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

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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 11 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 11 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. Unit 1 of the Palisades Nuclear Power Plant, the subject of this report, was categorized under Group 1.

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

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

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The SSRT was charged with the following responsibilities:

- To develop the general philosophy of review, setting forth seismic design criteria and evaluation concepts applicable to the review of older nuclear plants, and to develop an efficient, yet comprehensive, review process for NRC staff use in subsequent evaluations.
- To assess the safety of selected older nuclear power plants relative to those designed under current standards, criteria, and procedures, and to recommend generally the nature and extent of retrofitting to bring these plants to acceptable levels of capability if they are not already at such levels.

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

This report provides partial input into the SEP seismic evaluation of Unit 1 of the Palisades Nuclear Power Plant. The results of the seismic evaluation will be documented in a Safety Assessment Report prepared by the NRC staff that will address the capability of the Palisades systems to respond to seismic events or to mitigate the consequences of such events.

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

Safety for seismic excitation implies that certain elements and components of an entire system must continue to function under normal operating and test loads. The SSRT did not review all aspects of the plant's operation and the safety margins available to ensure 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 acknowledge M. Nitzel of EG&G, Idaho Falls, for his piping analysis. The authors also thank T. M. Cheng, technical monitor of this work at the NRC, for his continuing support.

ABSTRACT

A limited seismic reassessment of Unit 1 of the Palisades Nuclear Power Plant was performed by the Lawrence Livermore National Laboratory for the U.S. Nuclear Regulatory Commission as part of the Systematic Evaluation Program. The reassessment focused generally on the reactor coolant pressure boundary and on those systems and components necessary to shut down the reactor safely and to maintain it in a safe shutdown condition following a postulated earthquake characterized by a peak horizontal ground acceleration of 0.2 g. Unlike a comprehensive design analysis, the reassessment was limited to structures and components deemed representative of generic classes. Conclusions and recommendations about the ability of selected structures and equipment to withstand the postulated earthquake are presented.

CONTENTS

Foreword iii

Abstract vii

List of Illustrations xiii

List of Tables xvi

Chapter 1: Introduction 1

1.1 Scope and Depth of Review 1

1.2 Plant Description 3

1.2.1 Seismic Categorization 4

1.2.2 Equipment and Structures 5

1.3 Organization of Report 5

Chapter 2: Summary and Conclusions 8

2.1 Structures 8

2.1.1 Containment Building 9

2.1.2 Auxiliary Building 9

2.1.3 Field-Erected Tanks 9

2.1.4 Underground Piping 10

2.2 Mechanical and Electrical Equipment, and Fluid and Electrical Distribution Systems 10

2.3 Piping 11

2.4 Concluding Remarks 12

Chapter 3: Basis of Reevaluation of Structures and Equipment 14

3.1 General Approach to Reevaluation 14

3.2 Geology, Seismicity, and Site Conditions 15

3.3 Structures 15

3.3.1 Response Spectra, Damping, and Inelastic Behavior 16

3.3.2 Analysis Models and Procedures 17

3.3.3 Normal, Seismic, and Accident Loadings 17
3.3.4 Forces, Stresses, and Deformations 18
3.3.5 Relative Motions 18
3.4 Equipment and Distribution Systems 18
3.4.1 Seismic Qualification Procedures 19
3.4.2 Seismic Criteria 19
3.4.3 Forces, Stresses, and Deformations 19
3.5 Miscellaneous Items 19
3.6 Evaluation of Adequacy 20

Chapter 4: Previous Seismic Analyses 21
4.1 Introduction 21
4.2 Design Earthquake Motion 21
4.3 Seismic Analysis 24
4.3.1 Methods of Analysis 25
4.3.2 Damping 27
4.4 Stress Criteria 27
4.5 Seismic Analysis of Structures 33
4.5.1 Containment Building and Internal Structures 33
4.5.2 Auxiliary Building 40
4.5.3 Turbine Building 43
4.6 Seismic Analysis of Piping 45
4.7 Seismic Analysis of Equipment 47
4.7.1 Control Rod Drive Mechanism 47
4.7.2 Other Reactor Internals 47
4.7.3 Other Major Class 1 System Equipment 47
4.7.4 Other Class 1 Equipment 48

Chapter 5: Reassessment of Selected Structures 50
5.1 Introduction 50
5.2 Design Earthquake Motion 50
5.2.1 Peak Ground Acceleration 51
5.2.2 Ground Motion Characteristics 51
5.3 Seismic Design Methods 55
5.3.1 Soil-Structure Interaction 55

5.3.2 Combination of Earthquake Directional Components 60
5.3.3 Combination of Earthquake and Other Loads 62
5.4 Confirmatory Analysis of Structures 63
5.4.1 Containment Building 63
5.4.2 Auxiliary Building 71
5.4.3 Field-Erected Tanks 79
5.4.4 Underground Piping 83
5.5 Evaluation of Critical Structures 84
5.5.1 Containment Building 85
5.5.2 Auxiliary Building 86
5.5.3 Building-to-Building Interaction 87
5.5.4 Field-Erected Tanks 90
5.5.5 Underground Piping 93
5.6 Seismic Input Motion for Equipment and Piping 94

Chapter 6: Seismic Evaluation of Mechanical and Electrical Equipment and Fluid and Electrical Distribution Systems 96
6.1 Introduction 96
6.1.1 Purpose and Scope 96
6.1.2 Description of Selected Components 98
6.2 Seismic Input and Analytical Procedures 101
6.2.1 Original Seismic Input and Behavior Criteria 101
6.2.2 Seismic Input for SEP Reassessment 102
6.2.3 SEP Acceptance Criteria 103
6.3 Evaluation of Selected Components 104
6.3.1 Mechanical Equipment 104
6.3.2 Electrical Equipment 119
6.4 Piping 124
6.5 Summary and Conclusions 126

References 129

Appendix A: Evaluation of the Effects of Assuming 20% Maximum Modal Damping 135
A.1 Introduction 135
A.2 Method of Analysis 136
A.3 Building Response 138
A.4 In-Structure Response Spectra 141
References for Appendix A 147

Appendix B: In-Structure Response Spectra 149

LIST OF ILLUSTRATIONS

1-1 Plan view of the Palisades plant 6

1-2 Plan view of containment building, auxiliary building, and turbine building, showing the arrangement of major equipment . . 7

4-1 Seismic response spectra used for the OBE analyses of Palisades, for various levels of damping 22

4-2 Seismic response spectra used for the SSE analyses of Palisades, for various levels of damping 23

4-3 Ground response spectrum from the Taft 1952 earthquake, compared to the SSE response spectrum for 4% damping 24

4-4 Floor response spectrum (OBE, 0.5% damping) for the 646-ft level of the containment building 26

4-5 Transverse section of the containment and turbine buildings . . 34

4-6 (a) Preliminary single-stick model of the containment shell, with node weights; (b) inertial forces, shear envelope, and moment envelope from an OBE analysis of the preliminary model 35

4-7 Mathematical model used in the second analysis of the containment building 36

4-8 Response envelopes from a response-spectrum analysis using the model of 4-7 and an OBE of 0.1 g 36

4-9 Two-stick, lumped-mass model of the containment building and internal structure 37

4-10 Principal mode shapes for the two-stick containment building model shown in Fig. 4-9 38

4-11 Response envelopes from an OBE response-spectrum analysis using the model in Fig. 4-9 39

4-12 Mathematical model used in the analysis of the north-south seismic response of the auxiliary building 41

4-13 Mathematical model used in the analysis of the east-west seismic response of the auxiliary building 42

4-14 Maximum displacement, moment, and shear envelopes for the auxiliary building 43

5-1 Comparison of original 7.5% Housner design spectrum to R.G. 1.60 spectra for 10% and 20% of critical damping . . . 54

5-2 Two-stick, lumped-mass model of the containment building and internal structure 56

5-3 Containment building model used in the SEP reevaluation 64

5-4 Mode shapes for the first four modes of the containment building for the median soil case 66

5-5 Shear distribution in the containment building for the lower, median, and upper soil cases, compared to the distribution calculated in the original analysis 68

5-6 Moment distribution in the containment building for the lower, median, and upper soil cases, compared to the distribution calculated in the original analysis 69

5-7 Three-dimensional model used in the reanalysis of the auxiliary building 72

5-8 Mode shapes for the four most important modes of the auxiliary building for the median soil case 74

5-9 Distribution of E-W shear in the auxiliary building for the lower, median, and upper soil cases, compared to the results of the original analysis 75

5-10 Distribution of E-W moment in the auxiliary building for the lower, median, and upper soil cases, compared to the results of the original analysis 76

5-11 Distribution of N-S shear in the auxiliary building for the lower, median, and upper soil cases, compared to the results of the original analysis 77

5-12 Distribution of N-S moment in the auxiliary building for the lower, median, and upper soil cases, compared to the results of the original analysis 78

5-13 Response to overturning moment in Palisades storage tanks 82

5-14 Schematic plan of auxiliary feedwater line 84

5-15 Shear wall plan for the auxiliary building at El 590 ft 88

5-16 Relative displacements of the containment and auxiliary buildings, assuming soft-soil conditions 91

5-17 Linear and nonlinear tank wall force distributions resulting from seismic overturning moment M 92

A-1 Response spectrum (2% damping) used for the time-history analysis of the containment building, superposed on the corresponding smoothed spectrum from R.G. 1.60 137

A-2 Shear distribution in the containment building as determined from the response-spectrum analysis of Chapter 5 and two time-history analyses 139

A-3 Moment distribution in the containment building as determined from the response-spectrum analysis of Chapter 5 and two time-history analyses 140

A-4 Comparison of base slab response spectra 142

A-5 Comparison of response spectra for the concrete internals (El 649 ft) 143

A-6 Comparison of response spectra for the containment vessel . . . 144

A-7 Comparison of response spectra for the concrete internals (El 649 ft), for upper-bound and median soil conditions 145

A-8 Comparison of response spectra for the concrete internals (El 616 ft). 146

B-1 Comparison of horizontal in-structure response spectra for the containment building base slab 150

B-2 Horizontal in-structure response spectra for the containment building base slab, based on R.G. 1.60 spectra . . . 151

B-3 Vertical in-structure response spectra for the containment building base slab, based on R.G. 1.60 spectra . . . 152

B-4 Vertical in-structure response spectra for the containment building base slab, based on R.G. 1.60 spectra . . . 153

B-5 Comparison of horizontal in-structure response spectra for the internal structures (El 616 ft) 154

B-6 Horizontal in-structure response spectra for the internal structures (El 616 ft), based on R.G. 1.60 spectra 155

B-7 Horizontal in-structure response spectra for the internal structures (El 649 ft), based on R.G 1.60 spectra 156

B-8 Horizontal in-structure response spectra for the internal structures (El 649 ft), based on R.G. 1.60 spectra 157

B-9 Horizontal in-structure response spectra for the containment shell (El 730 ft), based on R.G. 1.60 spectra 158

B-10 Horizontal in-structure response spectra for the containment shell (El 730 ft), based on R.G. 1.60 spectra 159

B-11 Comparison of E-W horizontal in-structure response spectra for the auxiliary building base slab 160

B-12 Comparison of N-S horizontal in-structure response spectra for the auxiliary building base slab 161

B-13 East-west horizontal in-structure response spectrum for the auxiliary building (El 610 ft), based on R.G. 1.60 spectra . . . 162

B-14 North-south horizontal in-structure response spectrum for the auxiliary building (El 610 ft), based on R.G. 1.60 spectra . . . 163

B-15 Comparison of E-W horizontal in-structure response spectra for the auxiliary building (El 640 ft) 164

B-16 East-west horizontal in-structure response spectra for the auxiliary building (El 640 ft), based on R.G. 1.60 spectra . . 165

B-17 Comparison of N-S horizontal in-structure response spectra for the auxiliary building (El 640 ft) 166

B-18 North-south horizontal in-structure response spectra for the auxiliary building (El 640 ft), based on R.G. 1.60 spectra 167

B-19 Vertical in-structure response spectra for the auxiliary building (El 640 ft), based on R.G. 1.60 spectra 168

B-20 Vertical in-structure response spectra for the auxiliary building (El 640 ft), based on R.G. 1.60 spectra 169

LIST OF TABLES

3-1 Damping values from R.G. 1.61 compared to those recommended for the SEP evaluation 16

4-1 Seismic analysis methods used to evaluate Class 1 systems . . . 25

4-2 Damping values used in the design of Palisades 28

4-3 Summary of original load combinations and allowable stresses . . 29

4-4 Member properties of the two stick containment building model shown in Fig. 4-9 39

4-5 Maximum permissible spans for rigid piping systems 46

5-1 Original and currently recommended damping ratios, expressed as percent of critical damping 53

5-2 Comparison of median-soil spring rate constants and geometric damping values for Palisades containment building, calculated in different ways 58

5-3 Comparison of median-soil spring rate constants and geometric damping values for Palisades auxiliary building . . . 61

5-4 Containment building response frequencies, in Hz 65

5-5 Modal damping ratios for the containment building (median soil case) 67

5-6 Ratios of the loads calculated for the containment building and the concrete internals in the original analysis to those calculated in the present reanalysis 70

5-7 Auxiliary building response frequencies, in Hz 73

5-8 Modal damping ratios for the auxiliary building (median soil) . . 74

5-9 Ratios of the loads calculated for the auxiliary building in the original analysis to those calculated in the present reanalysis 79

5-10 Descriptions of field-erected tanks 81

5-11 Field-erected tank static design accelerations 82

5-12 Response characteristics of field-erected tanks 83

5-13 Typical member loads and capacities at El 590 ft in the auxiliary building for the postulated 0.2-g SSE 89

6-1 Mechanical and electrical components selected for seismic evaluation and the basis for selection 99

6-2 SEP acceptance criteria for determining seismic design adequacy of passive mechanical and electrical equipment and distribution systems 105

6-3 Stress levels induced in supporting pipes by motor-operated valves 113

6-4 Forces and moments in the steam generator supports caused by loads due to the SSE and a main coolant pipe rupture 116

6-5 Forces and moments in the primary coolant inlet and outlet nozzle due to seismic and pipe rupture loads 117

6-6 Original electrical and instrumentation seismic design qualifications 120

6-7 Summary of conclusions 127

CHAPTER 1: INTRODUCTION

This report describes work at the Lawrence Livermore National Laboratory (LLNL) to reassess the seismic design of Unit 1 of the Palisades Nuclear Power Plant. This limited reassessment includes a review of the original seismic design of selected structures, equipment, and components, and seismic analyses of selected items, using current modeling and analysis methods.

The LLNL work is being performed for the U.S. Nuclear Regulatory Commission (NRC) as part of the Systematic Evaluation Program (SEP). The purpose of the SEP is to develop a current documented basis for the safety of 11 older operating nuclear reactors, including Palisades. The primary objective of the SEP seismic review program is to make an overall seismic safety assessment of the plants and, where necessary, to recommend backfitting in accordance with the Code of Federal Regulations (10 CFR 50.109).[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[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 Palisades is considerably different in scope and depth from current reviews for construction permits and operating licenses. Its focus is limited to identifying safety issues and to providing an integrated, balanced approach to backfit considerations in accordance with 10 CFR 50.109, which specifies that backfitting will be required only if substantial additional protection can be demonstrated for the public health and safety. Such a finding requires an assessment of broad safety issues by considering the interactions of various systems in the context of overall plant safety.

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

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

The Palisades assessment focuses on the integrity of the reactor coolant pressure boundary--that is, components that contain coolant for the core, and piping or any component not isolable (usually by a double valve) from the core--and the capability of essential systems and components required to shut down the reactor safely and to maintain it in a safe shutdown condition during and after a postulated seismic disturbance. The assessment of this subgroup of equipment can be used to infer the capability of other safety-related systems, such as the emergency core cooling system (ECCS).

To review the selected systems, an evaluation was made of the reactor containment building (together with its internal structures) and the auxiliary building to demonstrate structural adequacy and to obtain seismic input to equipment. Field-erected tanks and a typical buried pipe were also analyzed. A zero-period peak horizontal ground acceleration of 0.2 g was employed along with R.G. 1.60 response spectra for the structural evaluations.

Mechanical and electrical equipment representative of items installed in the reactor coolant system and safe shutdown systems at the Palisades facility were examined for structural integrity and for electrical and mechanical functional operability. In order to develop a basis for evaluating the estimated lower-bound seismic capacity of mechanical and electrical components and distribution systems, we visited the site and identified components for review that potentially have a high degree of seismic fragility. The methods of selection of the representative equipment for this limited assessment are described in detail in Chapter 6.

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

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

The report addresses structures, systems, and components in the as-built condition and considers those modifications since the issuance of the operating license that have been made to the selected Class 1 components. Information about structures, systems, and components was primarily obtained from the Palisades docket (Docket 50255) maintained by the NRC in Bethesda, Maryland. Additional information was supplied by the utility and the architect-engineer either through correspondence or during site visits.

1.2 PLANT DESCRIPTION

The Palisades plant is part of the Michigan power pool, providing energy to both the Consumers Power Company (CPCo), which owns and operates the plant, and the Detroit Edison Company. It is located on the east shore of Lake Michigan, about 5 mi south of South Haven, Michigan, and about 16 mi north of Benton Harbor, Michigan. The plant's Unit 1 is a pressurized light-water moderated and cooled nuclear reactor, commonly designated as a PWR. The plant was designed to produce 2650 MW of heat and 845 MW of net electrical power.

Combustion Engineering, Inc., designed and supplied the nuclear fuel system and the nuclear steam supply system (NSSS), which includes the reactor vessel, steam generators, pressurizer, and pumps, plus auxiliary system components, instrumentation, and the reactor protective system. This NSSS was the first supplied by Combustion Engineering. Bechtel Corporation and its affiliate, Bechtel Power Company, designed and supplied the remaining plant structures, systems, and equipment. Bechtel Corporation actually constructed the entire plant, including the NSSS, for which Combustion Engineering gave technical advice. Westinghouse Electric Corporation supplied the turbogenerator.

The Atomic Energy Commission issued Construction Permit No. CPPR-25 to CPCo on March 13, 1967. Provisional Operating License No. DPR-20 was issued on March 24, 1971. CPCo filed for a full-term operating permit on January 22, 1974.

1.2.1 Seismic Categorization

According to Appendix A of the Final Safety Analysis Report (FSAR),[6] the plant equipment and structures were categorized in one of three seismic classes:

- Class 1: those structures, systems, and equipment whose failure could cause uncontrolled release of radioactivity, or those essential for immediate and long-term operation following a loss-of-coolant accident (LOCA).
- Class 2: those structures, systems, and equipment that can sustain limited damage without endangering safe shutdown of the NSSS following a reactor trip or normal shutdown. The failure of Class 2 items could not result in the uncontrolled release of radioactivity.
- Class 3: those structures and components whose failure would not result in the release of radioactivity and would not prevent reactor shutdown, but may interrupt power generation.

Note that these classifications differ from those in Regulatory Guide 1.29,[7] which was issued after the design of Palisades.

1.2.2 Equipment and Structures

Inherent to the design of a PWR, a closed-cycle reactor, are four barriers that prevent fission products from reaching the environment:

- Fuel matrix.
- Fuel cladding.
- Reactor vessel and coolant loops.
- Reactor containment building.

The reactor core comprises uranium dioxide pellets enclosed in Zircaloy tubes with welded end plugs. Two closed reactor coolant loops, connected in parallel to the reactor vessel, constitute the reactor coolant system (RCS). Each loop has two reactor coolant pumps and a steam generator.

The reactor containment building is a post-tensioned, prestressed concrete cylinder and dome structure. The walls are prestressed vertically and circumferentially, and the dome is prestressed with three groups of tendons oriented at 120° to each other. The containment building is described in more detail in Sec. 4.5.1. The structurally independent auxiliary and turbine buildings are located to the north and west of the containment building (Fig. 1-1). They are described briefly in Secs. 4.5.2 and 4.5.3, respectively. A schematic plan view of these three major structures is shown in Fig. 1-2.

1.3 ORGANIZATION OF REPORT

The report has six chapters. Chapter 2 is a summary of the overall assessment of the ability of Palisades to resist the stipulated SSE event. Included is an evaluation of the significance of any identified deficiencies or areas that may require further study. Chapter 3 contains a description of the general basis for reevaluation of structures and equipment. Chapter 4 includes a presentation of the original facility seismic design methods, models, and criteria for structures, equipment, and piping; it also summarizes the available original calculated seismic responses. Chapter 5 contains a comparison of the seismic loadings and responses for which the facility structures were originally designed with corresponding seismic loadings and

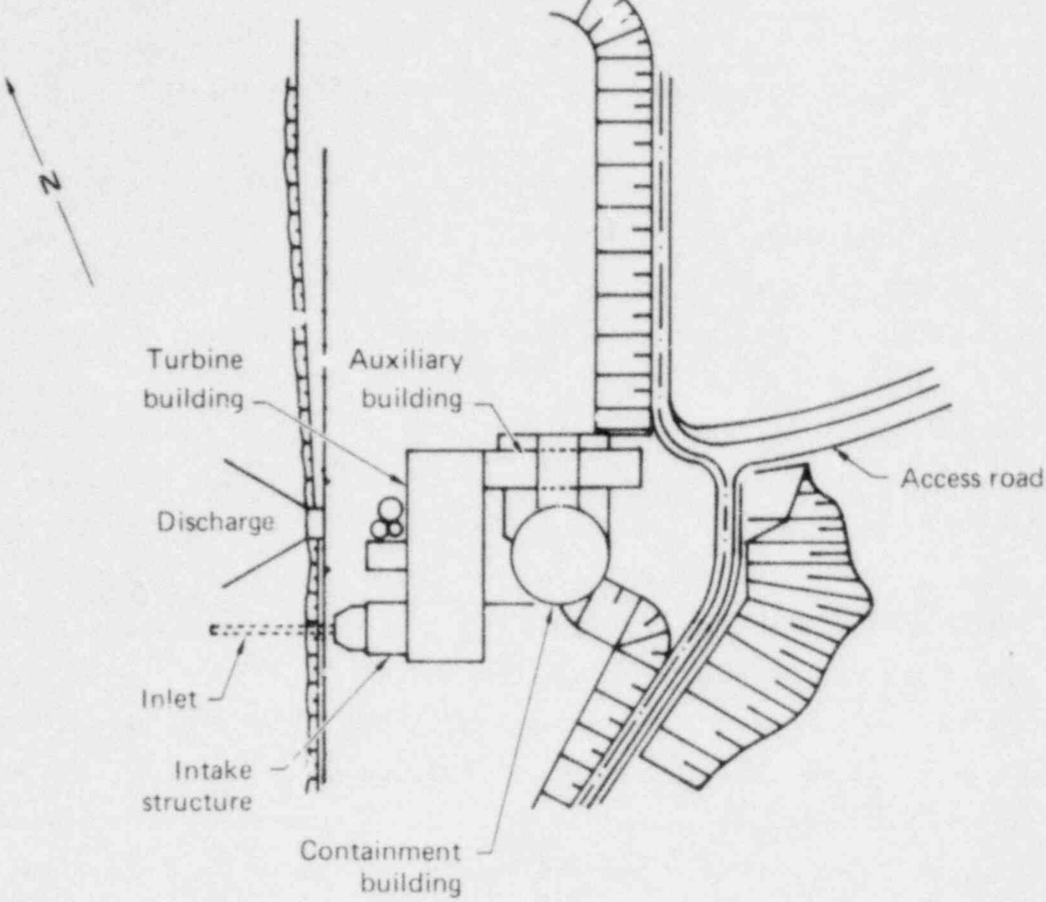


FIG. 1-1. Plan view of the Palisades plant.

responses derived using techniques thought more realistic in light of current knowledge. Chapter 6 uses the in-structure response spectra generated in Chapter 5, as well as other available information, to evaluate the capability of mechanical and electrical equipment and of fluid and electrical distribution systems to resist seismic loads and to perform their necessary safety functions.

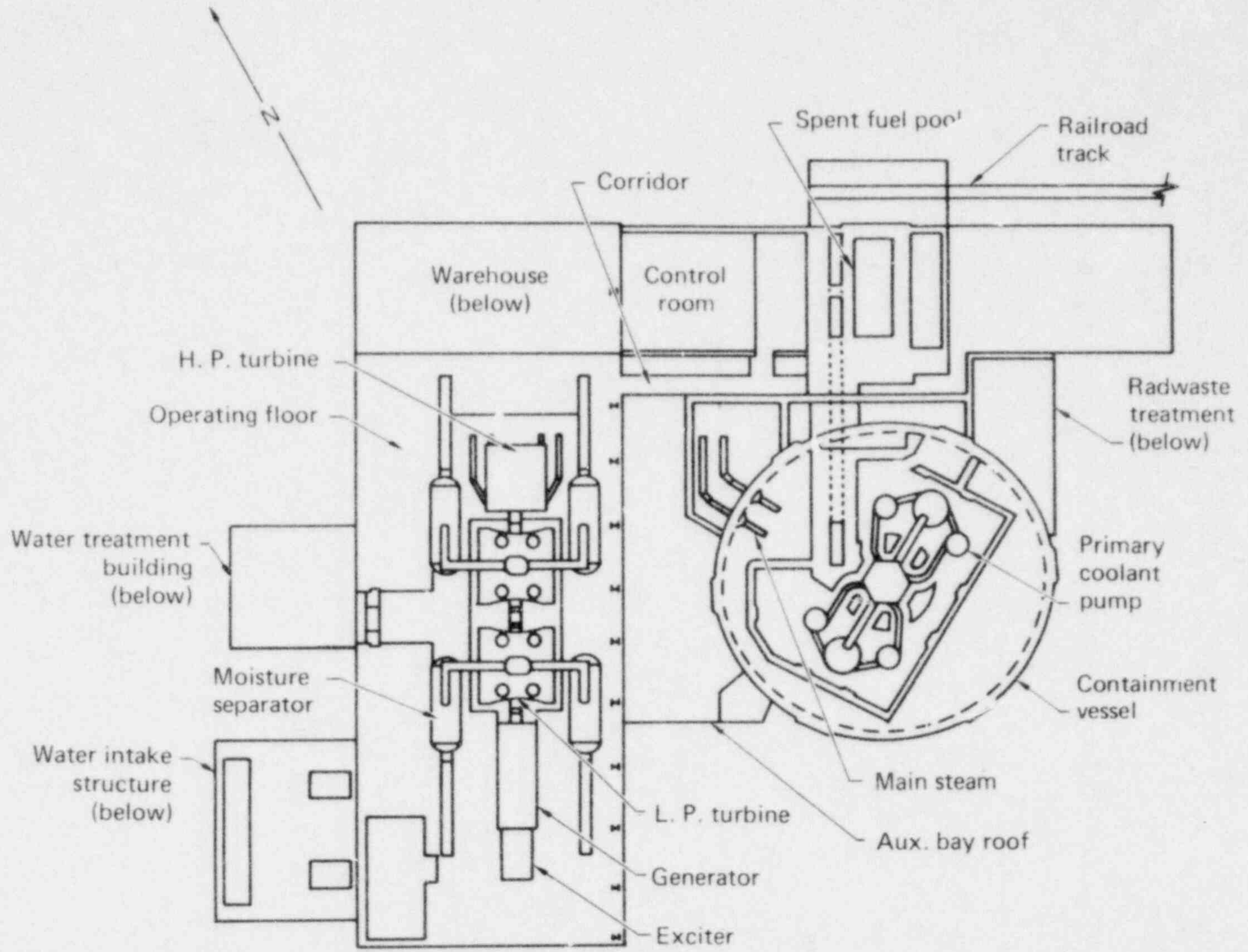


FIG. 1-2. Plan view of containment building, auxiliary building, and turbine building, showing the arrangement of major equipment.

CHAPTER 2: SUMMARY AND CONCLUSIONS

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

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

2.1 STRUCTURES

We evaluated the containment building (and its internal structures) and the auxiliary building to demonstrate structural adequacy and to obtain seismic input to equipment. We also reexamined the structural adequacy of field-erected tanks and a typical buried pipe. For the SSE structural evaluation, a peak horizontal ground acceleration of 0.2 g was used along with R.G. 1.60 response spectra.

New analytical models were developed for the containment and auxiliary buildings that accounted for the soil-structure interaction effects of the layered site. A fairly broad range of soil properties was used to account for the uncertainties in the soil characteristics.

For each structural analysis, seismic response loads were calculated and were compared to the seismic loads used in the design of the structure. Where the loads based on the reevaluation guidelines were less than those used for design, the structure was judged to be adequate without additional evaluation. Where the recalculated loads significantly exceeded the design loads or where the original design loads were not available, stress analyses of the controlling structural elements were conducted. Where the seismic stresses were found to be low compared to yield, the structure was again judged to be adequate. For structures with higher stresses, the effects of structure ductility were included as required.

2.1.1 Containment Building

The loads developed in the containment building for the stiffer soil conditions exceed those used in design; however, we found the resulting stresses to be well below yield for both the concrete internal structures and the containment vessel. Based on relatively low structural damping ratios consistent with the low seismic stresses, minimum factors of safety of 1.5 for the internals and 2.1 for the containment vessel were computed. We conclude that the containment building is capable of withstanding the 0.2-g SSE.

2.1.2 Auxiliary Building

Using a new three-dimensional analytical model, we found that the loads developed in the auxiliary building exceed the original seismic design loads. Based on the minimum concrete design strength but neglecting the ϕ factor for workmanship, our analysis showed that all elements of the auxiliary building structure remain well below yield for the SSE. If the ϕ factor of 0.85 is included, one shear wall at the northeast corner of the building at El 590 ft is expected to experience light cracking. The auxiliary building is considered capable of withstanding the 0.2-g SSE with no loss of function.

We also considered the consequences of possible interactions between the containment, auxiliary, and turbine buildings. No structural damage sufficient to cause any loss of function is predicted.

2.1.3 Field-Erected Tanks

Three field-erected tanks were evaluated for the SSE in terms of SEP guidelines. These were the T-2 condensate storage tank (CST), the T-58 safety injection and refueling water tank (SIRWT), and the T-81 primary water supply make-up storage tank (PWSMST). The T-2 CST and T-58 SIRWT were both found to be adequate to withstand the 0.2-g SSE with no stresses above yield. However, the SSE will produce loads in the T-81 PWSMST that will stress the anchor bolts well above yield. Failure of these bolts is expected to result in tank wall buckling and possible base plate failure, leading to a loss of water. If this tank is confirmed to be essential for safety, modifications should be implemented to increase the anchor capacity.

2.1.4 Underground Piping

We evaluated the auxiliary feedwater line to assess the adequacy of buried pipes at Palisades. Conservative assumptions were made concerning the soil strains expected during the SSE. Stresses were computed at discontinuities and at penetrations (where relative motion between structure and soil could occur), as well as in the straight-run sections. No stresses above ASME Code allowables were calculated in the auxiliary feedwater line. Assuming it to be a typical buried pipe, we conclude that critical buried pipelines will not fail as a result of the 0.2-g SSE.

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

As discussed in Chapter 6, typical mechanical and electrical equipment components were selected for review in large part on the basis of the judgment and experience of the SEP seismic review team comprising the authors and certain SSRT and NRC staff members. The documentation that exists regarding the original specifications applicable to procurement of equipment, as well as documentation concerning qualification of the equipment, varies greatly. In some cases the qualification for an item of equipment is quite specific, whereas in other cases the qualification pertains only to a class of equipment.

Because we lacked essential seismic design and qualification data, our review of the seismic design adequacy of mechanical and electrical equipment is incomplete. Additional data in the form of analysis or test results must be developed before definite conclusions can be drawn. Therefore, based upon the design review and independent calculations made for this reassessment, we were unable to confirm the capability of the following mechanical and electrical components to withstand the 0.2-g SSE without loss of structural integrity and required safety function:

- Essential service water pump: design details unavailable.
- Auxiliary feedwater pumps: design details unavailable.
- Diesel generator oil storage tanks: no evaluation performed because of lack of information.
- Safety injection tank: additional analysis of support structure required.

- Motor-operated valves: further analysis needed.
- Control rod drive mechanism: further analysis required to ensure active function.
- Steam generators: design details unavailable.
- Reactor coolant pumps: design details unavailable.
- Reactor vessel supports and internals: design details unavailable.
- Battery racks: lateral bracing should be replaced or strengthened.
- Motor control centers: design details unavailable.
- Switchgear: confirmation of anchorage design details necessary; other design details unavailable.
- Control room electrical panels: licensee to verify seismic design adequacy.
- Transformers: end units should be securely anchored; other design details unavailable.
- Electrical cable raceways: analysis of support systems needed.

2.3 PIPING

Pending completion of the final piping analysis report, only preliminary results are available. Portions of four piping systems were analyzed. Throughout, it was assumed that suitable stress analyses of the supports and substructure were performed for the original loads. In addition to the conclusions below, concern was expressed about the adequacy of pipe wall thicknesses for certain pipe sizes.

- Residual heat removal system. Seismic stresses in the piping were found to be well within allowable limits; however, support and anchor loads based on the SEP acceptance criteria are generally higher than those determined in the original analyses. Further analysis is warranted. Further consideration should be given to the nozzles, since the anchor moment loading is significantly greater than the design loading.
- Component cooling system. Piping stresses are well within allowable limits. Support loads were found to be generally higher than available design loads; however, no failure is anticipated during the postulated SSE. Anchor loads determined in the reanalysis were also

generally higher than original loads. Further consideration should be given to the nozzles in cases where the anchor loads have increased significantly.

- Auxiliary feedwater system (three models, including the steam line to the P-8B turbine). Reanalysis results for two portions of the system show that ASME Code stress limits will be exceeded during an SSE. Additional lateral and vertical dynamic supports are needed in sections of the steam line to pump P-8B; further analysis is necessary for overstressed support R227C near pump P-8A. Insufficient data were available for anchor load comparisons. Analysis of the third auxiliary feedwater model showed no excessive stresses; however, adequate data on original support and anchor loadings were not available for comparison.
- Regenerative heat exchanger letdown. Code allowable stresses will be exceeded in the piping, primarily due to a deficiency in axial and lateral restraint in a single vertical leg. Insufficient data were available for comparisons of support and anchor loads.

2.4 CONCLUDING REMARKS

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

- Structures and structural elements of the Palisades facility are adequate to resist an earthquake with a peak horizontal ground acceleration of 0.2 g, provided that the anchor capacity of the T-81 PWSMST is increased (Sec. 5.5.4).
- In view of the limited amount of both analysis and test documentation, no definitive statement can be made about the overall seismic design adequacy of mechanical and electrical equipment or of the piping systems. More data must be developed before equipment seismic design adequacy can be determined in accordance with evaluation criteria in this report.

We therefore recommend that

- Modifications be made as necessary to the mechanical and electrical equipment items listed in Sec. 2.2, to the auxiliary feedwater piping, and to the regenerative heat exchanger letdown piping; and where definitive conclusions could not be drawn, that additional analysis be performed.
- All safety-related electrical equipment in the plant be checked for adequate engineered anchorage; that is, the anchorage should be found to be adequate on the basis of analysis or tests employing design procedures (load, stress and deformation limits, materials, fabrication procedures, and quality acceptance) in accordance with a recognized structural design code.
- A general reconnaissance of the plant be made to identify items that are (1) overhead or suspended, (2) on rollers, or (3) capable of sliding or overturning. All such items, whether permanently installed or not, that could dislodge, fall, or displace during an earthquake and impair the capability of the plant to shut down safely should be upgraded so that they no longer jeopardize the plant.

CHAPTER 3: BASIS OF REEVALUATION OF STRUCTURES AND EQUIPMENT

3.1 GENERAL APPROACH TO REEVALUATION

The seismic reevaluation part of this study centers on

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

To accomplish this level of reevaluation, it is necessary to assess the factors of safety of essential structures, components, and systems of the older plant, relative to those designed under current standards, criteria, and procedures. Such evaluation should help to define the nature and extent of any retrofitting required or possible 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 implying an evaluation based on the norm of current criteria, standards, and procedures, but instead, an assessment made in the light of knowledge that led to such a level of design. It would be irrational to assume that an older plant would consist of structures, equipment, components, and systems that would meet current criteria in every instance; even so, those items that do not meet current criteria may be entirely adequate in the sense of meeting acceptable safety and reliability criteria.

Within the scope of the investigation, it was impossible to reexamine every item in detail. On the other hand, by examining structures, equipment, components, and systems individually, it was felt that it would be possible to assess their adequacy and general factors of safety for meeting the selected SSE hazard. Thereafter, on the basis of an evaluation of the 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 factor 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 GEOLOGY, SEISMICITY, AND SITE CONDITIONS

The seismicity information forms the basis for arriving at the effective peak transient ground motions (acceleration, velocity, and displacement) for use in deriving response spectra, time histories, etc., in the reevaluation. Thus, one important initial basis of reevaluation is a comparison of the original seismic design criteria with those selected for reevaluation. Another important basis for reevaluation is the treatment of soil-structure interaction (SSI). More accurate methods for computing SSI are available today than were in use when Palisades was designed. This is especially true of layered sites such as Palisades. Existing soils information was thus used in the reevaluation.

3.3 STRUCTURES

The first task in examining structures is to summarize the nature and makeup of the structures, based on knowledge about original design criteria and information on the as-constructed plant. Also required are summaries of the design analysis approaches employed, including loading combinations, stress and deformation criteria, and controlling response calculations. Chapter 4 provides these summaries. In evaluating the seismic design criteria, it is generally necessary to have information concerning the response spectra used originally, the applicable levels of damping, and the modeling approach used in the analyses. Also needed are details of input and methods of analysis used in designing mechanical equipment, piping, and electrical system supports.

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

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

3.3.1 Response Spectra, Damping, and Inelastic Behavior

One basis for evaluation will be comparison of the original response spectra with the response spectra applicable to the reevaluation, taking account of appropriate damping values. The damping values specified in R.G. 1.61[8] and those recommended in NUREG/CR-0098[4] for reevaluation purposes are summarized in Table 3-1. The reason for permitting higher damping values

TABLE 3-1. 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[a]
Reinforced concrete	7	7 to 10
Prestressed concrete	5	5 to 7[b] 7 to 10[c]
Welded assemblies	4	5 to 7
Bolted and riveted assemblies	7	10 to 15
Piping	2 or 3	2 to 3

[a]Recommended for yield level.

[b]Without complete loss of prestress.

[c]No prestress left.

is discussed in Ref. 4. Although there are limited data on which to base damping values, it is known that the R.G. 1.61 values are conservative to ensure that adequate dynamic response values are obtained for design purposes. The lower values recommended in NUREG/CR-0098 are, in most cases, close to the R.G. 1.61 values. The higher values in the NUREG/CR-0098 column are best-estimate values believed to be average or slightly above average values for typical structures. It is recommended that these higher values be used in design or evaluation for stresses at or near yield, and when moderately conservative estimates are made of the other parameters entering into the design or evaluation.

A second basis for evaluation is the level of inelastic response exhibited by the structures, as measured by ductility factors. It is recommended in Ref. 4 that low ductility factors (1.3 to 2) be used for conservatism and to help ensure that no gross deformation occurs in any critical safety elements. This, in turn, ensures that system ductility is maintained at a low value. Local inelastic behavior, which arises from deformation of a number of interconnected elements, may result in larger local ductility factors than those predicted at the system level. An assessment of the local element deformation and its role in system performance needs careful evaluation and is largely judgmental. Local element ductility should be permitted in equipment only if it can be clearly demonstrated that functional ability is not impaired and that a significant margin of strength still remains.

3.3.2 Analysis Models and Procedures

The reevaluation also considers the adequacy of the original analysis models, and assesses the possible effects of SSI, overturning, and torsion. Analysis procedures used in the reevaluation should be in keeping with the state of the art. In general, response-spectrum or time-history analyses are used unless other reasons dictate other approaches more or less sophisticated.

3.3.3 Normal, Seismic, and Accident Loadings

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

3.3.4 Forces, Stresses, and Deformations

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

3.3.5 Relative Motions

The reevaluation takes account of the effects of any gross relative motions that might influence piping entering buildings or spanning spaces between buildings, the effects of tilt, and other interaction effects.

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 of 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 the systems selected, representative items or systems were identified on the basis of

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

After identifying appropriate systems or items, and after ascertaining the nature of the seismic criteria used during procurement or qualification, the reevaluation effort turns to 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. Consideration is given to such items as the in-structure response spectra, 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 the seismic component of load, stress, or deformation.

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 regard to dams, intake structures, cooling water piping, etc.

3.6 EVALUATION OF ADEQUACY

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

CHAPTER 4: PREVIOUS SEISMIC ANALYSES

4.1 INTRODUCTION

This chapter presents the original seismic design criteria for Palisades. The seismic loadings and allowable stress criteria for Class 1 structures, equipment, and piping are defined, and the calculated seismic responses of critical structures are described. The data presented in this chapter are used to define the design basis and to form the basis for comparison with SEP acceptance criteria in Chapters 5 and 6. Most of the information has been drawn from the FSAR;[6] detailed references are given later, in the sections describing the individual analyses.

4.2 DESIGN EARTHQUAKE MOTION

Palisades was designed for an operating basis earthquake (OBE) with a peak ground acceleration (A_{max}) of 0.10 g and for a safe shutdown earthquake (SSE) with an A_{max} of 0.20 g. (The OBE values are called "design earthquake loads" in the FSAR, and SSE values are called "maximum credible earthquake" values.) The vertical component of acceleration, when considered in the dynamic analyses, was assumed to be two-thirds of the horizontal component.

Response spectra for structural design were developed from spectra in Ref. 9. Several acceleration spectra--including Taft 1952, Olympia 1949, El Centro 1934, and El Centro 1940--were normalized to A_{max} = 0.33 g, combined, and smoothed, then plotted on tripartite graph paper. The resulting response spectrum was multiplied by appropriate scaling factors, corresponding to the OBE and SSE accelerations. The results were the design spectra shown in Figs. 4-1 and 4-2. In-structure spectra developed from the Taft record alone were also used in several equipment analyses. Figure 4-3 compares the ground response spectrum from the Taft earthquake to the smoothed design spectrum for 4% damping.

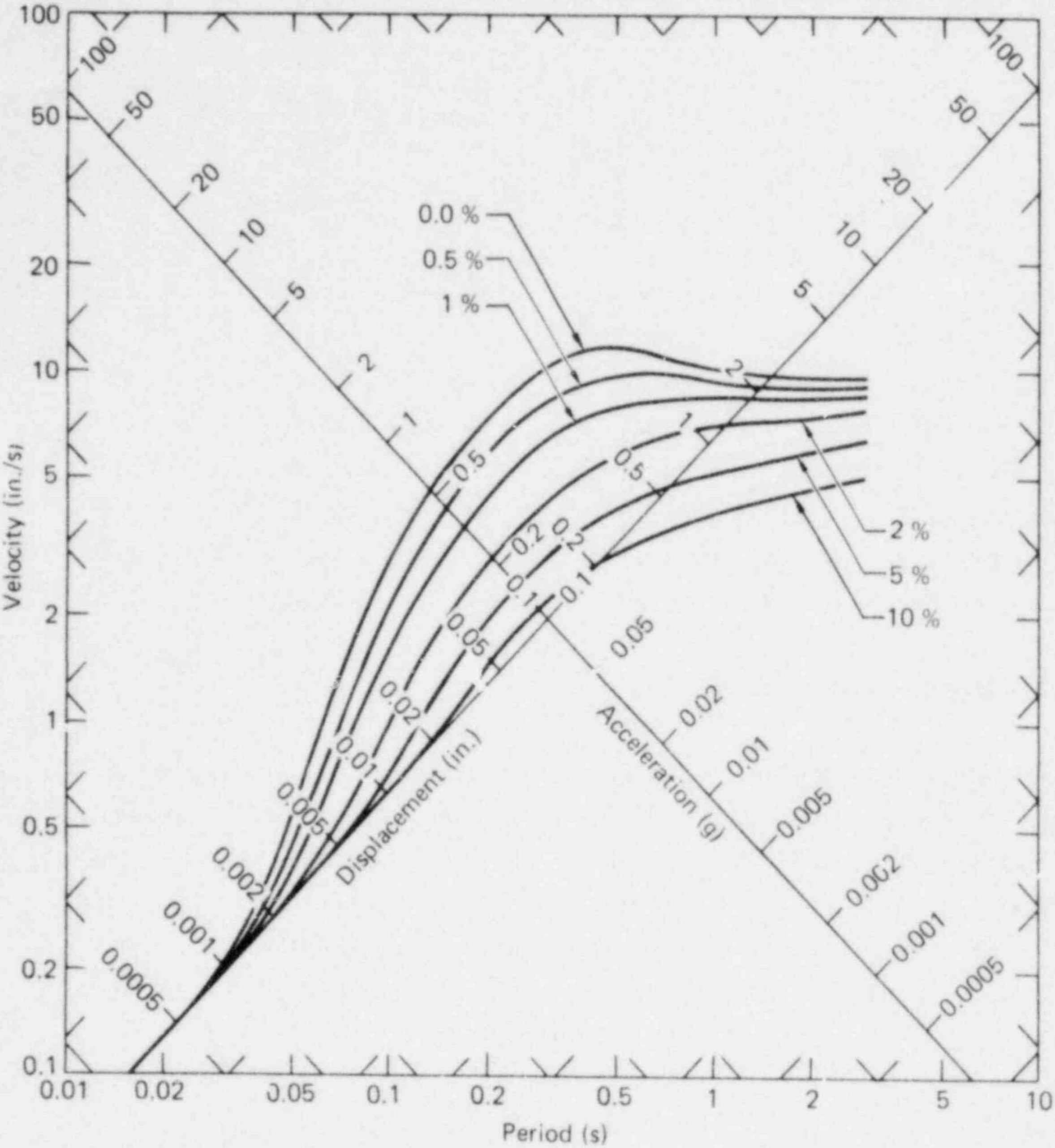


FIG. 4-1. Seismic response spectra used for the OBE analyses of Palisades, for various levels of damping (from FSAR, Appendix A).

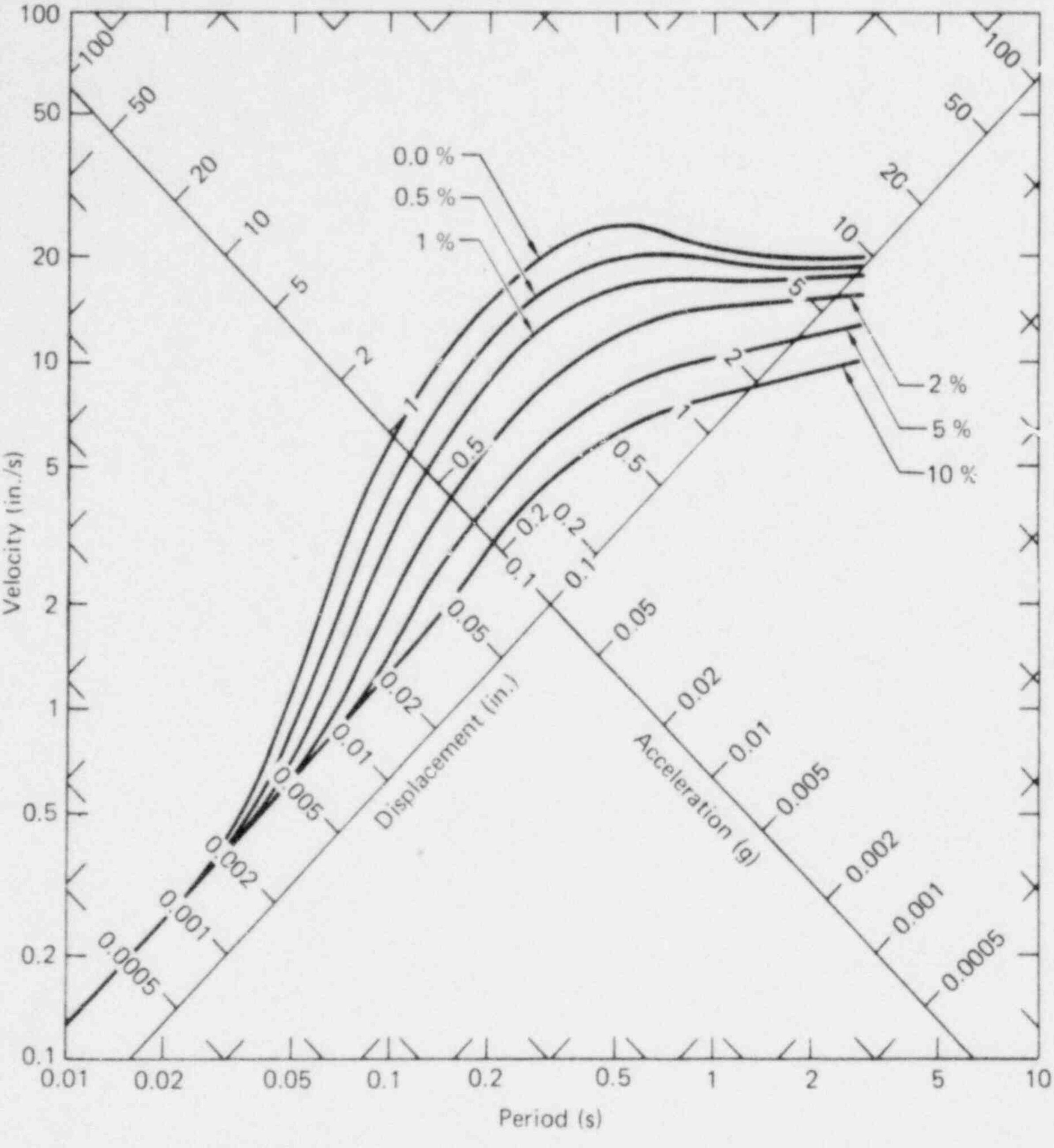


FIG. 4-2. Seismic response spectra used for the SSE analyses of Palisades, for various levels of damping (from FSAR, Appendix A).

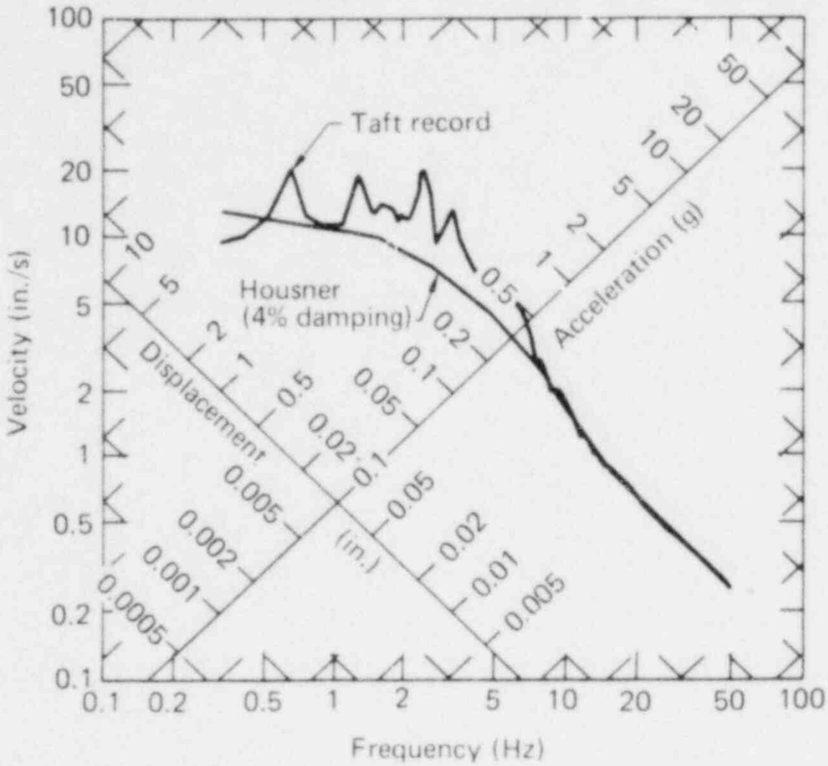


FIG. 4-3 Ground response spectrum from the Taft 1952 earthquake, compared to the SSE response spectrum for 4% damping (from Ref. 10).

4.3 SEISMIC ANALYSIS

Table 4-1 is a list of the Class 1 systems at Palisades, showing the seismic analysis method by which each was evaluated. In the brief descriptions of these analysis methods that follow, the emphasis is on the general characteristics of each method. Later, in the more detailed descriptions of the individual analyses, note will be taken of applicable details.

TABLE 4-1. Seismic analysis methods used to evaluate Class 1 systems.

Item	Type of analysis	Model
Structures:		
Containment building and internal structure	Response spectrum	Lumped mass and beam
Auxiliary building	Response spectrum	Lumped mass and frame
Turbine building	Response spectrum	Lumped mass and beam
Electrical penetration enclosure	Response spectrum	Lumped mass and beam
Intake structure and auxiliary feed pumps enclosure	Equivalent static	Unknown
Piping and equipment:		
Flexible piping	Response spectrum	Lumped mass and beam
Rigid piping	Equivalent static	Lumped mass and beam
Reactor internals	Unknown	Unknown
Control rod drive mechanism	Unknown	Unknown
Spent-fuel storage racks	Response spectrum	Lumped mass and grid
Other major equipment	Equivalent static	Unknown
Other equipment	Several (see Sec. 4.7.5)	Unknown

4.3.1 Methods of Analysis

4.3.1.1 Dynamic Methods

Dynamic response-spectrum analyses, using the smoothed response spectra shown in Figs. 4-1 and 4-2, were performed on the containment building (with its internal structure), the auxiliary building, the turbine building, and the electrical penetration enclosure in the containment. Response-spectrum analyses based on floor spectra derived from the 1952 Taft earthquake record were performed on all flexible piping systems and several pieces of Class 1 equipment. The dynamic structural analyses and the analysis of the spent-fuel storage racks considered only horizontal motion. (For the structures, horizontal responses were then combined with responses to static vertical loads in calculating the stresses. In the analysis of the spent-fuel storage racks, the vertical and horizontal stresses were combined by the square-root-of-the-sum-of-the-squares [SRSS] method.) Each piping system was analyzed twice, once for each principal direction of horizontal excitation; in each analysis, vertical excitation was applied simultaneously. The stress at each point was taken as the maximum obtained from the two analyses. Information is unavailable for the dynamic analyses of Class 1 appendages. Proprietary Bechtel computer programs were used for most of the dynamic analyses. The spent-fuel storage racks were analyzed using the STARDYNE code.

Floor response spectra were generated from time-history analyses. The input was the record of the 1952 Taft earthquake. The floor spectra were smoothed using straight-line segments, so that the peaks were broadened at the natural frequencies of the corresponding structures. A typical floor spectrum is shown in Fig. 4-4. Such floor spectra were used either as the basis for

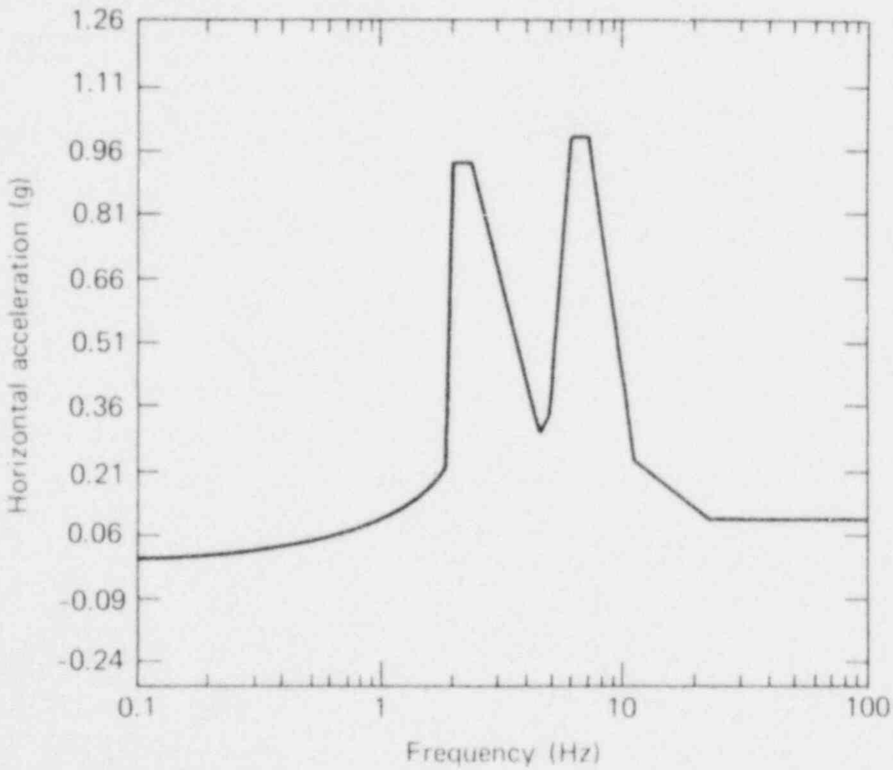


FIG. 4-4. Floor response spectrum (OBE, 0.5% damping) for the 646-ft level of the containment building (from Ref. 11).

dynamic analyses or as the source for accelerations to be used in static analyses. For piping or equipment between two or more floor levels, the floor spectrum for the level immediately above the center of mass was used.

4.3.1.2 Equivalent-Static Method

The equivalent-static 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 Palisades, the coefficients used for rigid piping and for equipment were at least equal to the peak floor accelerations. In lieu of a dynamic analysis, some Class 1 piping was designed for a horizontal static load equivalent to the peak of the horizontal spectrum, combined with two-thirds of this peak value applied vertically.

4.3.2 Damping

Damping values specified for the design of Palisades are given in Table 4-2, along with damping values for various Class 1 items.[6,12] The values account for both structural damping and, where applicable, soil damping.

4.4 STRESS CRITERIA

Stresses resulting from the 0.10-g OBE excitation, in combination with stresses imposed by nonseismic loads, were held to code-allowable levels. In addition, it was required that yield stresses not be exceeded during a 0.20-g SSE. The load combinations used in the design, compiled from the FSAR and Ref. 12, are listed in Table 4-3. (Revised stress criteria recently submitted by the licensee are currently being reviewed by the NRC.)

The loading combinations designated as "design loads" for the containment building were the basis of a "working stress" design. Stress criteria were generally those of ACI Code 318-63,[13] with the exceptions outlined in Appendix B.1 of the FSAR. Other loading combinations for the containment building were designated as "yield loads." These combinations include factored loads, which refle[illegible] evaluations as to the loads most

TABLE 4-2. Damping values used in the design of Palisades.

	% of critical damping	
	For OBE	For SSE
Structural types:		
Welded steel-plate assemblies	1.0	1.0
Welded steel-frame structures	2.0	2.0
Bolted steel-frame structures	2.0	2.0
Concrete equipment supports on another structure	2.0	2.0
Reinforced concrete structures on soil	5.0	7.5
Prestressed concrete structure on soil	--[a]	--[a]
Steel piping	0.5	0.5
Class 1 items:		
Containment building and internal structure	--[a]	--[a]
Auxiliary building	--[b]	--[b]
Turbine building	Unk	Unk
Electrical penetration enclosure	Unk	Unk
Flexible piping	0.5	0.5
Spent-fuel storage racks	4.0	7.5
Class 1 appendages	Unk	Unk

[a]The final analysis of the containment building and internal structure used the following values for damping: structural damping--2% (OBE) and 5% (SSE); soil damping--5% (OBE) and 10% (SSE); and composite modal damping--5% (OBE, modes 1 and 2), 2% (OBE, modes 3 and 4), and 7.5% (SSE, all modes).

[b]The OBE analysis of the auxiliary building used 5% damping for modes associated with reinforced concrete structures and 0.5% damping for modes associated with steel structures. SSE responses were inferred from the OBE results.

TABLE 4-3. Summary of original load combinations and allowable stresses.

Load combinations[a]	Design criterion
Containment building	
a. $D + F + 1.15P$	Design loads (see text). Thermal loads are due to the temperature gradient through the wall and to expansion of the liner.
b. $D + F + P + T_A$	
c. $1.05D + F + 1.5P + T_A$	Yield loads (see text). Thermal loads as above.
d. $1.05D + F + 1.25P + 1.25E + T_A$	
e. $D + F + P + E' + T_A$	
Containment internal structure	
a. $D + L + E$	Allowable stresses specified in ACI Building Code, ACI 318-63 (Ref. 13), or AISC Manual of Steel Construction, 6th ed.
b. $D + L + T_A$	Stresses to be less than 133% of (a) above.
c. $D + P + R + E'$	Local yielding allowed, but not to interfere with safe shutdown.

continued

[a]Abbreviations are explained at the end of the table.

TABLE 4-3 continued.

Load combinations	Design criterion
	Other Class I structures
a. 1.25D + R + 1.25E	Stresses limited to yield strengths of the effective load-carrying structural materials, reduced by an appropriate yield capacity reduction factor. Yield strength for steel was taken from ASTM specifications. Concrete structures were designed for ductile behavior whenever possible.
b. 1.25D + 1.25H + 1.25E	Same as above, except if the dead load decreases the total stress, 0.9D was used in place of 1.25D.
c. 1.25D + 1.25H + 1.25W	Same as (b) above.
d. D + R + E'	Same as (a) above.
e. D + H + E'	Same as (a) above.
	Reactor internals
a. D + E	Stress criteria of Sec. III, ASME Boiler and Pressure Vessel Code, Article 4.
b. D + E'	Small amount of yielding permitted.
c. D + R + E'	Permanent deformation is permitted.

continued

TABLE 4-3 continued.

Load combinations	Design criterion
	Spent-fuel racks
a. D + L	Allowable stress.
b. D + E	Allowable stress.
c. D + E + T_O	150% of allowable stress.
d. D + E' + T_O	160% of allowable stress.
e. D + E + T_A	160% of allowable stress.
f. D + E' + T_A	170% of allowable stress.
g. D + T_A + stuck fuel	160% of allowable stress.
h. D + T_A + fuel assy drop	160% of allowable stress.
	Other Class I systems and equipment
a. MOL + PTT + E	Applicable code-allowable stress.
b. MOL + MTT + E	Minimum yield stress at appropriate temperature.
c. MOL + MTT + E'	110% of minimum yield stress at appropriate temperature.

continued

TABLE 4-3 continued.

Abbreviations:

D = Dead loads of structures and equipment, plus any other permanent loading that contributes to stress, including hydrostatic or soil loads. In addition, a portion of the live load was added when it included items such as piping, cables, and trays suspended from floors. An allowance was made for future additions to the permanent load.

E = Design earthquake load (equivalent to OBE).

E' = Maximum earthquake load (equivalent to SSE).

F = Effective prestress loads.

H = Force on the structure due to the thermal expansion of pipes under operating conditions.

L = Live loads.

MOL = Maximum normal operating load, including design pressure, design temperature, and piping and support reactions.

MTT = Maximum thermal transients during emergencies such as full-power reactor trip, turbine generator trip, loss of auxiliary power, and the design accident.

P = Design accident pressure loads.

PTT = Normal thermal transients such as those associated with start-up, shutdown, and load swings.

R = Force or pressure on the structure due to the rupture of any one pipe.

T_O = Operating thermal loads.

T_A = Thermal loads due to the design accident.

W = Wind load.

subject to variation and most critical to safety. The stress criteria for the yield loads were based on the yield and ultimate stress values of ACI Code 318-63 and the appropriate ASTM specifications. These allowable stress limits were reduced by appropriate yield capacity reduction factors (ϕ factors) to account for "small adverse variations in material strengths, workmanship, dimensions, control, and degree of supervision."[13]

4.5 SEISMIC ANALYSIS OF STRUCTURES

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

4.5.1 Containment Building and Internal Structures (FSAR, Secs. 5.1.2, 5.1.3, and 5.1.9; FSAR, Amendment 14, answer to question 5.10; FSAR, Amendment 15, answers to questions 5.8, 5.20, and 5.21; Ref. 12, answer to question 2.A)

The reactor containment building, which houses the NSSS, is a vertical, cylindrical, reinforced concrete structure (Fig. 4-5). Its inside diameter is 116 ft, and its inside height is 189 ft. Containment walls are 3.5 ft thick, the dome is 3 ft thick, and the base slab thickness varies between 8 and 13 ft. The containment building was the first in the United States to be post-tensioned with fully prestressed walls and dome. Each of the 845 tendons, stressed to about 800,000 lb, comprises ninety 1/4-in.-diameter, high-tensile steel wires. The building was designed to withstand the internal pressure (55 psig at 283°F) that would result if the largest primary pipe ruptures. A 1/4-in. carbon-steel liner plate on the inside surface of the containment concrete ensures leak tightness.

The primary structures inside the containment building are the supports for the reactor vessel and steam generators, and the walls and slabs surrounding the steam generators and primary loop piping.

Three separate seismic analyses were performed on the containment building and internal structure, each analysis reflecting the state of the art and the design requirements at that stage in the engineering process.

In early 1967, during the preliminary design stage, the containment shell

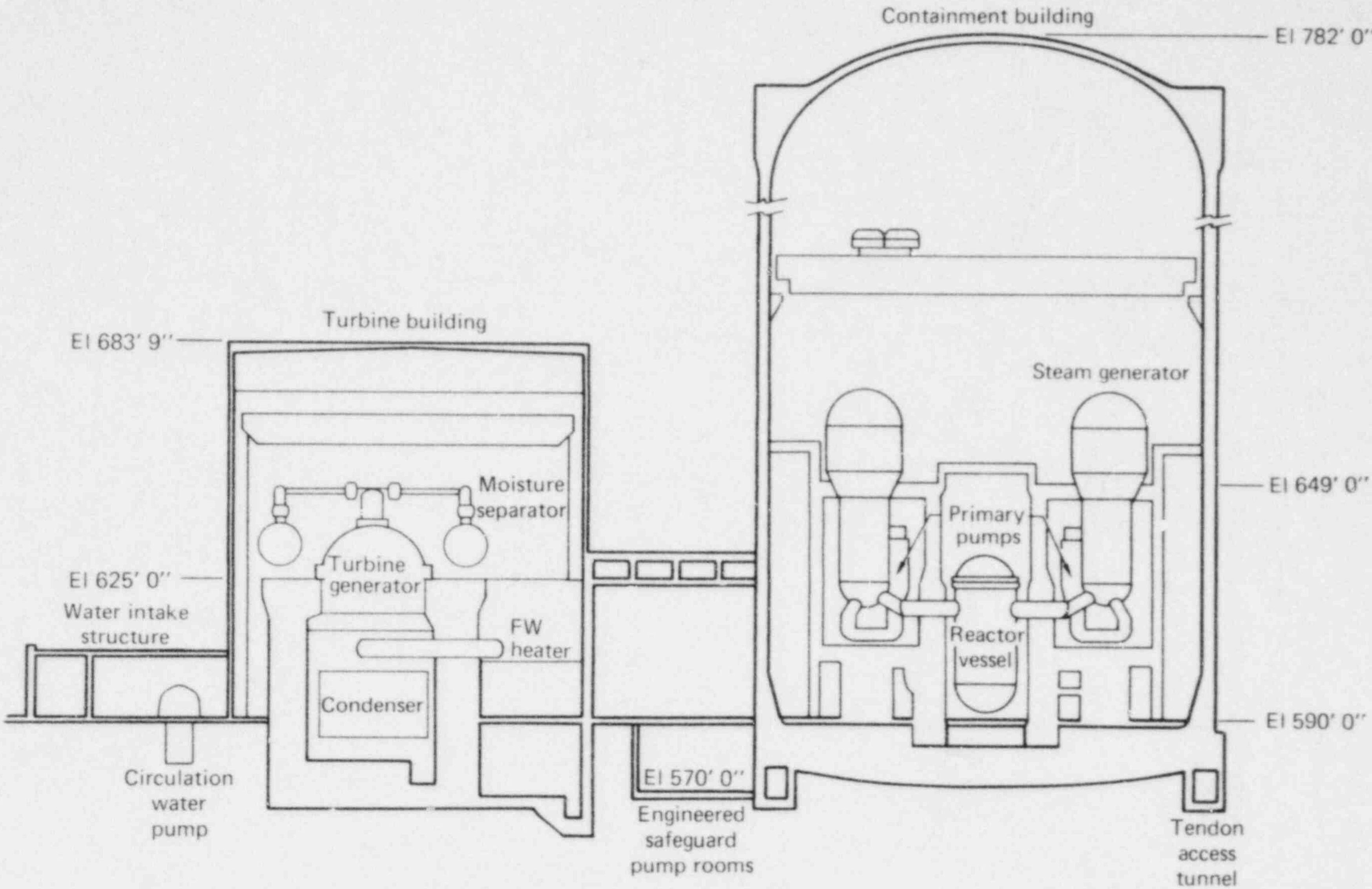


FIG. 4-5. Transverse section of the containment and turbine buildings.

was modeled separately from the internal structure as a single-stick, lumped-mass, fixed-base model (Fig. 4-6). The response-spectrum technique was used to generate modal responses that were added absolutely to get the structural responses. The forces so generated were used in the structural design of Palisades. Figure 4-6 shows the response envelopes for this OBE analysis, which used 2% structural damping.

Later, soil-structure interaction (SSI) was included in a model of the containment (Fig. 4-7). This modified model incorporated translational soil springs and offset vertical springs to account for rocking stiffness. The mass of the internal structure was lumped at the base of the containment. This model was analyzed by the response-spectrum technique, and the modal responses were combined by the SRSS method. Damping was 4% for the OBE analysis and 7.5% for the SSE analysis for all modes. The results shown in Fig. 4-8 were reported in Sec. 5.1.3 of the FSAR.

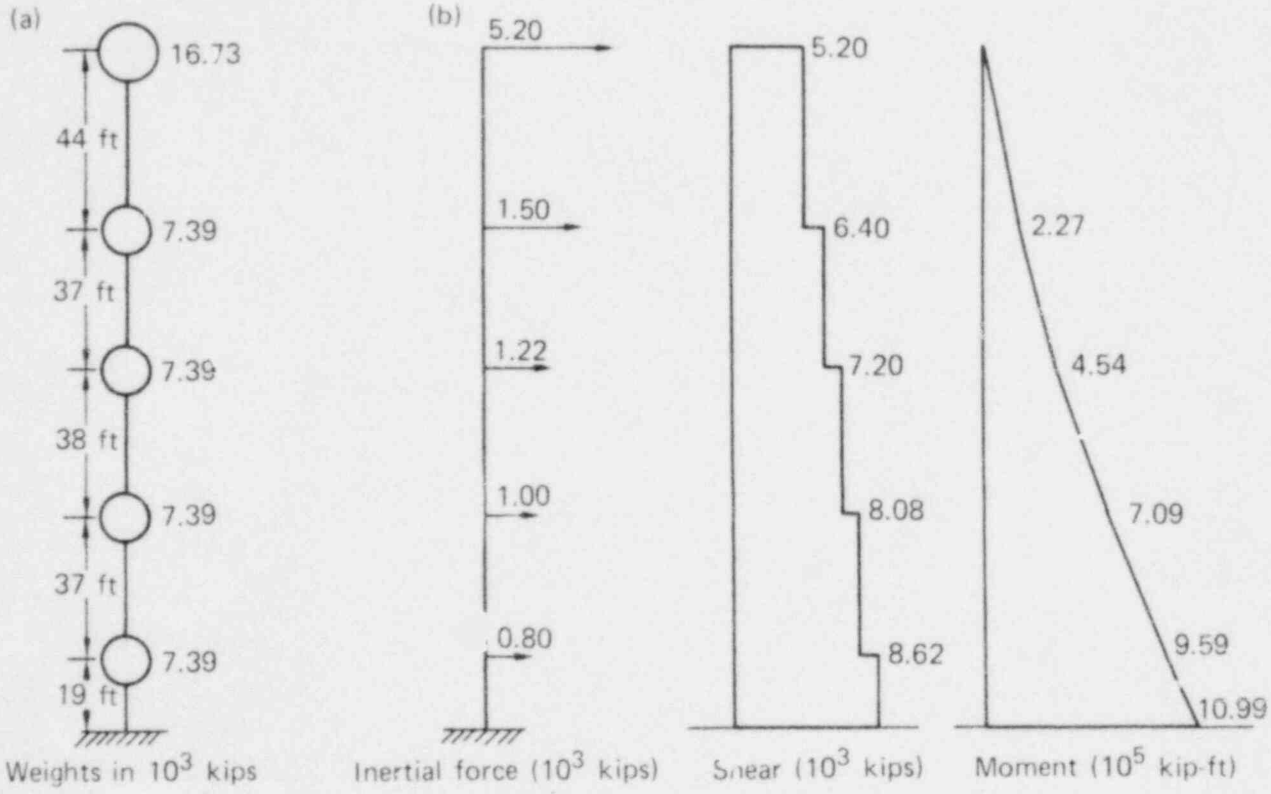


FIG. 4-6. (a) Preliminary single-stick model of the containment shell with node weights; (b) inertial forces, shear envelope, and moment envelope from an OBE analysis of the preliminary model (from Ref. 12)

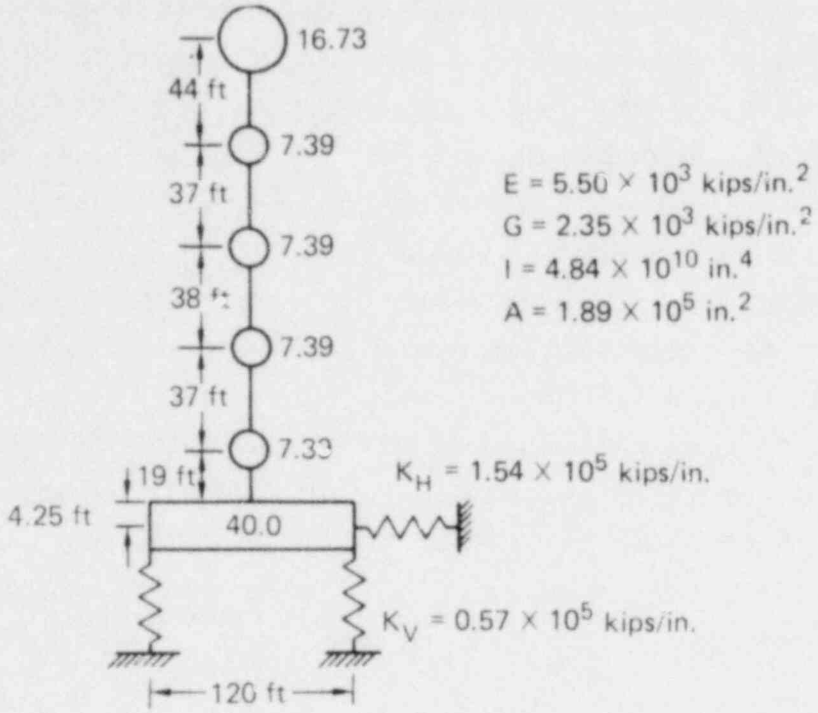


FIG. 4-7. Mathematical model used in the second analysis of the containment building (from FSAR, Sec. 5.1.3). The mass of the internal structures is lumped at the base of the model. Weights are given in 10^3 kips.

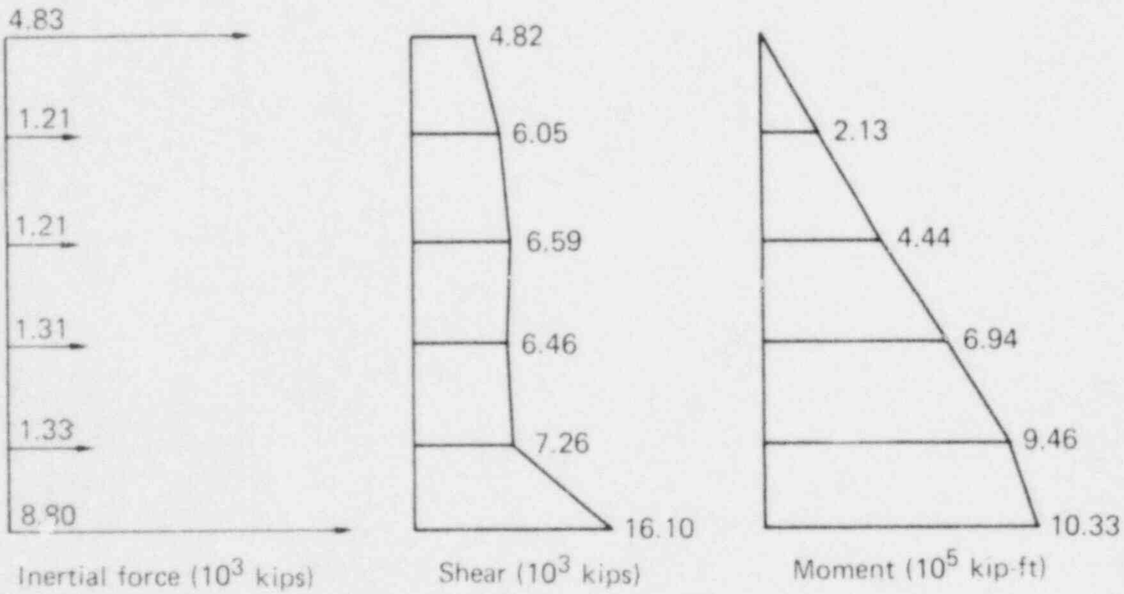


FIG. 4-8. Response envelopes from a response-spectrum analysis using the model of Fig. 4-7 and an OBE of 0.1 g (from FSAR Sec. 5.1.3).

In June 1969, the third and final seismic analysis was completed. Both SSI and coupling effects between the containment shell and the internal structure were included in a two-stick model (Fig. 4-9 and Table 4-4), which was analyzed by the response-spectrum technique. Responses of the first four modes (illustrated in Fig. 4-10) to OBE excitation were combined by the SRSS method to produce the envelopes shown in Fig. 4-11. Other details of the analysis are presented in Amendment 15 to the FSAR and in the response to question 2.A in Ref. 12. Amendment 15 states:

> This combined system produced lower forces on the containment building than were produced by the single-branch model of the containment used for design purposes. The design seismic forces on the containment building are therefore conservative.

This third model was also used to generate the in-structure response spectra for equipment qualification, as discussed in Sec. 4.3.1.1.

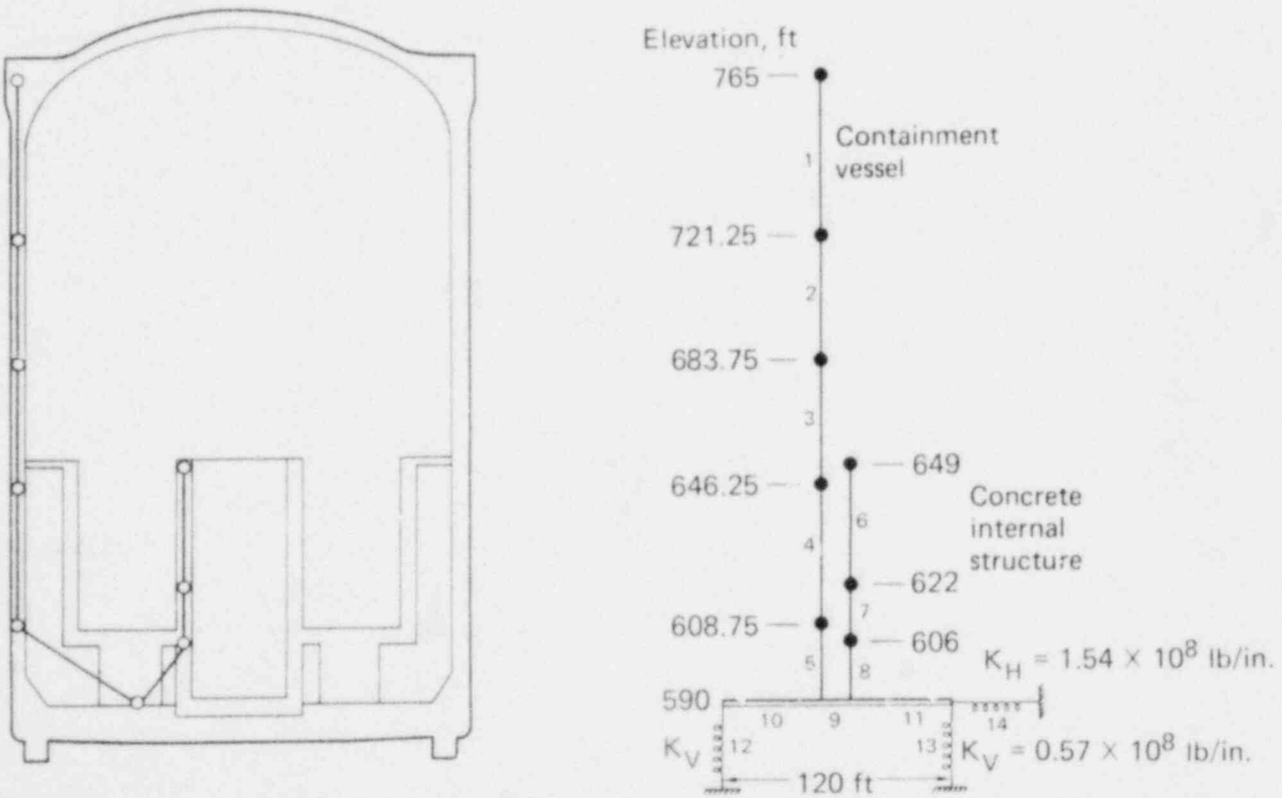


FIG. 4-.. Two-stick, lumped-mass model of the containment building and internal structure (from FSAR, Amendment 15).

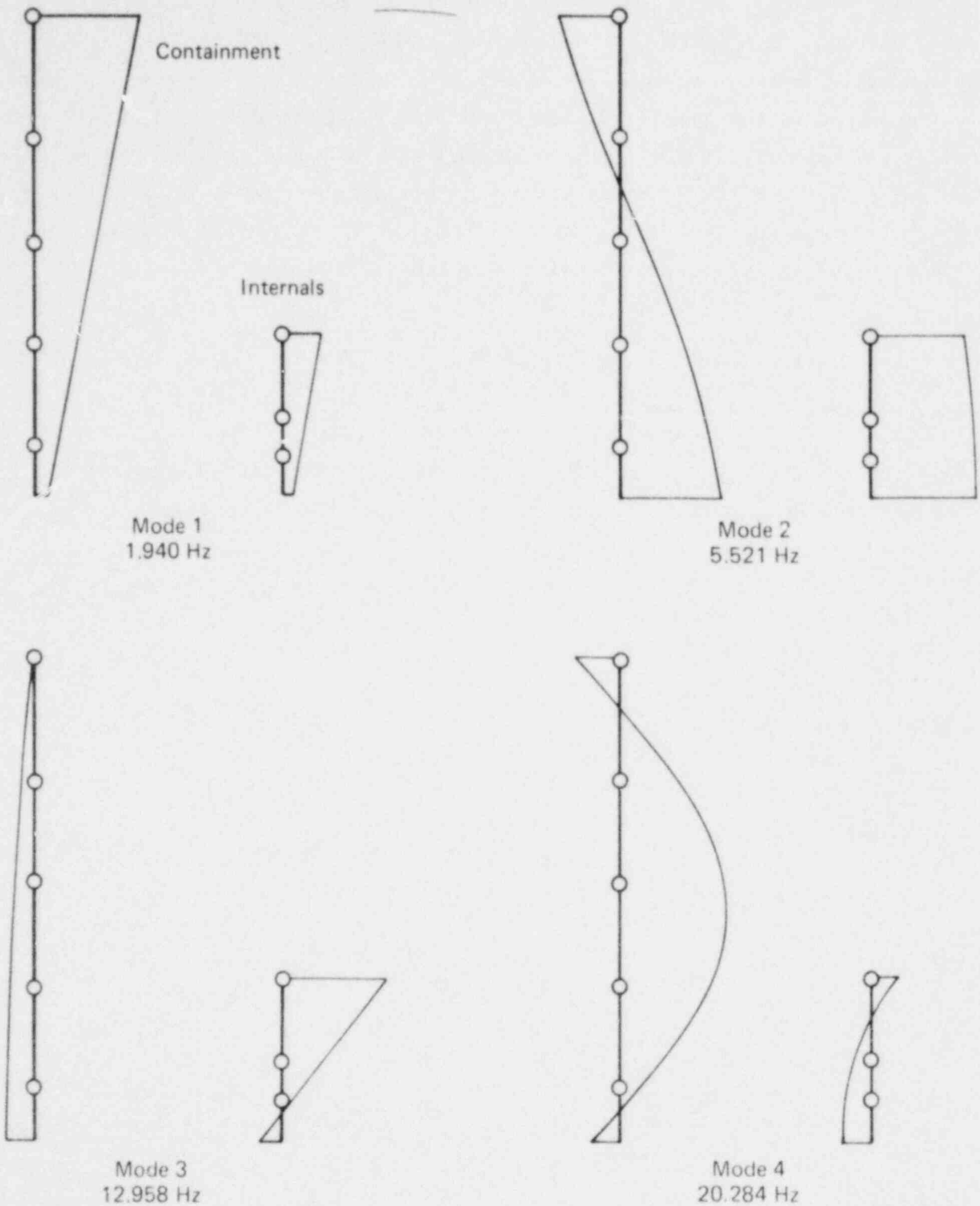


FIG. 4-10. Principal mode shapes for the two-stick containment building model shown in Fig. 4-9 (from FSAR, Amendment 15).

TABLE 4-4. Member properties of the two-stick containment building model shown in Fig. 4-9 (from FSAR, Amendment 15).[a]

Member	A_x, ft^2	A_y, ft^2	I_z, ft^4	E, 10^3 kip/ft^2	G, 10^3 kip/ft^2
1 - 5	1,310	874	2,340,000	792	338
6	1,429	814	833,513	576	246
7A	1,451.2	865	465,209	576	246
7B	1,188.7	811	384,312	576	246
8	1,376	865	336,503	576	246
9 - 11	278,000	185,000	9,999,999	792	338
12, 13	10.0	6.67	100.0	683	--
14	10.0	6.67	100.0	1,840	--

[a]Abbreviations: A, shear area; I_z, moment of inertia; E, modulus of elasticity; G, shear modulus.

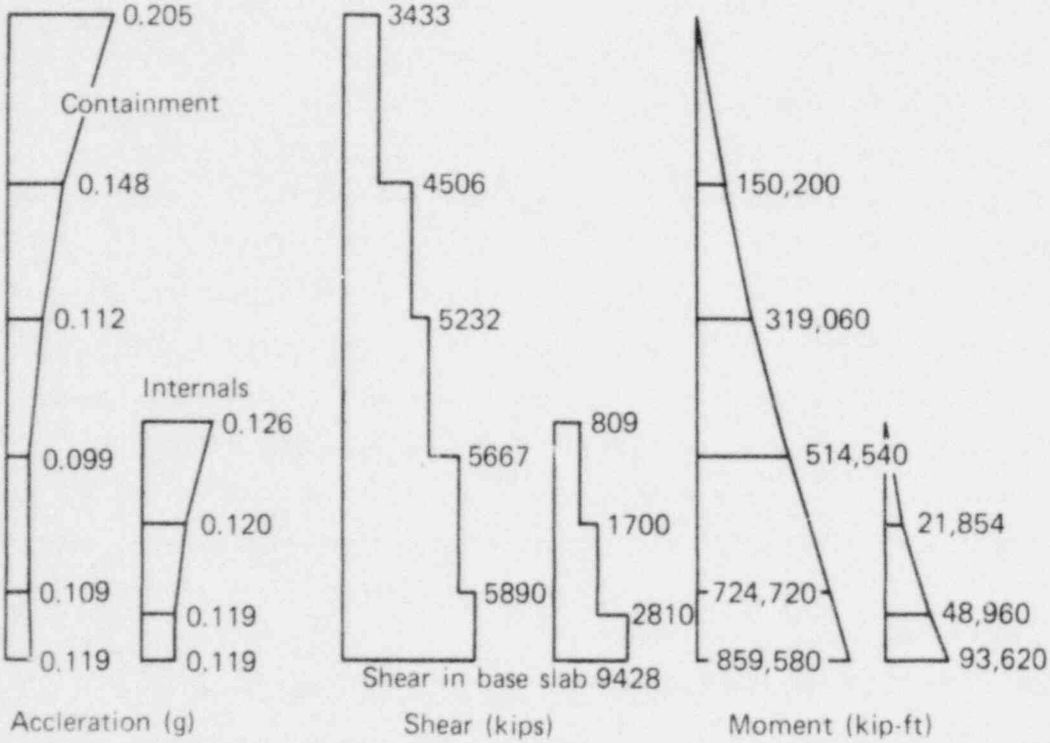


FIG. 4-11. Response envelopes from an OBE response-spectrum analysis using the model in Fig. 4-9 (from Ref. 12).

The containment was analyzed for axisymmetric and nonaxisymmetric loads. The axisymmetric loads--dead loads, pressure and thermal loads associated with an accident, and the prestress loads--were calculated using the FINEL code and an axisymmetric, finite element model of the containment that neglected buttresses, penetrations, brackets, and anchors.[12] Loads arising from these nonaxisymmetric features, seismic and wind loads, and concentrated loads were all considered in the nonaxisymmetric analysis described in Sec. 5.1.3.2 of the FSAR. Seismic loads were larger than wind loads in all cases. The resultant stresses are documented in Table 5-1 of the FSAR.

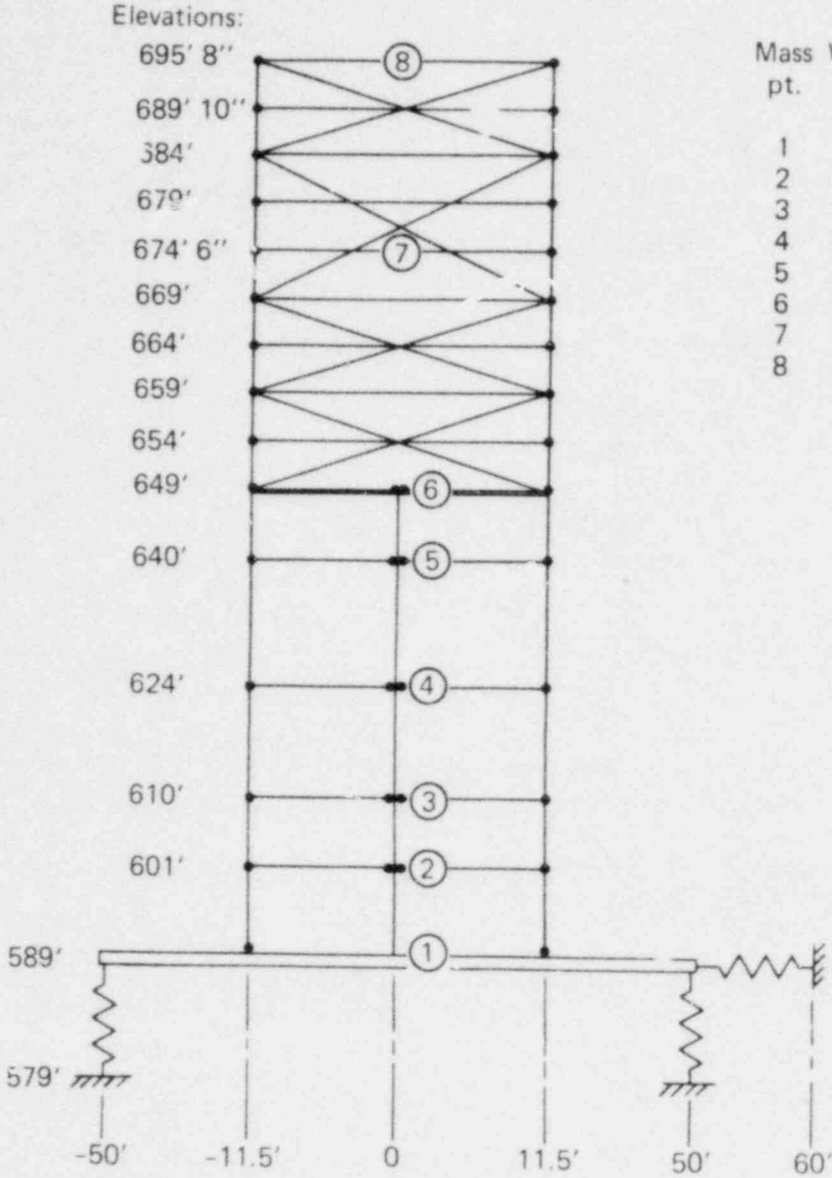
4.5.2 Auxiliary Building (FSAR, Sec. 5.2; FSAR, Amendment 15, answers to questions 5.8 and 5.16; Ref. 11, answer to question A.1; Ref. 12, answer to question 2.C)

The auxiliary building comprises a reinforced concrete system of interconnecting floors and walls to an elevation of about 649 ft, above which rise structural steel framing and attached nonstructural walls. The building is constructed on a mat foundation. Two two-dimensional models of lumped masses and beam elements were developed for the auxiliary building--one each for the north-south and east-west directions (Figs. 4-12 and 4-13). At the base of the structure, as for the containment building, offset vertical springs and translational springs simulate SSI. Detailed model properties are tabulated in Ref. 11.

Stiffness coefficients at the mass locations, which were determined from the model properties, were input to a Bechtel code that calculated system frequencies and mode shapes. The mode shapes and materials were evaluated to determine OBE modal damping values--0.5% for modes associated with steel portions of the structure and about 5% for those associated with reinforced concrete portions. The latter value accounts for the effect of soil damping.

An OBE analysis of both models, using the response-spectrum technique, resulted in the maximum structural responses shown in Fig. 4-14. Modal responses were combined by the SRSS method. SSE responses were calculated by doubling the OBE values--a simple method that produced conservative values because of the lower OBE damping.

Torsional effects, which arise from the asymmetry of the building, were considered in the design by distributing design horizontal loadings (obtained



Mass pt.	Weight (kips)
1	7,430
2	3,260
3	10,788
4	13,639
5	6,677
6	2,307
7	180
8	120

FIG. 4-12. Mathematical model used in the analysis of the north-south seismic response of the auxiliary building (from FSAR, Amendment 15).

from the decoupled seismic system analyses) in accordance with the actual shear-wall rigidity calculation.

No dynamic coupling was considered between the auxiliary and turbine buildings because the connections between them at the mezzanine and operating floors were designed with slotted bolt holes to provide a 3-in. gap.

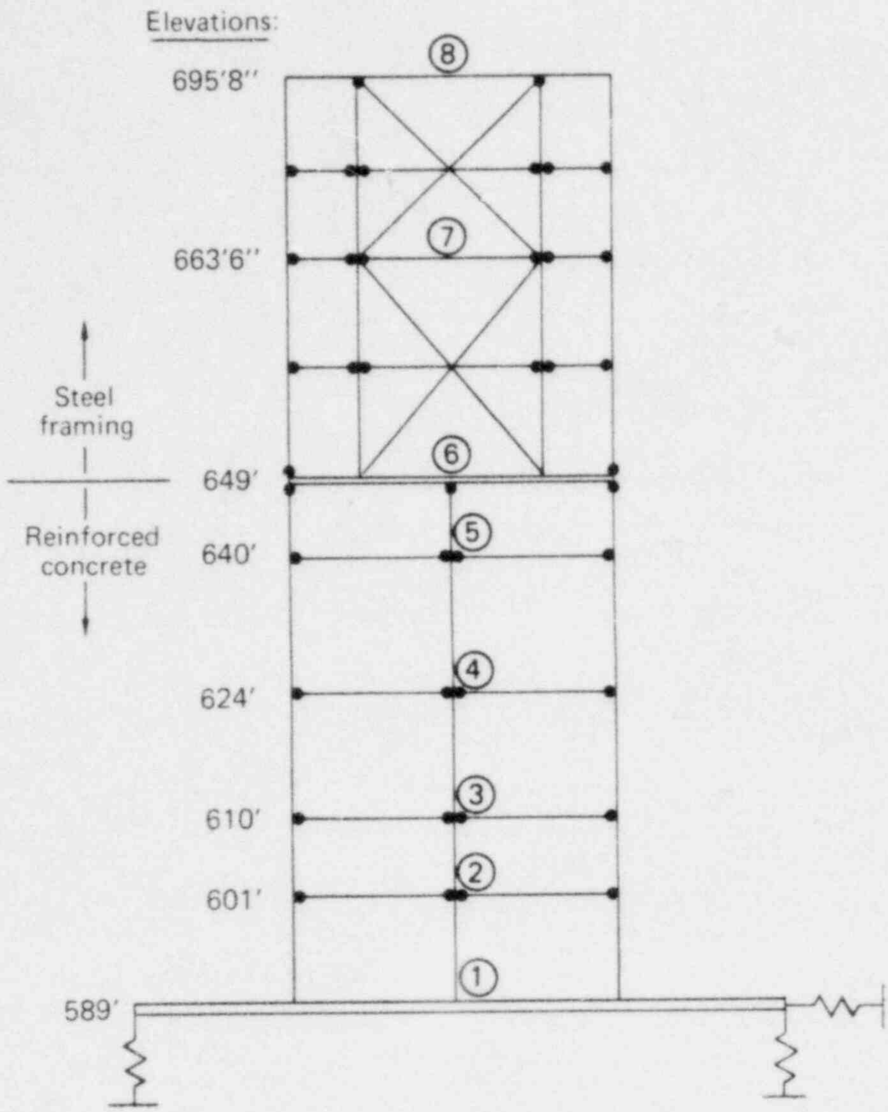


FIG. 4-13. Mathematical model used in the analysis of the east-west seismic response of the auxiliary building (from FSAR, Amendment 15).

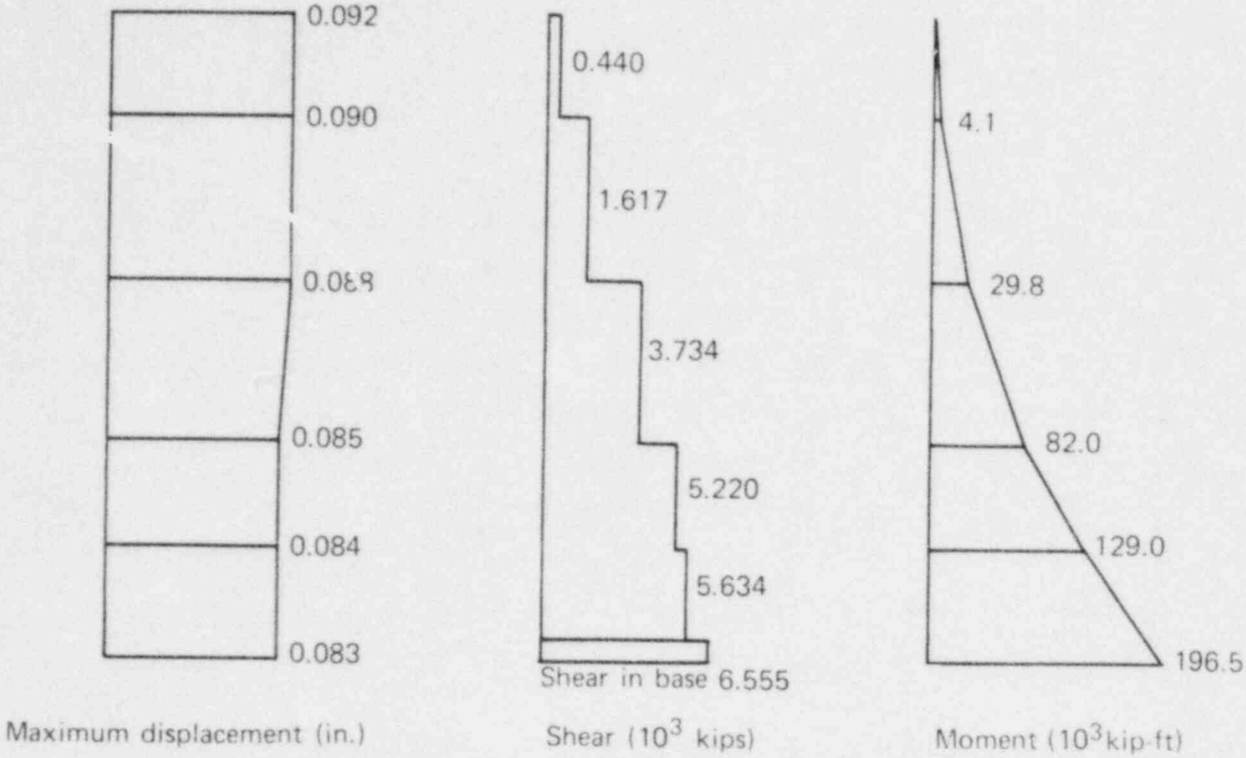


FIG. 4-14. Maximum displacement, moment, and shear envelopes for the auxiliary building (from FSAR, Amendment 15). Each is a composite of the north-south and east-west seismic analyses, showing the maximum value calculated at nodes 1 through 6.

4.5.3 Turbine Building (FSAR, Sec. 5.3.1; FSAR, Amendment 15, answers to questions 5.8, 5.16, and 5.21; FSAR, Amendment 17, answer to question 8.0; Ref. 11, answer to question A.1)

The turbine building (Fig. 4-5) is a steel-frame building with insulated siding. Within the building, reinforced concrete enclosures house Class 1 equipment. The building was analyzed for SSE excitation (A_{max} = 0.20 g) using the response-spectrum technique. The building was modeled only for the less-rigid east-west direction; the model was a system of lumped masses and stiffness coefficients. Three cases were analyzed:

- Case 1. The turbine building frame was considered to be restrained solely by its tie to the auxiliary building--by encasement of the secondary columns of the frame in the auxiliary building wall. It was concluded that this tie will not cause failure of the auxiliary building wall or the roof over the turbine building auxiliary bay.
- Case 2. The turbine building was considered to be a rigid frame, supported at the ground-floor level and unrestrained at the operating-floor level. Maximum frame deflection at the 625-ft elevation was calculated to be 3.4 in., enough to close the 0.75-in. gap between the turbine generator pedestal and the operating floor and to cause the pedestal to act as a restraint. This closure necessitated analysis of the third case.
- Case 3. The turbine generator pedestal was treated as a restraint to the building frame at the operating-floor level. It was concluded that the resulting lateral force would not affect the pedestal.

In all three cases, the crane was assumed to be unloaded and located at any bay of the turbine building. Mode shapes from the three analyses indicate that the crane support columns move in the same direction and have a maximum deflection of 2.75 in. at the roof. Based on the uniform movement of the columns at the crane rail and the fact that column stresses are below those allowed, it was concluded that the building will not collapse and that the crane bridge and trolley will remain in place.

In addition, steps were taken to ensure that the 2.75-in. deflection would not close the gap between the turbine and auxiliary buildings, inducing additional forces that would have to be borne by the concrete shear walls of the auxiliary building. Section 5.3.1 of the FSAR states:

> The results of the turbine building dynamic analysis for the 0.2-g earthquake showed that the auxiliary building floor slab would be overstressed due to the direct connection of the turbine building girders to the auxiliary building wall columns and the large openings in the auxiliary building floor slab. The overstress condition has been eliminated by cutting the associated turbine building girders, providing vertical supports with sliding surfaces at the girder cut points, and providing a 3" gap between the girders and the auxiliary building wall columns. The 3" gap is greater than the seismic displacement of the turbine building at the 625-ft elevation.

The earthquake-induced deflection would also close the gap between the turbine building and the intake structure on the west side of the turbine building. However, at the level of the intake structure roof, the predicted deflection is less than 2.75 in. The intake structure walls can absorb the induced stresses without exceeding 85% of the yield stress.

4.6 SEISMIC ANALYSIS OF PIPING (FSAR, Secs. 4.3.6, 6.1.2, 9.1 through 9.4, 9.7, and 10.1; FSAR, Amendment 15, answers to questions 5.4 and 5.8; Ref. 12, answers to questions 2.F and 2.G)

The following piping systems were considered in the original seismic analysis:

- Primary coolant piping.
- Safety injection system piping.
- Steam and power conversion system piping. Only part of this system was designed to Class 1 requirements.
- Service water system piping. Only part is Class 1.
- Reactor primary shield cooling system piping.
- Component cooling system piping. The portion of the system outside containment is Class 1.
- Fuel pool cooling system piping.
- Auxiliary feedwater system piping. Only part is Class 1.

Large piping, having an inside diameter greater than 3 in., was idealized as a series of lumped masses separated by elastic members. The masses were located to represent the dynamic and elastic properties of the system. For example, all valves, supports, elbows, tees, and other connections were represented by lumped masses. Seismic responses were calculated using the response-spectrum method and 0.5% damping. The floor response spectra were developed as described in Sec. 4.3.1.1. Typically, all modes with frequencies less than 20 Hz (up to a maximum of 10 modes) were considered. Modal responses were combined using the SRSS technique.

Smaller Class 1 piping systems were analyzed as rigid systems (defined as having fundamental frequencies greater than 20 Hz). The rigidity of these systems was ensured by permitting no unsupported insulated-pipe spans greater than those indicated in Table 4-5. Restraints were also placed as near as possible to bends and concentrated loads such as valves. Stresses on rigid systems were then calculated on the basis of static loads corresponding to the peak acceleration of the appropriate floor.

All static and dynamic piping analyses were carried out twice, once for each of two orthogonal horizontal excitations. A simultaneous vertical excitation equal to two-thirds of the horizontal was applied in each case. Piping between two structures was also designed to withstand stresses imposed by the maximum differential movement between horizontal restraints. These stresses were combined with other seismic stresses by the SRSS method. Detailed stress results were reported for only two piping systems: the auxiliary feedwater system and the main steam line (steam and power conversion system). The maximum OBE and SSE stresses on each were between 71% and 99% of allowable values. Stresses on rigid piping were reported to be "relatively low."[12]

TABLE 4-5. Maximum permissible spans for rigid piping systems (from Ref. 12, answer to question 2.F).

Piping i.d., in.	Span
0.5	4'3"
0.75	5'
1	5'10"
1.5	7'2"
2	8'2"
2.5	9'
3	9'10"
4	11'3"

4.7 SEISMIC ANALYSIS OF EQUIPMENT

4.7.1 Control Rod Drive Mechanism (FSAR, Sec. 3.3.4 and Amendment 15, answer to question 5.23)

The control rod drive mechanisms are mounted on flanged nozzles atop the reactor vessel closure head. Horizontal interconnections provide lateral stability and restrict the deformation of the control rod shrouds. It was determined by experiment that a shroud deformation greater than 0.76 in. would make it impossible to insert the corresponding control rod into the core. A conservative allowable deformation limit of 0.51 in. was adopted.

Under extreme loading, the shrouds of the first row of control rods were found to deform more than 0.51 in. Safe shutdown would not be jeopardized, however, since all other shroud deformations were within the allowable limit.

4.7.2 Other Reactor Internals (FSAR, Sec. 3.3.4 and Amendment 15, answers to questions 5.8 and 5.12)

The reactor internals comprise the core support barrel, the upper guide structure, and the flow skirt. Among their functions is the transmission of control rod dynamic loads and other loads to the the reactor vessel flange. The method used for analyzing the internals was not clearly specified, but the input acceleration was given as the acceleration of the concrete mass at the reactor support. Vertical and horizontal accelerations were applied simultaneously.

In all cases, the stresses due to pipe rupture were much larger than those due to seismic loads. Under extreme loading (D + R + E'; see Table 4-3), yielding was found to occur in the first row of control rod shrouds. Other stresses were within allowable limits.

4.7.3 Other Major Class 1 System Equipment (FSAR, Secs. 4, 6.1 through 6.3, 9.1 through 9.4, 9.7, and 10; FSAR, Amendment 15, answers to questions 5.8, 5.34, and 7.7)

The following Class 1 systems were analyzed using an equivalent-static method:

- Primary coolant system.
- Safety injection system.
- Containment spray system.
- Service water system.
- Reactor primary shield cooling system.
- Component cooling system.
- Auxiliary feedwater system.

Components were modeled as single-degree-of-freedom masses, and natural frequencies were estimated from the deflections produced by known loads. Based on these natural frequencies, seismic loads were then taken from the appropriate floor response spectra. In some cases, loads greater than those indicated by the floor responses were used in the analysis. Vertical and horizontal seismic accelerations were applied simultaneously. (The derivation of the floor response spectra is discussed in Sec. 4.3.1.1.)

No detailed analysis results are available; however, all components were reported to have been conservatively designed to withstand seismic forces greater than those specified. In several cases, seismic loads were insignificant compared to loads imposed by other postulated accident conditions.

4.7.4 Other Class 1 Equipment

Other Class 1 equipment includes:

- Electrical cable conduits and trays (FSAR, Amendment 15, answer to question 5.6; Ref. 12, answer to question 2.K).
- Appendages to Class 1 systems, piping, and equipment (FSAR, Amendment 15, answer to question 5.17).
- Battery racks (Ref. 12, answer to question 2.K).

Detailed analyses of the cable conduits and trays are not available; however, it is expected that seismic activity would only crack the concrete encasements of underground wires. The flexibility of the conduits and wires, both above and below ground, ensures that electrical continuity would not be disturbed. (Since 1971, cable tray supports have been explicitly designed to resist horizontal seismic ground accelerations of 0.05 g.)

Appendages were analyzed independently of the Class 1 components to which they are attached; however, their masses were accounted for in the analyses of the major components. Appendages to Class 1 piping and equipment were analyzed statically; appendages to major Class 1 systems were analyzed by the response-spectrum method.

Battery racks were analyzed by the equivalent-static method. For the OBE analysis, horizontal and vertical seismic accelerations were 0.15 and 0.067 g, respectively; for the SSE analysis, twice these values were used.

Additional information on the original seismic design qualification of electrical equipment is summarized in Chapter 6 (Table 6-6).

CHAPTER 5: REASSESSMENT OF SELECTED STRUCTURES

5.1 INTRODUCTION

In this chapter, the seismic loads and responses on which the Palisades 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 containment and auxiliary buildings, field-erected tanks, and a typical buried pipe. In addition, seismic design loadings are compared to seismic input motion based on current design practice for locations throughout the containment and auxiliary buildings where equipment and piping are supported.

Since the completion of the Palisades 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.

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 Palisades 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.[15] 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, Palisades structures were designed for an OBE and an SSE with peak ground accelerations of 0.1 and 0.2 g, respectively. A simultaneous vertical component of earthquake motion equal to two-thirds of the horizontal component was considered in the plant design.

For this reevaluation, an SSE characterized by a 0.2-g peak horizontal ground acceleration also was employed. Although a probabilistic evaluation of the seismicity of the Palisades site may justify a lower value, it was considered unlikely that a level higher than 0 2 g would be required.

5.2.2 Ground Motion Characteristics

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.[16] 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 directly compare response spectra for equal damping ratios, it is more informative to consider also the damping used in the design of Palisades. Table 5-1 lists the damping ratios used for Palisades, together with those from R.G. 1.61[8] for the SSE and those recommended in NUREG/CR-0098[4] for structures at or below the yield point and at approximately one-half the yield point.

In general, the damping ratios used in the design of Palisades are lower than those now in use for SSE design analyses. The amount of soil damping allowed by the NRC to be used in conjunction with R.G. 1.61 structural or component damping values has been the subject of extensive discussion in the past. However, including any reasonable amount of soil damping results in higher damping for all systems than was used in design.

In the original analysis, 7.5% of critical damping was used for the SSE for all modes for both reinforced and prestressed concrete structures. In the reanalysis, different values of damping were used for different modes, because more sophisticated methods for computing geometric (radiation) damping in the soil for layered sites were available. Composite modal damping was used to account for the variations between soil and structural damping. The actual geometric and composite modal damping values used are discussed in Secs. 5.3 and 5.4.

In general, the composite modal damping values used in the reanalysis range from 10% to 20% of critical. To obtain a rough estimate of the expected variation in response, the original Housner 7.5%-damped ground response spectrum used for design is shown in Fig. 5-1, together with the 10% and 20% (interpolated) spectra from R.G. 1.60. For the median soil case, the fundamental frequency of the containment building is approximately 2 Hz. As is apparent from Fig. 5-1, the response resulting from use of the original 7.5%-damped spectrum is approximately the same as would be expected if the 20% R.G. 1.60 spectrum were used for all modes. However, for modes with closer to 10% damping, an increase in response of as much as 50% can occur in the frequency range of interest.

Apparently, no analysis of buried pipe was conducted during the original design. Also, the incomplete soils data available do not include any determination of soil strains expected during the SSE. For the SEP reevaluation of the buried pipe, the soil strain $(\varepsilon_a)_{max}$ was conservatively determined to be approximately 2×10^{-4} in./in., as obtained from the relationship

TABLE 5-1. Original and currently recommended damping ratios, expressed as percent of critical damping.

	Original DBE	Original SSE	R.G. 1.61 (SSE)	NUREG/CR-0098 (yield levels)	NUREG/CR-0098 (working stress)
Welded steel-plate assemblies	1	1	4	5 to 7	2 to 3
Welded steel-frame structures	2	2	4	5 to 7	2 to 3
Bolted steel-frame structures	2	2	7	10 to 15	5 to 7
Concrete equipment supports on another structure	2	2	7	7 to 10	3 to 5
Reinforced concrete structures on soil	5	7.5	7 (+ soil)	7 to 10 (+ soil)	2 to 3 (slight cracking) (+ soil)
Prestressed concrete containment structure on soil	2 to 5	7.5	5 (+ soil)	7 to 10 (w/ loss of prestress) (+ soil)	2 to 3 (w/o loss of prestress) (+ soil)
Steel piping	0.5	0.5	2 to 3	2 to 3	1 to 2

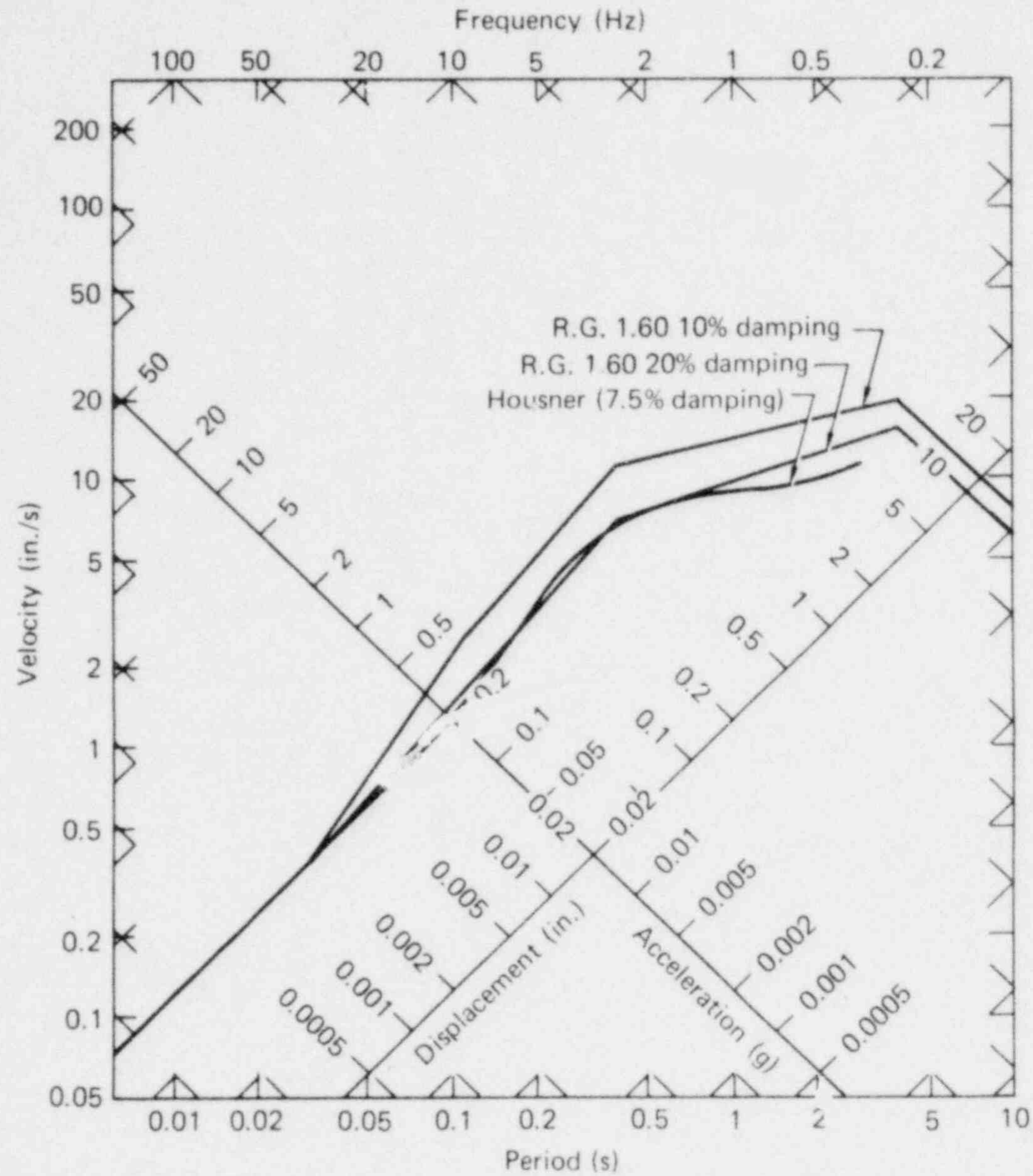


FIG. 5-1. Comparison of original 7.5% Housner design spectrum to R.G. 1.60 spectra for 10% and 20% of critical damping.

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

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. No geotechnic investigation was conducted to determine the apparent longitudinal horizontal-wave propagation speed. It was conservatively estimated at approximately 4000 ft/s, which accounts for the proximity to bedrock.

5.3 SEISMIC DESIGN METHODS

As previously discussed, seismic analysis methods have changed since the design of Palisades. Palisades was designed for both OBE and SSE conditions; for the concrete structures founded on soil, increased damping at the SSE response levels was considered. On the other hand, the conservative OBE damping values were used for both the OBE and SSE analyses of all other structures and components.

The analysis of the Palisades structures was conducted using two-dimensional models simulating the N-S and E-W responses. No vertical dynamic analysis was conducted. Planar 2-D models are adequate to compute the response of symmetric structures such as the containment building; however, such planar models cannot be used to calculate out-of-plane or torsional responses, which may be significant for such nonsymmetric structures as the auxiliary building. Current analytical techniques and computer models are considerably more sophisticated, and current licensing requirements would typically require additional load combinations resulting from other transients. Because these combinations are unlikely, however, our reevaluation has concentrated on the original design combinations, with primary attention devoted to the seismic margins. Several current assumptions and criteria are discussed below and compared with those used in the original design and analysis of Palisades.

5.3.1 Soil-Structure Interaction

The response of the Palisades structures was originally calculated on the basis of lumped-mass, two-dimensional models of the containment and auxiliary buildings. The structural models were supported on frequency-independent springs representing the soil flexibilities. These soil flexibilities, in turn, were calculated using elastic half-space theory, assuming a soil shear modulus G of 6.4×10^6 lb/ft^2 and a Poisson's ratio of 0.25. How these values were obtained is unclear.

The original design models were developed on the assumption of a rigid base slab. A horizontal spring simulating the horizontal soil stiffness was attached to the base slab, and the rocking stiffness was simulated by two equivalent vertical springs attached at opposite ends of the slab (Fig. 5-2). The vertical springs were used only to simulate the rocking stiffness; no vertical analysis was conducted. Also, no torsional response was computed for either the containment building or auxiliary building.

Much more sophisticated methods of analysis currently exist for the calculation of SSI effects, particularly for layered sites such as Palisades. However, it was decided that the soils information currently available for Palisades does not warrant a finite element or frequency-dependent compliance function analysis as part of the SEP. Rather, a frequency-independent analysis that included the effects of the layered site was selected as being consistent with the level of soils information available, as well as the sophistication of the structural models. Uncertainties were treated by considering a fairly broad range of soil properties.

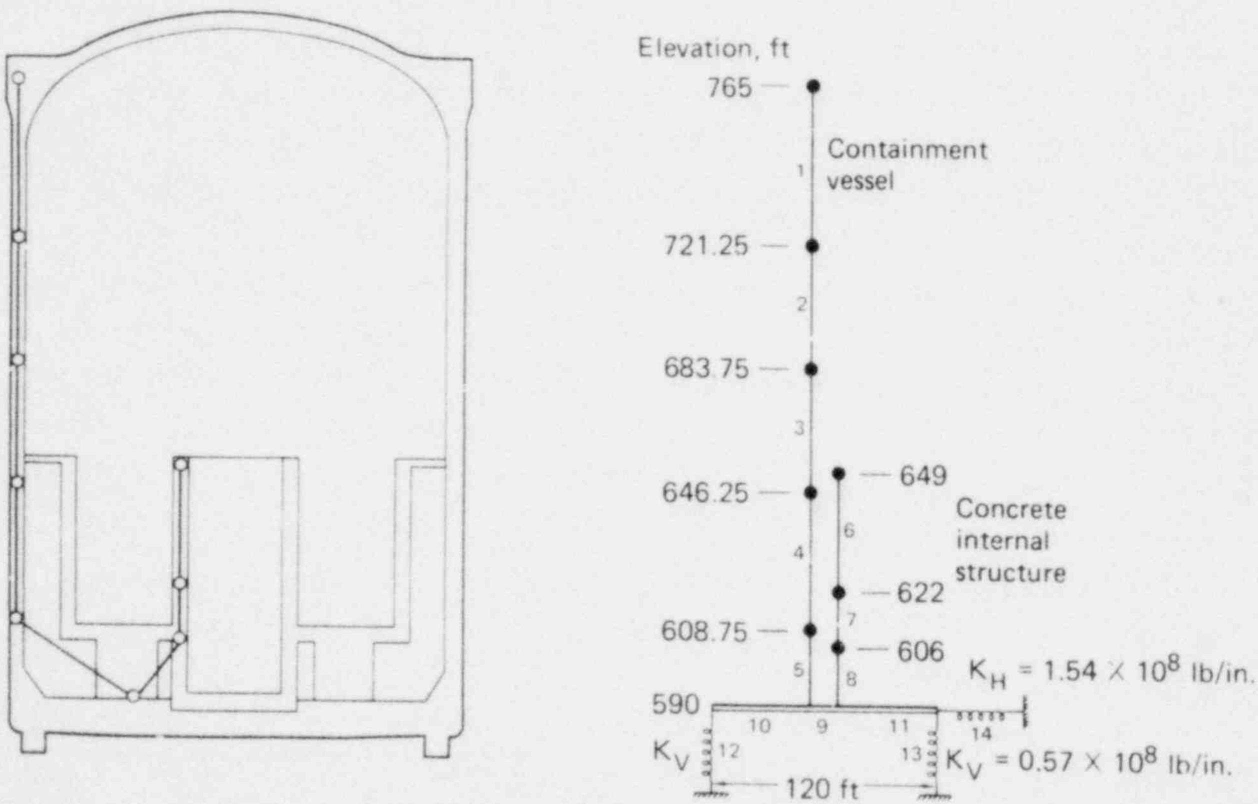


FIG. 5-2. Two-stick, lumped-mass model of the containment building and internal structure (from FSAR, Amendment 15).

Among the most useful of the current SSI analysis methods for layered sites are those of Kausel[17,18] and Luco.[19] Both have advantages and disadvantages when applied to the Palisades site. The original study by Kausel et al.[17] does not treat vertical response, while Luco's method does not directly include the effects of embedment and leads to a frequency-dependent solution for fairly wide parameter ranges. However, by utilizing both methods and obtaining a check of the parameters that can be computed independently by the two methods, a solution can be obtained that reflects the layered-site characteristics and is consistent with the information available for the soil.

In calculating the soil spring rates for the site, Poisson's ratio was chosen as 0.45, in accordance with recommendations in Refs. 20 and 21 for saturated sands. A median soil shear modulus of 4.38×10^6 lb/ft^2 was calculated using the approach outlined by Seed and Idriss[22] to account for the reduction in stiffness as a function of increased soil shear strain for the 0.2-g SSE. The radius of the containment building base slab is 60 ft, and the depth to bedrock is approximately 150 ft. An embedment depth of 17 ft was used in the analysis, which corresponds to the depth from grade to the bottom of the base slab, but neglects the sloping backfill on one side of the structure. In using Kausel's method, only half the embedment effects were used in computing the horizontal and rocking spring rates. This accounts for the fact that a cohesionless soil is unable to develop any significant tensile capacity. Cracks would, therefore, be expected to form between the soil and the vertical structure surfaces on the tensile side during an earthquake in the range of 0.2 g. Only the effects on the compression side would be felt. Similarly, no embedment effects were included for the vertical spring rate. After one or more strong lateral response cycles, cracks between the soil and the vertical surfaces could be expected over much of the total area for much of a vertical response cycle. Also, since the embedment is relatively small, the coupling that accounts for the interaction between the horizontal and rocking modes was neglected.

Table 5-2 presents a comparison of the frequency-independent containment building spring rates computed by the various techniques. Also shown for comparison are the horizontal and rocking springs used in the original analysis. The horizontal spring rate calculated using Kausel's method without embedment[17] is virtually identical with that computed using Luco's

TABLE 5-2. Comparison of median-soil spring rate constants and geometric damping values for Palisades containment building, calculated in different ways.[a] The subscripts h, ϕ, and v represent horizontal, rocking, and vertical modes, respectively. Values marked by asterisks were used in the reanalysis.

	Original design (no embedment)	Elastic half space (no embedment)	Kausel (no embedment)	Kausel (w/ embedment)	Luco (static) (no embedment)
Spring constants:					
k_h, lb/ft	1.85×10^9	1.3×10^9	1.63×10^9	$*1.92 \times 10^9$	1.63×10^9
k_ϕ, lb-ft/rad	4.92×10^{12}	4.58×10^{12}	4.89×10^{12}	$*6.59 \times 10^{12}$	4.77×10^{12}
k_v, lb/ft	--	1.97×10^9	2.89×10^9	3.25×10^9	$*2.87 \times 10^9$
Damping constants:[b]					
D_h, %	--	34	34	34	--
D_ϕ, %	--	2.5	1	1	0
D_v, %	--	60	--	6	3

[a]Values in the four rightmost columns are based, from left to right, on Refs. 21; 17; 17 and 18; and 19.

[b]5% soil material damping used for all modes.

approach. The rocking springs differ by less than 3%. These spring rates are higher than the values computed using elastic half-space theory. Including the embedment effects and a higher Poisson's ratio further stiffens the system. On the other hand, the lower shear modulus tends to soften the system compared to the original design values. The results for the median soil case are that the horizontal spring rate is approximately 4% higher than that originally used, while the rocking spring is about 34% higher. Most of the increase is directly attributable to including the embedment effects. The vertical spring rate used in the SEP reanalysis, calculated using Luco's method (assuming no frequency dependence) as for the horizontal and rocking springs, is almost identical to that determined using Kausel's method.[17] To further account for uncertainties resulting from the lack of soils information and the analytical approximations described above, the structural evaluation of Palisades was conducted assuming a ±50% variation in the soil shear modulus. This results in a range of soil modulus values from 30% to 90% of the low-strain value. The extreme soil values and the best estimate are referred to as the upper- and lower-bound soil cases and the median soil case.

Geometric (or radiation) damping considerations are also very important in the solution of SSI problems. However, such considerations are complicated by the layered-site characteristics at Palisades: Energy tends to be reflected from the soil-bedrock interface. The result is significantly lower damping for some modes than in an equivalent elastic half space. This is particularly true for the vertical response. In addition, the damping values tend to be more sensitive to small frequency variations than equivalent half-space values.

Depending on the frequency of the system and the stiffness of the bedrock, the damping may be limited to only the internal (material) damping of the soil; essentially no geometric damping may exist. The geometric damping values calculated for the Palisades containment building are shown in Table 5-2. Horizontal and rocking damping ratios of 34% and 1%, respectively, were calculated in accordance with Ref. 17. The horizontal damping corresponds to the half-space value, whereas the 1% rocking damping is somewhat below the 2.5% for an equivalent elastic half space. Luco's method indicates virtually no radiation damping for the rocking mode.

For the vertical response, the variation in the geometric damping is much more pronounced for the layered site, since the underlying bedrock tends to reflect most of the elastic wave energy back into the structure. The vertical damping for the Palisades site, as calculated from Kausel's recommended empirical relation,[18] is 6% of critical. Although none of the cases analyzed by Kausel corresponds exactly with the Palisades site parameters, the vertical damping obtained for the case closest to Palisades indicates that the 6% value is conservative. Using Luco's method[19] and a rigid bedrock interface, no geometric damping is indicated for the vertical response. However, when using the actual bedrock properties and an equivalent-static approach consistent with that used to compute the vertical spring rate, we obtained a vertical geometric damping of approximately 3%. (These values may be compared to the 60% vertical damping for an elastic half space.) Small changes in frequency result in very large changes in the calculated vertical damping for the layered-site values. Unfortunately, the limited soils information available does not permit an exact calculation of the system frequencies. Also, the results of Luco do not include a case for Poisson's ratio of 0.45. In view of these uncertainties, a vertical geometric damping of 5% of critical was used in the reanalysis.

A similar procedure was followed for the auxiliary building, with an equivalent rectangular base slab in place of the circular base slab used for the containment building. Table 5-3 shows the soil spring rates and geometric damping values calculated for the median soil case. No embedment exists for the auxiliary building. In addition to the geometric damping values discussed above, 5% soil material damping was used for all modes for both structures, in accordance with recommendations in Ref. 21.

5.3.2 Combination of Earthquake Directional Components

The design of Palisades was based on the absolute addition of one horizontal and one vertical load component. Current recommended practice is to combine the responses for the three principal simultaneous earthquake directions by the SRSS method, as described in R.G. 1.92.[23] Alternatively, it is recommended by Newmark and Hall[4] that directional effects be combined by taking 100% of the effects due to motion in one direction and 40% of the effects from the two remaining principal directions of motion.

The result of an SRSS combination of three components, compared to an absolute addition of two, depends on the relative magnitudes of the responses

TABLE 5-3. Comparison of median-soil spring rate constants and geometric damping values for Palisades auxiliary building. The subscripts h, ϕ, v, and θ represent horizontal, rocking, vertical, and torsional modes, respectively.

	Original design (E-W)	Original design (N-S)	Elastic half space (E-W)	Elastic half space (N-S)	Kausel[a] (E-W)	Kausel[a] (N-S)
Spring constants:						
K_h, lb/ft	1.91×10^9	2.01×10^9	1.42×10^9	1.49×10^9	1.90×10^9	1.90×10^9
K_ϕ, lb-ft/rad	1.19×10^{13}	6.35×10^{12}	9.84×10^{12}	5.06×10^{12}	1.07×10^{13}	6.49×10^{12}
K_v, lb/ft	--	--	1.99×10^9	1.99×10^9	3.42×10^9	3.42×10^9
K_θ, lb-ft/rad	--	--	8.82×10^{12}	8.82×10^{12}	8.82×10^{12}	8.82×10^{12}
Damping constants:						
D_h, %	--	--	67	67	67	67
D_ϕ, %	--	--	27	27	1.4	1.7
D_v, %	--	--	116	116	5	5
D_θ, %	--	--	26	26	26	26

[a]Luco spring used for vertical cases.

and the geometry of the structure. For instance, if the two horizontal load components are approximately equal and the vertical component is negligible, the SRSS method results in an increase in stress of approximately 40% for a square plant structure and 0% for a circular one. Combining the effects by the 100%/40%/40% method produces a 40% increase in stress (compared to direct addition) for a square building and an increase of about 8% for a circular structure, such as the Palisades containment building. If the two horizontal load components are approximately equal and result in stresses about equal to that from the vertical component, both SRSS and 100%/40%/40% stress combinations are less than the absolute sum of one horizontal component and the vertical component.

Conditions typically fall somewhere between the two cases discussed above. For various structures, with either rectangular or circular plans, the stresses calculated using current methods of load combinations can vary from about 30% less to 40% greater than the stresses on which the original design was based.

5.3.3 Combinations of Earthquake and Other Loads

The design and analysis of Palisades used the load combinations for Class 1 structures and equipment shown in Table 4-3. Load combinations are now specified in applicable design codes and standards, such as ASME Sec. III, Div. 2,[24] and ACI-349.[25] These codes, which describe the load combination procedures and cases to be considered, tend to be system dependent. In general, the trend has been to allow increased stresses for the specified seismic loading, but also to simultaneously increase the number of load conditions to be considered. The NRC has endorsed these load combinations, with the exceptions noted in Sec. 3.8 of the Standard Review Plan.[3] Because stresses resulting from load cases and combinations of loads from these more recent criteria are not available, this reevaluation has considered only the effects of changes in seismic loading produced by applying more recent methods. The allowable stresses used in the design of Class 1 structures are discussed in Chapter 4. These allowable stresses were used as the basis for seismic margin comparisons.

5.4 CONFIRMATORY ANALYSIS OF STRUCTURES

The original mathematical models of the containment and auxiliary buildings were two-dimensional planar representations. Such models are normally adequate for symmetric structures, but are not appropriate for structures or equipment where significant asymmetry exists. Because of the high degree of symmetry, a two-dimensional model was considered adequate for the Palisades containment building. Therefore, a new two-dimensional model of this structure was developed, incorporating the SSI effects discussed in the previous section. To evaluate the asymmetric effects of the auxiliary building, however, a new three-dimensional model was developed. These models were used to develop new structural loads from a response-spectrum analysis using R.G. 1.60 spectra, as well as new in-structure response spectra consistent with the SEP guidelines (see Ref. 4).

For the most part, the new models were similar to the original models, with the exception of the treatment of the SSI effects for the layered site. Mass and stiffness properties were computed and compared with the original values to the extent possible. While the mass of the structure could be computed from available drawings, neither the equipment weights nor the original design calculations were available. The masses used in the original calculations were, therefore, compared with the tributary masses of the structure, combined with a representative value simulating the weight of the equipment. In all cases, the original values agreed closely with the estimated masses; therefore, the original mass values were used in the new models. A similar approach was used for checking member stiffnesses, except that new torsional stiffness values were calculated for the 3-D auxiliary building model. In view of the close agreement between the dynamic properties originally used and those calculated in this reevaluation, the uncertainties in the structural models are expected to be overshadowed by the uncertainty in the soil properties. This greater uncertainty is accounted for by using a fairly wide range of soil values.

5.4.1 Containment Building

The model developed for the confirmatory analysis of the containment building is shown in Fig. 5-3. This model is essentially the same structural

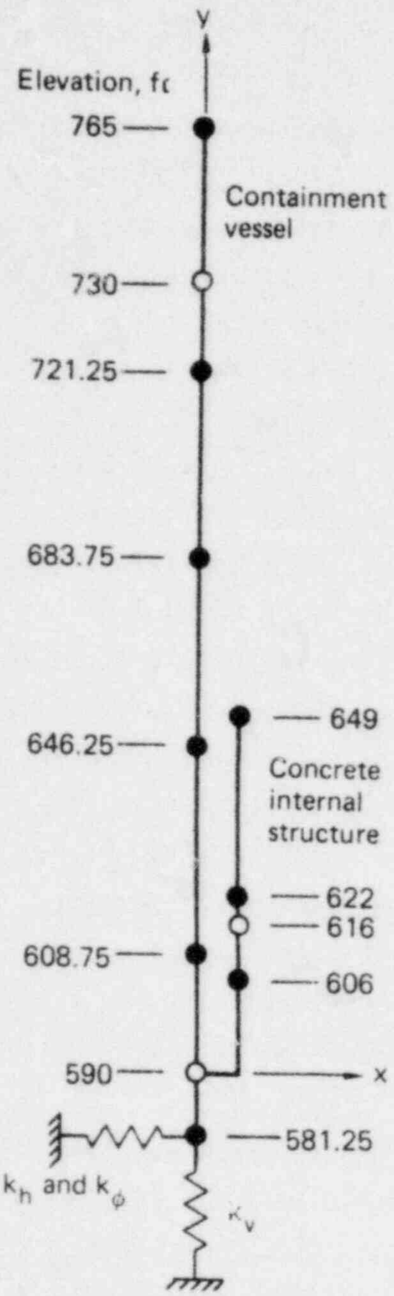


FIG. 5-3. Containment building model used in the SEP reevaluation. The open circles are massless nodes. The horizontal spring symbol depicts both a linear horizontal spring and a rocking spring (k_ϕ)--see Table 5-2.

model that was used for design except for new soil springs that account for the layered-site characteristics. Several new massless nodes were also added at locations where in-structure response spectra were desired.

The first several vibration frequencies of the containment building founded on the layered soil site are shown in Table 5-4 for the upper, median, and lower soil cases. Virtually all of the lateral response occurs as a result of the four modes shown. The vertical response is essentially a single-frequency response with very little amplification through the structure. Also shown for comparison are the frequencies of the first four response modes used in the original design analysis. The mode shapes for the median soil case are shown in Fig. 5-4.

In the original design, a modal damping ratio of 7.5% of critical was used for the SSE for all modes. In the SEP reevaluation, the large differences in the levels of geometric and structural damping for the various modes associated with SSI required the calculation of composite damping ratios. The method selected was originally developed by Roesset, Whitman, and Dobry.[26] In this approach, the composite damping ratios comprise two terms based on energy proportioning. One term accounts for energy assumed to dissipate in viscous form, and the other term is based on the energy assumed to dissipate in a hysteretic form. In the SEP reevaluation, the viscous portion is assumed to consist of the horizontal and vertical components of the soil damping. The remaining portion of the soil and structural damping is proportioned on the assumption that the energy is dissipated hysteretically.

In computing the composite modal damping ratios, the structural damping values recommended in NUREG/CR-0098,[4] and shown above in Table 5-1, were used. Composite damping ratios were originally calculated on the assumption

TABLE 5-4. Containment building response frequencies, in Hz.

Mode	Original design	Upper soil case	Median soil case	Lower soil case
1	1.94	2.42	2.06	1.52
2 (vert)	--	5.78	4.79	3.44
3	5.52	5.78	5.78	4.28
4	12.96	13.24	12.85	12.46
5 (vert)	--	17.93	17.74	17.56
6	20.28	20.23	20.02	19.81

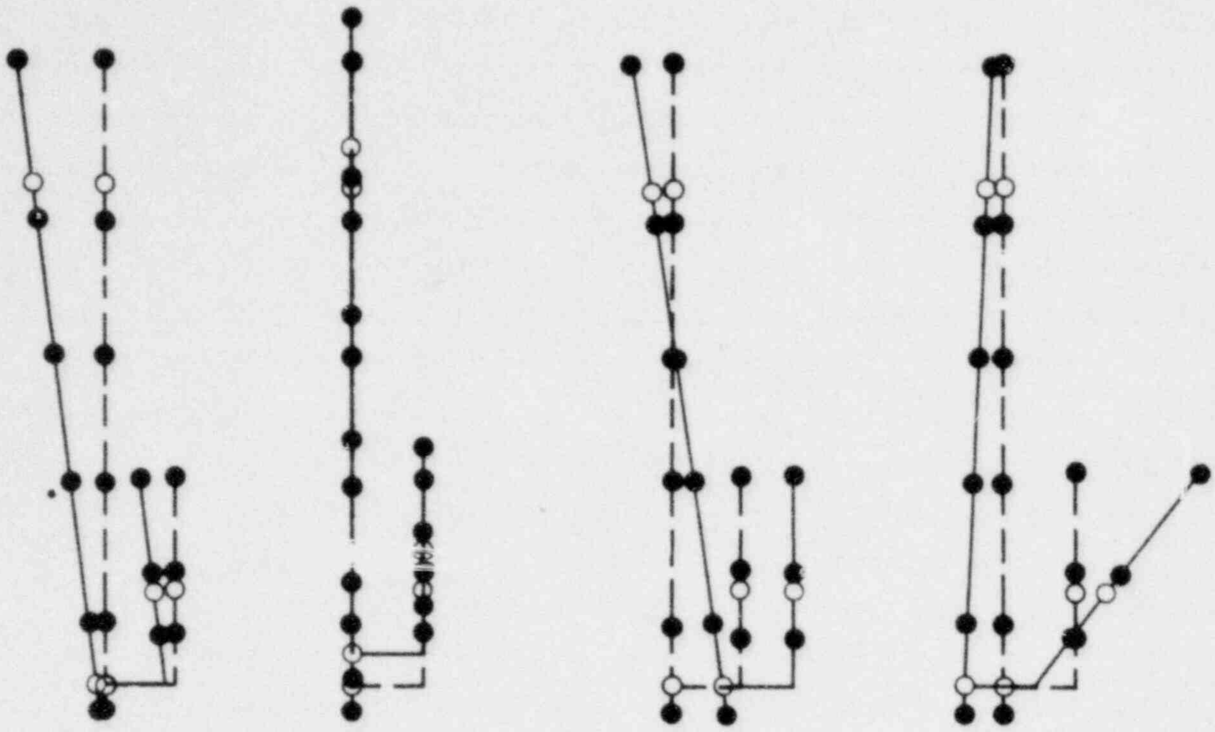


FIG. 5-4. Mode shapes for the first four modes of the containment building for the median soil case. The unperturbed model is shown with dashed lines.

that the dynamic response in the containment building would produce stresses at or near yield in the concrete. Based on the loads developed using this assumption, however, the stresses proved to be substantially less than the yield levels. The final analysis to determine the loads for the containment building was based on 3% of critical, in accordance with recommendations in Ref. 4 for prestressed concrete at about one-half the yield point. In view of the large amount of geometric damping predicted for the horizontal modes, a maximum composite modal damping ratio of 20% of critical was assumed in the analysis. (Appendix A discusses the consequences of this assumption.) Table 5-5 shows the composite modal damping ratios used in the analysis of the containment building for the median soil case.

The dynamic response of the containment building, computed using the methods and criteria discussed in the previous sections, is presented in the

TABLE 5-5. Modal damping ratios for the containment building (median soil case).

Mode	Frequency	Damping, % of critical
1	2.06	8
2	4.79	10
3	5.78	20
4	12.85	10
5	17.74	4
6	20.02	6

form of shear and bending moment diagrams for the containment vessel and concrete internal structure. Results for the median soil case, as well as for the upper- and lower-bound soil conditions, are shown in Figs. 5-5 and 5-6. Also shown in these same figures are the corresponding values used in design. As is evident from these figures, the dynamic loads resulting in the structure from application of the SEP acceptance criteria are, for the lower-bound soil case, not significantly different from the original design loads. The recalculated shears are less than the original loads throughout the containment vessel, and the recalculated bending moments are very slightly less than the original values for elevations below 600 ft. For the lower-bound soil condition, the new dynamic loads throughout the internal structure are essentially the same as the design values.

For both the median and upper-bound soil cases, the dynamic loads calculated in the SEP reanalysis exceed the original values throughout both the containment vessel and the concrete internal structure. Seismic loads resulting from vertical response were apparently not calculated in the original design analysis. Based on the present reanalysis, the vertical response throughout the containment building is 0.34 g for the lower-bound soil case and 0.27 g for both the median and upper-bound soil conditions.

Table 5-6 is a summary of the load ratios for the containment vessel and concrete internals. (The load ratio is defined as the ratio of the load calculated in the original analysis to that derived in the SEP reanalysis.)

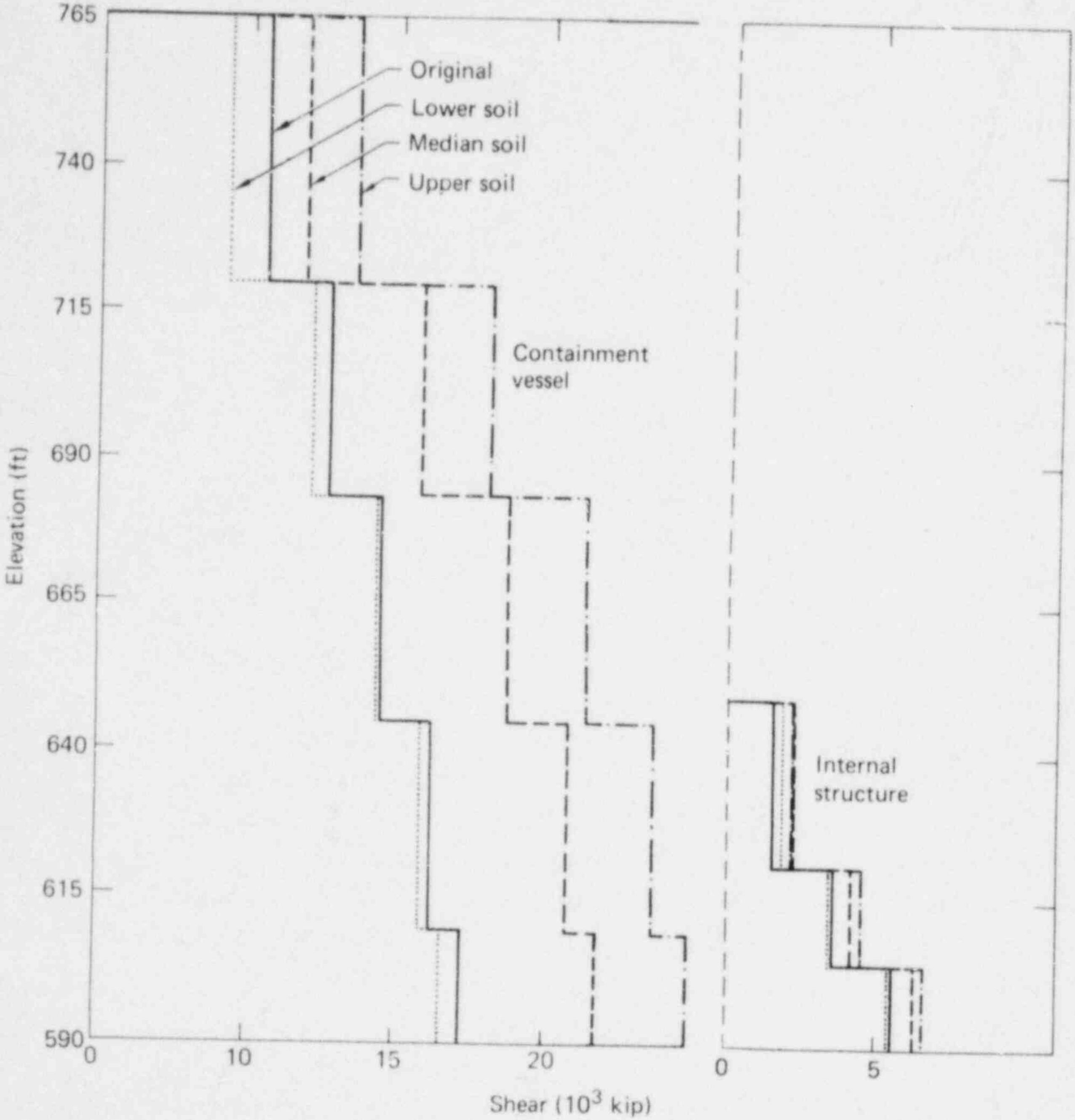


FIG. 5-5. Shear distribution in the containment building for the lower, median, and upper soil cases, compared to the distribution calculated in the original analysis.

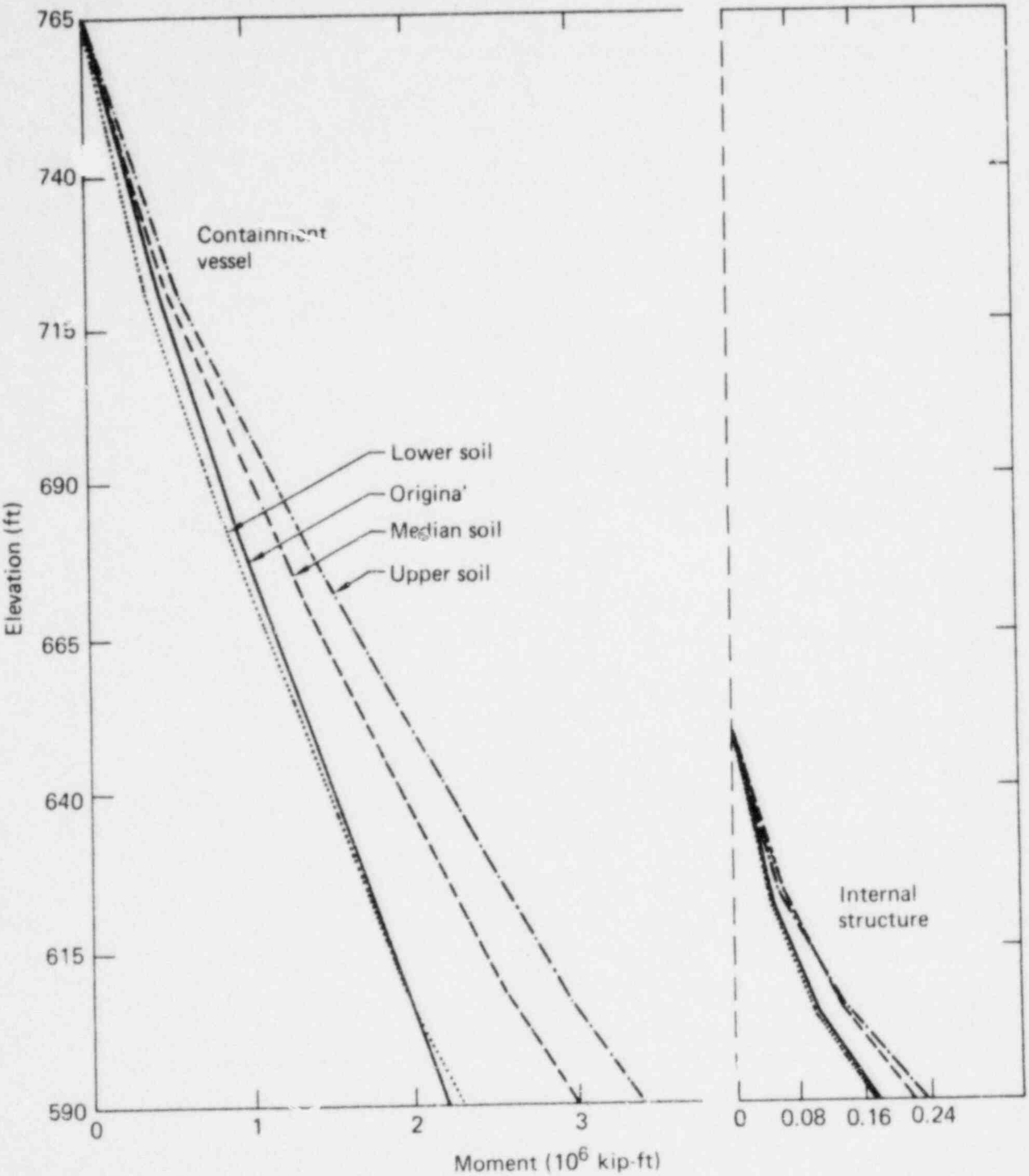


FIG. 5-6. Moment distribution in the containment building for the lower, median, and upper soil cases, compared to the distribution calculated in the original analysis.

TABLE 5-6. Ratios of the loads calculated for the containment building and the concrete internals in the original analysis to those calculated in the present reanalysis.

	Soil condition		
	Lower bound	Median	Upper bound
Containment vessel (El 590'):			
Shear	1.07	0.78	0.69
Moment	0.96	0.73	0.65
Containment vessel (El 610'):			
Shear	1.01	0.76	0.68
Moment	1.0	0.82	0.66
Containment vessel (El 645'):			
Shear	1.01	0.78	0.68
Moment	1.03	0.78	0.68
Containment vessel (El 685'):			
Shear	1.05	0.81	0.71
Moment	1.07	0.81	0.69
Containment vessel (El 720'):			
Shear	1.14	0.89	0.78
Moment	1.23	0.96	0.83
Concrete internals (El 590'):			
Shear	1.02	0.84	0.83
Moment	1.0	0.82	0.81
Concrete internals (El 605'):			
Shear	1.03	0.83	0.78
Moment	1.0	0.81	0.70
Concrete internals (El 625'):			
Shear	0.82	0.64	0.73
Moment	1.0	0.83	0.69

The original loads were based on the Housner response spectra with 7.5% modal damping; the recalculated loads are based on the R.G. 1.60 spectra with composite damping computed as previously discussed. The upper-bound soil case gives the highest response for the containment building. Load ratios of approximately 0.7 are typical for this condition. The effects that loads of this level have on the capacity of the structure are discussed in Sec. 5.5.1.

5.4.2 Auxiliary Building

The model developed for the reevaluation of the auxiliary building is a three-dimensional, lumped-mass model, as shown in Fig. 5-7. The locations of the masses in the model account for major discrete masses such as the refueling pool, but other equipment weights were taken to be uniformally distributed throughout the floor slabs. Both translational mass and rotational moments of inertia were included. Stiffness and center-of-rigidity calculations were based on the concrete shear walls and steel framing of the upper elevations, but any masonry-block wall stiffness values were neglected. All floor diaphragms were assumed rigid. New soil springs were based on the layered-site characteristics and on a rectangular base slab with the same area as the slightly irregular base slab of the actual structure.

The natural frequencies below 33 Hz for the auxiliary building are shown in Table 5-7 for the three soil cases considered. Mode shapes for the more important response modes are shown in Fig. 5-8 for the median soil case. No auxiliary building frequencies or mode shapes were available for the original design analysis.

Composite modal damping ratios based on the same energy-proportioning method used for the containment building were calculated for the auxiliary building.[26] The response was originally calculated using structural damping values for steel and reinforced concrete on the assumption that the response would be close to yield. However, the stresses developed throughout the structure using these values were substantially less than yield. Consequently, the final loads developed for this reevaluation were based on 3% of critical damping for reinforced concrete and 7% for bolted steel, which are consistent with Ref. 4. As in the case of the containment building, a maximum modal damping ratio of 20% of critical was used in the auxiliary building analysis. (See Appendix A for discussion.) The composite modal damping ratios

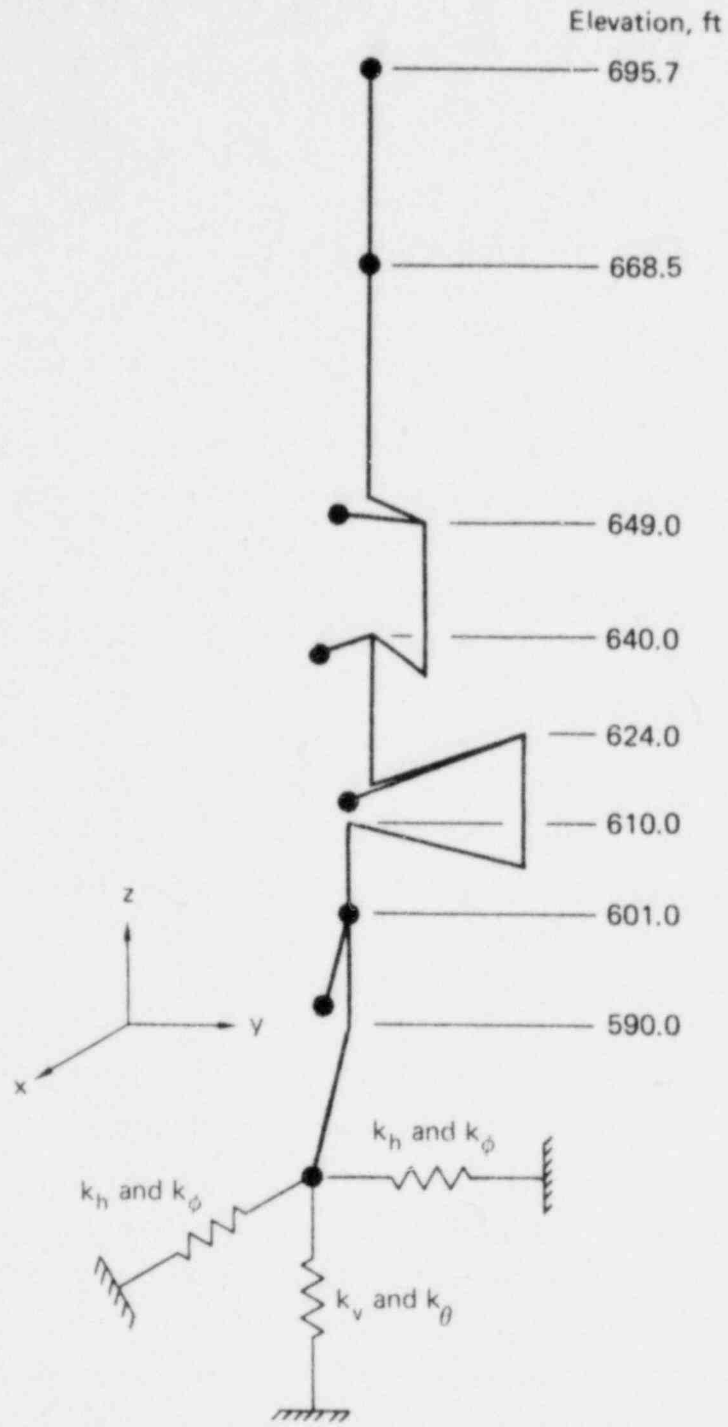


FIG. 5-7. Three-dimensional model used in the reanalysis of the auxiliary building. All nonvertical elements above 590 ft lie in the x-y plane. Each spring symbol depicts both a linear spring and either a rocking spring (k_ϕ) or a torsional spring (k_θ)--see Table 5-3.

TABLE 5-7. Auxiliary building response frequencies, in Hz. No frequencies were available for the original analysis.

Mode	Upper soil case	Median soil case	Lower soil case	Primary response direction (median soil)
1	2.24	2.24	2.23	N-S
2	2.28	2.28	2.27	E-W
3	3.76	3.76	2.60	Coupled
4	5.39	5.06	3.69	N-S
5	6.09	5.12	3.78	N-S
6	6.20	5.30	3.83	E-W
7	6.35	5.41	5.40	N-S
8	7.34	7.29	5.54	E-W
9	9.38	7.74	7.27	Vertical
10	10.4	10.4	10.4	Coupled
11	22.9	19.6	14.5	N-S
12	25.1	22.0	16.9	E-W
13	31.1	30.8	30.5	E-W

for the auxiliary building for the median soil case are shown in Table 5-8.

The shear and bending moment distributions throughout the auxiliary building are shown in Figs. 5-9 through 5-12 for both the N-S and E-W responses and for the three soil conditions. Also shown are the original design values, which were taken as the maxima of the N-S and E-W responses at each location in the structure.

The lateral loads in the auxiliary building increase as the assumed soil modulus increases. In general, however, the response of the auxiliary building is considerably less affected by soil stiffness than is the containment building. The ratios of the auxiliary building design loads to the loads calculated in the present reanalysis for the three soil cases are shown in Table 5-9. The minimum load ratios are at El 640 ft for bending moment; however, this is not a critical location as far as stress in the structure is concerned. No design loads were available for the steel framing

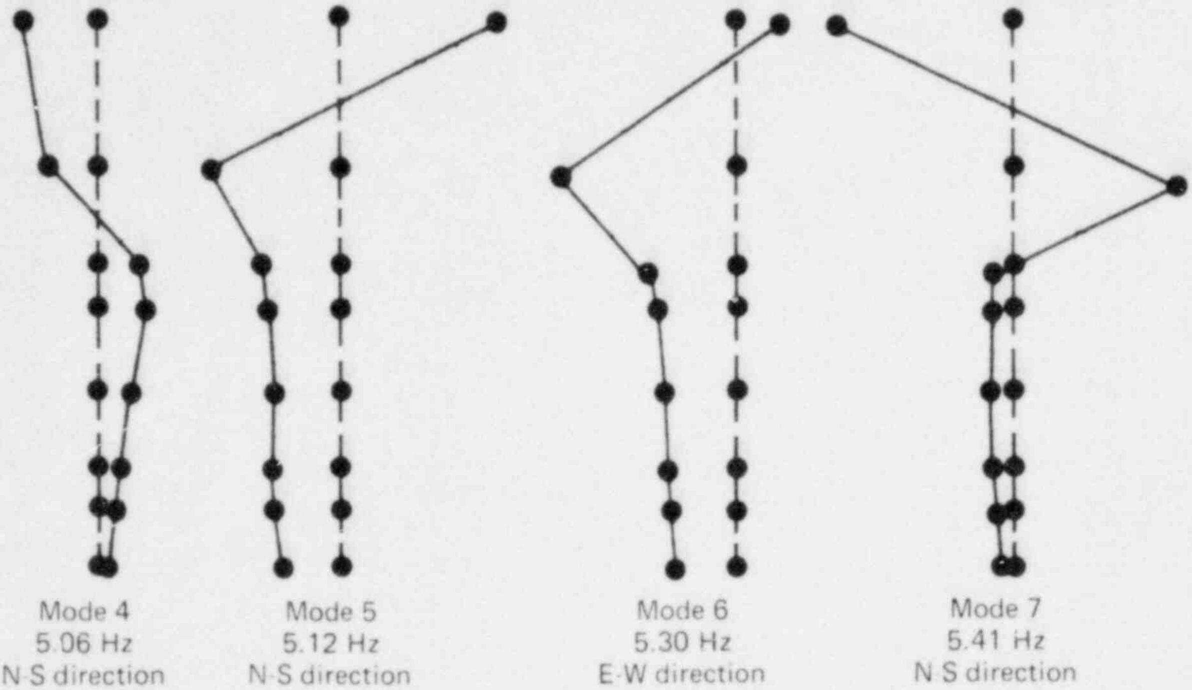


FIG. 5-8. Mode shapes for the four most important modes of the auxiliary building for the median soil case. The unperturbed model is shown with dashed lines.

TABLE 5-8. Modal damping ratios for the auxiliary building (median soil).

Mode	Frequency	Damping, % of critical	Mode	Frequency	Damping, % of critical
1	2.24	7	8	7.29	9
2	2.28	7	9	7.74	10
3	3.76	7	10	10.4	7
4	5.06	20	11	19.6	20
5	5.12	20	12	22.0	20
6	5.30	20	13	30.8	6
7	5.41	9			

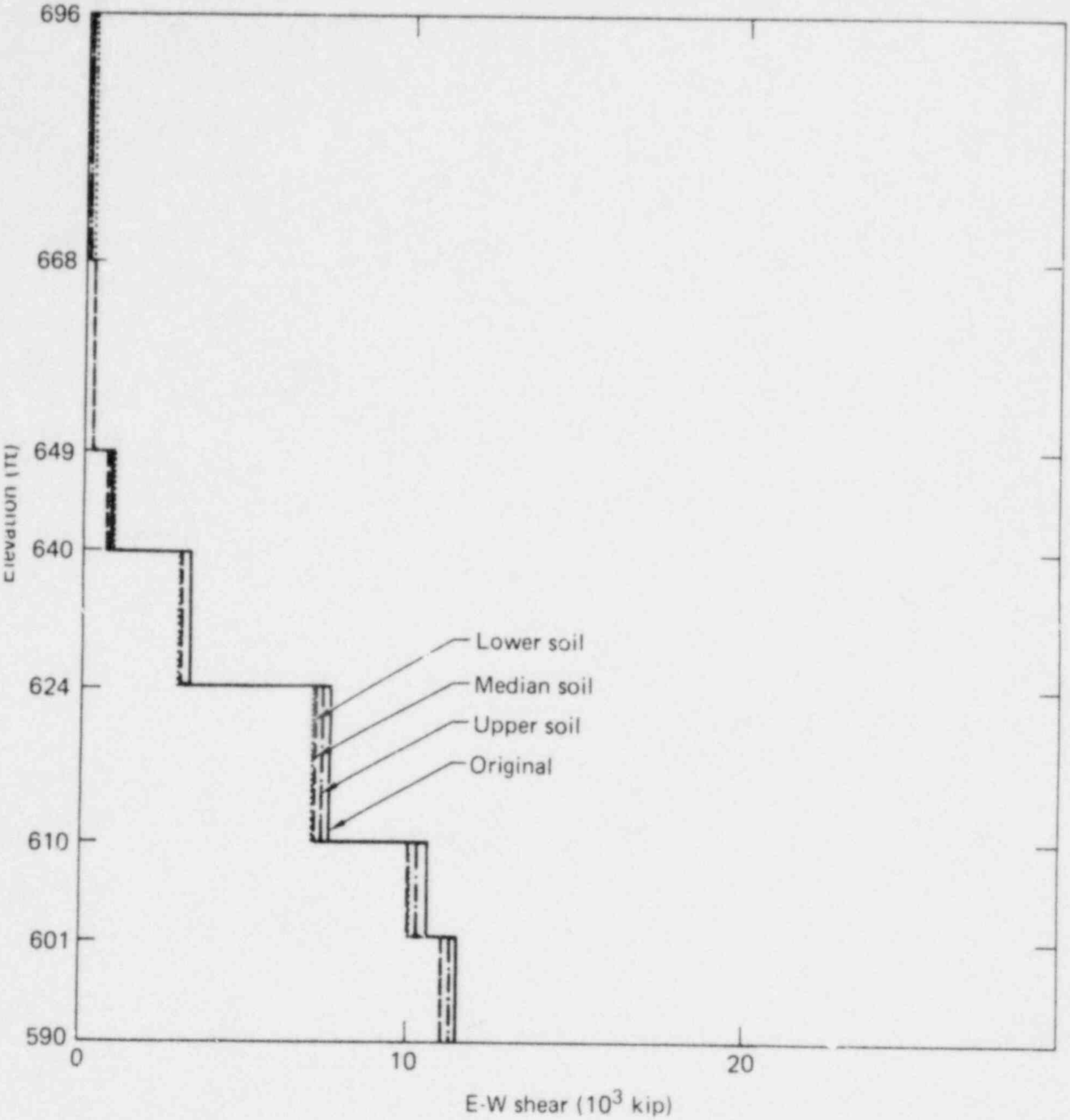


FIG. 5-9. Distribution of E-W shear in the auxiliary building for the lower, median, and upper soil cases, compared to the results of the original analysis.

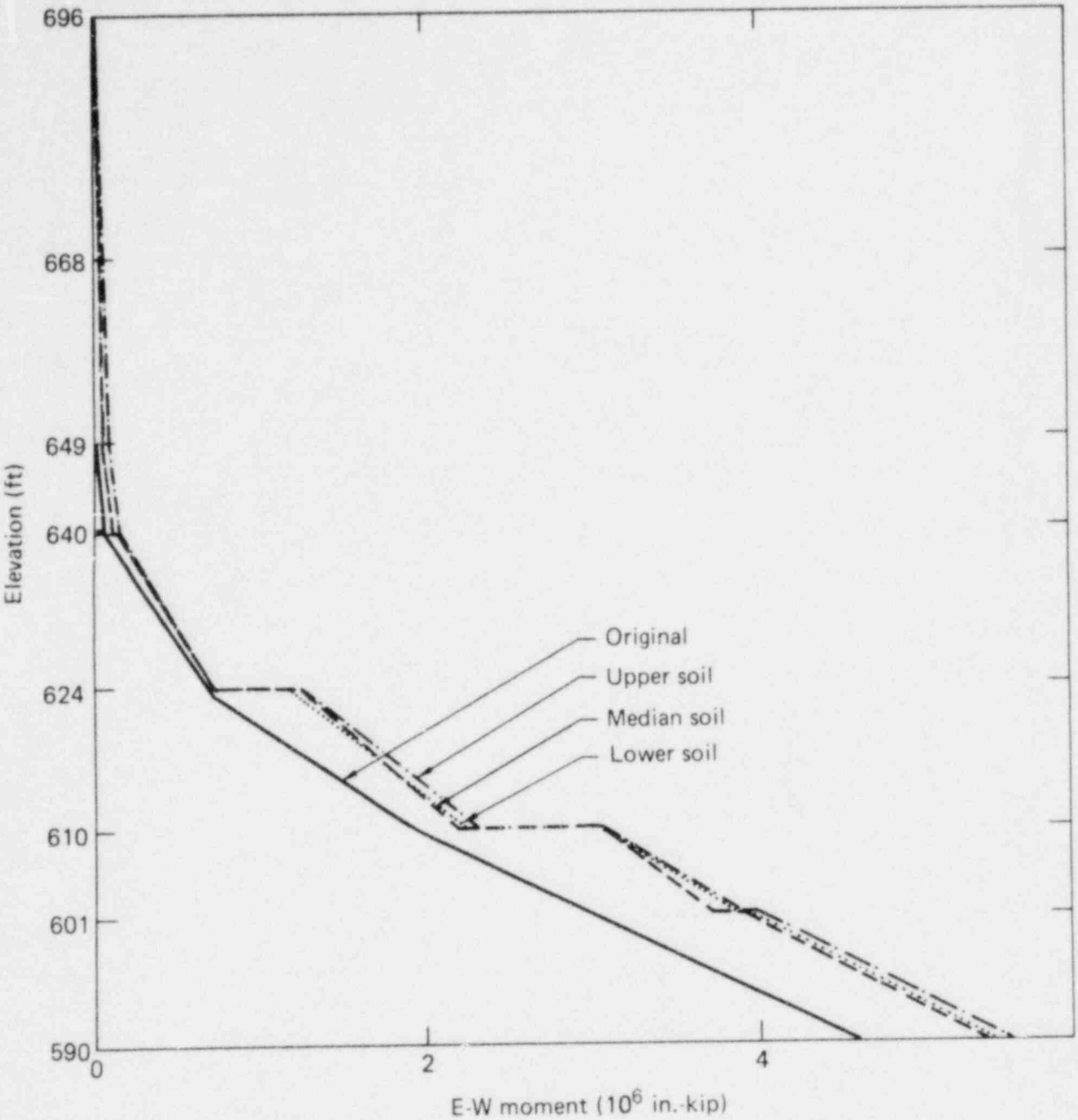


FIG. 5-10. Distribution of E-W moment in the auxiliary building for the lower, median, and upper soil cases, compared to the results of the original analysis.

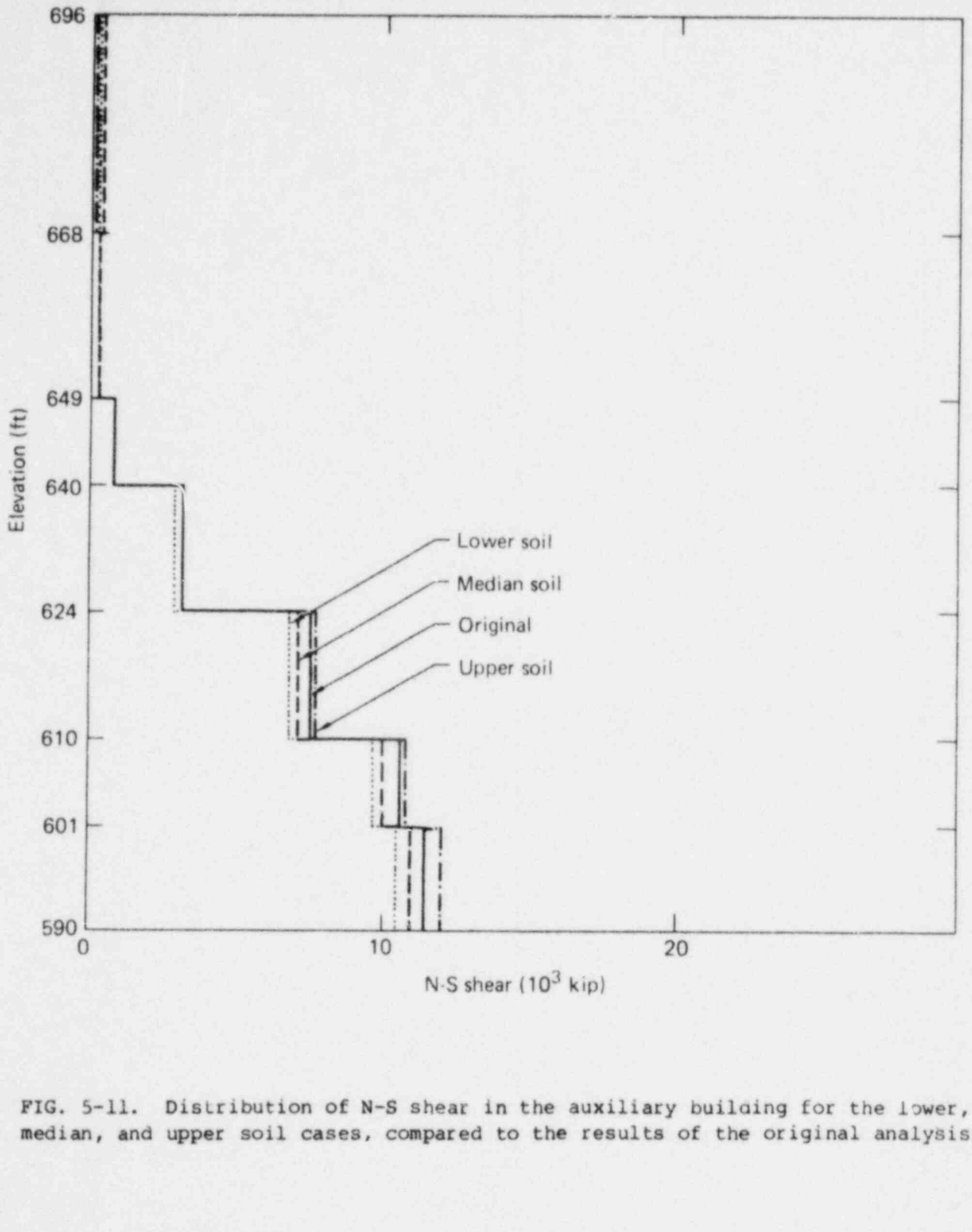


FIG. 5-11. Distribution of N-S shear in the auxiliary building for the lower, median, and upper soil cases, compared to the results of the original analysis.

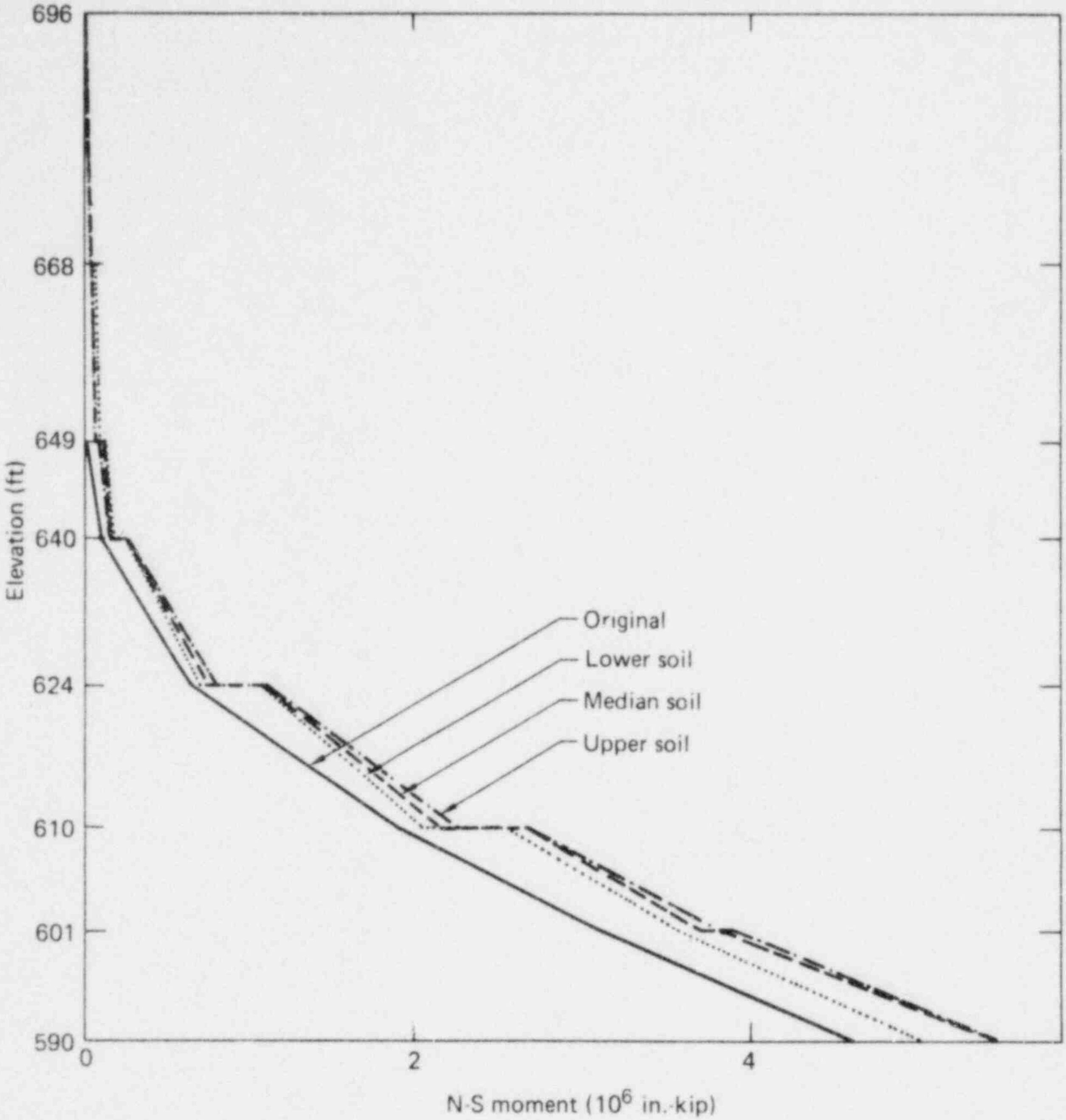


FIG. 5-12. Distribution of N-S moment in the auxiliary building for the lower, median, and upper soil cases, compared to the results of the original analysis.

TABLE 5-9. Ratios of the loads calculated for the auxiliary building in the original analysis to those calculated in the present reanalysis.

	Soil condition					
	Lower bound (N-S)	Lower bound (E-W)	Median (N-S)	Median (E-W)	Upper bound (N-S)	Upper bound (E-W)
El 590':						
Shear	1.09	1.04	1.03	1.05	0.96	1.03
Moment	0.92	0.86	0.87	0.87	0.85	0.85
El 601':						
Shear	1.10	1.05	1.04	1.06	0.98	1.04
Moment	0.87	0.79	0.83	0.82	0.82	0.79
El 610':						
Shear	1.10	1.07	1.04	1.07	0.97	1.04
Moment	0.98	0.87	0.89	0.89	0.87	0.85
El 624':						
Shear	1.10	1.10	1.0	1.10	0.99	1.0
Moment	0.93	0.97	0.99	0.99	0.88	0.92
El 640':						
Shear	1.0	1.0	1.0	1.0	1.0	1.0
Moment	0.8	0.8	0.8	0.8	0.7	0.7

above El 649 ft. Also, no vertical response was calculated in the original design analysis of the auxiliary building. Based on the SEP reanalysis, the vertical response throughout the structure is 0.23 g for the upper soil case, 0.24 g for the median case, and 0.28 g for the lower-bound condition. Virtually no amplification occurs within the structure for vertical excitation.

5.4.3 Field-Erected Tanks

The capability of three field-erected tanks to withstand seismically induced stresses was evaluated with regard to current methods and guidelines as set forth in Ref. 4. The tanks were the T-2 condensate storage tank (CST), the T-81 primary water supply make-up storage tank (PWSMST), and the T-58 safety injection and refueling water tank (SIRWT). Descriptions of these tanks are given in Table 5-10. The T-2 CST and T-81 PWSMST are located at ground level near the northwest corner of the turbine building. Part of the T-2 CST is located above the cover slab for the common valve pit for these two tanks. The T-58 SIRWT is located on one of the auxiliary building roof slabs at El 640 ft.

Several potential failure modes were investigated in the course of the SEP reevaluation of the three tanks. Among these were buckling of the tank sidewalls due to the seismic overturning moment; yielding, fracture, or pullout of the anchor bolts; collapse of the tank roof; sliding of the tank at the base, with subsequent rupture of connections; and failure due to high tensile stresses in the hoop direction that might result from hydrodynamic pressures occurring simultaneously with the hydrostatic pressure. Because of the geometry of the Palisades tanks, actual overturning is not a potential problem. However, if the anchor bolts fail, the overturning moment must be resisted by the tank walls and the weight of the internal fluid. For the fluid to be effective, a portion of the tank must separate from the foundation as shown in Fig. 5-13. This portion of the tank is highly stressed and is a potential source of failure in the presence of nonductile behavior (which might be caused, for example, by poor welds).

Calculation of the overturning moments and shears requires consideration of two important effects. The first is the impulsive force due to the tank shell and part of the fluid moving together. The second arises from a convective mode that occurs when part of the water near the free surface sloshes back and forth inside the tank. Table 5-11 shows the original design acceleration levels, which were the basis of an equivalent-static design.

The SEP reevaluation of the field-erected tanks was conducted using methods outlined by Veletsos[27,28] to calculate the impulsive mode frequencies. This approach is based on Rayleigh's method and includes both the shear and flexural deformations of the tanks. The convective (sloshing) frequencies were calculated following recommendations in Ref. 29. Table 5-12 lists the impulsive and convective frequencies for the three tanks. Spectral accelerations are also shown, together with the assumed modal damping ratios used in the analysis. Because it is unlikely that the maximum modal responses will occur simultaneously, the SRSS method was used to combine the impulsive and convective forces for all field-erected tanks.

TABLE 5-10. Descriptions of field-erected tanks.

Tank	Diameter	Height	Wall thickness and material	Base plate thickness and material	Number, diameter, and material of anchor bolts
T-2 CST	28 ft	28 ft	3/16 in., A36 steel	1/4 in., A36 steel	14, 2 in., A307 (not equally spaced)
T-81 PWSMST	22 ft	27 ft	3/16 in., A36 steel	1/4 in., A36 steel	6, 3/4 in., A307
T-58 SIRWT	46 ft	24 ft	13/32, 9/32, and 1/4 in.; 5454-0 aluminum in 8-ft courses	5/16 in., 5454-0 aluminum	52, 1-1/2 in., A325

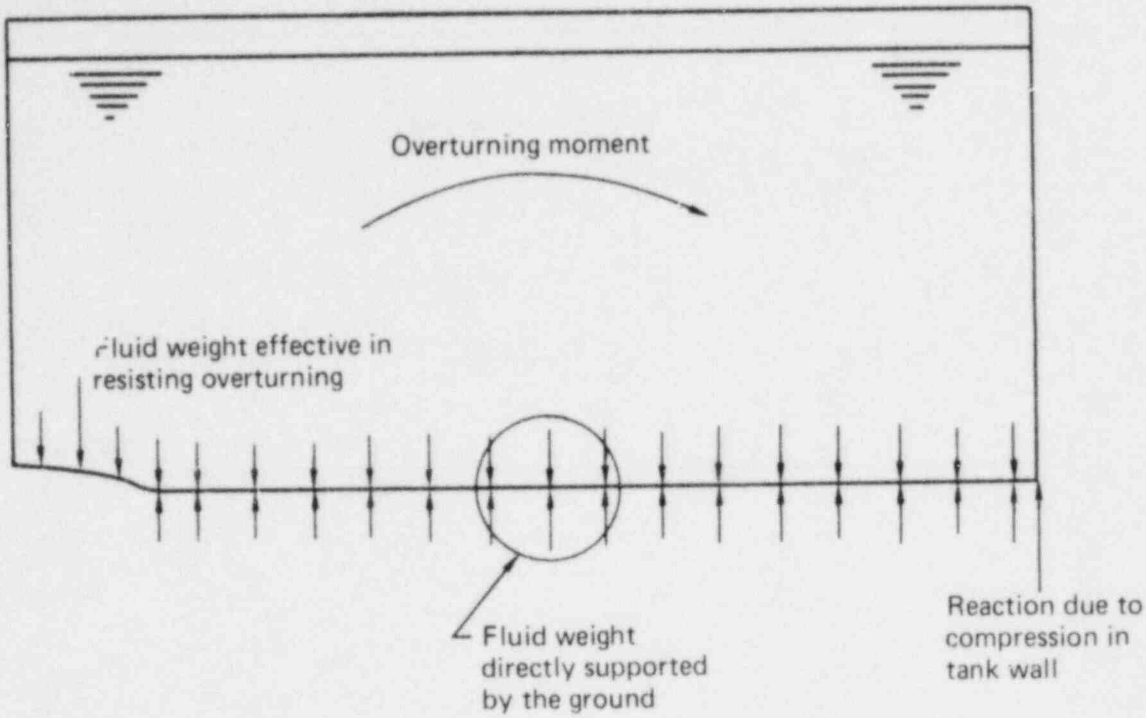


FIG. 5-13. Response to overturning moment in Palisades storage tanks.

TABLE 5-11. Field-erected tank static design accelerations.

	Design acceleration, g	
	Horizontal	Vertical
T-2 CST	0.20	0.133
T-81 PWSMST	0.025	0.025
T-58 SIRWT	0.23	0.17

TABLE 5-12. Response characteristics of field-erected tanks. Mode 1 is the fundamental (impulsive) mode; mode 2 is the convective mode.

	Frequency, Hz	Damping, %	Spectral acceleration, g
T-2 CST:			
Mode 1	11	7	0.40
Mode 2	0.33	0.5	0.18
T-81 PWSMST:			
Mode 1	11.7	7	0.37
Mode 2	0.37	0.5	0.19
T-58 SIRWT:			
Mode 1	20.3	7	0.42
Mode 2	0.25	0.5	0.22

5.4.4 Underground Piping

In conjunction with the SEP reevaluation, a typical buried pipeline was analyzed for the 0.2-g SSE. The pipeline selected was the auxiliary feedwater line, which is fabricated from 6-in.-diameter ASTM A-376 seamless stainless steel. The line runs from the condensate-tank valve pit, under the turbine building, to the auxiliary building, as shown in Fig. 5-14. The elevation of most of the line is 578 ft 5 in., which is approximately 11 ft 7 in. below the top of the turbine building base slab.

The reanalysis considered the stresses induced in the pipe both by strains in the soil resulting from the propagation of elastic waves and by the effects of discontinuities and the relative displacements of attachment points. Stresses may be caused by primary (compression) waves, secondary (shear) waves, and surface waves with various angles of incidence to the pipe. Stresses may also result from relative end-point displacements caused by the motion of the structure at a penetration location. The phasing of the stresses due to soil strains caused by the various types of wave motion is

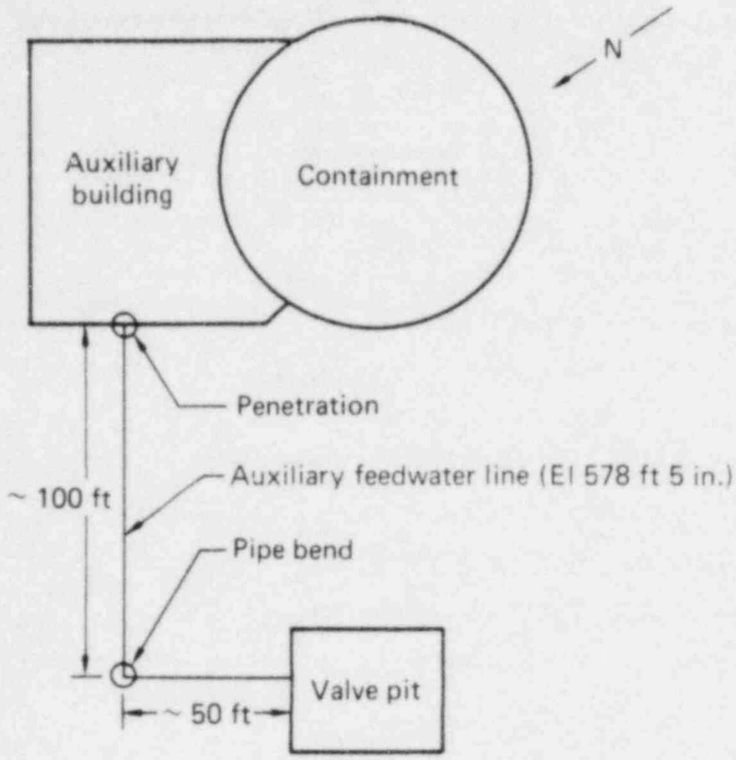


FIG. 5-14. Schematic plan of auxiliary feedwater line.

normally not known, nor is the phasing of soil-induced stresses with those due to end-point motion. Consequently, stresses were combined by the SRSS method. The analysis of the auxiliary feedwater line was conducted using the same median soil modulus and Poisson's ratio that were used for the SSI analysis of the containment and auxiliary buildings. A maximum soil strain of 2×10^{-4} in./in., corresponding to the 0.2-g SSE, was used, as discussed in Sec. 5.5.2.

5.5 EVALUATION OF CRITICAL STRUCTURES

A reevaluation of the seismic capability of the critical structures was conducted using loads developed in accord with SEP acceptance criteria. This reevaluation of the adequacy of the structures to withstand the 0.2-g SSE was based both on comparisons of these recalculated loads with the original design loads and, where necessary, on further stress analysis. Where loads based on the reevaluation guidelines are less than those used in the original design, the structure was in general judged to be adequate without additional evaluation. In cases where the recalculated loads exceed the original loads, but where the resulting stresses are low compared to yield, the structures were again judged to be adequate. In general, damping values based on NUREG/CR-0098[4] were used (see Sec. 5.2.2).

For those cases in which the seismic stresses are not low and where significantly low load ratios exist, conclusions were reached on the basis of ductility for Class 1 structures, as defined in Ref. 4. Accordingly, stresses above yield are considered acceptable provided the ability of the structure to perform its safety shutdown functions is not impaired. References 30 and 31 provide further discussion of rational ductility requirements for the inelastic response of critical structures.

Load ratios were used to provide an initial screening of the expected responses of the structures and do not imply that inelastic responses would actually be expected. (Inelastic response would be expected only if the original design load was at or close to the elastic limit of the structural element. In fact, most 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 between 0.8 and 0.6, and for those for which original and analytical results were not available, more detailed investigations, including stress analyses of critical components, were conducted. No load ratios were below 0.6. A ductility-modified response-spectrum analysis was not performed as part of this reevaluation, and all load ratios were based on linear analyses.

5.5.1 Containment Building

As summarized in Table 5-6, the loads developed in the containment building for the lower soil case are essentially the same as those used in the original design. For the median and upper soil conditions, however, the seismic response loads determined in the present reanalysis exceed the original loads. The load ratio never falls below the 0.8 to 0.6 range, but on the basis of the screening criteria adopted, some additional evaluation was necessary.

A limited stress analysis of the containment building was conducted using the loads developed with current methods and with the damping values recommended in Ref. 4 for structures with stress levels at approximately one-half yield (see Sec. 5.4.1). Concrete strengths (f_c') of 4000 and 5000 psi were used for the concrete internals and containment vessel, respectively. The results indicate a minimum factor of safety of nearly 1.5 for the reactor internal structure (in E-W shear at El 590 ft). The minimum factor of safety for the cracking moment of the containment vessel is over 2.1; other factors of safety throughout the structure are even higher. The finding that all stresses were well below yield justifies the choice of damping values compatible with low stress levels. Details of equipment anchor capacity were not included as part of the structure evaluation--they were considered as part of the equipment evaluation. However, it was concluded that the containment structure is capable of withstanding the 0.2-g SSE, based on SEP acceptance criteria.

5.5.2 Auxiliary Building

The original design loads for the auxiliary building are available only in the form of an envelope of maximum loads, with no indication as to whether they arose from the E-W or N-S response. (The load ratios presented in Table 5-9 are based on this load envelope.) Since the design of the structure was apparently based on this envelope, the comparison of load ratios as a means of evaluation is considered conservative. The response tends to increase as the soil modulus increases. In no case does a load ratio fall below the 0.8 to 0.6 range. As in the case of the containment building, however, some confirmatory stress analysis was conducted to verify the adequacy of the auxiliary building.

Shear, moment, torsion, vertical seismic, and dead loads were considered. Dead loads were assumed uniformly distributed over the floor area. Various load combinations were considered: the combination of the total horizontal response load (for either principal direction) with 40% of the loads due to the remaining horizontal and vertical responses, and the total vertical load plus 40% of the two horizontal loads. Concrete strength was taken as 3000 psi, and 40-ksi yield was assumed for the A-15 intermediate-grade reinforcing steel.

In general, the loads decrease rapidly with increased elevation: All the most highly stressed members are located at El 590 ft. Also, although the torsional response is not the major contribution to the load, the presence of moderate torsion causes elements located near the corners of the structure to be the most highly stressed For primary response in the E-W direction, the two E-W shear walls in the northwest and southeast corners of the structure (elements Nos. 22 and 39 in Fig. 5-15) were found to be the most highly stressed. Similarly, for primary response in the N-S direction, the N-S shear walls on the east side and in the northwest corner of the building (elements Nos. 1 and 17 in Fig. 5-15) are the most highly stressed. Table 5-13 shows a summary of the loads, stresses, and ultimate capacities of the most highly stressed elements. With the exception of element 17, the maximum shear stress, including torsion, is less than the code value, including the ϕ factor for workmanship of 0.85. If the ϕ factor is neglected, the stresses in all elements would be less than code allowables. Maximum corresponding flexural stresses of approximately 160 psi occur, and no net flexural tension results. All walls interior to the elements discussed above are less stressed, as are elements at higher elevations. Consequently, the auxiliary building is considered capable of withstanding the postulated 0.2-g SSE with no loss of function and with possible light cracking in only one shear wall.

5.5.3 Building-to-Building Interaction

Since the containment, auxiliary, and turbine buildings are supported on individual foundations, consideration was given to the possibility of impact between the buildings. The turbine and auxiliary buildings are framed together at the lower elevations, but at El 624 ft, only slotted connections exist in the N-S direction, and very light brackets resist E-W motion; the structures are essentially uncoupled above El 610 ft. At El 610 ft, the maximum auxiliary building displacement normal to the turbine building (assuming the soft-soil condition) is approximately 0.18 in., of which approximately 0.16 in. is due to soil deformation. Since the slotted connections will not transfer large seismic loads, no damage that would impair the functional capacity of the auxiliary building is expected to result from the interaction between the turbine and auxiliary buildings.

A 1-in. rattle space exists between the auxiliary building and the containment building. The locations of concern in any interaction between

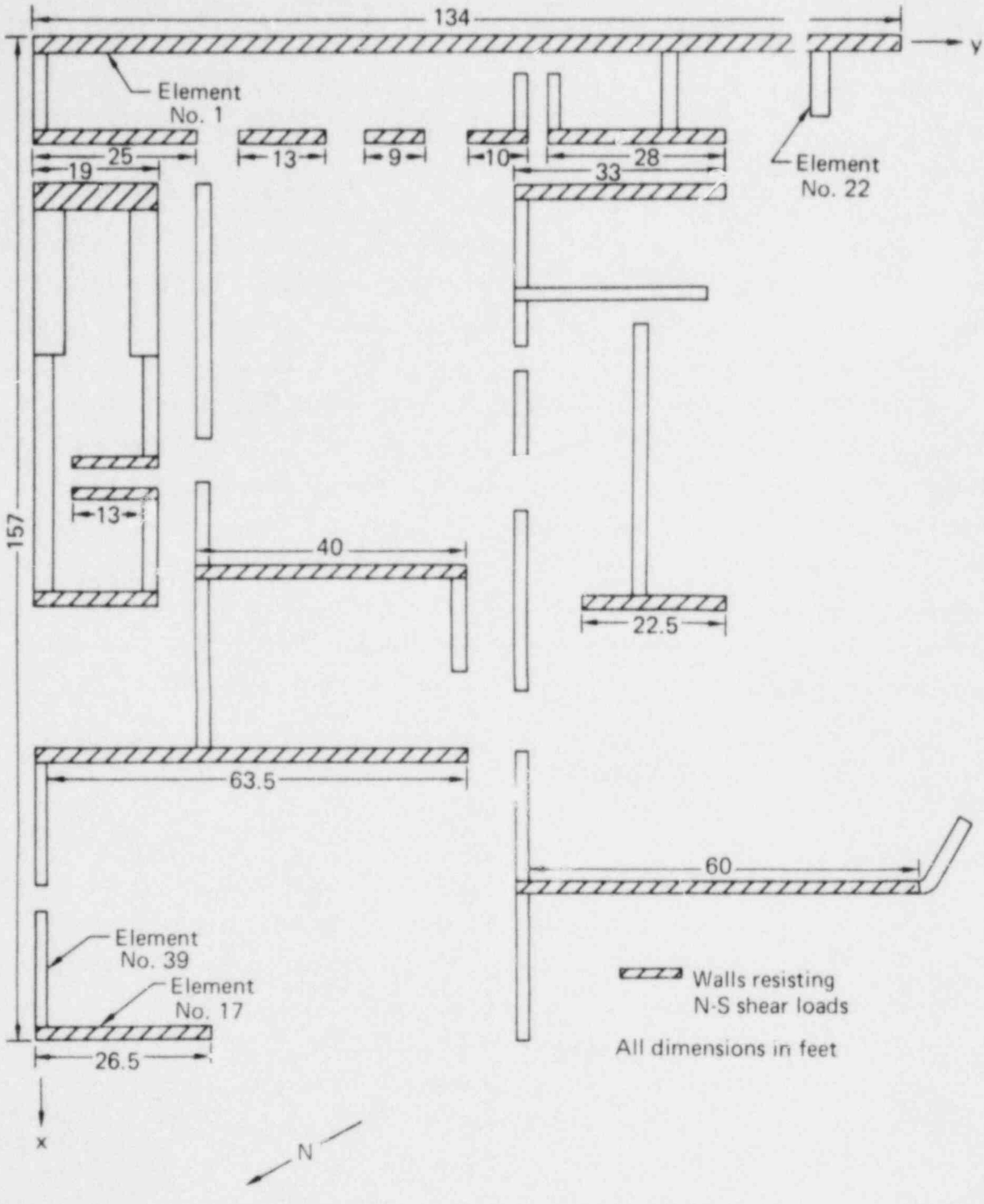


FIG. 5-15. Shear wall plan for the auxiliary building at El 590 ft.

TABLE 5-13. Typical member loads and capacities at El 590 ft in the auxiliary building for the postulated 0.2-g SSE.

Direction of loading	Member number (Fig. 5-15)	Axial stress on shear walls due to moment and vertical load,[a] ksi		Maximum shear load, including torsion, kips	Calculated capacity in shear, kips		
		for σ_{max}	for σ_{min}		concrete	steel	total[b]
E-W	39	0.121	0.048	468	407	402	688
E-W	22	0.162	0.007	732	454	456	744
N-S	1	0.123	0.046	4714	3380	3770	6080
N-S	17	0.157	0.013	954	502	509	8[illegible]0

[a]Positive numbers indicate compression.

[b]Total capacity was taken as 85% of the sum of the concrete and steel capacities, which were calculated assuming f_c' = 3000 psi and f_y = 40 ksi (for A-15 intermediate-grade reinforcing steel).

these two buildings are limited to elevations below 649 ft, since the relatively lightweight and flexible steel framing of the auxiliary building above El 649 ft is not expected to cause severe damage, even if it should strike the containment building. The soft-soil assumption produces the maximum structural deflection for both the auxiliary and the reactor buildings. At El 649 ft, the auxiliary building deflects about 0.23 in. in the direction of the containment building. The deformation at the base slab (El 590 ft) due to soil flexibility accounts for about 0.16 in. of this value. The corresponding deflections in the containment building are approximately 0.88 and 0.36 in., respectively. Although some of the natural frequencies of the auxiliary building are close to the fundamental of the containment building, the auxiliary building modes involve essentially only the steel superstructure. Containment building response frequencies are widely separated from all auxiliary building modes that involve significant displacements of the concrete shear wall structure. Consequently, the most appropriate estimate of relative displacement is obtained by combining the two structural displacements by the SRSS method. On this basis, the maximum relative displacement was calculated to be approximately 0.91 in., indicating that impact would not occur. Even if the displacements of the two structures were exactly out of phase, a maximum interference of approximately 0.11 in. (the absolute sum of relative displacements) would be expected for the soft-soil condition. Figure 5-16 illustrates these results graphically. Although some local concrete cracking and spalling could be anticipated, major structural damage sufficient to cause any loss of functionality is not predicted for the soft-soil condition, and no impact is expected in the median and upper soil cases, even on an absolute sum basis.

Although no analysis of the turbine building was conducted, and although no through-soil coupling effects were considered in the analysis of the building-to-building interaction, no response modes that would cause loss of structural function of the auxiliary building are expected for the 0.2-g SSE.

5.5.4 Field-Erected Tanks

Tank shell and base plate integrity during the postulated 0.2-g SSE was evaluated for each of the three tanks. Initially, the assumption was made that the tanks behaved linearly, as shown in Fig. 5-17a. Tensile and

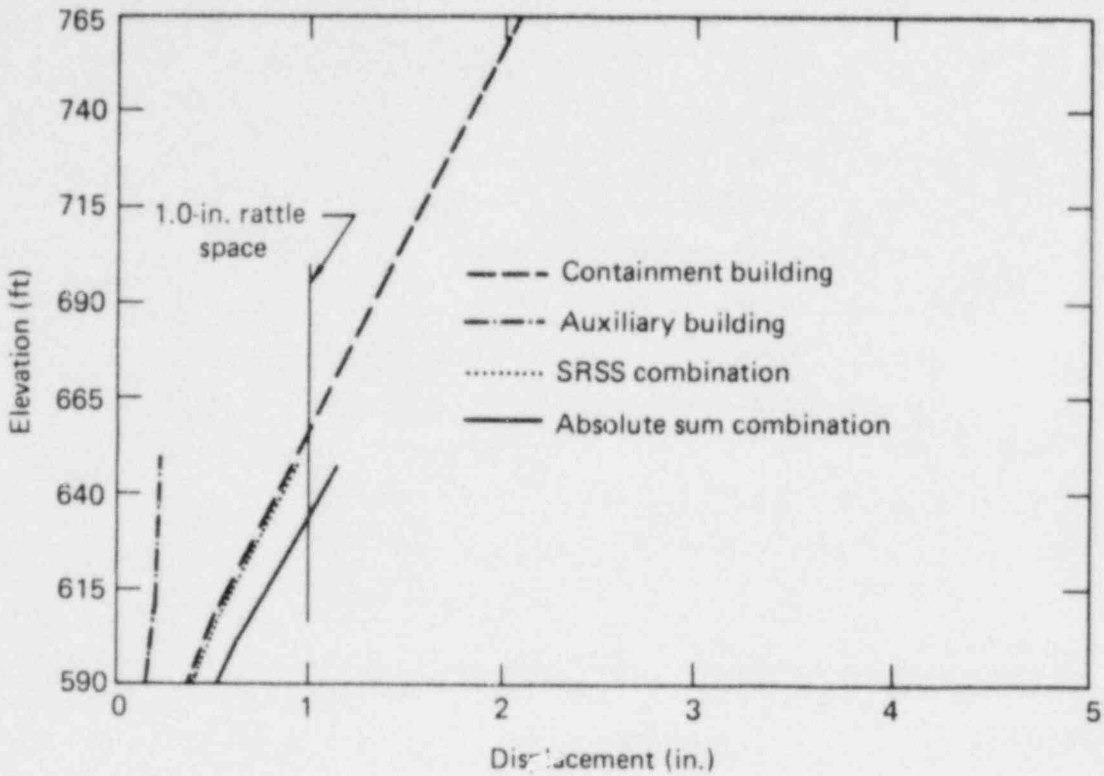


FIG. 5-16. Relative displacements of the containment and auxiliary buildings, assuming soft-soil conditions.

compressive forces in the tank shell were evaluated, and attachment bolts and tank side-wall buckling were checked. If the number of hold-down bolts was determined to be insufficient to ensure linear behavior, then a nonlinear stress distribution, as shown in Fig. 5-17b, was assumed. In the latter case, the hold-down force resisting uplift is provided by the weight of the tank fluid on the base plate, as well as by the yielded anchor bolts.

The T-2 CST and T-58 SIRWT were both found to have adequate shell thickness and a sufficient number of anchor bolts to ensure a linear response. However, the T-81 PWSMST does not have sufficient anchor bolt capacity to ensure linear behavior. During a 0.2-g SSE, the T-81 tank is expected to undergo substantial yielding and deformation of both the tank and the anchor bolts. Since the tank is constructed of A36 steel, with double fillet welds at the wall-base junction, ductile behavior of the tank is expected. The bolts have adequate shear capacity and the attachment brackets

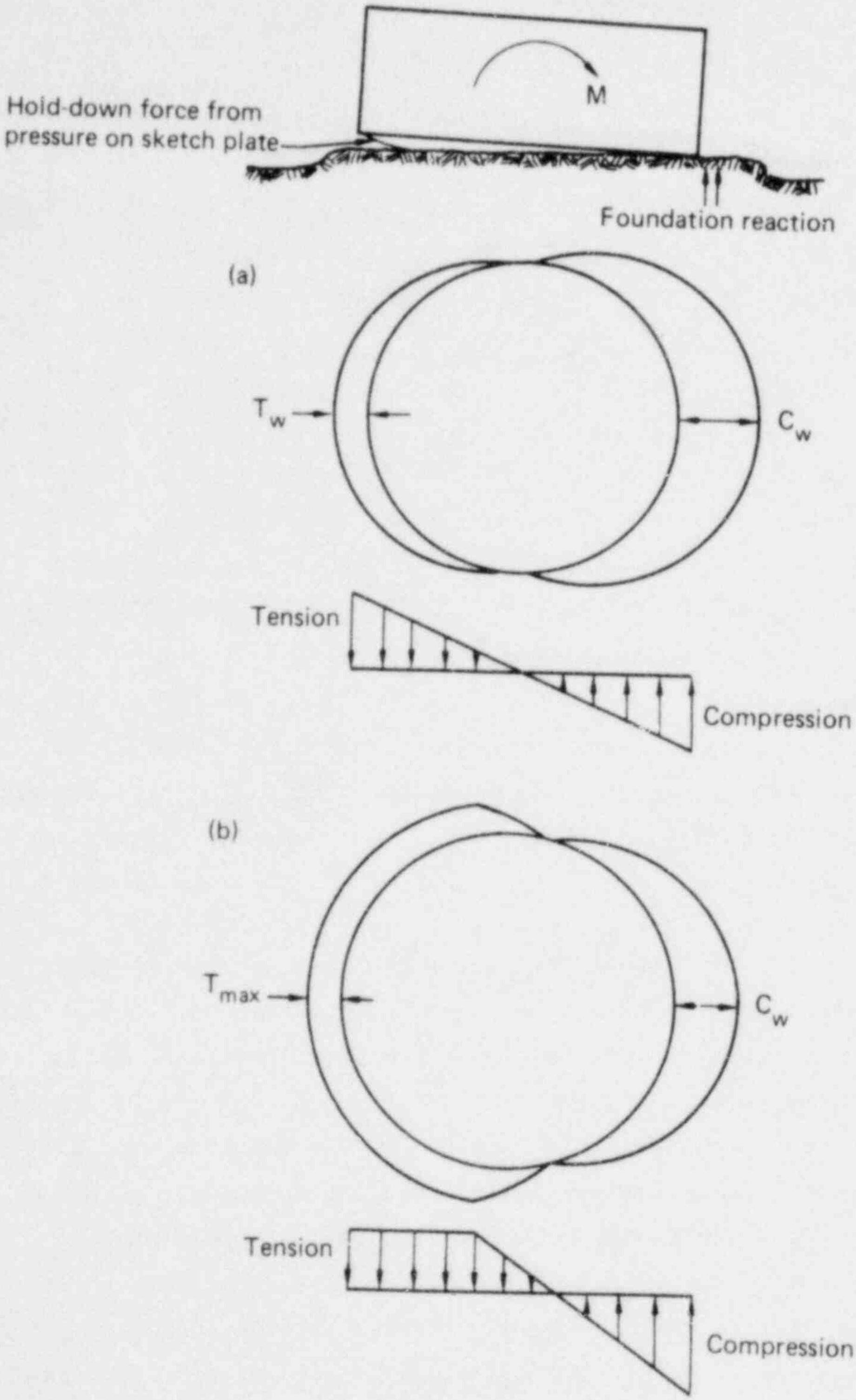


FIG. 5-17. Linear (a) and nonlinear (b) tank wall force distributions resulting from seismic overturning moment M. T_w and C_w signify longitudinal wall tension and compression, respectively.

are more than adequate to develop the anchor bolt ultimate strength. However, the anchor bolt stresses for the T-81 tank are well above yield, as well as above the ASME Code allowable for the faulted condition (70% of the ultimate strength). For the A307 bolt material, the strain hardening across the thread stress area is sufficient to develop some yielding in the bolt shank before failure. Tank anchor bolts are not subjected to load reversals, so the bolts may not fracture. Nevertheless, bolt failure is possible, and since tank buckling with subsequent loss of function is expected upon anchor bolt failure, increased anchor capacity should be installed.

Stresses in the anchor bolts of the T-2 and T-58 tanks are acceptable. Stresses that might cause pullout of the anchor bolts and stresses in the concrete ring girders for all three tanks were evaluated and found to be adequately low. The moment capacities of the ring girders were found to be sufficient, and the soil was found to have a large safety factor against bearing capacity failure for all tanks. Frictional forces between the tanks and the underlying footings were high enough to preclude any relative displacement between the tanks and ground. Hoop stresses were evaluated by combining absolutely the tank hydrostatic pressures with the SRSS of the pressures induced by the impulsive, convective, and vertical modes. All three tanks were found to have shell thicknesses sufficient to prevent yielding. Thus, with the exception of the possible failure of the T-81 anchor bolts, the critical elements of all three tanks are considered adequate to withstand the 0.2-g SSE. It is recommended that modifications be implemented to increase the anchor capacity of the T-81 PWSMST.

5.5.5 Underground Piping

In the evaluation of the adequacy of the buried pipelines at Palisades, the auxiliary feedwater line--a typical pipeline--was analyzed. To account for possibly higher stresses in other lines with somewhat different configurations and stress allowables, conservative assumptions were made throughout the analysis. For instance, the assumed soil strain of 2×10^{-4} in./in. is higher than would be expected from a detailed geotechnic investigation of the Palisades soil conditions for the 0.2-g SSE. Also, stresses were typically compared with ASME Code allowables. No stresses above code-allowable values were computed for the auxiliary feedwater line.

Maximum axial and shear stresses of 27 and 15.7 ksi, respectively, were calculated in the pipe due to seismic wave propagation. These values do not account for the effects of discontinuities or end-point motion.[32,33] The resulting principal stress of 34.2 ksi may be compared with the ASME Code allowable of 44.9 ksi for the faulted condition.

When a buried pipe changes direction abruptly, as in the case of the auxiliary feedwater line (Fig. 5-14), 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 the soil surrounding the transverse leg, which in turn creates shear and bending stresses at the elbow. These stresses were calculated assuming that the transverse leg acts as a beam on an elastic foundation. The coefficient of subgrade reaction was determined from Ref. 34. Based on the calculated deformation of the pipe (relative to the soil) of 0.16 in. at this location and including the code stress-intensity factor, a maximum bending stress of 29 ksi results, together with a shear stress of 9.8 ksi. The principal stress of 29 ksi is well below the 44.9-ksi allowable.

The final location at which stresses in buried piping might be concentrated is the penetration where the pipe enters a building. Stresses here are caused by the building structure moving relative to the soil and imposing either lateral or axial displacements, or both, on the pipe. The auxiliary feedwater line was analyzed for stresses induced by the maximum lateral auxiliary building motion of 0.18 in. and a maximum lateral deformation of 0.15 in. Maximum axial, bending, and shear stresses are 12, 25.2, and 9 ksi, respectively. The principal stress of 40 ksi is less than the 44.9-ksi code allowable. It is therefore concluded that critical buried pipelines at Palisades are not expected to fail as a result of the postulated 0.2-g SSE.

5.6 SEISMIC INPUT MOTION FOR EQUIPMENT AND PIPING

Seismic input motion for piping and equipment is typically defined by means of in-structure (or floor) response spectra.[35,36] Currently, floor response spectra are usually generated by means of time-history analyses, or by direct generation using random vibration techniques that use the response of the building structure as input. Before being used for design, these spectra are normally smoothed and the peaks broadened to account for modeling and material uncertainties. Unless a parametric study of the building is conducted, the peaks of the floor response spectra are normally widened by ±15%.

Horizontal response spectra were generated for the original Palisades design analyses of the containment and auxiliary building equipment and piping, but no vertical spectra were developed for either structure. Spectra for 0.5%, 2%, and 5% of critical damping for equipment were originally developed. The original spectra for the auxiliary building are envelopes of the maximum responses from the E-W and N-S directions.

As part of the SEP reevaluation, both horizontal and vertical in-structure response spectra were generated for both the containment and the auxiliary buildings, using the new models and criteria described in Sec. 5.4. In accord with current recommendations,[4] spectra were generated for 3%, 5%, and 7% equipment damping. These values reflect the somewhat higher damping expected. Spectra were generated for each of the three soil cases considered. Smoothed and broadened envelopes encompassing the complete soil range were developed, as well as spectra for the individual soil cases. The spectra generated for the auxiliary building included the effects of torsion, although this contributes little to the response.

Plots of the the new in-structure response spectra are presented in Appendix B. Where applicable, the corresponding original design spectra are shown for comparison. In general, the new spectra exceed the original spectra at both high and low frequencies, whereas the original spectra tend to be slightly above the new spectra in the resonant range. Load ratios for equipment in the rigid range are expected to be approximately 0.71 to 0.86 for the containment building and approximately 0.71 to 0.9 for the auxiliary building. For flexible equipment with frequencies of about 4 Hz or less, load ratios as low as 0.34 for the containment building and 0.23 for the auxiliary building can occur, depending on the soil modulus used. The effects of the new floor spectra and of these load ratios on equipment and piping are discussed in Chapter 6.

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

6.1 INTRODUCTION

6.1.1 Purpose and Scope

This chapter reviews the reevaluation of selected mechanical and electrical equipment and fluid and electrical distribution systems at the Palisades Nuclear Power Plant. Based on that review, this chapter also evaluates the ability of the reactor to shut down safely and to remain in a safe shutdown condition in the event of an SSE. The SEP review team purposely selected for reevaluation those components that were expected to have a high 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 also of other seismic Category I systems, such as engineered safeguards. Thus, evaluation of these selected components establishes an estimated lower-bound seismic capability for the mechanical and electrical components and the distribution systems of the Palisades plant.

Considered in terms of seismic design adequacy, nuclear power plant equipment and distribution systems fall into two main categories--active and passive. Within each main category, equipment and systems are further categorized as rigid or flexible. As discussed in R.G. 1.48[37] and Sec. 3.9.3. of the Standard Review Plan,[3] active components are those that must perform a mechanical motion to accomplish a system safety function. For the purpose of this report, this definition is expanded to include electrical or mechanical components that are required for safe shutdown and that must move during or after a seismic event to perform their design safety function. Typically found in the active category are pumps, valves, motors and associated motor control centers, and switchgear.

Seismic design adequacy of active components, which depends upon function as well as structural integrity, may be demonstrated by either analysis or test. Testing is generally the preferred method, but because of size or weight restrictions or difficulty in monitoring function, many active components are seismically evaluated by analysis. To ensure active component function by analysis, deformations must be limited and predictable. Therefore, total stresses in such components are normally limited to the elastic linear range of 0.67 to 0.8 times the yield stress of the material, and in no case would the total stress in a component be allowed to exceed the yield stress.

Passive components considered in this report are those components that are required for safe shutdown and for which the only safety functions are to remain leak tight or to maintain structural integrity during or following the SSE. Typically found in the passive category are pressure vessels, heat exchangers, tanks, piping and other fluid distribution systems, transformers, and electrical distribution systems.

In determining seismic design adequacy by analysis, the most important distinction between active and passive components is the stress level that the component is allowed to reach in response to the SSE excitation. For passive components, higher total stress limits, ranging from yield to 0.7 times the ultimate strength of the material, are permitted by current design procedures and codes.[24]

The designation of components and distribution systems as flexible or rigid is important in developing the magnitude of seismic input for component evaluation. The seismic inertial acceleration of the equipment depends upon potential resonance with the supporting building structure, structure and equipment damping values, and equipment support elevations. Whether a component is designated as rigid or flexible may also depend on how it is supported. Many rigid components must be considered and evaluated as flexible because of their support flexibility.

For the Palisades auxiliary building and containment internal structures, in-structure response spectra are such that equipment may be considered rigid for frequencies greater than 10 Hz. The maximum floor acceleration is approximately 0.4 g (twice the SSE zero-period ground acceleration). For flexible components with fundamental frequencies less than 20 Hz, the maximum seismic inertial acceleration is approximately 2 g.

After the components were categorized as active or passive and as rigid or flexible, a representative sample from each group was evaluated to establish the seismic design margin or degree of adequacy of each group. In this way, seismic design margins for groups of similar components were established without the need for detailed reevaluations of hundreds of individual components.

Representative samples of components were selected for review by one of two methods:

- Selection based on a walk-through inspection of the Palisades facility by a team comprising NRC staff, members of the SSRT, and the authors. Based on their experience, team members selected components in each group that appeared to have a high potential degree of seismic fragility. Particular attention was paid to the components' support structures.
- Categorization of the safe shutdown components into generic groups such as horizontal tanks, heat exchangers, and pumps; vertical tanks, heat exchangers, and pumps; motor control centers and motors.

The licensee was asked to provide seismic qualification data on the selected components.

The rest of this chapter reviews the seismic capacity of the selected components and recommends, if necessary, additional analysis or hardware changes to qualify them for the 0.2-g SSE. Based on the detailed review of the seismic design adequacy of these representative components, conclusions are drawn as to the overall seismic design adequacy of seismic Category I equipment installed at Palisades.

6.1.2 Description of Selected Components

Table 6-1 lists and describes the components selected by the team on the basis of its plant walk-through--components that are representative of the listed generic groups of safety-related components. Table 6-1 also gives the basis for each selection.

The review in this chapter emphasizes what are normally listed as auxiliary components. Such components are typically supplied by manufacturers who--unlike the NSSS vendors--may not have routinely designed and fabricated components for the nuclear power industry, particularly during the time this

TABLE 6-1. Mechanical and electrical components selected for seismic evaluation and the basis for selection.

Item No.	Description	Reason for selection
	Mechanical Components	
1	Essential service water pump	Has a long, vertical, unsupported intake section which was originally statically analyzed for seismic effects.
2	Auxiliary feedwater pumps	Represents a horizontal component which is rigidly connected to a foundation mat.
3	Component cooling heat exchangers	Unique component in that one heat exchanger is stacked on top of the other and connected to it by two saddle supports. Concern was expressed about the heat exchangers' ability to withstand overturning effects in the transverse direction.
4	Component cooling surge tank	A column-supported vertical tank.
5	Diesel generator oil tanks	Anchor-bolt system for flexible in-structure flat-bottom tanks may be overstressed if tank and fluid contents were assumed rigid in the original analysis.
6	Boric acid storage tank	A column-supported vertical tank with bracing.
7	Hydrazine tank	A tall, column-supported vertical tank. Concern was expressed about overturning effects.
8	Sodium hydroxide tank	A horizontally mounted component supported by two saddles that do not appear to be seismically restrained. Concern was expressed about the saddles' ability to carry required seismic loads, particularly in the longitudinal direction.

continued

TABLE 6-1 continued.

Item No.	Description	Reason for selection
9	Safety injection tank	Supported by a truss structure which is mounted at the top of the containment building. Concern was expressed about the increased acceleration values that the tanks will experience at that elevation.
10	Motor-operated valves	A general concern with respect to externally unsupported, motor-operated valves, particularly for lines 4 in. or less in diameter, is that the relatively large eccentric mass of the motor will cause excessive stress in the attached piping.
11	Control rod drive mechanism	Particularly critical to ensure reactor coolant system integrity.
12	Pressurizer	Same as item 11.
13	Steam generators	Same as item 11.
14	Reactor coolant pumps	Same as item 11.
15	Reactor vessel	Same as item 11.
	Electrical Components	
16	Battery racks	Bracing required to develop lateral load capacity may not be sufficient to carry the seismic load.
17	Motor control centers	Typical seismically qualified electrical equipment. Functional design adequacy may not have been demonstrated. In addition, anchorage to floor structure may not be adequate.
18	Switchgear	Same as item 17.

continued

TABLE 6-1 continued.

Item No.	Description	Reason for selection
19	Control room electrical panels	Appear adequately anchored at the base; however, there appear to be many components cantilevered from the front panel. The lack of front panel stiffness may permit significant seismic response of the panel, resulting in high acceleration of the attached components.
20	Transformers	Same as item 17.
21	Electrical cable raceways	Cable tray support system does not appear to have positive lateral restraint and load-carrying capacity.

plant was under construction. Therefore, if there is a reduction in seismic design adequacy, it would tend to be found in the auxiliary equipment, rather than in the major nuclear components. However, because of its importance to safety, the seismic design adequacy of the reactor coolant system components and support structures, to the extent information has been provided, is also evaluated in this report. In addition, portions of four piping systems were analyzed. The results of these analyses will be reported independently,[38] but a preliminary summary is provided in Sec. 6.4.

6.2 SEISMIC INPUT AND ANALYTICAL PROCEDURES

6.2.1 Original Seismic Input and Behavior Criteria

For seismic Category I mechanical equipment, all components and systems originally classified as Class 1 were designed in accordance with the criteria for load combinations and stresses listed in Table 6-3 under "Other Class 1 systems and equipment."

The manual Nuclear Reactors and Earthquakes,[39] was used as the basic design guide for seismic analysis. Class 1 equipment and their supports were analyzed for accelerations at least equal to the acceleration of the floor on which they are supported. The seismic loads were applied to the equipment centers of gravity.

The reactor protective system and nuclear instrumentation were specified to operate throughout a disturbance equivalent to a horizontal acceleration of 0.8 g. The seismic design criteria for the safety-related equipment controls and the emergency electric power systems were such that all controlling devices and systems would withstand the seismic disturbance without malfunction or improper action. Furthermore, it was specified that a seismic disturbance would not affect the operation of safety systems either momentarily or permanently. Some relays, breakers, emergency generators, and various other controlling devices similar to the type used in the Palisades plant have been shop tested and have been shown capable of withstanding seismic shock loads without malfunction. However, there is no information currently available as to the level of shock loads sustained, nor is there information to indicate that the specified criteria have been explicitly implemented in the design of the electrical components.

For seismic Category I cable conduits, the flexibility of the cables and the supporting trays for above-ground cables was designed to accommodate differential seismic motions inside and outside the structure. Cables leaving structures below ground are placed in plastic conduits which are subsequently encased in concrete for mechanical protection.

Appendages (small masses elastically attached to large masses) were not considered dynamically coupled in the seismic analysis of large masses, because their inertial forces were assumed to be too small to affect the behavior of large masses. Their weights, however, were included in those of the large masses. Class 1 appendages to piping and equipment (valve operators, for example) were analyzed statically to evaluate their effect on the piping and equipment.

6.2.2 Seismic Input for SEP Reassessment

Seismic input requirements for determining the seismic design adequacy of mechanical and electrical equipment and distribution systems are normally based on floor or equipment response spectra for the various elevations at which the equipment is supported. These in-structure spectra, which are based on R.G. 1.60 spectra modified by the dynamic characteristics of the building, are shown in Appendix B. The in-structure spectra are based on the building models shown in Figs. 5-3 and 5-7.

For mechanical and electrical equipment in general, a composite 7% equipment damping, as suggested in Sec. 5.3.1, is used in the evaluation for the 0.2-g SSE. For piping evaluations, the equipment damping associated with the SSE is limited to 3%. These damping ratios are also consistent with a recent summary of data presented to define damping as a function of stress level.[40] For cable trays, recent tests seem to indicate that the damping ratios to be used in design depend greatly on the tray and support construction and the manner in which the cables are placed in the trays. Damping may be as high as 20% of critical damping.[41] Horizontal seismic input loads have been assumed in this evaluation to be simultaneously applied independent components. Depending on the geometry of the component being evaluated, the resultant horizontal load varies from 1.0 to 1.4 times the individual component load. Except where design adequacy is in question, we have conservatively applied the 1.4 factor to the check evaluations performed in this reanalysis.

6.2.3 SEP Acceptance Criteria

Seismic Category I components that are designed to remain leak tight or to retain structural integrity in the event of an SSE are now typically designed to ASME Section III Code, Class 1, 2, or 3 stress limits for service condition D. The stress limits for supports for ASME leak-tight components are shown in Appendix F or Appendix XVII to the ASME Section III Code.[24]

When qualified by analysis, active ASME Section III components that must perform a mechanical motion to accomplish their safety functions typically must meet ASME Section III Code, Class 1, 2, or 3 stress limits for service condition B. (Recent increases to ASME Section III, Class 1, service level B limits have not been considered.) Supports for these components are also typically restricted to service condition B limits.

For other equipment, which is not designed to ASME Section III Code requirements, and for which the design, material, fabrication, and examination requirements are typically less rigorous than ASME Section III Code requirements, the allowable stresses are limited to yield values for passive components and to the normal working stress (typically 0.5 to 0.67 times yield) for active components. The SEP acceptance criteria used to evaluate various equipment and distribution systems for the Palisades passive components are given in Table 6-2. For active electrical components such as switches and relays, functional adequacy should be demonstrated by test.

Experience in the design of such pressure-retaining components as vessels, pumps, and valves (designed to meet ASME Section III Code requirements at 0.2 g) indicates that, except at the supports and nozzels, stresses induced by earthquakes seldom exceed 10% of the dead weight and pressure-induced stresses in the component body.[42] Therefore, design adequacy of such equipment is seldom dictated by seismic design considerations.

Seismically induced stresses in nonpressurized mechanical and electrical equipment, in fluid and electrical distribution systems, and in all component supports may be significant in determining design adequacy. However, because of the more restrictive stress and damping limits, the OBE rather than the SSE normally controls the design of piping systems.

6.3 EVALUATION OF SELECTED COMPONENTS

6.3.1 Mechanical Equipment

6.3.1.1 Essential Service Water Pump

The essential service water (ESW) pump and motor unit is oriented vertically in the intake structure and supported at El 590 ft. As shown on Layne & Bowler, Inc., drawing 950X9 SH-1, the intake portion of the pump extends downward from the discharge head and pump base for 37 ft 10 in. The seismic analysis, as given by Layne & Bowler, Inc.,[43] was performed for simultaneous equivalent-static loads of 0.90 g acting in the horizontal direction and 0.14 g acting in the vertical direction.

The pump and motor unit is located at grade; therefore, the seismic input is essentially the R.G. 1.60 ground response spectrum. However, to be conservative, the response spectra for the auxiliary building base slab was used in the analysis (see Figs. B-11 and B-12). Overturning tensile and shear stresses in the pump base anchor bolts were determined, as well as stresses at the attachment of the intake column pipe to the discharge head.

TABLE 6-2 SEP acceptance criteria for determining seismic design adequacy of passive mechanical and electrical equipment and distribution systems.

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

Because the intake portion of the pump is oriented vertically as a cantilever beam, the dynamic characteristic of the intake suction pipe was determined. It was found to have a fundamental frequency of 1.0 Hz, assuming a weight distribution that includes the shaft and contained water, as shown in Ref. 44. At this natural frequency, the spectral acceleration for 7% damping is 0.32 g.

The seismic accelerations were applied to the pump, and the resulting anchor bolt stresses were determined, considering simultaneous N-S and E-W loading. The effect of attached piping nozzle loads was not considered, since they were not available. (They are generally not significant in determining the overall adequacy of the pump body and support system.) Based on the ASME condition D stress limits, the analysis established a factor of safety of 2.04 for the assumed A307 anchor bolts. The stress calculated at the attachment of the discharge head to the intake column pipe was 10.8 ksi, which is well below the yield stress of 35.0 ksi, which is given in the original seismic design calculations for the column pipe. Therefore, we believe that the ESW pump will withstand a 0.2-g SSE seismic event without loss of structural integrity, provided the discharge head stresses are within code allowables. Insufficient information was provided to determine the stresses and material used in the discharge head; thus, allowable stresses there are unknown. Also, too few details were available to evaluate the functional adequacy of the pump in terms of motor impeller shaft deformations or bearing failure.

6.3.1.2 Auxiliary Feedwater Pumps

The auxiliary feedwater pumps are horizontal components supported on concrete pedestals and a mat foundation that is located at El 571 ft in the turbine building. The components consist of two pumps, one motor driven and one turbine driven. The motor-driven pump was supplied by the Bingham Pump Company and is shown on drawing FD-270663; the turbine-driven pump was supplied by the Elliot Company and is shown on drawing 602464. The original seismic design,[45] considered an SSE seismic load resulting from a 0.20-g horizontal acceleration and a 0.14-g vertical acceleration, acting simultaneously.

Since response spectra for the turbine building are not available, the response spectra considered applicable for verifying seismic design adequacy are those calculated for the auxiliary building at El 590 ft (see Figs. B-11, B-12, and B-19). The corresponding floor acceleration values for the N-S, E-W, and vertical directions are 0.32, 0.34, and 0.28 g, respectively. The seismic accelerations were applied simultaneously to the pumps, and the resulting mounting-bolt stresses were determined for three separate attachment locations: the base plate mounting, the pump mounting, and the turbine mounting. Based on ASME condition D stress limits for the assumed A307 bolts, the factors of safety are 11.6, 5.76, and 4.70, respectively.[46] In the analysis, nozzle loads due to the attached piping were neglected, since they were not available. However, considering the relatively large safety factors, nozzle loads would not be expected to control design. We believe that the auxiliary feedwater pumps will withstand a 0.2-g SSE seismic event without loss of structural integrity; however, due to the lack of design detail, no attempt was made to evaluate the functional adequacy of the pumps.

6.3.1.3 Component Cooling Heat Exchangers

The component cooling heat exchangers (CCHXs) are two horizontal heat exchangers located in the auxiliary building, one stacked on top of the other and supported there by two saddles. The pair is supported on the floor by four saddles at El 590 ft. Three of the four floor saddles are slotted in the longitudinal direction to permit thermal expansion. The heat exchangers are shown on Industrial Process Engineers drawing F-5628-3. The response spectra for 7% damping (see Figs. B-11 and B-12) are considered applicable for verifying seismic design adequacy.

The seismic qualification of the CCHXs was performed as described in Ref. 47. The seismic evaluation determined the dynamic response characteristics of the exchangers and their saddle support system. The evaluation indicated that the system is relatively rigid and has no response frequencies below 33 Hz. As a result, horizontal seismic input accelerations in orthogonal directions are 0.34 and 0.32 g, respectively, corresponding to the 0.2-g SSE.

The seismic accelerations were applied simultaneously to the heat exchangers, and the resulting anchor bolt and support saddle stresses were determined. The analysis established factors of safety of 2.35 for the anchor bolts and 16.5 for the support saddle, based on ASME III-1 condition D stress limits .

In addition to evaluating the CCHX saddle and anchor bolt support system, the seismic stresses induced in the tubes of the heat exchanger were determined, combined with other applicable loads, and compared to code allowables.[47] The factors of safety determined for the heat exchanger tube was 40.8; for the heat exchanger shell, it was 2.13. Both were controlled by hoop stresses due to internal pressure rather than seismic stresses. It should be noted that no evaluation was made of nozzle loads in the heat exchanger, since they were determined from the attached piping system analysis that was not available for evaluation. It has been generally found that such piping loads, which can be a limiting load to the nozzle, seldom have a significant effect on the heat exchanger support loads.

In conclusion, we believe that the CCHXs will withstand a 0.2-g SSE seismic event without loss of structural integrity or function. Our conclusion is based on

- Evaluation of the dynamic characteristics of the heat exchanger support system and the supplemental analysis given in Ref. 47.
- Experience in reviewing similar saddle-supported heat exchangers.

6.3.1.4 Component Cooling Surge Tank

The component cooling surge tank is a column-supported component located in the auxiliary building at El 649 ft. The surge tank is shown on Niles Steel Tank Company drawing 5935-M38-A. The response spectra for 7% damping (Appendix B) are considered applicable for verifying seismic design adequacy. The seismic qualification of the surge tank was originally performed for a 0.30-g horizontal acceleration and a 0.14-g vertical accelera , applied simultaneously.

We have reviewed the tank and support system to determine seismic design adequacy.[48] The dynamic analysis considered the effective impulsive and convective response of the contained fluid and determined fundamental response frequencies for the tank: 0.71 Hz for convective loading and 7.89 Hz for the tank and support system. For the convective mode, a damping value of 0.5% was used; for the impulsive mode, 7% damping was used. The analysis determined gross dynamic characteristics of the tank and established minimum factors of safety of 10.0 for compressive stresses in the tank legs and 9.43 for combined stresses in the anchor bolts. As in the case of other components with attached piping, we did not evaluate nozzle capacities, since piping loads were not available. We believe that the component cooling surge tank will withstand the 0.2-g SSE without loss of structural integrity or function, based on

- Check of the dynamic characteristics of the tank and an evaluation of support leg and anchor bolt stresses.
- Experience in reviewing similar tanks.

6.3.1.5 Diesel Generator Oil Storage Tanks

The diesel generator oil storage tanks were not evaluated, since no drawings or design calculations were available.

6.3.1.6 Boric Acid Storage Tank

The boric acid storage tank is a column-supported tank with cross bracing, as shown on Nooter Corporation drawing JN-D-31011. The seismic qualification of the tank and support system was performed as described in Ref. 49. We have reviewed the tank and support system and its anchors to determine seismic design adequacy.[50] The tank, which is supported at El 590 ft in the auxiliary building, was evaluated for the corresponding floor response spectra from Appendix B. The dynamic analysis considered the effective impulsive and convective response of the contained fluid and determined the fundamental response frequencies for the tank: 0.56 Hz for convective loading and 18.8 Hz for the tank and support system horizontal mode (including impulsive loading). For the convective mode, a damping value of 0.5% was used, and for the impulsive mode, 7% damping was used. The analysis determined gross dynamic characteristics of the tank and established minimum factors of safety of 5.64 for compressive stresses in the support legs and 50.0 for combined stresses in the anchor bolts, in accordance with ASME III-2 condition D stress limits. Again, we did not evaluate nozzle capacities, since piping loads were not available. We believe that the boric acid storage tank will withstand the 0.2-g SSE without loss of structural integrity or function, based on

- Review of the stress analysis of the tank support supplied by the licensee.
- Check of the dynamic characteristics of the tank and an evaluation of the tank and support system and anchor bolt stresses performed in connection with this report.
- Experience in review of similar tanks.

6.3.1.7 Hydrazine Tank

The hydrazine tank,[51] as shown on Buffalo Tank Company drawings SK-1031-2 and SK-1031-4, is a tall, column-supported vertical vessel (12 ft long and 30 in. in diameter). The tank is supported at El 640 ft of the auxiliary building and was evaluated for the corresponding floor response spectra, shown in Figs. B-15, B-17, and B-19.

The dynamic analysis considered the effective impulsive and convective responses of the fluid and tank system.[52] The fundamental sloshing (convective) frequency was found to be 1.10 Hz, and the fundamental tank and support system frequency was found to be 27.1 Hz. For the convective and impulsive modes, the analysis used damping values of 0.5% and 7%, respectively. The analysis determined gross dynamic characteristics of the tank and established minimum factors of safety of 14.1 for compressive stresses in the tank legs and (assuming ASME III-2 condition D stress limits) 7.14 in the anchorage system,. We did not evaluate nozzle capacities, since piping loads were not available. We believe that the hydrazine tank will withstand the 0.2-g SSE without loss of structural integrity, based on

- Check of the dynamic characteristics of the tanks and evaluation of support leg and anchor bolt stresses.
- Experience in reviewing similar tanks.

6.3.1.8 Sodium Hydroxide Tank

The sodium hydroxide tank is a horizontal vessel located on the auxiliary building and supported by two saddle supports at El 640 ft. The tank is shown on Buffalo Tank drawing SK-M-1054. The response spectra for 7% damping (see Figs. B-15, B-17, and B-19) are considered applicable for verifying seismic design adequacy.

The seismic qualification of the sodium hydroxide tank was performed as described in Ref. 53. This analysis was reviewed, and an independent evaluation of the dynamic response characteristics of the heat exchanger and its saddle support system was made. The review indicates that the system is relatively rigid and has no response frequencies below 17.8 Hz. As a result, horizontal seismic input accelerations in orthogonal directions were determined to be 0.49 and 0.42 g, corresponding to the 0.2-g SSE.

The seismic accelerations were applied simultaneously to the sodium hydroxide tank, and the resulting support saddle stresses and anchor bolt stresses were determined. The analysis established factors of safety of 3.52 for the support saddles and (assuming ASME III-2 condition D stress limits) 15.9 for the anchor bolts. Therefore, we believe that the sodium hydroxide tank will withstand a 0.2-g SSE seismic event without loss of structural integrity, based on

- Review of the analysis in Ref. 53.
- Evaluation of the dynamic characteristics of the tank and support system and the supplemental analysis given in Ref. 54.
- Experience in reviewing similar saddle-supported tanks.

6.3.1.9 Safety Injection Tank

The safety injection tank is a tall vertical vessel, 32 ft 2 in. in length and 9 ft 0 in. in diameter. The tank is shown on Nooter Corporation drawing F-6171, and the support system is shown on Bechtel Corporation drawing C-246. The support system for the tanks consists of a series of trusses which are supported by the containment structure near the springline and connected together at the dome centerline. The tanks are connected to the support trusses by means of a structural framework and vertical hanger members. The tanks were originally designed assuming a horizontal acceleration of 1.5 g and a vertical acceleration of 0.2 g, applied simultaneously to the center of gravity of the vessel.[55,56]

The tank, assumed to be full of water and with its truss and hanger support system assumed rigid, was reevaluated dynamically as shown in Ref. 57

and found to have a fundamental frequency of 25 Hz. Based on this frequency and the in-structure response spectra determined at El 730 ft (Figs. B-3 and B-9), a resultant response spectrum floor acceleration of 0.6 g for 7% damping was obtained. The SRSS value for horizontal acceleration was then 0.85 g, which is less than the original design value of 1.5 g. For the vertical direction, an acceleration value of 0.34 g was obtained from the rigid end of the vertical response spectrum. This represents a load ratio of 1.34/1.20 = 1.117, an 11.7% increase in dead weight plus seismic load. With this revised seismic load, the tank and support trusses and structural framework will remain within ASME III-2 condition D stress limits, given that the tank and support system is rigid.

However, the tank truss and structural framework support system may not be rigid. This system is highly complex and the determination of its frequency characteristics is beyond the scope of this report. If the support system is flexible, such that the tank is in the near-resonant range, the horizontal response acceleration would be approximately 4.5 g and the vertical response 1.5 g. This would result in seismic loads on the support system three times that considered in the horizontal design and twice that considered vertically. As a result, ASME condition D stress limits would be exceeded. We recommend that a detailed reanalysis of the tank and its support system be performed by the licensee to determine the resultant dynamic characteristics and stresses in the system for the redefined seismic response spectra.

6.3.1.10 Motor-Operated Valves

The motor-operated valves are shown on Velan Engineering Company drawings P-33345, P-33345-4, and P33345-3 and Philadelphia Gear Corporation drawings 02-405-0039 and 02-405-0085-4. The response spectra considered applicable for the motor-operated valves are those for the auxiliary building and containment building internal structure at 3% damping (see Appendix B).

It has been our experience that, for lines 4 in. in diameter and smaller, the eccentricity of motor-operated valves may cause additional significant piping stresses (in excess of 10% of code allowable) that should be considered in the computation of total stresses. The applicable stress levels are specified by Class 2, condition B, for active valves and by condition D when only pressure boundary integrity is required. The stresses induced by valve eccentricity increases as the line size decreases.

Calculations performed on randomly selected motor-operated valves (2 in., 3 in., and 4 in. in diameter) installed in the Palisades Nuclear Power Plant demonstrate that the stress levels reached are well in excess of the above-mentioned 10%, regardless of service condition.

For a typical ferritic piping material (S_h = 15,000 psi), the condition B and D stress limits would be 18,000 and 36,000 psi, respectively. Preliminary calculations[58] indicate that the stress levels shown in Table 6-3 would be reached in the pipe if a peak acceleration of 1.5 g were applied to the valves. Based on these values, it is recommended that the licensee evaluate the stresses induced from motor-operated valves in supporting pipe 4 in. in diameter and smaller. The licensee should show that stresses induced in the piping by these valves are less than 10% of the pertinent service condition allowable stresses. Otherwise, the total stresses at motor-operated valve locations should be calculated to determine if they are within the established allowables. Alternatively, we recommend that a requirement to support the valve operators externally be developed and implemented. In addition, the licensee should provide an evaluation in the form of either test or analytical results which demonstrate the functional adequacy of the valves.

TABLE 6-3. Stress levels induced in supporting pipes by motor-operated valves.

Pipe diam., in.	Stress, psi	% of condition B	% of condition D
4	8,200	45%	23%
3	15,300	85%	43%
2	20,800	116%	58%

6.3.1.11 Control Rod Drive Mechanism

A summary of the original stress analysis of the control rod drive mechanism (CRDM) is given in Ref. 59. The mechanism and seismic support are shown on Combustion Engineering drawings 2966-E-2869, 2966-SE-2554, and 2966-SE-2557. A static seismic analysis of the drive mechanism was performed to determine if the seismic support allowables were exceeded. A static horizontal SSE seismic load of 1.35 g was applied to the mechanism, and a stress evaluation of the various parts of the seismic support was performed. The stress evaluation was based upon a moment restraint placed 51 in. below the center of gravity of the mechanism.

The response spectra for the CRDM, which correspond to the reactor vessel support elevation, are given in Figs. B-1 and B-3. Since the fundamental frequency of the drive mechanism is 8.3 Hz, the peak acceleration in the two horizontal directions is 1.06 g for 2% damping. Therefore, the resultant horizontal acceleration is 1.49 g, and the ratio to the original design value of 1.35 g is 1.10. If the original CRDM seismic stresses are multiplied by 1.10, the resulting stress values are less than the allowable stress values, except for the tension stress in the plate bolts. For the plate bolts, the revised tension stress is 183 ksi, as compared to an original allowable stress of 176 ksi. The latter value was based on 110% of yield. Furthermore, there are two sources of additional margin not accounted for in the evaluation:

- Two percent damping of the CRDM was assumed, whereas Combustion Engineering testing has reportedly indicated that higher damping could be justified.
- Nonlinearities and friction in the seismic support hardware were neglected; these effects would tend to further reduce the predicted response.

We believe that the CRDM will withstand the 0.2-g seismic load without loss of structural integrity. However, since some resultant stresses exceed yield, as well as current ASME condition B stress limits, the active function of the mechanism cannot be assured, based on the reviewed calculations.

6.3.1.12 Pressurizer

The pressurizer is a vertical cylindrical vessel with a skirt-type support attached to the lower head. The lower part of the skirt terminates in a bolting flange with sixteen 2-in. bolts, which secure the vessel to its foundation. A summary of the stress analysis of the pressurizer is given in Ref. 60. In 1967 a seismic analysis of the pressurizer shell and internal tubes, support skirt, shock lugs, and pressurizer support bolts was performed. The SSE evaluation assumed simultaneous horizontal and vertical accelerations of 0.20 g. These accelerations were applied statically at the center of gravity of the pressurizer model.

Since the pressurizer is at El 626 ft of the internal structure, the response spectra which correspond to El 616 ft and El 649 ft of the internal structure are considered applicable (Figs. B-5 and B-7). The fundamental frequency of the pressurizer is 18 Hz,[61] which indicates a spectral acceleration of 0.32 g (7% damping) for the horizontal directions. For the vertical direction (Fig. B-3), the spectral acceleration is 0.34 g. Therefore, the resultant horizontal acceleration is 0.45 g, 2.25 times the original design value of 0.20 g. For the vertical direction, the ratio of the revised acceleration to the original design acceleration value of 0.20 g is 1.70.

For the pressurizer heater-tube assemblies, the currently predicted maximum bending stresses are 2.25 times the results of the analysis given in Ref. 60, but the stresses are small and well below the ASME III-2 condition D stress values. For the support skirt, the currently predicted axial stress is 0.70 ksi, and the bending stress is 1.27 ksi; again, both are small and well within the ASME III-1 condition D limits. The pressurizer support lugs were designed for loads due to pipe rupture plus the SSE. Since the pipe rupture loads control the design and are much larger than the SSE loads, we believe that an increase in seismic loads will not affect the design of the shock lugs. It should be noted that the design loads given are for pipe rupture plus SSE; no individual loads were given. The original stresses in the support bolts due to overturning moment effects were multiplied by the ratio of 2.25; the resulting stress was 18.6 ksi, which is less than the original allowable value of 55 ksi.

Based upon review of the Combustion Engineering calculations and independent evaluation, we believe that the pressurizer support system will withstand the 0.2-g SSE seismic event without loss of structural integrity. Combination of SSE with LOCA loads was not evaluated.

6.3.1.13 Steam Generators

The steam generators are vertical cylindrical vessels, supported by the internal structure at El 615 ft 1 in. and laterally restrained at the operating deck (El 649 ft 0 in.). A summary of the stress analysis of the steam generator is given in Ref. 62. The original seismic design specified 0.2 g in both horizontal and vertical directions, applied simultaneously.

The response spectra for the steam generators, which correspond to El 649 ft of the internal structure, are given in Figs. B-3 and B-7. The fundamental frequency of the steam generator was not given but was assumed to be 10 Hz or greater; therefore, the corresponding spectral accelerations are 0.64 g for the horizontal direction and 0.49 g for the vertical direction. The resultant value is 0.90 g for the horizontal direction, and the ratio of this acceleration to the original design value of 0.20 g is approximately 4.50. For the vertical direction, the ratio to the original design acceleration value of 0.20 g is 2.45.

The steam generator supports were originally designed to withstand a load combination which includes pipe rupture loads in addition to seismic loads. A comparison of the maximum forces and moments for the support structures is given in Table 6-4. In addition, a comparison of the primary coolant nozzle

TABLE 6-4. Forces and moments in the steam generator supports caused by loads due to the SSE and a main coolant pipe rupture.

Support component	SSE	Pipe break
Support skirt:		
Horizontal force	0.3×10^6 lb	3.0×10^6 lb
Vertical force	1.4×10^6 lb	3.0×10^6 lb
Moment about horizontal axis	5.0×10^6 ft-lb	12.0×10^6 ft-lb
Moment about vertical axis	0	12.0×10^6 ft-lb
Upper support key:		
Force per key	0.12×10^6 lb	0.3×10^6 lb
Upper support snubber:		
Force per snubber	0.1×10^6 lb	0.2×10^6 lb

loads for maximum seismic loads and pipe rupture loads is given in Table 6-5. This table compares design loads, but a comparison of allowable forces or stresses for the various load combinations is not available. Likewise, no information concerning pipe nozzle loads, tubing lateral supports, or tubes and tube sheets was supplied. Because this information was not provided by the licensee, we do not feel we can comment on the design adequacy of the steam generator and supports.

TABLE 6-5. Forces and moments in the primary coolant inlet and outlet nozzle due to seismic and pipe rupture loads.

	Force, 10^6 lb			Moment, 10^7 in.-lb		
Nozzle	F_x	F_y	F_z	M_x	M_y	M_z
			SSE load			
Inlet	± 2.0	± 0.4	± 0.8	± 2.0	± 0.2	± 2.0
Outlet	± 0.12	± 0.02	± 1.2	± 1.2	± 0.4	± 1.6
			Pipe rupture load			
Inlet	-3.46	+1.98	--	--	--	-4.85
Outlet	-1.45	± 1.77	+1.35	+7.35	+7.35	+7.35

6.3.1.14 Reactor Coolant Pumps

The reactor coolant pumps are vertical components supported by the internal structure at El 608 ft 6 in. A summary of the stress analysis of the reactor coolant pumps is given in Ref. 63. The original seismic design specified 0.55 g in both horizontal and vertical directions, applied simultaneously.

The response spectra for the reactor coolant pumps, which correspond to El 616 ft of the internal structure, are given in Figs. B-3 and B-5. The fundamental frequency of the reactor coolant pump was not given but was assumed to be in the flexible range; therefore, the corresponding spectral acceleration is 0.9 g (7% damping) for the horizontal direction. The pump was assumed rigid in the vertical direction; the corresponding spectral acceleration is 0.34 g. The resultant horizontal acceleration is 1.27 g, 2.30 times the original design value of 0.55 g. The ratio of the revised vertical acceleration to the original design acceleration value of 0.55 g is 0.62.

The reactor coolant pump supports were originally designed to withstand a load combination which includes pipe rupture loads in addition to seismic loads. However, a comparison of seismic and pipe rupture design loads and a comparison of allowable forces or stresses for the various load combinations was not provided. Therefore, based on the limited information provided by the licensee, we cannot comment on the design adequacy of the reactor coolant pump and supports.

6.3.1.15 Reactor Vessel

The reactor vessel for the Palisades plant is supported at the nozzles (centerline El 618 ft 2-1/2 in.) by steel brackets, which are supported in turn by the primary shield wall. A summary of the stress analysis of the reactor vessel is given in Ref. 64. The original seismic design specified a 0.468-g horizontal acceleration and a 0.312-g vertical acceleration, acting simultaneously.

The response spectra for the reactor vessel, corresponding to El 616 ft of the internal structure, are given in Figs. B-3 and B-5. Assuming the reactor vessel to be rigid, the corresponding spectral acceleration is 0.28 g for the horizontal direction and 0.34 g for the vertical direction. The resultant horizontal acceleration is 0.395 g, whose ratio to the original design value of 0.468 g is 0.84. For the vertical direction, the ratio of the revised acceleration to the original design acceleration value of 0.312 g is 1.09. Based upon the above spectral acceleration ratios, it appears that the reactor vessel will withstand a 0.2-g SSE without loss of structural integrity. However, due to the limited information provided by the licensee, we do not feel we can comment on the actual design adequacy of the reactor vessel and vessel internals.

6.3.2 Electrical Equipment

The seismic qualification performed on the Palisades plant electrical equipment, as provided by the licensee in Amendment No. 15 to the FSAR, p. 7.7-1, is summarized in Table 6-6. The qualification documentation listed in the third column was not provided for this evaluation.

6.3.2.1 Battery Racks

The battery racks used for the Palisades plant were manufactured by Gould-National Batteries, Inc.[65] They appear to be similar in design to the 125-V racks installed in the Dresden 2 and Ginna stations,[1,66] except that additional diagonal bracing was added at the time of installation at the request of the architect/engineer. The floor response spectra for the auxiliary building (El 610 ft, Figs. B-13 and B-14) are assumed applicable to the battery racks. Given the rigidity of the racks, accelerations applicable to the racks are essentially the same as the floor accelerations. On this basis, we recommend that the wooden battens which now laterally restrain the batteries be strengthened or replaced so that friction between the batteries and their support rails no longer need be relied upon to carry the seismic load.

6.3.2.2 Motor Control Centers

The ac and dc motor control centers (MCCs) are located in the auxiliary building at El 607 ft. The ac MCCs were supplied by Cutler-Hammer, Inc., and are shown on drawing 94-D9801ED-837, sheets 1, 2, 3, 4, 9, and 10. The dc MCCs were supplied by Westinghouse and are shown on Westinghouse Electric Corporation drawing E13AC-950PB10, sheet 14.

The original seismic design for the ac MCCs considered a 0.25-g horizontal acceleration and a 0.14-g vertical acceleration, acting simultaneously.[67] For the dc MCCs, the values were 0.283 g (horizontal) and 0.144 g (vertical), again acting simultaneously.[68]

The response spectra considered for seismic design adequacy were those for the auxiliary building at El 610 ft (see Figs. B-13, B-14, and B-19). The peak floor accelerations for the N-S, E-W, and vertical directions are 0.38,

TABLE 6-6. Original electrical and instrumentation seismic design qualifications.[a]

Equipment	Specified	Design
Emergency generators	0.23 g horiz. 0.13 vert.	2.5 g for locomotive and marine service.
2400-V switch-gear:	0.25 g horiz. 0.14 vert.	
Breakers		3.0-g vibration test by supplier.
Relays		5.0-g vibration test and dynamic analysis by supplier.
Structure		Bechtel analysis: structure is rigid.
480-V load centers:	0.25 g horiz. 0.14 g vert.	Prototype unit shock tested by supplier at 5 g max.
Transformers Breakers Relays Structure		6 g; shock tested at more than 6 g.
Preferred ac bus, Battery chargers, inverters	0.28 g horiz.	Dynamic analysis by supplier: unit remained operable at 0.75 g.
Batteries	0.30 g horiz. 0.14 g vert.	Designed for more than 0.3 g with cell impact spacers and braces.
Battery rack	0.30 g horiz. 0.14 g vert.	Bechtel analysis: structure braced and rigid.
480-V MCC: Breakers Starting Structure	0.25 g horiz. 0.14 g vert.	Unit designed for 1.3 g for marine service (momentary interruption only).

continued

[a]All seismic Class 1 equipment supported directly on the floor levels have been analyzed statically for the floor acceleration, and the support structures, including the anchor bolt systems, have been designed to withstand the shear load and the equipment overturning moment. Where seismic Class 1 components are installed within structures, such as control panels or racks (which are supported from a concrete floor or wall), the structures have been analyzed and are rigid or restrained such that acceleration is not amplified above the specified floor-level acceleration. Component anchorages within a structure, such as instrumentation mounts, have been determined to be adequate, since (1) a conservative component mass was assumed, (2) a minimum standard anchor system was provided, and (3) it was determined that an acceleration greater than 5.0 g, which is far above the design acceleration, would be required to reach yield stress in the anchorage system.

TABLE 6-6 continued.

Equipment	Specified	Design
Main control boards	0.30 g horiz.	Bechtel analysis: structure is rigid.
Shutdown panel	0.20 g horiz. 0.13 g vert.	Bechtel analysis: structure is rigid.
Transmitters	0.30 g horiz. 0.14 g vert.	Prototype shock tested at more than 0.5 g by supplier.
Switches	0.30 g horiz. 0.14 g vert.	Prototvpe shock tested at 15 g by supplier.
Cable trays	--	Support system braced and rigid.

0.36, and 0.28 g, respectively. The peak spectral accelerations at 3% damping for the N-S and E-W directions are 1.65 and 1.57 g, respectively. The MCCs were considered flexible in the transverse direction and rigid longitudinally. Thus, typical ac and dc MCCs units were analyzed for the peak spectral acceleration of 1.65 g in the transverse direction and for the floor acceleration value of 0.36 g in the longitudinal direction. The seismic accelerations were applied simultaneously to the MCCs, and the resulting anchor bolt stresses were determined.

The analysis established factors of safety of 20.0 for the ac MCC anchor bolts and 12.0 for the dc MCC anchor bolts, when compared to ASME III-2 condition D stress limits. Therefore, we believe that the ac and dc MCCs will withstand a 0.2-g SSE seismic event without failure of the control centers' anchorage system. However, no information was supplied concerning the design adequacy of the control cabinets or the functional adequacy of the contained electrical components. Hence, additional analysis or test information is required before structural integrity and design adequacy can be assured.

6.3.2.3 Switchgear

The switchgear for the Palisades plant are located in the auxiliary building at El 590 ft and El 607 ft. They were supplied by the Allis Chalmers Mfg. Co. and are shown on Allis Chalmers drawings 18-463-546-417, 18-463-546-418, 18-463-546-419, 72-422-906, and 72-422-907.

The original seismic design for the switchgear specified a 0.25-g horizontal acceleration and a 0.14-g vertical acceleration, acting simultaneously.[69] Allis Chalmers has stated that, since the switchgear has withstood recorded input shipping shocks of 3 g (horizontal) and 1 g (vertical), the switchgear should withstand the seismic accelerations specified. This would be true if the support of the switchgear during shipment were similar to that of the installed switchgear. In general, however, this is not the case. Also, the shock during shipment gives no information as to functional adequacy during a seismic disturbance.

The response spectra considered for seismic design adequacy are those for the auxiliary building at El 610 ft (Figs. B-13, B-14, and B-19). The peak floor accelerations for the N-S, E-W, and vertical directions are 0.38, 0.36, and 0.28 g, respectively. The peak spectral accelerations at 3% damping for the N-S and E-W directions are 1.65 and 1.57 g, respectively.

The switchgear cabinet support anchorage was analyzed for the peak spectral acceleration of 1.65 g in the transverse direction and the floor acceleration value of 0.36 g in the longitudinal direction. The seismic accelerations were simultaneously applied to the switchgear, and the resulting anchor bolt stresses were determined. The analysis established a factor of safety of 1.18 for the assumed 7/8-in.-diameter A-307 bolts, based on ASME stress limits. If the bolt diameter is actually less than 7/8 in., the factor of safety would be less than one, and the design would be inadequate. Also, the number of anchor bolts per unit has been taken as four, whereas the drawings indicate the possibility of six anchor bolts per unit. Therefore, we recommend that the licensee verify the size and number of anchor bolts used to secure the switchgear to the floor. Furthermore, no analysis or test results have demonstrated the structural integrity of the switchgear racks or the functionality of the contained electrical components. Hence, additional analysis or tests are required before structural integrity and functional adequacy can be assured.

6.3.2.4 Control Room Electrical Panels

The control room electrical panels, which were supplied by the Harlo Corporation, are located in the control room at El 625 ft; their arrangement and location are shown on Bechtel Corporation drawing M-183. The original seismic design specified a 0.3-g horizontal acceleration and a 0.14-g vertical acceleration. The method used to determine design adequacy assumed that the structure is rigid.[70]

Since response spectra for El 625 ft of the auxiliary building are not available, the floor acceleration values were taken as the average of the El 610 ft and El 640 ft values (Figs. B-13, B-14, B-15, B-17, and B-19). The corresponding peak floor accelerations for the N-S, E-W, and vertical directions are 0.40, 0.39, and 0.28 g, respectively. Since these values are higher than the original spectral design values, it is recommended that the licensee verify the seismic design adequacy of the panels for a simultaneous seismic acceleration of 0.4 g in both horizontal directions and 0.3 g in the vertical direction.

6.3.2.5 Transformers

The transformers for the Palisades plant, which were supplied by the ITE Circuit Breaker Company, are located in the auxiliary building at El 607 ft and are shown on ITE drawings 33-42924-P-02, 33-42924-P-01, and 73488-B31. The original seismic design specified a 0.25-g horizontal acceleration and a 0.14-g vertical acceleration, acting simultaneously.[71]

The response spectra considered for seismic design adequacy are those for the auxiliary building at El 610 ft (Figs. B-13, B-14, and B-19). The peak floor accelerations for the N-S, E-W, and vertical directions are 0.38, 0.36, and 0.28 g, respectively. The seismic accelerations were applied simultaneously to the transformer, and the resulting anchor bolt stresses were determined. The analysis considered the center section to act separately from the end units. The resultant factor of safety, as measured against ASME III-2 condition D stress limits for the assumed A307 anchor bolts, is 22.5. For the end units, the overturning effect produces uplift, and since the end units are not tied down, we recommend that they be anchored. Insufficient information was supplied to evaluate either the structural adequacy of the framework which supports the transformers or the functional adequacy of the transformers.

6.3.2.6 Electrical Cable Raceways

Seismic loads were not considered in the original design of cable tray supports. Recent tests[41] have indicated that damping values of 20% or more may be justified for cable trays and their supports; nonetheless, an evaluation of the existing cable tray system required for safe shutdown, including supporting documentation of the design assumptions used, is required before design adequacy can be assured.

6.4 PIPING

The results of the Palisades piping analyses will be published separately.[38] This brief summary is intended only as a preliminary overview. Portions of four major piping systems were analyzed:

- Residual heat removal (RHR) system.
- Component cooling system.
- Auxiliary feedwater system (three portions were modeled, including the steam line to the P-8B turbine).
- Regenerative heat exchanger (RHE) letdown system.

Throughout, it was assumed that suitable stress analyses of the supports and substructure were performed for the original loads.

The results of the analyses performed on the RHR piping indicate that stresses in this piping will be well within allowable limits during a seismic event equivalent to the postulated SSE. However, where comparison was possible, support loads were found to be generally higher than those determined in the original analysis. Further examination of the support members subjected to significantly higher loads is warranted. In addition, the anchor loads determined in the reanalysis are generally higher than the original loads. Further consideration should be given to the nozzles since the anchor moment loading has increased significantly.

Piping stresses for the component cooling model are well within allowable limits for an SSE event. The support loads determined in the reanalysis are generally higher than the known original loads, but the increases do not appear large enough to warrant anticipation of failures. No conclusions were drawn for those supports where original loads were unknown. The anchor loads based on the SEP acceptance criteria are generally higher than the original loads. Further consideration should be given to the nozzles in cases where the loads have increased significantly.

The reanalysis results for a portion of the auxiliary feedwater piping between pumps P-8A and P-8B show that ASME Code stress limits will be exceeded during an SSE event. The overstressed points occur near pump P-8A. These high stresses result from a deficiency in east-west lateral restraint. When relief valve (RV 0783) discharge loads are considered in conjunction with the SSE loading, a large increase in the loads on support R227C is indicated. This support should be reevaluated for this increased load. Insufficient data were available for anchor load comparisons; thus, no conclusions were drawn concerning nozzle capabilities.

Analysis of a model of a second portion of the auxiliary feedwater system showed no piping stresses above Code allowables for the SSE. Insufficient data were available for comparisons of support and anchor loading. No conclusions concerning support or nozzle structural adequacy were drawn.

In the third analysis of the auxiliary feedwater system, the piping between the steam line and the P-8B turbine was modeled. Results show that ASME Code stress limits will be exceeded during an SSE event. Additional lateral and vertical dynamic supports are needed in the vicininty of the piping upstream from valve PCV 0521A. Additional vertical and lateral dynamic supports are also needed in the vicinity of valve MS 0522A. Support loads for the SSE case are also generally higher than the known original loads. Results indicate that rod hangers H11, H13, and H14 will probably buckle and become ineffective. Nonetheless, the increased support loads are not great enough to warrant anticipation of failure. No conclusions regarding support structural adequacy could be drawn in those cases where original loads were unknown. Insufficient data were available for anchor load comparison; therefore, no conclusions regarding anchor or nozzle structural adequacy were drawn.

The results for the RHE letdown piping show that ASME Code stress limits will be exceeded during an SSE event. The high stresses are primarily due to a deficiency in axial and lateral restraint near the downstream vertical leg of the expansion loop nearest valve CV 2003. Insufficient data were available for support or anchor load comparisons. No conclusions were drawn concerning support or nozzle structural adequacy.

6.5 SUMMARY AND CONCLUSIONS

Table 6-7 summarizes our findings on the sample of mechanical and electrical components and distribution systems that were evaluated to determine the seismic design adequacy of items required for the safe shutdown of the Palisades nuclear steam supply system. As discussed in Sec. 6.1, this sample includes components the review team selected, based on judgment and experience, as representative of the lower-bound seismic design capacity of Palisades. The components of the sample were also chosen to represent important generic groups.

Based upon the design review and independent calculations for the SEP seismic load condition, we recommend design modifications or reanalysis of several mechanical and electrical components to ensure that they can withstand the 0.2-g SSE without loss of structural integrity as required to perform safety functions. In general, no information was provided which demonstrated the functional adequacy of mechanical and electrical equipment evaluated at the Palisades plant. The specific mechanical and electrical components which require additional evaluation and possible design modification are marked by asterisks in Table 6-7.

TABLE 6-7. Summary of conclusions.

Item	Description	Conclusion and recommendation
1	Essential service water pump*	OK for structural integrity if discharge head stresses are within code allowables (no use of cast iron). Aside from the anchor bolts, functional integrity was not evaluated because of lack of design detail.
2	Auxiliary feedwater pumps*	OK for structural integrity. Functional integrity was not evaluated because of lack of design detail.
3	Component cooling heat exchangers	OK.
4	Component cooling surge tank	OK.
5	Diesel generator oil storage tanks*	No evaluation was performed, since no drawings or design calculations were available.
6	Boric acid storage tank	OK.
7	Hydrazine tank	OK.
8	Sodium hydroxide tank	OK.
9	Safety injection tank*	OK if tank support structure is rigid. Complex support structure should be evaluated for dynamic characteristics to ensure that rigidity assumption is correct.
10	Motor-operated valves*	Generic analysis of motor-operated valves on lines 4 in. in diameter or smaller should be performed to show that resulting stresses in the pipe 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 a procedure should be implemented whereby all such motor valves are externally supported. Also, verification of structural adequacy and function of the valves themselves were not demonstrated.

continued

TABLE 6-7 continued.

Item	Description	Conclusion and recommendation
11	Control rod drive mechanism*	OK for structural integrity. Based on the calculations reviewed, active function cannot be assured.
12	Pressurizer	OK.
13	Steam generators*	Insufficient information provided to verify design adequacy.
14	Reactor coolant pumps*	Insufficient information provided to verify design adequacy.
15	Reactor vessel supports and internals*	Insufficient information provided to verify design adequacy.
16	Battery racks*	Racks OK, except wooden lateral bracing should be replaced or strengthened to carry full seismic inertial loads.
17	Motor control centers*	Anchorage OK. No information available to evaluate rack structural adequacy or electrical component functionality.
18	Switchgear*	Anchorage OK if anchor bolts are 7/8 in. in diameter; otherwise, design modifications may be necessary. No information available to evaluate switchgear rack structural adequacy or electrical component functionality.
19	Control room electrical panels*	Licensee to verify seismic design adequacy.
20	Transformers*	End units of transformers should be securely anchored. No information available to evaluate structural adequacy or electrical functionality.
21	Electrical cable raceways*	Cable tray support systems should be evaluated for seismic loads induced by 0.2-g SSE.

*Components requiring additional evaluation and possible design modifications. See text.

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

***Available in the NRC Public Document Room for inspection and copying for a fee.

APPENDIX A: EVALUATION OF THE EFFECTS OF ASSUMING 20% MAXIMUM MODAL DAMPING

A.1 INTRODUCTION

In the evaluation of the seismic response of the Palisades containment and auxiliary buildings, composite modal damping ratios were computed by considering both the expected structure damping and the energy dissipation within the soil. The latter effect included both geometric (or radiation) damping and soil material damping. Calculation of the composite modal damping values was based on energy proportioning, as discussed in Chapter 5.

It was decided to limit the composite modal damping to no more than 20% of critical for both the containment and auxiliary buildings. Since both structures have calculated horizontal geometric damping ratios significantly above the 20% limit, less energy was assumed to be dissipated than would be predicted theoretically for modes involving significant horizontal soil response. Furthermore, the use of modal damping often leads to a substantial difference between the location where energy is assumed to be dissipated and the location where it is actually dissipated. For instance, the assumption of modal damping may lead to the prediction that less energy is dissipated in the soil and more in the structure than is actually the case for modes with large amounts of soil damping and significant structural response.

In order to investigate these effects, an independent, confirmatory analysis was conducted for the containment building, using a discrete dashpot to simulate the energy dissipation in the soil. This analysis was conducted for one case, using the full theoretical value computed for the horizontal damping factor. A second case was investigated using 75% of the theoretical horizontal geometric damping value. The 75% level was based on the review of one set of test results conducted for the San Onofre Nuclear Generating Station, Units 2 and 3.[A-1] These tests involved concrete slabs embedded different amounts in the soil. The full theoretical value of rocking damping was used in the analyses for both cases.

A.2 METHOD OF ANALYSIS

The analysis of the containment building with a discrete dashpot simulating the energy dissipation that results from horizontal translational soil modes was performed by direct integration of the equations of motion using the computer program DRAIN-2D.[A-2] For case 1, the full theoretical 34% horizontal geometric damping was combined with 5% soil material damping to give a horizontal energy dissipation corresponding to 39% of critical damping. For the second case, 26% of critical was used for the horizontal geometric damping. This was combined with the 5% soil material damping to give an overall value of 31% of critical for the horizontal damping. (These values were determined as described in Sec. 5.3.1.) The remainder of the two-dimensional analytical model was the same as was used in the modal analysis described in Chapter 5. The same rocking damping was used, and the structural damping was again assumed to be 3% of critical for all modes (as recommended in Ref. A-3) for prestressed and well-reinforced concrete with stresses at no more than one-half the yield point. The analysis was conducted for the median soil case with a limited check for the upper-bound soil.

Damping was incorporated into the model for the time-history analysis differently than for the modal analysis. For the time-history analysis using DRAIN-2D, the viscous damping matrix was assumed to be of the form

$$[C] = \alpha[M] + \beta[K] + [C_D],$$

where

$[C]$ = system damping matrix,

$[M]$ = system mass matrix,

$[K]$ = system stiffness matrix,

α,β = damping parameters,

$[C_D]$ = concentrated dashpot matrix.

The dashpot was added along the diagonal of the system damping matrix at the degree of freedom with which the dashpot is associated.

The input for the time-history analysis consisted of an artificial earthquake whose response spectra are close to the smoothed spectra in

R.G. 1.60 but do not necessarily envelop the R.G. 1.60 spectra at all frequencies (Fig. A-1). The direct integration of the equations of motion was performed using a technique which assumes constant acceleration within the time step. The method is stable for all frequencies and does not introduce numerical damping, but small time steps are required. Integration time steps

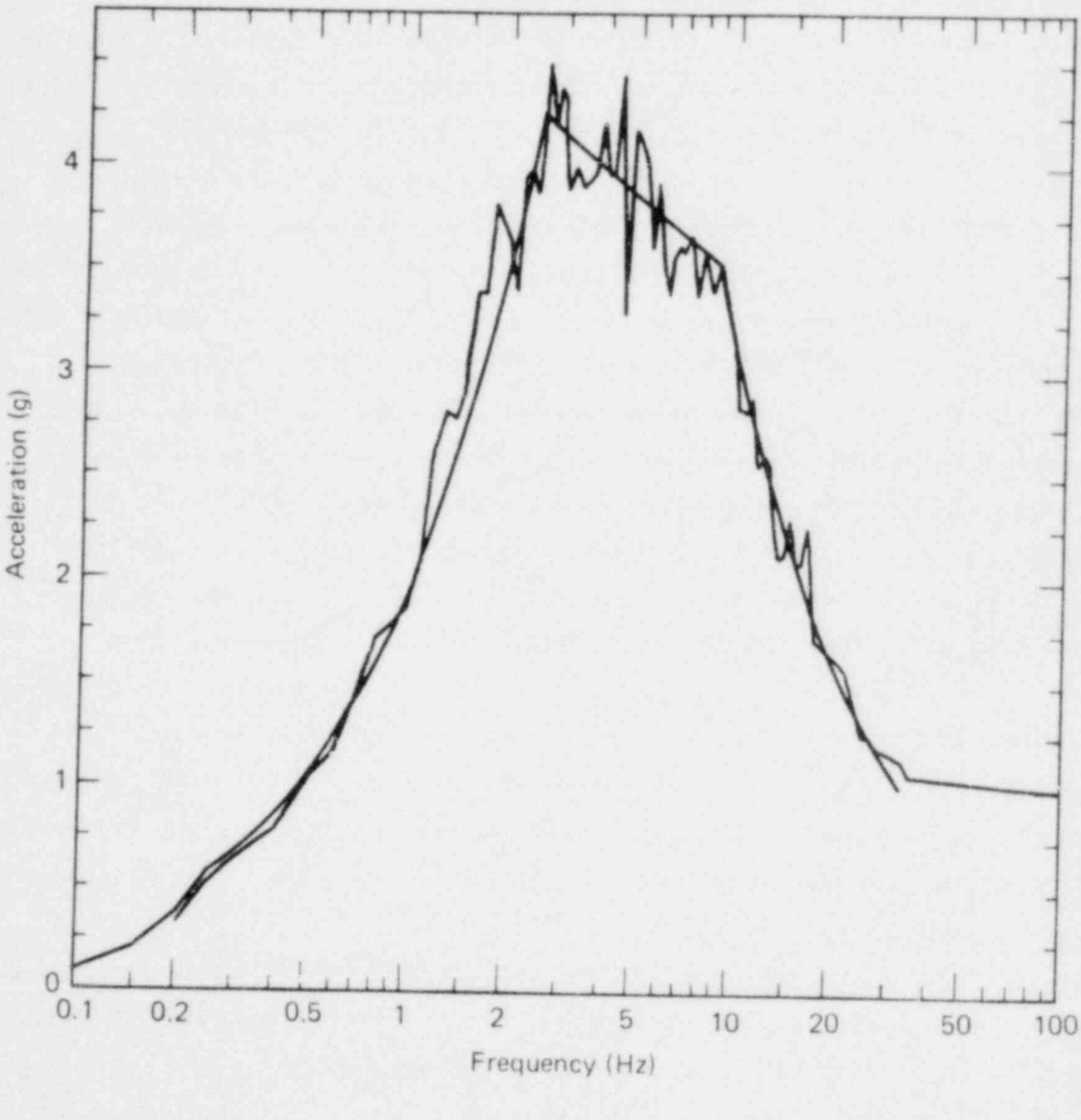

FIG. A-1. Response spectrum (2% damping) used for the time-history analysis of the containment building, superposed on the corresponding smoothed spectrum from R.G. 1.60.

of 0.01 s were used in the analysis. This time step is expected to provide good accuracy for frequencies up to near 20 Hz, which is adequate for the region of interest for the reactor containment building.

A.3 BUILDING RESPONSE

Maximum shear and moment distributions throughout the structure, including the containment vessel and concrete internal structures, as determined from the time-history analysis are shown in Figs. A-2 and A-3. For the purpose of comparison, the corresponding shear and moment distributions obtained from the response-spectrum modal analysis (using composite modal damping with a 20% upper limit) are shown in the same figures. This comparison indicates that the latter analysis results in somewhat higher response throughout the structure, compared with either of the time-history cases. For case 1 (full theoretical geometric damping), base shear in the containment is approximately 10% greater using the 20% damping limit; shear remains about 4% greater at the higher elevations. Shear in the concrete internals is about 9% to 13% greater for the 20% damping case. Bending moments are approximately 5% greater at the base of the containment vessel and about 11% greater in the concrete internals. For case 2, using 75% of the theoretical damping, somewhat less reduction in response is indicated; however, both shear and moment response throughout the structure are somewhat less than computed using the composite modal damping limited to 20% of critical. These comparisons are not expected to be exact, since the response spectrum produced by the time history is not identical with the R.G. 1.60 spectrum. Consequently, identical responses would not be expected, even if the damping were treated in exactly the same manner. However, the results are considered representative in determining the effects of the 20% limit on modal damping.

From these results, it is apparent that for the Palisades analysis the use of composite modal damping limited to 20% of critical produces slightly conservative structural response results. Such an approach is therefore acceptable.

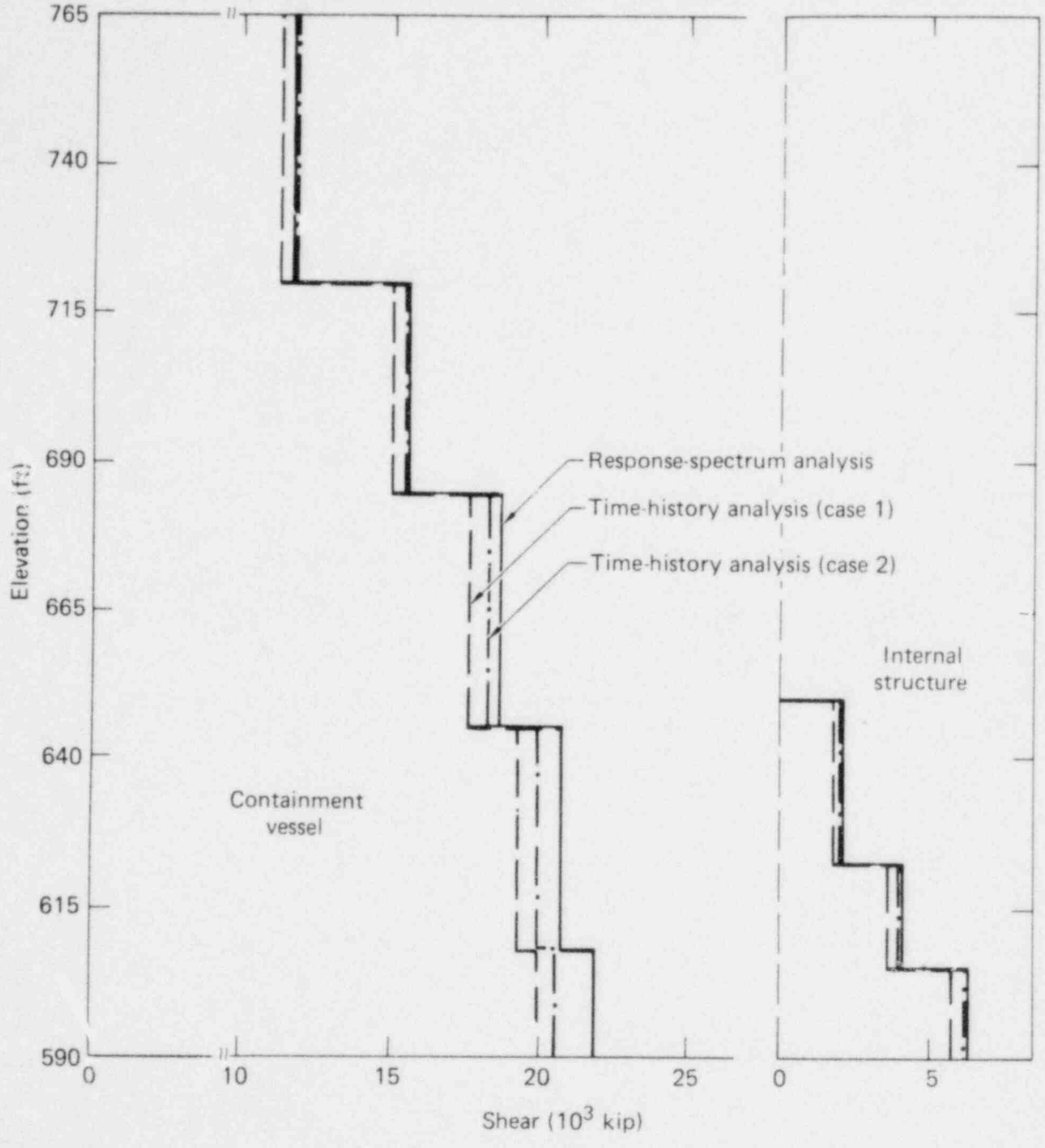

FIG. A-2. Shear distribution in the containment building as determined from the response-spectrum analysis of Chapter 5 and two time-history analyses.

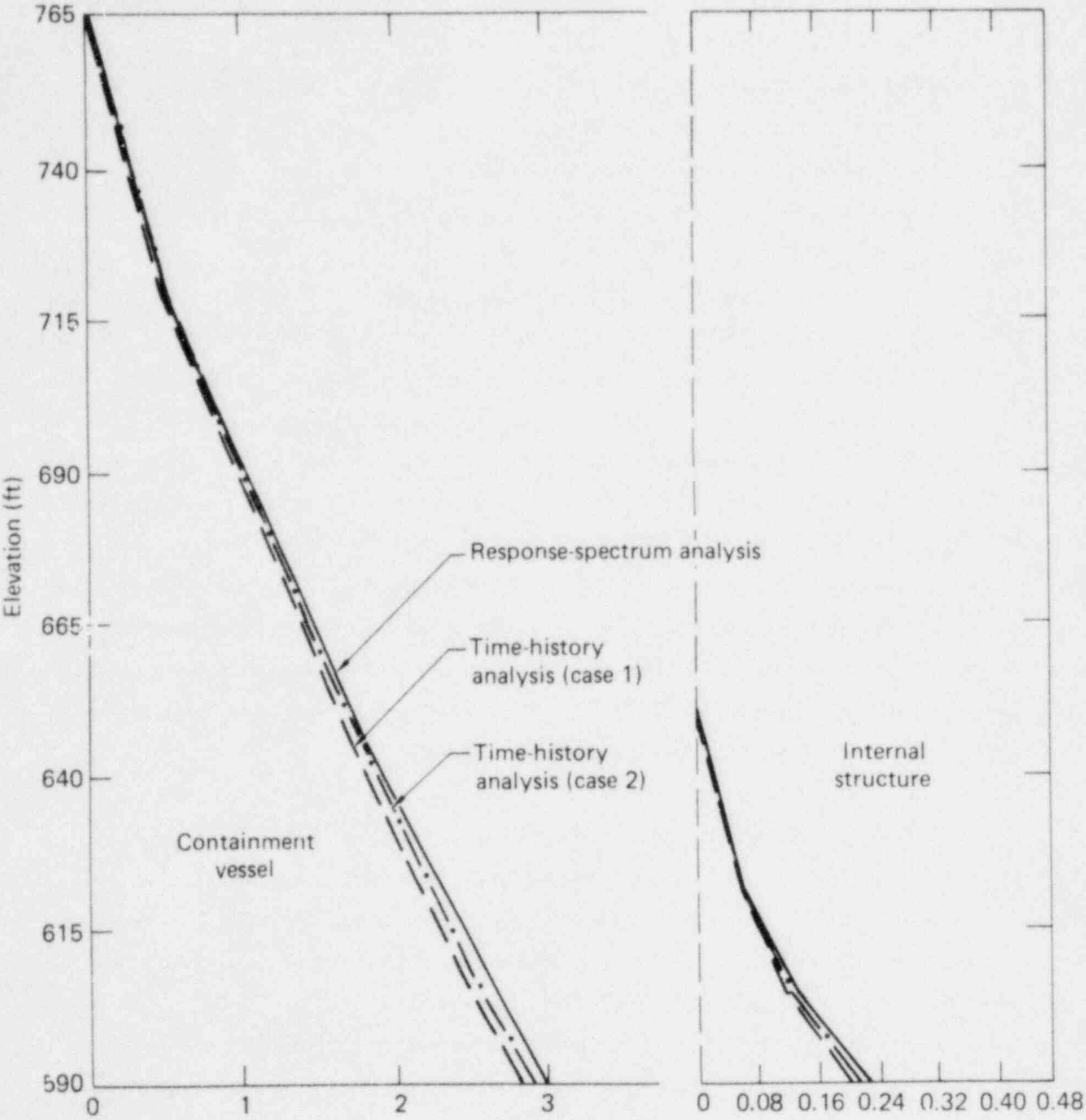

FIG. A-3. Moment distribution in the containment building as determined from the response-spectrum analysis of Chapter 5 and two time-history analyses.

A.4 IN-STRUCTURE RESPONSE SPECTRA

To evaluate the effects of composite modal damping (as used in Chapters 5 and 6) on the response of equipment as well as on the response of the structure, in-structure response spectra were generated for several locations, using the response obtained from the two time-history analysis cases described above. In-structure response spectra for 3% equipment damping were generated for the median soil case at the containment building base slab (El 590 ft), at the top of the concrete internals (El 649 ft), and at El 730 ft in the containment vessel; and for the upper-bound soil case at El 616 ft and El 649 ft in the internals.

These spectra were smoothed and broadened as described in Sec. 5.6 and are shown in Figs. A-4 through A-8. For comparison, the corresponding spectra for 3% equipment damping, developed using the 20% maximum modal damping results, are shown in the same figures. At the base slab and containment vessel locations, the in-structure spectra produced by the two methods are similar in shape and magnitude, though the time-history results are, in general, somewhat lower than those developed using the 20% limit on modal damping. This is particularly true in the high-frequency regions, where the decrease in the spectra corresponds to a similar decrease in the structural response acceleration levels. Only very slight differences are evident when comparing spectra generated using the full theoretical geometric damping with those generated using 75% of the theoretical value.

At the top of the concrete internal structure, some modification in the shape of the response spectrum is noted. This occurs because of changes in the response contributions from the second and third horizontal modes. The second horizontal mode (5.8 Hz) is basically a soil translation mode, whereas the third horizontal mode (12.9 Hz) is primarily shear deformation of the concrete internals structure, with little soil displacement. Use of the discrete soil dashpot tends to add a significant amount of damping to the second mode, while the damping of the third mode is decreased in comparison with the composite modal damping limited to 20% of critical. This is reflected in the in-structure response spectra shown in Fig. A-5, where, for the time-history results, the response near 13 Hz is amplified while that near 6 Hz is attenuated.

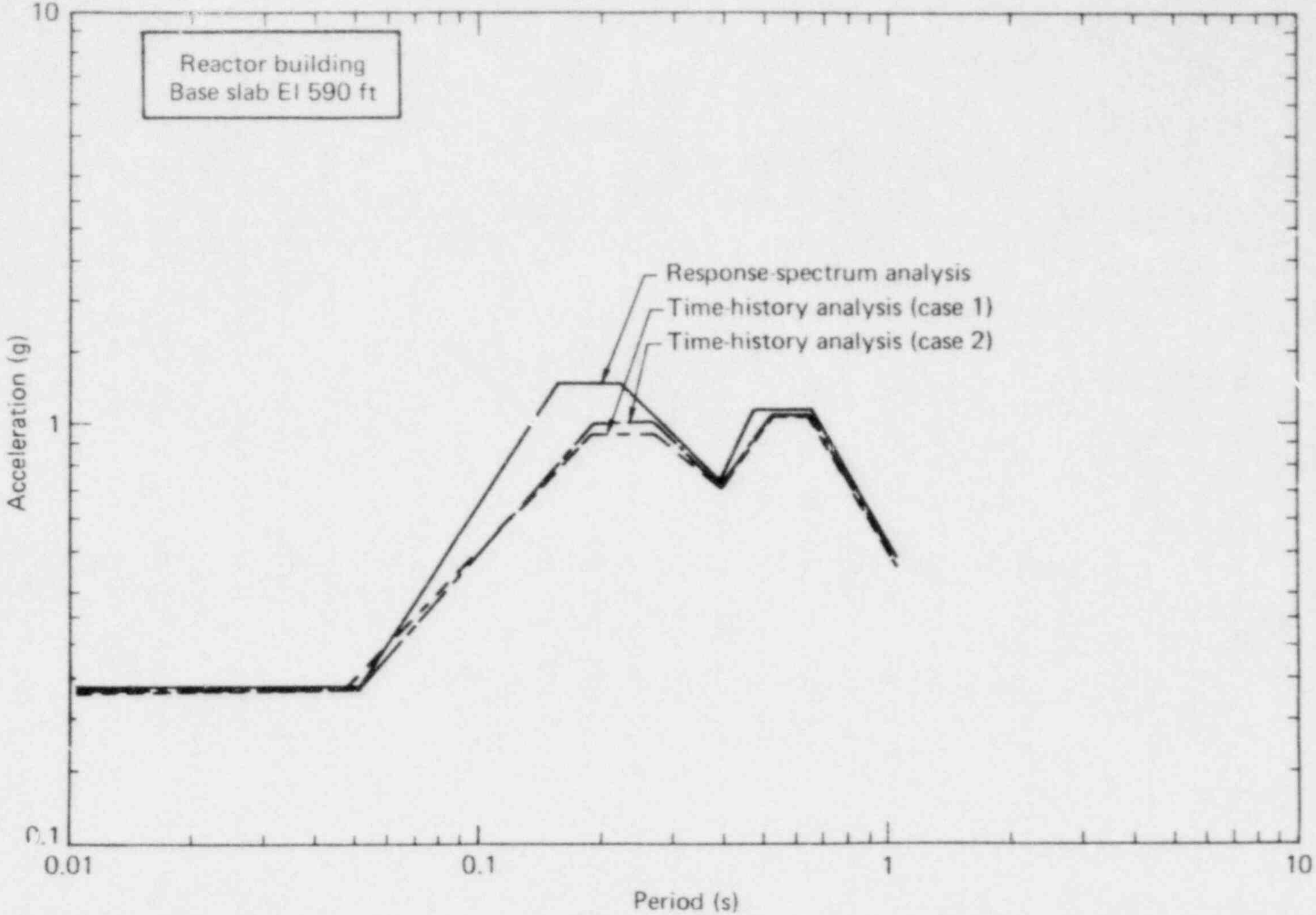

FIG. A-4. Comparison of base slab response spectra. All three assume 3% equipment damping and median soil conditions.

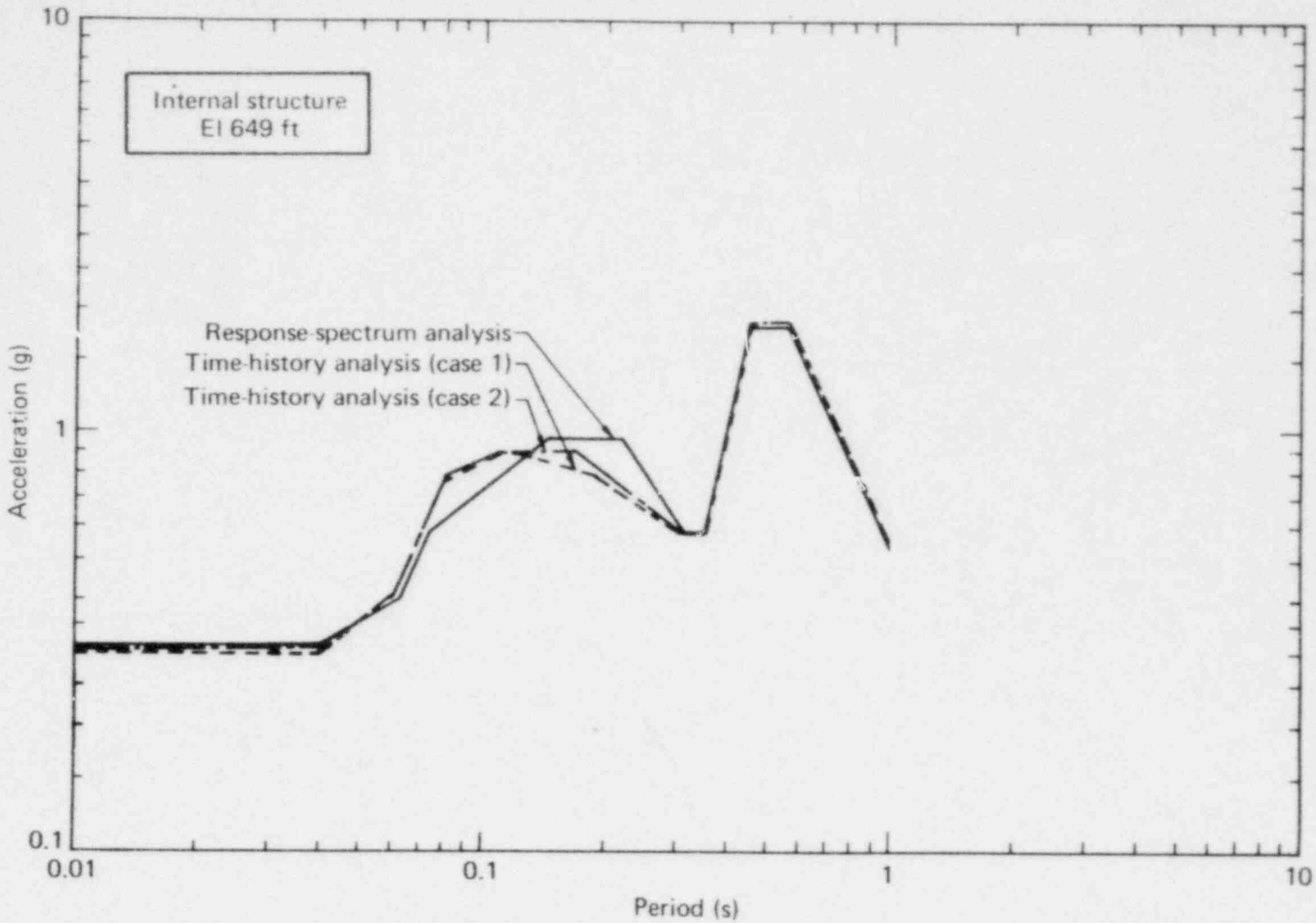

FIG. A-5. Comparison of response spectra for the concrete internals (El 649 ft). All three assume 3% equipment damping and median soil conditions.

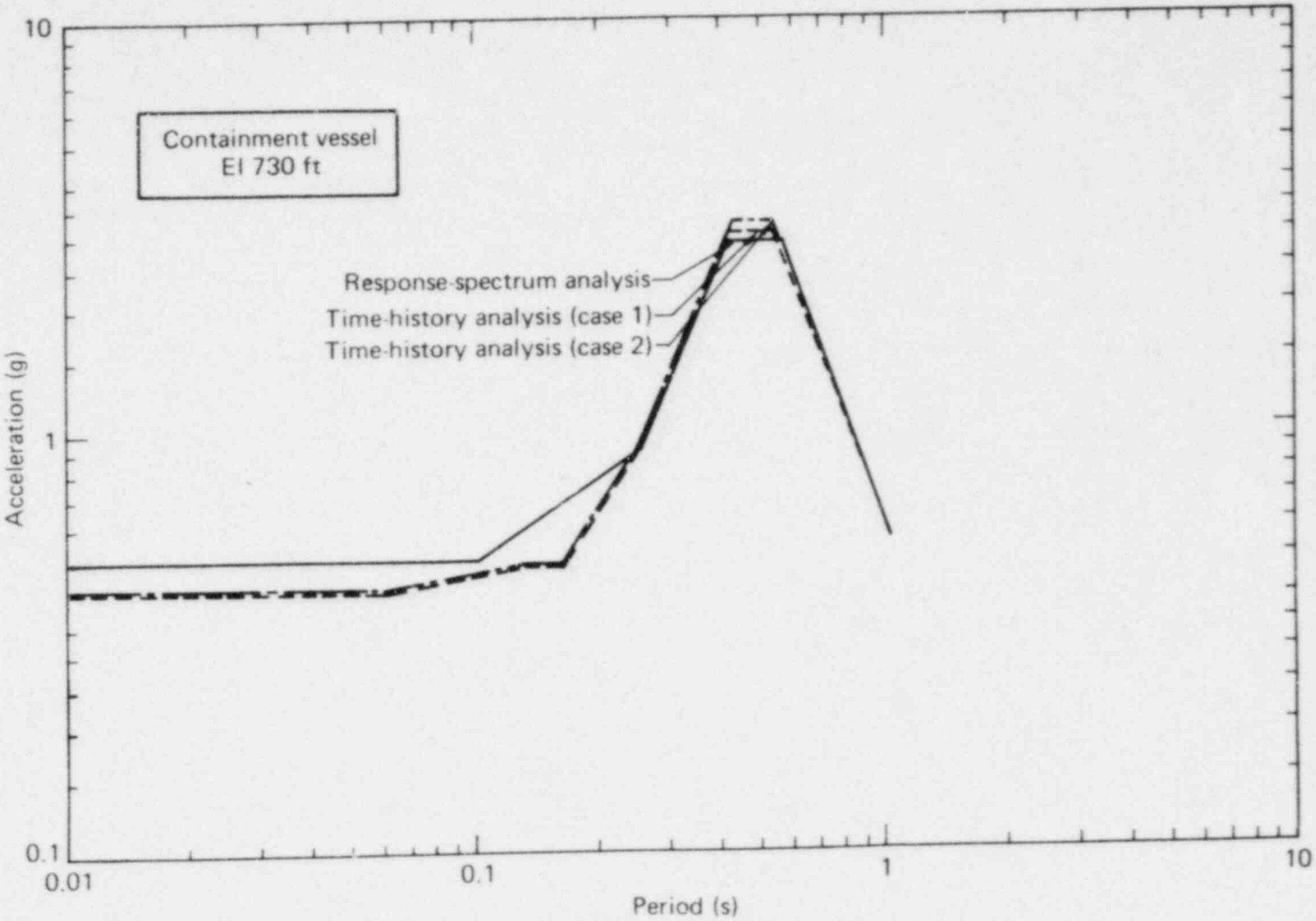

FIG. A-6. Comparison of response spectra for the containment vessel. All three assume 3% equipment damping and median soil conditions.

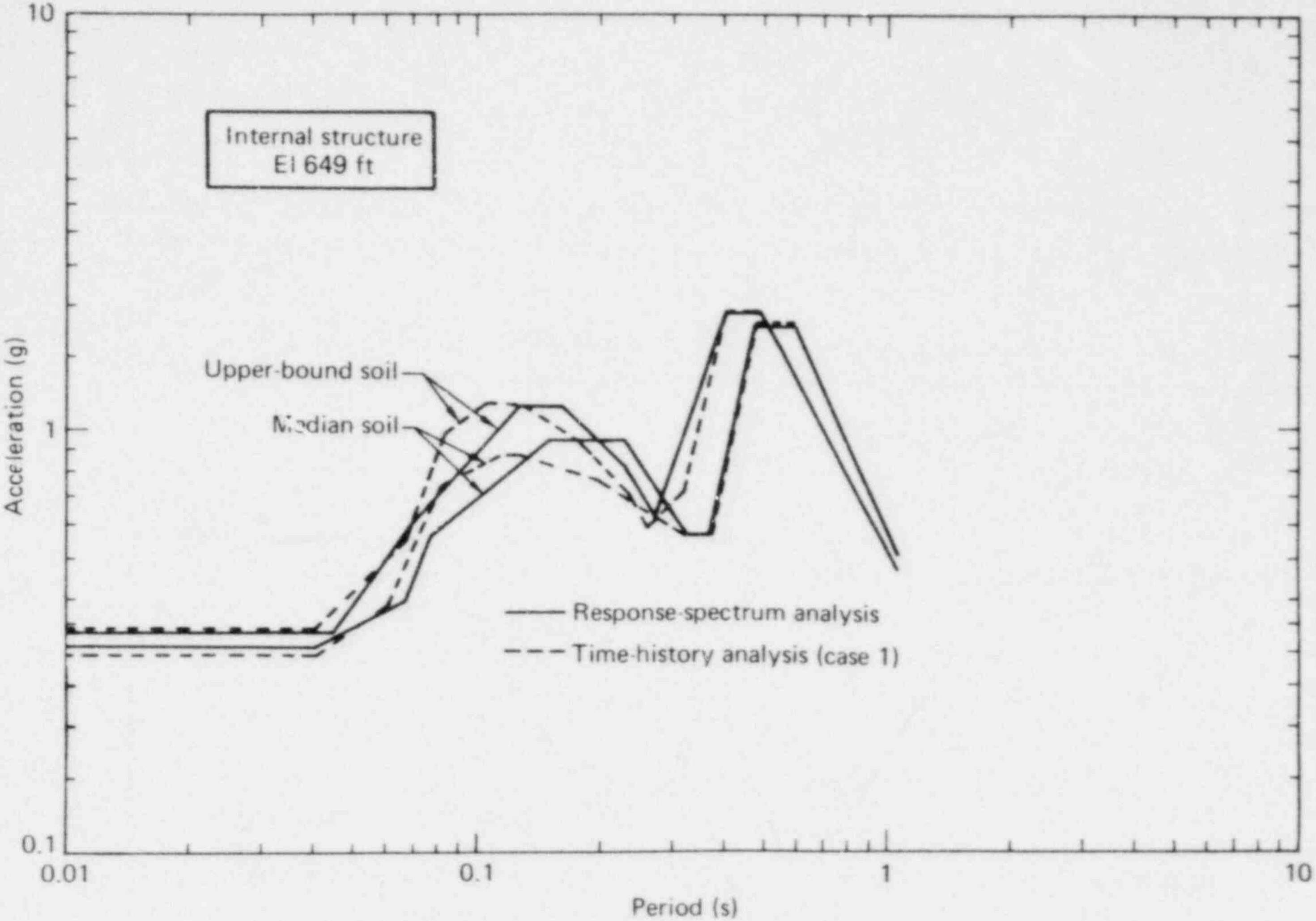

FIG. A-7. Comparison of response spectra for the concrete internals (El 649 ft), for upper-bound and median soil conditions. All four assume 3% equipment damping.

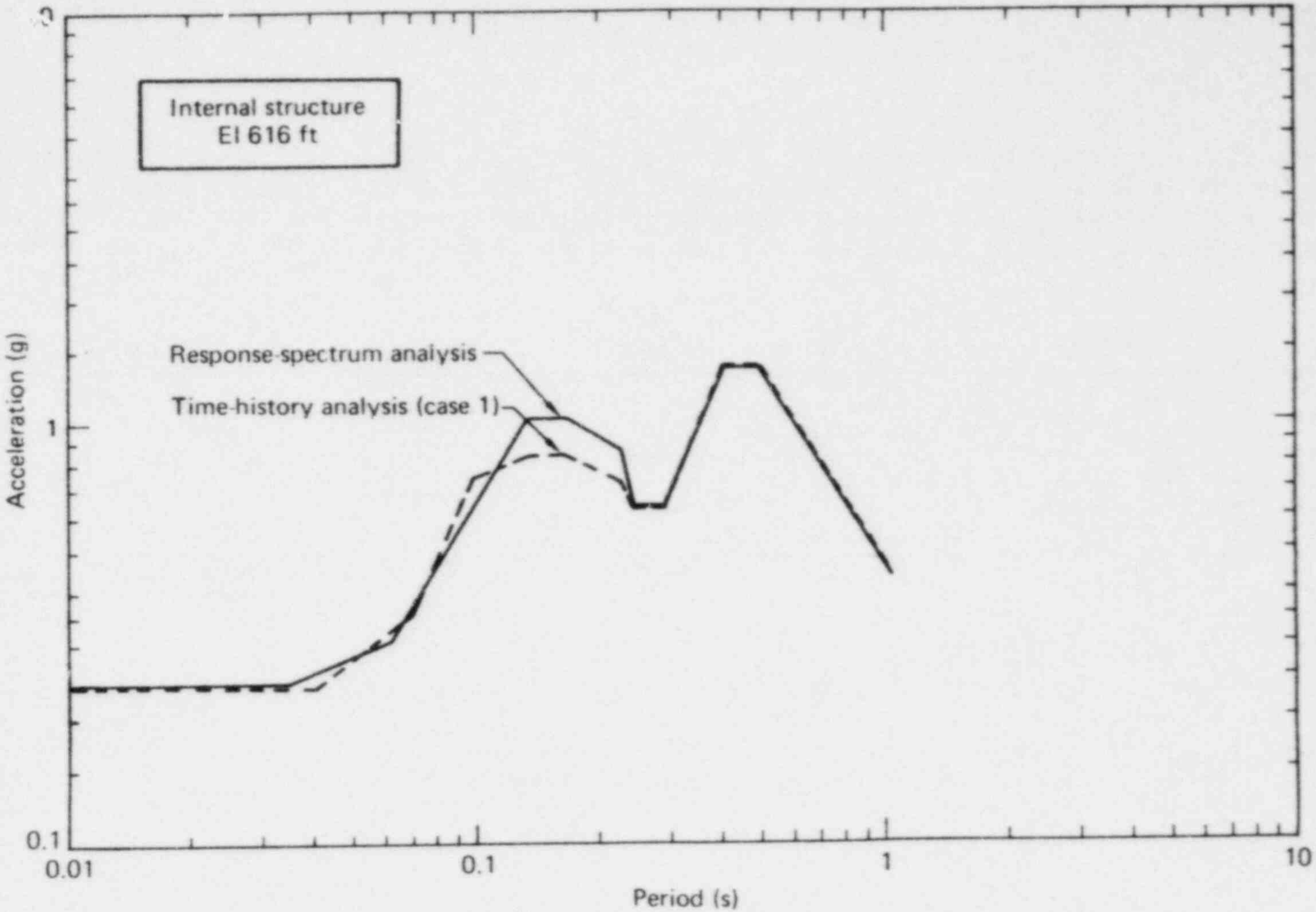

FIG. A-8. Comparison of response spectra for the concrete internals (El 616 ft). Both assume 3% equipment damping and upper-bound soil conditions.

In the region of 11 Hz, the response spectra generated using the discrete dashpot approach exceed somewhat the corresponding spectra based on the composite modal damping method. This is true for the median soil case and for the upper-bound case (Fig. A-7). If the actual soil modulus is near or less than the median value used in the analysis, the envelope of spectra accounting for the soil range will cover the increase in response obtained by using the discrete dashpot method. In any event, as shown in Figs. A-4 through A-8, the discrepancies between the spectra generated by the two methods are limited to very narrow frequency ranges. Thus, it is concluded that the response spectra generated using composite modal damping limited to 20% of critical are adequate for evaluations of piping and equipment.

REFERENCES FOR APPENDIX A

A-1. Woodward-McNeill and Associates, Soil-Structure Interaction Parameters: Development of Soil-Structure Interaction Parameters for Proposed Units 2 and 3, San Onofre Generating Station for Southern California Edison Co., Appendix 3.7c (January 31, 1971).

A-2. G. H. Powell, DRAIN-2D Users Guide, University of California, Berkeley, Calif. (August 1975).

A-3. N. M. Newmark and W. J. Hall, Development of Criteria for Seismic Review of Selected Nuclear Power Plants, U.S. Nuclear Regulatory Commission, NUREG/CR-0098 (1978).**

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

APPENDIX B: IN-STRUCTURE RESPONSE SPECTRA

The following 20 figures depict the in-structure response spectra developed from the models shown in Figs. 5-3 and 5-7. The spectra are discussed briefly in Sec. 5.6. They are the basis of the reassessment of equipment and piping in Chapter 6.

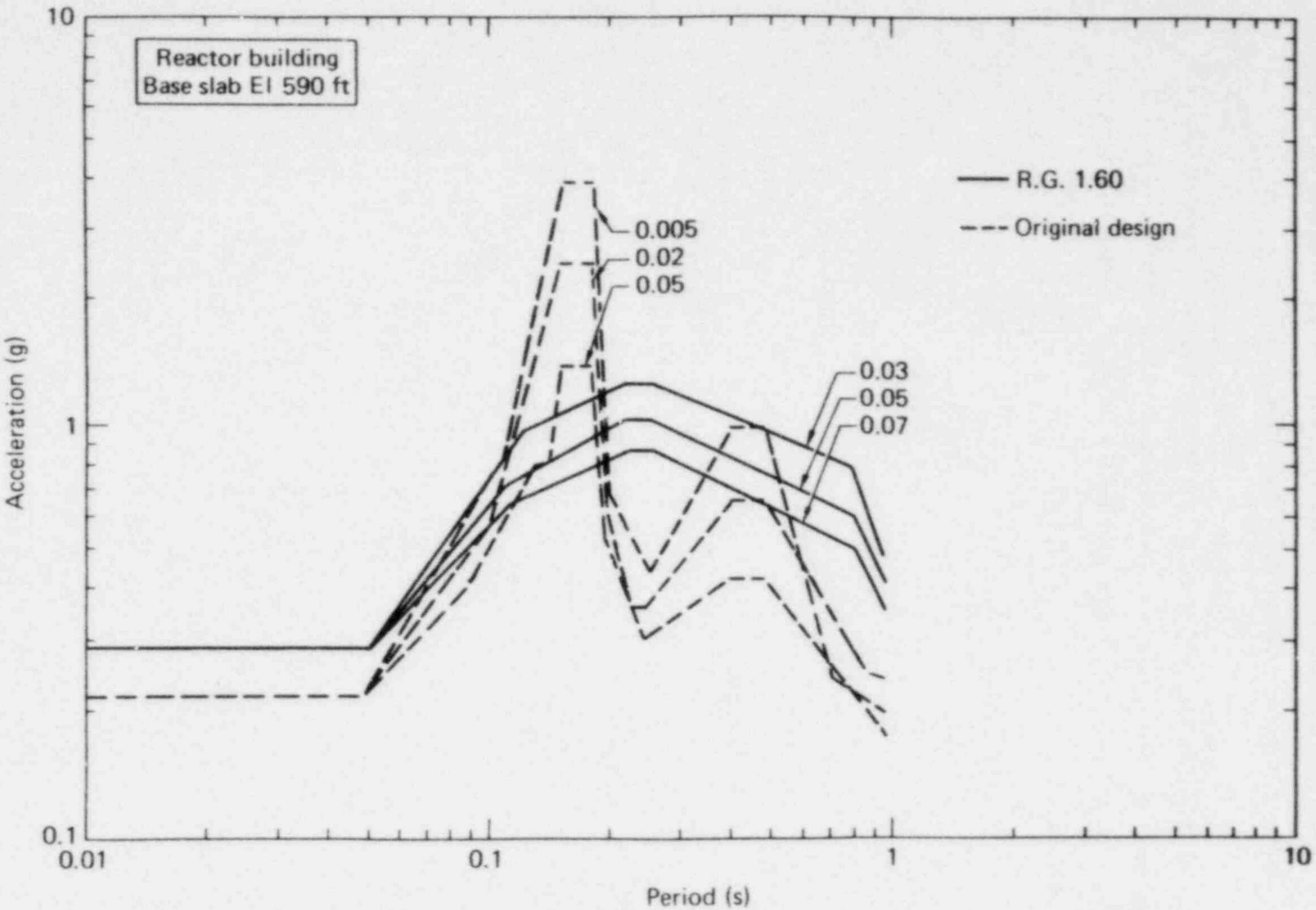

FIG. B-1. Comparison of horizontal in-structure response spectra for the containment building base slab. The spectra recalculated on the basis of R.G. 1.60 envelop the total range of soil properties. Labels indicate equipment damping ratios.

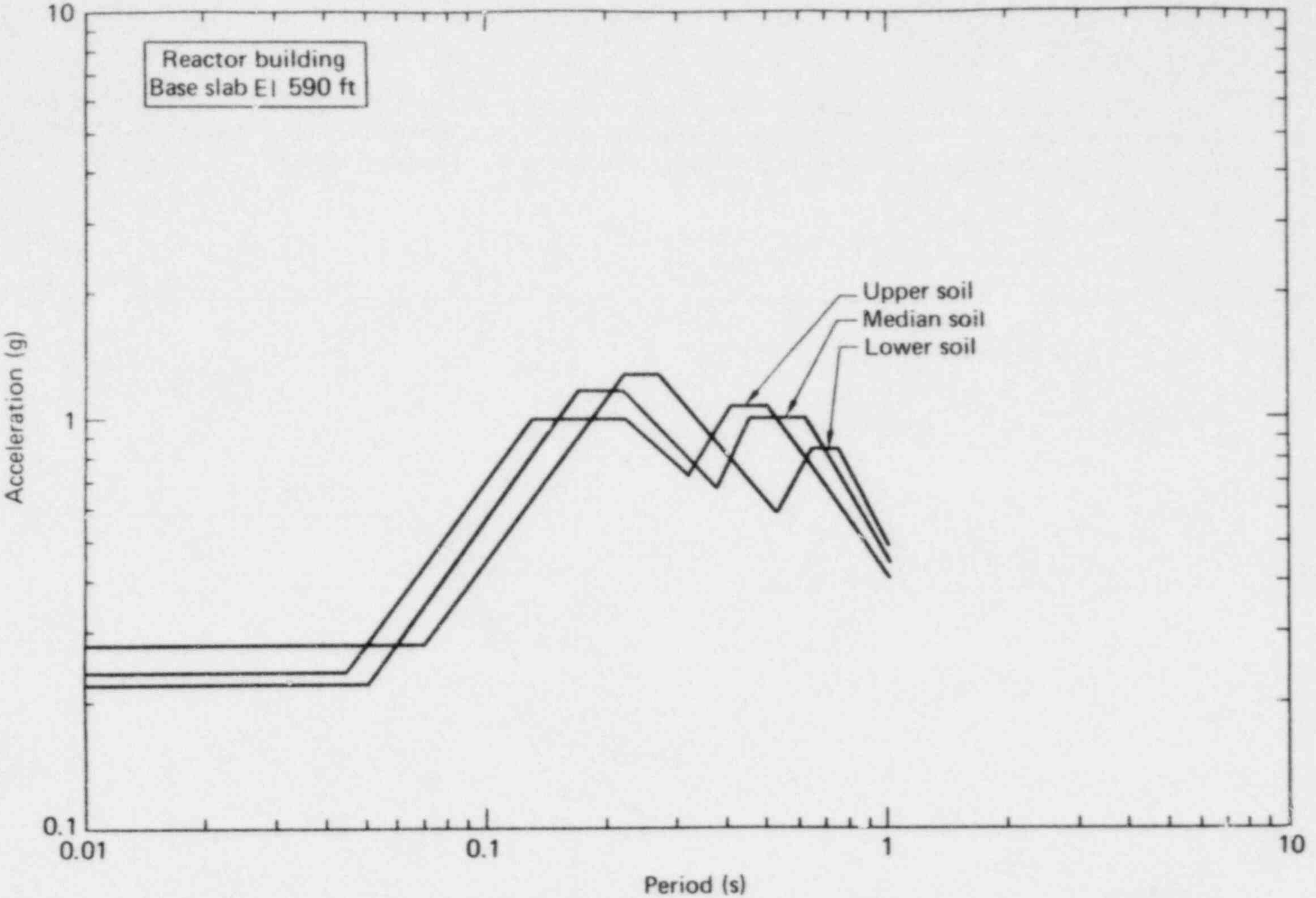

FIG. B-2. Horizontal in-structure response spectra for the containment building base slab, based on R.G. 1.60 spectra. The spectra illustrate the variation with the different soil conditions discussed in the text. The equipment damping ratio was 0.03 for all three spectra.

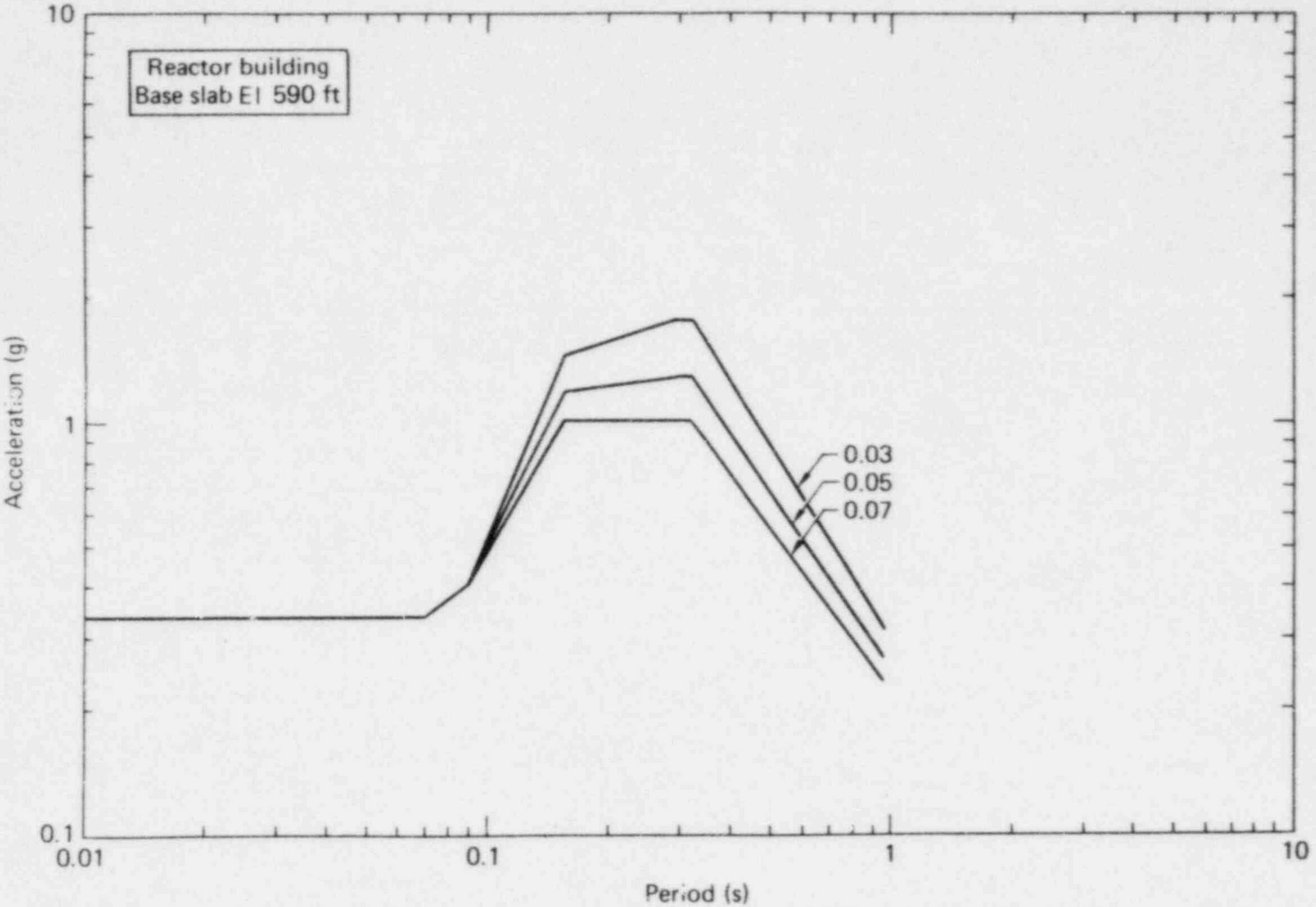

FIG. B-3. Vertical in-structure response spectra for the containment building base slab, based on R.G. 1.60 spectra. The spectra envelop the total range of soil properties. Labels indicate equipment damping ratios. These spectra are typical for all elevations in the containment building.

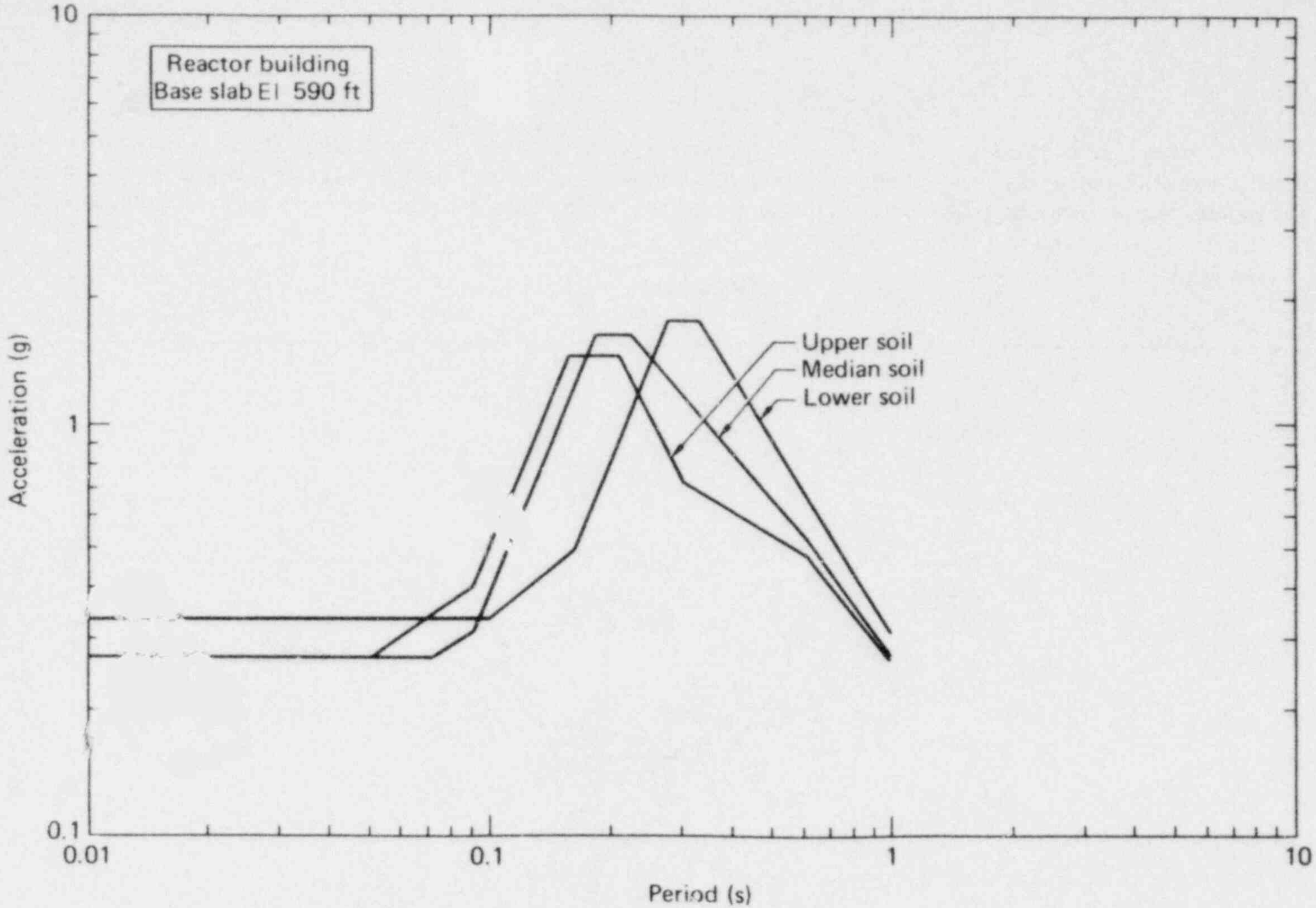

FIG. B-4. Vertical in-structure response spectra for the containment building base slab, based on R.G. 1.60 spectra. The spectra illustrate the variation with the different soil conditions discussed in the text. The equipment damping ratio was 0.03 for all three spectra. The spectra are typical for all elevations in the containment building.

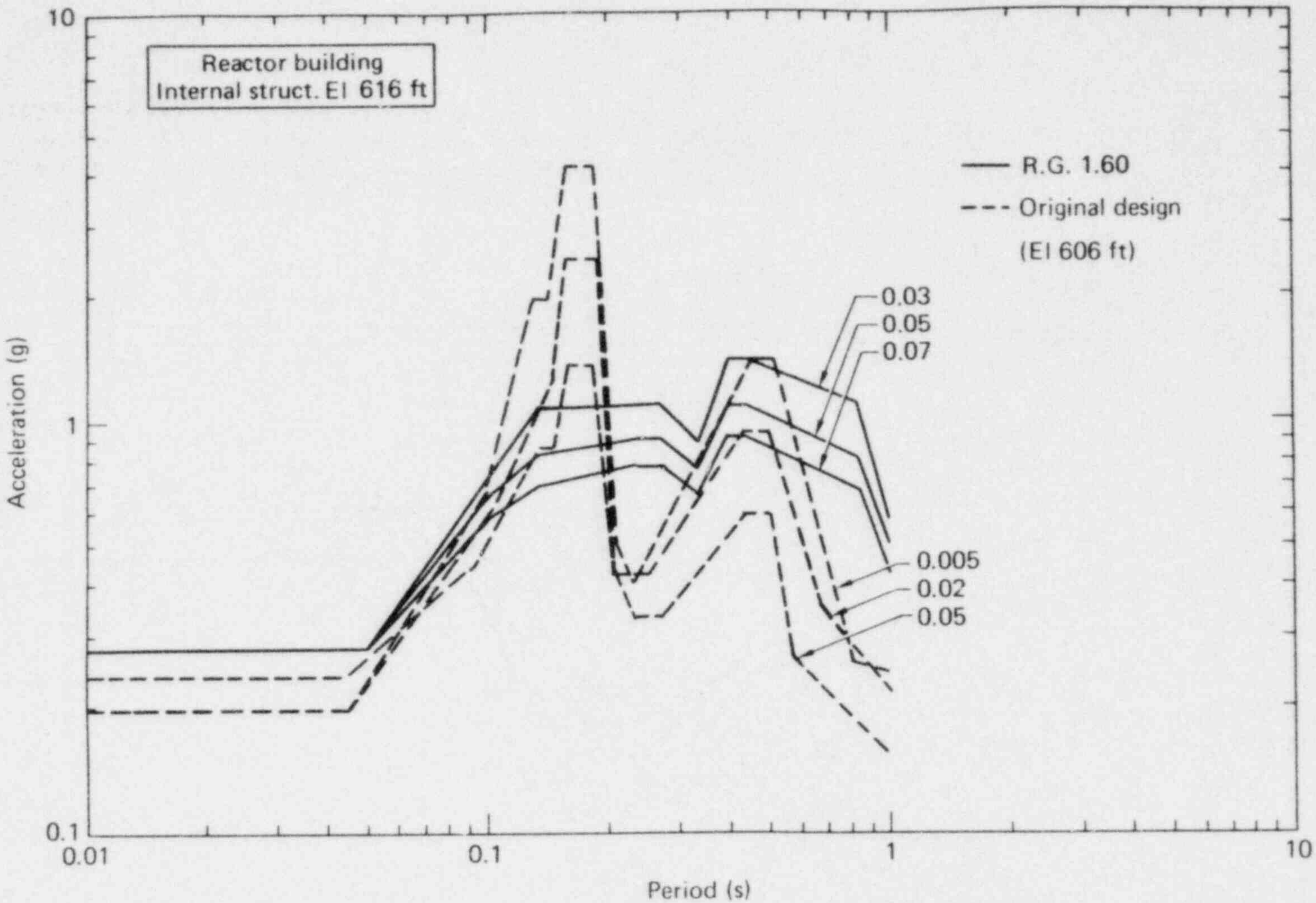

FIG. B-5. Comparison of horizontal in-structure response spectra for the internal structures (El 616 ft). The spectra recalculated on the basis of R.G. 1.60 envelop the total range of soil properties. Labels indicate equipment damping ratios.

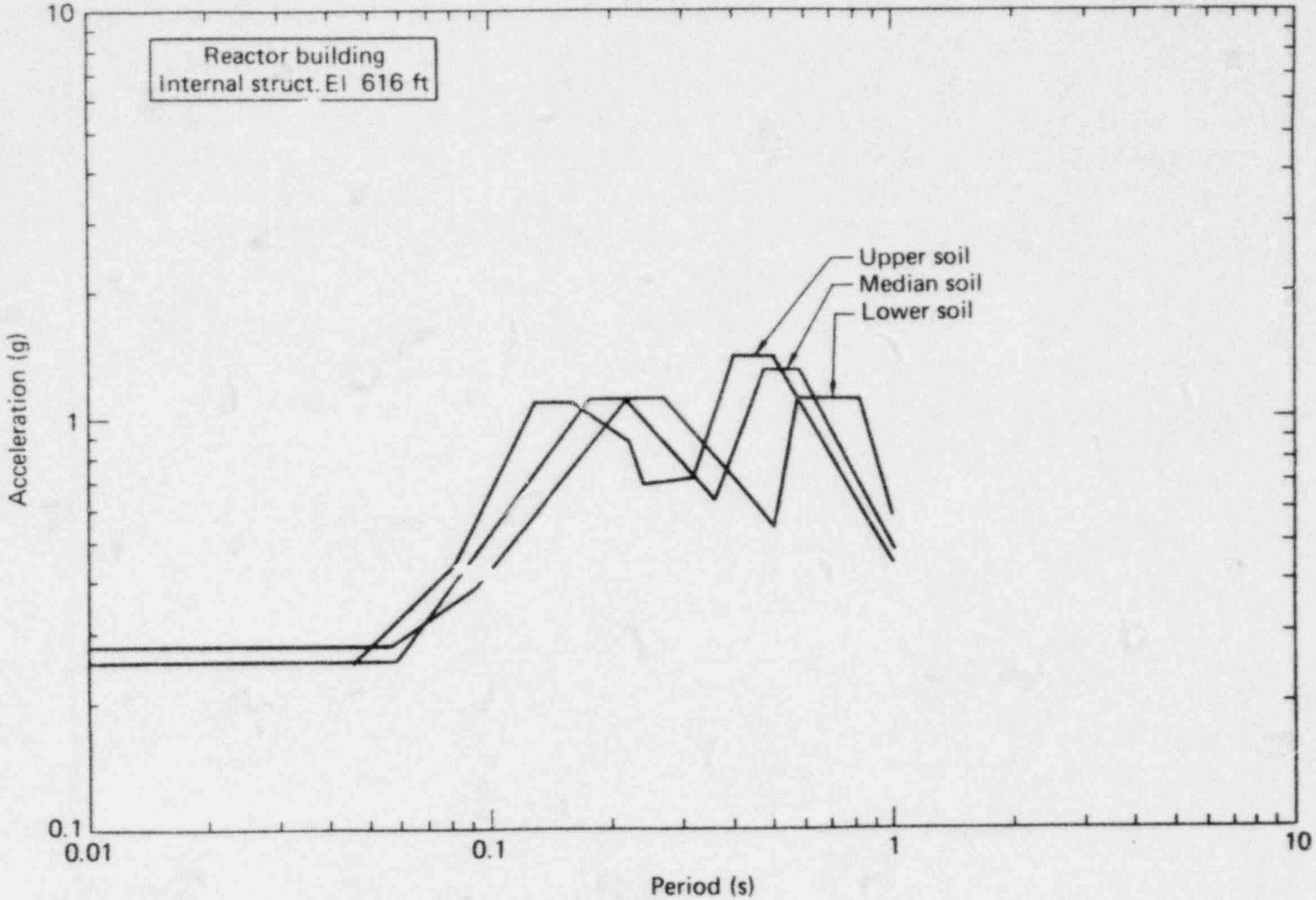

FIG. B-6. Horizontal in-structure response spectra for the internal structures (El 616 ft), based on R.G. 1.60 spectra. The spectra illustrate the variation with the different soil conditions discussed in the text. The equipment damping ratio was 0.03 for all three spectra.

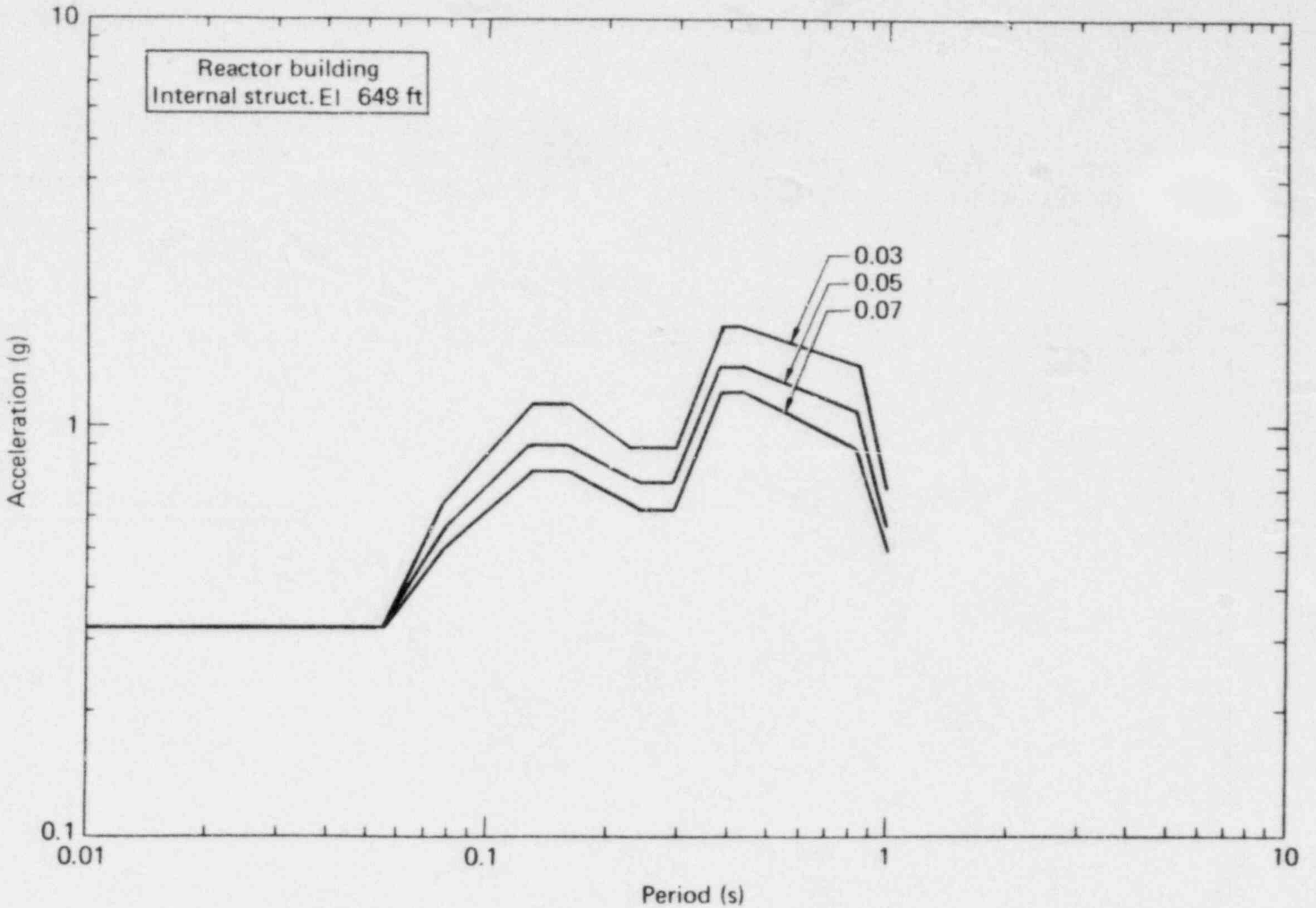

FIG. B-7. Horizontal in-structure response spectra for the internal structures (El 649 ft), based on R.G. 1.60 spectra. The spectra envelop the total range of soil properties. Labels indicate equipment damping ratios.

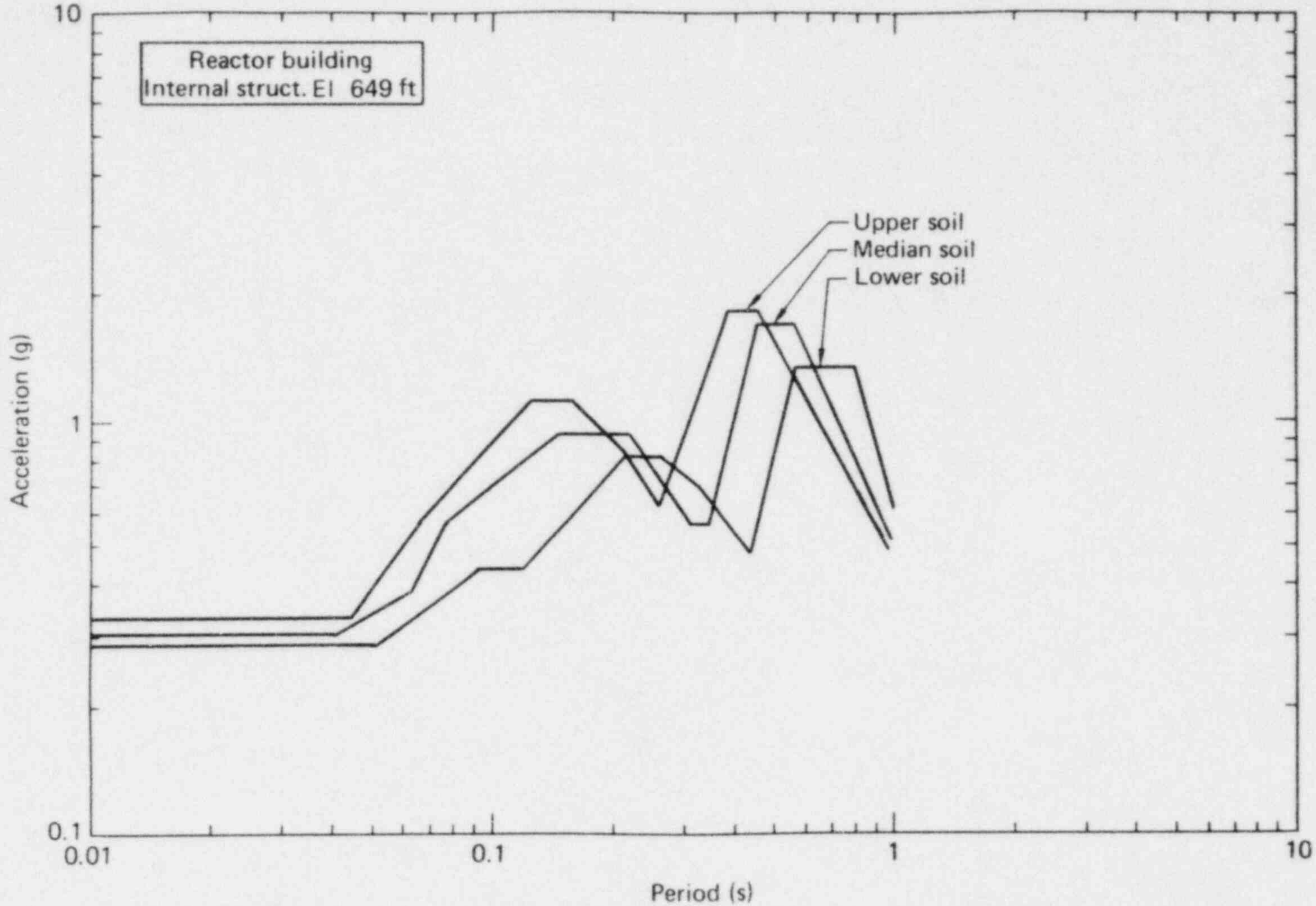

FIG. B-8. Horizontal in-structure response spectra for the internal structures (El 649 ft), based on R.G. 1.60 spectra. The spectra illustrate the variation with the different soil conditions discussed in the text. The equipment damping ratio was 0.03 for all three spectra.

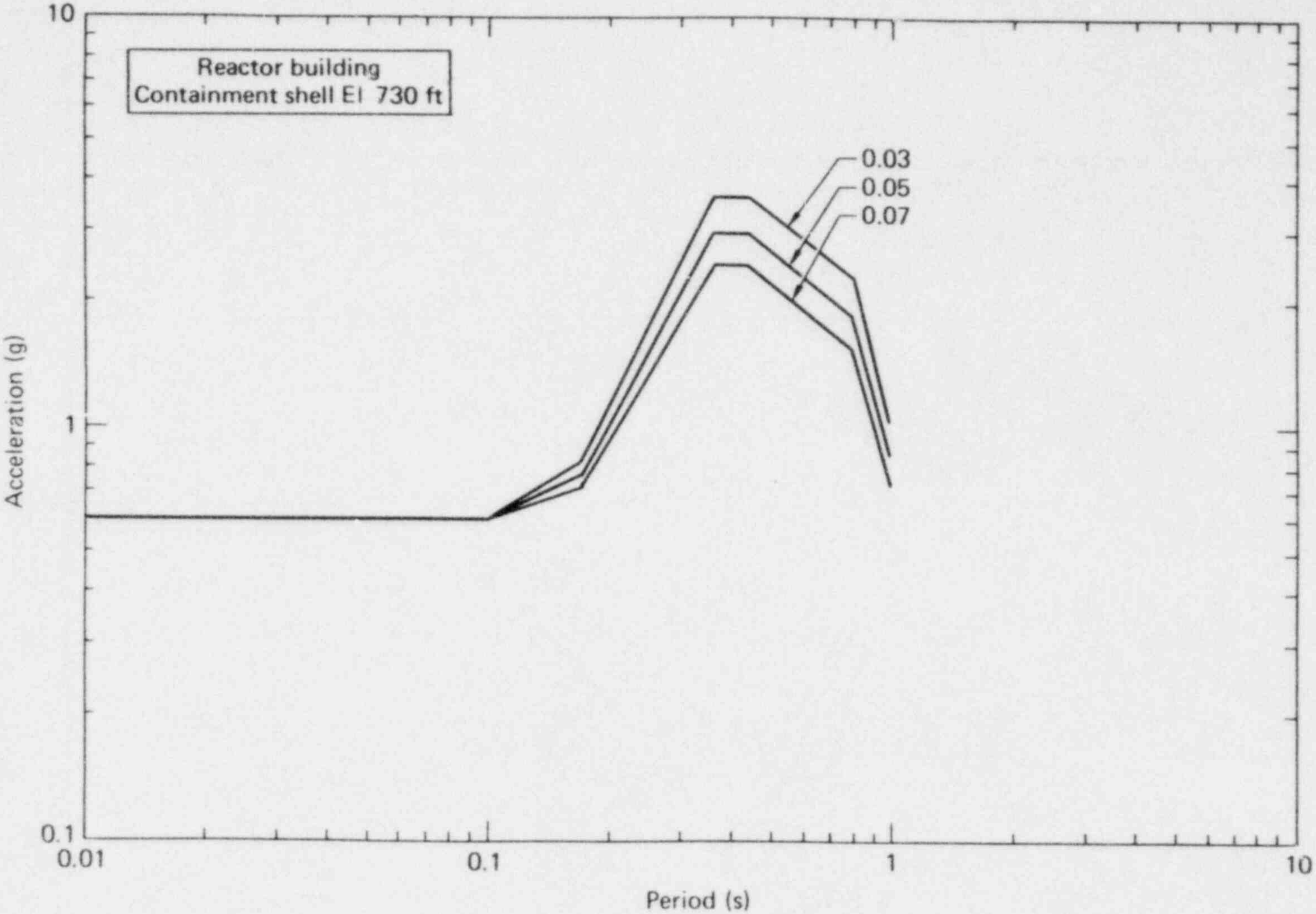

FIG. B-9. Horizontal in-structure response spectra for the containment shell (El 730 ft), based on R.G. 1.60 spectra. The spectra envelop the total range of soil properties. Labels indicate equipment damping ratios.

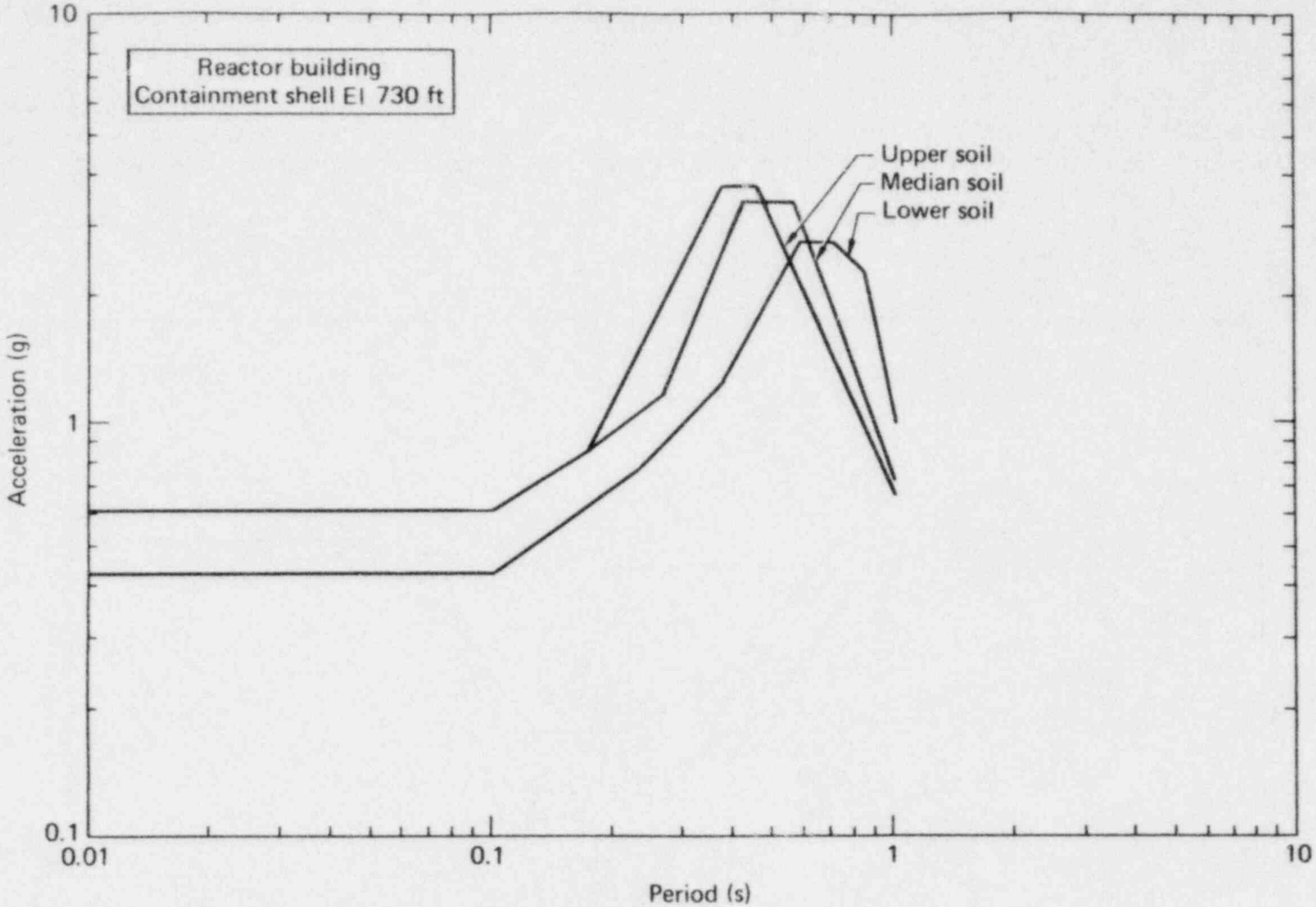

FIG. B-10. Horizontal in-structure response spectra for the containment shell (El 730 ft), based on R.G. 1.60 spectra. The spectra illustrate the variation with the different soil conditions discussed in the text. The equipment damping ratio was 0.03 for all three spectra.

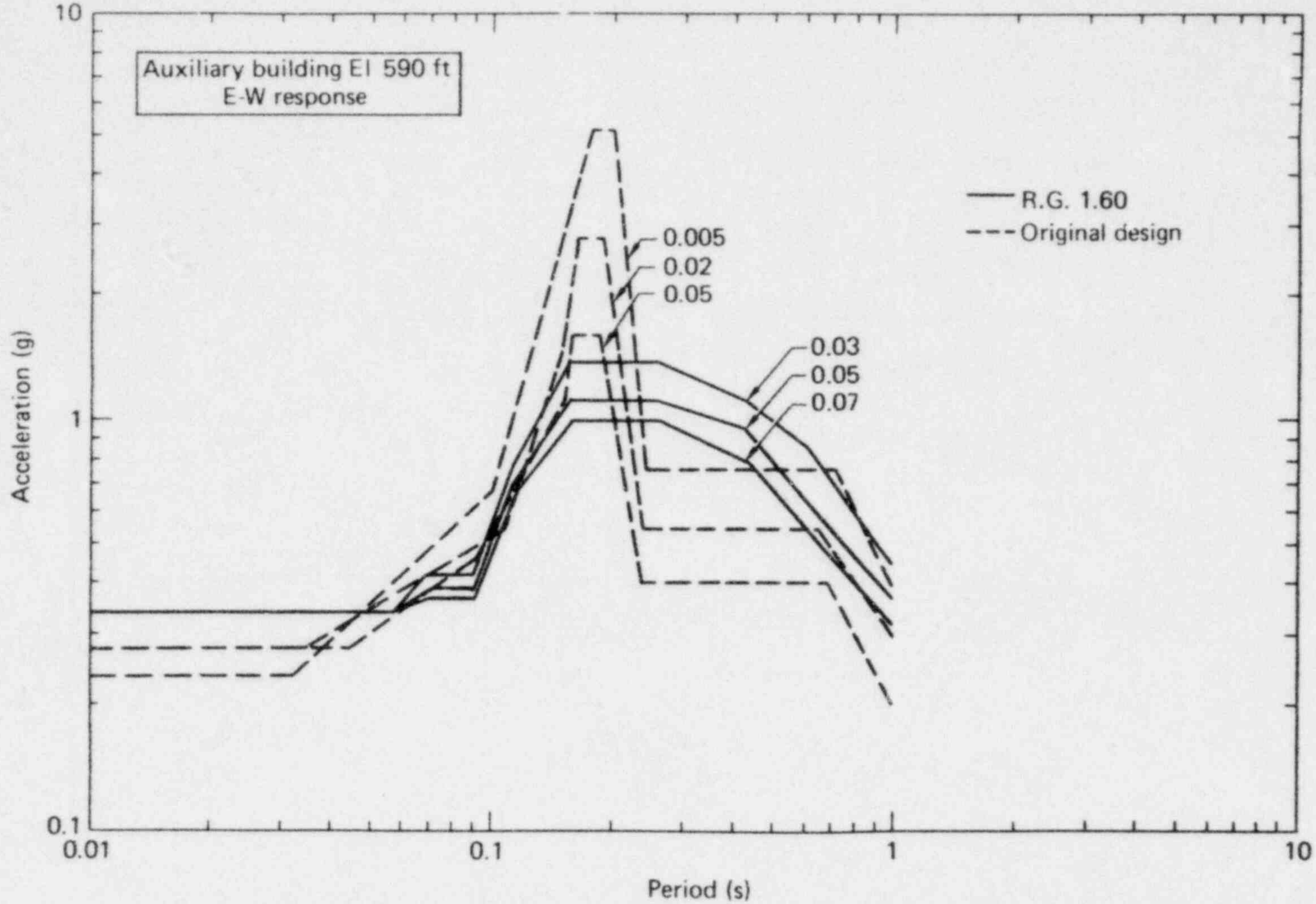

FIG. B-11. Comparison of E-W horizontal in-structure response spectra for the auxiliary building base slab. The spectra recalculated on the basis of R.G. 1.60 envelop the total range of soil properties. Labels indicate equipment damping ratios.

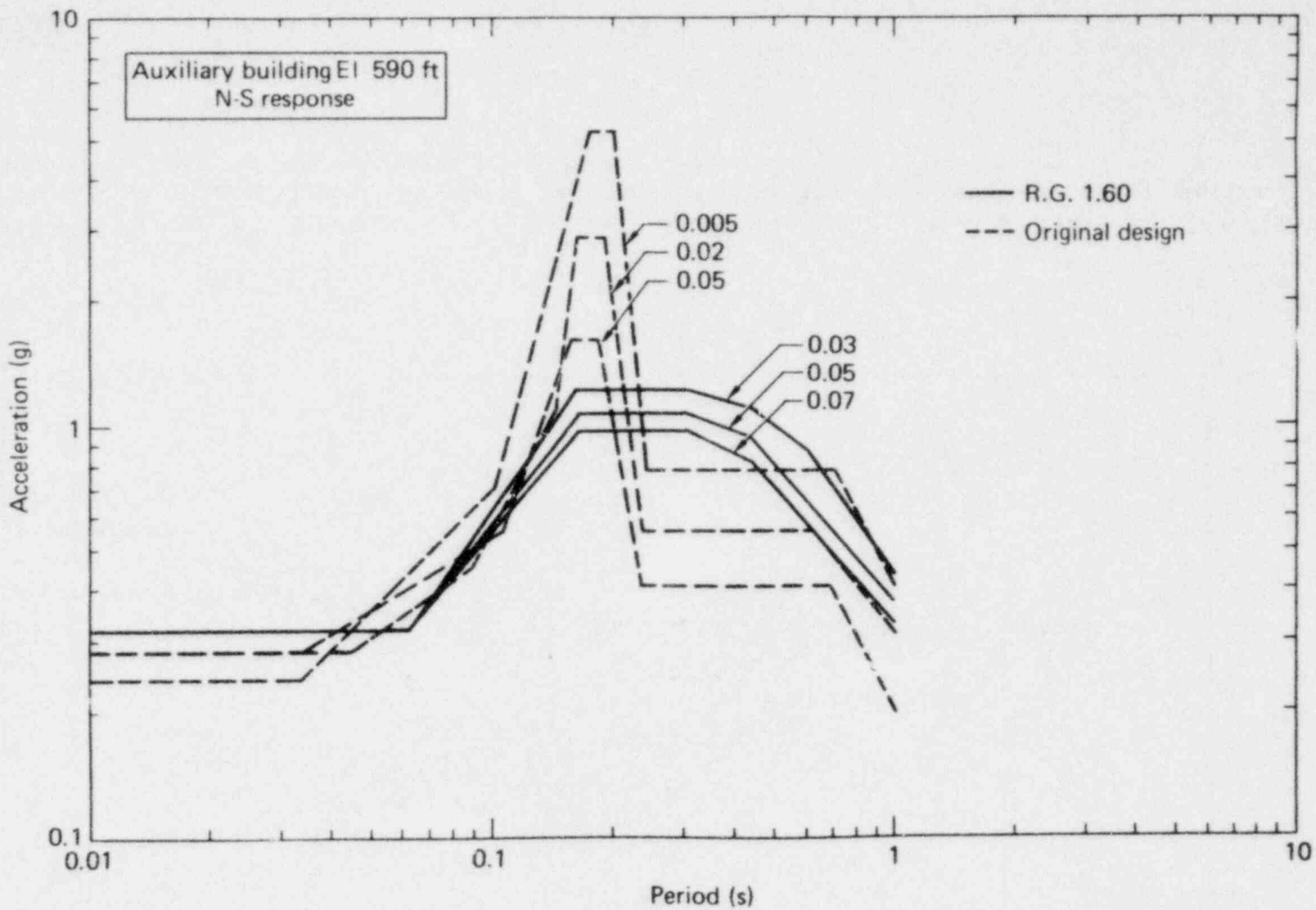

FIG. B-12. Comparison of N-S horizontal in-structure response spectra for the auxiliary building base slab. The spectra recalculated on the basis of R.G. 1.60 envelop the total range of soil properties. Labels indicate equipment damping ratios.

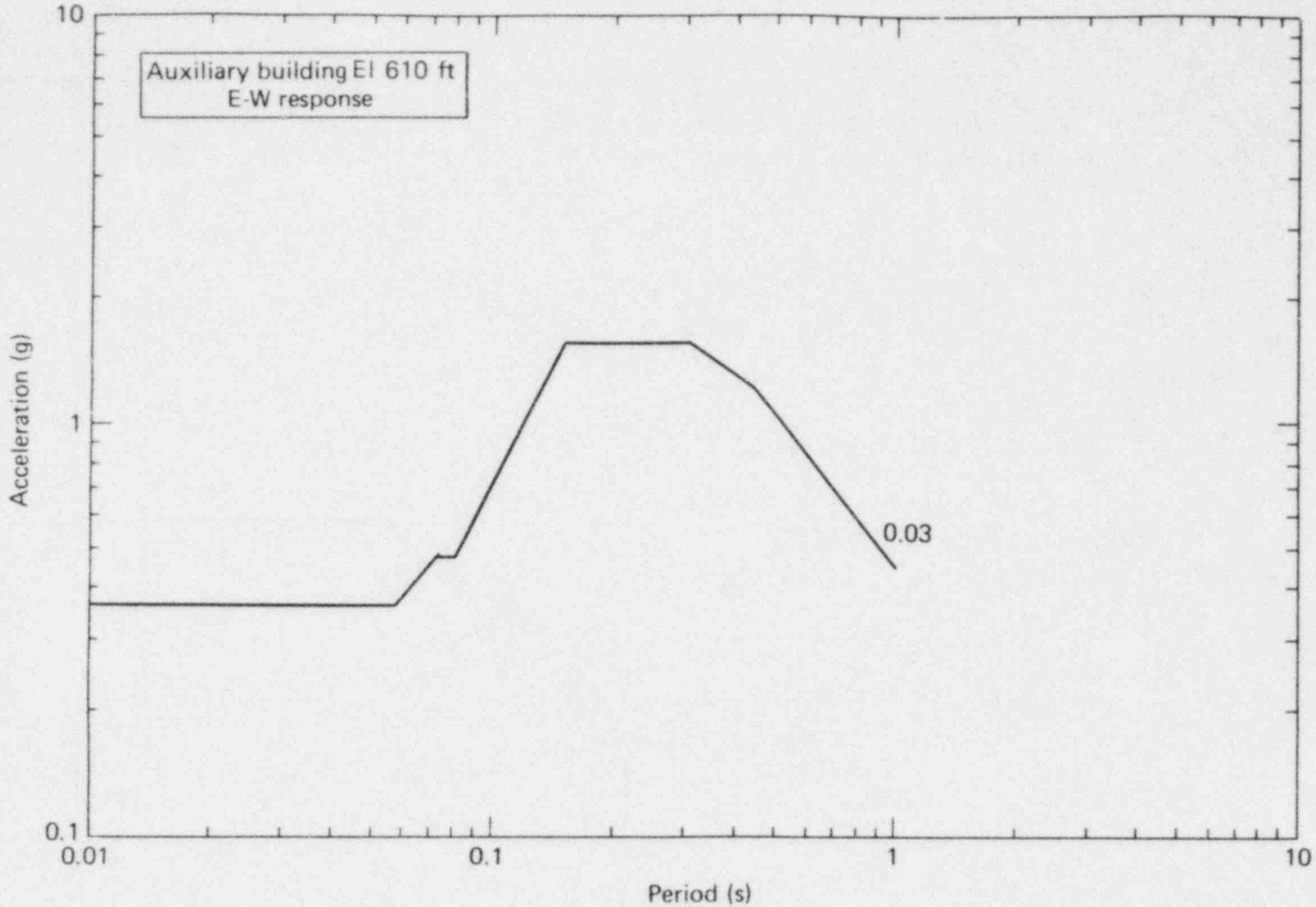

FIG. B-13. East-west horizontal in-structure response spectrum for the auxiliary building (El 610 ft), based on R.G. 1.60 spectra. The spectrum envelops the total range of soil properties. The equipment damping ratio is 0.03.

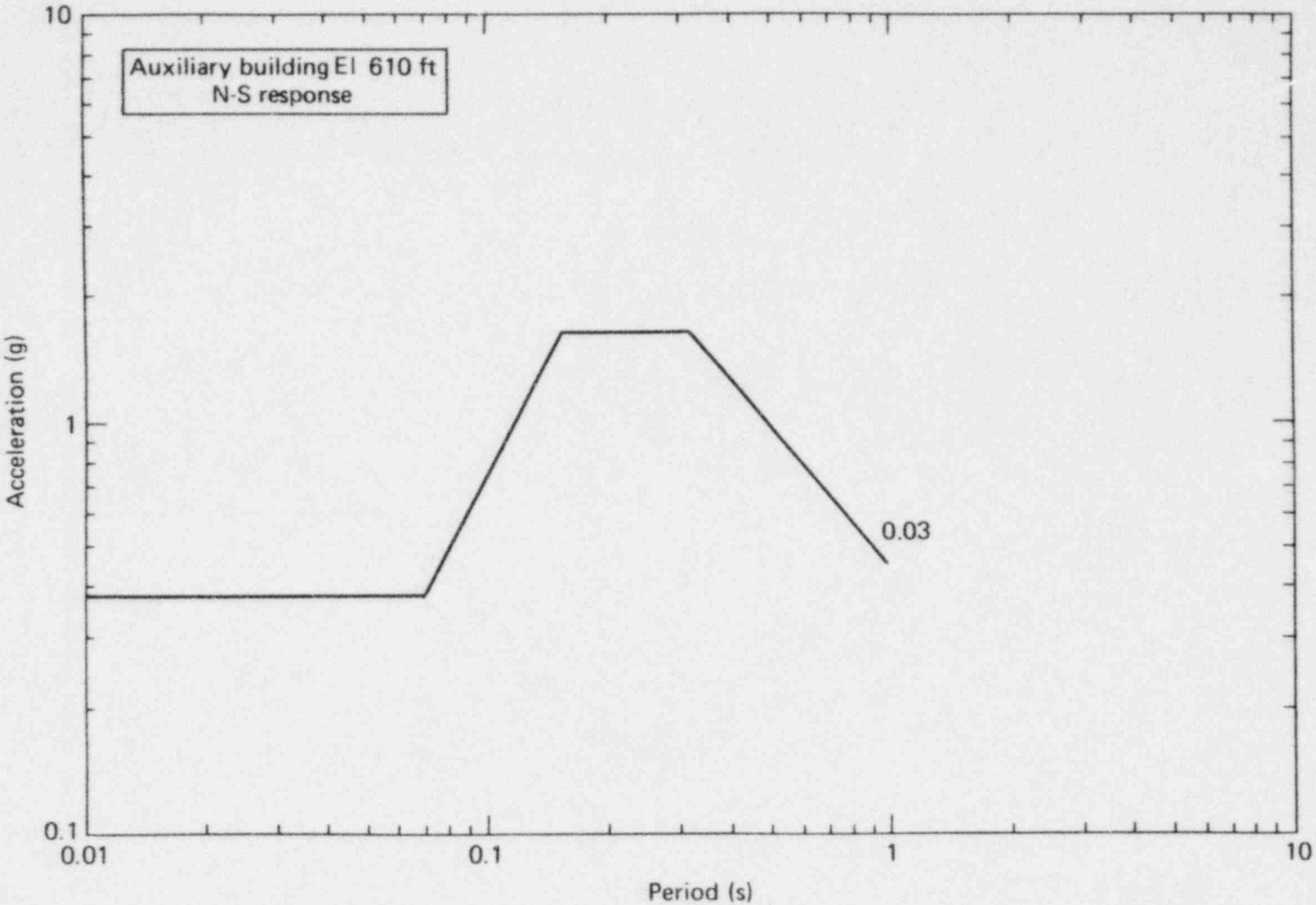

FIG. B-14. North-south horizontal in-structure response spectrum for the auxiliary building (El 610 ft), based on R.G. 1.60 spectra. The spectrum envelops the total range of soil properties. The equipment damping ratio is 0.03.

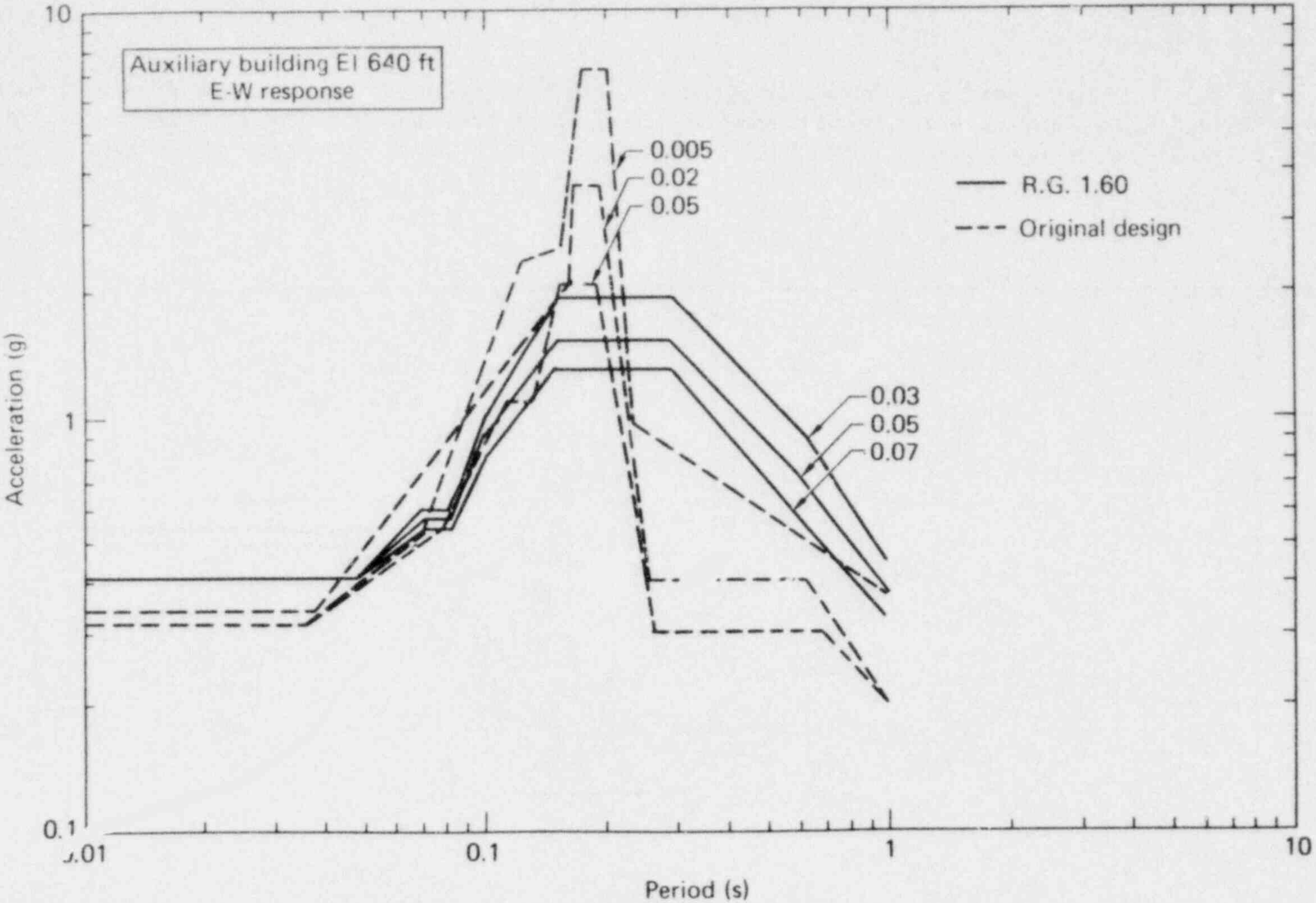

FIG. B-15. Comparison of E-W horizontal in-structure response spectra for the auxiliary building (El 640 ft). The spectra recalculated on the basis of R.G. 1.60 envelop the total range of soil properties. Labels indicate equipment damping ratios.

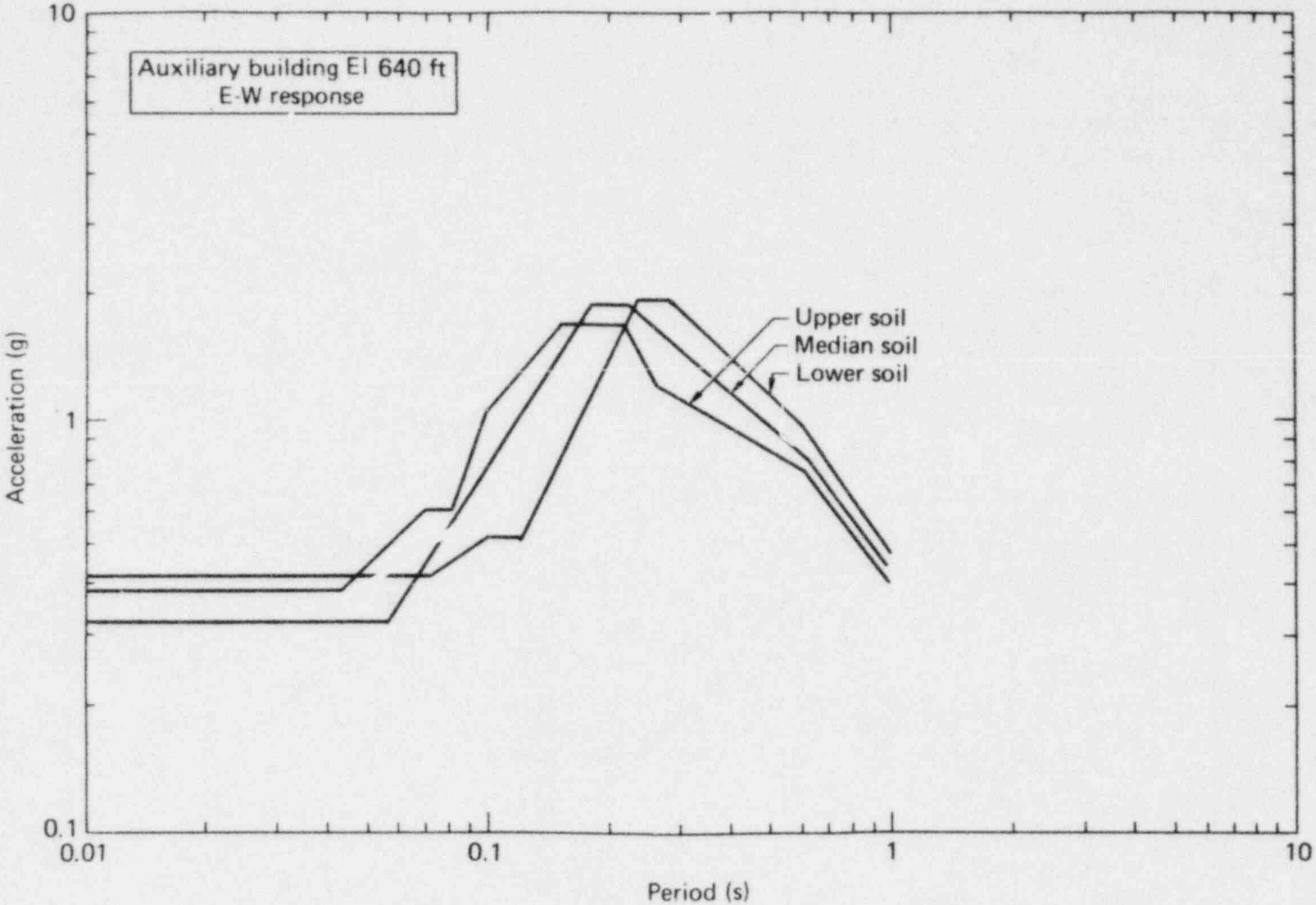

FIG. B-16. East-west horizontal in-structure response spectra for the auxiliary building (El 640 ft), based on R.G. 1.60 spectra. The spectra illustrate the variation with the different soil conditions discussed in the text. The equipment damping ratio was 0.03 for all three spectra.

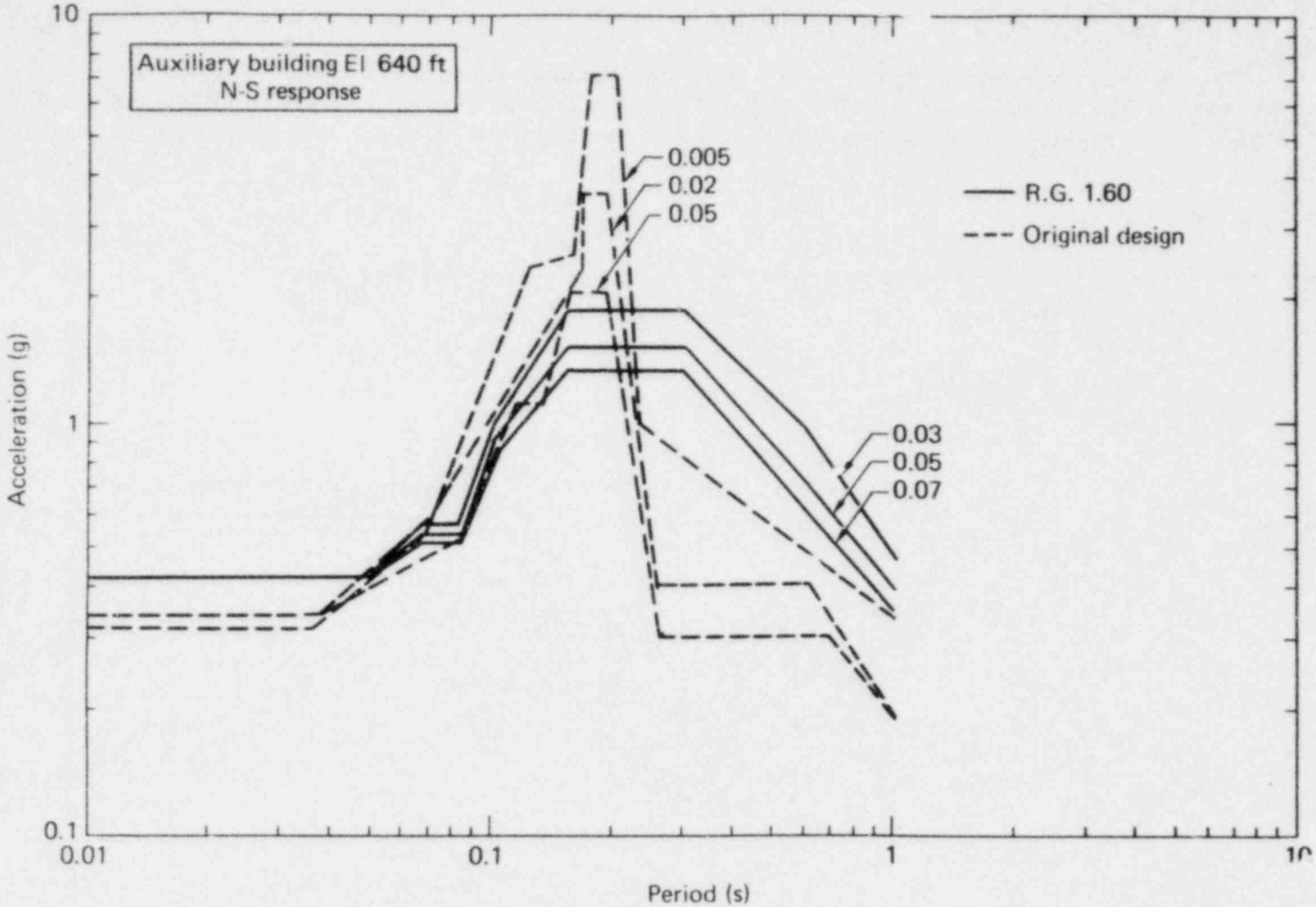

FIG. B-17. Comparison of N-S horizontal in-structure response spectra for the auxiliary building (El 640 ft). The spectra recalculated on the basis of R.G. 1.60 envelop the total range of soil properties. Labels indicate equipment damping ratios.

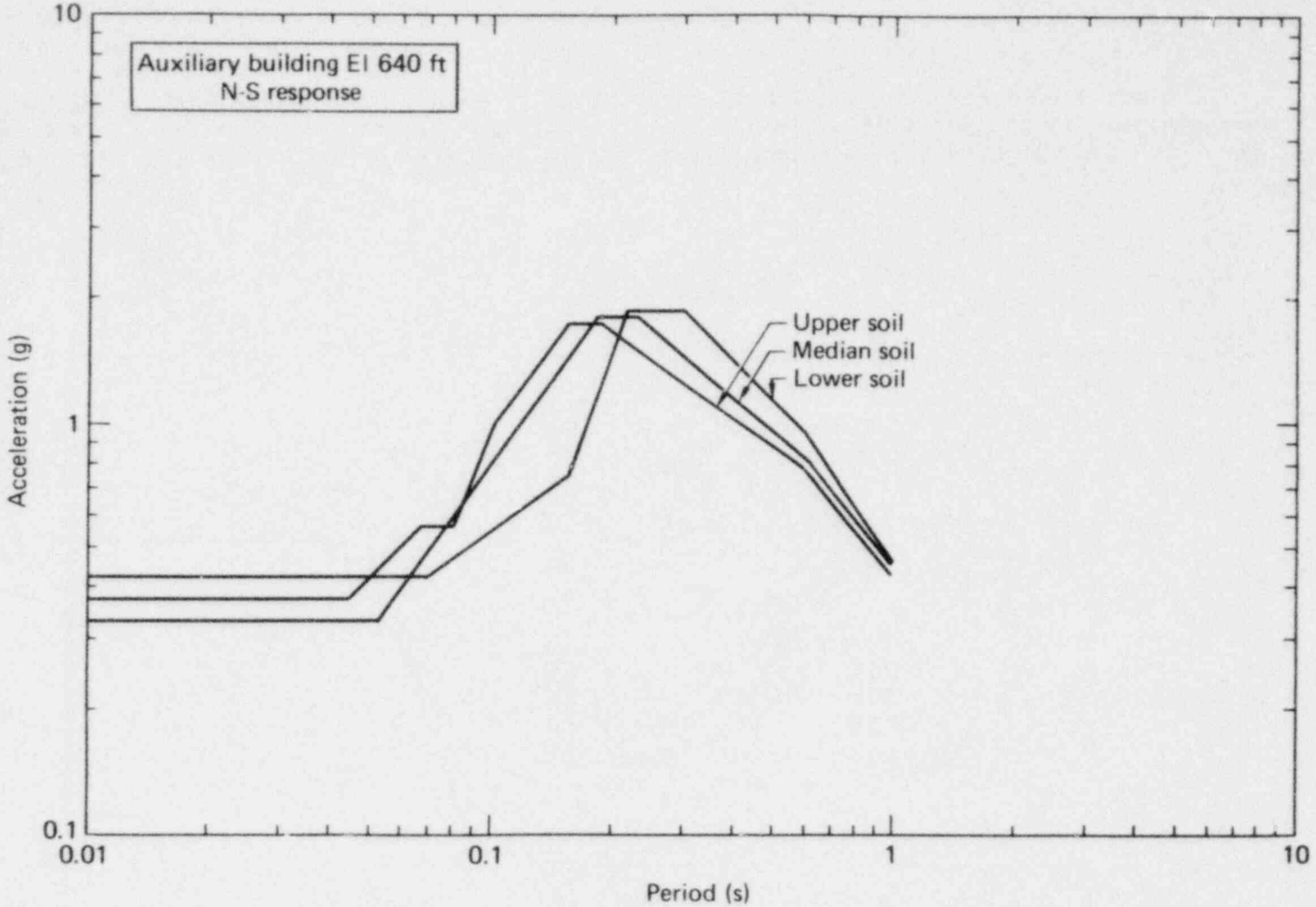

FIG. B-18. North-south horizontal in-structure response spectra for the auxiliary building (El 640 ft), based on R.G. 1.60 spectra. The spectra illustrate the variation with the different soil conditions discussed in the text. The equipment damping ratio was 0.03 for all three spectra.

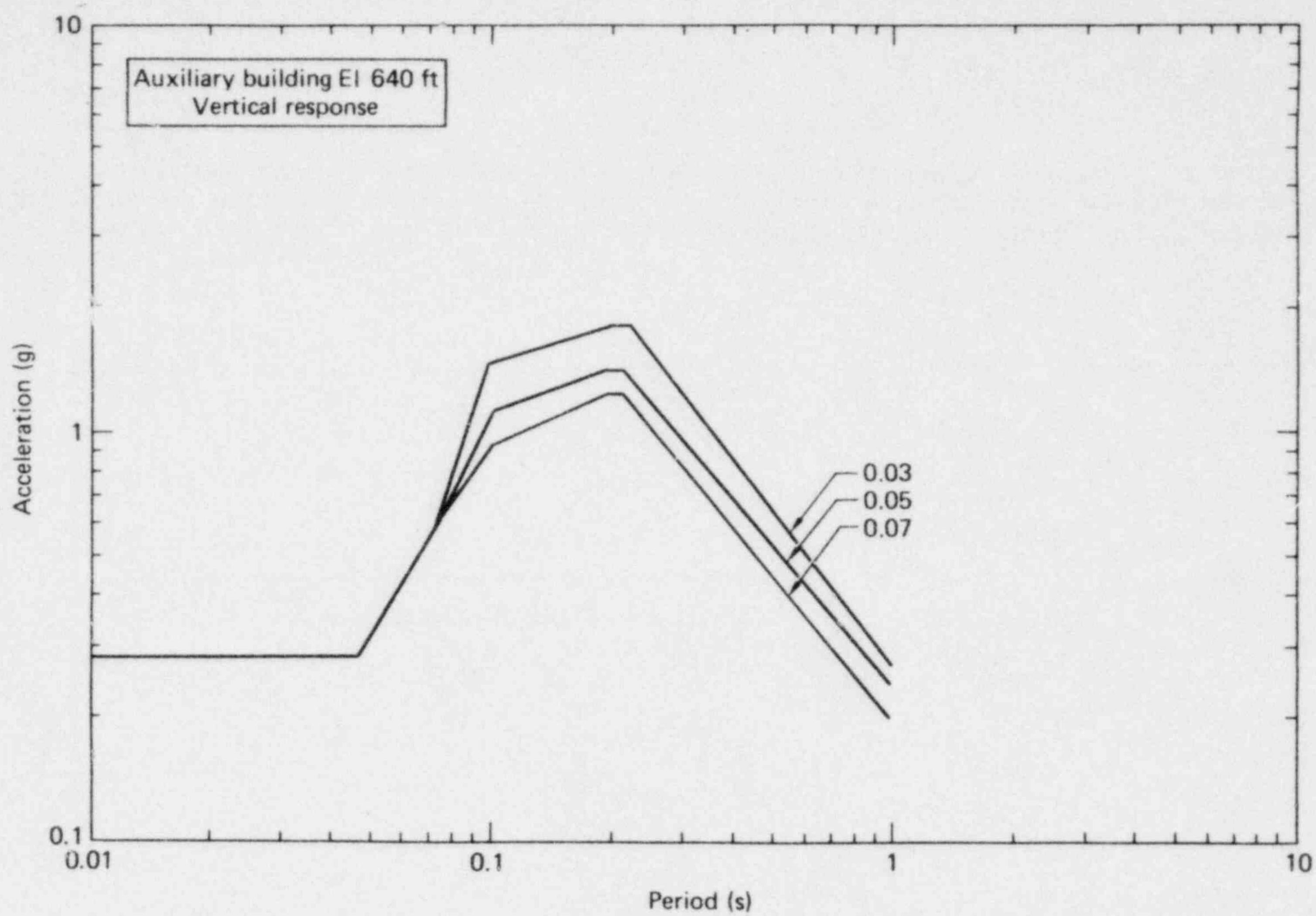

FIG. B-19. Vertical in-structure response spectra for the auxiliary building (El 640 ft), based on R.G. 1.60 spectra. The spectra envelop the total range of soil properties. Labels indicate equipment damping ratios. These spectra are typical for all elevations in the auxiliary building.

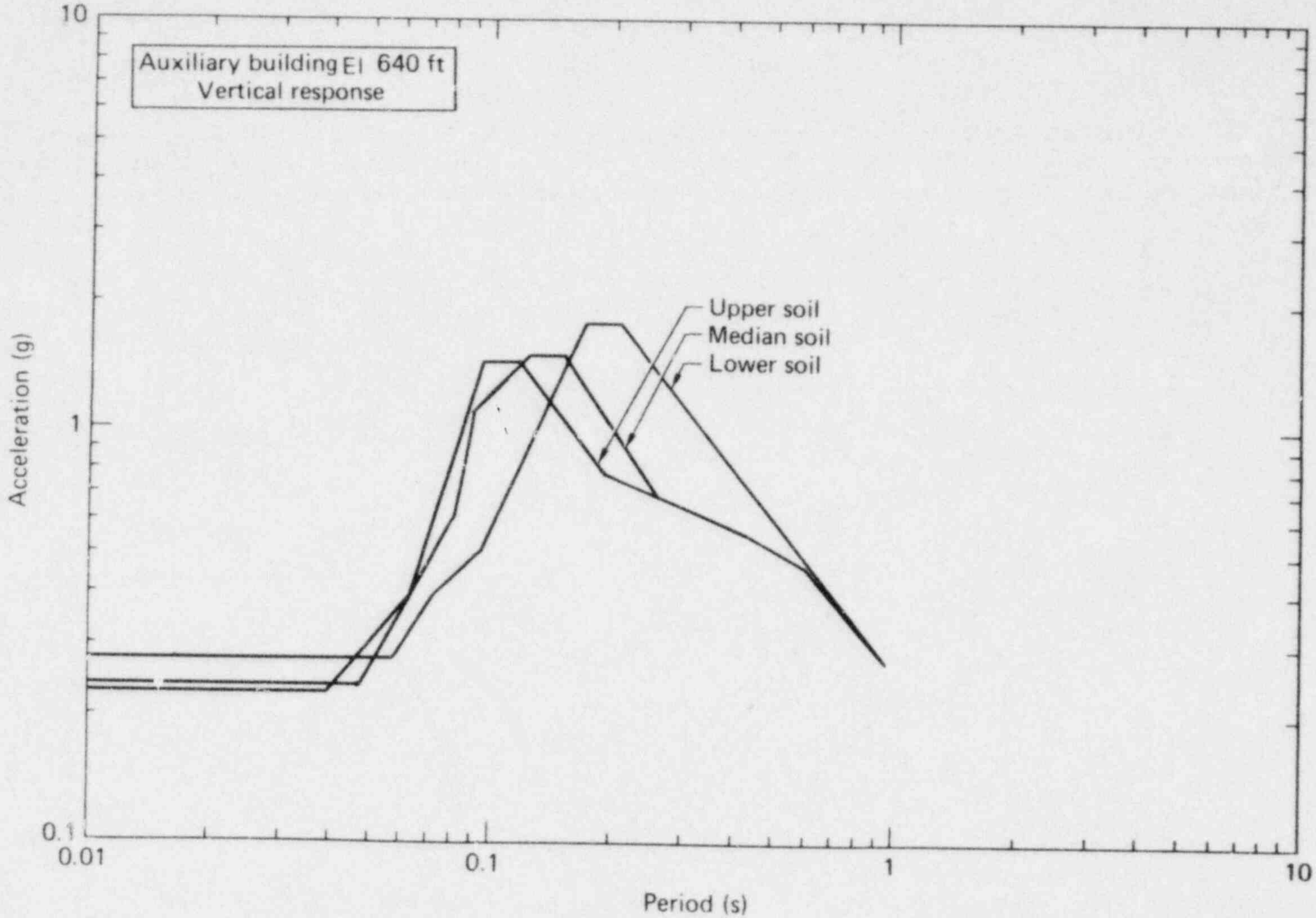

FIG. B-20. Vertical in-structure response spectra for the auxiliary building (El 640 ft), based on R.G. 1.60 spectra. The spectra illustrate the variation with the different soil conditions discussed in the text. The equipment damping ratio was 0.03 for all three spectra. The spectra are typical for all elevations in the auxiliary building.

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16. ABSTRACT (200 words or less)

A limited seismic reassessment of Unit 1 of the Palisades Nuclear Power Plant was performed by the Lawrence Livermore National Laboratory for the U. S. Nuclear Regulatory Commission as part of the Systematic Evaluation Program. The reassessment focused generally on the reactor coolant pressure boundary and on those systems and components necessary to shut down the reactor safety and to maintain it in a safe shutdown condition following a postulated earthquake characterized by a peak horizontal ground acceleration of 0.2g. Unlike a comprehensive design analysis, the reassessment was limited to structures and components deemed representative of generic classes. Conclusions and recommendations about the ability of selected structures and equipment to withstand the postulated earthquake are presented.

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