency of the heater rods was found to be greater than 33 Hz.

The in-structure response spectra used in the reassessment of the pressurizer are shown in Fig. 26. Since the fundamental frequency of the pressurizer may be as low as 3 Hz, peak spectral accelerations were used--0.55 g for the N-S direction and 0.60 g for the E-W direction. The SRSS value is 0.81 g, and the ratio of this SRSS value to the original design value of 0.48 g is 1.7.

Table 1 of Ref. 45 gives total stress resultants, including thermal, based on the 1969 analysis of the pressurizer. Table 2 of the same reference gives primary stress resultants for a 6.7 g SSE seismic input load based on a 1973 analysis. The revised SRSS seismic input of 0.81 g determined in this evaluation is well within the design limits presented in Table 2 of Ref. 45.

5.3.1.11 Control Rod Drive Mechanism

The previous stress analysis of the control rod drive mechanism was a static seismic analysis of the drive mechanism to determine if the bending moment allowable levels were exceeded.[46] A static horizontal SSE load of 0.8 g was applied to the mechanism center of gravity, and moments were calculated at several sections along the length of the mechanism. The 0.8 g load was also used to determine if the seismic support attached to the top of the rod-travel housing was adequately designed.

The response spectra for our reevaluation of the control rod drive mechanism are given in Figs. 27 and 28. Assuming that the fundamental frequency of the drive mechanism is less than 12.5 Hz, the peak spectral acceleration in both the N-S and E-W directions is 0.60 g and the SRSS value is 0.85 g. The ratio of this SRSS value to the original design value of 0.8 g is 1.1. If the previous seismic moments (from Ref. 46) are multiplied by 1.1, the results are less than the allowable bending moment levels. Similarly, if the previous seismic support stresses are multiplied by 1.1, the results are less than the allowable stress levels.

5.3.1.12 Reactor Coolant System Supports

For the steam generator and reactor coolant pump, the stress resultants given in Refs. 43 and 44 are for the components only and do not consider the supports. In Ref. 47, which we reviewed, an evaluation of the steam generator and pump supports indicates that they are heavily dominated by LOCA-induced loads. We therefore conclude that these supports are adequate for SSE-induced loads acting alone.

5.3.1.13 Piping

Because the licensee is currently reviewing piping design, piping was not part of this SEP reevaluation.

5.3.2 Electrical Equipment

5.3.2.1 Battery Racks

A sketch of the battery racks installed in Ginna is shown in Ref. 48. These racks were manufactured by Gould-National Batteries, Inc., and appear to be of the same design as the 125-V racks installed in Dresden 2. Comparison of the input response spectra used to evaluate Dresden 2 (Ref. 1) to the in-structure response spectra generated for the battery-racks location at Ginna (Fig. 40) reveals that the Dresden spectra are equal to or greater than the Ginna spectra. As was the case for Dresden 2, we recommend that the wooden battens which now laterally restrain the batteries be strengthened or replaced so that friction between the batteries and their support rail no longer need be relied upon to carry seismic loads.

5.3.2.2 Motor Control Centers 1L and 1M

A previous computer analysis was made of a Westinghouse Type-W ac motor control center which was originally tested at Wyle Laboratories in October, 1972, to meet the seismic requirements recommended by IEEE Std. 344-1971.[49] The calculations determined the acceleration levels and type of motion response that was excited in the equipment by a simultaneous horizontal and vertical sine beat type of motion input (5 cycles/beat). Subsequently, a similar dynamic analysis was made of the equipment as modified for Ginna, with attention focused on the new panelboard and distribution transformers.

The original Ginna response spectra, as specified for the SSE condition, give a total rms vector input acceleration of 0.79 g calculated as 0.56 times the SRSS value of the following three components:

x-direction (front to rear) = 0.707 x 0.56 g = 0.4 g
y-direction (side to side) = 0.707 x 0.56 g = 0.4 g
z-direction (vertical) = 1.0 x 0.56 g = 0.56 g.

The value of 0.56 g was specified for the Ginna test in Fig. 3 of Ref. 49. The Wyle Laboratory response spectra, on the other hand, give a total rms vector input acceleration of 1.49 g.

We compared the response spectra at the auxiliary building platform and operating floor centers of gravity (Figs. 35 and 39)--which should be limiting for the motor control centers--to the Wyle Laboratories spectrum. Above 5 Hz, the acceleration levels throughout the equipment were greater when calculated for the 5 cycles/beat test at the 8.5 Hz fundamental natural frequency, compared to an envelope of the Ginna in-structure response spectra in Figs. 35 and 39.

Therefore, based on review of the test results and comparison of input response spectra, as well as corresponding acceleration levels sustained in the equipment, we believe that the existing fragility level tests performed at Wyle Laboratories can be used to qualify the Ginna motor control centers, which have fundamental frequencies above 5 Hz.

5.3.2.3 Switchgear

The previous seismic qualification of Westinghouse Type DB-50 Reactor Trip Switchgear for Ginna was performed according to Refs. 50 and 51. The reports present results of seismic simulation testing for the "low seismic" (SSE peak acceleration not exceeding 0.2 g) and "high seismic" (SSE between 0.2 g and 0.4 g) classes of plants over the frequency range 1 to 35 Hz. The simulated seismic tests consisted of three elements:

- Inputing a sine beat type acceleration to the base of the equipment being tested.
- Monitoring the resulting accelerations at various locations in the equipment.
- Monitoring the electrical functions of the equipment both during and after the tests to check for any loss of function.

Each sine beat of the vibration input consisted of ten cycles of the test frequency with the amplitude of the beat (i.e., the acceleration of the vibration) increasing from a small value to the specified maximum value and returning to the initial value in sine wave fashion. The maximum required vertical input acceleration of the sine beat as a function of test frequency for the "low seismic" plant classification was 0.5 g up to 10 Hz and reduced to a minimum value of 0.2 g at 25 Hz. For horizontal excitation, the maximum required acceleration level of the sine beat was 0.8 g up to 10 Hz and reduced to a minimum value of 0.2 g at 25 Hz. Corresponding values for the "high seismic" plant classification were 0.93 g up to 10 Hz, reducing to 0.32 g at 25 Hz for vertical excitation and 1.4 g up to 10 Hz, reducing to 0.5 g at 25 Hz for horizontal excitation.

The applicable reassessment response spectra for the switchgear (Fig. 39) are higher than both the "low seismic" and "high seismic" horizontal acceleration input curves for frequencies between 15 and 30 Hz. Based on our review of the tests performed at Westinghouse Astronuclear Laboratory, as given in Refs. 50 and 51, we believe that the Westinghouse Type DB-50 Reactor Trip Switchgear will maintain its electrical function during an SSE event. However, this conclusion is based on the assumption that there are no resonant frequencies in the 15 to 30 Hz range, or, if such resonances exist, that the response spectra developed from the sine beat test at the resonant frequency for 7% of critical damping envelop the Ginna spectra.

5.3.2.4 Constant Voltage Transformers

The constant voltage transformers (CVTs) are located in the battery rooms of the control building at El. 253.7 ft. The seismic qualification of the CVTs is based upon a seismic test of transformers that were designed by Battelle Columbus Laboratories for the Snupps Nuclear Power Plant Project.[52] The Snupps transformers are identical to those installed in Ginna. The horizontal response spectra for the Ginna control building are shown in Figs. 40 and 41, whereas the peak acceleration of the Snupps test response spectra for SSE excitation in the horizontal direction is 13 g, and the minimum response acceleration is 2.4 g. Therefore, we believe the CVTs will maintain their functional performance and structural integrity during a 0.2 g SSE since the test response spectra everywhere envelop the reassessment in-structure response spectra of Fig. 40.

5.3.2.5 Control Room Electrical Panels

No evaluation has been performed since no drawings or design calculations are currently available.

5.3.2.6 Electrical Cable Raceways

No evaluation has been performed since no drawings or design calculations are currently available.

5.4 SUMMARY AND CONCLUSIONS

Table 10 summarizes our findings on the sample of mechanical and electrical components and distribution systems that were evaluated to determine the seismic design adequacy of such items required for safe shutdown of the Ginna nuclear steam supply system. As discussed in Sec. 5.1, the sample includes components that the seismic review team selected, based on judgement and experience, as representative of lower-bound seismic design capacity for representative categories.

Based upon the design review and independent calculations made for this reassessment for the SEP seismic load condition, we recommend design modifications or reanalysis for the following mechanical and electrical components to be able to withstand the 0.2 g SSE without loss of structural integrity and required safety function:

1. Essential service water pump
2. Motor-operated valves
3. Component cooling surge tank
4. Refueling water storage tank
5. Battery racks.

Because we lacked essential seismic design/qualification data as of the writing of this report, our review the seismic design adequacy of electrical equipment is incomplete. We have only reviewed battery racks, motor control centers, and switchgear; therefore, we can not conclude whether design modifications will be necessary for control panels or electrical distribution systems.

Note also that our conclusions about the following components are based on data that have not been independently verified:

Reactor control rod drive
Reactor vessel supports
Steam generator
Reactor coolant pump
Pressurizer and its supports.

In particular, the stress summaries that were made available for these components (Refs. 43 through 46) did not contain enough information for us to verify the assumed input loads and review the analyses performed.

TABLE 10. Seismic review team conclusions and recommendations regarding equipment seismic design adequacy.

Equipment item	Conclusion and recommendation
1. Essential Service Water (ESW) Pump	The ESW pump-motor system requires a support brace to be added to the pipe column at approximately El. 229.5 ft if the pump can be treated as a passive component. If active, the cast iron discharge bowl may require replacement by steel.
2. Component Cooling Heat Exchanger	O.K.
3. Component Cooling Surge Tank	The tank is not positively restrained in the longitudinal direction. Either a more rigorous analysis is required or the tank requires addition of a longitudinal restraint.
4. Diesel Generator Air Tanks	O.K.
5. Boric Acid Storage Tank	O.K.
6. Refueling Water Storage Tank (RWST)	High stresses develop in the anchor bolts because of the 0.2 g SSE and the flexible response of the tank. In addition, the shell will buckle from overturning moment effects. Therefore, we recommended that a design modification be made for the RWST to be able to withstand the 0.2 g SSE without loss of structural integrity.
7. Motor-Operated Valves	Generic analysis of motor-operated valves on lines $\leq$4 in. in diameter should be performed to show that resulting stresses are less than 10% of the applicable Condition B (active) or Condition D (passive) allowable stresses. Otherwise, stresses induced by valve eccentricity should be introduced into piping analysis to verify design adequacy or provide a procedure whereby all motor valves $\leq$4 in. in diameter be externally supported. Seismic testing results supplied on motor operators do not demonstrate functional adequacy for Ginna.

TABLE 10 (Continued).

	Equipment item	Conclusion and recommendation
8.	Steam Generators	Insufficient information was provided to evaluate seismic design adequacy and reach a definitive conclusion. However, assuming that the stress summary provided is accurate and limiting the seismic design is adequate.
9.	Reactor Coolant Pumps	Same as Item 8.
10.	Pressurizer	Same as Item 8.
11.	Control Rod Drive Mechanism	Same as Item 8.
12.	Reactor Coolant System Supports	O.K. for SSE acting alone.
13.	Battery Racks	Racks O.K. with the exception of wooden lateral bracing, which should be replaced or strengthened to carry full seismic inertia loads.
14.	Motor Control Center Designated 1L and 1M	O.K., assuming no resonant natural frequencies below 5 Hz.
15.	Switchgear	O.K., assuming no resonant natural frequencies between 15 and 30 Hz. If such resonances exist, additional analysis is required.
16.	Tranformers (CVTs)	O.K.
17.	Control Room Electrical Panels	No evaluation has been performed since no drawings or design calculations are currently available.
18.	Electrical Cable Raceways	Same as Item 17.

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APPENDIX A: EQUIPMENT SELECTED AND CRITERIA FOR SELECTION

This appendix lists specific pieces of mechanical and electrical equipment necessary to ensure the integrity of the reactor coolant pressure boundary (RCPB) and to shut down the reactor safely and maintain it in a safe shutdown condition during and after a postulated seismic event. RG&E provided the criteria that it considered appropriate for evaluating the seismic classification of Ginna structures, systems, and components.[A-1] These criteria, which are listed below, reflect plant-specific requirements, not the more general light water reactor standards now in effect. Tables of the mechanical and electrical equipment, which are based on tables supplied by RG&E as a revision to Ref. A-1, follow the list of criteria.

CRITERIA FOR SELECTION

1. Seismic classification will be restricted to those structures, systems, and components required for safe shutdown, and to maintain RCPB integrity, and to prevent other design basis accidents which could potentially result in offsite exposures comparable to the guideline exposures of 10CFR100. These latter systems and components include, for example, the steam, feedwater, and blowdown piping up to first isolation valve, and the spent fuel pool, including fuel racks. (Also included, though not explicitly defined in the list of seismically classified equipment, are all structures, systems, and components not required to function, but whose failure could irreversibly prevent the functioning of required safe shutdown equipment or cause a design basis accident.) Seismic design of these items will ensure a very low probability of failure in the event of an SSE.

   System boundaries, for purposes of seismic reevaluation will be considered to terminate at the first normally closed, auto-close, or remote-manual valve in connected piping.

2. Safe shutdown is defined as the capability to control residual heat removal under all plant conditions resulting from a seismic event (with the consequential loss of function of nonseismic equipment) and a loss of off-site power. Safe shutdown may be the maintenance of an extended hot shutdown condition, or a gradual cooldown to cold shutdown conditions. For Ginna, safe shutdown assumes gradual cooldown and depresurization in the event of an SSE. Safe shutdown would be in the following manner:

a. Maintenance of the pressurizer level to ensure continued natural circulation capability (this does not require use of the pressurizer heaters. Loss of heaters would result in a net heat loss through the pressurizer insulation, resulting in a slow natural depressurization). Reactor coolant system (RCS) subcooling would be maintained through use of the auxiliary feedwater (AFW) system and atmospheric dump valves.

b. Auxiliary feedwater control to the steam generators (on-off pump control) and depressurization by local manual control of the atmospheric dump valves.

c. Inventory and reactivity control by use of the charging pumps taking suction from the refueling water storage tank (RWST). Letdown is to be isolated. Boric acid concentration is such that the reactor is maintained subcritical with the most reactive rod stuck out of the core. Charging is via the reactor pump seal integrity while the RCS is above 450°F. Below 450°F, the seals do not require either component cooling water (CCW) or seal injection.

d. RCS depressurization is accomplished by shrinking of reactor coolant due to contraction cooldown, then filling the pressurizer. This would condense the steam in the pressurizer by mixing with the relatively cooler RCS water entering the pressurizer.

e. Service water (SW) is the seismic source for the auxiliary feedwater system (credit is taken for the standby AFW system only. Sufficient inventory is available in the steam generators following a loss of off-site power to maintain the RCS within safe limits until the operator can align the SW system to the suction of the standby AFW pumps (the time for this is conservatively estimated at 10 minutes.).

f. Eventually, when the RCS is depressurized and cooled to about 350°F and 360 psig, the residual heat removal system (RHRS) can be placed in operation. Containment access may be necessary to manually open a RHRS suction valve (Valve 700 or 701) and an RHRS discharge valve (Valve 720 or 721) in the event of a single failure (such as a failed valve or power supply). Containment access is considered acceptable. There is no need to expedite this operation, since the AFW source is essentially infinite, heat loss from the pressurizer is very gradual, and residual heat can continue to be removed by operation of the steam generators.

g. Component cooling water must be established to the RHRS cooler and the RHRS pump oil coolers concurrent with this operation.

3. Although categorized as seismic Class I in the original plant design, the chemical and volume control system (CVCS) is not required for safe shutdown except as noted in 2.c. above, and, therefore, most portions need not be classified as seismic Category I for purposes of seismic reevaluation.

4. As noted in 1. above, systems required only for accident mitigation, such as the safety injection, containment spray, containment isolation, containment purge, HEPA and charcoal filter systems are not classified as seismic Category I for purposes of seismic reevaluation.

5. The only component cooling water functions required, per 2.g. above, are cooling the RHRS heat exchangers and pumps. All other CCW functions will not be classified as seismic Category I for purposes of

seismic reevaluation (such as cooling the seal injection and charging pumps, core injection valves, etc.). However, system integrity must be maintained to the extent needed to perform the safety function.

6. The waste disposal system, though largely considered seismic Class I in the original plant design, will not be classified as seismic Category I for purposes of seismic reevaluation because of the relatively low potential consequences that could result from failures in the system.

7. Ventilation systems for safe shutdown equipment rooms are designated seismic Class I in the FSAR. However, for purposes of seismic reevaluation, no ventilation systems in the plant except the standby AFW building coolers are designated seismic Category I. Other cooling systems, such as for safety equipment rooms, the control room, and the battery rooms, are not needed immediately since the rooms will heat up slowly. It is considered acceptable to provide for the use of portable air conditioning/ventilation units to effect cooling of safe shutdown equipment in the time required. Containment access might be required only for a short time; protective clothing and breathing apparatus could be worn by personnel for the time required.

8. The FSAR has designated portions of the fire protection system as seismic Class I. However, for purposes of seismic reevaluation, and consistent with Appendix A to Ref. A-2 and with Ref. A-3, no credit for seismic design of the fire protection system is needed or claimed.

The above criteria are presently under review by the NRC SEP Branch (under Topics III-1 and "Safe Shutdown").

Table A-1 lists the mechanical equipment items that should be classified seismic Category I consistent with the above criteria. Figure A-1 identifies the area where each item is located.

SEISMIC QUALIFICATION OF CLASS IE ELECTRICAL EQUIPMENT

RG&E supplied the following information on electrical equipment at Ginna:[A-1]

> At the time that the Ginna Nuclear Plant was designed and constructed no industry framework of standards for seismic qualification existed. Certain critical components were required by specification to be capable of withstanding the maximum seismic loads postulated for the plant site as shown in Table 1-1 (A-2 in this appendix). Most components in the Class IE electric power distribution system are designed to withstand forces due to electrical faults, which are much larger than the inertial forces due to a severe seismic event. In addition, many components used at Ginna have undergone subsequent seismic testing or analysis using inputs equal to or greater than those postulated for their location.
>
> It has been the policy of RG&E Engineering, when making modifications at Ginna Station, to require seismic qualification in accordance with the current standard when possible. In practice this means that when major Class IE components, which are independently anchored to Category I structures, are designed and procured, it is done in accordance with the current seismic standard.
>
> This has resulted in an evolution of seismic qualifcation in Ginna electrical equipment to increasingly severe standards including IEEE 344-1975.

Table A-2 shows the major components in the Class IE electrical system and the basis for seismic qualification.

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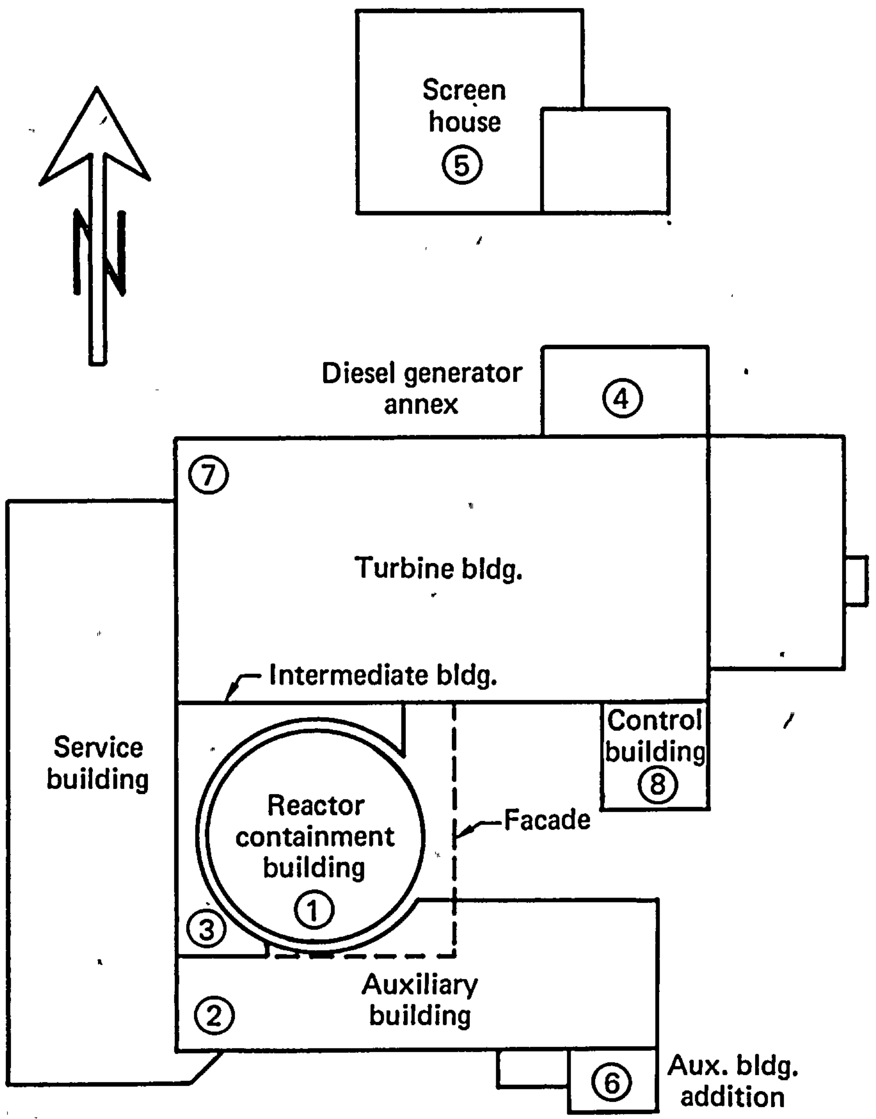

FIG. A-1. Schematic plan view of the major Ginna structures shows the locations of equipment items in Table A-1.

Table A-1. Mechanical equipment necessary to ensure the integrity of the reactor coolant pressure boundary and to shut down the reactor safely and maintain it in a safe shutdown condition during and after a postulated seismic event, from a supplement to Ref. A-1.

System	Equipment description		Description	Location		Function	Seismic input level, g		Qualification references
	Name and number	Manufacturer and model no.		Area	El., ft		OBE	SSE	
Charging	LCV 112B	Continental (Butterfly) w/ASCO Solenoid LBX 831616	4 in.	2[a]	236	Suction from RWST			
	Pump 1A, 1B, 1C	AJAX	60 gpm	2	236	Inventory and reactivity control		0.52/0.52[b]	E-Spec 676370, Rev. 0.
	286	(Globe) Rockwell	2 in.	2		Manual valve used to isolate normal charging		3.0/2.0	E-676241, Rev. 1
	323	Rockwell P-58	2 in.	2		Manual valve used to isolate alternate charging		3.0/2.0	E-676241, Rev. 1
	348B	(Grinnell)	2 in.	2		Manual valve used to isolate unneeded portion			
	356	(Grinnell)	1 in.	2		Manual valve used to isolate unneeded portion			
	RWST	Pittsburgh-Des Moines Steel Co.	335,000 gal 26.5 ft ϕ 81 ft high	2	236	Suction source for charging pumps	0.08/0.08	0.2/0.2	Pittsburgh-Des Moines Steel Co. No. 56063-02 2/26/78
	Pulsation dampener			2		Integrity only			
	Seal injection filters	Commercial filter dwg. #		2		Integrity only		0.6/0.6	E-Spec 676436, Rev. 1
	AOV427	Copes-Vulcan 2-1D58-D with operator D-100-160	2 in.	1	236	Integrity only			
	Regen. heat exchanger	Sentry Equipment		2	236	Integrity only		0.52/0.52	E-Spec 676224 (ASME III)
	Letdown orifices		2 in.	1	236	Integrity only	0.08	0.20	GSM-3
	AOV 200 A	Rockwell-Edward Valve Copes-Vulcan Operator ASCO Solenoid LBX-831616	2 in.	1	236	Isolation letdown		3.0/2.0	E-676270. Also GSM-3

[a]Area designated by number on Fig. A-1.

[b]Horizonatal/vertical.

Table A-1. Continued

System	Equipment description		Description	Location		Function	Seismic input level, g		Qualification references
	Name and number	Manufacturer and model no.		Area	El., ft		OBE	SSE	
Charging	AOV 200 B	Rockwell-Edward Valve Copes-Vulcan Operator ASCO Solenoid LBX-831616	2 in.	1	236.	Isolation letdown		3.0/2.0	E-676270. Also GSM-3
	AOV 202	Rockwell-Edward Valve Copes-Vulcan Operator ASCO Solenoid LBX-831616	2 in.	1	236	Isolation letdown		3.0/2.0	E-676270. Also GSM-3
	AOV 310	Rockwell-Edward Valve Copes-Vulcan Operator ASCO Solenoid LBX-831616	3/4 in.	1	236	Integrity (fail closed)		3.0/2.0	E-676270.
RHR	Pump A, B	Pump: Pacific SVC-6L Motor: Westinghouse 445 TS TBDP, Class B Insul.	2500 gpm	2	219	RHR suction from A hot leg		0.52/0.52	E-676228, Rev. 0
	Heat Exch. A, B	Jos. Oat. Dwg. 4807, Rev. 4	24 MBTU/hr	2	236	RHR suction from A hot leg		0.52/0.52	E-676228
	MOV 700	Velan Valve Limitorque SMB-1-25	10 in. 2260 lb	1	236	RHR suction from A hot leg		3.0/2.0	E-676258, Rev. 2
	MOV 701	Velan Valve Limitorque SMB-1-25	10 in. 2260 lb	1	236	RHR suction from A hot leg		3.0/2.0	E-676258, Rev. 2
	HCV 624	Continental Valve Copes-Vulcan Operator Fisher Actuator	8 in.	2	236	RHR discharge to B cold leg		3.0/2.0	E-676270
	HCV 625	Continental Valve Copes-Vulcan Operator Fisher Actuator	8 in.	2	236	RHR discharge to B cold leg		3.0/2.0	E-676270
	MOV 720	Velan Valve Limitorque SMB-1-25	10 in. 2260 lb	1	246	RHR discharge to B cold leg		3.0/2.0	E-676258, Rev. 2
	MOV 721	Velan Valve Limitorque SMB-1-25	10 in. 2260 lb	1	236	RHR discharge to B cold leg		3.0/2.0	E-676258, Rev. 2
	MOV 850A	Darling Valve Limitorque SMB-00 Reliance Motor	10 in.	2	230	Integrity only		3.0/2.0	E-676258, Rev. 2
	MOV 850B	Limitorque SMB-00 Reliance Motor	10 in.	2	230	Integrity only		3.0/2.0	E-676258, Rev. 2

Table A-1. Continued

System	Equipment description		Description	Location		Function	Seismic input level, g		Qualification references
	Name and number	Manufacturer and model no.		Area	El., ft		OBE	SSE	
	MOV 704A	Darling Valve Limitorque SMB-1 Peerless Motor	10 in.	2	226.4	Integrity only		3.0/2.0	E-676258, Rev. 2
	MOV 704B	Darling Valve Limitorque SMB-1 Peerless Motor	10 in.	2	226.4	Integrity only		3.0/2.0	E-676258, Rev. 2
	MOV 857A	Limitorque SMB-00 Reliance, Class B Insulation	6 in.	2	236	Integrity only		3.0/2.0	E-676258, Rev. 2
	MOV 857B	Limitorque SMB-00 Reliance, Class B Insulation	6 in.	2	236	Integrity only		3.0/2.0	E-676258, Rev. 2
	FCV 626	Continental Valve Copes-Vulcan Operator Fisher Actuator	8 in.	2		Integrity only		3.0/2.0	E-676270
	MOV 852A & MOV 852B	Velan Valve Limitorque SMB-1 Reliance, Class B Insul.	6 in. 1300 lb	1	236	Integrity only		3.0/2.0	E-676258, Rev. 2
	HCV 133	Copes-Vulcan	2 in.	1		Integrity only			E-676270. Also GSM-3
Component cooling	Pump 1, 2	Ingersoll-Rand 8SD Motor: Westinghouse 444TS TBDP, Class B Insulation	3000 gpm	2	270	Cold shutdown cooling		0.52/0.52	E-676370, Rev. 0
	CCW Heat Exch. A, B	Atlas, Dwg. B-1260	25 MBTU/hr	2	281.5	Cold shutdown cooling		0.52/0.52	E-676454
	Surge Tank	Minotte Man. Corp.	2000 gal	2	281.5	Maintain CCW pressure boundary			
	MOV 817	Crane Valve Limitorque SMB-000 Peerless Motor	8 in.	2		Isolate CCW to RCP, reactor supports, excess letdown heat exch.		3.0/2.0	E-676258, Rev. 2
	MOV 738A	Crane Valve Limitorque SMB-000 Reliance Motor	10 in.	2		CCW to RHR heat exch.		3.0/2.0	E-676258, Rev. 2

Table A-1. Continued

System	Equipment description: Name and number	Equipment description: Manufacturer and model no.	Description	Location: Area	Location: El., ft	Function	Seismic input level, g: OBE	Seismic input level, g: SSE	Qualification references
	MOV 738B	Crane Valve Limitorque SMB-000 Reliance Motor	10 in.	2		CCW to RHR heat exch.		3.0/2.0	E-676258, Rev. 2
	772A	Velan T-36	3 in.	2		Isolate CCW from sample heat exchangers		3.0/2.0	E-676241, Rev. 1
	772B	Velan T-36	3 in.	2		Isolate CCW from sample heat exchangers		3.0/2.0	E-676241, Rev. 1
Service water	Pump 1A, 1B 1C, 1D	Worthington 20H-500-WZ Motor: Westinghouse 509-UPH ABDP, Class F Insul.	5300 gpm	5	253	Safe shutdown cooling		0.52/0.52	E-Spec 626370
	MOV 4615	Chapman List 155 Limitorque SMB-2-60 Peerless Motor	20 in.	2	253	Isolation valves between two loops			
	MOV 4616	Chapman List 155 Limitorque SMB-2-60 Peerless Motor	20 in.	2	253	Isolation valves between two loops			
	MOV 4734	Rockwell Valve Limitorque SMB-00-2	14 in., 590 lb	2	271	To CC heat exch.			
	MOV 4735	Rockwell Valve Limitorque SMB-00-2	18 in., 890 lb	2	271	To CC heat exch.			
	MOV 4670	Crane 47-1/2 x R Limitorque SMB-0-25 Peerless Motor	10 in.	4	236	Isolation to ensure diesel cooling			
	MOV 4664	Crane 47-1/2 x R Limitorque SMB-0-25 Reliance Motor	10 in.	3	236	Isolation between seismic/nonseismic portions			
	MOV 4663	Crane 101 x U Limitorque SMB-000-5 Reliance Motor		3	236	Isolate air conditioner water chiller from required seismic portion			
	4651A 4652A 4651, 4652	Fisher 657-ES Crane 143-1/2 x R	4 in. 4 in. 120 lb	3		Isolate discharge of water chiller. Integrity only			

Table A-1. Continued

System	Equipment description: Name and number	Equipment description: Manufacturer and model no.	Description	Location: Area	Location: El., ft	Function	Seismic input level, g: OBE	Seismic input level, g: SSE	Qualification references
	MOV 4609	Crane Valve, Reliance Motor	14 in.	5	253	Isolate supply to travelling screens			
	Containment Fan Coolers (coils)	Sturtevant	8 in.	1	253	Integrity only	0.25/0.25	0.64/0.64	SP-5342, RO-2328
Service water	Reactor Compartment Cooler (coils)		2.5 in.	1	235	Integrity only	0.08/0.08	0.20/0.20	SP-5342
	Spent fuel pool heat exch.	Westinghouse	5.3 MBTU/hr 7600 lb			Cool spent fuel		0.52/0.52	E-Spe 676228
Main Steam	AOV 3516	Valve: Atwood Morrill Operator: Chicago Fluid A31 Solenoids: Laurence 110114W, 125434W	30 in. 7400 lb	3	278	Main steam isolation valves			
	AOV 3517	Valve: Atwood Morrill Operator: Chicago Fluid A31 Solenoids: Laurence 110114W, 125434W	30 in. 7400 lb	3	278	Main steam isolation valves			
	3410	Fisher Governor Co. Type 476AA (Spec. D)	6 in.	3	278	Power operated relief valve used for cooldown by manual operation of handwheel			
	3411	Fisher Governor Co. Type 476AA (Spec. D)	6 in.	3	278	Power operated relief valve used for cooldown by manual operation of handwheel			
	MOV 3504	Rockwell 604NY	6 in.	3	278	Isolation from TDAFP			
	MOV 3505	Rockwell 604NY	6 in.	3	278	Isolation from TDAFP			
	3508-3515	Crosby 6R-10, HC-65	6 in.	3	278	Main steam safeties			

Table A-1. Continued

System	Equipment description: Name and number	Equipment description: Manufacturer and model no.	Description	Location: Area	Location: El., ft	Function	Seismic input level, g: OBE	Seismic input level, g: SSE	Qualification references
Feed-water	AOV 5737	Masoneilan 38-20521AB Solenoid: ASCO 2300A56R	2 in. 167 lb	3	235	Blowdown isolation			
	AOV 5738	Masoneilan 38-20521AB Solenoid: ASCO 2300A56R	2 in. 167 lb	3	235	Blowdown isolation			
Feed-water	AOV 5735	Masoneilan 38-20521AB Solenoid: ASCO 2300A56R	3/8 in.	3	235	Blowdown sample isolation			
	AOV 5736	Masoneilan 38-20521AB Solenoid: ASCO 2300A56R	3/8 in.	3	235	Blowdown sample isolation			
Standby aux. feed-water system	Standby Aux. Feedpump 1C, 1D	Ingersoll Rand 2HM7A-9 w/GE Motor Type R Class B Insulation	200 gpm	6	271	Aux. feed to S.G. 1A, 1B	0.08 ground	0.20	Standby AFW system seismic analysis is referenced in 10/6/78 letter. See also Ref. B-4
	MOV 9629A,B		4 in.	6	271	Supply SAFW system	0.08 ground	0.20	
	MOV 9701A,B	Fisher Control Valve Limitorque SMB-005	3 in.	6	271	AFW discharge	0.08 ground	0.20	
	MOV 9704A,B	Edwards Univalve Limitorque SMB-000-10	3 in.			Discharge flow control (stop check valve)	0.08 ground	0.20	
	MOV 9703A,B	Edwards Univalve Limitorque SMB-00-10	3 in.			Cross tie	0.08 ground	0.20	
	AOV 9710A,B	Fisher Valve	1 in.			Recirculation valves (integrity only)	0.08 ground	0.20	
	AOV 9732A,B	Fisher Valve	1.5 in.			Service water to cooling unit	0.08 ground	0.20	

Table A-1. Continued

System	Equipment description: Name and number	Equipment description: Manufacturer and model no.	Description	Location: Area	Location: El., ft	Function	Seismic input level, g: OBE	Seismic input level, g: SSE	Qualification references
	Cooling system					Maintain proper environment	0.08 ground	0.20	
Diesel generator auxiliary systems	Fuel Oil (FO) Storage Tank	Buffalo Tank ASTM A-283 Grade C	8000 gal	4	Buried	Storage for diesel oil			
	FO Transfer Pumps	DeLaval Model A3133AD-187 Westinghouse Motor Model TEFC	24.3 gpm 0.96 BHP	4	236	Pump fuel oil from storage tank to day tank			
Diesel generator auxiliary systems	FO Day Tank	Integral w/diesel	350 gal approx.	2	236	Store fuel oil	0.19/0.19	0.47/0.47	GAI RO 2239 seismic criteria
	Piping & Valves	ASCO Valve ASCO Solenoid HT 82107	-	4	236	Transfer of fuel oil to diesel			
	Lube Oil (LO) Heat Exchanger	American Standard Model 1205-6	16350 btu/min	4	236	Lube oil cooling	0.19/0.19	0.47/0.47	GAI RO 2239 seismic criteria
	LO Prelube Pump	Worthington Model 2-GAUFTM	10 gpm at 50 psi	4	236	Circulate oil	0.19/0.19	0.47/0.47	GAI RO 2239 seismic criteria
	Air Inlet Silencer	Kittel-Model ABRK20-st		4		Diesel air inlet silencing	0.19/0.19	0.47/0.47	GAI RO 2239 seismic criteria
	Exhaust Gas Silencer	Kittel-Model TI-20		4		Diesel exhaust silencing	0.19/0.19	0.47/0.47	GAI RO 2239 seismic criteria
Standby aux. feedwater system	AOV 1710B					Integrity only			Standby AFW system seismic analysis is referenced in 10/6/78 letter. See also Ref. B-4
	Cooling system					Function			Standby AFW system seismic analysis is referenced in 10/6/78 letter. See also Ref. B-4
Diesel fuel oil (FO)	FO storage tank	Buffalo Tank ASTM A-283 Grade C	8000 gal	Buried in diesel-building vicinity		Storage tank (2) for No. 2 diesel oil			

Table A-1. Continued

System	Equipment description: Name and number	Equipment description: Manufacturer and model no.	Description	Location: Area	Location: El., ft	Function	Seismic input level, g: OBE	Seismic input level, g: SSE	Qualification references
	FO transfer pumps	DeLaval Model A3133AD-187	24.3 gpm 0.96 BHP	-		Transfer pump from FO storage tank to day tank			
	FO day tank		350 gal approx.	Diesel generator building - diesel subbase		Storage of FO at engine	0.19/0.19	0.47/0.47	GAI RO 2239 seismic criteria
	Piping and valves			Storage to day tank buried - day tank in diesel subbase					
Diesel lube oil (LO)	LO heat exchanger	American Standard Model 1205-6	16350 btu/min			LO cooling	0.19/0.19	0.47/0.47	GAI RO 2239 seismic criteria
	LO prelube pump	Worthington Model 2-GAUFTM	10 gpm at 50 psi	Mounted on diesel base			0.19/0.19	0.47/0.47	GAI RO 2239 seismic criteria
Diesel combustion air intake & exhaust	Air inlet silencer	Kittel-Model ABRK20-st		Diesel generator building		Diesel air inlet silencing	0.19/0.19	0.47/0.47	GAI RO 2239 seismic criteria
	Exhaust gas silencer	Kittel-Model TI-20		Diesel generator build-		Diesel exhaust gas silencing	0.19/0.19	0.47/0.47	GAI RO 2239 seismic criteria

TABLE A-2. The major components in the Class IE electrical system and the basis for seismic qualification.

System/component	Basis for seismic qualification
I EMERGENCY POWER SYSTEM	
A. Low Voltage (600 V) Switchgear (excluding unit transformer) (Westinghouse DB 15, 25, 50 & 75 Breakers).	Post-construction testing.
B. Motor Control Centers (Westinghouse type W)	Post-construction testing and analyses in accordance with IEEE 344-1971. Upgraded by analysis to IEEE 344-1975.
C. MOV Operators (ac/dc)	Post-construction testing.
D. Vital 120 VAC	
1. Distribution panels 1A & 1C 2. Inverters (Solidstate Controls Inc.) 3. Const voltage xformers	Post-construction testing. Installed in 1978, qualified by test in accordance with IEEE 344-1975.
E. 125 VDC Power System	
1. 125 V, 60-cell batteries (Gould) and racks 2. Battery chargers	Design specification; 0.52 g simultaneous horizontal and vertical. Currently under review.
F. Diesel Generators (Alco/Westinghouse)	Design specification; 0.47 g simultaneous horizontal and vertical acceleration.
G. Reactor Bldg. Cable Penetrations (Crouse-Hinds)	Post-construction testing.
H. Conduit Supports & Tray Supports	Currently under review.
I. Electrical Equipment Anchors	Currently under review.

TABLE A-2. Continued.

System/component	Basis for seismic qualification
II. SAFEGUARDS INSTRUMENTATION AND CONTROL	
1.0 Transmitters (Barton, Foxboro)	Post-construction testing.
2.0 Reactor Trip Switchgear (DB 50)	Post-construction testing.
3.0 Main Control Board (Wolf and Mann)	Design specification; 0.52 g simultaneous horizontal and vertical acceleration.
4.0 Reactor Trip System Racks (A/D conversion)	Design specification; 0.52 g simultaneous horizontal and vertical acceleration.
5.0 Protective Relay Racks. (SI and reactor trip logic)	Design specification; 0.52 g simultaneous horizontal and vertical acceleration.
6.0 Safeguards Racks (ESF actuation output)	Design specification; 0.52 g simultaneous horizontal and vertical acceleration.
7.0 Control Switches (Westinghouse type W2 and OT2)	Post-construction testing.
8.0 Under Voltage Relay Cabinet (Westinghouse CV-7 & MG-6)	Currently under review.
9.0 Safety Related Indication	Currently under review.

APPENDIX B: GENERAL BASIS OF SEP REEVALUATION OF STRUCTURES AND EQUIPMENT*

B.1 GENERAL APPROACH TO REEVALUATION

The seismic reevaluation part of the SEP centers on:

- Assessment of the general integrity of the reactor coolant pressure boundary.
- Evaluation of the capability of essential structures, systems, and components required to shut down the reactor safely and maintain it in a safe shutdown condition, including removal of residual heat, during and after a postulated seismic disturbance, which in this case is the Safe Shutdown Earthquake (SSE).

To accomplish this level of reevaluation, it is necessary to assess the factors of safety of essential structures, components, and systems of the older plant relative to those designed under current standards, criteria, and procedures. Such evaluation should help to define the nature and extent of retrofitting, if any, required or possible to make these plants acceptable, if they are not already at such levels.

As used in the previous paragraph, the term "relative" is not to be construed as evaluation based on the norm of current criteria, standards, and procedures, but, instead, in the light of knowledge that led to such a level of design. It would be irrational to assume that an older plant would consist of structures, equipment, components, and systems that would meet current criteria in every instance; even so, those items that do not meet current criteria may be entirely adequate in the the sense of meeting acceptable safety and reliability criteria.

Within the scope of the investigation, it was impossible to reexamine every item in detail. On the other hand, by examining structures, equipment, components, and systems individually, it was felt it would be possible to assess their adequacy and general margin of safety for meeting the selected SSE hazard. Thereafter, on the basis of evaluation of the structures, items of equipment, or systems, as appropriate, it should be possible to provide:

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*This appendix is essentially Chapter 3 of the Dresden 2 report (Ref. 1 of this report).

- Judgmental assessment of the adequacy of the existing plant to function properly during and following the SSE hazard, including judgmental assessment of the overall margin of safety with regard to seismic resistance.
- Specific comments pertaining to upgrading or retrofitting as may be appropriate.

The detailed basis of the reevaluation approach to be followed generally is presented in Refs. B-1 and B-2. The specific bases of reevaluation are described next.

B.2 GEOLOGY, SEISMICITY, AND SITE CONDITIONS

The seismicity information forms the basis for arriving at the effective peak transient ground motions (acceleration, velocity, and displacement) for use in arriving at response spectra, time histories, etc. in the reevaluation. Thus, one important initial basis of reevaluation is a comparison of the original basic seismic design criteria and those selected for reevaluation.

B.3 STRUCTURES

The first task in examining structures is to summarize the nature and makeup of the structures that are to be examined in the light of knowledge about original design criteria and information on the as-constructed plant. Also required is a summary of design analysis approaches employed, including loading combinations, stress and deformation criteria, and controlling response calculations. In evaluating the seismic design criteria, generally it is necessary to have information concerning the seismic input employed originally, the applicable levels of damping, and the modeling approach used in the analyses. Also needed are details of input and methods of analysis used in designing mechanical equipment, piping, and electrical system supports.

Thereafter, with the seismic criteria applicable to the reevaluation known, and with knowledge of other normal loading criteria deemed necessary, it is possible to estimate the response to the seismic excitation. In some cases it may be necessary to carry out new seismic analyses with the original model or new models as deemed appropriate.

The final basis of evaluation will involve consideration of many factors, including the following items.

B.3.1 Response Spectra, Damping, and Nonlinear Behavior

One basis of evaluation will be comparison of the original response spectra and the response spectra applicable to the reevaluation along with appropriate damping values and ductility factors. The damping values specified in R.G. 1.61 (Ref. B-3) and those recommended in NUREG/CR-0098 (Ref. B-1) for reevaluation purposes are summarized in Table B-1.

TABLE B-1. Damping values from R.G. 1.61 compared to those recommended for the SEP evaluation.

	Damping (% of critical damping)	
	R.G. 1.61 (SSE)	NUREG/CR-0098 (recommended)
Reinforced concrete	7	7 to 10
Prestressed concrete	5	5 to 7
Welded assemblies	4	5 to 7
Bolted and riveted assemblies	7	10 to 15
Piping	2 or 3	2 to 3

The reason for permitting higher damping values is discussed in Ref. B-1. Although there are limited data on which to base damping values, it is known that the R.G. 1.61 values are conservative to ensure that adequate dynamic response values are obtained for design purposes. The lower values in the NUREG/CR-0098 column of values in Table B-1 in most cases are close to the R. G. 1.61 values. The upper values in the NUREG/CR-0098 column are bestestimate values believed to be average or slightly above average values; it is recommended that these values be used in design or evaluation for stresses at or near yield, and when moderately conservative estimates are made of the other parameters entering into the design or evaluation.

As for ductility factors, it is recommended (Ref. B-1) that low values (1.3 to 2) be used for conservatism and to help ensure that no gross deformation occurs in any critical safety elements, which, in turn, ensures that system ductility is maintained at a low value. System ductility, which arises from deformation of a number of interconnected elements, may be slightly larger by virtue of the interacting deformation of the connected elements. An assessment of the local element deformation and its role in system performance requires careful evaluation and is largely judgmental in assessing the factor of safety.

Ductility factors in excess of 1 should not be permitted in active equipment unless it can be clearly demonstrated that functional ability is not impaired and a significant margin of strength still remains. In passive mechanical and electrical equipment and distribution systems, component ductility limits should conservatively range between 3 and 5.

B.3.2 Analysis Models

Another basis of the reevaluation involves consideration of the adequacy of representation of the original analysis models, along with assessment of the possible effects of such factors as soil-structure interaction, overturning, and torsion. Analysis procedures used in the reevaluation should be in keeping with the state of the art.

B.3.3 Normal, Seismic, and Accident Loadings

Those loading combinations of particular importance in the reevaluation process involve the usual combinations incorporating normal loadings (dead load, live load, pressure, temperature, etc., as appropriate) with seismic loadings. Design basis accident load effects were not considered; however, one criterion examined was that the reactor coolant pressure boundary be maintained to preclude an earthquake-initiated loss-of-coolant accident.

B.3.4 Forces, Stresses, and Deformations

A significant aspect of the reevaluation involves assessment of the reasonableness of the forces (axial and shear forces, and moments) and associated stresses and deformations used in the original design and their adequacy in the light of the seismic criteria applicable to the reevaluation. Such studies involve consideration of effects arising from horizontal and vertical excitation and take into account the proportion of total effects attributed to seismic factors. Also, the amount of limited nonlinear behavior that is to be accommodated is evaluated as may be appropriate.

B.3.5 Relative Motions

The effect of any gross relative motions that might influence piping entering buildings, or spanning between buildings, tilt, or interaction effects is taken into account as a part of the reevaluation.

B.4 EQUIPMENT AND DISTRIBUTION SYSTEMS

Of particular importance in the reevaluation process is the assessment of the adequacy of critical mechanical and electrical equipment, and fluid- and electrical-distribution systems. The reevaluation centers on those items or systems essential to meeting the general criteria described earlier.

A major task of the reevaluation process is to identify the critical safety related systems and the criteria originally used for procurement and seismic qualification of equipment. For such systems selected, representative items or systems were identified on the basis of:

- Physical inspection of the facility (where specific items were identified as appearing possibly to have nearly lower bound seismic resistance).
- Representative sampling.

After system or item identification, and after ascertaining the nature of the seismic criteria used during procurement or qualification, the reevaluation effort involves a detailed assessment of the original design in the light of current knowledge about equipment vulnerability to seismic excitation. Specifically, the evaluation involves consideration of the following items.

B.4.1 Seismic Qualification Procedures

The initial reevaluation assessment is concerned with the original seismic qualification of the equipment item or system, in terms of the seismic test performance (level and extent of testing), or analyses that may have been made, or both.

B.4.2 Seismic Criteria

The second major aspect of reassessment involves comparison of the original seismic design criteria with those currently applicable. This area of assessment involves consideration of such items as the in-structure response spectra, dynamic coupling, and damping.

B.4.3 Forces, Stresses, and Deformations

For those items of equipment for which loads, stresses, or deformations may be a major factor in design and performance, the reevaluation involves:

- Examination of the original loading combinations and analyses.
- Calculation or estimation of the situation that exists under the reevaluation criteria. Particular attention is directed to the effect of any increase in seismic component of load, stress, or deformation.

B.5 MISCELLANEOUS ITEMS

In a subsequent step of the reevaluation, it may be appropriate to evaluate such items as sources of water for emergency core cooling and to assess whether or not any potential problems could occur with regard to dams, intake structures, cooling water piping, etc.

B.6 EVALUATION OF ADEQUACY

On the basis of the reevaluation assessments made as a part of the foregoing studies, an overall evaluation of the adequacy of the critical structures and representative equipment items and systems is made. Such an evaluation takes into account judgmental or factual assessment of the margin of safety, as the case may be, and consideration of the adequacy of individual items in a system in terms of overall system performance.

B.7 CITED REFERENCES

B-1. N. M. Newmark and W. J. Hall, Development of Criteria for Seismic Review of Selected Nuclear Power Plants, U.S. Nuclear Regulatory Commission, NUREG/CR-0098 (1978).*

B-2. T. A. Nelson, Seismic Analysis Methods for the Systematic Evaluation Program, Lawrence Livermore National Laboratory, Livermore, CA, UCRL-52528 (1978).

B-3. U.S. Nuclear Regulatory Commission, Damping Values for Seismic Design of Nuclear Power Plants, Washington, D. C., Regulatory Guide 1.61 (1973).

*Available for purchase from the National Technical Information Service, Springfield, VA 22161.

APPENDIX C

STRUCTURAL AND MODELING DETAILS OF THE INTERCONNECTED BUILDING COMPLEX

AUXILIARY BUILDING

Construction Details

The auxiliary building is a three-story rectangular structure, 70 ft, 9 in. by 214 ft, 5 in. It is located south of the containment and intermediate buildings and adjacent to the service building (See Figs. 1 and 2 of the report). The structure has a concrete basement floor that rests on a sandstone foundation at El. 235 ft, 8 in., and two concrete floors--an intermediate floor at El. 253 ft and an operating floor at El. 271 ft. The floors have a minimum thickness of 1.5 ft, and are supported by 2.5-ft-thick concrete walls at the south, east, and part of the north sides of the building. The northwest corner of the building is adjacent to the circular wall of the containment building. The west concrete wall, which separates the auxiliary building and the spent fuel storage pit, is 6 ft thick.

The spent fuel storage pit is a rectangular swimming-pool-type concrete structure. Its bottom is at El. 236 ft, 8 in. Walls are 6-ft thick at the north and west sides and 3-ft thick at the east and south sides, which are below the ground surface and also serve as retaining walls.

The auxiliary building has two roofs constructed of steel truss and bracing systems and supported by frame bracing systems. The high roof (El. 328 ft) covers the west part of the operating floor and the spent fuel storage pit. The low roof (El. 312 ft) covers the east part of the operating floor. Insulated siding is used for the wall above the operating floor.

A platform that supports a component cooling surge tank and a heat exchanger rises from the operating floor to El. 281.5 ft. The platform is supported by columns and bracings. There are also a number of 2.5 to 3.5 ft thick concrete shield walls on the floors.

Model Details

The elevations and masses of the five rigid diaphragms in the auxiliary building are given in Table C-1.

TABLE C-1. Elevations and masses of the five rigid diaphragms in the auxiliary building.

	Elevation, ft	Translational mass, kip-s^2/ft	Torsional mass moment of inertia, kip-ft-s^2/rad
High roof	328.0	19.24	33793.9
Low roof	312.0	10.56	9581.8
Operating floor	270.0	400.93	1022560.0
Intermediate floor	252.0	371.43	947319.0
Platform	283.83	1.53	

The translational mass data and center of gravity locations were based on information supplied by Gilbert Associates, Inc. (Ref. C-1). The torsional mass moments of inertia were calculated assuming uniformly distributed mass. The spent fuel pit was assumed to be rigid and was not included in the model. The properties and locations of the two equivalent beams that represent the two stories of the concrete structure are the same as those in Ref. C-1.

INTERMEDIATE BUILDING

Construction Details

The intermediate building, which encloses the cylindrical containment building, is north of the auxiliary building and is connected to the part of the auxiliary building that is under the high roof.

The building is a 136 ft, 7 in. by 140 ft, 11 in. steel frame structure with facade structures on each side. The facade structures are steel frame bracing systems covered with shadowall aluminum sidings (see Figs. 3 and 4 of report). The concrete basement floor slab (El. 253.5 ft) is supported by a set of 2 ft, 10 in. square concrete columns and a concrete retaining wall on the west side. The columns have individual concrete footings embedded in the rock foundation. The top elevations of the footings vary from 238 ft to 236.5 ft.

In the north part of the building, there are three floors at Els. 278.33 ft, 298.33 ft, and 315.33 ft, and a high roof at El. 335.5 ft. In the south part of the building there are two floors at Els. 271 ft and 293 ft, and the low roof at El. 318 ft. All floors are made of composite steel girders and 5-in.-thick concrete slabs (see Fig. 3 of report). Built around the circular containment building, the floors extend completely through the west side of the intermediate building, a major portion of the north side and a small portion of the south side. There are no floors on the east side. The roofs are supported by steel roof girders. The floors and roofs are also supported vertically on a set of interior steel columns which are continuous from the basement floor to the roof. Concrete block walls surround all the floor space between the basement floor and the roofs.

The top of the four facade structures is at El. 387 ft. There is no roof at the top, only a horizontal truss connecting the four sides to provide out-of-plane strength (Fig. 4 of text). One special characteristic of the west facade is that the horizontal floor or roof girders are connected not to the bracing joints but somewhere between joints. In such a design, the columns must transform significant shears and moments when the structure is subject to lateral loads.

Model Details

The elevations and masses of the seven rigid diaphragms in the intermediate building are given in Table C-2.

The translational mass values for the E-W and N-S directions at Els. 298.33 ft and 293 ft differ because only the E-W mass of the service building roof is lumped to the intermediate floors. The N-S roof mass is distributed to four nodal joints of the west facade structure at El. 287.33 ft. In the N-S direction, These four nodal joints are further assumed to have the same motion because of the additional horizontal girders connecting the four joints on the service building side of the facade.

In calculating the mass inertia, the floor or roof dead weight is assumed to be uniformly distributed over the diaphragm. The concrete block walls and

TABLE C-2. Elevations and masses of the five rigid diaphragms in the intermediate building.

	Elevation, ft	Translational mass, $kip\text{-}s^2/ft$		Torsional mass moment of inertia, $kip\text{-}ft\text{-}s^2/rad$
		E-W	N-S	
North	278.33	20.01	20.01	41531.6
floors	298.33	19.64	27.80	36832.4
	315.33	18.38	18.38	37365.3
High roof	335.33	7.63	7.63	14839.2
South	271	10.91	10.91	8159.1
floors	293	14.81	12.97	9420.1
Low roof	318	4.84	4.84	3419.0

equipment masses are computed from their actual locations. Other important modeling details are as follows.

- The concrete basement slab (El. 253.5 ft) is assumed to be rigid. The sub-basement and the retaining wall at the west end of the intermediate building are not included in the model.
- Unlike most of the other columns, four columns of the west facade have fixed base supports.
- The horizontal truss structure that acts as an inner stiffener for the facade structures is modeled by an additional set of continuous horizontal beams.
- The floor at El. 271 ft is attached to the concrete wall of the spent fuel pit at the south end. It is assumed to be rigid in the model.

TURBINE BUILDING

Construction Details

The turbine building is a 257.5 ft by 124.5 ft rectangular building on the north side of the building complex. It has a concrete basement at El. 253.5 ft, two concrete floors (a mezzanine floor at El. 271 ft and an operating floor at El. 289.5 ft. The roof includes a roof truss structure from El. 342.66 ft to El. 257 ft composed of top and bottom chords connected by vertical bracing. The roof and floors are supported by steel framing and bracing systems on all four sides of the building. The floors are also supported by additional interior framing at various locations under the floors.

Part of the south wall frame also serves as the north wall of the intermediate building. The north facade structure (from El. 357 ft to El. 387 ft) is actually on the top of the south frame of the turbine building. The west frame is the continuation of the west facade structure of the intermediate building. This west frame is also part of the service building. Except between buildings, the walls of the turbine building have insulated aluminum siding.

Inside the building and parallel to the south and north frames, there is an interior frame system supporting the crane from the basement elevation to El. 330 ft. The crane frame is designed like the exterior frame system with vertical columns, horizontal beams, and cross bracing bolted to columns. Each interior column is welded to the corresponding exterior column at the joints and mid-points of columns by a series of girder connections.

The south frame of the turbine building is designed like the west facade structure of the intermediate building; that is, horizontal floor girders are connected to columns somewhere between joints.

Model Details

The elevations and masses of the three rigid diaphragms in the turbine building are given in Table C-3.

TABLE C-3. Elevations and masses of the three rigid diaphragms in the turbine building.

	Elevation, ft	Translational mass, kip-s^2/ft	Torsional mass moment of inertia, kip-ft-s^2/rad
Mezzanine floor	270.0	207.91	1437610.0
Operating floor	288.5	168.16	1489340.0
Operating roof	342.67	19.91	135744.0

The dead weight masses were calculated based on the uniform mass data supplied in Ref. C-2. Equipment masses were based on estimated weights and actual locations.

The turbine building model includes steel frames up to El. 342.66 ft. The roof truss structure above this elevation is modeled by a single rigid diaphragm at this elevation.

The pressurization walls (see below) between column lines 3 and 5 on both the north and south sides were represented by a set of cross bracing systems with the lateral stiffnesses equal to the shear stiffnesses of the walls. In calculating the shear stiffness of the pressurization wall, the frame and stiffener of the wall were ignored; only the 1/4-in. armor plate was considered.

The inside crane frame system was not included in the model because it was not considered to have strong enough connections to transmit forces to the exterior frame.

CONTROL BUILDING

Construction Details

Located adjacent to the south side of the turbine building, the control building is a 41 ft, 11-3/4 in. by 54 ft, 1-3/4 in. three-story structure with concrete foundation mat at El. 253 ft. The common wall is reinforced with 1/4-in. armor plate, stiffeners, and siding to form a pressurization wall or "super wall" . The other three sides have reinforced concrete walls, and the roof is also reinforced concrete. The control room floor at El. 289.75 ft and the relay room floor at El. 271 ft are 6-in.-thick reinforced concrete slabs supported by steel girders that are tied to turbine building floors at the respective elevations. The basement is the battery room.

Model Details

The control building model is adapted from Ref. C-2. Three equivalent sticks that are located at the centers of rigidity for each story represent the control building. However, only the roof and basement floor masses are retained as master nodes. The relay room and control room floors are considered to be rigidly attached to the mezzanine and operating floors of the turbine building, and their masses are, therefore, transformed to the corresponding master nodes of the turbine building. No rotational inertia is included because of the relatively small floor sizes. The lumped masses and their elevations are given in Table C-4.

TABLE C-4. Lumped masses and elevations for the three-stick control building model.

	Elevation, ft	Translational mass, $kip\text{-}s^2/ft$
Basement	250	47.58
Relay room floor	270	29.49
Control room floor	290	26.57
Roof	309	30.21

Six elastic springs were attached at the basement to simulate the soil effect. The soil spring constants for the foundation were taken from Ref. C-2:

Lateral translations	2342.9×10^3 kip/ft
Vertical translations	6355.2×10^3 kip/ft
Rocking about E-W	1361572×10^3 kip-ft/rad
Rocking about N-S	3192889×10^3 kip-ft/rad
Torsion	2591911×10^3 kip-ft/rad

DIESEL GENERATOR BUILDING

Construction Details

The diesel generator building is a one-story reinforced concrete structure that has two cable vaults underneath the floor. The south wall, which is common with the turbine building, is reinforced to be a pressurization wall or "super wall" like the one described above. The building roof is a reinforced concrete slab supported by four shear walls that sit on concrete spread footings.

Model Details

The four shear walls of the diesel generator building are represented by four elastic springs attached to the north frame of the turbine building at the diesel generator building roof. The first three shear walls from the east end of the diesel generator building have an equivalent stiffness of 355.6×10^3 kip/ft in the N-S direction at El. 270 ft. The stiffness of the fourth wall, which is lower than the other three, is 467.9×10^3 kip/ft.

SERVICE BUILDING

The service building is located on the west side of the building complex. It extends from the south end of the auxiliary building, through the intermediate building, and ends a little before the north end of the turbine building. The building is a two-story steel structure with spread footings, steel columns, and concrete-steel framing floors and roof. The basement is at

El. 253.66 ft, the floor is at El. 271 ft, and the roof is at El. 287.33 ft. The walls between the service building and the other buildings as well as the partitions in the building are made of concrete blocks.

Model Details

Half of the service building mass was lumped to the intermediate building floors (Els. 293 and 298.3 ft) in the E-W direction, and to the west facade structure in N-S direction. The rest is lumped to the turbine building floor (El. 271 ft) in both directions.

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C-2. L. D. White, Jr., Rochester Gas and Electric Corporation, Rochester, N. Y., Letter Report to D. L. Ziemann, U. S. Nuclear Regulatory Commission (May 22, 1979).

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16. ABSTRACT *(200 words or less)*

A limited seismic reassessment of the Robert E. Ginna Nuclear Power Plant was performed by the Lawrence Livermore National Laboratory (LLNL) for the U. S. Nuclear Regulatory Commission (NRC) as part of the Systematic Evaluation Program. The reassessment focused generally on the reactor coolant pressure boundary and on those systems and components necessary to shut down the reactor safely and to maintain it in a safe shutdown condition following a postulated earthquake characterized by a peak horizontal ground acceleration of 0.2 g. Unlike a comprehensive design analysis, the reassessment was limited to structures and components deemed representative of generic classes. Conclusions and recommendations about the ability to selected structures and equipment to withstand the postulated earthquake are presented.

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