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Seismic Review of the Millstone 1 Nuclear Power Plant as Part of the Systematic Evaluation Program

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

Prepared for
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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 Millstone 1 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 report reflects a collective effort on the part of the following persons:

- T. A. Nelson and R. C. Murray, (Lawrence Livermore National Laboratory (LLNL)), who provided project management support and compiled the report.
- C. Y. Liaw, (EG&G/San Ramon Operations), who conducted the seismic reevaluation of major structures.
- D. A. Wesley, (Structural Mechanics Associates, Inc.), who conducted the seismic reevaluation of ancillary structures.
- J. D. Stevenson, (Structural Mechanics Associates, Inc.), who conducted the seismic reevaluation of mechanical and electrical equipment and of the fluid and electrical distribution systems.

This limited assessment of the Millstone 1 facility relied in large part upon the guidance, procedures, and recommendations of recognized seismic

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*The licensee has proposed deferring SEP review of Dresden 1. The facility is currently shutdown and will not restart until 1986.

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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 (Deceased, January 25, 1981)
Nathan M. Newmark Consulting Engineering Services
Urbana, IL

William J. Hall
William J. Hall Consulting Engineering Services
Urbana, IL

Robert P. Kennedy (Donald A. Wesley, Alternate)
Structural Mechanics Associates, Inc.
Newport Beach, CA

John D. Stevenson (Frank A. Thomas, Alternate)
Structural Mechanics Associates, Inc.
Cleveland, OH

Robert C. Murray (member since October 1, 1980)
Lawrence Livermore National Laboratory
Livermore, CA

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.

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The SSRT developed its general philosophy and presented it in the first SEP report, which reviews Unit 2 of the Dresden Nuclear Power Station (Ref. 1). The limited assessment of Millstone 1 reported here is the fifth in the series of SEP seismic reviews of Group 1 plants.

This report provides partial input into the SEP seismic evaluation of the Millstone 1 Nuclear Power Plant. The results of the seismic evaluation will be documented in a Safety Evaluation Report, prepared by the NRC staff, that will address the capability of the Millstone 1 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 its 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 and W. T. Russell, Chief, SEP Branch, at the NRC, for their continuing support. Thanks also go to C. A. Meier of LLNL and D. Vaughan of EG&G/San Ramon Operations for publications support.

The authors especially wish to acknowledge the accomplishments of Dr. Nathan M. Newmark (Deceased January 25, 1981), Chairman of the SSRT, for his guidance and direction during the course of this study. He will be deeply missed by all who worked with him.

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ABSTRACT

A limited seismic reassessment of the Millstone 1 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 (SEP). 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, equipment, and piping to withstand the postulated earthquake are presented.

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CONTENTS

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LIST OF FIGURES

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DRAFT

LIST OF TABLES

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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 Millstone 1 Nuclear Power Plant. This limited reassessment includes a review of the original seismic design of selected structures, equipment, and components and includes 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 eleven** older operating nuclear reactors, including Millstone 1. The primary objective of the SEP seismic review program is to make a seismic safety assessment of the plants based on a limited sample of structures, systems, and components and, where necessary, to 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 Millstone 1 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 it can be demonstrated that such backfitting will provide substantial, additional protection for public health and safety. Adequate demonstration requires an assessment of broad safety issues by considering the interactions of various systems in the context of overall plant safety.

**The licensee has proposed deferring SEP review of Dresden 1. The facility is currently shutdown and will not restart until 1986.

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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 Millstone 1 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 perform its safety function and to maintain a safe shutdown condition.

This seismic reevaluation of Millstone 1 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 to 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 other such safety-related systems as the Emergency Core Cooling System.

Not all equipment was examined as part of this reassessment. The intent was to examine mechanical and electrical equipment representative of items installed in the reactor coolant system and safe shutdown systems at the Millstone 1 facility for structural integrity and electrical and mechanical functional operability. Components that potentially have a high degree of seismic fragility were selected for review in order to estimate the lower-bound seismic capacity in generic classes of equipment. The selection was made during a site visit by representatives of the NRC, the SSRT, LLNL, and its subcontractors. The methods of selecting representative equipment for this limited assessment are described in detail in Chapter 6.

Structures housing the selected systems were analyzed to demonstrate structural adequacy and to generate seismic input to equipment. The major structures reviewed for the Millstone 1 plant include the reactor building, with its related internal structures, turbine building, and radwaste/control building. 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 (Ref. 6).

The SSE is the only earthquake level considered in the review 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 one-half of the SSE, controlling the design of structures, systems, and components for which operation, rather than 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.

To ensure safety in a seismic evaluation, certain elements and components of an entire system must continue to function under normal operating and test loads both during and following an earthquake. The seismic review team did not review all aspects of the plant's operation, nor did they review the safety margins available to assure that vital elements and components would withstand unusual operating conditions, sabotage, operator error, or other nonseismic events.

This report addresses structures, systems, and components in the as-built condition, including modifications made to all seismic Category I components since the issuance of the operating license. Information about structures, systems, and components was primarily obtained from the Millstone 1 docket (Docket 50245) 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.

1.2 PLANT DESCRIPTION

Jointly owned and operated by the Connecticut Light and Power Company, the Hartford Electric Light Company, Western Massachusetts Electric Company, and the Millstone Light Company, the Millstone 1 plant (Fig. 1) is located on

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the Millstone Point peninsula which projects from southeastern Connecticut into Long Island Sound. Unit 1 is a Mark 1 boiling water reactor, commonly designated as a BWR. The plant was designed to produce a maximum output of 2011 MW of heat and 652 MW of net electrical power.

General Electric Company, the prime contractor for the plant, engaged Ebasco Services, Incorporated, as their architect-engineer. Most seismic analyses were conducted by John A. Blume and Associates, Engineers.

The plant has been in operation since 1970 under Provisional Operating License No. DPR-21. Application for a full-term operating license is under consideration.

1.2.1 Seismic Categorization

Using Appendix A of the Safety Analysis Report (Ref. 7) as a guide, the plant equipment and structures were categorized into one of two seismic classes as follows:

Class 1 structures and equipment are those whose failure could cause significant release of radioactivity or which are vital to a proper shutdown of the plant and the removal of decay heat. These include:

Structures

- Drywell, Vents, Suppression Chamber (Torus), and Penetrations
- Reactor Building
- Control Room (and supporting part of Radwaste Building)
- Gas Turbine Building
- Ventilation Stack
- Radwaste Building
- Intake and Discharge Structure

Equipment

- Nuclear Steam Supply Systems (NSSS)
- Reactor Vessel
- Reactor Vessel Supports
- Control Rods and Drive System (including equipment necessary for scram operation)
- Control Rod Drive Thimble Supports
- Fuel Assemblies
- Core Shroud
- Core Supports

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Steam Dryer
Recirculating Piping System including valves and pumps*
All piping connections from the Reactor Vessel up to and including the first isolation valve external to the drywell.
Isolation Valves
Condensor (hot well only)

Reactor Emergency Systems
Isolation Condenser System
Standby Liquid Control System
Core Spray System
Core Reflooding System
Reactor Building Closed Loop Cooling
Containment Spray System
Service Water System

Standby Gas Treatment System

Fuel Storage Facilities including spent fuel and new fuel storage equipment

Electrical Power Systems
Standby Diesel Generator
Station Battery
Diesel Generator
Emergency Busses and other electrical gear that supply power to critical equipment, including startup transformer

Instrumentation and Controls
Reactor Pressure and Level Instrumentation
Feedwater Control Instrumentation
Standby Liquid Control System Instrumentation
Manual Reactor Control System
Control Rod Instrumentation
Control Rod Position Indicating System
Reactor Protection System
Neutron Monitor System
In-Core Neutron Monitors
Area Monitors
Turbine Building Secondary Cooling Water System

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*Piping was here considered as one type of equipment.

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Class 2 structures and equipment are those that are not essential to a proper shutdown but are related to the operation of the station. These include:

Structures

- Turbine Building
- Service Building
- Office Building
- Radioactive Waste Building (that portion not Class I)

Equipment

- Turbine Generator
- Cranes
- Shutdown Cooling System
- Reactor Cleanup System
- Waste Disposal System
- Turbine Moisture Separators
- Air Compressors and Receivers
- All other Piping and Equipment not listed under Class 1

Note that these classifications differ from those in Regulatory Guide 1.29, (Ref. 8) which was issued after the design of Millstone 1 was completed.

1.2.2 Principal Structures

The primary structures of the Millstone 1 plant are shown in Fig. 1 and identified in Fig. 2. The reactor building (Fig. 3) is a reinforced concrete structure that houses the reactor and its auxiliary systems. Its square base measures 142 ft 6 in along each side. The reactor vessel and the recirculation system are contained inside the drywell of a pressure suppression containment system. The primary containment system consists of the drywell, vent pipes, and a pool of water contained in the suppression chamber. The reactor building encloses the primary containment system, thereby providing a second containment. In addition, all refueling equipment is inside the building, including the spent fuel storage pool and the new fuel storage vault.

The reactor service and refueling area is served by an overhead bridge crane designed to handle the reactor vessel head, the steam separators and dryers, the drywell head, the drywell shielding blocks, and the spent fuel shipping casks during refueling and maintenance. A refueling service platform

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with necessary handling and grappling fixtures serves the refueling area and spent fuel storage pool. A passenger-freight elevator provides access to the various floor levels of the building.

The turbine building is a reinforced concrete structure founded on rock. The base of the building is approximately 178 ft by 300 ft. This building houses the turbine-generator and the associated equipment for generating electric power. It also houses emergency cooling components.

The radwaste building is a reinforced concrete structure also founded on rock. Located north of and adjacent to the reactor building, it includes equipment and tankage space below grade and houses the plant control room above grade. The area below grade is of reinforced concrete walls with a steel framed roof. The control room has reinforced concrete walls and a steel-frame roof.

The gas turbine building is a reinforced concrete structure on top of piles driven to rock. The roof is a concrete slab, supported on steel beams. The gas turbine flood protection requirements are met by placing the gas turbine and the control equipment at elevation 17 ft and providing the building access doors with flood gates to elevation 19 ft.

The stack is an unlined, axi-symmetric reinforced concrete structure which is free-standing. Foundation supporting media information was not given. The stack was modeled as a flexible cantilever system with the base fixed at the foundation surface. Forty lumped masses were connected by beam members in the model.

1.3 ORGANIZATION OF REPORT

This report contains six chapters. Chapter 2 summarizes our assessment of the ability of Millstone 1 to resist the stipulated SSE event. The chapter also identifies potential deficiencies and areas that may require further study.

Chapter 3 describes the general basis for reevaluation of structures and equipment.

Chapter 4 summarizes the original facility's seismic design criteria for structures, equipment, and piping. The chapter also includes a summary of the original calculated seismic response and acceptance criteria.

Chapter 5 compares the seismic loadings and responses for which the facility structures were originally designed with corresponding seismic loadings and responses derived using techniques thought to be more realistic in light of current knowledge.

Chapter 6 contains an evaluation of the capability of mechanical and electrical equipment, and fluid- and electrical-distribution system to resist seismic loads and to perform their necessary functions. Evaluations are based on the floor input generated in Chapter 5, along with other available information.

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CHAPTER 2: SUMMARY AND CONCLUSIONS

Within the limited scope of this reevaluation, we examined typical structures, equipment, components, and systems:

- To assess the adequacy of the existing plant to perform necessary safety functions during and following an SSE.
- To qualitatively judge the overall factor of safety with regard to seismic resistance.
- To make specific recommendations on upgrading or retrofitting, as appropriate.

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

Structural reassessment results are reported for the reactor building and drywell, the radwaste/control building, the ventilation stack, the turbine building, the condensate storage tank, a typical buried pipe, a typical buried tank, the suppression chamber, ring header, torus, and support system, and the gas turbine building.

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

A. Reassessment loads are less than original design loads. Here we assumed that the structures were designed and constructed adequately to resist the design seismic loads.

B. Stress evaluations indicate that combined static and dynamic stresses (including seismic) do not exceed SEP acceptance limits.

C. For cases in which structural stresses exceed yield, the estimated reserve capacity of the structures would permit inelastic deformations without failure.

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2.1.1 Reactor Building and Drywell

The reactor building was originally analyzed for a Taft time history record with 0.07 g SSE, then checked for 0.17 g SSE. The reassessment was based on a 0.2 g SSE which resulted in higher shear forces and overturning moments. Results indicated that the highest stressed elements in the reactor building were the exterior shear walls and the reactor shield walls. The predicted stresses in the shear walls were below the calculated capacities. Therefore, it is considered that the structure has sufficient strength to resist the 0.2 g SSE loads.

The reassessment seismic moments and shears in the drywell were larger than the original values. However, the totals of the predicted static stresses plus seismic stresses were within the current code allowables, and the drywell is considered acceptable for the 0.2 g SSE.

2.1.2 Radwaste/Control Building

The reanalysis indicated that the walls of the above grade structure housing the control room have sufficient strength to resist the 0.2 g SSE loads. Results indicated that the highest stressed elements in this building are in the stairway structure which is independent of the control room portion of the building.

2.1.3 Ventilation Stack

The stack was seismically reassessed since collapse of the stack might damage the nearby control building and the essential equipment within. The seismic (SSE) moments in the stack predicted by the reassessment were larger than the original analysis values. While elastic analysis indicated that a significant portion of the stack might develop cracks along the cross section, the stresses from the combined static and SSE load conditions remained within the current code allowables. The adequacy of the connection of the piles to the footing should be verified to ensure that the 35 kips tension per pile can be safely resisted.

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2.1.4 Turbine Building

Subsequent to the original design of the turbine building, the diagonal bracing of its lateral load resisting system was reinforced in the north-south direction. The reassessment analysis is based on this modified system. Results indicated that the stresses in the frames and bracing were generally below yield. Although the diagonal bracing in one end wall resisting east-west loading as stressed at yield, the rigid frames in this direction should have enough reserve capacity to allow the structure to resist a 0.2 g SSE load. However, the bracing should be investigated for:

- non-ductile failure modes; and
- sufficient strength in the connections to develop yield stress of the bracing members.

2.1.5 Condensate Storage Tank

The reanalysis of the condensate storage tank indicated that anchor bolt pullout would not be expected during an SSE if the embedment length meets minimum building code requirements. However, details of the embedment were not available for review. Although code allowable stresses are exceeded in the bolt thread area, failure of the bolts is not expected. The tank wall is sufficiently thick to prevent buckling. Frictional resistance between the tank and the underlying footings was high enough to preclude any relative displacement between the tank and the ground. Therefore, the above critical elements of the condensate storage tank are considered adequate to withstand the 0.2 g SSE.

However, it is considered likely that without the presence of the anchor bolt chairs as originally specified, failure of the welds at the intersection of the bottom plate and the cylindrical shell should be expected for the 0.2 g SSE. Although a double fillet weld is specified, the bottom plate is quite thin and large local bending moments as well as uplift tension forces must be resisted. It is recommended that the tank be modified to include the originally specified anchor chairs or an equivalent capacity restraint.

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2.1.6 Underground Piping

In the reanalysis, stresses were typically compared with ASME Code allowables. These comparisons indicated that the maximum pipe stress due to seismic wave propagation without the effects of discontinuities or end point motion is small. In addition, the stresses resulting from abrupt directional change of buried pipe are expected to be well below the allowable stress level. Strains induced by relative motion where pipe enters a building may cause stresses above yield; however, failure is not expected. The critical buried pipelines are therefore concluded to be adequate to resist the 0.2 g SSE.

2.1.7 Buried Tank

No analysis of buried tanks was coducted during the design of Millstone 1. As part of the reassessment, a typical underground tank (the gas turbine fuel oil storage tank) was analyzed for the 0.2 SSE. The maximum principal stress in the tank was estimated to be well below the ASME allowable; therefore, it is concluded that the buried tanks will sufficiently resist the 0.2 g SSE load.

2.1.8 Suppression Chamber - Ring Header - Torus, and Support System

A reevaluation to determine the seismic capacity of principal components of the suppression chamber was conducted using the fundamental frequency predicted by the original design model. Seismic response accelerations were determined in accordance with SEP guidelines. SSE induced end plate bearing stresses, pin shear stresses, and tensile stresses in the tie rods are within acceptable limits. If the ends of the rods do not have upset threads, the minimum yield strength may be exceeded in the threaded region. Inelastic deformations in this region may be acceptable provided non-ductile failure modes do not exist. The configuration of the tie-rod ends needs to be determined. The predicted seismic stresses in the columns and torus are expected to be small compared to the stresses resulting from deadweight and the postulated loss-of-coolant accident (LOCA). Therefore, we recommend that the adequacy of the torus and column system be based on an evaluation which includes both the pool dynamic loads due to LOCA and seismic loads. The

IMAGE EVALUATION
TEST TARGET (MT-3)

6″

IMAGE EVALUATION
TEST TARGET (MT-3)

6"

IMAGE EVALUATION
TEST TARGET (MT-3)

6"

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seismic loads used for this evaluation should be those resulting from an analysis performed in accordance with current seismic criteria.

2.1.9 Gas Turbine Building

The gas turbine building is a reinforced concrete structure on top of piles which are driven to competent rock. The analysis of the building was conducted using the response spectrum from the 1952 Taft time history record. Only the critical (E-W) direction results were reported, although the analysis was performed in two directions. Based on a comparison with Regulatory Guide 1.60 spectra, increases in the response of up to approximately 40 percent could be expected if no frequency shift or amplification resulting from overburden flexibility is considered. No details of the soil characteristics were available to assess these effects, and it is unlikely that these effects will be of sufficient magnitude to result in structure damage to the extent that the critical system's functionality would be impaired. It is recommended, however, that the effects of the structure foundation flexibility should be considered when considering the seismic input for these equipment systems.

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

As discussed in Chapter 6, typical mechanical and electrical equipment, components, and distribution systems were selected for review by an SEP review team composed of the authors plus SSRT and NRC staff members. The review was largely based on the judgment and experience of team members. There is wide variation in the documentation of original specifications applied to procurement of equipment, as well as current qualification standards for equipment. While some qualifications for an item of equipment are quite specific, others are generic and apply to a class of equipment rather than specific items at Millstone 1.

Because we lacked essential seismic design and qualification data on the Millstone 1 plant, our review of the seismic design adequacy of mechanical and electrical equipment is incomplete. Additional tests and analyses which

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demonstrate functionality of active components must be developed before definite conclusions can be drawn. In many cases the minimum design details needed to review the design and make appropriate calculations were unavailable. Therefore, we were unable to confirm the capability of a number of mechanical and electrical components to withstand the 0.22 g SSE without loss of structural integrity and required safety function. A summary of the qualification status of various electrical and mechanical components follows. Chapter 6 contains detailed discussion of the adequacy of these items.

- Emergency service water pump: bolting must be revised; no design details available to determine functional integrity; if cast iron material was used, it must be replaced.
- Emergency condenser: OK.
- Shutdown heat exchanger: OK.
- Emergency cooling water heat exchanger: provide additional longitudinal restraint or conduct further analysis to determine the adequacy of the restrainer without the restraint.
- Recirculation pump support: OK for structural integrity; further data needed to evaluate functional integrity and to determine seismic loads on pump snubber supports.
- Emergency diesel oil storage day tank: OK.
- Motor operated valves: functional adequacy not demonstrated; further analysis needed to verify design adequacy on lines 4 inches or less.
- CRD hydraulic control system: insufficient data available; no evaluation made.
- Reactor vessel, supports, and internals: OK.
- Battery racks: an additional longitudinal restraint is required; further analysis is needed.
- Motor control centers: OK, contingent upon demonstration of design adequacy of block wall to which supports are attached; no information on function.
- Transformers: no evaluations; design details unavailable.
- Switchgear panels: no evaluations; design details unavailable.
- Control room electrical panels: no evaluations; design details unavailable.

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- Diesel generator remote control boards: OK contingent upon demonstration of design adequacy of block wall to which supports are attached; no information on function.
- Battery room distribution panels: OK contingent upon demonstration of design adequacy of block wall to which supports are attached; no information on function.
- Electrical cable raceways: no evaluation made; however, it is recommended that lateral restraint be provided unless design adequacy is demonstrated.

2.3 PIPING

Piping calculations were performed for the SEP evaluation by EG&G/Idaho and are summarized in Ref. 9. Portions of the feedwater, shutdown cooling, condensate transfer, and diesel oil piping have been analyzed using independently developed finite element models. Original analysis methods as described in Ref. 9 were simulated and new analyses incorporating current ASME Code and Regulatory Guide requirements have been performed and comparisons made. It was assumed for all the following piping systems that a suitable stress analysis of the supports, substructures and anchor nozzles was performed for the original loads. A reanalysis of these items was beyond the scope of this effort and was not performed. Since no original load data were provided, no comparisons of support or anchor loads could be made. In addition, no conclusions regarding structural adequacy of supports or anchors could be drawn for any of the subject piping systems. Major conclusions from the comparison studies include:

- Feedwater Piping: From the analyses performed on the feedwater model it can be concluded that stresses in the piping will not be excessive during a seismic event. Adjacent piping and/or structure should be checked to ascertain that the three inch deflections indicated by the "current criter " analysis do not cause unanticipated impact loads.

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- Shutdown Cooling Piping: The piping stresses will exceed allowable limits at two locations during an a postulated seismic event. These high stressed areas are at the junction of the eight inch and fourteen inch piping and are primarily caused by insufficient lateral restraint of the eight inch piping included in the model. Adjacent piping and/or structures should be checked to ascertain that the two inch deflections indicated by the "current criteria" analysis do not cause unanticipated impact loads.

- Condensate Transfer Piping: The results of the analyses performed on the subject portion of the condensate transfer piping indicate that stresses in this piping will not exceed allowable limits during a seismic event. As previously recommended, clearance should be checked near areas of high deflection to ascertain that impacts will not occur.

- Diesel Oil Line: The piping stresses for the diesel oil piping model were well within allowable limits for an SSE event. However, the presence of the assumed spacers between the oil line and the jacket piping should be verified so that the possibility of impact loading between the two pipes may be assessed more fully.

The spectra used to analyze the feedwater and shutdown cooling piping were generated from reactor building floor motions at an elevation near these systems. After completion of these analyses, new spectra were generated inside the drywell. In the new spectra, the accelerations were reduced in the high frequency regions; however, the general conclusions drawn for these systems remain the same. Both spectra are included in Appendix A.

2.4 CONCLUDING REMARKS

Based on the combined experience and judgment of the authors and the SSRT, reviews of the original design analyses, and comparisons with similar items of equipment and components in more recently designed nuclear power plant facilities, we conclude that:

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- Structures and structural elements of the Millstone 1 facility are adequate to resist an earthquake with a peak horizontal ground acceleration of 0.2 g, provided additional analysis, design modification, or verification as stated above are conducted.
- While much of the equipment proved to be adeequate for seismic loading, some items needed to be modified and insufficient data was available for other items. Therefore, no definitive statement can be made about the overall seismic design adequacy of mechanical and electrical equipment.

We therefore recommend:

- As discussed in section 2.1, the following actions are necessary to demonstrate seismic adequacy of structures:
  - Provide additional restraint to the bottom of the condensate storage tank by adding the originally specified anchor bolt chairs or a restraint of equivalent capacity. If the original bolts are used in this tank, verify their embedment length. Outlet piping details should be examined to ensure integrity.
  - Evaluate the tie-rod ends and columns of the torus for SSE loading and evaluate SSE and LOCA loading.
  - Assess the forthcoming results of gas turbine building analysis considering foundation flexibility.
- That modifications and/or additional analyses be made as necessary to the mechanical and electrical equipment items listed in Sec. 2.2 in order to demonstrate seismic design adequacy.
- Modifications to piping or additional analysis be made as listed in Sec. 2.3.
- That 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 tests or analysis employing design procedures (load, stress and deformation limits, materials, fabrication procedures, and quality acceptance) in accordance with a recognized structural code. In addition, the equipment inside the cabinets and panels should be checked for seismic design adequacy.

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- That 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 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 either be modified or moved so that they no longer jeopardize the plant.

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CHAPTER 3: GENERAL BASIS OF SEP REEVALUATION OF STRUCTURES AND EQUIPMENT

3.1 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 needed to shut down the reactor safely and maintain it in a safe shutdown condition during and after a postulated Safe Shutdown Earthquake (SSE). This includes the capability to remove residual heat after the earthquake.

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 define the nature and extent of retrofitting, if any, required to make these plants acceptable if they are not already at acceptable 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 unreasonable 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 sense of meeting the spirit of current criteria.

Within the scope of the investigation, it was impossible to reexamine every item in detail. On the other hand, by examining selected 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

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structures, items of equipment, or systems, as appropriate, it should be possible to provide:

- 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. 4 and 5. The specific bases of reevaluation are described next.

3.2 SEISMIC INPUT AND SITE CONDITIONS

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. The original regional and site geologic and seismic information used to establish seismic input parameters has been reviewed in light of current knowledge. A final comparison can be made between the site specific seismic input and the seismic input assumed in this evaluation when the data become available. Preliminary results from the SEP Site Specific Spectra Program (Ref. 10) indicate that the predicted SSE peak ground acceleration (PGA) is larger than that used in the original analysis (0.17 g). Therefore, a PGA of 0.2 g, along with R.G. 1.60 spectra, was used for reassessment analyses reported herein. As mentioned in Ref. 11, the specified OBE was less than half the original SSE. It should be verified that items for which the design is normally controlled by the OBE, possess adequate margins of safety to ensure that they are appropriately conservative for SSE loading.

3.3 STRUCTURES

In examining a structure, the first task is to summarize the nature and makeup of the structure in the light of both the original design criteria and information on the as-constructed plant. Also required is a summary of the

DRAFT

design analysis approaches that were used, including loading combinations, stress and deformation criteria, and controlling response calculations. For evaluating the seismic design criteria, it is generally necessary to know the seismic input employed originally. the applicable levels of damping, and the modeling approach used in the analyses.

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

Overall reevaluation of a structure will involve consideration of many factors, including those discussed in the following subsections.

3.3.1 Response Spectra, Damping, and Nonlinear Behavior

Reevaluation of a structure will include comparison of original response spectra to [illegible]alysis response spectra, along with appropriate damping values and [illegible]ctors. Table 1 compares the damping values specified in R.G. 1.61 (Ref. 12) with those recommended in NUREG/CR-0098 (Ref. 4) for reevaluation purposes.

TABLE 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 when stresses are close to yield)
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 for the reassessment is discussed in Ref. 4. Although there are limited data on which to base damping values, it is believed 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 1 are in most cases close to the R.G. 1.61 values. The upper values in the NUREG/CR-0098 column are best-estimate values believed to be average or slightly above average values. We recommend that these upper values be used in evaluations of existing facilities for stresses at or near yield, and when moderately conservative estimates are made of the other parameters entering into the evaluation.

It is recommended in Ref. 4 that low values of ductility factors (1.3 to 2) be used for conservatism and to help ensure that no gross deformation occurs in any critical safety elements. An assessment of the local element deformation and its role in system performance requires careful evaluation and is largely judgmental in evaluating safety.

3.3.2 Analysis Models

The reevaluation includes a consideration of the adequacy of the models used in the original analysis. This consideration must include an assessment of possible effects of such factors as overturning and torsion. In the reevaluation of Millstone 1, state-of-the-art analysis procedures were used wherever feasible.

3.3.3 Normal, Seismic, and Accident Loadings

In the reevaluation we considered the usual combinations of normal loadings (dead load, live load, pressure, temperature, etc., as appropriate) with seismic loadings. Design basis accident load effects were not considered. However, to preclude an earthquake-initiated loss-of-coolant accident, the reactor coolant pressure boundary was examined to make certain that under an SSE event it would remain within code prescribed behavior limits.

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3.3.4 Forces, Stresses, and Deformations

The reevaluation assessed the reasonableness of the forces (axial and shear forces, and moments) and associated stresses and deformations used in the original design along with 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.

3.3.5 Relative Motions

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

3.4 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.

3.4.1 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.

3.4.2 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, modeling, dynamic coupling, and damping.

3.4.3 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.

3.4.4 Functionality

For those items of equipment that are defined as active in R.G. 1.48, qualification testing or analysis, when performed, has been used to demonstrate the structural integrity of such equipment. Operability of such equipment has become a generic concern for all power reactors. In the interim period until the completion of this generic activity, maintenance of the structural integrity and judgments reached concerning operability of such equipment provide reasonable assurance that they will function following the occurrence of an earthquake up to and including the specified SSE.

3.4.5 Nonlinear Behavior

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

3.5 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 dams, intake structures, cooling water piping, or other items that form part of the ultimate heat sink.

3.6 EVALUATION OF ADEQUACY

Based on the reevaluation assessments as described above, an overall evaluation of the adequacy of the critical structures and representative equipment items and systems is made. Such an evaluation takes into account analytical assessment of factors of safety, as well as judgment. We also considered the adequacy of individual items as they pertain to overall system performance.

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CHAPTER 4: ORIGINAL SEISMIC DESIGN CRITERIA

4.1 INTRODUCTION

This chapter presents the original seismic desi... criteria for Millstone 1. The seismic loadings for seismic Class I structures, equipment, and piping are defined, and the seismic response and allowable stress criteria for critical structures are outlined. The data presented in this chapter define the design basis and are used to form the basis for comparison with current seismic criteria in Chapter 5. Most of the information has been drawn from the FSAR; detailed references are given, in the sections describing the individual analyses.

4.2 DESIGN EARTHQUAKE MOTION

Millstone was designed for an earthquake (equivalent to the operating basis earthquake, or OBE) with a peak horizontal ground acceleration, A_{max}, of 0.07 g (Ref. 7, Sec. XII and Appendix F). The maximum earthquake (equivalent to the safe shutdown earthquake, or SSE) was defined by scaling the OBE by 17/7, thus generating an earthquake record with an A_{max} of 0.17 g. A simultaneous vertical acceleration equal to two-thirds of the horizontal acceleration was also considered in the plant design, with the loads added absolutely where applicable.

Dynamic response-spectrum analyses were performed using the smoothed response spectrum (Fig. 4) recommended in Ref. 12. The origin of this design response spectrum is unknown. Dynamic time-history analyses were based on the first 12 seconds of an accelerogram from the July 21, 1952, Kern County earthquake as recorded in the Taft, Lincoln School Tunnel (N69W component), normalized to a maximum acceleration of 0.07 g. The response spectrum (Fig. 5) calculated from the normalized time history for 0.5% damping generally envelops the smooth response spectrum of Fig. 4 (Ref. 7, Am. 17, question VII-A.11).

DRAFT

4.3 SEISMIC ANALYSES

Table 2 is a list of the Class I systems at Millstone, showing the original seismic analysis method by which each was evaluated. In the brief descriptions of these analysis methods that follow, the emphasis will be on the general characteristics of each method. Later, in the more detailed descriptions of the individual analyses, applicable details will be noted.

TABLE 2. Seismic analysis methods used by the licensee to evaluate Class I systems.

Item	Type of analysis	Model
Structures:		
Reactor building	Time history	Lumped mass and beam
Ventilation stack	Time history	Lumped mass and beam
Radwaste building/control room	Time history	Lumped mass and beam
Condensate storage tank	Time history	Lumped mass and beam
Gas turbine building	Response spectrum	Single mass and spring
Intake structures	Equivalent static	Unknown
Drywell	Time History	Lumped mass and beam
Suppression chamber	Response spectrum	Single mass and spring
Piping and equipment:		
Recirculation loop piping	Response spectrum	Lumped mass and beam
Main steam lines and other Class I piping in reactor bldg	Equivalent static	Lumped mass and beam
Class I piping in turbine bldg	Equivalent static	Unknown
Class I instrumentation piping	Lateral deflection and force evaluation curves	None
Reactor pressure vessel	Response spectrum	Lumped mass, beam, and spring
Isolation condenser, LPCI pump, containment heat exchanger	Response spectrum	Single-degree-of-freedom[a]

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DRAFT

TABLE 2 continued.

Item	Type of analysis	Model
Class I equipment in reactor bldg	Equivalent static	Unknown
Class I equipment in turbine bldg	Equivalent static	Unknown
Batteries and racks	Equivalent static	None
Fuel racks in fuel storage pool	Response spectrum[b]	Plate and beam
Components of off-gas modifications	Unknown	Unknown
Xe-Kr equipment	Unknown	Unknown
Safety instrumentation	Unknown[c]	Unknown

[a]Further details unavailable.

[b]Boron carbide neutron-absorber plates were qualified by test.

[c]A test program was proposed for seismic qualification, but it is not known whether it was carried out.

4.3.1 Methods of Analysis

4.3.1.1 Dynamic Methods

The seismic designs of the reactor building, the ventilation stack, the drywell, and the radwaste building/control room were based on time-history analyses, using as input the record of the Kern County earthquake normalized to 0.07 g. A constant vertical acceleration of 0.05 g was also applied. Seismic responses, such as member internal forces, displacements, and accelerations, were computed for each mode at each instant of time. Modal responses were then added together directly. Little information is available on the computer programs used for the analyses. For the drywell, the size of the integration interval was given as 0.005 s.) In the analysis of the radwaste building/control room, the structural model was analyzed twice, once along each horizontal building axis. The maximum horizontal response in conjunction with the vertical response was used as the basis for design. The

DRAFT

condensate storage tank was also analyzed using the time-history method, but no details of the analysis are available.

Dynamic response-spectrum analyses, using the smoothed response spectra shown in Fig. 4, were performed on the gas turbine building, the pressure suppression chamber, the recirculation loop piping, the reactor pressure vessel, and other pieces of equipment. Vertical loads were applied simultaneously in the analyses of the gas turbine building, the torus, the piping, the reactor pressure vessel, and the fuel racks in the fuel storage pool. In the analysis of the gas turbine building and the piping, the maximum of two orthogonal horizontal analyses was used as the basis for design. For the fuel racks, analyses were performed for the three components of motion, and the responses were combined as the square root of the sum of squares.

The computer programs used for the dynamic analyses were not identified, except for the analysis of the fuel racks in the fuel storage pool, where the STARDYNE code was used.

4.3.1.2 Static-Equivalent Method

The static-equivalent method depends on seismic coefficients (in g's) to obtain static lateral forces for structural design. The forces are simply the products of the seismic coefficients and structural weights. In the analysis of Millstone 1, the coefficients for the piping and equipment in the reactor building were based on the dynamic analysis of the building itself. The absolute acceleration results from the time-history analysis were used to generate the curve shown in Fig. 6. Seismic coefficients for rigidly attached equipment or piping at a given elevation were then taken directly from the curve. The seismic coefficients for other components that were analyzed by the equivalent-static method are discussed in the sections on the individual analyses.

4.3.1.3 Lateral Deflection and Force Evaluation Curves

Design curves developed by John A. Blume and Associates were used in the analysis of Class I instrumentation piping. Pipe spans were chosen as specified in Power Piping USAS B31.1.0 so that stresses were less than

1500 psi at loads of 0.5 g. The period of each piping span, which depends on the length of the span and the size of the pipe, was then established from the Blume design curves. Each span was required to respond rigidly: its period was required to be less than half that of the supporting structure.

4.3.2 Damping

Damping values specified for the design of Millstone (Ref. 7, Sec. XII-1.7) are given in Table 3, along with damping values for various Class I items. Because the evaluation for the SSE was based on scaling the loads developed for the OBE by 17/7, the same damping values were used for both earthquakes.

A rocking mode was considered in the analysis of the reactor building, but no further note was taken of soil damping. (Most of the Class I structures have foundations supported on bedrock or on piles driven to bedrock.) No method was developed for estimating the damping of structures in which more than one structural material was used.

4.4 STRESS CRITERIA

Millstone was primarily designed for a 0.07-g OBE. Stresses resulting from this excitation, in combination with stresses imposed by nonseismic loads, were held to code-allowable levels. The allowable stresses for reinforcing steel, concrete, and structural steel in Class I structures are listed in Table 5 (after Ref. 7, Table XII-1). In addition, it was required that the plant be able to shut down safely following a 0.17-g SSE, as described in Sec. 4.2. The load combinations used in the design are listed in Table 4. At the time Millstone was designed, emphasis was placed on the less severe OBE. As a result, the safe shutdown acceptance criterion was not very specific as to allowable stresses or loads. In general, stresses for the SSE were limited to the yield level.

DRAFT

TABLE 3. Damping values used in the design of Millstone.

	% of critical damping
Structural types:	
Reinforced-concrete structures	5.0
Steel frame structures	2.0
Welded assemblies	1.0
Bolted and riveted assemblies	2.0
Vital piping systems	0.5
Class I items:	
Reactor building	5.0
Ventilation stack	5.0
Radwaste building/control room	5.0
Condensate storage tank	2.0
Condensate storage tank fluid	0.5
Gas turbine building	5.0
Recirculation loop piping	0.5
Drywell	3.0
Suppression chamber	2.0
Reactor pressure vessel	2.0
Isolation condenser, LPCI pump, and containment heat exchanger	2.0
Fuel racks in fuel storage pool	Unknown
Xe-Kr equipment structure	2.0 for OBE 5.0 for SSE

TABLE 4. Summary of original load combinations and allowable stresses.

Load combinations[a]	Design criterion
	Primary containment (including penetrations)
D + P + H + T + E	ASME, Sec. III, Class B, without the usual increase for seismic loading.
D + P + R + H + T + E	Same as above, except local yielding is permitted in the area of the jet force where the shell is backed by by concrete. In areas not backed by concrete, primary local membrane stresses at the jet force must not exceed 90% of the yield point of the material at 300^{o}F.

continued

[a]Abbreviations:

- D = Dead load of structure and equipment, plus any other permanent loads contributing to stress, such as soil, hydrostatic, or temperature loads, or operating pressures and live loads expected to be present when the plant is operating.
- P = Pressure due to loss-of-coolant accident.
- H = Force on structure due to thermal expansion of pipes under operating conditions.
- T = Thermal loads on containment due to loss-of-coolant accident.
- E = Design earthquake load (equivalent to OBE). Twenty-five percent of the live load was considered concurrent with the seismic load.
- R = Jet force or pressure on structure due to rupture of any one pipe.
- E' = Maximum earthquake load (equivalent to SSE).

TABLE 4 continued.

Load combinations	Design criterion
D + P + R + H + T + E'	Primary membrane stresses, in general, must not exceed the yield point of the material. If the total stress exceeds the yield point, an analysis must be done to determine that the energy absorption capacity exceeds the energy input from the earthquake. The same criteria as in the previous combination are applied to the effect of jet forces for this loading condition.
Reactor building and all other Class I structures	
D + R + E	Normal code-allowable stresses (AISC for structural steel, ACI for reinforced concrete). The customary increase in design stresses, when earthquake loads are considered, is not permitted.
D + R + E'	In general, stresses are limited to the minimum yield point. However, in a few cases, stresses may exceed the yield point. If so, an analysis must be done, using the limit-design approach, to ensure that the energy absorption capacity exceeds the energy input. This method is discussed in AEC TID-7024. The resulting distortion is limited to assure no loss of function and an adequate factor of safety against collapse.

continued

TABLE 4 continued.

Load combinations	Design criterion
	Reactor vessel supports
D + H + E	Normal code-allowable stresses (AISC for structural steel, ACI for reinforced concrete). The customary increase in design stresses, when earthquake loads are considered, is not allowed.
D + H + R + E	Stresses must not exceed • 150% of AISC-allowable levels for structural steel • 90% of the yield stress for reinforcing bars • 85% of the ultimate stress for concrete.
D + H + E'	No functional failure. Usually, stresses do not exceed the yield point of steel or the ultimate strength of concrete. If these limits are exceeded, energy absorption capacity must be shown to exceed the energy input from the earthquake.
	Reactor vessel internals
D + E	Stresses that result from the maximum possible combination of loadings encountered under operational conditions must not exceed the stress criteria of ASME, Sec. III, Class A Vessel.

continued

TABLE 4 continued.

Load combinations	Design criterion
D + E'	The secondary and the primary plus secondary stresses are examined rationally, taking into account elastic and plastic strains. These strains must not allow any failure by deformation that would either compromise any engineered safeguard or that would prevent safe shutdown of the reactor.
	<u>Emergency core cooling systems (ECCS)</u>
D + T + H + E	Stresses must remain within code-allowable limits: Piping — ANSI B31.1 (1955) plus code cases.[b] Pumps — ASME, Sec. III, Class C. Shell side — ASME, Sec. III, Class C and TEMA C. Tube side (LPCI, heat exchanger) — ASME, Sec. VIII and TEMA C.
D + T + H + E'	Same as for (b) under <u>Reactor vessel internals</u>, above. Also, primary stresses must be within the stress limits of ASME III, Class A.

[b]Conforms to ANSI B31.1 (1967) levels.

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DRAFT

TABLE 5. Allowable stresses for Class I structures (after Ref. 7, Table XII-1.7).

	Allowable limit[a]
Reinforcing steel	$0.5F_y$
Concrete:	
Compression stress	$0.45f'_c$
Shear stress	$1.1\ f'_c$
Bearing stress	$0.25f'_c$
Structural steel:	
Tension on net section	$0.60F_y$
Shear on gross section	$0.40F_y$
Compression on gross section	varies with slenderness ratio
Bending	$0.66F_y$ to $0.60F_y$

[a] F_y = minimum yield point of the material;
f'_c = compressive strength of concrete.

DRAFT

4.4 SEISMIC RESPONSE OF STRUCTURES

This section presents the results of analyses used in the original design and is the basis of comparisons with current criteria in Chapter 5. The original design analyses were not verified as part of this program.

Figure 7 is a plot plan of the plant, showing the structures discussed here.

4.4.1 Reactor Building (Ref. 7, Secs. XII-1.2.1 through 1.2.3; Am. 17, question VII-A.10)

A section of the reactor building is shown in Fig. 3. It is a cast-in-place, reinforced-concrete structure, founded on rock at an elevation of -32 ft. Its square, reinforced-concrete foundation measures 142 ft 6 in. along each side. The primary containment, an integral part of the building, occupies the central area of the building. The reactor building also encloses the reactor cleanup system, the reactor shutdown system, the reactor isolation condenser system, the supplemental cooling systems, and the control-rod-drive hydraulic system. All refueling is also done inside the reactor building.

The reactor service and refueling area is serviced by an overhead bridge crane, which is supported by a laterally-braced internal steel frame at an elevation of 108 ft 6 in. (This framing also supports the roof of the building.) Access to the drywell and reactor head space is gained by removing a large concrete plug in the refueling floor with the bridge crane. The crane also handles the drywell head, the reactor vessel head, the pool plugs, and the spent-fuel shipping cask. A refueling service platform with necessary handling and grappling fixtures serves the refueling area and the spent-fuel storage pool.

The seismic response of the reactor building was determined from a dynamic time-history analysis using the lumped-mass model shown in Fig. 8 and the digitized Taft record. In the reactor building model, single masses were lumped at each floor level, and the top concrete story was approximated by an equivalent two-mass system. Each mass represents the mass of concrete and equipment at that level and the tributary mass of the concrete and equipment between adjacent floors. (The top-story masses include the tributary mass of the walls, crane, and mechanical equipment of the top story.) The average area and moment of inertia of the concrete between floors was used to determine the stiffness of the beams, using the elastic modulus of uncracked concrete (3×10^6 psi).

Only the first three modes were evaluated because the predominant response of the building arises from the first mode (0.22 s); the period of the third mode (0.05 s) approaches that of a rigid system.

The subgrade modulus of elasticity for the bedrock material was evaluated by assuming 14,000 ft/s as the shear wave velocity in the bedrock and 0.2 as Poisson's ratio. From the subgrade modulus, the rocking mode period of vibration was calculated to be 0.13 s. These data were included in the computer analysis of rocking and elastic vibrations.

Figure 8 shows the maximum envelopes of building design shears and moments that were used in the seismic (OBE) design of the reactor building, without the usual increase in stress for short-term loadings. Stresses due to the SSE were handled as described in Table 5.

4.4.2 Ventilation Stack (Ref. 7, Sec. XII-1.2.5; Am. 17, question VII-A.10)

The ventilation stack is an unlined, free-standing, axisymmetric, reinforced-concrete structure.

The seismic response of the ventilation stack was determined from a dynamic time-history analysis using the digitized Taft input and a lumped-mass, flexible-cantilever model with its base rigidly fixed to the foundation (Fig. 9). Ten natural vibrational modes, between 0.035 and 2.289 s, were considered in the seismic analysis.

Design envelopes for shear and moment based on the OBE, are also shown in Fig. 9. The analysis also concluded that the stack would not fail, collapse, or hinder safe shutdown following an SSE.

4.4.3 Radwaste Building/Control Room (Ref. 7, Sec. XII-1.2.4; Am. 17, question VII-A.10; Am. 14, question III-A.6)

The waste treatment facility (Fig. 10) is north of and adjacent to the reactor building. The building contains equipment and tankage space below grade and the plant control room above grade. The area below grade is

DRAFT

constructed of reinforced concrete, and shielded compartments are provided for various pieces of equipment. The control room has reinforced-concrete walls and a steel-frame roof.

The building below grade is founded on rock. The allowable bearing pressure of the rock was assumed to be 10 tons/ft^2. The exterior walls are of cast-in-place reinforced concrete and are designed for an earth pressure (per square foot) at any depth equal to the depth (in feet) times 90 lb. The exterior walls and the base slab are designed to resist hydrostatic pressure and uplift due to exterior flooding to an elevation of 19 ft. Interior walls of the substructure are of cast-in-place concrete, and those of the superstructure were either cast in place or made of concrete masonry units. With few exceptions, the floors are poured-in-place concrete slabs.

The radwaste building/control room model (Fig. 11) comprises lumped masses connected by weightless elastic beam elements. The base was fixed. The masses of the concrete and equipment at each floor level were lumped together, along with the tributary mass of the concrete and equipment between floors. The moments of inertia and effective shear areas of the equivalent beam members were obtained by calculating the properties of horizontal sections through the building between the mass points. The time-history analysis used the Taft time history. Periods of the first three modes were calculated to be 0.08, 0.06, and 0.05 s in the strong direction, and 0.10, 0.05, and 0.04 s in the other horizontal direction. The original results are compared with the reanalysis results in Chapter 5.

4.4.4 Condensate Storage Tank (Ref. 7, Am. 17, question VII-A.10)

The condensate storage tank, located east of the reactor building (see Fig. 7) was analyzed by the time-history method. The damping values given in Table 3 were reported separately, in answer to question VII-A.14, Am. 17 of Ref. 7. No other information was reported.

4.4.5 Gas Turbine Building (Ref. 7, Sec. XII-1.2.7; Am. 14, question A.16; Am. 17, question VII-A.10; Am. 18, answer to question B-2)

The gas turbine building is a reinforced-concrete structure founded on piles driven to competent rock. The roof is an concrete slab, supported on

DRAFT

steel beams. The gas turbine flood-protection requirements were met by placing the turbine and the control equipment at an elevation of 17 ft and by providing the building access doors with flood gates to an elevation of 19 ft.

The gas turbine building, modeled as a single-degree-of-freedom mass-and-spring system, was analyzed by the response-spectrum technique, using the response spectrum from the Taft earthquake (Fig. 5). An SSE seismic load of 0.32 g was calculated for the structure (Ref. 14).

4.4.6 Turbine Building (Ref. 7, Sec. XII-1.2.1; Am. 14, questions III-A.1 and III-A.7; Am. 18, question B-4)

The turbine building (Fig. 12) is basically a Class II structure, but the parts that support and protect the emergency cooling subsystem were reviewed to meet Class I specifications where required. The foundation is a reinforced-concrete mat supported on rolled H-section, structural-steel bearing piles. All piles were driven to rock or to refusal in the dense strata immediately above rock. Reinforced-concrete shield walls extend to the operating deck at an elevation of 54 ft 6 in. Portions of the building outside the shield walls are protected by reinforced-concrete flood walls (to an elevation of 19 ft). Doors and access apertures in the flood wall are provided with flood gates. The rest of the building is constructed of steel framing with metal siding.

The turbine building ground floor consists of a reinforced-concrete slab supported on sand fill over the foundation mat. The turbine generator is supported on a massive reinforced-concrete pedestal, which is supported in turn on a 6-ft-thick mat that is an integral part of the foundation mat. The roof is covered with metal decking, insulation, and five-ply tar-and-felt roofing material, flashed at the parapet walls. An overhead rolling door at the south end of the building provides rail car access.

The seismic analysis was done by the equivalent-static method, using OBE coefficients of 0.07, 0.11, 0.14, and 0.25 g for the ground floor, mezzanine, operating floor, and roof, respectively. Equivalent static loads were calculated by multiplying these coefficients by the masses at the respective levels. Scaling by 17/7 gave the SSE loads. No detailed mathematical model was constructed for the turbine building, but presumably, two orthogonal

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horizontal analyses were performed. A separate dynamic analysis found the period of the first mode to be 0.10 s.

A few items were unable to meet the design criteria for both OBE and SSE loads and were modified to reduce the stresses to allowable limits. Modifications included an additional bay of vertical bracing above the operating floor and reinforcing structural T's on the 8-in.-thick concrete-block walls that make up the battery and switchgear rooms.

4.4.8 Drywell (Ref. 7, Am. 13, Appendix D; Am. 17, question A.8)

The pressure suppression containment system housed in the reactor building consists of a drywell; a pressure suppression chamber, which stores a large volume of water; and a connecting vent system between the drywell and the water pool. The drywell is embedded in concrete to an elevation of 8 in.; the concrete provides uniform support by following the contours of the vessel. An embedment transition is provided for the shell between El 2'2" and El 0'8". The material for the shell of the drywell, suppression chamber, and interconnecting vent system is ASTM-A516, grade-70 FBX plate to SA-300. Materials, design, fabrication, inspection, and testing conformed with the ASME Boiler and Pressure Vessel Code, Sec. III, Subsec. NB, 1965 ed.

Originally, the drywell seismic response was determined from an equivalent-static analysis, using the seismic coefficients shown in Fig. 13 for two different support conditions. A vertical acceleration--0.05 g (OBE) or 0.12 g (SSE)--was applied simultaneously. The seismic coefficients are said to arise from the results of the reactor building dynamic analysis, but details are not known.

The model was apparently an axisymmetric shell for which six cases were considered, each with different loading conditions:

- Initial test condition at ambient temperature; drywell cantilevered.
- Final test condition at ambient temperature.
- Normal operating condition.
- Refueling condition with drywell head removed.
- Accident condition.
- Flooded condition.

The maximum primary membrane stress in the shell (19 ksi) resulted from the accident condition, but was less than the 19.25-ksi code-allowable level. The internal pressure load was the greatest contributor to the stress. The containment vessel was also reviewed for the SSE condition by scaling the OBE seismic load by 2.4. The maximum stress increased to 19.6 ksi, still below the yield stress of the material (26.25 ksi). Discontinuity stresses and buckling stresses were also within allowable limits. Subsequent to the static analysis, a dynamic analysis was performed as shown in Ref. 14. It is presumed that SSE stress levels from this analysis were checked.

4.4.9 Pressure Suppression Chamber (Ref. 7, Am. 13, Appendix D; Am. 17, questions A.1 and A.10; Am. 18, question B.5; Am. 18, technical report TR-2138, docket 50245-776)

The pressure suppression chamber is a toroidal, steel pressure vessel, located below and encircling the drywell. It is constructed of the same material as the drywell (Sec. 4.4.8). Inside the chamber, also in the shape of a torus, is the vent system distribution header, from which 96 downcomer pipes project. The upward reaction from the downcomers is resisted by columns extending from and attached to the bottom of the suppression chamber. The columns are pinned top and bottom to accommodate differential horizontal movement of the header and the suppression chamber. The suppression chamber is supported on 16 pairs of columns, spaced equally around the torus. These supports transmit vertical and seismic loading to the reinforced-concrete foundation of the reactor building. Cross-bracing is provided between the outer support columns to provide lateral stability (Fig. 14).

The torus and its support were modeled as a single-degree-of-freedom system, as shown in Fig. 4-26. The rigid frame and the cross-bracing contributed to the spring constant of the system. The masses of the steel vessel, baffles, fluid content, supporting structures, and appurtenances were all lumped into a single mass, and the period of the system was calculated using the formula for a simple oscillator. The response-spectrum method was used to analyze the model. Spectral acceleration was taken from the response spectrum curve for 2% damping (Fig. 4.1) and multiplied times the mass to calculate the total equivalent seismic force. A vertical loading of 0.05 g (OBE) was applied simultaneously with the horizontal loading. Four load cases were considered:

- Initial test condition at ambient temperature.

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- Accident condition; pressure 62 psig.
- Accident condition; pressure -2 psi.
- Flooded condition.

Results showed that the maximum stress on the torus shell occurs at the transverse ring girders and that all stresses remain within ASME code-allowable limits. The inside columns supporting the torus experience the largest axial and bending stresses under seismic loading. Depending on the postulated conditions, the calculated stresses ranged between 79% and 99% of the code-allowable stress limit. The seismic response of the suction header was evaluated by comparison with the seismic analysis of a similar header. Allowance was made for possible higher-mode effects and potential resonance between the header and one of the modes of the torus. The vent system, comprising the vent header, the downcomers, and the vents and their bellows, were designed to withstand accident conditions that either include seismic loads or impose loads far greater than the postulated seismic loads.

4.5 SEISMIC RESPONSE OF PIPING SYSTEMS

4.5.1 Recirculation Loop Piping (Ref. 7, Am. 14, question III-A.7; Am. 18, question B-2; Am. 19, question B-1c; Am. 21, question B-2)

In the response-spectrum analysis of the recirculation loop piping, the smoothed response spectrum of Fig. 4 was used as input at the rigid pipe supports, together with a vertical input two-thirds of the horizontal. The piping was modeled as a lumped-mass system (Fig. 15). The stiffness was determined by considering axial, shear, flexural, and torsional deformation, as well as the effect of curvature. Modal frequencies and mode shapes were not reported, but the inertial loads for each mode were calculated separately, then combined as the square root of the sum of squares. The maximum total stresses on the recirculation loop piping, calculated for the OBE and SSE loads, were 59.4% and 83.7% of the allowable (yield) stress.

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4.5.2 Main Steam Lines and Other Class I Piping in the Reactor Building (Ref. 7, Am. 17, Sec. VII and question A.2)

The seismic coefficient curve in Fig. 6 was used as the basis of the equivalent-static analyses of equipment and piping in the reactor building. For Class I piping, as well as nonrigidly attached equipment, the coefficients from the curve were multiplied by an amplification factor, which is the ratio of the average peak acceleration from the Taft response spectrum (Fig. 5) and the peak ground acceleration (0.7 g). Three seismic coefficients were calculated:

- 0.90 g--for piping above an elevation of 90 ft.
- 0.68 g--for piping between 90 and 55 ft.
- 0.50 g--for piping below 55 ft.

For each piping system, the coefficient was selected that corresponds to the elevation range of most of the system mass.

The lumped-mass model of the main steam line was analyzed in two orthogonal horizontal directions, with a concurrent vertical load two-thirds of the horizontal applied in each analysis. In calculating stresses, stress intensification factors were considered, as was the relative motion between the reactor building and the turbine building. The latter effect proved to be negligible.

The total stresses, under OBE loading, ranged between 39.6% of the allowable stresses (for the reactor-head cooling spray system) and 69.4% of the allowable (for the core spray system). Under SSE loading, the total stresses ranged between 36.9% and 93.0% of the yield stresses, for the main steam system and the standby liquid control system, respectively. The analysis also produced estimates of the stresses to be expected at the piping restraints; the restraints were designed to withstand these loads.

4.5.3 Class I Piping in the Turbine Building (Ref. 7, Am. 14, question III-A.7)

Seismic coefficients for piping in the turbine building were based on the smoothed response curve shown in Fig. 4 and the calculated period (0.10 s) of the building. For rigid piping (period 0.05 s or less), the coefficient of 0.18 g was taken directly from the curve for 0.5% damping; where no analysis was done to establish the rigidity of the piping, the seismic coefficient was taken to be 0.28 g.

No results of the analyses were found.

4.5.4 Class I Instrumentation Piping (Ref. 7, Am. 14, question III-A.7)

Class I instrumentation piping was designed using the lateral deflection and force evaluation curves described in Sec. 4.3.1.3 of this report.

4.6 SEISMIC RESPONSE OF MECHANICAL AND ELECTRICAL EQUIPMENT

4.6.1 Reactor Pressure Vessel System (Ref. 7, Am. 13, Appendix E, Exhibits A and G; Am. 14, question III-A.7; Am. 17; questions A.6, A.14, and A.16)

The reactor pressure vessel system (Fig. 16) comprises the vessel, the surrounding steel wall (a cylindrical concrete-filled steel shell), the vessel support system, and the concrete support pedestal. The mathematical model developed for the response-spectrum analysis of the system is shown in Fig. 17. Reactor vessel masses are represented by mass points 1 through 7, shield wall masses by points 9 through 12. Springs k_1 and k_2 represent connecting structural elements between the vessel and the shield, and between the shield and the reactor building, respectively.

Rayleigh's method was used to calculate the fundamental period (estimated to be 0.145 s) and the first mode shape. The modal acceleration was taken from the smoothed 2% response spectrum shown in Fig. 4, and the maximum acceleration of each mass point (relative to the building) was then calculated. The maximum absolute acceleration at each mass point was obtained by combining this value with the maximum building acceleration (from the earlier analysis of the reactor building) as the square root of the sum of squares. Seismic loads at each mass point were then computed and applied as static loads on the reactor pressure vessel system.

The calculated OBE shears and moments are shown in Figs. 18 and 19 for the vessel, shield, and support pedestal. Internal moments and forces stemming from the relative displacement of the building were combined with those from inertial loads by summing absolutely.

The data indicate that stresses produced by the SSE were not evaluated in the design stress checks. Furthermore, no seismic stresses were included in the nozzle stress check.

4.6.3 Other Instrumentation

4.6.3.1 Isolation Condensor, LPCI Pump, and Containment Heat Exchanger (Ref. 7, Am. 17, question A.10)

No information was reported, except that summarized in Table 4-1.

4.6.3.2 Class I Equipment in the Reactor Building (Ref. 7, Am. 14, question III-A.7)

For rigidly attached equipment (period 0.05 s or less), the curve of Fig. 6 was used to obtain seismic coefficients for the equivalent-static analyses. For nonrigidly attached equipment, the coefficients were calculated as they were for piping (Sec. 4.5.2). No results were reported.

4.6.3.3 Class I Equipment in the Turbine Building (Ref. 7, Am. 14, question III-A.7)

Seismic coefficients for equipment in the turbine building were based on the smoothed response curve shown in Fig. 4 and the calculated period (0.10 s) of the building. For rigid equipment, the coefficient of 0.16 g was taken directly from the curve for 1% damping; where no analysis was done to establish the rigidity of the equipment, the seismic coefficient was taken to be 0.22 g. No results of the analyses were reported.

4.6.3.4 Batteries and Battery Racks (Ref. 7, Am. 14, question III-A.7)

The batteries and battery racks were designed to withstand lateral and vertical seismic loads of 0.12 and 0.046 g, respectively, based on equivalent-static analyses. No further details of the analyses were available.

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4.6.3.5 Off-Gas System Modifications

No details of the assumed Category I qualification were available.

4.6.3.6 Safety Instrumentation (Ref. 7, Am. 18, Sec. A.7)

A seismic testing program was prescribed for the following Class I systems:

- Reactor protection system;
- Nuclear boiler system;
- CRD hydraulic system;
- Standby liquid control system;
- Neutron monitoring system;
- Emergency core cooling system;
- Process radiation monitoring system.

No information was available to indicate whether the test program was carried out.

CHAPTER 5: REASSESSMENT OF SELECTED STRUCTURES

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5.1 INTRODUCTION

In this chapter, the seismic loads and responses on which the Millstone 1 structural designs were based (see Chapter 4) are compared to corresponding seismic loads and responses derived using SEP seismic evaluation methods. This comparison is made to identify those structures that essentially meet SEP seismic criteria and those that need to be investigated further. Seismic loadings and responses are examined for the reactor building and drywell, radwaste/control building, turbine building, ventilation stack, field-erected tank, a typical buried pipe, a typical buried tank, the suppression chamber, and the gas turbine building.

In addition, seismic input motions for equipment (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 as described in Chapter 6.

Since the completion of the Millstone 1 plant, a number of changes in seismic design methods and qualification criteria for structures and equipment have occurred. These changes do not necessarily imply that old seismic qualification criteria were inadequate, merely that the criteria are now better defined and require less interpretation by the designer. The general trend has been to increase:

- Allowable stresses for the specified seismic loading function

- Allowable damping

- Number of loading conditions to be considered simultaneously

- Degree of sophistication to be used in the analyses

- Quality assurance requirements

DRAFT

5.2 DESIGN EARTHQUAKE MOTION

In describing the design earthquake motion for a given site, several items of information are required:

- Peak ground acceleration, together with either design ground response spectra or a design time history
- How and where in the structure the design inputs are specified (such as at the base slab, in the free field, etc.)
- Simultaneous directional components
- Duration or number of strong motion cycles

This section compares the ground motion parameters specified for Millstone 1 with the SEP acceptance criteria.

5.2.1 Peak Ground Acceleration

The regulation currently governing seismic design of commercial nuclear power plants is 10 CFR 100, Appendix A (Ref. 16). It sets forth the principal seismic and geological considerations to be used in determining such design bases as requirements for the OBE and SSE, for peak acceleration levels, and for design response spectra. As discussed in Chapter 4, Millstone 1 structures were designed for an OBE and an SSE with peak ground accelerations (PGA) of 0.07 and 0.17g, respectively. A simultaneous vertical component of earthquake motion equal to two-thirds of the horizontal component was considered in the plant design.

For this reassessment, an SSE characterized by a 0.2-g peak horizontal ground acceleration was employed. Although a probabilistic evaluation of the seismicity of the Millstone 1 site may justify a slightly lower value, it was considered unlikely that a level higher than 0.2 g would be required.

5.2.2 Ground Motion Characteristics

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In addition to the peak ground acceleration, either a design time history (or histories) or ground response spectra are needed to define a design earthquake. Typical current practice is to specify either site-dependent spectra or, more often, ground response spectra like those in R.G. 1.60 (Ref. 17). These latter 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. Currently, time-history analyses are based mostly on artificial earthquakes whose response spectra envelop the smoothed R.G. 1.60 design spectra.

Rather than compare response spectra directly for equal damping values, it is more informative to include the damping used in the design of Millstone 1. Table 6 lists the damping values used for Millstone 1 together with those from R.G. 1.61 (Ref. 12) for the SSE and those values recommended in NUREG/CR-0098 for structures at or below the yield point. The damping values used in the design of Millstone 1 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 of PGA. Because higher response and, consequently, increased damping are expected for the SSE, a significant degree of conservatism was typically introduced over current practice.

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TABLE 6. Original and currently recommended damping values.

Structure or component	Percent of critical damping		
	Millstone 1	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	2	4 or 7	10 to 15
Welded assemblies	1	4	5 to 7
Bolted and riveted assemblies	2	7	10 to 15
Vital piping	0.5	2 or 3	2 to 3

[a]Ref. 12.

[b]Ref. 4.

A comparison of the original response spectrum scaled to 0.17g peak acceleration for 2% damping with the 7% spectrum from R.G. 1.60 indicates the relative magnitudes of the 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 20 shows these comparisons.

5.3 SEISMIC DESIGN METHODS

Current licensing requirements would typically require load combinations resulting from transients other than those considered when Millstone 1 was designed. This re-evaluation 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 Millstone 1.

5.3.1 Soil-structure Interaction

Sophisticated methods of treating soil-structure interaction exist today. However, for structures that are founded on competent rock, as is Millstone 1, the effects of soil-structure interaction are probably relatively small and were neglected in the reananlysi of all structures except the gas turbine building. 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. For the gas turbine building, still under investigation, the flexibility of the pile foundation is being considered.

5.3.2 Combination of Earthquake Directional Components

The design of Millstone 1 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. 17).

5.3.3 Combinations of Earthquake and Other Loads

The design and analysis of Millstone 1 used the load combinations for Class I structures shown in Sec. 4.4. Load combinations are now specified in applicable design codes and standards such as ASME Sec. III, Div. 2, and ACI-349 (Refs. 18 and 19). 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 (Ref. 3).

Because stresses resulting from load cases and combinations of loads from these more recent criteria are not available, the re-evaluation of the selected 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.

5.4 ANALYSIS OF MAJOR STRUCTURES

The major structures included in this analysis are the reactor building and drywell, the radwaste/control building, the turbine building, and the ventilation stack. The three buildings are adjacent to each other, with the north sides of the radwaste and turbine buildings connected to structures of Unit 2. Modeled as three independent structures, there is no significant structural interaction between the three buildings. Since the connections between Units 1 and 2 are slotted, mass or stiffness effects of the interconnections were ignored in this analysis.

An evaluation of the seismic capability of these structures was conducted using loads developed in accordance with current practice. A response spectrum analysis, using the R.G. 1.60 spectrum, was performed to check the structural adequacy of each structure. A time history analysis, using the same synthetic time history as was used in other SEP plants, (Ref. 20) was used to generate in-structure spectra. In the case of the reactor building, radwaste/control building or stack, the original model as specified or with some modifications was used. For the turbine building, a new model was developed. Where loads based on current criteria are less than those used in the original design, the structure in general was judged to be adequate without the need for additional evaluation. For cases where loads resulting from current criteria exceeded the original loads, if the resulting stresses were low compared to yield, these structures were also judged to be adequate. In general, damping values based on NUREG/CR-0098 were used. These values are higher than those in R.G. 1.61, the current basis for new license applications.

For those cases in which the seismic stresses were not low compared to yield, conclusions were reached on the basis of ductility for Class I structures as defined in Ref. 4. In accordance with Ref. 4, stresses above yield are considered acceptable provided the ability of the structure to perform its safe shutdown functions is not impaired.

5.4.1 Analysis of the Reactor Building and Drywell

The vertical-beam model for the analysis of the reactor building and drywell was developed from information shown in Refs. 15, 21, and 22 is shown in Fig. 21. This model is a lumped-mass three-dimensional model which includes torsional effects. The model has eight nodes representing the centers of gravity of floors at eight elevations in the reactor building. The floors are considered rigid and translational and rotational masses contributing to each floor level are lumped at these nodes. The model is rigidly fixed to the ground at el. -26'0". A uniform damping of 10% of critical was assumed in calculating the responses of the reactor building, since the walls were stressed above 50% of yield.

The drywell was modeled by 14 lumped masses interconnected by beam elements, rigidly connected to the reactor building model at el. 0'0", and simply supported at approximately 73'. Two cases were considered in modeling the drywell: the flooded drywell condition and the empty drywell condition. The mass and stiffness properties of the drywell were the same as those of the original model. A uniform damping of 7% of critical was assumed in calculating the responses of the drywell.

Structural seismic responses were obtained by using the response spectrum analysis method. In this analysis the R.G. 1.60 spectral curves were scaled to 0.2 g peak acceleration for the horizontal components, and 0.13 g for the vertical. The directional responses were then combined using the square root of the sum of the squares (SRSS) method.

The time-history method was used to generate in-structure response spectra for the reactor building. The R.G. 1.60 spectral curves were scaled as in the response spectrum analysis. In-structure spectra were generated independently in both the horizontal and vertical directions, and like components were combined by means of the SRSS method.

The first twenty modes of the coupled reactor building-drywell system were extracted from the mathematical model. The natural frequencies of the

DRAFT

first twenty modes range from 5.1 to 30.0 Hz, for the flooded drywell case, and range from 5.2 to 38.1 Hz for the empty drywell case. Table 7 shows the first four frequencies in the N-S and E-W directions. For the flooded drywell case, the fundamental modes for the reactor building are at 5.1 and 5.3 Hz. At 9.1 and 9.3 Hz, the system shows the first clear drywell modes. Mode 6 (12.2 Hz) and 9 (15.6 Hz) are the first significant building-drywell coupled modes. Mode 6 has also a significant component in the vertical direction. Torsional coupling between two horizontal components does not become noticeable until the 10th mode at 18.6 Hz.

TABLE 7. Reactor building/drywell model modal frequencies (drywell flooded or empty)

Mode	Frequency (Hz) Empty	Frequency (Hz) Flooded	Direction of Major Response Empty	Direction of Major Response Flooded
1	5.2	5.1	N-S	N-S
2	5.4	5.3	E-W	E-W
3	9.7	9.1	N-S	N-S
4	12.2	9.3	E-W	E-W
5	14.8		E-W	
6	15.6	12.2	N-S	E-W
7	21.6		E-W	
8	22.6		N-S	
9		15.6		N-S
11		20.1		E-W
16		23.7		N-S

For the empty drywell case, the frequencies and mode shapes for the first few sigificant modes of the reactor building remain almost unchanged from the flooded case, whereas, the drywell modes were shifted to higher frequencies.

The flooded drywell case was used throughout the analyses. This case resulted in somewhat larger inertial loading for the reactor building and drywell than that from the empty case. In the context of a screening analysis, the flooded drywell was considered the worst case. After completion of the structural analysis of the reactor building and drywell, it was discovered that in-structure spectra were required for piping systems inside the drywell. These were generated using the empty drywell case.

DRAFT

The seismically induced shears and bending moments in the reactor building are presented in Fig. 22 for the flooded case. The results for the empty case are very close to the flooded case only slightly lower. As a comparison, the original seismic analysis results using the time history method with lower damping were also plotted in the respective figures. The reactor building was originally analyzed for 0.07 g Taft record, and the structure was then checked for SSE condition which had a peak acceleration of 0.17 g. The new results for shear force based on 0.2 g R.G. 1.60 spectrum are quite close to the original SSE (0.17 g) result. However, the new moments are significantly higher than the original results.

To evaluate the stress results of the reactor building, we referred to the stress summary report prepared by EBASCO Services Inc. (Ref. 22). The EBASCO report evaluates the stresses in the concrete shield walls, columns and exterior walls at elevation 14.5', based on the original 0.07 g design earthquake analysis. The new shear force based on the R.G. 1.60 0.2 g spectrum is about three times the original OBE shear, and the moment is about four times the original moment. When the dead load and live load stresses from the EBASCO report are added to the new seismic shear force and moment distributed according to the member rigidities, the results show that the maximum predicted shear stresses in the exterior shear walls and the reactor shield walls are both about 180 psi. Calculated capacities were greater than 200 psi. Also, the total loads in the columns including axial loads and moments are less than their ultimate capacities. Thus, the reactor building should be adequate to resist the 0.2 g SSE.

Figure 23 shows the shear diagram of the flooded drywell subjected to the 0.2 g SSE excitation. Also plotted in the same figure are the results of the original time history analysis using the Taft record with a PGA of 0.17 g corresponding to the SSE. As expected, the shear stresses in the 0.2 g SSE case are higher than the original 0.17 g SSE case. Appendix D of Amendment 13, FSAR gave the stress evaluation of the drywell under different load combinations. When the new seismic loads were combined with loads induced under normal operating conditions, the maximum primary membrane stress and buckling stress in the drywell were found to be within allowable stress limits. The drywell can therefore be considered adequate for loads from normal operating conditions and the 0.2 g SSE.

DRAFT

5.4.2 Analysis of the Radwaste/Control Building

The mathematical model used in the analysis of the radwaste/control building (Fig. 24) was developed from the original Blume model (Ref. 23). Modifications to the original model included offsets between mass centers and centers of rigidity at each floor. Floors were assumed to be rigid with centers of mass and rigidity connected by rigid links. The foundation, also assumed to be rigid, was set at elevation -20'. Between the foundation and 14'9" elevation, the structure separates into a dual beam system. One system consists of a beam for the west wing of the building which has no intermediate floor between the foundation and 14'9" elevation. The other, for the east wing, consists of two beams with an intermediate floor at elevation at -1'6". Both systems are rigidly connected at elevation 14'9".

The building separates into two towers above the 14'9" elevation. One tower supports the control room (el. 34'6"); the other consists of the stairway structure. The building model contains a total of eight lumped masses.

The method of analysis was similar to that used for the reactor building. Again, the response spectrum method was used to evaluate the building's structural responses to the 0.2 g SSE using 10% of critical damping. The R.G. 1.60 horizontal spectral curve was scaled to 0.2 g and input as the excitation in both N-S and E-W directions. Since the structure was assumed to be rigid in the vertical direction, no dynamic amplification was considered. The SRSS method was used to combine structural responses to different directions of excitation. The time-history approach was used to generate in-structure response at the control room floor. The analysis included only the horizontal components of ground excitation. Because we assumed no dynamic amplification in the vertical direction, the vertical floor spectra were the same as the ground spectra.

The first sixteen modes of the structure were extracted from the mathematical model. The first three modes in the weak (N-S) direction have frequencies 10.9, 16.7, and 24.6 Hz, respectively. In the strong (E-W) direction, the first three frequencies are 12.4, 13.8 and 21.5 Hz. The mode at 16.7 Hz is the first mode to exhibit noticeable torsional behavior.

The radwaste building is primarily a shear-wall type structure. The shear forces calculated from this analysis for the 0.2 g SSE are presented in Fig. 25 for both the N-S and E-W directions. In the same figures, shear forces

DRAFT

from the original time history analysis for a 0.17 g SSE are plotted for comparison. The shear wall structures of the radwaste building supporting the control room all have low shear stresses (factor of safety greater than 1.3). Only the stairway structure, which is independent of the control room structure, has high shear stresses. The control room together with its supporting radwaste building can therefore be considered to be acceptable in light of current criteria.

5.4.3 Analysis of the Ventilation Stack

The model used in the original seismic analysis of the ventilation stack (Ref. 24) was used without modification in the reanalysis. The model includes a foundation assumed to be rigid and has forty mass points connected by vertical beam elements. The damping ratio for all modes was 10%. The response spectrum method using R.G. 1.60 spectra as input was again used.

The first ten modes of the stack have frequencies ranging from 0.44 to 29.36 Hz. The calculated shear force and moment diagram are shown in Fig. 26. Compared to the original time history analysis using 0.17 g for the SSE horizontal component, this analysis gave slightly higher shear forces over the height except near the base. The bending moments computed in this analysis are much higher than those of the original time history analysis. Stress calculations based on a cracked section assumption and ACI-307-79, (Ref. 25) showed that forces resulting from the combined dead and seismic loads were less than ultimate capacities of the reinforced concrete sections. The adequacy of the connection of the piles to the footing should be verified to ensure that the 35 kip tension per pile can be safely resisted.

Although a significant portion of the stack may develop cracks through the cross section, it is considered to be marginally acceptable.

5.4.4 Analysis of the Turbine Building

The turbine building consists of three floors: the ground floor at elevation 14.5'; a mezzanine at elevation 34.5'; and the operating floor at elevation 54.5'. There is a small low roof (el. 77.13') above the operating floor and part of the ventilation equipment enclosure. A higher roof stands at 104.75' and is supported by a series of rigid frames (Fig. 27). The

DRAFT

operating and mezzanine floors are supported by a concrete reinforced shear wall which is part of the condensor room located inside the turbine building between the ground and operating floors. After the seismic review performed by EBASCO (Ref. 26, Enclosure) in 1979, additional bracing members were installed in accordance with their recommendations (Figs. 28 and 29). Steel member connections were not constructed to resist moment, except for those connections in the rigid frame. The turbine pedestal is not structurally connected to the turbine building above ground level, although it shares a common foundation.

No original seismic model existed for the building; therefore, a model had to be constructed for this analysis (Fig. 30). The model constructed included the condensor room and the frame structures above ground level and incorporated the added bracing. The foundation was assumed to be rigid, i.e., no soil-structure interaction effect was considered. The model was a four lumped mass model, with masses located at the upper roof, the lower roof, the operating floor, and the mezzanine floor. Each lumped mass had three dynamic degrees of freedom, two horizontal translations and one torsional rotation. All steel structural members were represented by beam elements. All joints, except those joints in the rigid frames, were modeled as hinges. Since members with angle sections have very low buckling loads, only half of the actual cross sectional area for angle bracing was included in the model. Only horizontal degrees of freedom were assumed for the seismic dynamic analysis. Dynamic amplification in the vertical direction was not considered.

The condensor room was modeled by massless equivalent beam elements located at each floor's center of rigidity. The horizontal degrees of freedom of all structural members at each floor were rigidly connected to the lumped masses of that floor. A uniform critical damping of 10% was assumed for the entire model.

The response spectrum method was used to evaluate the dynamic responses to seismic excitations of the building. In-structure floor response spectra were generated by the time history analysis method.

The mathematical model for the turbine building posseses twelve lateral modes, with frequencies ranging from 1.3 Hz to 36.6 Hz. The two significant modes for the high roof are the E-W mode at 1.3 Hz and torsional modes at 7.9 Hz. The first two significant operating/mezzanine floor modes are at 10.3 Hz (E-W direction) and 119 Hz (N-S direction). At 2.5, and 2.8 and 6.9 Hz, the model has mixed high and low roof modes.

DRAFT

To evaluate the stress in the bracing systems, the compressive members were assumed to take no load, with all lateral load being taken by the tensile members. The results indicate that the highest stress in all turbine bracing was about 36 ksi and occurred at the bracing of Column line 3 near the high roof. The stresses in columns, including the dead load, were generally less than 20 ksi. Locally, at Column line E near the low roof, the columns experienced higher ...s resulting from combined bending and axial load in the 30 ksi range primarily from flexure. The roof girders were not highly stressed, the maximum bending stress being less than 10 ksi.

The lateral load resisting system of the turbine building includes the rigid frames, diagonal bracing and the reinforced concrete shield wall. Results indicated that the stresses in the frames and bracing were generally below yield. Although the diagonal bracing in one end wall resisting east-west loading was stressed at yield, the rigid frames in this direction should have enough reserve capacity to allow the structure to resist a 0.2 g SSE load. The tracing should b investigated for non-ductile failure modes and sufficient strength in the connections to develop yield stress of the bracing members.

The concrete shield wall for the condenser room acted mainly as shear wall. The calculated shear stresses in the concrete were less than 125 psi. This stress level results in a safety factor greater than 1.0 for the wall.

5.5 In-Structure Response Spectra

As discussed in the previous sections, the in-structure response spectra were generated using time history methods. The ground excitation was a time history record compatible with the R.G. 1.60 spectrum curves. The responses to excitations which were input independently in two horizontal directions were combined by the SRSS method. The response spectral curves were generated for four different equipment damping ratios, 2, 3, 5 and 7%. The spectral curves generated were smoothed and broadened ±15% according to the current SEP practice, to account for the uncertainties of structural modeling and material properties.

The in-structure response spectral curves were generated for the following locations.

DRAFT

1. Control Room Floor, elevation 34'-6" of the radwaste/control building.

2. Reactor Building Floor elevations - 82.75', 65.75', 42.5' and 14.5'.

3. Reactor Shield Wall, elevations 73' and 28'.

4. Turbine Building Floor elevation 34'-6".

Table 8 gives the peak acceleration computed at those locations for the 0.2g SSE. The final broadened spectra are shown in Appendix A.

TABLE 8. Peak accelerations at selected locations.

LOCATION	ELEVATION	DIRECTION	PEAK ACCELERATION (g)
Control Room	34.5	N-S	0.45
		E-W	0.35
		Vertical	0.13
Reactor Building	82.75	N-S	0.43
		E-W	0.43
		Vertical	0.24
	65.75	N-S	0.38
		E-W	0.38
		Vertical	0.22
	42.50	N-S	0.30
		E-W	0.30
		Vertical	0.18
	14.50	N-S	0.23
		E-W	0.23
		Vertical	0.15
Reactor Shield Wall	73.0	N-S	0.40
		E-W	0.40
		Vertical	0.21
	28.0	N-S	0.25
		E-W	0.25
		Vertical	0.16
Turbine Building	34.5	N-S	0.30
		E-W	0.32
		Vertical	0.13

5.6 Analysis of Ancillary Structures

DRAFT

In the design of Millstone, the field erected tanks were analyzed using a time history approach. The details of this analysis were not available for review, however. Apparently, no analysis of the seismic response of the buried pipelines or buried tanks was conducted during the design. The lateral bracing and steel columns of the Millstone pressure suppression chamber (torus) were designed using a single-degree-of-freedom analysis in conjunction with the ground response spectra. The suction header design was based on a single-degree-of-freedom analysis of a similar suction header for the Monticello Nuclear Plant and the response increased to account for possible additional amplification of the Millstone torus. Current licensing requirements would typically require additional load combinations resulting from other transients. Because these combinations are unlikely, this evaluation has concentrated on the original design combinations with primary attention devoted to the seismic margins.

The reevaluation of the seismic capacity of the Millstone 1 ancillary structures was conducted in accordance with SEP acceptance criteria (Ref. 4). The reevaluation of the adequacy to withstand the 0.2 g SSE was based on comparisons of recalculated loads with the original design loads and, where necessary, on further stress analysis. Where possible, an initial screening of the expected responses of the structures was conducted on the basis of load ratios where the load ratio is defined as the ratio of the load calculated in the original analysis to that derived in the SEP reanalysis. Where original design loads were not available or where significantly low load ratios exist, conclusions were based on the ductility of Class 1 structures as defined in Ref. 4. Load ratios less than unity do not imply that inelastic responses would be expected. Inelastic responses would be expected only if the original design loads were well below the elastic limit. Therefore, structures that do not exhibit load ratios less than 0.8 are considered acceptable. For structures with load ratios less than 0.8 or for those where design analysis was neither conducted nor available, more detailed investigations, including stress analyses of critical components, were conducted.

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5.6.1 Condensate Storage Tank

5.6.1.1 Condensate Storage Tank Confirmatory Analysis

The Condensate Storage Tank (CST) is located north of the reactor building on a concrete slab. The tank is 49 feet high and has an inside diameter of 40 feet. The tank is fabricated from 5454 aluminum, "O" temper. The wall thickness varies from 0.25 inches to 1.335 inches in six separate courses (Fig. 31). Sixty 1-1/2 inch diameter anchor bolts fabricated from A36 steel provide resistance to uplift forces.

We investigated a number of potential failure modes in the course of the SEP evaluation of the CST. Among these were: buckling of the tank sidewalls due to the seismic overturning moment; yielding; fracture, or pullout of the anchor bolts; sliding of the tank at the base with subsequent rupture of connections; and failure due to high tensile hoop stresses as a result of hydrodynamic pressures occurring simultaneously with the hydrostatic pressure. Because of the geometry of the Millstone CST, actual overturning is not a potential problem. However, if the anchor bolts do not have sufficient strength, the tank walls must resist the overturning moment and the weight of the internal fluid. In order for the fluid to be effective, a portion of the tank must separate from the foundation as shown in Fig. 32. This highly stressed portion of the tank is a potential source of failure if poor welds or other nonductile behavior mechanisms are present.

Two important effects should be considered when calculating the overturning moments and shears. The first is the impulsive force caused by the combined motion of the tank shell and part of the fluid. A second, or convective, mode occurs when part of the water near the free surface sloshes back and forth inside the tank.

In the SEP evaluation of the CST, we used Veletsos' methods (Refs. 27, 28) to calculate the impulsive mode frequencies. This approach is based on Rayleigh's method and includes both the shear and flexural deformation of the tank. The convective, or sloshing, frequencies were calculated following recommendations in Ref. 29. Frequencies for the impulsive and convective modes were calculated to be 5.8 Hz and 0.27 Hz, respectively. A spectral acceleration of 0.48 g at 7% of critical damping was used for the impulsive mode, and a spectral acceleration of

DRAFT

0.16g at 0.5% damping was used for the convective mode. Due to the unlikelihood that the maximum modal responses will occur simultaneously, we used the SRSS method to combine the impulsive and convective forces.

5.6.1.2 Evaluation of Condensate Storage Tank

The integrity of the tank shell and base plate under the assumed 0.20g SSE conditions was evaluated for the CST. The loads in the tank were based on the spectral accelerations discussed in Sec. 5.2.1 of this report together with a 0.13 g vertical component. Initially, the assumption was made that the tank behaves in a linear manner as shown in Fig. 33. Tensile and compressive forces in the tank shell were evaluated, and the attachment bolt stresses and tank side wall buckling were checked.

The reevaluation of the CST was conducted based on drawings (Ref. 30) supplied by the licensee. These drawings specify the use of anchor bolt chairs, and the reevaluation was based on these drawings. Subsequent to the completion of the reevaluation, it was determined that the anchor bolt chairs were not used on the CST, and the anchor bolt restraint is provided only through the tank bottom plate. Since the foundation concrete is poured flush with the bottom plate upper surface, it was not possible to verify the tank bottom plate thickness.

Linear behavior was found to adequately model the response of the CST provided the anchor chairs are present. The maximum tensile stress across the bolt thread area is above yield for the A36 material and also above the 0.7 ultimate strength ASME Code allowable for the faulted condition for the 0.2 g SSE. For the A36 bolt material, the bolt shank is not expected to yield. Tank anchor bolts are not subjected to load reversals. Since the maximum stress in the thread area is less than the ultimate material strength, failure of the bolts is not expected although the code allowable is exceeded. The anchor bolt chairs as originally specified would have adequate capacity to transfer the bolt forces into the foundation. The analysis indicated that anchor bolt pullout would not be expected during an SSE if the embedment length meets minimum building code requirements. However, details of the embedment were not available for review. The tank wall is sufficiently thick to prevent buckling. Frictional resistance between the tank and the

DRAFT

underlying footings was high enough to preclude any relative displacement between the tank and ground. Hoop stresses were evaluated by combining the SRSS of the pressures induced by the impulsive, convective, and vertical modes absolutely with the tank hydrostatic pressure. The CST was found to have shell thickness sufficient to prevent yielding. Thus, the critical elements of the CST are considered adequate to withstand the 0.20 g SSE based on the presence of the anchor bolt chairs.

5.6.2 Underground Piping

5.6.2.1 Underground Piping Confirmatory Analysis

During the original design, buried pipe was apparently not analyzed. The available soils data is incomplete and does not include any determination of anticipated soil strains expected during the SSE. For the current evaluation of the buried pipe, the soil strain was conservatively determined to be approximately 1.8×10^{-4} in/in as obtained from the relationship:

$$(\varepsilon_a)_{max} = \frac{\nu_{max}}{C_\varepsilon}$$

where ν_{max} is the maximum ground velocity and C_ε is the apparent longitudinal horizontal propagation speed of the seismic waves with respect to the structure. The apparent longitudinal horizontal wave propagation speed was not determined, since no geotechnic investigation was made. We conservatively estimated the speed at approximately 2600 fps accounting for the proximity to bedrock. The maximum ground velocity ν_{max} at a rock site corresponding to an earthquake with peak ground acceleration of 1 g is 28 in/sec (Ref. 31).

The SEP evaluation for the 0.20 g SSE included the analysis of a typical buried pipe. The pipeline selected was the gas turbine fuel supply line. This pipe was field run from the gas turbine building to the gas turbine oil storage tank (Fig. 34). The line is 1.5 inches in diameter with the bottom of pipe located at Elevation 11'-0".

The analysis considered the stresses induced in the pipe from two mechanisms. Stresses may result from strains in the soil caused by the propagation of elastic waves or from the effects of discontinuities and attachment point relative displacements. Wave propagation stresses may occur in the pipe as a result of primary (compression) waves, secondary (shear) waves, and surface waves with various angles of incidence to the pipe. Displacement-induced stresses may result from relative motion of the structure at a pipe attachment point or penetration location. The phasing of the stresses due to soil strains resulting from the various types of wave motion is normally not known, nor is the phasing of soil induced stresses with those due to end point motion known. Consequently, the assumption was made to combine these stresses on an SRSS basis. The analysis of the gas turbine fuel supply line was conducted using conservatively estimated soil properties since the actual overburden properties were unavailable.

5.6.2.2 Evaluation of Underground Piping

We made conservative assumptions throughout the analysis to provide a margin to account for possibly higher stresses resulting in other lines with somewhat different configurations and stress allowables. For instance, the soil strain of 1.8×10^{-4} in/in used is higher than expected from a detailed geotechnic investigation of the Millstone soil conditions for the 0.20 g SSE. Stresses were typically compared with ASME Code allowables; however, stresses above allowables were not computed for the gas turbine fuel supply line.

Maximum normal and shear stresses of 12.4 ksi and 5.4 ksi, respectively, were calculated in the pipe due to seismic wave propagation without the effects of discontinuities or end point motion (Refs. 32, 33). The resulting principal stress of 14.4 ksi may be compared with the ASME Code allowable of 45 ksi for the accident condition.

When an abrupt change in direction exists in a buried pipe, as in the case of the gas turbine fuel supply line (Fig. 34), axial strains induced in one leg will impose a normal load on the transverse leg. This load must be resisted by the stiffness of the pipe and soil surrounding the transverse leg which creates shear and bending stresses at the elbow. We calculated these stresses assuming the transverse leg acts as a beam on an elastic foundation with the coefficient of subgrade reaction determined from Ref. 34. The calculated deformation of the pipe relative

DRAF

to the soil at this location is 0.095 inch. The corresponding maximum stresses, calculated by including the code stress intensity factor were 32.6 ksi in the normal direction together with a shear stress of 2.9 ksi. The resulting principal stress of 33 ksi is well below the 45 ksi allowable.

Stresses in buried piping may also concentrate where the pipe enters a building or other large structure which may have some motion relative to the soil. The relative motion of the gas turbine building is unknown. It was originally analyzed as a fixed-base, single-degree-of-freedom system. Insufficient information is available concerning the soil and pile foundation system on which the building rests to permit a refinement of this analysis. We do not expect the relative motion of the gas turbine building to result in strains great enough to cause failure of the pipe, although some strains above yield could occur. Most of the other structures are founded on rock and the motions of the base slabs of these structures relative to the rock are very small. We, therefore, conclude that critical buried pipelines at Millstone are not expected to fail as a result of the postulated 0.20 g SSE.

5.6.3 Buried Tank

5.6.3.1 Buried Tank Confirmatory Analysis

As in the case of the buried piping, buried tanks apparently were not analyzed during the design of Millstone 1. As part of the SEP evaluation, a typical underground tank was analyzed in conjunction with the 0.20 g SSE. The tank selected for this evaluation was the gas turbine fuel oil storage tank (Fig. 35). The tank consists of a cylindrical shell with shallow spherical end caps. It is 12'-6" in diameter and 40 feet long. The tank was fabricated from 0.375 inch thick A283, Grade C, steel plate.

Tank deformations caused by the propagation of the elastic seismic waves were estimated from soil strains in the transverse, longitudinal, and vertical directions. Soil strains for the tank evaluation were determined in the same manner (Refs. 32, 33, 34) as described in Section 5.6.2 for the buried pipe, using a maximum soil

DRAFT

strain of $1.8x10^{-4}$ in/in in the analysis. Displacement limited stresses corresponding to these deformations were determined, and local stresses due to nozzle loads from the buried pipeline described in Section 5.7 were included. The stresses due to different directional components were combined on an SRSS basis.

5.6.3.2 Evaluation of Buried Tank

Normal stresses in the meridional and tangential directions together with shear stresses were determined for the tank deformations due to seismic wave propagation in the transverse, longitudinal, and vertical directions. Stresses due to nozzle loads were then superposed. Conservative assumptions were made throughout the analysis in order to account for the uncertainties in the soil properties. Seismically induced meridional, tangential, and shear stresses were found to be 4.4 ksi, 2.5 ksi, and 2.8 ksi, respectively. The maximum principal stress for the 0.20g SSE is 8.6 ksi which is well below the ASME Code allowable value of 42 ksi for the accident condition. Therefore, we conclude that the buried tanks will not be damaged by the postulated 0.20 g SSE.

5.6.4 Suppression Chamber - Ring Header, Torus, and Support System

An evaluation to determine the seismic capacity of principal components of the suppression chamber was also conducted using the fundamental frequency predicted by the original design model. Seismic response accelerations were determined in accordance with SEP guidelines (Ref. 4). This new response value was used to develop the resultant seismic stresses from the original dynamic analyses. The original dynamic analysis (Ref. 35) was performed for both 3 and 3.25 inch diameter tie rods. A subsequent report (Ref. 36), developed in conjunction with an evaluation of the pool dynamic loads, lists the sway-bar diameters as 3.75 O.D. The report also discusses modifications to increase the capacity of the inner columns and column end plates and pins. It is not clear that these modifications have been implemented. Therefore, the stresses reported here correspond to 3.25 inch diameter tie rods, since these stresses are larger.

DRAFT

Based on the design analysis, the fundamental frequency of the torus is 3.3 Hz. The spectral acceleration for the SSE using the 7% damped Regulatory Guide 1.60 spectrum is approximately 0.5 g. We compared this to the spectral acceleration of 0.41 g for 2% damping used in the original analysis. The results showed a load ratio of 0.82 assuming no higher frequency mode contributions. In other words, the seismic stresses are increased approximately 22%. Stresses for both the OBE and SSE for the torus shell are given (Ref. 7, Am. 17) for operating, accident, and flooded conditions, but no breakdown between seismic stress and operating stress is given. A load ratio of 0.82 is generally considered acceptable if non-ductile failure modes do not exist.

Elements of the torus support system including the pipe columns, tie rods, and tie rod connections were investigated further to the extent that detailed information was available. The result of the AISC interaction formulas for beam columns was calculated to be 1.0 compared to the short-term allowable value of 1.33. Maximum tensile stress across the gross tie rod cross-section was determined to be approximately 34 ksi. If the ends of the rods do not have upset threads, minimum yield strength of 36 ksi will likely be exceeded in the threaded region. Inelastic deformations in this region may be acceptable provided nonductile failure modes do not exist. End plate bearing stress and pin shear stress were determined by increasing the design values listed in Ref. 7, Am. 17. The end plate bearing stress of 23 ksi and the pin shear stress of 16 ksi are within allowable limits. Insufficient information was available to evaluate the vent system. If the modifications described in Ref. 36 have been or will be implemented, the seismic capacity of the torus support system will be increased above what is discussed here which is based on information in the FSAR. If the modifications have not been implemented, we recommend that the configuration of the tie-rod ends be investigated.

The predicted seismic stresses in the columns and torus are expected to be small compared to the stresses resulting from deadweight and the postulated loss-of-coolant accident (LOCA). As part of a separate program, the entire suppression chamber system is being evaluated for pool dynamic loads resulting from a LOCA or safety relief valve (SRV) actuations, as well as for seismic loads. This review is not part of the Systematic Evaluation Program. Therefore, we recommend that

DRAFT

the adequacy of the torus and column system be based on an evaluation which includes both the pool dynamic loads due to LOCA and seismic loads. The seismic loads used for this evaluation should be those resulting from an analysis performed in accordance with current seismic criteria.

5.6.5 Gas Turbine Building

The gas turbine building was originally analyzed as a single-degree-of-freedom system using a fixed-base cantilever beam model (Ref. 14). No effects of the flexibility of the overburden and pile system on which the structure is founded were considered. The analysis was conducted using the response spectrum from the 1952 Taft north 69° west earthquake record normalized to 0.07 g with 5% damping. The analysis was performed for the earthquake in two directions but only the critical (E-W) direction results were reported (Ref. 14). The period was calculated to be 0.089 seconds and the response at the roof was computed to be 0.121 g. It was recommended in Ref. 14, that the structure be checked for safe shutdown for twice the loads developed for the 0.07 g case in conjunction with a 0.10 g vertical component.

Based on a comparison with Regulatory Guide 1.60 spectra for 10% critical damping, we could expect a load ratio of approximately 0.7 for the structure if no frequency shift or amplification resulting from the overburden flexibility is considered. Details of the soil characteristics of the overburden, the depth to bedrock, or the building pile system details were unavailable. We consider it unlikely that these effects will be of sufficient magnitude to result in structure damage to the extent that the functionality of the critical systems would be impaired. However, the effects of the structure foundation flexibility should be included when considering the seismic input for these equipment systems; and the structural integrity should be verified once the pile and overburden information is available. When this seismic input is determined, the thrust load in the generator should be reviewed.

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CHAPTER 6: SEISMIC EVALUATION OF MECHANICAL AND ELECTRICAL EQUIPMENT AND FLUID AND ELECTRICAL DISTRIBUTION SYSTEMS

6.1 INTRODUCTION

6.1.1 Purpose and Scope

In this chapter, selected seismic evaluation data developed to qualify certain mechanical and electrical equipment will be reviewed along with fluid and electrical distribution systems of the Millstone 1 Nuclear Power Plant. Based on that review, the ability of the reactor to safely shutdown and remain in a safe shutdown condition in the event of an SSE will be evaluated. The SEP seismic review team purposely identified those components that are expected to have a higher degree of seismic fragility; moreover, the review team believes that these components are representative not only of those installed in the safe shutdown systems, but those in other seismic Category I systems, 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 the Millstone 1 Plant.

It should be noted that a majority of the electrical equipment calculations, furnished by Northeast Utilities, were recently made; as a result, substantial modifications in the support and anchorage of each of the electrical components was required. As such, the sample of electrical components evaluated in this report should not be considered representative, unless it can be demonstrated that all similar equipment required for safe shutdown or as part of an engineered safeguard has undergone a comparable review.

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

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As discussed in R.G. 1.48 (Ref. 37), and "Standard Review Plan," Sec. 3.9.3 (Ref. 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 that are required for safe shutdown and which must move during or after a seismic event, in order to perform their design safety function.

Typically found in the active category are:

- Pumps.
- Valves.
- Motors and associated motor-control centers.
- Switchgear.

Seismic design adequacy of active components should be shown by demonstration of safety function as well as by structural integrity. Adequacy may be determined by either analysis or by physical testing, but testing is generally preferred. 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 through analysis, deformations must be limited and predictable. Therefore, total stresses in such components are normally limited to the elastic linear range of 0.67 to 0.8 times the yield stress of the material, and in no case should total stress in the component be allowed to exceed yield stress.

Components determined to be passive considered in this report are those components required for safe shutdown for which the only safety functions are maintenance of leak-tight pressure boundaries or structural integrity during and following the SSE. Typically found in the passive category are:

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

DRAFT

- Transformers.

- Electrical-distribution systems.

In determining seismic design adequacy by analysis, the major 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, ranging from 1.0 times yield to 0.7 times ultimate strength of the material, are permitted (Ref. 18). As in the case of active components, higher stress limits are used for components constructed or manufactured in accordance with the ASME, BPVC-III (Ref. 18). Lower stress limits are used for components constructed or manufactured to other codes or standards.

In selecting the magnitude of seismic input for component evaluation, it is important to determie whether a component or distribution system is rigid or flexible. Seismic acceleration of equipment depends upon:

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

The designation of "rigid" or "flexible" may also depend on how a particular component is supported. Many rigid components must be evaluated as though flexible because of the flexibility of their support.

A review of the Millstone 1 reactor and turbine building floor response spectra, shown in Fig. A-1 to A-14, shows that equipment contained in the buildings may be considered rigid for frequencies greater than 25 Hz. For flexible components with fundamental frequencies less than 25 Hz, the maximum horizontal response occurs between 10 and 12 Hz for the reactor building. The maximum vertical response occurs between 12 and 14 Hz. The maximum spectral acceleration for 3% damping is approximately 15 times the SSE peak ground acceleration (PGA) of 0.2 g.

Components were first grouped as either active or passive, and rigid or flexible; then a representative sample of each group was evaluated to establish the factor of safety or degree of adequacy of that group's seismic design. In this way, factors of safety within groups of similar components were established without detailed reevaluation of hundreds of individual components.

DRAFT

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

- Selection was based on a walk-through inspection of the Millstone 1 facility by the review team. Based on their experience, team members selected components that appeared to have high seismic fragility for each component's category. Particular attention was paid to the component's support structure.

- Safe shutdown components were categorized into generic groups: tanks, heat exchangers, and pumps, all horizontally mounted; tanks, heat exchangers, and pumps, all vertically mounted; and motors and motor control centers.

The licensee was asked to provide seismic qualification data on selected components representative of each generic group.

The remainder of this chapter reviews the seismic capacity of the selected components and, where necessary, recommends additional analysis or hardware changes needed to qualify them for the SSE defined in this report. Based on detailed review of the seismic design adequacy of representative components, conclusions are developed as to the overall seismic design adequacy of seismic Category I equipment installed in Millstone 1. Table 12 and Sec. 6.4 summarize these conclusions.

6.1.2 Description of Components Selected for Review

Table 9 lists and describes components that the review team selected following its plant walk-through, as well as components representative of the generic groups of safety related components. Table 9 also provides the basis for each selection.

The review in this chapter emphasizes what are normally listed as auxiliary components. Deficiencies in seismic design condition tend to be found in the auxiliary equipment rather than in the major nuclear components. Auxiliary components are typically supplied by manufacturers who--unlike the nuclear steam-supply system vendors--may not have routinely designed and fabricated components for the nuclear power industry, particularly the time this plant was under construction. However, because of its importance to

DRAFT

safety, the seismic design adequacy of the reactor coolant system components and support structures are also evaluated, to the extent information was provided.

TABLE 9. Mechanical and electrical components selected by the review team for seismic evaluation and the basis for selection.

Item No.	Description	Reason for selection
	Mechanical Components	
1.	Emergency service water pump	This item has a long, vertical unsupported intake section which may be limiting for lateral loads which are seismically induced. In addition, material may be cast iron.
2.	Isolation (emergency) condenser	This item is a horizontally mounted component supported by three saddles that do not appear to be seismically restrained except at the center support. Concern was expressed about the saddle's ability to carry required seismic loads, particularly in the longitudinal direction.
3.	Shutdown heat exchanger	Horizontally mounted leading to concern regarding capability to carry the load in longitudinal direction.
4.	Emergency cooling water heat exchanger	This item is horizontally mounted on two saddles which do not appear to be seismically restrained.

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TABLE 9. Mechanical and electrical components selected by the review team for seismic evaluation and the basis for selection.

Item No.	Description	Reason for selection
5.	Recirculation pump support	This item is a vertical component supported by spring hangers whose functionality is critical in insuring reactor coolant system integrity.
6.	Emergency diesel oil storage day tank	Anchor-bolt system 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 air and electric motor-operated valves, particularly for lines 4 in. or less in diameter, is that the relatively large eccentric mass of the motor, when not externally supported, will cause excessive stresses in the attached piping. In addition, overstress and excess deformation of valve yoke and stem may also occur.
8.	CRD hydraulic control system including tubing and supports	Item is particularly critical to insure reactor coolant system integrity.
9.	Reactor vessel, supports, and internals	Same as Item 8.
	Electrical Components	
10.	Battery racks	The bracing required to develop lateral load capacity may not be sufficient to carry the seismic load.

DRAFT

TABLE 9. Mechanical and electrical components selected by the review team for seismic evaluation and the basis for selection.

Item No.	Description	Reason for selection
11.	Motor control centers	Typical seismically qualified electrical equipment. Functional design adequacy may not have been demonstrated. In addition, anchorage to support structure may not be adequate and might permit sliding or overturning during a seismic event.
12.	Transformers	Same as Item 11.
13.	Switchgear panels	Same as Item 11.
14.	Control room electrical panels	The control panels appear adequately anchored at the base. However, there appear to be many components cantilevered off 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.
15.	Diesel generator remote control board	Same as Item 11.
16.	Battery room distribution panels	Same as Item 11.
17.	Electrical cable raceways	The cable tray support system does not appear to have positive lateral-restraint load-carrying capacity.

DRAFT

6.2 SEISMIC INPUT AND ANALYTICAL PROCEDURES

6.2.1 Original Seismic Input and Behavior Criteria

6.2.1.1 Piping and Equipment

The governing design and construction codes, used for piping and equipment are summarized below:

TABLE 10. Design and construction codes for piping and equipment.

Component	Design code	
Containment	ASME BPVC Section III, Class B, 1965	
Reactor pressure vessel and internals	ASME BPVC Section III, Class A, 1965	
Reactor pressure vessel supports		
Structural steel-----------------	AISC - 1963	(Allowable behavior criteria for SSE loading case not defined explicitly)
Reinforced concrete--------------	ACI - 318 - 1963	
Recirculation loop, piping	ANSI-B31.1 - 1955 & Code Cases (Equiv. B31.1 - 1967)	
Emergency core cooling system (ECCS)	Stresses remain within code allowable,	
Piping---------------------------	ANSI-B31.1 - 1955 plus code cases	
Pump-----------------------------	ASME Section III, Class C	
LPCI exchangers------------------	ASME Section III, Class C and TEMA C, shell side. ASME Section VIII, TEMA C on tube side. (The secondary and primary plus secondary stresses are examined on a rational basis taking into account elastic and plastic strains. These strains are limited so as to preclude failure by deformation which would not only compromise any of the engineered safeguards but also prevent safe shutdown of the reactor.)	

DRAFT

The allowable stresses for Category I piping are as given below:

	Loading condition	Allowable stress
1.	Thermal Expansion	S_A
2.	M.O.L. + S.L.	S_h
3.	M.O.L. + 2 x S.L.	(Stress such that safe shutdown can be achieved)

where

M.O.L. = Maximum Operating Loads including design pressure and temperature, weight of piping and contents including insulation and the effect of supports and other sustained external loadings

S.L. = Seismic loads due to the design earthquake (OBE)

2 x S.L. = Seismic loads due to twice the design earthquake (SSE)

S_A = $f(1.25\ S_C + 0.25\ S_h)$

where:

f = stress range reduction factor for cyclic conditions

S_C = allowable stress in cold condition per ASA B31.1

S_h = allowable stress in the hot condition (design temperature) per ASA B31.1

For the reactor vessel supports, the allowable stresses are given below:

1. OBE: = normal AISC allowable stresses.
2. OBE + Jet: 150% of normal AISC allowable stresses.
3. 2 OBE: 150% of normal AISC allowable stresses.

DRAFT

Except for the containment, reactor pressure vessel and the isolation condenser, which are considered rigid (fundamental frequency 20 Hz), equipment in the reactor building was analyzed for a single horizontal component of acceleration as a function of height (determined from Fig. 4-3) combined with a constant vertical acceleration of 0.05 g for OBE and 0.15 g for SSE. The horizontal acceleration was applied along each of the two principal axes of the equipment components. The worst case combination of horizontal and vertical excitation was used to evaluate design adequacy of the equipment. For flexible equipment, an amplification factor as determined for piping in the reactor building was used to increase the horizontal seismic input to equipment. For Seismic Category I or Category I equipment in the turbine building which is considered rigid, a horizontal seismic coefficient equal to 0.16 g (for OBE) was specified. For equipment where the degree of rigidity was not determined, the horizontal seismic coefficient specified was assumed to be 0.22 g for OBE.

6.2.1.2 Safety Instrumentation Seismic Qualification Test Program

The proposed original seismic program for the qualification of procured Safety Instrumentation is described in Sec. A.7, AM. 18 of the PSAR and is repeated below.

1. <u>Introduction</u>

   The following describes the proposed program for assuring that Category I instrumentation meets the seismic requirements.

2. <u>Systems</u>

   The Category I instruments for the following essential systems are designed, and will be analyzed and tested to ensure performance of their primary functions without spurious response during and after an earthquake:

   Reactor Protection System

   Nuclear Boiler System

   CRD Hydraulic System

   Standby Liquid Control System

   Neutron Monitoring System

   Emergency Core Cooling Systems

   Process Radiation Monitoring Systems

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3. Criteria

a. Design Basis Earthquake, DBE (Current SSE)

For the design Basis Earthquake for rigid body calculations, the seismic force assumed to act on the equipment's center of mass will have the following components:

Horizontal	1.5 times the weight
Vertical	0.14 times the weight

b. Operational Basis Earthquake (OBE)

The maximum stresses from combined seismic and normal loads shall not exceed allowable stresses without the usual one-third increase of allowable stress for short term loading. The seismic loads for such analyses are:

Horizontal	0.75 times the weight
Vertical	0.07 times the weight

c. Acceptance

The product being evaluated must perform its prescribed functions during and after the application of seismic forces.

d. Schedule

The above will be accomplished for G.E. supplied instrumentation prior to startup. The EBASCO supplied instrumentation is so similar to the G.E. supplied instrumentation that the results of this seismic test program are applicable without extrapolation.

4. Evaluation

a. Devices

All types of Category I devices (relays, switches, amplifiers, power supplies, sensors, etc.) which make up the Category I systems will be testea for proper performance under the simulated seismic accelerations of the Design Basis Earthquake of 3.1 g. Each device tested will be energized and, as applicable, will have a simulated input signal applied and will have its output monitored during and after the test.

DRAFT

The test will consist of vibrating the devices to the DBE accelerations over the DBE frequency range on each of the devices' three rectilinear axes.

b. Racks and Panels

Category I racks and panels complete with all internal wiring and devices mounted will be vibrated at low accelerations over the DBE frequency range and measurements made to determine the presense of resonances. If resonances are present which affect Category I devices, steps will be taken to shift their frequencies out of the band of interest or dampen them to an acceptable level. Once this is accomplished, the panel can be considered a rigid body and analyzed statically.

c. Code Devices

All instrument devices required to conform to ASME Boiler code requirements will be analyzed as required by the applicable Code. In general, these devices are large, strong structural or pressure bearing instruments which would not be noticeably stressed at the low seismic accelerations but, rather, should be analyzed at the combined loading of their in-situ forces plus the seismic loads.

6.2.2 Current Seismic Input

Current seismic input requirements for determining the seismic design adequacy of mechanical and electrical equipment and of distribution systems are normally based on floor or equipment response spectra for the various elevations at which the equipment is supported. The floor spectra, which are based on R.G. 1.60 spectra modified by the dynamic characteristics of the building, are shown in Figs. A-1 through A-14. The floor spectra are based on the building models shown in Figs. 21, 23, 24, and 30.

For evaluating mechanical and electrical equipment, a composite 7% damping, is used for the 0.2 g SSE. For piping evaluation, the damping associated with the SSE is limited to 3%. These values are consistent with a recent summary of data directed toward defining damping as a function of stress level (Ref. 38). For cable trays, recent tests seem to indicate that damping levels depend greatly on the tray and support construction and on the

DRAFT

TABLE 11. Current structural behavior criteria for determining seismic design adequacy of passive mechanical and electrical equipment and distribution systems.

Components	Current Criteria (0.2 g ZPGA input)	
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)
	$S_{m_{all}} \leq 0.5\ S_u$ and $1.25\ S_y$	ASME III Class 2 (NC 3321)
	$S_{m_{all}} \leq 0.5\ S_u$ 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	
	$S_{m_{all}} \leq S_u$ 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	$S_{all} \leq 1.2\ S_y$ or $0.7\ S_u$	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 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

manner in which cables are placed in the trays. Damping may be as high as 20% of critical damping (Ref. 39). A damping factor of 0.5% was used for analyzing the sloshing mode of fluids contained in tanks. Horizontal seismic loads are assumed to act simultaneously. Depending on the geometry of the component being evaluated, the resultant horizontal load will vary from 1.0 to 1.4 times the individual component load. We have applied the conservative 1.4 factor in this evaluation except where design adequacy is in question.

6.2.3 Current Behavior Criteria

Seismic Category I components designed to remain leak-tight or retain structural integrity in the event of an SSE are now typically designed to ASME, Sec. III Code, Class 1, 2 or 3 stress limits for Service Condition D. Stresses in supports for ASME leak-tight components are limited as shown in Appendix F or Appendix XVII of the ASME, Sec. III Code (Ref. 18).

If qualification is to be by analysis, active ASME, Sec. III components that must perform a mechanical motion to accomplish safety functions must typically meet ASME Sec. III Code, Class 1, 2, or 3 stress limits for Service Condition B. Supports for these components are also typically restricted to Service Condition B limits.

For other passive and active equipment not designed to ASME, Sec. III Code requirements, and for which the design, material, fabrication, and examination requirements are typically less rigorous than ASME, Sec. III Code requirements, allowable stresses for passive components are limited to yield values and to normal working stresses (typically 0.5 to 0.67 of yield). SEP behavior criteria used in reevaluation of various equipment and distribution systems for Millstone 1 passive components are given in Table 11. For electrical components such as switches, relays, etc., functional adequacy should be demonstrated by test.

Experience in designing such pressure retaining components as vessels, pumps, and valves to ASME, Sec. III code requirements for 0.2 g, indicates that stresses induced by earthquakes seldom exceed 10% of the dead weight and pressure-induced stresses in the component (Ref. 40). Therefore, design adequacy of such equipment is seldom dictated by seismic design.

DRAFT

Seismically induced stresses in nonpressurized mechanical and electrical equipment, fluid and electrical distribution systems, and in all component supports may be significant in determining design adequacy. Note that SSE loadings seldom control design of piping systems. Because of more restrictive stress and damping limits, the OBE normally controls design of piping.

6.3 EVALUATION OF SELECTED COMPONENTS FOR SEISMIC DESIGN ADEQUACY

6.3.1 Mechanical Equipment

6.3.1.1 Emergency Service Water Pump

The emergency service water pump and motor unit is oriented vertically in the Intake Structure. As shown on Ingersoll-Rand Co., Drawing W-55APMA86X10-A, the intake portion of the pump extends downward from the discharge head and pump base for a distance of 27 ft 0 in. Although the original seismic analysis was not furnished, the equipment specification indicates that the equipment was originally designed to withstand seismic accelerations of 0.11 g in the horizontal direction and 0.05 g in the vertical direction.

The pump-motor unit is located at grade; therefore, the seismic input is essentially the R.G. 1.60 ground response spectrum normalized to 0.2 g. Tensile and shear stresses in the pump base anchor bolts as a result of overturning were determined, as well as stresses at the attachment of the intake column pipe to the discharge head (Ref. 41).

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 and shaft was found to have a fundamental frequency of 2.17 Hz. At this natural frequency, the spectral acceleration from the R.G. 1.60 response spectra normalized to 0.2 g for 7% damping is 0.54 g. The seismic accelerations were applied to the pump considering simultaneous N-S and E-W loading, and the resulting anchor bolt stresses were determined. The effects of attached piping nozzle loads due to normal operation were not available. However, the emergency service water line is a cold line; therefore, it tends to transfer small pressure and thermal loads onto the pump.

The stress calculated at the flange connecting the discharge head to the intake column pipe is 8,200 psi. This stress level is within Condition D service limits allowable stresses even if the pump head is of cast iron. However, the shear stresses (49,940) on the anchor bolts, which are assumed to be A307, exceeds the stress limit set by ASME III for a Service Condition D.

DRAFT

Insufficient detail has been provided to evaluate the functional adequacy of the pump to include motor impeller shaft deformities and for bearing or coupling failure.

We believe that the ESW pump-motor, designed as a passive component, will not withstand a 0.20 g SSE, unless the bolting is revised.

6.3.1.2 Emergency (Isolation) Condenser

The emergency condenser, supplied by Struthers Wells Corporation, is located in the Reactor Building at El. 82'9". It is 49 ft. long, mounted horizontally, and supported by three saddles. The original seismic design, based on 0.18 g horizontal acceleration and 0.10g vertical acceleration, was performed by John A. Blume and Associates, and is given in Refs. 42 and 43.

The response spectra for 7% damping (Figs. A-4 and A-6) at El. 82'9" of the reactor building are considered applicable for verifying seismic design adequacy. When the component and its support system were assumed to be rigid, the resultant input horizontal and vertical seismic accelerations are 0.42 and 0.25 g, respectively.

Our evaluation assumed that only the center support would carry longitudinal shearing stress, because the bolt holes in the other supports are slotted to provide for thermal growth. Since the center support is assumed to take the total longitudinal shear load plus one-third the transverse shear load, the shear stress in the 8-2 1/2 in. A307 support bolts indicated in Ref. 44 is 4.36 ksi per bolt for combined N-S and E-W earthquake loading. Since this stress is within the allowable ASME Service Conditions D shear stress of 17.4 ksi, we believe that the anchorage system for the emergency condenser is adequate to withstand the 0.2 g PGA SSE seismic loading.

6.3.1.3 Shutdown Heat Exchanger

The shutdown heat exchanger is a horizontal component, 20 ft. in length and supported by three saddles. The exchanger is located in the Reactor Building at El. 42.5' and was supplied by Struthers Wells Corporation. Although the original seismic design was not supplied, the equipment specification required that the equipment and supports be designed to withstand a 0.10 g horizontal acceleration.

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The response spectra for 7% damping (Figs. A-2 and A-5) generated at El. 42'6" of the Reactor Building are considered applicable for verifying seismic design adequacy.

An independent review of the dynamic characteristics of the exchanger (Ref. 45) confirms that in both the transverse and longitudinal directions the equipment and its supports are rigid. The frequency characteristics in the longitudinal direction are based on the assumption that only the middle saddle can restrain base shear and moment reactions in the longitudinal direction, because it is assumed that the holes in the two outside support saddles are slotted to permit thermal movement. The determination of exchanger support system frequencies in the relatively rigid range that includes torsion gives a maximum horizontal seismic input of 0.30 g. The analysis established a safety margin to ASME Condition D stress limits of 0.15 for the anchor bolts. No analysis of the exchanger internals was performed, since the necessary information was not made available. It should also be noted that no evaluation has been made of nozzle loads in the exchanger, since the piping system analysis is not currently available. It has been generally found that such piping loads seldom have a significant effect on the support loads. Therefore, we believe that the shutdown cooling heat exchanger will withstand a 0.2 g SSE seismic event without loss of structural integrity.

6.3.1.4 Emergency Cooling Water Heat Exchanger

The emergency cooling water heat exchanger is located in the Reactor Building at El. 42'6". It is 34'0" long, mounted horizontally, and supported by two saddles. It was supplied by Struthers Wells Corporation. The original seismic design calculations, based on a 0.30 g horizontal and 0.05 g vertical acceleration (Ref. 46) were not furnished.

The response spectra for 7% damping (Figs. A-2 and A-5) at El. 42'6" of the reactor building were used for verifying seismic design adequacy. When the component and its support system are assumed to be rigid, the resultant input horizontal and vertical seismic accelerations are 0.3 g and 0.18 g, respectively.

The seismic qualification of the exchanger was performed as described in Ref. 47. In the transverse direction, the exchanger and its support system are considered rigid. The seismic forces in the transverse direction were

DRAF

applied to the exchanger, and the resulting shell, saddle, and anchor bolt stresses were determined (Ref. 47). No attempt was made to evaluate nozzle loads on the exchanger as they were not furnished and seldom affect support integrity. Stresses calculated were well within ASME III Service Condition D code allowables.

In the longitudinal direction, the exchanger is restrained by slotted holes only. Since it is not positively anchored against sliding, lateral stability of the exchanger in the longitudinal direction must be provided by friction alone.

We believe that all seismically qualified components should be positively anchored against earthquake forces. Alternatively, the component may be analyzed considering both friction forces and the potential for non-linear response associated with rocking and sliding. If this analysis were performed, potential impact effects on the anchor bolts should also be considered. However, we recommend that the slotted holes in one of the tank saddle supports be modified to provide positive lateral restraint in the longitudinal direction. The remaining slotted holes would allow thermal expansion at the other saddle.

6.3.1.5 Recirculation Pump Support

The stress report reviewed (Ref. 48) for recirculation pump support integrity addressed a pump which was specified for Pilgrim Station plant. Presumably, the specified pump is similar to that installed in Millstone 1. Results of indicate that the DVSS recirculation pump is well within code allowable limits for a seismic input of 1.5 g in the horizontal direction and 0.14g in the vertical direction. This horizontal input is approximately 1.4 times that applicable to the Millstone 1 pump, while the vertical input is approximately 0.33 that applicable at Millstone 1. Using a factor of 1.6 times the normal allowable stress as shown in Table 6-2, the minimum safety factor in the bolts in the pump body was 1.6. The vertical seismic response has little effect on bending moments applied to the pump body; hence, it is not a controlling factor in its seismic capacity.

Each pump is supported by three spring hangers and five snubbers as shown in G.E. Drawing 718E979. Seismic design loadings in the snubbers as a function of acceleration level have not been defined.

DRAFT

As a result of the review of the seismic design adequacy of the referenced recirculation pump, it has been determined that the pump body is capable of carrying the seismic loads defined in this report with no loss of structural or leak-tight integrity. Insufficient information has been provided to determine seismic loads on the pump snubber supports; hence, their seismic design adequacy has not been determined. The licensee should verify that the pump support reviewed is applicable to the recirculation pumps at Millstone 1.

Insufficient detail concerning operating characteristics of the pump and internal design has been provided to determine continued function of the pump in the event of an earthquake.

6.3.1.6 Oil Storage Day Tank

The emergency diesel oil storage day tank is a cylindrical vessel 14' 2" tall and 5' 0" in diameter. The tank, which has a wall thickness of 1/4", is restrained by bolts. Since original design calculations for the tank were not furnished, the number and size of bolts are unknown.

The tank, which is supported at El. 34' 6" of the Turbine Building, was evaluated as shown in Ref. 49 for the horizontal response spectra at 7% damping (Fig. A-13). The dynamic analysis considered the effective convection and impulsive response of the contained fluid. The resulting fundamental frequencies were 0.77 Hz for the tank under convective loading (0.5% damping) and 39.9 Hz for the tank bending and shear deformation under impulsive loading (tank considered full). Therefore, the tank can be considered rigid for the impulsive loading effect.

The analysis determined gross dynamic characteristics of the tank. The evaluation shows that the oil storage tank will not slide or overturn even without anchor bolts. If friction were to be overcome, the resulting anchor bolt safety factor in shear would be 1.77, using ASME Condition D stress limits. The safety factor is 34.13 for compressive stress in the tank wall due to combined seismic overturning and deadweight stresses.

Therefore, we believe that the emergency diesel oil storage tank will withstand the 0.2 g SSE loading without loss of structural integrity.

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6.3.1.7 Motor Operated Valves

No data, other than a General Electric Purchase Specification (Ref. 50), was furnished by the licensee. The conclusions reached indicate that the original seismic acceleration levels were 0.17 g in the horizontal direction and 0.05 g in the vertical. We considered this level of seismic excitation for a 0.2 g SSE to be several times smaller than would typically be determined if the piping systems were evaluated using floor response spectra.

It has been the experience of the review team (see Ref. 51) that, for lines smaller than 4 to 6 in. in diameter, the eccentricity of motor-operated valves may cause additional and significant piping stresses (in excess of 10% of code allowable) and should be considered in the computation of total stresses. The applicable 10% stress levels (Ref. 18) are Class 2, Condition B for active valves and Condition D for valves where only pressure boundary integrity is required (~ 1800 and 3600 psi, respectively, for typical ferritic piping material). This tendency is increased as the lines become smaller.

Therefore, it is recommended that the licensee evaluate the stresses induced in the supporting piping from motor-operated valves required to move or change state when performing their safety function. The licensee should show that the stresses are less than 10% in typical pipe lines in the 1-in. to 4-in. diameter range. If not, the total stresses at motor operated valve locations should be calculated to determine that they are within Condition B code allowables (Ref. 18). For nonactive valves, Condition D service levels would apply. Alternatively, the review team recommends that a requirement to support motor-operated valves external to the pipe on lines smaller than 4 in. in diameter should be developed and implemented on Millstone 1.

6.3.1.8 CRD Hydraulic Control System Including Tubing and Support System

We reviewed the generic design specification for the control rod drive system, (Ref. 52). Section 5.5 of this specification states that the system shall be designed and constructed to withstand earthquake loads "per project requirements," and that structural design shall be "based on all loads including seismic." However, neither the specific seismic design loads for Millstone 1 nor the acceptance criteria for seismic load is defined.

DRAFT

A number of drawings showing system layout and fluid system design parameters have also been reviewed. One of the drawings provided (G.E. APED 846D692) gives structural details of the hydraulic control unit. In particular, Section B-B shows anchor details for the units; structural dimensions for the unit frames are not defined. While no calculations or analysis were submitted for review on Millstone 1, we are aware of a generic General Electric Co. report No. DAR 149 dated November, 1972 which covers the seismic design of hydraulic control units. This report may not be applicable to the units installed at Millstone 1; therefore, we have not attempted to review its content.

Insufficient information is available on the hydraulic control units and their structural supports to permit evaluation of their seismic design adequacy.

6.3.1.9 Reactor Vessel Supports

The seismic design of the reactor vessel support is discussed in answer to SAR Question A.6 AM.17. (Ref. 53). The stresses are reported as 11.2 Ksi in the anchor bolts for the SSE equivalent loading case of 0.8 g, well within the code allowable for tension. The stresses in the ring to skirt bolt appear to be within the pretension levels at the 0.8 g level. Stresses in the truss members for the upper lateral support have not been defined.

From page A-97 of CENC 1126 (Ref. 54) the stresses determined in the reactor vessel support skirt are 3.5 Ksi for the dead load plus OBE seismic load. Assuming that the same 2.4 factor used in evaluating the vessel is applicable to the skirt support, the applied stress in the skirt would be 2.4 x 3.5 or 8.4 ksi, which is less than the 18.7 ksi allowable.

The basis of the 11.2 Ksi stress reported in answer to SAR Question A.6 AM.17. has not been reviewed nor have the resultant stresses in the upper lateral support truss been defined. Except for these uncertainties it would appear the reactor support system is adequate for the SSE level defined in this report.

6.3.1.10 Reactor Vessel and Internals

Based on our review of the analytical report for the Millstone 1 reactor vessel as given in Ref. 54 and the Earthquake Analysis reported in Ref. 55, the original analysis appears to be as described below.

DRAFT

The reactor vessel was designed for a 455 Kip shear load and 12079 K-ft overturning moment corresponding to the 0.07 g PGA horizontal earthquake. These values were modified results of a seismic analysis which considered building movement. The final design accelerations were determined by the SRSS combination of rigid body ground acceleration and a one degree of freedom relative acceleration. The effects of earthquake loads were converted to an equivalent uniform vertical load on the vessel. In this conversion the effect of shear on stress intensity seems to have been neglected. In addition, there appears to have been a calculation error on page A-81 which would increase the seismic effect by a factor of 2.0 or the effective vertical load by a factor of one third.

Since the original seismic loads contribute less than 10 percent of the total primary stresses, changes in seismic loads have a relatively minor effect on the reactor vessel. Even when accounting for the increases in seismic loading indicated above, the total stresses in the vessel are well within allowable values.

The analysis of the reactor internals and in particular the shroud support as shown in Ref. 54 has been reviewed. Original Seismic loadings on the shroud support structure are the same 0.4 g horizontal and 0.08 g vertical load as were used in the vessel analysis hence no amplification of seismic input by the vessel was considered. On page A-629 of the Ref. 54 calculations the maximum primary stress in the shroud support was determined to be 5000 psi. Given that the maximum vertical earthquake load may be as high as 1.6 times the dead weight load and vertical loading is the principal primary stress contributor to stresses in the shroud support the maximum resultant stress should not exceed 8000 psi which is well within code allowables.

It can be concluded, therefore, that the reactor and reactor internals (shroud-support) should be able to withstand the SSE level of seismic loading defined in this report without loss of function within the allowable behavior limits prescribed in Table 10.

6.3.2 Electrical Equipment

6.3.2.1 Battery Racks

The calculations and drawings, furnished by the licensee, for the

DRAFT

battery racks used in the Millstone 1 plant are from a recent redesign (Ref. 56). These stationary battery cells where analyzed using horizontal and vertical accelerations of 1.0 g.

The horizontal response spectra for the battery racks, which corresponds to El. 34' 6" of the turbine building are given in Fig. A-13.

Peak spectral accelerations are substantially lower than those used in the analyses; however, it would appear that positive anchorage against battery sliding has not been included and that the only lateral stability of the rack in the longitudinal direction is developed by friction. In our opinion, positive anchorage against earthquake forces is required, unless the component is analyzed considering both friction and the potential for non-linear response associated with rocking and sliding.

In addition, indicated stresses account for the occurrence of one horizontal and one vertical earthquake only and do not consider the seismic input horizontal loads as independent components simultaneously applied.

We believe that the positive anchorage indicated above should be added. A new analysis, using the proper load combinations should be performed before determining the structural design adequacy of the racks.

6.3.2.2 Motor Control Centers

General Electric Co., which supplied the 250V DC motor control centers for Millstone 1, performed a generic test on motor control units (Ref. 57). These tests were performed using an input of 0.5 g at the base, through a frequency band width from 5 to 500 Hz. The tests demonstrated that the resonant level of the equipment structure appears to be at 5-6 Hz. In this frequency range, the floor spectra for the Turbine Building at El. 34' 6" (Fig. A-13) indicate accelerations of 0.85 g for the N-S direction and 0.75 g for the E-W direction. These accelerations are substantially higher than those used in the test. However, the licensee recently performed new calculations (Ref. 58) which resulted in additional supports and braces.

Based on a review (Ref. 59) of both the furnished test results and calculations, the review team believes that the D.C. Motor Control Centers in the Reactor and Turbine Buildings will withstand the 0.2 g SSE, if their supporting systems are modified as indicated in Ref. 58.

DRAFT

6.3.2.3 Transformers

No evaluation has been performed since insufficient drawings or design calculations are currently not available. However, the available drawings indicate that the 480V A.C. switchgear emergency transformer is not positively anchored. It is recognized that modifications are in progress; however, anchorage details have not been provided.

6.3.2.4 Switchgear Panels

The Switchgear located in the Turbine Building at EL. 34' 6", was recently analyzed by Northeast Utilities Service Co. (NEU) for a 1.0 g SSE (Ref. 60).

The peak horizontal acceleration levels obtained from Fig. A-13, are 0.85 g and 0.75 g for the E-W and N-S directions, respectively. Since these are below the levels used by NEU in their calculations, a review of the NEU data was performed with the following observances:

- The additional bracing added has been attached to a concrete block wall.
- No proof has been furnished that the above referenced block wall has been seismically qualified.
- The switchgear and its anchorage, as modified, will withstand the 0.2 g SSE without loss of structural integrity.

As a result, the review team believes that the switchgear panels, when modified as indicated in Ref. 60 will withstand the postulated SSE, if it can be demonstrated that the attachment walls are qualified.

6.3.2.5 Control Room Control Panels

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

DRAFT

6.3.2.6 Diesel Generator Remote Control Board

As with the switchgear panels (Section 6.3.2.5) the diesel generator remote control board was recently analyzed by Northeast Utilities Service Co. and modifications were introduced (Ref. 61).

The control board, located at El. 14' 6" of the Turbine Building, was analyzed by NEU for an SSE seismic acceleration of 1.0 g which compares favorably with the N-S and E-W peak response at El. 34' 6" of the Turbine Building--0.85 g and 0.75 g, respectively.

A review of Ref. 61, indicates that the additional bracing added is attached to a concrete block wall. It is not clear that this wall has been designed to withstand the 0.2 g SSE. Based on the above and on Ref. 62, the review team believes that the diesel generator remote control board, when modified as indicated in Ref. 61, and if it can be demonstrated that the attachment walls are qualified, will withstand the postulated SSE.

6.3.2.7 Battery Charger

The battery chargers (1, 1A, 11A) were recently analyzed by NEU Service Co. (Ref. 63) and modifications were made. The NEU analysis was performed using a 1.0 g acceleration for both horizontal directions and an acceleration somewhat less than 1.0 g for the vertical direction. A comparison to the peak response obtained for the Turbine Building at El. 34' 6" (7% damping), which is 0.85 g and 0.75 g for the E-W and N-S directions, respectively (Fig. A-13), together with a thorough review (Ref. 64) of the NEU calculations and modifications demonstrates that the battery chargers would withstand the 0.2 g SSE without loss of structural integrity.

6.3.2.8 Electrical Cable Raceways

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

6.4 SUMMARY AND CONCLUSIONS

Table 12 summarizes our findings on the sample of mechanical and

DRAFT

electrical components and of distribution systems that were evaluated to determine the seismic design adequacy of such items required for the saf shutdown of the Millstone 1 nuclear steam supply system. As discussed in Sec. 6.1 of this report, the sample includes components the review team selected, based on judgement and experience, as representative of lower-bound seismic design capacity of Millstone 1, as well as the grouping of components into representative categories.

Based upon the design review and independent calculations for the SEP seismic load condition, we recommend that design modifications or reanalysis may be required for particular mechanical and electrical components to withstand the 0.2 g SSE without loss of structural integrity as required for maintaining safety functions. In general, no information that has been provided demonstrates the functional adequacy of mechanical and electrical equipment evaluated on the Millstone 1 Plant. Based on design data we have evaluated, the particular mechanical and electrical components that require additional evaluation and possible design modification are as follows:

1. Emergency service water pump.
2. Emergency cooling water heat exchanger.
3. Recirculation pump.
4. Motor operated valves.
5. CRD hydraulic control units.
6. Battery racks.
7. Motor control centers.
8. Transformers.
9. Switchgear panels.
10. Diesel generator remote control board.
11. Battery chargers.
12. Electric cable raceways.

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TABLE 12. Conclusions regarding equipment selected for review for seimic design adequacy of Millstone Unit 1.

Item	Description	Conclusion and Recommendation
1.	Emergency service water pump	Bolting must be revised. Functional integrity has not been evaluated due to lack of design detail. We also recommend the replacement of any cast iron components used.
2.	Emergency condenser	O.K.
3.	Shutdown heat exchanger	O.K.
4.	Emergency cooling water heat exchanger	The exchanger is not positively restrained in the longitudinal direction. Either a more rigorous analysis is required or the tank requires the addition of a longitudinal restraint.
5.	Recirculation pump support	O.K. for structural integrity; however, no evaluation has been performed to determine seismic loads on the pump snubber supports or to evaluate its functional integrity since sufficient data is unavailable.
6.	Emergency diesel oil storage day tank	O.K.

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TABLE 12. Conclusions regarding equipment selected for review for seismic design adequacy of Millstone Unit 1.

Item	Description	Conclusion and Recommendation
7.	Motor operated valves	Generic analysis on motor-operated valves on lines 4 in. or less in diameter should be performed to show 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 4 in. or less in diameter would be externally supported. Seismic testing results supplied on motor operators do not demonstrate functional adequacy for Millstone 1.
8.	CRD hydraulic control system including tubing and supports	There is insufficient information concerning the Millstone 1 hydraulic control units and their structural supports to permit evaluation of their seismic design adequacy.
9.	Reactor vessel, supports, and internals	O.K.
10.	Battery racks	New analysis is required along with the addition of longitudinal restraints for the batteries.
11.	Motor control centers	O.K. for structural integrity if block wall to which supports are attached can be demonstrated to be seismically qualified. No information on function.

DRAFT

TABLE 12. Conclusions regarding equipment selected for review for seismic design adequacy of Millstone Unit 1.

Item	Description	Conclusion and Recommendation
12.	Transformers	No evaluation has been performed since insufficient drawings or structural integrity design calculations are currently available.
13.	Switchgear panels	No evaluation has been performed since insufficient drawings or structural integrity design calculations are currently available.
14.	Control room electrical panels	No structural integrity evaluation has been performed since no drawings or design calculations are currently available. Functionality has not been demonstrated.
15.	Diesel generator remote control boards	O.K. for structural integrity if block wall to which supports are attached can be demonstrated to be seismically qualified. No information on function.
16.	Battery room distribution panels	O.K. for structural integrity if block wall to which supports are attached can be demonstrated to be seismically qualified. No information on function.
17.	Electrical cable raceways	No evaluation has been made since no drawing or design calculations are currently available. However, it is recommended that lateral restraint be provided unless design adequacy is demonstrated.

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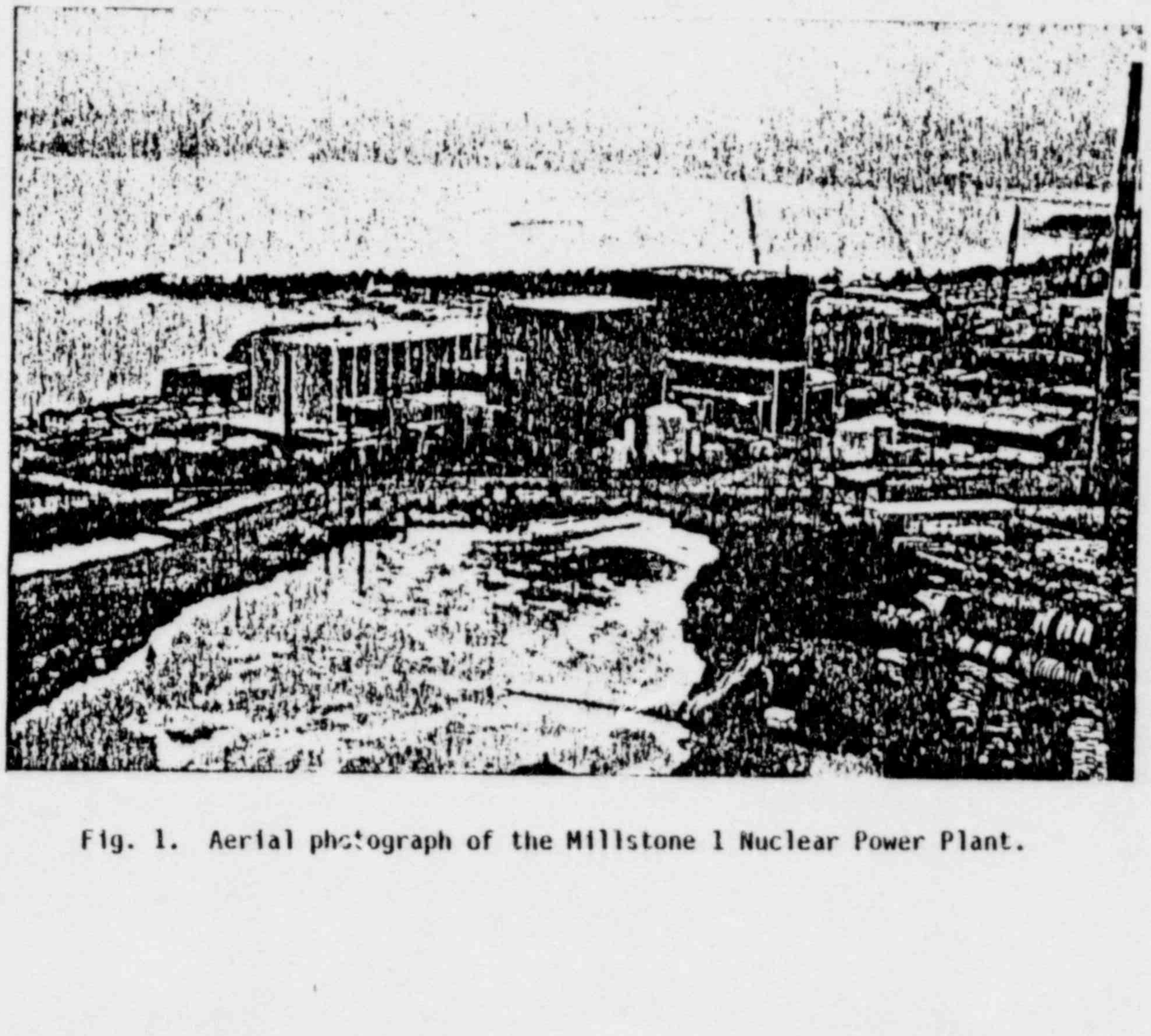

Fig. 1. Aerial photograph of the Millstone 1 Nuclear Power Plant.

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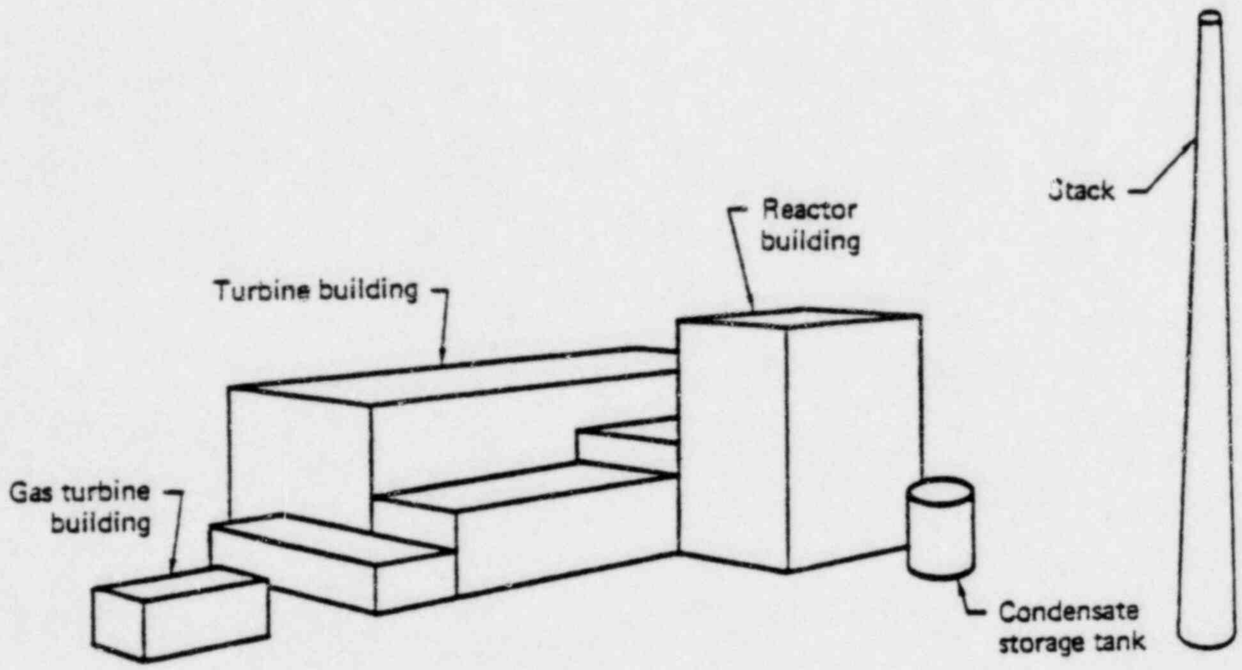

Fig. 2. Isometric of Millstone 1 plant showing major structures.

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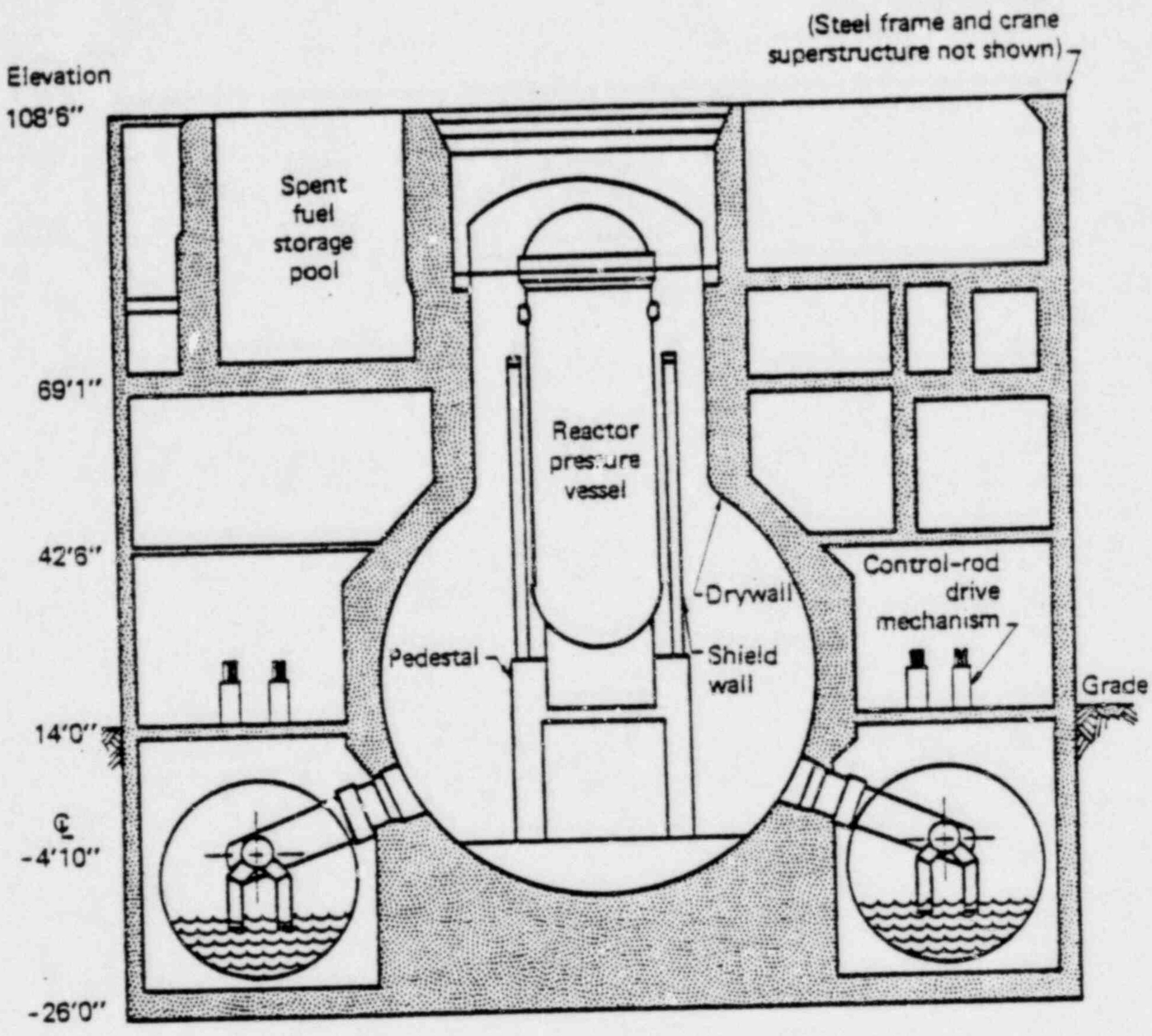

Fig. 3. Cross section of the reactor building.

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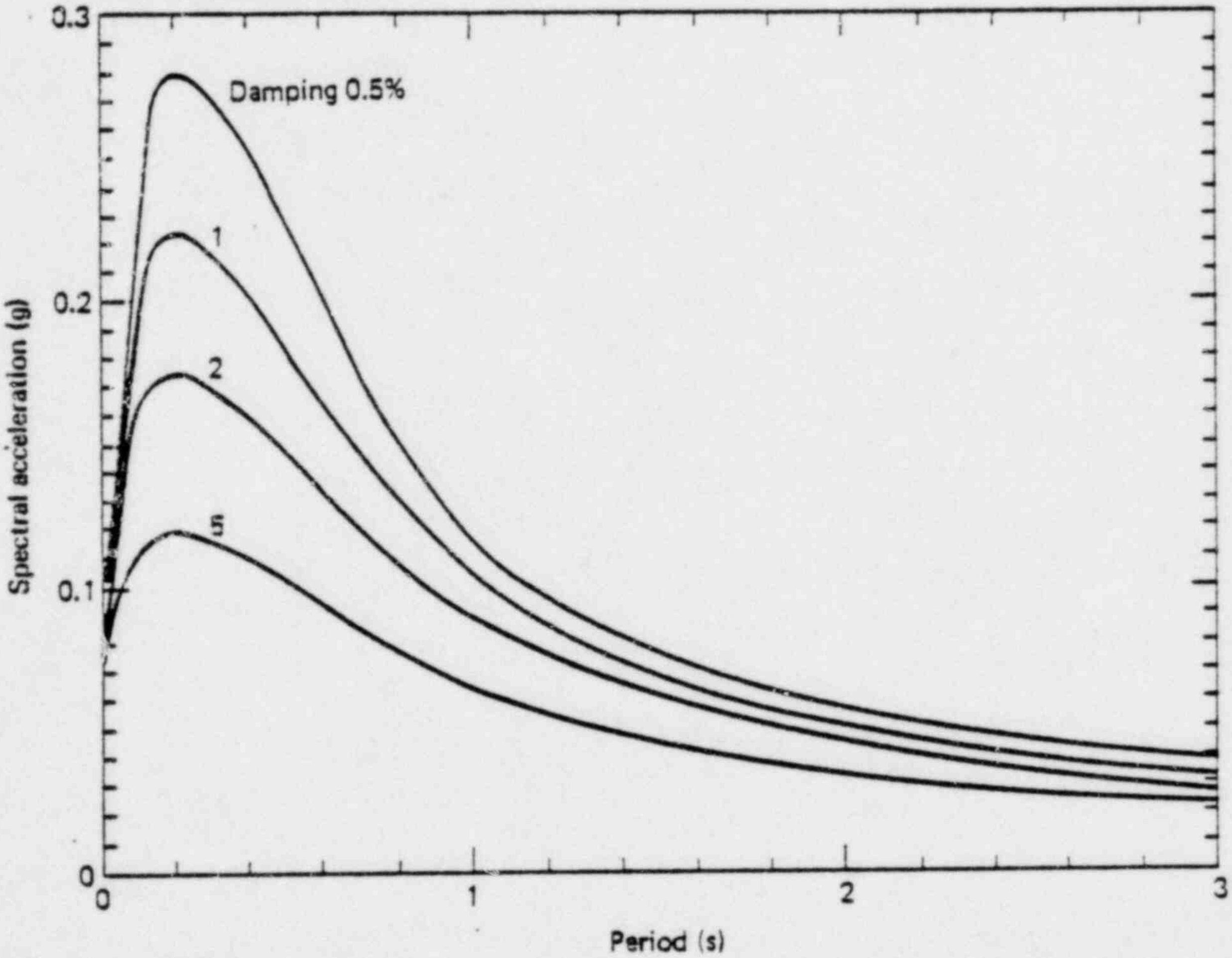

Fig. 4. Smoothed acceleration response spectra used in the design of Millstone 1.

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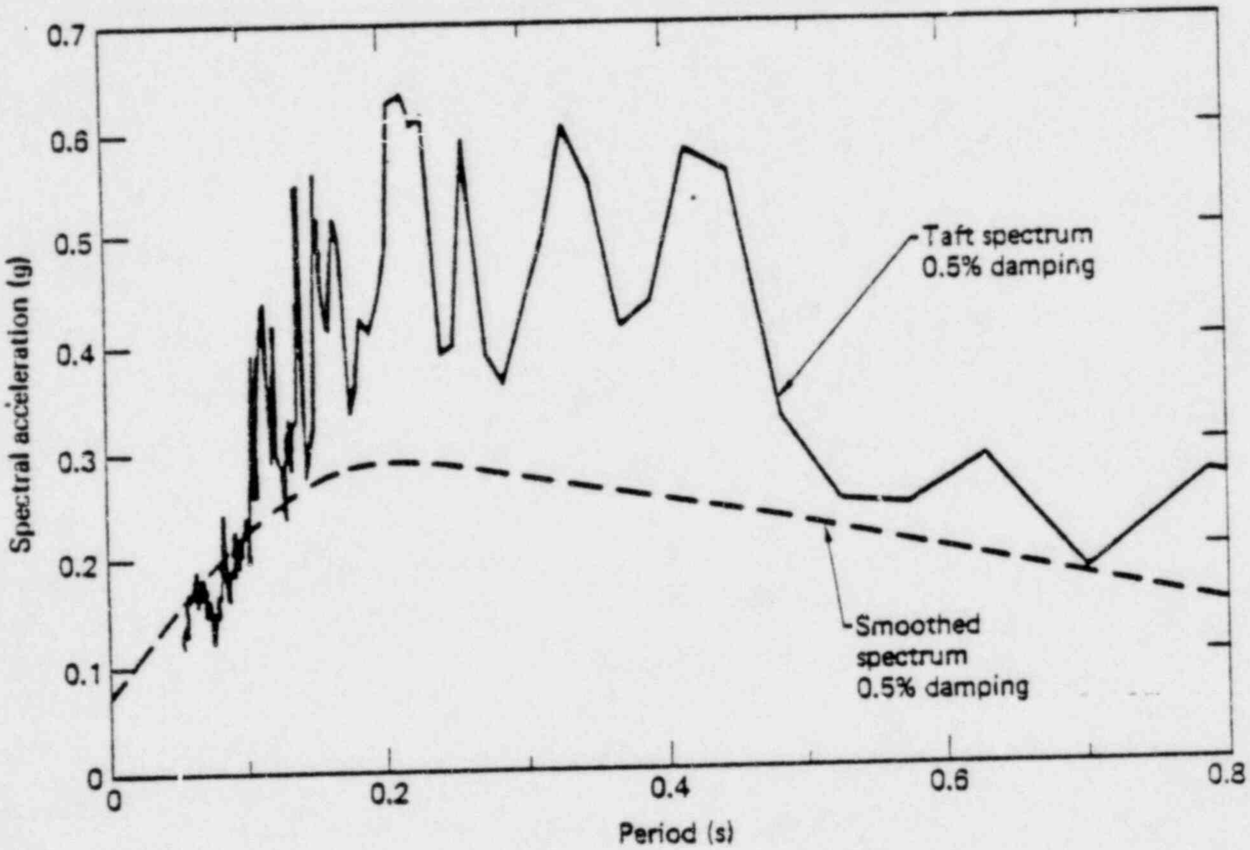

Fig. 5. Acceleration response spectrum from the normalized record of the July 21, 1952, Kern County earthquake (N69W component from Taft, Lincoln School Tunnel).

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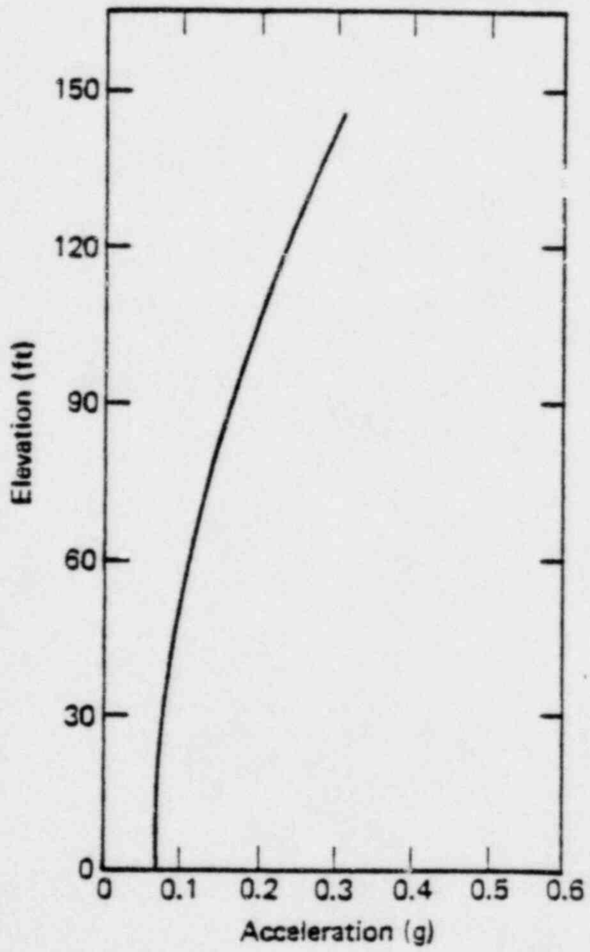

Fig. 6. Seismic coefficient curve for rigid equipment and piping in the reactor building.

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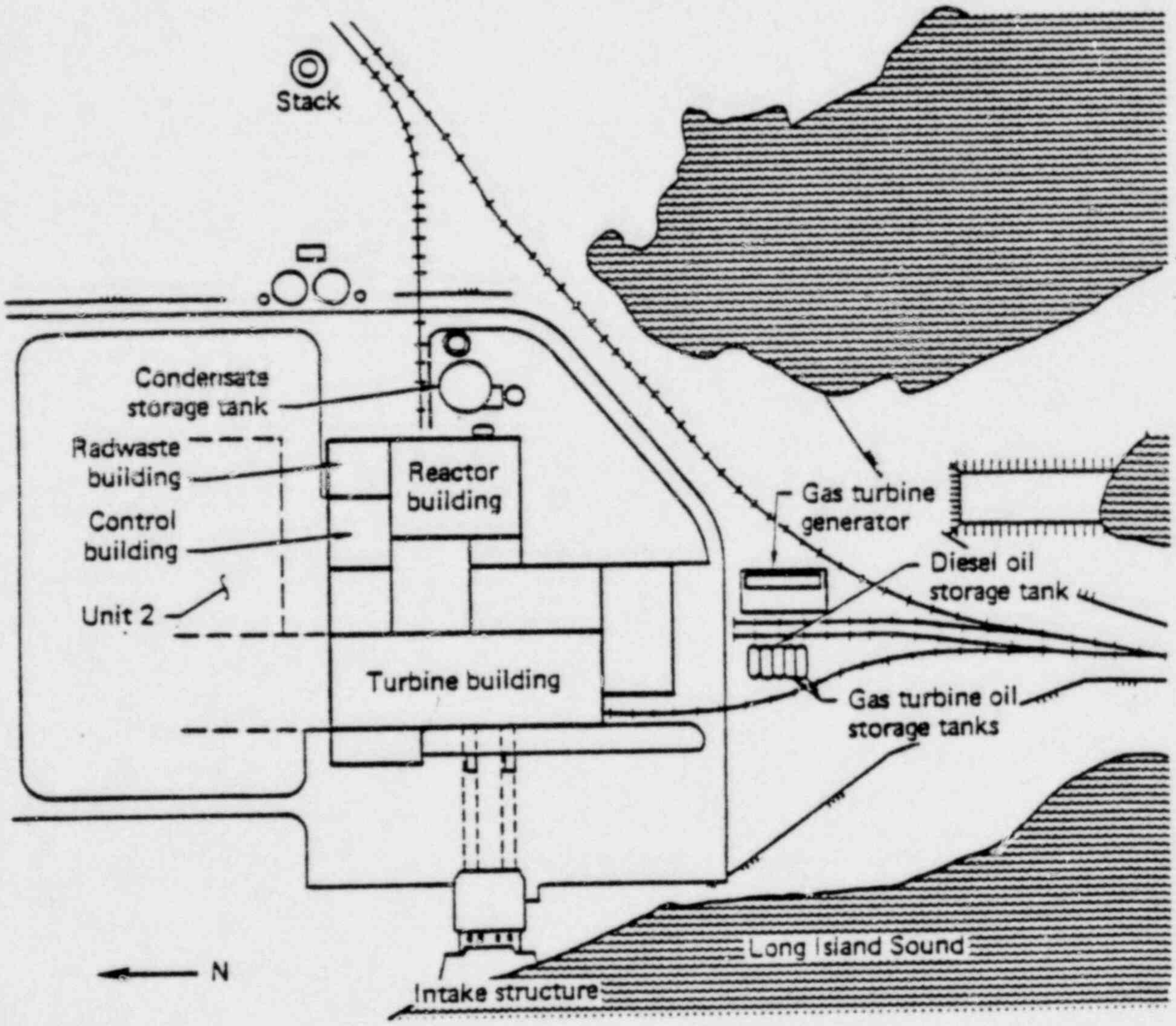

Fig. 7. Plot plan of Millstone Nuclear Power Station Unit 1.

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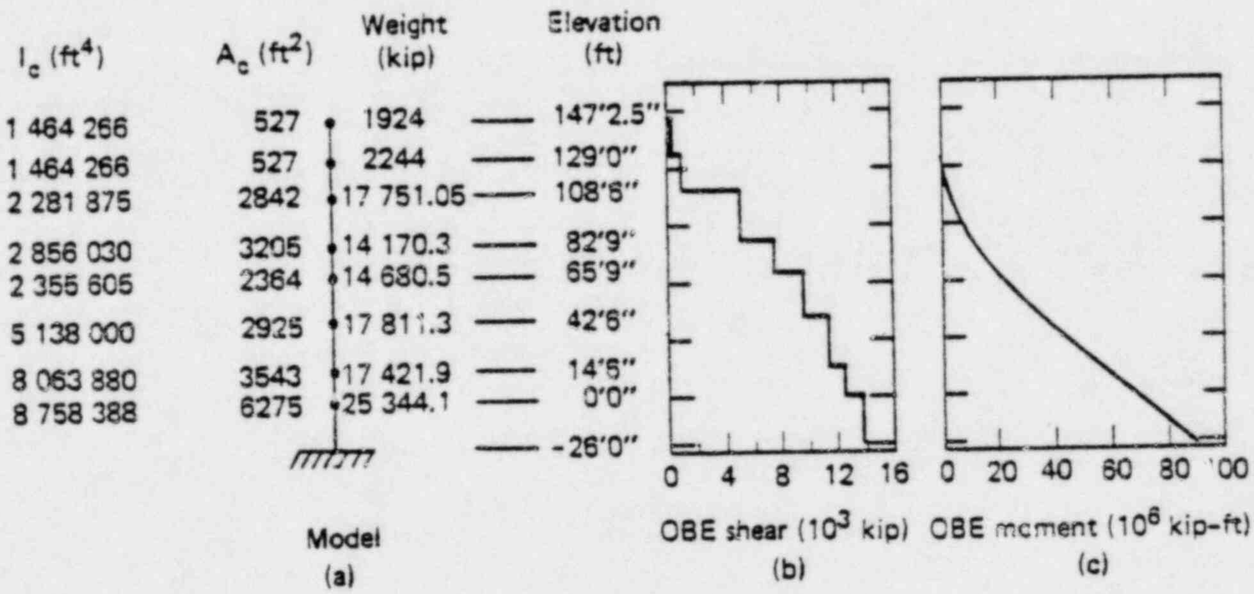

Fig. 8. Lumped mass model and seismic responses of the reactor building. (a). The model consisted of 8 masses. (b). OBE shear calculated by the time history technique using the Taft record. (c). OBE calculated using the same conditions.

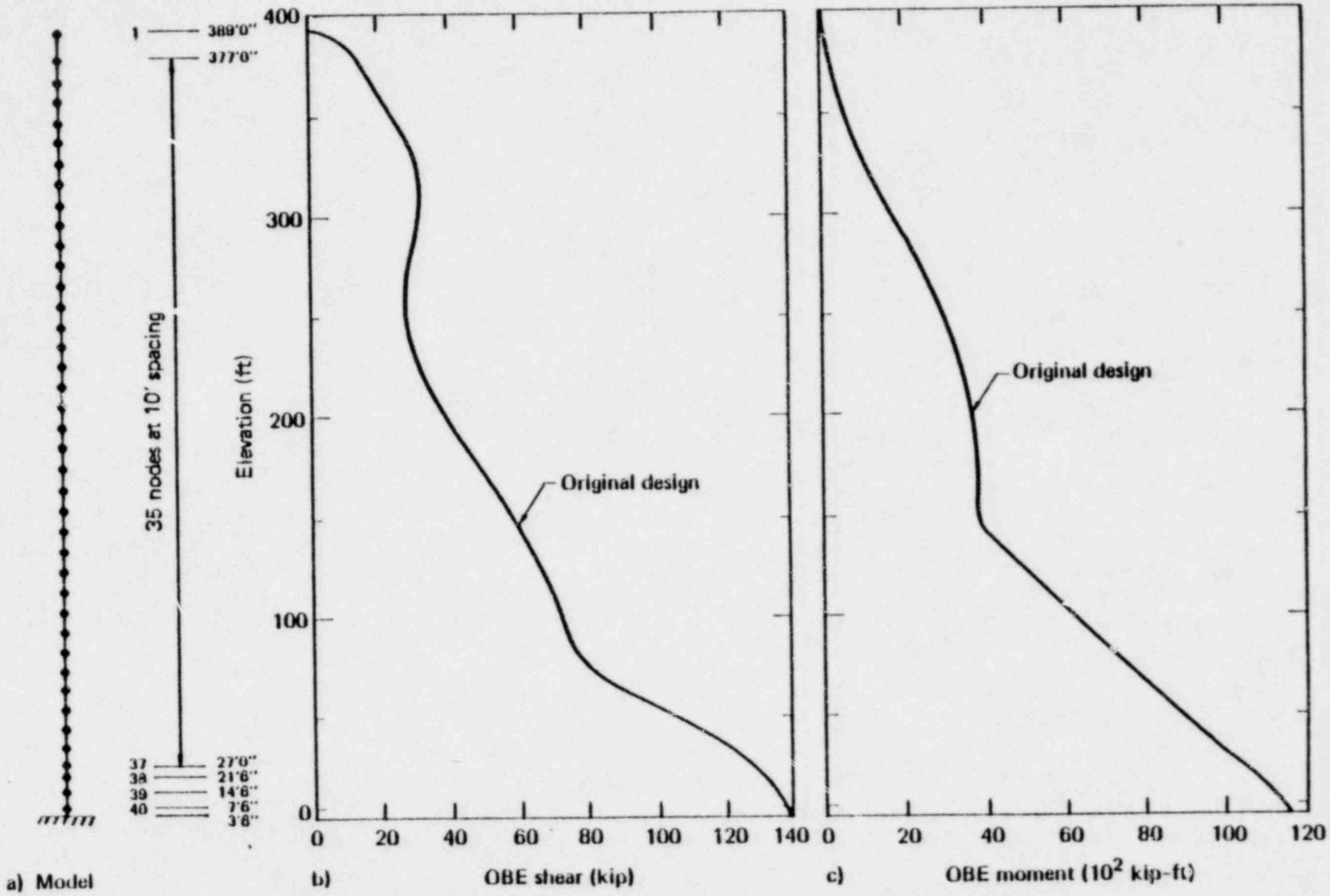

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Fig. 9. Lumped mass model and seismic responses of the ventilation stack. (a). The model consisted of 40 masses. (b). OBE shear calculated by the time history technique using the Taft record. (c). OBE moment

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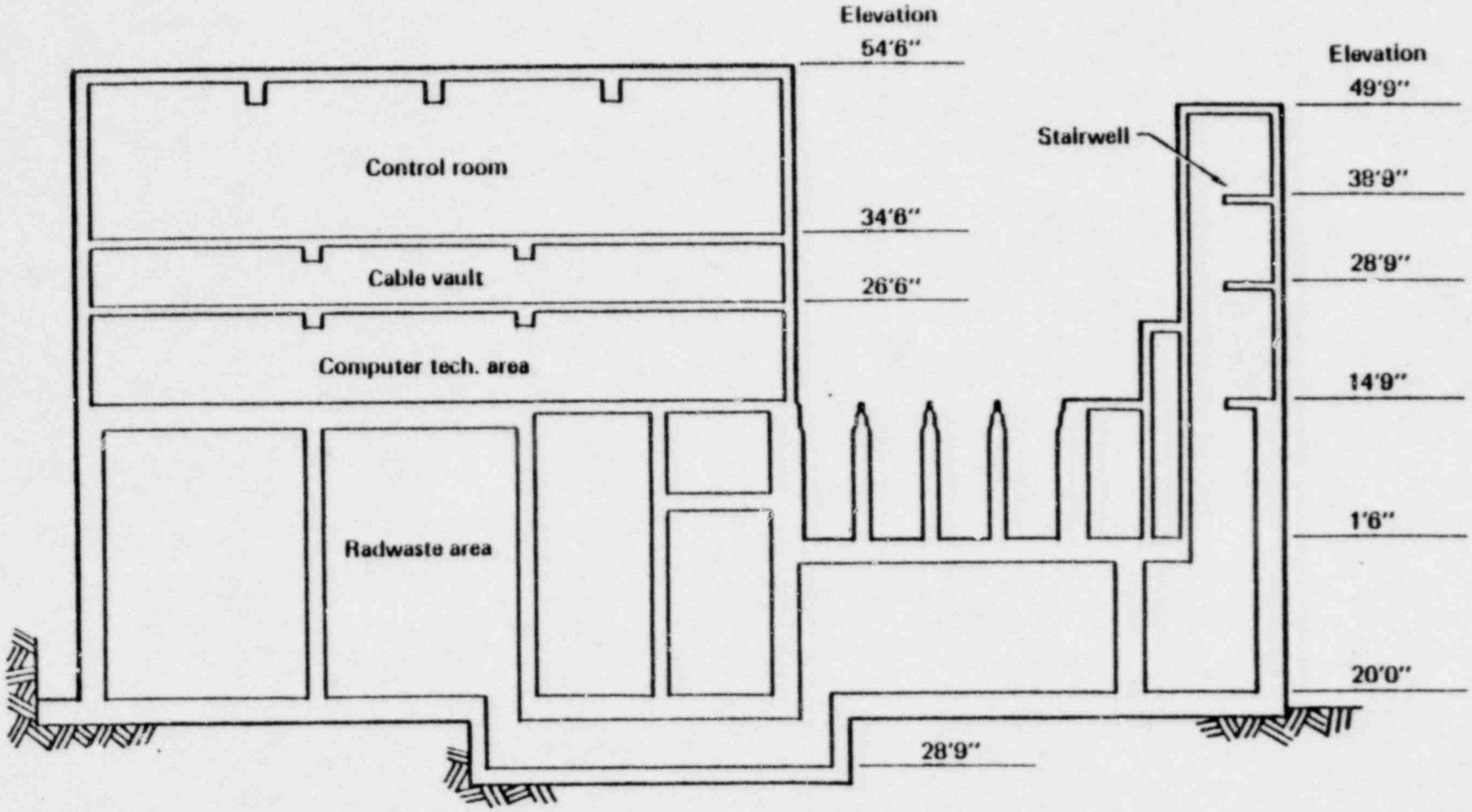

Fig. 10. Sections of the radwaste building/control room.

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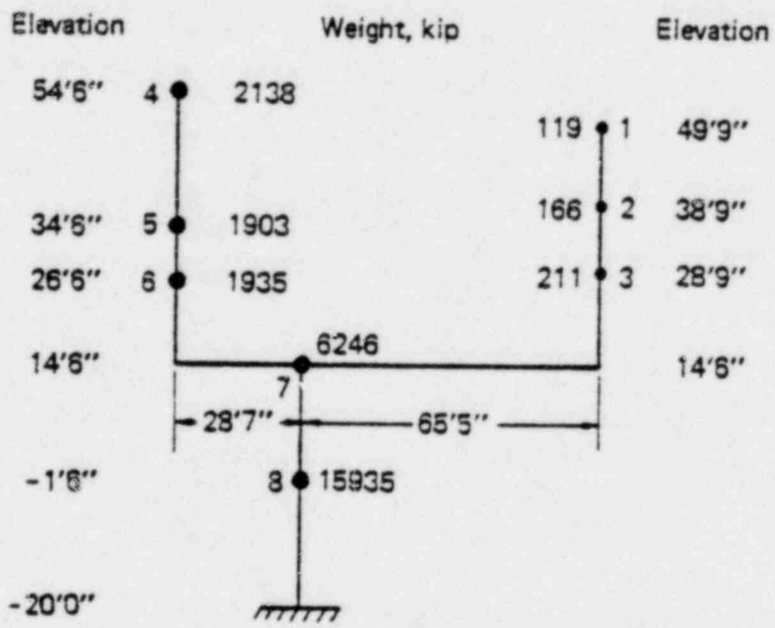

Fig. 11. Mathematical model used in the original analysis of the radwaste building/control room.

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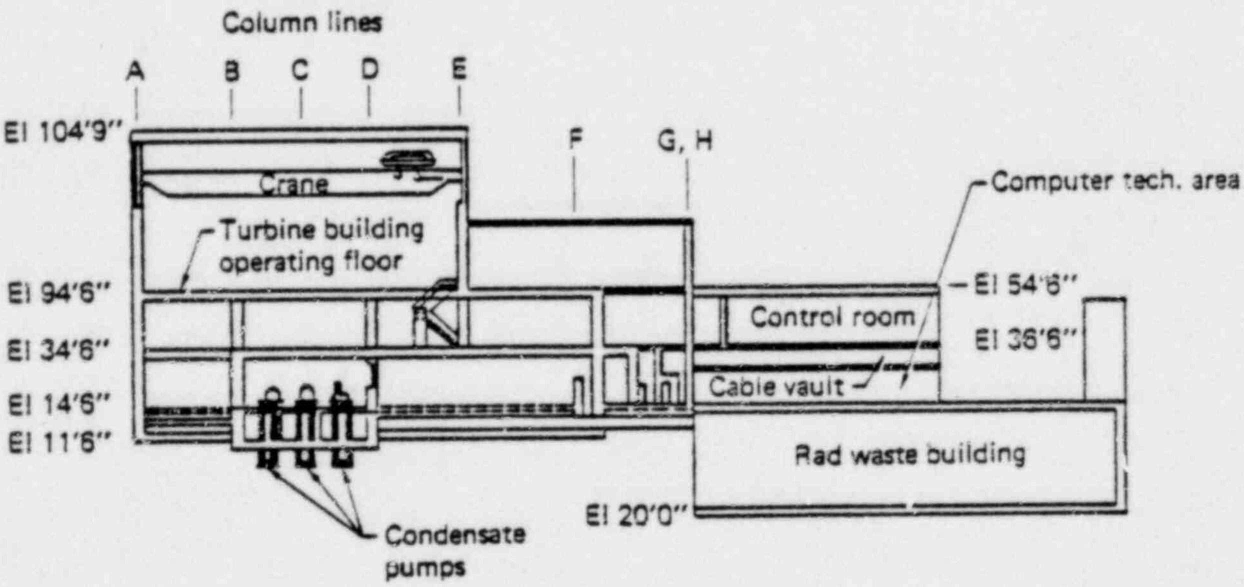

Fig. 12. East-west section of turbine and radwaste buildings.

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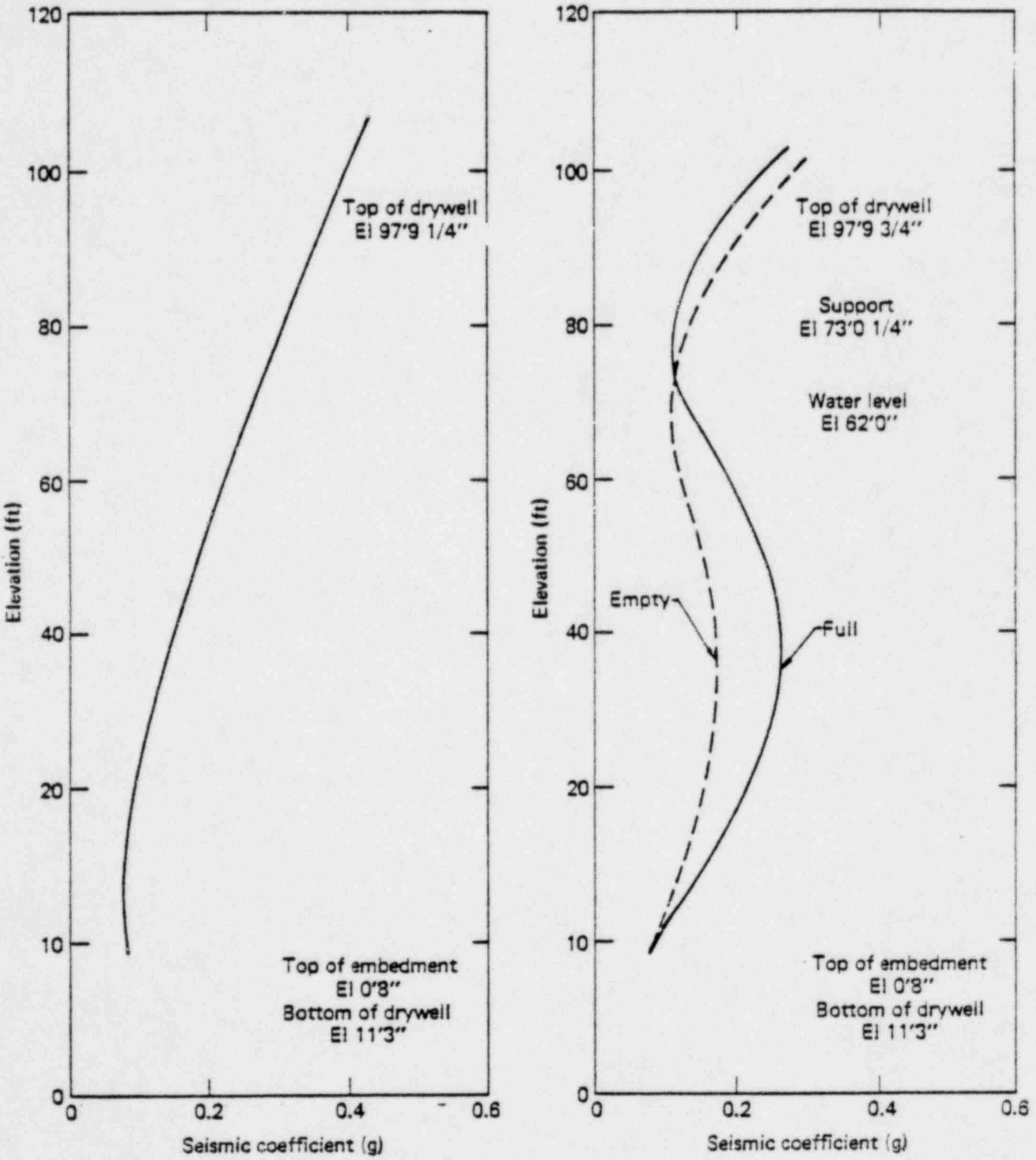

Fig. 13. Seismic coefficients for the drywell: a) cantilevered from the skirt support or the embedment (initial test condition only); and b) supported laterally (normal operation).

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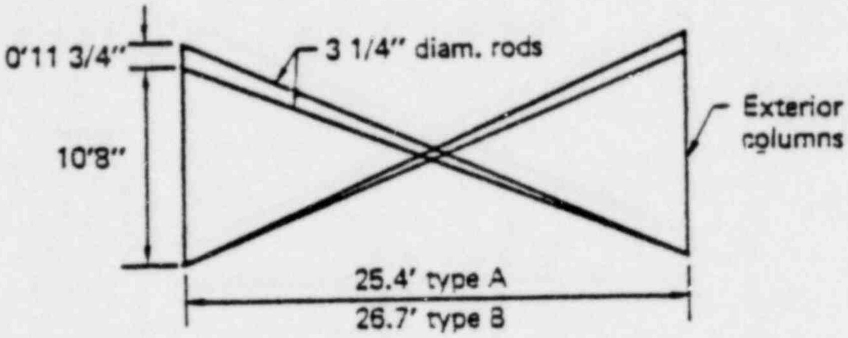
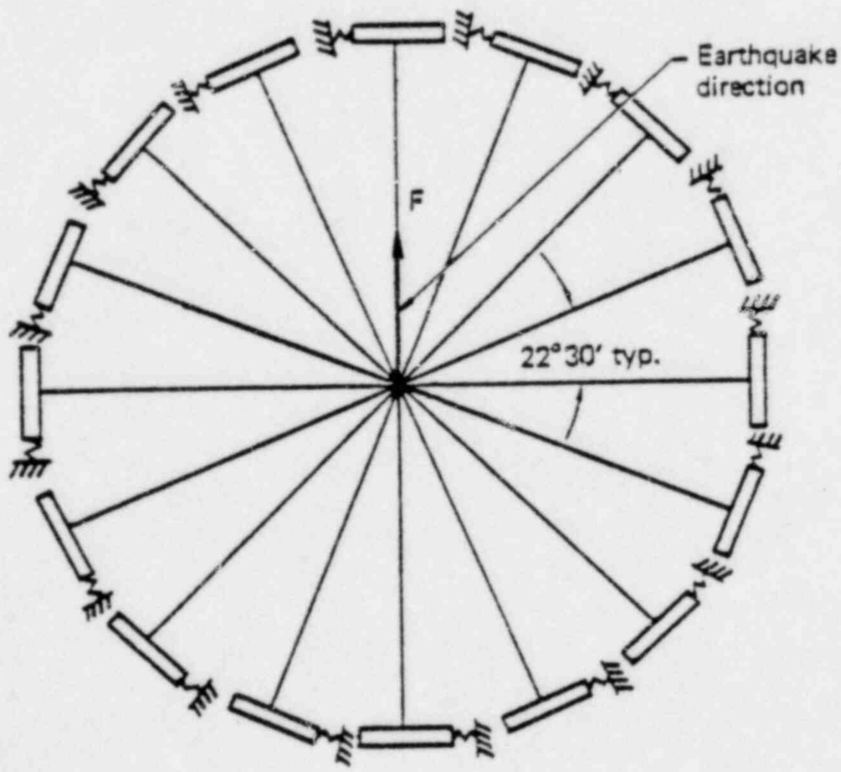


a. Cross bracing represented by springs



b. Single degree-of-freedom model

Fig. 14. Seismic analysis model of the torus. (a). Typical cross bracing; and (b). Single degree-of-freedom model.

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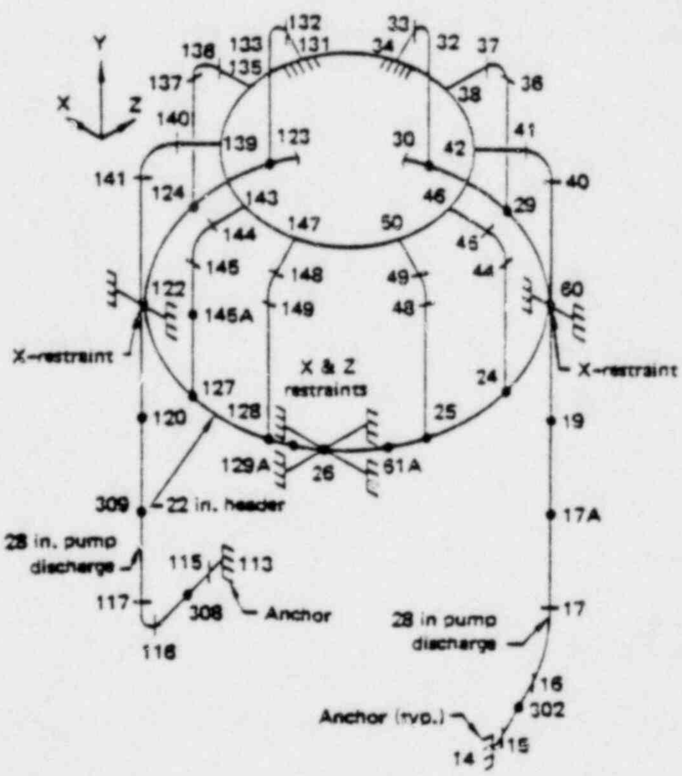

Fig. 15. Mathematical model used in the analysis of the reactor recirculation loop piping.

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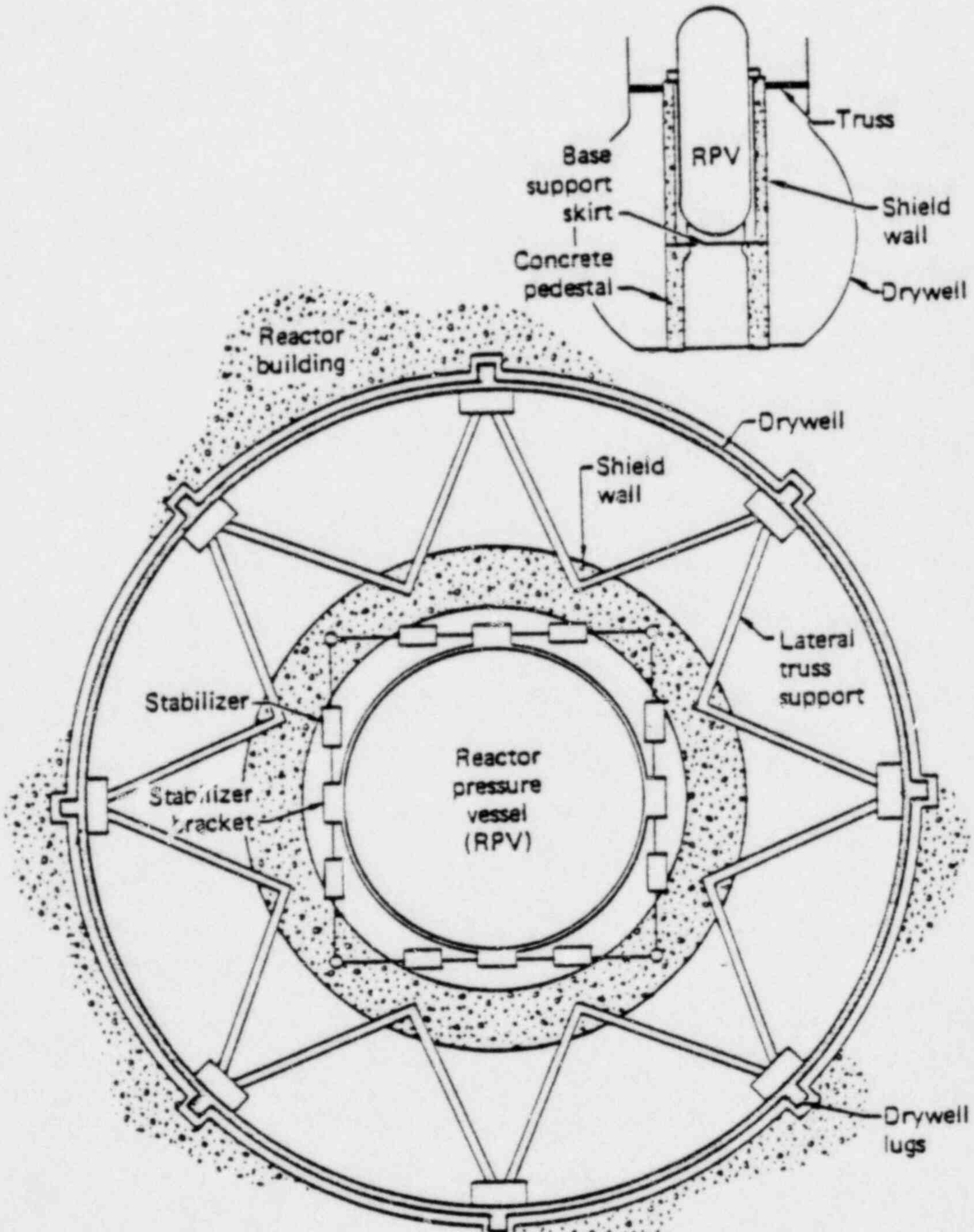

Fig. 16. Schematic of the drywell truss system.

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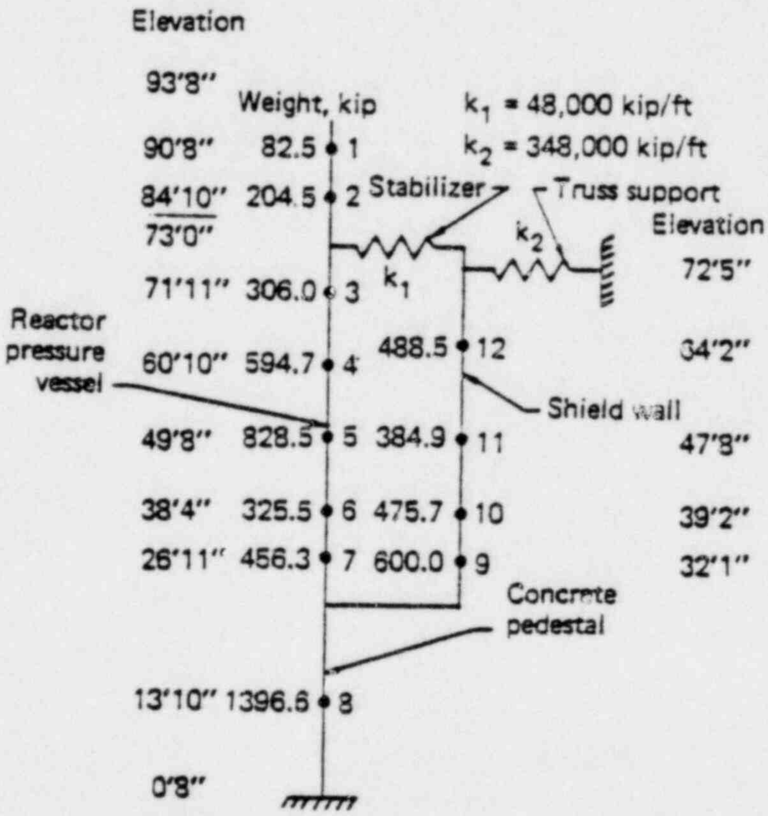

Fig. 17. Mathematical model used in the original analysis of the reactor pressure vessel.

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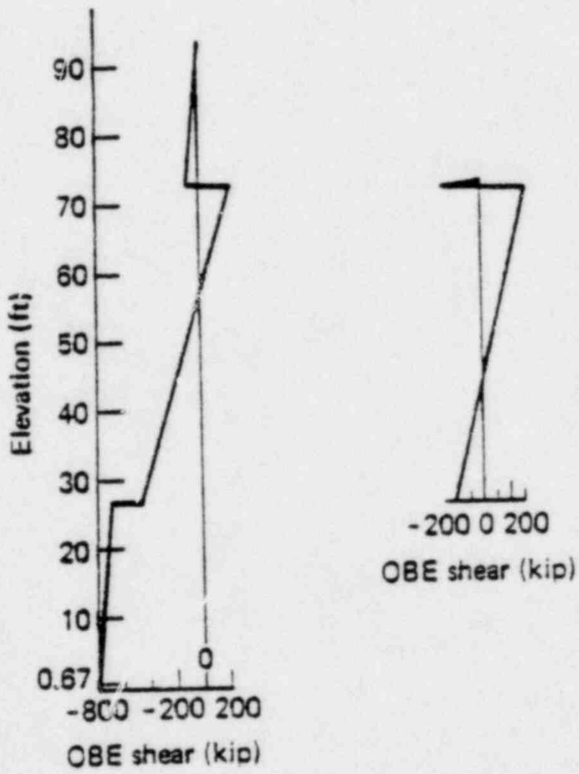

Fig. 18. Calculated OBE seismic shears in the reactor pressure vessel.

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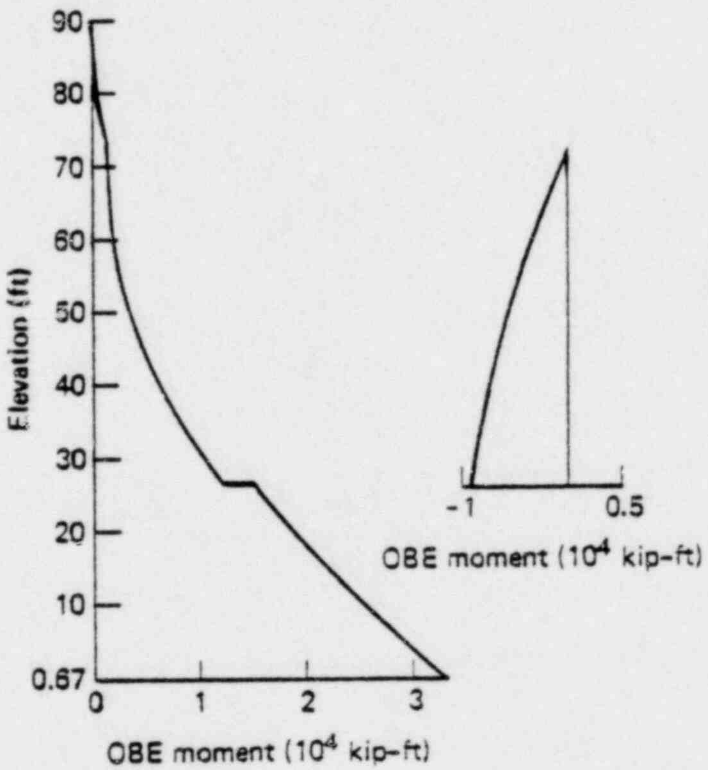

Fig. 19. Calculated OBE seismic moments in the reactor pressure vessel.

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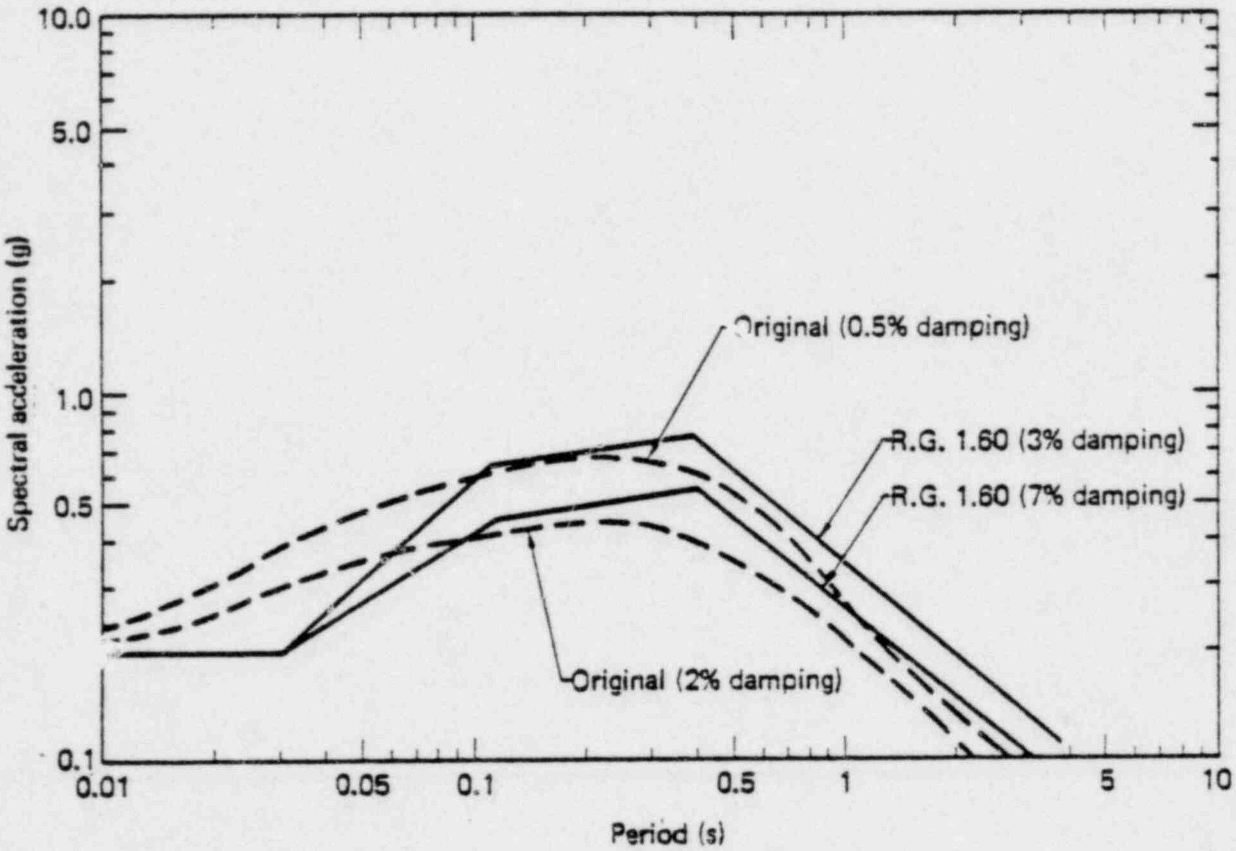

Fig. 20. Comparison of the original response spectrum for 2% of critical damping with the 7% R.G. 1.60 spectrum indicates the relative magnitude of the response of bolted steel structures and equipment designed to Millstone 1 criteria to that from current criteria. Similarly, expected levels of response for base-level-mounted large piping for the two criteria can be made by comparing the 0.5% original response spectrum and the 3% R.G. 1.60 spectrum.

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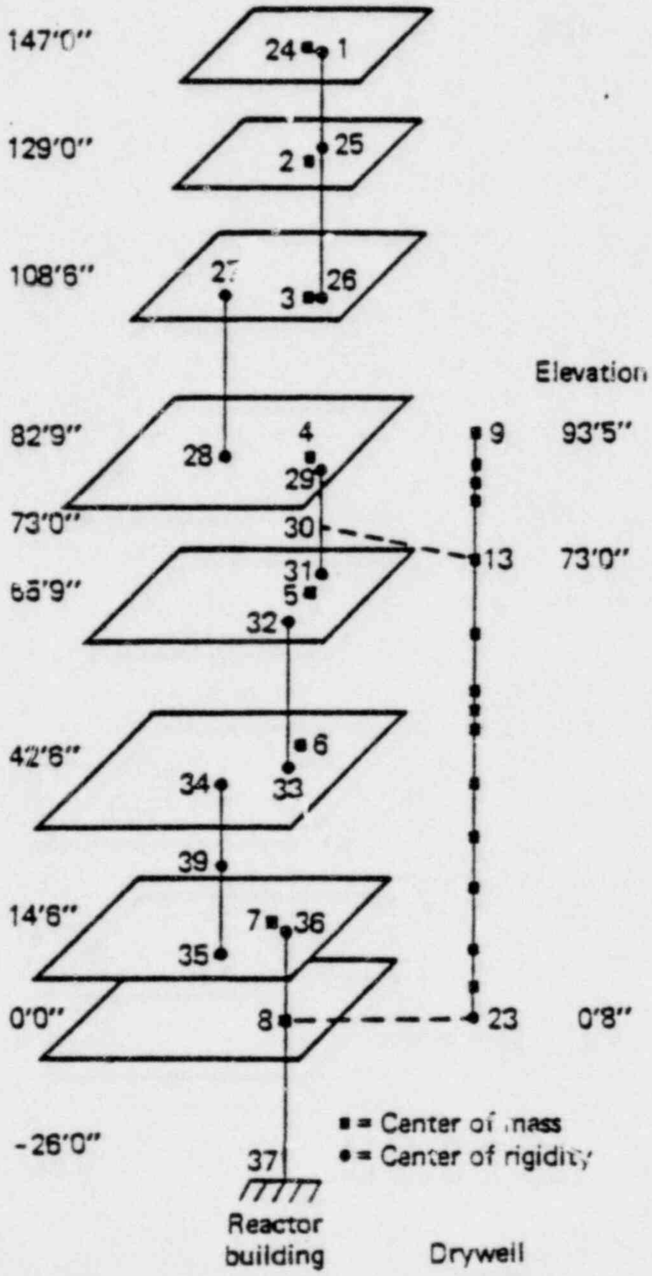

Fig. 21. Mathematical model for reactor building and drywell.

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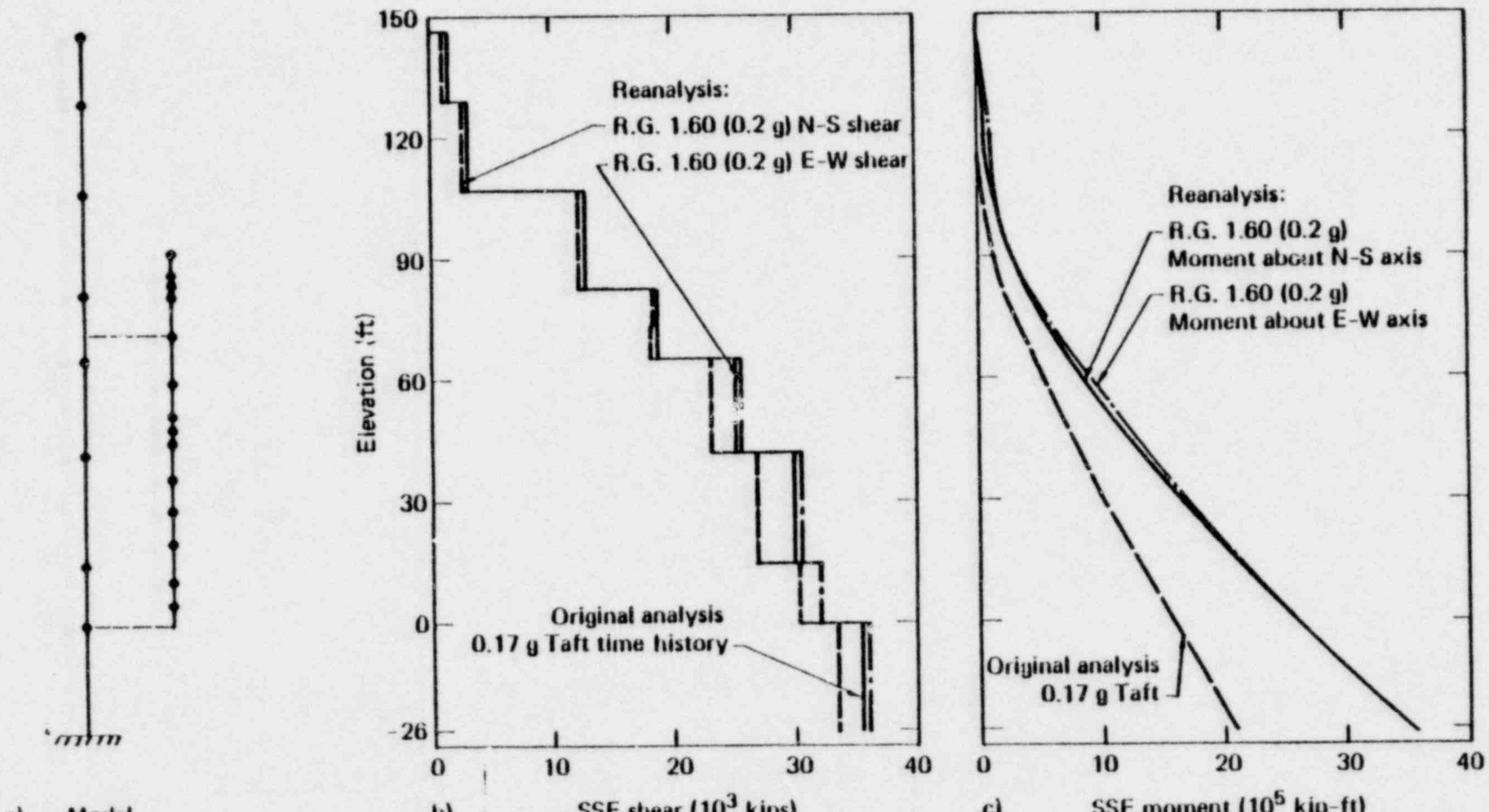
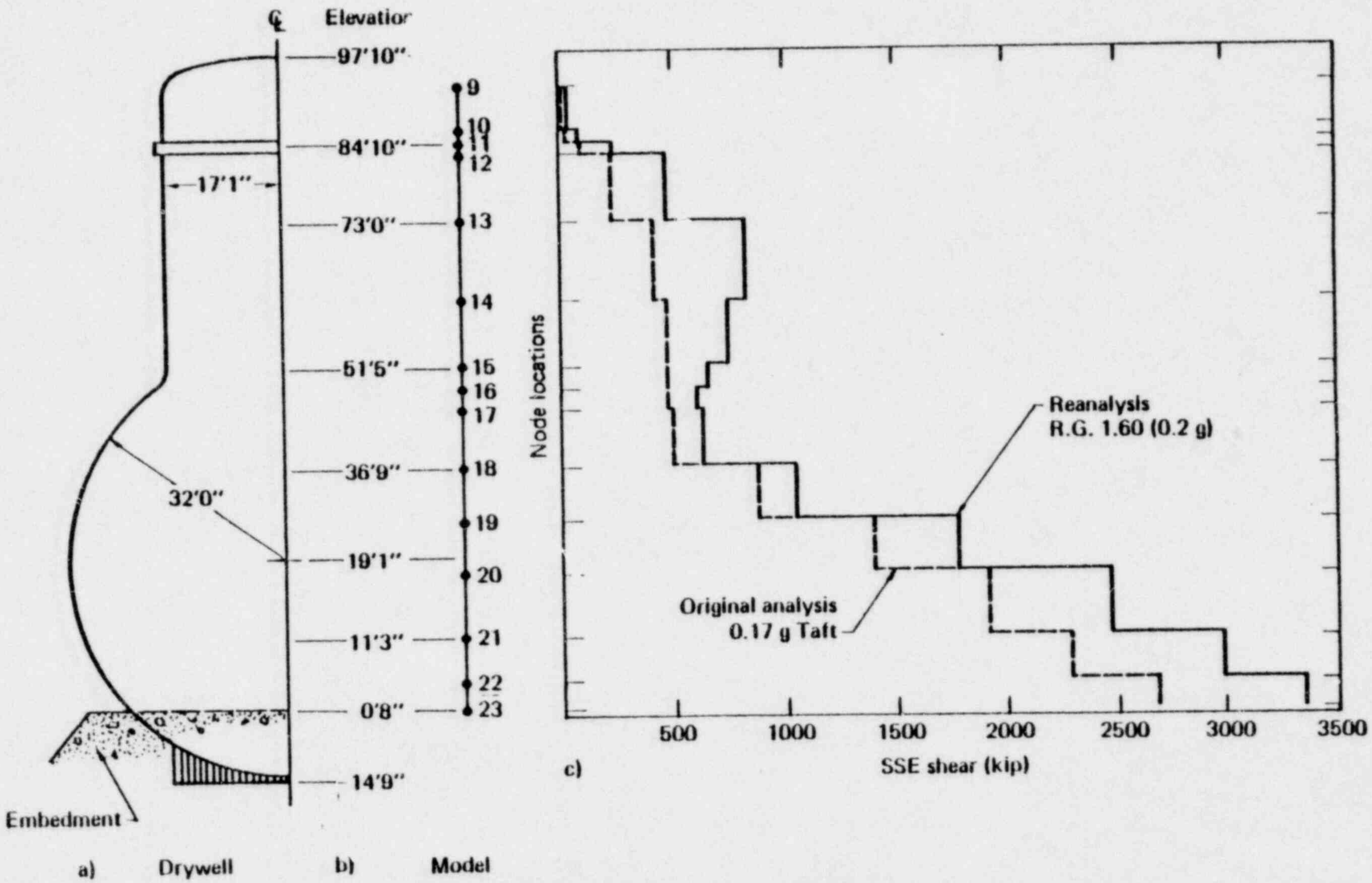


Fig. 22. SSE shear and moment responses for the reactor building.



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Elevatior
97'10"
84'10"
17'1"
73'0"
51'5"
36'9"
32'0"
19'1"
11'3"
0'8"
14'9"
Embedment
9
10
11
12
13
14
15
16
17
18
19
20
21
22
23
Node locations
Reanalysis
R.G. 1.60 (0.2 g)
Original analysis
0.17 g Taft
500
1000
1500
2000
2500
3000
3500
SSE shear (kip)
a) Drywell
b) Model
c)

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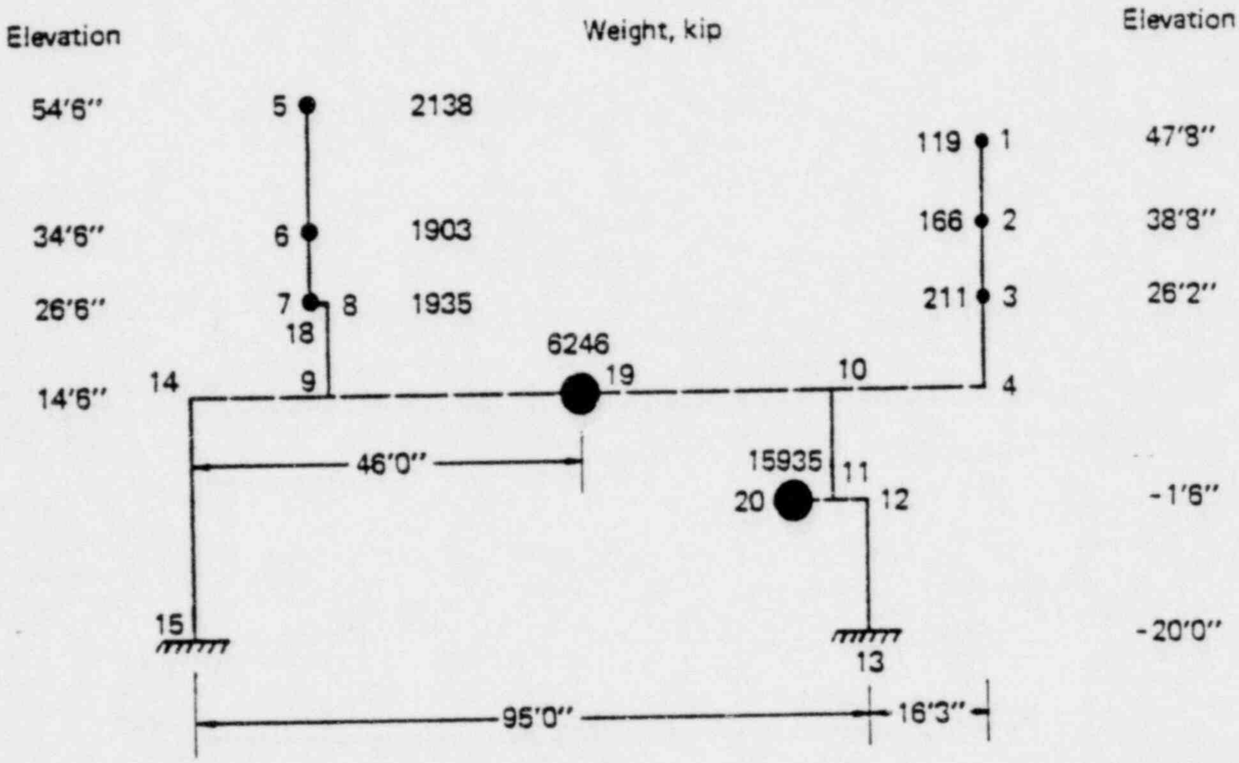

Fig. 24. Mathematical model of the radwaste building.

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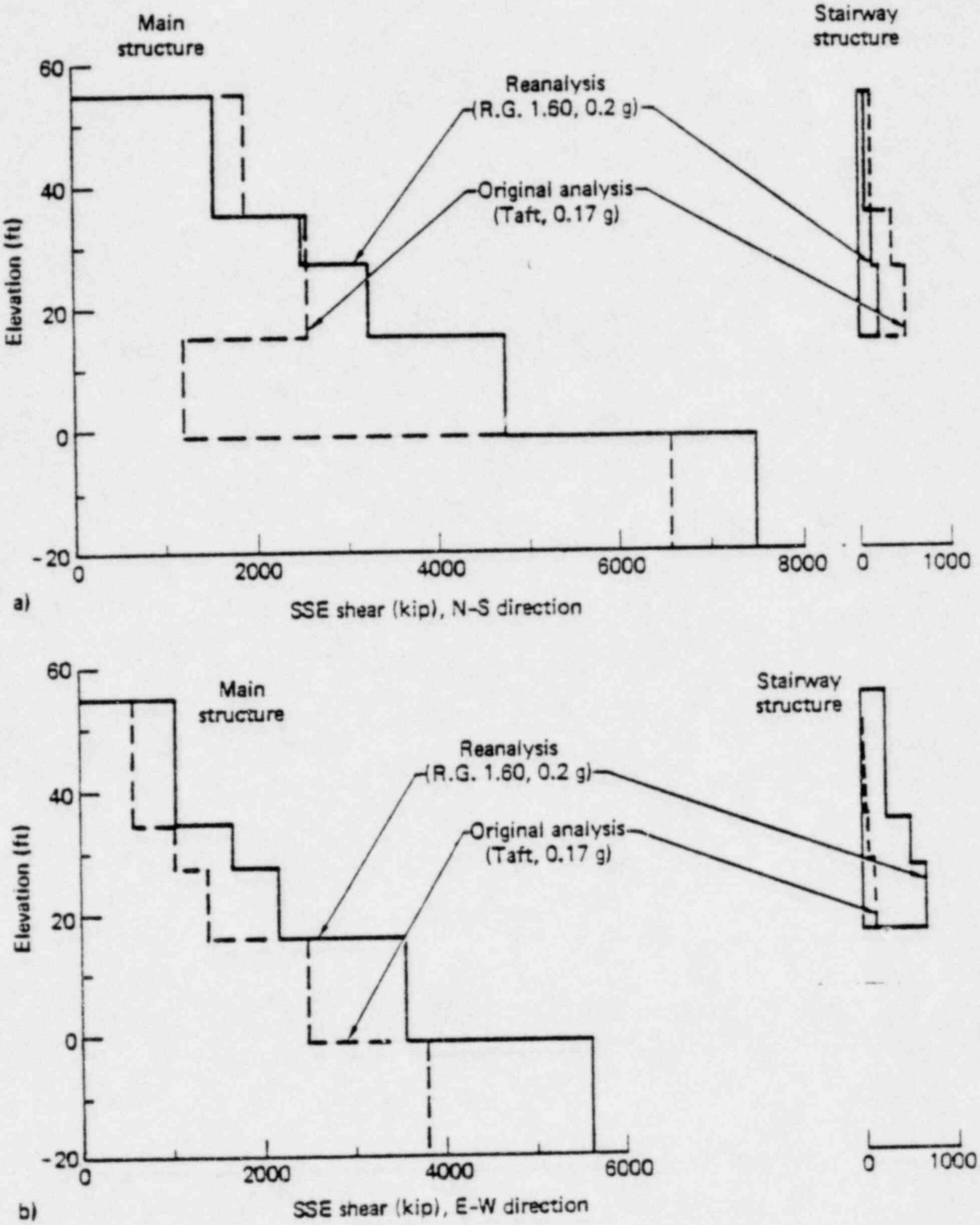

Fig. 25. SSE shear responses of the radwaste building. (a). Shears in the N-S direction. (b). Shears in the E-W direction.

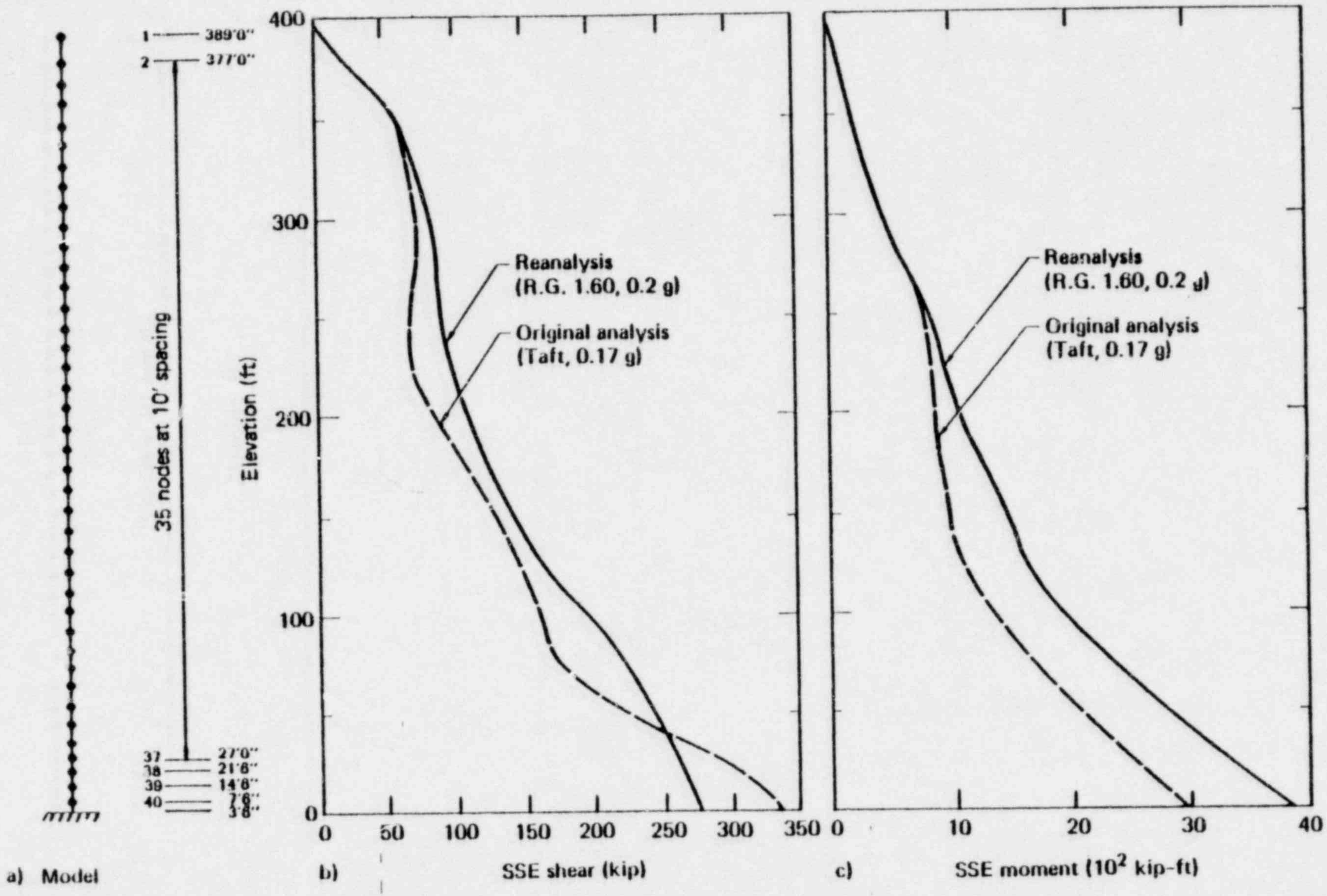

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Fig. 26. Mathematical model and SSE shear and moment responses of the stack.

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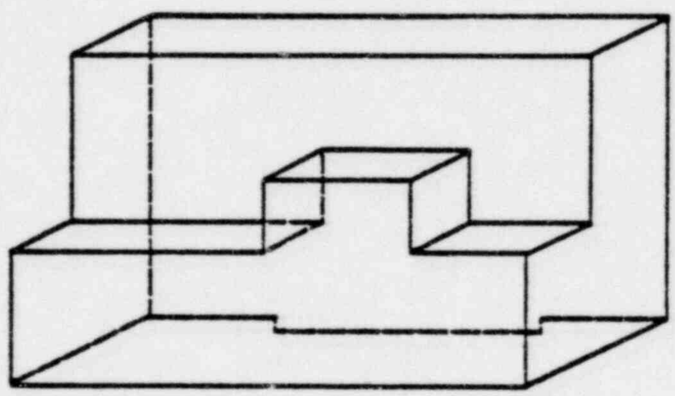

Fig. 27. Isometric view of the turbine building.

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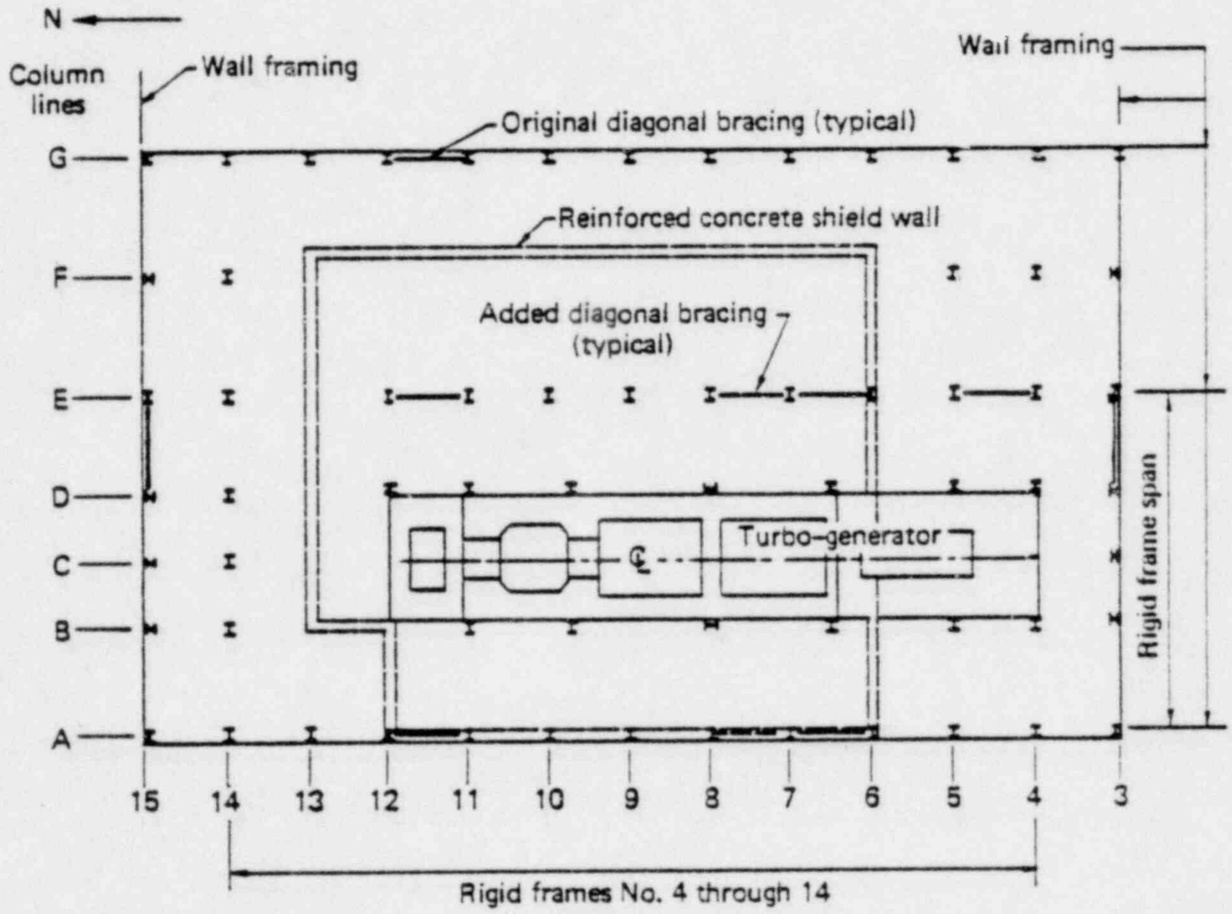

Fig. 28. Plan view, turbine building operating floor showing diagonal bracing locations.

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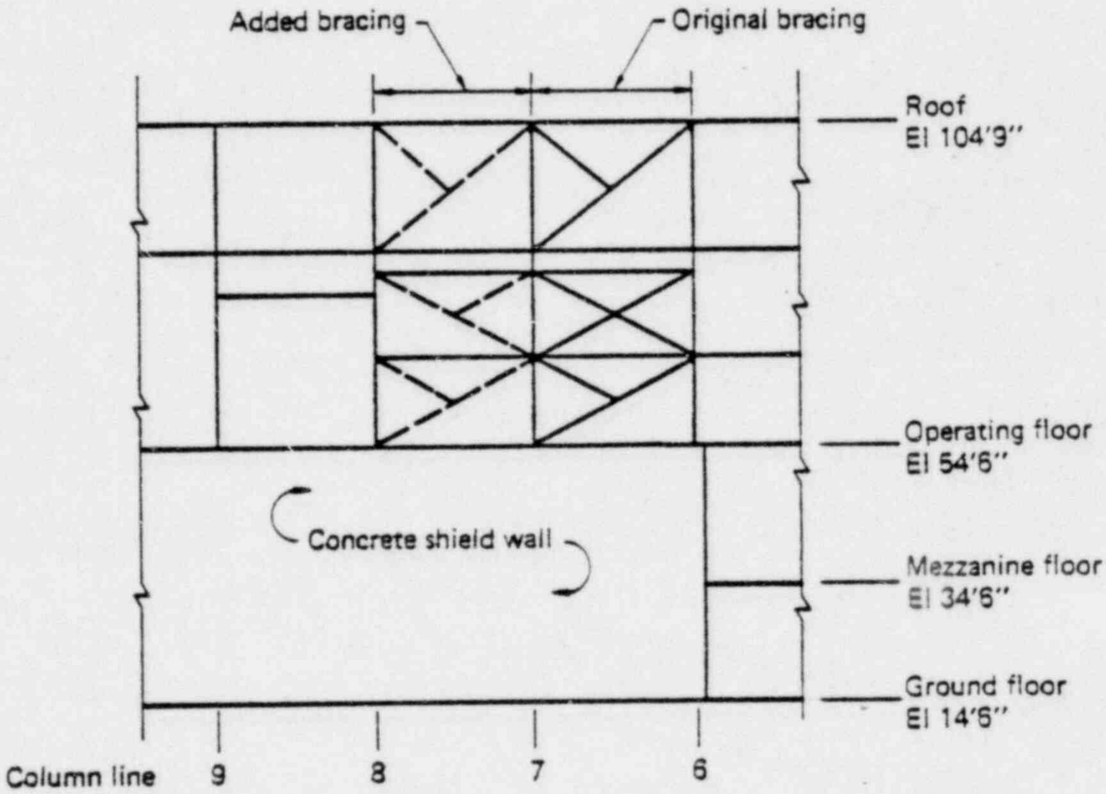

Fig. 29. Typical bracing along column line E.

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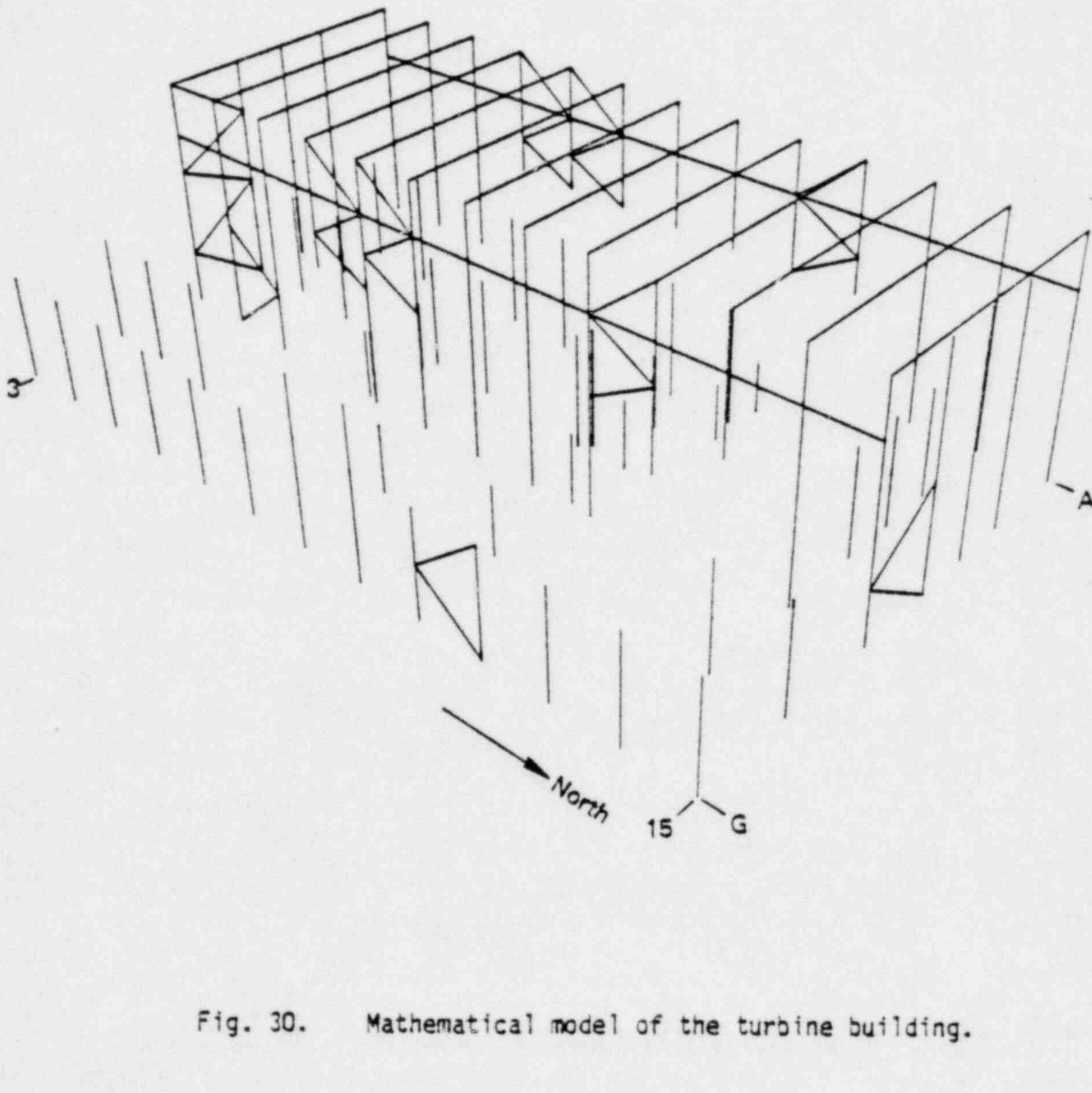

Fig. 30. Mathematical model of the turbine building.

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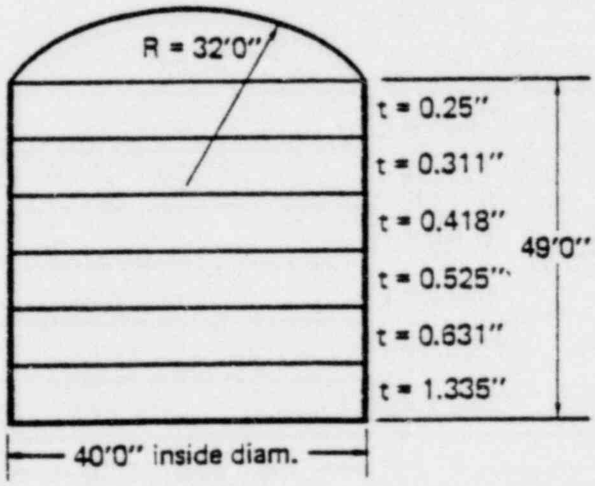

Fig. 31. Condensate storage tank.

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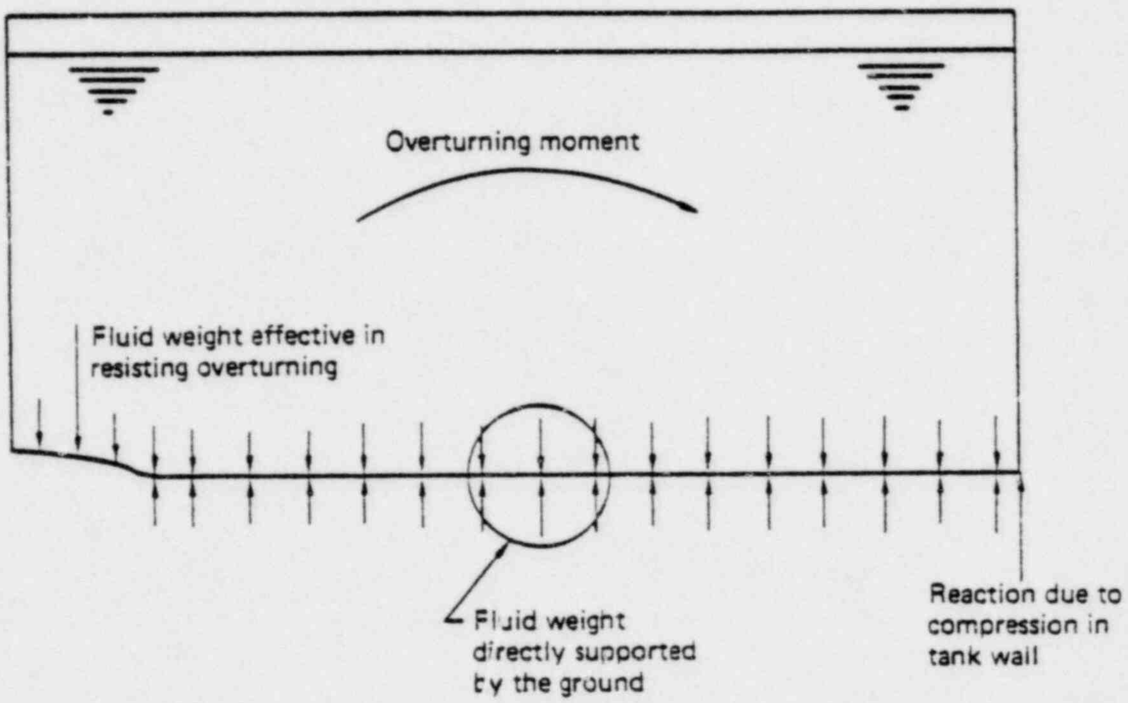

Fig. 32. Response to overturning moment in condensate storage tank.

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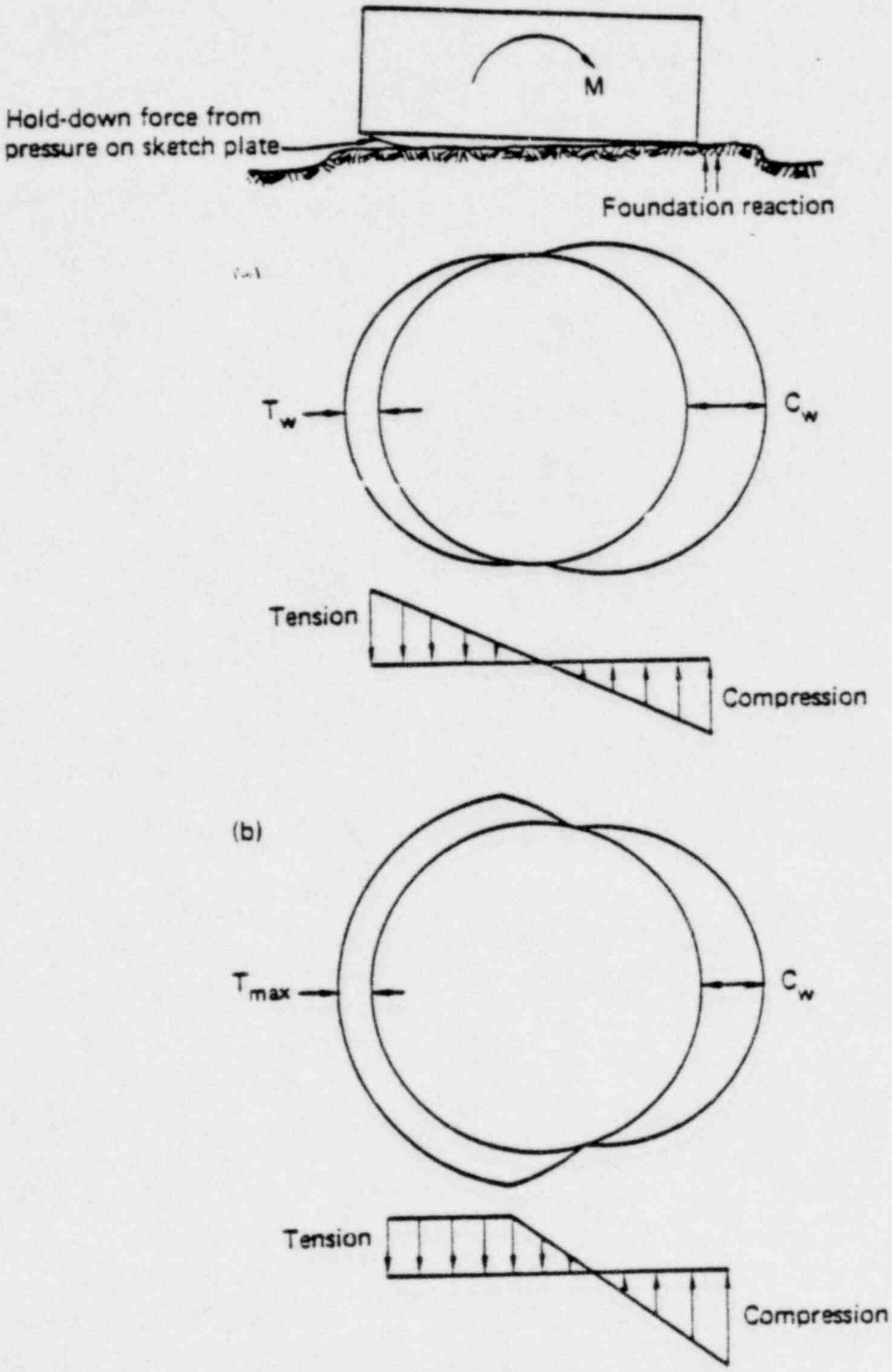

Fig. 33. Linear and nonlinear tank wall force distributions resulting from seismic overturning moment.

DRAFT

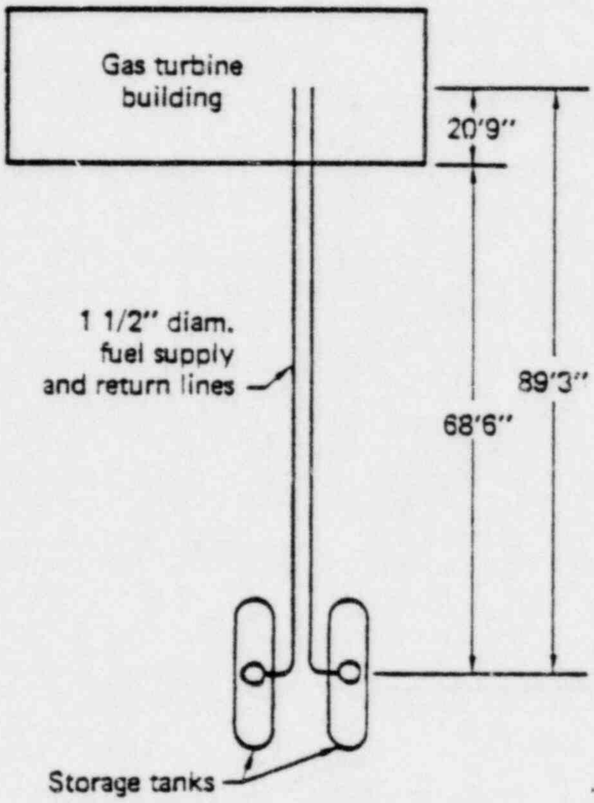

Fig. 34. Schematic plan view of gas turbine buried fuel supply and return lines.

DRAFT

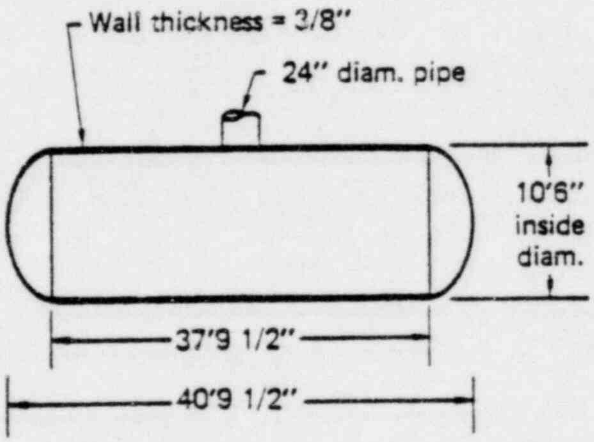

Fig. 35. Gas turbine fuel oil storage tank.

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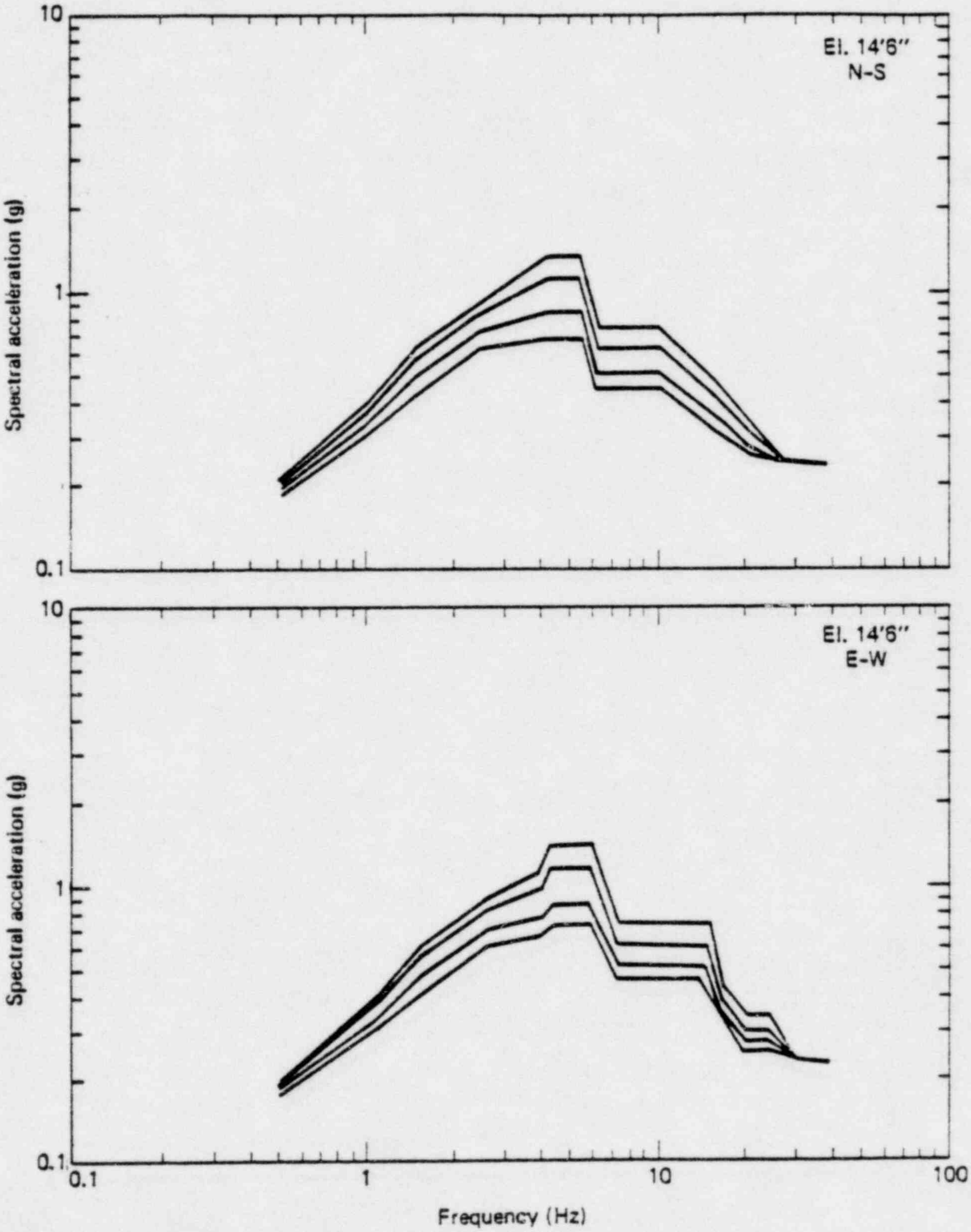

Fig. A-1. Horizontal in-structure response spectra at 2, 3, 5, and 7% of critical damping for equipment in the reactor building at el. 14'6".

DRAFT

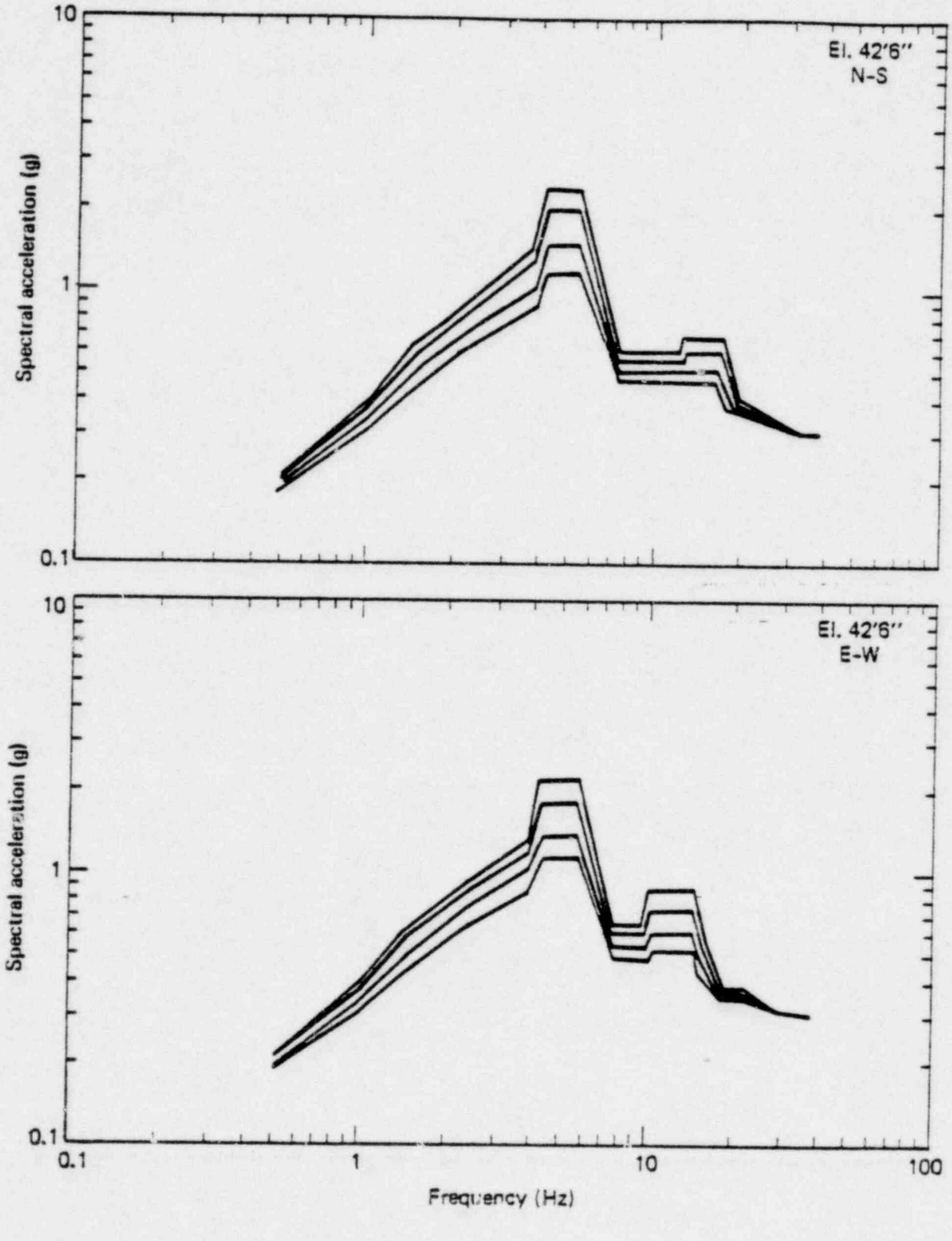

Fig. A-2 Horizontal in-structure response spectra at 2, 3, 5, and 7% of critical damping for equipment in the reactor building at el. 42'6".

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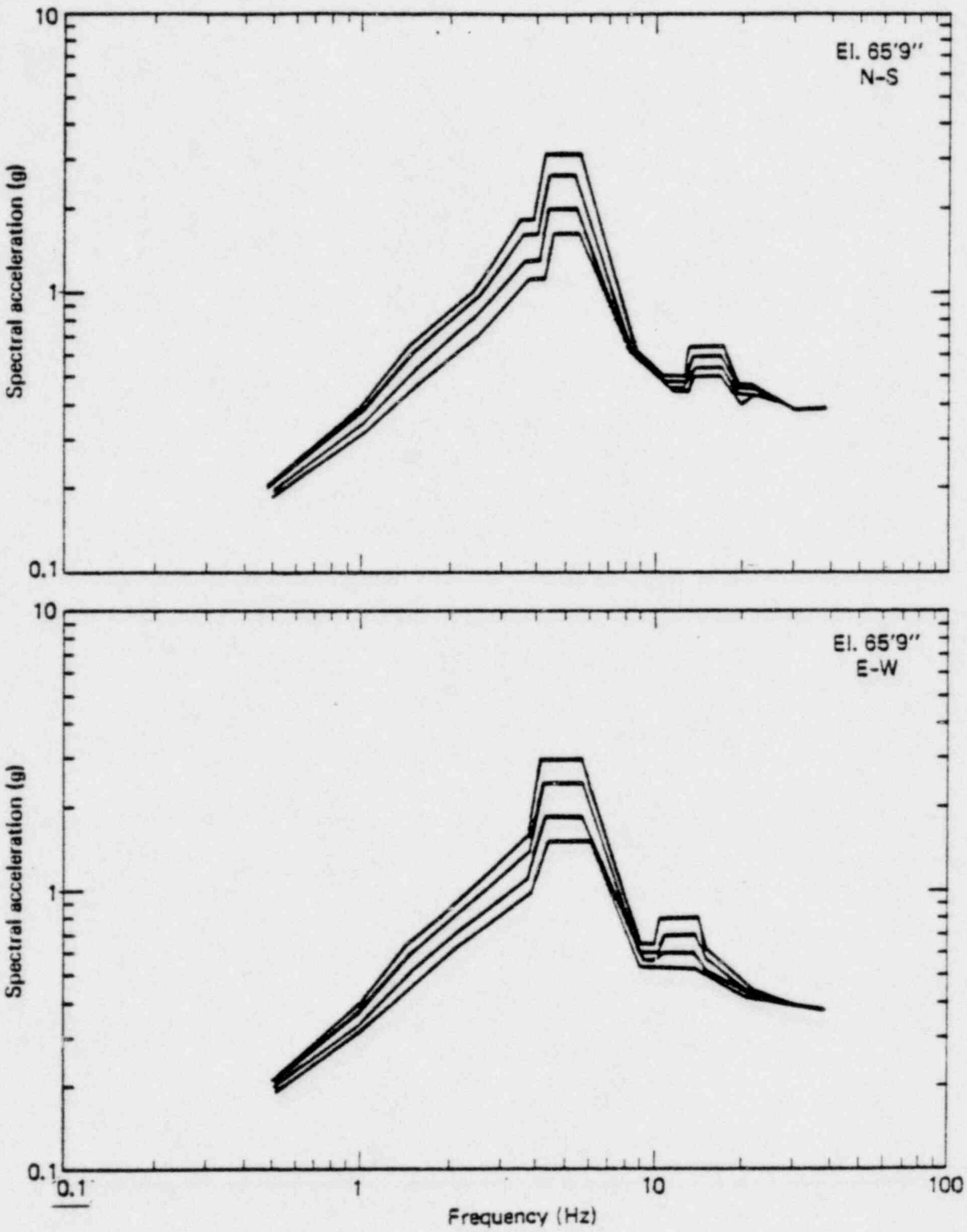

Fig. A-3. Horizontal in-structure response spectra at 2, 3, 5, and 7% of critical damping for equipment in the reactor building at el. 65'9".

DRAFT

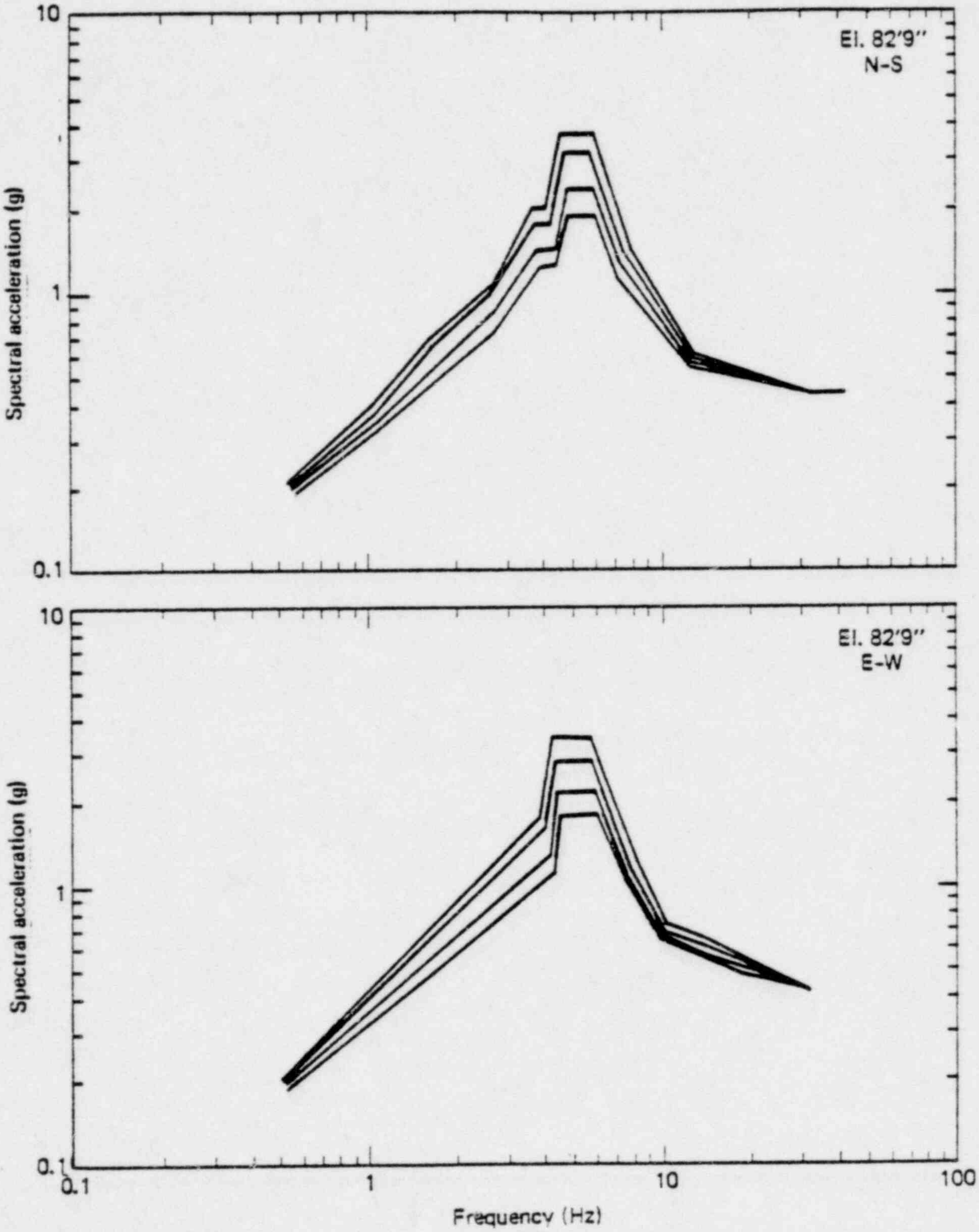

Fig. A-4. Horizontal in-structure response spectra at 2, 3, 5, and 7% of critical damping for equipment in the reactor building at el. 82'9".

DRAFT

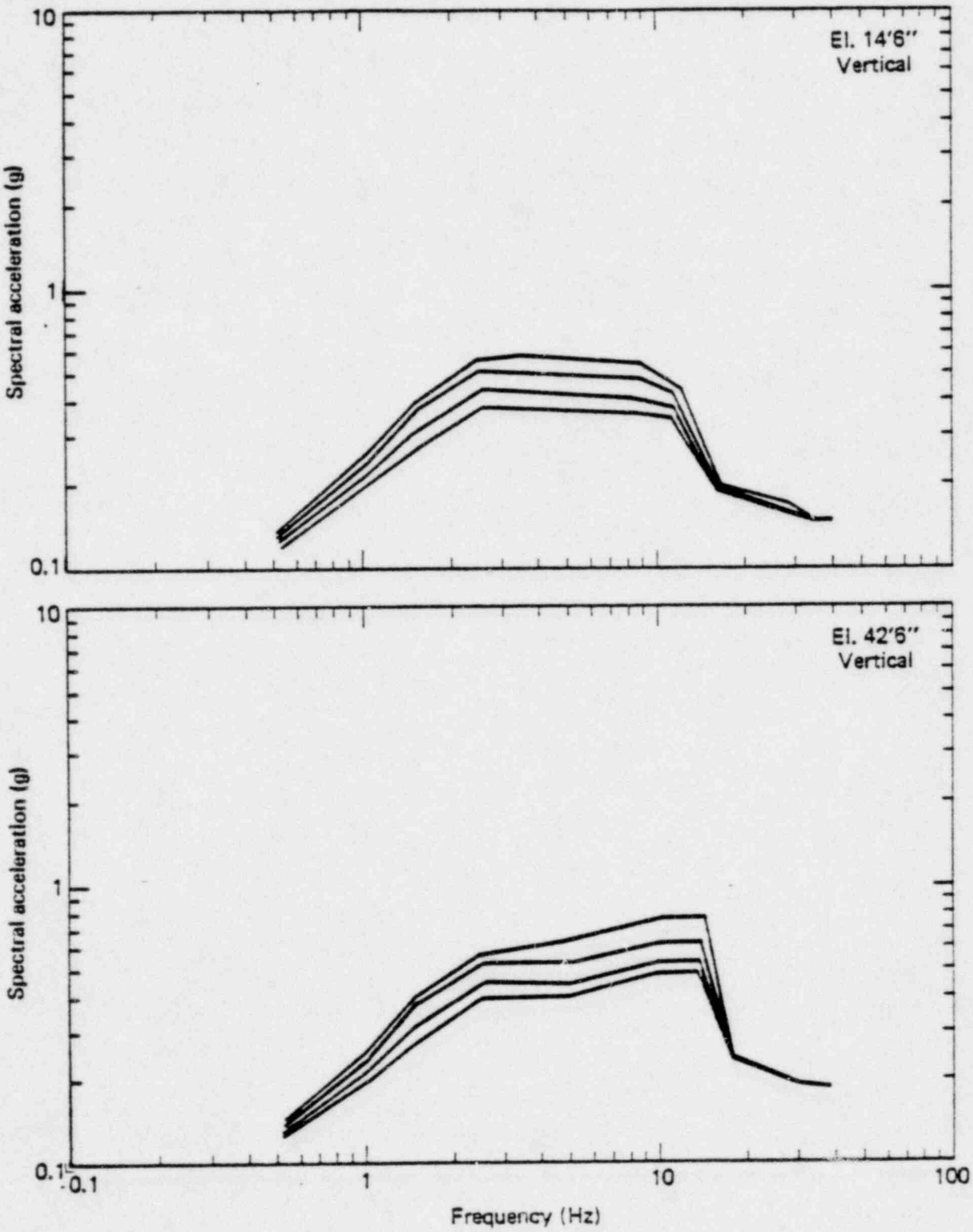

Fig. A-5. Vertical in-structure response spectra at 2, 3, 5, and 7% of critical damping for equipment in the reactor building at els. 14'6" and 42'6".

DRAFT

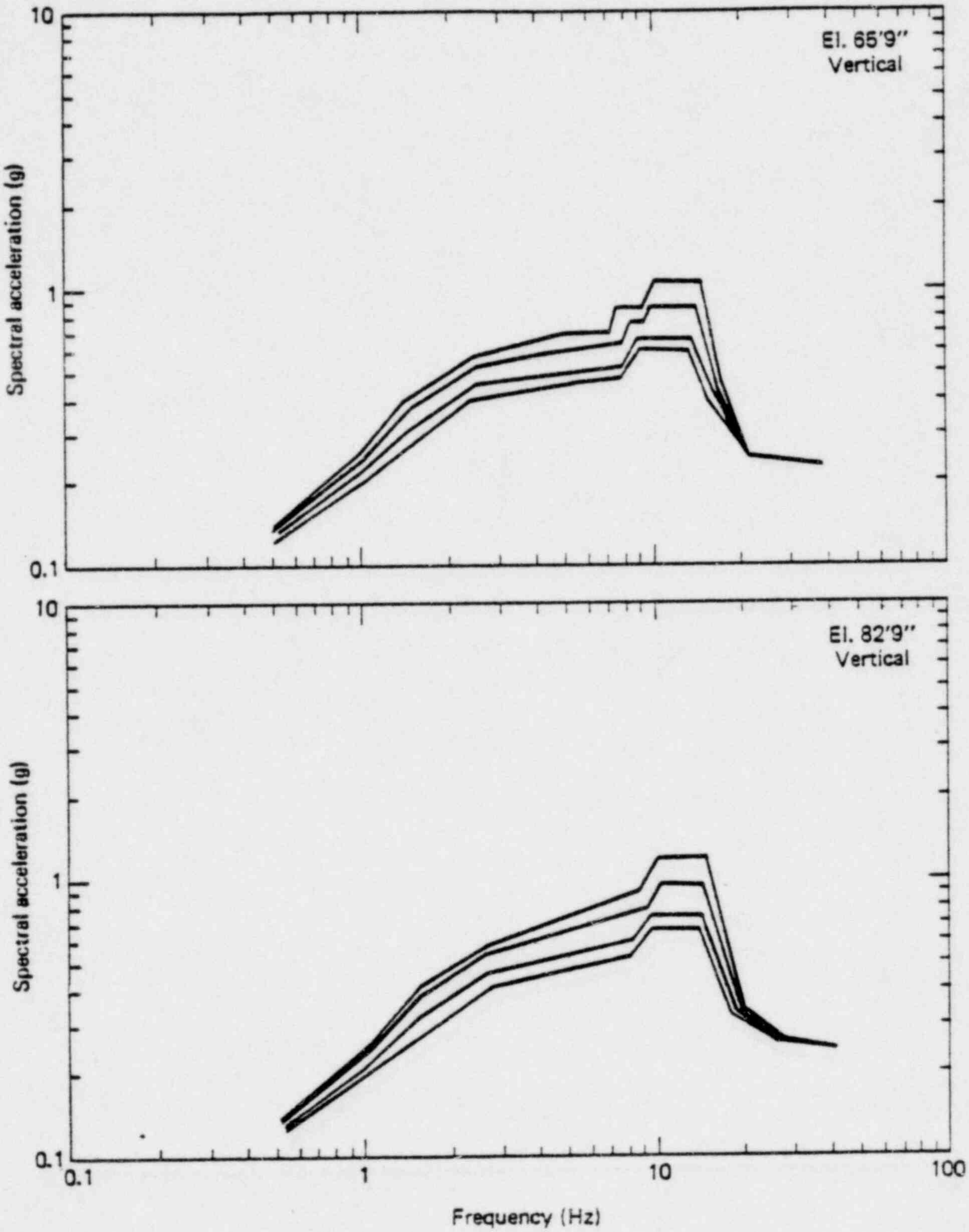

Fig. A-6. Vertical in-structure response spectra at 2, 3, 5, and 7% of critical damping for equipment in the reactor building at els. 65'9" and 82'9".

DRAFT

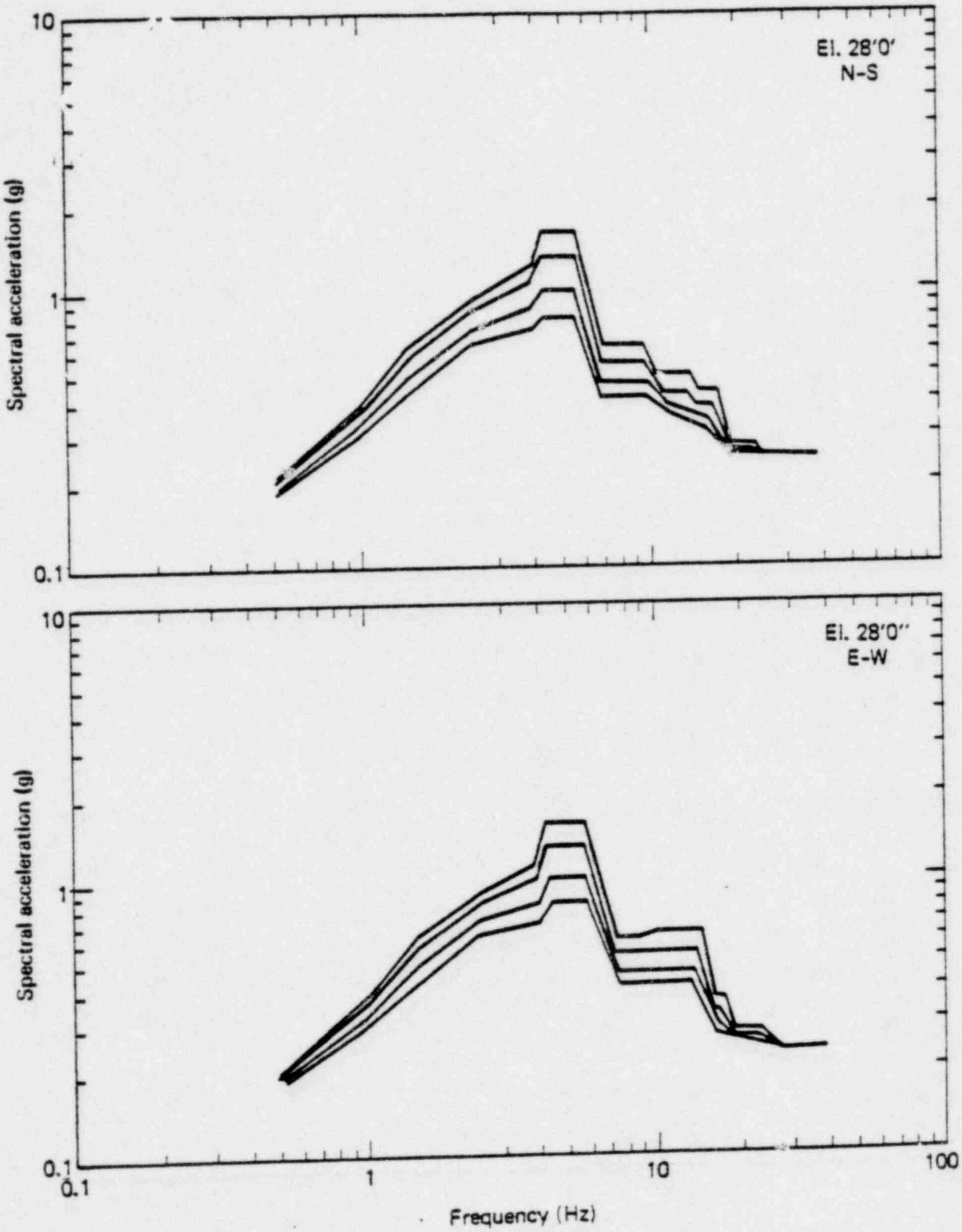

Fig. A-7. Horizontal in-structure response spectra at 2, 3, 5, and 7% of critical damping for equipment on the reactor shield wall at el. 28'0".

DRAFT

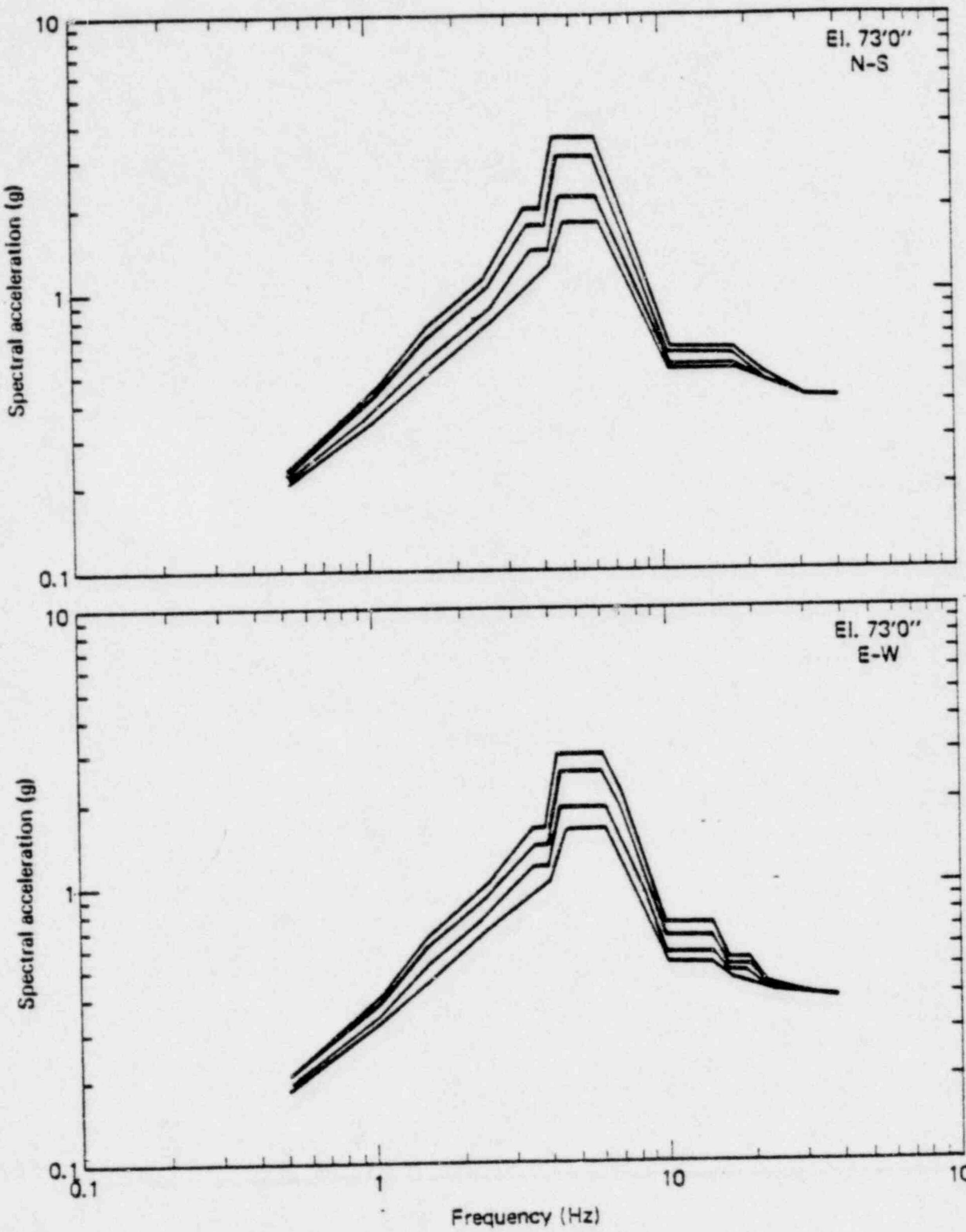

Fig. A-8. Horizontal in-structure response spectra at 2, 3, 5, and 7% of critical damping for equipment on the reactor shield wall at el. 73'0".

DRAFT

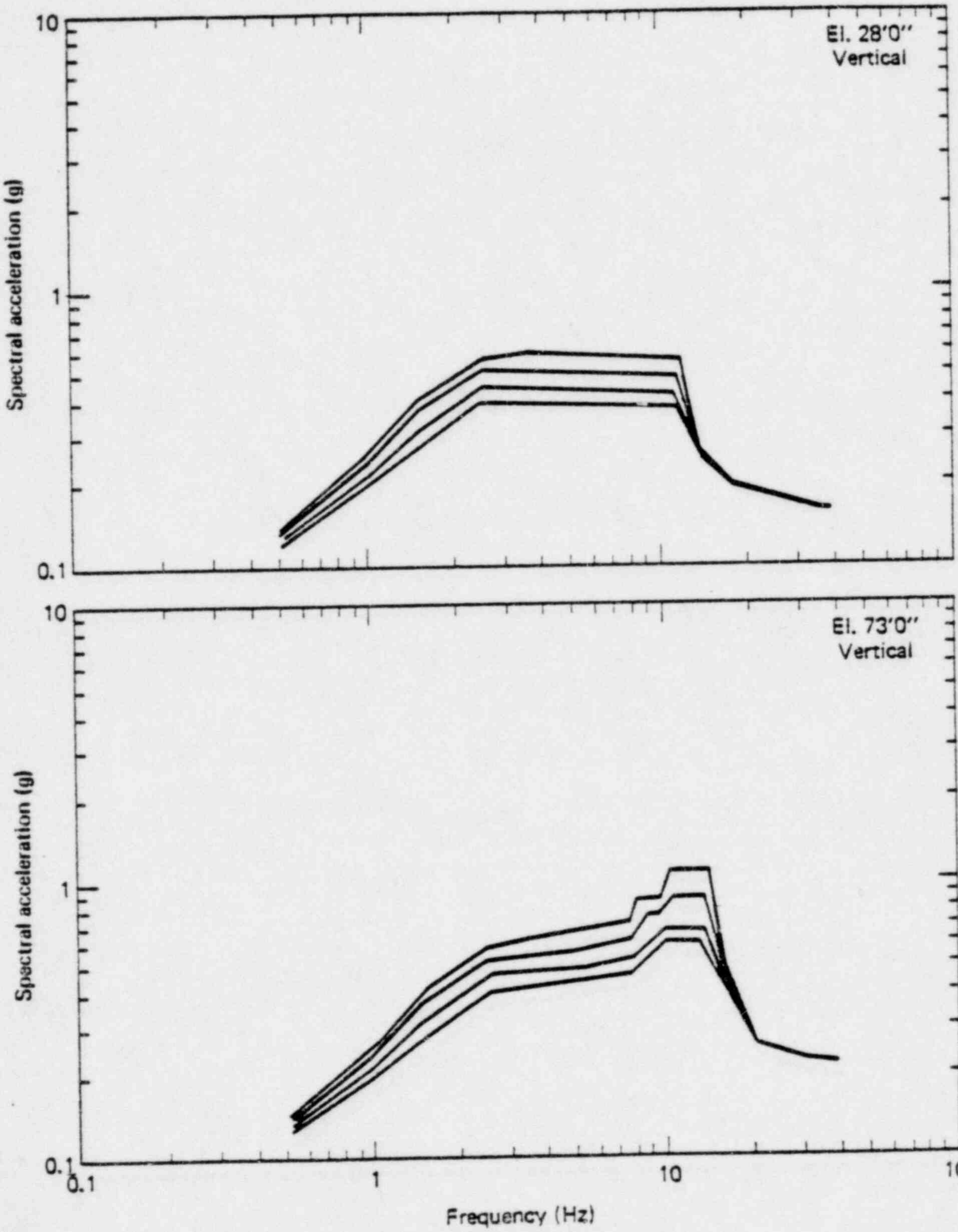

Fig. A-9. Vertical in-structure response spectra at 2, 3, 5, and 7% of critical damping for equipment on the reactor shield wall at els. 28'0" and 73'0".

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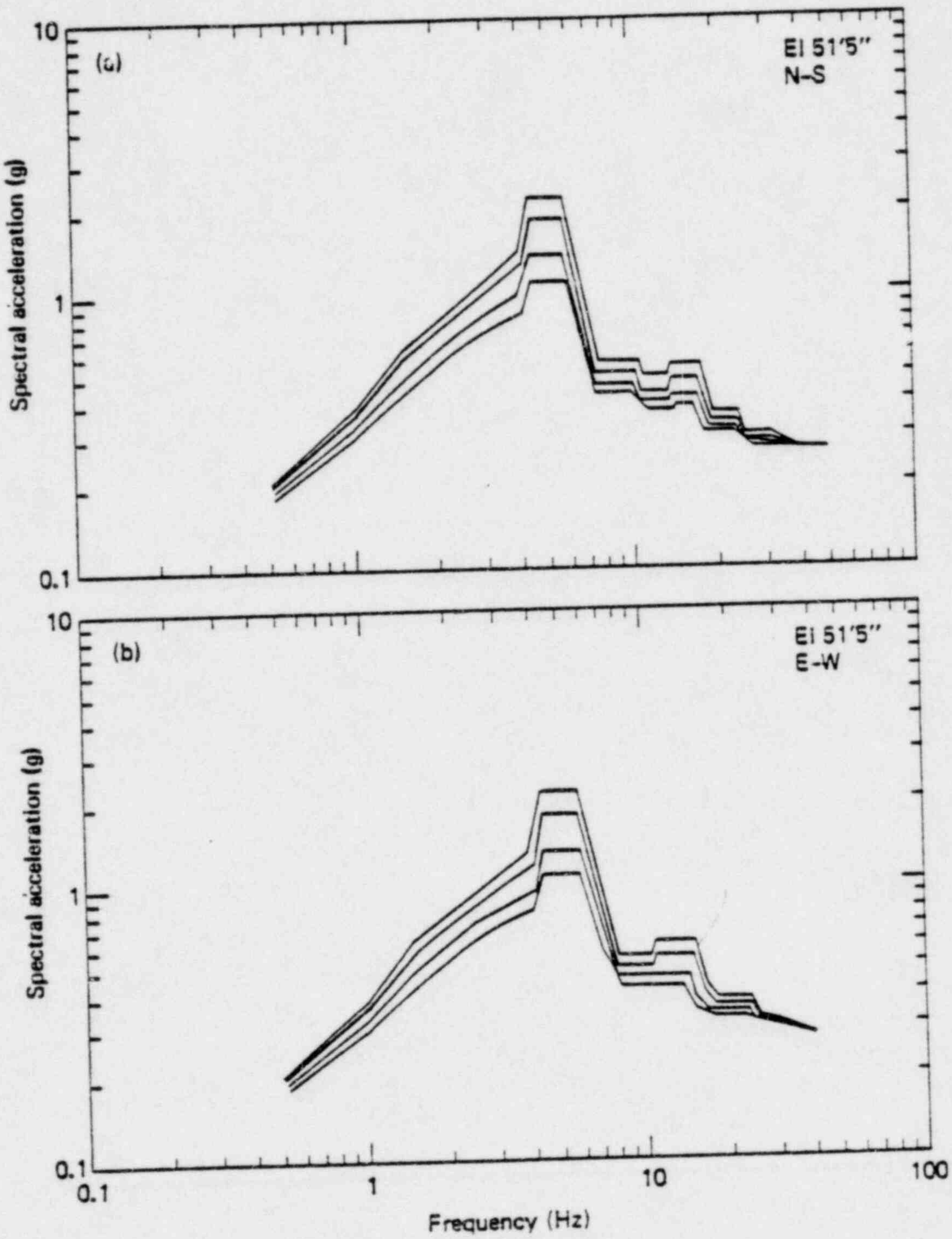

Fig. A-10. Horizontal in-structure response spectra at 2, 3, 5, and 7% of critical damping for equipment in the drywell at el. 51'5".

DRAFT

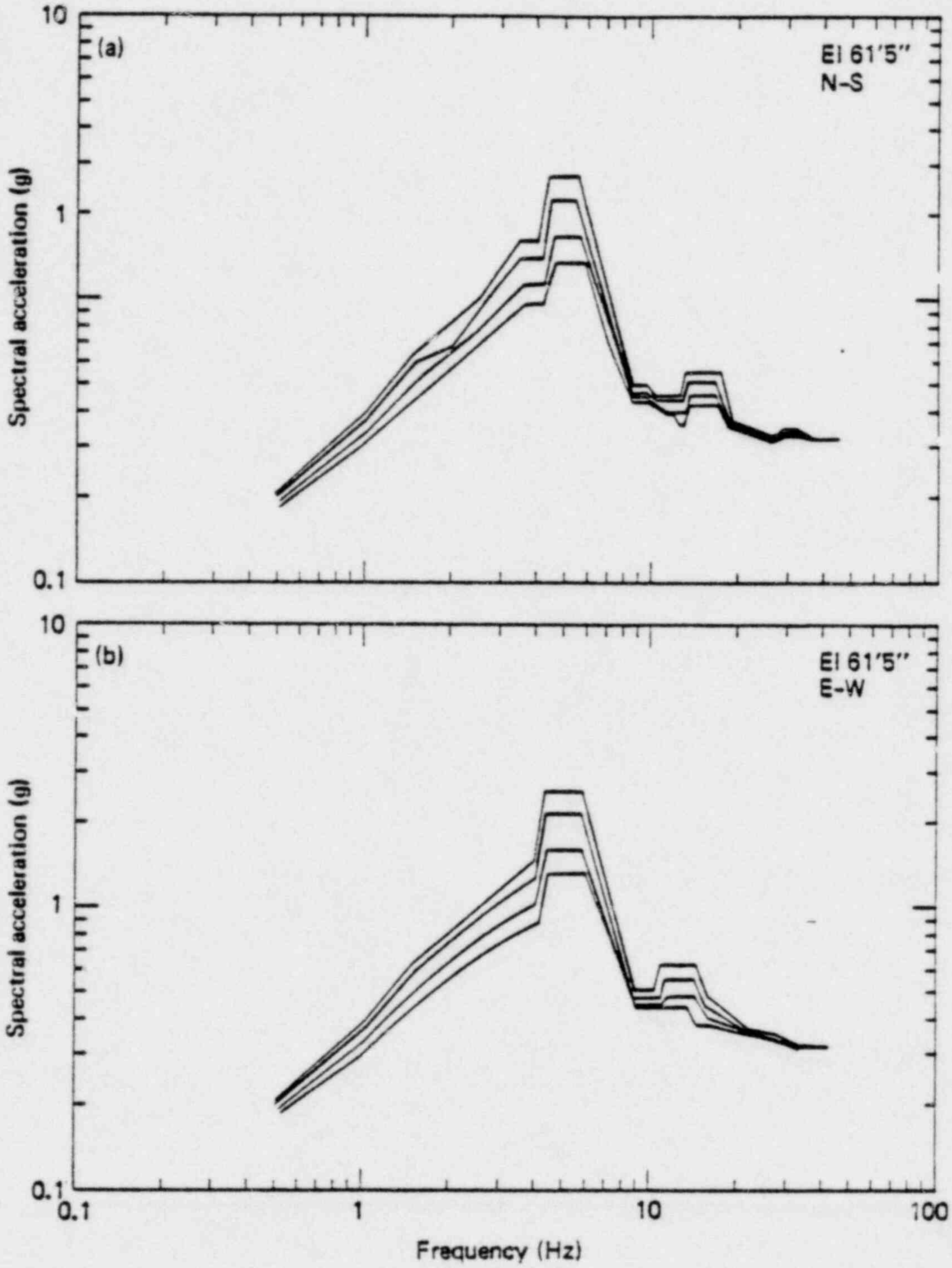

Fig. A-11. Horizontal in-structure response spectra at 2, 3, 5, and 7% of critical damping for equipment in the drywell at el. 61'5".

DRAFT

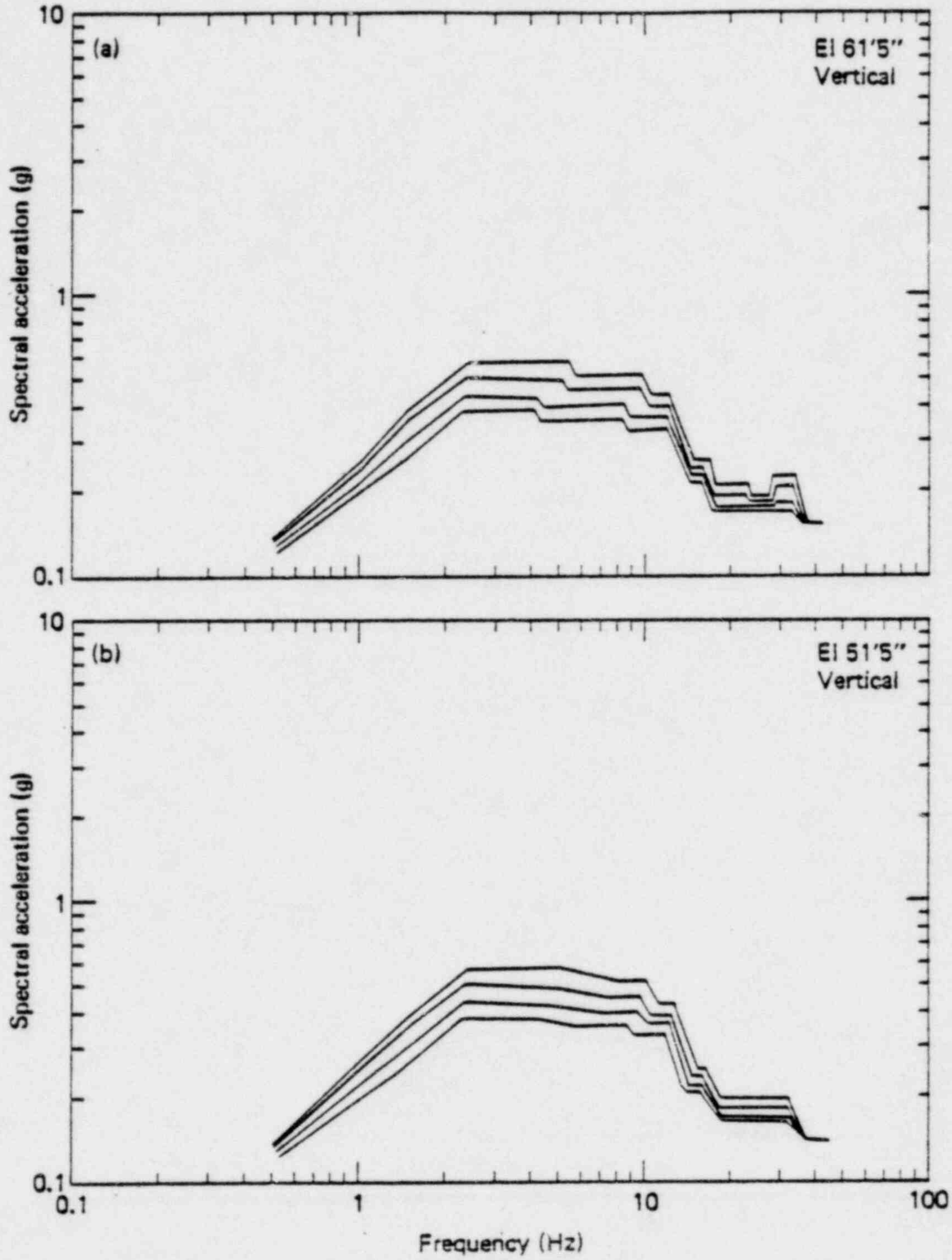

Fig. A-12. Vertical in-structure response spectra at 2, 3, 5, and 7% of critical damping for equipment in the drywell at els. 51'5" and 61'5".

DRAFT

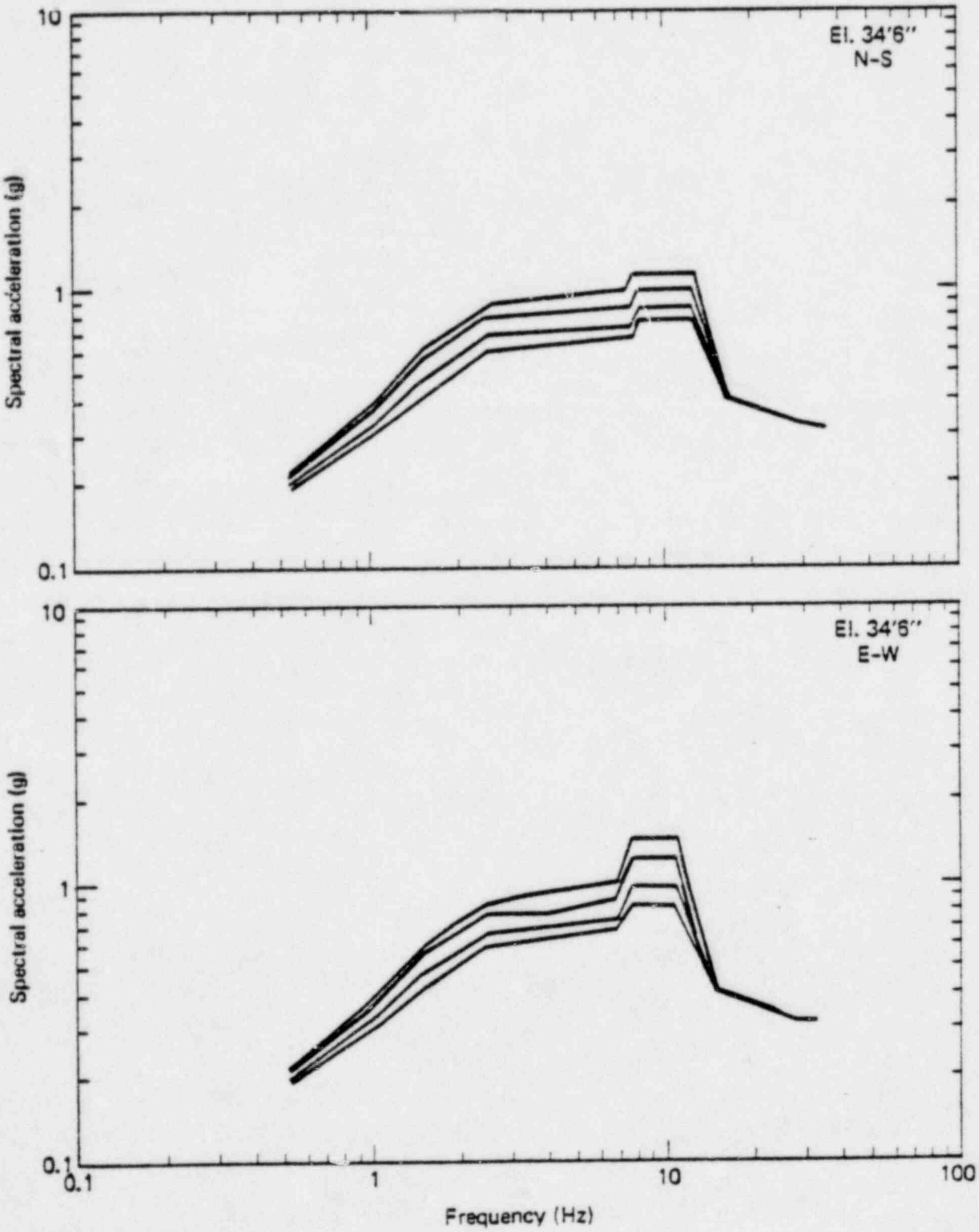

Fig. A-13. Horizontal in-structure response spectra at 2, 3, 5, and 7% of critical damping for equipment in the turbine building at el. 34'6".

DRAFT

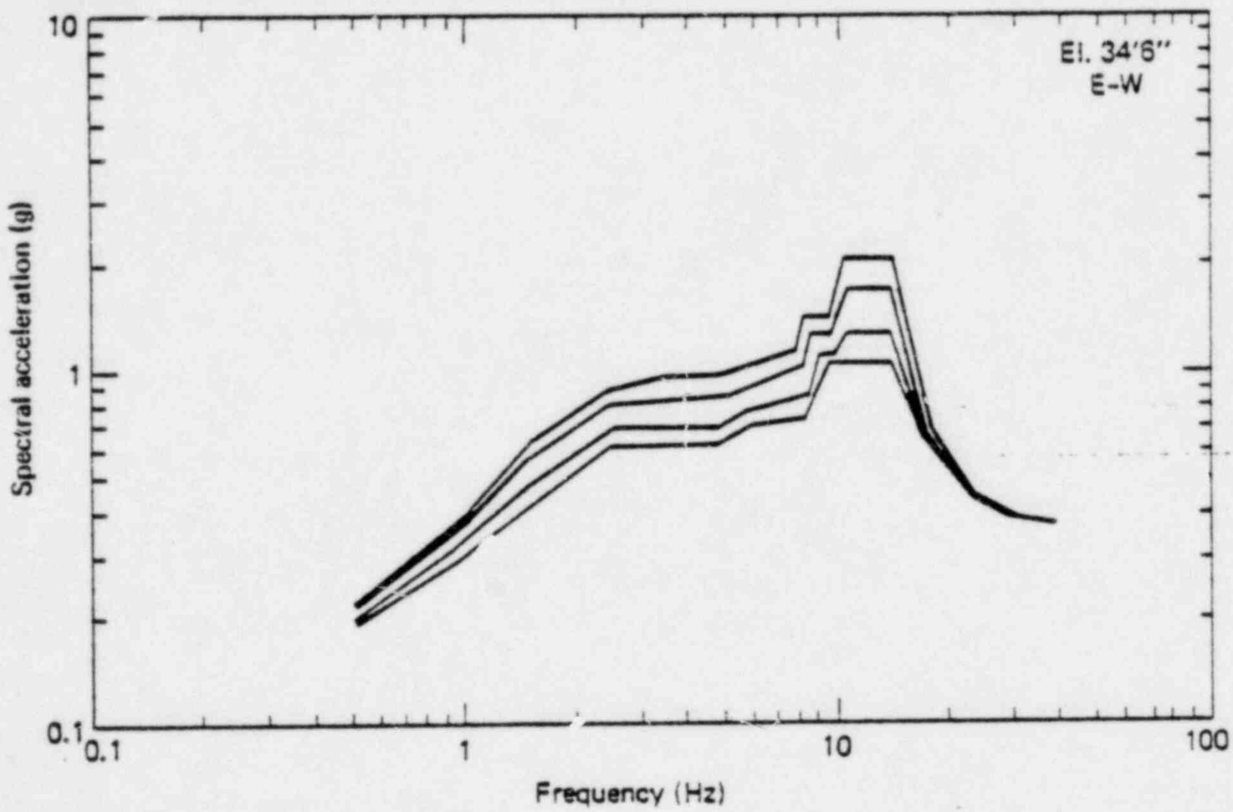

Fig. A-14. Horizontal in-structure response spectra at 2, 3, 5, and 7% of critical damping for equipment in the control room floor at el. 34'6".