FINAL SAFETY ANALYSIS REPORT

SAN ONOFRE NUCLEAR GENERATING STATION
UNITS 2 & 3 · VOLUME 6

Southern California Edison Company

SAN DIEGO GAS & ELECTRIC COMPANY

FOREWORD

The Final Safety Analysis Report (FSAR) for the San Onofre Nuclear Generating Station Units 2 and 3 was prepared based upon Nuclear Regulatory Commission (NRC) Regulatory Guide 1.70, Standard Format and Content of Safety Analysis Reports for Nuclear Power Plants, Revision 2. In addition, appendices have been added to facilitate the organization or presentation of information and to provide additional information.

Standards used for editorial abbreviations and symbols are the latest editions of the following IEEE-approved American National Standards Institute publications: ANSI-Y1.1, Abbreviations; ANSI-Y10.19, Letter Symbols for Units Used in Science and Technology; and ANSI-Y10.5, Letter Symbols for Quantities Used in Electrical Science and Electrical Engineering.

All text pages are numbered by chapter and section. Tables and illustrations are numbered in a similar manner; e.g., table 1.1-1 is the first table in section 1.1. Each table is placed in the text following the page on which it is first referenced; figures are placed at the end of each section.

Appendices are identified by section or chapter number with a suffixed letter and are placed following the applicable section or chapter.

Ammendments to the FSAR are identified by a bold line and the amendment number in the outside margin. The number and date of the most recent amendment affecting a page is placed at the bottom of that page. A list of effective pages is submitted with each amendment to provide a guide for inserting and removing pages.

Questions and Responses initiating amendments to the FSAR appear in separate volumes subdivided by tabs identifying the functional branches originating the questions. References are provided indicating corresponding changes to the text.

CONTENTS

VOLUME 1

1. INTRODUCTION AND GENERAL DESCRIPTION OF PLANT
1.1 Introduction
1.2 General Plant Description
1.2.1 Site Description
1.2.2 Plant Arrangement
1.2.3 Nuclear Steam Supply System (NSSS)
1.2.4 Engineered Safety Features
1.2.5 Instrumentation and Controls
1.2.6 Electrical Systems
1.2.7 Power Conversion System
1.2.8 Heating, Ventilating, and Air Conditioning Systems
1.2.9 Fuel Handling and Storage
1.2.10 Auxiliary Systems
1.2.11 Radioactive Waste Management Systems
1.2.12 Particularly Difficult Engineering Problems
1.2.13 Extrapolation of Technology
1.3 Comparison Tables
1.3.1 Comparisons with Similar Facility Designs
1.3.2 Comparison of Final and Preliminary Information
1.4 Identification of Agents and Contractors
1.4.1 Applicants
1.4.2 Nuclear Steam Supply System (NSSS) Supplier
1.4.3 Engineer-Constructor
1.4.4 Turbine-Generator Supplier
1.4.5 Consultants
1.5 Requirements for Further Technical Information
1.5.1 Fretting and Vibrations Tests of Fuel Assemblies
1.5.2 Departure from Nucleate Boiling (DNB) Testing
1.5.3 Fuel Assembly Structural Tests
1.5.4 Fuel Assembly Flow Mixing Tests
1.5.5 Reactor Flow Model Testing and Evaluation
1.5.6 Fuel Assembly Flow Tests
1.5.7 Control Element Drive Mechanism (CEDM) Tests
1.5.8 DNB Improvement
REFERENCES
1.6 Material Incorporated by Reference
1.7 Electrical, Instrumentation, and Control Drawings
2. SITE CHARACTERISTICS
2.1 Geography and Demography
2.1.1 Site Location and Description
2.1.2 Exclusion Area Authority and Control
2.1.3 Population Distribution
REFERENCES
2.2 Nearby Industrial, Transportation, and Military Facilities

(NOTE: Tables and figures are listed in each chapter table of contents)

CONTENTS

VOLUME 1 (cont)

2.2.1 Location and Routes
2.2.2 Descriptions
2.2.3 Evaluation of Potential Accidents
REFERENCES

VOLUME 2

2.3 Meteorology
2.3.1 Regional Climatology
2.3.2 Local Meteorology
2.3.3 Onsite Meteorological Measurement Program
2.3.4 Short-Term (Accident) Diffusion Estimates
REFERENCES
BIBLIOGRAPHY
APPENDIX 2.3A Study of California Tornadoes
APPENDIX 2.3B Annual Joint Frequency Wind Speed-Wind Direction Summaries for Pasquill Stability Categories at the 10-Meter Level and the 36.6- and 40-Meter Levels Combined for the Plant Site (Period of Record 1/25/73 - 1/24/76) (Tables 2.3B-1 through 2.3B-14)
APPENDIX 2.3C Annual Joint Frequency Wind Speed-Wind Direction Summaries by Pasquill Stability Category for San Diego, California (Period of Record 1955-1964) from the Star Program (Tables 2.3C-1 through 2.3C-8)
APPENDIX 2.3D Annual Joint Frequency Wind Speed-Wind Direction Summaries by Pasquill Stability Category for Los Angeles, California (Period of Record 1955-1964) from the Star Program (Tables 2.3D-1 through 2.3D-8)
APPENDIX 2.3E Annual Joint Frequency Wind Speed-Wind Direction Summaries by Pasquill Stability Category for San Diego and Los Angeles, California (Period of Record 1973-1974) from the Star Program (Tables 2.3E-1 through 2.3E-16)
APPENDIX 2.3F Monthly Joint Frequency Wind Speed-Wind Direction Summaries by Pasquill Stability Category at the 10-Meter Wind Level for the Plant Site (Period of Record 1/25/73 - 1/24/76) (Tables 2.3F-1 through 2.3F-84)

CONTENTS

VOLUME 2 (cont)

APPENDIX 2.3G Monthly Joint Frequency Wind Speed-Wind Direction Summaries by Pasquill Stability Category at the Upper Wind Levels (36.6 m and 40 m) for the Plant Site (Period of Record 1/25/73-1/24/76) (Tables 2.3G-1 through 2.3G-84)

APPENDIX 2.3H Annual and Monthly Joint Frequency Wind Speed-Wind Direction Summaries at the 10 Meter and 36.6/40 Meter Wind Levels for the San Onofre Plant Site (Tables 2.3H-1 through 2.3H-26)

APPENDIX 2.3I Annual and Monthly Wind Direction Persistence (Duration) Summaries at 10 Meter and 36.6/40 Meter Wind Levels for the San Onofre Plant Site (Tables 2.3I-1 through 2.3I-26)

APPENDIX 2.3J Diurnal Wind Variation Tables for the 10 Meter Level at the San Onofre Plant Site (Tables 2.3J-1 through 2.3J-26)

APPENDIX 2.3K Diurnal Wind Variation Tables for the 36.6/40 Meter Levels at the San Onofre Plant Site (Tables 2.3K-1 through 2.3K-26)

APPENDIX 2.3L Diurnal Variation and Persistence of Pasquill Stability Categories and Low Level Inversions at the San Onofre Plant Site (Tables 2.3L-1 through 2.3L-26)

2.4 Hydrologic Engineering

2.4.1 Hydrologic Description

2.4.2 Floods

2.4.3 Probable Maximum Flood on Streams and Rivers

2.4.4 Potential Dam Failures, Seismically Induced

2.4.5 Probable Maximum Surge and Seiche Flooding

2.4.6 Probable Maximum Tsunami Flooding

2.4.7 Ice Effects

2.4.8 Cooling Water Canals and Reservoirs

2.4.9 Channel Diversions

2.4.10 Flooding Protection Requirements

2.4.11 Low Water Considerations

2.4.12 Dispersion, Dilution, and Travel Times of Accidental Releases of Liquid Effluents in Surface Waters

2.4.13 Groundwater

2.4.14 Technical Specification and Emergency Operation Requirements

REFERENCES

APPENDIX 2.4A Report on Pumping Test San Onofre Nuclear Generating Station Units 2 and 3

CONTENTS

VOLUME 3

2.5 Geology, Seismology, and Geotechnical Engineering
2.5.1 Basic Geologic and Seismic Information
2.5.2 Vibratory Ground Motion
2.5.3 Surface Faulting
2.5.4 Stability of Subsurface Materials and Foundations
2.5.5 Slope Stability
2.5.6 Embankments and Dams
REFERENCES
BIBLIOGRAPHY

VOLUME 4

APPENDIX 2.5A Soil Liquefaction
APPENDIX 2.5B Chronology of the Development of the DBE Response Spectrum
APPENDIX 2.5C Soil Borings
APPENDIX 2.5D Laboratory Testing and Soil Property Evaluation
APPENDIX 2.5E Dynamic Switchyard Slope Stability Confirmation

VOLUME 5

3. DESIGN OF STRUCTURES, COMPONENTS, EQUIPMENT, AND SYSTEMS
3.1 Conformance with NRC General Design Criteria
3.1.1 Overall Requirements
3.1.2 Protection by Multiple Fission Product Barriers
3.1.3 Protection and Reactivity Control Systems
3.1.4 Fluid Systems
3.1.5 Reactor Containment
3.1.6 Fuel and Radioactivity Control
3.2 Classification of Structures, Components, and Systems
3.2.1 Seismic Classifications
3.2.2 System Quality Group Classifications
3.2.3 Quality Assurance Program Classifications
3.3 Wind and Tornado Loadings
3.3.1 Wind Loadings
3.3.2 Tornado Loadings
REFERENCES
3.4 Water Level (Flood) Design
3.4.1 Flood Protection
3.4.2 Analysis Procedures
3.5 Missile Protection
3.5.1 Missile Selection and Descriptions

CONTENTS

VOLUME 5 (cont)

3.5.2 Systems to be Protected
3.5.3 Barrier Design Procedures
REFERENCES
APPENDIX 3.5A Analysis of Probability of Ejecting Turbine Missiles
3.6 Protection Against Dynamic Effects Associated with the Postulated Rupture of Piping
3.6.1 Postulated Piping Failures in Fluid Systems
3.6.2 Determination of Break Locations and Dynamic Effects Associated with the Postulated Rupture of Piping
REFERENCES
APPENDIX 3.6A High Energy Pipe Break Analysis
3.7 Seismic Design
3.7.1 Seismic Input
3.7.2 Seismic System Analysis
3.7.3 Seismic Subsystem Analysis
3.7.4 Seismic Instrumentation Program
REFERENCES

VOLUME 6

APPENDIX 3.7A Seismic Response Spectra
APPENDIX 3.7B Seismic Analysis of Piping Systems
APPENDIX 3.7C Soil Structure Interaction Parameters
3.8 Design of Category I Structures
3.8.1 Concrete Containment
3.8.2 Steel Containment
3.8.3 Concrete and Steel Internal Structures of Steel or Concrete Containments
3.8.4 Other Seismic Category I Structures
3.8.5 Foundations
REFERENCES

VOLUME 7

3.9 Mechanical Systems and Components
3.9.1 Special Topics for Mechanical Components
3.9.2 Dynamic System Analysis and Testing
3.9.3 ASME Code Class 1, 2, and 3 Components, Component Supports and Core Support Structures
3.9.4 Control Element Drive Mechanisms
3.9.5 Reactor Pressure Vessel Internals

CONTENTS

VOLUME 7 (cont)

3.9.6 Inservice Testing of Pumps and Valves
REFERENCES
3.10 Seismic Qualification of Seismic Category I Instrumentation and Electrical Equipment
3.10.1 Seismic Qualification Criteria
3.10.2 Methods and Procedures for Qualifying Electrical Equipment and Instrumentation
3.10.3 Methods and Procedures of Analysis or Testing of Supports of Electrical Equipment and Instrumentation
3.10.4 Operating License Review
REFERENCES
APPENDIX 3.10A Criteria for Seismic Qualification of Seismic Category I Equipment
APPENDIX 3.10B Results of Seismic Tests and Analyses, Seismic Category I Instrumentation and Electrical Equipment
3.11 Environmental Design of Mechanical and Electrical Equipment
3.11.1 Equipment Identification and Environmental Conditions
3.11.2 Qualification Tests and Analyses
3.11.3 Qualification Test Results
3.11.4 Loss of Ventilation
3.11.5 Estimated Chemical and Radiation Environment
REFERENCES
APPENDIX 3.11A Results of Qualification Tests for Safety-Related Mechanical and Electrical Equipment
APPENDIX 3A Comparison of Design with NRC Regulatory Guides
APPENDIX 3B A Functional Evaluation of the Components of the Systems Shared by the Two Units
APPENDIX 3C Computer Programs Used in Structural Analysis
4. REACTOR
4.1 Summary Description
REFERENCES
4.2 Fuel System Design
4.2.1 Design Bases
4.2.2 Description and Design Drawings
4.2.3 Design Evaluation
4.2.4 Testing and Inspection Plan
REFERENCES

VOLUME 8

4.3 Nuclear Design
4.3.1 Design Bases

CONTENTS

VOLUME 8 (cont)

4.3.2 Description
4.3.3 Analytical Methods
4.3.4 Changes
REFERENCES
4.4 Thermal and Hydraulic Design
4.4.1 Design Bases
4.4.2 Description of Thermal and Hydraulic Design of the Reactor Core
4.4.3 Description of the Thermal and Hydraulic Design of the Reactor Coolant System (RCS)
4.4.4 Evaluation
4.4.5 Testing and Verification
4.4.6 Instrumentation Requirements
REFERENCES
4.5 Reactor Materials
4.5.1 Control Element Drive Structural Materials
4.5.2 Reactor Internals Materials
REFERENCES
4.6 Functional Design of Reactivity Control Systems
4.6.1 Information for CRDS
4.6.2 Evaluations of the CRDS
4.6.3 Testing and Verification of the CRDS
4.6.4 Information for Combined Performance of Reactivity Systems
4.6.5 Evaluations of Combined Performance
REFERENCES
5. REACTOR COOLANT SYSTEM AND CONNECTED SYSTEMS
5.1 Summary Description
5.1.1 Schematic Flow Diagram
5.1.2 Piping and Instrument Diagram
5.1.3 Elevation Drawings
5.2 Integrity of Reactor Coolant Pressure Boundary (RCPB)
5.2.1 Compliance with Codes and Code Cases
5.2.2 Overpressurization Protection
5.2.3 Reactor Coolant Pressure Boundary Materials
5.2.4 Inservice Inspection and Testing of Reactor Coolant Pressure Boundary
5.2.5 Detection of Leakage through Reactor Coolant Pressure Boundary
REFERENCES
APPENDIX 5.2A Overpressure Protection for Combustion Engineering 3410 MWt - Pressurized Reactors

CONTENTS

VOLUME 9

5.3 Reactor Vessel
5.3.1 Reactor Vessel Materials
5.3.2 Pressure-Temperature Limits
5.3.3 Reactor Vessel Integrity
REFERENCES
5.4 Component and Subsystem Design
5.4.1 Reactor Coolant Pumps
5.4.2 Steam Generators
5.4.3 Reactor Coolant Piping
5.4.4 Main Steam Line Flow Restrictions
5.4.5 Main Steam Line Isolation System
5.4.6 Reactor Core Isolation Cooling System
5.4.7 Residual Heat Removal System
5.4.8 Reactor Water Cleanup System (BWRs)
5.4.9 Main Steam Line and Feedwater Piping
5.4.10 Pressurizer
5.4.11 Quench Tank (Pressurizer Relief Tank)
5.4.12 Valves
5.4.13 Safety and Relief Valves
5.4.14 Components Support
6. ENGINEERED SAFETY FEATURES
6.1 Engineered Safety Feature Materials
6.1.1 Metallic Materials
6.1.2 Organic Materials
6.1.3 Post-Accident Chemistry
6.2 Containment Systems
6.2.1 Containment Functional Design
6.2.2 Containment Heat Removal Systems
6.2.3 Secondary Containment Functional Design
6.2.4 Containment Isolation System
6.2.5 Combustible Gas Control in Containment
6.2.6 Containment Leakage Testing
REFERENCES
BIBLIOGRAPHY

VOLUME 10

6.3 Emergency Core Cooling System
6.3.1 Design Basis
6.3.2 System Design
6.3.3 Performance Evaluation
6.3.4 Tests and Inspections
6.3.5 Instrumentation Requirements

VOLUME 10 (cont)

REFERENCES
BIBLIOGRAPHY
6.4 Habitability Systems
6.4.1 Design Bases
6.4.2 System Design
6.4.3 System Operational Procedures
6.4.4 Design Evaluation
6.4.5 Testing and Inspection
6.4.6 Instrumentation Requirement
REFERENCES
6.5 Fission Product Removal and Control Systems
6.5.1 Engineered Safety Feature (ESF) Filter Systems
6.5.2 Containment Air Purification and Cleanup--Iodine Removal System
6.5.3 Fission Product Control Systems
6.5.4 Ice Condenser as a Fission Product Cleanup System
REFERENCES
6.6 Inservice Inspection of Class 2 and 3 Components
6.6.1 Components Subject to Examination
6.6.2 Accessibility
6.6.3 Examination Techniques and Procedures
6.6.4 Inspection Intervals
6.6.5 Examination Categories and Requirements
6.6.6 Evaluation of Examination Results
6.6.7 System Pressure Tests
6.6.8 Augmented Inservice Inspection to Protect Against Postulated Piping Failures
7. INSTRUMENTATION AND CONTROLS
7.1 Introduction
7.1.1 Identification of Safety-Related Systems
7.1.2 Identification of Safety Criteria
REFERENCES
7.2 Reactor Protective System
7.2.1 Description
7.2.2 Analysis

VOLUME 11

7.3 Engineered Safety Features Systems
7.3.1 Description
7.3.2 Analysis
7.4 Systems Required for Safe Shutdown
7.4.1 Description
7.4.2 Analysis
7.5 Safety-Related Display Instrumentation
7.5.1 Description
7.5.2 Analysis

CONTENTS

VOLUME 11

7.6 All Other Instrumentation Systems Required for Safety
7.6.1 Description
7.6.2 Analysis
7.7 Control Systems Not Required for Safety
7.7.1 Description
7.7.2 Design Comparison
7.7.3 Analysis
REFERENCES
8. ELECTRIC POWER
8.1 Introduction
8.1.1 Utility Grid Description
8.1.2 Onsite Power System Description
8.1.3 Safety-Related Loads
8.1.4 Design Bases
8.2 Offsite Power System
8.2.1 Description
8.2.2 Analysis
APPENDIX 8.2A Bulk Power Transmission System Reliability Criteria
8.3 Onsite Power Systems
8.3.1 AC Power Systems
8.3.2 DC Power Systems
8.3.3 Fire Protection for Cable Systems

VOLUME 12

9. AUXILIARY SYSTEMS
9.1 Fuel Storage and Handling
9.1.1 New Fuel Storage
9.1.2 Spent Fuel Storage
9.1.3 Spent Fuel Pool Cooling and Cleanup System
9.1.4 Fuel Handling System
9.2 Water Systems
9.2.1 Saltwater Cooling System
9.2.2 Component Cooling Water System
9.2.3 Makeup Demineralizer System
9.2.4 Domestic Water System
9.2.5 Ultimate Heat Sink
9.2.6 Condensate Storage and Transfer System
9.2.7 Nuclear Service Water System
9.2.8 Turbine Plant Cooling Water System
9.3 Process Auxiliaries
9.3.1 Compressed Air System
9.3.2 Process Sampling System

CONTENTS

VOLUME 12 (cont)

9.3.3 Sump and Drain Systems
9.3.4 Chemical and Volume Control System
REFERENCES
9.4 Air Conditioning, Heating, Cooling, and Ventilation Systems
9.4.1 Containment Building Ventilation Systems
9.4.2 Auxiliary Building Ventilation Systems
9.4.3 Support Building Ventilation Systems

VOLUME 13

9.5 Other Auxiliary Systems
9.5.1 Fire Protection System
9.5.2 Communication Systems
9.5.3 Lighting Systems
9.5.4 Diesel Generator Fuel Oil Storage and Transfer System
9.5.5 Diesel Generator Cooling Water System
9.5.6 Diesel Generator Starting System
9.5.7 Diesel Generator Lubrication System
9.5.8 Diesel Generator Combustion Air Intake and Exhaust System
10. STEAM AND POWER CONVERSION SYSTEM
10.1 Summary Description
10.1.1 General Description
10.1.2 Protective Features
10.2 Turbine-Generator
10.2.1 Design Bases
10.2.2 Description
10.2.3 Turbine Disc Integrity
10.2.4 Evaluation
10.3 Main Steam Supply System
10.3.1 Design Bases
10.3.2 System Description
10.3.3 Evaluation
10.3.4 Inspection and Testing Requirements
10.3.5 Secondary Water Chemistry (PWR)
10.3.6 Steam and Feedwater System Materials
REFERENCES
10.4 Other Features of Steam and Power Conversion System
10.4.1 Main Condenser
10.4.2 Main Condenser Evacuation System
10.4.3 Turbine Gland Sealing System
10.4.4 Turbine Bypass System
10.4.5 Circulating Water System
10.4.6 Condensate Cleanup System

CONTENTS

VOLUME 13 (cont)

10.4.7 Condensate and Feedwater System
10.4.8 Steam Generator Blowdown Processing System
10.4.9 Auxiliary Feedwater System
10.4.10 Turbine Plant Chemical Addition System
11. RADIOACTIVE WASTE MANAGEMENT
11.1 Source Terms
11.1.1 Design Basis Source Terms
11.1.2 Deposited Corrosion Products
11.1.3 Neutron Activation Products
11.1.4 Tritium Production in Reactor Coolant
11.1.5 Tritium Concentrations and Releases
11.1.6 Spent Fuel Fission Product Activities
11.1.7 Leakage Sources
11.1.8 Steam Generator Activity Model
REFERENCES
APPENDIX 11.1A Derivation of Residence Times
11.2 Liquid Waste Management Systems
11.2.1 Design Bases
11.2.2 System Descriptions
11.2.3 Radioactive Releases
11.3 Gaseous Waste Management Systems
11.3.1 Design Bases
11.3.2 System Descriptions
11.3.3 Radioactive Releases
11.4 Solid Waste Management System
11.4.1 Design Bases
11.4.2 System Description
11.5 Process and Effluent Radiological Monitoring and Sampling Systems
11.5.1 Design Bases
11.5.2 System Description
11.5.3 Effluent Monitoring and Sampling
11.5.4 Process Monitoring and Sampling

VOLUME 14

12. RADIATION PROTECTION
12.1 Ensuring that Occupational Radiation Exposures are as Low as is Reasonably Achievable (ALARA)
12.1.1 Policy Considerations
12.1.2 Design Considerations
12.1.3 Operational Considerations
REFERENCES

CONTENTS

VOLUME 14 (cont)

12.2 Radiation Sources
12.2.1 Contained Sources
12.2.2 Airborne Radioactive Material Sources
12.3 Radiation Protection Design Features
12.3.1 Facility Design Features
12.3.2 Shielding
12.3.3 Ventilation
12.3.4 Area Radiation and Airborne Radioactivity Monitoring Instrumentation
REFERENCES
12.4 Dose Assessment
12.4.1 Exposures Within the Plant
12.4.2 Exposures at Locations Outside Plant Structures
12.4.3 Exposures to Construction
REFERENCES
12.5 Health Physics Program
12.5.1 Organization
12.5.2 Equipment, Instrumentation, and Facilities
12.5.3 Procedures
13. CONDUCT OF OPERATIONS
13.1 Organizational Structure of Applicant
13.1.1 Management and Technical Support Organization
13.1.2 Operating Organization
13.1.3 Qualifications of Nuclear Plant Personnel
APPENDIX 13.1A Resumes
13.2 Training
13.2.1 Plant Staff Training Program
13.2.2 Replacement and Retraining
13.2.3 Applicable NRC Documents
13.3 Emergency Planning
13.4 Review and Audit
13.4.1 Onsite Review
13.4.2 Independent Review
13.4.3 Audit Program
13.5 Plant Procedures
13.5.1 Administrative Procedures
13.5.2 Operating and Maintenance Procedures
13.6 Industrial Security
14. INITIAL TEST PROGRAM
14.1 Specific Information to be Included in PSAR
14.2 Specific Information to be Included in FSAR
14.2.1 Summary of Test Program and Objectives
14.2.2 Organization and Staffing
14.2.3 Test Procedures
14.2.4 Conduct of Test Program

CONTENTS

VOLUME 14 (cont)

14.2.5 Review, Evaluation and Approval of Test Results
14.2.6 Test Records
14.2.7 Comparison of Test Programs with Regulatory Guides
14.2.8 Utilization of Reactor Operating and Testing Experiences in Development of Test Program
14.2.9 Trial Use of Plant Operating and Emergency Procedures
14.2.10 Initial Fuel Load and Initial Criticality
14.2.11 Test Program Schedule
14.2.12 Individual Test Descriptions

VOLUME 15

15.0 TRANSIENT ANALYSES
15.0.1 Identification of Causes and Frequency Classification
15.0.2 Systems Operation
15.0.3 Core and System Performance
15.0.4 Barrier Performance
15.0.5 Radiological Consequences
REFERENCES
15.1 Increase in Heat Removal by the Secondary System (Turbine Plant)
15.1.1 Moderate Frequency Incidents
15.1.2 Infrequent Incidents
15.1.3 Limiting Faults
REFERENCES
15.2 Decrease in Heat Removal by the Secondary System (Turbine Plant)
15.2.1 Moderate Frequency Incidents
15.2.2 Infrequent Incidents
15.2.3 Limiting Faults
REFERENCES

VOLUME 16

15.3 Decrease in Reactor Coolant Flowrate
15.3.1 Moderate Frequency Incidents
15.3.2 Infrequent Incidents
15.3.3 Limiting Faults
REFERENCES
15.4 Reactivity and Power Distribution Anomalies
15.4.1 Moderate Frequency Incidents
15.4.2 Infrequent Events
REFERENCES

CONTENTS

VOLUME 16 (cont)

15.5 Increase in Reactor Coolant System Inventory
15.5.1 Moderate Frequency Incidents
15.5.2 Infrequent Incidents
15.5.3 Limiting Faults

VOLUME 17

15.6 Decrease in Reactor Coolant Inventory
15.6.1 Moderate Frequency Incidents
15.6.2 Infrequent Incidents
15.6.3 Limiting Faults
REFERENCES
15.7 Radioactive Release from a Subsystem or Component
15.7.1 Moderate Frequency Incidents
15.7.2 Infrequent Incidents
15.7.3 Limiting Faults
REFERENCES
15.8 Anticipated Transients Without SCRAM (ATWS)
REFERENCES
APPENDIX 15A Specification of LCO on Required Margin to DNB
APPENDIX 15B Dose Models Used to Evaluate the Environmental Consequences of Accidents
16. TECHNICAL SPECIFICATIONS
16.1 Definitions
16.1.1 Defined Terms
16.1.2 Thermal Power
16.1.3 Rated Thermal Power
16.1.4 Operational Mode
16.1.5 Action
16.1.6 Operable - Operability
16.1.7 Reportable Occurrence
16.1.8 Containment Integrity
16.1.9 Channel Calibration
16.1.10 Channel Check
16.1.11 Channel Functional Test
16.1.12 Core Alteration
16.1.13 Shutdown Margin
16.1.14 Identified Leakage
16.1.15 Unidentified Leakage
16.1.16 Pressure Boundary Leakage
16.1.17 Controlled Leakage
16.1.18 Azimuthal Power Tilt - T_q
16.1.19 Dose Equivalent I-131
16.1.20 E - Average Disintegration Energy

CONTENTS

VOLUME 17 (cont)

16.1.21 Staggered Test Basis
16.1.22 Frequency Notation
16.1.23 Axial Shape Index
16.1.24 Unrodded Planar Radial Peaking Factor - F_r^P
16.1.25 Engineered Safety Features (ESF)
16.1.26 Reactor Trip System Response Time
16.1.27 Engineered Safety Feature Response Time
16.1.28 Physics Tests
16.2 Safety Limits and Limiting Safety System Settings
16.2.1 Safety Limits
16.2.2 Limiting Safety System Settings (LSSS)
16.3/4 Limiting Conditions for Operation and Surveillance Requirements
16.3/4.1 Reactivity Control Systems
16.3/4.2 Power Distribution Limits
16.3/4.3 Instrumentation
16.3/4.4 Reactor Coolant System
16.3/4.5 Emergency Core Cooling Systems (ECCS)
16.3/4.6 Containment Systems
16.3/4.7 Plant Systems
16.3/4.8 Electrical Power Systems
16.3/4.9 Refueling Operations
16.3/4.10 Special Test Exceptions
APPENDIX 16.3/4A Bases for Limiting Conditions for Operation and Surveillance Requirements
16.5 Design Features
16.5.1 Site
16.5.2 Containment
16.5.3 Reactor Core
16.5.4 Reactor Coolant System
16.5.5 Emergency Core Cooling Systems
16.5.6 Fuel Storage
16.5.7 Seismic Classification
16.5.8 Meteorological Tower Location
16.5.9 Component Cyclic or Transient Limits
16.6 Administrative Controls
16.6.1 Responsibility
16.6.2 Organization
16.6.3 Facility Staff Qualifications
16.6.4 Training
16.6.5 Review and Audit
16.6.6 Reportable Occurrence Action (Not Applicable)
16.6.7 Safety Limit Violation
16.6.8 Procedures

CONTENTS

VOLUME 17 (cont)

16.6.9 Reporting Requirements
16.6.10 Record Retention
16.6.11 Radiation Protection Program
16.6.12 Respiratory Protection Program
17. QUALITY ASSURANCE
17.1 Quality Assurance During Design and Construction
17.2 Quality Assurance (QA) During the Operations Phase
17.2.1 NSSS Vendor Quality Assurance During Preoperational and Startup Testing Activities
17.2.2 Engineer-Constructor (Bechtel) Quality Assurance During Preoperational and Startup Testing

APPENDIX 3.7A

SEISMIC RESPONSE SPECTRA

FIGURES

3.7A-1 Design Basis Earthquake Vertical Acceleration Response Spectra, Parallel to Hot Leg Reactor Vessel Column Support

3.7A-2 Design Basis Earthquake Horizontal Acceleration Response Spectra, Parallel to Hot Leg Reactor Vessel Column Support

3.7A-3 Design Basis Earthquake Horizontal Acceleration Response Spectra, Parallel to Hot Leg Reactor Vessel Base Shear Key

3.7A-4 Design Basis Earthquake Horizontal Acceleration Response Spectra, Parallel to Hot Leg Reactor Vessel Nozzle Restraint

3.7A-5 Design Basis Earthquake Vertical Acceleration Response Spectra, Parallel to Hot Leg Steam Generator Base Support

3.7A-6 Design Basis Earthquake Horizontal Acceleration Response Spectra, Parallel to Hot Leg Steam Generator Snubber Support

3.7A-7 Design Basis Earthquake Vertical Acceleration Response Spectra, Parallel to Hot Leg RCP Base Support

3.7A-8 Design Basis Earthquake Horizontal Acceleration Response Spectra, Parallel to Hot Let RCP Lower Horizontal Support

3.7A-9 Design Basis Earthquake Horizontal Acceleration Response Spectra, Parallel to Hot Leg RCP Upper Horizontal Support

3.7A-10 Design Basis Earthquake Horizontal Acceleration Response Spectra, Parallel to Hot Leg RCP Snubber Support

3.7A-11 Design Basis Earthquake Vertical Acceleration Response Spectra, Parallel to Hot Leg Pressurizer Base Support

3.7A-12 Design Basis Earthquake Horizontal Acceleration Response Spectra, Parallel to Hot Leg Pressurizer Base Support

3.7A-13 Design Basis Earthquake Horizontal Acceleration Response Spectra, Parallel to Hot Leg Pressurizer Shear Key Support

3.7A-14 Design Basis Earthquake Vertical Acceleration Response Spectra, Perpendicular to Hot Leg Reactor Vessel Column Support

3.7A-15 Design Basis Earthquake Horizontal Acceleration Response Spectra, Perpendicular to Hot Leg Reactor Vessel Column Support

3.7A-16 Design Basis Earthquake Horizontal Acceleration Response Spectra, Perpendicular to Hot Leg Reactor Vessel Base Shear Key

3.7A-17 Design Basis Earthquake Horizontal Acceleration Response Spectra, Perpendicular to Hot Leg Reactor Vessel Nozzle Restraints

3.7A-18 Design Basis Earthquake Vertical Acceleration Response Spectra, Perpendicular to Hot Leg Steam Generator Base Support

3.7A-19 Design Basis Earthquake Horizontal Acceleration Response Spectra, Perpendicular to Hot Leg Steam Generator Base Support

3.7A-20 Design Basis Earthquake Horizontal Acceleration Response Spectra, Perpendicular to Hot Leg Steam Generator Shear Key Support

3.7A-21 Design Basis Earthquake Vertical Acceleration Response Spectra, Perpendicular to Hot Leg RCP Base Support

3.7A-22 Design Basis Earthquake Horizontal Acceleration Response Spectra, Perpendicular to Hot Leg RCP Lower Horizontal Support

3.7A-23 Design Basis Earthquake Horizontal Acceleration Response Spectra, Perpendicular to Hot Leg RCP Upper Horizontal Support

3.7A-24 Design Basis Earthquake Horizontal Acceleration Response Spectra, Perpendicular to Hot Leg RCP Snubber Support

FIGURES (cont)

3.7A-25 Design Basis Earthquake Vertical Acceleration Response Spectra, Perpendicular to Hot Leg Pressurizer Base Support

3.7A-26 Design Basis Earthquake Horizontal Acceleration Response Spectra, Perpendicular to Hot Leg Pressurizer Base Support

3.7A-27 Design Basis Earthquake Horizontal Acceleration Response Spectra, Perpendicular to Hot Leg Pressurizer Base Support

3.7A-28 Operating Basis Earthquake Vertical Acceleration Response Spectra, Parallel to Hot Leg Reactor Vessel Column Support

3.7A-29 Operating Basis Earthquake Horizontal Acceleration Response Spectra, Parallel to Hot Leg Reactor Vessel Column Support

3.7A-30 Operating Basis Earthquake Horizontal Acceleration Response Spectra, Parallel to Hot Leg Reactor Vessel Base Shear Key

3.7A-31 Operating Basis Earthquake Horizontal Acceleration Response Spectra, Parallel to Hot Leg Reactor Vessel Nozzle Restraint

3.7A-32 Operating Basis Earthquake Vertical Acceleration Response Spectra, Parallel to Hot Leg Steam Generator Base Support

3.7A-33 Operating Basis Earthquake Horizontal Acceleration Response Spectra, Parallel to Hot Leg Steam Generator Snubber Support

3.7A-34 Operating Basis Earthquake Vertical Acceleration Response Spectra, Parallel to Hot Leg RCP Base Support

3.7A-35 Operating Basis Earthquake Horizontal Acceleration Response Spectra, Parallel to Hot Leg RCP Lower Horizontal Support

3.7A-36 Operating Basis Earthquake Horizontal Acceleration Response Spectra, Parallel to Hot Leg RCP Upper Horizontal Support

3.7A-37 Operating Basis Earthquake Horizontal Acceleration Response Spectra, Parallel to Hot Leg RCP Snubber Support

3.7A-38 Operating Basis Earthquake Vertical Acceleration Response Spectra, Parallel to Hot Leg Pressurizer Base Support

3.7A-39 Operating Basis Earthquake Horizontal Acceleration Response Spectra, Parallel to Hot Leg Pressurizer Base Support

3.7A-40 Operating Basis Earthquake Horizontal Acceleration Response Spectra, Parallel to Hot Leg Pressurizer Shear Key Support

3.7A-41 Operating Basis Earthquake Vertical Acceleration Response Spectra, Perpendicular to Hot Leg Reactor Vessel Column Support

3.7A-42 Operating Basis Earthquake Horizontal Acceleration Response Spectra, Perpendicular to Hot Leg Reactor Vessel Column Support

3.7A-43 Operating Basis Earthquake Horizontal Acceleration Response Spectra, Perpendicular to Hot Leg Reactor Vessel Base Shear Key

3.7A-44 Operating Basis Earthquake Horizontal Acceleration Response Spectra, Perpendicular to Hot Leg Reactor Vessel Nozzle Restraints

3.7A-45 Operating Basis Earthquake Vertical Acceleration Response Spectra, Perpendicular to Hot Leg Steam Generator Base Support

3.7A-46 Operating Basis Earthquake Horizontal Acceleration Response Spectra, Perpendicular to Hot Leg Steam Generator Base Support

3.7A-47 Operating Basis Earthquake Horizontal Acceleration Response Spectra, Perpendicular to Hot Leg Steam Generator Shear Key Support

3.7A-48 Operating Basis Earthquake Vertical Acceleration Response Spectra, Perpendicular to Hot Leg RCP Base Support

FIGURES (cont)

3.7A-49 Operating Basis Earthquake Horizontal Acceleration Response Spectra, Perpendicular to Hot Leg RCP Lower Horizontal Support
3.7A-50 Operating Basis Earthquake Horizontal Acceleration Response Spectra, Perpendicular to Hot Leg RCP Upper Horizontal Support
3.7A-51 Operating Basis Earthquake Horizontal Acceleration Response Spectra, Perpendicular to Hot Leg RCP Snubber Support
3.7A-52 Operating Basis Earthquake Vertical Acceleration Response Spectra, Perpendicular to Hot Leg Pressurizer Base Support
3.7A-53 Operating Basis Earthquake Horizontal Acceleration Response Spectra, Perpendicular to Hot Leg Pressurizer Base Support
3.7A-54 Operating Basis Earthquake Horizontal Acceleration Response Spectra, Perpendicular to Hot Leg Pressurizer Shear Key Support
3.7A-55 Design Basis Earthquake Vertical Acceleration Response Spectra for Containment Interior Structure Basemat
3.7A-56 Design Basis Earthquake Horizontal Acceleration Response Spectra for Containment Interior Structure Basemat
3.7A-57 Design Basis Earthquake Vertical Acceleration Response Spectra for Containment Interior Structure - Elevation 63'-6"
3.7A-58 Design Basis Earthquake Horizontal Acceleration Response Spectra for Containment Interior Structure - Elevation 63'-6"
3.7A-59 Design Basis Earthquake Vertical Acceleration Response Spectra for Containment Interior Structure - Elevation 80'-6"
3.7A-60 Design Basis Earthquake Horizontal Acceleration Response Spectra for Containment Interior Structure - Elevation 80'-6"
3.7A-61 Design Basis Earthquake Vertical Acceleration Response Spectra for Containment Exterior Shell Basemat
3.7A-62 Design Basis Earthquake Horizontal Acceleration Response Spectra for Containment Exterior Shell Basemat
3.7A-63 Design Basis Earthquake Vertical Acceleration Response Spectra for Containment Exterior Shell - Elevation 177'-6"
3.7A-64 Design Basis Earthquake Horizontal Acceleration Response Spectra for Containment Exterior Shell - Elevation 177'-6"
3.7A-65 Operating Basis Earthquake Vertical Acceleration Response Spectra for Containment Interior Structure Basemat
3.7A-66 Operating Basis Earthquake Horizontal Acceleration Response Spectra for Containment Interior Structure Basemat
3.7A-67 Operating Basis Earthquake Vertical Acceleration Response Spectra for Containment Interior Structure - Elevation 63'-6"
3.7A-68 Operating Basis Earthquake Horizontal Acceleration Response Spectra for Containment Interior Structure - Elevation 63'-6"
3.7A-69 Operating Basis Earthquake Horizontal Acceleration Response Spectra for Containment Interior Structure - Elevation 80'-6"
3.7A-70 Operating Basis Earthquake Horizontal Acceleration Response Spectra for Containment Interior Structure - Elevation 80'-6"
3.7A-71 Operating Basis Earthquake Vertical Acceleration Response Spectra for Containment Exterior Shell Basemat
3.7A-72 Operating Basis Earthquake Horizontal Acceleration Response Spectra for Containment Exterior Shell Basemat

FIGURES (cont)

3.7A-73 Operating Basis Earthquake Vertical Acceleration Response Spectra for Containment Exterior Shell - Elevation 177'-6"

3.7A-74 Operating Basis Earthquake Horizontal Acceleration Response Spectra for Containment Exterior Shell - Elevation 177'-6"

3.7A-75 Design Basis Earthquake Vertical Acceleration Response Spectra at Node 1 Elevation 9'-0" of Auxiliary Building

3.7A-76 Design Basis Earthquake Horizontal Acceleration Response Spectra at Node 1 Elevation 9'-0" of Auxiliary Building

3.7A-77 Design Basis Earthquake Vertical Acceleration Response Spectra at Node 12 Elevation 85'-0" of Auxiliary Building

3.7A-78 Design Basis Earthquake Horizontal Acceleration Response Spectra at Node 12 Elevation 85'-0" of Auxiliary Building

3.7A-79 Operating Basis Earthquake Vertical Acceleration Response Spectra at Node 1 Elevation 9'-0" of Auxiliary Building

3.7A-80 Operating Basis Earthquake Horizontal Acceleration Response Spectra at Node 1 Elevation 9'-0" of Auxiliary Building

3.7A-81 Operating Basis Earthquake Vertical Acceleration Response Spectra at Node 12 Elevation 85'-0" of Auxiliary Building

3.7A-82 Operating Basis Earthquake Horizontal Acceleration Response Spectra at Node 12 Elevation 85'-0" of Auxiliary Building

3.7A-83 Design Basis Earthquake Vertical Acceleration Response Spectra at Node 12A Elevation 85'-0" of Central Control Area Auxiliary Building

3.7A-84 Design Basis Earthquake Vertical Acceleration Response Spectra at Node 12A Elevation 85'-0" of Central Control Area Auxiliary Building

3.7A-85 Design Basis Earthquake Vertical Acceleration Response Spectra at Node 1 Elevation 17'-6" of Fuel Handling Building

3.7A-86 Design Basis Earthquake Horizontal Acceleration Response Spectra at Node 1 Elevation 17'-6" of Fuel Handling Building

3.7A-87 Operating Basis Earthquake Vertical Acceleration Response Spectra at Node 1 Elevation 17'-6" of Fuel Handling Building

3.7A-88 Operating Basis Earthquake Horizontal Acceleration Response Spectra at Node 1 Elevation 17'-6" of Fuel Handling Building

3.7A-89 Design Basis Earthquake Vertical Acceleration Response Spectra at Node 6 Elevation 114'-0" of Fuel Handling Building

3.7A-90 Design Basis Earthquake Horizontal Acceleration Response Spectra at Node 6 Elevation 114'-0" of Fuel Handling Building

3.7A-91 Operating Basis Earthquake Vertical Acceleration Response Spectra at Node 6 Elevation 114'-0" of Fuel Handling Building

3.7A-92 Operating Basis Earthquake Horizontal Acceleration Response Spectra at Node 6 Elevation 114'-0" of Fuel Handling Building

3.7A-93 Design Basis Earthquake Vertical Acceleration Response Spectra at Elevation -15'-6" of Safety Equipment Building (Safety Injection Area)

3.7A-94 Design Basis Earthquake E-W Horizontal Acceleration Response Spectra at Elevation -15'-6" of Safety Equipment Building (Safety Injection Area)

FIGURES (cont)

3.7A-95 Design Basis Earthquake N-S Horizontal Acceleration Response Spectra at Elevation -15'-6" of Safety Equipment Building (Safety Injection Area)

3.7A-96 Design Basis Earthquake Vertical Acceleration Response Spectra at Elevation -5'-3" of Safety Equipment Building (Component Cooling Area)

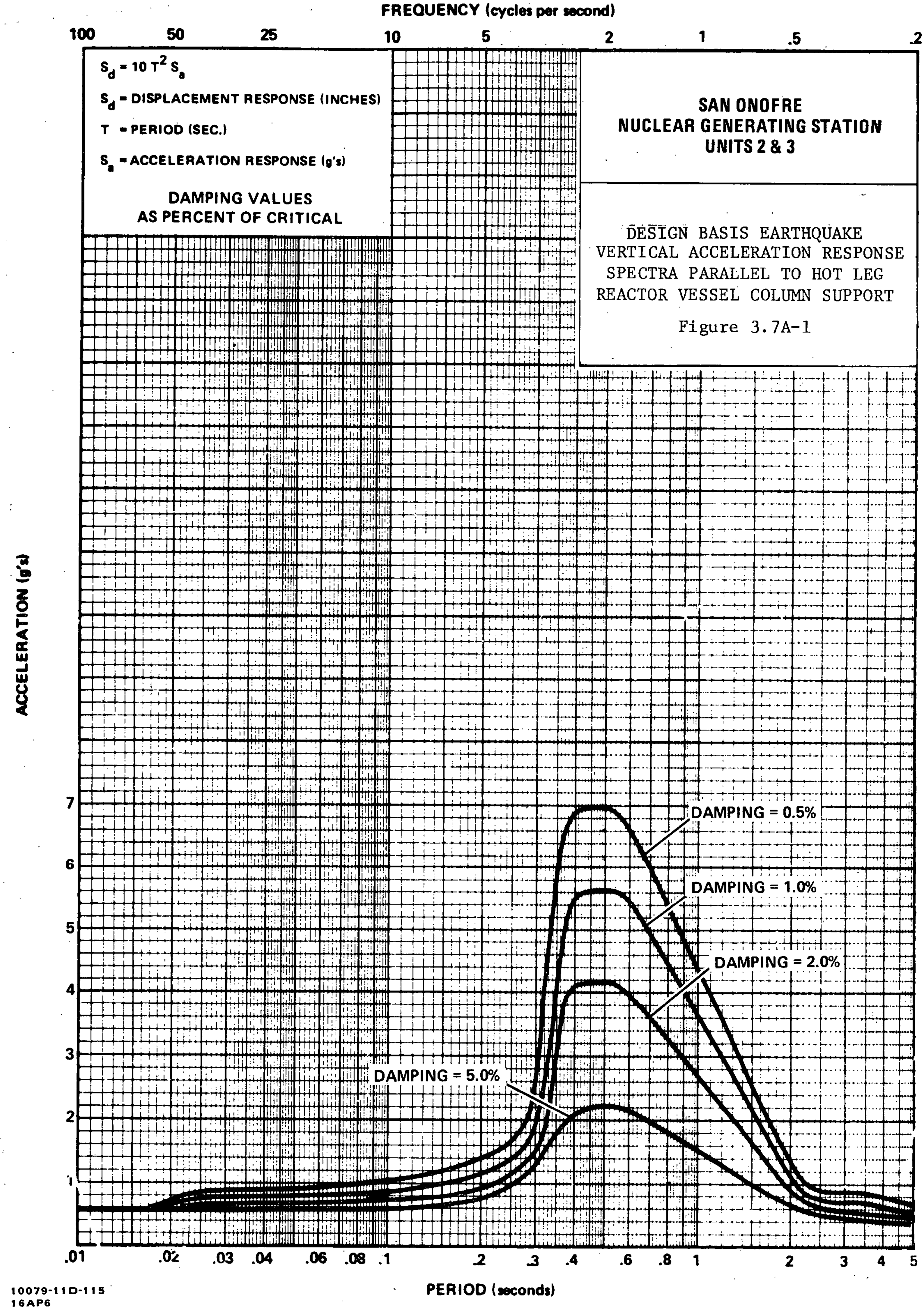
3.7A-97 Design Basis Earthquake E-W Horizontal Acceleration Response Spectra at Elevation -5'-3" of Safety Equipment Building (Component Cooling Area)

3.7A-98 Design Basis Earthquake N-S Horizontal Acceleration Response Spectra at Elevation -5'-3" of Safety Equipment Building (Component Cooling Area)

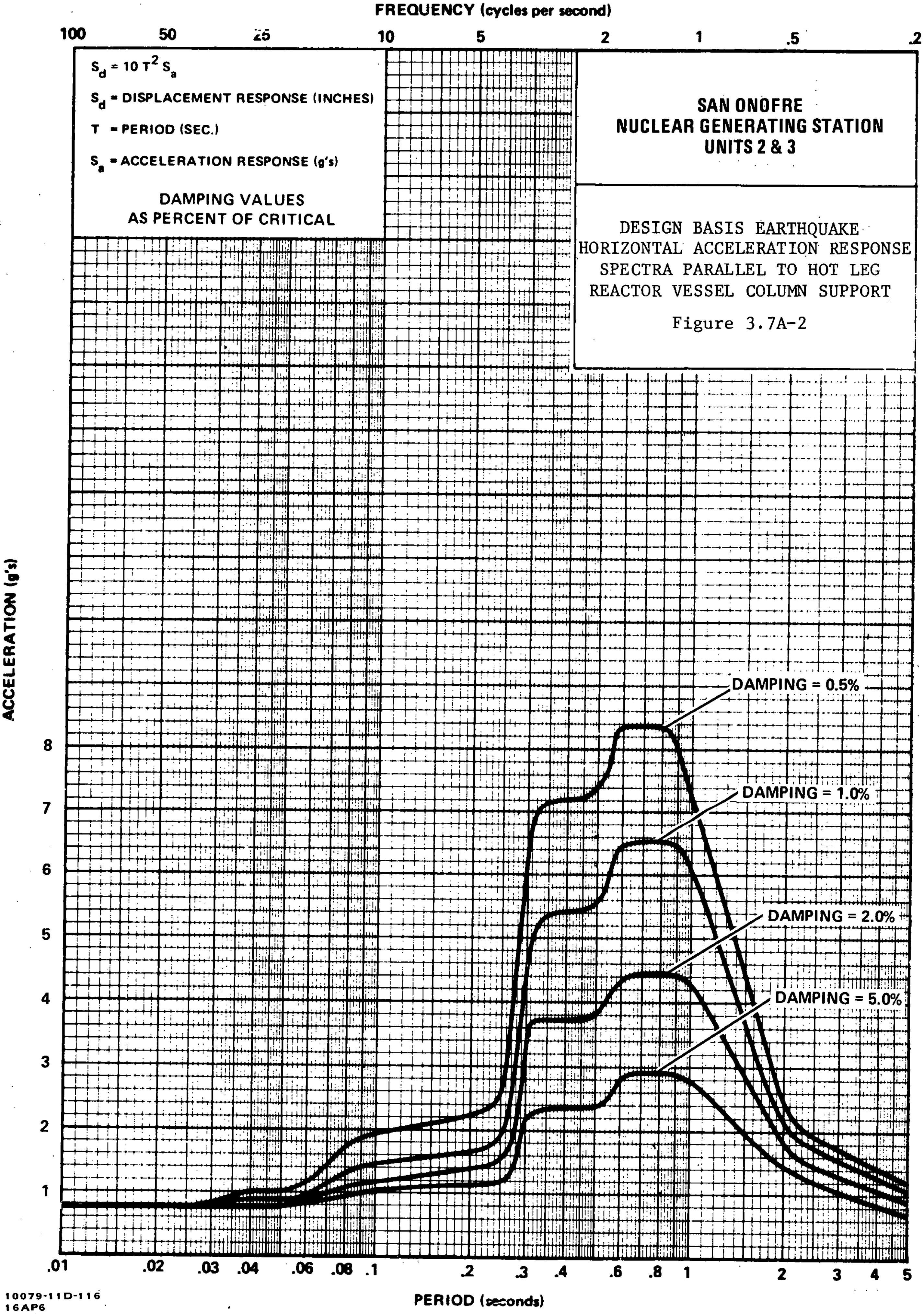
3.7A-99 Design Basis Earthquake Vertical Acceleration Response Spectra at Elevation +50'-6" of Safety Equipment Building

3.7A-100 Design Basis Earthquake E-W Horizontal Acceleration Response Spectra at Elevation +50'-6" of Safety Equipment Building

3.7A-101 Design Basis Earthquake N-S Horizontal Acceleration Response Spectra at Elevation +50'-6" of Safety Equipment Building



10079-11D-115
16AP6

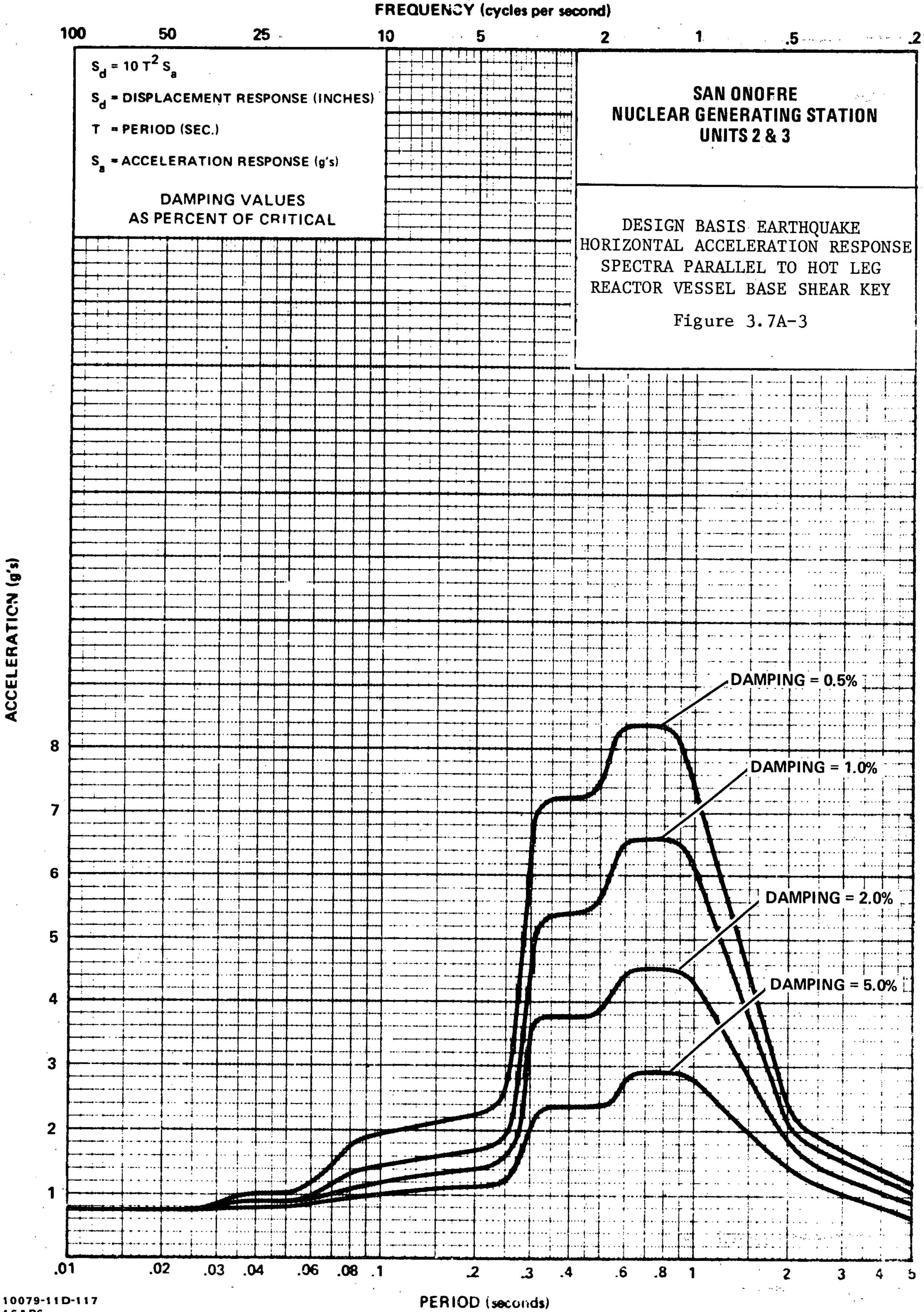


SAN ONOFRE
NUCLEAR GENERATING STATION
UNITS 2 & 3

DESIGN BASIS EARTHQUAKE
HORIZONTAL ACCELERATION RESPONSE
SPECTRA PARALLEL TO HOT LEG
REACTOR VESSEL COLUMN SUPPORT

Figure 3.7A-2

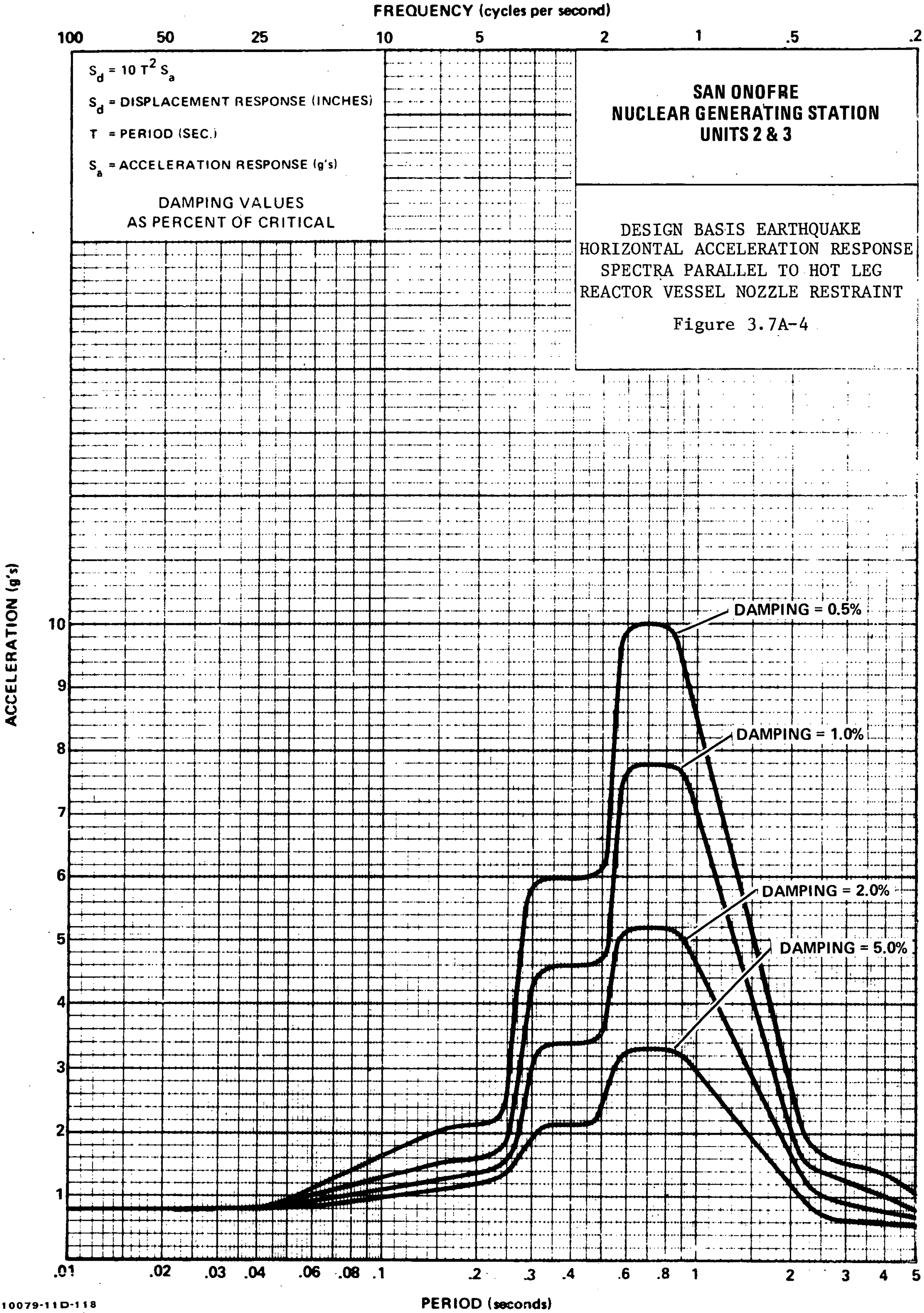
10079-11D-116
16AP6



DESIGN BASIS EARTHQUAKE
HORIZONTAL ACCELERATION RESPONSE
SPECTRA PARALLEL TO HOT LEG
REACTOR VESSEL BASE SHEAR KEY

Figure 3.7A-3

10079-11D-117
16AP6

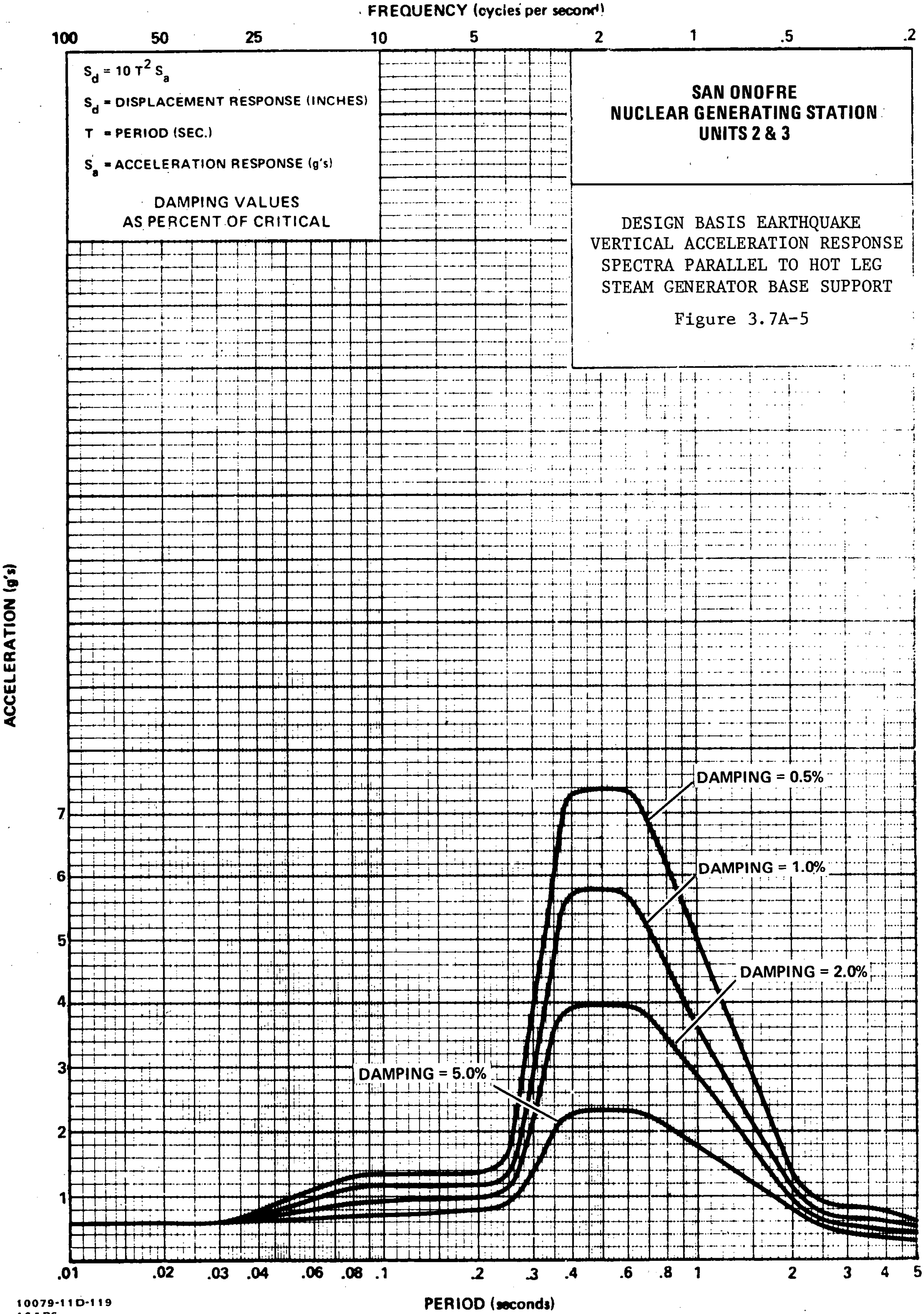


SAN ONOFRE
NUCLEAR GENERATING STATION
UNITS 2 & 3

DESIGN BASIS EARTHQUAKE
HORIZONTAL ACCELERATION RESPONSE
SPECTRA PARALLEL TO HOT LEG
REACTOR VESSEL NOZZLE RESTRAINT

Figure 3.7A-4

10079-11D-118
16AP6

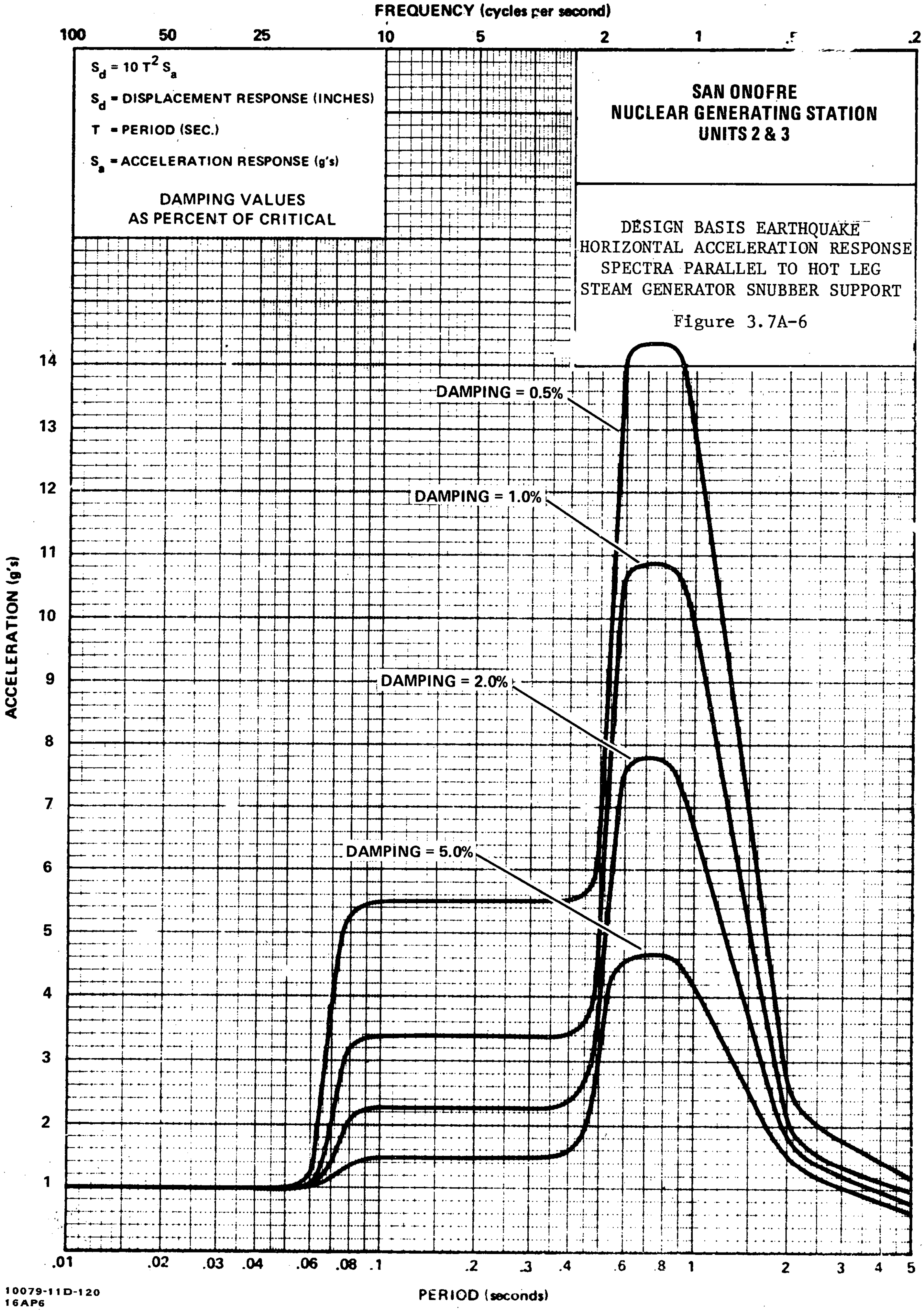


SAN ONOFRE NUCLEAR GENERATING STATION UNITS 2 & 3

DESIGN BASIS EARTHQUAKE VERTICAL ACCELERATION RESPONSE SPECTRA PARALLEL TO HOT LEG STEAM GENERATOR BASE SUPPORT

Figure 3.7A-5

10079-11D-119
16AP6

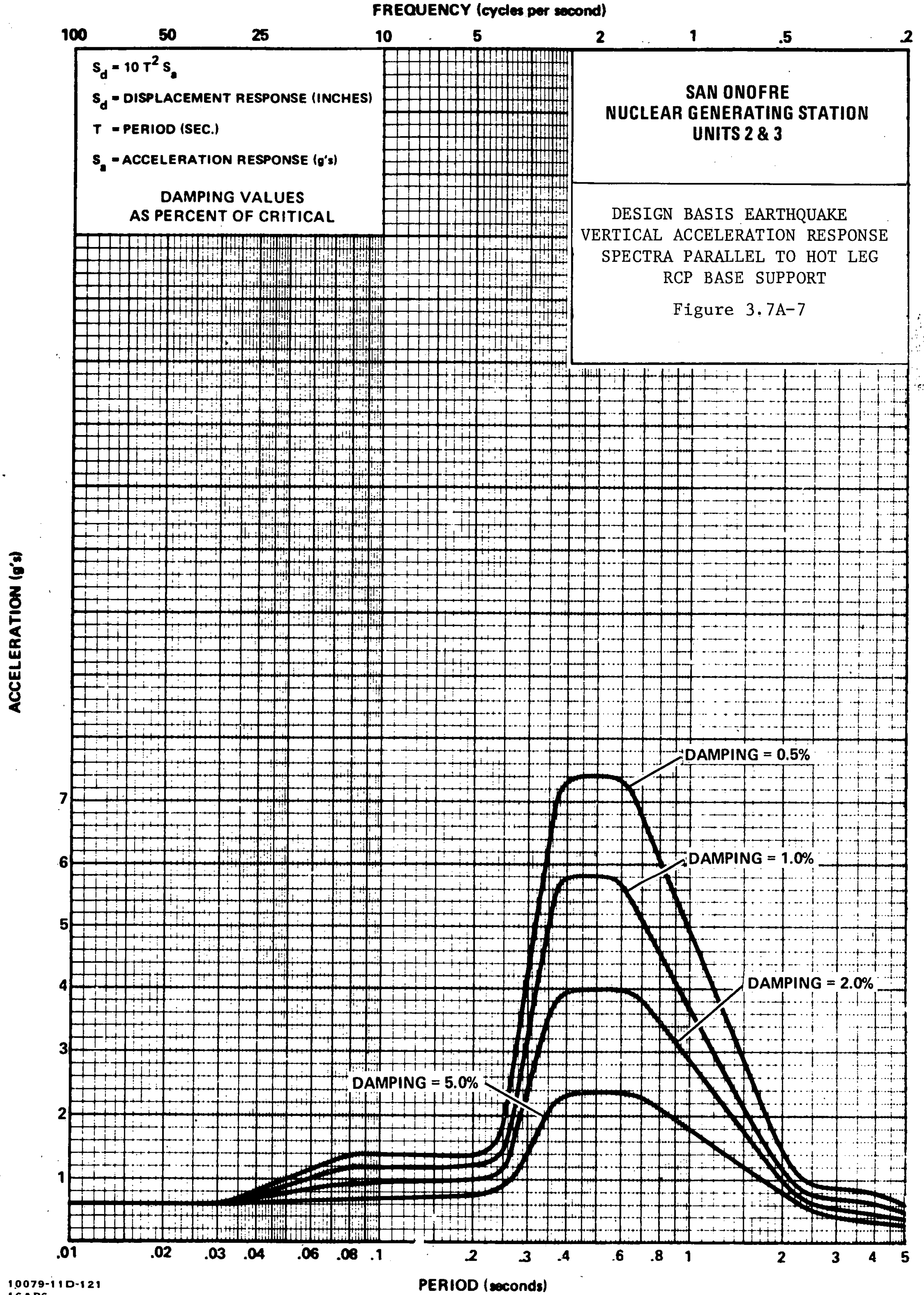


SAN ONOFRE
NUCLEAR GENERATING STATION
UNITS 2 & 3

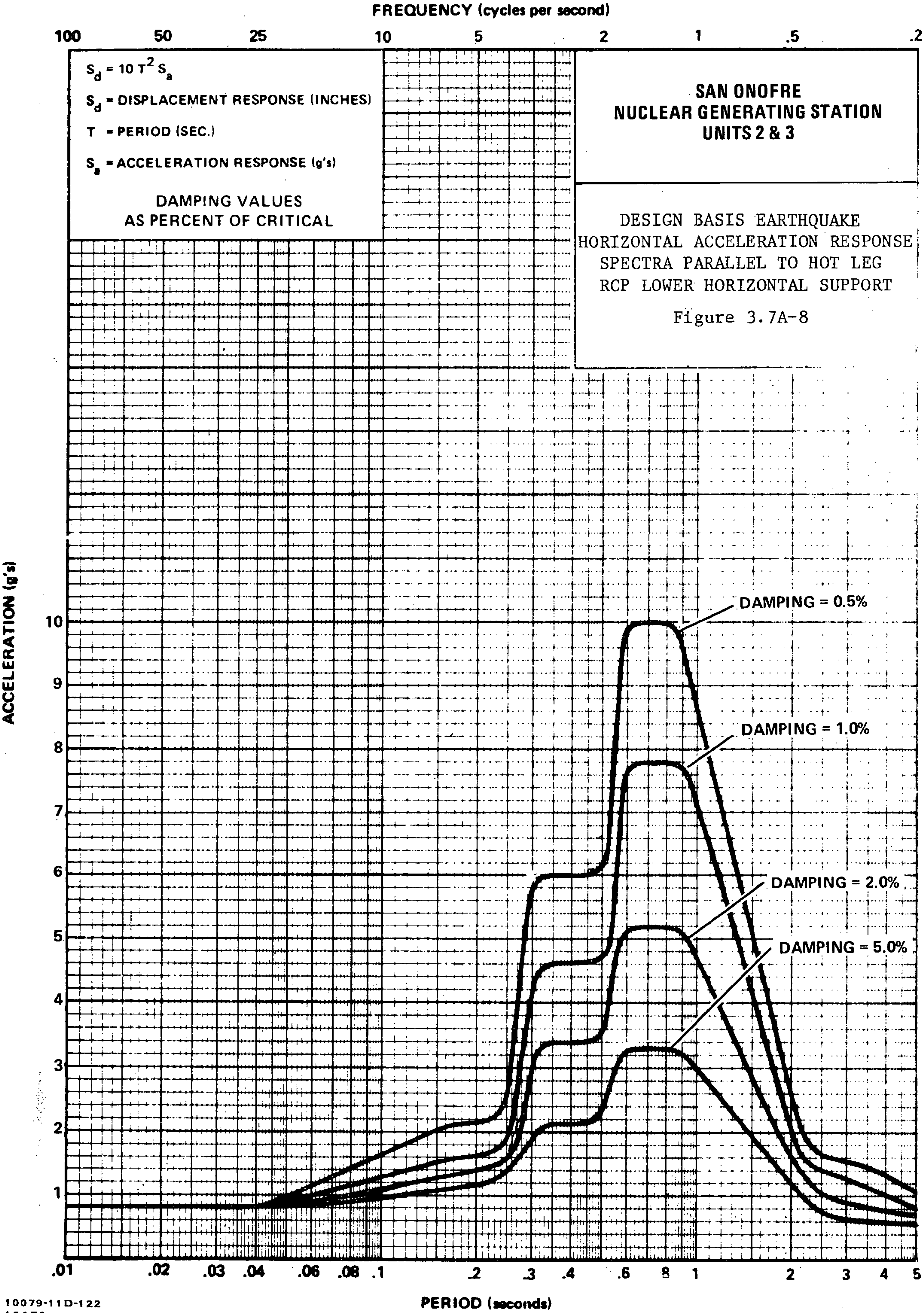
DESIGN BASIS EARTHQUAKE
HORIZONTAL ACCELERATION RESPONSE
SPECTRA PARALLEL TO HOT LEG
STEAM GENERATOR SNUBBER SUPPORT

Figure 3.7A-6

10079-11D-120
16AP6



10079-11D-121
16AP6

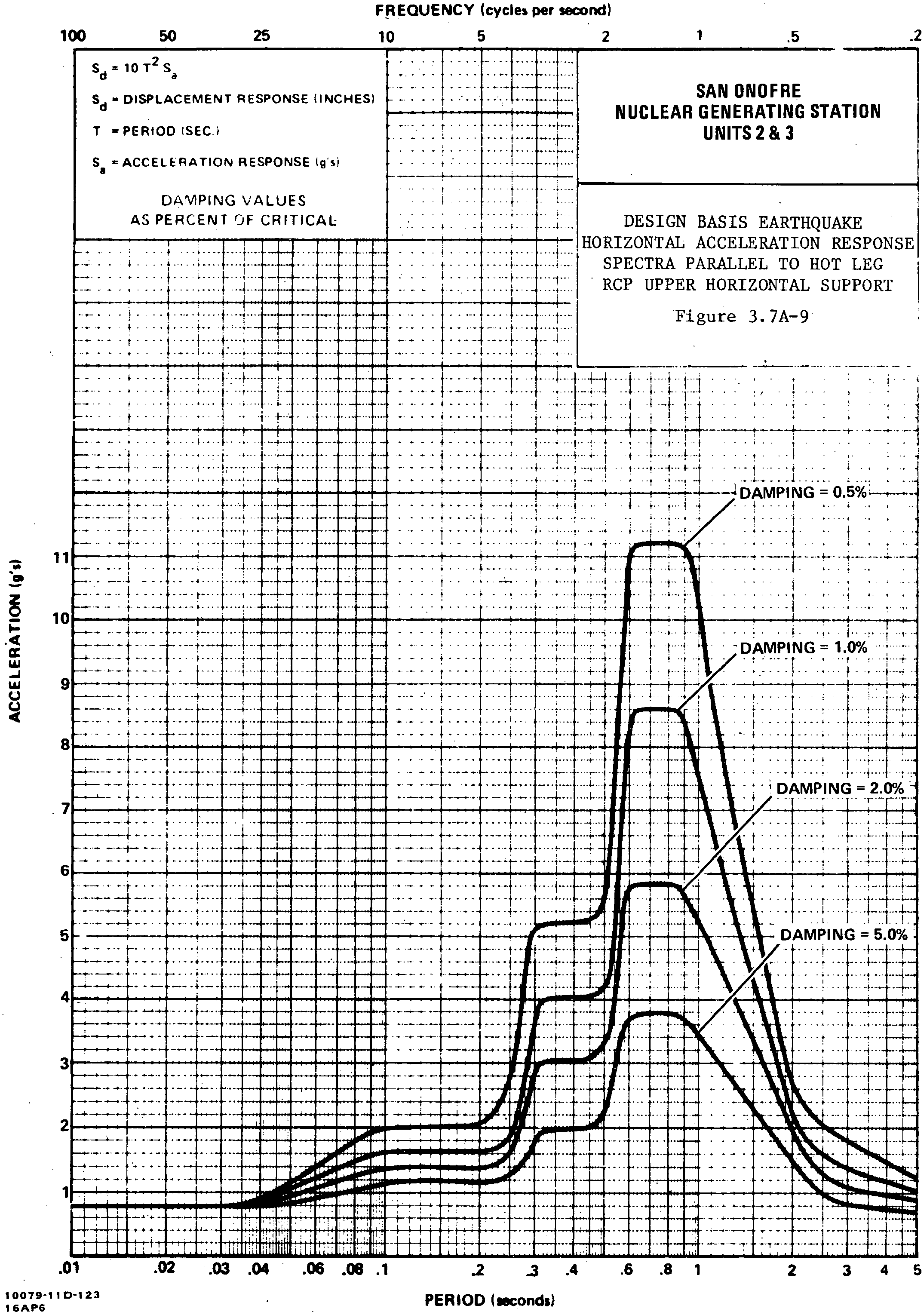


SAN ONOFRE
NUCLEAR GENERATING STATION
UNITS 2 & 3

DESIGN BASIS EARTHQUAKE
HORIZONTAL ACCELERATION RESPONSE
SPECTRA PARALLEL TO HOT LEG
RCP LOWER HORIZONTAL SUPPORT

Figure 3.7A-8

10079-11D-122
16AP6

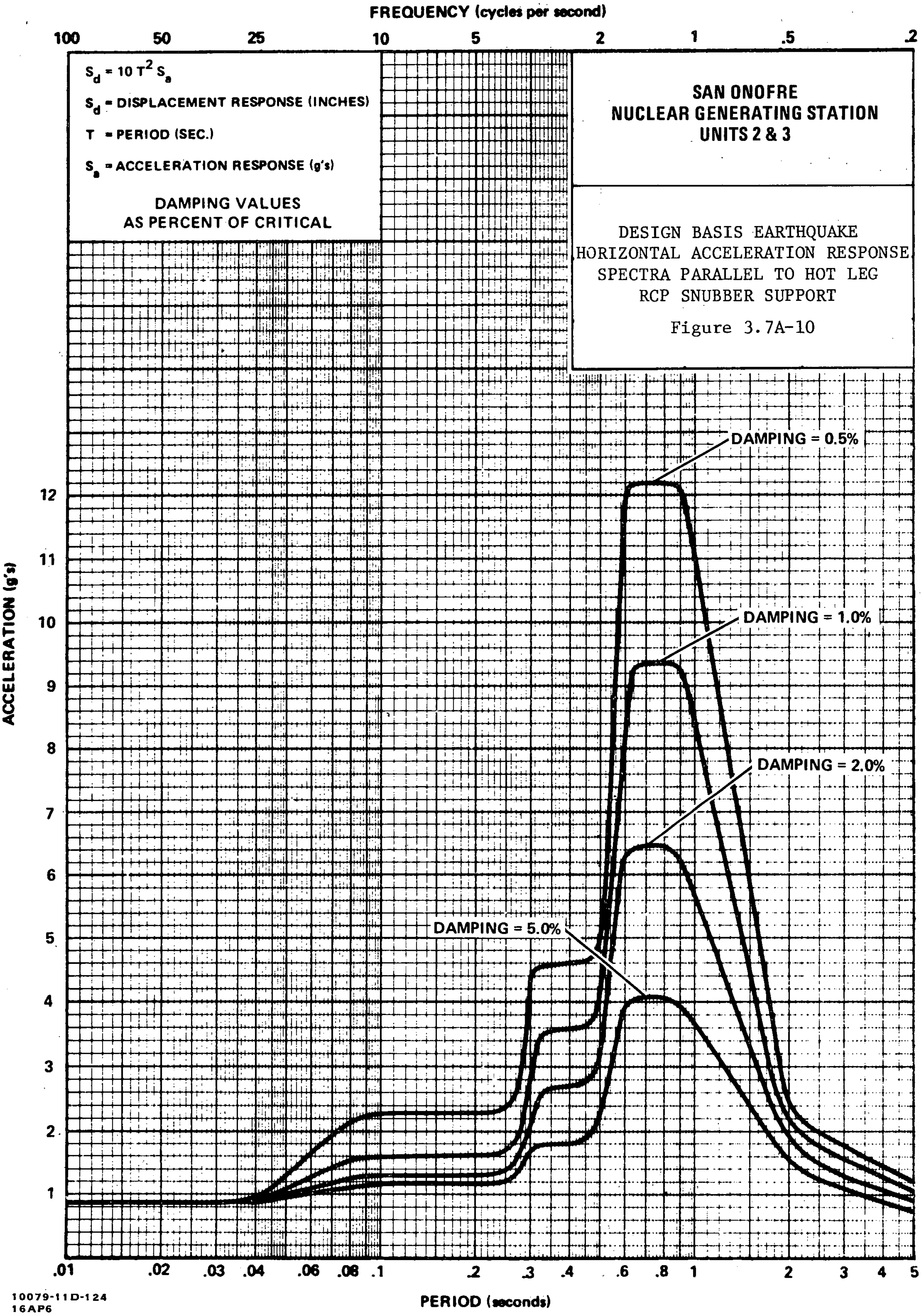


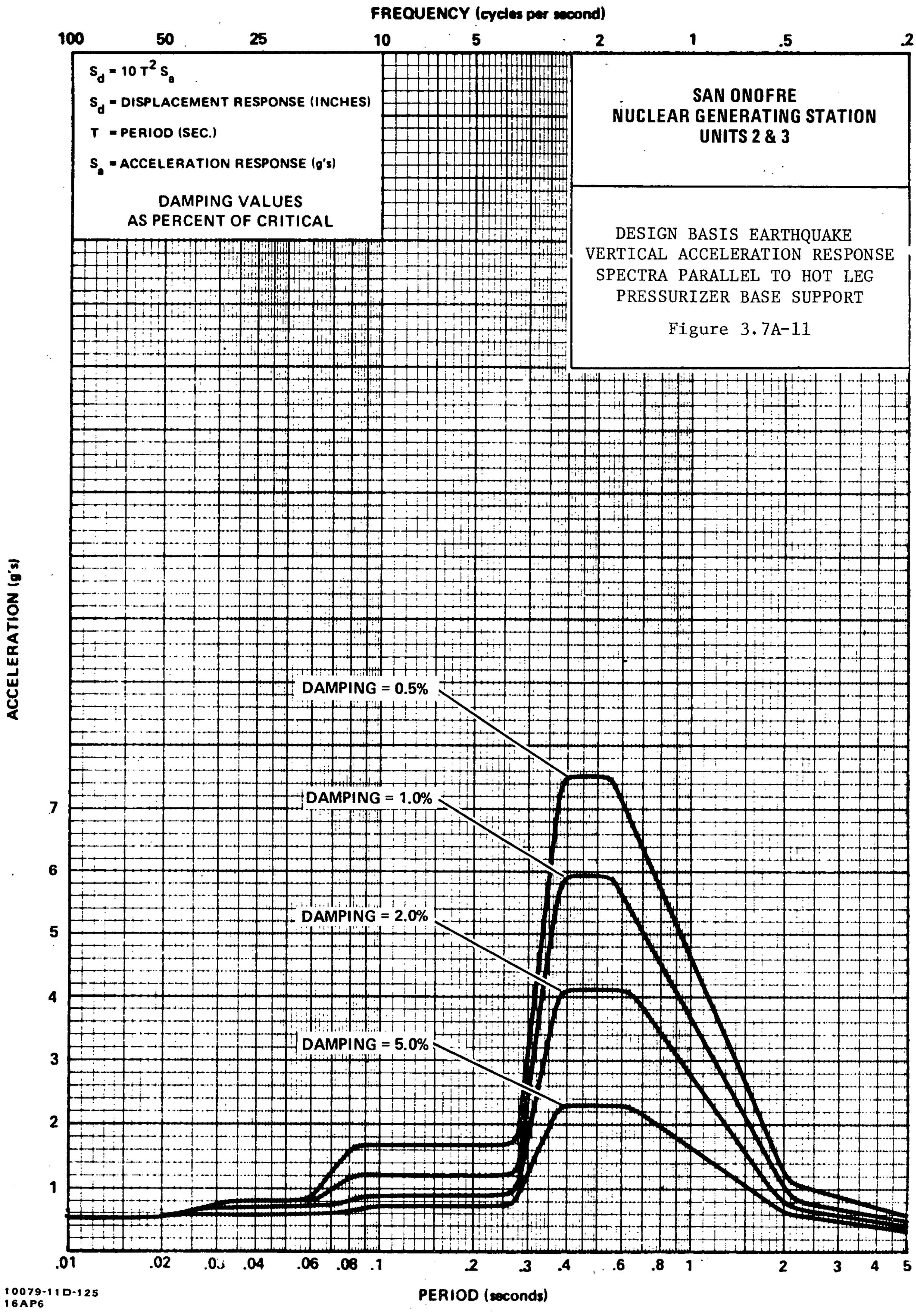
SAN ONOFRE
NUCLEAR GENERATING STATION
UNITS 2 & 3

DESIGN BASIS EARTHQUAKE
HORIZONTAL ACCELERATION RESPONSE
SPECTRA PARALLEL TO HOT LEG
RCP UPPER HORIZONTAL SUPPORT

Figure 3.7A-9

10079-11D-123
16AP6

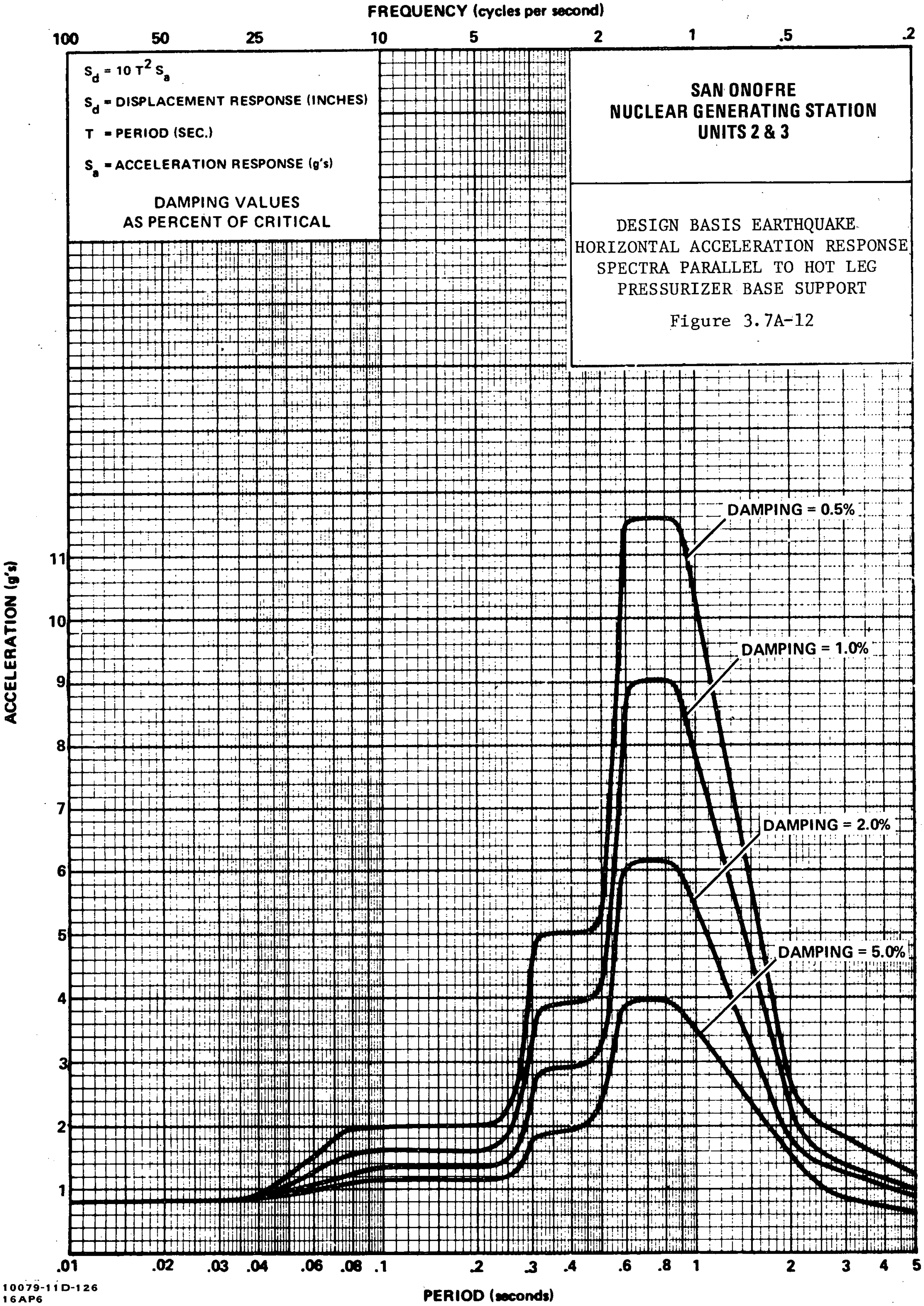

FREQUENCY (cycles per second)
100
50
25
10
5
2
1
.5
.2
$S_d = 10\,T^2\,S_a$
S_d = DISPLACEMENT RESPONSE (INCHES)
T = PERIOD (SEC.)
S_a = ACCELERATION RESPONSE (g's)
DAMPING VALUES
AS PERCENT OF CRITICAL
SAN ONOFRE
NUCLEAR GENERATING STATION
UNITS 2 & 3
DESIGN BASIS EARTHQUAKE
HORIZONTAL ACCELERATION RESPONSE
SPECTRA PARALLEL TO HOT LEG
RCP SNUBBER SUPPORT
Figure 3.7A-10
DAMPING = 0.5%
DAMPING = 1.0%
DAMPING = 2.0%
DAMPING = 5.0%
ACCELERATION (g's)
12
11
10
9
8
7
6
5
4
3
2
1
.01
.02
.03
.04
.06
.08
.1
.2
.3
.4
.6
.8
1
2
3
4
5
PERIOD (seconds)
10079-11D-124
16AP6



SAN ONOFRE NUCLEAR GENERATING STATION UNITS 2 & 3

DESIGN BASIS EARTHQUAKE VERTICAL ACCELERATION RESPONSE SPECTRA PARALLEL TO HOT LEG PRESSURIZER BASE SUPPORT

Figure 3.7A-11

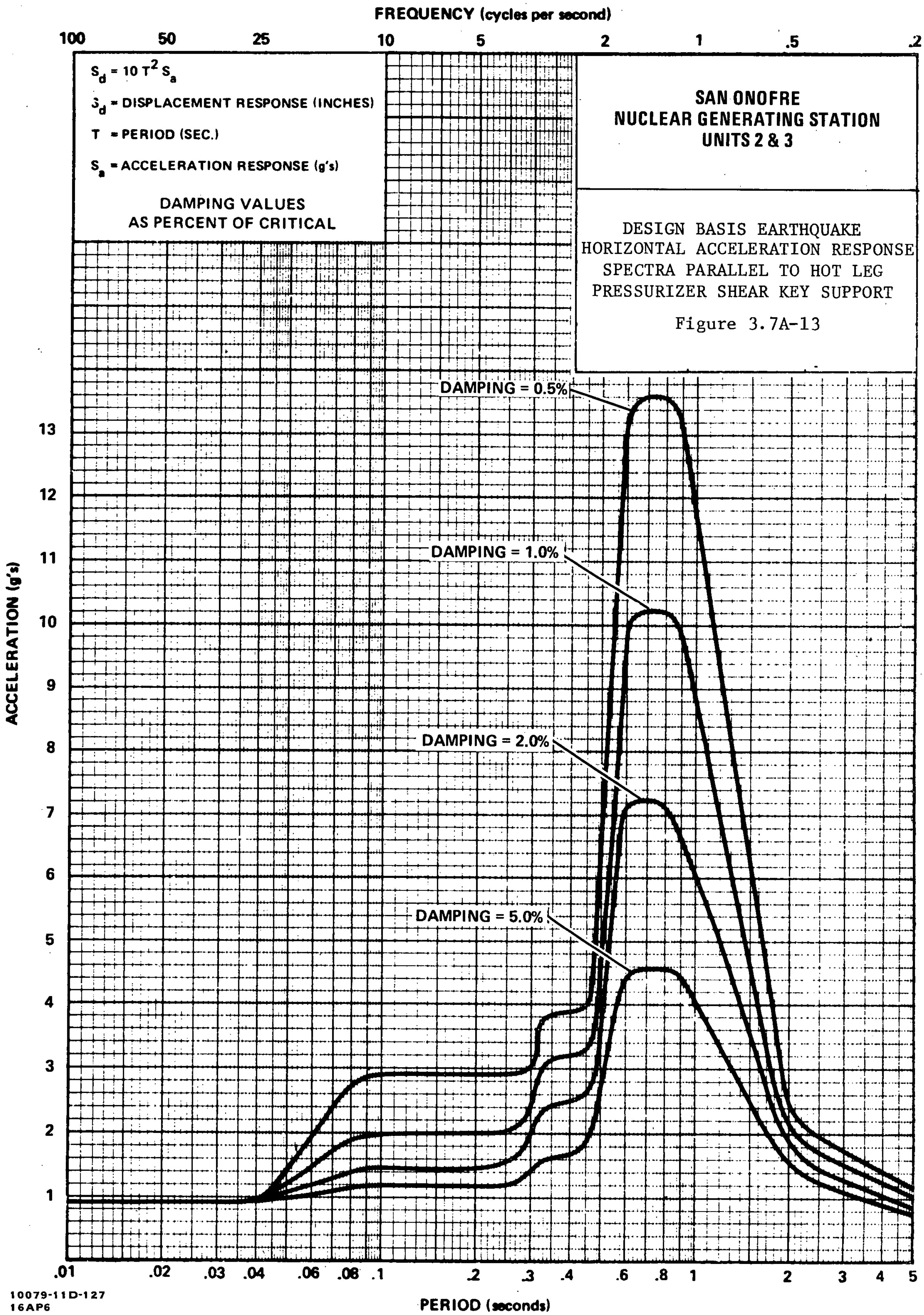


SAN ONOFRE
NUCLEAR GENERATING STATION
UNITS 2 & 3

DESIGN BASIS EARTHQUAKE
HORIZONTAL ACCELERATION RESPONSE
SPECTRA PARALLEL TO HOT LEG
PRESSURIZER BASE SUPPORT

Figure 3.7A-12

10079-11D-126
16AP6

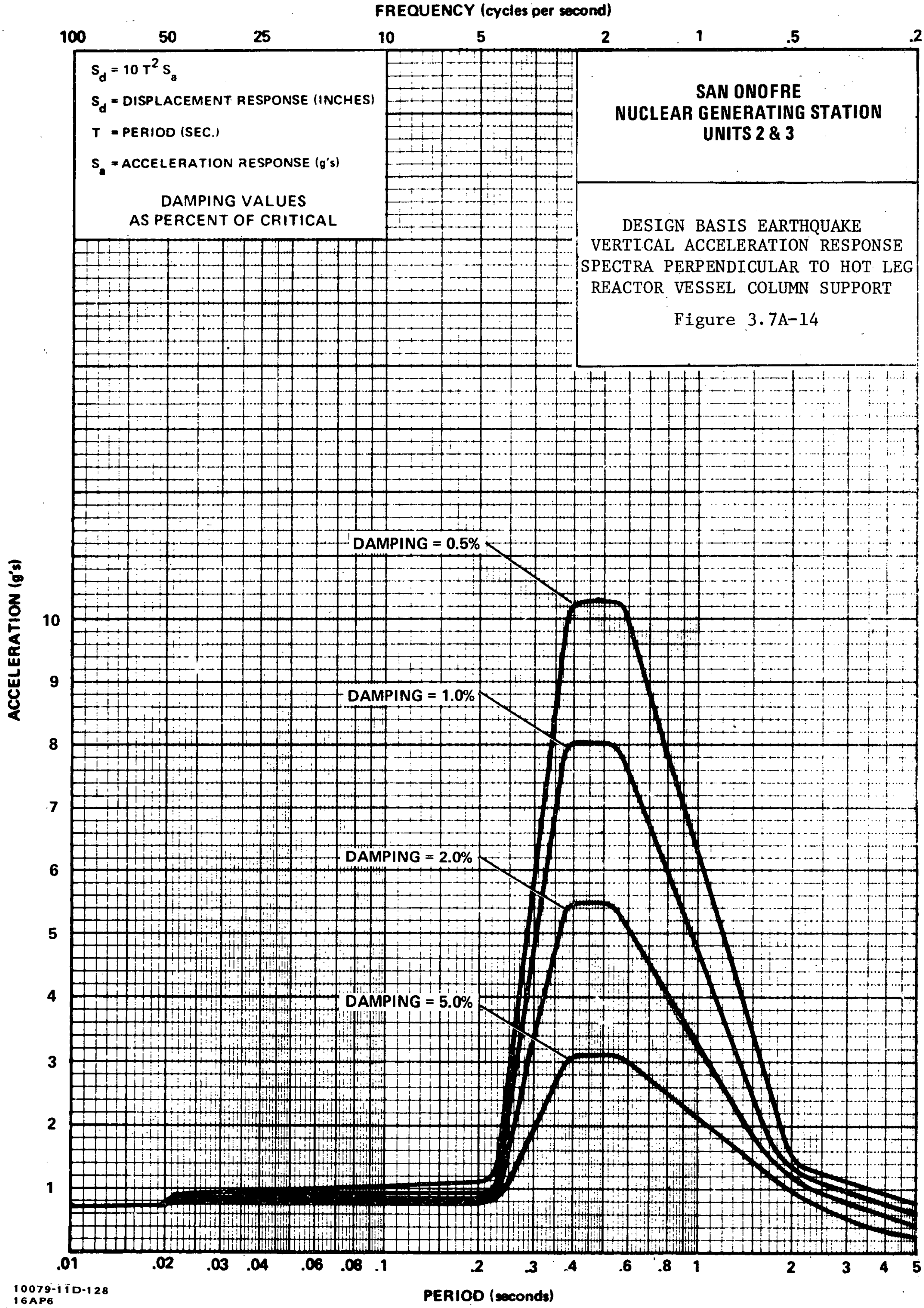


SAN ONOFRE
NUCLEAR GENERATING STATION
UNITS 2 & 3

DESIGN BASIS EARTHQUAKE
HORIZONTAL ACCELERATION RESPONSE
SPECTRA PARALLEL TO HOT LEG
PRESSURIZER SHEAR KEY SUPPORT

Figure 3.7A-13

10079-11D-127
16AP6

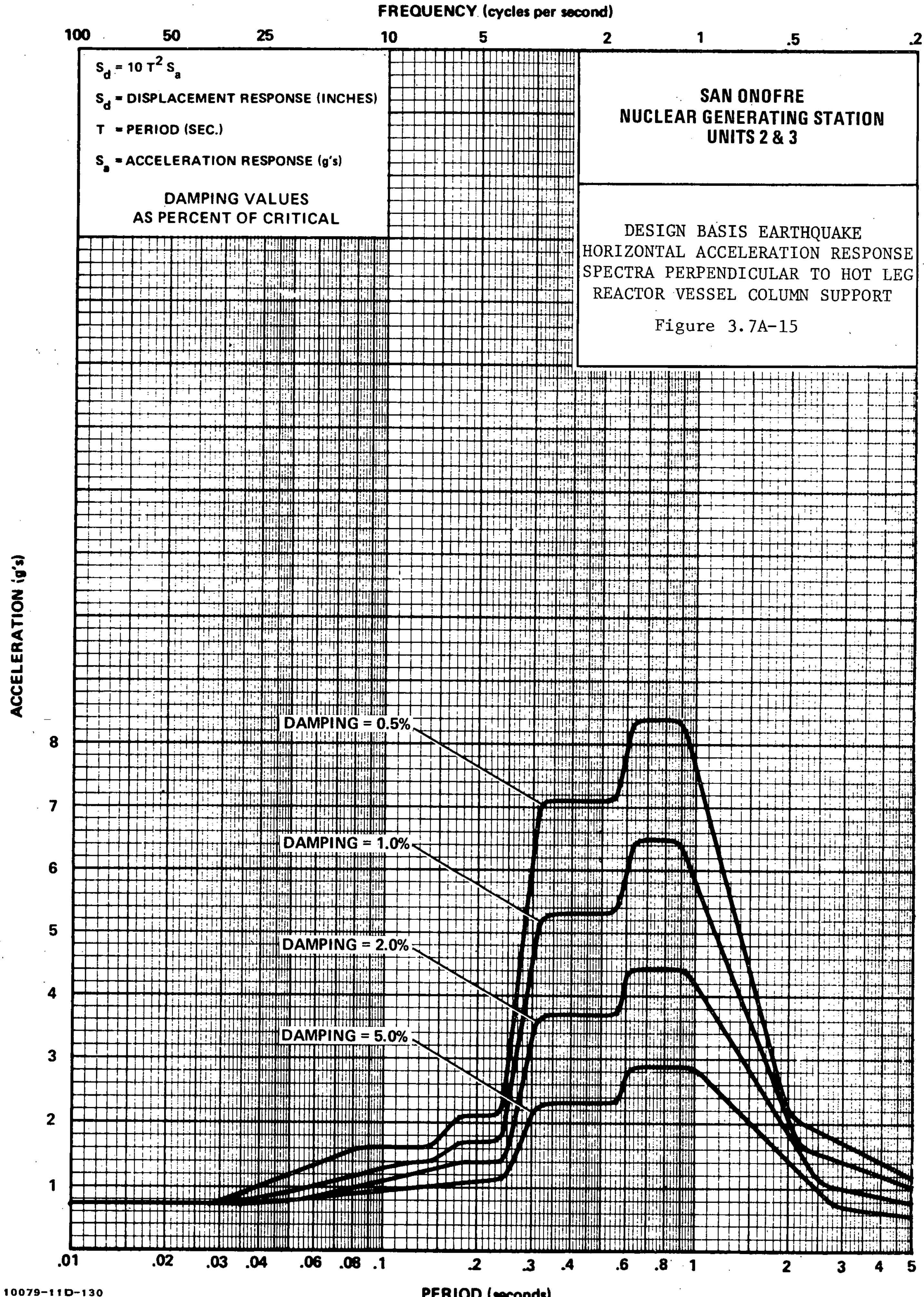


SAN ONOFRE NUCLEAR GENERATING STATION UNITS 2 & 3

DESIGN BASIS EARTHQUAKE VERTICAL ACCELERATION RESPONSE SPECTRA PERPENDICULAR TO HOT LEG REACTOR VESSEL COLUMN SUPPORT

Figure 3.7A-14

10079-11D-128
16AP6

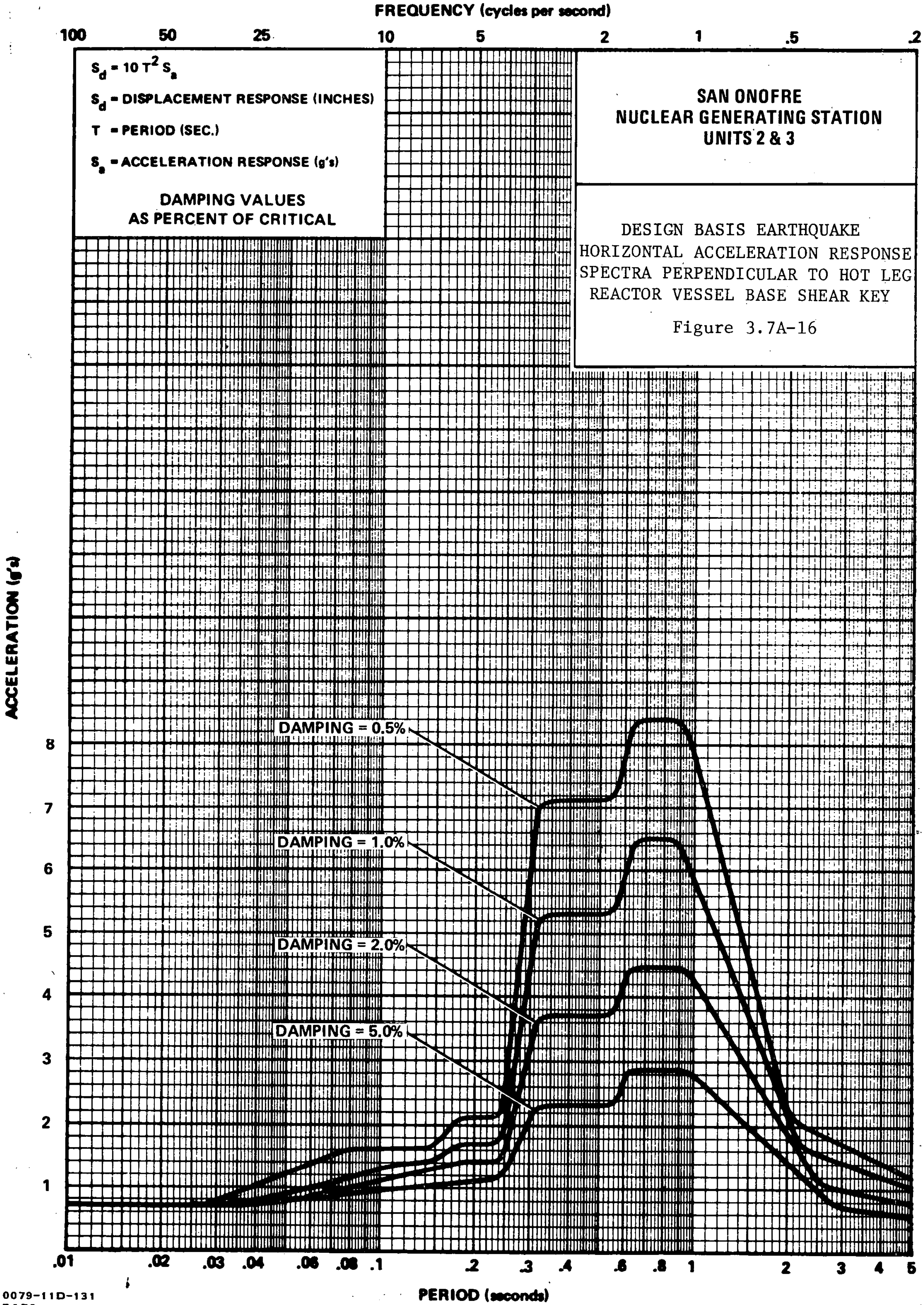


SAN ONOFRE
NUCLEAR GENERATING STATION
UNITS 2 & 3

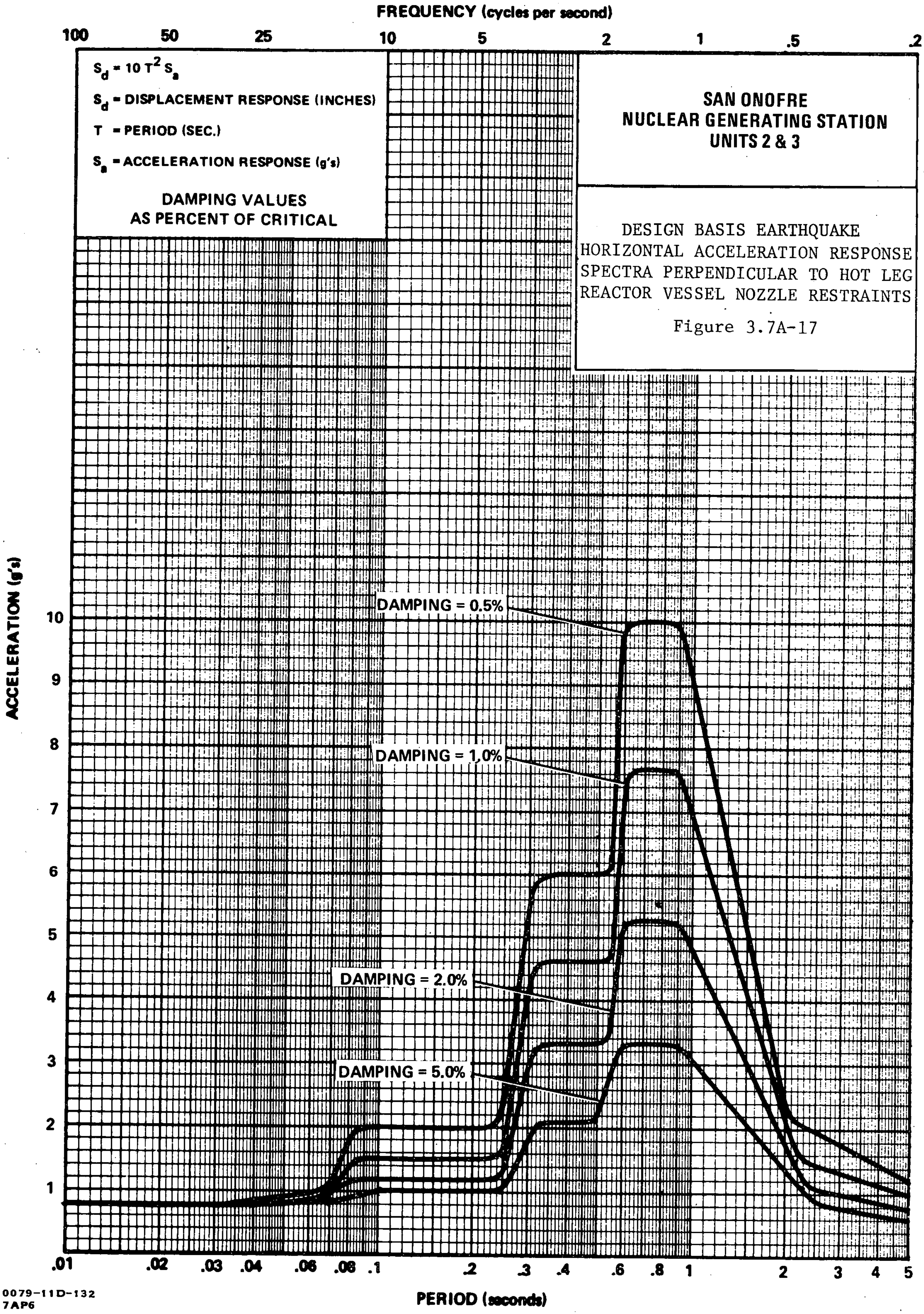
DESIGN BASIS EARTHQUAKE
HORIZONTAL ACCELERATION RESPONSE
SPECTRA PERPENDICULAR TO HOT LEG
REACTOR VESSEL COLUMN SUPPORT

Figure 3.7A-15

10079-11D-130
17AP6



10079-11D-131
17AP6

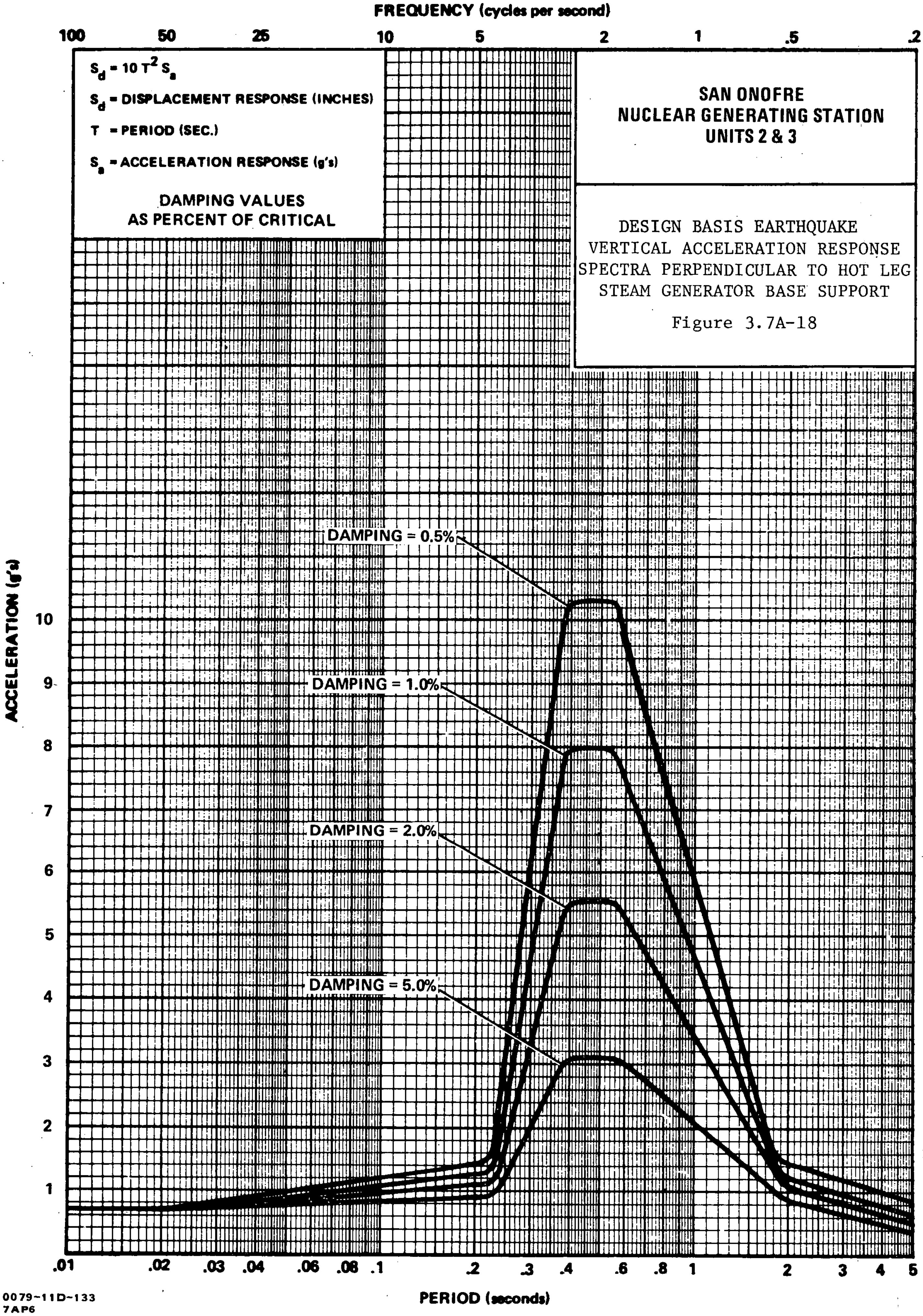


SAN ONOFRE
NUCLEAR GENERATING STATION
UNITS 2 & 3

DESIGN BASIS EARTHQUAKE
HORIZONTAL ACCELERATION RESPONSE
SPECTRA PERPENDICULAR TO HOT LEG
REACTOR VESSEL NOZZLE RESTRAINTS

Figure 3.7A-17

10079-11D-132
17AP6

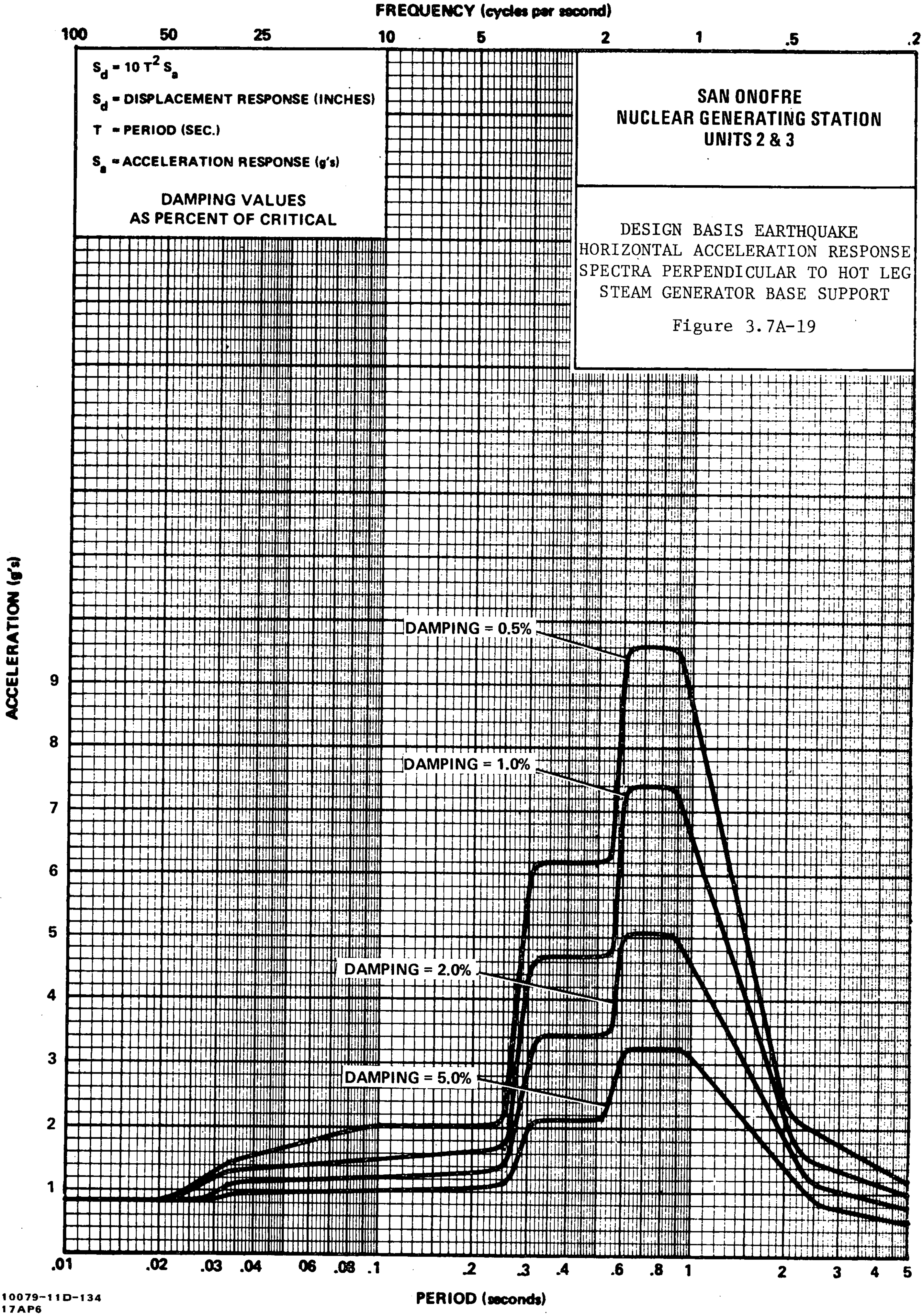


SAN ONOFRE
NUCLEAR GENERATING STATION
UNITS 2 & 3

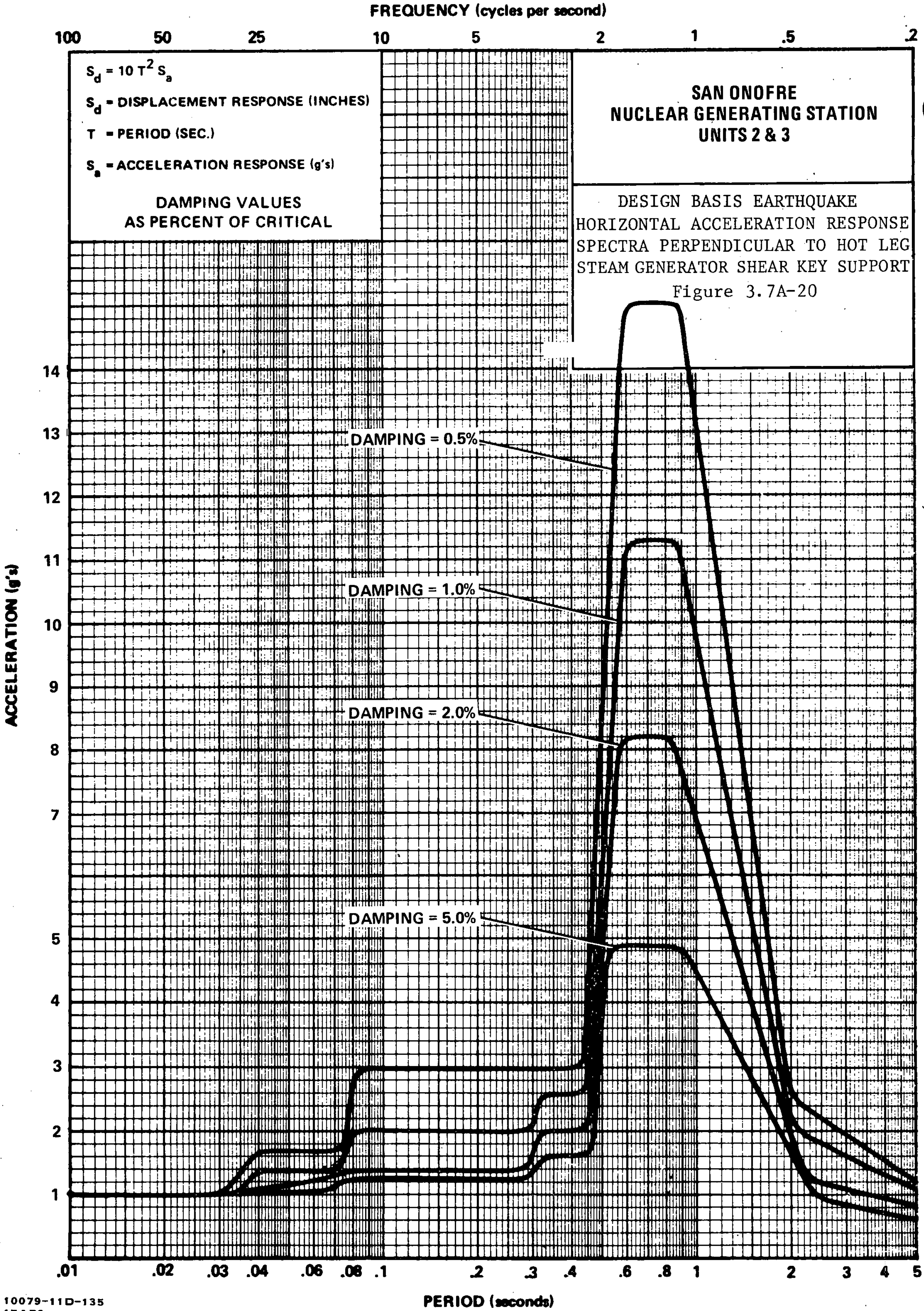
DESIGN BASIS EARTHQUAKE
VERTICAL ACCELERATION RESPONSE
SPECTRA PERPENDICULAR TO HOT LEG
STEAM GENERATOR BASE SUPPORT

Figure 3.7A-18

10079-11D-133
17AP6



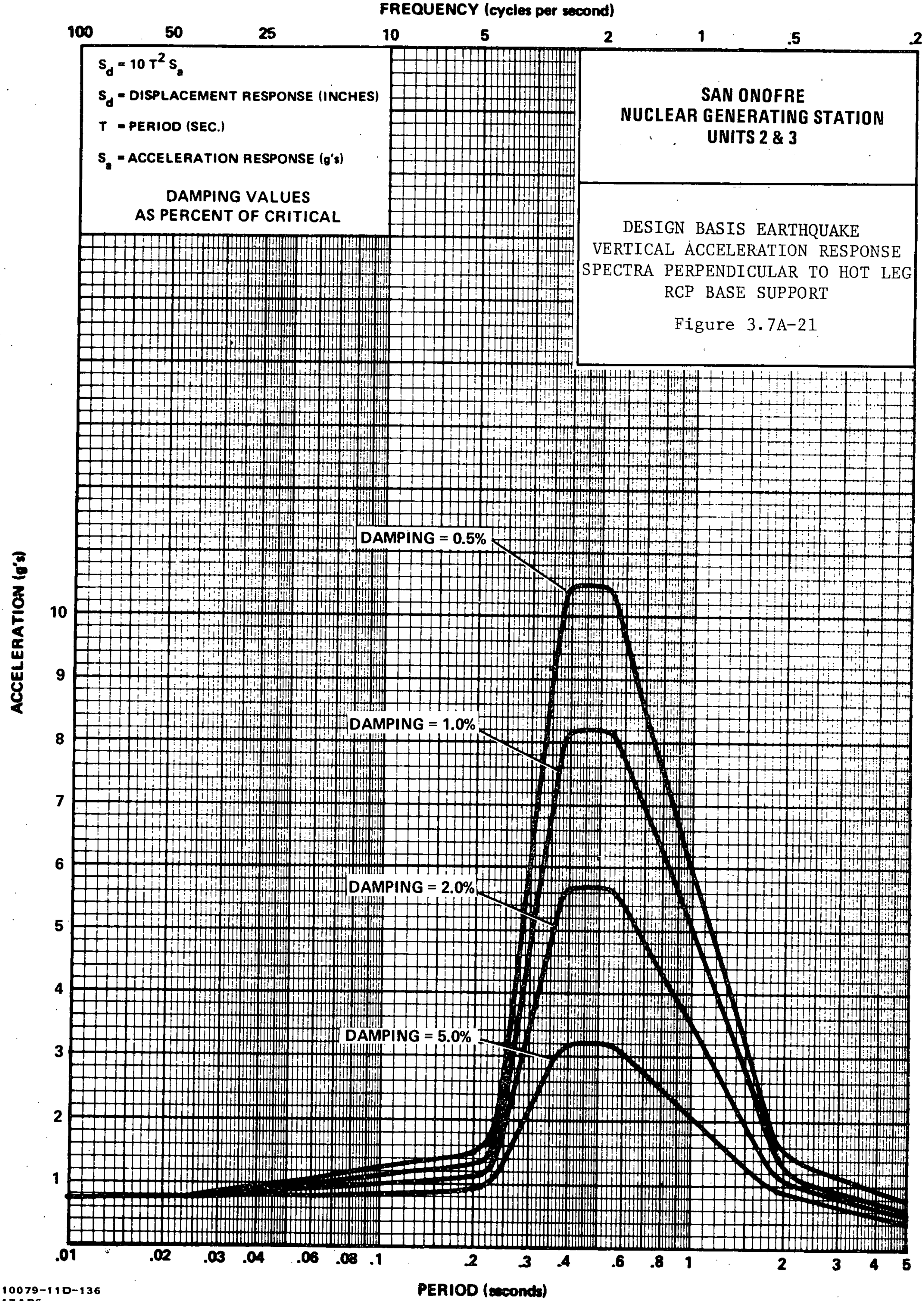
10079-11D-134
17AP6



SAN ONOFRE
NUCLEAR GENERATING STATION
UNITS 2 & 3

DESIGN BASIS EARTHQUAKE
HORIZONTAL ACCELERATION RESPONSE
SPECTRA PERPENDICULAR TO HOT LEG
STEAM GENERATOR SHEAR KEY SUPPORT
Figure 3.7A-20

10079-11D-135
17AP6

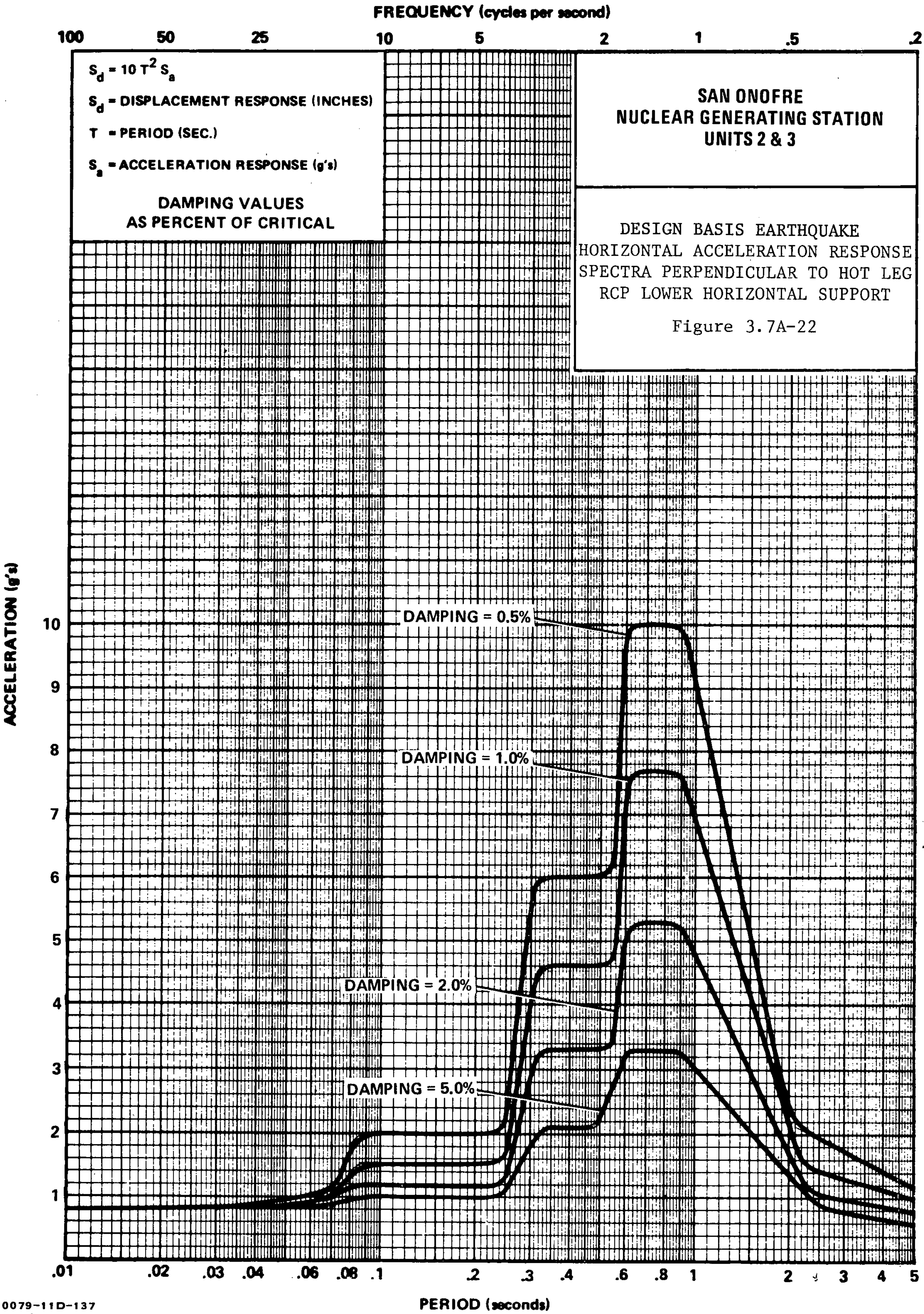


SAN ONOFRE
NUCLEAR GENERATING STATION
UNITS 2 & 3

DESIGN BASIS EARTHQUAKE
VERTICAL ACCELERATION RESPONSE
SPECTRA PERPENDICULAR TO HOT LEG
RCP BASE SUPPORT

Figure 3.7A-21

10079-11D-136
17AP6

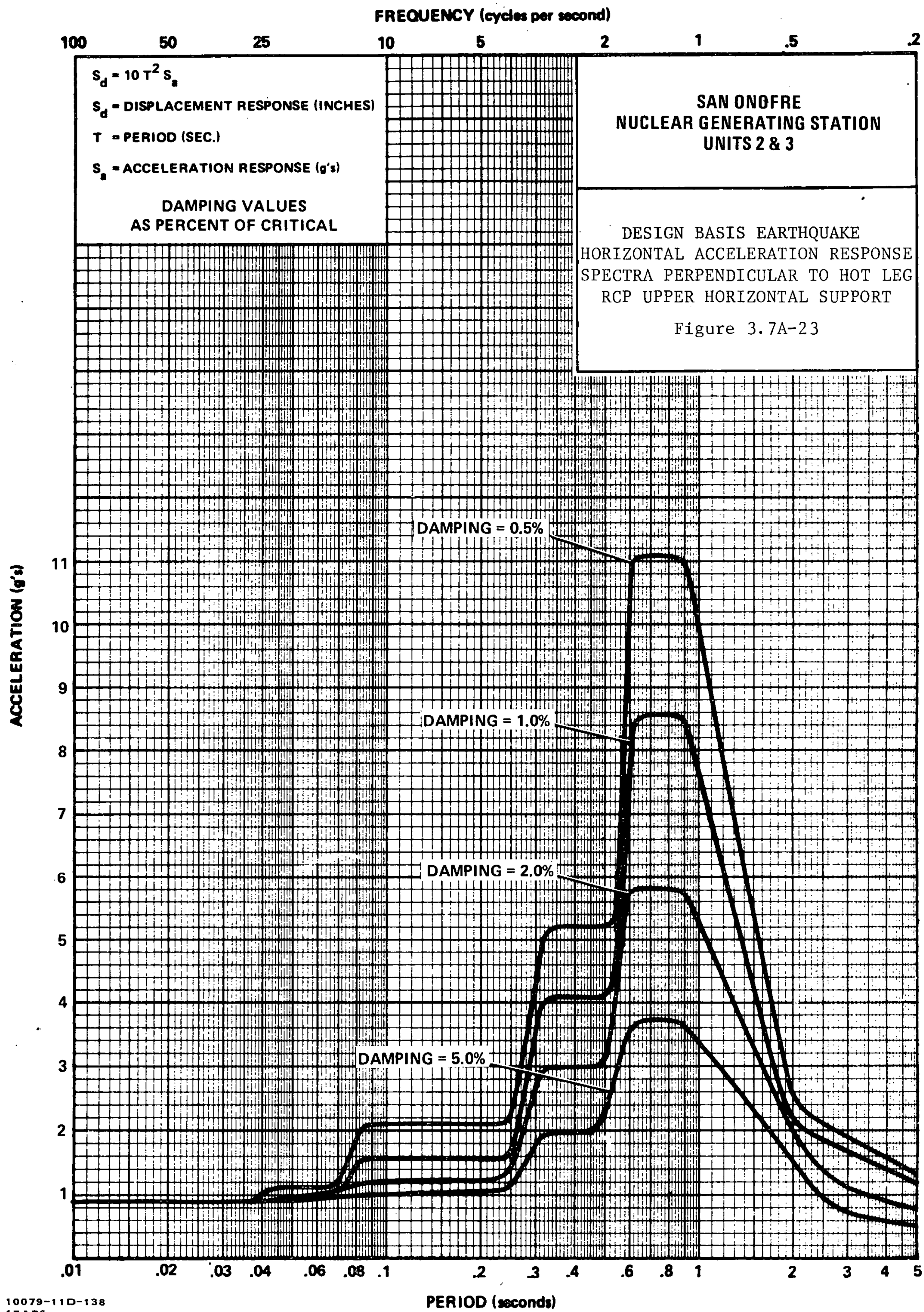


SAN ONOFRE
NUCLEAR GENERATING STATION
UNITS 2 & 3

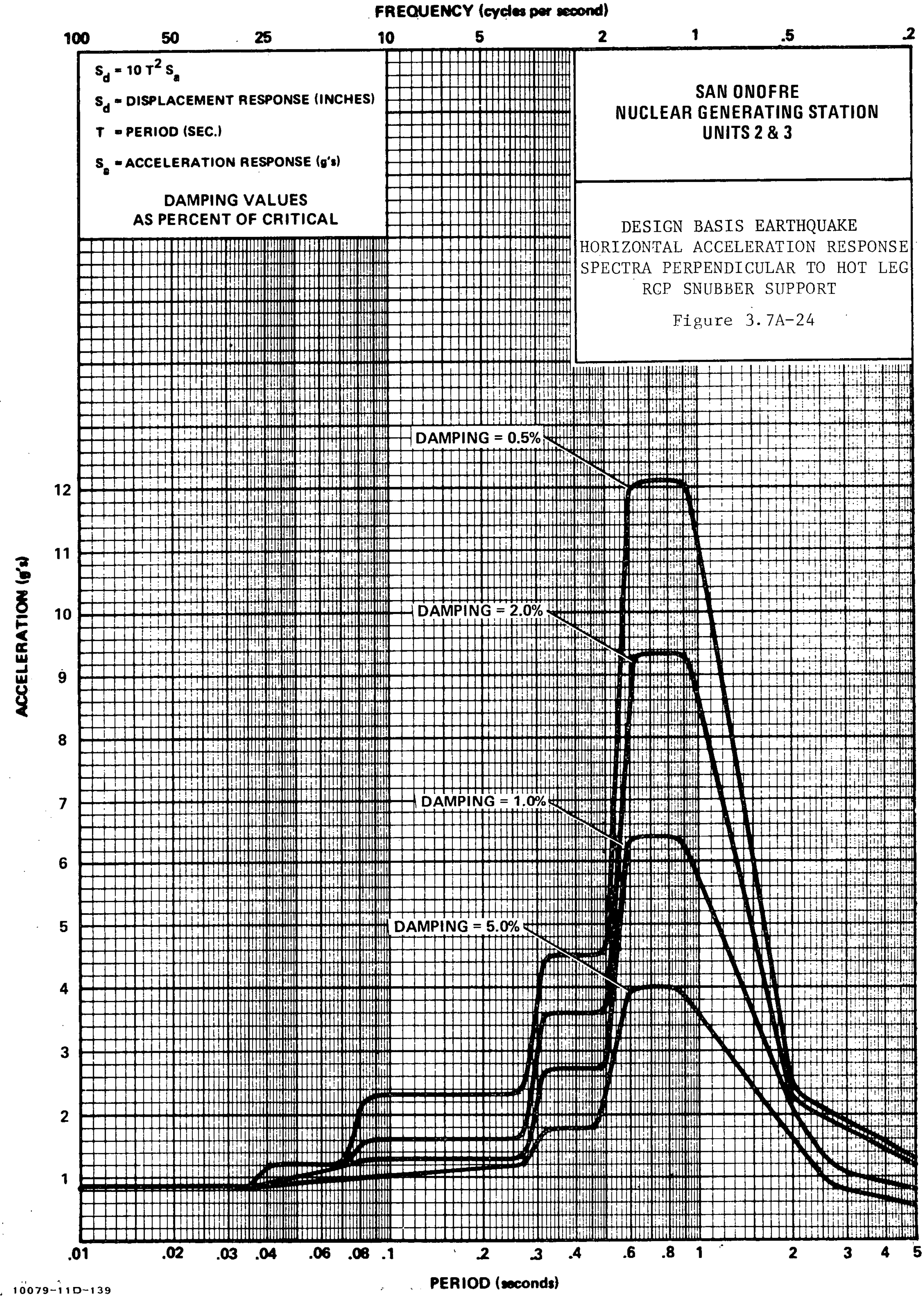
DESIGN BASIS EARTHQUAKE
HORIZONTAL ACCELERATION RESPONSE
SPECTRA PERPENDICULAR TO HOT LEG
RCP LOWER HORIZONTAL SUPPORT

Figure 3.7A-22

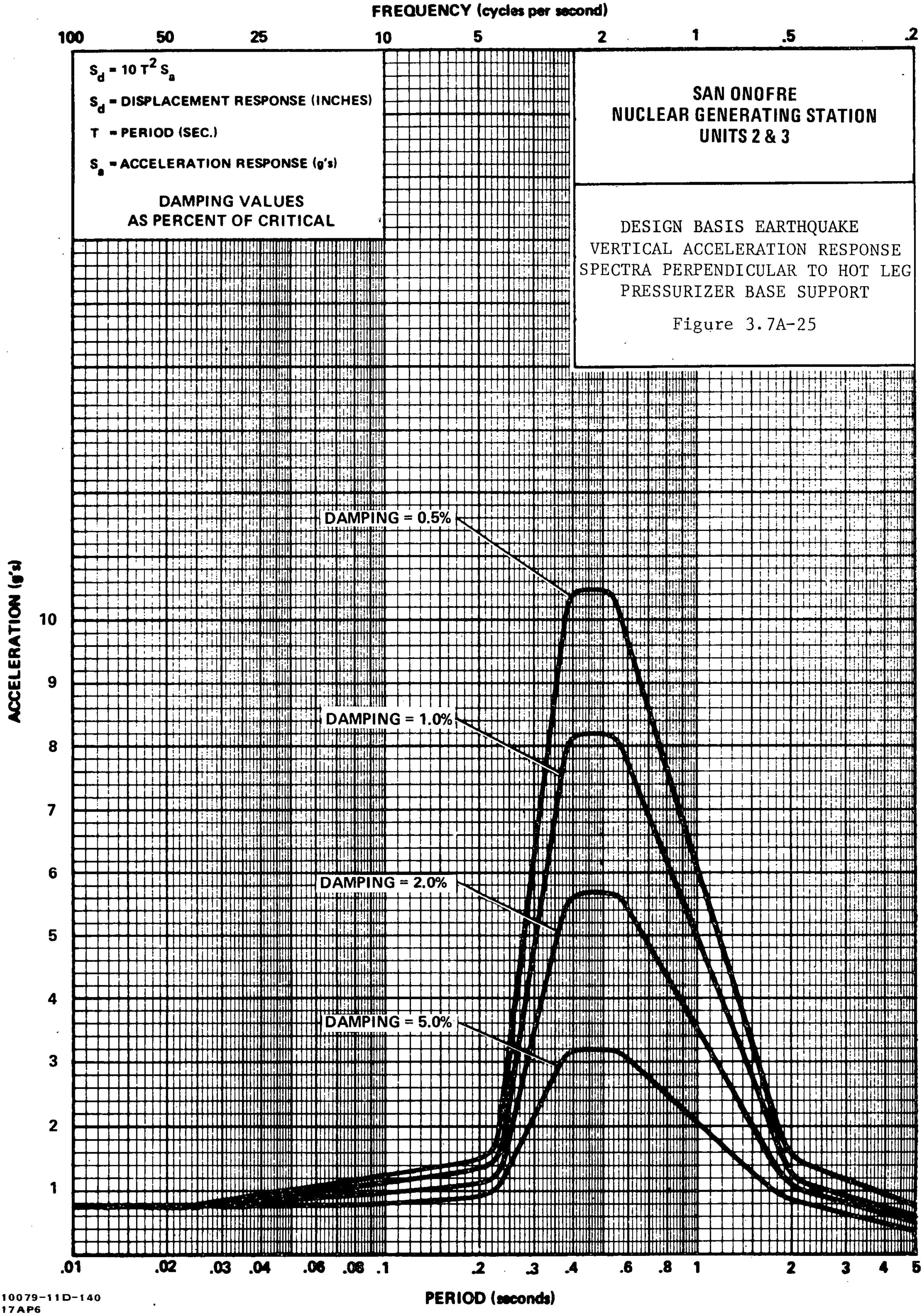
10079-11D-137
17AP6



10079-11D-138
17AP6



10079-11D-139
17AP6

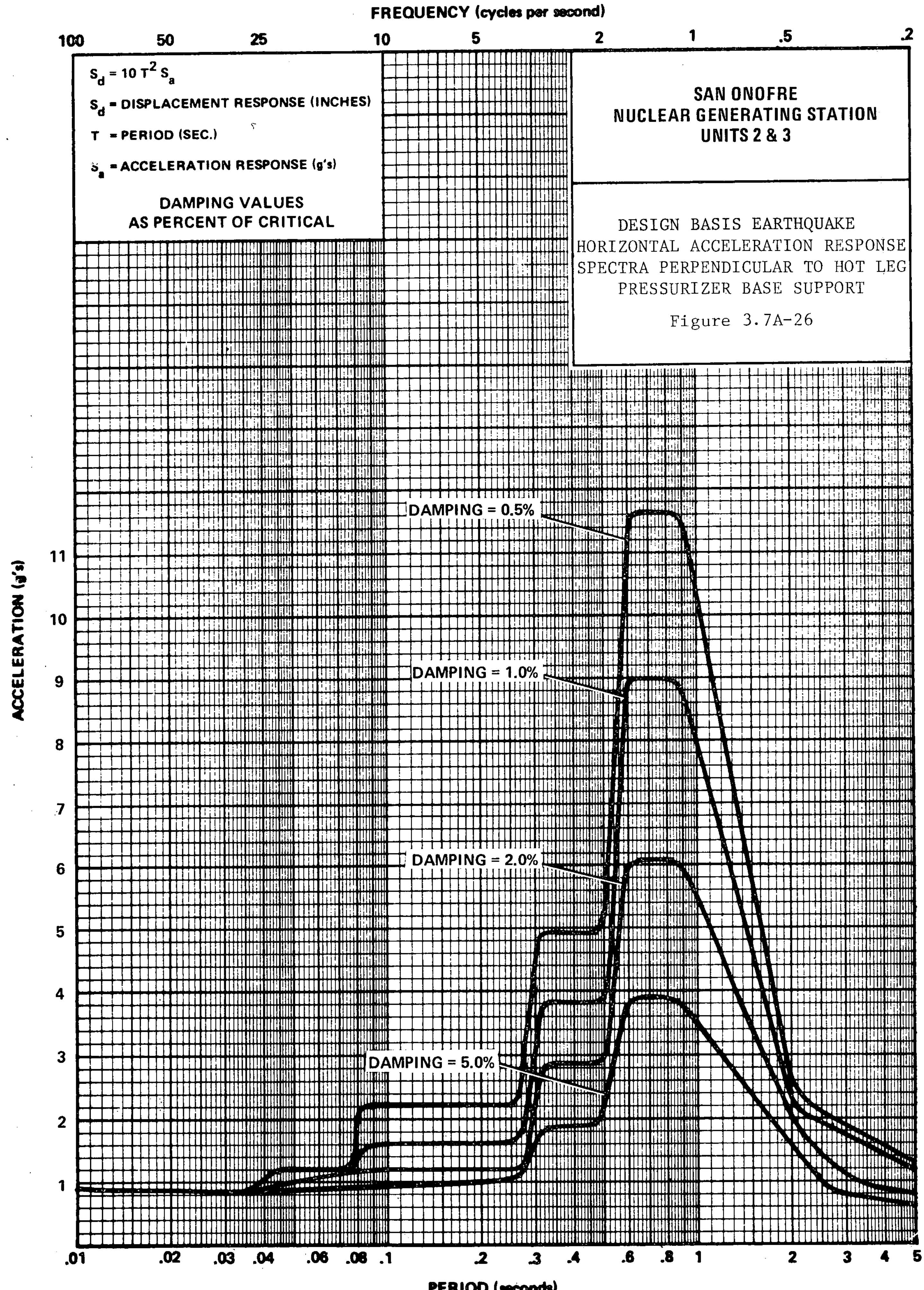


SAN ONOFRE
NUCLEAR GENERATING STATION
UNITS 2 & 3

DESIGN BASIS EARTHQUAKE
VERTICAL ACCELERATION RESPONSE
SPECTRA PERPENDICULAR TO HOT LEG
PRESSURIZER BASE SUPPORT

Figure 3.7A-25

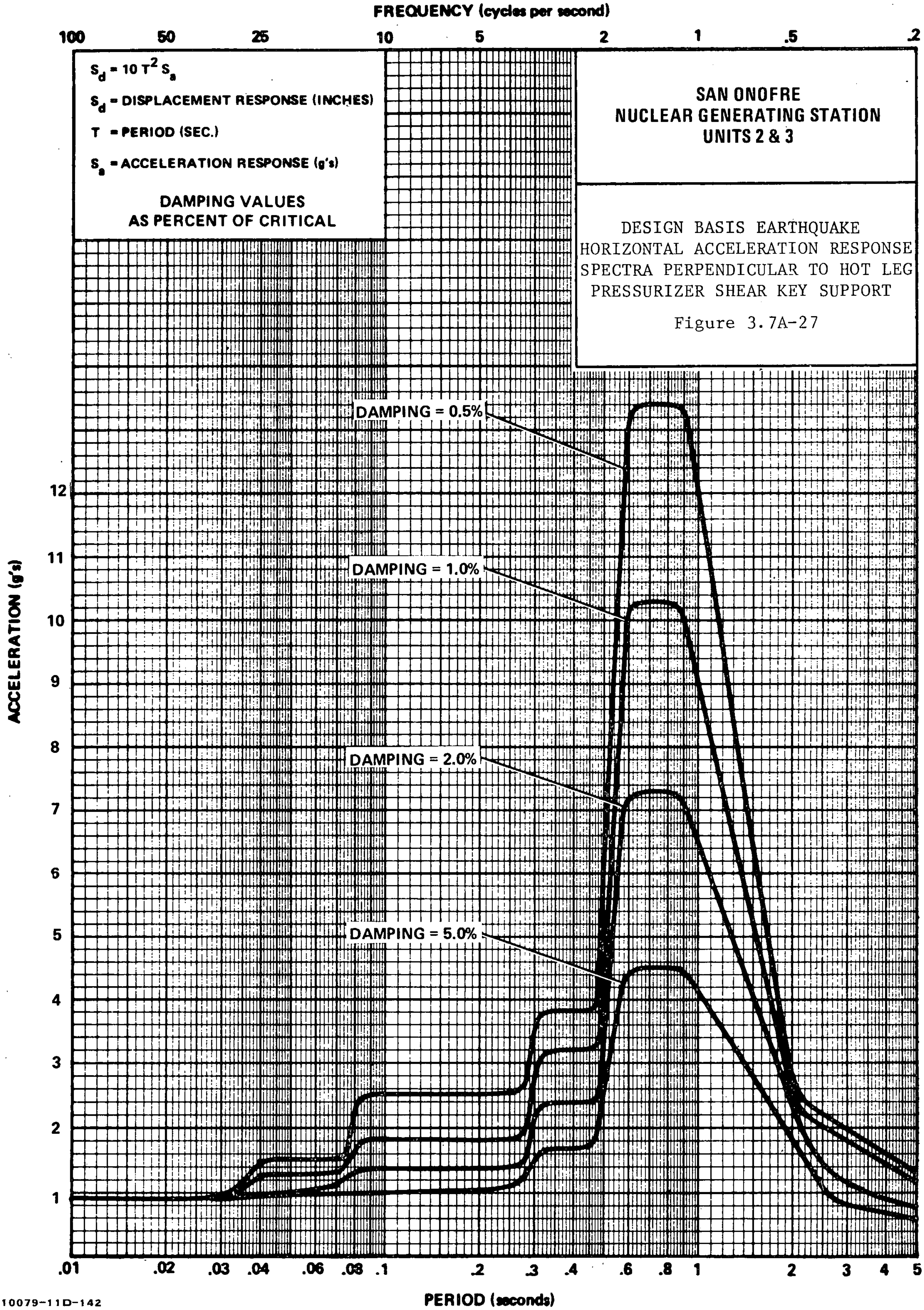
10079-11D-140
17AP6



DESIGN BASIS EARTHQUAKE
HORIZONTAL ACCELERATION RESPONSE
SPECTRA PERPENDICULAR TO HOT LEG
PRESSURIZER BASE SUPPORT

Figure 3.7A-26

10079-11D-141
17AP6

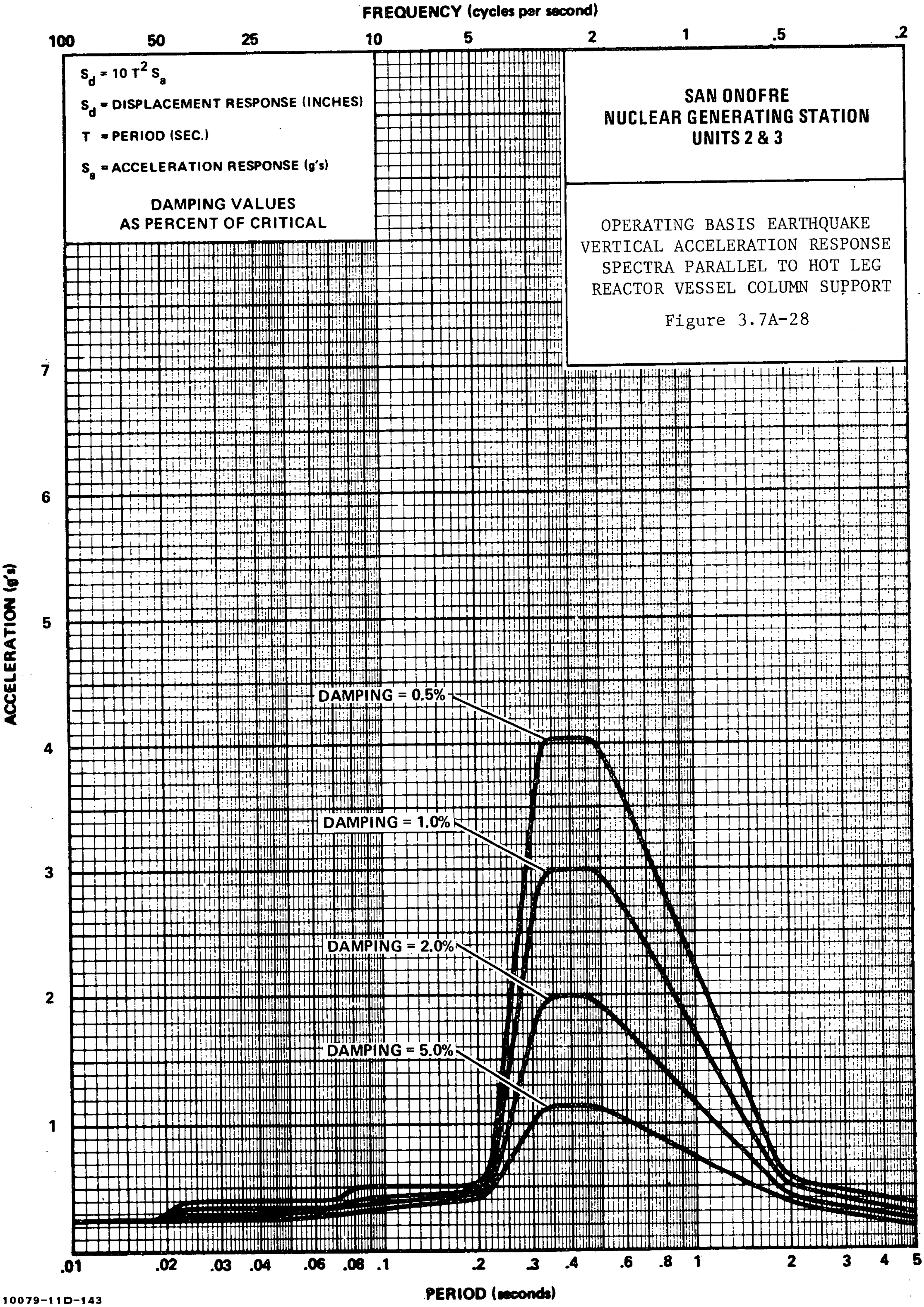


SAN ONOFRE
NUCLEAR GENERATING STATION
UNITS 2 & 3

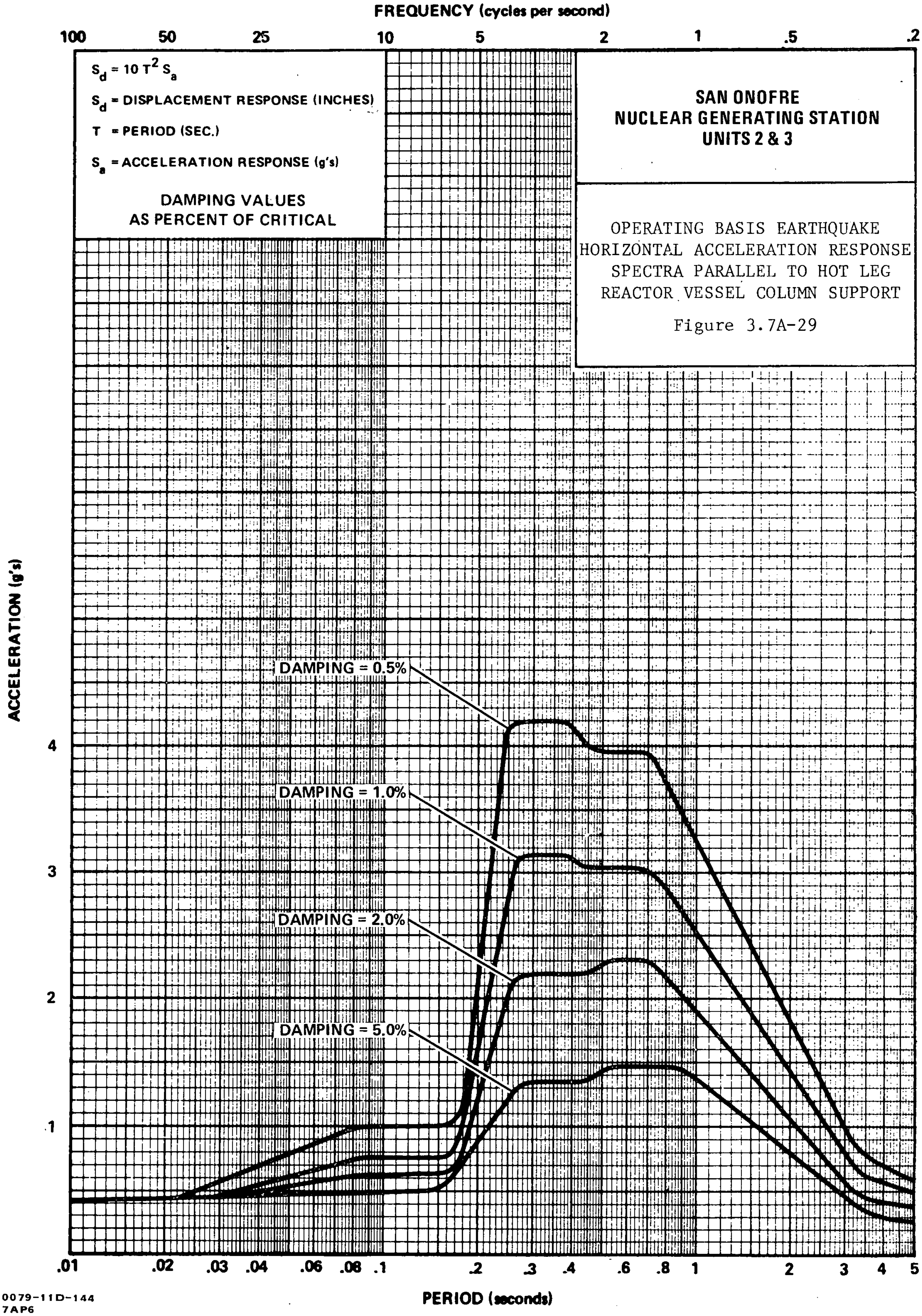
DESIGN BASIS EARTHQUAKE
HORIZONTAL ACCELERATION RESPONSE
SPECTRA PERPENDICULAR TO HOT LEG
PRESSURIZER SHEAR KEY SUPPORT

Figure 3.7A-27

10079-11D-142
17AP6



10079-11D-143
17AP6

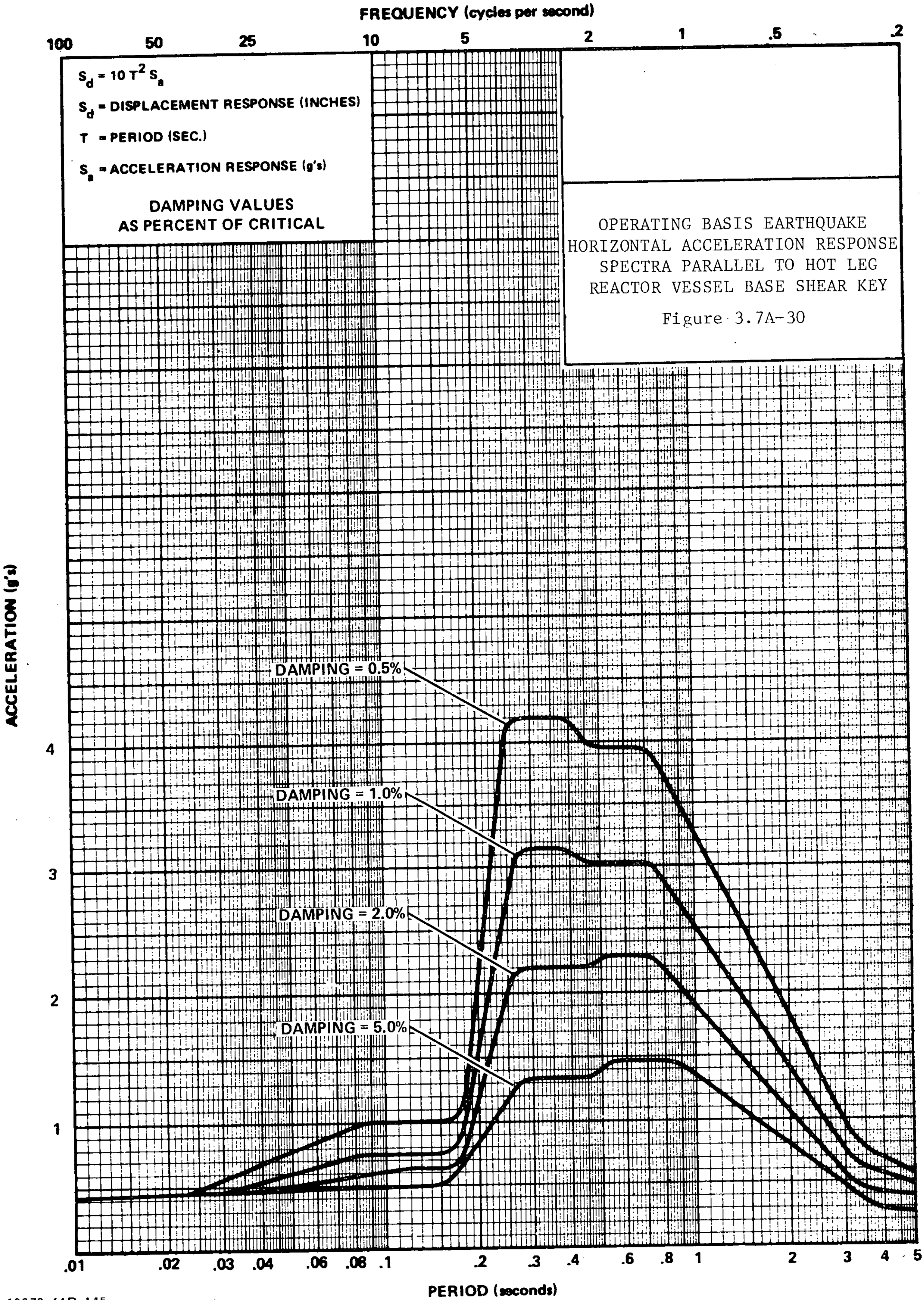


SAN ONOFRE
NUCLEAR GENERATING STATION
UNITS 2 & 3

OPERATING BASIS EARTHQUAKE
HORIZONTAL ACCELERATION RESPONSE
SPECTRA PARALLEL TO HOT LEG
REACTOR VESSEL COLUMN SUPPORT

Figure 3.7A-29

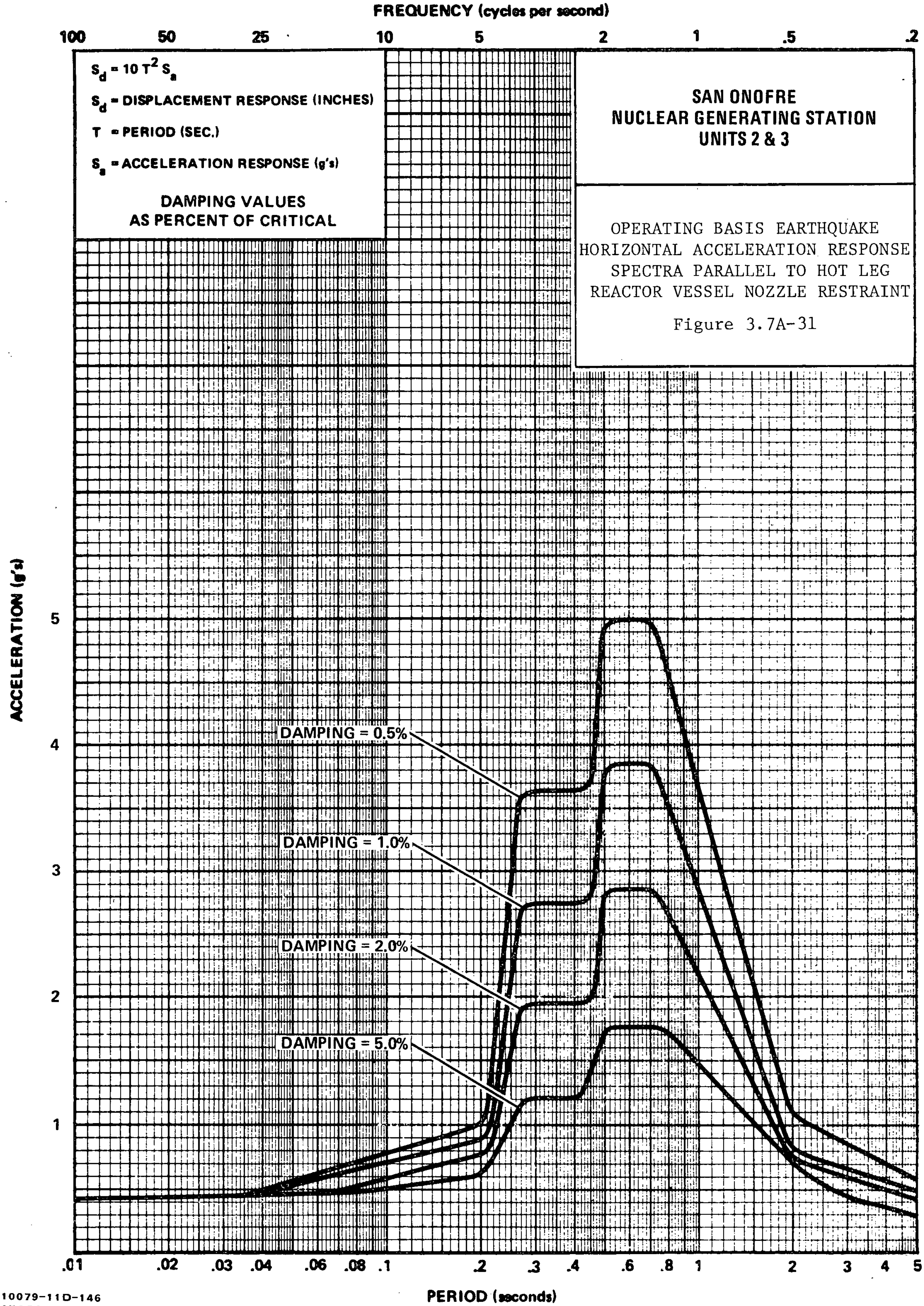
10079-11D-144
17AP6



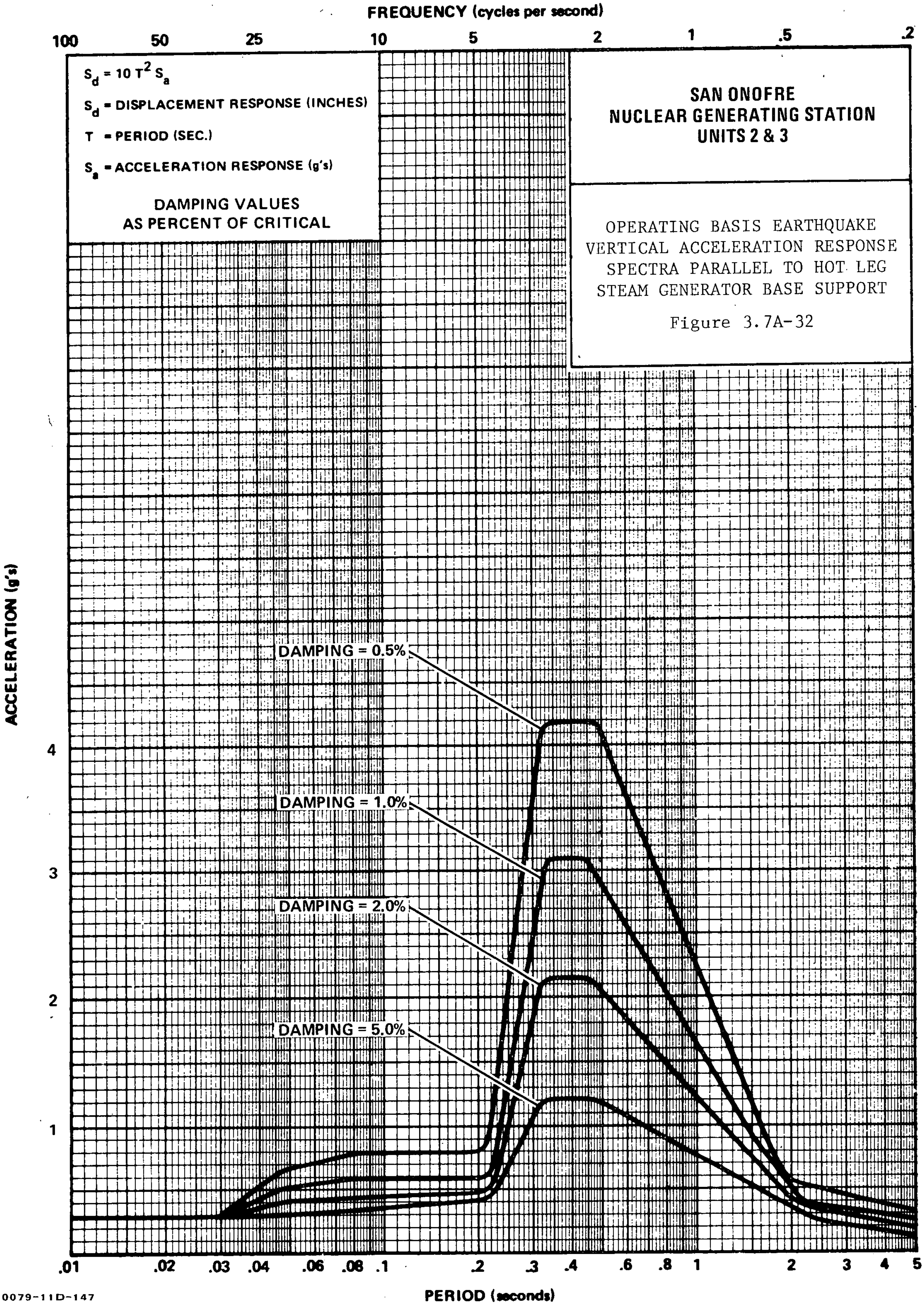
OPERATING BASIS EARTHQUAKE
HORIZONTAL ACCELERATION RESPONSE
SPECTRA PARALLEL TO HOT LEG
REACTOR VESSEL BASE SHEAR KEY

Figure 3.7A-30

10079-11D-145
17AP6



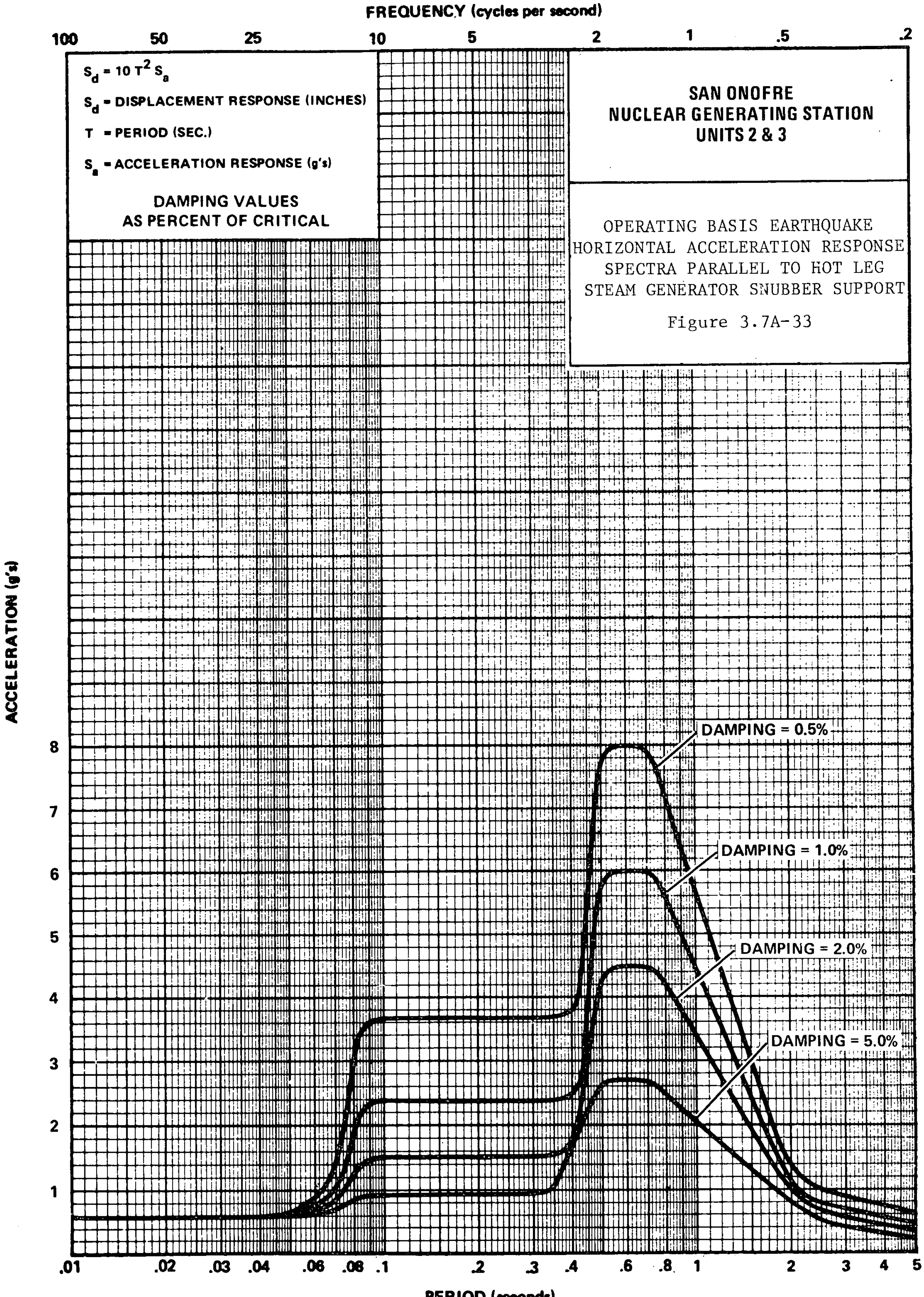
10079-11D-146
17AP6



OPERATING BASIS EARTHQUAKE
VERTICAL ACCELERATION RESPONSE
SPECTRA PARALLEL TO HOT LEG
STEAM GENERATOR BASE SUPPORT

Figure 3.7A-32

10079-11D-147
17AP6

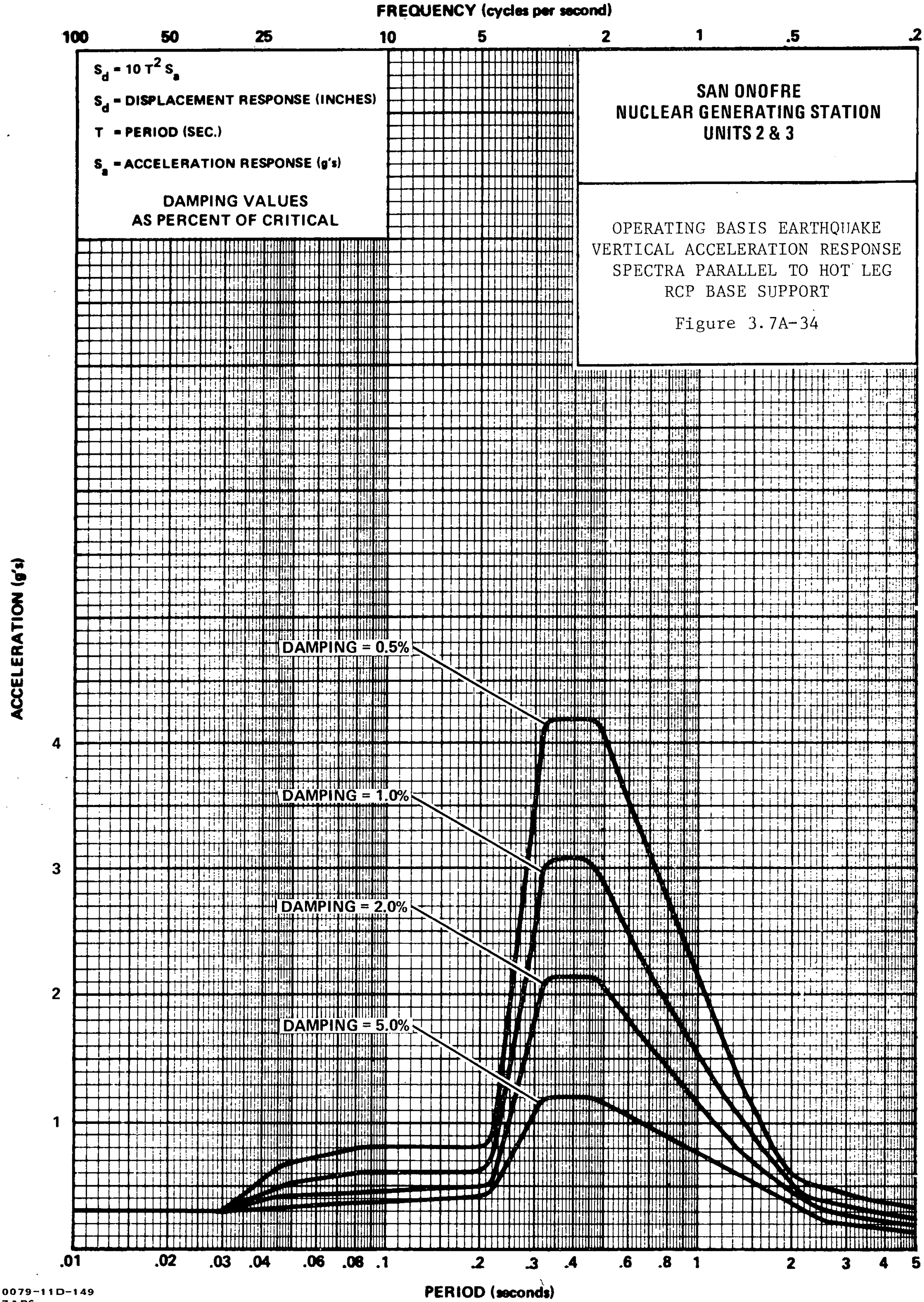


SAN ONOFRE
NUCLEAR GENERATING STATION
UNITS 2 & 3

OPERATING BASIS EARTHQUAKE
HORIZONTAL ACCELERATION RESPONSE
SPECTRA PARALLEL TO HOT LEG
STEAM GENERATOR SNUBBER SUPPORT

Figure 3.7A-33

10079-11D-148
17AP6

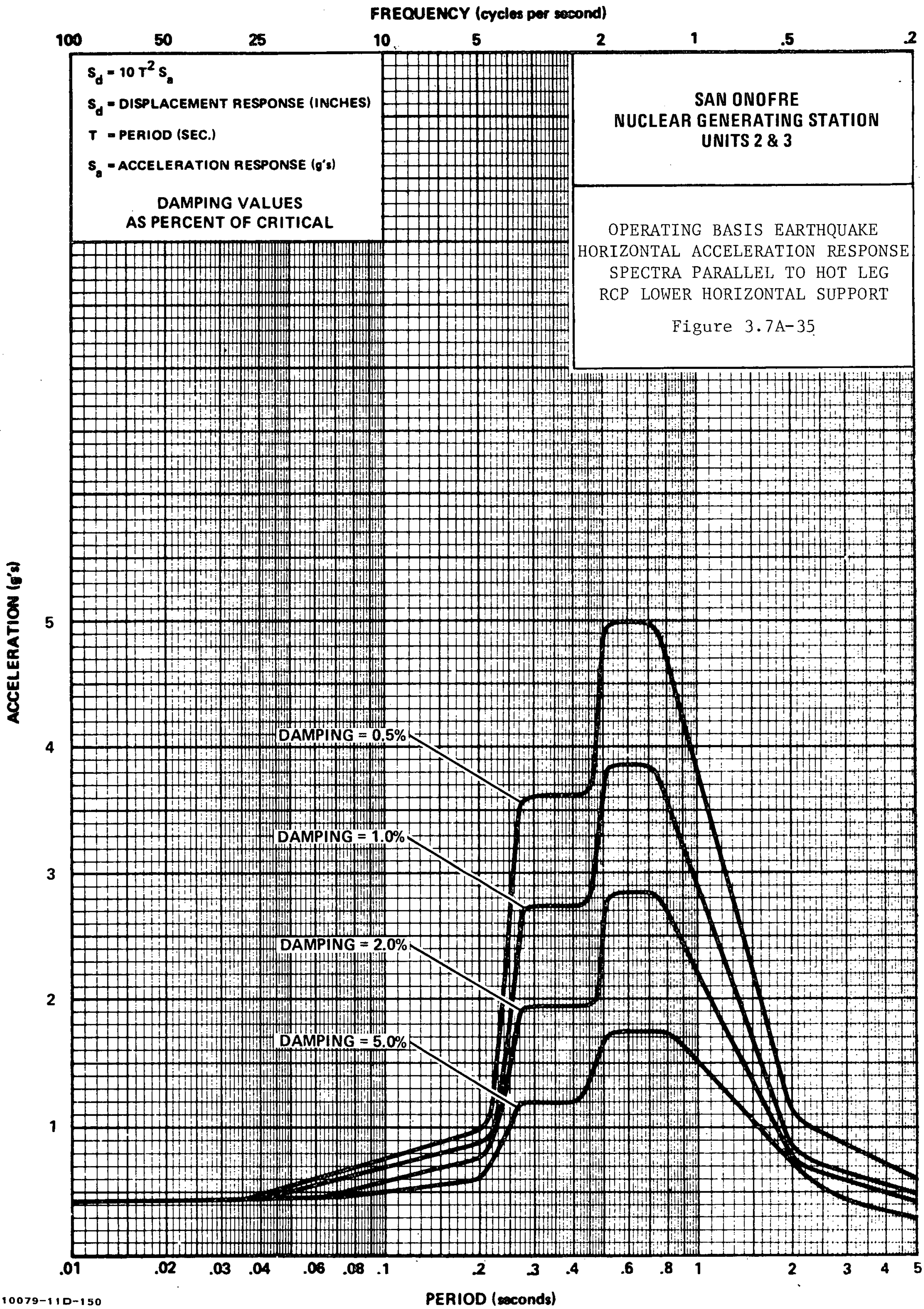


SAN ONOFRE
NUCLEAR GENERATING STATION
UNITS 2 & 3

OPERATING BASIS EARTHQUAKE
VERTICAL ACCELERATION RESPONSE
SPECTRA PARALLEL TO HOT LEG
RCP BASE SUPPORT

Figure 3.7A-34

10079-11D-149
17AP6

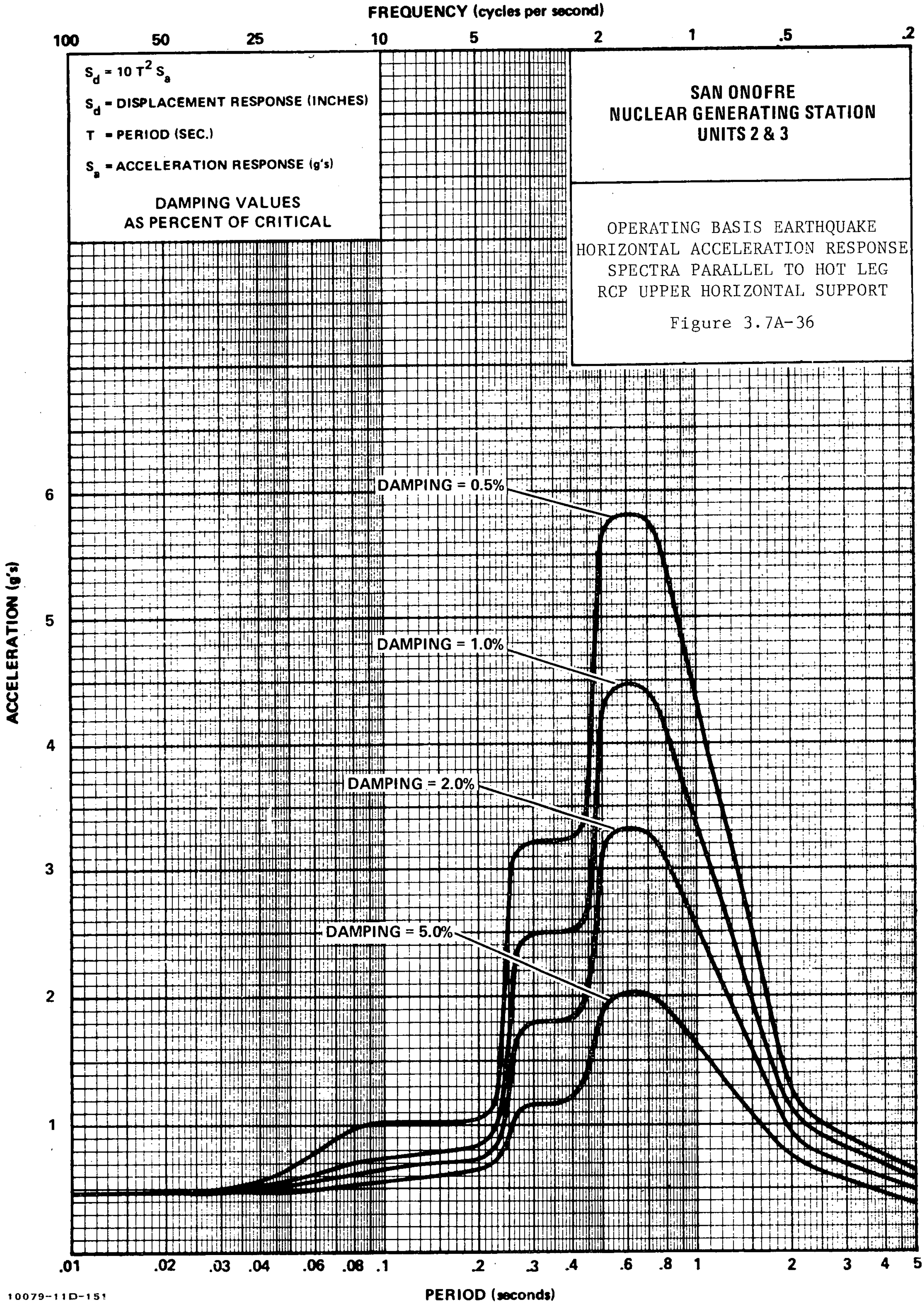


SAN ONOFRE
NUCLEAR GENERATING STATION
UNITS 2 & 3

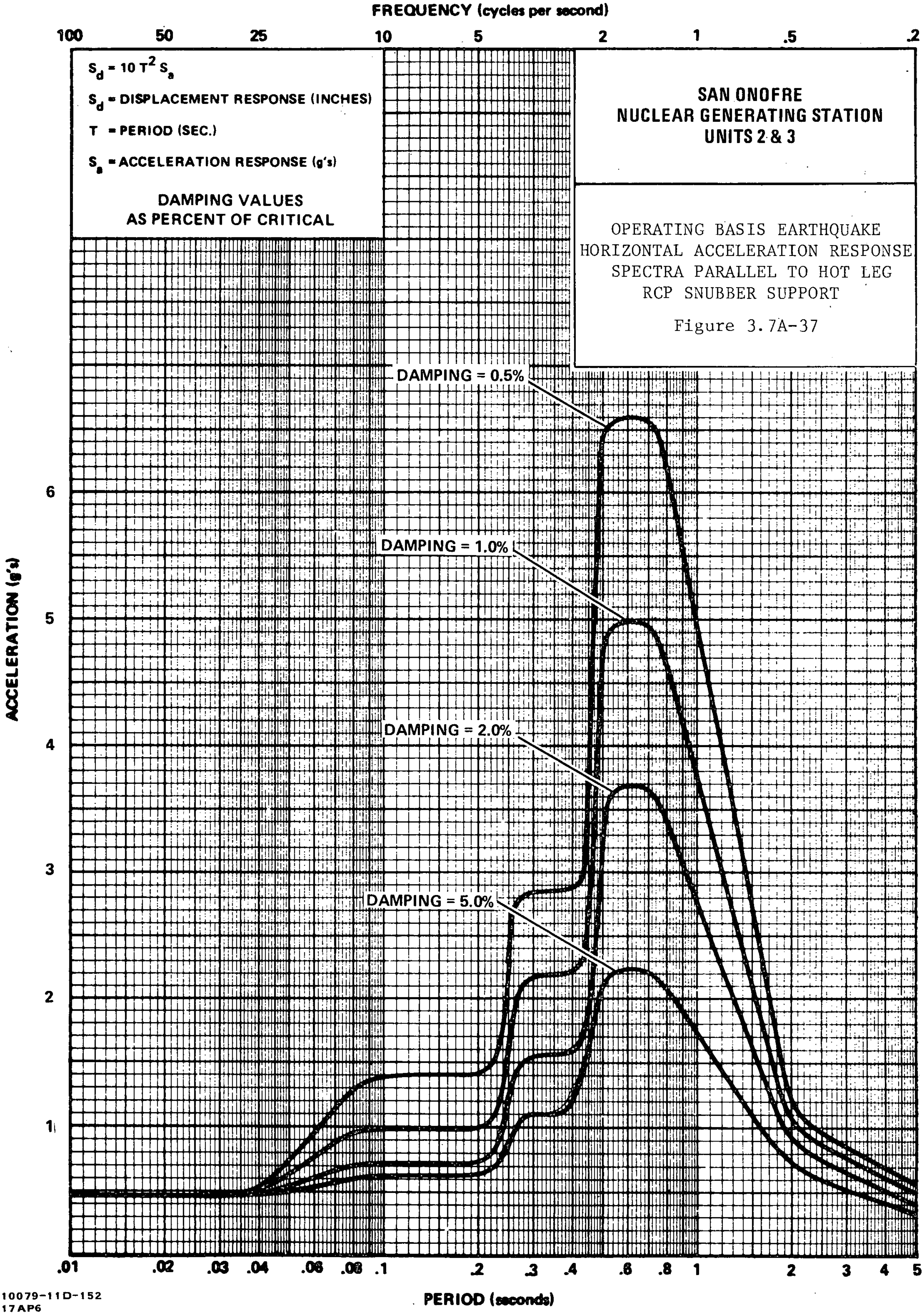
OPERATING BASIS EARTHQUAKE
HORIZONTAL ACCELERATION RESPONSE
SPECTRA PARALLEL TO HOT LEG
RCP LOWER HORIZONTAL SUPPORT

Figure 3.7A-35

10079-11D-150
17AP6



10079-11D-151
17AP6

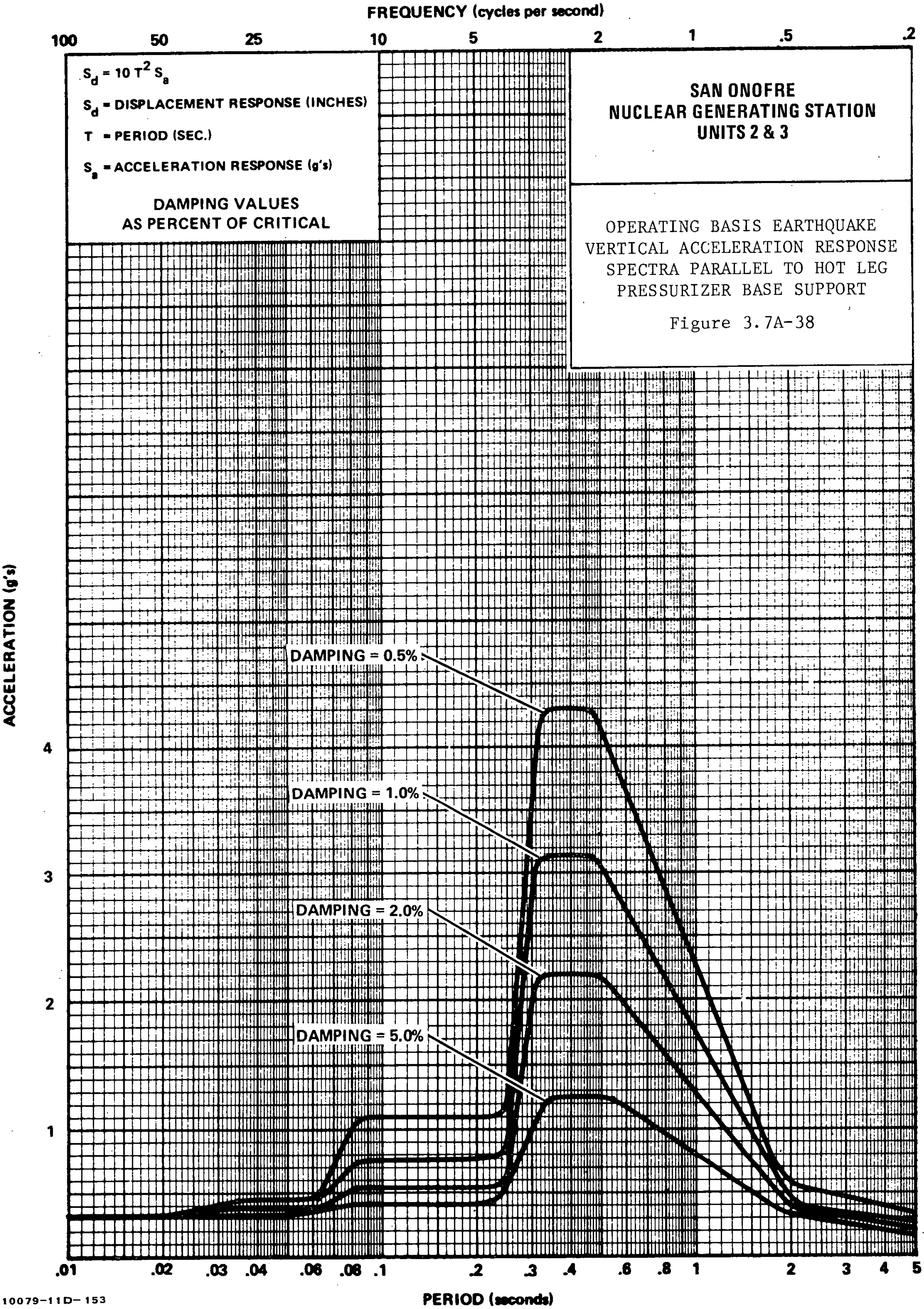


SAN ONOFRE
NUCLEAR GENERATING STATION
UNITS 2 & 3

OPERATING BASIS EARTHQUAKE
HORIZONTAL ACCELERATION RESPONSE
SPECTRA PARALLEL TO HOT LEG
RCP SNUBBER SUPPORT

Figure 3.7A-37

10079-11D-152
17AP6

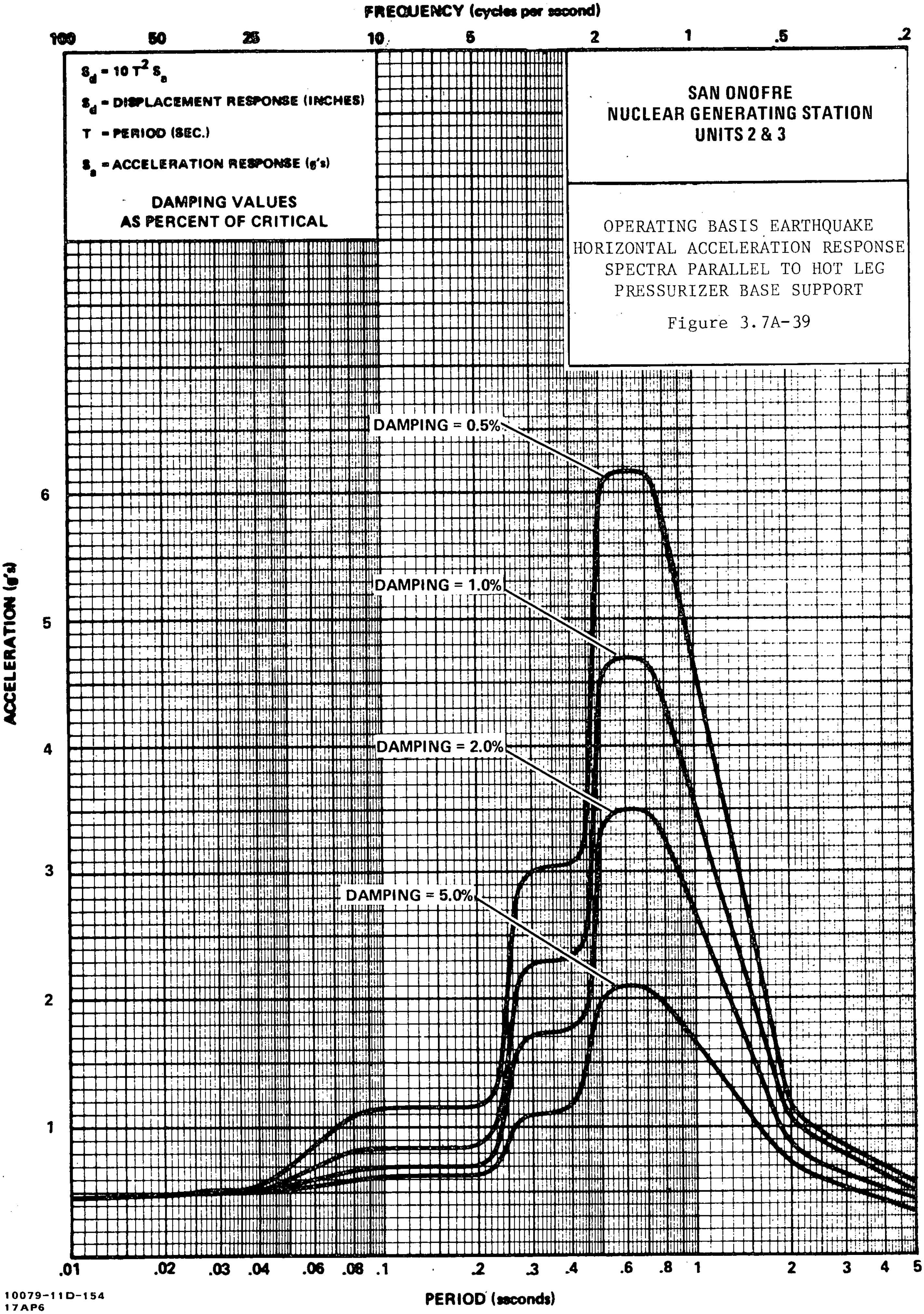


SAN ONOFRE
NUCLEAR GENERATING STATION
UNITS 2 & 3

OPERATING BASIS EARTHQUAKE
VERTICAL ACCELERATION RESPONSE
SPECTRA PARALLEL TO HOT LEG
PRESSURIZER BASE SUPPORT

Figure 3.7A-38

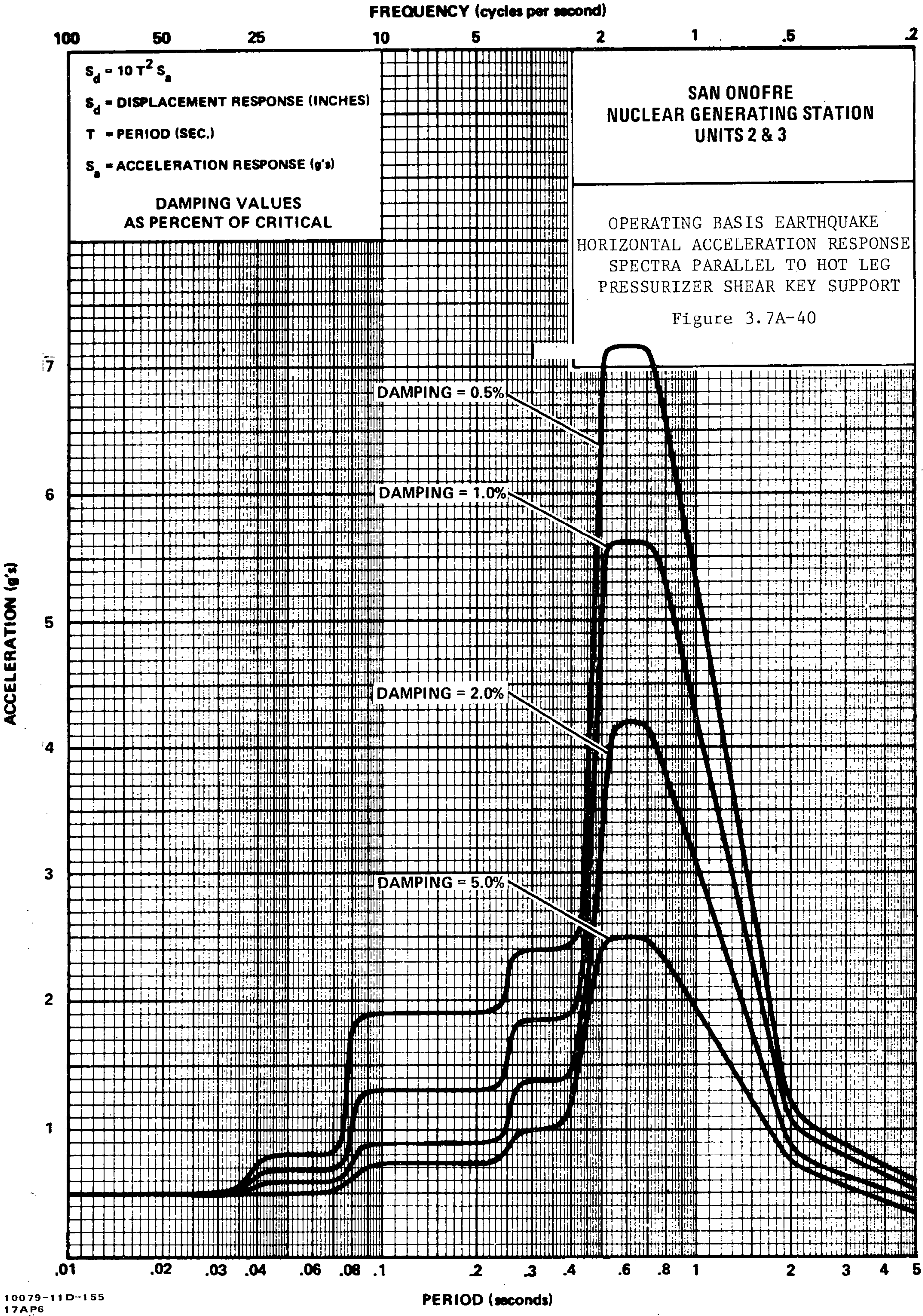
10079-11D-153
17AP6



OPERATING BASIS EARTHQUAKE
HORIZONTAL ACCELERATION RESPONSE
SPECTRA PARALLEL TO HOT LEG
PRESSURIZER BASE SUPPORT

Figure 3.7A-39

10079-11D-154
17AP6

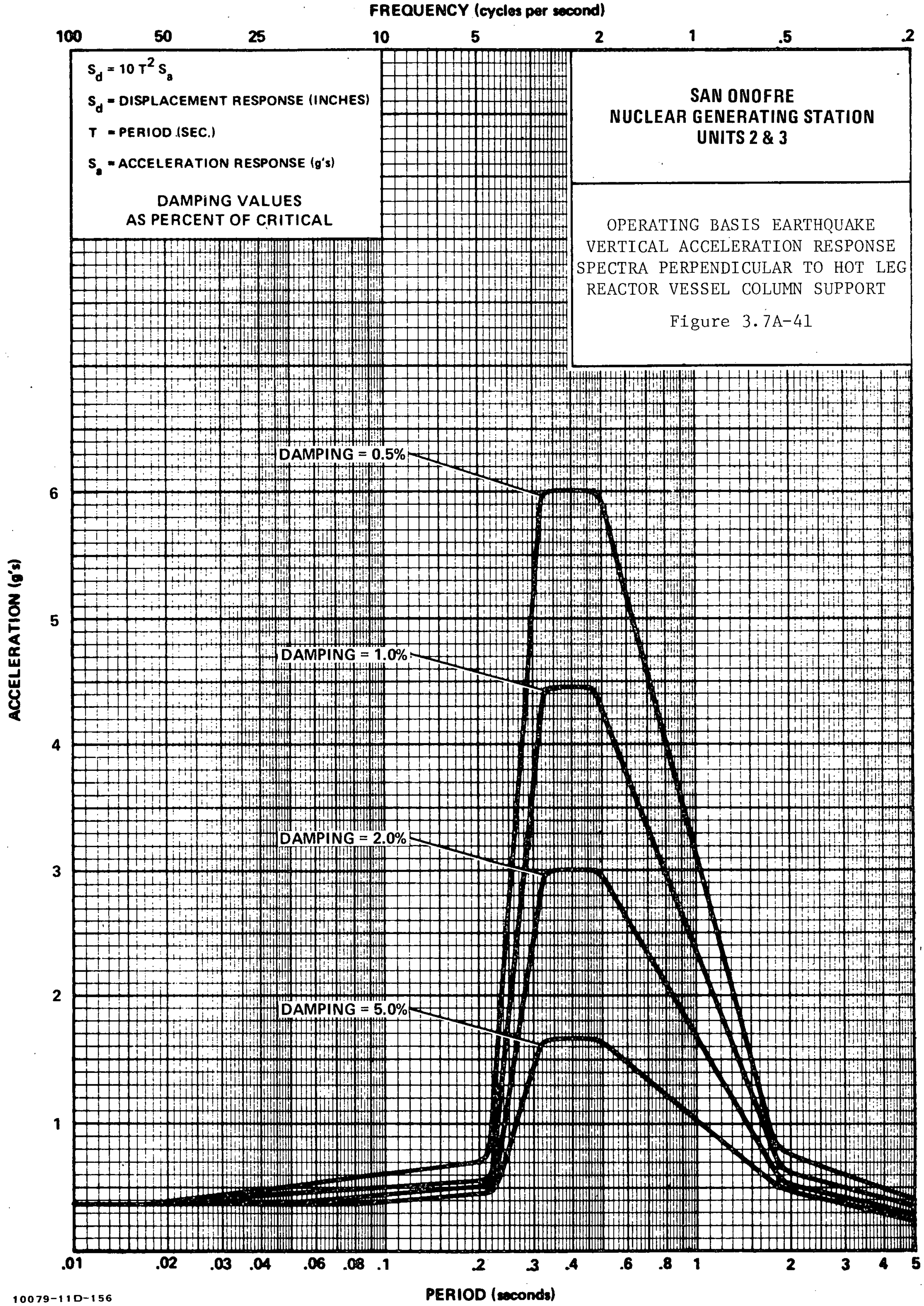


SAN ONOFRE
NUCLEAR GENERATING STATION
UNITS 2 & 3

OPERATING BASIS EARTHQUAKE
HORIZONTAL ACCELERATION RESPONSE
SPECTRA PARALLEL TO HOT LEG
PRESSURIZER SHEAR KEY SUPPORT

Figure 3.7A-40

10079-11D-155
17AP6

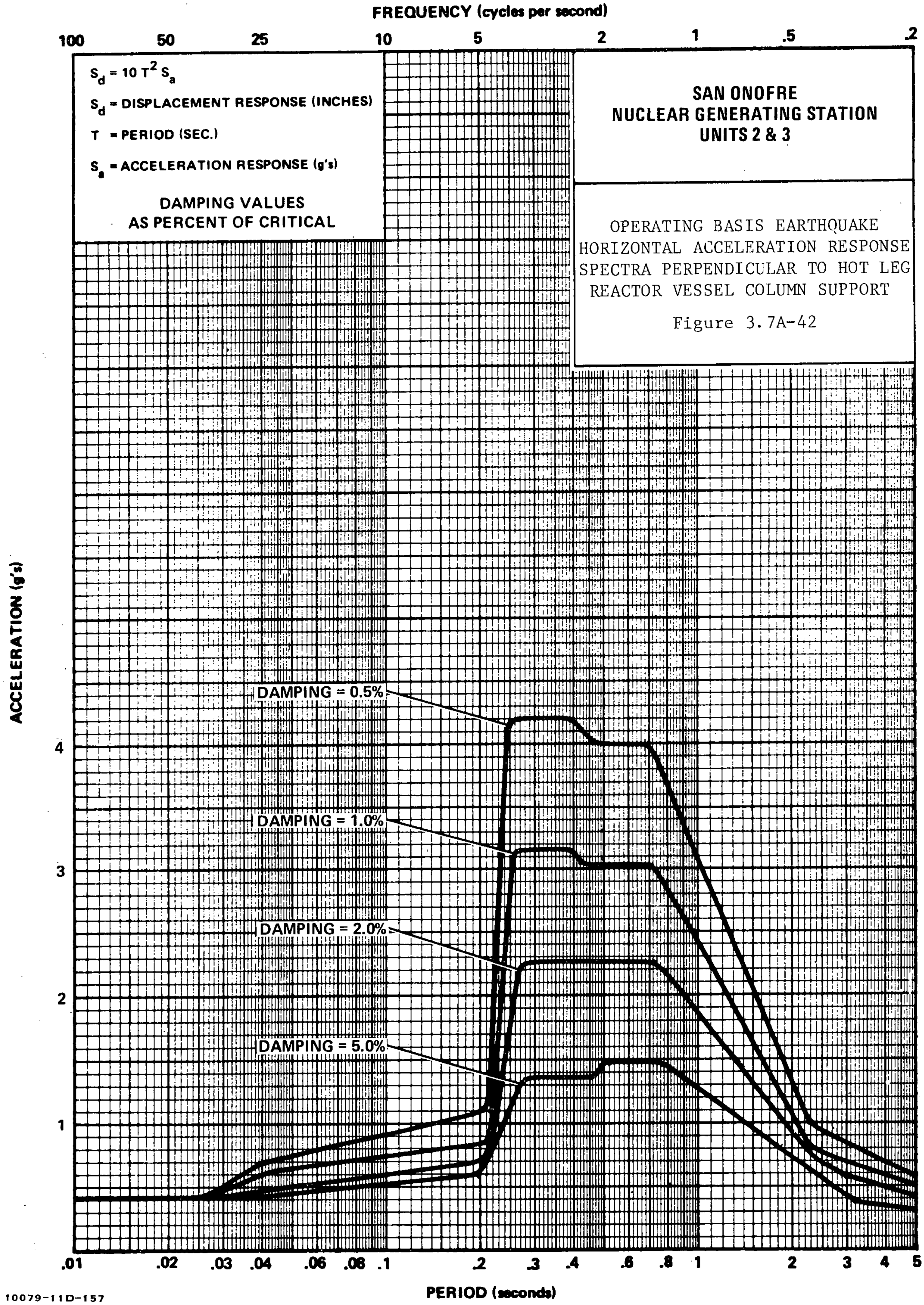


SAN ONOFRE
NUCLEAR GENERATING STATION
UNITS 2 & 3

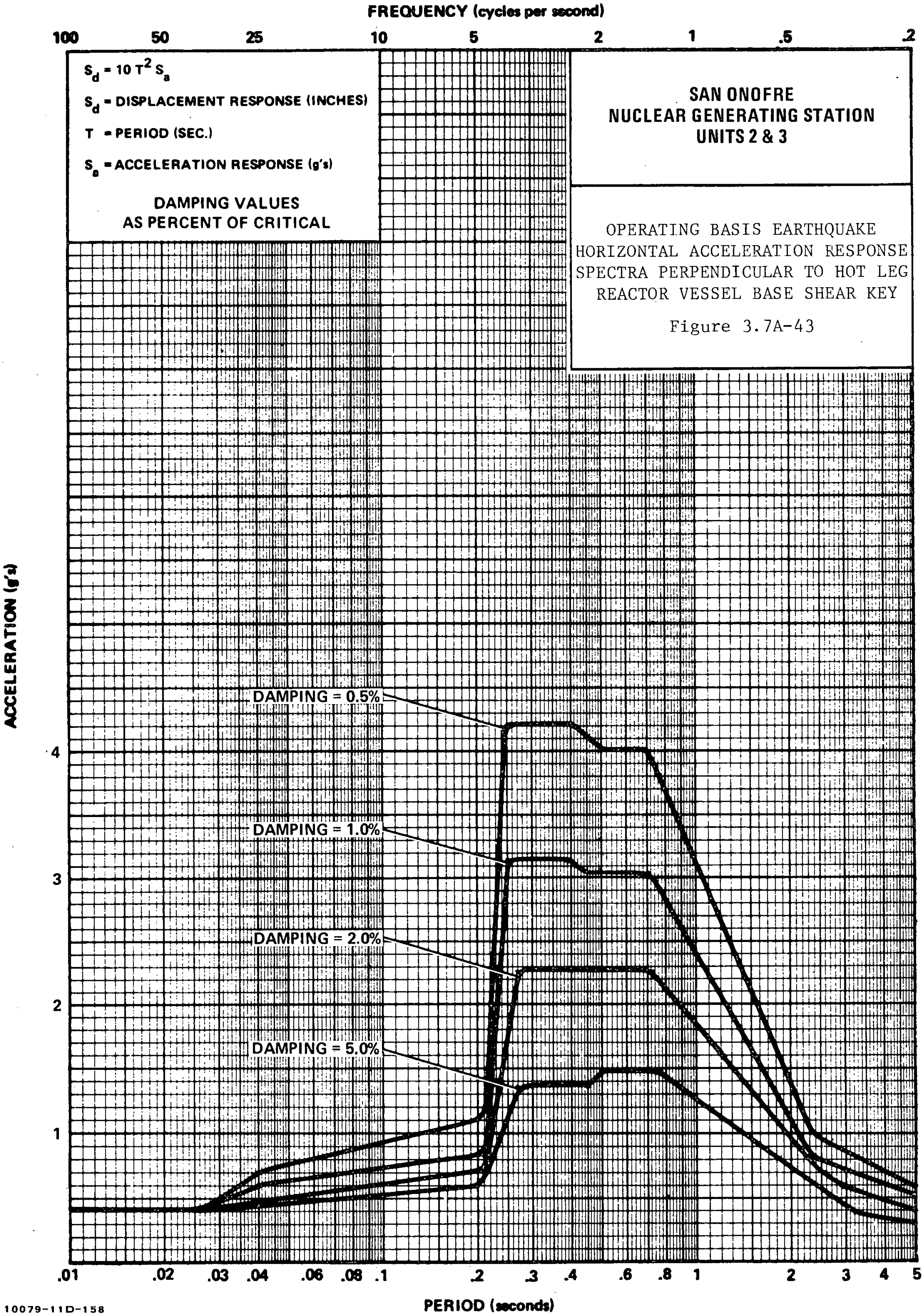
OPERATING BASIS EARTHQUAKE
VERTICAL ACCELERATION RESPONSE
SPECTRA PERPENDICULAR TO HOT LEG
REACTOR VESSEL COLUMN SUPPORT

Figure 3.7A-41

10079-11D-156
17AP6



10079-11D-157
17AP6

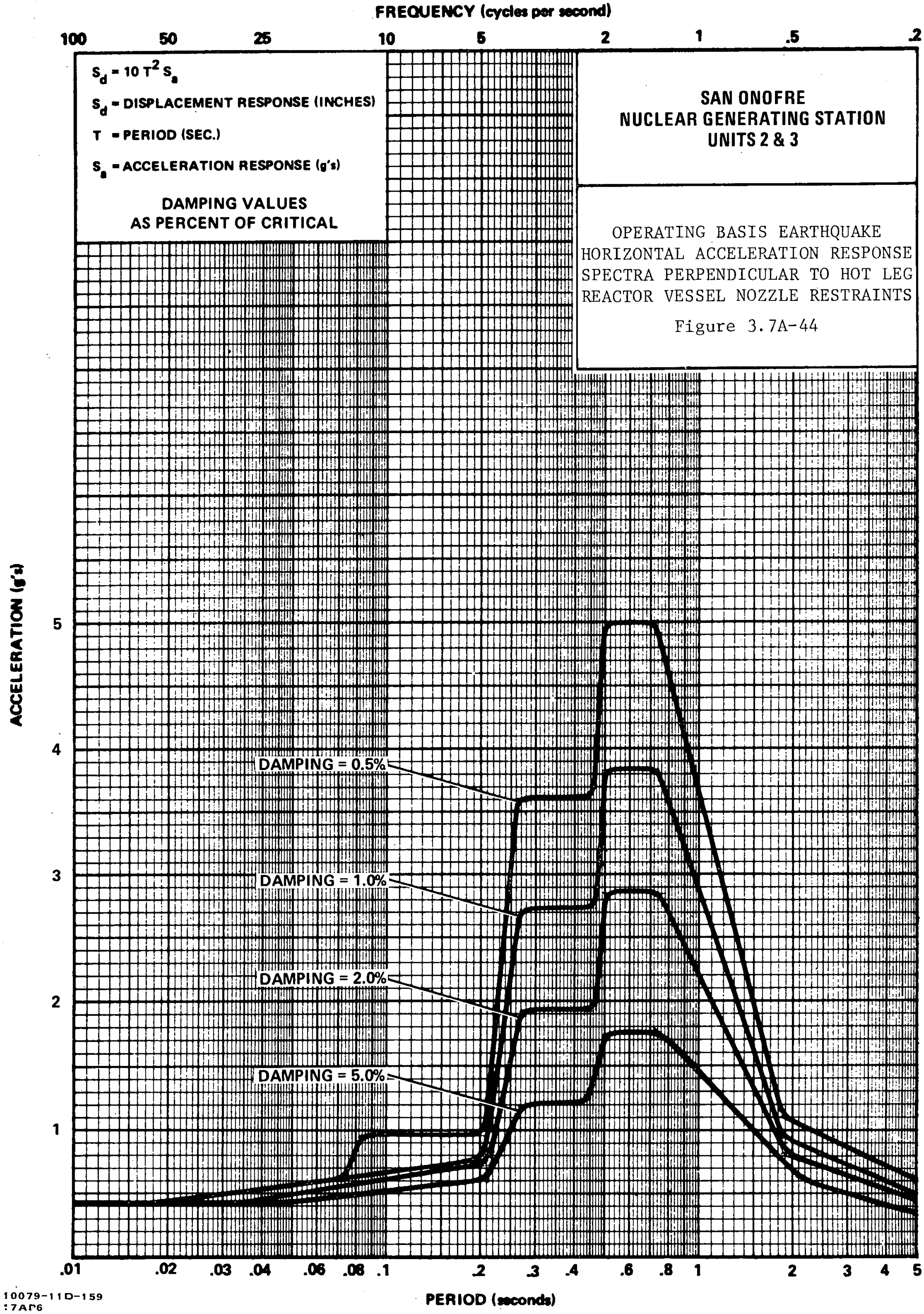


SAN ONOFRE
NUCLEAR GENERATING STATION
UNITS 2 & 3

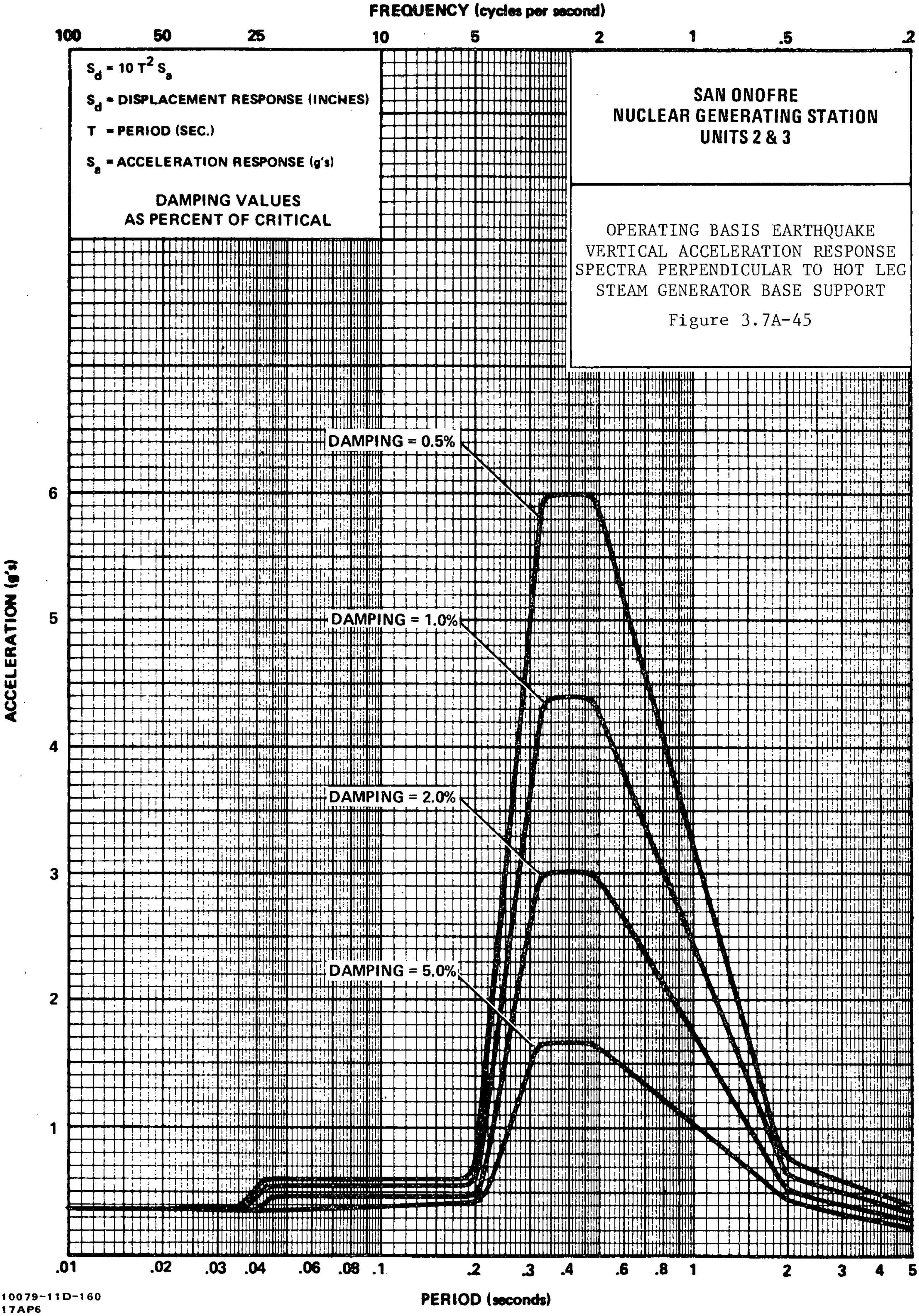
OPERATING BASIS EARTHQUAKE HORIZONTAL ACCELERATION RESPONSE SPECTRA PERPENDICULAR TO HOT LEG REACTOR VESSEL BASE SHEAR KEY

Figure 3.7A-43

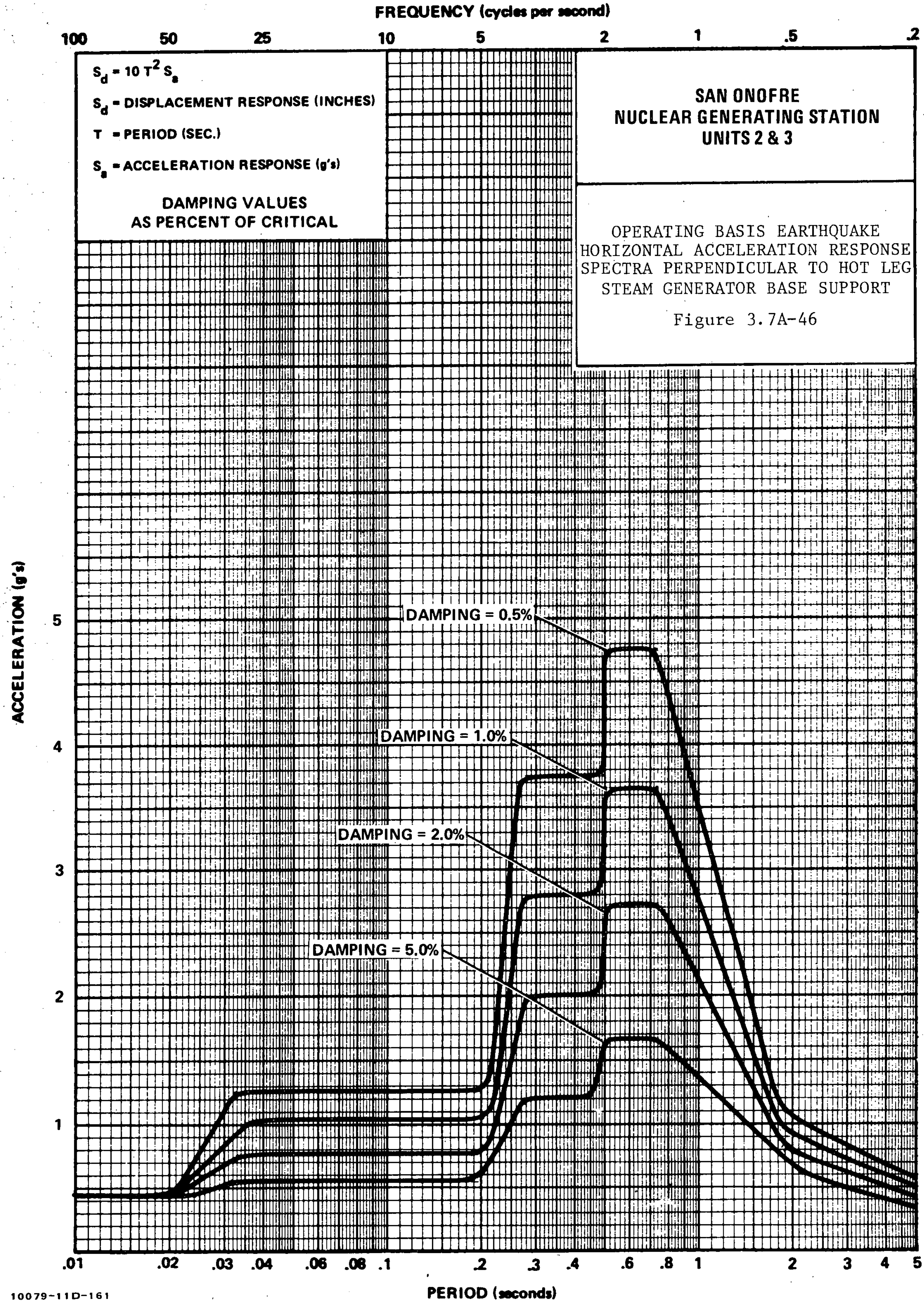
10079-11D-158
17AP6



10079-11D-159
17AP6



10079-11D-160
17AP6

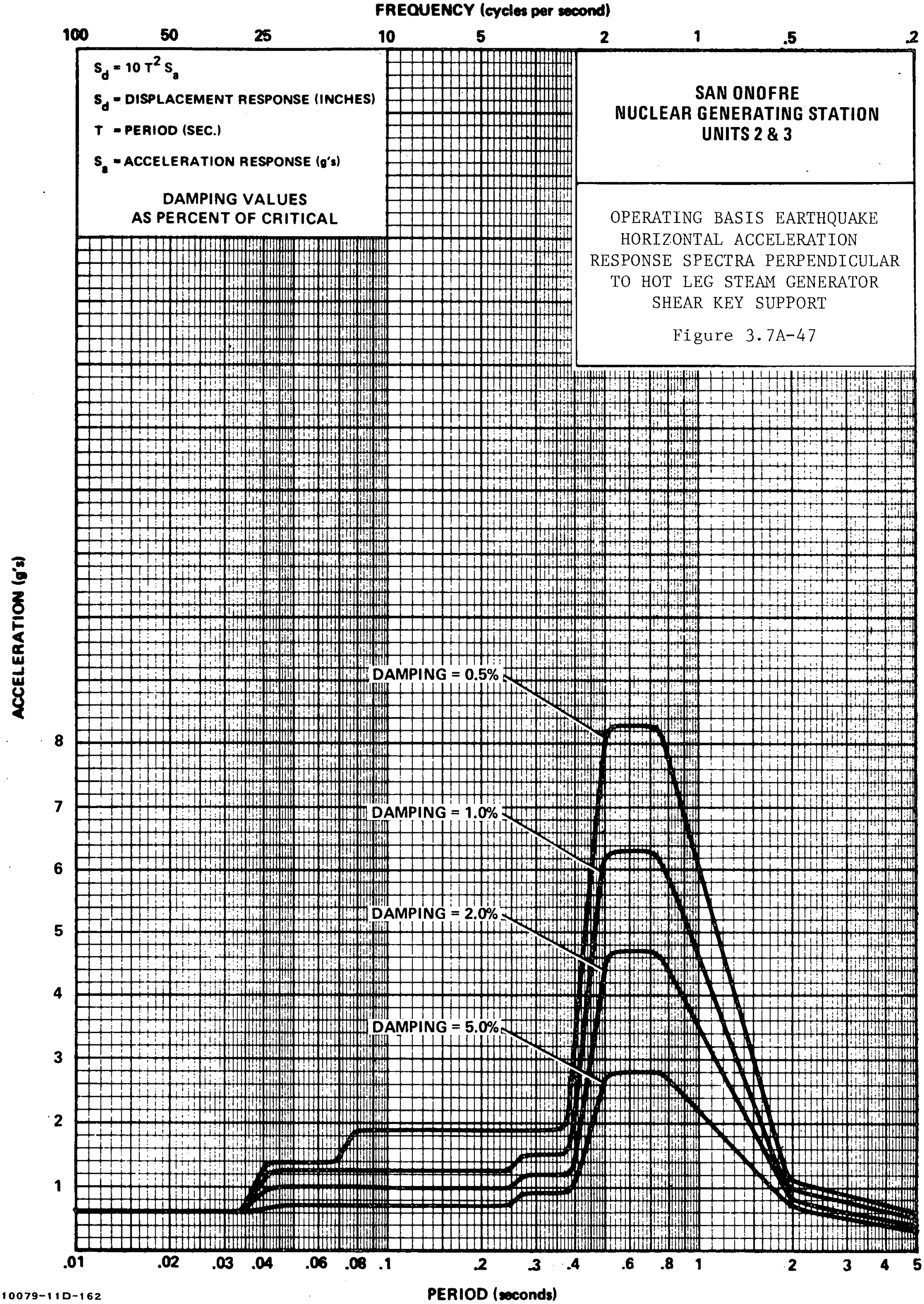


SAN ONOFRE
NUCLEAR GENERATING STATION
UNITS 2 & 3

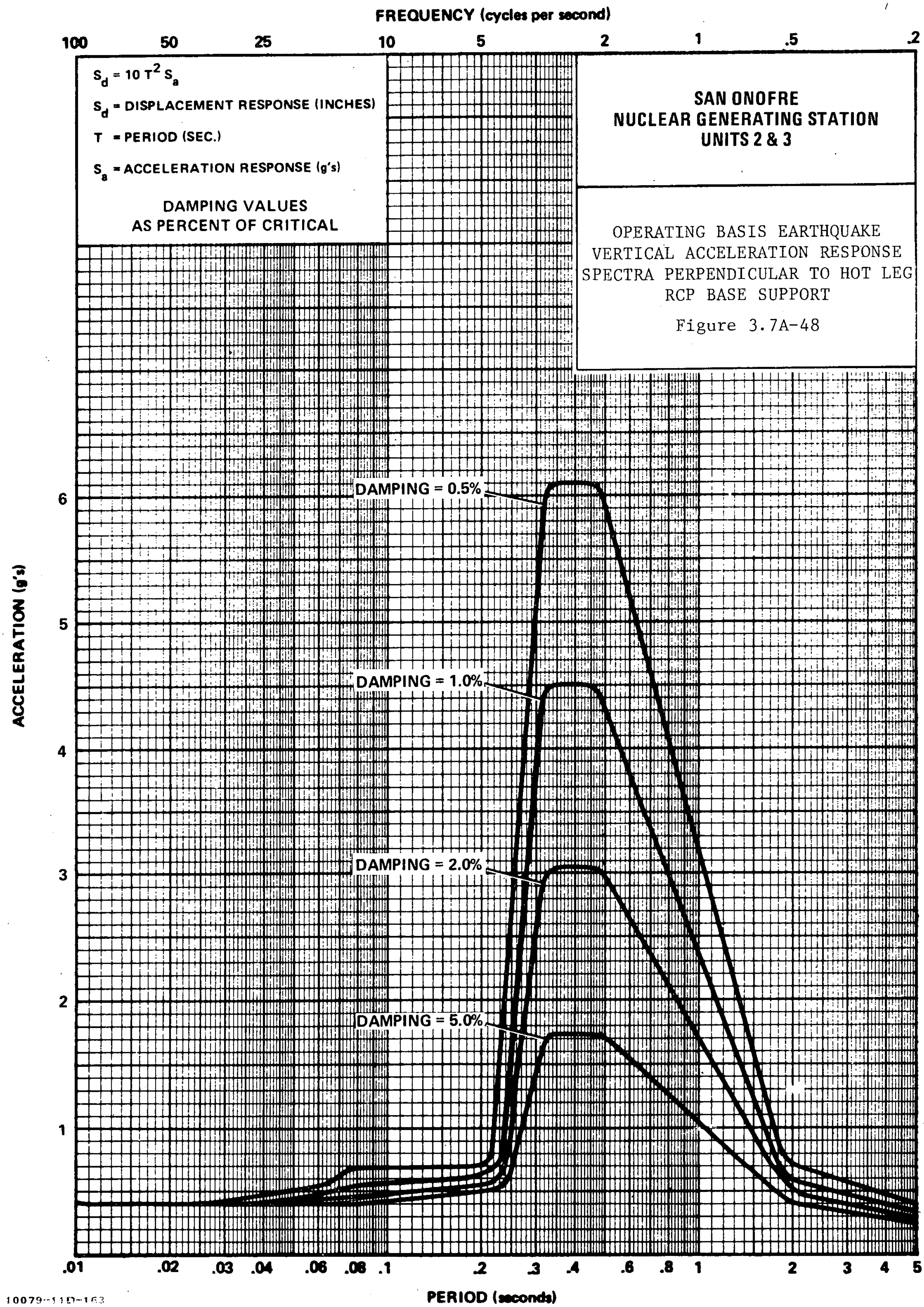
OPERATING BASIS EARTHQUAKE
HORIZONTAL ACCELERATION RESPONSE
SPECTRA PERPENDICULAR TO HOT LEG
STEAM GENERATOR BASE SUPPORT

Figure 3.7A-46

10079-11D-161
17AP6



10079-11D-162
17AP6

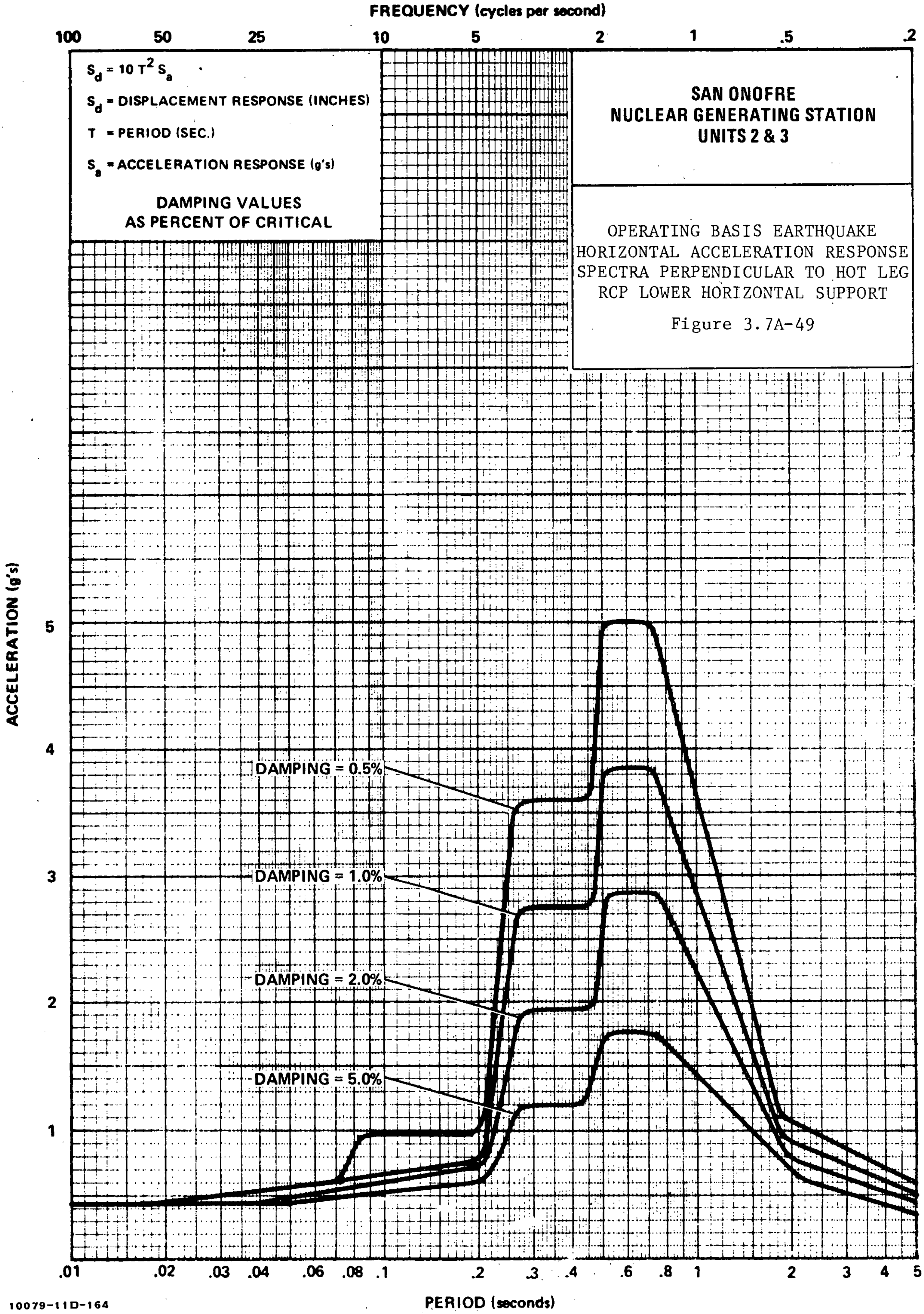


SAN ONOFRE
NUCLEAR GENERATING STATION
UNITS 2 & 3

OPERATING BASIS EARTHQUAKE
VERTICAL ACCELERATION RESPONSE
SPECTRA PERPENDICULAR TO HOT LEG
RCP BASE SUPPORT

Figure 3.7A-48

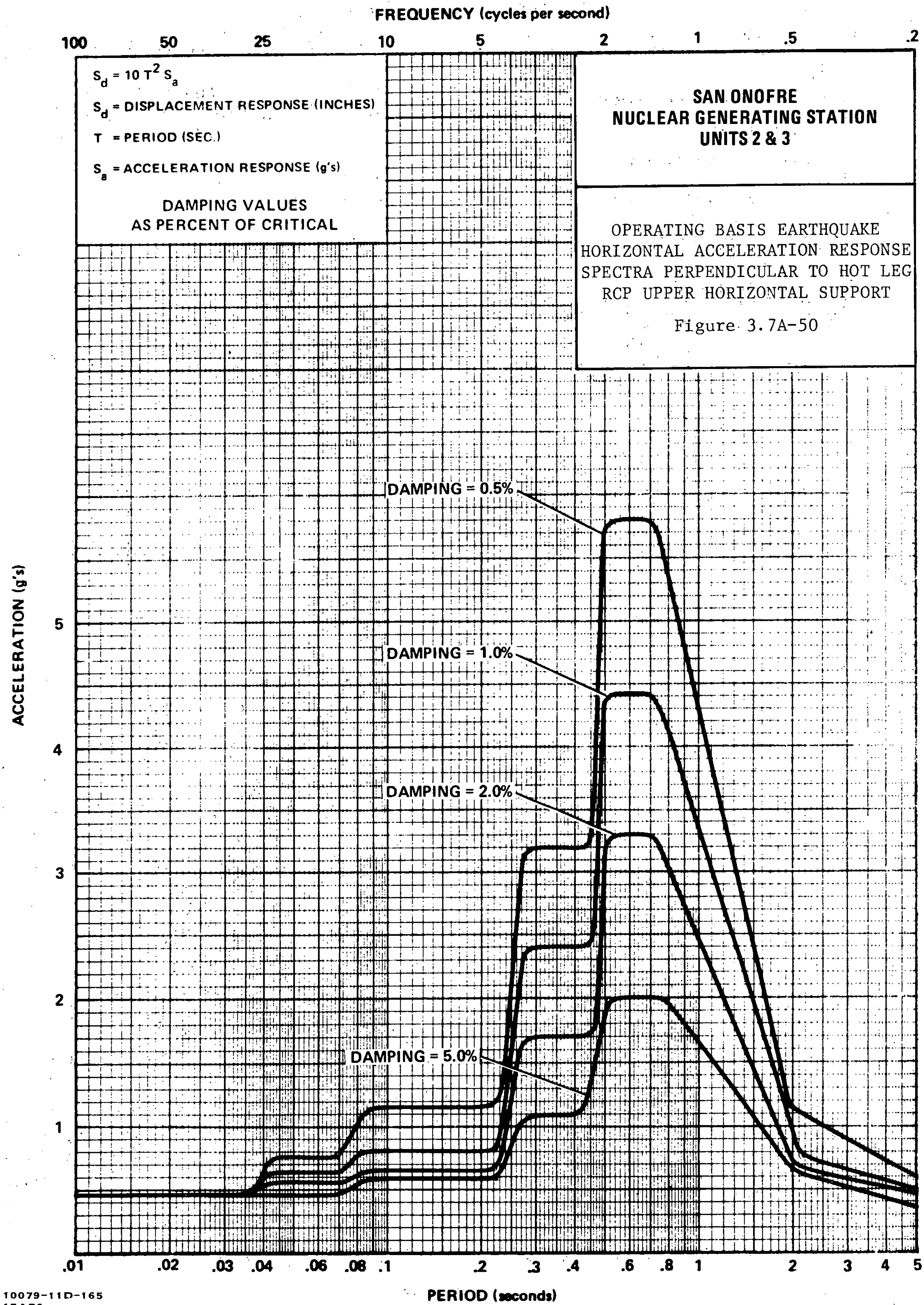
10079-11D-163



OPERATING BASIS EARTHQUAKE
HORIZONTAL ACCELERATION RESPONSE
SPECTRA PERPENDICULAR TO HOT LEG
RCP LOWER HORIZONTAL SUPPORT

Figure 3.7A-49

10079-11D-164
17AP6

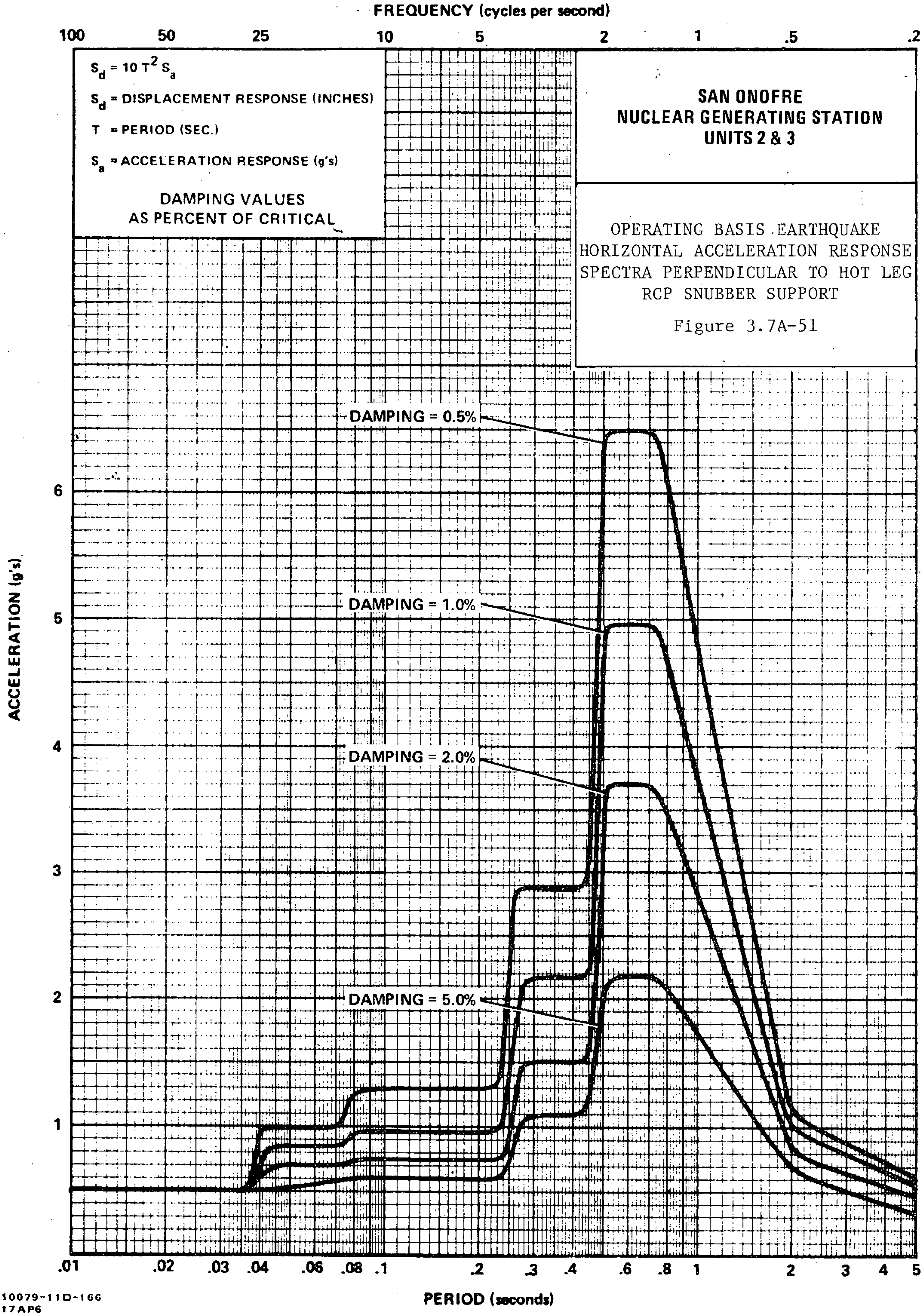


SAN ONOFRE
NUCLEAR GENERATING STATION
UNITS 2 & 3

OPERATING BASIS EARTHQUAKE
HORIZONTAL ACCELERATION RESPONSE
SPECTRA PERPENDICULAR TO HOT LEG
RCP UPPER HORIZONTAL SUPPORT

Figure 3.7A-50

10079-11D-165
17AP6

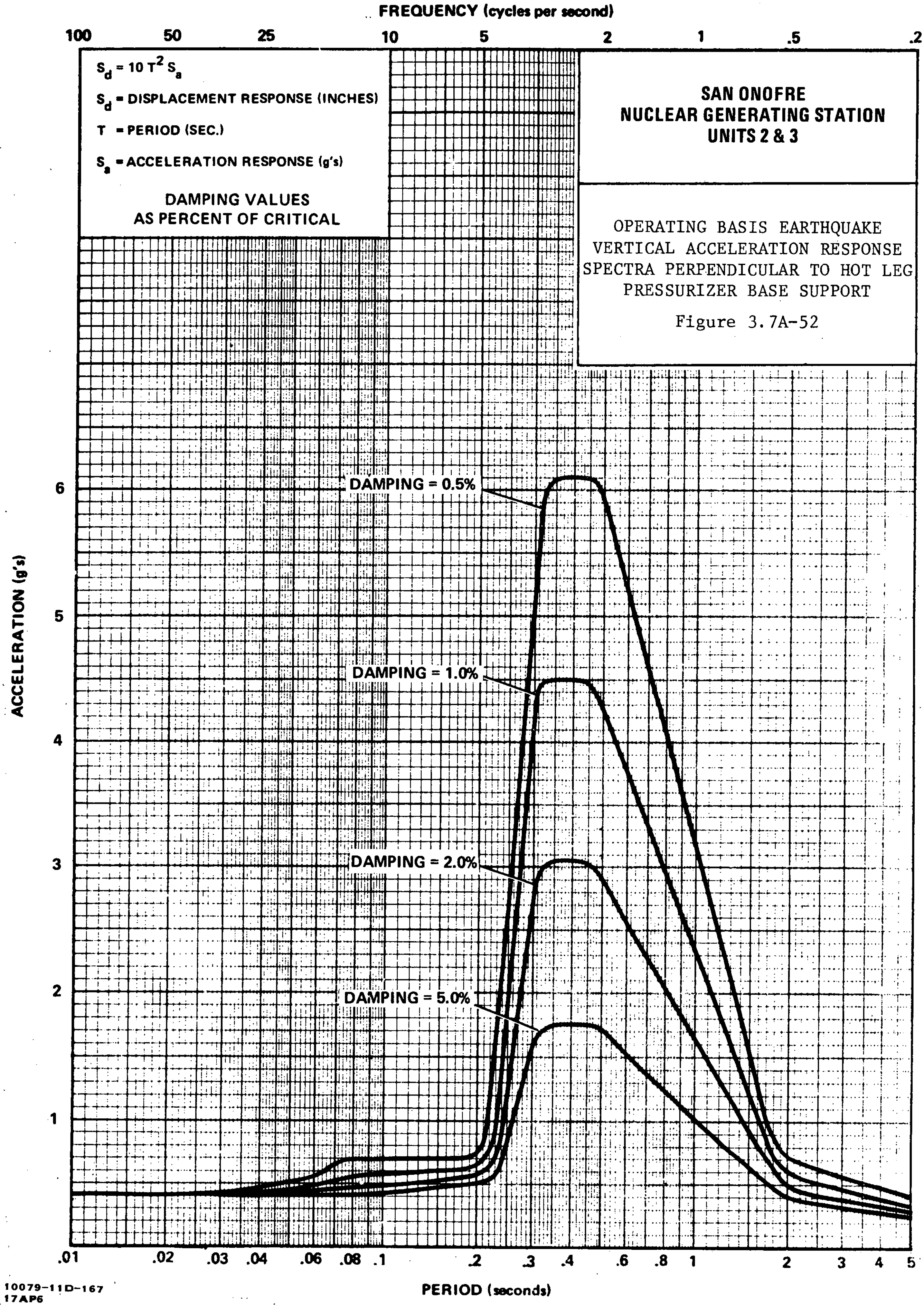


SAN ONOFRE NUCLEAR GENERATING STATION UNITS 2 & 3

OPERATING BASIS EARTHQUAKE HORIZONTAL ACCELERATION RESPONSE SPECTRA PERPENDICULAR TO HOT LEG RCP SNUBBER SUPPORT

Figure 3.7A-51

10079-11D-166
17AP6

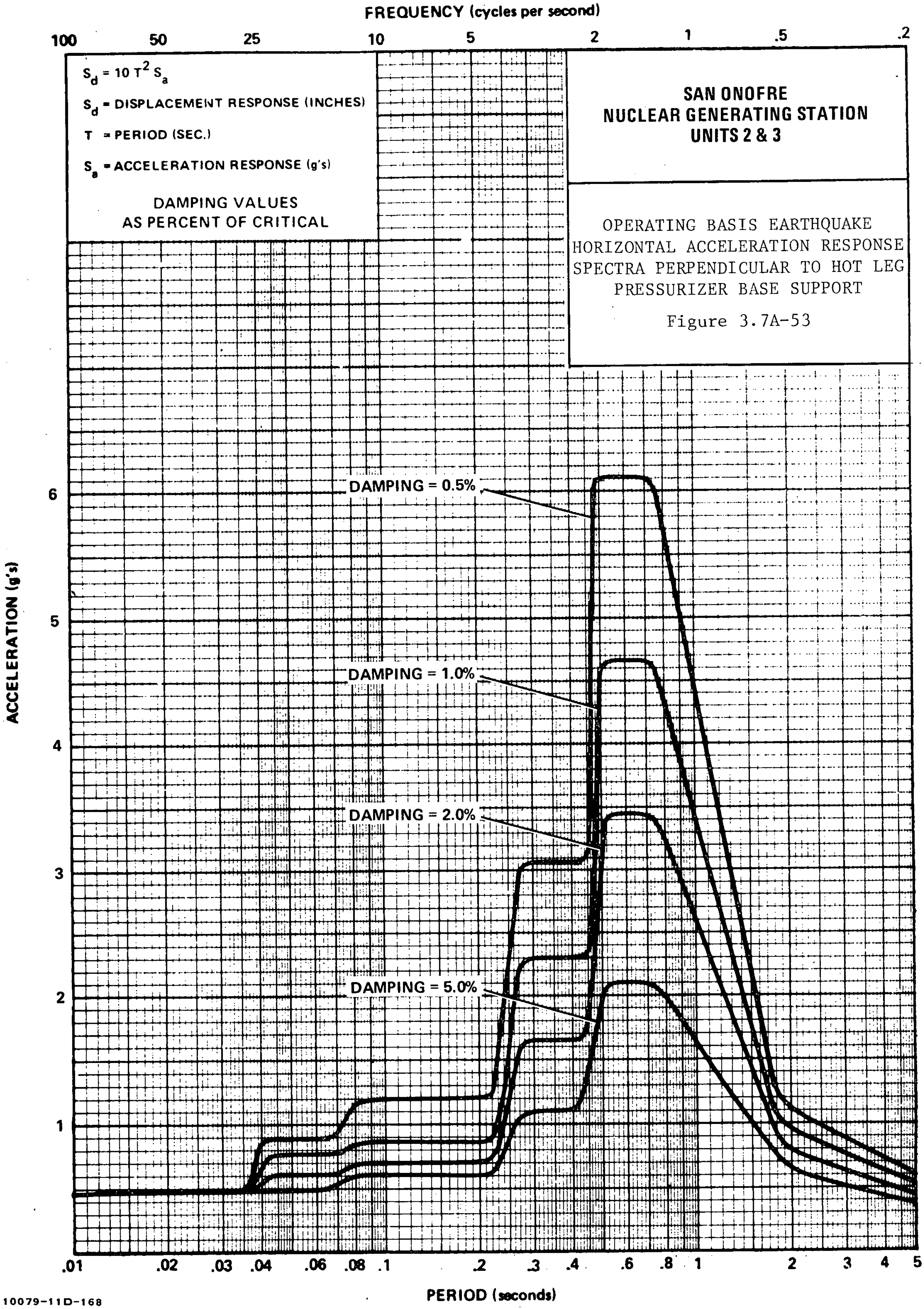


SAN ONOFRE
NUCLEAR GENERATING STATION
UNITS 2 & 3

OPERATING BASIS EARTHQUAKE
VERTICAL ACCELERATION RESPONSE
SPECTRA PERPENDICULAR TO HOT LEG
PRESSURIZER BASE SUPPORT

Figure 3.7A-52

10079-11D-167
17AP6

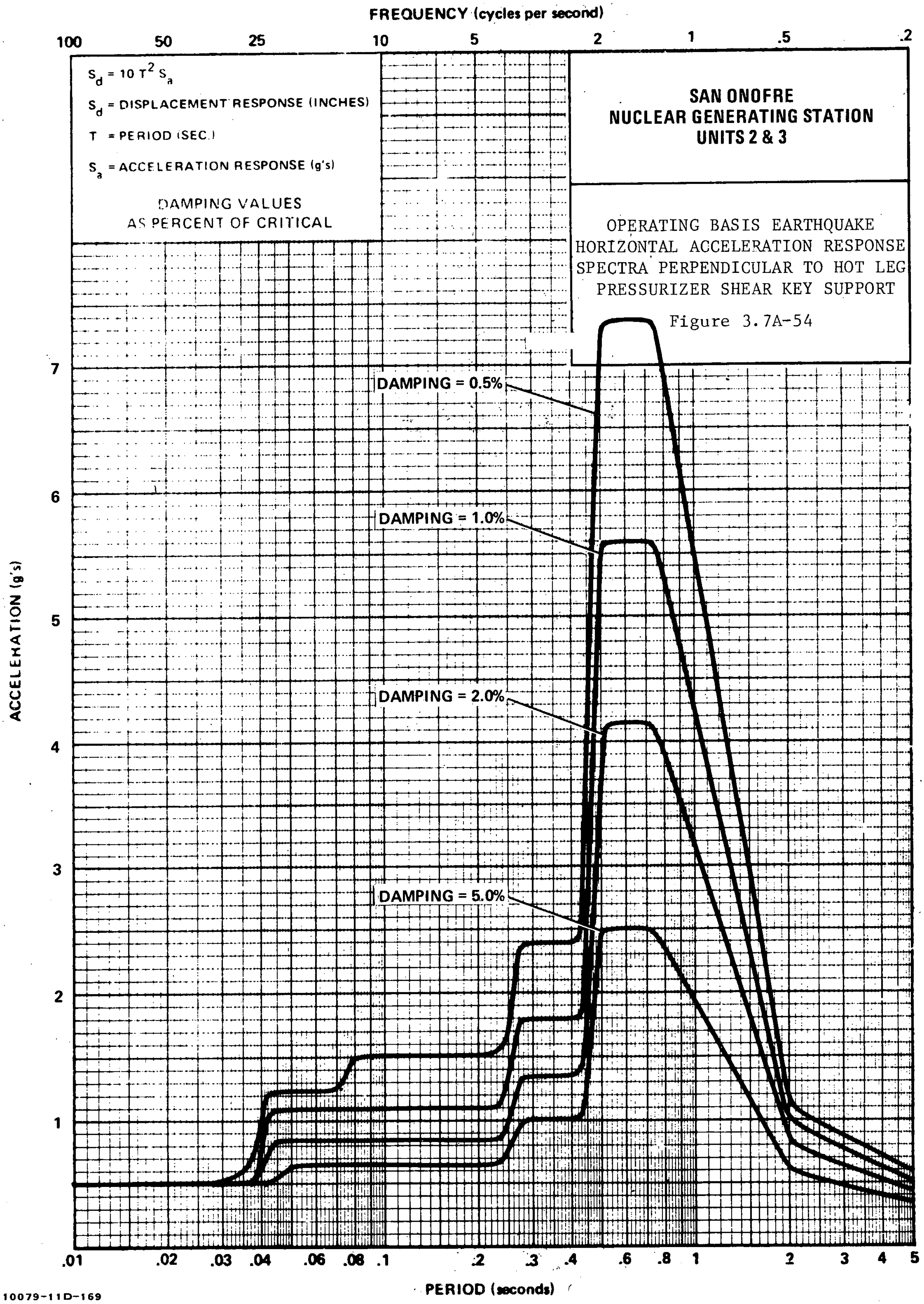


SAN ONOFRE
NUCLEAR GENERATING STATION
UNITS 2 & 3

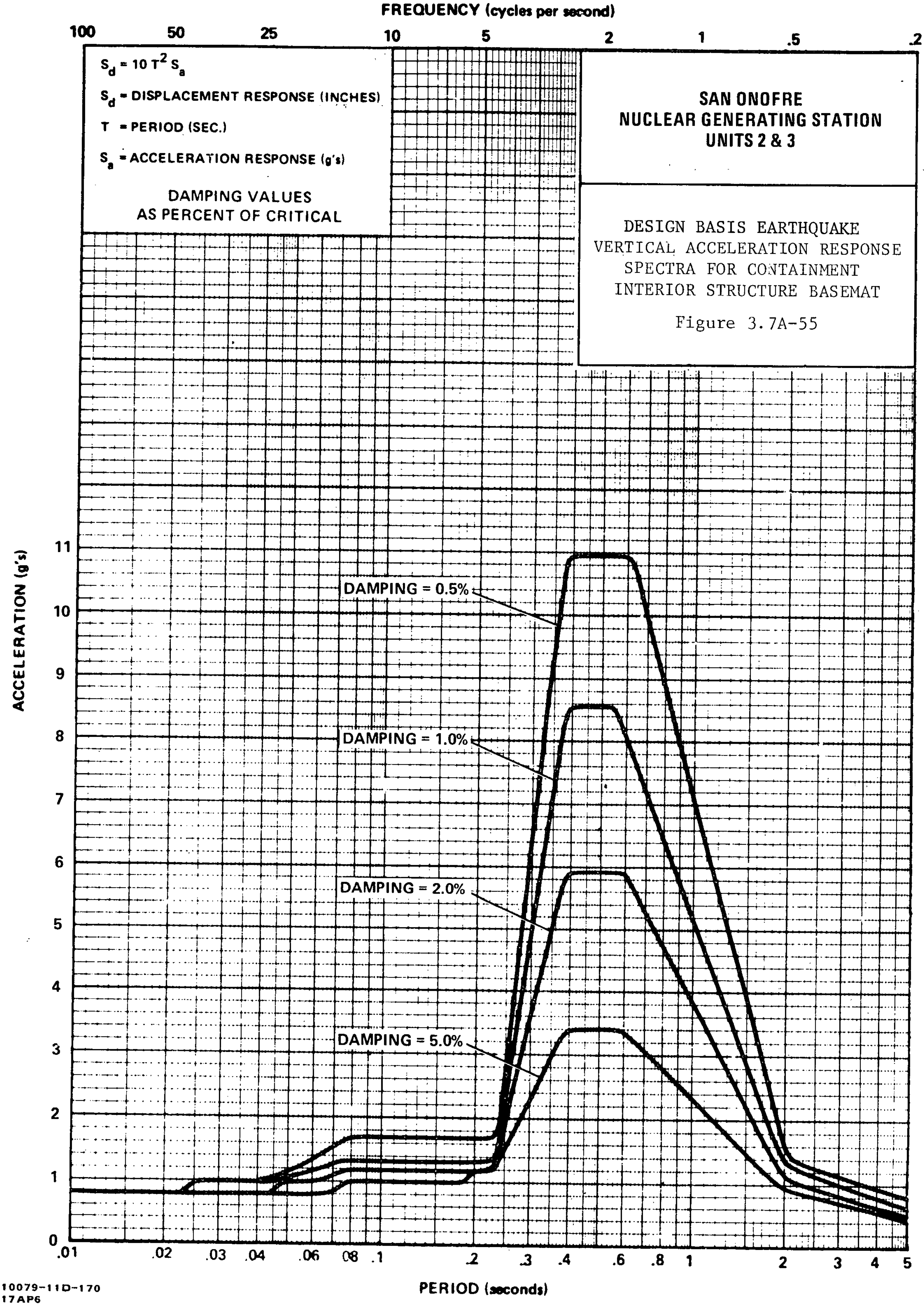
OPERATING BASIS EARTHQUAKE
HORIZONTAL ACCELERATION RESPONSE
SPECTRA PERPENDICULAR TO HOT LEG
PRESSURIZER BASE SUPPORT

Figure 3.7A-53

10079-11D-168
17AP6



10079-11D-169
17AP6

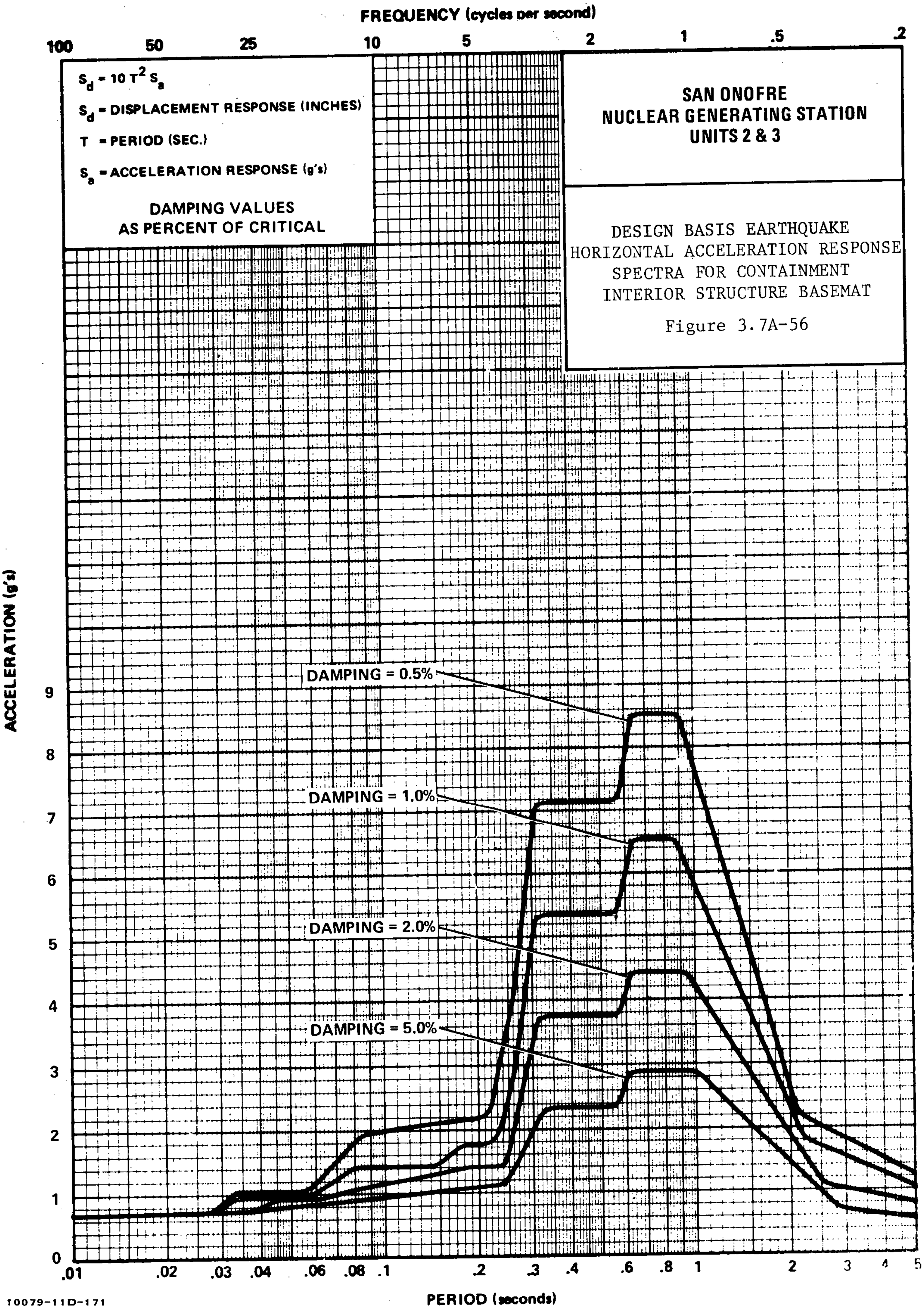


SAN ONOFRE
NUCLEAR GENERATING STATION
UNITS 2 & 3

DESIGN BASIS EARTHQUAKE
VERTICAL ACCELERATION RESPONSE
SPECTRA FOR CONTAINMENT
INTERIOR STRUCTURE BASEMAT

Figure 3.7A-55

10079-11D-170
17AP6

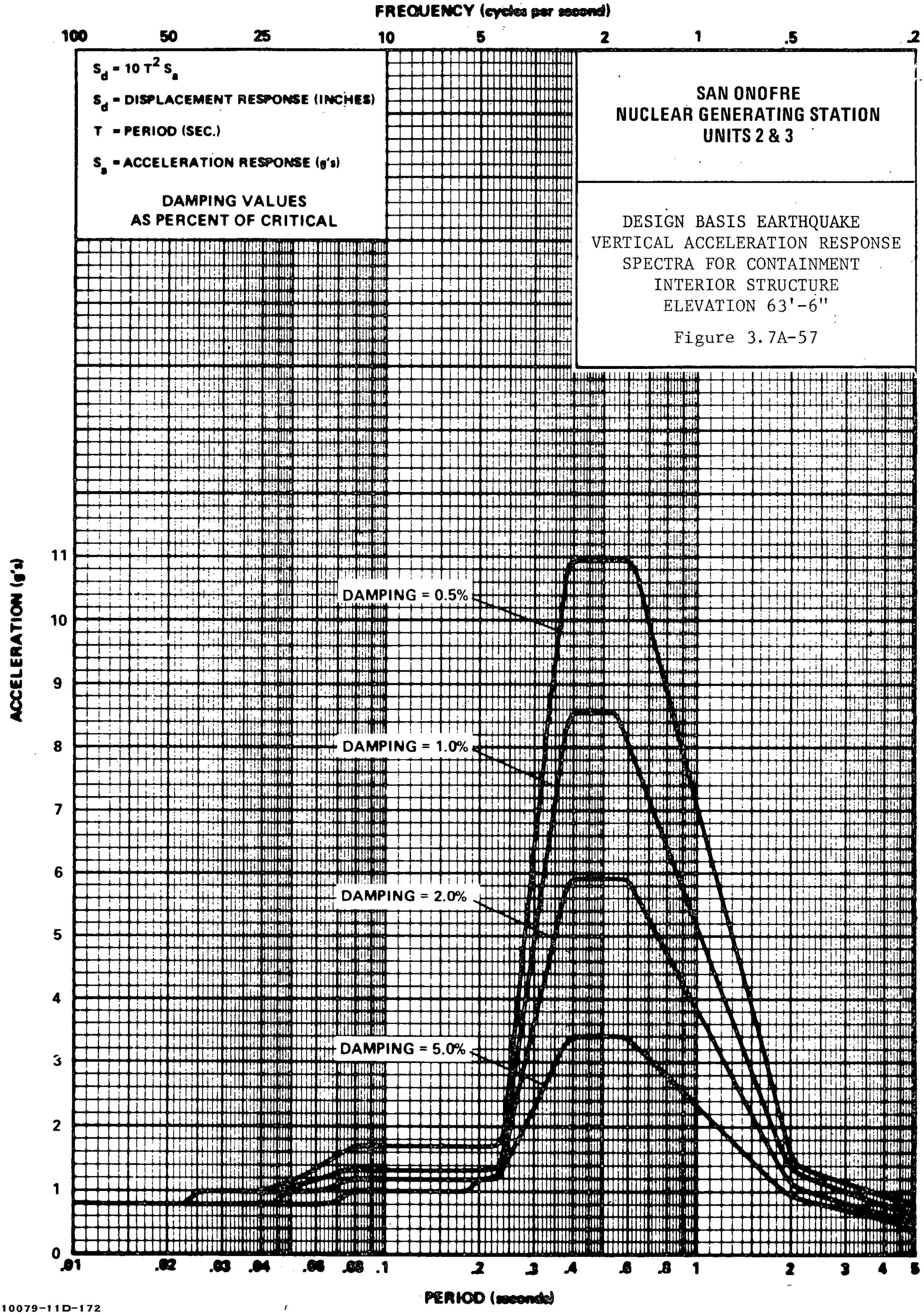


SAN ONOFRE
NUCLEAR GENERATING STATION
UNITS 2 & 3

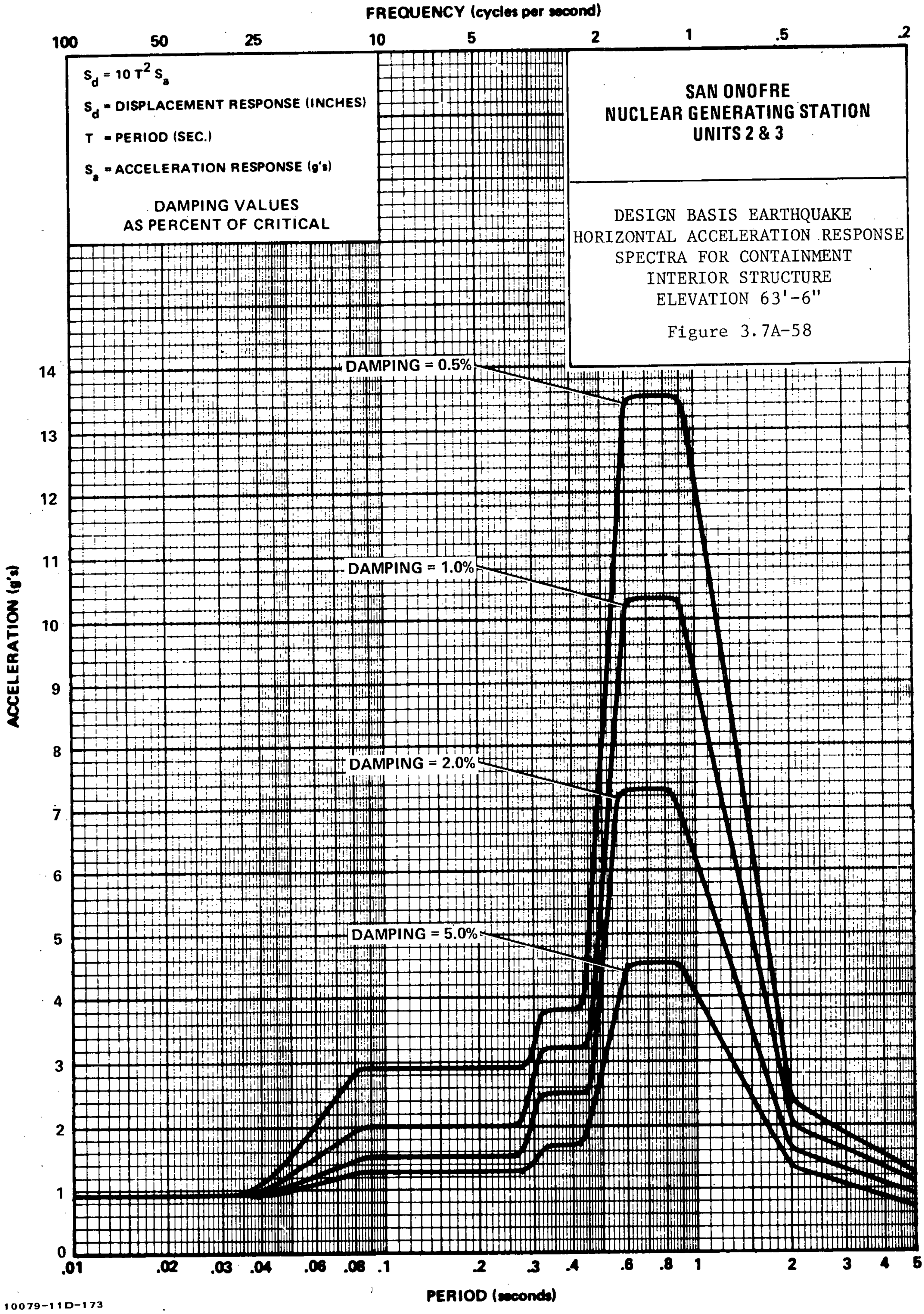
DESIGN BASIS EARTHQUAKE
HORIZONTAL ACCELERATION RESPONSE
SPECTRA FOR CONTAINMENT
INTERIOR STRUCTURE BASEMAT

Figure 3.7A-56

10079-11D-171
17AP6



10079-11D-172
19AP6

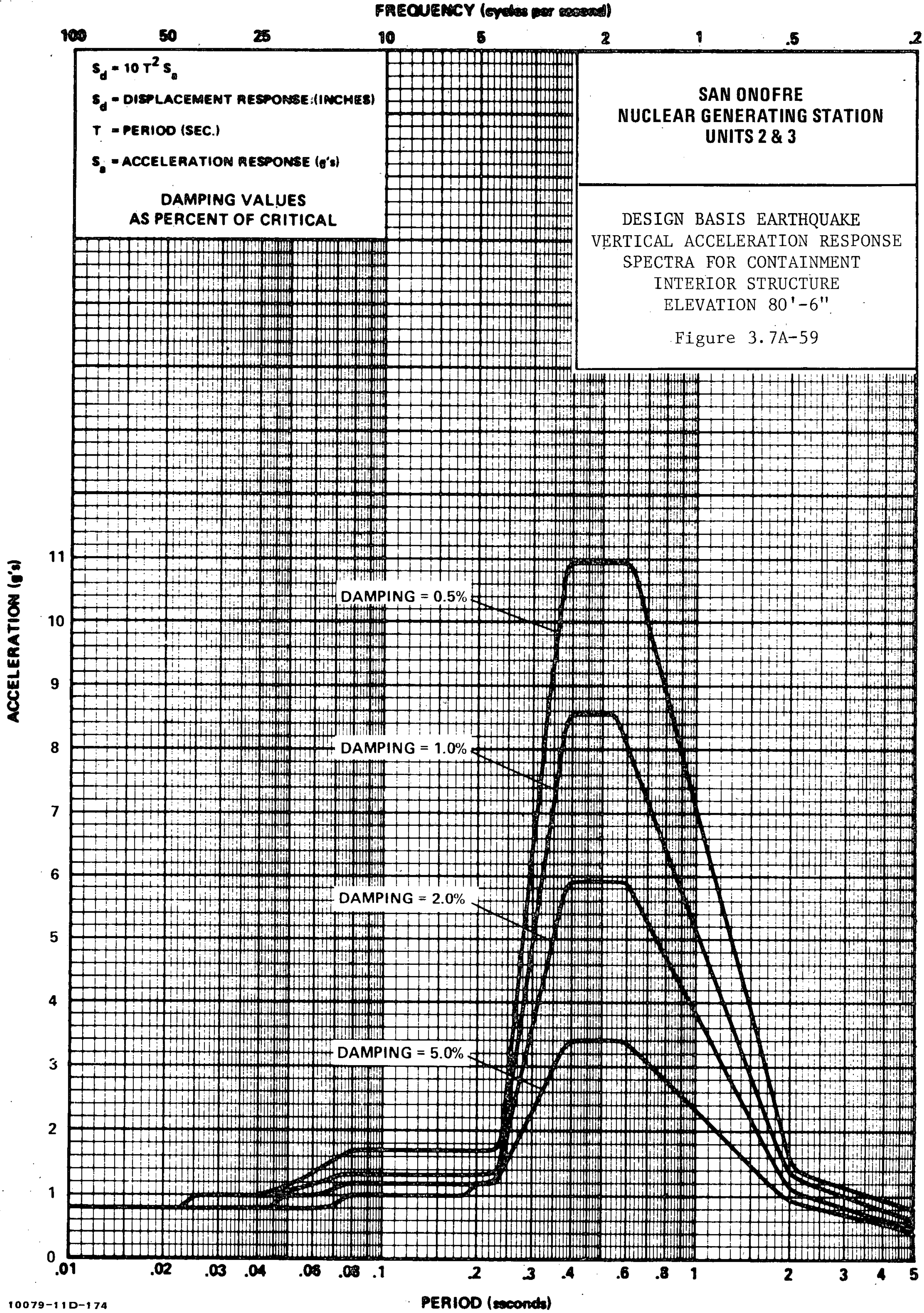


SAN ONOFRE
NUCLEAR GENERATING STATION
UNITS 2 & 3

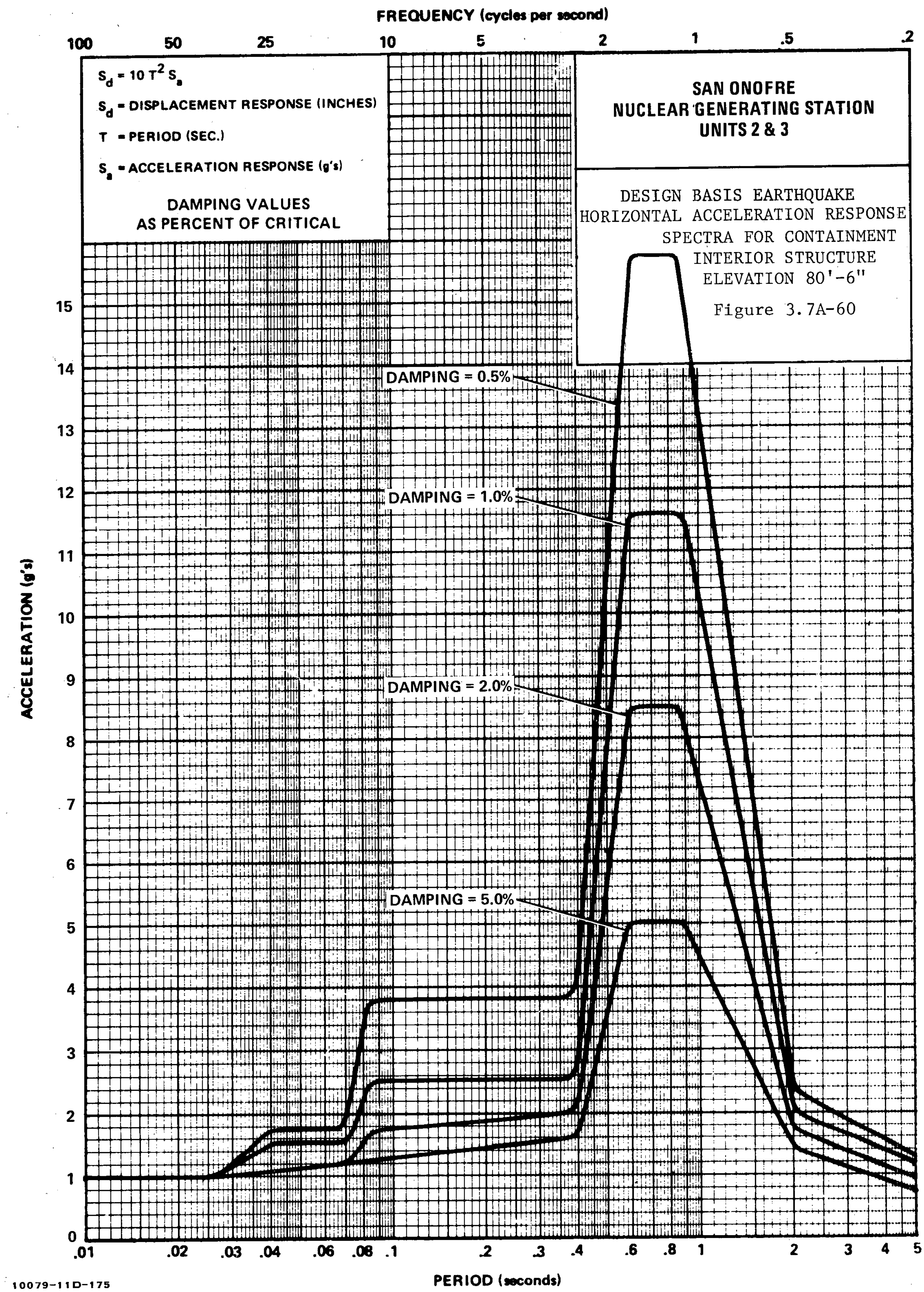
DESIGN BASIS EARTHQUAKE
HORIZONTAL ACCELERATION RESPONSE
SPECTRA FOR CONTAINMENT
INTERIOR STRUCTURE
ELEVATION 63'-6"

Figure 3.7A-58

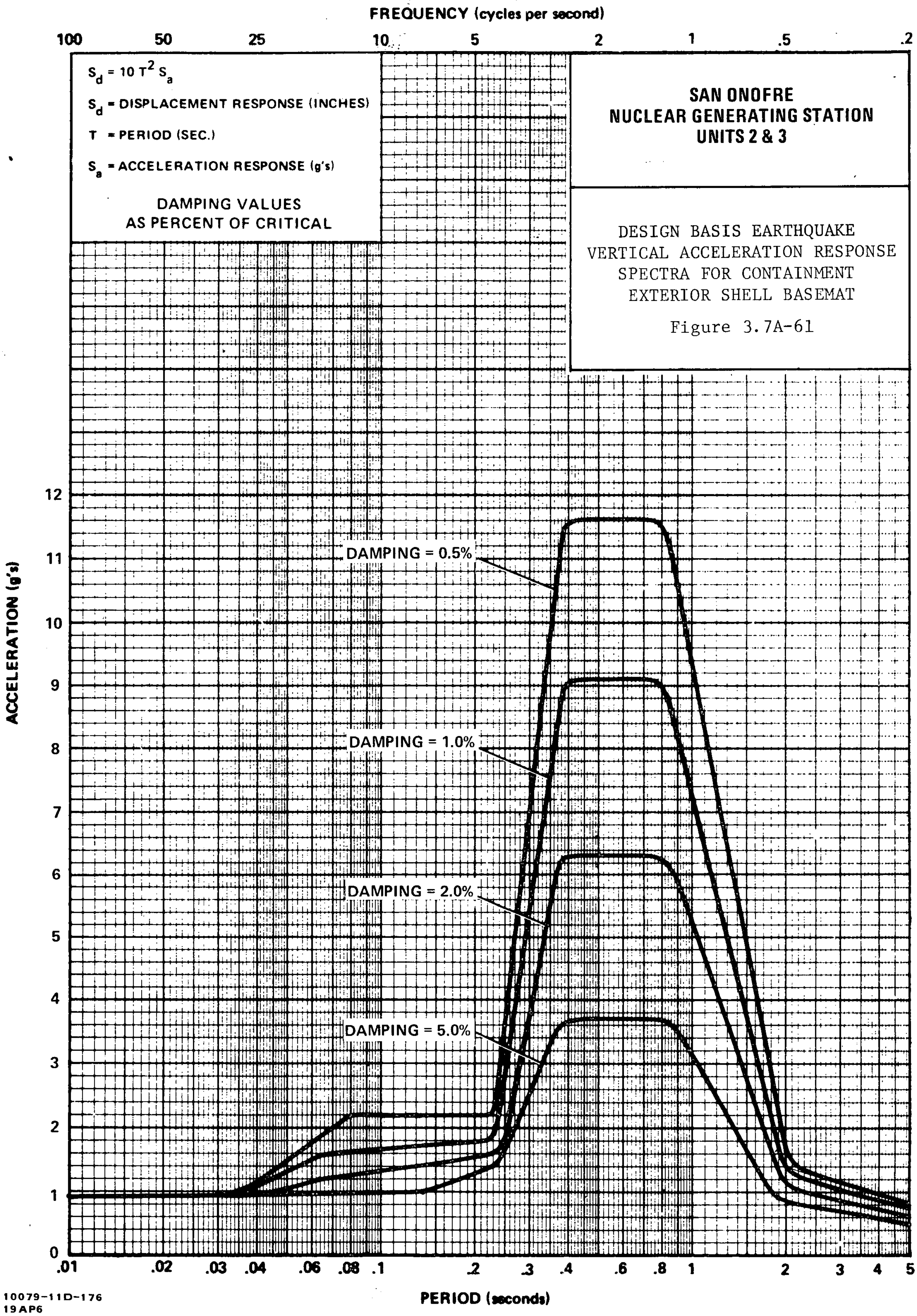
10079-11D-173
19AP6



10079-11D-174
19AP6



10079-11D-175
19AP6

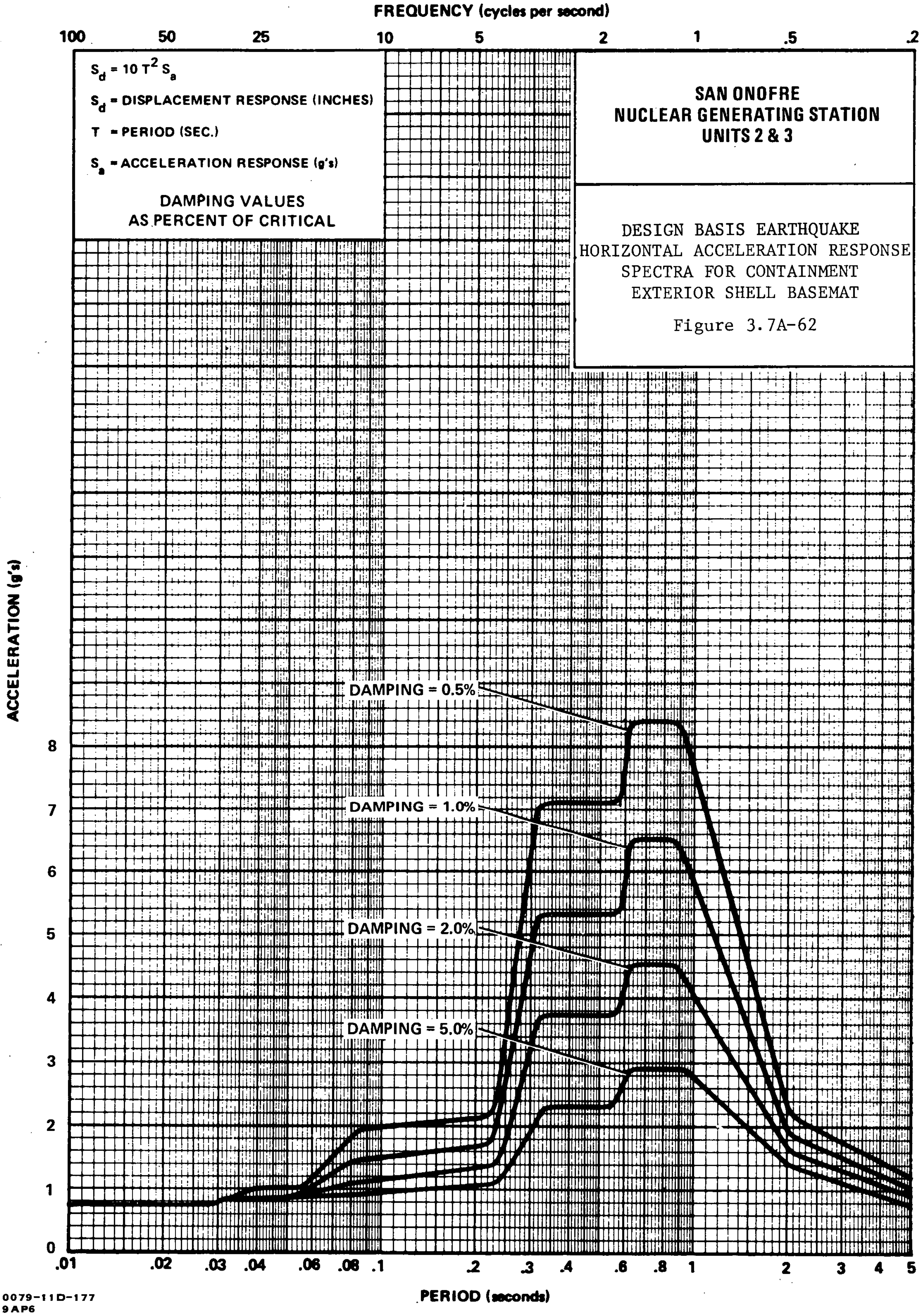


SAN ONOFRE
NUCLEAR GENERATING STATION
UNITS 2 & 3

DESIGN BASIS EARTHQUAKE
VERTICAL ACCELERATION RESPONSE
SPECTRA FOR CONTAINMENT
EXTERIOR SHELL BASEMAT

Figure 3.7A-61

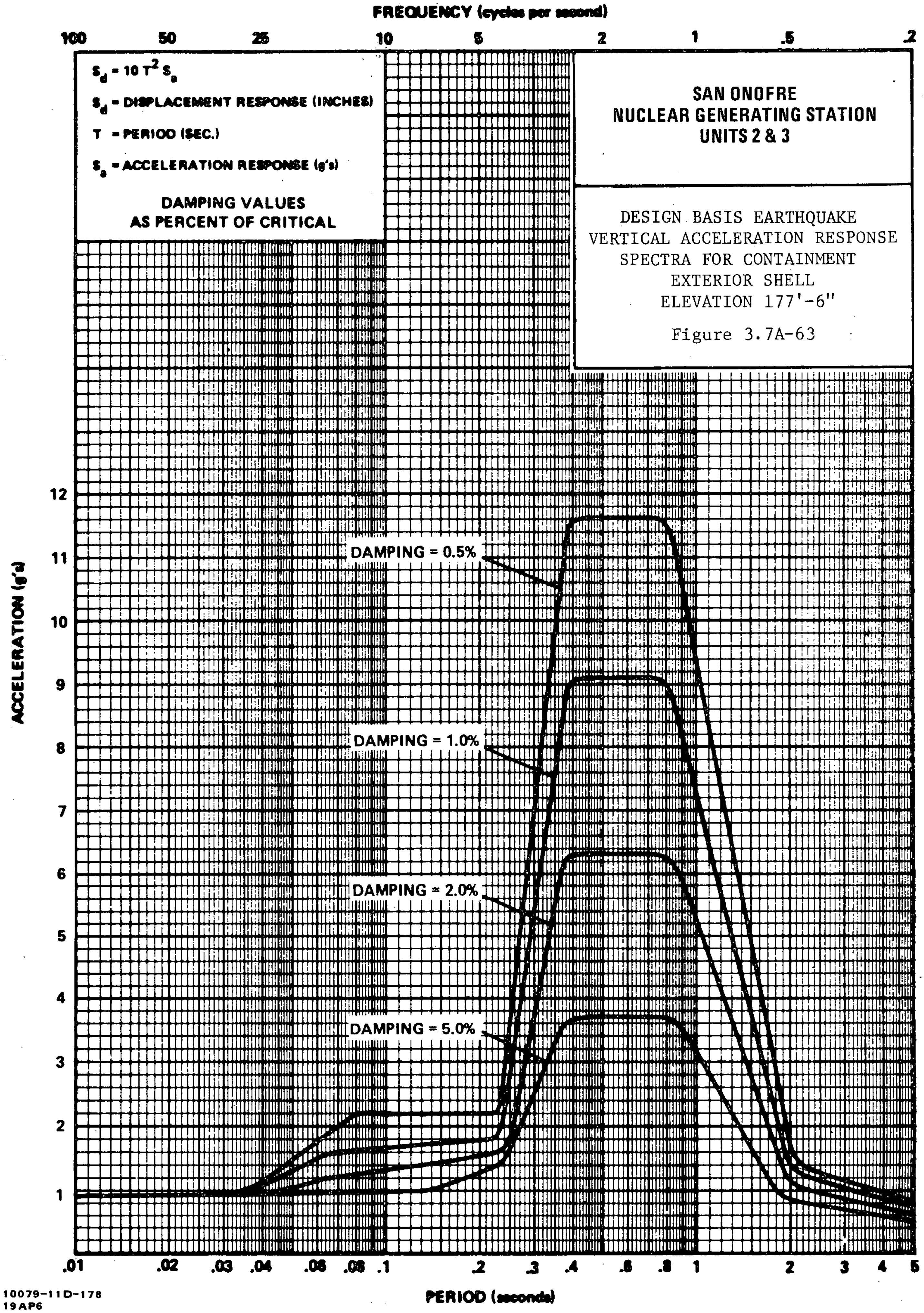
10079-11D-176
19AP6



SAN ONOFRE
NUCLEAR GENERATING STATION
UNITS 2 & 3

DESIGN BASIS EARTHQUAKE
HORIZONTAL ACCELERATION RESPONSE
SPECTRA FOR CONTAINMENT
EXTERIOR SHELL BASEMAT

Figure 3.7A-62

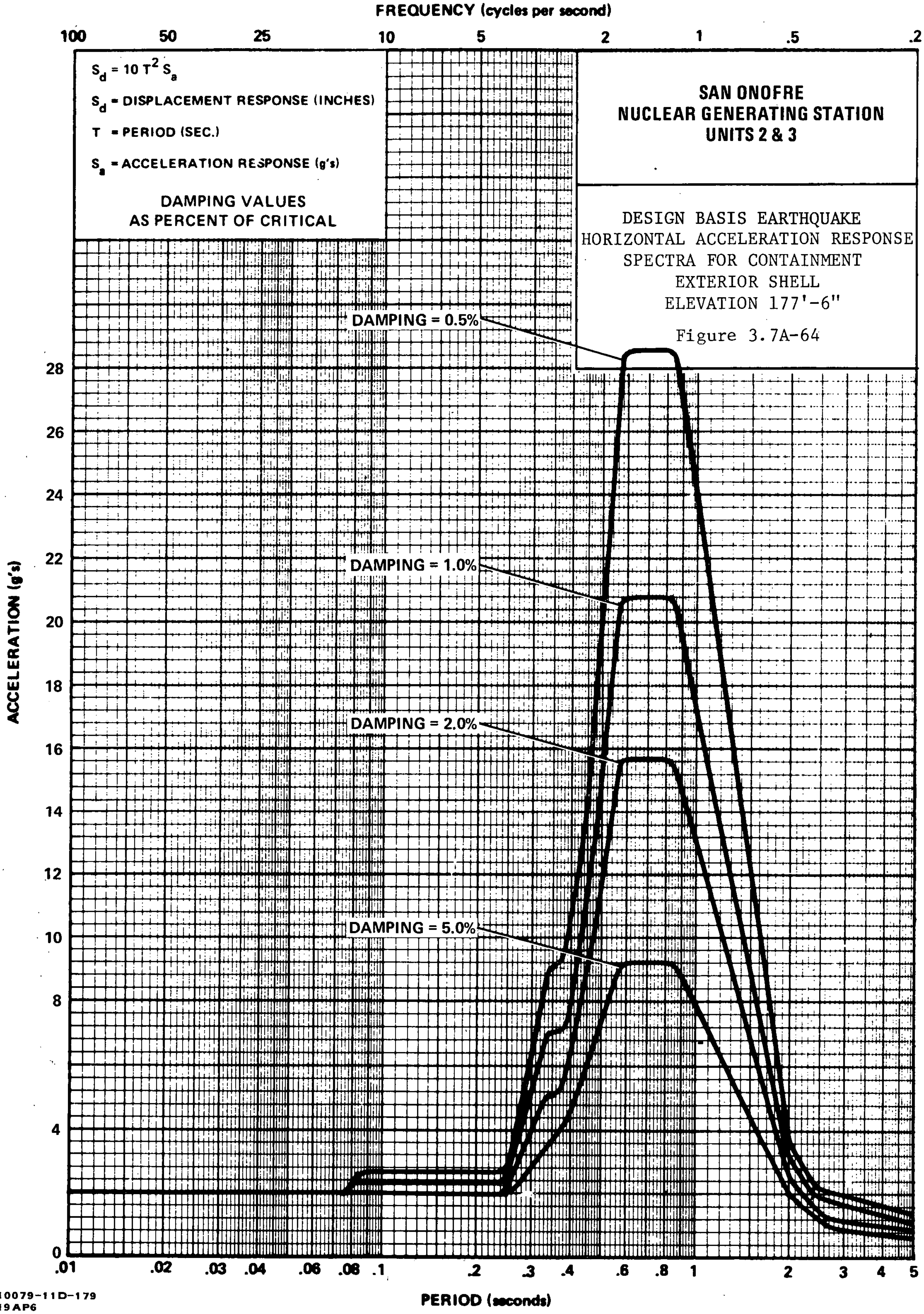


SAN ONOFRE
NUCLEAR GENERATING STATION
UNITS 2 & 3

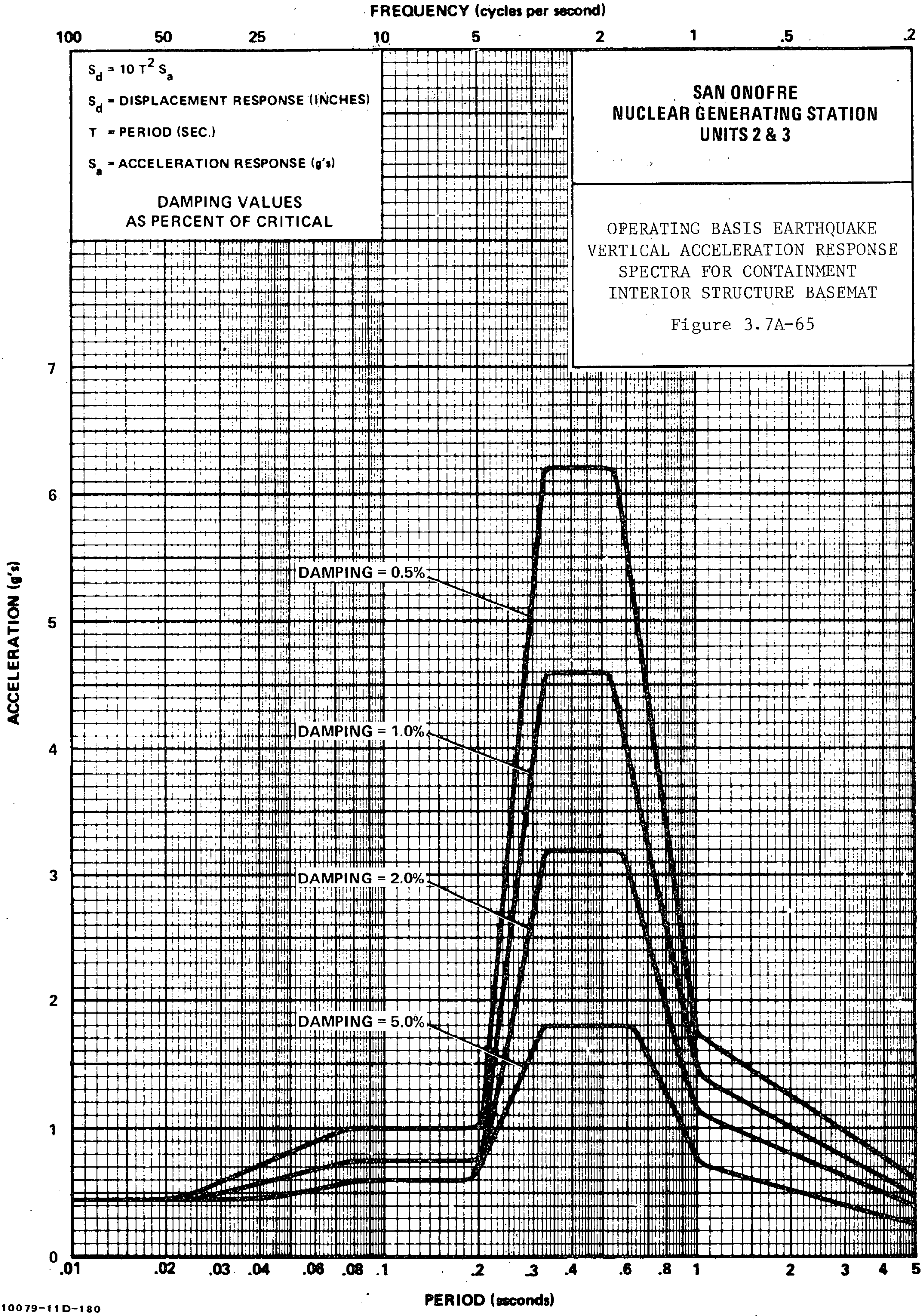
DESIGN BASIS EARTHQUAKE
VERTICAL ACCELERATION RESPONSE
SPECTRA FOR CONTAINMENT
EXTERIOR SHELL
ELEVATION 177'-6"

Figure 3.7A-63

10079-11D-178
19AP6



10079-11D-179
19AP6

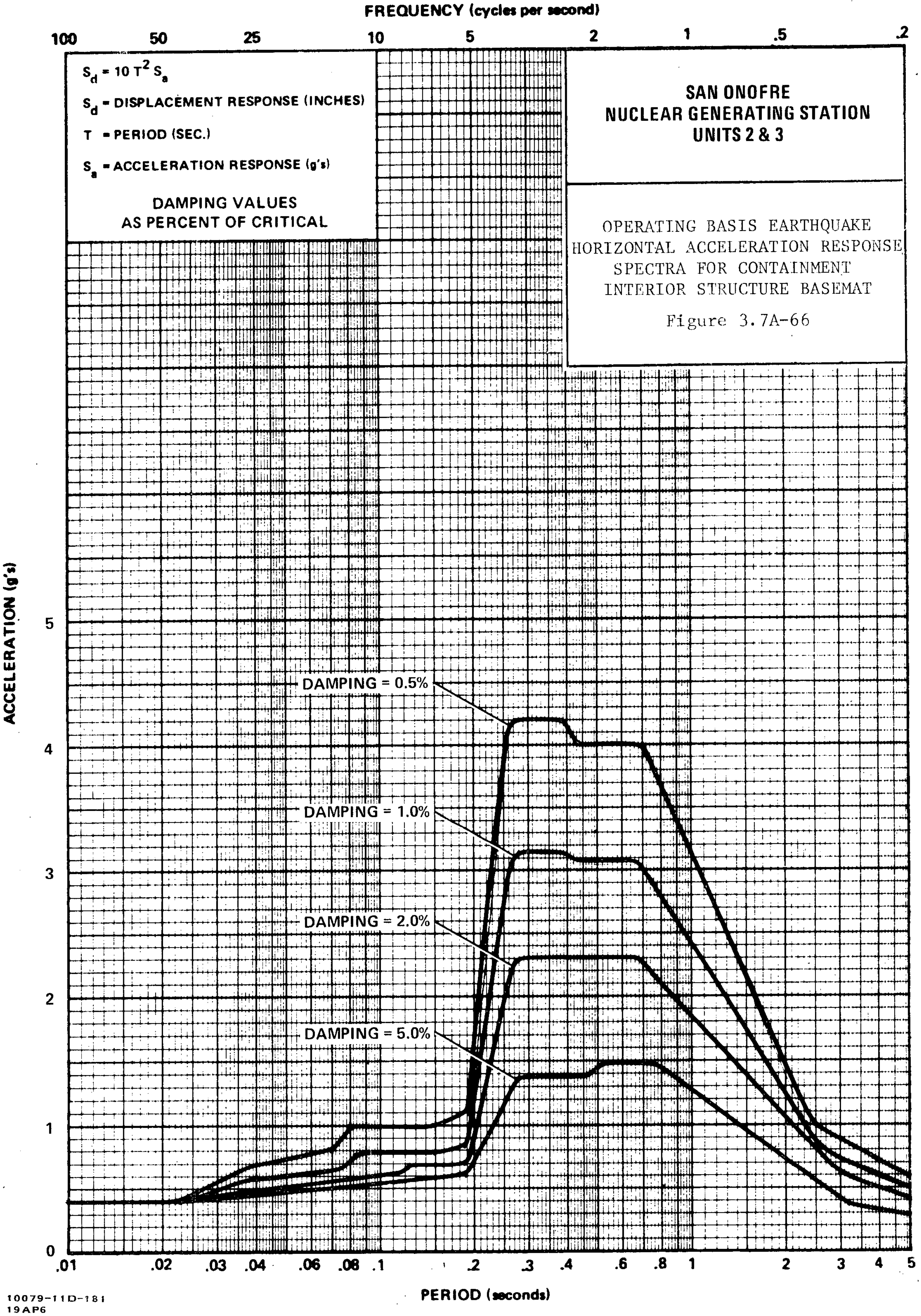


SAN ONOFRE
NUCLEAR GENERATING STATION
UNITS 2 & 3

OPERATING BASIS EARTHQUAKE
VERTICAL ACCELERATION RESPONSE
SPECTRA FOR CONTAINMENT
INTERIOR STRUCTURE BASEMAT

Figure 3.7A-65

10079-11D-180
19AP6

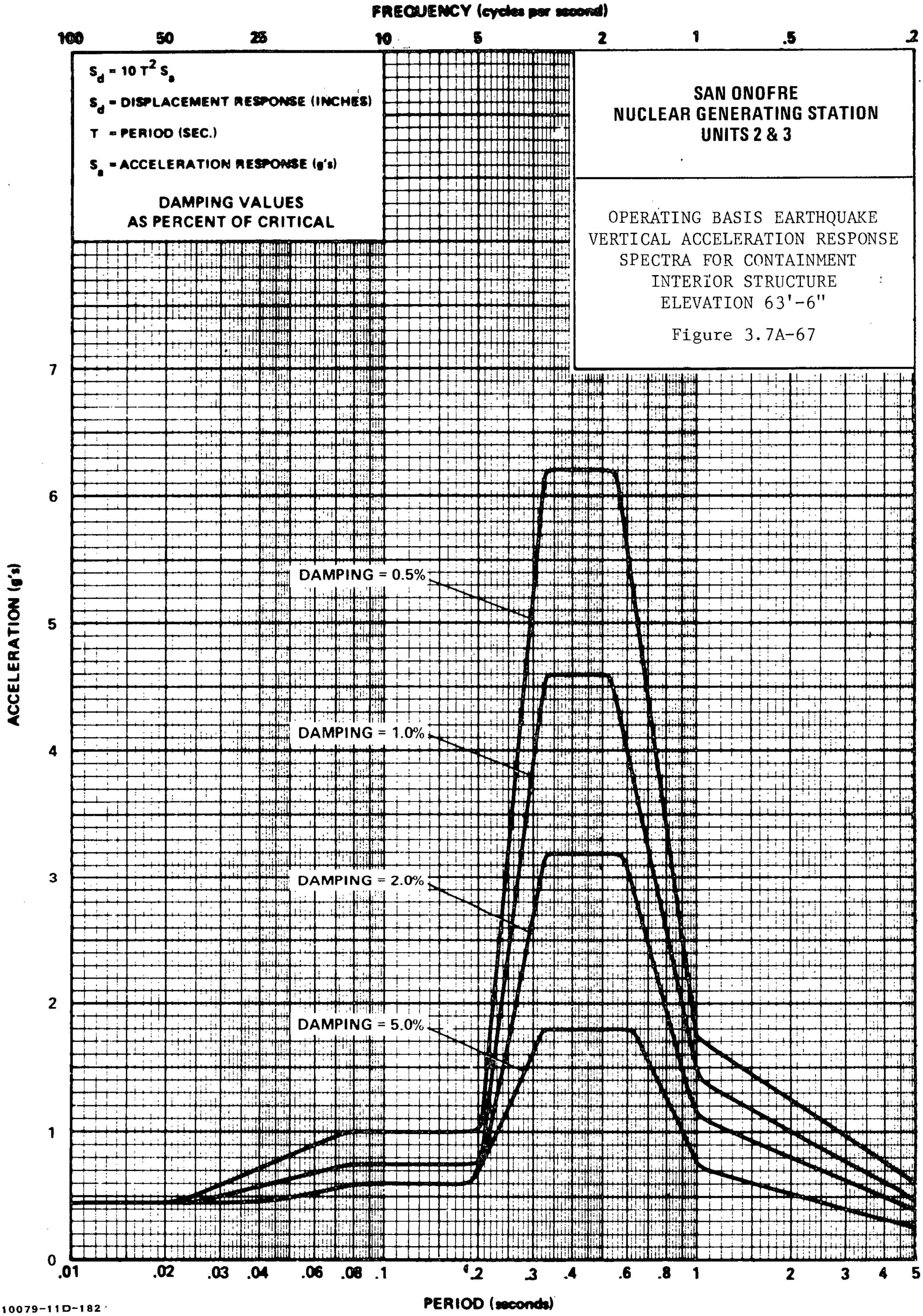


SAN ONOFRE
NUCLEAR GENERATING STATION
UNITS 2 & 3

OPERATING BASIS EARTHQUAKE
HORIZONTAL ACCELERATION RESPONSE
SPECTRA FOR CONTAINMENT
INTERIOR STRUCTURE BASEMAT

Figure 3.7A-66

10079-11D-181
19AP6

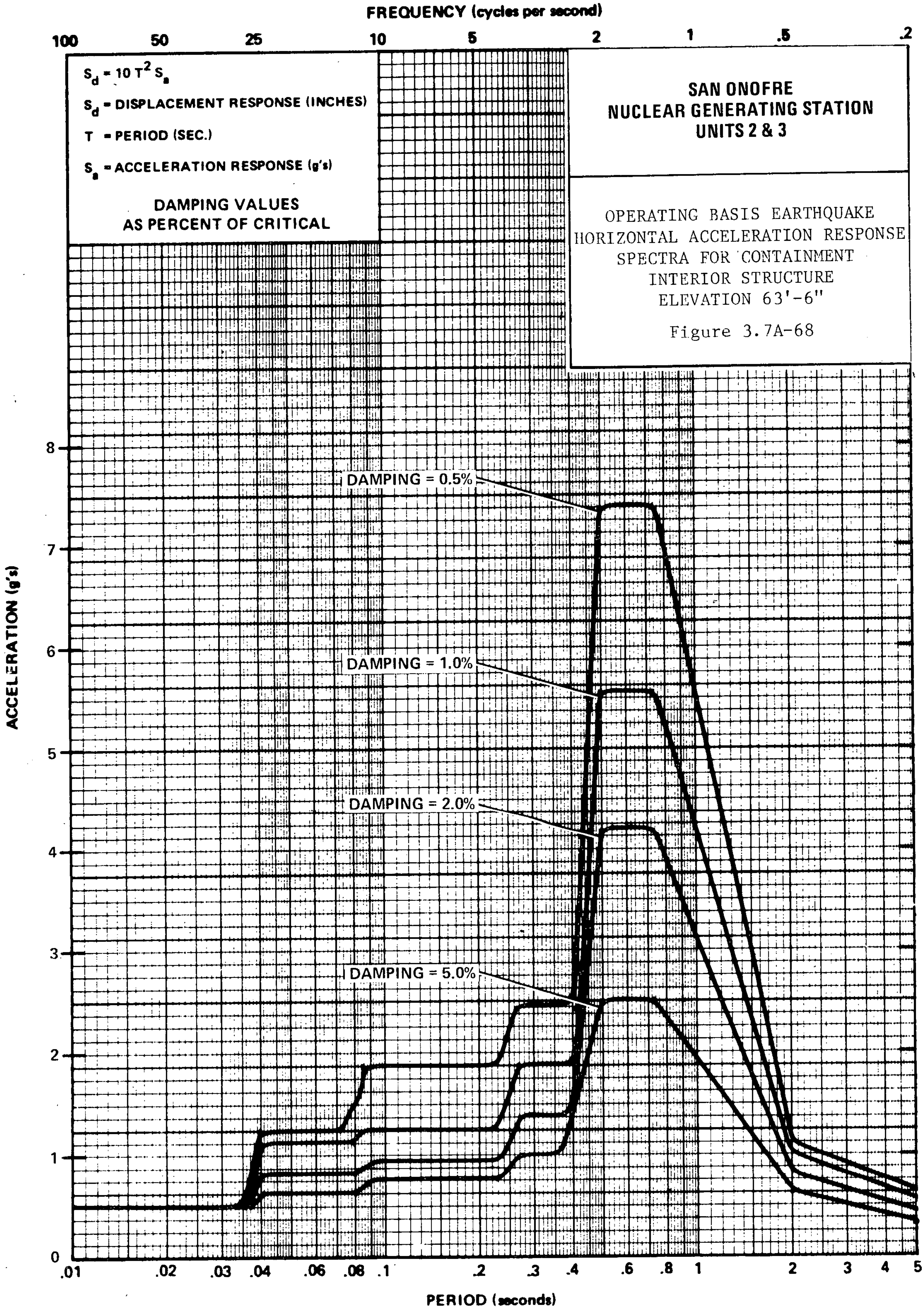


SAN ONOFRE
NUCLEAR GENERATING STATION
UNITS 2 & 3

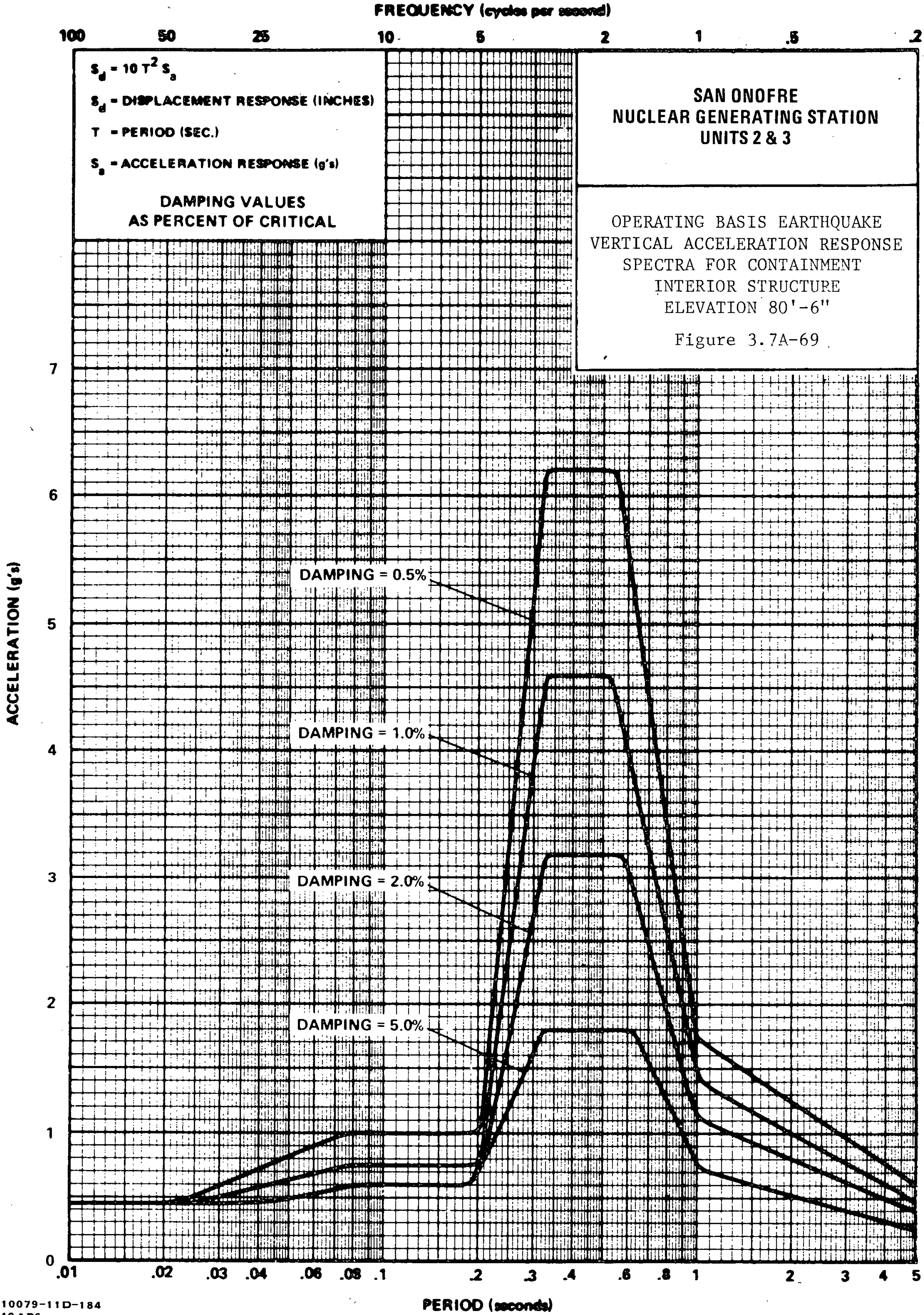
OPERATING BASIS EARTHQUAKE
VERTICAL ACCELERATION RESPONSE
SPECTRA FOR CONTAINMENT
INTERIOR STRUCTURE
ELEVATION 63'-6"

Figure 3.7A-67

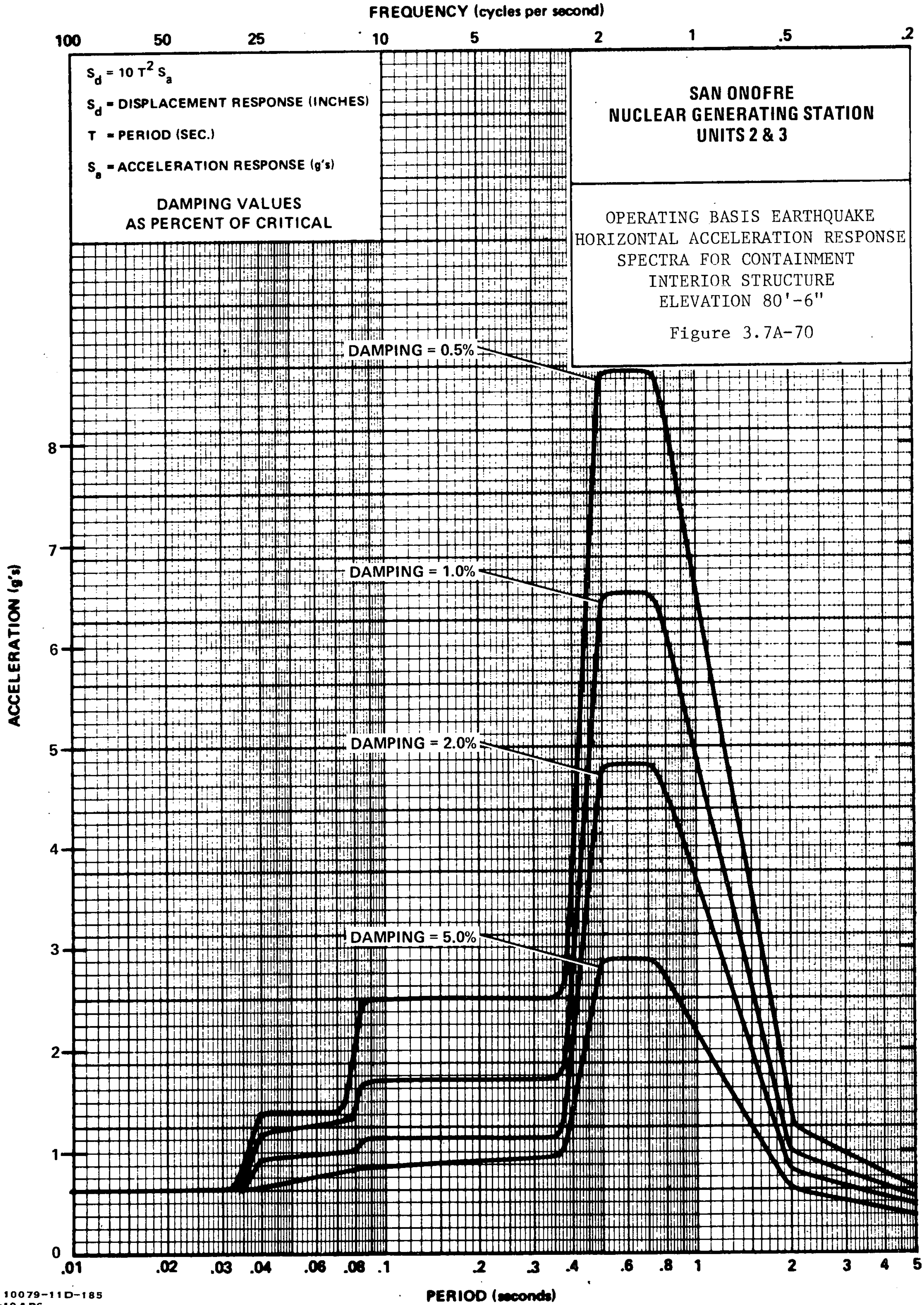
10079-11D-182
19AP6



10079-11D-183
19AP6



10079-11D-184
19AP6

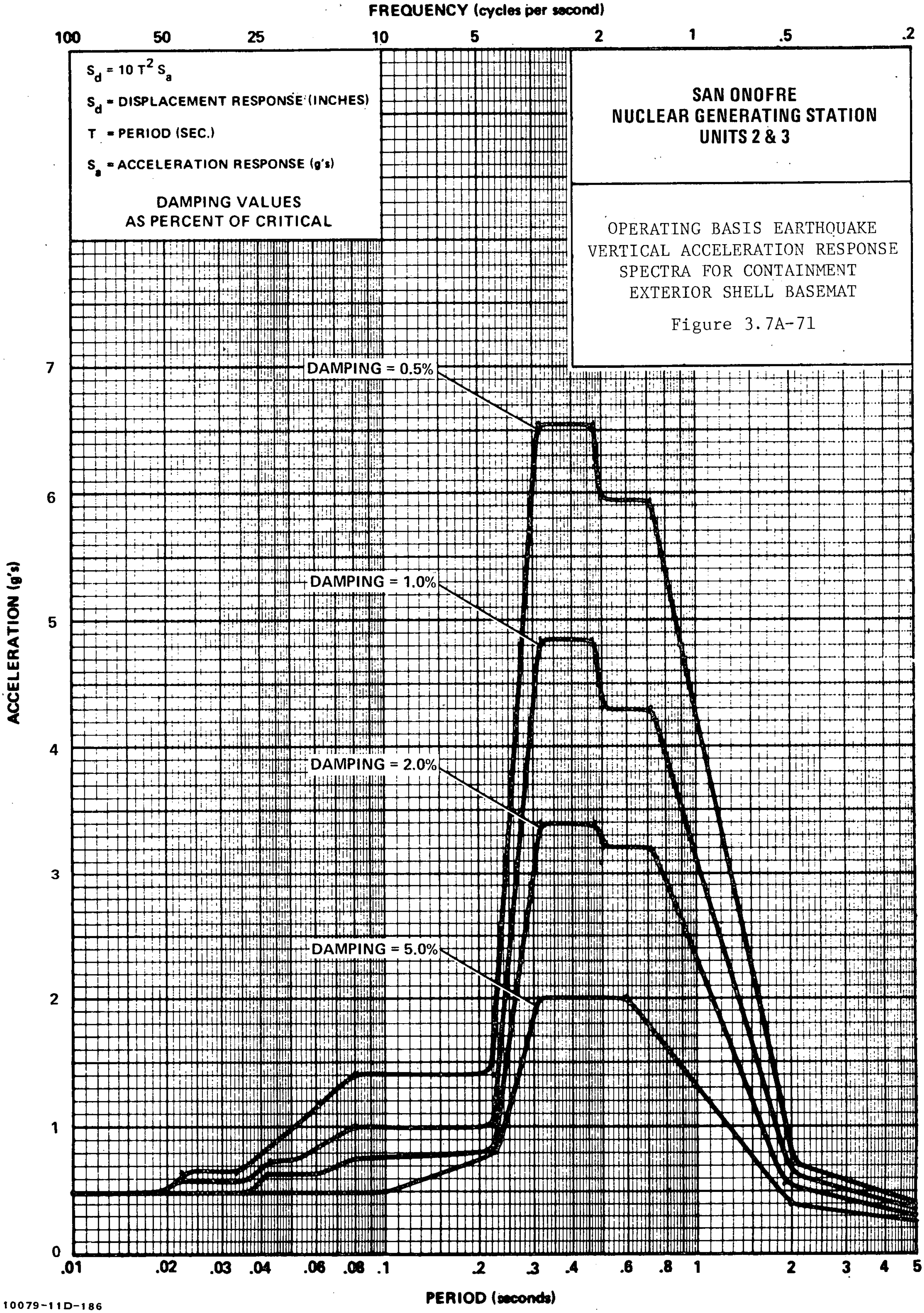


SAN ONOFRE
NUCLEAR GENERATING STATION
UNITS 2 & 3

OPERATING BASIS EARTHQUAKE
HORIZONTAL ACCELERATION RESPONSE
SPECTRA FOR CONTAINMENT
INTERIOR STRUCTURE
ELEVATION 80'-6"

Figure 3.7A-70

10079-11D-185
19AP6

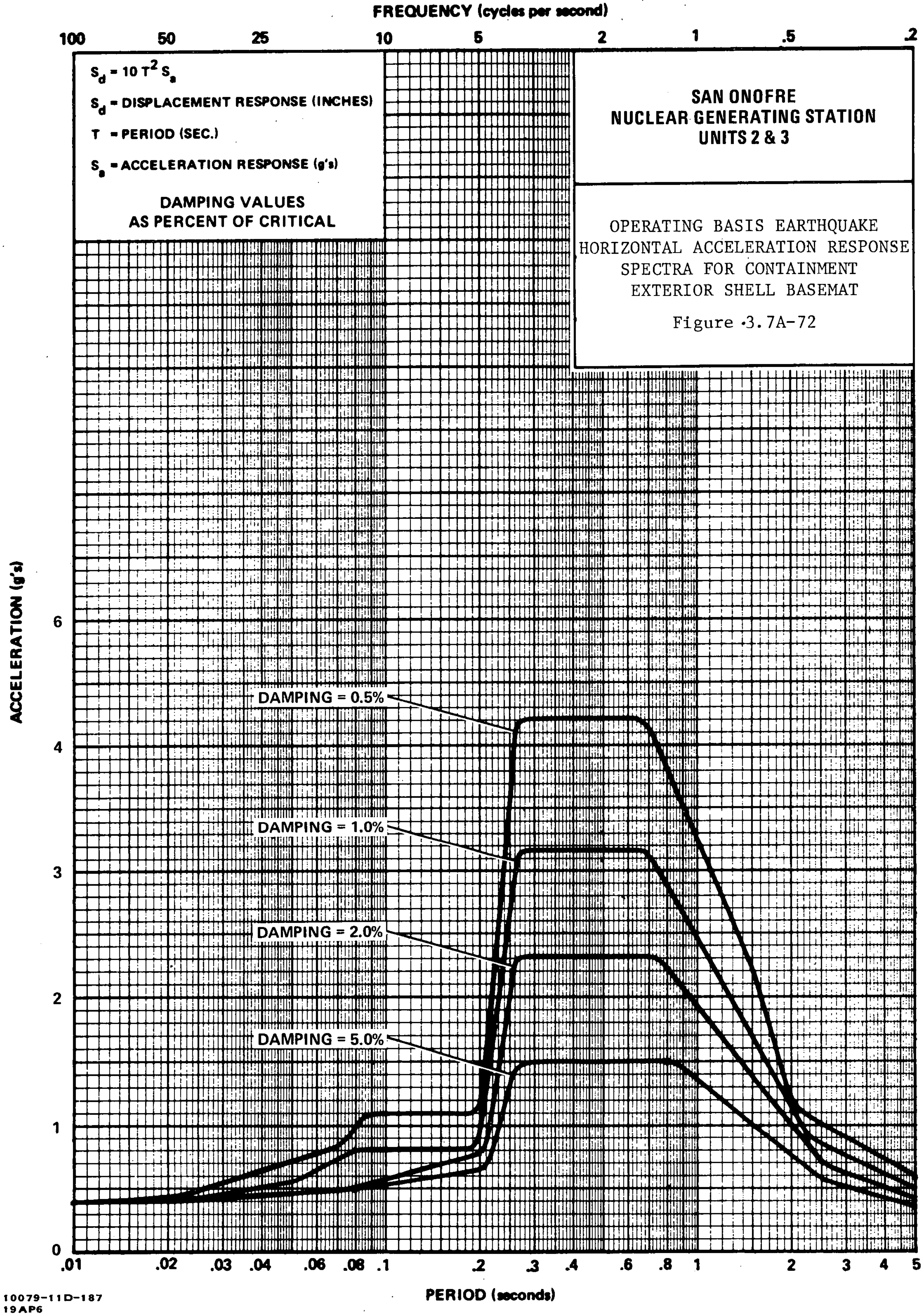


SAN ONOFRE
NUCLEAR GENERATING STATION
UNITS 2 & 3

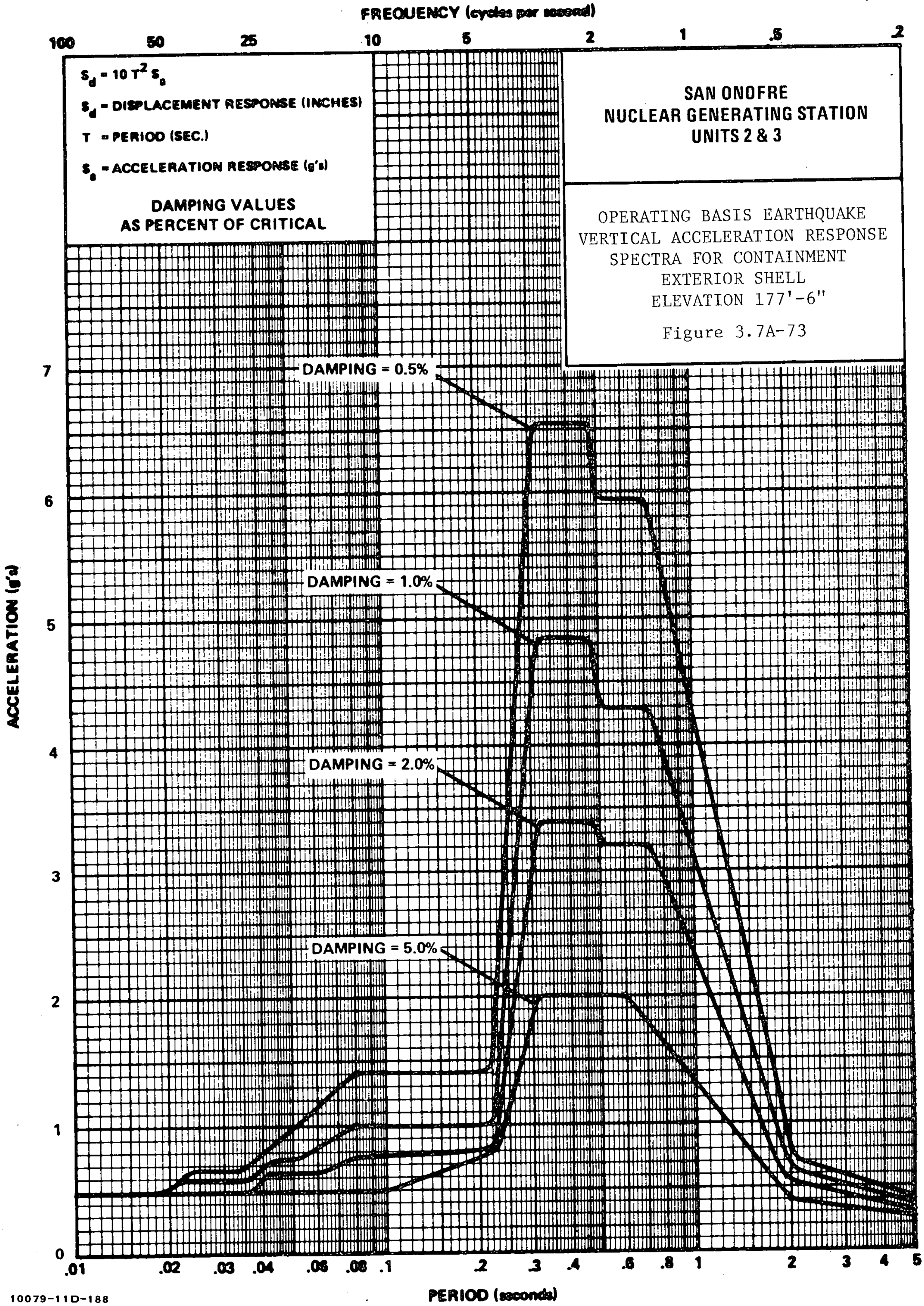
OPERATING BASIS EARTHQUAKE
VERTICAL ACCELERATION RESPONSE
SPECTRA FOR CONTAINMENT
EXTERIOR SHELL BASEMAT

Figure 3.7A-71

10079-11D-186
19AP6



10079-11D-187
19AP6

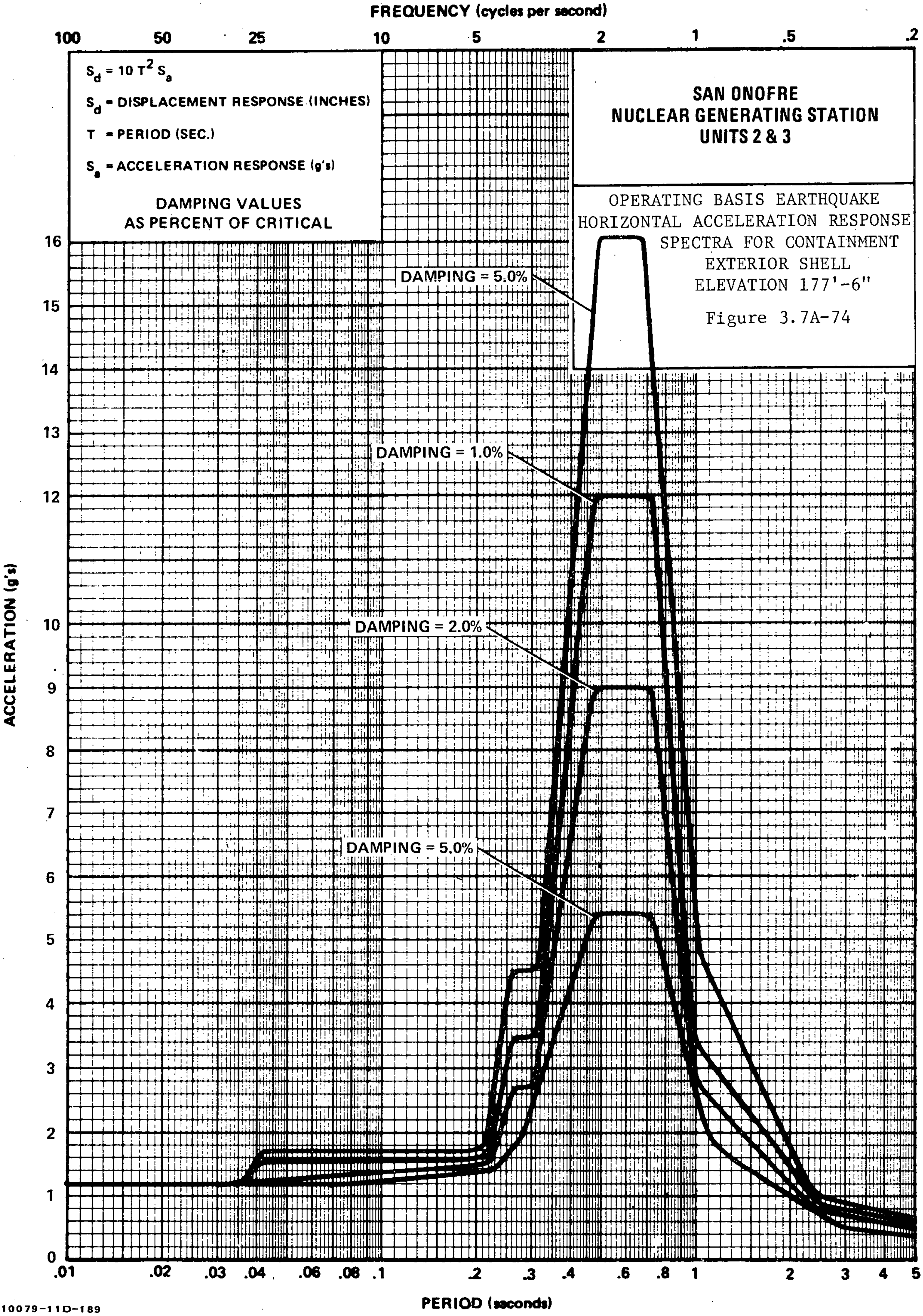


SAN ONOFRE
NUCLEAR GENERATING STATION
UNITS 2 & 3

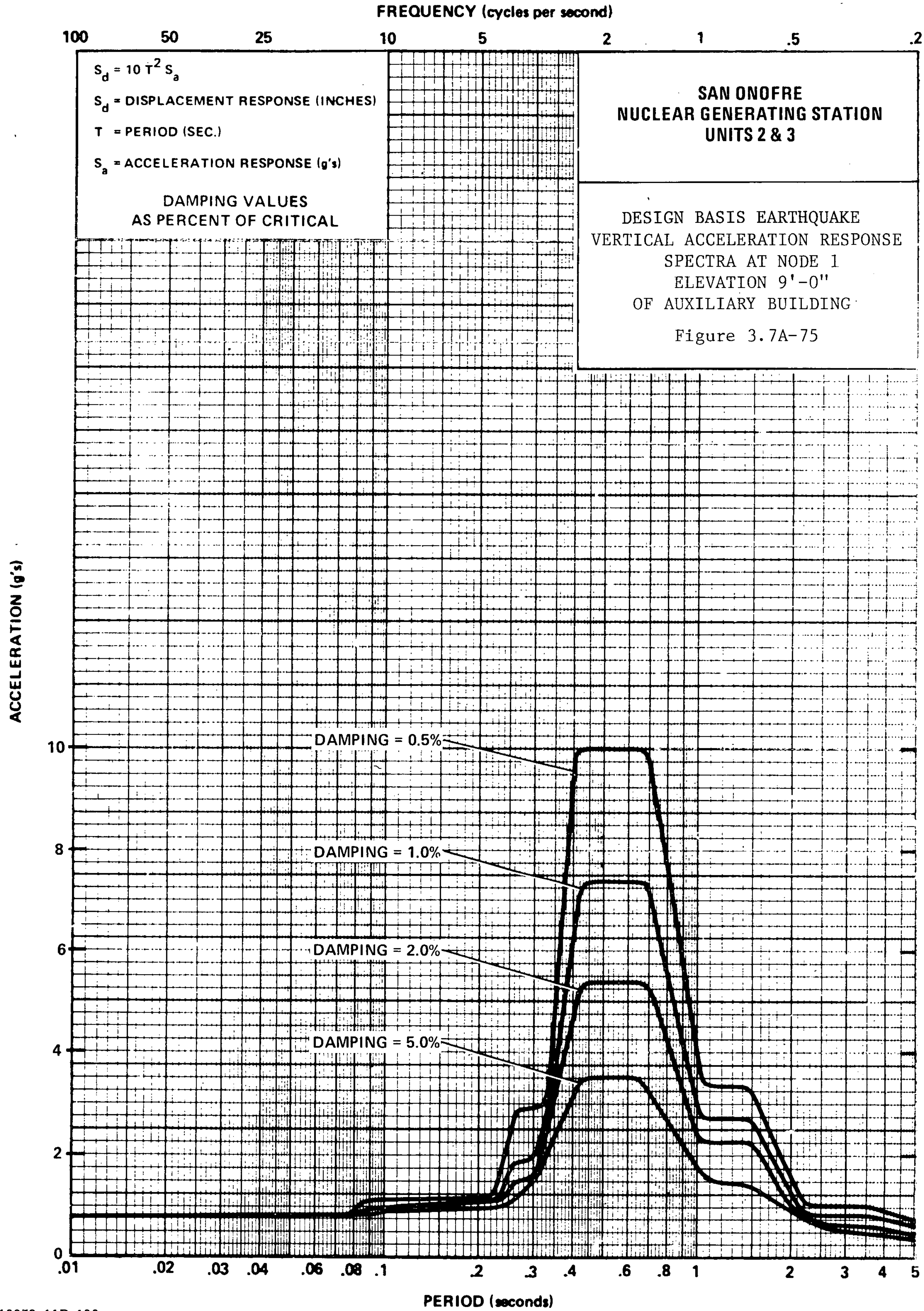
OPERATING BASIS EARTHQUAKE
VERTICAL ACCELERATION RESPONSE
SPECTRA FOR CONTAINMENT
EXTERIOR SHELL
ELEVATION 177'-6"

Figure 3.7A-73

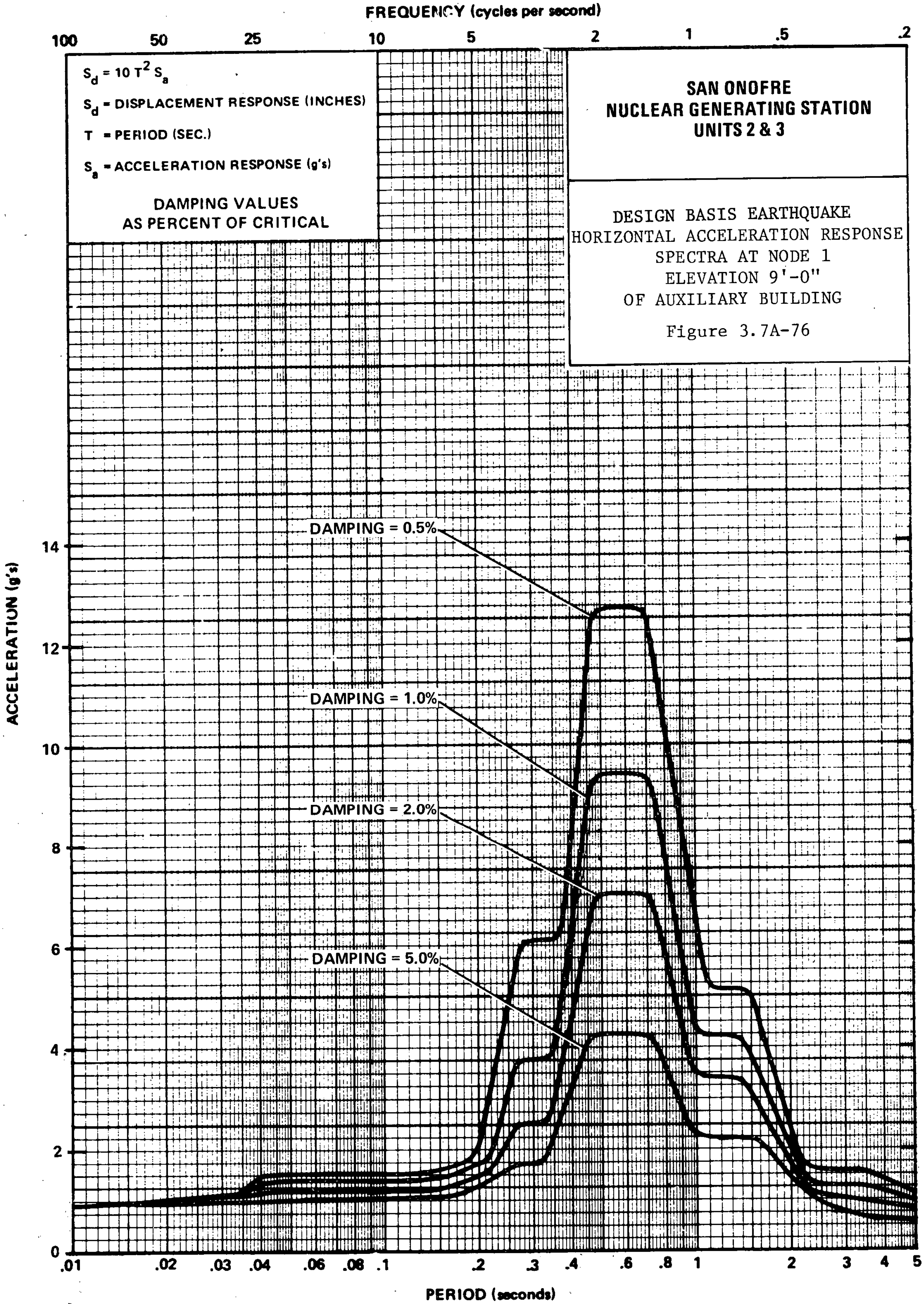
10079-11D-188
19AP6



10079-11D-189
19AP6



10079-11D-190
19AP6

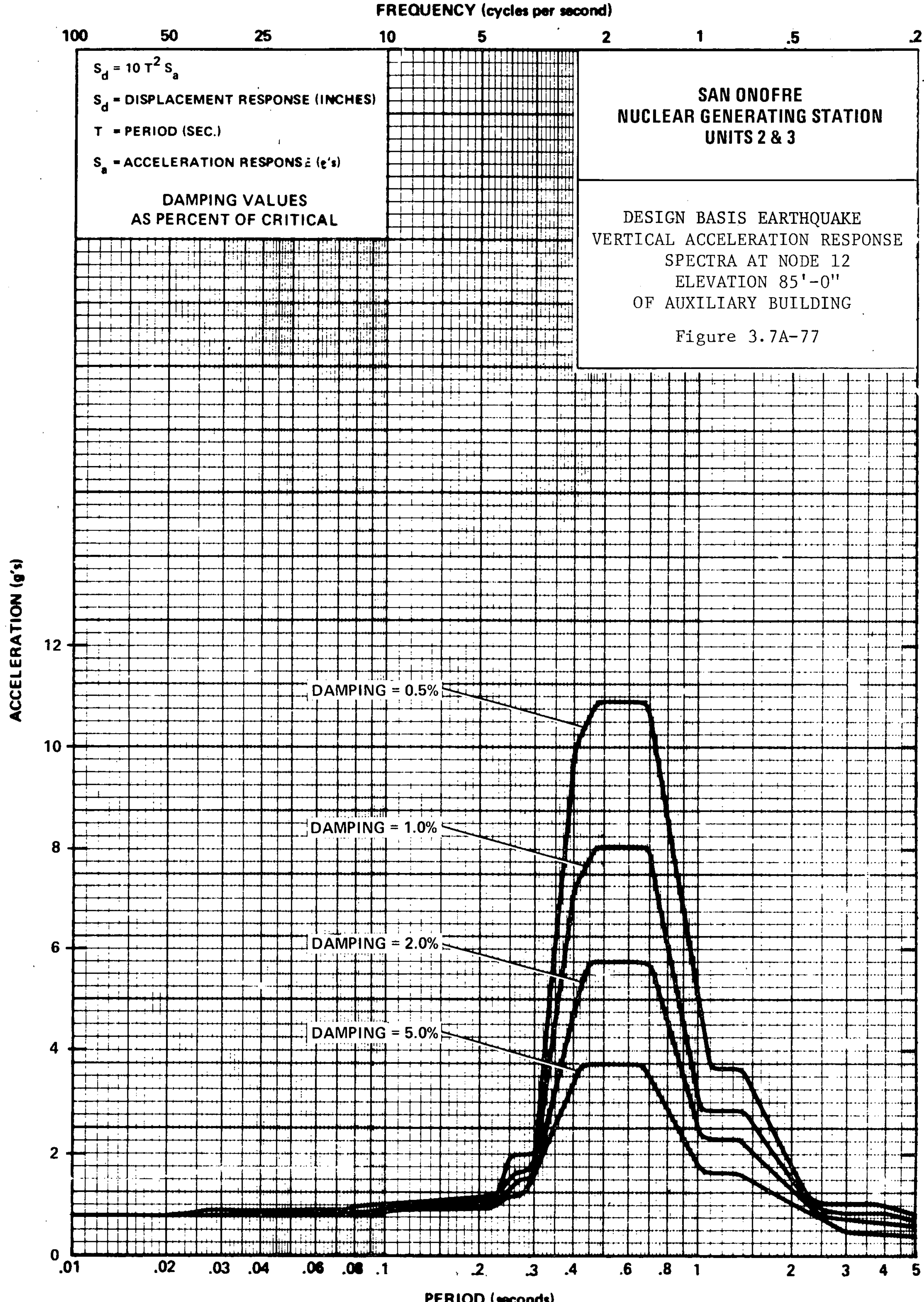


SAN ONOFRE
NUCLEAR GENERATING STATION
UNITS 2 & 3

DESIGN BASIS EARTHQUAKE
HORIZONTAL ACCELERATION RESPONSE
SPECTRA AT NODE 1
ELEVATION 9'-0"
OF AUXILIARY BUILDING

Figure 3.7A-76

10079-11D-191
19AP6

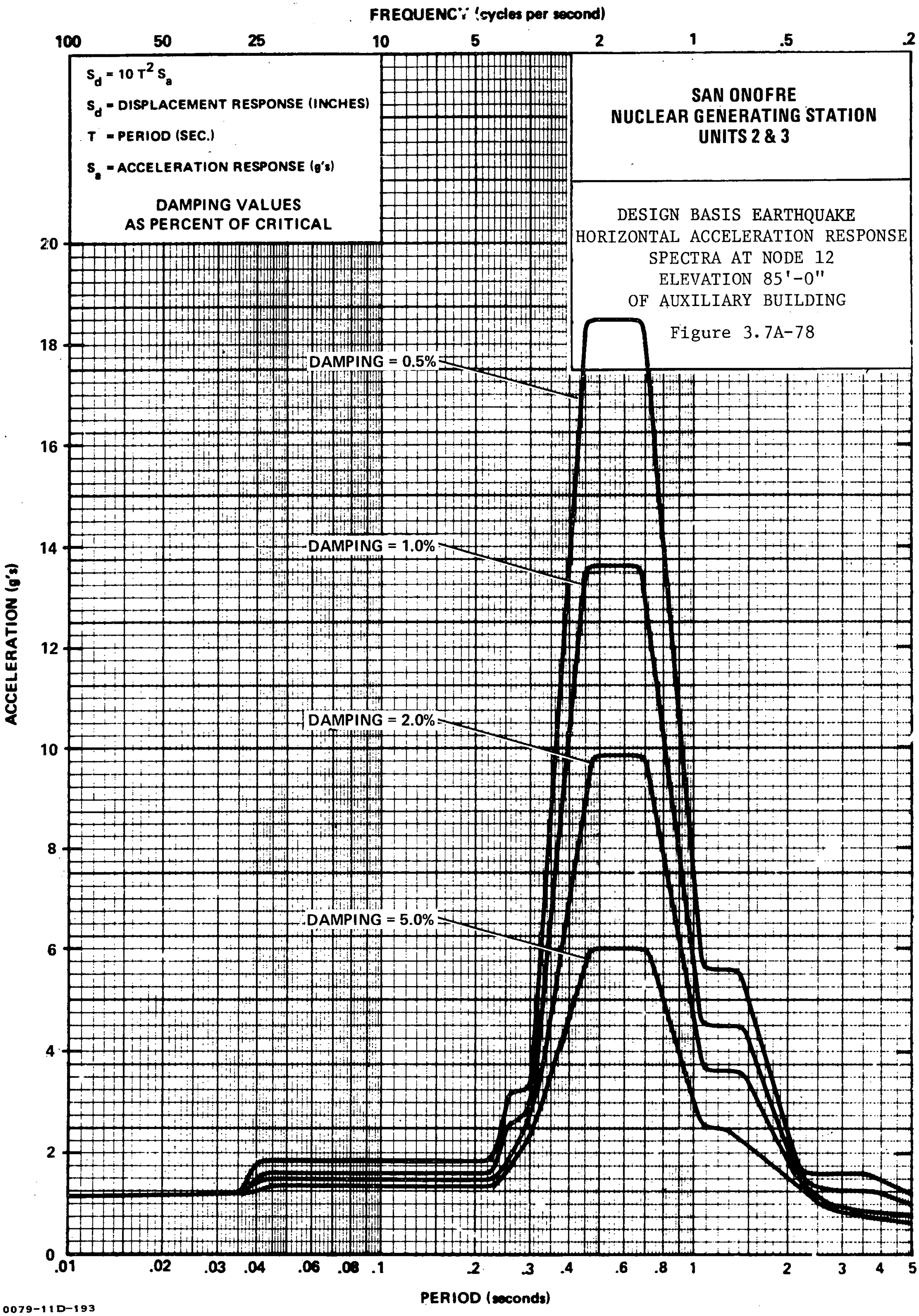


SAN ONOFRE
NUCLEAR GENERATING STATION
UNITS 2 & 3

DESIGN BASIS EARTHQUAKE
VERTICAL ACCELERATION RESPONSE
SPECTRA AT NODE 12
ELEVATION 85'-0"
OF AUXILIARY BUILDING

Figure 3.7A-77

10079-11D-192
19AP6

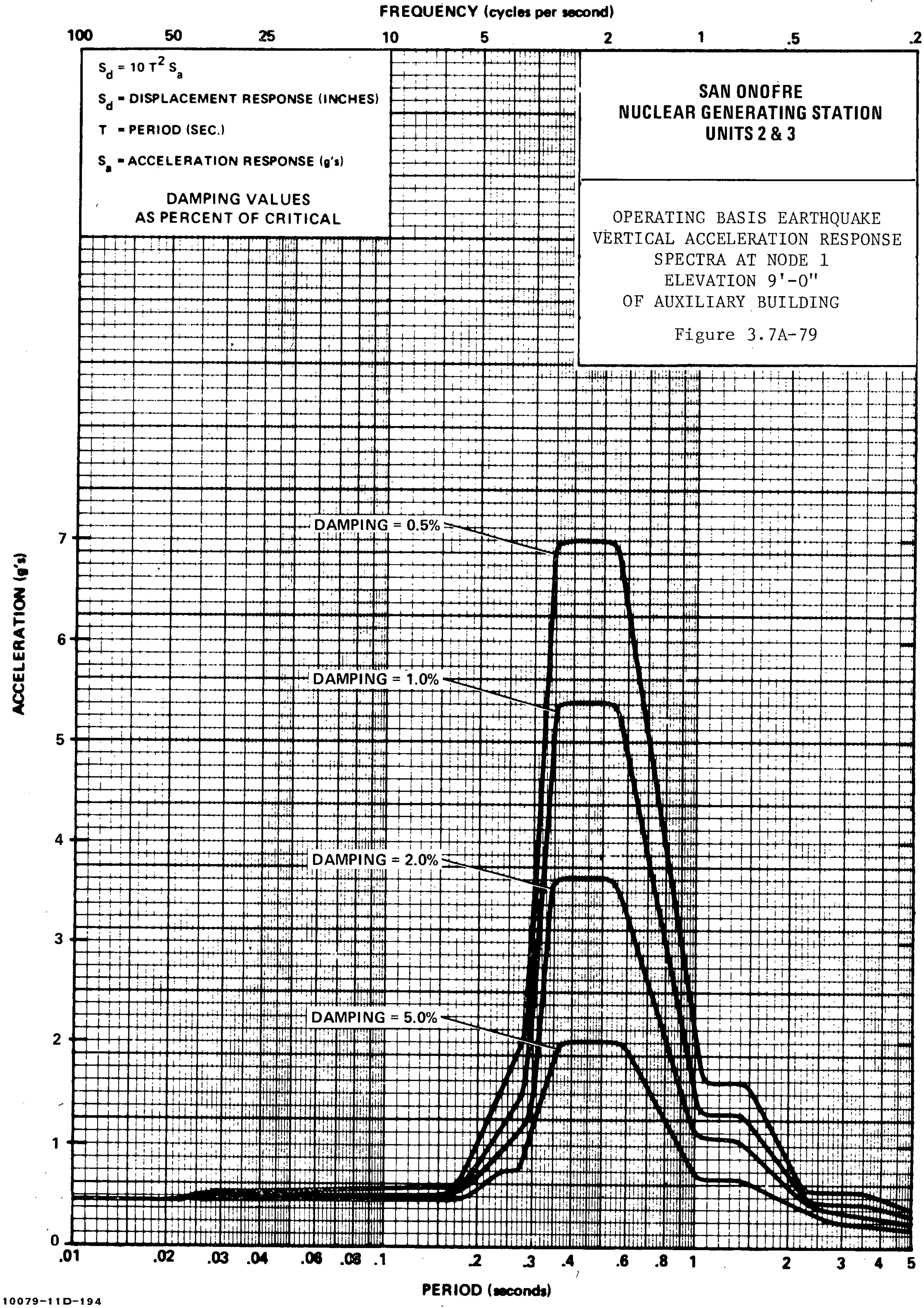


SAN ONOFRE
NUCLEAR GENERATING STATION
UNITS 2 & 3

DESIGN BASIS EARTHQUAKE
HORIZONTAL ACCELERATION RESPONSE
SPECTRA AT NODE 12
ELEVATION 85'-0"
OF AUXILIARY BUILDING

Figure 3.7A-78

10079-11D-193
19AP6

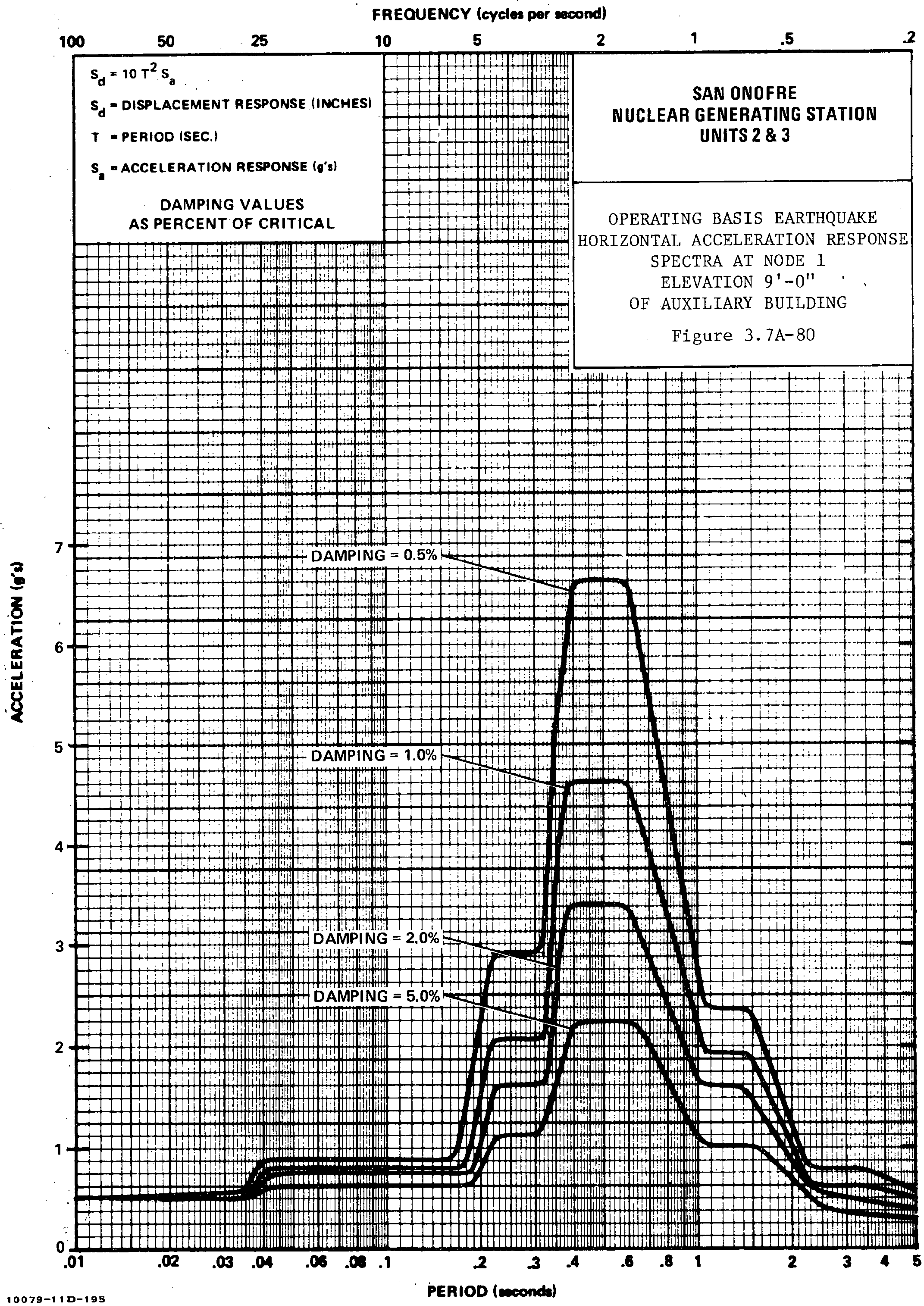


SAN ONOFRE
NUCLEAR GENERATING STATION
UNITS 2 & 3

OPERATING BASIS EARTHQUAKE
VERTICAL ACCELERATION RESPONSE
SPECTRA AT NODE 1
ELEVATION 9'-0"
OF AUXILIARY BUILDING

Figure 3.7A-79

10079-11D-194
19AP6

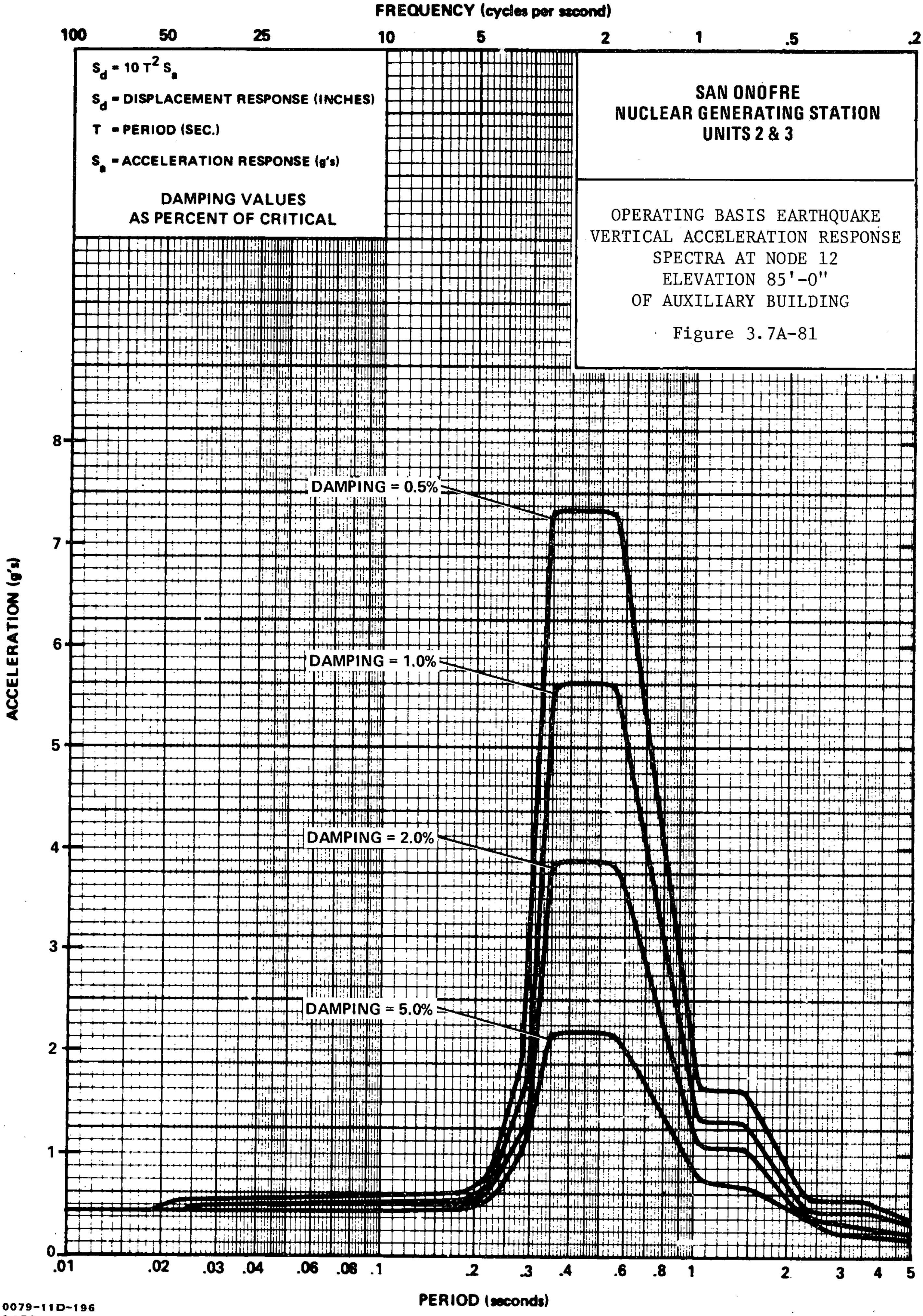


SAN ONOFRE
NUCLEAR GENERATING STATION
UNITS 2 & 3

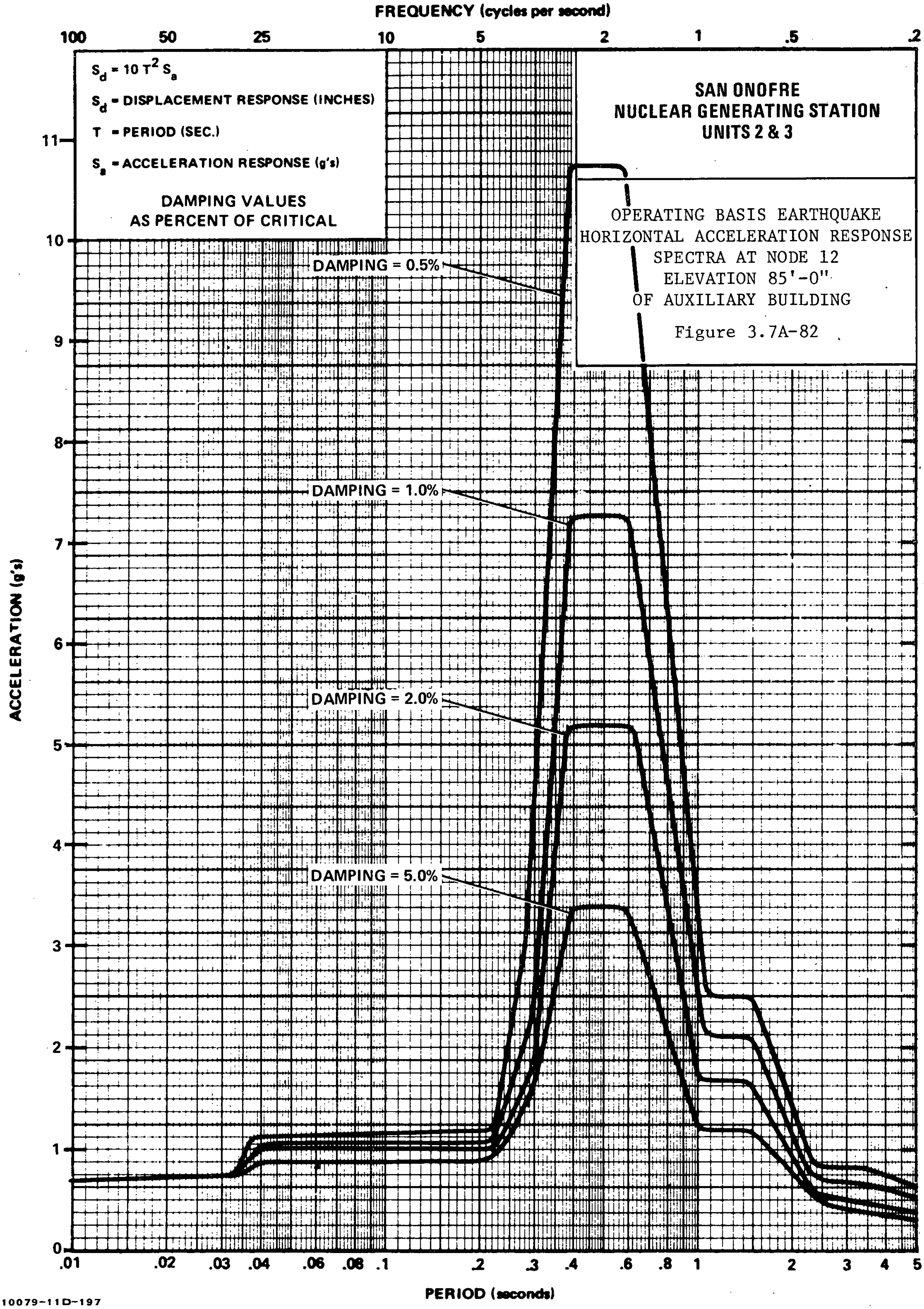
OPERATING BASIS EARTHQUAKE
HORIZONTAL ACCELERATION RESPONSE
SPECTRA AT NODE 1
ELEVATION 9'-0"
OF AUXILIARY BUILDING

Figure 3.7A-80

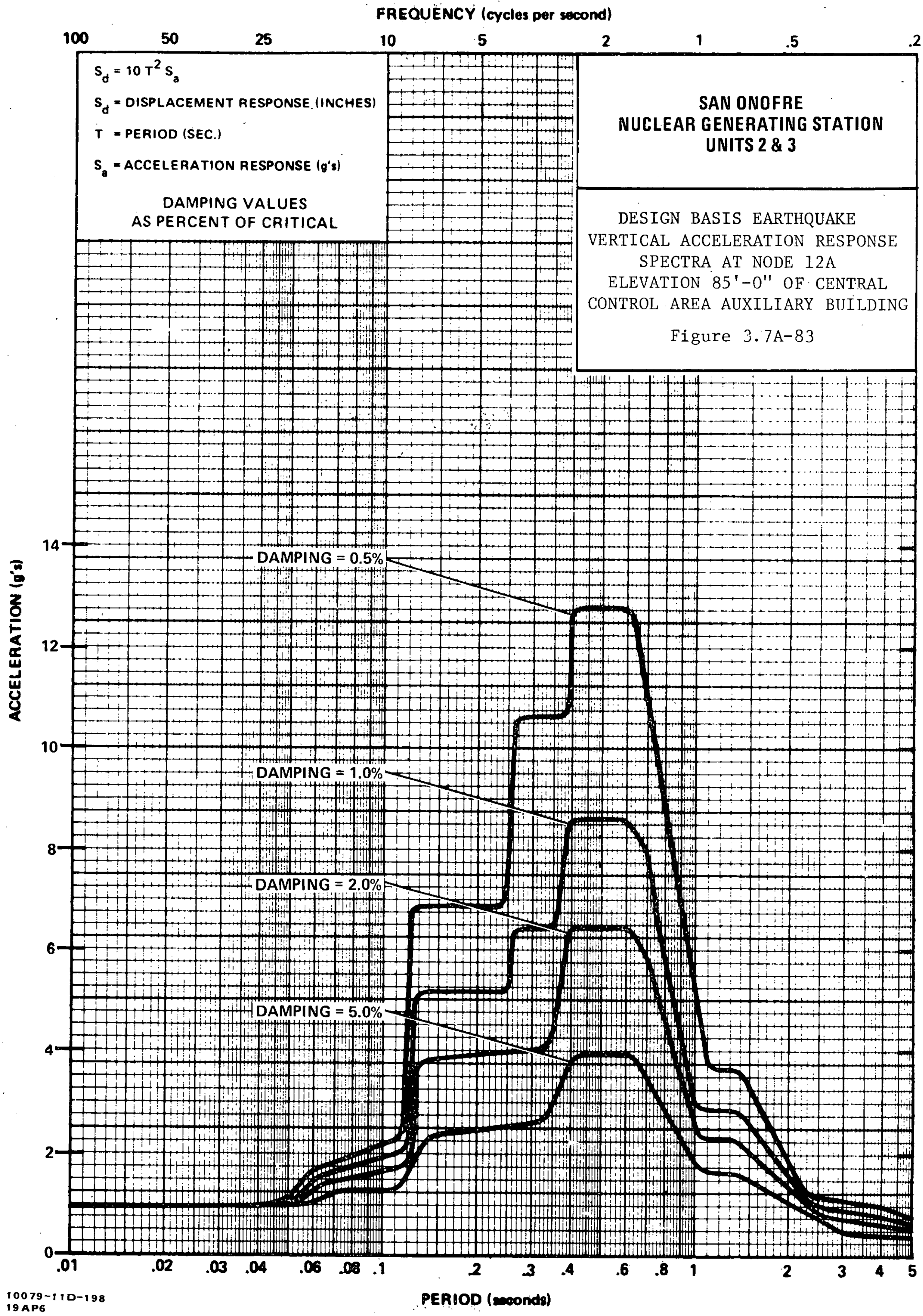
10079-11D-195
19AP6



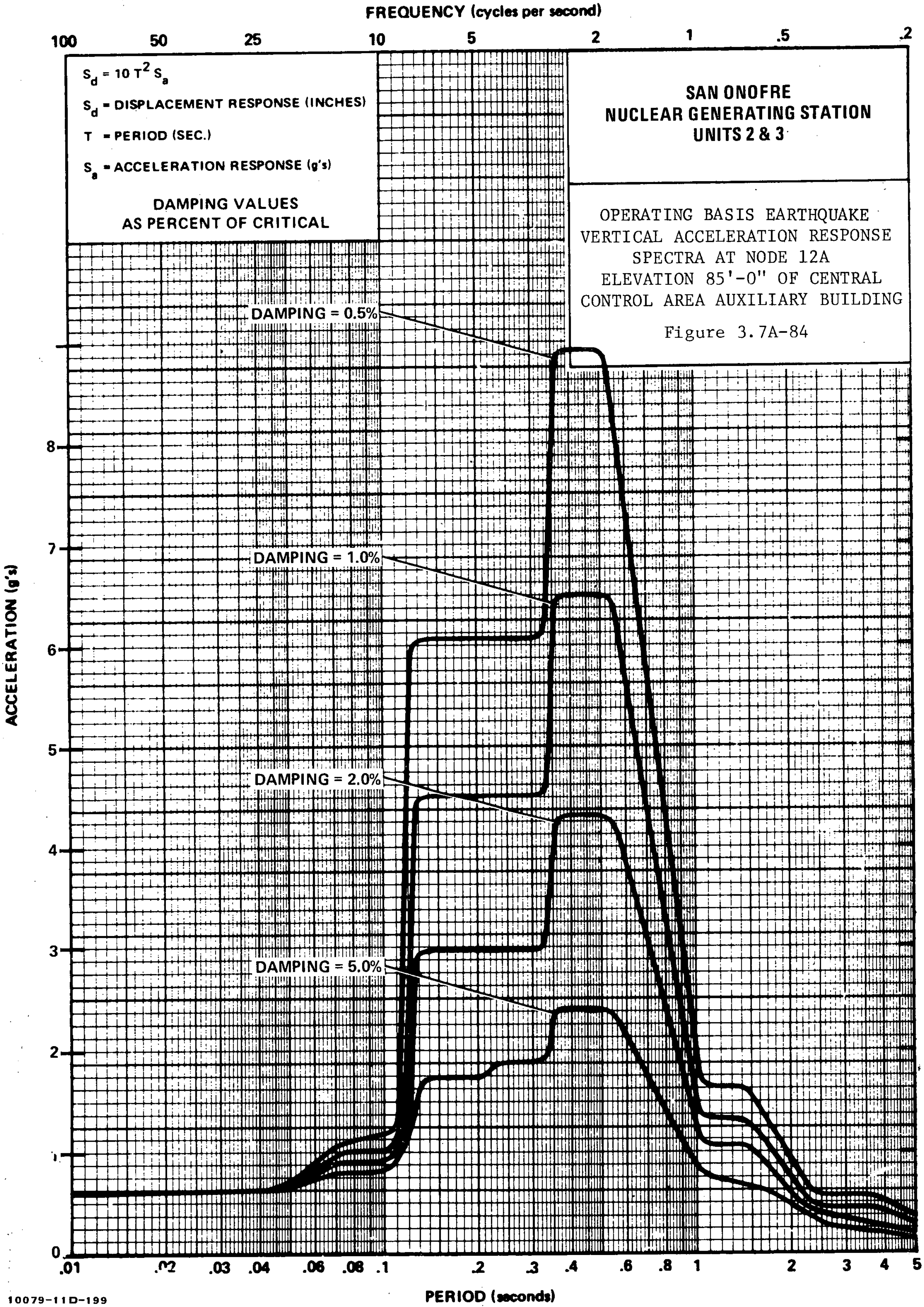
10079-11D-196
19AP6



10079-11D-197
19AP6



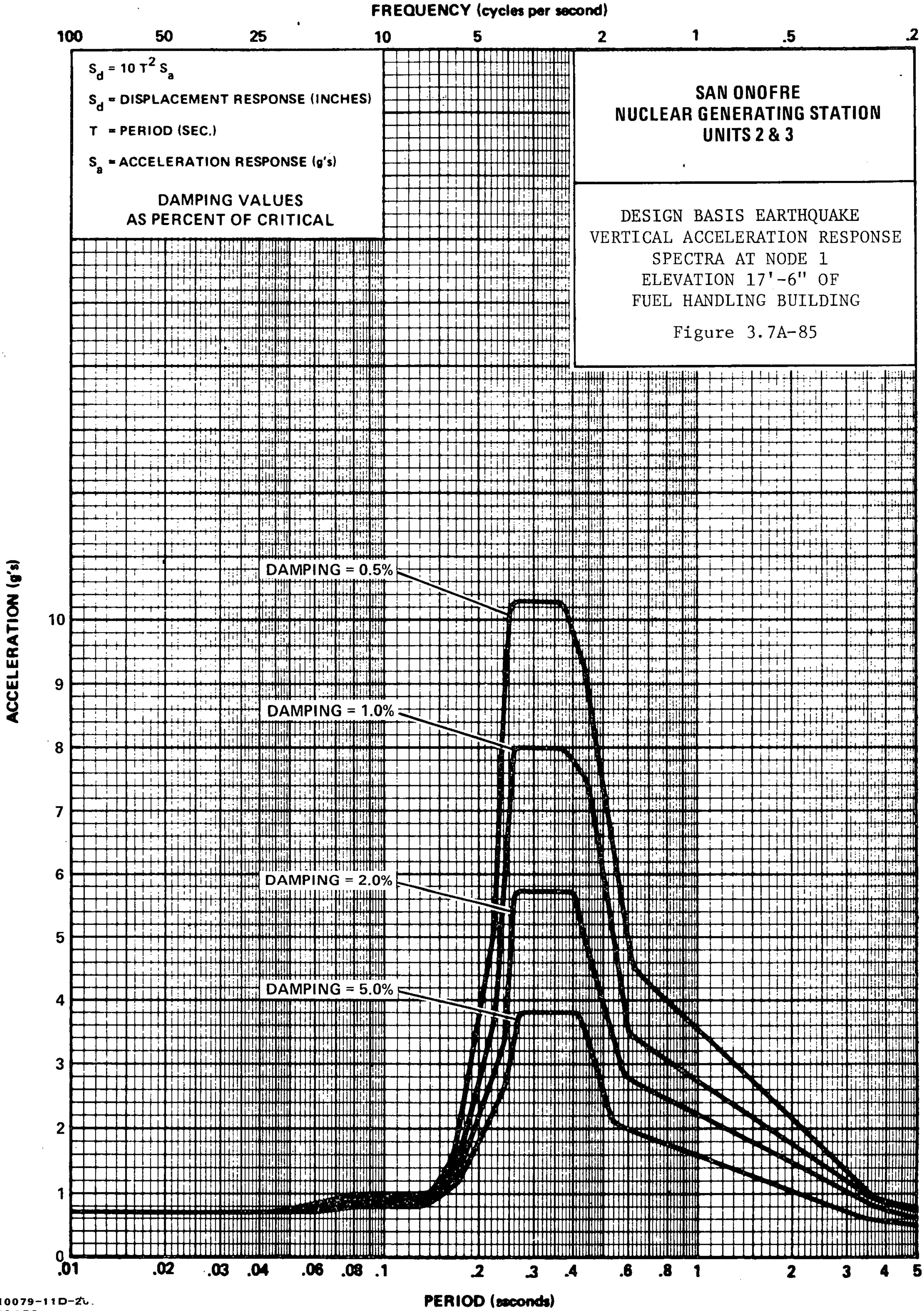
10079-11D-198
19AP6



OPERATING BASIS EARTHQUAKE
VERTICAL ACCELERATION RESPONSE
SPECTRA AT NODE 12A
ELEVATION 85'-0" OF CENTRAL
CONTROL AREA AUXILIARY BUILDING

Figure 3.7A-84

10079-11D-199
19AP6

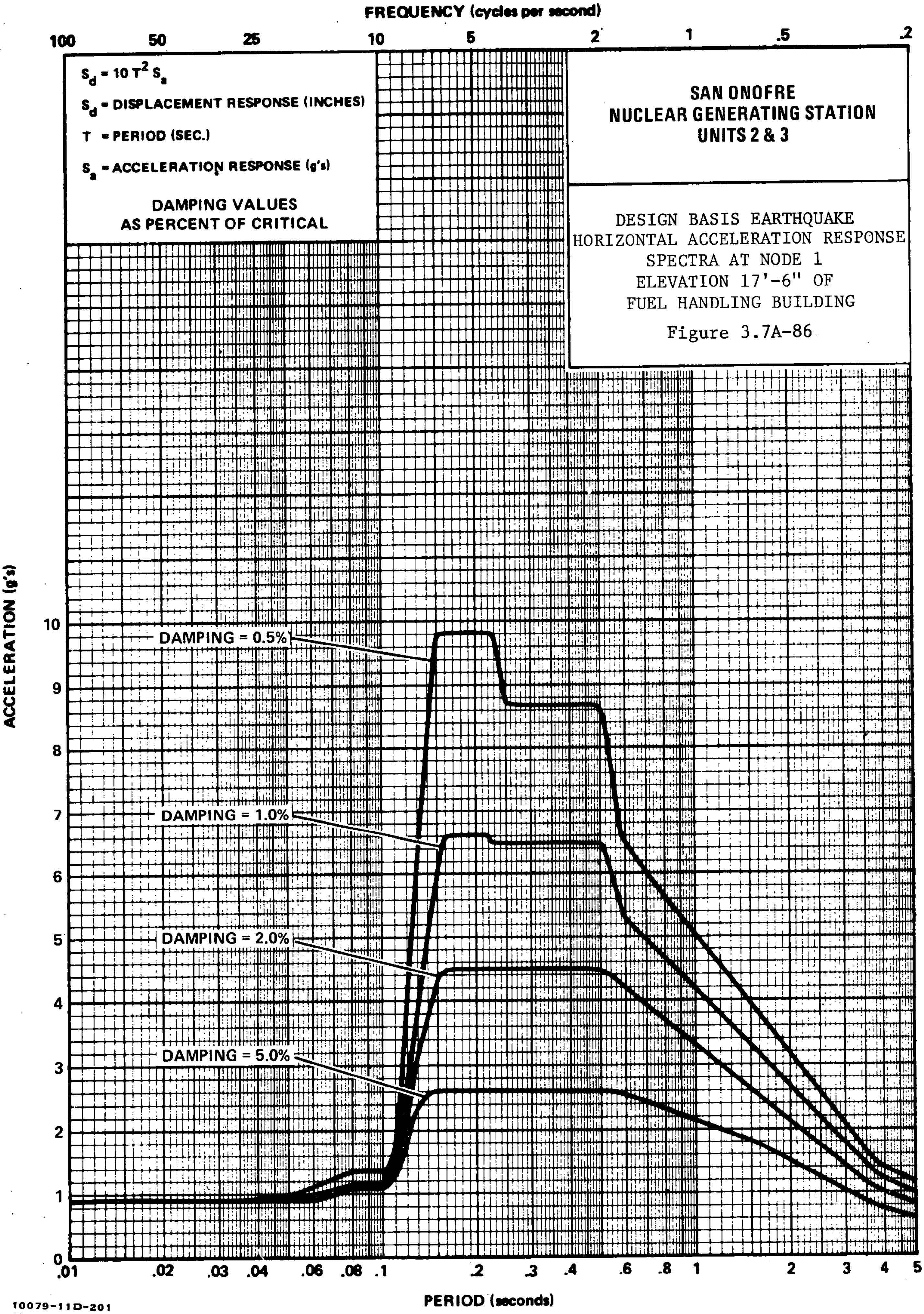


SAN ONOFRE
NUCLEAR GENERATING STATION
UNITS 2 & 3

DESIGN BASIS EARTHQUAKE
VERTICAL ACCELERATION RESPONSE
SPECTRA AT NODE 1
ELEVATION 17'-6" OF
FUEL HANDLING BUILDING

Figure 3.7A-85

10079-11D-20.
19AP6

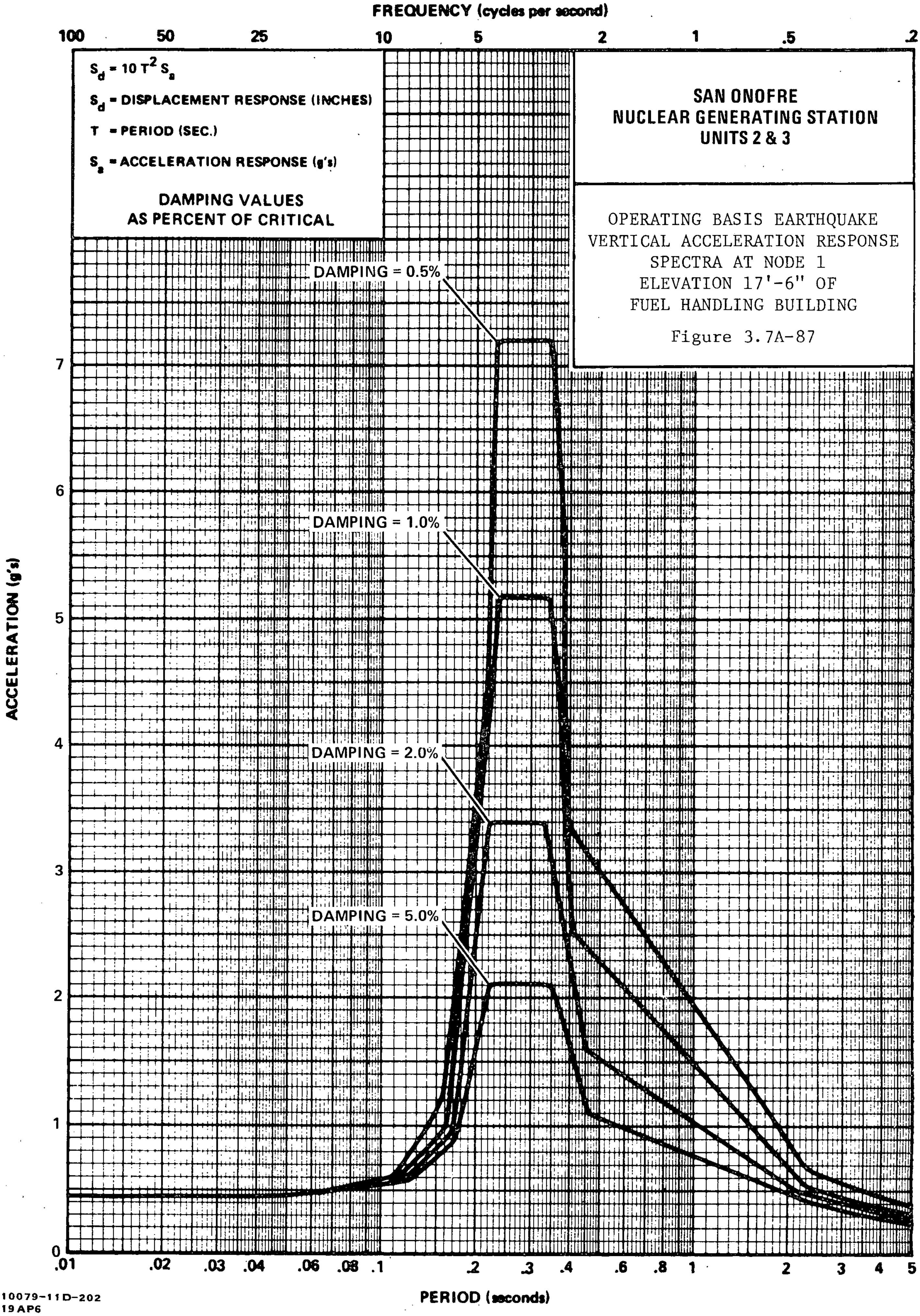


SAN ONOFRE
NUCLEAR GENERATING STATION
UNITS 2 & 3

DESIGN BASIS EARTHQUAKE
HORIZONTAL ACCELERATION RESPONSE
SPECTRA AT NODE 1
ELEVATION 17'-6" OF
FUEL HANDLING BUILDING

Figure 3.7A-86

10079-11D-201
19AP6

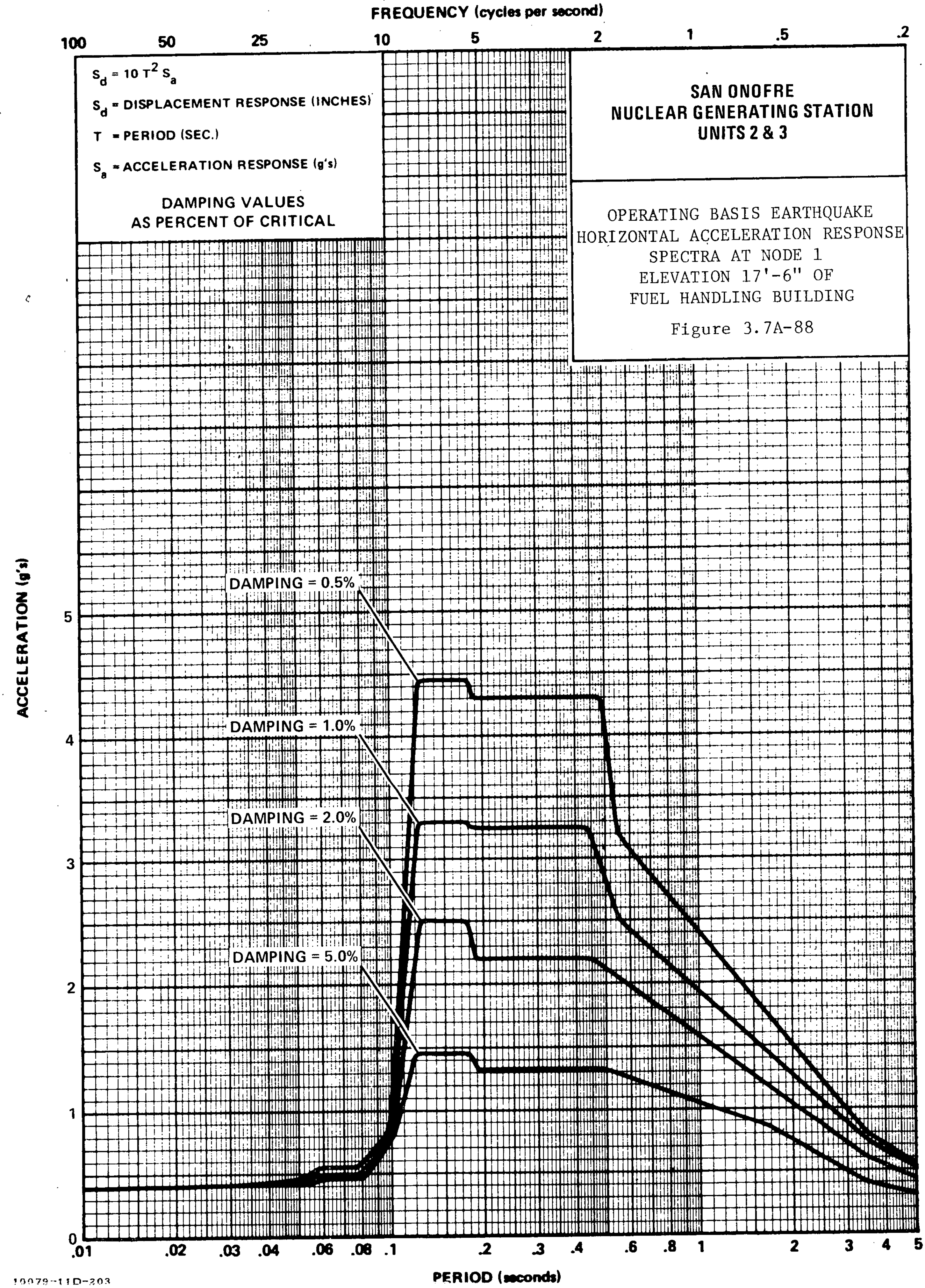


SAN ONOFRE
NUCLEAR GENERATING STATION
UNITS 2 & 3

OPERATING BASIS EARTHQUAKE
VERTICAL ACCELERATION RESPONSE
SPECTRA AT NODE 1
ELEVATION 17'-6" OF
FUEL HANDLING BUILDING

Figure 3.7A-87

10079-11D-202
19AP6

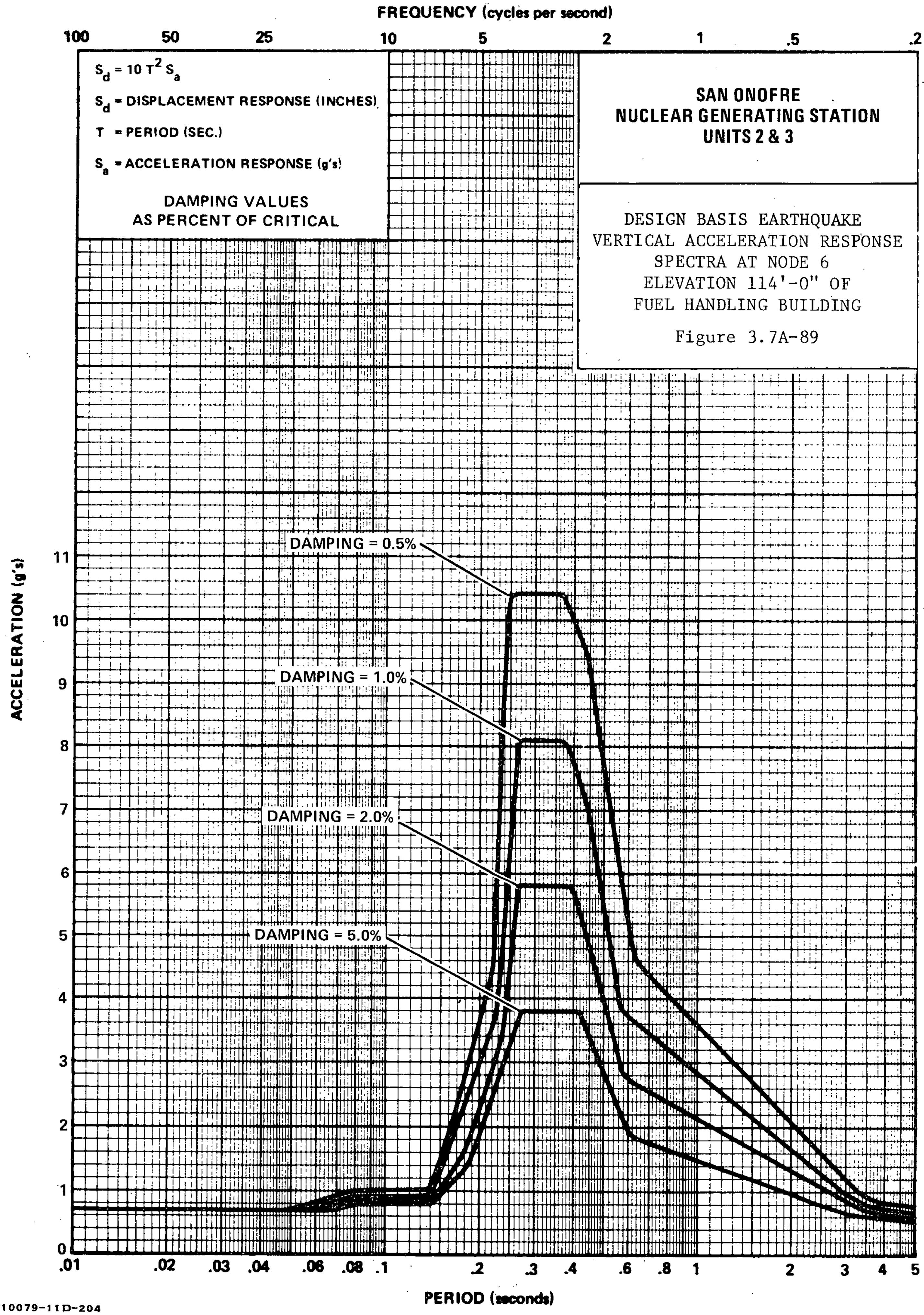


SAN ONOFRE
NUCLEAR GENERATING STATION
UNITS 2 & 3

OPERATING BASIS EARTHQUAKE
HORIZONTAL ACCELERATION RESPONSE
SPECTRA AT NODE 1
ELEVATION 17'-6" OF
FUEL HANDLING BUILDING

Figure 3.7A-88

10079-11D-303
1SAPG

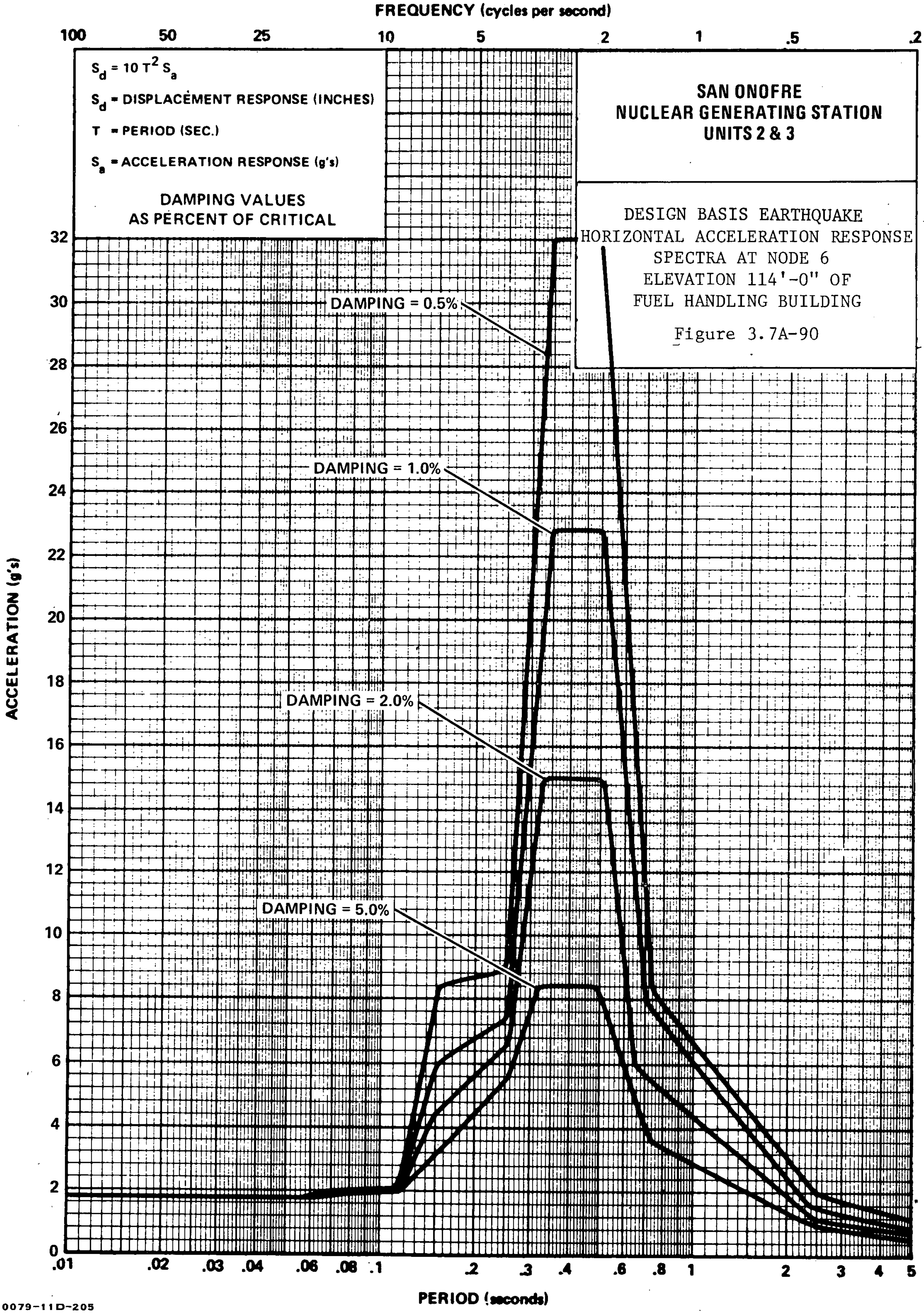


SAN ONOFRE
NUCLEAR GENERATING STATION
UNITS 2 & 3

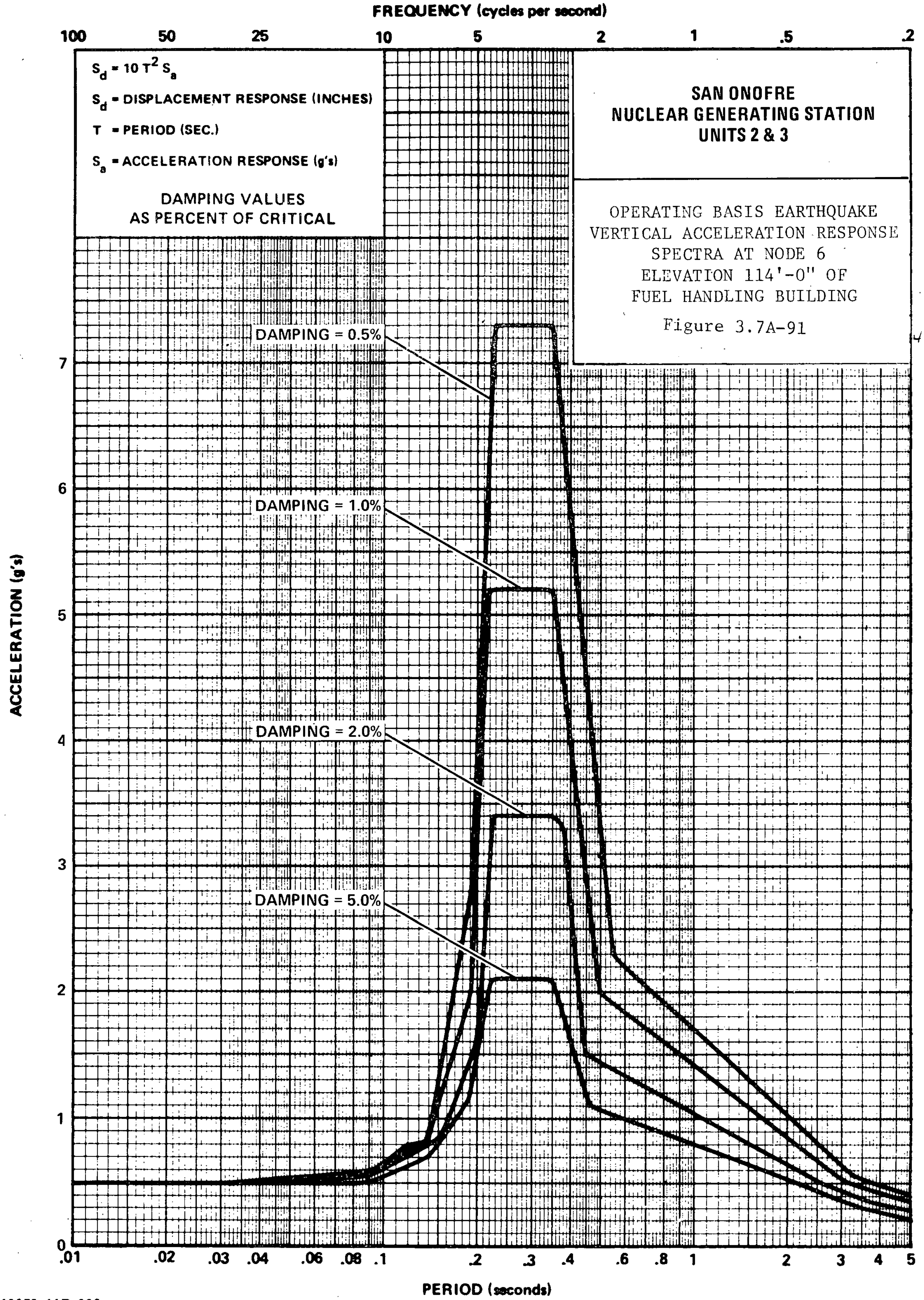
DESIGN BASIS EARTHQUAKE
VERTICAL ACCELERATION RESPONSE
SPECTRA AT NODE 6
ELEVATION 114'-0" OF
FUEL HANDLING BUILDING

Figure 3.7A-89

10079-11D-204
19AP6



10079-11D-205
19AP6

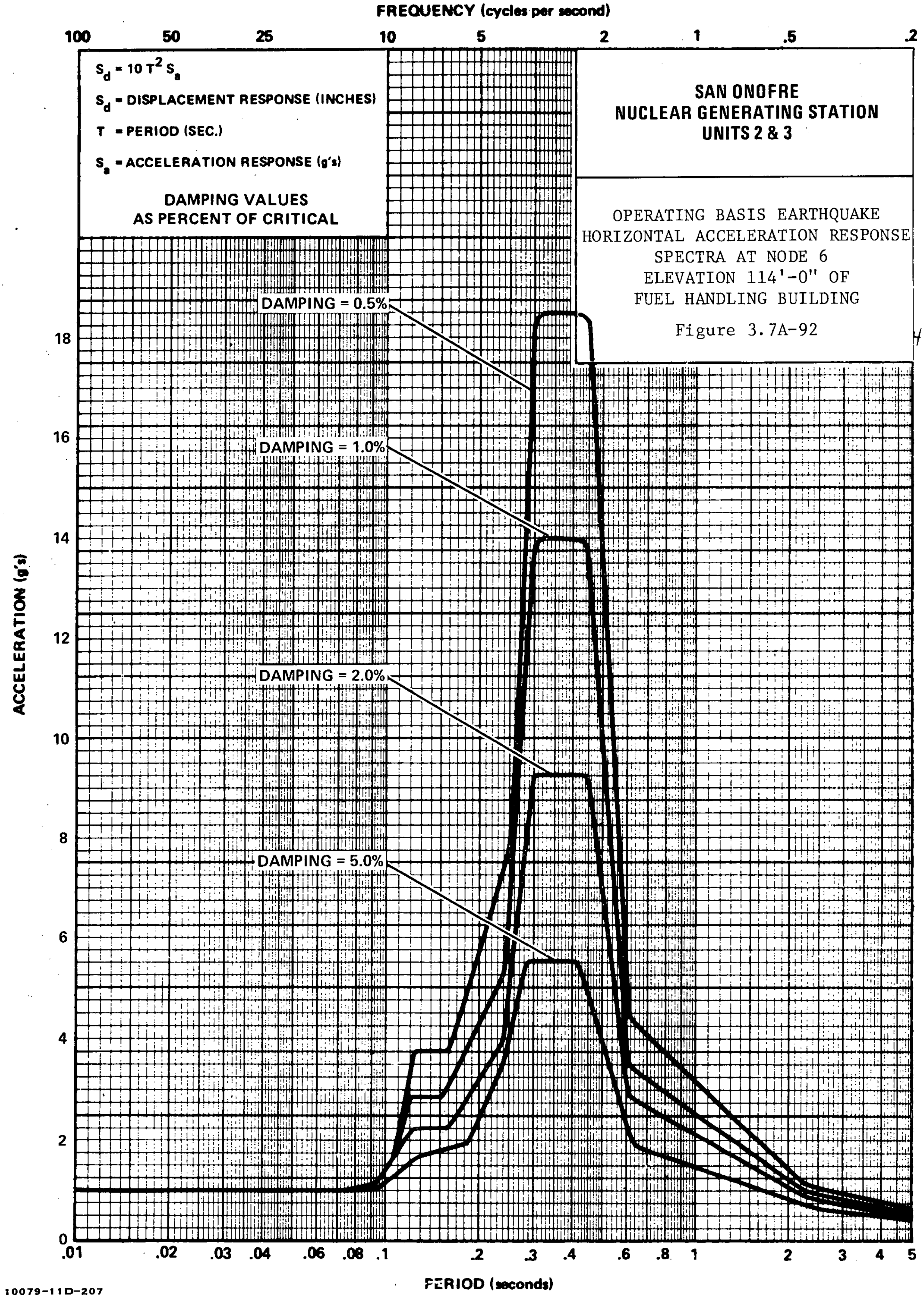


SAN ONOFRE
NUCLEAR GENERATING STATION
UNITS 2 & 3

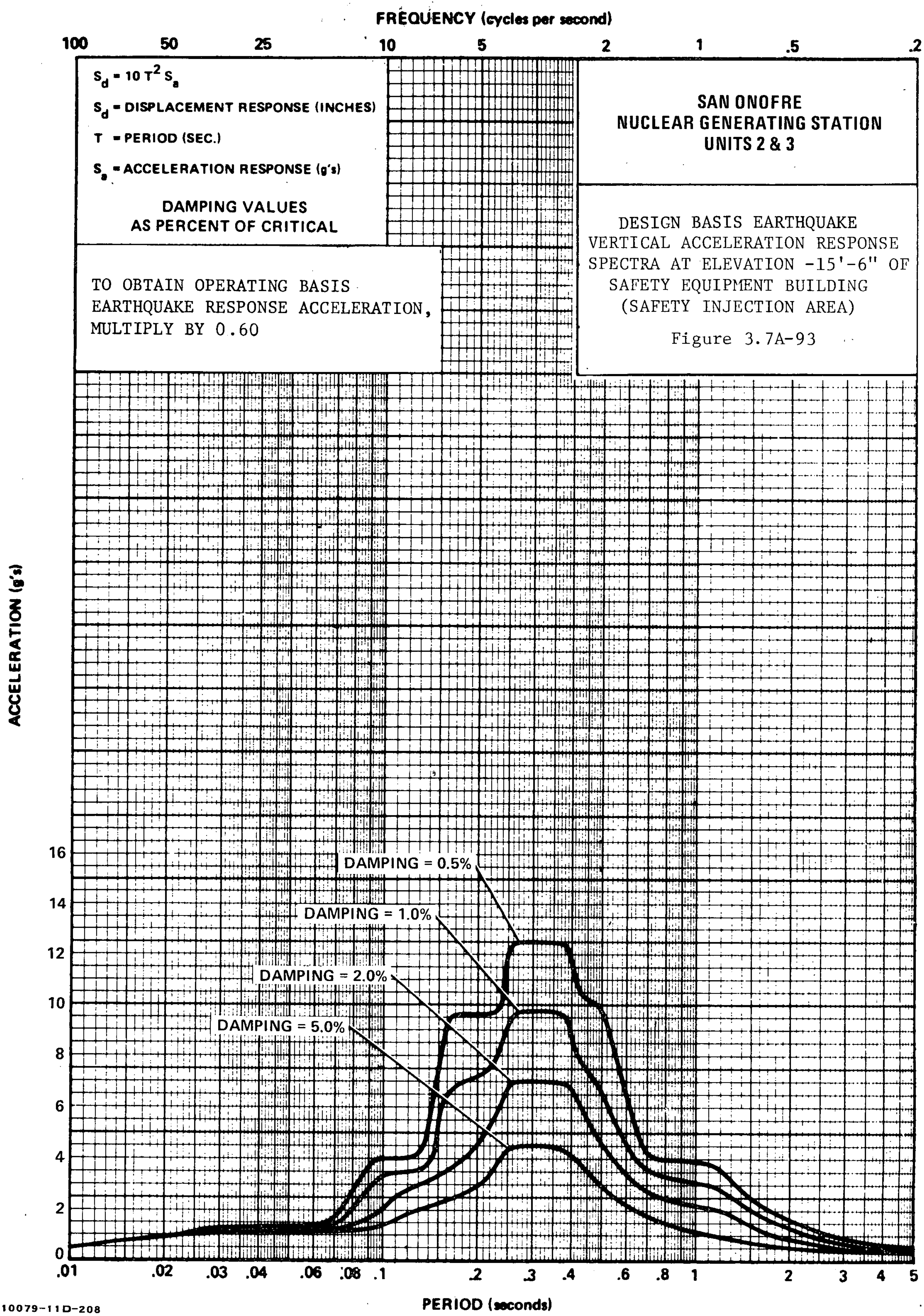
OPERATING BASIS EARTHQUAKE
VERTICAL ACCELERATION RESPONSE
SPECTRA AT NODE 6
ELEVATION 114'-0" OF
FUEL HANDLING BUILDING

Figure 3.7A-91

10079-11D-206
19AP6



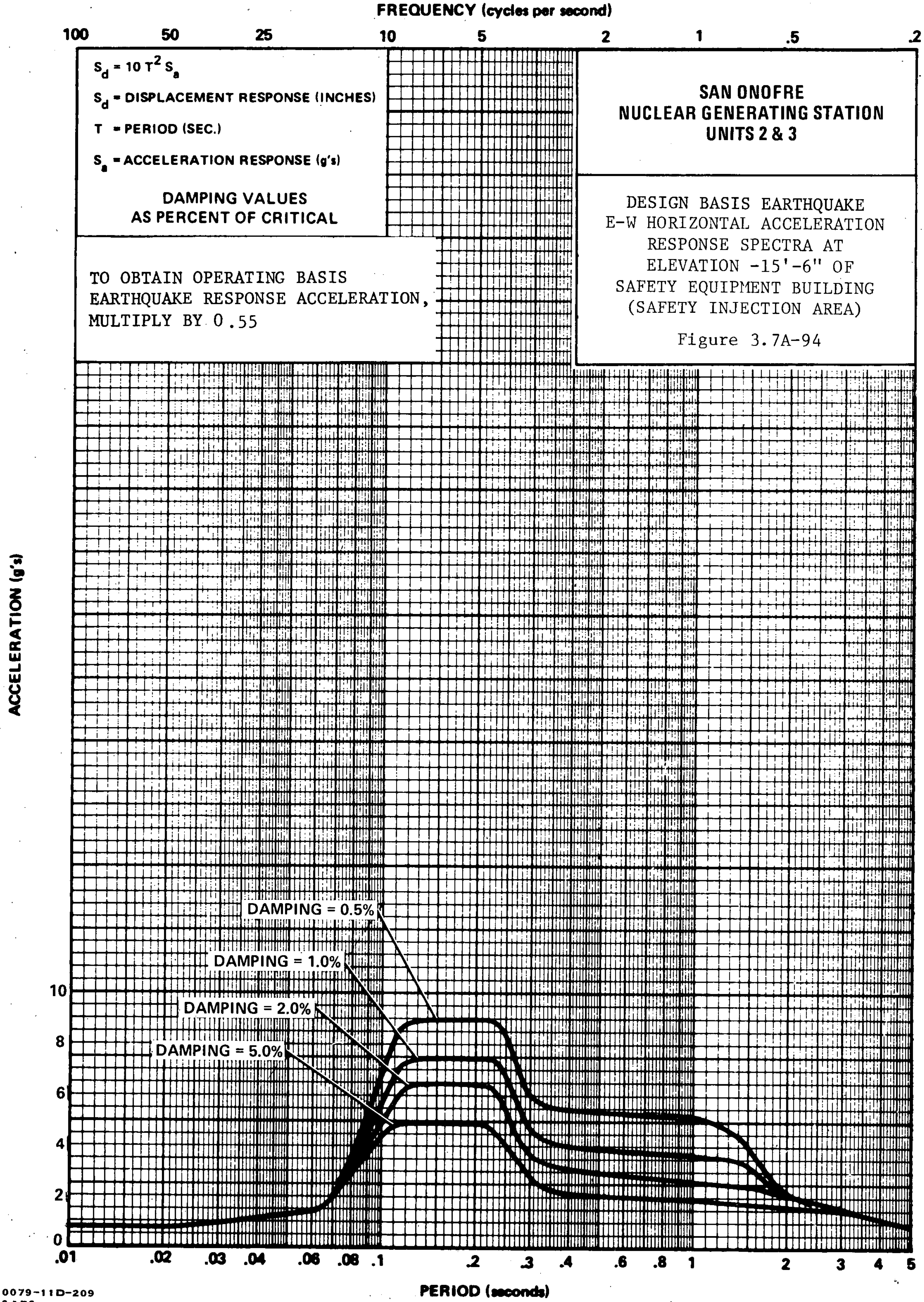
10079-11D-207
19AP6



DESIGN BASIS EARTHQUAKE
VERTICAL ACCELERATION RESPONSE
SPECTRA AT ELEVATION -15'-6" OF
SAFETY EQUIPMENT BUILDING
(SAFETY INJECTION AREA)

Figure 3.7A-93

10079-11D-208
19AP6

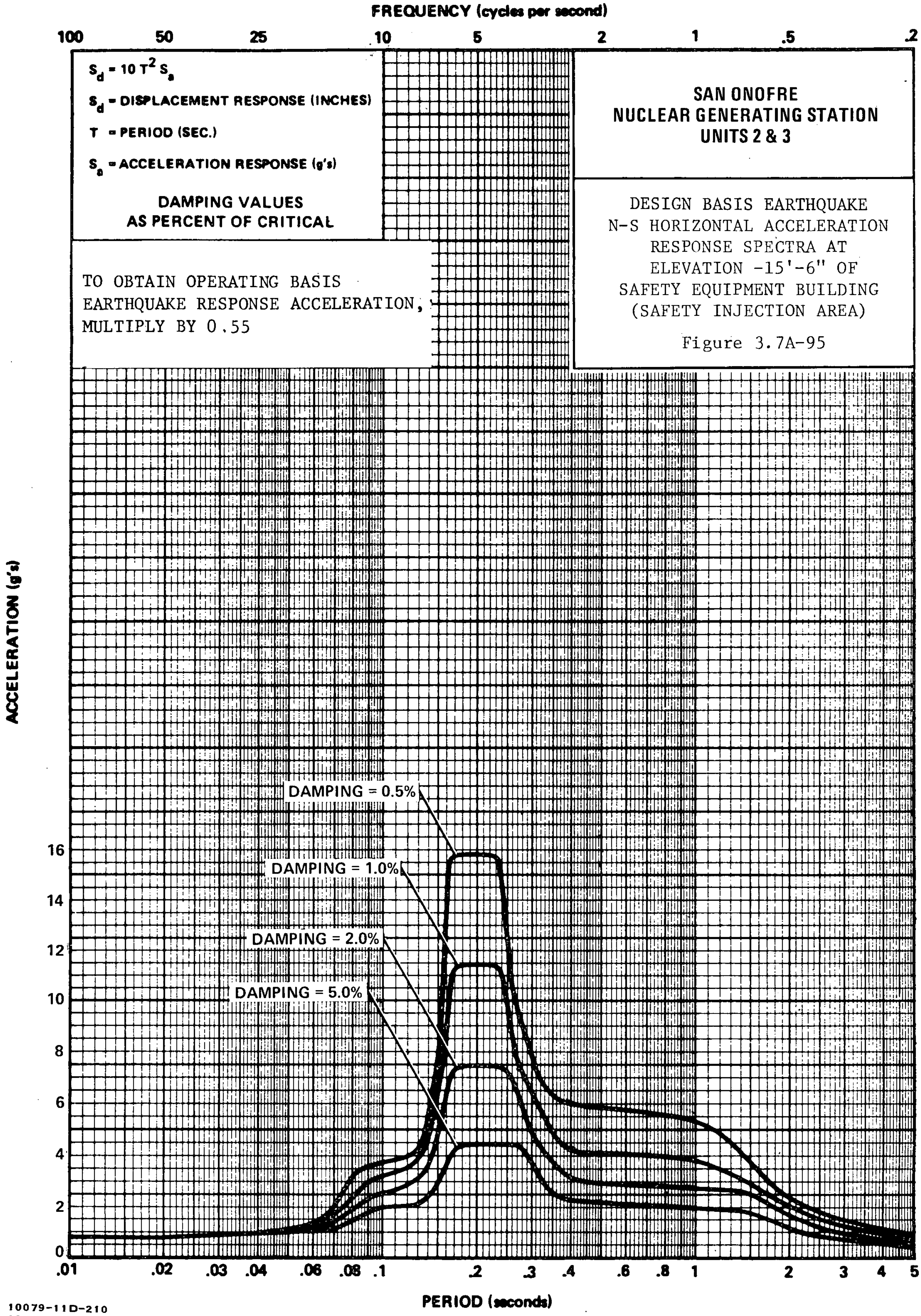


SAN ONOFRE
NUCLEAR GENERATING STATION
UNITS 2 & 3

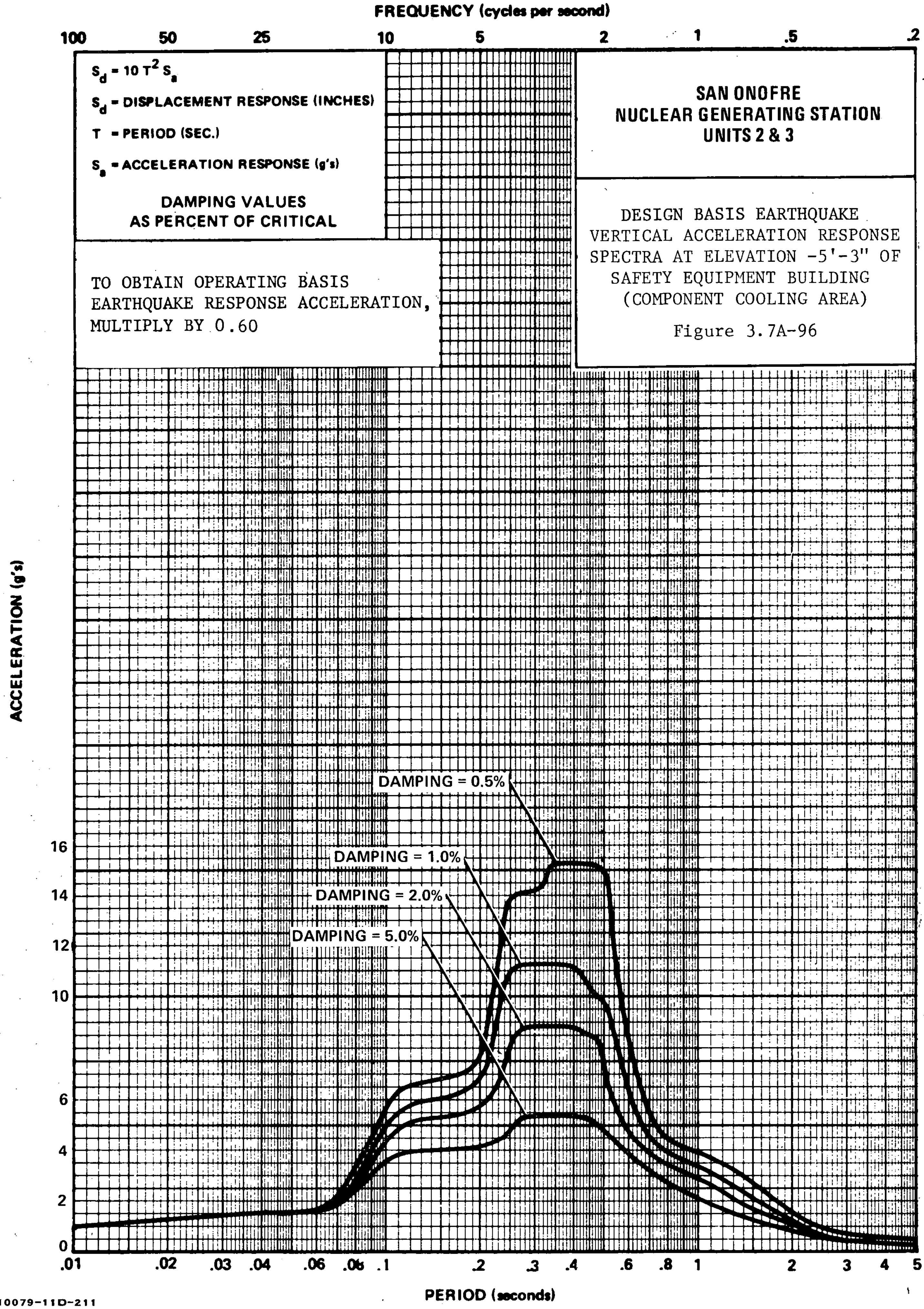
DESIGN BASIS EARTHQUAKE
E-W HORIZONTAL ACCELERATION
RESPONSE SPECTRA AT
ELEVATION -15'-6" OF
SAFETY EQUIPMENT BUILDING
(SAFETY INJECTION AREA)

Figure 3.7A-94

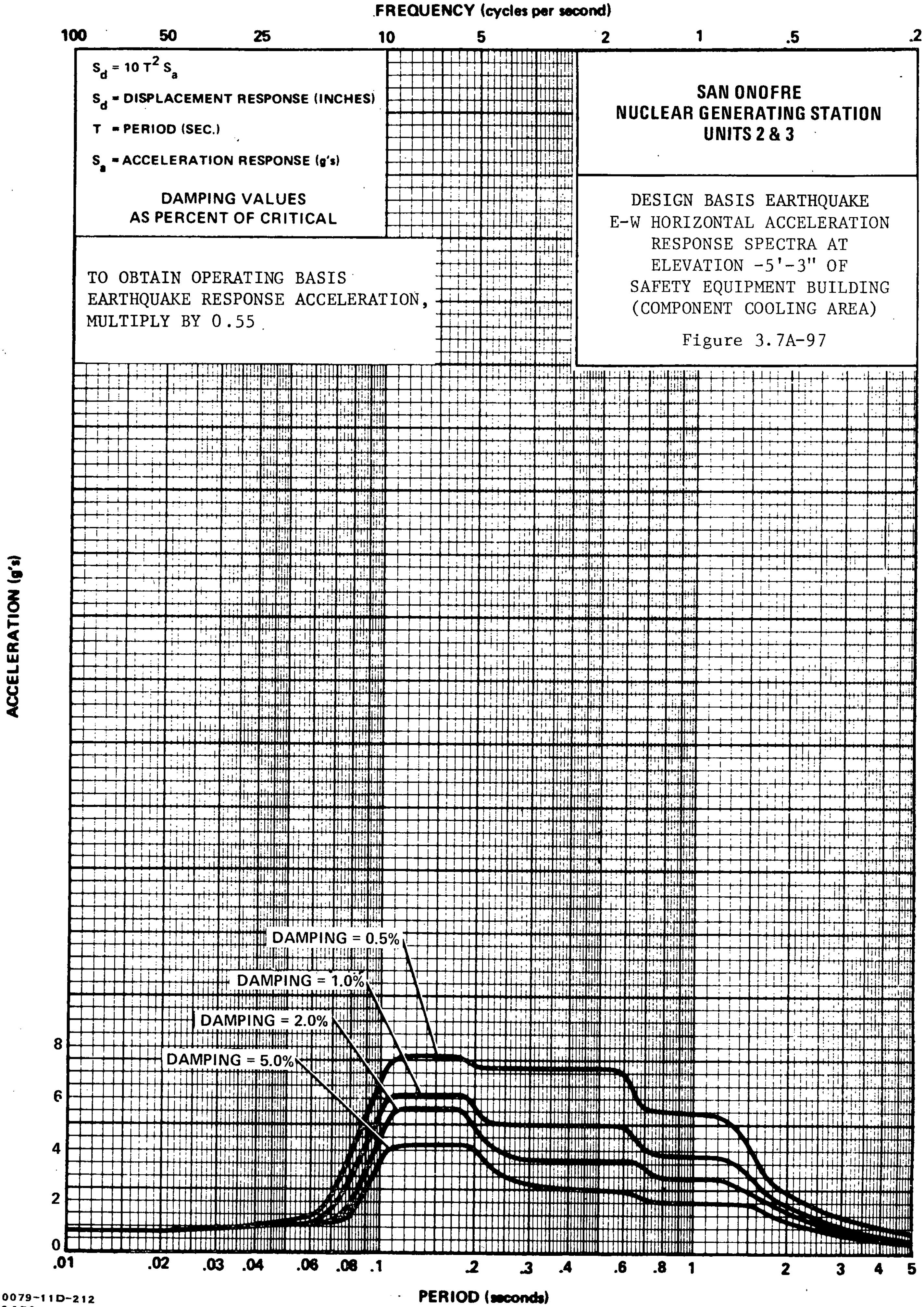
10079-11D-209
19AP6



10079-11D-210
19AP6



10079-11D-211
19AP6

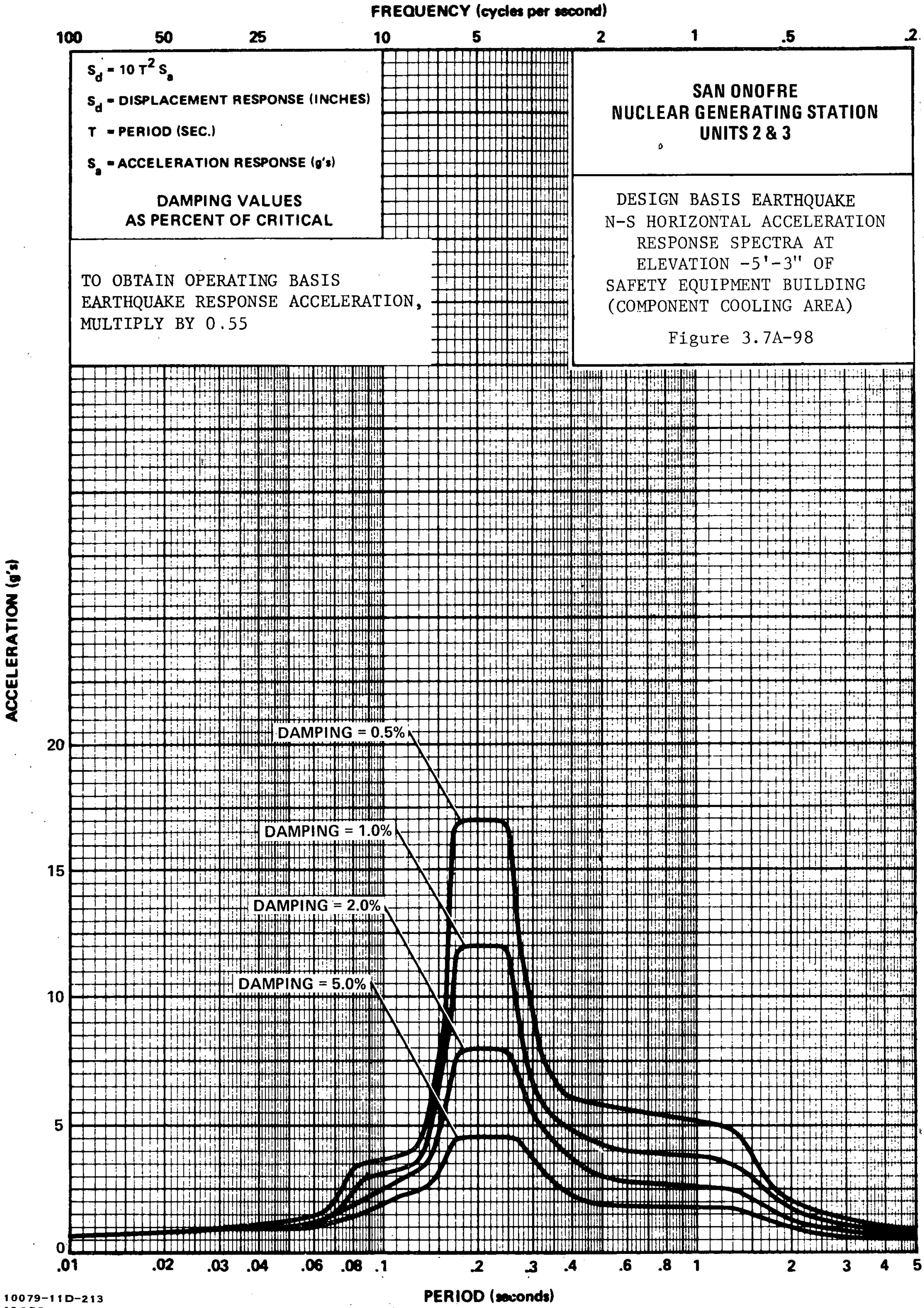


SAN ONOFRE NUCLEAR GENERATING STATION UNITS 2 & 3

DESIGN BASIS EARTHQUAKE E-W HORIZONTAL ACCELERATION RESPONSE SPECTRA AT ELEVATION -5'-3" OF SAFETY EQUIPMENT BUILDING (COMPONENT COOLING AREA)

Figure 3.7A-97

10079-11D-212
19AP6

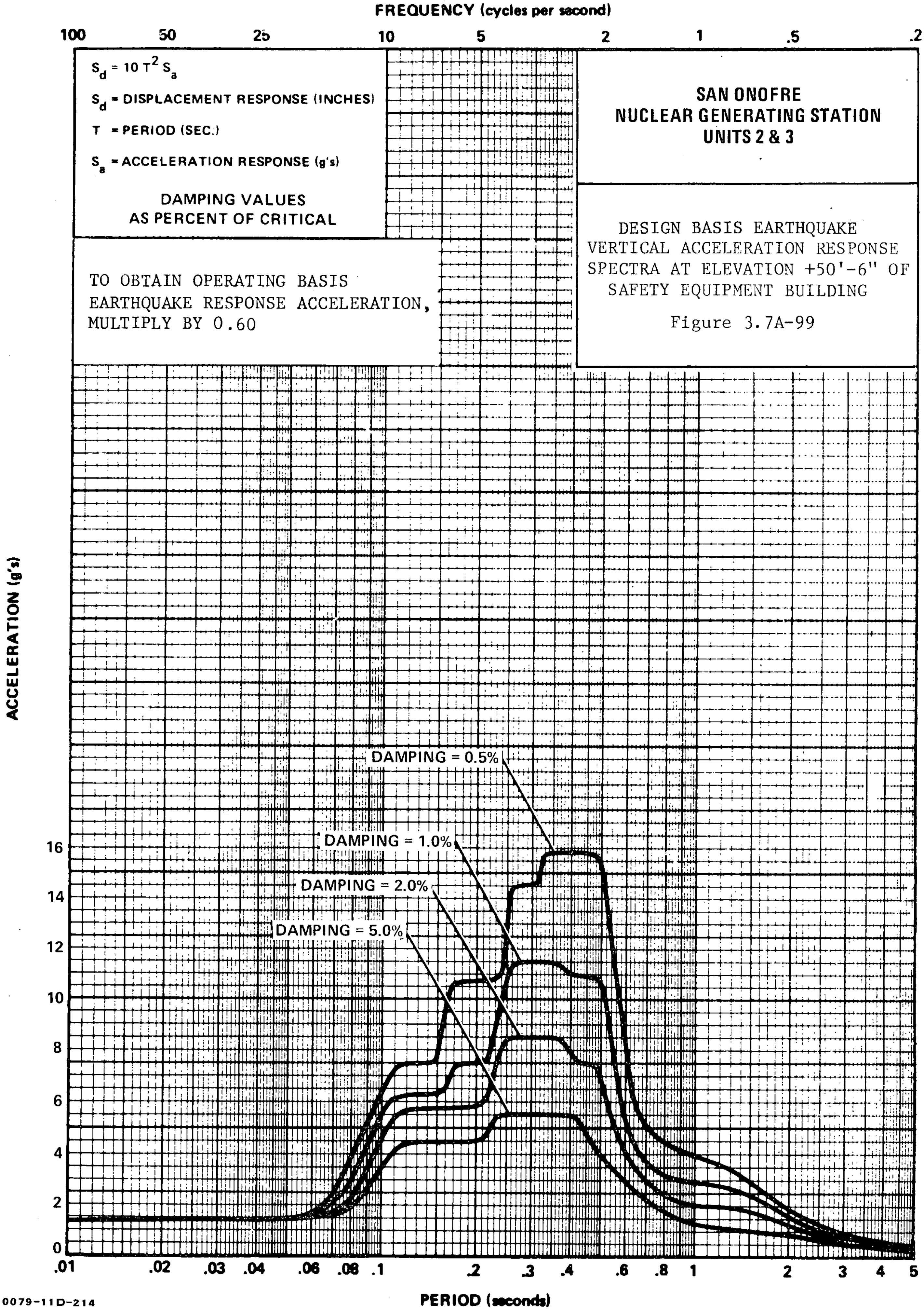


SAN ONOFRE
NUCLEAR GENERATING STATION
UNITS 2 & 3

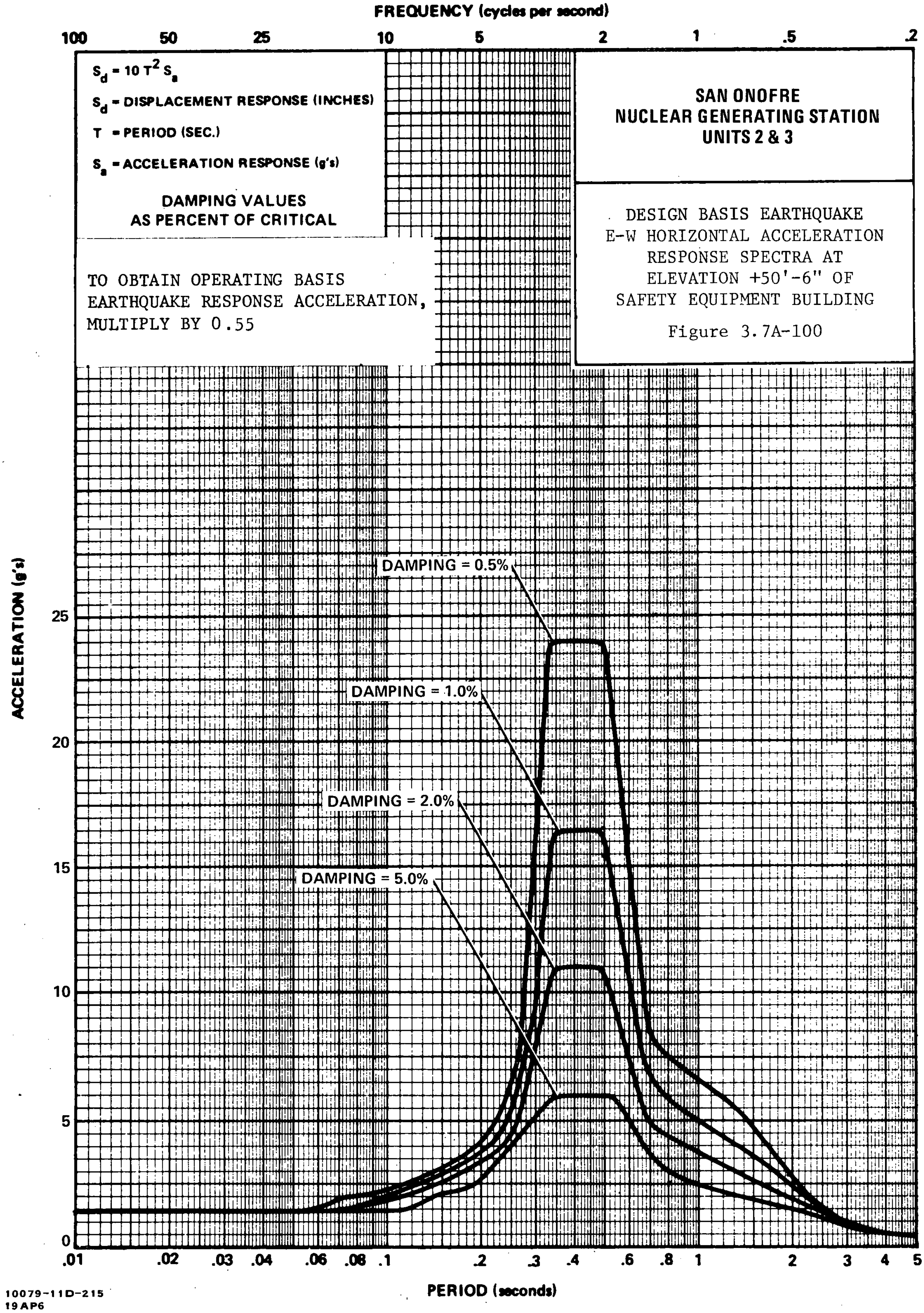
DESIGN BASIS EARTHQUAKE
N-S HORIZONTAL ACCELERATION
RESPONSE SPECTRA AT
ELEVATION -5'-3" OF
SAFETY EQUIPMENT BUILDING
(COMPONENT COOLING AREA)

Figure 3.7A-98

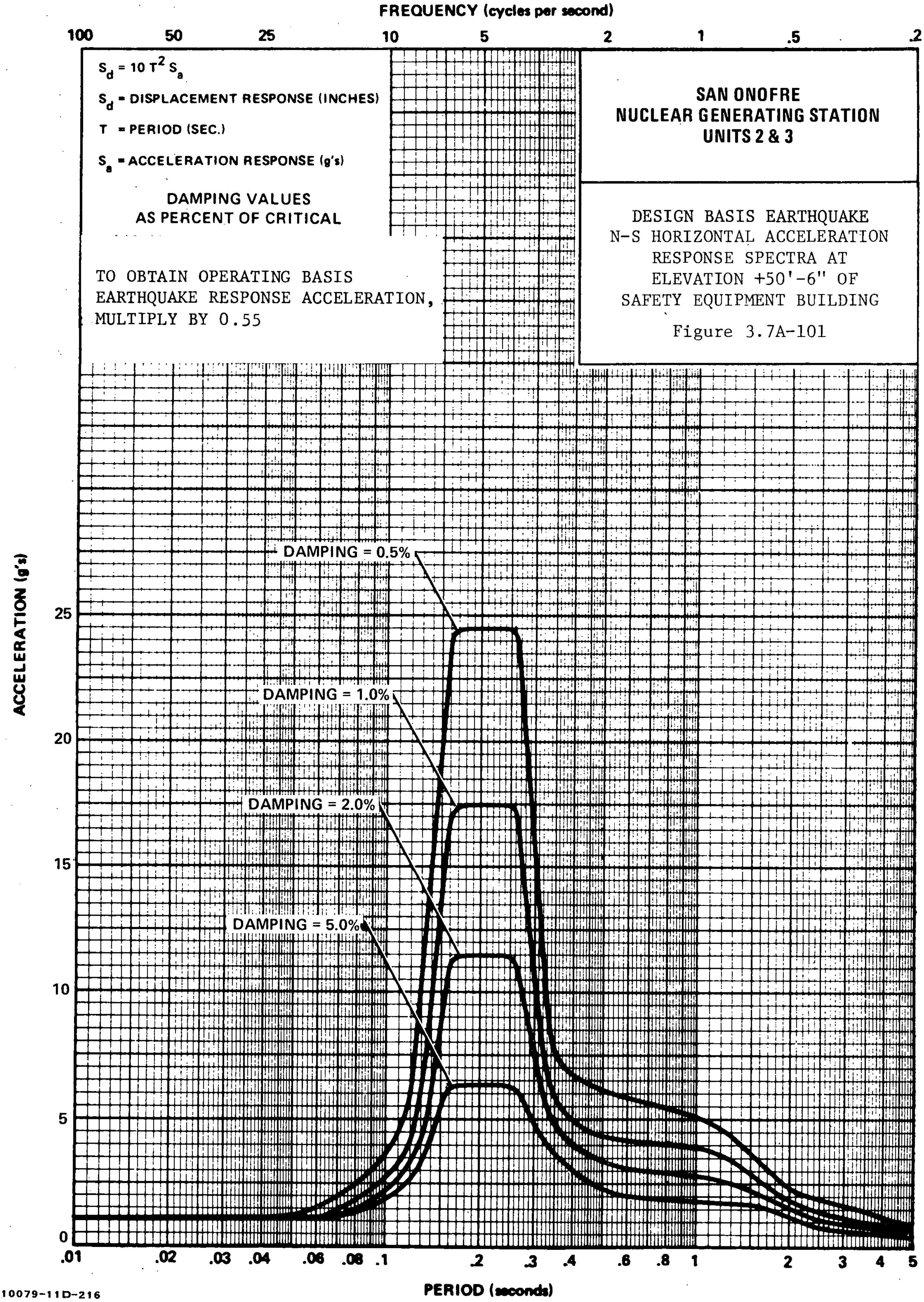
10079-11D-213
19AP6



10079-11D-214
19AP6



10079-11D-215
19AP6



SAN ONOFRE
NUCLEAR GENERATING STATION
UNITS 2 & 3

DESIGN BASIS EARTHQUAKE
N-S HORIZONTAL ACCELERATION
RESPONSE SPECTRA AT
ELEVATION +50'-6" OF
SAFETY EQUIPMENT BUILDING

Figure 3.7A-101

APPENDIX 3.7B

SEISMIC ANALYSIS OF PIPING SYSTEMS

APPENDIX 3.7B

CONTENTS

		Page
3.7B	SEISMIC ANALYSIS OF PIPING SYSTEMS	3.7B-1
3.7B.1	Introduction	3.7B-1
3.7B.2	Analytical Techniques	3.7B-1
3.7B.2.1	Summary	3.7B-1
3.7B.2.2	Rigid Piping	3.7B-2
3.7B.2.3	Flexible Piping	3.7B-2
3.7B.2.3.1	Dynamic Analysis	3.7B-2
3.7B.2.3.2	Equivalent Dynamic Analysis	3.7B-5
3.7B.2.4	Damping Ratio	3.7B-9
3.7B.2.5	General Guidelines	3.7B-9
3.7B.3	Modeling Techniques for Dynamic Analysis	3.7B-9
3.7B.3.1	Summary	3.7B-9
3.7B.3.2	Selection of Mass Points	3.7B-9
3.7B.3.3	Selection of Spectrum Curves	3.7B-10
3.7B.3.4	Interface of Seismic Category I and Other Piping Systems	3.7B-10
3.7B.4	Effect of Differential Building Movements	3.7B-11
3.7B.4.1	Summary	3.7B-11
3.7B.4.2	Effect on Piping Stresses	3.7B-11
3.7B.5	Modal and Directional Responses Combination	3.7B-11
3.7B.5.1	Summary	3.7B-11
3.7B.6	Cyclic Criteria	3.7B-12
3.7B.6.1	Summary	3.7B-12
3.7B.6.2	Nuclear Class I Cyclic Criteria	3.7B-12
3.7B.7	Computer Codes for Seismic Analysis	3.7B-12
3.7B.7.1	Seismic Analysis of Piping Systems	3.7B-12
3.7B.8	Verification of Simplified Approach	3.7B-12
3.7B.8.1	Fundamental Frequency	3.7B-12
3.7B.8.2	Evaluation of Calculation Methods	3.7B-13
3.7B.8.3	Comparison of Results of Dynamic Analysis and Modified Spectrum Method for a Typical Problem	3.7B-13
REFERENCES		3.7B-15

TABLES

		Page
3.7B-1	Sample Chart and Table for Modified Spectrum Method Reactor Building	3.7B-8

FIGURES

3.7B-1	Typical Modified Response Spectrum Curve
3.7B-2	Damping Values of Piping Systems
3.7B-3	Dynamic Analysis of Typical Piping System
3.7B-4	Response Spectrum Curve for Figure 3.7B-3

APPENDIX 3.7B

SEISMIC ANALYSIS OF PIPING SYSTEMS

3.7B.1 INTRODUCTION

In the event of a seismic occurrence near the San Onofre 2&3 power plant, the integrity of certain piping systems within the plant must be ensured. To that end, specific systems are classified as Seismic Category I and are analyzed to withstand the earthquake specified for the plant site.

Piping classified as Seismic Category I is designed to withstand levels of loading imposed by two hypothetical earthquakes: the design basis earthquake (DBE) and the operational basis earthquake (OBE). The criteria for each are delineated in this appendix.

Seismic Category I piping systems are those required to retain their integrity in the event of a DBE. These systems are necessary to:

A. Ensure the integrity of the reactor coolant pressure boundary

B. Shut down the reactor, and maintain it in a safe shutdown condition

C. Prevent or mitigate the consequences of accidents that could result in potential offsite exposures comparable to the guideline exposures of 10CFR100.

Analysis of buried piping is described in BC-TOP-4, section 6.

Section 3.7B.7 contains a cross reference of this appendix and the applicable sections of NRC "Standard Format and Content of Safety Analysis Reports for Nuclear Power Plants," October 1972.

Terms in the various equations given in the text are not assigned units for reasons of simplicity.

3.7B.2 ANALYTICAL TECHNIQUES

3.7B.2.1 SUMMARY

Piping may be generally classified according to the dynamic response of the system. Systems are considered rigid if they are supported and restrained so as to cause the first mode of vibration to occur in the rigid range of the response spectrum curve (see paragraph 3.7B.2.2 for definition). All other piping is considered flexible. General guidelines can be stated which relate pipe diameter to analytical technique.

3.7B.2.2 RIGID PIPING

The rigid range of the response spectrum curve is loosely defined as that portion in which there is no significant change in spectral acceleration with increasing frequencies (see point "A" on figure 3.7B-1). If piping is supported and restrained so that the first mode of vibration occurs in this range, it is classified as rigid.

Rigid piping systems are analyzed with static equivalent loads corresponding to the acceleration in the rigid range of the response spectrum curves for the applicable floor elevations. Both horizontal and vertical static equivalent loads are applied to the rigid piping systems. The response of the component for two horizontal and one vertical directions are combined on a square root of the sum of squares (SRSS) basis. The stresses are then computed in accordance with "ASME Boiler and Pressure Vessel Code Section III - Nuclear Power Plant Components," hereafter referred to as ASME Section III. The rigid range is dependent on building response and as such will be determined on a case basis. The rigid range floor spectrum typically begins between 20 to 33 Hz.

Classification of a specific piping system may be made in either of the following ways:

A. Restraints may be located such that no span between rigid restraints exceeds the length of a simply supported beam with a rigid range frequency. In addition, restraints are located at changes in direction, concentrated masses, and extended masses.

B. A dynamic analysis may be run to obtain the mode shapes of the piping system. If the first mode frequency is found to be in the rigid range, the system can be assumed rigid.

3.7B.2.3 FLEXIBLE PIPING

Piping that cannot be classified as rigid by the method defined above is assumed to be flexible and the analytical technique incorporates consideration of pipe natural frequencies in addition to the fundamental frequency.

3.7B.2.3.1 Dynamic Analysis

The dynamic analysis of flexible piping systems is performed using the response spectrum method. A flexible piping system is idealized as a mathematical model consisting of lumped masses connected by massless elastic members. The lumped masses are carefully located so as to adequately represent the dynamic and elastic properties of the piping system (see section 3.7B.3.2). The three-dimensional stiffness matrix of the mathematical model is determined by the direct stiffness method. Axial, shear, flexural, and torsional deformations of each member are included. For curved members a decreased stiffness is used in accordance with ASME Section III. The mass matrix is also calculated.

After the stiffness and mass matrix of the mathematical model are calculated, the natural frequencies of piping system and corresponding mode shapes are determined using the following equation:

$$\left(\underline{K} - W_n^2\underline{M}\right)\phi_n = \underline{0} \qquad (2\text{-}1)$$

where

$\underline{K}$ = stiffness matrix

W_n = natural circular frequency for the nth mode

$\underline{M}$ = mass matrix

$\underline{\phi}_n$ = mode shape matrix for the nth mode

$\underline{0}$ = zero matrix.

The Givens or the Jacobi method is used in the solution of the above equation. The mode shapes are normalized as follows:

$$\underline{\phi}_n^t \, \underline{M} \, \underline{\phi}_n = 1 \qquad (2\text{-}2)$$

A generalized mass matrix is calculated, and should correspond to:

$$\underline{\phi}^t \, \underline{M} \, \underline{\phi} = \underline{I} \qquad (2\text{-}3)$$

where

$\underline{\phi}$ = matrix of mode shapes

$\underline{\phi}^t$ = transpose of $\underline{\phi}$

$\underline{I}$ = identity matrix.

If any one of the off diagonal terms in the generation of the left hand side of equation (2-3) is greater than 1×10^{-4}, the problem is aborted. This occurs when poor or improper modeling of the piping system exists.

The response spectrum method is then used to find the maximum response of each mode:

$$Y_n\,(t)_{max} = \frac{\underline{\phi}_n^t\,\underline{M}\,\underline{D}\,Sa_n}{W_n^2\,M_n} \qquad (2\text{-}4)$$

where

Sa_n = spectral acceleration value for the nth mode

$\underline{D}$ = earthquake vector matrix, used to introduce earthquake direction to the response analysis

$\underline{\phi}_n^t$ = transpose of the nth mode shape

M_n = generalized mass of the nth mode; equals one by equation (2-2)

Y_n = generalized coordinate for the nth mode.

Using the maximum generalized coordinate for each mode, the maximum displacements associated with each mode are calculated:

$$\underline{V}_n = \underline{\phi}_n\,Y_n\,(t)_{max} \qquad (2\text{-}5)$$

Once the appropriate maximum modal displacements have been determined for each mass point, the effective inertia forces for each mode are computed:

$$\underline{Q}_n = \underline{K}\,\underline{V}_n \qquad (2\text{-}6)$$

where

$\underline{Q}_n$ = effective inertia force matrix due to nth mode

$\underline{V}_n$ = displacement matrix due to nth mode.

The effective acceleration for each mode is calculated:

$$\underline{a}_n = \underline{M}^{-1}\,\underline{Q}_n \qquad (2\text{-}7)$$

where

a_n = effective acceleration matrix due to nth mode

$\underline{M}^{-1}$ = the inverse of mass matrix.

After the effective inertia forces have been determined, the internal forces and moments for each mode are also calculated:

$$\underline{S}_n = \underline{b}\ \underline{Q}_n \qquad (2\text{-}8)$$

where

$\underline{S}_n$ = internal force and moment matrix due to the nth mode

$\underline{b}$ = force transformation matrix.

The modal stresses are then calculated from the modal internal forces and moments in accordance with ASME Section III. The analysis is made three times: once for the vertical direction, and once for each of the two principal horizontal directions of the building. The method of combining the modal responses (i.e., displacements, effective inertia forces, effective accelerations, internal forces and moments, support reactions and stresses) and the responses due to three (one vertical and two horizontal) directions is described in section 3.7B.5.

3.7B.2.3.2 Equivalent Dynamic Analysis

As differentiated from the dynamic analysis described in paragraph 3.7B.2.3.1, which produces a unique analysis for each piping system, this approach results in charts and tables for each site showing span lengths and restraint forces for pipes at various building elevations.

Since the technique uses a modified response spectrum curve, the method uses the same title.

3.7B.2.3.2.1 Modified Spectrum Method

A piping system may be considered seismically acceptable in accordance with ASME Section III, if it can be divided into a series of spans. These spans are limited by guides: two mutual perpendicular restraints normal to the pipe at each change of direction, at all concentrated masses (e.g., valves), at all extended masses, at each tee, and at a maximum spacing on straight runs of piping determined by dynamic calculations based on a modified spectrum curve.

The spectrum curve for a particular building elevation is modified so that the flexible side of the peak of the curve remains constant at the peak spectral acceleration for decreasing frequencies (see figure 3.7B-1).

If a dynamic analysis were performed using the above spectrum, the results would, by inspection, be conservative. The fundamental frequency of the piping system, supported as stated above, is greater-than-or-equal-to the fundamental frequency of a simply supported beam of maximum seismic span which is calculated as follows (see section 3.7B.10):

$$f = \frac{\pi}{2}\sqrt{\frac{EI}{mL^4}} \qquad (2\text{-}9)$$

where

f = fundamental frequency

E = modulus of elasticity

I = moment of inertia

m = mass per unit length

L = maximum seismic span (maximum distance between two seismic guides).

The justification of this approach, as well as a study demonstrating conservatism by comparing results of this approach with a dynamic analysis of a typical piping system, is presented in section 3.7B.10.

The following is a description of the development of the modified spectrum method.

The circular frequency of a simply supported beam is calculated:

$$W_n = (n\pi)^2\sqrt{\frac{EI}{mL^4}} \qquad (2\text{-}10)$$

where W_n = natural circular frequency for the nth mode.

The response spectrum method is then used to find the maximum response of each mode:

$$V_{2n-1} = \frac{4mSa_{2n-1}}{\pi(2n-1)\,W_{2n-1}^2}, \quad V_{2n} = 0 \qquad (2\text{-}11)$$

where

Sa_n = spectral acceleration value of the modified spectrum curve for the nth mode

V_n = maximum displacement (at midspan) due to nth mode

n = 1,2,3 ... etc.

The maximum internal moment (at mid span) and the maximum restraint force (at the support) are determined:

$$M_{2n-1} = \frac{4mL^2\ Sa_{2n-1}}{\pi^3\ (2n-1)^3},\quad M_{2n} = 0 \tag{2-12}$$

$$R_{2n-1} = \frac{4mL\ Sa_{2n-1}}{\pi^2\ (2n-1)^2},\quad R_{2n} = 0 \tag{2-13}$$

where

M_n = maximum internal moment (at mid-span) due to the nth mode

R_n = maximum restraint force (at the support) due to the nth mode.

The modal displacements, the modal internal moments and restraint forces are combined by the square root of the sum of squares (SRSS) method.

From the nature of the modified spectrum curve, the spectral acceleration for the first mode is always the largest value of the spectral accelerations of any mode. The first mode frequency for a given span is calculated and the resultant spectral acceleration is obtained. This maximum acceleration is then applied to all higher modes giving conservative results. The variables in equations (2-11), (2-12), and (2-13) can then be eliminated and the equations reduced to:

$$V = \frac{0.0131\ mL^4 Sa}{EI} \tag{2-14}$$

$$M = 0.1291\ mL^2 Sa \tag{2-15}$$

$$R = 0.8164\ mLSa \tag{2-16}$$

where

- Sa = spectral acceleration of modified spectrum curve corresponds to the fundamental frequency of the maximum seismic span
- V = maximum displacement
- M = maximum internal moment
- R = maximum restraint force.

The analysis is made for two horizontal and one vertical excitation. The horizontal and vertical responses are then combined on the SRSS basis.

3.7B.2.3.2.2 Sample Chart and Table for Modified Spectrum Method

A sample chart and table for the modified spectrum method is given in table 3.7B-1.

Table 3.7B-1
SAMPLE CHART AND TABLE FOR MODIFIED SPECTRUM METHOD
REACTOR BUILDING

Pipe Size (in.)	Below El 195'-0"	Below El 165'-0"	Below El 135'-0"	
1/2	4'-6"	4'-9"	5'-0"	Span
	5.0 lb	5.0 lb	4.5 lb	Load/support
3/4	5'-0"	5'-6"	5'-6"	Span
	8.0 lb	7.0 lb	7.0 lb	Load/support
1	6'-0"	6'-6"	6'-9"	Span
	9.0 lb	8.0 lb	8.0 lb	Load/support
1-1/2	7'-0"	7'-3"	7'-6"	Span
	20.0 lb	20 lb	18 lb	Load/support
2	9'-0"	9'-6"	10'-0"	Span
	50 lb	55 lb	58 lb	Load/support

3.7B.2.4 DAMPING RATIO

The damping ratio (percentage of critical damping) of piping systems employed in design and analysis is shown in figure 3.7B-2. The term, working stress, as used in defining the abscissa of figure 3.7B-2 is interpreted as the combined stress due to operating conditions and the respective seismic condition (OBE or DBE). The damping ratios are more conservative than those of Regulatory Guide 1.61.

3.7B.2.5 GENERAL GUIDELINES

Although it is difficult to categorize which analytical classification a specific piping system fits, certain generalizations can be made.

The major part of the larger diameter piping systems is analyzed using a full dynamic analysis. This is especially true where process fluid temperatures are high. This also applies to small-diameter high-temperature systems. For piping ≤2-inch diameter, wherein socket weld fittings are used, seismic acceptability was determined using modified spectrum methods. In these small piping systems, a curve derived from dynamic analysis has been used to determine equivalent straight spans for corner sections.

This distinction is necessary since the inherent conservatism in the other approaches described herein requires the addition of large numbers of restraints. The restraints could restrict normal thermal expansion. Therefore, a dynamic analysis is performed and snubbers are added as required.

Rigid-range piping techniques are typically reserved for instrumentation and some small-diameter piping. As previously stated, many conditions affect the selection of the appropriate technique. For example, a large-diameter cold operating system may be given a rigorous dynamic analysis to reduce the number of restraints required if the system is located so that installation of the restraints would be difficult.

3.7B.3 <u>MODELING TECHNIQUES FOR DYNAMIC ANALYSIS</u>

3.7B.3.1 SUMMARY

If a dynamic analysis is used to predict the actual response of a piping system to the specified forcing function, the dynamic model must adequately represent the system. This representation includes correct mass point selection to represent all significant modes, selection of the proper response spectrum curves, and proper location of anchors to separate Seismic Category I from non-Seismic Category I piping systems.

3.7B.3.2 SELECTION OF MASS POINTS

When a dynamic analysis is performed, a piping system is idealized as a mathematical model consisting of lumped masses connected by elastic members. The elastic members are given the properties of the piping system being analyzed. The lumped masses are carefully located to adequately represent the dynamic and elastic properties of the piping system. A lumped mass is located at the beginning and end of every elbow, valve, at the extended valve operator, and at the intersection of every tee. On straight runs, lumped masses are located at spacings no greater than the span length corresponding to 33 Hz. A mass point is located at every extended mass to account for torsional effects on the piping system. (The valve purchase specifications require the extended top works to have a fundamental frequency on the rigid side of the response spectrum curve.) In addition, the increased stiffness and mass of valves is considered in the modeling of a piping system.

3.7B.3.3 SELECTION OF SPECTRUM CURVES

In selecting the spectrum curve to be used for dynamic analysis of a particular piping system, a curve is chosen which will most closely describe the accelerations existing at the end points and restraints of the system. For a piping system spanning a large elevation difference within one structure, the worst single floor response spectrum is used as input to all supports. In cases of piping systems which go between different structures, a single response spectrum enveloping all spectral acceleration values is used.

3.7B.3.4 INTERFACE OF SEISMIC CATEGORY I AND OTHER PIPING SYSTEMS

In certain instances, Seismic Category I piping may be connected to non-Seismic Category I piping at locations other than a piece of equipment, which, for purposes of analysis, could be considered an anchor. These transition points typically occur at Seismic Category I valves. Since a dynamic analysis must be modeled from pipe anchor point to anchor point, two options exist:

A. Specify a structural anchor at the Seismic Category I valve and analyze the Category I system; or if impractical to design an anchor:

B. Analyze the system from the anchor point in the Seismic Category I system through the valve and to the first anchor point in the non-Seismic system.

Where small, non-Seismic Category I piping is directly attached to Seismic Category I piping, its effect on the Seismic Category I piping is accounted for by lumping a portion of its mass with the Seismic Category I piping at the point of attachment.

3.7B.4 EFFECT OF DIFFERENTIAL BUILDING MOVEMENTS

3.7B.4.1 SUMMARY

In most cases, piping systems are anchored and restrained to floors and walls of buildings that may have differential movements during a seismic event. These may range from insignificant differential displacements between rigid walls of a common building at low elevations to relatively large displacements between separate buildings at a high seismicity site.

3.7B.4.2 EFFECT ON PIPING STRESSES

Differential end-point or restraint deflections cause forces and moments to be induced into the piping system. The stress thus produced is a secondary stress. It is justifiable to place this stress, which results from restraint of free end displacement of the piping system, in the secondary stress category because the stresses are self-limiting, and when the stresses exceed yield strength, minor distortions or deformations within the piping system, satisfy the condition which caused the stress to occur.

This contribution of the earthquake produces a stress exhibiting properties much like a thermal expansion stress; and as such, a static analysis may be used to obtain actual stresses. The differential displacements are obtained from the dynamic analysis of the building. These displacements are applied to the piping anchors and restraints corresponding to the maximum differential displacements which could occur. The static analysis is made three times; once for one of the horizontal differential displacements, once for the other horizontal differential displacements, and once for the vertical.

3.7B.5 MODAL AND DIRECTIONAL RESPONSES COMBINATION

3.7B.5.1 SUMMARY

The basis for combining the modal responses (e.g., displacements, effective inertia forces and accelerations, internal forces and moments, support reactions, and stresses) as described in section 3.7B.2.3 is the square root of the sum of squares (SRSS). In order to obtain most conservative results, the three-directional (one vertical and two horizontal) responses obtained by the modal combination of each direction are then combined by the SRSS method.

After the total internal moments, support reactions, and stresses are obtained by combining the modal and directional internal moments, support reactions, and stresses, they are then combined with other loadings (e.g., thermal, weight, and pressure) in accordance with ASME Section III.

3.7B.6 CYCLIC CRITERIA

3.7B.6.1 SUMMARY

The number of cycles for a given load set must be obtained in the fatigue evaluation of ASME Section III, Nuclear Class I Piping.

3.7B.6.2 NUCLEAR CLASS I CYCLIC CRITERIA

To calculate the number of cycles, the following procedure is employed: the rigid range frequency limit (20 cps) is multiplied by the duration of strong shaking for the OBE to determine an upper bound of the number of cycles for a single occurrence of the OBE.

Two occurrences of the OBE are assumed over the plant life. The number of earthquake design cycles is equal to 40 times the duration (in seconds) of strong shaking for the OBE. The corresponding number of design cycles is given in the Design Specification.

The DBE condition is used, however, in the calculation of primary stresses (NB-3656 ASME Section III).

3.7B.7 COMPUTER CODES FOR SEISMIC ANALYSIS

3.7B.7.1 SEISMIC ANALYSIS OF PIPING SYSTEMS - ME632

The ME632 program, used in the seismic analysis of piping systems was developed by Bechtel International Corporation in San Francisco, California. Verification of computer program ME632 was performed after significant program changes. In the past, the verification was done by comparing results with an independent piping program used at EDS Nuclear, Incorporated. EDS Nuclear has verified computer program PISOLIA V. through benchmarks against STRESS (MIT, IBM) for stiffness computation, SAP (US Berkeley) and STARDYNE (CDC) for dynamic analyses and extensive hand calculations. Recently, verification was performed by using the ASME Pressure Vessel and Piping 1972 Computer Program benchmarks.

3.7B.8 VERIFICATION OF SIMPLIFIED APPROACH

3.7B.8.1 FUNDAMENTAL FREQUENCY

The verification of the fundamental frequency of a piping system restrained as outlined in paragraph 3.7B.2.3.2.1 is greater-than-or-equal-to the fundamental frequency of a beam with pin-connected ends (simply supported beam, SSB) of maximum seismic span (L).

The fundamental frequency of a multi-equal-span continuous beam is equal to the fundamental frequency of an SSB of the single span length of the continuous beam (see reference 1). For a multi-unequal-span continuous beam, the fundamental frequency of the maximum span using SSB formula is less than the fundamental frequency of the multi-unequal-span continuous beam. This can be easily proved by considering a three-span continuous beam. Suppose that the middle span is longer than two side spans: when the side spans are made smaller, the system approaches the fixed-fixed end case. Suppose that one of the side spans is the longest span: when the middle span is made smaller, the sytem is approaching the hinged-fixed end case. From the analytical results of a single span with various end conditions, it can be concluded that the SSB formula gives the smallest frequency value of the three cases (see references 1 and 2). The same argument can be applied to multi-span continuous beam. Therefore, the fundamental frequency of a piping system restrained as described in paragraph 3.7B.2.3.2.1 is greater-than-or-equal-to the fundamental frequency of an SSB of the maximum seismic span (L).

3.7B.8.2 EVALUATION OF CALCULATION METHODS

The modified spectrum method is conservative for piping supported in accordance with paragraph 3.7B.2.3.2.1.

As developed in section 3.7B.10.1, the SSB results in a lower frequency than either the fixed-fixed end or the fixed-hinged end.

Owing to the characteristics of the modified spectrum curve (see figure 3.7B-1), the span with the lowest fundamental frequency will always have a spectral acceleration equal-to-or-greater-than spans with higher frequencies.

It can be demonstrated by the same techniques as used in paragraph 3.7B.2.3.2.1 that both the fixed-fixed end and fixed-hinged end cases result in a lower dynamic response than for the SSB.

Dynamic analyses were done for five, six, and seven equal-span beams, and in all cases the stresses resulting were smaller than those obtained when the simple beam formula is used.

See section 3.7B.10.3 for a comparison with a dynamic analysis.

3.7B.8.3 COMPARISON OF RESULTS OF DYNAMIC ANALYSIS AND MODIFIED SPECTRUM METHOD FOR A TYPICAL PROBLEM

A piping system (see figure 3.7B-3) has been analyzed through the dynamic analysis using response spectrum method (see figure 3.7B-4 for the response spectrum curve). The following results have shown that the modified spectrum method described in paragraph 3.7B.2.3.2.1 yields a very conservative result:

	Dynamic Analysis	Modified Spectrum Method (Section 2.3.2.2)
Fundamental frequency (Hz)	11.05	6.47
Maximum stress (lb/in.^{2})	1300.00	5703.00
Maximum displacement (in.)	0.05	0.438
Maximum reaction (lb)	16.00	22.00

This typical example analyzed through both dynamic analysis and modified spectrum method (see figure 3.7B-2) has:

No. of degrees of freedom = 112

No. of modes considered in dynamic analysis = 20

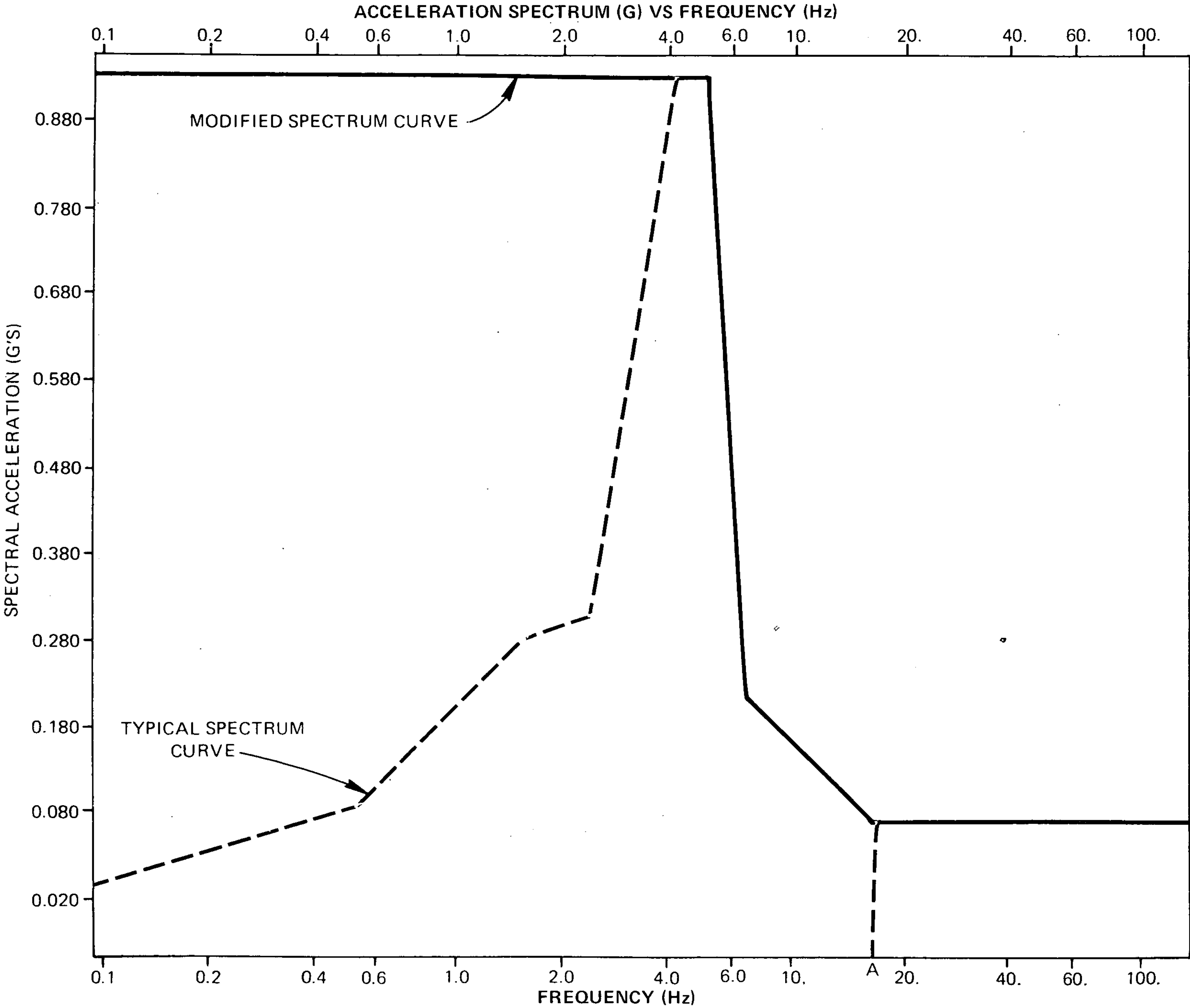
Least significant period = 0.03 sec.

REFERENCES

1. Biggs, J. M., Introduction to Structural Dynamics, McGraw-Hill, New York, N.Y.

2. Hurty, W. C. and Rubinstein, M. F., Dynamics of Structures, Prentice-Hall, Inc., Englewood Cliffs, N.J.

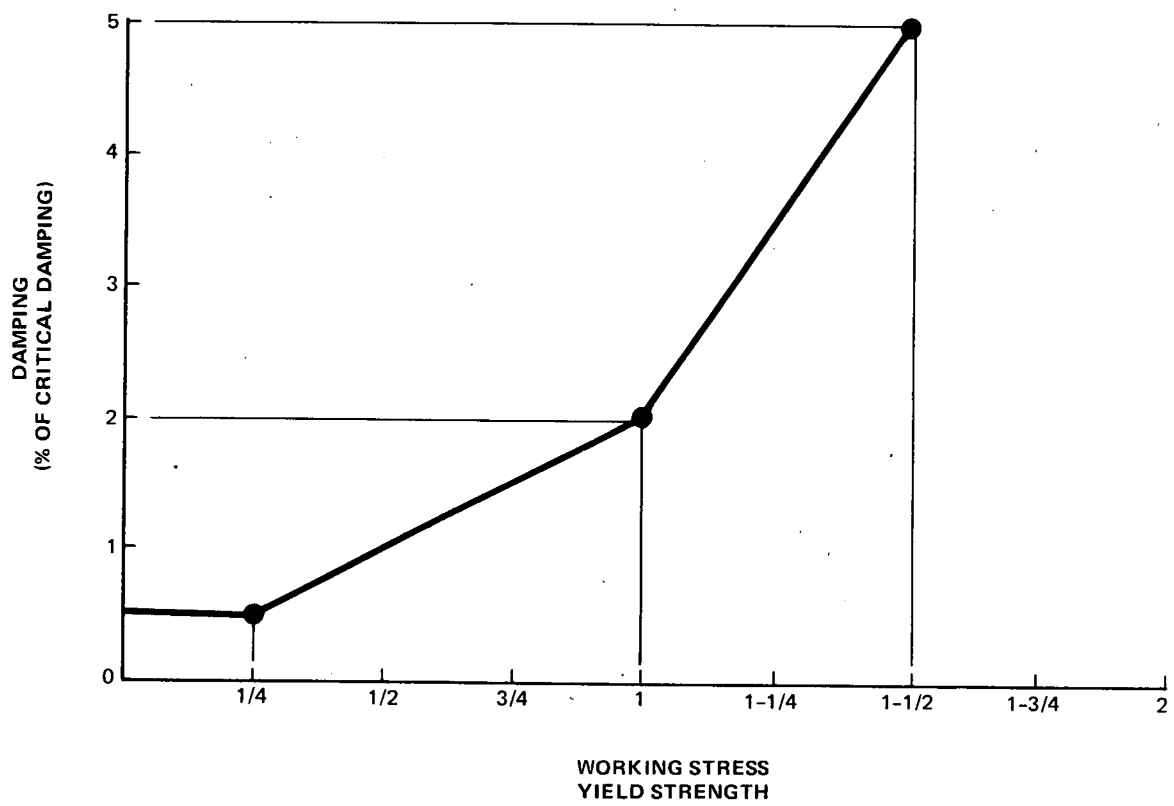
10079-11E-377
18JN6



SAN ONOFRE NUCLEAR GENERATING STATION
Units 2 & 3

TYPICAL MODIFIED RESPONSE SPECTRUM CURVE

Figure 3.7B-1



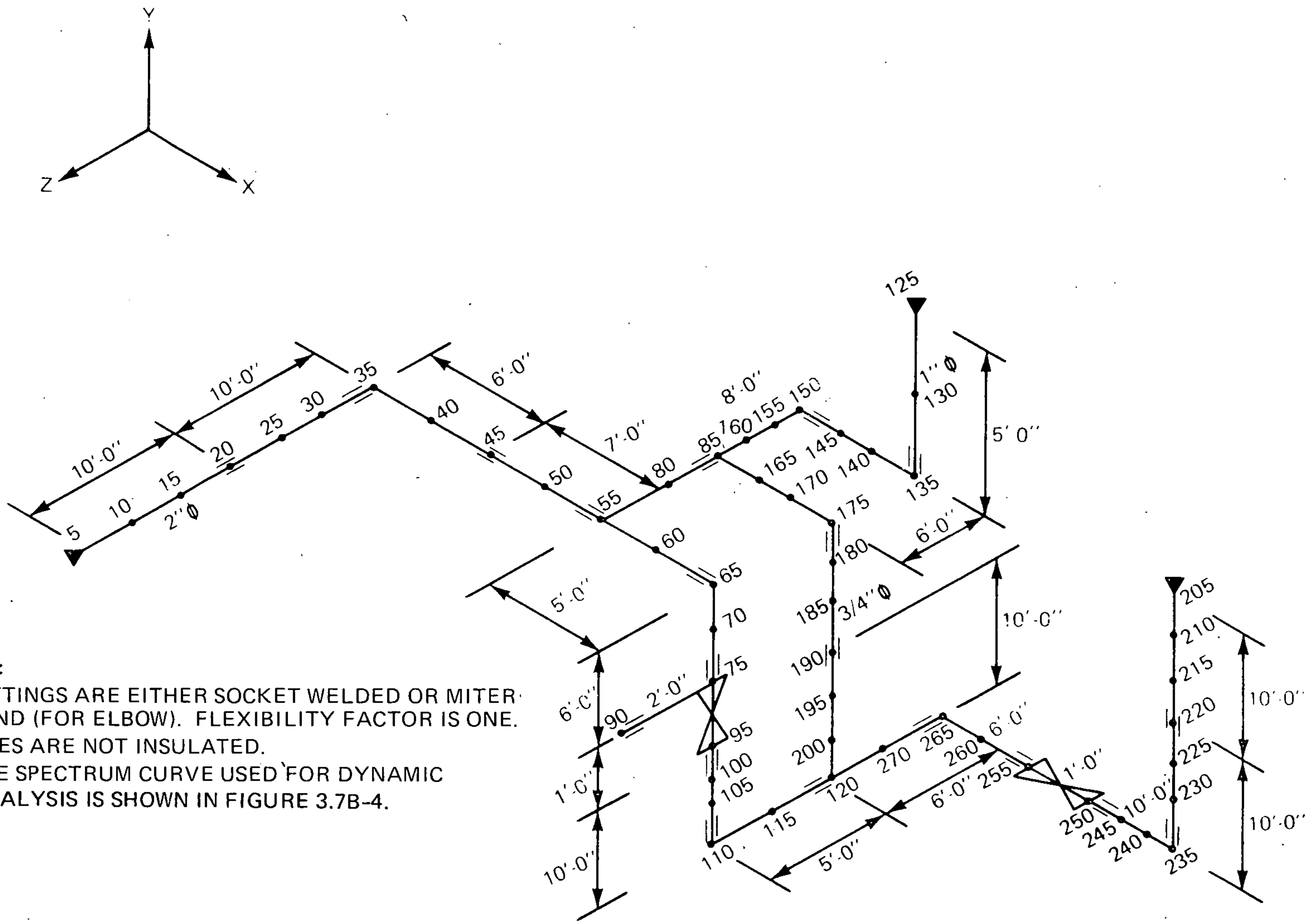
SAN ONOFRE
NUCLEAR GENERATING STATION
Units 2 & 3

DAMPING VALUES
OF PIPING SYSTEMS

Figure 3.7B-2

10079-11D-895
12OC6

10079-11E-378
110C6



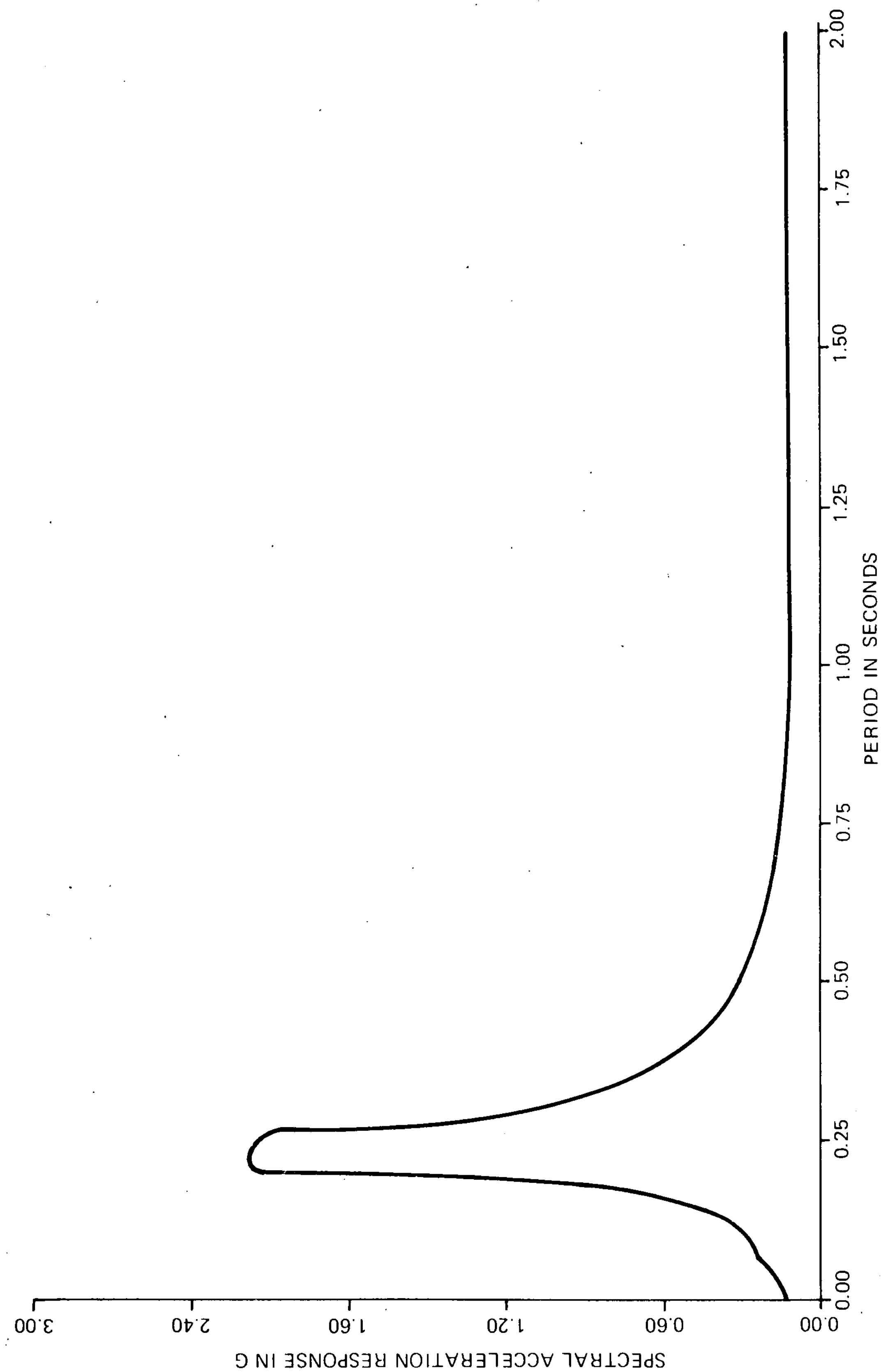
NOTE:

1. FITTINGS ARE EITHER SOCKET WELDED OR MITER BEND (FOR ELBOW). FLEXIBILITY FACTOR IS ONE.
2. PIPES ARE NOT INSULATED.
3. THE SPECTRUM CURVE USED FOR DYNAMIC ANALYSIS IS SHOWN IN FIGURE 3.7B-4.

SAN ONOFRE
NUCLEAR GENERATING STATION
Units 2 & 3

DYNAMIC ANALYSIS OF TYPICAL PIPING SYSTEM

Figure 3.7B-3



SAN ONOFRE
NUCLEAR GENERATING STATION
Units 2 & 3

RESPONSE SPECTRUM CURVE
FOR FIGURE 3.7B-3

Figure 3.7B-4

10079-11E-379
11OC6

APPENDIX 3.7C

SOIL-STRUCTURE INTERACTION PARAMETERS

APPENDIX 3.7C

SOIL-STRUCTURE INTERACTION PARAMETERS

DEVELOPMENT OF SOIL-STRUCTURE
INTERACTION PARAMETERS
PROPOSED UNITS 2 AND 3
SAN ONOFRE GENERATING STATION
SAN ONOFRE, CALIFORNIA

for

Southern California Edison Company
P. O. Box 800
Rosemead, California 91770

by

WOODWARD-McNEILL & ASSOCIATES
Consulting Engineers and Geologists

31 January 1971
(Final Revision)

APPENDIX 3.7C

TABLE OF CONTENTS

	Page
1.0 INTRODUCTION	1
1.1 Scope	1
1.2 Organization of Report	2
2.0 LABORATORY AND FIELD TESTING	2
2.1 Laboratory Testing	3
2.2 Field Testing	3
2.2.1 General	3
2.2.2 Slab-Response Tests	4
2.2.3 Rayleigh-Wave Tests	5
2.2.4 Attenuation Tests	5
3.0 DISCUSSION AND CONCLUSIONS	5
3.1 Modulus and Damping Parameters	6
3.2 Soil Damping and Spring Constant	6
3.2.1 Soil Damping	7
3.2.2 Spring Constant	8
3.3 Lateral Pressures on Structure Walls	9
3.4 Evaluation of Structural Sliding	10
3.5 Evaluation of Critical Instantaneous Displacement Profile	11
4.0 SUMMARY OF SOIL-STRUCTURE INTERACTION PARAMETERS	12
4.1 Parameters for Dynamic Analyses	12
4.2 Static Analyses Parameters	12
5.0 REVIEW OF THE USE OF RECOMMENDED PARAMETERS	13

TABLE I - DESIGN PARAMETERS

TABLE II - SUMMARY OF DETERMINATION OF STRESSES ON WALLS

TABLE III - CASES STUDIED FOR STRUCTURE SLIDING ANALYSES

FIGURE 1 - SITE PLAN

FIGURE 2 - MODULUS AND DAMPING VS. STRAIN, SAN MATEO FORMATION SAND

FIGURE 3 - INSTRUMENTATION

TABLE OF CONTENTS

FIGURE 4 - TESTS IN PROGRESS

FIGURE 5 - MAXIMUM SLAB RESPONSE MEASUREMENTS

FIGURE 6 - SOIL DAMPING FOR STRUCTURES (SCHEMATIC)

FIGURE 7 - SHEAR MODULUS DETERMINATION (SCHEMATIC)

FIGURE 8 - EFFECT OF EMBEDMENT ON SPRING CONSTANT

FIGURE 9 - LATERAL PRESSURES DUE TO ADJACENT STRUCTURE

FIGURE 10 - EXAMPLE OF STRUCTURAL SLIDING RESULTS

FIGURE 11 - CRITICAL DISPLACEMENT PROFILE FOR OFFSHORE CONDUIT EXTENDING WEST OF PERMANENT SEA WALL

FIGURE 12 - OBE & DBE INDUCED STRAINS

FIGURE 13 - ESTIMATED SETTLEMENT FOR FOUNDATIONS BEARING ON SAN MATEO FORMATION SANDS

APPENDIX A - SLAB RESPONSE TESTS

APPENDIX B - RAYLEIGH WAVE TESTS

APPENDIX C - ATTENUATION TESTS

APPENDIX D - DAMPING

APPENDIX E - EVALUATION OF SPRING CONSTANT

APPENDIX F - LATERAL PRESSURES ON STRUCTURE WALLS

APPENDIX G - STRUCTURE SLIDING

APPENDIX H - EVALUATION OF CRITICAL INSTANTANEOUS DISPLACEMENT PROFILE

BIBLIOGRAPHY

DEVELOPMENT OF SOIL-STRUCTURE INTERACTION PARAMETERS, PROPOSED UNITS 2 AND 3 SAN ONOFRE GENERATING STATION SAN ONOFRE, CALIFORNIA

1.0 INTRODUCTION

1.1 Scope

As part of the geotechnical studies for the proposed expansion of the San Onofre Nuclear Generating Station (SONGS) for Units 2 and 3, this report presents the results of field testing and analyses used to develop soil-structure interaction parameters. The field tests were initiated to both confirm the laboratory-derived modulus and damping parameters and set the rules by which those parameters should be used in evaluating soil-structure interaction. The need for this extensive effort is realized when considering the requirements to supply soil-structure interaction parameters for state-of-the-art level structures analyses. These analyses are required to design the Seismic Category 1 structures to withstand the large Design Basis Earthquake (DBE) motions. The SONGS 2 and 3 Category 1 structures exhibit unusual shape and inertia characteristics for which there is no precedence available in the literature or design experience.

The present study can be separated into three basic areas to indicate the specific items considered in detail. These items are listed below:

- field tests - slab response
  R-wave
  attenuation

- lab tests - modulus and damping parameters from previous studies

- Analytical studies with field and laboratory test results as basic data to develop 5 aspects of soil-structure interaction

  - spring constants and modulus values
  - dampings (hysteretic and spatial)
  - structural sliding
  - most critical instantaneous displacement profile for structural design and interaction between structures
  - consideration of lateral stresses on structural walls

1.2 Organization of Report

The field and laboratory investigations including reference to previous, parallel work pertinent to this study are described in Section 2. Section 3 describes the five aspects of soil-structure interaction considered; Section 4 presents the general soil-structure interaction parameters developed for the SONGS 2 and 3 site. Finally, Section 5 summarizes final review procedures of how parameters are utilized.

The necessary back-up data for the sections described above are assembled in the appendices, as follows:

Appendix A	Slab Response Tests
Appendix B	Rayleigh Wave Tests
Appendix C	Attenuation Tests
Appendix D	Evaluation of Damping
Appendix E	Evaluation of Spring Constant
Appendix F	Evaluation of Stresses on Walls
Appendix G	Evaluation of Structure Sliding
Appendix H	Evaluation of Critical Instantaneous Displacement Profile

2.0 LABORATORY AND FIELD TESTING

The data upon which the analyses reported here are based were derived from both laboratory and field testing. Because all Category 1 structures are to be founded on San Mateo Formation Sand, all tests were limited to that material. The laboratory testing was reported in a previous study (Ref. 4) on modulus and damping parameters. The field tests, including slab response measurements were conducted at the site (Fig. 1) and are presented in this report. The paragraphs which follow discuss both the laboratory and field testing.

2.1 Laboratory Testing

The results of tests performed to develop strain-dependent modulus and damping parameters are presented in the 14 October 1971 report entitled, *Elastic and Damping Properties, Laydown Area, San Onofre Nuclear Generating Station* (Ref. 4). The data presented in that report resulted in the development of the strain-dependent modulus and damping curves presented in Fig. 2. These relationships were verified by the field attenuation and Rayleigh-wave tests described below and were used in the detailed analyses of the slab response tests.

2.2 Field Testing

2.2.1 General

All field tests were performed in the Unit 1 laydown area as located on the Site Plan, Fig. 1, due to the flat working area and easy access to the exposed native San Mateo Sand material. Testing was completed between 5 and 12 September 1972, and included a series of three types of tests: slab-response tests, Rayleigh-wave tests, and attenuation tests. These tests are described in detail below.

2.2.2 *Slab-Response Tests*

The slab-response tests were conducted by setting five concrete slabs of different size, shape, and embedment configurations into transient motion and measuring the intensity and the decay of the response motion. The test slabs ranged from 4 to 10 ft in diameter and 2 to 5 ft in thickness. The sizes and shapes were chosen to evaluate the effects of geometry, scaling and embedment on response. Response measurements were made with velocity-sensing geophones. A typical instrumentation layout is shown in the photograph in Fig. 3a. The slabs were set into transient motion by tensioning a cable and weak link with a crane or tractor until the weak link failed. The intensity of response was controlled by using weak links with tensile load capacities varying from 3 to 16 kips as shown on Fig. 3b. Typical vertical and horizontal response tests using a crane and a tractor are indicated on Figs. 4a and 4b, respectively. For the 105 tests performed, response of the slabs generally ranged from accelerations of 0.2 to 1.0 times the acceleration of gravity over a frequency range of 10 to 100 cycles per second, as indicated from peak response points plotted in Fig. 5. A description of the details of the slab response tests and the test results are presented in Appendix A. Basically, the stiffness was evaluated by the response frequency, and the damping by the decay of the response motion.

2.2.3 *Rayleigh-Wave Tests*

Rayleigh-wave traverses were performed in two mutually perpendicular, 5-ft deep and about 100-ft long trenches. These tests consisted of measuring the wave length of vibratory input motion at various frequencies. The Rayleigh-wave velocity was calculated from the measured wave length and input frequency and is essentially equal to shear-wave velocity. The measurements were carried out to verify earlier measurements of near-surface shear-wave velocity and estimates of low-strain level shear modulus. The details of Rayleigh-wave tests and the test results are presented in Appendix B.

2.2.4 *Attenuation Tests*

An attenuation test was performed using a vibrating sheepsfoot roller as a source of vibratory energy and monitoring the motion simultaneously at two distances from this source. These measurements yield the hysteretic damping of the soil and were used to verify the laboratory-determined relationship between damping and strain. The details of this test and the test results are presented in Appendix C.

3.0 DISCUSSION AND CONCLUSIONS

The purpose of all testing completed during the present investigation was twofold: (1) to verify the previously determined damping and modulus parameters; and (2) to develop the soil-structure interaction parameters. The paragraphs which follow discuss the verification and the development of the soil-structure interaction parameters.

3.1 Modulus and Damping Parameters

The relationship between modulus and strain presented on Fig. 2 was developed from a combination of dynamic laboratory tests and field seismic tests. The verification of the low-strain level value of modulus near the ground surface was of primary concern in the present study. This was accomplished by the measurement of the Rayleigh-wave velocity described above and in Appendix B. Results of the test are presented in Appendix B. They indicate a range of Rayleigh-wave velocities between 850 and 1200 fps, with a velocity of 930 fps as a representative average for the near-surface (upper 15 ft) soils. This range in values is consistent with the shear-wave velocity used to develop the modulus curve on Fig. 2.

The relationship between hysteretic damping and strain presented in Fig. 2 was developed entirely from dynamic laboratory testing. The field verification of this relationship was done by performing attenuation tests in the field as described above and in Appendix C. A comparison of the field results to the laboratory determined curve, as presented in Appendix D, indicates good agreement between the two.

3.2 Soil Damping and Spring Constant

The development of soil damping and spring constant parameters for structures was necessary for the response evaluation of structures at the SONGS 2 and 3 sites. These parameters were developed from a combination of the slab response tests and the strain-dependent modulus and damping curves (Fig. 2) as discussed below.

3.2.1 *Soil Damping*

There are basically four tasks involved in the evaluation of soil damping: (1) determination of the manner in which to combine hysteretic and spatial damping; (2) evaluation of the effective radius of the foundation which depends on stress distribution; (3) evaluation of the effect of embedment of the foundation; and (4) evaluation of shape and scaling effects. The details of analyses of field data to evaluate these tasks are presented in Appendix D. In brief, it was found that for the soil conditions at the SONGS 2 and 3 site: (1) total damping could be determined by adding the spatial and hysteretic damping algebraically; (2) an effective radius of about 60% of the actual radius (corresponds to parabolic stress distribution) could be utilized in the theoretical equations to determine a conservative value of geometrical damping for the translational modes (horizontal and vertical) while 80% (corresponds to uniform stress distribution) could be used for rotational modes (rocking and twisting); (3) the effects of embedment are negligible on the amount of damping; and (4) the effects of scaling and shapes (for regular shapes like square or circular) were negligible on the amount of damping.

The general methods of obtaining damping are summarized on Fig. 6 with the general equations included in Table I.

It is noted that the strain for which the hysteretic damping is calculated should be the seismic induced free-field strain for OBE and DBE analysis, as it would likely dominate over the local strains caused by response of the structure. This should be checked by the method to evaluate strain compatibility suggested in Section 4.2.

3.2.2 Spring Constant

There are basically four tasks involved in the evaluation of spring constants: (1) determination of the confinement to be used in the calculation of modulus; (2) evaluation of the effective radius (stress distribution); (3) evaluation of the effect of embedment; and (4) evaluation of the shape and scaling effects. Slab-response tests were carried out in the field to respond to these tasks. Details of analysis-of field data, obtained from these tests, to evaluate these tasks are presented in Appendix E. Task-1 was answered by determining the shear modulus from the strain and confinement-dependent curve (Fig. 2), and then the other tasks were evaluated. If the results of the evaluation of task-2 were consistent with judgment, then the assumption could be considered reasonable. The evaluation of shear modulus for these tests was done by assuming the strain in the soil was accommodated within one radius below the test slab. Therefore, the strain was calculated as the measured response deflection divided by the radius of the foundation and the mean confinement was calculated at an average depth of one-half a radius below the foundation as indicated in Fig. 7. Tasks-3 and 4 were answered by field tests of slabs with different embedments and shapes. These tests are also described in Appendix E. In summary, using the task-1 assumption, it was found that: (1) the evaluation of effective radius leads to a correction factor on the theoretical equation consistent with uniform stress distribution (see Table I, factor C_1); (2) the effect of embedment was significant as is indicated on Fig. 8 (correction indicated as C_2); and (3) the effects of shape (for regular shapes) and scaling were negligible.

The general equations for obtaining spring constants are presented in Table I. Also, included in Table I are specific equations for the containment model which was studied separately due to its complex base geometry. As in the case of damping, the strain to be used in the actual structure model should be equal to the seismic induced free-field strain for OBE and DBE analyses. This should be checked by the method suggested in Section 4.2.

3.3 Lateral Pressures on Structure Walls

Three basic pressures must be considered in the evaluation of seismic-induced stresses on structure walls: (1) active pressure acting on the side of the structure tending to move the structure away from the soil; (2) developed passive pressure acting on the opposite side of the structure (due to inertial loads) tending to move the structure into the soil; and (3) pressures due to the proximity of adjacent structures. The details of analyses to determine these stresses are presented in Appendix F. A summary of the method of evaluation is presented in Table II with a schematic diagram indicating the effects of the proximity of the adjacent structures presented on Fig. 9. Because all sides of the structure could be subjected to both passive and active pressures, all walls should be designed for whichever analysis yields the highest pressure for each wall element as indicated on Table II.

3.4 Evaluation of Structural Sliding

During a large earthquake the horizontal forces developed due to inertial loads from large structures with shallow embedments may be significant and tend to cause structural sliding. The geometry of the Auxiliary Building makes it the most critical Category 1 structure from this standpoint. Therefore, a dynamic finite-element program was utilized to evaluate this problem by constructing the element mesh with very thin elements just below the structure and calculating a time-history of the ratio of the shear and normal stresses in that soil just below the structure to compare with the available frictional resistance. A series of cases were calculated considering various factors as summarized on Table III. The details of the analysis and the results of all cases are presented in Appendix G. The most critical case as indicated on Table III is reported as Fig. 10. For this case the stress ratio is always less than the available frictional resistance ($^2/_3$ tan ϕ) of 0.59; therefore, we conclude that the Auxiliary Building is safe against sliding for DBE-induced loading.

Based on the results of the analyses performed for the various cases summarized in Table III, several general conclusions can be drawn:

1. The phase relationship between the horizontal and vertical motion does not appear to influence sliding appreciably.

2. Inclusion of rotational inertia does not increase the sliding potential.

3. Sliding forces are largest at the center of the slab and decrease toward the edges.

4. The most critical combination of conditions involves a surface slab (i.e., no embedment) subjected to a combination of horizontal and vertical base motions.

3.5 Evaluation of Critical Instantaneous Displacement Profile

For structures with unusual geometry or small anticipated inertial loading the use of a seismic instantaneous displacement may be critical for design. The critical instantaneous displacement profile (CIDP) is defined as the deflected shape that the structure would assume at an instant during the earthquake which would cause maximum stresses in structural elements. The determination of the CIDP is made using a travelling shear wave finite-element computer program and evaluating the displacement profile along the base of structural elements in the finite-element mesh at every instant in time. The critical profile is found using maximum slope change across the profile (maximum bending) as a criterion. The details of the procedure are presented in Appendix H. An example of results is presented for the Intake Conduit in Fig. 11.

4.0 SUMMARY OF SOIL-STRUCTURE INTERACTION PARAMETERS

Interaction parameters for both static and dynamic analyses are summarized in the paragraphs which follow.

4.1 Parameters for Dynamic Analyses

Using the procedures outlined in the foregoing sections, the soil-structure interaction parameters can be developed for each Category 1 structure. The actual parameters will depend on the geometry, inertia and embedment of the structure together with the shear modulus and hysteretic damping of the soil. As noted in Fig. 2, the shear modulus and hysteretic damping of the soil are strain dependent. To determine the appropriate value of these parameters, it was necessary to determine the DBE and the OBE induced free-field strains. These strains were evaluated on the basis of finite element analyses as presented in Fig. 12.

4.2 Static Analyses Parameters

The denseness of the San Mateo Sand material indicates that a static spring model analysis is appropriate for the consideration of static deflections of the base mat. Further, by comparing dynamic modulus values for dense sands (Seed and Idriss, 1970) and static reloading modulus values for the same material (Kulhawy, Duncan and Seed, 1969) for the same strain level, it is found that there is essentially no difference. For this reason, it is concluded that the static analyses can be performed using the same stiffness parameters as calculated for dynamic analyses. Strain compatibility should be attained in performing the analysis as described in the following steps:

1. Choose initial value of modulus for the spring constant equation in Table I as appropriate.

2. Complete analysis calculating the maximum deflection of the foundation.

3. Calculate strain by dividing the deflection from Step 2 by the radius of the foundation.

4. Compare modulus from Fig. 2 corresponding to the strain from Step 3 to the modulus chosen in Step 1.

5. If the values of modulus are different, repeat Steps 1 through 4 until compatibility is attained.

It is important that the net bearing pressure be used as the applied load in calculating strain (i.e., total pressure at the structure subgrade minus pressure due to soil removed between adjacent grade and the structure subgrade). If the net pressure is zero or negative use K_m= 590 (see Fig. 2). For net pressures greater than 10 ksf (approximately equal to the initial overburden removed to attain plant grade at el. +30 ft) the amount of net pressure over 10 ksf should not be included in the calculation of σ_m. Parametric studies using these procedures yielded results for the evaluation of settlement of isolated footings for net loads of up to 10,000 kips and bearing pressures of up to 20 ksf as presented graphically on Fig. 13. These can be used in conjunction with the equation for net allowable bearing capacity as given on Fig. 13 for design of isolated footings. Evaluation of structures should be done by an acceptable mat deflection criterion using the techniques presented above.

5.0 REVIEW OF THE USE OF RECOMMENDED PARAMETERS

Due to the complexity of structures and of the soil structure interaction concepts, interpretation and simplifying assumptions must be made to utilize the general rules for design parameters.

For this reason it is imperative that a detailed review be made by this firm of how all parameters are calculated and used in the analyses completed by Bechtel Corporation. By mutual agreement between Southern California Edison Company, Bechtel Corporation, and Woodward-McNeill & Associates, this review is to consist of at least one working meeting for each major structural analysis with appropriate representation of each company, and should be ultimately followed up with written documentation.

TABLE I
DESIGN PARAMETERS

	Mode of Motion			
Parameters	VERTICAL TRANSLATION	HORIZONTAL TRANSLATION	ROCKING	TWISTING
Inertia	m, mass of foundation and machine	m, mass of foundation and machine	I_r, mass moment of inertia about rocking axis	I_t, mass moment of inertia about twist axis
Radius	$r = \sqrt{\frac{BL}{\pi}}$	$r = \sqrt{\frac{BL}{\pi}}$	$r = \sqrt[4]{\frac{BL^3}{3\pi}}$	$r = \sqrt[4]{\frac{BL(B^2 + L^2)}{6\pi}}$
Inertia Ratio	$B_v = \frac{(1-\nu)m}{4\rho r_e^3}$	$B_h = \frac{(7-8\nu)m}{32(1-\nu)\rho r_e^3}$	$B_r = \frac{3(1-\nu)I_r'}{8\rho r_e^5}$	$B_t = \frac{I_t}{\rho r_e^5}$
Effective Inertia for Design	m	m	$I'_r = \eta_r I_r$ See Appendix E for value of η_r	I_t
Stiffness Coefficient	$k_v = \frac{4GrC}{(1-\nu)}$	$k_h = \frac{32(1-\nu)\ GrC}{(7-8\nu)}$	$k_r = \frac{8Gr^3C}{3(1-\nu)}$	$k_t = \frac{16Gr^3C}{3}$
Geometric Damping	$D_v = \frac{0.425}{\sqrt{B_v}}$	$D_h = \frac{0.288}{\sqrt{B_h}}$	$D_r = \frac{0.15}{(1+B_r)\sqrt{B_r}}$	$D_t = \frac{0.50}{1+2B_t}$
General Case $C = C_1\ C_2$	$C_1 = 0.81$ C_2 = See Fig. 8	$C_1 = 1.0$ C_2 = See Fig. 8	$C_1 = 0.66$ C_2 = See Fig. 8	$C_1 = 0.41$ C_2 = See Fig. 8
Containment Structure Value of C	1.08	1.09	0.60	insufficient data

* For rectangular shaped foundations, calculation should be based on equations of pgs. 350 and 351, Richart, Hall and Woods, "Vibrations of Soil and Foundation."

$r_e = 0.6r$ for translation modes

$r_e = 0.8r$ for rotational modes

Note: for square or rectangular footing-

B = width of foundation in plan (parallel to axis of rotation)

L = length of foundation in plan (perpendicular to axis of rotation)

TABLE II

SUMMARY OF DETERMINATION OF STRESSES ON WALLS

1) Active Pressure on Structure Walls (σ_A) (at-rest condition assumed as a minimum condition)

Case	Equivalent Fluid Pressure Above Water Table (pcf)	Equivalent Fluid Pressure Below Water Table (pcf)
Static (at rest)	45	23*
Seismic** DBE	75	39*
Seismic** OBE or lower	45	23*

Note: * Hydrostatic head should be added.
** These values include static stresses. Seismic lateral stresses should be checked by the inertial load method, presented in 2 below.

2) Passive Pressure Developed on Structure Walls Due to Inertial Loads. (uniform stress assumed)

For Horizontal Translation: $\sigma_P = \frac{C_2 - 1}{C_2}\left(\frac{P}{A}\right)$

For Rocking Rotation : $\sigma_P = \frac{C_2 - 1}{C_2}\left(\frac{4M}{hA}\right)$

where: σ_P = Stress against wall

P = 70% of the maximum total horizontal inertial load

M = 70% of the maximum total inertial moment

A = Area of side of structure

h = Depth of embedment

C_2 = Embedment correction factor (See Fig. 8)

3) Stresses Due to Adjacent Structures (σ_L); See Fig. 9

4) Lateral Stress for Design (σ_T); $\left.\begin{aligned}\sigma_T &= \sigma_A + \sigma_L \\ \sigma_T &= \sigma_P + \sigma_L\end{aligned}\right\}$ Use Whichever is Larger

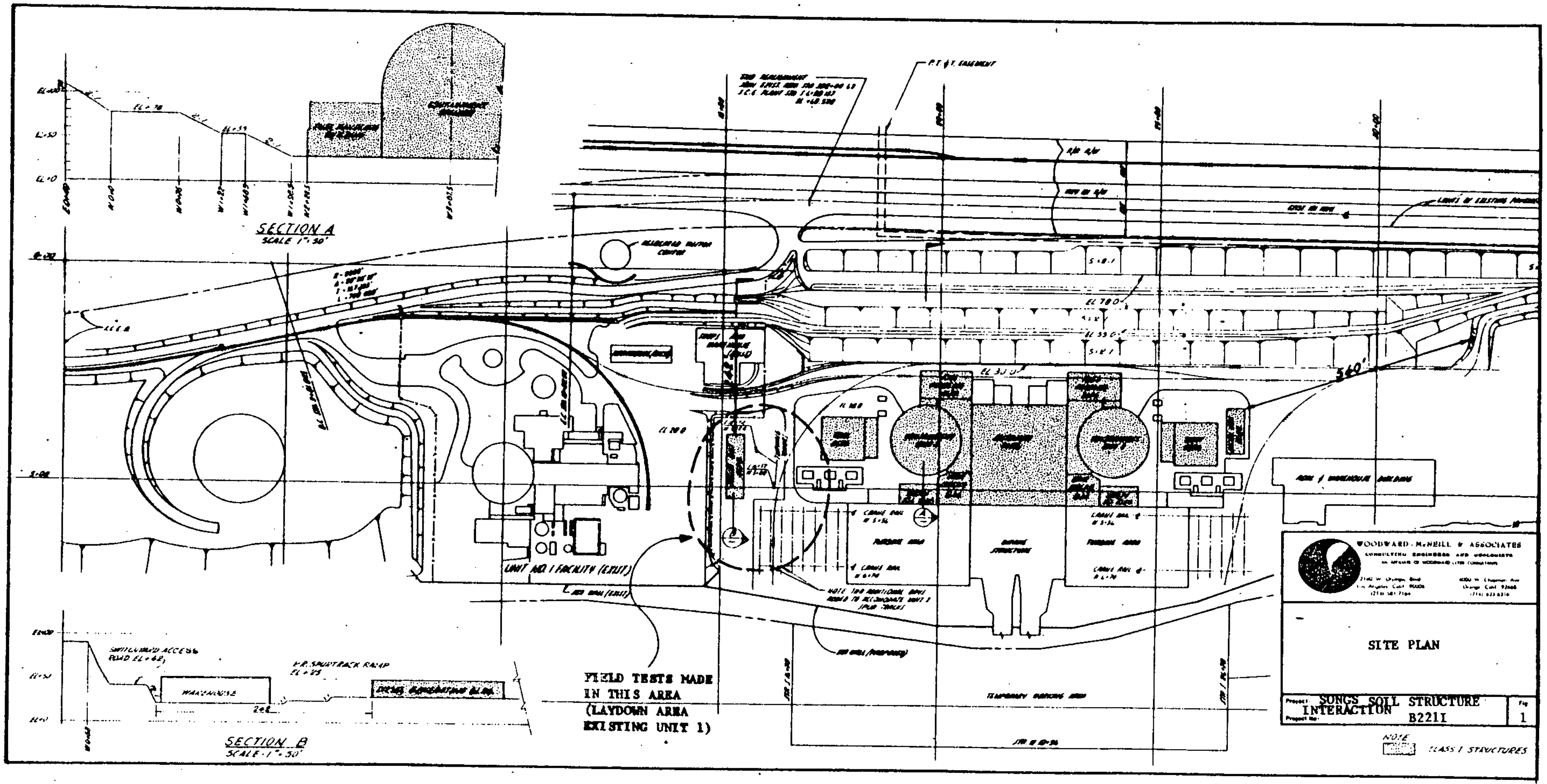
TABLE III

CASES STUDIED FOR
STRUCTURE SLIDING ANALYSES

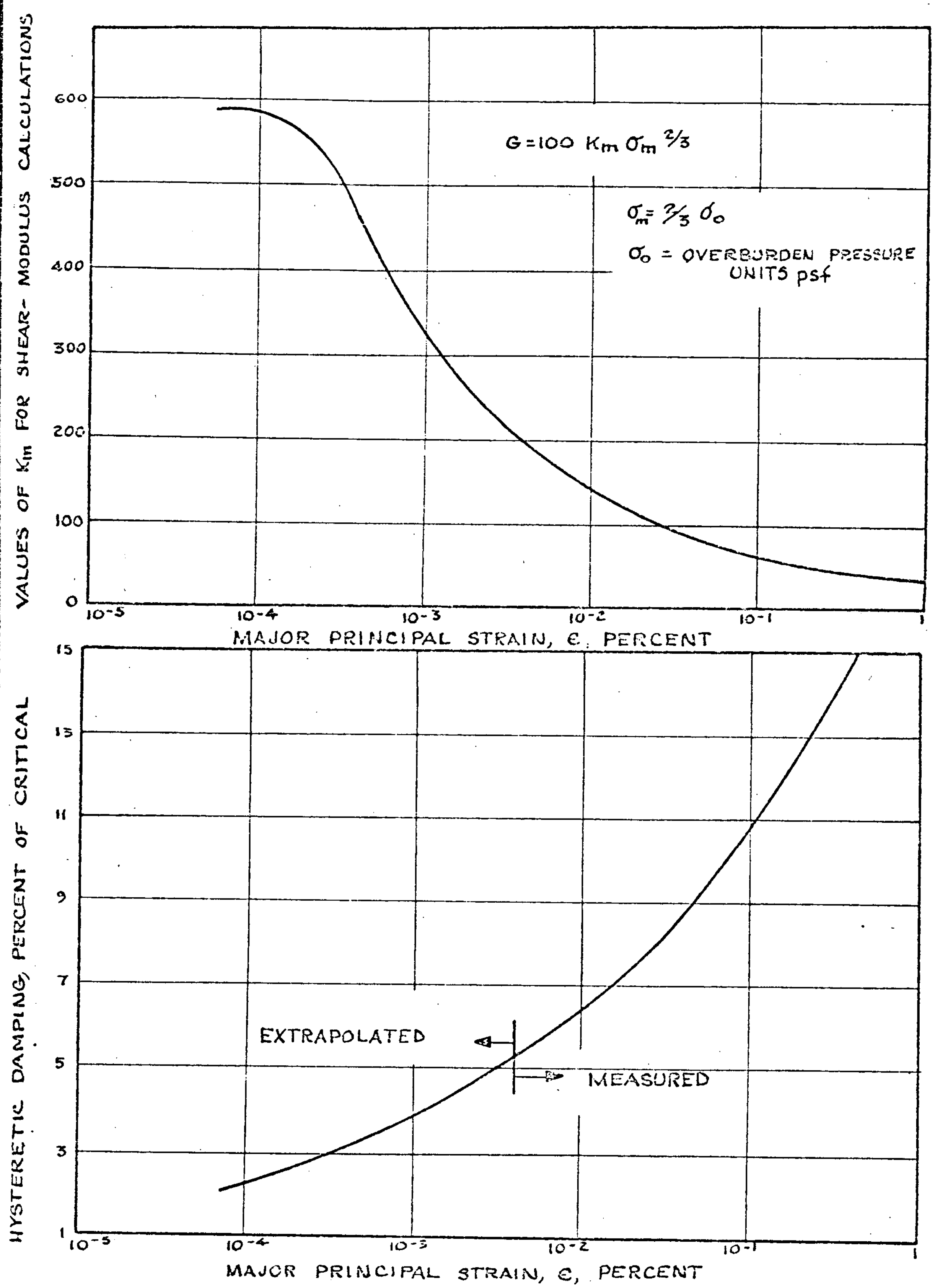
	Input Motion			Condition				
Case No.	H only	H & V in phase	H & V out of phase	Mass only	Mass plus Inertia	Embedment	Figure Nos. for Results	Location of Output
1	✓			✓			G-3	L, C, R
2		✓		✓			G-4	L, C, R
3			✓	✓			G-5	L, C, R
4	✓				✓		G-6	L, C, R
5		✓			✓		G-7	L, C, R
6	✓			✓		✓	G-8	L, C
7		✓		✓		✓	G-9	L, C, R

Where: H = Horizontal
V = Vertical
L = Left end of slab
C = Center of slab
R = Right end of slab

NOTE: Check is for item included; blank is for item not included.

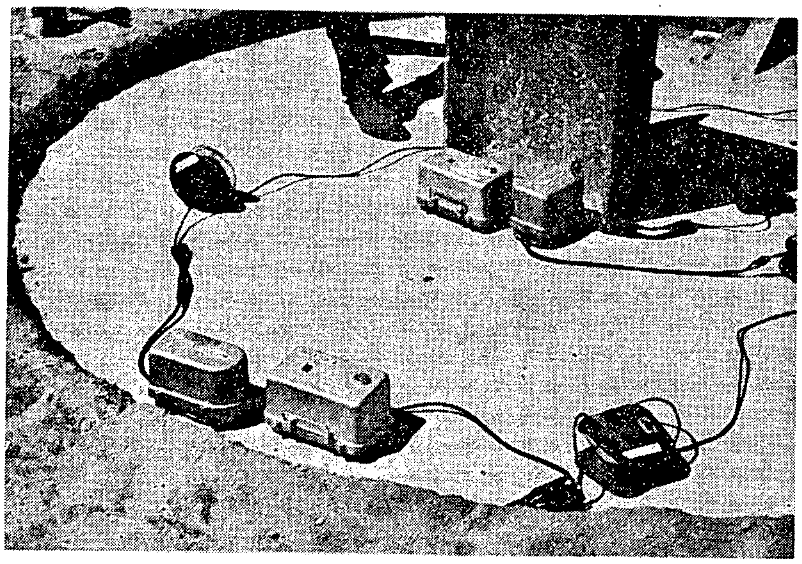


SECTION A
SCALE 1" = 50'
UNIT NO. 1 FACILITY (EXIST)
FIELD TESTS MADE
IN THIS AREA
(LAYDOWN AREA
EXISTING UNIT 1)
SECTION B
SCALE 1" = 50'
WAREHOUSE
WOODWARD-McNEILL & ASSOCIATES
SITE PLAN
SONGS SOIL STRUCTURE INTERACTION
B2211
1
NOTE
CLASS I STRUCTURES

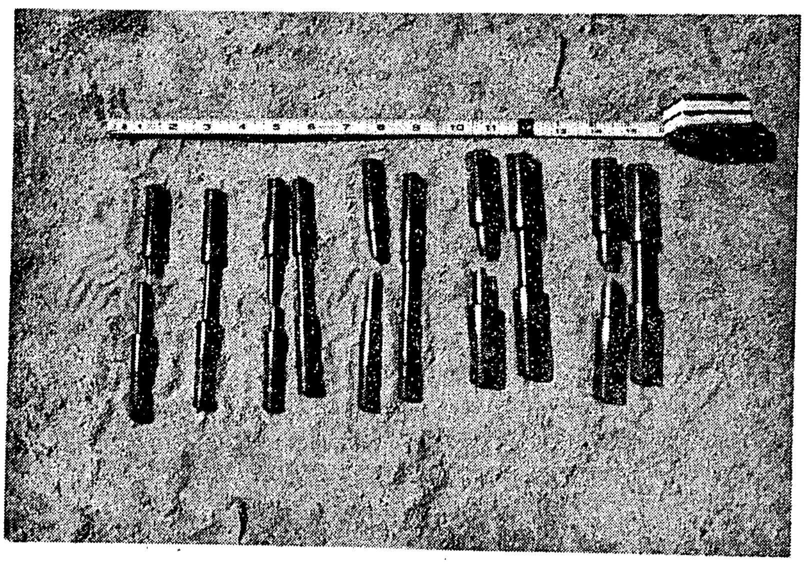


Project: SONGS SOIL STRUCTURE INTERACTION Job Number: E221I	MODULUS AND DAMPING VS STRAIN SAN MATEO FORMATION SAND	Fig. 2

WOODWARD - McNEILL & ASSOCIATES



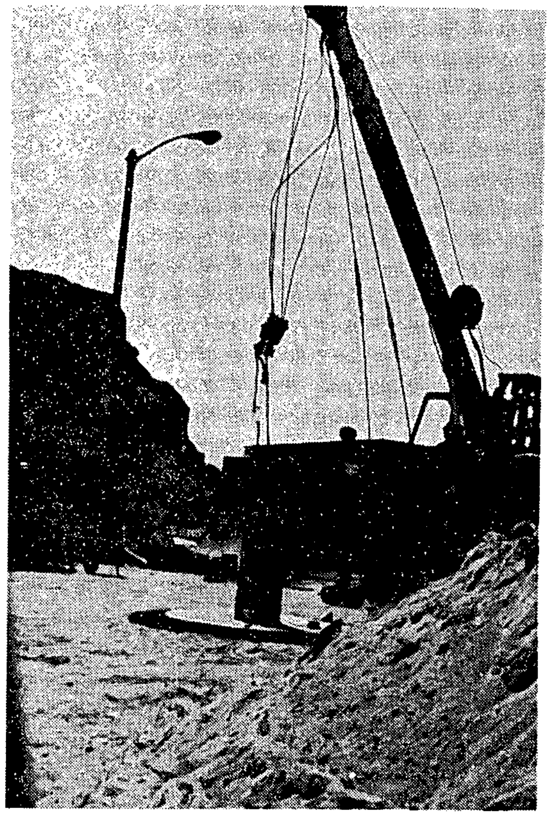
(a) INSTRUMENTATION LAYOUT FOR
FOUNDATION RESPONSE TESTS



(b) WEAK LINKS USED IN TESTS
(3 TO 16 KIPS TENSILE STRENGTH)

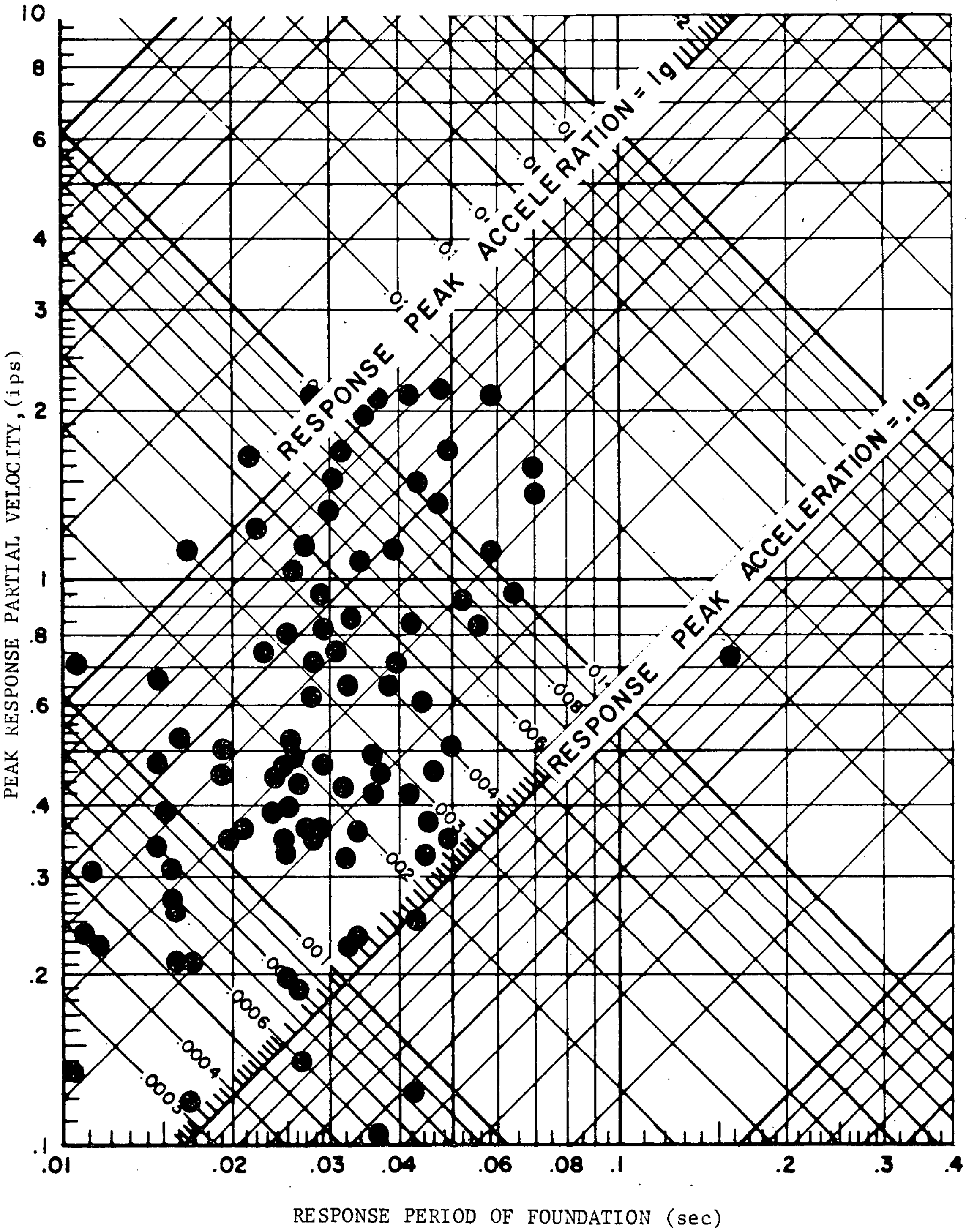
Project: SONGS SOILS STRUCTURE INTERACTION Job Number: B221I	INSTRUMENTATION	Fig. 3

(a) VERTICAL RESPONSE TEST



(b) HORIZONTAL RESPONSE TEST

Project: SONGS SOIL STRUCTURE INTERACTION Job Number: B221I	TESTS IN PROGRESS	Fig. 4



Project: SONGS SOIL STRUCTURE INTERACTION Job Number: B2211	MAXIMUM SLAB RESPONSE MEASUREMENTS	Fig. 5

WOODWARD-McNEILL & ASSOCIATES

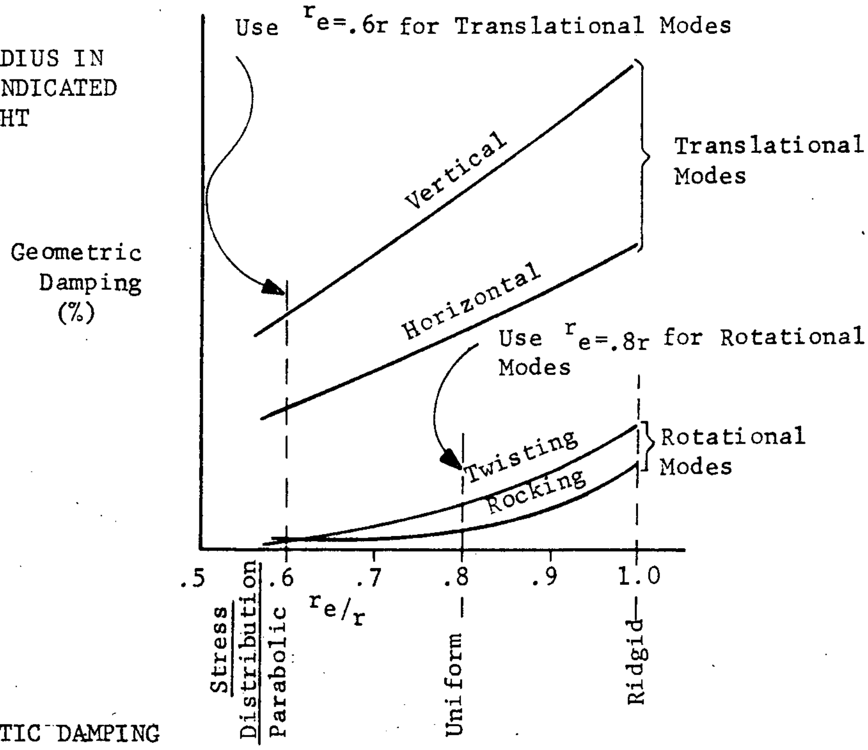
STEP 1

EVALUATE GEOMETRIC DAMPING FROM APPROPRIATE EQUATIONS OF TABLE-1

USE EFFECTIVE RADIUS IN CALCULATION AS INDICATED ON CURVES TO RIGHT

r_e = Effective Radius

r = Structure Radius



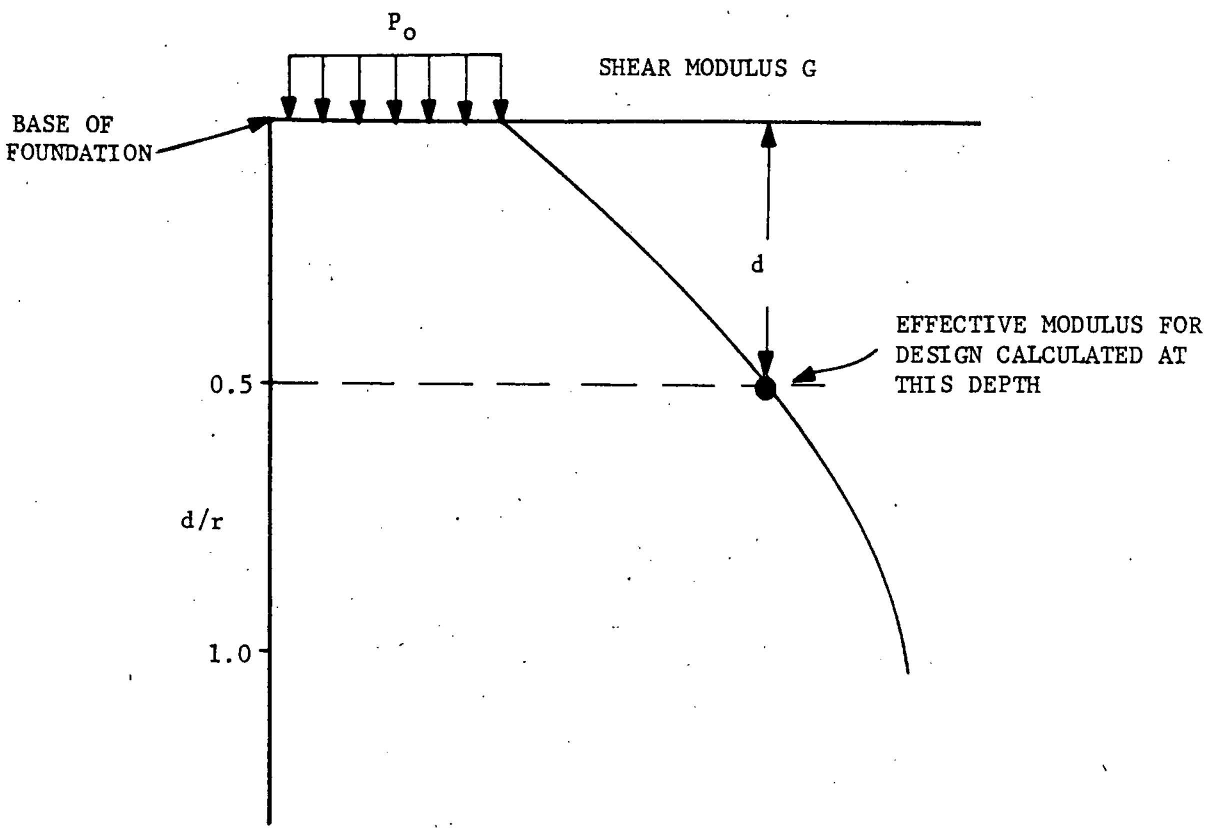
STEP 2

EVALUATE HYSTERETIC DAMPING FROM FIG. 2 USING FREE FIELD EARTHQUAKE INDUCED STRAIN

STEP 3

ADD GEOMETRIC AND HYSTERETIC DAMPING TO OBTAIN CONSERVATIVE ESTIMATE (LOW) OF DAMPING FOR DESIGN

Project: SONGS SOIL STRUCTURE INTERACTION **Job Number**: B221I	SOIL DAMPING FOR STRUCTURES (SCHEMATIC)	**Fig.** 6



WHERE:

d = Depth below base of foundation

r = Radius of foundation

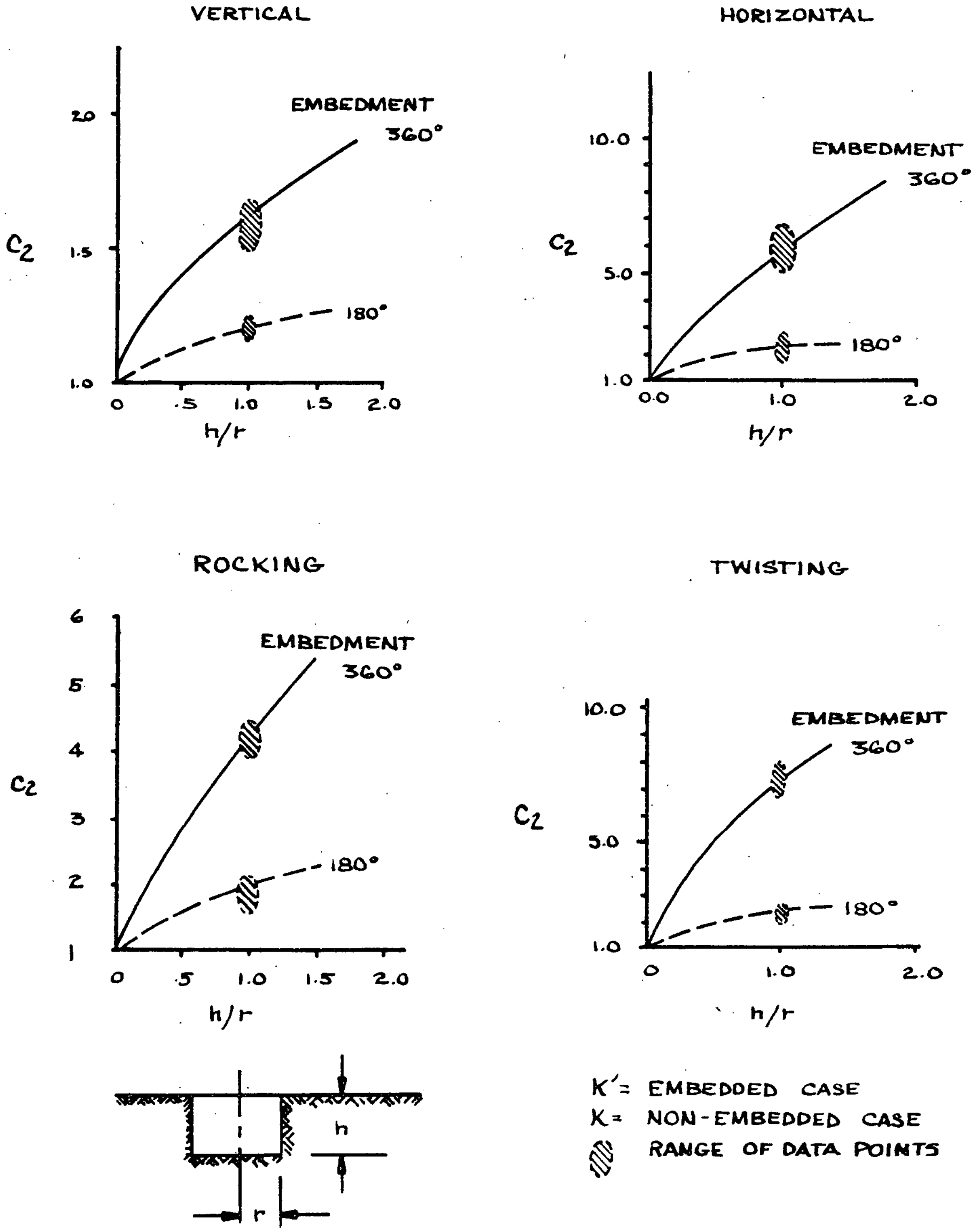
G = Shear modulus

$\sigma_m = 2\,(\gamma' d + 0.9\ P_o)/3$

P_o = Total bearing pressure

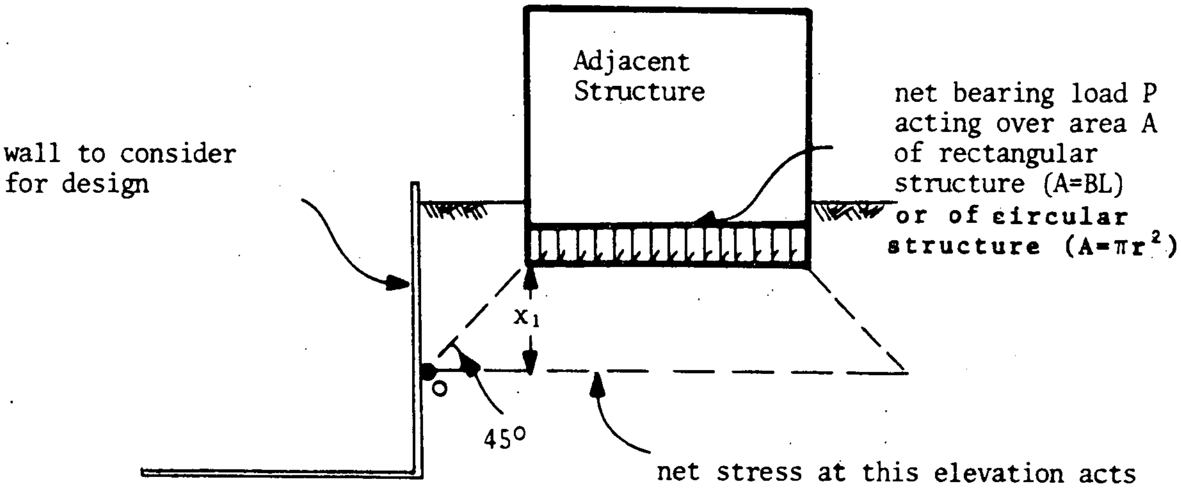
Project: SONGS SOIL STRUCTURE INTERACTION **Job Number:** B221I	SHEAR MODULUS DETERMINATION (SCHEMATIC)	**Fig.** 7

WOODWARD-McNEILL & ASSOCIATES



Project: SONGS SOIL STRUCTURE INTERACTION **Job Number:** B221I	EFFECT OF EMBEDMENT ON SPRING CONSTANT	**Fig.** 8

LATERAL PRESSURE DUE TO ADJACENT STRUCTURES



additional lateral pressure on wall $= \frac{A}{A'} \cdot PM$ (uniform)

(applied below point .O)

where: M = 0.34 for Static and OBE condition

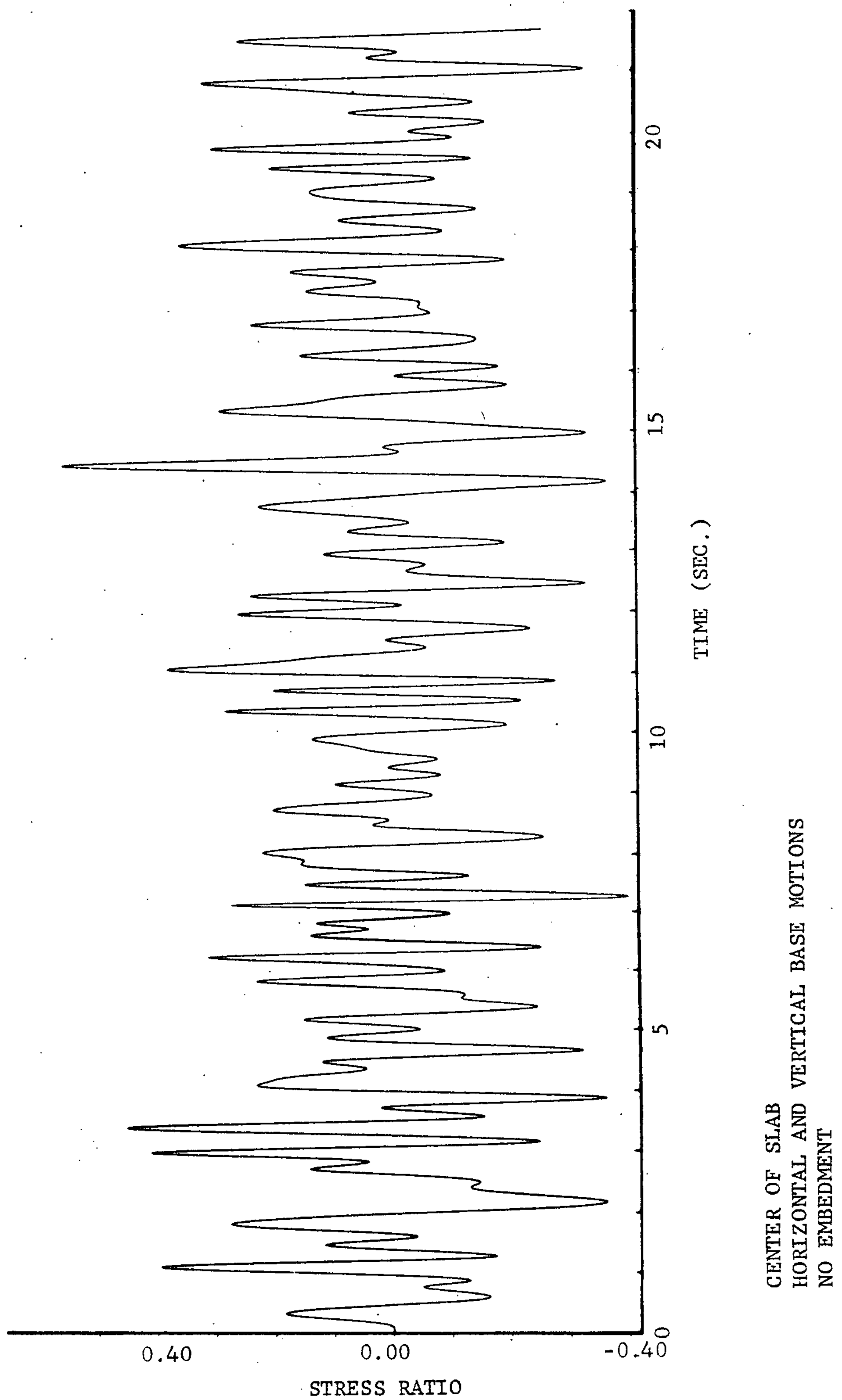
M = 0.57 for DBE condition

Notes: 1) The example presented is for a rectangular structure. For circular structures adjust the radius by x_1, to calculate A'.

2) The lateral pressure calculated here should be added to either the active earth pressure or the inertial load for design.

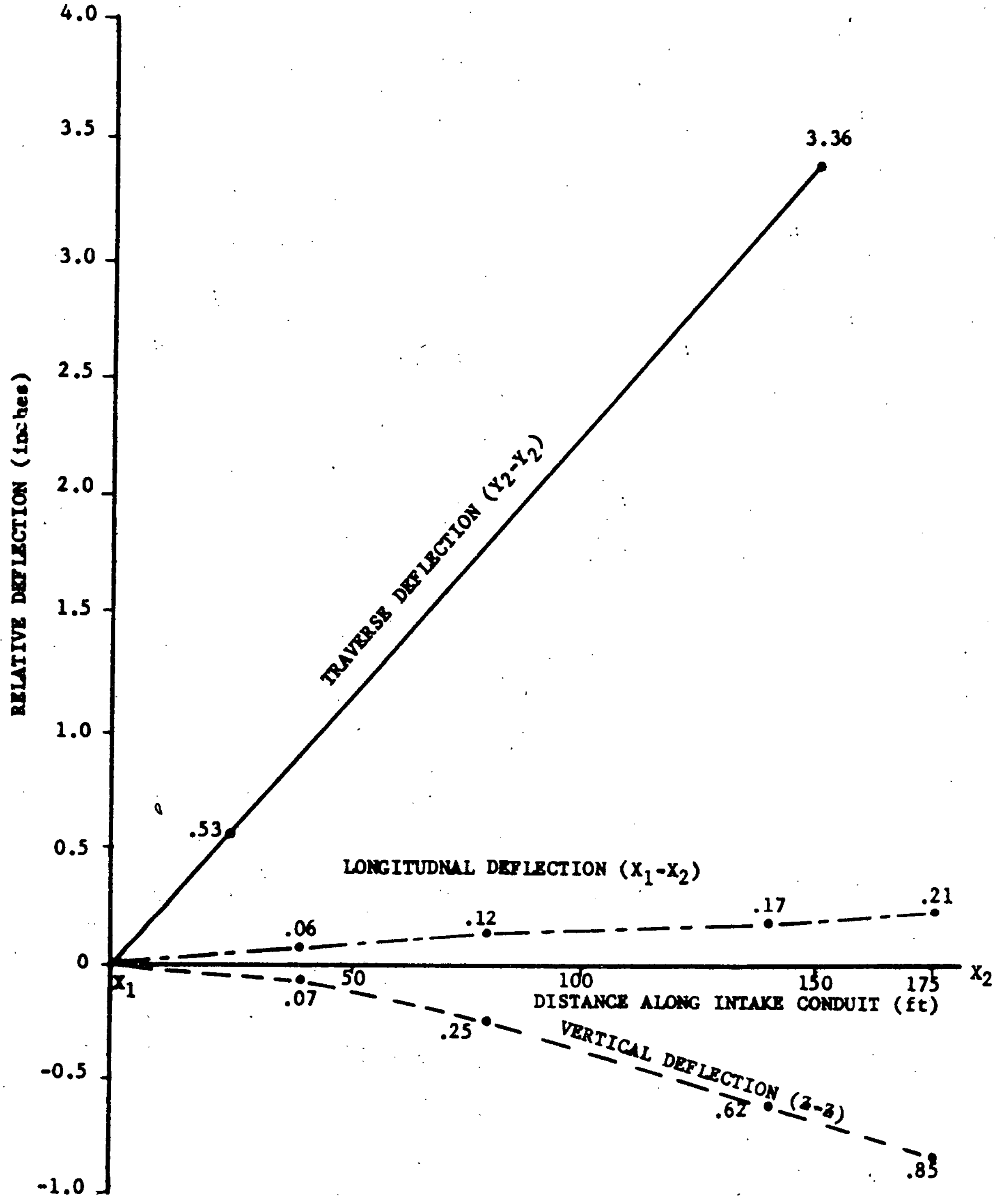
Project: SONGS SOIL STRUCTURE INTERACTION Job Number: B221I	LATERAL PRESSURES DUE TO ADJACENT STRUCTURES	Fig. 9

WOODWARD - McNEILL & ASSOCIATES



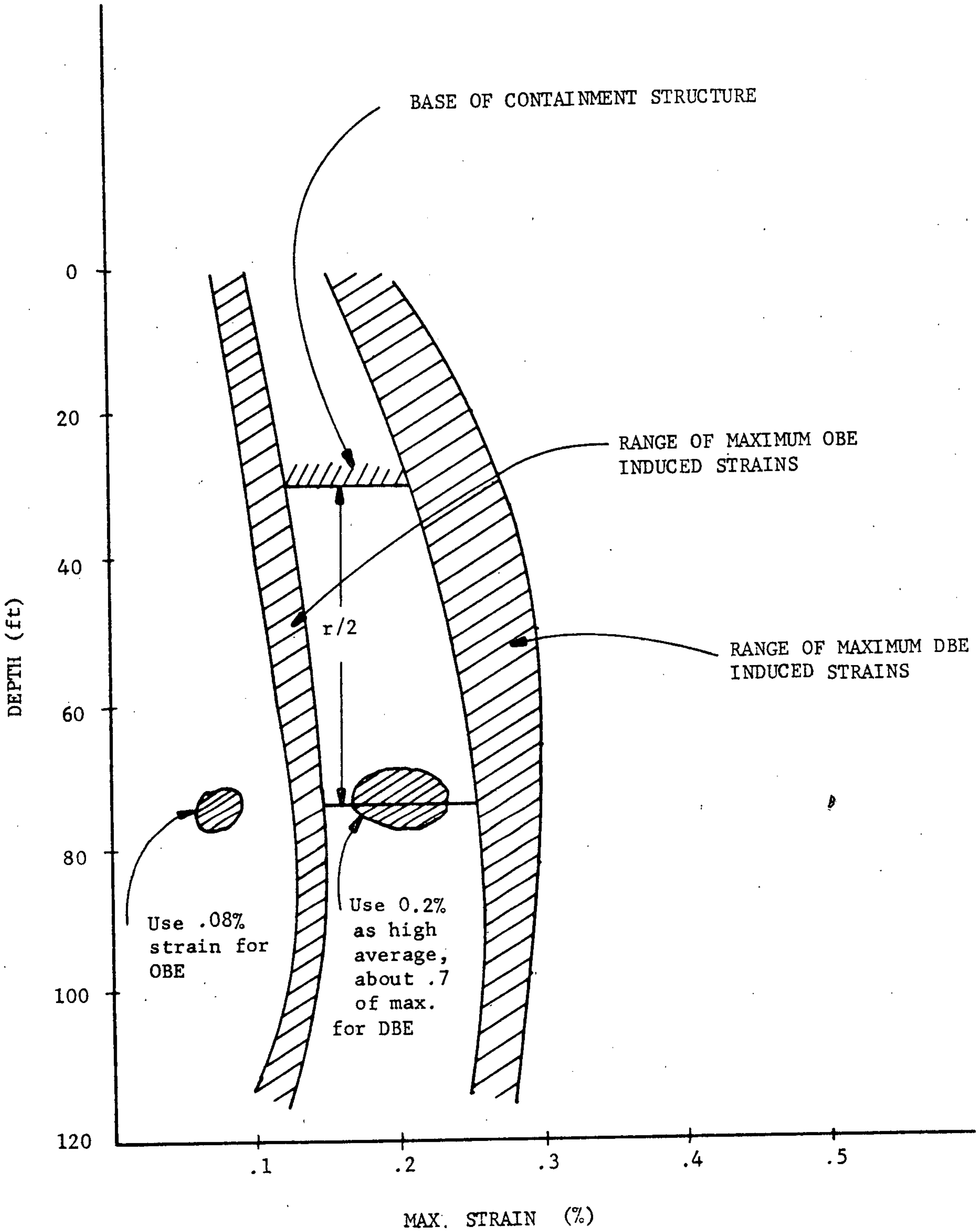
Project: SONGS SOIL STRUCTURE INTERACTION **Job Number:** B221I	EXAMPLE OF STRUCTURAL SLIDING RESULTS	**Fig.** 10

WOODWARD - McNEILL & ASSOCIATES



Project: SCE POST ACRS TASK 12 Job Number: B221I	CRITICAL DISPLACEMENT PROFILE FOR OFF-SHORE CONDUIT EXTENDING WEST OF PERMANENT SEA WALL	Fig. 11

WOODWARD - McNEILL & ASSOCIATES

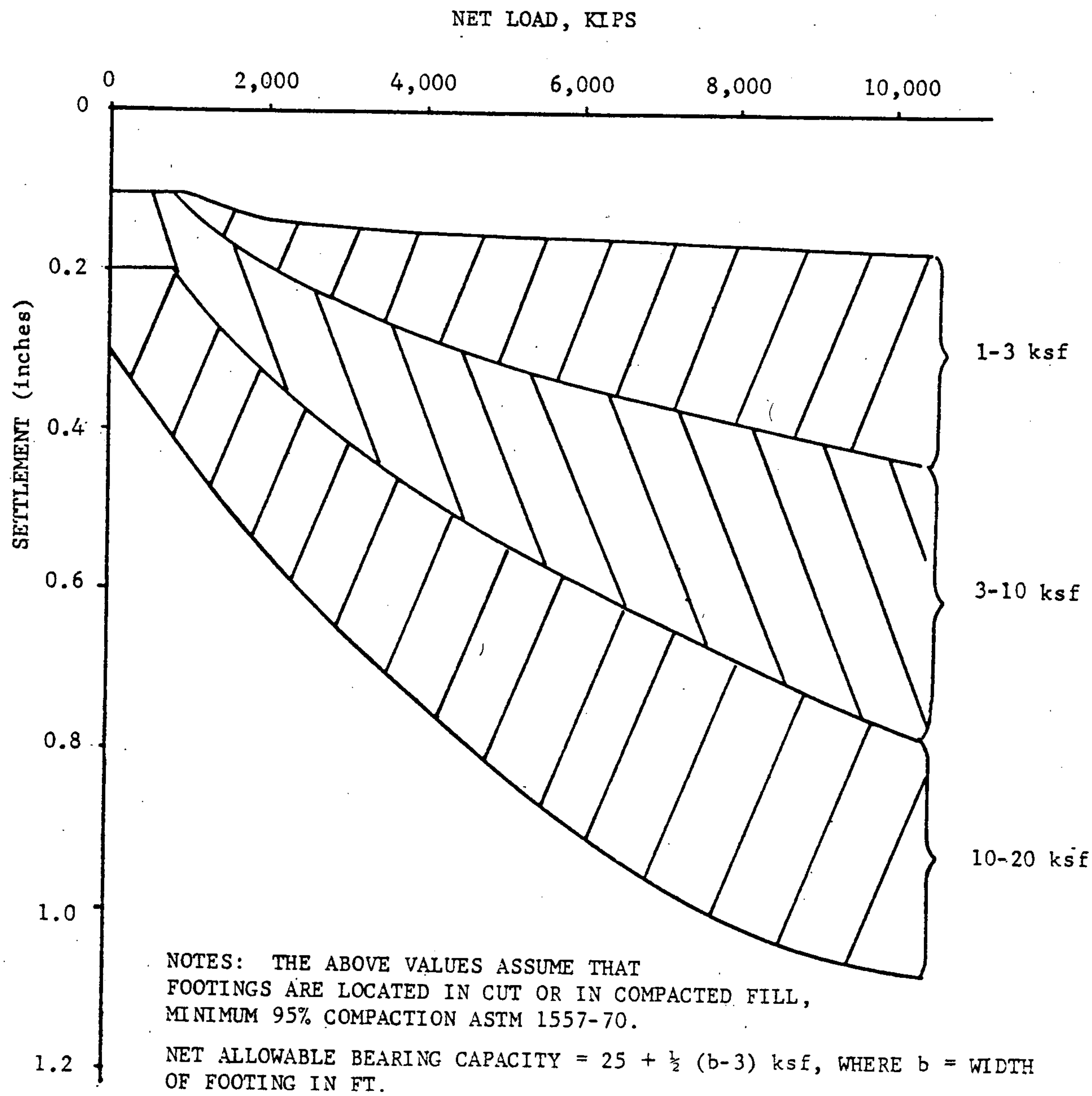


Project: SONGS SOIL STRUCTURE INTERACTION
Job Number: B221I

OBE & DBE INDUCED STRAINS

Fig. I2

WOODWARD - McNEILL & ASSOCIATES



NOTES: THE ABOVE VALUES ASSUME THAT FOOTINGS ARE LOCATED IN CUT OR IN COMPACTED FILL, MINIMUM 95% COMPACTION ASTM 1557-70.

NET ALLOWABLE BEARING CAPACITY = 25 + ½ (b-3) ksf, WHERE b = WIDTH OF FOOTING IN FT.

NET LOAD = TOTAL LOAD-Wt. OF SOIL DISPLACED BETWEEN THE FOOTING BASE AND ADJACENT GRADE OR TOP OF ADJACENT FLOOR SLAB (WHICH EVER IS LESS)

MINIMUM WIDTH OF FOOTING = 3'

DIFFERENTIAL SETTLEMENTS COULD BE UP TO ½ THE VALUE INDICATED ABOVE.

Project: SONGS SOIL STRUCTURE INTERACTION Job Number: B221I	ESTIMATED SETTLEMENT FOR FOUNDATIONS BEARING ON SAN MATEO FM. SANDS	Fig. 13

WOODWARD-McNEILL & ASSOCIATES

APPENDIX A

SLAB RESPONSE TESTS

Five test slabs were constructed in the laydown area of Unit 1 between 5 September and 12 September 1972. The various slab configurations are shown on Figs. A-1 and A-2. Slab Nos. 1 and 2 are of similar construction, geometry, and dimensions. Slab No. 1 was constructed above the existing ground surface, while slab No. 2 was fully embedded in the ground. Slab No. 3, shown on Fig. A-2, was constructed below the ground surface. The configuration of this slab was designed to model the basic shape of the proposed containment structure foundation. Slab Nos. 4 and 5 were of smaller dimensions then the other slabs, and were constructed primarily to evaluate scaling and shape effects on slab response.

Test Procedures

It was desired to measure slab response in the horizontal, vertical, twisting, and rocking modes for each slab under various embedment conditions after a transient vibration was input to each slab. Each slab was set into motion by attaching it with a cable to a loader or crane by means of a weak link, as illustrated on Fig. A-3. Vibrations with various amplitudes were induced in the slabs by tensioning a cable between the slab and tractor until it broke at the weak link. The response of the slab to the transient load was measured with horizontally and vertically oriented velocity-sensitive geophones used in conjunction with a CEC 124 Oscillograph recorder. From the response frequency and attenuation characteristics of the slab vibrations it is possible to calculate soil stiffness and damping factors, knowing the mass of the slab, its geometry, and the mode of vibration.

Previous research has indicated that embedment characteristics, i.e., depth of slab embedment and the nature of the surrounding soil, affect the slab response. For this reason, slabs Nos. 2 and 3 were tested under various conditions of embedment. Slab No. 2 was first tested with complete embedment below the existing ground surface (in undisturbed soil). The soil around one-half of the slab was then excavated, and the slab was tested again. The remainder of the soil around the slab was then excavated, and the slab was again tested. As it was desired to determine if there were any differences between slab response due to embedment in undisturbed native soils versus compacted backfill native soils, the soil which had been excavated from around slab No. 2 was then backfilled in thin lifts and compacted with hand-held mechanical vibrators. A final set of response tests was then made on this slab. Slab No. 3 was tested with two embedment conditions; (1) non-embedment, and (2) full embedment in compacted backfill of native soils. The various embedment conditions for these slabs are illustrated on Fig. A-4.

Examples of the recorded field test traces for various slabs are presented on Figs. A-5, A-6, A-7 and A-8.

It can be noted in Fig. A-5, which is a record for horizontal input motion, that some vertical motion occurred during this test, i.e., there was some rocking. This occurred to some extent in all horizontal tests. Although coupling did occur between the horizontal and rocking modes, careful interpretation was done in order to determine the dominant form of motion.

Fig. A-8 presents the test record for twisting input motion of slab No. 5. It can be noted on this figure that geophone H_2 registered motion. This is due to the fact that because of the size of the geophone with respect to the slab, it was not possible to center the geophone, as can be seen on the schematic test setup plan shown on Fig. A-8.

A summary of representative frequency response data determined for each mode of motion for each slab is presented in Table A-1.

TABLE A-1

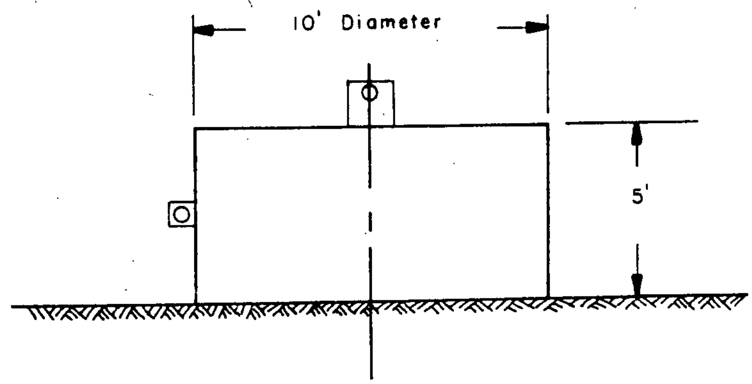
SUMMARY OF SLAB RESPONSE MEASUREMENTS

Slab No.	Embedment Condition	Representative Frequency Response (cps) F_v	F_h	F_r	F_t
1	Non-embedment	21	20	20	19
2	Full Embedment in Natural Ground	27	50	39	52
2	180° Embedment, Natural Ground	23	30	27	29
2	Non-embedment (slightly undercut)[1]	23	18	17	14
2	Full Embedment in[2] Backfill	--	24	25	36
3	Below Ground Surface, but non-embedded	32	29	32	29
3	Full Embedment in[2] Backfill	(19-24)	(33-37)	39	39
4	Full Embedment	(40-49)	--	(59-67)	(80-90)
5	Full Embedment	(36-48)	--	(59-65)	90

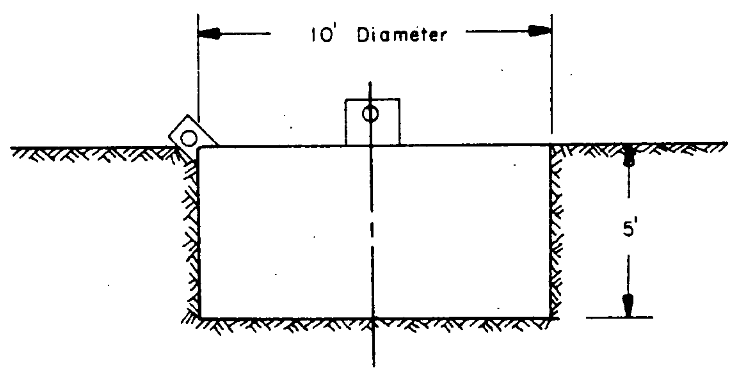
Notes:

[1] During full perimeter excavation, the slab was accidently undercut locally. Therefore results were likely low and were not utilized.

[2] Because of close working conditions and time constraints neither a high nor a uniform degree of compaction was achieved. Therefore results were likely low and were not utilized.



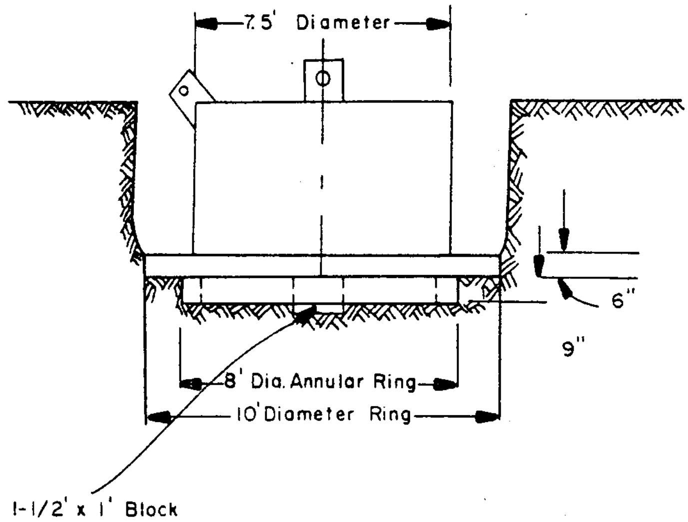
SLAB NO. 1



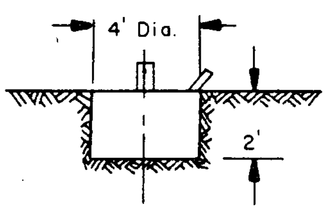
SLAB NO. 2

Project: SONGS Job Number: B221I	TEST SLAB CONFIGURATIONS	Fig. A-1

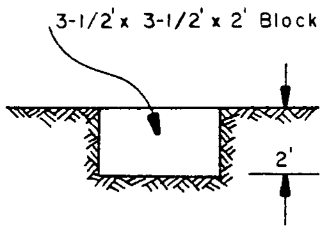
WOODWARD-McNEILL & ASSOCIATES



SLAB NO. 3



SLAB NO. 4

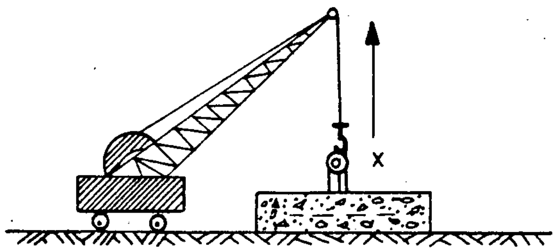


SLAB NO. 5

Project: SONGS **Job Number**: B221I	TEST SLAB CONFIGURATIONS	**Fig.** A-2

WOODWARD-McNEILL & ASSOCIATES

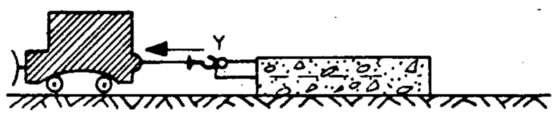
VERTICAL MODE



(ELEVATION)

Pull upward with crane using a "weak link" attached to crane cable

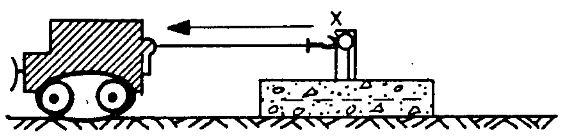
HORIZONTAL MODE



(ELEVATION)

Pull horizontally with cat using "weak link" attached to slab with a cable

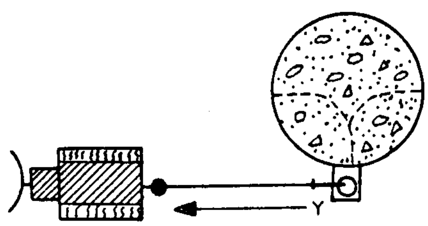
ROCKING MODE



(ELEVATION)

Pull as indicated for the horizontal mode

TWISTING MODE

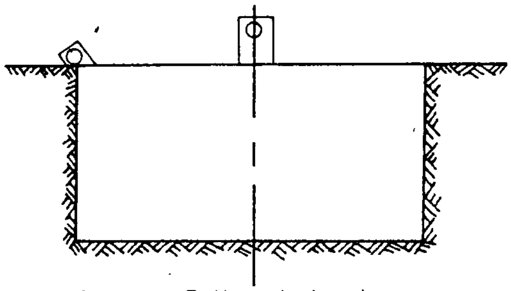


(PLAN)

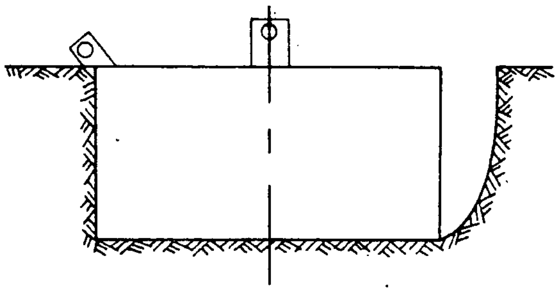
Pull as indicated for the horizontal mode

Project: SONGS **Job Number:** B221I	METHODS OF DYNAMIC EXCITATION OF TEST SLABS	Fig. A-3

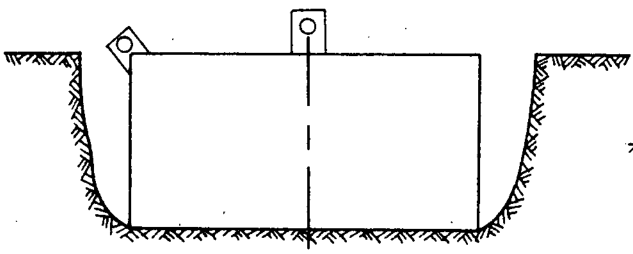
SLAB NO. 2



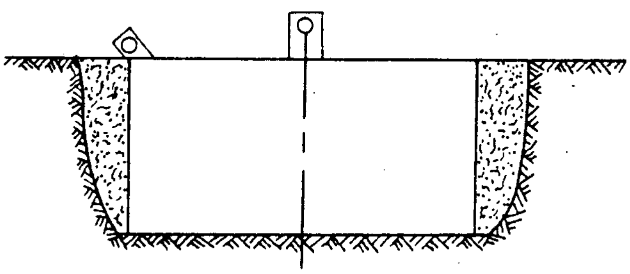
Case 1: Full embedment in natural ground



Case 2: Half embedment in natural ground

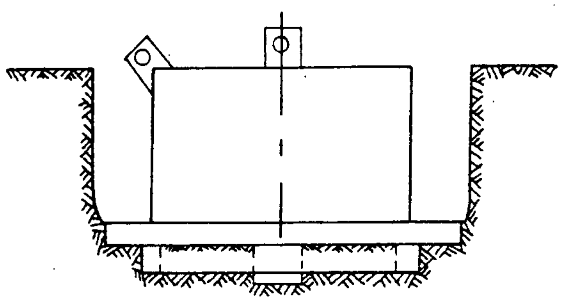


Case 3: Soil around slab excavated

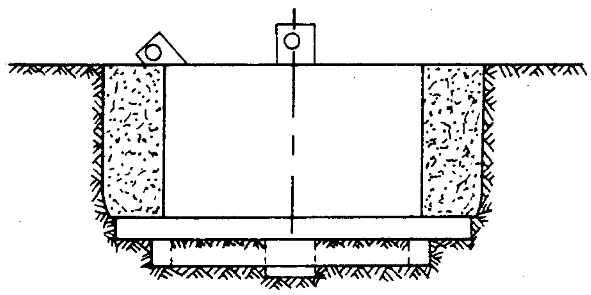


Case 4: Soil backfilled and compacted around slab

SLAB NO. 3



Case 1: Soil around slab excavated



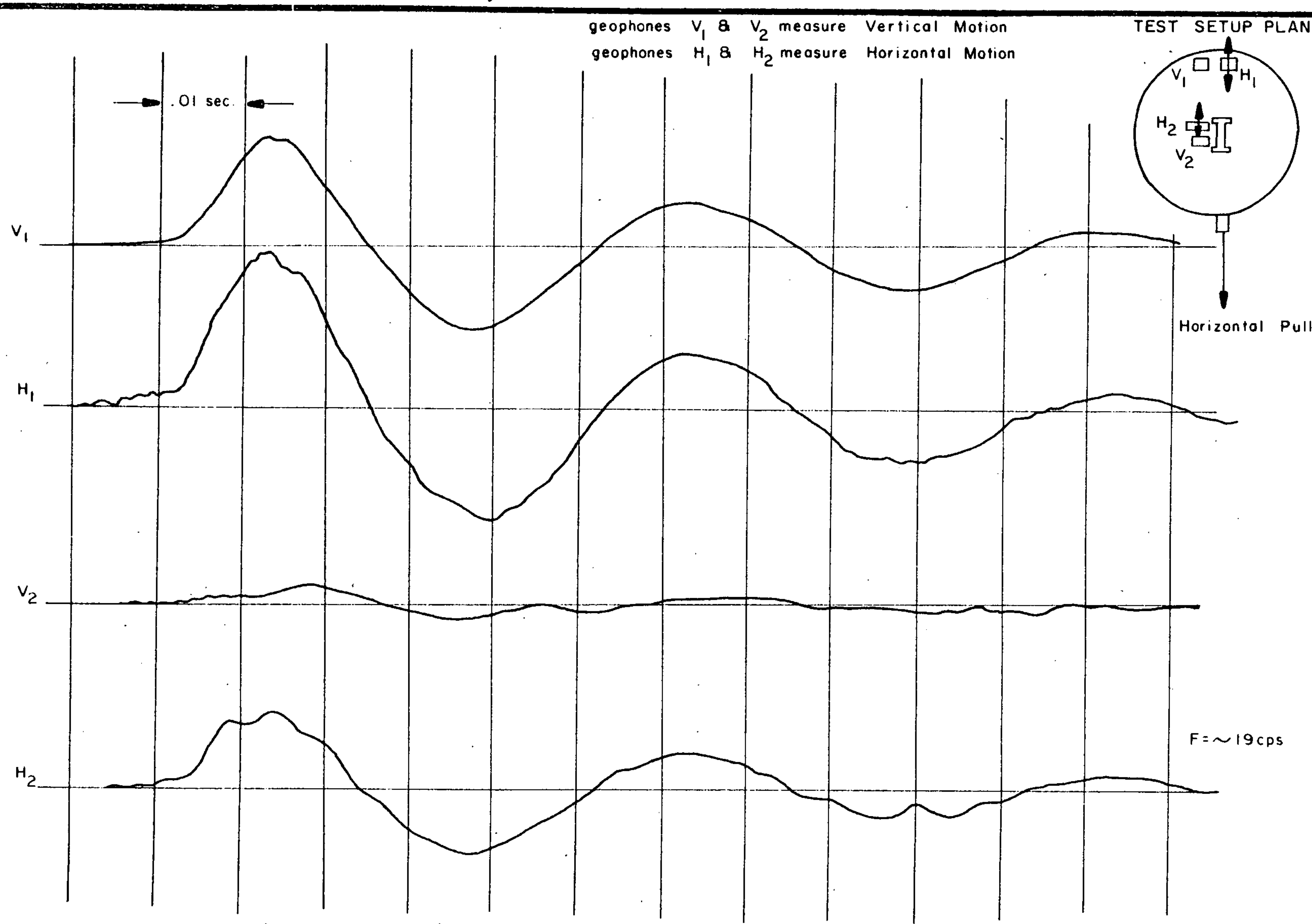
Case 2: Soil backfilled and compacted around slab

Project: SONGS **Job Number**: B221I	EMBEDMENT CONDITIONS SLABS NO. 2 AND 3	Fig. A-4

WOODWARD-McNEILL ASSOCIATES

Project: SONGS
Job Number: 82211

RECORDED DATA, SLAB RESPONSE
TEST HORIZONTAL MODE

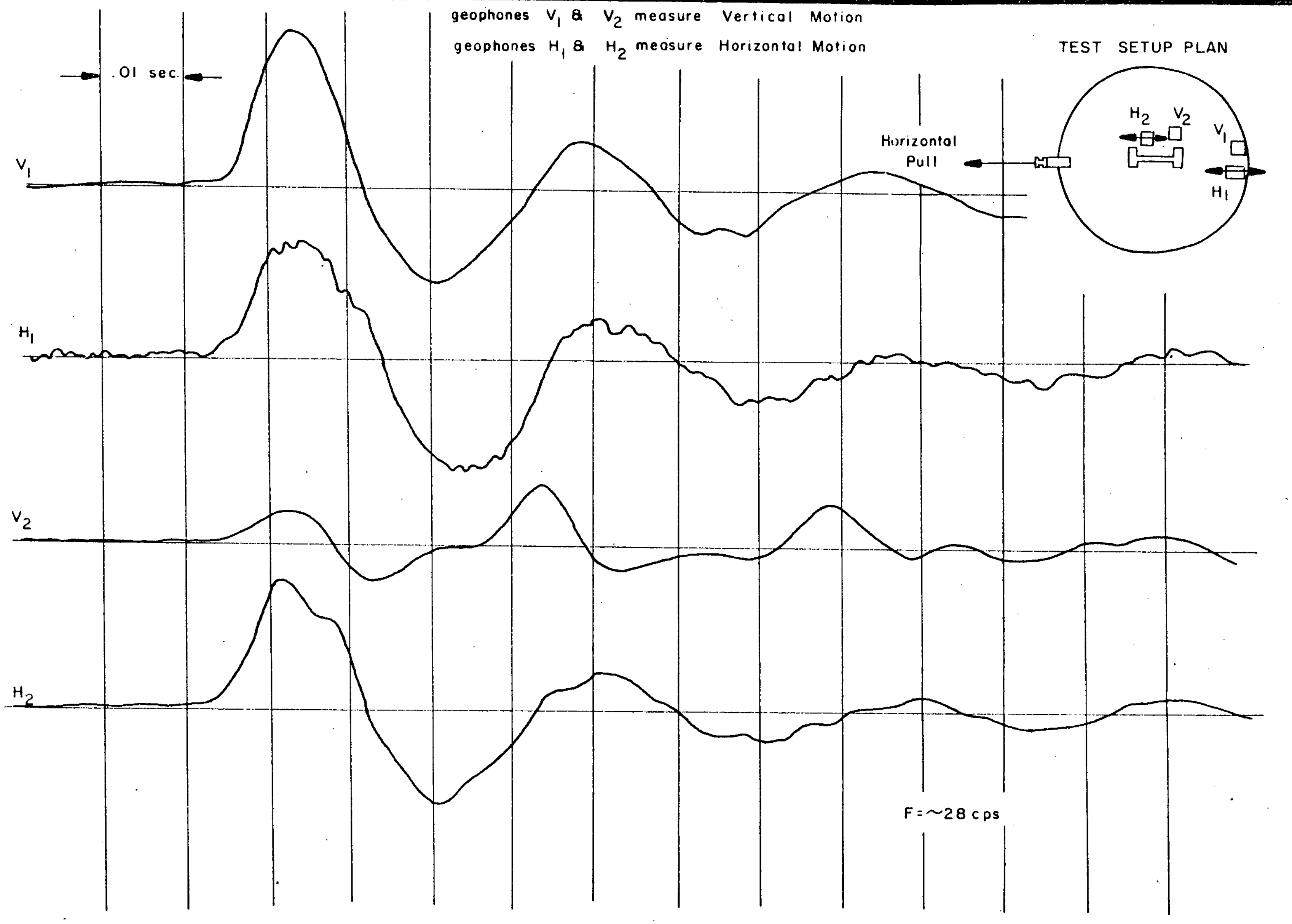


SLAB NO. 1, HORIZONTAL MODE (with coupled Rocking)
Slab Velocity versus Time

WOODWARD-McNEILL & ASSOCIATES

Fig. A-5

0-1



SLAB NO. 2, HALF EMBEDMENT CASE, ROCKING MODE

Slab Velocity versus Time

Project: SONGS

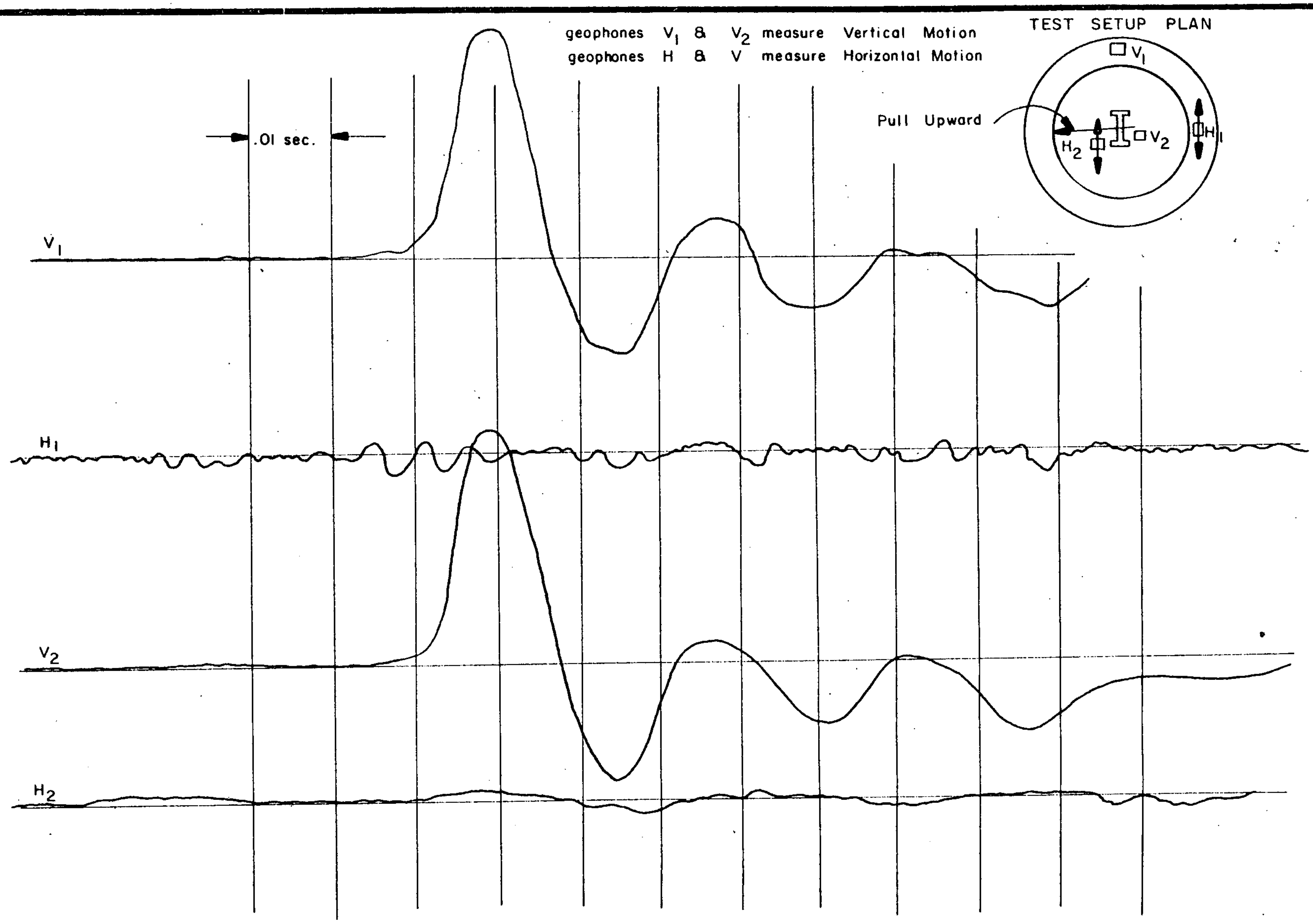
Job Number: B221 I

RECORDED DATA, SLAB RESPONSE
TEST ROCKING MODE

Fig. A-6

WOODWARD-McNEILL & ASSOCIATES

O-1



SLAB NO. 3 VERTICAL MODE
Slab Velocity versus Time

F = ~ 35 cps

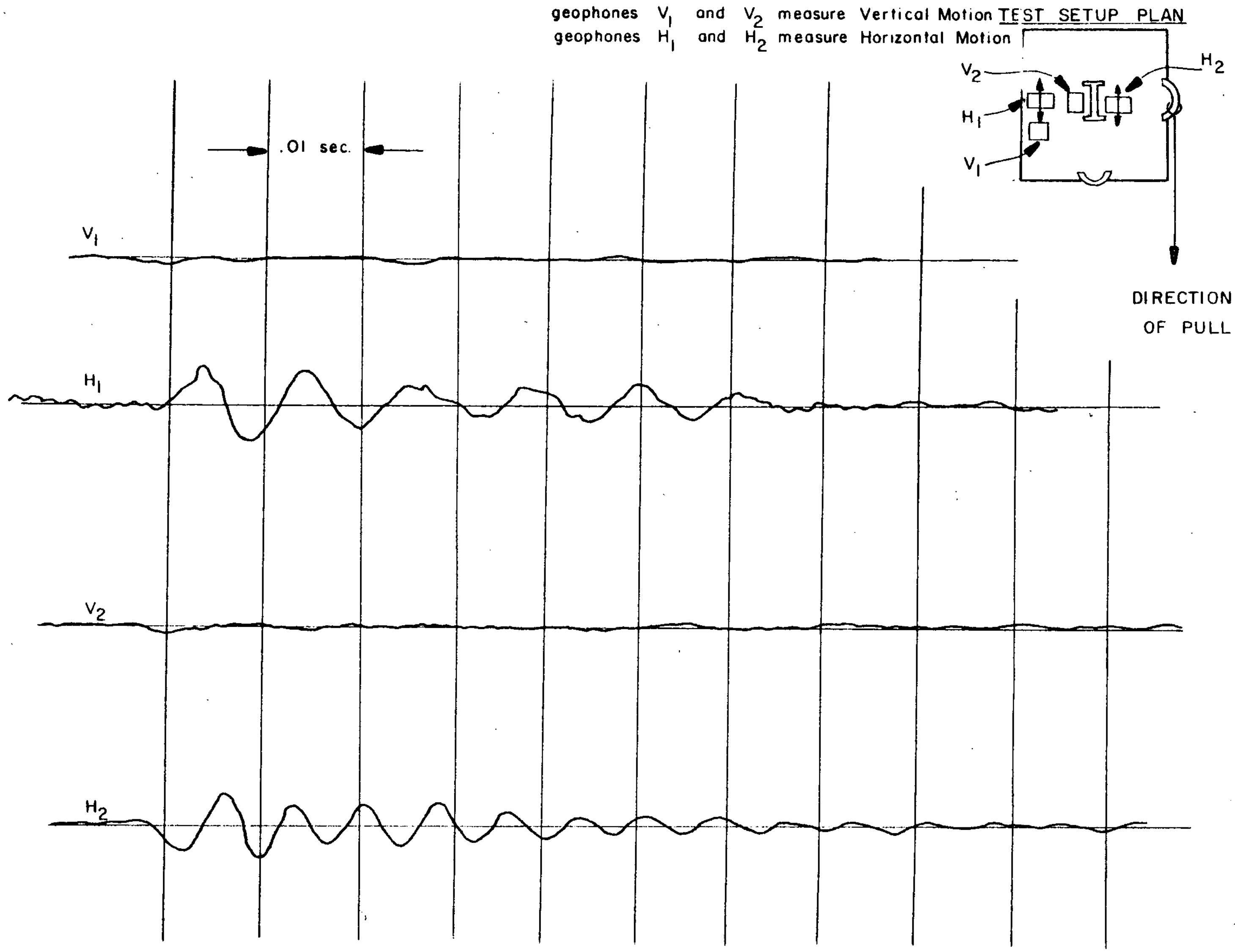
Project: SONGS SOIL STRUCTURE INTERACTION
Job Number: B2211

RECORDED DATA, SLAB RESPONSE
TEST VERTICAL MODE

WOODWARD-McNEILL & ASSOCIATES

Fig. A-7

0-1



SLAB NO. 5, TWISTING MODE
Slab Velocity vs. Time

f = ~90 cps

Project: SONGS SOIL STRUCTURE INTERACTION
Job Number: B221I

0-1

RECORDED DATA, SLAB RESPONSE
TEST TWISTING MODE

Fig. A-8

WOODWARD-McNEILL & ASSOCIATES

APPENDIX B

RAYLEIGH WAVE TESTS

Rayleigh wave traverses were conducted in the laydown area at the site at the locations shown on Fig. B-1. The purpose of performing these traverses was to provide data for the calculation of the Rayleigh wave velocity for the near-surface soil at the site.

Two 5-ft deep trenches were excavated in order to improve coupling between the input source and the subsurface soils and to insure that tests would be made in undisturbed native soil (San Mateo Formation Sand). The first trench was dug parallel to the beach, and the other perpendicular.

For the first two traverses conducted, Rayleigh waves were created by allowing a small electromechanical vibrator to vibrate in the vertical mode in the bottom of the trench. The third traverse utilized a vibrating sheepsfoot roller as the source of the Rayleigh waves. This vibrator produced large amplitude motions which also provided data for the attenuation measurements, described in Appendix C.

The Rayleigh wave motions were sensed with a pair of vertically oriented geophones. The procedure involved two steps: first, setting the two geophones at the same distance from the vibrator, and checking that they are exactly in phase; and second, moving one geophone to a progressively larger distance from the vibrator and measuring the time phase changes at several vibrator frequencies at each new location. An examples of the data recorded in the field is presented in Fig. B-2.

Appendix B

Page 2

The data are analyzed by using the simple relationship:

$$V_r = \frac{D}{t}$$

Where: V_r = Rayleigh-wave velocity

D = distance between geophones

t = time phase change

The wave length corresponding to each frequency can then be determined by the relationship:

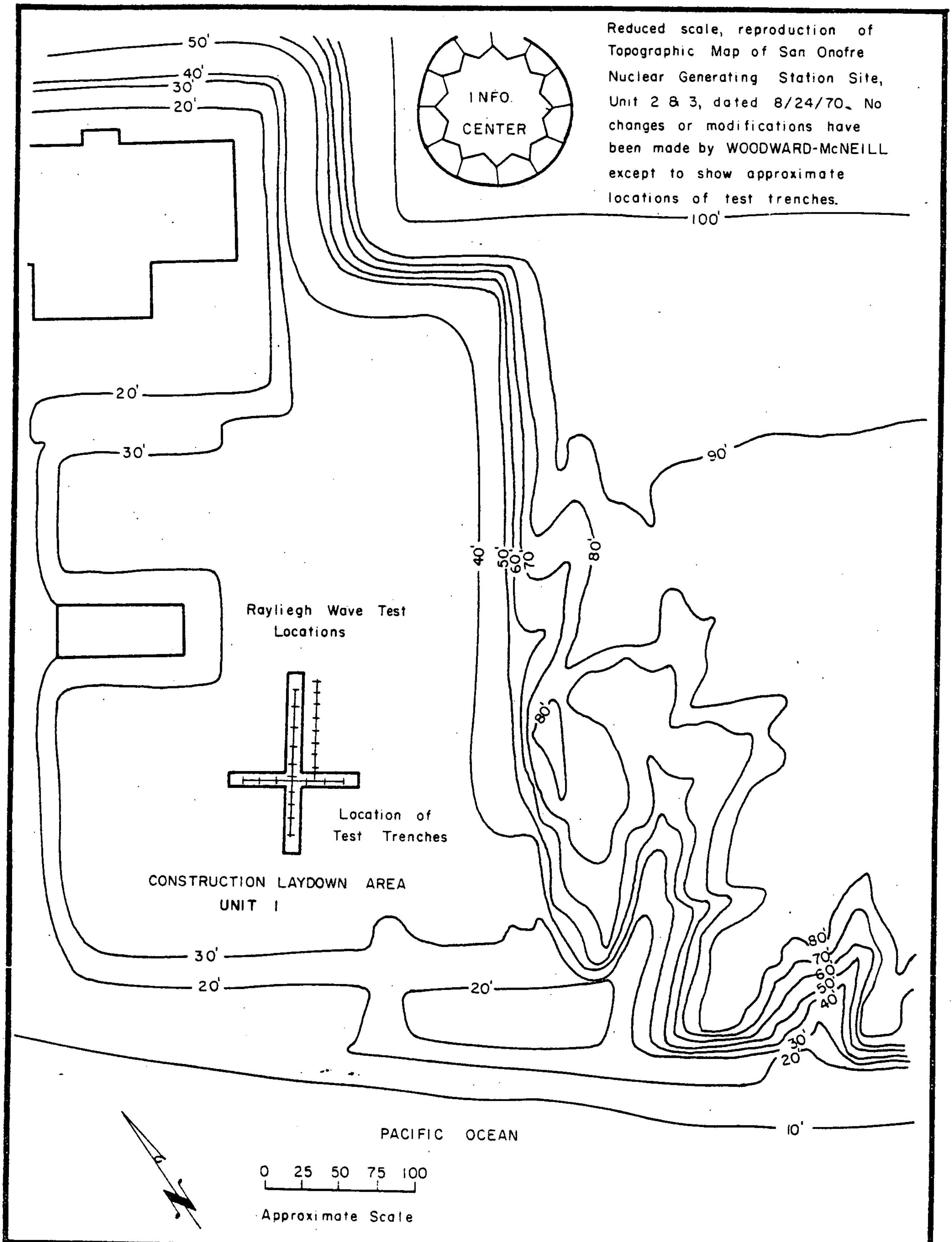
$$\lambda = \frac{D}{tF}$$

Where: λ = wave length

F = frequency

The Rayleigh wave velocity V_r is a property of the near-surface material, and is, for practical purposes, equal to the shear wave velocity for material in this zone. The test averages the soil property to a depth of about 0.5 times the wave length. Thus the high frequencies (short wave lengths) provide properties of only the surface materials; and the low frequencies integrate the properties of surface, near-surface, and deeper materials.

For this site, the Rayleigh wave velocity was found to range from 850 to 1200 fps, with a velocity of 930 fps as a representative average for the near-surface (upper 15 ft) soils. This value was confirmed by direct measurement of surface wave velocity from attenuation tests as discussed in Appendix C.



Project: SONGS SOIL STRUCTURE Job Number: B221I	SITE PLAN	Fig. B-1

0-1

WOODWARD-McNEILL & ASSOCIATES

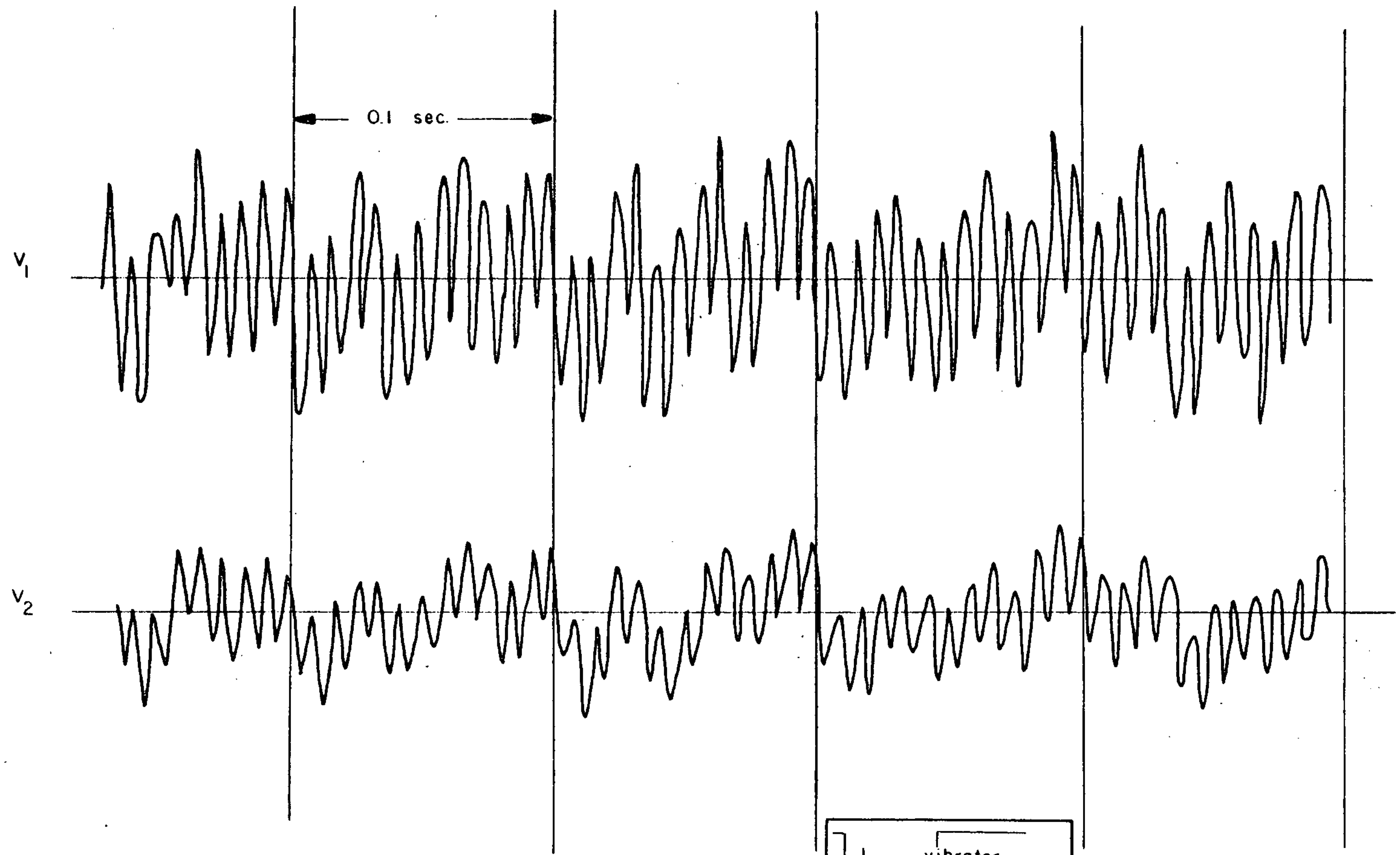
Project: SONGS SOIL STRUCTURE INTERACTION
Job Number: B2211

RECORDED DATA, RAYLIEGH WAVE TEST

Fig. B-2

WOODWARD-McNEILL & ASSOCIATES

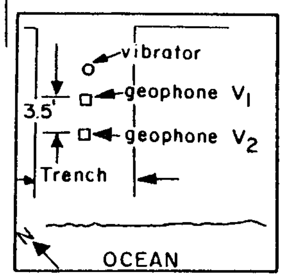
O-1



F = 30 cps

Test No. R-9

Geophone V_2 at 3.5 ft. from V_1



LOCATION PLAN

APPENDIX C

ATTENUATION TEST

Of the three Rayleigh wave traverses described in Appendix B, the third traverse, which utilized a vibrating sheepsfoot roller as the source of motion, also provided data for wave attenuation measurements. The test was carried out by inducing a large amplitude vibration in the ground and measuring the resulting vibration levels simultaneously with two geophones located on a straight line at different distances from the source. For uniform soils the attenuation of vibrations can be described by the following equation:

$$\frac{A_2}{A_1} = \left(\sqrt{\frac{r_1}{r_2}}\right) e^{-\alpha(r_2 - r_1)}$$

where the terms are defined on Fig. C-1.

The part of the equation under the radical indicates the portion of attenuation due to geometric energy dispersion with distance. The α term in the exponential part of the equation indicates the portion of attenuation due to the frequency-dependent internal damping or hysteresis of the soil.

Since the San Mateo Formation Sand at the site is fairly uniform, this equation should yield an adequate description of the soil damping properties. The results of the evaluation of "α" are presented graphically on Fig. C-2 for the test data shown on Fig. C-1 as well as another test performed with V_2 moved to a distance r_2 = 100 ft from the vibration source. Damping values can be calculated for the range of "α" indicated on Fig. C-2 by the following equation:

Appendix C

Page 2

$$\frac{D}{1-D^2} = \frac{V\alpha}{2\pi F}$$

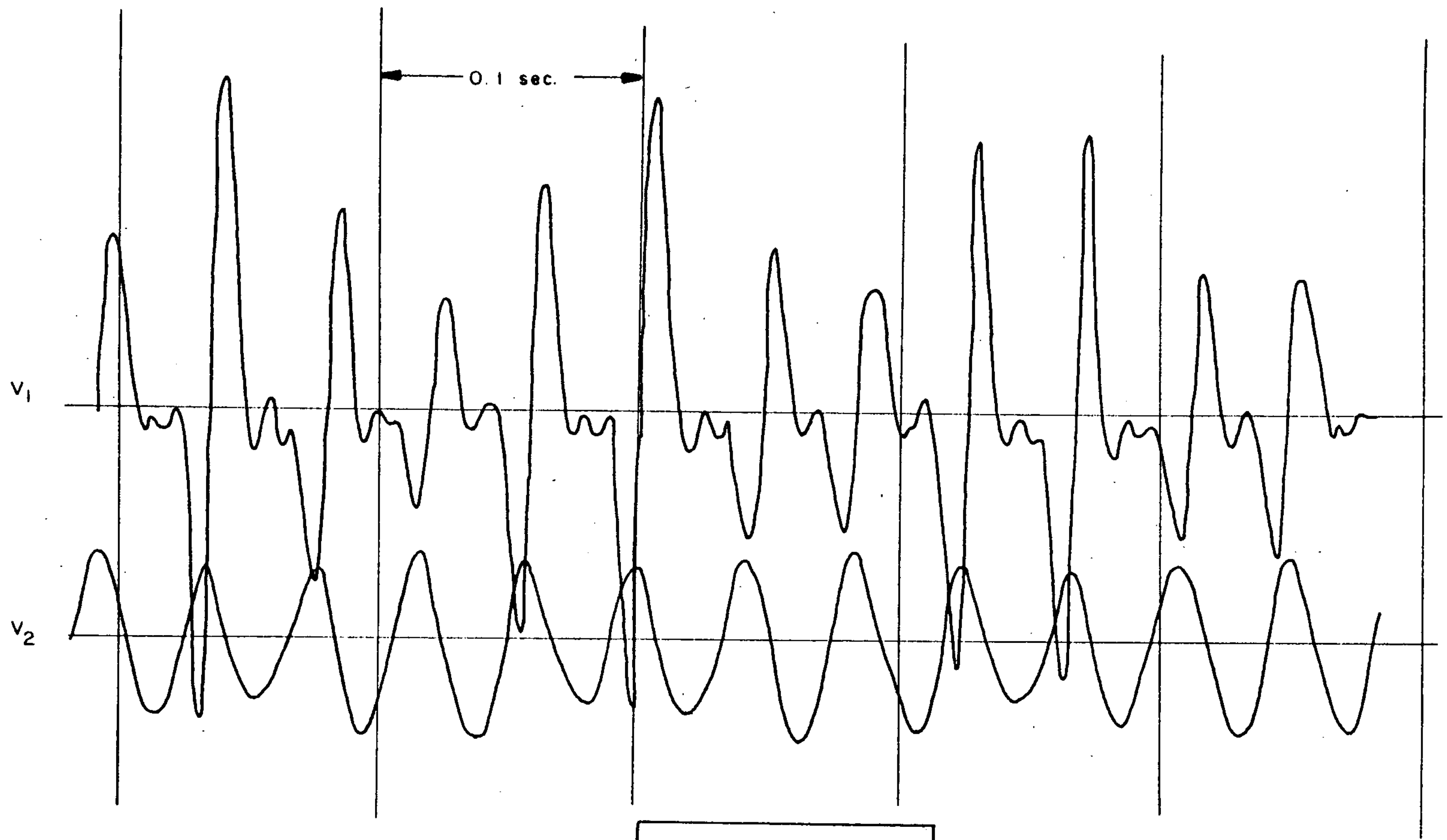
where D = damping
V = wave velocity
F = frequency of vibration
α = attenuation constant

This calculation yields a hysteretic damping of 4-1/2 to 7% of critical. These values are comparable to those determined by laboratory tests as outlined in the report entitled *Elastic and Damping Properties, Laydown Area, San Onofre Nuclear Generating Station,* Fig. F-2 and F-3 (Ref. 4).

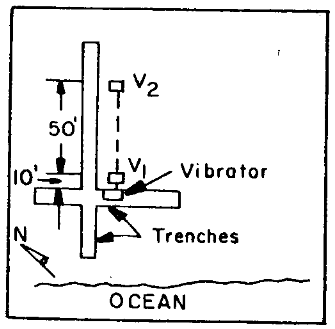
Project: SONGS SOIL STRUCTURE INTERACTION
Job Number: B221I

RECORDED FIELD DATA
ATTENUATION TEST

Fig. C-1



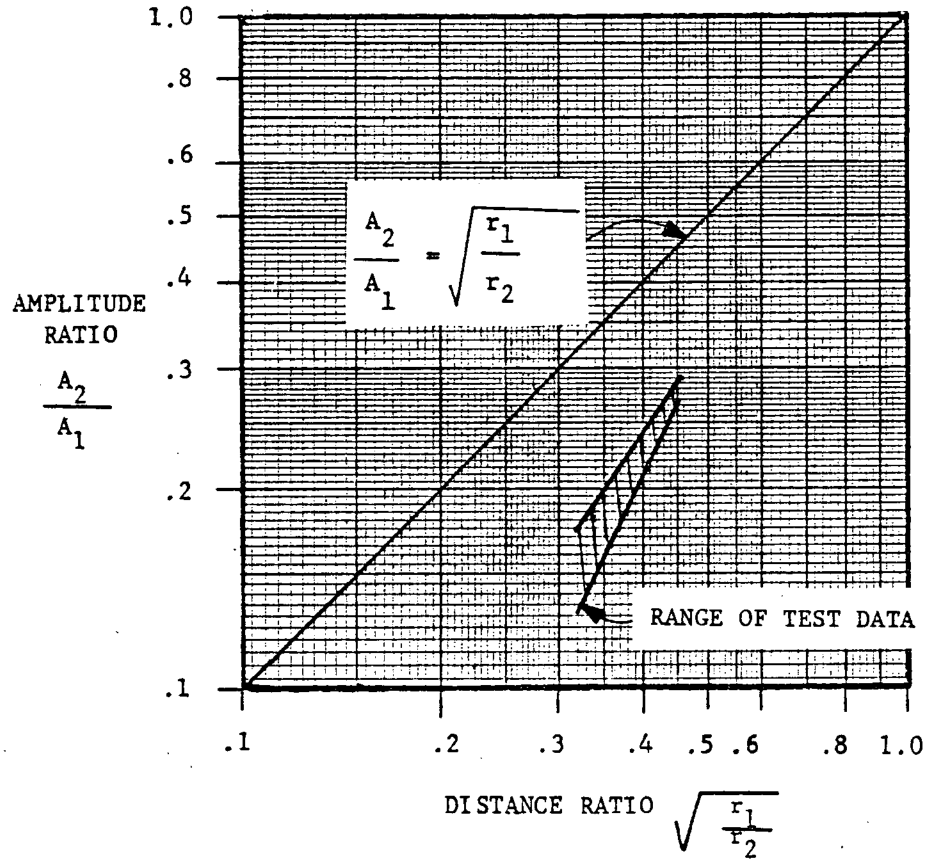
$F = 23$ cps
Test No. R-14
V_2 at 50 ft. from V_1
$r_1 = 10'$
$r_2 = 60'$
$\left.\begin{array}{l} A_1 \propto V_1 \\ A_2 \propto V_2 \end{array}\right\}$ for equal values of frequency



LOCATION PLAN

WOODWARD-McNEILL & ASSOCIATES

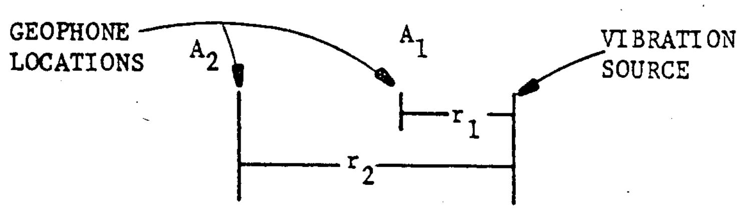
C-1



$$\frac{A_2}{A_1} = \sqrt{\frac{r_1}{r_2}}\, e^{-\alpha\,(r_2-r_1)}$$

WHERE α = .008 TO .012 FOR THE RANGE OF TEST DATA

DEFINITION OF TERMS



Project: SONGS SOIL STRUCTURE INTERACTION Job Number: B221I	RESULTS OF ATTENUATION TESTS	Fig. C-2

WOODWARD-McNEILL & ASSOCIATES

APPENDIX D

DAMPING

D-1 General

Both hysteretic and spatial damping are of importance for design. Hysteretic damping refers to that portion of the damping influenced by the internal behavior of a soil mass. It is a function of the stress-strain hysteresis of the soil. Spatial, or geometric damping refers to the decay of vibrations with distance from the source of motion. The determination of these quantities is discussed below.

D-2 Hysteretic Damping

The hysteretic damping of the site soils has been calculated from attenuation test results as described in Appendix C, and has been previously studied and reported on in Ref. 4. The data presented in that report were based on dynamic strain-control triaxial test results. A summary plot of the results of those tests is presented on Fig. D-1.

The field tests performed provided supportive data for the laboratory results. The range of results of attenuation tests calculated in Appendix C is indicated on Fig. D-1. These values show general agreement with the laboratory determined curve.

D-3 Spatial Damping

For most dynamic design conditions spatial (geometric) damping is more significant than hysteretic damping. The design equations for the calculation of damping values are presented in Table D-1 (Ref. 16). The effective radius, r_e, in these equations depends on the stress distribution below the slab

(Ref. 16). Therefore, it is necessary to know the stress distribution beneath the slab. The values of effective radius for the assumed stress distributions are given below (Ref. 16):

Distribution of Contact Pressures	Values of Effective Radius (r_e)
Rigid	$r_e = r$
Uniform	$r_e = .808\ r$
Parabolic	$r_e = .606\ r$

Figures D-2, D-3, and D-4 present the calculated damping curves for geometric damping for the range of effective radius values listed above, as well as the field test results. It should be noted that the field results are for total damping (hysteretic plus geometric). Therefore, the data points could be about 4-1/2 to 7% lower than shown on Figs. D-2, D-3, and D-4. From these results, it can be seen that there is a fairly wide range of results for the translational modes, while the range is relatively narrow for the rotational modes. Based on these results, it is concluded that an effective radius of 0.6 and 0.8 of the actual foundation radius results is a conservative estimate of damping for the translational and rotational modes, respectively.

The scatter in the test-results indicates there is no grouping of test-results according to embedment conditions; therefore, it is concluded that embedment conditions do not appreciably affect damping for this site.

TABLE D-1

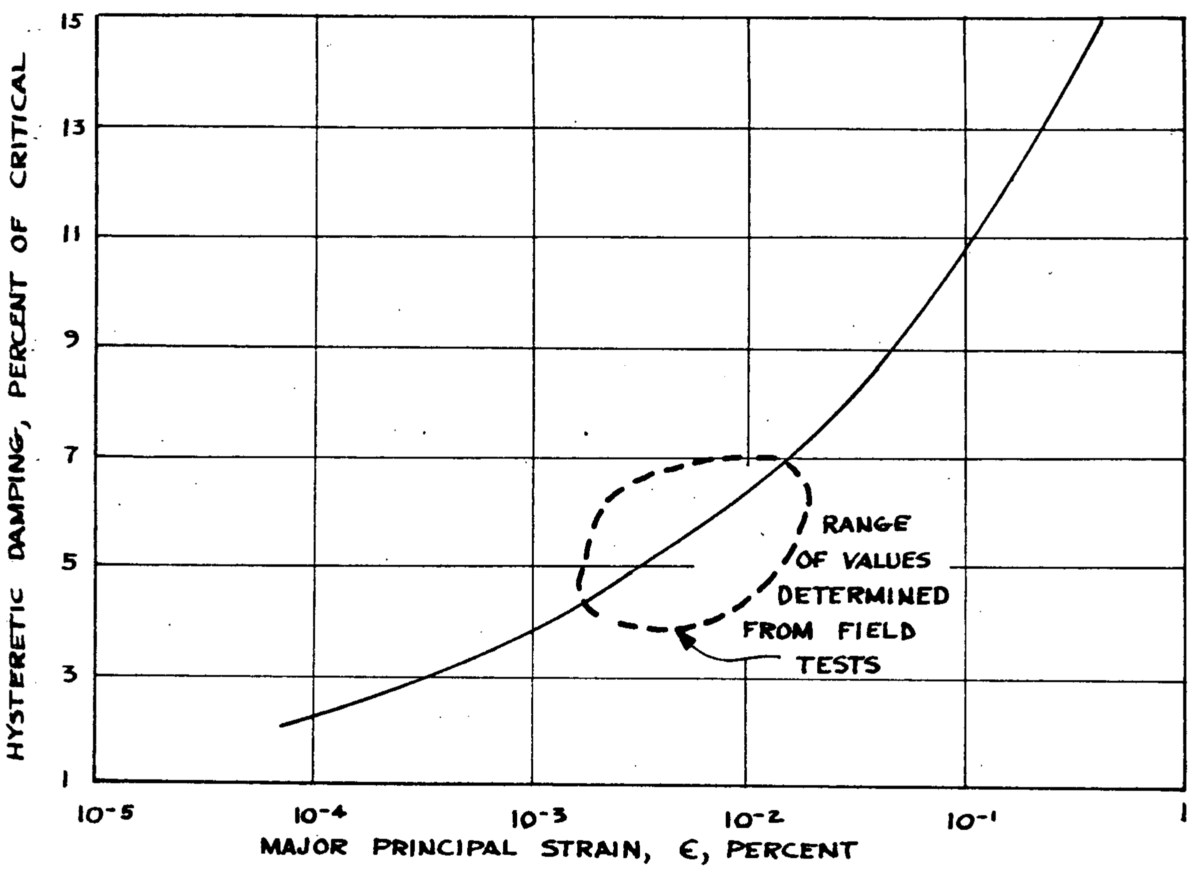
DESIGN PARAMETERS

Parameters	Mode of Motion: VERTICAL TRANSLATION	HORIZONTAL TRANSLATION	ROCKING	TWISTING
Inertia	m, mass of foundation and machine	m, mass of foundation and machine	I_r, mass moment of inertia about rocking axis	I_t, mass moment of inertia about twist axis
Radius	$r = \sqrt{\frac{BL}{\pi}}$	$r = \sqrt{\frac{BL}{\pi}}$	$r = \sqrt[4]{\frac{BL^3}{6\pi}}$	$r = \sqrt[4]{\frac{BL(B^2 + L^2)}{6\pi}}$
Inertia Ratio	$B_v = \frac{(1-\nu)m}{4\rho r_e^3}$	$B_h = \frac{(7-8\nu)m}{32(1-\nu)\rho r_e^3}$	$B_r = \frac{3(1-\nu)I_r}{8\rho r_e^5}$	$B_t = \frac{I_t}{\rho r_e^5}$
Effective Inertia for Design	m	m	$I'_r = \eta_r I_r$	I_t
Geometric Damping Coefficient	$c_v = \frac{3.4 r^2 \sqrt{G\rho}}{(1-\nu)}$ ($B_v \geq 0.36$)	$c_h = \frac{18.4(1-\nu) r^2 \sqrt{G\rho}}{(7-8\nu)}$ ($B_h \geq 0.17$)	$c_r = \frac{0.80 r^4 \sqrt{G\rho}}{(1-\nu)(1+B_r \eta_r)}$	$c_t = \frac{2.31 r^4 \sqrt{G\rho} \sqrt{B_t}}{1 + 2B_t}$
Geometric Damping	$D_v = \frac{0.425}{\sqrt{B_v}}$	$D_h = \frac{0.288}{\sqrt{B_h}}$	$D_r = \frac{0.15}{(1+B_r)\sqrt{B_r}}$	$D_t = \frac{0.50}{1+2B_t}$

Note : for square or rectangular foundation - B = width of foundation in plan (parallel to axis of rotation)

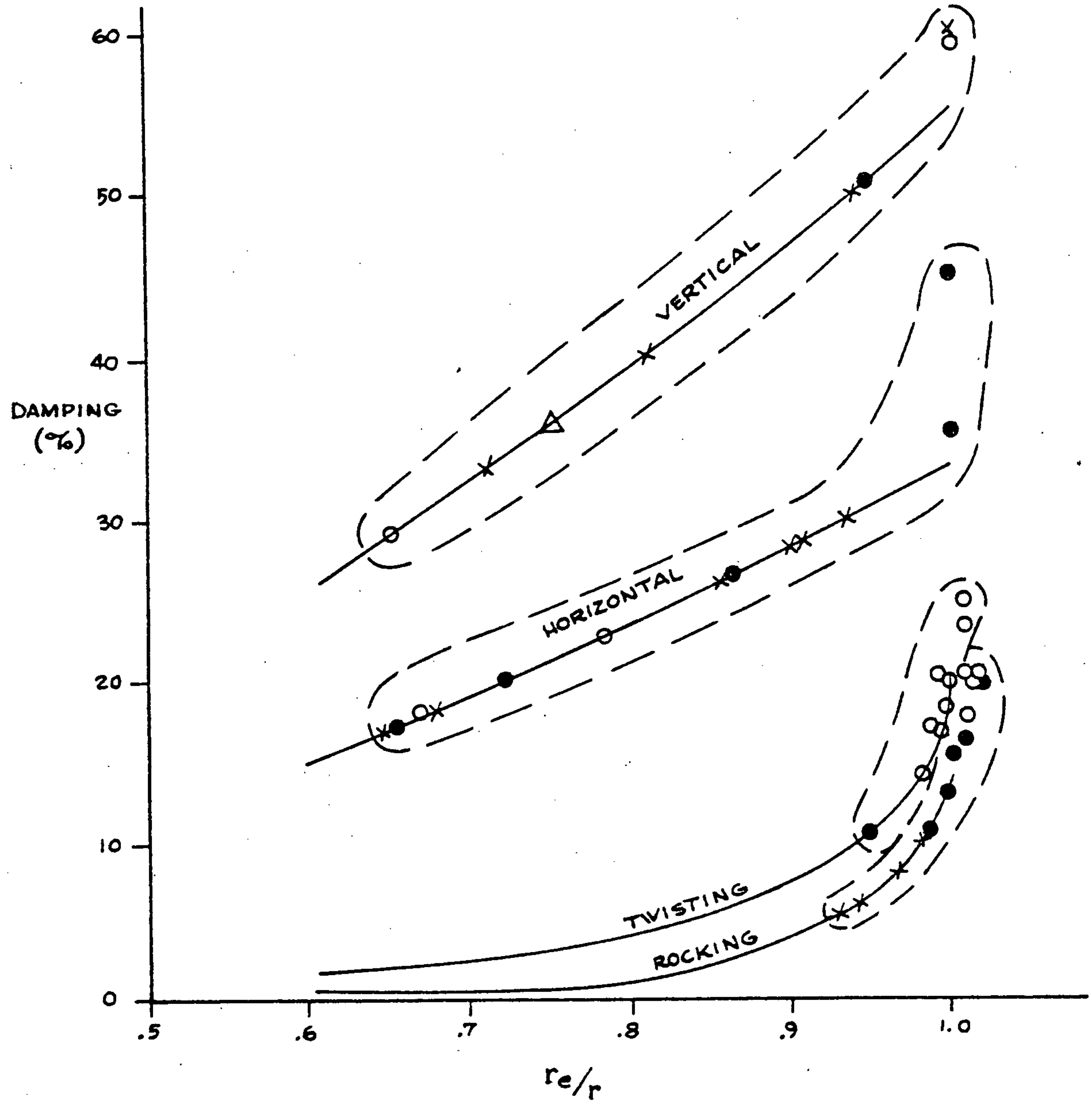
L = length of foundation in plan (perpendicular to axis of rotation)

Ref. 16



Project: SONGS SOIL STRUCTURE INTERACTION **Job Number:**	VALUES OF HYSTERETIC DAMPING	**Fig.** D-1

WOODWARD - McNEILL & ASSOCIATES



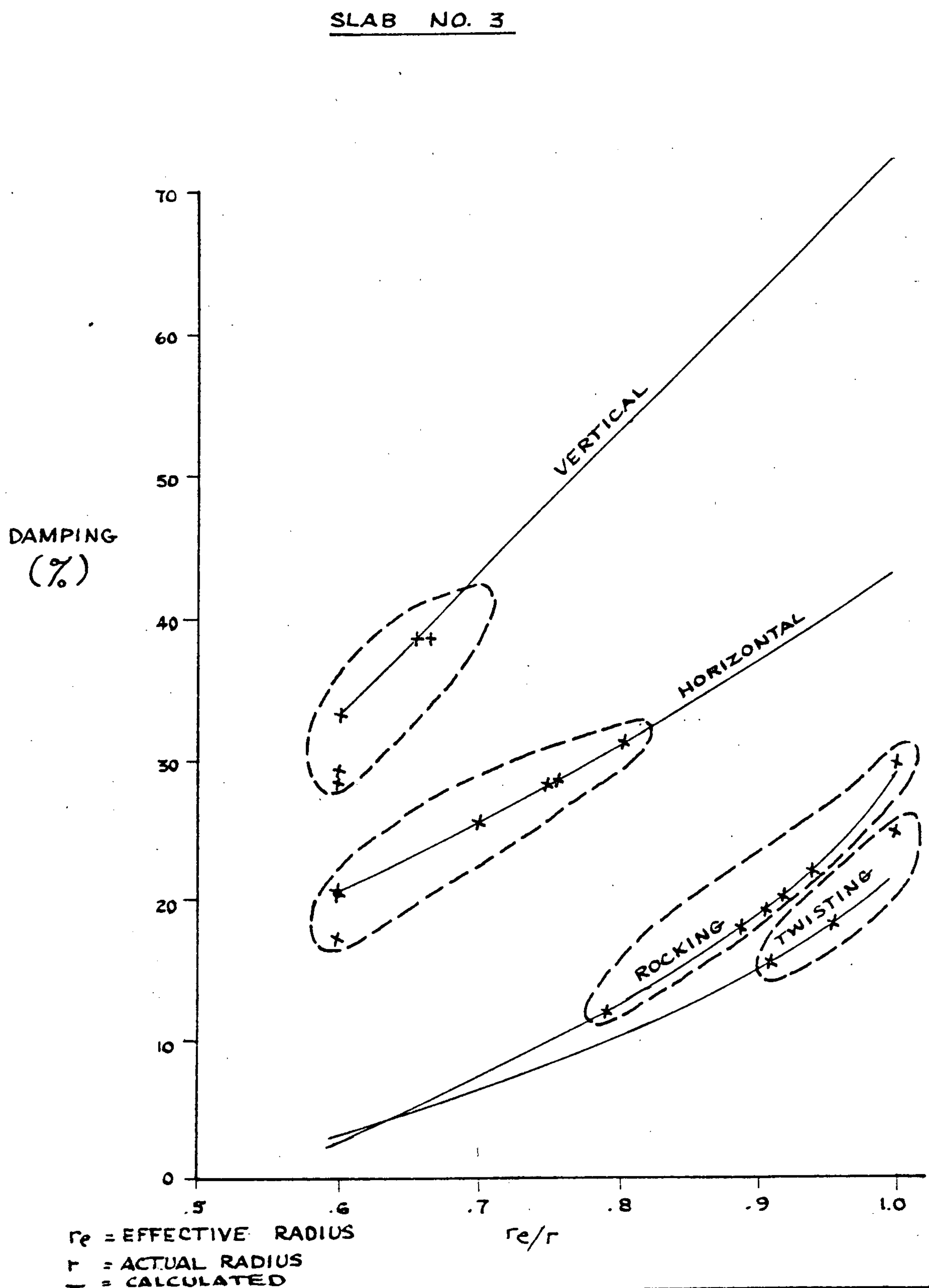
r_e = EFFECTIVE RADIUS
r = ACTUAL RADIUS
— = CALCULATED

SYMBOL	SLAB NO.
● X	1 & 2
O	2A
△	2C

Project: SONGS SOIL STRUCTURE INTERACTION Job Number: B221I	DAMPING VERSUS EFFECTIVE RADIUS	Fig. D-2

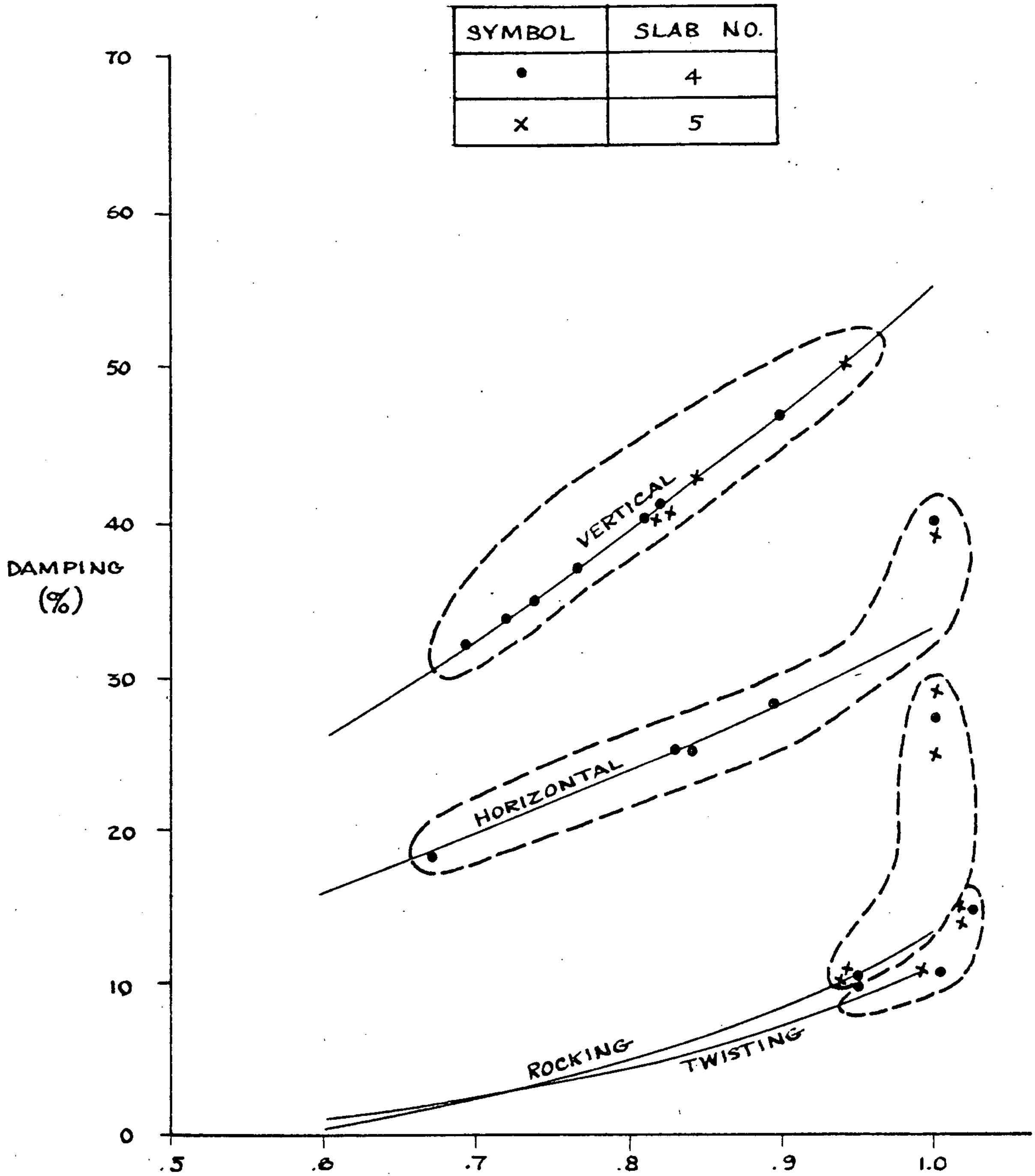
0-1

WOODWARD-McNEILL & ASSOCIATES



Project: SONGS SOIL STRUCTURE INTERACTION Job Number: 3221I	DAMPING VERSUS EFFECTIVE RADIUS SLAB NO. 3	Fig. D-3

WOODWARD - McNEILL & ASSOCIATES



r_e = EFFECTIVE RADIUS
r = ACTUAL RADIUS
— = CALCULATED

Project: SONGS SOIL STRUCTURE INTERACTION **Job Number:** B221I	DAMPING VERSUS EFFECTIVE RADIUS SLAB NO. 4 AND 5	Fig. D-4

WOODWARD - McNEILL & ASSOCIATES

APPENDIX E

EVALUATION OF SPRING CONSTANTS

E-1 Introduction

The choice of an appropriate soil stiffness (of spring constant K), is fundamentally important to dynamic foundation design because its value is the most significant unknown in the determination of fundamental frequency, and it governs the static displacement of the foundation and the maximum amplitude of dynamic motion. The field tests performed on the Laydown Area at the site (Unit 1) provided information required to evaluate the compatibility of structural response calculations with data previously presented in the materials report (Ref. 4). In addition, because of the unique geometry of some of the proposed structure foundations, for which analytical solutions are not available, these tests were performed to develop adjustment factors for existing solutions and to determine the effects of various embedment conditions on structural response.

Whereas damping values were determined by analysis of the attenuation of vibrations after each slab was set into motion (as described in Appendix D), the evaluation of spring constants was made by analyzing frequency response of each slab.

The previous values given for spring constants (see Appendix H, Ref. 4) have been further studied and refined and are reported on in this report. Figure E-4 of that report has been included in this report as Fig. E-1, for ease of reference, because it has been used for the determination of effective Shear Modulus.

E-2 Shear Modulus Determination

In order to evaluate spring constants for the soil-structure systems at the site, it is necessary to determine shear modulus values (G) for each test pad. Since G varies with strain (see Fig. E-1), it is necessary to determine the strain developed in the soil during dynamic loadings. In the field tests performed for this study, the slabs were set into transient motion, which produced a displacement and corresponding strain in the supporting soils. Based on our experience, we have assumed that the strain is accommodated within a depth of one radius below the slab, and that an average shear modulus can be calculated at a depth of half the radius below the base of the slab. Based on our previous work at this site, we have determined that for the San Mateo Formation Sand, the shear modulus should be calculated according to the following equation:

$$G = 100\ K_m\ (\sigma_m)^{2/3}$$

where $\sigma_m = \frac{2}{3}\ (x\gamma' + mp)$

where K_m = strain dependent parameter as determined from Fig. E-1
x = depth below base
γ' = effective unit weight
m = stress reduction factor
p = net bearing pressure

Figure E-2 presents the relationship between confinement (σ_m), strain (ε), and shear modulus (G) for the soils at the site. The values determined for each test slab are also shown on the figure.

E-3 Basic Equations

Analytical solutions have been developed for the calculation of spring constants for the vertical, horizontal, rocking, and twisting modes of motion of a rigid circular footing resting on an elastic half space (Ref. 16). These relations are presented in Table E-1.

In terms of frequency response, the basic relationship between undamped natural frequency (F_n) and stiffness is given by:

$$F_n = \frac{1}{2\pi} \sqrt{\frac{\text{stiffness}}{\text{inertia}}}$$

Where stiffness = k_v, k_h, k_r, or k_t

inertia = m_v, m_h, I_r, or I_t

The undamped natural frequency (F_n) and damped natural frequency (F_d), which is measured in our field tests, are related as given below:

$$F_d = F_n \sqrt{1-D^2}$$

Where D = damping factor

It can be seen by review of the damping factors determined from the field tests (Appendix D) that F_n and F_d are approxi-

TABLE E-1

DESIGN PARAMETERS

Parameters	Mode of Motion: VERTICAL TRANSLATION	HORIZONTAL TRANSLATION	ROCKING	TWISTING
Inertia	m, mass of foundation and machine	m, mass of foundation and machine	I_r, mass moment of inertia about rocking axis	I_t, mass moment of inertia about twist axis
Radius	$r = \sqrt{\frac{BL}{\pi}}$	$r = \sqrt{\frac{BL}{\pi}}$	$r = \sqrt[4]{\frac{BL^3}{3\pi}}$	$r = \sqrt[4]{\frac{BL(B^2 + L^2)}{6\pi}}$
Inertia Ratio	$B_v = \frac{(1-\nu)m}{4\rho r^3}$	$B_h = \frac{(7-8\nu)m}{32(1-\nu)\rho r^3}$	$B_r = \frac{3(1-\nu)I'_r}{8\rho r^5}$	$B_t = \frac{I_t}{\rho r^5}$
Effective Inertia for Design	m	m	$I'_r = \eta_r I_r$	I_t
Stiffness Coefficient	$k_v = \frac{4Gr}{(1-\nu)}$	$k_h = \frac{32(1-\nu)Gr}{(7-8\nu)}$	$k_r = \frac{8Gr^3}{3(1-\nu)}$	$k_t = \frac{16Gr^3}{3}$
Stress Distribution	rigid	uniform	rigid	rigid

Note: for square or rectangular foundation - B = width of foundation in plan (parallel to axis of rotation)

L = length of foundation in plan (perpendicular to axis of rotation)

Figure E-2 presents the relationship between confinement (σ_m), strain (ε), and shear modulus (G) for the soils at the site. The values determined for each test slab are also shown on the figure.

E-3 Basic Equations

Analytical solutions have been developed for the calculation of spring constants for the vertical, horizontal, rocking, and twisting modes of motion of a rigid circular footing resting on an elastic half space (Ref. 16). These relations are presented in Table E-1.

In terms of frequency response, the basic relationship between undamped natural frequency (F_n) and stiffness is given by:

$$F_n = \frac{1}{2\pi}\sqrt{\frac{\text{stiffness}}{\text{inertia}}}$$

Where stiffness = k_v, k_h, k_r, or k_t

inertia = m_v, m_h, I_r, or I_t

The undamped natural frequency (F_n) and damped natural frequency (F_d), which is measured in our field tests, are related as given below:

$$F_d = F_n\sqrt{1-D^2}$$

Where D = damping factor

It can be seen by review of the damping factors determined from the field tests (Appendix D) that F_n and F_d are approxi-

TABLE E-1

DESIGN PARAMETERS

Parameters	Mode of Motion: VERTICAL TRANSLATION	HORIZONTAL TRANSLATION	ROCKING	TWISTING
Inertia	m, mass of foundation and machine	m, mass of foundation and machine	I_r, mass moment of inertia about rocking axis	I_t, mass moment of inertia about twist axis
Radius	$r = \sqrt{\frac{BL}{\pi}}$	$r = \sqrt{\frac{BL}{\pi}}$	$r = \sqrt[4]{\frac{BL^3}{3\pi}}$	$r = \sqrt[4]{\frac{BL(B^2 + L^2)}{6\pi}}$
Inertia Ratio	$B_v = \frac{(1-\nu)m}{4\rho r^3}$	$B_h = \frac{(7-8\nu)m}{32(1-\nu)\rho r^3}$	$B_r = \frac{3(1-\nu)I'_r}{8\rho r^5}$	$B_t = \frac{I_t}{\rho r^5}$
Effective Inertia for Design	m	m	$I'_r = \eta_r I_r$	I_t
Stiffness Coefficient	$k_v = \frac{4Gr}{(1-\nu)}$	$k_h = \frac{32(1-\nu)Gr}{(7-8\nu)}$	$k_r = \frac{8Gr^3}{3(1-\nu)}$	$k_t = \frac{16Gr^3}{3}$
Stress Distribution	rigid	uniform	rigid	rigid

Note: for square or rectangular foundation - B = width of foundation in plan (parallel to axis of rotation)

L = length of foundation in plan (perpendicular to axis of rotation)

mately equal. Table E-2, below, presents equations for spring constants as a function of frequency.

TABLE E-2

Motion	Spring Constant
Vertical and Horizontal	$k_{v,h} = 4\pi^2F^2m$
Rocking	$k_r = 4\pi^2F^2I_r$
Twisting	$k_t = 4\pi^2F^2I_t$

Where m = mass of slab

I_r = moment of inertia in Rocking Mode

I_t = moment of inertia in Twisting Mode

From the equations of Tables E-1 and E-2, spring constants can be calculated for each slab as a function of G and F, respectively. These relationships are presented in Table E-3 below:

Appendix E

Page 5

TABLE E-3
k=f(G,F)

Motion	SLABS 1 & 2 Spring Constant k		SLAB 3 Spring Constant k		SLABS 4 & 5 Spring Constant k	
	k=XGC	k=YF^2	k=XGC	k=YF^2	k=XGC	k=YF^2
Vertical	k_v=30.8GC	k_v=72.4F^2	k_v=30.8GC	k_v=42.4F^2	k_v=12.1GC	k_v=4.65F^2
Horizontal	k_h=25GC	k_h=72.4F^2	k_h=25GC	k_h=42.4F^2	k_h=9.7GC	k_h=4.65F^2
Rocking	k_r=512GC	k_r=1040F^2	k_r=512GC	k_r=390F^2	k_r=32.4GC	k_r=10.9F^2
Twisting	k_t=665GC	k_t=905F^2	k_t=665GC	k_t=330F^2	k_t=42.7GC	k_t=9.3F^2

where $C = C_1 x\, C_2 x\, C_3 x\, C_4$ = correction factor

C_1 = Empirical Correction Factor
C_2 = Embedment Correction Factor
C_3 = Scaling Correction Factor
C_4 = Shape Correction Factor

These correction factors are discussed in the sections which follow.

E-4 Empirical Correction Factor-C_1

The value of C_1 is determined by comparing the theoretical value of k (based on k = XGC, Table E-3) with the value of k determined from actual frequency response measurements obtained in the field (k = YF^2) from a circular slab on the ground surface (slab - 1). From this comparison, values for the empirical correction factor, C_1, were calculated for each mode. These values are presented below:

Appendix E

Page 6

Mode	Empirical Correction Factor - C_1
Vertical	0.81
Horizontal	1.0
Rocking	0.66
Twisting	0.41

The values for C_1 presented above are probably due to the actual stress distributions. For the vertical mode, theoretical values are consistent to a rigid base stress distribution (Table E-1). Field tests indicated that a correction factor of .81 should be applied, which is consistent to a uniform stress distribution (r_e = .808r).

For the horizontal mode field tests, the values calculated from measured frequency agreed with the theoretical values; therefore C_1 equals 1.0. For this mode, uniform stress distribution is consistent to the theoretical equation (Table E-1).

For the rotational modes, stiffness is proportional to radius to the third power, therefore for uniform stress distribution where $r_e = .81r$, $r_e^3 = 0.53r^3$. The values determined for C_1 for the rocking and twisting modes were 0.66 and 0.41, respectively, which are reasonably close to the calculated values of 0.53

E-5 Embedment Correction Factor - C_2

Because there is relatively little data available which describe the effect of embedment on stiffness, field tests were performed for this purpose. Test-results for Slab No. 2 (embedded) were compared with those for Slab No. 1 (non-embedded), and the results plotted as a graph of slab embedment to radius ratio (h/r) versus spring constant ratio $\left(\frac{k \text{ embedded}}{k \text{ non-embedded}}\right)$. The values of k were calculated from frequency response data according to the equations of Table E-3. These graphs are presented on Figs. E-3, E-4, E-5 and E-6. It should be noted that these curves are based on one set of data points (at h/r=1.0), and the shape of the curves has been estimated from previous work by Kaldjian (1968). From the data of Figs. E-3 through E-6 an embedment correction factor, C_2, is determined as a function of the degree of embedment.

E-6 Scaling Correction Factor-C_3

To evaluate the effects of scaling, tests were made on circular slabs 4 to 10 ft in diameter as shown in Appendix A (Slabs 4 and 2, respectively). The ratio of frequency responses for two slabs of similar geometry but different size is given by:

$$\frac{F_2}{F_1} = \sqrt{\frac{k_2 \ m_1}{m_2 \ k_1}}$$

Where m = mass or inertia
k = stiffness

For the two slabs tested to evaluate the effects of scaling, a calculation was made of the theoretical frequency response ratio for each mode of vibration. Taking into account the difference in shear modulus for the two slabs, these calculations indicate that the response of the 10 ft diameter slab (No. 2) should have a frequency of 65 to 70% of that of the 4 ft diameter slab (No. 4) for all modes. A review of the frequency response test-results presented in Appendix A indicates that the ratio of the measured responses for these slabs did not deviate appreciably from the theoretical ratio calculated for each case. This indicates that a scaling correction factor (C_3) of 1.0 can be used for engineering analysis.

E-7 Shape Correction Factor - C_4

A number of tests were made on slabs with various shape-characteristics in order to determine differences in responses. Two of the slabs (Nos. 4 and 5) had approximately the same mass, thickness, inertia, and embedment conditions. The significant difference between the two was in their shapes. One was circular (No. 4) while the other was square (No. 5). A review of the test-results presented in Appendix A, Table A-1, indicates that there was no appreciable difference in measured frequency response; therefore, it is concluded that a correction factor (C_4) of 1.0 for foundation shape can be utilized for engineering design for normal shaped foundations (i.e., circular, square, rectangular).

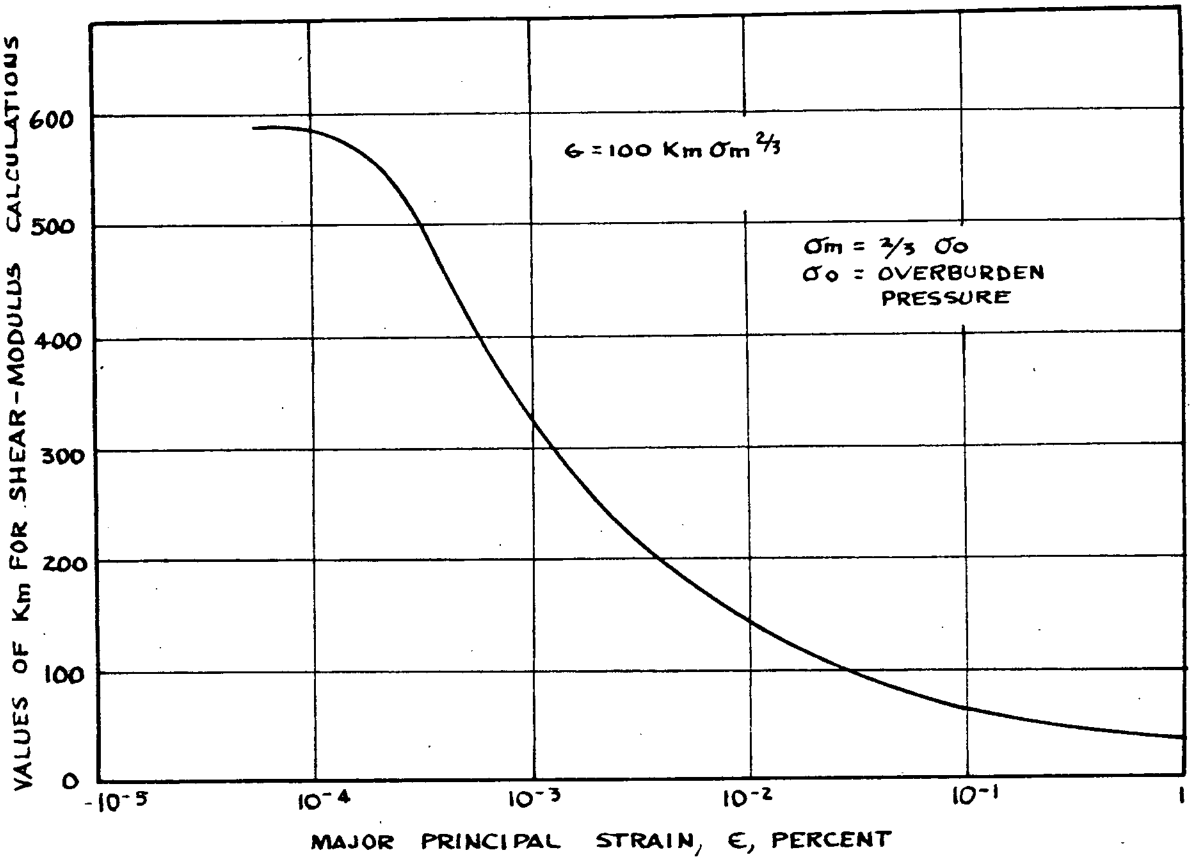
Tests were also made on slabs to evaluate the effects on response of the special foundation shape of the proposed containment structures for Units 2 and 3. Slab No. 3, shown in Appendix A, was constructed for this purpose. Tests were made on this slab and compared to the theoretical calculation for a slab with the same overall dimensions, but of uniform geometry (cylindrical). For the shape of the containment structure, the overall correction factors presented on the following table were determined:

Mode	Correction Factor-C
Vertical	1.075
Horizontal	1.09
Rocking	0.60
Twisting	N.A.

It should be noted that the correction factors presented above take into account not only the correction for the complex shape of the foundation, but also the unique embedment conditions of the containment structure (see Appendix A, Fig. A-4) and the empirical correction factor. The scaling correction factor was assumed to be 1.0 as discussed in Section E-6 above.

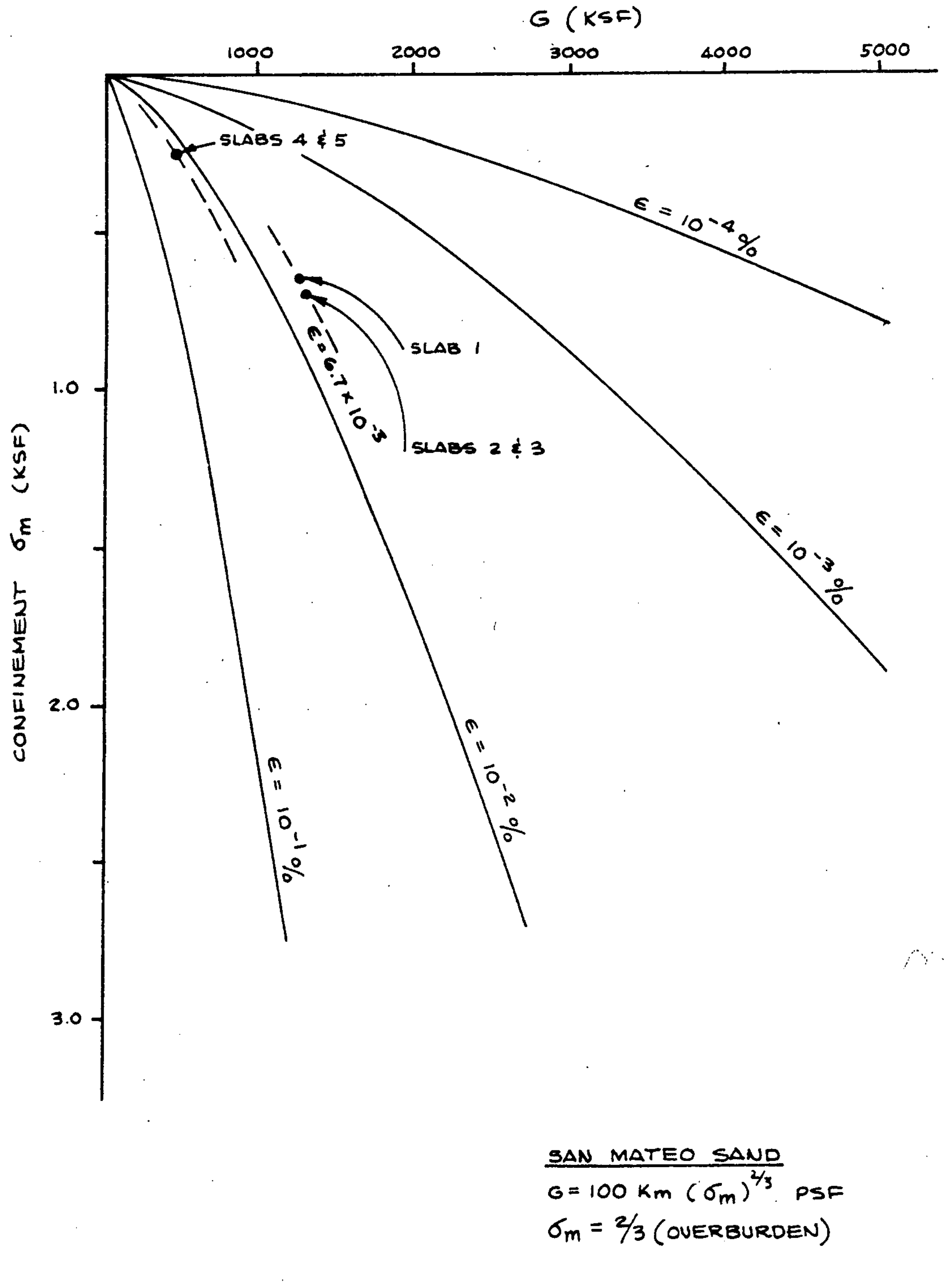
E-8 Special Consideration for Rocking Mode Inertia

Table E-4 presents design equations for stiffness calculations. For the rocking mode, the effective inertia for the rocking mode is given as a function of η_r and B_r. For the slabs tested in this study η_r was not significantly greater than 1.0; however for design, the value of η_r should be calculated and the moment of inertia adjusted by η_r as indicated from Fig. E-7.



Project: SONGS SOIL STRUCTURE INTERACTION **Job Number:** B221I	RECOMMENDED MODULUS VALUES	Fig. E-1

WOODWARD - McNEILL & ASSOCIATES

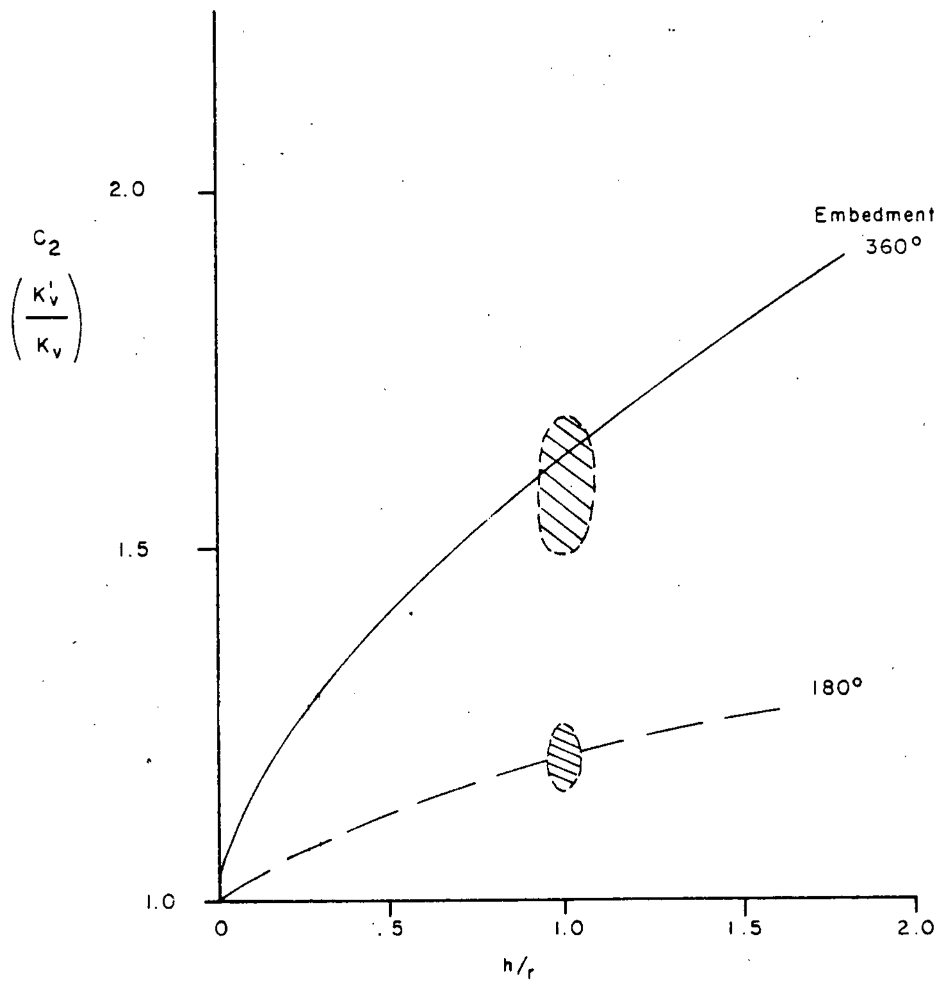


Project: SONGS SOIL STRUCTURE INTERACTION Job Number: B221I	SHEAR MODULUS FOR TEST SLABS	Fig. E-2

WOODWARD-McNEILL & ASSOCIATES

VERTICAL MODE

Note: Shape of curve estimated from previous work by Kaldjian (1968)

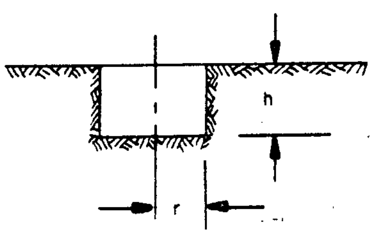


K' = embedded case

K = non-embedded case

C_2 = embedment correction factor

= range of data points

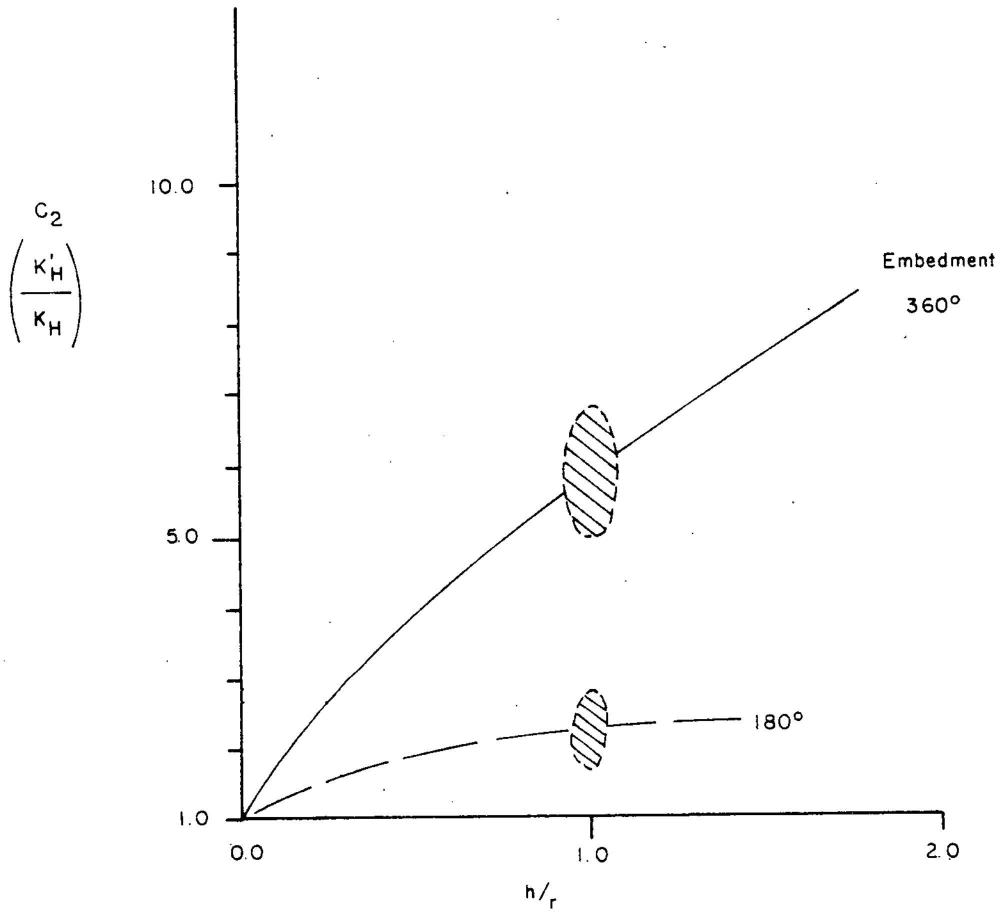


Project: SONGS SOIL STRUCTURE INTERACTION Job Number: B221I	EFFECT OF EMBEDMENT ON SPRING CONSTANT VERTICAL MODE	Fig. F-3

WOODWARD-McNEILL & ASSOCIATES

HORIZONTAL MODE

Note: Shape of curve estimated from previous work by Kaldjian (1968)

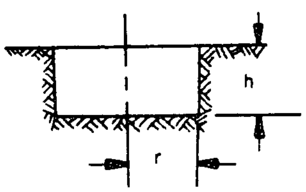


C_2 = embedment correction factor

K' = embedded case

K = non-embedded case

= range of data points

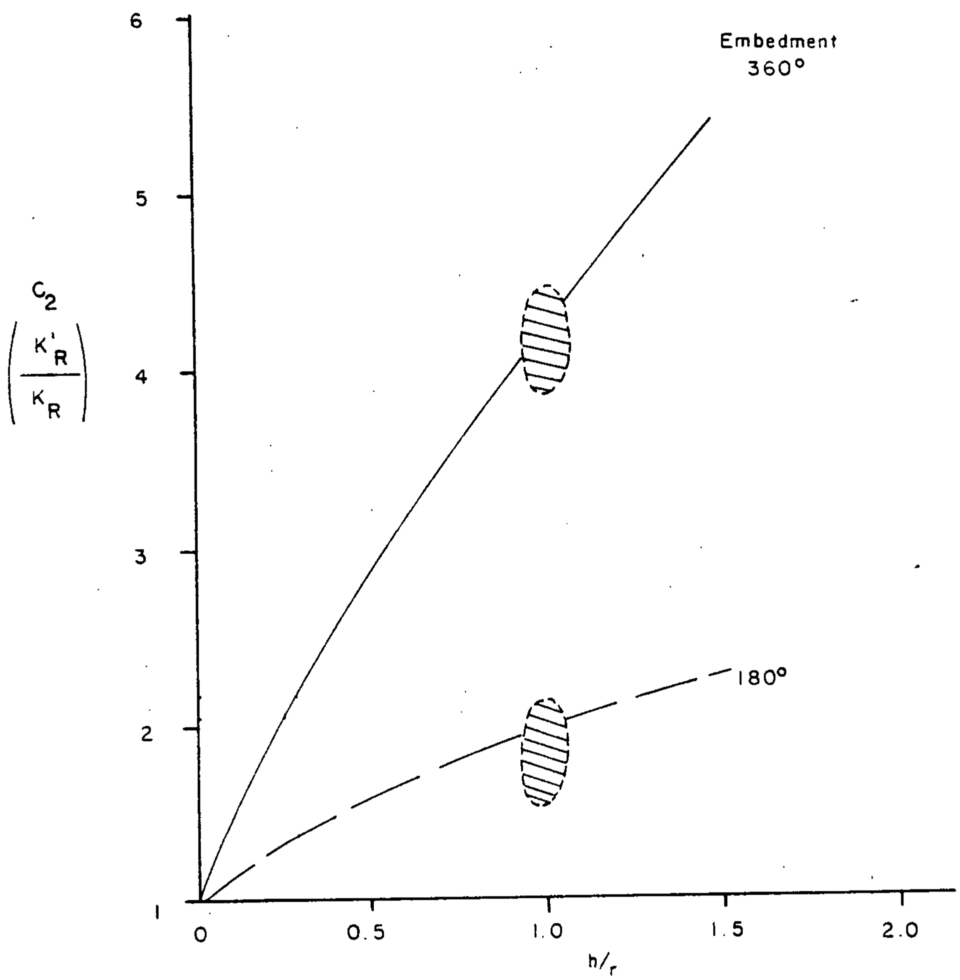


Project: SONGS SOIL STRUCTURE INTERACTION **Job Number** B221I	EFFECT OF EMBEDMENT ON SPRING CONSTANT, HORIZONTAL MODE	**Fig.** E-4

WOODWARD-McNEILL & ASSOCIATES

ROCKING MODE

Note: Shape of curve estimated from previous work by Kaldjian (1968)

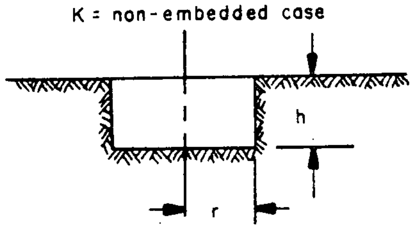


C_2 = embedment correction factor

K' = embedded case

K = non-embedded case

= range of data points



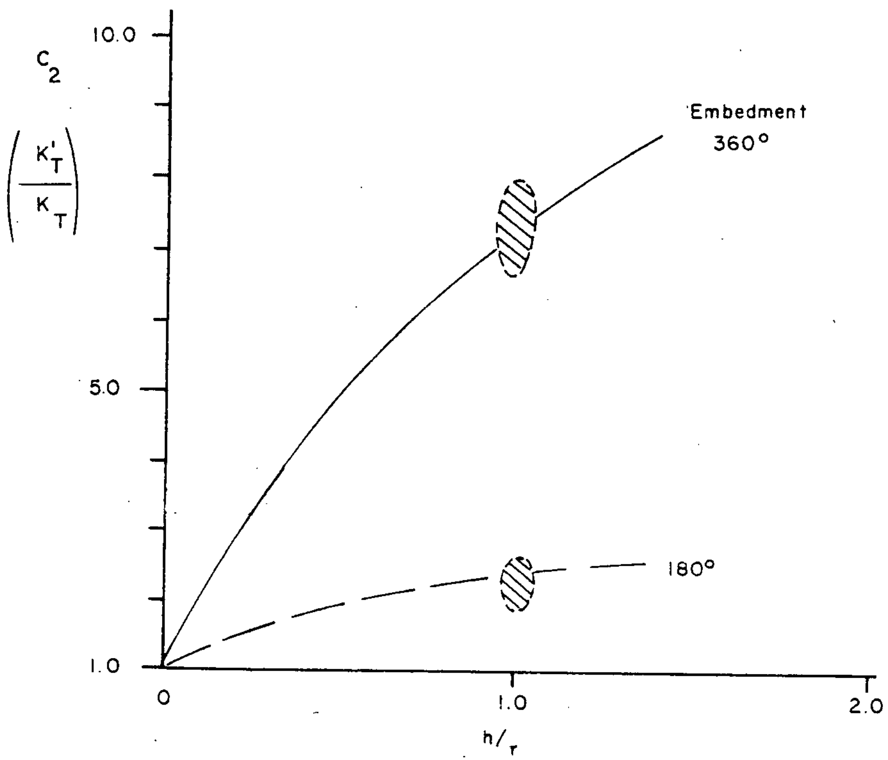
Project: SONGS SOIL STRUCTURE INTERACTION Job Number: B221I	EFFECT OF EMBEDMENT ON SPRING CONSTANT ROCKING MODE	Fig. E-5

WOODWARD-McNEILL & ASSOCIATES

0-1

TWISTING MODE

Note: Shape of curve estimated from previous work by Kaldjian (1968)

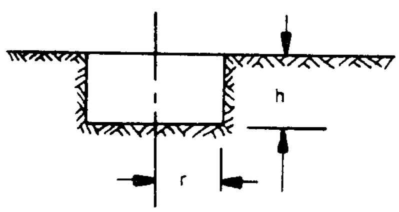


C_2 = embedment correction factor

K' = embedded case

K = non-embedded case

= range of data points



Project: SONGS SOIL STRUCTURE INTERACTION

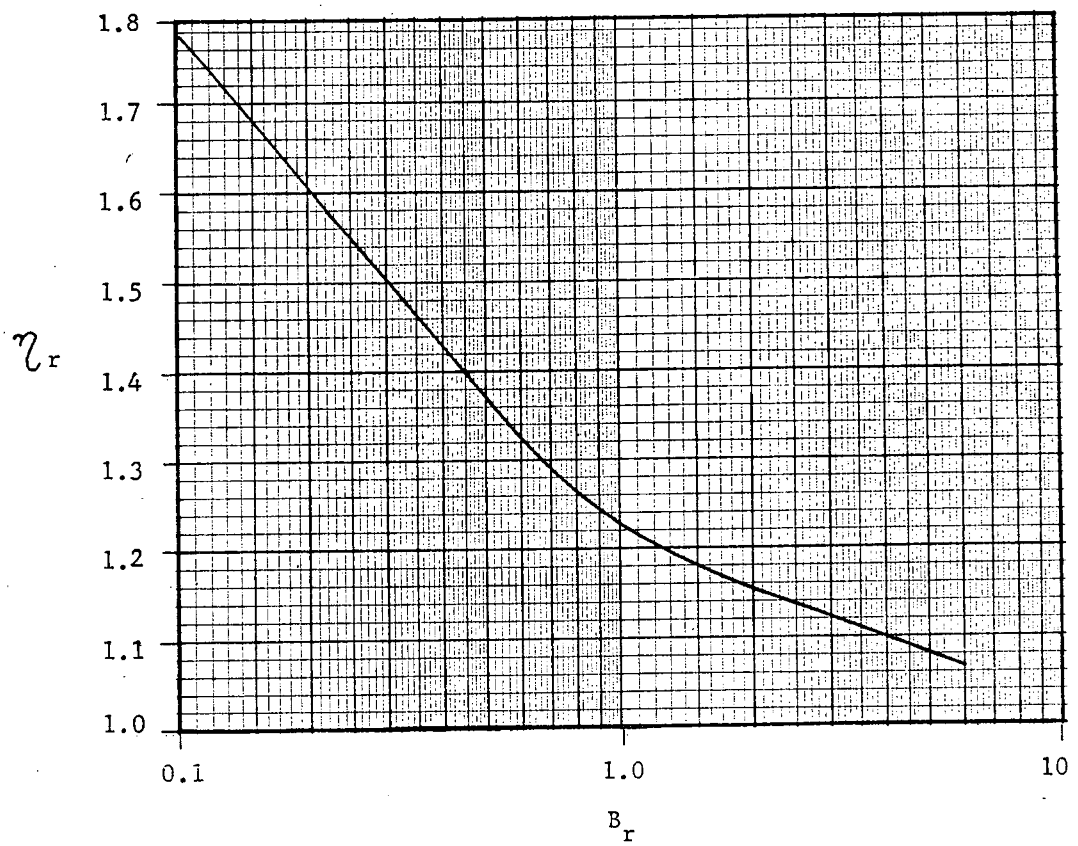
Job Number: B221I

EFFECT OF EMBEDMENT ON SPRING CONSTANT, TWISTING MODE

Fig. E-6

O-I

WOODWARD-McNEILL & ASSOCIATES



Project: SONGS SOIL STRUCTURE INTERACTION **Job Number:** B221I	INERTIAL EFFECTS ON ROCKING MODE (η_r vs B_r)	**Fig.** E-7

WOODWARD - McNEILL & ASSOCIATES

APPENDIX F

LATERAL PRESSURES ON STRUCTURE WALLS

F-1 General

Structures at the site will be constructed in the San Mateo Formation Sand. Field measurements indicate that the in-situ density of this material is on the order of 100% relative compaction as determined by ASTM D1557-70, and a dry density of about 120 pcf. Backfill will be compacted to a minimum of 95% relative compaction above the water table and 100% compaction below the water table. Laboratory tests indicate the soil has an angle of internal friction of 41.5^{0} and an effective cohesion of 750 psf, however in the spirit of conservatism the cohesion has been neglected for this analysis.

Three conditions causing lateral stresses on walls are discussed. The first, described by Seed and Whitman (1970), is a force-equilibrium analysis for which the critical angle of slope of the base of the wedge is determined to obtain maximum (active) earth pressure on walls. The second involves the calculation of lateral (passive) pressures mobilized due to inertial loads. The third involves the calculation of additional lateral pressures due to nearby structures. It is expected that during an earthquake the structure would be acted upon by both the active stress component on the side of the structure that at an instant in time was tending to move away from the soil, and by the passive stress component on the opposite side of the structure which was tending to move into the soil. Both conditions should be analyzed to evaluate the most critical stress for each wall element. The lateral stress caused by a nearby structure should then be added to the larger of these two stresses for design. Details of these techniques of analysis are presented in the following sections of this appendix.

F-2 Lateral Earth Pressures

The at-rest earth-pressure coefficient is considered applicable for evaluation of static stresses in the San Mateo Formation Sand against rigid side walls of structures. This coefficient (k_o) is obtained from the expression: $k_o = 1 - \sin \phi$ (Jaky, 1944). For the San Mateo Formation Sand this calculation yields $k_o = 0.34$. For the dynamic DBE and OBE loading conditions a wedge analysis approach was used to determine seismic earth pressure coefficients. The steps involved in this procedure are presented in Attachment F-1. The equivalent fluid pressures determined from these analyses are presented in Table F-1 below.

It should be noted that for static calculation, the at-rest earth-pressure coefficient was used, because of the assumption that the structures will be essentially rigid.

F-3 Lateral Stresses Due to Inertial Loading

The equivalent fluid pressures presented in the preceding section will act on one side of the structure during earthquake loading while stresses due to inertial loads will act on the opposite side, as the structure moves differentially with respect to the soil. Calculations should be made of both stresses, and the larger value used for design for each element of the wall.

The stress due to inertial loading can be determined as follows:

For Horizontal Translation,

$$\sigma_p = \frac{(C_2 - 1)\ P}{C_2\ A} \qquad \text{(uniform stress distribution).}$$

For Rocking Rotation,

$$\sigma_p = \frac{(C_2 - 1)\ 4M}{C_2 hA}$$

where σ_p = Stress against wall

P = 70% of the maximum total horizontal inertial load

M = 70% of the maximum total inertial moment

A = Area of side of structure

h = Depth of embedment

C_2 = Embedment correction factor (see Appendix E)

In the spirit of conservatism it has been assumed that walls parallel to the direction of earthquake induced pressures do not contribute to the resistance to the induced motion, i.e., all the stress is concentrated on the wall perpendicular to the direction of motion. An explanation of the equations presented above is given in Attachment F-2.

F-4 Lateral Loads Due to Adjacent Structures

For the special case of walls close to adjacent structures, additional load should be considered in design to take into account the pressure caused by the bearing load of the adjacent structure. Our recommendations for determination of how much additional load to consider for this circumstance is presented on Fig. F-1.

TABLE F-1

Lateral Earth Pressures
San Mateo Sand

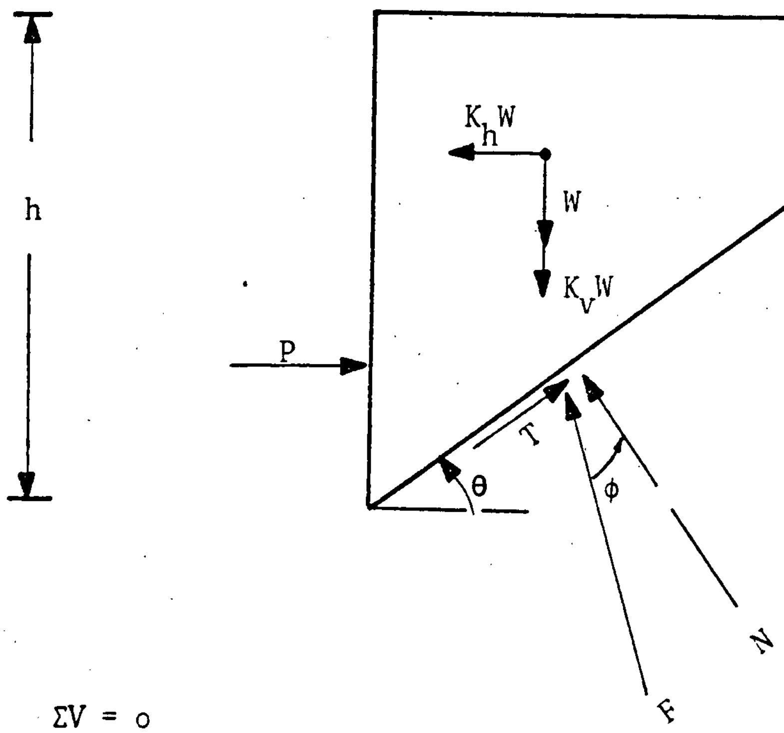
Case	Equivalent Fluid Pressure Above Water Table (pcf)	Equivalent Fluid Pressure Below Water Table (pcf)
Static (at-rest)	45	23*
Seismic** DBE	75	39*
Seismic** OBE or lower	45	23*

Note: * Hydrostatic head should be added.
** These values include static stresses. Seismic lateral stresses should be checked by the inertial load method, presented in Section F-3.

ATTACHMENT F-1

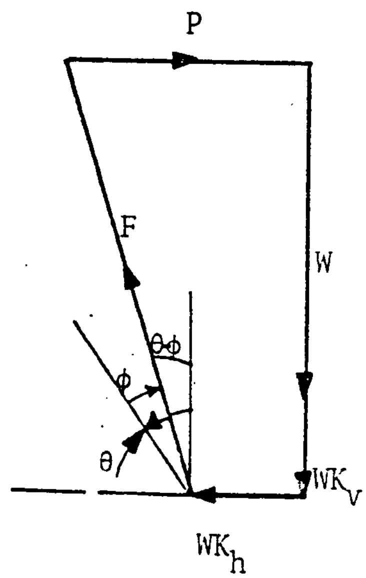
Wedge Analysis -- Active Earthpressures due to Earthquake Loading.

ACTIVE PRESSURE -- Horizontal Acceleration = $K_h g$
Vertical Acceleration = $K_v g$



$\Sigma V = o$

$\Sigma H = o$



(1) $W + W K_v = F \cos(\Theta - \phi)$

$$F = \frac{W(1+K_V)}{\cos(\theta-\phi)}$$

(2) $P = WK_h + F \sin(\Theta - \phi)$

$$= WK_h + \frac{W(1+K_V)\sin(\theta-\phi)}{\cos(\theta-\phi)}$$

$$= W\left[K_h + (1+K_v)\tan(\Theta - \phi)\right]$$

$$= \tfrac{1}{2}\gamma h^2 \cot\Theta \left[K_h + (1+K_v)\tan(\Theta - \phi)\right]$$

$P = 1/2 \quad \gamma h^2 \; K$

Where,

$$K = \cot \Theta \quad \left[K_h + (1+K_v) \tan (\Theta - \phi)\right]$$

K_{AE} is the maximum value at $\Theta = \Theta$ critical

$$K_{AE} = \cot \Theta_{cr} \left[K_h + (1+K_v) \tan (\Theta_{cr} - \phi)\right]$$

The values of K_{AE} calculated for upward and downward assumed vertical seismic coefficients for DBE and OBE conditions, are presented below. The resulting equivalent fluid pressures are presented in Table F-1.

Earthquake Loading	K_{AE} (due to $\uparrow K_V$)	Θ_{cr}	K_{AE} (due to $\downarrow K_V$)	Θ_{cr}
DBE K_h = .47g K_v = .31g	0.57	27	0.56	49
OBE K_h = .2g K_v = .13g	0.29	56	0.33	59

ATTACHMENT F-2

Explanation of Inertial Loading Equations
Presented in Section F-3

For a structure on the gound surface:

$$K_{BASE} = K' \; C_1 \quad \text{(See Appendix E)}$$

For an embedded structure:

$$K_{TOTAL} = K' \; C_1 \; C_2 \quad \text{(See Appendix E)}$$

$$\therefore \; K_{SIDES} = K_{TOTAL} - K_{BASE} = K'C_1 \; (C_2 - 1)$$

$$P_{TOTAL} = K_{TOTAL} \; \delta$$

$$P_{SIDES} = K_{SIDES} \; \delta$$

$$= K'C_1 \; (C_2 - 1) \; \delta$$

$$= \frac{K'C_1 \; (C_2 - 1) \; P_{TOTAL}}{K_{TOTAL}}$$

$$= \frac{K'C_1 \; (C_2 - 1) \; P_{TOTAL}}{K'C_1 \; C_2}$$

$$P_{SIDES} = \frac{(C_2 - 1) \; P_{TOTAL}}{C_2}$$

$$\sigma_{SIDES} = \frac{P_{SIDES}}{A_{SIDES}}$$

$$\therefore \; \sigma_{SIDES} \; \frac{(C_2 - 1)}{C_2} \; \frac{P_{TOTAL}}{A_{SIDES}} \quad \text{(horizontal translation)}$$

Attachment F-2
Page 2

$$P_{TOTAL} = \frac{M}{X}$$

$$\therefore \sigma_{SIDES} = \frac{(C_2-1)}{C_2} \frac{M}{AX} \quad \text{(Rocking Rotation)}$$

Definition of Terms :

K_{BASE} = Soil stiffness along base of structure

K' = Uncorrected stiffness

K_{SIDES} = Soil stiffness on sides of structure

K_{TOTAL} = Overall soil stiffness

C_1 = Empirical correction factor (See Appendix E)

C_2 = Embedment correction factor (See Appendix E)

P_{SIDES} = Horizontal inertial load on sides of structure

P_{TOTAL} = Total horizontal inertial load

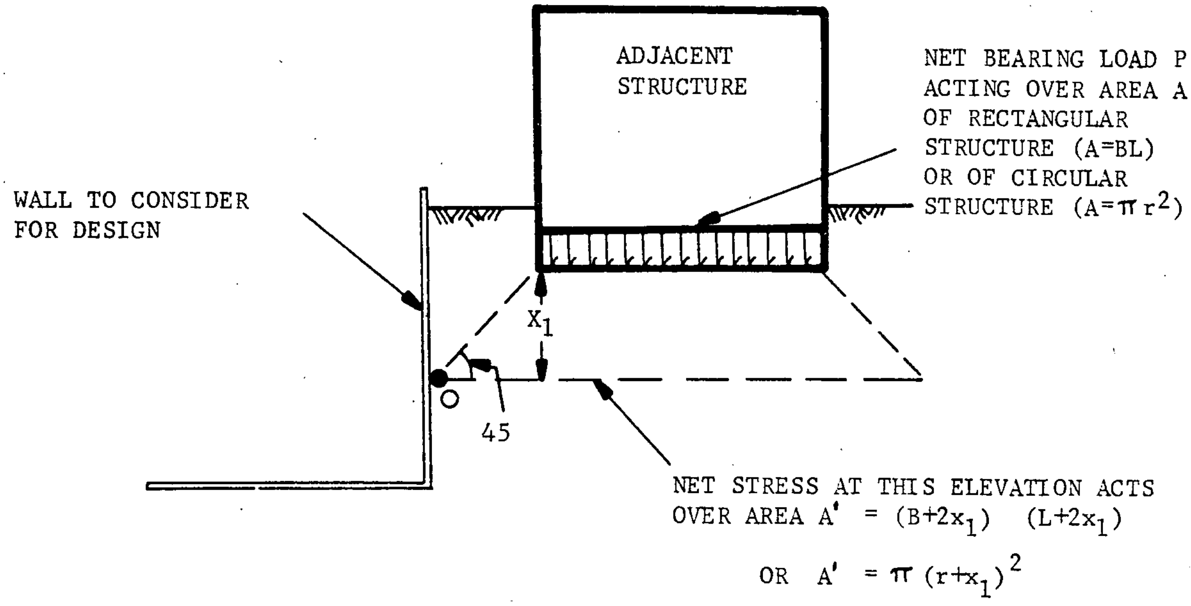
δ = Deflection of structure

A_{SIDES} = Area of Wall

M = Total inertial moment

X = Distance from center of base mat to center of gravity

LATERAL PRESSURE DUE TO ADJACENT STRUCTURES



ADDITIONAL LATERAL PRESSURE ON WALL $= \frac{A}{A'} \cdot PM$ (UNIFORM)

(APPLIED BELOW POINT .0)

WHERE: M = 0.34 FOR STATIC AND OBE CONDITION

M = 0.57 FOR DBE CONDITION

NOTES: 1. THE EXAMPLE PRESENTED IS FOR A RECTANGULAR STRUCTURE. FOR CIRCULAR STRUCTURES ADJUST THE RADIUS BY x_1, TO CALCULATE A'.

2. THE LATERAL PRESSURE CALCULATED HERE SHOULD BE ADDED TO EITHER THE ACTIVE EARTH PRESSURE OR THE INERTIAL LOAD FOR DESIGN.

Project: SONGS SOIL STRUCTURE INTERACTION Job Number: B221I	LATERAL PRESSURES DUE TO ADJACENT STRUCTURES	Fig. F-1

WOODWARD-McNEILL & ASSOCIATES

APPENDIX G

STRUCTURE SLIDING

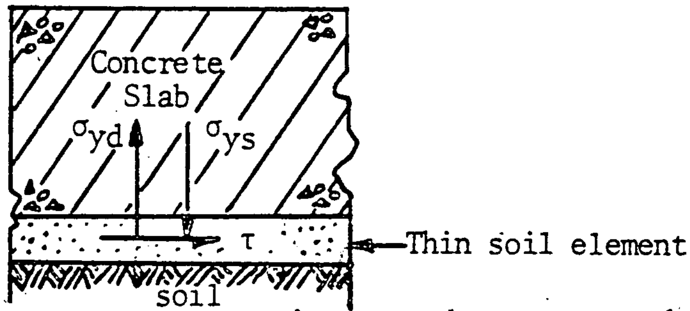
G-1 Introduction

During an earthquake horizontal forces are developed between a slab and the supporting soil which may cause the slab to slide with respect to the soil. Normally, if the slab is supporting building loads and the earthquake motion is not large, the frictional resistance mobilized between the slab and the soil is enough to prevent sliding. However, if a large earthquake input is considered, the possibility of potential sliding should be investigated. The auxiliary building has a proposed mat-foundation which has a fairly large flat area in contact with soil with little or no embedment. As the design base motion for the structure is a very strong motion, it is considered essential that a study of potential sliding between mat foundation and the supporting soil be made.

Though the analyses presented here were done for the specific case of the auxiliary building, some general conclusions can be drawn from the results of these analyses.

G-2 Evaluation of Potential Sliding Ratio

Consider a soil-element just below a slab. In the static condition, a vertical stress σ_{ys}, due to the load from the slab, is acting on this element. Normal and shear stresses would be induced in this soil element due to the effect of the imposed base motion on the slab-soil system. These stress conditions can be represented as shown below:



Thus, the shear stress, τ, acting at the contact between the slab and the soil, can be taken as the dynamically induced shear stress in the thin soil element just below the slab and the net normal stress can be obtained by evaluating $\sigma_n=(\sigma_{yd}-\sigma_{ys})$. The ratio of τ/σ_n can then be easily evaluated. This ratio can be taken as a measure of friction mobilized during the imposed base motion. So long as this friction is smaller than the available angle of friction between the concrete and soil, no sliding would be anticipated. A time-history of τ/σ_n would indicate if the mobilized friction exceeds the available friction at any time, and if so, for how long.

G-3 Analyses for Auxiliary Building

In order to study potential sliding of the mat-foundation of the auxiliary building during the ground motions, a two-dimensional finite-element model was prepared to incorporate interaction between foundation and the underlying soil. The soil deposit was assumed to be 120 ft thick and to be resting on a rigid base. This layer extended far enough horizontally from the edges of the foundation so as to minimize the influence of restrained vertical boundaries. The finite-element representation of the soil-foundation system is shown on Fig. G-1.

The mat foundation for the auxiliary building is 230' X 220' in size as shown on Fig. G-2 from Bechtel. The total normal load on the soil is about 3500 psf. In representing this foundation in the finite-element mesh, a concrete block was assumed to have appropriate thickness to represent this normal pressure.

In order to obtain the base motion which would yield a response time-history, at 30 ft depth below the ground surface which was the same as the DBE, a top-down analysis using wave propagation techniques was conducted. From this analysis, an acceleration time-history of 80 sec. duration at a depth of 120 ft was obtained. Because of core limitation of the available computer storage, appropriate sections, which govern the response spectral characteristics, were selected from this time-history to give a 22 sec. duration acceleration time-history. This 22 sec. motion was used as the base motion.

To study sliding at the contact between the foundation and the soil, normal and shear stresses were obtained in thin layers close to the contact. We feel that these stresses reasonably represent the stresses at the contact between the soil and the foundation. The ratio of the dynamic shear stress and the total normal stress at any instant was considered to be a measure of the potential sliding. If this ratio exceeded the value $\delta = \frac{2}{3} \tan \phi$ where δ is the angle of friction between soil and concrete, sliding is indicated. Thus, a time-history of the ratio τ/σ_n was obtained at three different locations in the foundation. A study of the time-history of τ/σ_n ratio at these three locations would indicate not only the potential sliding but also the phase differences between these points; i.e., whether these points would slide simultaneously or at different times. These time-histories were obtained for all cases studied, and are presented in this report.

Similar analyses were done using a finite-element model in which the vertical dimension of the slab was selected to give the same rotational inertia as that of the actual structure. In addition to the above, influences of a nominal embedment of 5 ft as well as of combined horizontal and vertical base motions were studied. Table G-1 presents a tabulation of the various cases studied. The resulting time-histories of τ/σ_n are presented on Figs. G-3 through G-9.

G-4 Results of Analyses

On the basis of the results of the cases studied, the following observations are made.

a) The maximum potential sliding develops at the center of the slab. (Fig. G-3, (a), (b), and (c))

b) The effect of embedment is to reduce the potential sliding, i.e., the maximum value of the stress ratio is smaller for embedded slabs than for slabs with no-embedment. (Figs. G-4 (a) and G-9 (a))

Appendix G

Page 5

c) The influence of prescribed vertical motion (i.e., 2/3 the horizontal motion) with the horizontal motion was to increase the potential sliding. (Fig. G-3 (b) and G-4 (b))

d) The maximum potential for sliding was not significantly influenced when the horizontal and vertical motions were out of phase. (Figs. G-4 (b) and G-5 (b))

e) Inclusion of the rotational inertia in modeling the slab for the analysis did not increase the sliding potential. (Figs. G-4 (b) and G-6 (b))

f) From all the cases studied, the maximum potential for sliding was obtained for non-embedded slabs subjected to horizontal and vertical inphase motions simultaneously. The corresponding stress ratio (τ/σ_n) was found to be 0.57. The time-history for this case is presented on Fig. G-10.

g) The available friction between the slab and soil is 2/3 (tan 41.5°) = 0.59. As this is larger than the mobilized friction (0.57) no sliding is anticipated. Furthermore, the peak value of 0.57 occurs only for a short duration during the time-history of stress ratio. The average value of stress ratio is about 2/3 of the peak value. Thus the average mobilized friction is considerably smaller than the available friction.

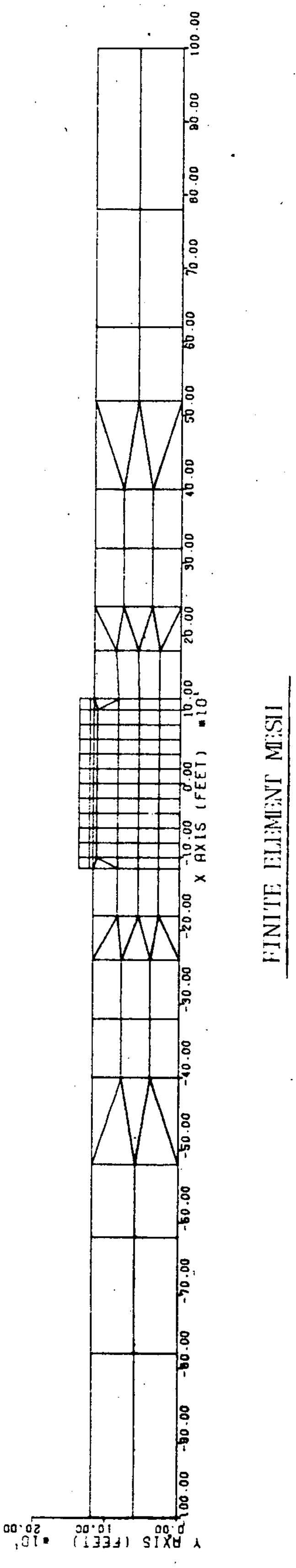
TABLE G-1

CASES STUDIED FOR
STRUCTURE SLIDING ANALYSES

Case No.	Input Motion			Condition			Figure Nos. for Results	Location of Output
	H only	H & V in phase	H & V out of phase	Mass only	Mass plus Inertia	Embedment		
1	✓			✓			G-3	L, C, R
2		✓		✓			G-4	L, C, R
3			✓	✓			G-5	L, C, R
4	✓				✓		G-6	L, C, R
5		✓			✓		G-7	L, C, R
6	✓			✓		✓	G-8	L, C
7		✓		✓		✓	G-9	L, C, R

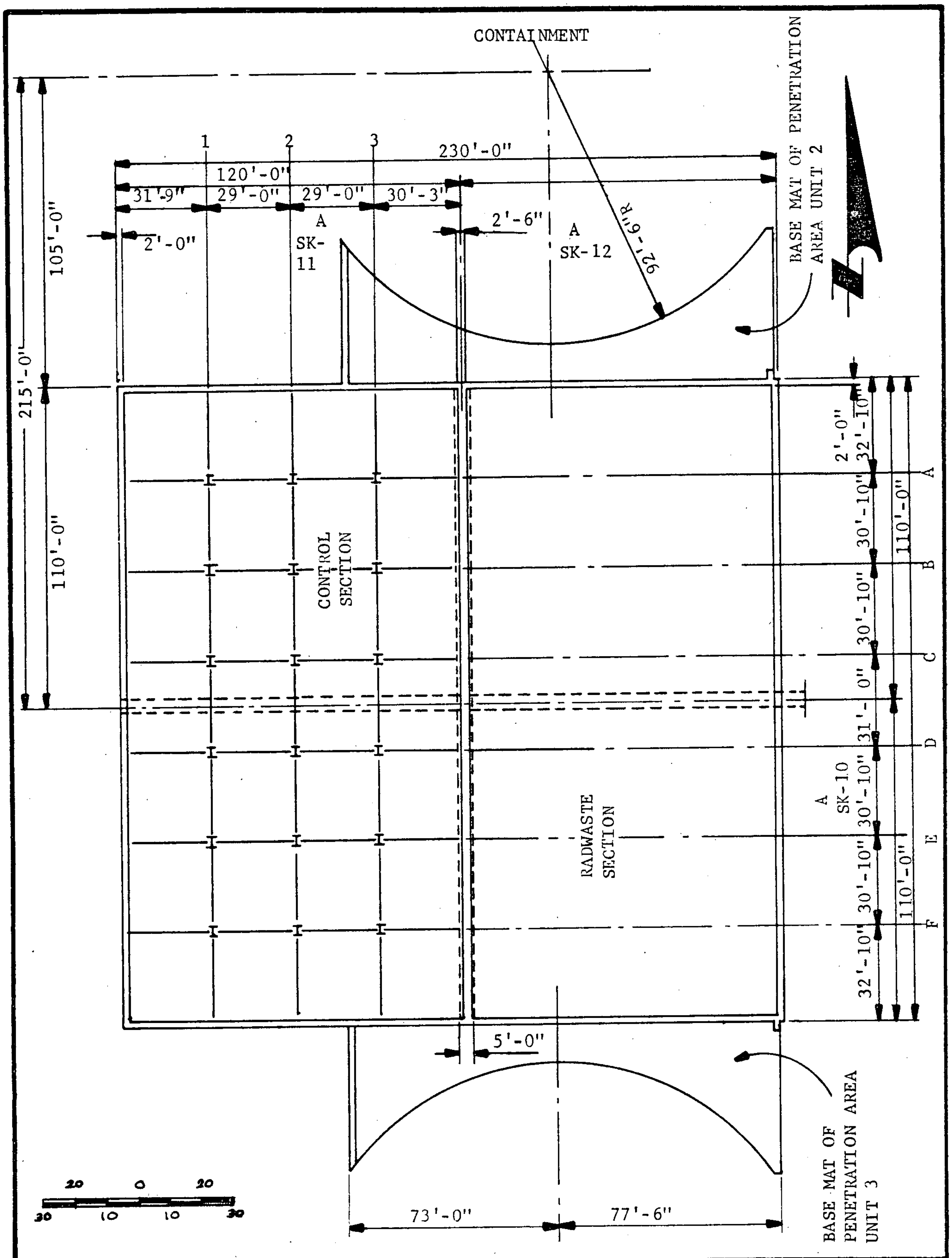
Where: H = Horizontal
V = Vertical
L = Left end of slab
C = Center of slab
R = Right end of slab

NOTE: Check is for item included; blank is for item not included.



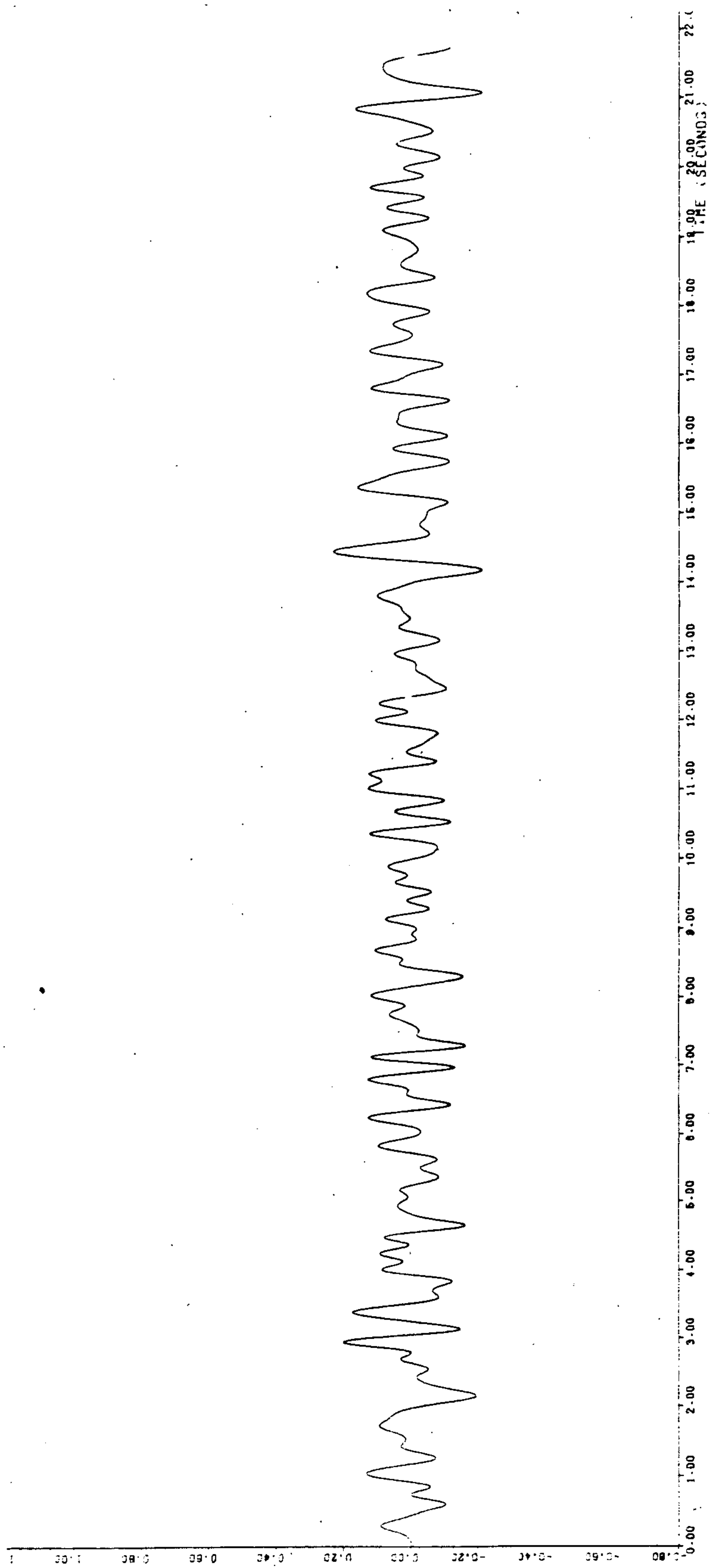
Project: SONG SOIL STRUCTURE INTERACTION **Job Number** B221I	FINITE ELEMENT MESH	**Fig.** G-1

WOODWARD - McNEILL & ASSOCIATES



Project: SONGS SOIL STRUCTURE INTERACTION Job Number: B221I	AUXILIARY BUILDING PLAN	Fig. G-2

WOODWARD - McNEILL & ASSOCIATES



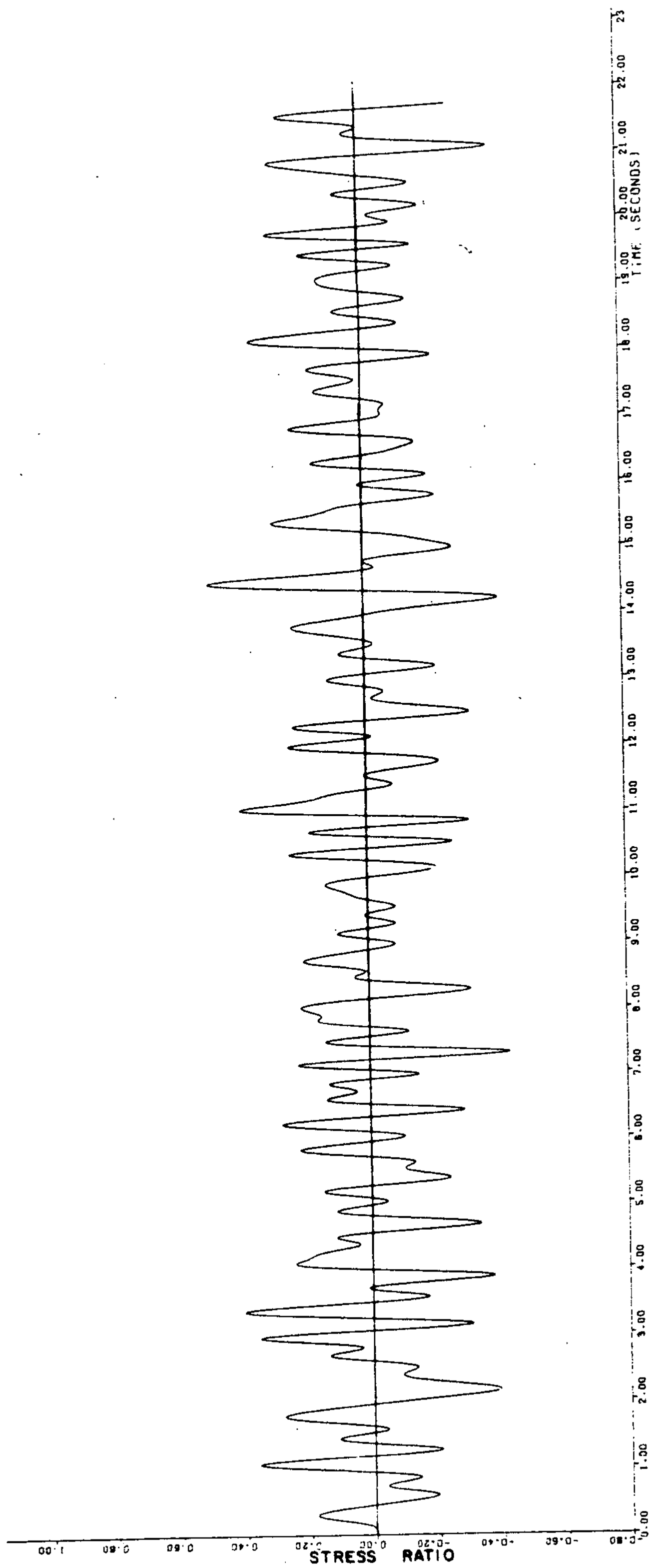
Project: SONGS SOIL STRUCTURE INTERACTION

Job Number: B221I

NO EMBEDMENT-LEFT END OF SLAB
HORIZONTAL BASE MOTION ONLY

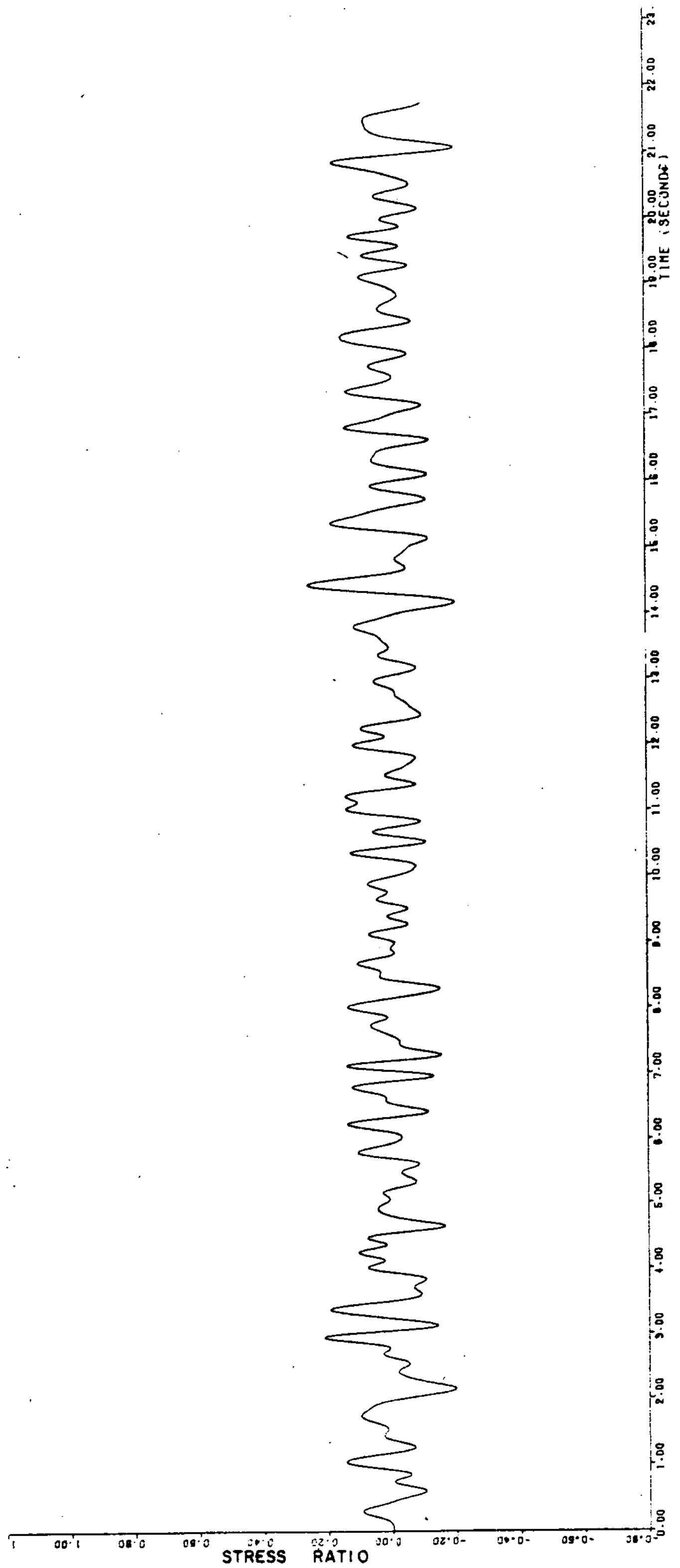
Fig. G3a

WOODWARD-McNEILL & ASSOCIATES



Project: SONGS SOIL STRUCTURE INTERACTION **Job Number:** B221I	NO EMBEDMENT-CENTER OF SLAB HORIZONTAL BASE MOTION ONLY	Fig. G3b

WOODWARD-McNEILL & ASSOCIATES



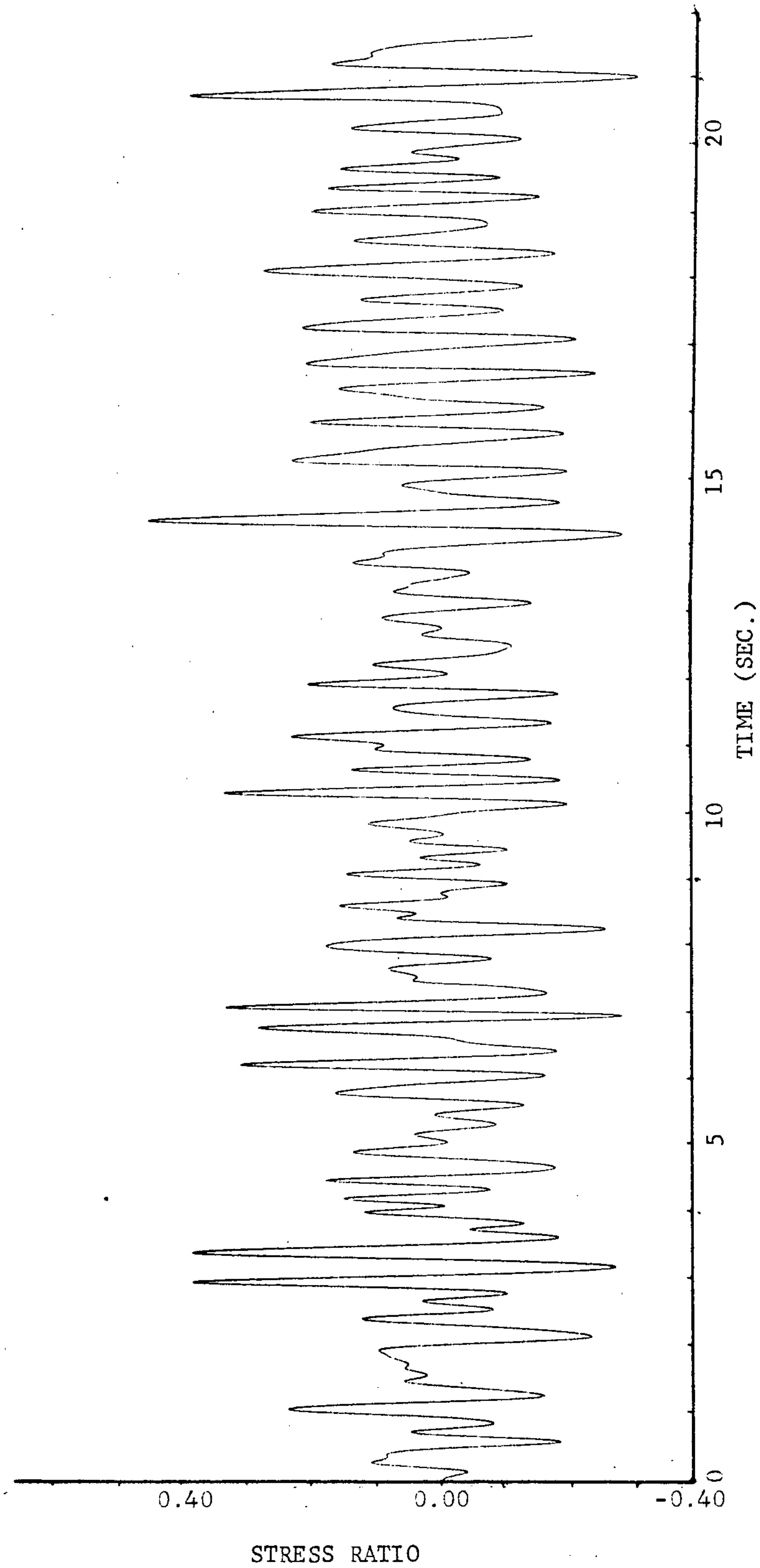
Project: SONGS SOIL STRUCTURE INTERACTION

Job Number: B221I

NO EMBEDMENT-RIGHT END OF SLAB
HORIZONTAL BASE MOTION ONLY

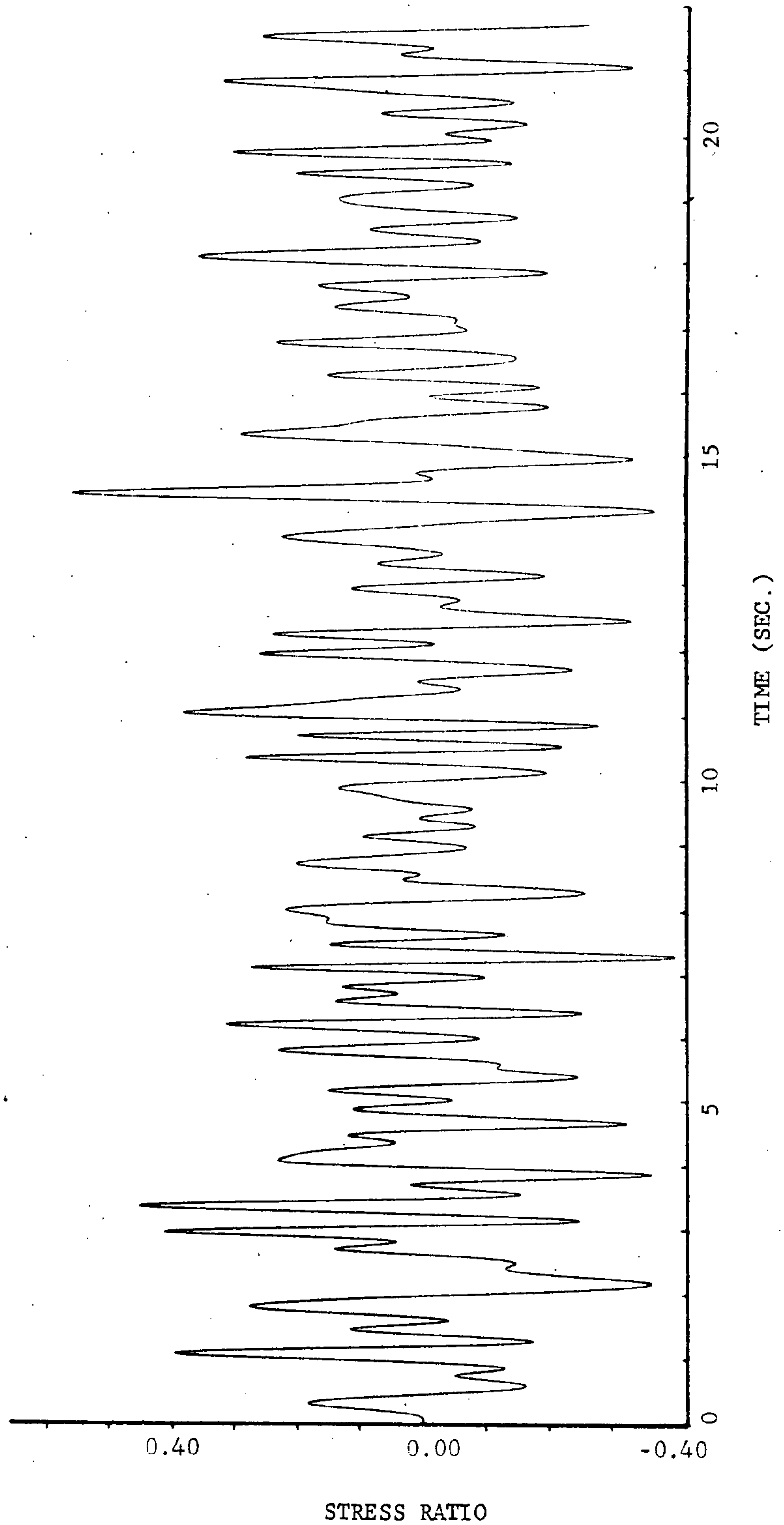
Fig. G3c

WOODWARD-McNEILL & ASSOCIATES



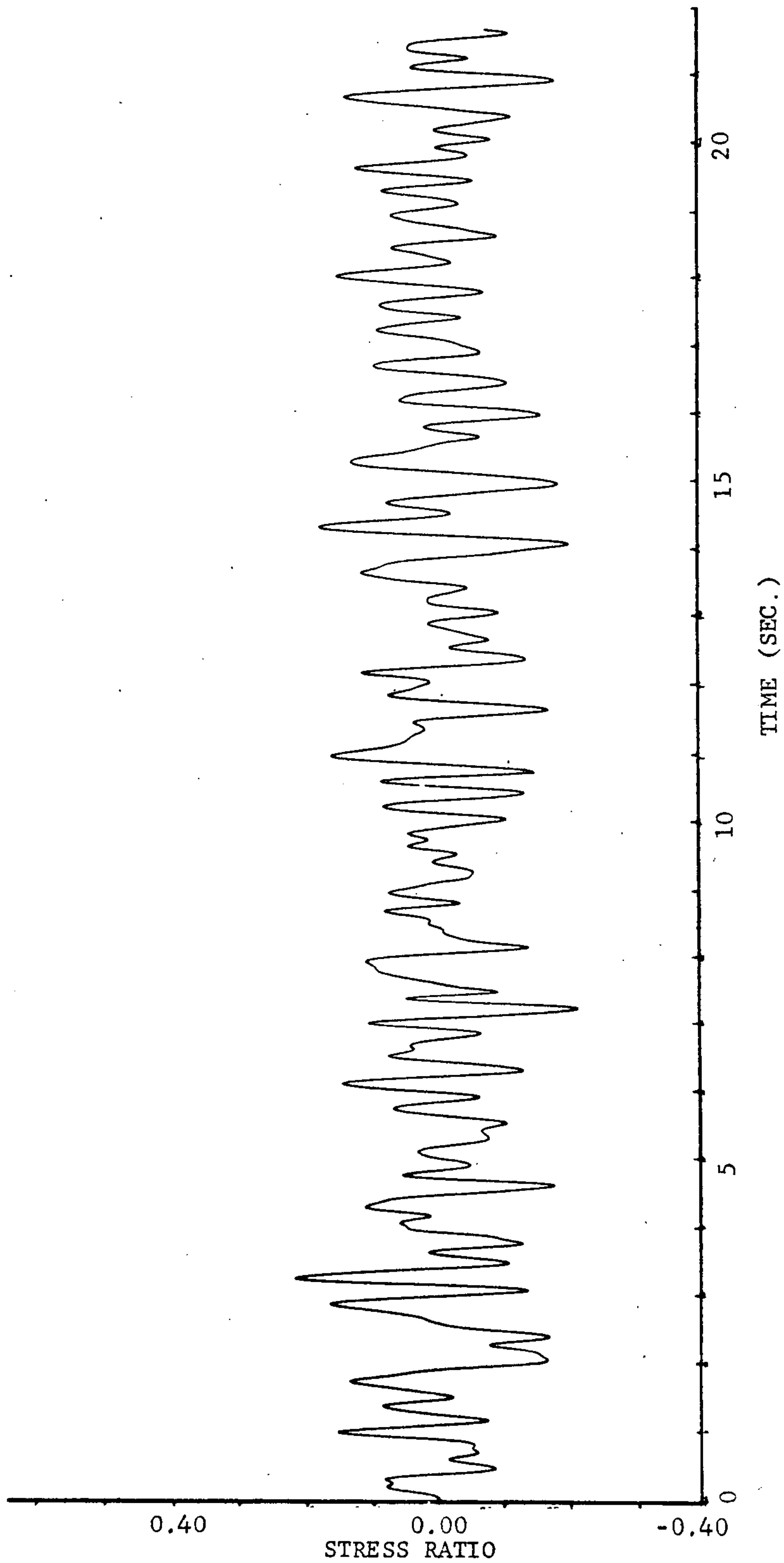
Project: SONGS SOIL STRUCTURE INTERACTION **Job Number:** B221I	NO EMBEDMENT LEFT EDGE OF SLAB HORIZONTAL AND VERTICAL BASE MOTIONS	**Fig.** G-4 a

WOODWARD - McNEILL & ASSOCIATES



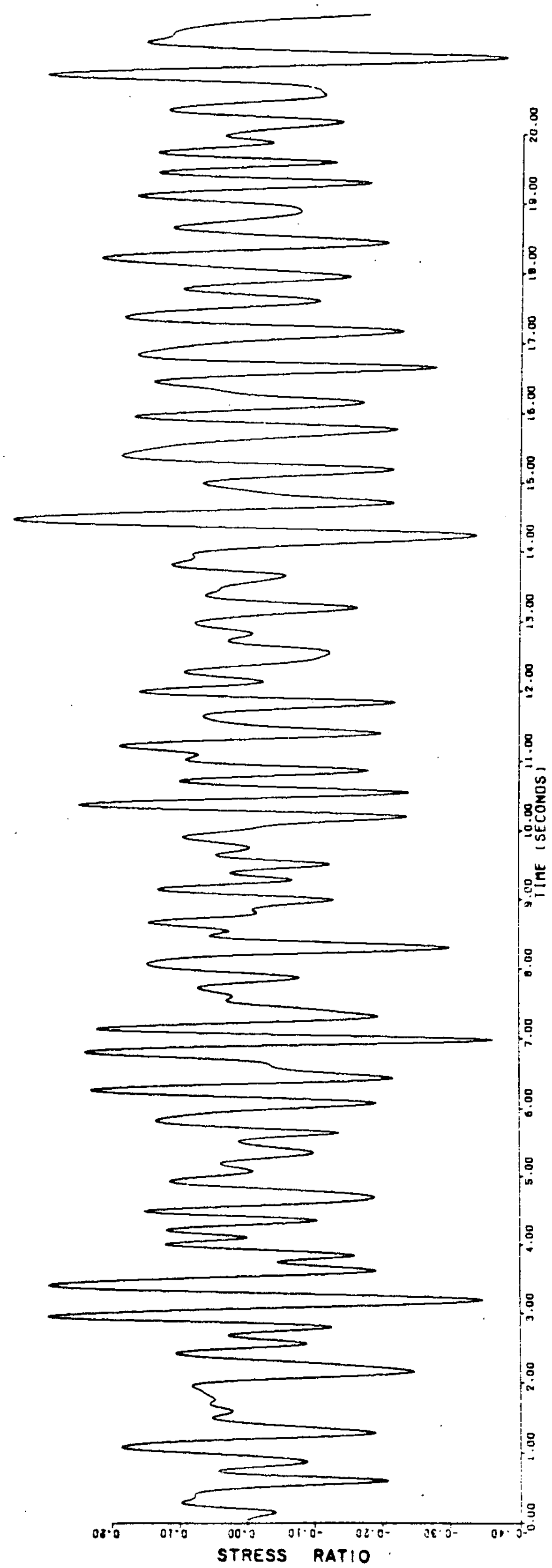
Project: SONGS SOIL STRUCTURE INTERACTION **Job Number:** B221I	NO EMBEDMENT CENTER OF SLAB HORIZONTAL AND VERTICAL BASE MOTIONS	**Fig.** G-4 b

WOODWARD - McNEILL & ASSOCIATES



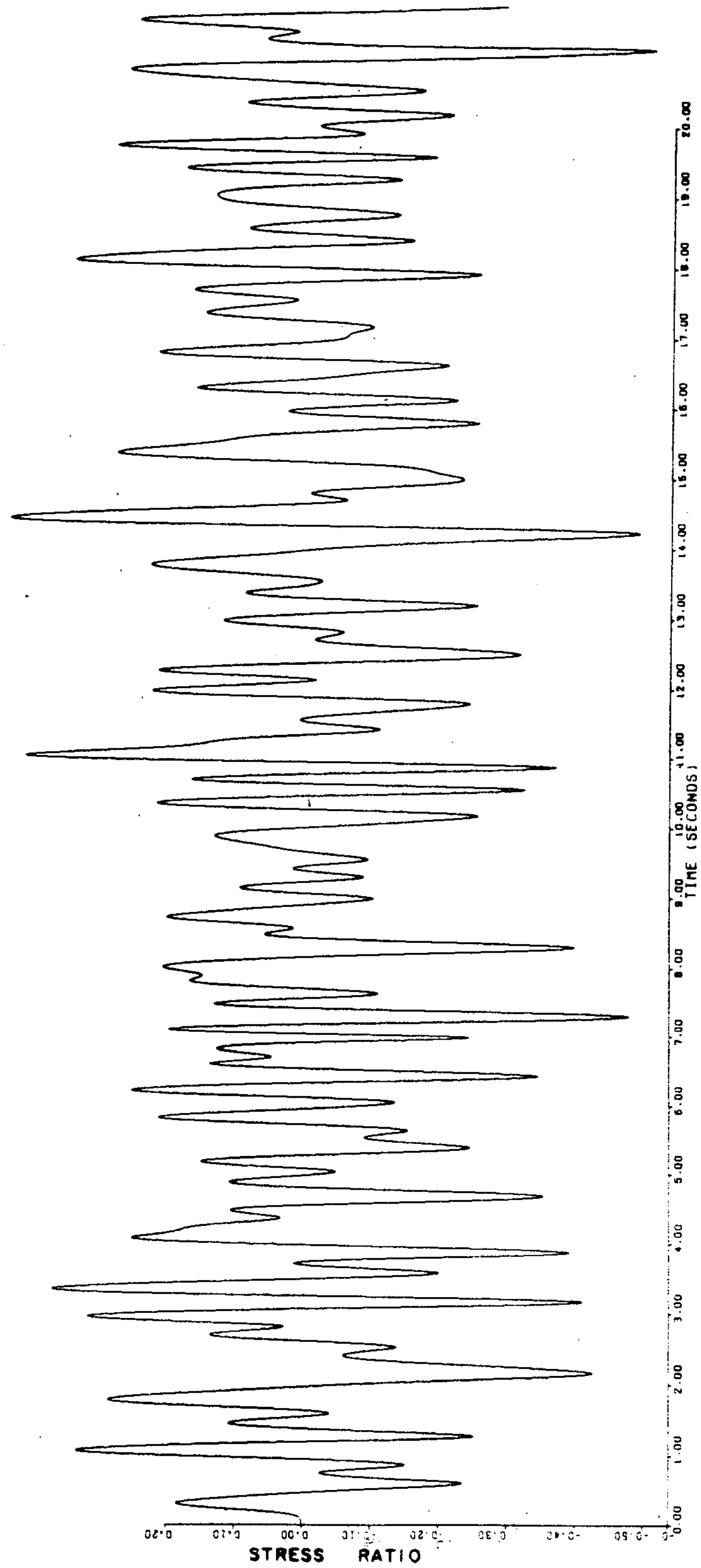
Project: SONGS SOIL STRUCTURE INTERACTION **Job Number:** B221I	NO EMBEDMENT RIGHT EDGE OF SLAB HORIZONTAL AND VERTICAL BASE MOTIONS	**Fig.** G-4c

WOODWARD - McNEILL & ASSOCIATES



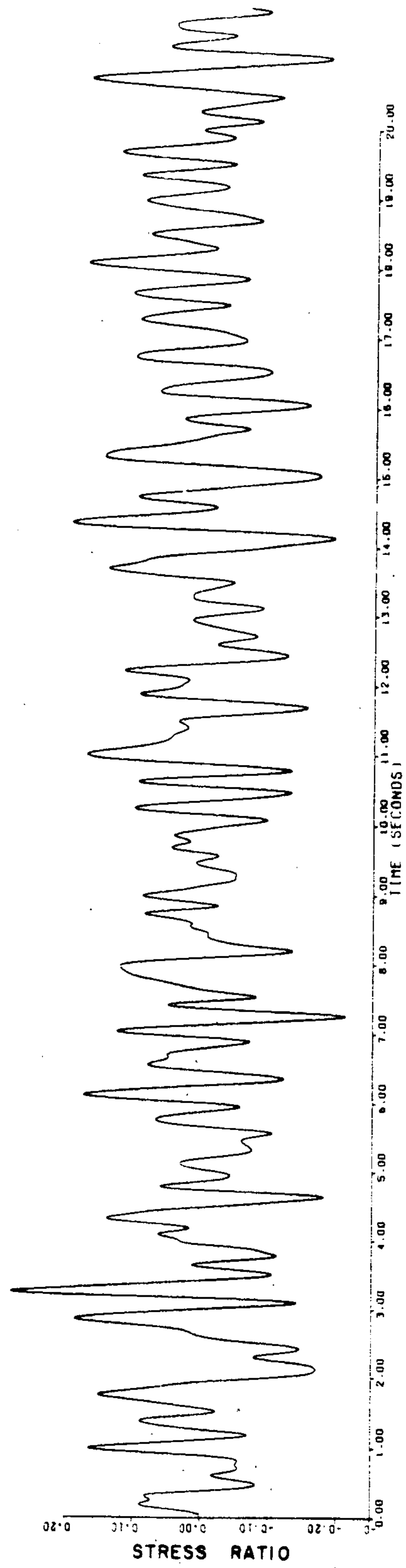
Project: SONGS SOIL STRUCTURE INTERACTION **Job Number:** B221I	NO EMBEDMENT-LEFT END OF SLAB HORIZONTAL AND VERTICAL (OUT OF PHASE) BASE MOTIONS	**Fig.** G5a

WOODWARD-McNEILL & ASSOCIATES



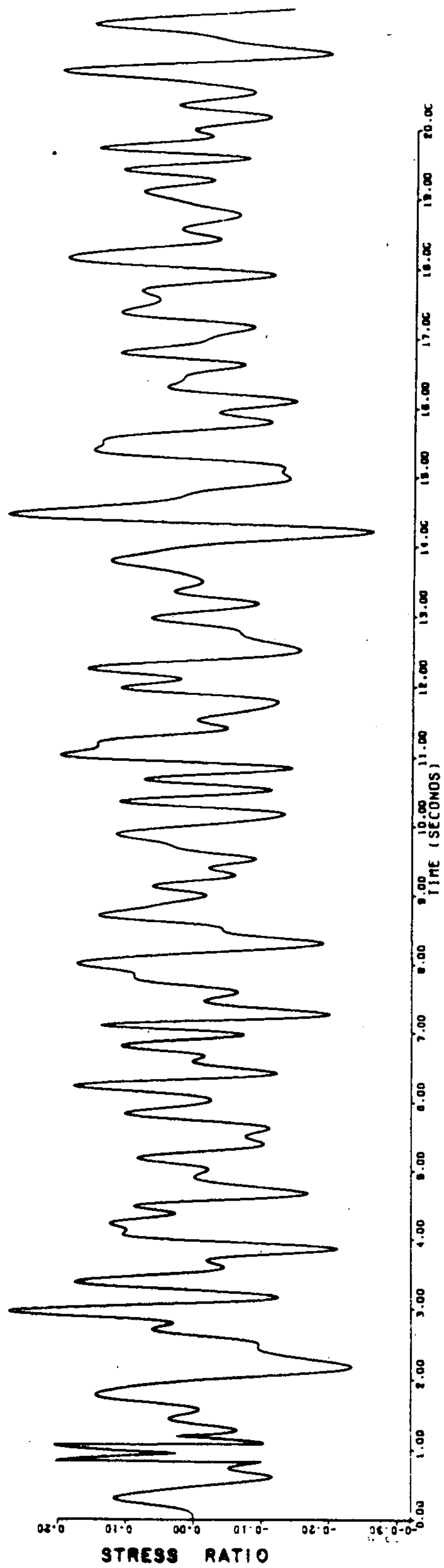
Project: SONGS SOIL STRUCTURE INTERACTION **Job Number:** B221I	NO EMBEDMENT-CENTER OF SLAB HORIZONTAL AND VERTICAL (OUT OF PHASE) BASE MOTION	Fig. G5b

WOODWARD-McNEILL & ASSOCIATES



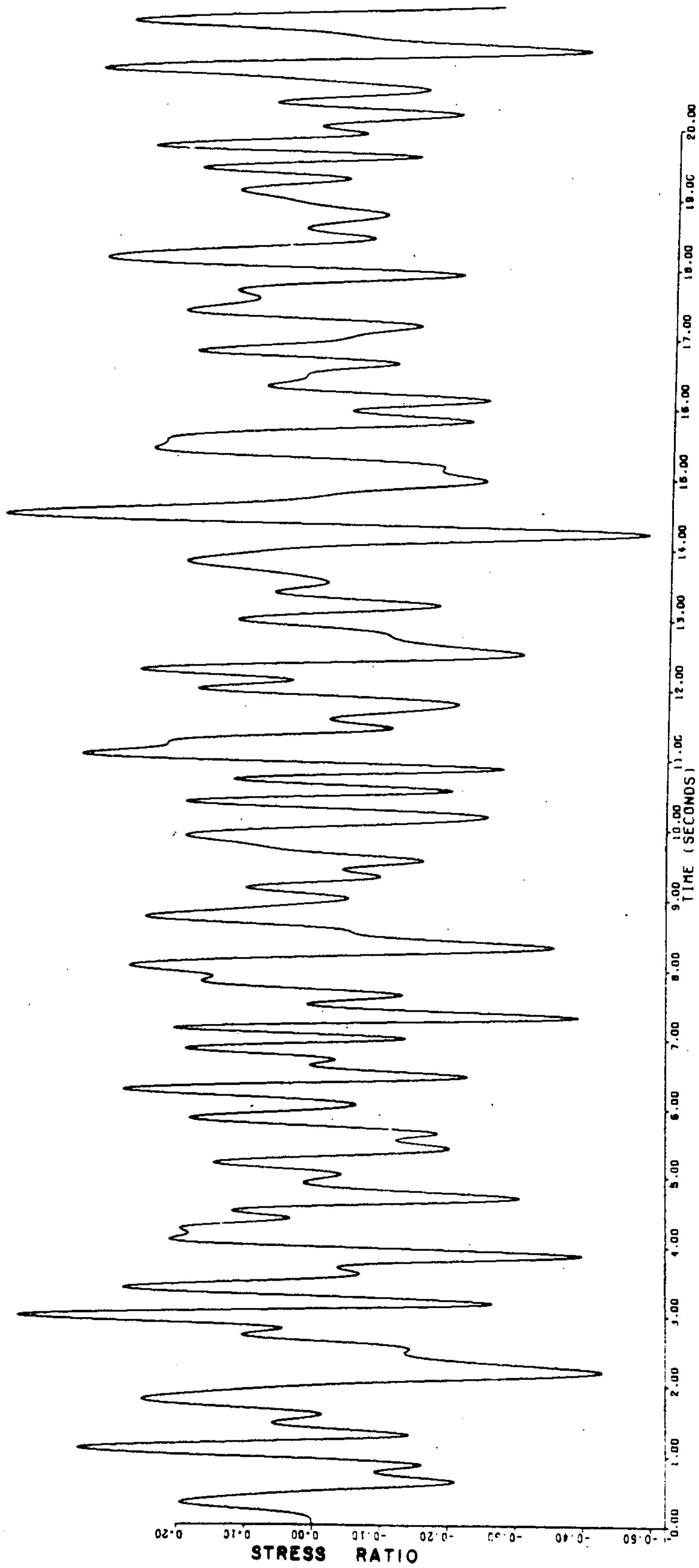
Project: SONGS SOIL STRUCTURE INTERACTION Job Number: B2211	NO EMBEDMENT-RIGHT END OF SLAB HORIZONTAL AND VERTICAL (OUT OF PHASE) BASE MOTIONS	Fig. G5c

WOODWARD-McNEILL & ASSOCIATES



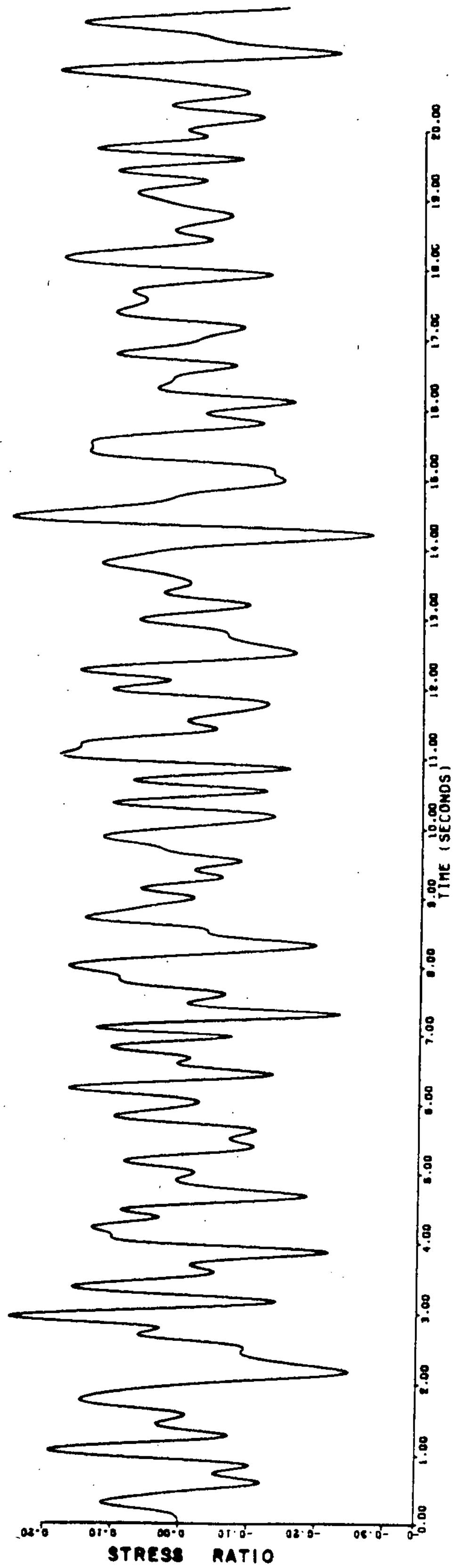
Project: SONGS SOIL STRUCTURE INTERACTION	NO EMBEDMENT-ROTATIONAL INERTIA INCLUDED	Fig.
Job Number: B221I	LEFT END OF SLAB HORIZONTAL BASE MOTION ONLY	G6a

WOODWARD-McNEILL & ASSOCIATES



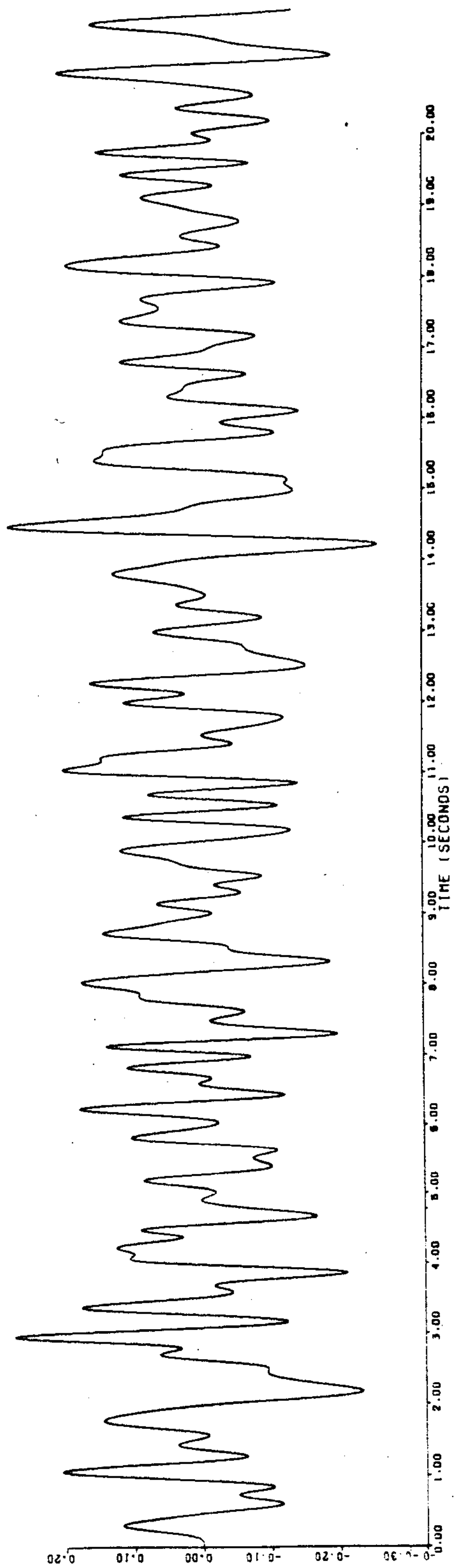
Project: SONGS SOIL STRUCTURE INTERACTION **Job Number:** B221I	NO EMBEDMENT-ROTATIONAL INERTIA INCLUDED CENTER OF SLAB HORIZONTAL BASE MOTION ONLY	Fig. G6b

WOODWARD-McNEILL & ASSOCIATES



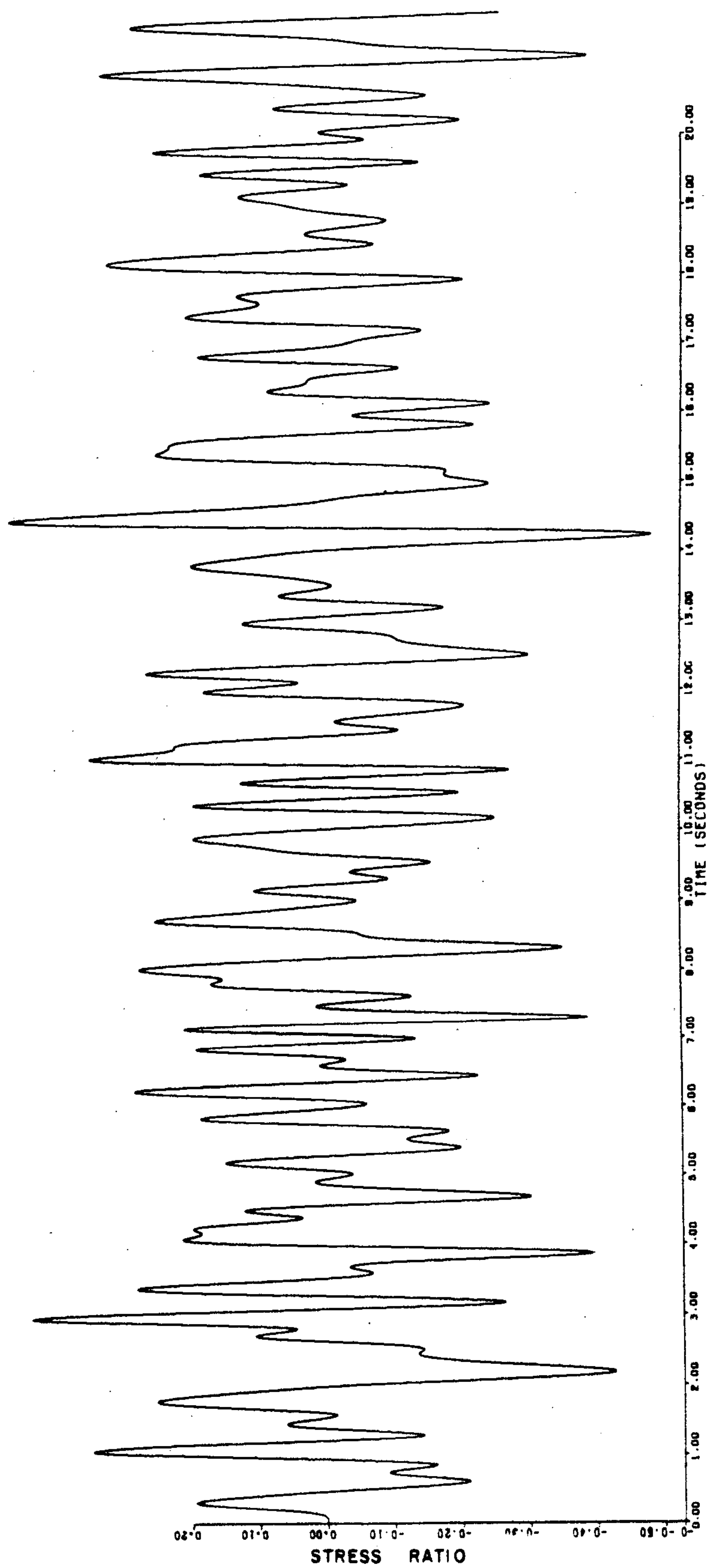
Project: SONGS SOIL STRUCTURE INTERACTION **Job Number:** B221I	NO EMBEDMENT-ROTATIONAL INERTIA INCLUDED RIGHT END OF SLAB HORIZONTAL BASE MOTION ONLY	Fig. G6c

WOODWARD-McNEILL & ASSOCIATES



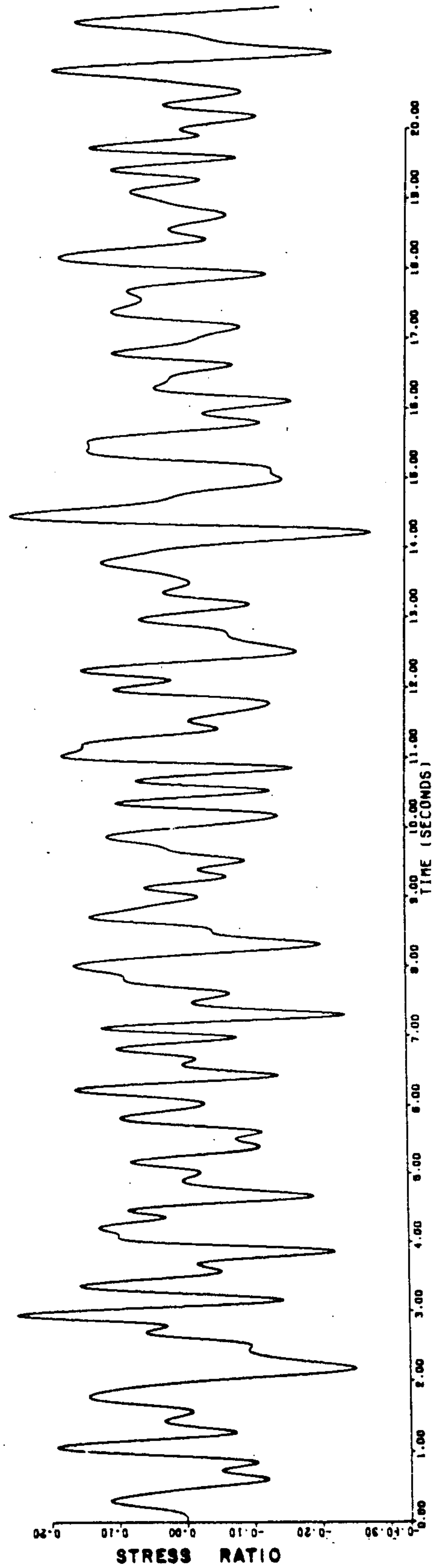
Project: SONGS SOIL STRUCTURE INTERACTION Job Number: B221I	NO EMBEDMENT-ROTATIONAL INERTIA INCLUDED LEFT END OF SLAB HORIZONTAL AND VERTICAL BASE MOTIONS	Fig. G7a

WOODWARD-McNEILL & ASSOCIATES



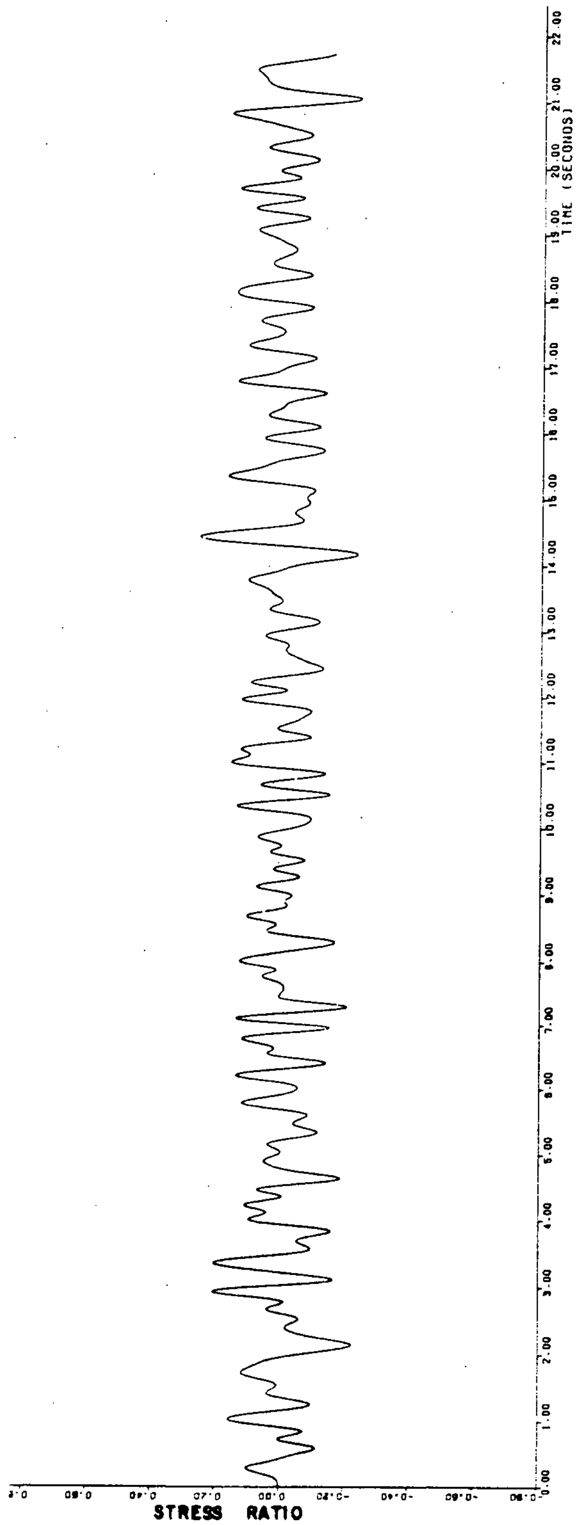
Project: SONGS SOIL STRUCTURE INTERACTION **Job Number:** B221I	NO EMBEDMENT-ROTATIONAL INERTIA INCLUDED CENTER OF SLAB HORIZONTAL AND VERTICAL BASE MOTIONS	Fig. G7b

WOODWARD-McNEILL & ASSOCIATES



Project: SONGS SOIL STRUCTURE INTERACTION **Job Number:** B2211	NO EMBEDMENT ROTATIONAL INERTIA INCLUDED RIGHT END OF SLAB HORIZONTAL AND VERTICAL BASE MOTIONS	**Fig.** G7c

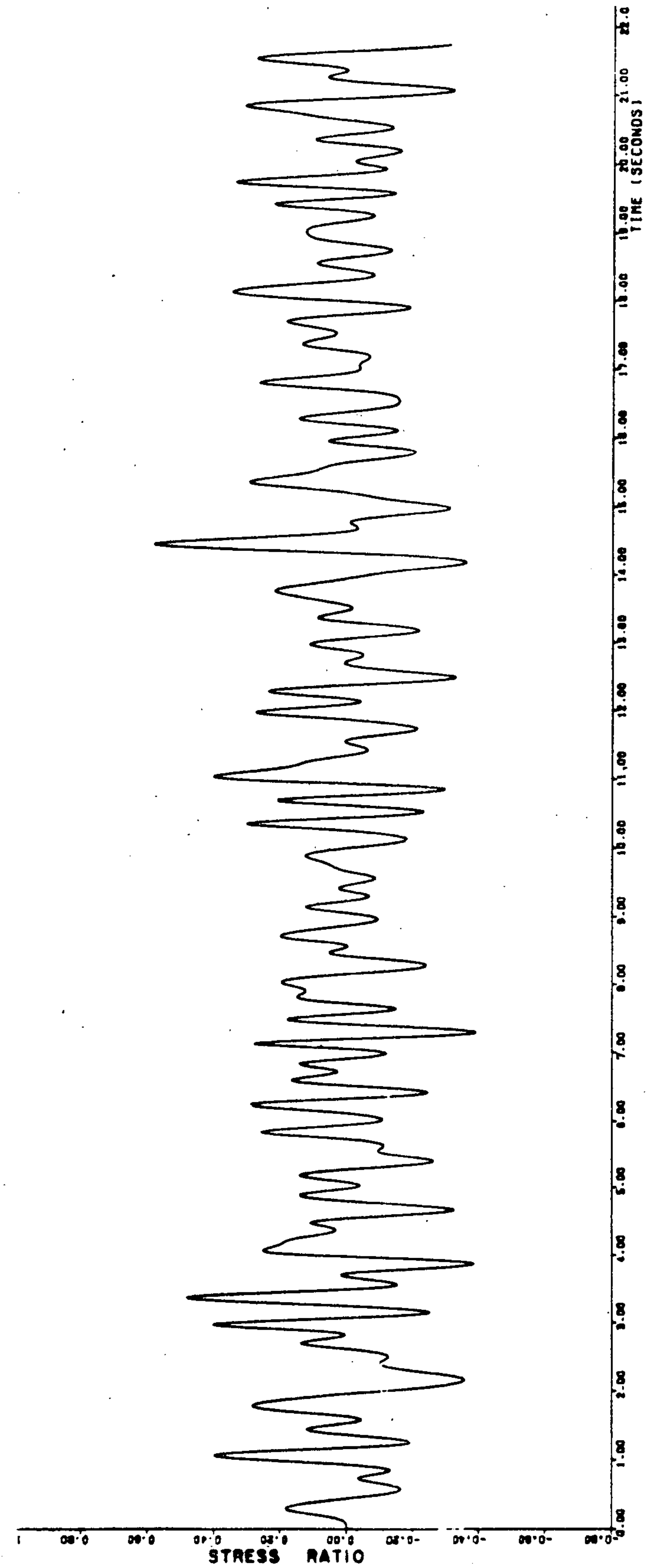
WOODWARD - McNEILL & ASSOCIATES



Project: SONGS SOIL STRUCTURE INTERACTION **Job Number:** B221I	EMBEDDED SLAB LEFT END OF SLAB HORIZONTAL BASE MOTION ONLY

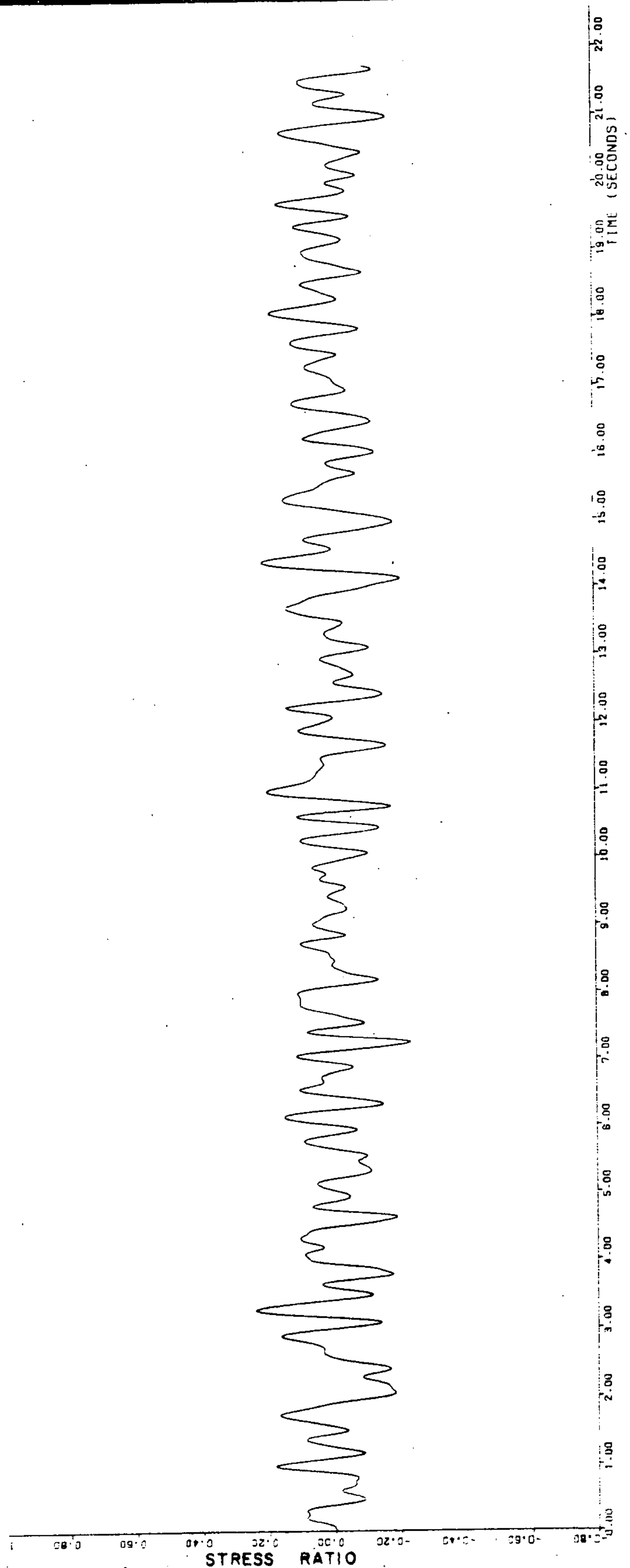
Fig. G8a

WOODWARD-McNEILL & ASSOCIATES



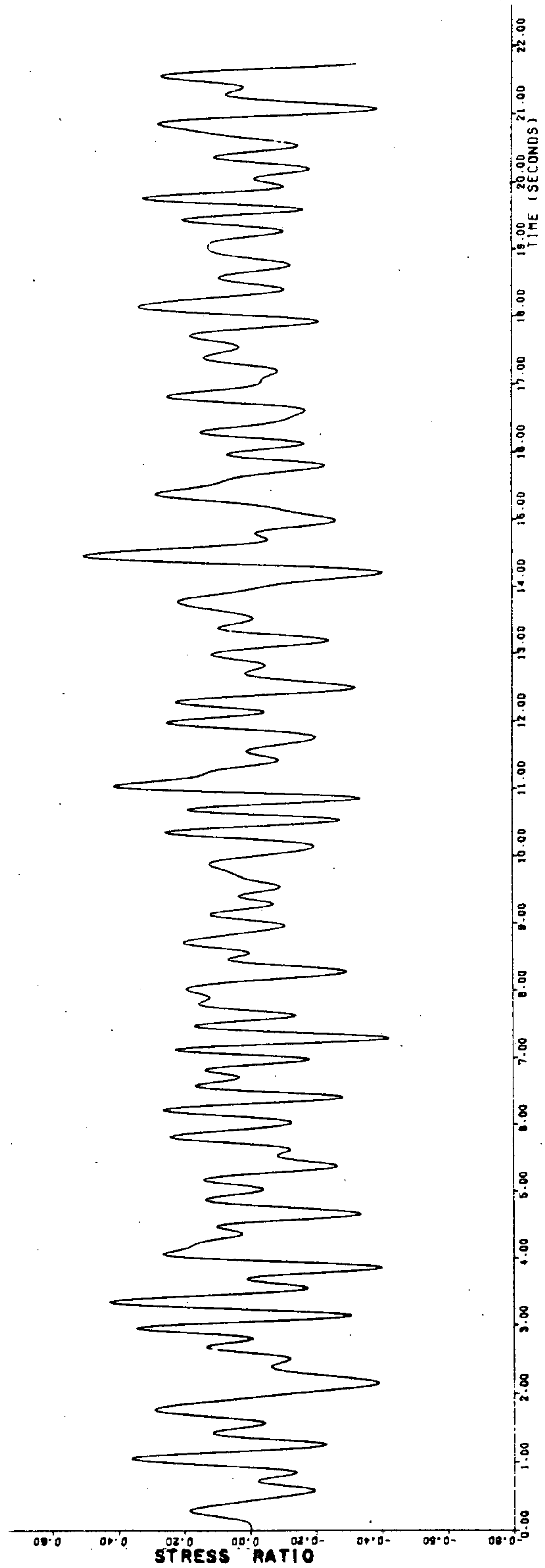
Project: SONGS SOIL STRUCTURE INTERACTION **Job Number**: B221I	EMBEDDED SLAB CENTER OF SLAB HORIZONTAL BASE MOTION ONLY	Fig. G8b

WOODWARD-McNEILL & ASSOCIATES



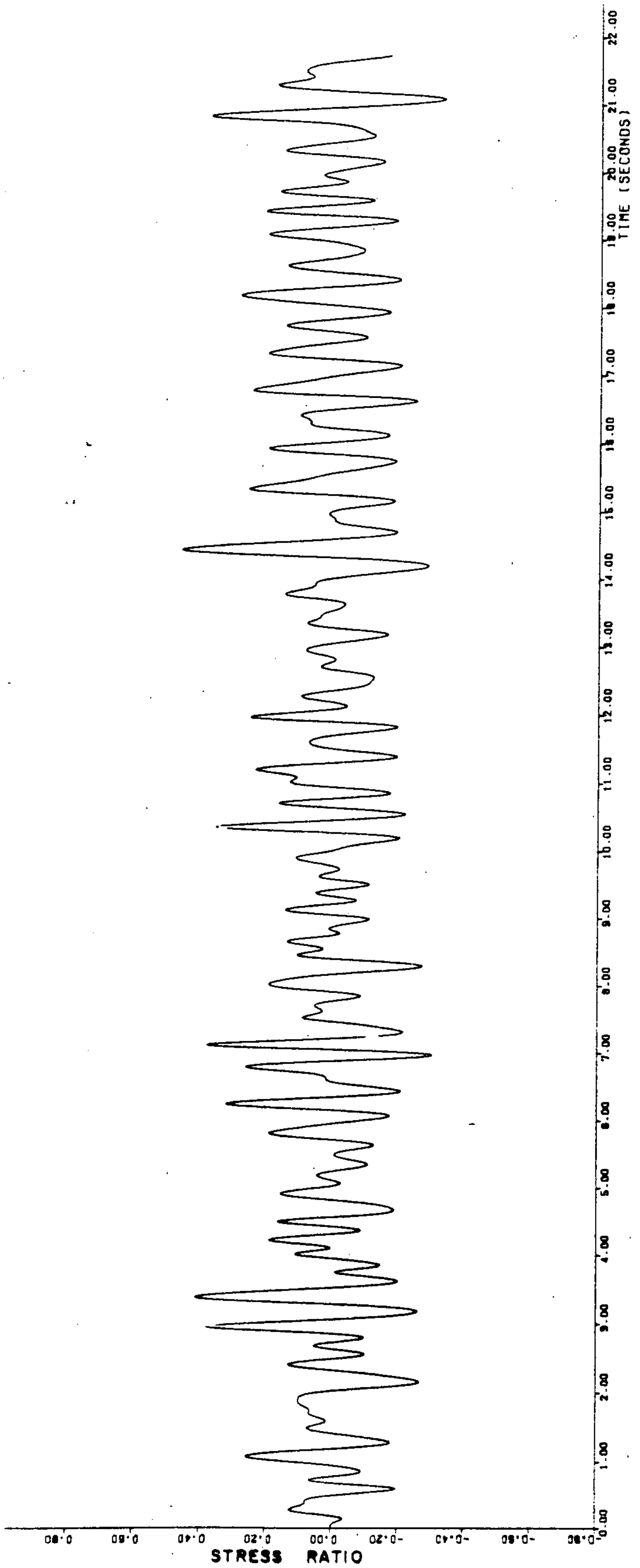
Project: SONGS SOIL STRUCTURE INTERACTION Job Number: B221I	EMBEDDED SLAB LEFT END OF SLAB HORIZONTAL AND VERTICAL BASE MOTIONS	Fig. G9a

WOODWARD - McNEILL & ASSOCIATES



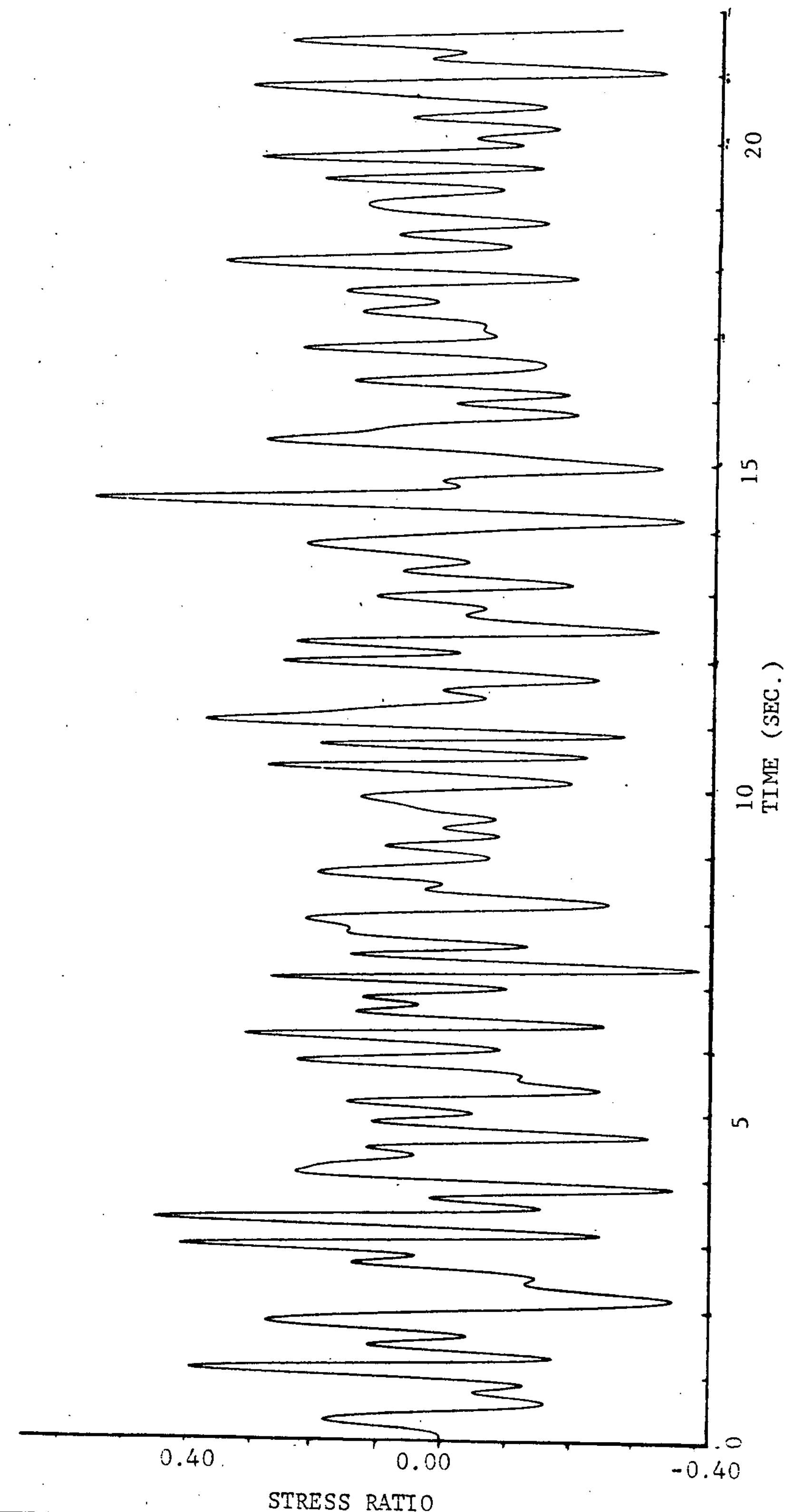
Project: SONGS SOIL STRUCTURE INTERACTION **Job Number:** B221I	EMBEDDED SLAB CENTER OF SLAB HORIZONTAL AND VERTICAL BASE MOTIONS	**Fig.** G9b

WOODWARD – McNEILL & ASSOCIATES



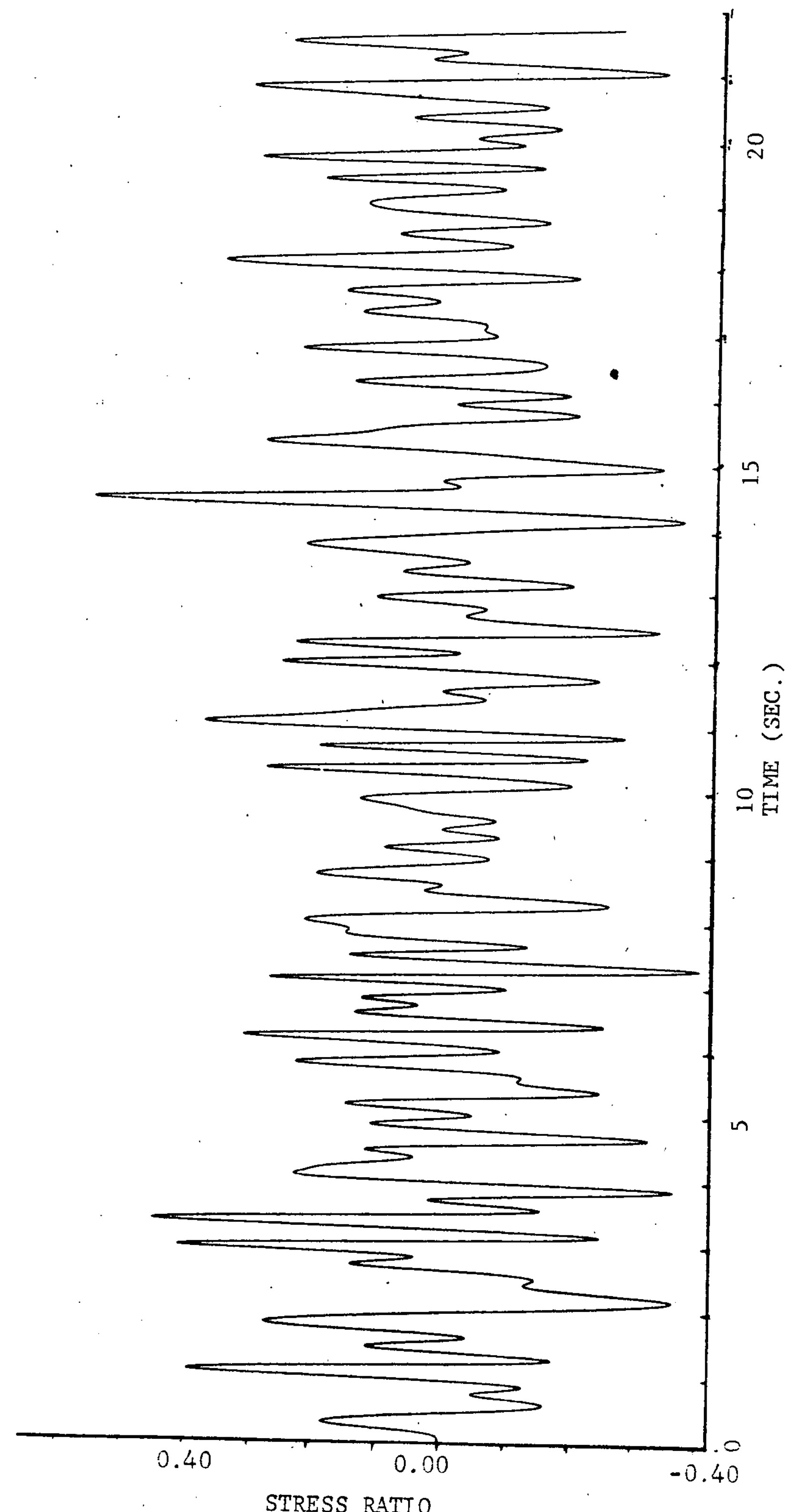
Project: SONGS SOIL STRUCTURE INTERACTION Job Number: B221I	EMBEDDED SLAB RIGHT END OF SLAB HORIZONTAL AND VERTICAL BASE MOTIONS	Fig. G9c

WOODWARD - McNEILL & ASSOCIATES



Project: SONGS SOIL STRUCTURE INTERACTION **Job Number:** B2211	NO EMBEDMENT CENTER OF SLAB HORIZONTAL AND VERTICAL BASE MOTIONS	**Fig.** G-10

WOODWARD - McNEILL & ASSOCIATES



Project: SONGS SOIL STRUCTURE INTERACTION
Job Number: B221I

NO EMBEDMENT
CENTER OF SLAB
HORIZONTAL AND VERTICAL BASE MOTIONS

Fig. G-10

WOODWARD - McNEILL & ASSOCIATES

APPENDIX H

EVALUATION OF CRITICAL INSTANTANEOUS DISPLACEMENT PROFILE

H-1 General

Determination of the DBE induced critical instantaneous displacement profile was done so that a pseudo-static calculation of stresses in a structure could be facilitated. This was done for structures grouped into either of two categories:

(1) Structures with complicated geometry and difficult to model.

(2) Structures below ground with the same mass or inertia as the displaced soil, for which response modeling is not appropriate for dynamic analysis.

The method of analysis is outlined in the following five (5) steps:

(1) Select appropriate section(s) of structure for modeling.

(2) Make suitable finite-element model(s) to represent actual soil-structure configuration(s).

(3) Determine dynamic properties of soil and structure for computer program input.

(4) Input the DBE time-history to the model and obtain and plot displacement time-history output at selected nodal points.

(5) Input the output from Step 4 into another computer program to obtain the critical instantaneous displacement profile(s) based on a maximum differential slope criteria.

These steps will be discussed in more detail in the section that follows.

H-2 Analysis

The computer program to study dynamic response of earth structures subjected to single travelling wave inputs was developed at the University of California, Berkeley in 1967. DBE earthquake wave motions were input, propagating along the base of the finite-element model. A schematic diagram of the analysis is shown in Fig. H-1(a) through (c).

Procedures involved for this analysis are described below:

(1) Determine a representative cross-section of the structure to be studied. This is shown schematically by the shaded elements presented in Fig. H-1a.

(2) The configuration is then divided into suitable number of finite-elements. The mesh- consisting of these elements is extended to a sufficient distance to eliminate the influence of fixed vertical boundaries on the response value in the vicinity of the location of interest. Nodal points 1 through 5 represent five select points, at which displacement time-histories are desired at the base of the structure.

(3) Dynamic properties of soil are obtained as well as properties of concrete. These properties include Young's modulus of the soil at the strain level of interest and the equivalent modulus of structural elements, assuming a solid structure. The structural element modulus values are chosen by consideration of section modulus of the structure and/or judgment. Then properties are input to the computer program.

(4) Output from the computer program consists of vertical and horizontal displacement time-histories of the five selected nodal points at interface of the soil-structure system. Typical displacement-time histories of the five points are shown in Fig. H-1b. Delay time, t_i, is time required for wave front movement from one point to the next, and is expressed by the equation:

$$t_i = \frac{X_i}{V_s} \quad , \; i = 1, 2, 3, 4.$$

Where X_i = distance between two adjacent points.

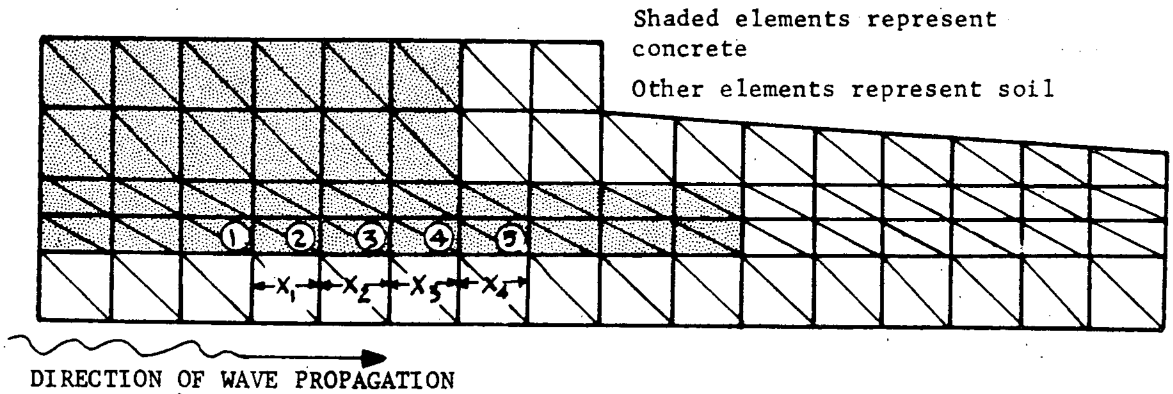
t_i = time required for wave front movement from one point to the next point.

V_s = shear wave velocity.

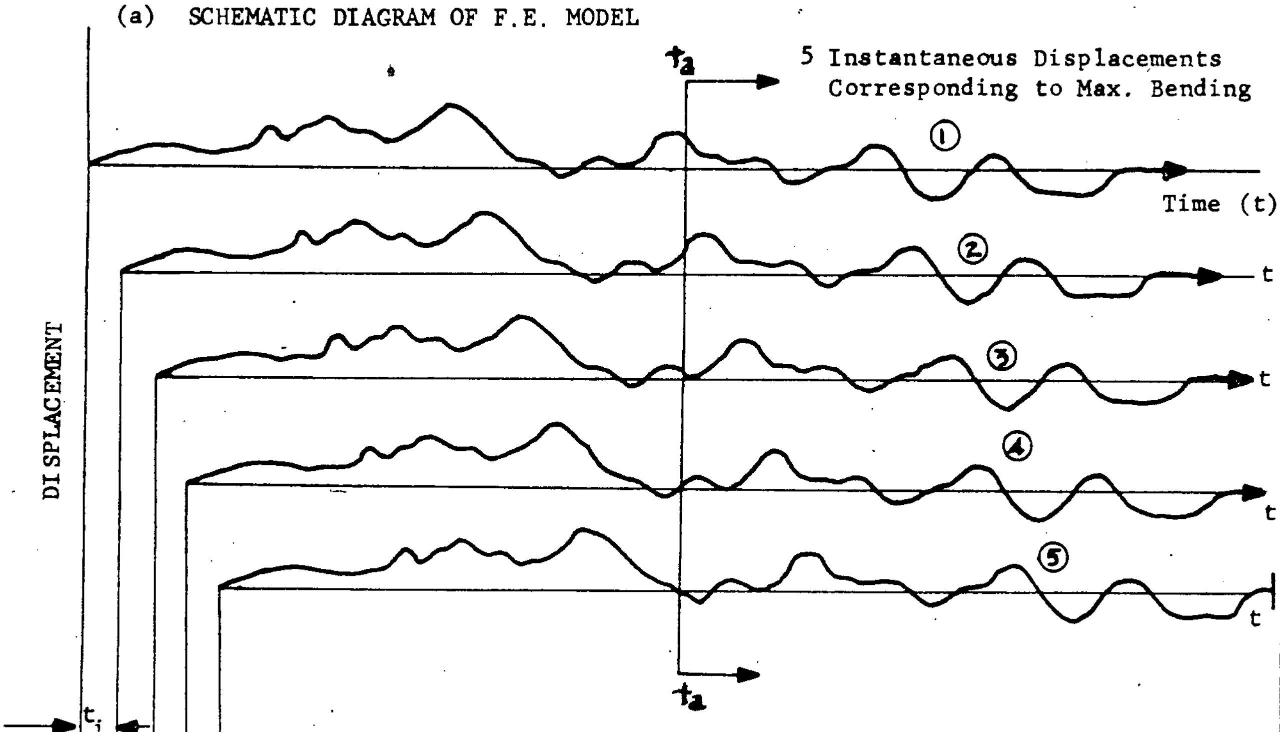
Appendix H

Page 3

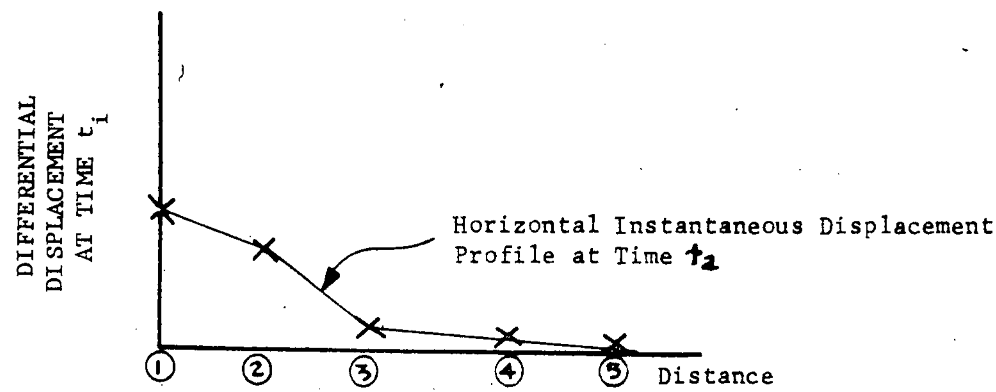
(5) To determine the most critical bending conditions of the system during DBE earthquake motion, the displacement time-histories are input into another computer program in the sequence described in Step 4 and Fig. H-1b to obtain instantaneous displacement profiles corresponding to maximum slope change of any two adjacent points. The typical result of the computer output for horizontal displacement is presented schematically in Fig. H-1c.



(a) SCHEMATIC DIAGRAM OF F.E. MODEL



(b) DISPLACEMENT - TIME HISTORIES (Horizontal)



(c) TYPICAL INSTANTANEOUS DISPLACEMENT PROFILE

Project: SONGS SOIL STRUCTURE INTERACTION **Job Number:** E221I	SUMMARY OF PROCEDURE FOR DETERMINATION OF INSTANTANEOUS DISPLACEMENT PROFILE	**Fig.** H-1

WOODWARD - McNEILL & ASSOCIATES

BIBLIOGRAPHY

1. Barneich, J. A., Johns, D. H., and McNeill, R. L. (1974) "Soil Structure Interaction Parameters for Aseismic Design of Nuclear Power Stations," Pre-print 2182, ASCE National Meeting on Water Resources Engineering, Los Angeles, Jan. 1974.

2. Clough, R. W., and Chopra, A. K. (1966) "Earthquake Stress Analysis in Earth Dams," Journal of the Engineering Mechanics Division, ASCE, Vol. 92, No. EM2, Proc. Paper 4793, April, pp. 197-212.

3. Duncan, J. M., and Chang, C. Y. (1970) "Nonlinear Analysis of Stress and Strain in Soils," Journal of the Soil Mechanics and Foundations Division, ASCE, Vol. 96, No. SM5, September.

4. "Elastic and Damping Properties, Laydown Area, San Onofre Nuclear Generating Station," Report by Woodward-McNeill & Associates, dated 14 October 1971, presented as "Material Property Studies, San Onofre Nuclear Generating Station," PSAR, Amendment 11, Appendix 2E.

5. Hardin, B. O., and Drnevich, V. P. (1972) "Shear Modulus and Damping in Soils: Design Equations and Curves," Journal of the Soil Mechanics and Foundations Division, ASCE, Vol. 98, No. SM7, Proc. Paper 9006, July, pp. 667-692.

6. Idriss, I. M., Dezfulian, H., and Seed, H. B. (1969) "Computer Programs for Evaluating the Seismic Response of Soil Deposits with Non-Linear Characteristics Using Equivalent Linear Procedures," Research Report, Geotechnical Engineering, University of California, Berkeley.

7. Idriss, I. M., and Seed, H. B. (1974) "Seismic Response by Variable Damping Finite Elements," Journal of the Geotechnical Engineering Division, ASCE, Vol. 100, No. GT1, January.

8. Idriss, I. M., Lysmer, J., Hwang, R., and Seed, H. B. (1972) "Computer Programs for Evaluating the Seismic Response of Soil Structures by Variable Damping Finite Elements," Report, Earthquake Engineering Research Center, University of California, Berkeley, August.

9. Kaldjian, M. J. (1969) "Discussion of Design Procedures for Dynamically Loaded Foundations," by R. V. Whitman and F. E. Richart Jr., Paper No. 5569, Journal of the Soil Mechanics and Foundations Division, Proc. Paper, ASCE, Vol. 95, No. SM1, January.

10. Kulhawy, R. H., Duncan, J. M., and Seed, H. B. (1969) "Finite Element Analyses of Stresses and Movements in Embankments During Construction," Report No. TE 69-4, Office of Research Services, University of California, Berkeley.

11. Lysmer, J., Seed, H. B., and Schnabel, P. B. (1970) "Influence of Base Rock Characteristics on Ground Response," Earthquake Engineering Research Center Report No. EERC 70-7, University of California, Berkeley, November.

12. McNeill, R. L., and Mathur, J. N. (1973) "Soil Structure Interaction--A State of the Art Review," Presented at the Structural Engineers Association of Southern California Convention, San Diego, California, 1973.

13. McNeill, R. L. (1969) "Machine Foundations--The State of the Art," Proc. 7th International Conference on Soil Mechanics and Foundation Engineering, Mexico City, Mexico, August, 1969.

14. Newmark, N. M., and Hall, W. J. (1969) "Seismic Design Criteria for Nuclear Reactor Facilities," Proceedings, 3rd World Conference on Earthquake Engineering, Vol. II, Santiago, Chile.

15. Newmark, N. M., and Rosenblueth, E., "Fundamentals of Earthquake Engineering," Prentice Hall, 1971.

16. Richart, R. E., Jr., Hall, J. R., and Woods, R. D. (1970) "Vibrations of Soil and Foundations," Prentice Hall Inc., New York.

17. Schnabel, P., Seed, H. B., and Lysmer, J. (1971), "Modification of Seismograph Records for Effects of Local Soil Conditions," Earthquake Engineering Research Center, University of California, Berkeley, December.

18. Seed, H. B., and Idriss, I. M. (1970) "Soil Moduli and Damping Factors for Dynamic Response Analyses," Earthquake Engineering Research Center, Report No. EERC 70-10, University of California, Berkeley, December.

19. Seed, H. B., and Idriss, I. M. (1969) "Rock Motion Accelerograms for High Magnitude Earthquakes," Earthquake Engineering Research Center Report No. EERC 69-7, University of California, Berkeley, April.

20. Seed, H. B., Idriss, I. M., and Kiefer, R. W. (1969) "Characteristics of Rock Motions During Earthquakes," Journal of the Soil Mechanics and Foundations Division, ASCE, Vol. 95, No. SM5, September.

3.8 DESIGN OF CATEGORY I STRUCTURES

3.8.1 CONCRETE CONTAINMENT

The containment structure is designed to house the reactor coolant system and is referred to as the containment in the following paragraphs. The containment is part of the containment system whose functional requirements are summarized by the following criteria:

A. The containment must withstand the peak pressure and time-varying thermal gradient resulting from a hypothetical failure of the reactor coolant system or main steam system as discussed in subsection 6.2.1.

B. The containment must provide biological shielding during normal operation and following a postulated loss-of-coolant accident (LOCA) to minimize radiation exposure.

C. The containment must also be leaktight in order to minimize leakage of airborne radioactive materials.

D. The containment must provide approximately 150 penetrations for piping and electrical cabling, as well as personnel and equipment access and provides rigid anchor points for all piping entering or leaving the containment.

This section describes the structural design considerations for the containment. Further information relative to the containment is covered in Topical Report BC-TOP-5[(1)] which provides the bases for design, construction, testing, and surveillance for the prestressed concrete containment.

3.8.1.1 Description of the Containment

3.8.1.1.1 General

The basic configuration of each containment structure consists of a prestressed, reinforced concrete cylindrical structure with a hemispherical dome and a conventionally reinforced concrete basemat with a reactor cavity approximately at its center. Figure 3.8-1 illustrates this configuration and also shows the relationship between the external shell and the internal floors and walls. The internal structure is separated from the shell by a peripheral gap of 6 inches at each floor to avoid any interaction between the floors and the shell during a seismic event. The walls are connected to the basemat by means of reinforcing steel and cadwelds welded to the liner plate. The arrangement of the containment in relation to its' surrounding buildings is illustrated in figures 1.2-4, 1.2-11, and 1.2-12. As indicated in these figures, the containment is separated from the surrounding buildings by means of a 12-inch gap to avoid any interaction with the surrounding building during a seismic event.

The dome and cylinder are reinforced with bonded reinforcing steel as required by the design loading conditions. The quantity of reinforcing steel provided, always satisfied the minimum requirement specified for crack control (refer to figures 3.8-3 and 3.8-4 for typical details). Additional bonded reinforcing is provided at discontinuities and around openings in the shell. A continuous tendon access gallery below the basemat is provided for installation and inspection of the vertical post-tensioning system (refer to figure 3.8-2). A welded carbon steel liner plate is provided on the inside surface of the basemat, shell wall, and dome (refer to figures 3.8-8, 3.8-9, and 3.8-10 respectively). The basemat liner plate system is covered with concrete for protection. Typical basemat, shell wall, and dome reinforcing steel details are shown in figures 3.8-2 through 3.8-4.

Principal nominal dimensions of the containment are as follows:

Interior diameter	150 ft
Interior height (above filler slab)	170 ft
Cylindrical wall thickness	4 ft - 4 in. (nominal)
Dome thickness	Varies from 4 ft - 4 in. (nominal) at the springline to 3 ft - 9 in. (minimum) at the top of the dome
Basemat thickness	9 ft
Liner plate thickness	1/4 in.
Internal free volume	2,305,000 ft^3 minimum

3.8.1.1.2 Post-Tensioning System (Figures 3.8-5 through 3.8-7)

The tendon system employed in each of the two containment structures is shown in figures 3.8-5 and 3.8-6. The system uses VSL's E5-55 tendons each consisting of 55, 1/2-inch diameter, high-strength seven-wire strands and VSL anchorage components consisting of wedges and wedge blocks (refer to figure 3.8-7). The tendons transfer load to the structure through bearing plates.

The unbonded tendons are installed in tendon sheathing, which form ducts through the concrete between anchorage points. The tendon sheathing is a galvanized, spiral-wrapped, semi-rigid corrugated steel tubing. It is designed to retain its shape and resist construction loads. Trumpets, which are enlarged ducts attached to the bearing plate, allow the strands to spread out at the anchorage to suit wedge block spacing requirements.

Tendon sheathing provides an enclosed space surrounding each tendon. A valved vent at the highest points of curvature permits release of entrapped air during greasing operations. Drains are provided at the lowest points of curvature to remove accumulated water prior to installing tendons. After the greasing operation, the vents and drains are closed and sealed.

The prestressing tendons are protected against atmospheric corrosion during shipment and installation, and during the life of the containment. Prior to shipment, the tendons are coated with a thin film of petrolatum containing rust inhibitors. The interior surface of the sheathing is coated with a suitable material during manufacture to minimize removal of the petrolatum from the tendon wires during pulling through the sheathing. The sheathing filler material used for permanent corrosion protection is a modified, refined petroleum-base product. The material is pumped into the sheathing after stressing. Details of the corrosion protection are given in paragraph 3.8.1.6.3.4.

3.8.1.1.2.1 Cylindrical Wall Prestressing. The vertical tendons are comprised of 90 inverted U-shaped tendons, which extend through the full height of the containment shell wall and over the dome, and are anchored at the bottom of the basemat. Eighty-four horizontal tendons are anchored at three buttresses equally spaced around the cylinder. Each horizontal tendon is anchored at buttresses located 240° apart. The successive horizontal tendons are anchored at alternate buttresses, resulting in two complete hoops for three consecutive horizontal tendons. Refer to figure 3.8-5, sheet 1, for buttress arrangements and to figure 3.8-5, sheet 2, for schematic arrangements of hoop tendons.

3.8.1.1.2.2 Dome Prestressing. Prestressing of the hemispherical dome is achieved by a two-way pattern of tendons, which are an extension of the continuous vertical U-shaped tendons and 30 hoop tendons which start at the springline and continue up to the 90° solid angle of the dome. Refer to figure 3.8-6 for schematic arrangements of dome tendons.

3.8.1.1.3 Liner Plate System (Figures 3.8-9 through 3.8-11)

3.8.1.1.3.1 Liner Plate and Anchors. A welded steel liner plate covers the entire inside surface of the containment (excluding penetrations) to satisfy the leaktight criteria. The liner is typically 1/4-inch thick and is thickened locally around penetration sleeves, large brackets, and attachments to the basemat and shell wall. The stability of the liner plate, including the thickened plate, is controlled by anchoring it to the concrete structure. The shellwall and dome liner plate system is also used as a form for construction. Typical details of the liner plate system and anchors are shown in figures 3.8-9 and 3.8-10.

3.8.1.1.3.2 Equipment and Personnel Penetration Assemblies. Penetration assemblies consist of steel penetration sleeves, reinforcing plates, and anchors. A circular equipment hatch and two personnel airlock assemblies penetrate the concrete cylinder walls. They are anchored to the concrete walls and are welded to the steel liner. Hatch and lock doors are provided with double-gasketed flanges with provisions for leak testing the flange-gasket combinations. Details are shown in figure 3.8-11.

One of two personnel locks is for emergency access only. Each personnel lock has a door at each end and is an ASME coded steel pressure vessel. A quick-acting equalizing valve connects the personnel lock with the interior and exterior of the containment to equalize pressure in the two systems.

During plant operation, the two doors of each personnel lock are interlocked to prevent both being opened simultaneously. Remote indicating lights and annunciators in the control room indicate the doors operational status. Provision is made to bypass the interlock system during plant cold shutdown.

3.8.1.1.3.3 Process Pipe Penetration Assemblies. Single barrier piping penetrations are provided for all piping passing through the containment walls. The closure for process piping to the liner plate is accomplished with a special flued head welded into the piping system and to the penetration sleeve which is, in turn, welded to a reinforced section of the liner plate. In the case of piping carrying hot fluid, the pipe is insulated to prevent excessive concrete temperatures and to prevent excessive heat loss from the fluid. Closures to these penetration assemblies are provided by the piping systems that are served by the penetrations. For typical details of the cold and hot pipe penetration assemblies used in the shell wall, refer to figure 3.8-11, sheet 2.

3.8.1.1.3.4 Electrical Penetration Assemblies. The two electrical penetration assembly configurations used for all electrical conductors passing through the shell wall are the canister-type and the modular (pancake) type assemblies. The canister-type assemblies are used for the medium-voltage power circuits, 6900 volts. The modular-type assemblies are used for the low-voltage power circuits, 600 volts and below.

The penetration canister is a hollow cylinder bolted at one end to the steel penetration sleeve flange and closed at both ends with sealed header plate assemblies. The canister is provided with a test plug on the outside of the containment to allow test pressurization of the penetration assembly.

The modular-type assemblies consist of a header plate in which a group of small, interchangeable, modular penetrations are fitted. The header plate mates with the flange welded to the nozzle, which is bolted to the penetration sleeve flange. Dougle silicone O-rings provide a monitorable seal between the header plate and the flange. The header plate is provided with a monitoring hole on the outside of the containment to allow test pressurization of the penetration assembly. The method of sealing the electrical penetration assemblies depends upon the type of cable and connector assembly involved. In general, three types are used:

- Type 1 - Medium-voltage power, 6900 volts (canister-type assembly)
- Type 2 - Low-voltage power and control, 600 volts and below (modular-type assembly)
- Type 3 - Instrumentation, thermocouple leads, coaxial, and other special wires (modular-type assembly)

Type 1 penetrations consist of insulated conductors that are passed through the header plates and potted to effect a pressure seal. Mechanical splices within the potting compound provide gas stops. High-voltage insulating bushings and seals are also used to provide the barrier.

Type 2 and 3 penetrations have conductors that are passed through the modules and potted to effect a pressure seal. Epoxy-encapsulated splices within the potting compound provide gas stops. The module is fitted into the header plate with the O-rings in each pressure barrier acting to make the necessary pressure seal.

For typical details of an electrical penetration assembly, refer to figure 3.8-11, sheet 1.

3.8.1.1.3.5 <u>Fuel Transfer Tube</u>. A fuel transfer tube penetration is provided for refueling. An inner pipe acts as the refueling tube with an outer pipe as the housing. The tube is fitted with a double-gasketed blind flange in the refueling canal and a standard gate valve in the spent fuel pool. This arrangement prevents leakage through the refueling tube. Outer sleeves permit the transfer tube to penetrate the secondary shield wall, the containment shell, and the exterior wall of the fuel handling building, while maintaining a pressure-tight boundary at each wall. The sleeves are anchored into each wall respectively and welded to each wall's liner plate. Sleeve bellows at the interior face of both the containment shell and the exterior wall of the fuel handling building permit thermal expansion of the transfer tube. The same expansion bellows permit differential movement between structures. Details are shown in figure 3.8-12.

3.8.1.1.3.6 <u>Attachments and Brackets</u>. Attachments to the shell wall are brackets for support of the service polar crane, electrical conduit and cable tray, spray piping, dome lighting, dome ventilation, and safety injection valves. The polar crane support brackets consist of built-up steel plate, the top flange penetrating the thickened liner plate, and are anchored in the concrete of the shell wall. For details, see figure 3.8-13.

Attachments to the base slab include anchor bolts for equipment support and reinforcing steel for internal structures support. Attachment is accomplished by connectors welded to both top and bottom of thickened liner plate.

3.8.1.1.4 Shell Discontinuities

Significant discontinuities in the shell structure are at the wall-to-base-slab connection, the buttresses, and the large penetration openings.

3.8.1.1.4.1 Wall-to-Base-Slab Connection. The shell wall interface at the base slab incorporates a haunched design in order to accommodate large moments due to horizontal seismic excitation. Refer to figures 3.8-2, and 3.8-3, sheet 2, for details of the lower wall configuration.

3.8.1.1.4.2 Buttresses. Buttresses project out from the shell wall and dome surface to provide adequate space for hoop tendon anchorage and tendon stressing equipment. The anchorage surfaces of the buttress are normal to the tangent line of hoop tendons anchored. Details are shown in figure 3.8-5, sheet 1.

3.8.1.1.4.3 Large Penetration Openings. The concrete shell around the equipment hatch opening is thickened by the method shown in figure 3.8-3, sheet 2.

3.8.1.2 Applicable Codes, Standards and Specifications

The following codes, standards, and regulations, specifications, design criteria, and NRC Regulatory Guides constitute the basis for the design, fabrication, construction, testing, and inservice inspection of both containment structures. Modifications to these codes, standards, etc. are made when necessary, to meet the specific requirements of the structure. These modifications are indicated in the sections where references to the codes, standards, etc. are made.

3.8.1.2.1 Codes

A. Uniform Building Code (UBC), 1970 Edition.

B. American Institute of Steel Construction (AISC), Manual of Steel Construction, 1970 Edition.

C. American Concrete Institute (ACI) 318-71, Building Code Requirements for Reinforced Concrete.

D. American Society of Mechanical Engineers (ASME), Boiler and Pressure Vessel Code, 1971 Edition and Addenda through Winter 1972.

1. Section II, Material Specifications - Part A - Ferrous

2. Section III, Nuclear Power Plant Components, Division 1

3. Section V, Nondestructive Examination

4. Section VIII, Pressure Vessels - Division 1

5. Section IX, Welding and Brazing Qualifications

E. American Welding Society (AWS), AWSD1.1-72, Structural Welding Code.

3.8.1.2.2 Standards and Regulations

A. Occupational Safety and Health Act (OSHA)

B. State of California, Division of Industrial Safety, General Industry Safety Orders

C. Property Loss Prevention Standard for Nuclear Generating Stations, Nuclear Mutual Limited (NML), June 1974 Edition.

D. National Fire Protection Association (NFPA), NFPA No. 24, 1973 Edition, Outside Protection.

3.8.1.2.3 Specifications

A. Industry Specifications

1. American Society for Testing and Materials (ASTM)

ASTM standard specifications are used whenever possible to describe material properties, testing procedures, and fabrication and construction methods. The standards used and the exception to these standards, if any, are identified in the applicable sections

2. American Concrete Institute (ACI), ACI 301, Specification for Structural Concrete for Buildings, May 1972

3. American Iron and Steel Institute (AISI), Specification for the Design of Light Gage, Cold-Formed Steel Structural Members, 1968 Edition

4. Crane Manufacturers Association of America (CMAA), CMAA Specification No. 70, 1971

B. Project Design and Construction Specifications

Project design and construction specifications are prepared to cover the areas related to design and construction of the containment. These specifications, prepared specifically for the San Onofre Nuclear Generating Station, Units 2 and 3, emphasize important points of the industry standards for the design and construction of the containment, and reduce options that otherwise would be permitted by the industry standards. Unless specifically noted otherwise, these specifications do not deviate from the applicable industry standards. They cover the following subject headings.

1. Excavation and Backfill
2. Concrete Placement
3. Inspection of Concrete Production
4. Reinforcement Steel Placement
5. Structural Steel Erection
6. Miscellaneous Metalwork Installation
7. Stainless Steel Liner Plate System Installation
8. Post-Tensioning System Embedded Items Installation
9. Concrete and Concrete Products
10. Reinforcing Steel and Associated Products
11. Prestressing Steel and Related Accessories
12. Structural Steel
13. Miscellaneous Steel and Embedded Materials
14. Stainless Steel Liner Plate
15. Containment Polar Cranes
16. Containment Liner Plate System including Locks and Hatches
17. Fuel transfer tube.

3.8.1.2.4 Design Criteria

A. Project Design Criteria

Project design criteria are prepared to include comprehensive design requirements of the containment, and contain specific references to prescribed Bechtel internal design guides, applicable industry standards, and pertinent technical texts, journals, and published reports.

B. Bechtel Topical Reports

1. BC-TOP-1, Containment Building Liner Plate Design Report, Revision 1, December 1972 with additional information dated September 1973.

2. BC-TOP-4, Seismic Analysis of Structures and Equipment for Nuclear Power Plants, Revision 1, September 1972

3. BC-TOP-5, Prestressed Concrete Nuclear Reactor Containment Structures, Revision 1, December 1972

4. BC-TOP-7, Full-Scale Buttress Test for Prestressed Nuclear Containment Structures, August 1971 (reprinted September 1972)

5. BC-TOP-8, Tendon End Anchor Reinforcement Test, November 1971

6. BC-TOP-9A, Design of Structures for Missile Impact, Revision 2, September 1974

7. BN-TOP-1, Testing Criteria for Integrated Leakage Rate Testing of Primary Containment Structures for Nuclear Power Plants, Revision 1, November 1972

8. BN-TOP-2, Design for Pipe Break Effects, Revision 1, September 1973

9. BP-TOP-1, Seismic Analysis of Piping Systems, Revision 0, April 1973

C. Project Reports

1. Seismic and Foundation Studies, April 15, 1970, Dames and Moore.

2. Methods of Direct Application of Element Damping - San Onofre Units 2 and 3, January 1972, Bechtel Power Corporation, Los Angeles office.

3. Development of Soil-Structure Interaction Parameters, Proposed Units 2 and 3 San Onofre Generating Station, January 31, 1974, Woodward-McNeil & Associates.

4. Elastic and Damping Properties, Laydown Area, San Onofre Nuclear Generating Station, Woodward-McNeil & Associates, Orange, CA, October 14, 1971.

5. Preliminary Safety Analysis Report - San Onofre Units 2 and 3.

3.8.1.2.5 NRC Regulatory Guides

A. Regulatory Guide 1.10, Mechanical (Cadweld) Splices for Reinforcing Bars of Category 1 Concrete Structures, Revision 1, January 1973

B. Regulatory Guide 1.15, Testing of Reinforcing Bars for Category I Concrete Structures, Revision 1, December 1972

C. Regulatory Guide 1.18, Structural Acceptance Test for Concrete Primary Reactor Containments, Revision 1, December 1972

D. Regulatory Guide 1.19, Nondestructive Examination of Primary Containment Liner Welds, Revision 1, August 1972

E. Regulatory Guide 1.55, Concrete Placement in Category I Structures, June 1973.

3.8.1.3 Loads and Load Combinations

3.8.1.3.1 Load Definitions

The containment is designed for all credible loading conditions. The design load categories are identified as preoperational pressure test loads, normal loads, severe environmental loads, extreme environmental loads, and abnormal loads.

3.8.1.3.1.1 Preoperational Pressure Test Load. Upon completion of construction, the containment and its penetrations are tested at 115% of the design LOCA pressure as discussed in subsection 6.2.6. This pressure is considered in the design.

3.8.1.3.1.2 Normal Loads. Normal loads are those loads to be encountered during normal plant operation and shutdown. They include the following:

A. Dead Loads

Dead load consists of the weight of the concrete wall, dome, base slab, steel, and permanently attached equipment, and in addition includes hydrostatic loads that consist of lateral hydrostatic pressure resulting from ground or flood water, as well as buoyant forces resulting from the displacement of ground or flood water by the structure.

B. Live Loads

Live loads consist of any movable equipment loads and other loads with variable intensity and occurrence, such as soil pressures.

C. Prestressing Loads

Prestressing loads consist of the compressive forces due to prestressing tendons.

D. Normal Thermal Loads

Normal thermal loads are produced due to the temperature distribution through the wall during normal operating or shutdown conditions, based on the most critical transient or steady-state condition.

E. Normal Pipe Expansion Loads

Normal pipe expansion loads consist of local forces on the structure caused by thermal expansion of piping during normal operating or shutdown conditions, based on the most critical transient or steady-state condition.

3.8.1.3.1.3 <u>Severe Environmental Loads</u>. Severe environmental loads are those loads that could infrequently be encountered during the plant life. Included in this category are:

A. Operating Basis Earthquake (OBE)

The OBE consists of a static equivalent seismic load for which the dynamic effects have been included in its determination. A more detailed discussion is presented in subsection 3.7.1.

B. Wind Loads

Refer to subsection 3.3.1 for a detailed description of wind loads.

3.8.1.3.1.4 <u>Extreme Environmental Loads</u>. Extreme environmental loads are those loads that are credible but are highly improbable. They include the following.

A. Design Bases Earthquake (DBE)

The DBE consists of a static equivalent seismic load for which the dynamic effects have been included in its determination. A more detailed discussion is presented in subsection 3.7.1.

B. Tornado Loads

Tornado loads consist of the combined effects of tornado wind pressure, pressure differential, and missile impingement. Refer to subsection 3.3.2 for a detailed description.

3.8.1.3.1.5 Abnormal Loads. Abnormal loads are those loads generated by a postulated high-energy pipe break accident within a building and/or compartment thereof. Included in this category are the following:

A. Pipe Rupture and Miscellaneous Missile Loads

Pipe rupture loads consist of local loads on the structure generated by either jet impingement from, or the reaction of, a ruptured high-energy pipe, and missile impact due to or during a postulated pipe break (e.g. pipe whipping), all of which are applied as a static equivalent load that includes an appropriate dynamic factor. Pipe rupture effects are further discussed in section 3.6.

Miscellaneous missile loads are described in detail in section 3.5.

B. Design Pressure Load

The design pressure load of the containment is greater than the calculated peak pressure occurring as the result of any rupture of the reactor coolant system or main steam system. The basis for the containment design pressure of 60 $lb/in.^2g$ is presented in subsection 6.2.1.

C. Abnormal Thermal Loads

Abnormal thermal loads are produced due to the temperature gradient through the wall and expansion of the liner during a LOCA or main steam line break (MSLB).

D. Abnormal Pipe Expansion Loads

Abnormal pipe expansion loads consist of local forces on the structure caused by thermal expansion of piping during a LOCA or MSLB, based on the most critical transient or steady-state condition.

3.8.1.3.2 Load Combinations

Two types of loading cases are considered in the design of the containment

A. The service load conditions for which the working stress method is used.

B. The factored load conditions for which the strength design method is used.

The following nomenclature is used in the loading combination equations:

C = required capacity of the containment to resist factored loads.

ϕ = capacity reduction factor (defined in paragraph 3.8.1.3.2.4)

D = dead loads

L = appropriate live load

F_T = prestress at transfer load

F = sustained prestress load

T_o = normal thermal loads

H_o = normal pipe expansion load

E = operating basis earthquake load

W = wind load

E' = design basis earthquake load

W_T = tornado load

R = pipe rupture and miscellaneous missile loads

P = LOCA or MSLB pressure load

P_T = preoperational pressure test load

T_A = abnormal thermal loads

H_A = abnormal pipe expansion load

3.8.1.3.2.1 <u>Service Load Conditions</u>

A. Preoperational Pressure Test Case

$D + F + P_T$

B. Normal Case

$D + F + L + T_o$

C. Abnormal Case

$D + F + L + P + T_A$

3.8.1.3.2.2 <u>Factored Load Conditions</u>

A. Abnormal Case

$1.0\ D + 1.5\ P + 1.0\ T_A + 1.0\ F$

B. Abnormal/Severe Environmental Case

$1.0\ D + 1.25\ P + 1.0\ T_A + 1.0\ H_A + 1.25\ E + 1.0\ F$

C. Abnormal/Severe Environmental Case

$1.0\ D + 1.25\ P + 1.0\ T_o + 1.25\ H_o + 1.25\ E + 1.0\ F$

D. Abnormal/Severe Environmental Case

$1.0\ D + 1.0\ H_A + 1.0\ R + 1.0\ F + 1.25\ E + 1.0\ T_A$

E. Abnormal/Severe Environmental Case

$1.0\ D + 1.25\ H_o + 1.0\ R + 1.0\ F + 1.25\ E + 1.0\ T_o$

F. Abnormal/Extreme Environmental Case

$1.0\ D + 1.0\ P + 1.0\ T_A + 1.0\ H_A + 1.0\ E' + 1.0\ F$

G. Abnormal/Extreme Environmental Case

$1.0\ D + 1.0\ P + 1.0\ T_o + 1.25\ H_o + 1.0\ E' + 1.0\ F$

H. Abnormal/Extreme Environmental Case

$1.0\ D + 1.0\ H_A + 1.0\ R + 1.0\ E' + 1.0\ F + 1.0\ T_A$

I. Abnormal/Extreme Environmental Case

$$1.0\ D + 1.25\ H_o + 1.0\ R + 1.0\ E' + 1.0\ F + 1.0\ T_o$$

J. Extreme Environmental Case

$$1.0\ D + 1.25\ H_o + 1.0\ F + 1.0\ T_o + 1.0\ W_T$$

3.8.1.3.2.3 Capacity Reduction Factors. The capacity reduction factor provides for the possibility that small adverse variations in material strengths, workmanship, dimensions, control, and degree of supervision, while individually within required tolerances and the limits of good practice, occasionally may combine to result in undercapacity. It is applied to the ultimate strength capacity of the section being designed and to the allowable stresses. Capacity reduction factors are:

ϕ = 0.90 for concrete in flexure with or without axial tension

ϕ = 0.85 for shear and torsion

ϕ = 0.75 for spirally reinforced concrete compression members

ϕ = 0.70 for tied compression members

ϕ = 0.90 for reinforcing steel in direct tension

ϕ = 0.90 for mechanical splices of reinforcing steel

ϕ = 0.95 for prestressed tendons in direct tension

For members subject to flexure and axial compression, the provisions of ACI 318-71 apply.

3.8.1.4 Design and Analysis Procedures

The containment is analyzed for various loading combinations, considering the values of individual loads that generate the most significant stress condition for each component and member of the structure.

The critical areas for analysis are the basemat, the intersection between cylinder wall and basemat, the liner plate system, the tendon anchorage zones, and the penetration openings.

Computer programs are relied upon to perform many of the computations required for the containment analysis. However, classical theory, empirical equations, and numberical methods are applied as necessary for analysis of localized areas and for preliminary proportioning. They are described in Section 7 of BC-TOP-5.[1]

The design methods incorporate several phases as described in Section 6 of BC-TOP-5. Improved assumptions as to material properties, including the effects of creep, shrinkage, and cracking on concrete, are used in design. Analysis and design of tendon anchorage zones and reinforcement in buttresses are discussed in BC-TOP-5, BC-TOP-7,(2) and BC-TOP-8.(3) The method of analyzing the effects of penetrations, the thickening of walls, reinforcements, and embedments, etc., is discussed in Section 7 of BC-TOP-5. The design of the liner and its anchorage system is covered in BC-TOP-1(4) and BC-TOP-5. Information on analyses for computation of seismic loads is provided in section 3.7.

3.8.1.4.1 Analytical Techniques

The analysis of the containment consists of two parts: the overall analysis of the containment and the local analysis. The overall analysis, given axisymmetric loads, is performed by utilizing the FINEL finite-element computer program for combinations of the individual loading cases of dead, live, thermal, pressures, and prestress loads. In the case of nonaxisymmetric loads (i.e., Seismic Loads), the analysis is performed using the ASHSD finite-element computer program.

The axisymmetric finite-element representation of the containment assumes that the structure is axisymmetric. This does not account for the buttresses, penetrations, brackets, and liner plate anchors. These items are considered in the local analysis using either computer programs (e.g., equipment hatch analysis), or principles of structural analysis (e.g., polar crane bracket analysis). In addition, some of the design (e.g., buttresses) is based on test results.

3.8.1.4.1.1 <u>Overall Analysis</u>. The containment is considered an axisymmetric structure for the overall analysis. Although there are deviations from this ideal shape, the deviations are usually localized and can be handled by special analyses; hence, axisymmetric analyses are considered acceptable.

The overall analysis of the containment, given the application of axisymmetric loads, is performed by Bechtel's nonlinear FINEL finite-element computer program. A detailed description of this program is provided in appendix 3C, section 3C.1. The entire containment is modeled with one finite-element mesh consisting of the shellwall, basemat, internal structure and soil.

The entire concrete structure is modeled by continuously interconnected elements. The geometry of the mesh allows the representation of reinforcing steel superimposed on the corresponding concrete elements.

The liner plate is simulated by a layer of elements attached to the interior surfaces of the concrete structure.

The finite-element mesh of the structure is extended into the soil to account for the elastic nature of the soil material and its effect on the behavior of the basemat. The tendon access gallery is analyzed as a separate structure.

The use of the nonlinear finite-element analysis permits accurate determination of the stress pattern at any location of the structure.

The FINEL finite-element mathematical model for axisymmetric loads is shown on figure 3.8-14.

The overall analysis of the containment, given the application of non-axisymmetric loads, is performed by Bechtel's linear elastic ASHSD finite-element computer program. A detailed description of this program is provided in appendix 3C, subsection 3C.2. Details of the seismic analysis are described in section 3.7. Wind and tornado loadings are discussed in subsections 3.3.1 and 3.3.2.

3.8.1.4.1.2 Local Analysis. The local analyses of the containment include the following:

A. Buttress and Tendon Anchorage Zones

The containment has three buttresses. At each buttress, two out of any group of three hoop tendons are anchored on the opposite faces of the buttress, with the third tendon continuous through the buttress.

Between the opposite anchorages in the buttress, the compressive forces exerted by the anchored tendons are larger than elsewhere in the shell wall. This value, combined with the effect of the tendon, which is continuous throughout the buttress, is 1.5 times the prestressing forces acting outside the buttress. The thickness of the buttress is approximately 1.5 times the thickness of the wall. Hence, the hoop stresses and strains, as well as the radial displacements, may be considered as being nearly constant all around the structure.

The design of the tendon anchorage zones is based on two test programs conducted by Bechtel to demonstrate the adequacy of several reinforcing patterns for use in anchorage-zone concrete in the basemat and buttresses. These tests have been undertaken to develop a more efficient design to reduce reinforcement congestion, and thereby facilitate the placement of high quality concrete around the tendon anchorages. The test programs are as follows:

1. A full-scale model of a simulated containment buttress containing several patterns of reinforcement and types of tendon anchorages was constructed and tested. A detailed description of the test is presented in BC-TOP-7.[2]

2. Two large concrete test blocks containing two patterns of reinforcement with different proportions of reinforcing bars were constructed and tested. A detailed description of the test is presented in BC-TOP-8.[3]

The test results demonstrate satisfactory performance of the test anchorages. The design of the tendon anchorage zones is based on the results and recommendations of these tests.

B. Large Penetration Openings

Large penetrations are defined as those having an inside diameter equal to or greater than 2.5 times the containment nominal shell wall thickness. The equipment hatch falls into this category.

The stresses at the opening are predicted by an analysis performed using Bechtel's computer program SAP, described in appendix 3C, subsection 3C.5, which is capable of performing a static analysis of linear elastic three-dimensional structures utilizing the finite-element method. The points delineating the outermost boundaries of the analytical model are located at approximately two penetration diameters beyond the center of the opening, so that the behavior of the model along the boundaries is compatible with that of the undisturbed cylindrical wall.

Typical details of the equipment hatch are shown in figure 3.8-3, sheets 1 and 2. Figure 3.8-15 shows the equipment hatch boundary conditions. Figure 3.8-16 shows the finite-element model geometry. Figure 3.8-17 through 3.8-34 show the finite-element mesh for truss elements and brick elements, respectively, used for the analyses of the equipment hatch. The brick elements are used to model the concrete, and the truss elements are used to represent the post-tensioning system. Figure 3.8-18 shows a section through the equipment hatch wall showing the layered elements.

C. Small Penetration Openings

Small penetration openings are defined as those having an inside diameter less than 2.5 times the containment nominal shell wall thickness. The stresses at the openings due to applied moments and forces are determined using the methods outlined in reference 5.

Results of these analyses show the stresses to be well within the allowable limits. Typical details of small penetrations are shown in figure 3.8-11.

3.8.1.4.1.3 <u>Variation in Analytical Assumptions and Material Properties</u>. The treatment of the effects of expected variation in assumptions and material properties on the analysis results is discussed in paragraphs 3.7.2.1.9 and 3.8.1.3.2.4.

3.8.1.4.2 Steel Liner Plate and Penetrations

The analytical techniques and procedures used in the design of the liner plate are outlined in detail in reference 4. The steel liner plate and penetrations are designed to serve as the leakage barrier for the containment. Typical details for the liner plate and penetrations are shown in figure 3.8-11.

The design of the liner plate considers the composite action of the liner and the concrete structure, and includes the transient effects on the liner due to temperature changes during construction, normal operation, and the postulated LOCA. The changes in strains to be experienced by the liner due to these effects, and those at the pressure testing of the containment, are considered.

The stability of the liner is achieved by anchoring it to the concrete structure. At all penetrations, the liner is thickened to reduce stress concentration. The thickened plate is also anchored to the concrete.

Insert plates and/or bracket embeds are provided in the liner to transfer concentrated loads to the wall, slab, and dome of the containment. The polar crane brackets are an example of these concentrated loads. A typical bracket detail is shown in figure 3.8-13.

3.8.1.4.3 Description of Computer Programs

Computer programs used in the design and analysis of the containment are described in appendix 3C.

3.8.1.5 Structural Acceptance Criteria

The fundamental acceptance criterion for the containment as a prototype is the successful completion of the structural integrity test, with measured responses within the allowable limits, including strain measurements in the concrete sufficient to permit a complete evaluation of strain distribution at prescribed locations. Strain measurements are taken in accordance with NRC Regulatory Guide 1.18 with the exception that no measurement is taken under a vertical tendon anchor, since full-scale tests on the size of tendon used have shown the adequacy of the anchorage system as discussed in BC-TOP-7(2) and -8.(3) The limits for allowable values for stress and strain are given in table 3.8-1. In this way, the margins of safety associated with the design and construction of the containment are, as a minimum, the accepted margins associated with nationally recognized codes of practice. The accepted margins will be compatible with the provisions of ASME Section III, Division I, and the ACI-318-71 codes. In addition, the measured responses are compared to those predicted by the analyses.

Table 3.8-1
ALLOWABLE STRESSES AND STRAINS

Loading Condition	Allowables: Concrete $(f_c')_{90} = 6$ k/in.^{2}	Reinforcement $f_y = 60$ k/in.^{2}	Liner $f_y = 24$ k/in.^{2} and 38 k/in.^{2}
At transfer of prestress	Mem. = $0.3\ f_c'$		
	Mem. + Bend. = $-0.6\ f_c'$	$f_s = \pm 0.5\ f_y$	$f_s = 0.5\ f_y$ (ten.)
	Memb. Ten. = $\sqrt{f_c'}$		$\varepsilon_s = 0.004$ (Comp.)
Design loads	Mem. + Bend. = $-0.6\ f_c'$	$f_s = \pm 0.5\ f_y$	$f_s = 0.5\ f_y$ (Ten.)
	Mem. Ten. = 0		$\varepsilon_s = 0.004$ (Comp.)
Factored loads(a)	Mem. + Bend. = $-0.9\ f_c'$		
	Comp. Strain = -0.003	$f_s = \pm 0.9\ f_y$	$\varepsilon_s = \pm 0.005$

a. In the San Onofre Units 2 and 3 PSAR, the capacity reduction factors were applied to the loading combinations. In this table, the capacity reduction factors are applied to the allowable stresses.

The structural integrity test is planned to yield information on both the overall response of the containment and the response of localized areas, such as major penetrations and buttresses, which are important to the design functions, of the containment.

The design and analysis methods, as well as the type of construction and construction materials, are chosen to allow assessment of the capability of the structure throughout its service life. Additionally, surveillance testing provides further assurances of the continuing ability of the structure to meet its design functions.

Table 3.8-2 shows the calculated stresses and strains, respectively, as well as the allowables, taken from critical sections of the containment structure. The ratios of the allowable stresses and strains to the calculated stresses and the strains yield the margins of safety at selected critical sections.

Table 3.8-2
STRESS ANALYSIS RESULTS (Sheet 1 of 10)

FINAL ANALYSIS LOADING COMBINATIONS
OF CONTAINMENT SHELL

LOADING CONDITIONS(A)	LOADING COMBINATIONS(A)	ANALYSIS PERFORMED (YES OR NO)
AT TRANSFER OF PRESTRESS	$D + F_T$	YES
LOAD UNDER SUSTAINED PRESTRESS	$D + F$	NO(B)
PREOPERATIONAL PRESSURE TEST	$D + F + P_T$	YES
SERVICE LOADS	$D + F + T_O$	YES
	$D + F + T_A + P$	YES
FACTORED LOADS	$D + F + T_A + 1.5P$	YES
	$D + F + T_A + 1.25P + 1.25E$	YES
	$D + F + T_O + 1.25P + 1.25E$	NO(C)
	$D + F + T_A + 1.25E$	YES
	$D + F + T_O + 1.25E$	NO(C)
	$D + F + T_A + P + E'$	YES
	$D + F + T_O + P + E'$	NO(C)
	$D + F + T_A + E'$	YES
	$D + F + T_O + E'$	NO(C)
	$D + F + T_O + W_T$	NO(D)

NOTATION

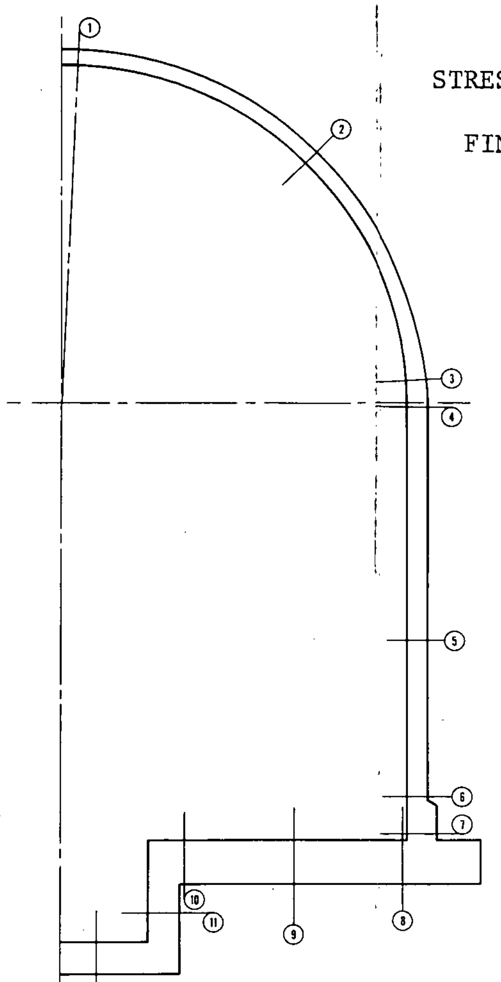
D	= DEAD LOAD	P	= LOCA/MSLB PRESSURE LOAD
E	= OPERATING BASIS EARTHQUAKE	P_T	= PREOPERATIONAL PRESSURE TEST LOAD
E'	= DESIGN BASIS EARTHQUAKE	T_A	= ABNORMAL THERMAL LOAD
F	= PRESTRESS LOAD	T_O	= NORMAL THERMAL LOADS
F_T	= PRESTRESS AT TRANSFER	W_T	= TORNADO LOAD

A. LOADING CONDITIONS AND CORRESPONDING LOADING COMBINATIONS ARE TAKEN FROM PARAGRAPHS 3.8.1.3.2.1 AND 3.8.1.3.2.2 AND SINCE THEY ARE INTENDED TO REFLECT AN OVERALL STRESS AND STRAIN DISTRIBUTION, THEY EXCLUDE THE LOCAL EFFECTS OF LIVE LOAD (L), NORMAL AND ABNORMAL PIPE REACTION LOADS (H_O AND H_A), AND PIPE RUPTURE AND MISCELLANEOUS MISSILE LOADS (R).

B. THE CASE OF LOADING UNDER SUSTAINED PRESTRESS WAS NOT ANALYZED SINCE CONCRETE STRESSES WILL BE LOWER THAN THE PRECEDING CASE. REINFORCEMENT STRESSES WILL BE HIGHER HOWEVER, BUT THEY ARE MAINLY COMPRESSIVE AND ARE WELL WITHIN THE ALLOWABLE LIMITS. LINER PLATE STRESSES ALSO WILL BE HIGHER (IN COMPRESSION); HOWEVER, LINER STRESSES AND STRAINS UNDER SUSTAINED PRESTRESS (INCLUDING CREEP AND SHRINKAGE EFFECTS) HAVE ALREADY BEEN EVALUATED AND TAKEN INTO ACCOUNT PER BECHTEL TOPICAL REPORT (BC-TOP-1)

C. EACH OF THESE LOADING CASES IS LESS CRITICAL THAN THE PRECEDING ONE SINCE ACCIDENT THERMAL EFFECTS ARE MORE SIGNIFICANT.

D. THE EFFECTS OF TORNADO LOADS WERE PREVIOUSLY EVALUATED AND WERE FOUND TO BE LESS CRITICAL THAN THE SEISMIC FORCES. THUS, THIS LOADING CASE IS, FOR EXAMPLE, LESS CRITICAL THAN THAT OF THE LAST CASE ANALYZED.



KEY ELEVATION
(SHOWING LOCATION OF THE TWELVE REFERENCE SECTIONS)

THE FOLLOWING NOTES ARE COMMON TO ALL TABULAR MATERIAL ON SHEETS 2 THROUGH 10

1. RESULTS GIVEN IN TABLES ARE FROM THE NONLINEAR FINITE ELEMENT ANALYSIS EXCEPT FOR THE SEISMIC LOADS.
2. SEISMIC ANALYSIS RESULTS ARE TAKEN FROM SAN ONOFRE 2 AND 3 PRELIMINARY CONTAINMENT ANALYSIS AND SUPERIMPOSED ON FINITE ELEMENT ANALYSIS RESULTS.
3. ONLY THE COMPRESSIVE CONCRETE STRESSES ARE INCLUDED.
4. FULLY CRACKED SECTIONS ARE SHOWN THUS (*). PARTIALLY CRACKED SECTIONS ARE NOT INDICATED.
5. DEFLECTIONS FOR THE BASEMAT ARE VERTICAL; FOR THE WALL RADIAL. DEFLECTIONS FOR THE DOME ARE NORMAL TO THE SURFACE.
6. RADIAL SHEARS AND DEFLECTIONS DO NOT INCLUDE THE EFFECTS OF SEISMIC LOADS.
7. ALLOWABLE STRESSES ARE BASED ON SONGS 2 AND 3 PSAR EXCEPT FOR THE LINER ALLOWABLE COMPRESSIVE STRAINS SHOWN IN THE TABLES, WHICH ARE BASED ON ASME CODE, SECTION III, DIVISION 2.
8. MATERIAL PROPERTIES ARE:
   - fc' = 6 ksi = COMPRESSIVE STRENGTH OF CONCRETE (90 DAYS)
   - fy = 60 ksi = REINFORCEMENT YIELD STRENGTH
   - fy = 24 ksi = 1/4 INCH LINER PLATE YIELD POINT
   - fy = 38 ksi = YIELD POINT OF LINER PLATE GREATER THAN 1/4 INCH
9. SIGN CONVENTIONS ARE:

STRESSES AND STRAINS	(+) TENSILE	(-) COMPRESSIVE
DEFLECTIONS	(+) OUTWARD	(-) INWARD
SECTION AXIAL FORCES	(+) TENSILE	(-) COMPRESSIVE
MOMENTS	(+) TENSION ON THE OUTSIDE FACE	(-) COMPRESSION ON THE OUTSIDE FACE

Table 3.8-2
STRESS ANALYSIS RESULTS (Sheet 2 of 10)

DEAD LOAD + PRESTRESS AT TRANSFER LOAD
$(D + F_t)$

PORTION	SECTION	CONCRETE STRESS		LINER STRAINS		REINFORCING STRESS				SECTION RESULTANTS					DEFLECTION
						INSIDE		OUTSIDE							
		MER PSI	HOOP PSI	MER $\times 10^{-6}$ IN./IN.	HOOP $\times 10^{-6}$ IN./IN.	MER KSI	HOOP KSI	MER KSI	HOOP KSI	MER FORCE K/FT	HOOP FORCE K/FT	MER MOMENT K/FT/FT	HOOP MOMENT K/FT/FT	RADIAL SHEAR K/FT	INCHES
ALLOWABLE		-3600	-3600	+400 -2000	+400 -2000	±30	±30	±30	±30	--	--	--	--	--	--
DOME	1	-893	-890	-210	-200	-4.6	-4.5	-4.7	-4.7	-504	-500	15	22	-2	-0.60
	2	-1190	-1050	-270	-250	-6.1	-5.3	-6.5	-5.0	-662	-585	13	49	13	-0.46
	3	-1270	-1070	-310	-260	-6.5	-5.3	-5.1	-5.0	-641	-590	95	57	6	-0.21
WALL	4	-1070	-1211	-250	-290	-5.3	-6.5	-5.0	-6.2	-652	-769	116	95	24	-0.20
	5	-1090	-1542	-240	-360	-4.5	-8.7	-5.2	-8.3	-687	-1004	-19	80	6	-0.27
	6	-1100	-775	-220	-190	-5.2	-3.8	-6.2	-3.6	-709	-521	-47	33	-38	-0.13
	7	-1550	-495	-450	-180	-5.9	-0.7	-0.7	-0.1	-698	-288	1110	302	13	-0.03
BASE SLAB	8	-253	-140	-30	-50	-1.4	-0.5	5.1	0.5	-20	-55	643	264	47	0.20
	9	-192	-101	-30	-60	-1.0	-0.3	0.7	0.2	-28	-33	410	188	-34	0.26
	10	-44	-39	-10	-10	-0.2	-0.2	0.2	0	-20	-28	95	52	2	0.27
REACTOR CAVITY	11	-83	-24	-10	-10	-0.4	0	-0.5	0	-70	-21	-12	2	18	0.27
	12	-4	5	0	0	0	0.1	0.2	0.1	9	7	20	-2	21	0.27

Table 3.8-2
STRESS ANALYSIS RESULTS (Sheet 3 of 10)

DEAD LOAD + PRESTRESS LOAD + PREOPERATIONAL PRESSURE TEST LOAD
$(D + F + P_T)$

PORTION	SECTION	CONCRETE STRESS		LINER STRAIN		REINFORCING STRESS				SECTION RESULTANTS					DEFLECTION
						INSIDE		OUTSIDE							
		MER PSI	HOOP PSI	MER $\times 10^{-6}$ IN./IN.	HOOP $\times 10^{-6}$ IN./IN.	MER KSI	HOOP KSI	MER KSI	HOOP KSI	MER FORCE K/FT	HOOP FORCE K/FT	MER MOMENT K/FT/FT	HOOP MOMENT K/FT/FT	RADIAL SHEAR K/FT	INCHES
ALLOWABLE		-3600	-3600	+400 -2000	+400 -2000	±30	±30	±30	±30	--	--	--	--	--	--
DOME	1	-154	-151	-19	0	-0.5	-0.5	-0.8	-0.8	-74	-71	-9	-8	-1	-0.20
	2	-405	-269	-73	-62	-1.9	-2.0	-2.4	-1.1	-212	-147	-9	15	11	-0.13
	3	-378	-139	-85	-34	-2.1	-0.3	-1.8	-0.3	-196	-72	25	12	2	-0.01
WALL	4	-358	-128	-80	-32	-2.1	-0.3	-1.8	-0.3	-208	-73	49	20	4	-0.01
	5	-404	-177	-91	-43	-2.3	-0.5	-1.9	-0.5	-242	-105	32	19	0	-0.02
	6	-785	-309	18	-30	-0.5	-1.0	-4.7	-1.0	-264	-160	-260	-33	25	-0.03
	7	-527	-156	343	151	0.9	0.1	-3.4	0.5	-284	21	-509	-48	104	0.01
BASE SLAB	8	-353	-62	563	128	13.9	-0.3	-2.3	2.8	9	48	-1082	55	-197	0.10
	9	-702	-374	-242	-124	-3.6	-1.0	8.7	3.0	-75	-11	1277	616	-123	0.47
	10	-331	-161	-94	-47	-1.9	-0.5	4.1	0.7	-25	-28	687	279	39	0.57
REACTOR CAVITY	11	-199	-7	-32	0	-1.0	0.3	-1.3	0.2	-165	0	-38	-7	42	0.58
	12	101	95	25	25	0.5	0.5	0.7	0.6	98	98	3	-6	6	0.58

/

Table 3.8-2
STRESS ANALYSIS RESULTS (Sheet 4 of 10)

DEAD LOAD + PRESTRESS LOAD + NORMAL THERMAL LOAD
$(D + F + T_o)$

PORTION	SECTION	CONCRETE STRESS MER PSI	CONCRETE STRESS HOOP PSI	LINER STRAINS MER $\times 10^{-6}$ IN./IN.	LINER STRAINS HOOP $\times 10^{-6}$ IN./IN.	REINFORCING STRESS INSIDE MER KSI	REINFORCING STRESS INSIDE HOOP KSI	REINFORCING STRESS OUTSIDE MER KSI	REINFORCING STRESS OUTSIDE HOOP KSI	SECTION RESULTANTS MER FORCE K/FT	SECTION RESULTANTS HOOP FORCE K/FT	SECTION RESULTANTS MER MOMENT K/FT/FT	SECTION RESULTANTS HOOP MOMENT K/FT/FT	SECTION RESULTANTS RADIAL SHEAR K/FT	DEFLECTION INCHES
ALLOWABLE		-3600	-3600	+400 -2000	+400 -2000	±30	±30	±30	±30	--	--	--	--	--	--
DOME	1	-1490	-1488	-450	-450	-6.7	-6.7	0.2	0.3	-438	-440	371	378	-2	-0.40
DOME	2	-1720	-1656	-510	-500	-8.0	-7.3	-1.3	0	-576	-514	369	401	11	-0.29
DOME	3	-1880	-1639	-550	-500	-8.5	-7.0	0	0.3	-559	-491	456	418	3	-0.10
WALL	4	-1650	-1817	-490	-530	-7.7	-8.9	-1.5	-2.6	-570	-671	526	554	15	-0.10
WALL	5	-1610	-2119	-480	-600	-7.3	-11.0	-0.2	-2.4	-604	-873	467	562	8	-0.17
WALL	6	-1600	-1345	-480	-420	-8.4	-7.0	-0.9	-3.0	-626	-594	513	343	-41	0.02
WALL	7	-1830	-829	-590	-320	-7.1	-2.7	0.4	0.5	-614	-277	1512	807	26	0.10
BASE SLAB	8	-651	-388	-210	-160	-4.6	-2.5	5.2	3.4	-217	-120	1256	518	25	0.22
BASE SLAB	9	-923	-548	-340	-230	-6.3	-3.3	3.5	2.6	-307	-162	1462	926	-60	0.23
BASE SLAB	10	-680	-418	-240	-180	-5.2	-3.1	0.8	2.3	-333	-144	1202	754	-11	0.22
REACTOR CAVITY	11	34	(*)	20	80	-0.1	2.3	0.4	4.2	22	38	-16	-28	39	0.23
REACTOR CAVITY	12	89	85	-100	-90	-0.3	-0.3	8.8	7.6	40	34	101	85	22	0.25

Table 3.8-2
STRESS ANALYSIS RESULTS (Sheet 5 of 10)

DEAD LOAD + PRESTRESS LOAD + LOCA/MSLB PRESSURE LOAD + ABNORMAL THERMAL LOAD
$(D + F + P + T_A)$

PORTION	SECTION	CONCRETE STRESS		LINER STRAINS		REINFORCING STRESS				SECTION RESULTANTS					DEFLECTION
						INSIDE		OUTSIDE							
		MER PSI	HOOP PSI	MER $\times 10^{-6}$ IN./IN.	HOOP $\times 10^{-6}$ IN./IN.	MER KSI	HOOP KSI	MER KSI	HOOP KSI	MER FORCE K/FT	HOOP FORCE K/FT	MER MOMENT K/FT/FT	HOOP MOMENT K/FT/FT	RADIAL SHEAR K/FT	INCHES
ALLOWABLE		-3600	-3600	+400 -2000	+400 -2000	±30	±30	±30	±30	--	--	--	--	--	--
DOME	1	-845	-830	-380	-380	-1.8	-1.7	8.5	8.6	-115	-114	211	212	-1	0.13
	2	-1370	-1364	-500	-510	-5.4	-4.1	1.3	7.4	-257	-216	377	355	8	0.14
	3	-1510	-905	-550	-420	-5.3	-1.0	7.3	10.0	-242	-113	384	248	-1	0.22
WALL	4	-1400	-1012	-510	-430	-5.8	-2.3	0.7	7.7	-255	-157	469	313	2	0.21
	5	-1620	-1256	-570	-490	-6.3	-3.6	6.7	8.4	-289	-189	535	397	-3	0.17
	6	-1150	-1620	-430	-540	-5.2	-7.9	-0.4	-0.5	-311	-451	410	546	25	0.11
	7	-404	-504	-90	170	-1.8	-2.0	-2.4	1.4	-322	-71	47	425	138	0.13
BASE SLAB	8	-251	-206	-180	-60	4.8	-2.2	-1.7	5.7	-124	-32	-556	262	-191	0.12
	9	-1240	-690	-470	-300	-7.8	-3.5	9.7	5.7	-267	-118	2054	1157	-140	0.40
	10	-870	-446	-310	-210	-6.3	-3.0	6.0	3.7	-241	-97	1560	796	15	0.42
REACTOR CAVITY	11	-38	(*)	20	130	-0.5	-0.5	-0.7	4.9	-59	57	-64	-57	66	0.50
	12	-3	(*)	30	50	3.0	3.0	10.0	9.9	57	60	54	42	10	0.52

Table 3.8-2
STRESS ANALYSIS RESULTS (Sheet 6 of 10)

DEAD LOAD + PRESTRESS LOAD + ABNORMAL THERMAL LOAD + 150% LOCA/MSLB PRESSURE LOAD
$(D + F + T_A + 1.5P)$

PORTION	SECTION	CONCRETE STRESS		LINER STRAINS		REINFORCING STRESS				SECTION RESULTANTS					DEFLECTION
						INSIDE		OUTSIDE							
		MER PSI	HOOP PSI	MER $\times 10^{-6}$ IN./IN.	HOOP $\times 10^{-6}$ IN./IN.	MER KSI	HOOP KSI	MER KSI	HOOP KSI	MER FORCE K/FT	HOOP FORCE K/FT	MER MOMENT K/FT/FT	HOOP MOMENT K/FT/FT	RADIAL SHEAR K/FT	INCHES
ALLOWABLE		-5400	-5400	±5000	±5000	±54	±54	±54	±54	--	--	--	--	--	--
DOME	1	(*)	(*)	290	330	11.4	11.8	18.2	17.5	55	59	-16	-26	0	0.48
	2	-656	-511	-310	-300	-2.9	0.6	0.9	12.5	-100	-60	192	150	9	0.43
	3	-660	(*)	-250	90	-1.8	11.5	7.6	22.0	-85	75	161	46	-2	0.62
WALL	4	-758	(*)	-200	290	-3.5	16.7	0.8	26.1	-98	93	244	-45	-6	0.81
	5	-1100	(*)	-320	300	-3.7	25.0	14.6	36.0	-132	146	365	-55	-3	1.08
	6	-453	-880	-200	-380	-1.7	-4.3	-3.5	2.8	-154	-166	-46	306	47	0.22
	7	-800	-296	370	30	4.8	-1.1	-5.6	2.8	-182	32	-826	82	156	0.17
BASE SLAB	8	-465	-124	590	50	16.3	-1.7	-2.4	7.4	-113	22	-1404	120	-302	-0.01
	9	-1410	-841	-540	-340	-8.6	-4.1	11.9	7.2	-272	-127	2346	1399	-169	0.48
	10	-1050	-531	-370	-240	-7.4	-3.3	9.2	4.5	-248	-104	1911	931	36	0.63
REACTOR CAVITY	11	-203	(*)	20	140	-0.8	4.2	-1.2	5.3	-117	63	-93	-66	76	0.65
	12	(*)	(*)	80	90	3.6	3.9	10.8	10.8	73	75	23	16	6	0.68

Table 3.8-2
STRESS ANALYSIS RESULTS (Sheet 7 of 10)

DEAD LOAD + PRESTRESS LOAD + ABNORMAL THERMAL LOAD
+ 125% LOCA/MSLB PRESSURE LOAD + 125% OPERATING BASIS EARTHQUAKE
$(D + F + T_A + 1.25P + 1.25E)$

PORTION	SECTION	CONCRETE STRESS MER PSI	CONCRETE STRESS HOOP PSI	LINER STRAINS MER $\times 10^{-6}$ IN./IN.	LINER STRAINS HOOP $\times 10^{-6}$ IN./IN.	REINFORCING STRESS INSIDE MER KSI	REINFORCING STRESS INSIDE HOOP KSI	REINFORCING STRESS OUTSIDE MER KSI	REINFORCING STRESS OUTSIDE HOOP KSI	SECTION RESULTANTS MER FORCE K/FT	SECTION RESULTANTS HOOP FORCE K/FT	SECTION RESULTANTS MER MOMENT K/FT/FT	SECTION RESULTANTS HOOP MOMENT K/FT/FT	SECTION RESULTANTS RADIAL SHEAR K/FT	DEFLECTION INCHES
ALLOWABLE		-5400	-5400	±5000	±5000	±54	±54	±54	±54	--	--	--	--	--	--
DOME	1	-260	-230	-230	-230	1.3	1.5	11.1	10.9	-40 -14	-12 -38	86 94	84 90	-2	0.02
	2	-1120	-1090	-430	-440	-4.5	-2.6	1.6	1.3	-206 -148	-106 -194	302 300	261 259	7	0.26
	3	-1270	-500	-480	-310	-4.1	3.0	9.9	14.0	-231 -95	56 -104	298 262	143 133	-1	0.33
WALL	4	-1160	-440	-430		-5.3	2.6	1.4	12.6	-231 -121	45 -111	372 354	127 117	-1	0.34
	5	-1720	-410	-580	-300	-7.2	3.0	10.6	14.9	-405 -17	-1 -67	480 422	161 145	-4	0.37
	6	-1940	-1910	-390	-400	-4.0	-8.9	5.0	3.9	-513 +47	-578 -70	-67 553	442 572	37	0.14
	7	-1520	-790	30	-90	2.1	-2.3	2.8	4.1	-525 -23	-240 180	-1269 559	79 457	151	0.14
BASE SLAB	8	-600	-340	420	-130	11.1	-3.0	-3.3	6.8	-137 -79	-227 -151	-501 -1317	472 -10	-244	0.07
	9	-1620	-1070	-560	-390	-9.7	-4.5	11.8	7.6	-297 -207	-166 -64	2714 1548	1783 767	-149	0.44
	10	-1250	-600	-540	-240	-8.1	-3.7	9.5	4.5	-204 -270	-115 -67	2249 1237	1037 685	28	0.56
REACTOR CAVITY	11	-260	(*)	40	120	-1.1	4.3	-1.1	5.3	-67 -119	86 36	-137 -29	-75 -49	71	0.58
	12	(*)	(*)	50	65	3.2	3.5	10.6	10.6	97 29	115 15	77 5	65 -1	9	0.61

Table 3.8-2
STRESS ANALYSIS RESULTS (Sheet 8 of 10)

DEAD LOAD + PRESTRESS LOAD + ABNORMAL THERMAL LOAD
+ 125% OPERATING BASIS EARTHQUAKE
$(D + F + T_A + 1.25E)$

PORTION	SECTION	CONCRETE STRESS		LINER STRAINS		REINFORCING STRESS				SECTION RESULTANTS					DEFLECTION
						INSIDE		OUTSIDE							
		MER PSI	HOOP PSI	MER $\times 10^{-6}$ IN./IN.	HOOP $\times 10^{-6}$ IN./IN.	MER KSI	HOOP KSI	MER KSI	HOOP KSI	MER FORCE K/FT	HOOP FORCE K/FT	MER MOMENT K/FT/FT	HOOP MOMENT K/FT/FT	RADIAL SHEAR K/FT	INCHES
ALLOWABLE		-5400	-5400	±5000	±5000	±54	±54	±54	±54	--	--	--	--	--	--
DOME	1	-2075	-2061	-680	-680	-8.0	-8.0	3.1	3.0	-449 -423	-427 -453	533 541	543 549	-2	-0.28
	2	-2378	-2345	-750	-750	-10.1	-9.3	1.0	3.1	-603 -545	-483 -571	599 597	598 596	10	-0.18
	3	-2736	-2354	-840	-750	-11.0	-9.0	4.3	4.2	-627 -491	-400 -560	685 649	603 593	1	0.02
WALL	4	-2350	-2575	-750	-800	-10.0	-11.3	0.5	0.6	-625 -515	-577 -733	776 758	821 811	14	-0.04
	5	-2700	-2899	-820	-820	-11.6	-13.3	4.5	3.2	-798 -410	-838 -904	812 754	885 869	4	-0.12
	6	-3270	-2670	-730	-730	-10.0	-12.1	6.0	0.9	-906 -346	-972 -466	460 1080	566 696	-26	0.03
	7	-3050	-1390	-810	-440	-8.8	-3.2	6.0	2.9	-887 -339	-484 -64	638 2466	423 1101	51	0.11
BASE SLAB	8	-840	-540	-250	-190	-5.6	-3.2	5.5	3.9	-229 -171	-143 -67	1556 840	735 253	23	0.22
	9	-1244	-840	-410	-300	-8.0	-4.9	4.6	3.7	-332 -242	-193 -91	2021 855	1423 409	-61	0.23
	10	-940	-430	-340	-230	-6.5	-3.7	3.5	2.8	-283 -349	-144 -96	1708 696	941 589	-12	0.21
REACTOR CAVITY	11	-50	(*)	50	100	0.5	2.9	0.3	4.1	59 7	67 17	-82 26	-48 -22	39	0.23
	12	190	-200	-120	-100	-0.4	0.4	9.1	8.8	77 9	108 8	120 48	71 5	23	0.25

Table 3.8-2
STRESS ANALYSIS RESULTS (Sheet 9 of 10)

DEAD LOAD + PRESTRESS LOAD + ABNORMAL THERMAL LOAD
+ LOCA/MSLB PRESSURE LOAD + DESIGN BASIS EARTHQUAKE LOAD
$(D + F + T_A + P + E')$

PORTION	SECTION	CONCRETE STRESS		LINER STRAINS		REINFORCING STRESS				SECTION RESULTANTS					DEFLECTION
						INSIDE		OUTSIDE							
		MER PSI	HOOP PSI	MER $\times 10^{-6}$ IN./IN.	HOOP $\times 10^{-6}$ IN./IN.	MER KSI	HOOP KSI	MER KSI	HOOP KSI	MER FORCE K/FT	HOOP FORCE K/FT	MER MOMENT K/FT/FT	HOOP MOMENT K/FT/FT	RADIAL SHEAR K/FT	INCHES
ALLOWABLE		-5400	-5400	±5000	±5000	±54	±54	±54	±54	--	--	--	--	--	--
DOME	1	-896	-869	-390	-390	-1.9	-1.9	8.8	8.8	-131 -99	-99 -129	204 218	207 217	-1	0.13
	2	-1446	-1476	-510	-530	-5.8	-4.8	1.7	8.1	-295 -219	-159 -273	379 375	357 353	8	0.14
	3	-1739	-1116	-600	-450	-6.6	0.2	8.0	11.3	-329 -155	-8 -218	407 361	255 241	-1	0.22
WALL	4	-1539	-1194	-540	-460	-6.6	-3.3	1.3	8.8	-327 -183	-55 -259	480 458	320 306	2	0.21
	5	-2110	-1347	-680	-500	-9.3	-4.0	8.9	9.0	-542 -36	-145 -233	573 497	406 388	-3	0.17
	6	-2646	-2345	-500	-620	-5.7	-10.8	8.2	4.0	-678 56	-784 -118	1 819	460 632	25	0.11
	7	-1782	-988	-230	-180	0.8	-2.8	5.8	4.4	-682 48	-346 -204	-1154 1248	177 673	138	0.13
BASE SLAB	8	-525	-407	-240	-100	6.1	-3.3	-2.5	6.2	-160 88	-81 17	-78 -1034	580 -56	-191	0.12
	9	-1666	-1072	-560	-380	-10.1	-5.6	11.2	6.9	-324 -210	-184 -52	2797 1311	1801 513	-140	0.40
	10	-1230	-586	-380	-230	-7.8	-3.8	7.7	4.1	-200 -282	-130 -64	1198 -78	1020 572	15	0.42
REACTOR CAVITY	11	-164	(*)	50	150	0.1	-0.9	-0.9	5.1	-24 -94	92 22	-138 10	-73 -41	66	0.50
	12	-7	(*)	30	-30	2.4	3.3	10.6	10.6	106 8	139 -19	116 -8	91 -7	10	0.52

Table 3.8-2
STRESS ANALYSIS RESULTS (Sheet 10 of 10)

DEAD LOAD + PRESTRESS LOAD + ABNORMAL THERMAL LOAD
+ DESIGN BASIS EARTHQUAKE LOAD
$(D + F + T_A + E')$

PORTION	SECTION	CONCRETE STRESS		LINER STRAINS		REINFORCING STRESS				SECTION RESULTANTS					DEFLECTION
						INSIDE		OUTSIDE							
		MER PSI	HOOP PSI	MER $\times 10^{-6}$ IN./IN.	HOOP $\times 10^{-6}$ IN./IN.	MER KSI	HOOP KSI	MER KSI	HOOP KSI	MER FORCE K/FT	HOOP FORCE K/FT	MER MOMENT K/FT/FT	HOOP MOMENT K/FT/FT	RADIAL SHEAR K/FT	INCHES
ALLOWABLE		-5400	-5400	±5000	±5000	±54	±54	±54	±54	--	--	--	--	--	--
DOME	1	-2091	-2067	-680	-680	-8.0	-8.1	3.2	3.0	-452 -420	-424 -455	530 544	541 551	-2	-0.28
	2	-2396	-2372	-750	-750	-10.2	-9.5	1.1	3.3	-612 -536	-470 -584	600 596	599 595	10	-0.18
	3	-2789	-2405	-850	-760	-11.4	-9.3	4.5	4.5	-646 -472	-375 -585	690 644	605 591	1	0.02
WALL	4	-2379	-2630	-750	-800	-10.2	-11.6	0.6	0.8	-642 -498	-553 -757	778 756	823 809	14	-0.04
	5	-2820	-2921	-850	-820	-12.3	-13.4	5.0	3.4	-857 -351	-827 -915	821 745	886 868	4	-0.12
	6	-3626	-2835	-750	-750	-10.1	-12.8	9.1	2.0	-993 -259	-1052 -386	361 1179	545 717	-26	0.03
	7	-3378	-1604	-840	-440	-9.4	-3.4	8.4	3.6	-973 -253	-549 1	351 2753	663 1159	51	0.11
BASE SLAB	8	-906	-580	-270	-210	-6.0	-3.6	5.7	4.0	-236 -164	-154 -56	1676 720	812 176	23	0.22
	9	-1335	-924	-440	-310	-8.5	-5.4	5.0	4.0	-344 -230	-208 -76	2181 695	1561 273	-61	0.23
	10	-1062	-564	-350	-230	-6.8	-3.8	3.8	2.9	-275 -357	-153 -87	1840 564	989 541	-12	0.21
REACTOR CAVITY	11	170	(*)	50	100	0.7	3.0	0.4	4.2	68 -2	77 7	-102 46	-41 -19	39	0.23
	12	(*)	(*)	-120	-110	-0.1	0.5	9.3	9.0	92 -6	137 -21	146 22	87 -11	23	0.25

In addition, interaction diagrams for each critical section identified in table 3.8-2, sheet 1, together with a graphical plot of the actual axial load with its accompanying moment for each principal load combination are furnished in figure 3.8-19, sheet 1 through 18 inclusive.

The effect of three-dimensional stress/strain fields on the behavior of the structure has been considered in the FINEL computer program described in appendix 3C, subsection 3C.1.

3.8.1.6 Materials, Quality Control, and Special Construction Techniques

The following basic materials are used in the construction of the containment structure.

A. Concrete

Tendon access gallery		f'_c (lb/in.^{2}) = 6,000 at 90 days
Base slab		f'_c (lb/in.^{2}) = 6,000 at 90 days
Cylindrical wall and dome		f'_c (lb/in.^{2}) = 6,000 at 90 days

B. Reinforcing Steel

Deformed bars	ASTM A-615 Grade 60	f_y (lb/in.^{2}) = 60,000

C. Structural and Miscellaneous Steel

Structural steel shapes, plates, and bars	ASTM A-36	f_y (lb/in.^{2}) = 36,000
High-strength structural steel shapes, plates, and bars	ASTM A-572	f_y (lb/in.^{2}) = 42,000 to 65,000 (varies depending on grade of the material)
Pipe used as structural members	ASTM A-53	f_y (lb/in.^{2}) = 25,000 to 35,000 (varies depending on grade used)
Forgings	ASTM A-237 Class C	f_y (lb/in.^{2}) = 58,000 to 60,000 (varies depending on material thickness)

Pins	ASTM A-307	f_t (lb/in.^{2}) = 60,000 minimum
	ASTM A-325	f_y (lb/in.^{2}) = 81,000 to 92,000 (varies depending on diameter of pins)
	ASTM A-449	f_y (lb/in.^{2}) = 58,000 to 92,000 (varies depending on diameter of material)
	ASTM A-490	f_y (lb/in.^{2}) = 130,000 (varies depending on diameter of pins)
	ASTM A-540	f_y (lb/in.^{2}) = 105,000 to 150,000 (varies depending on diameter, class, and grade of material)
Polar crane rail	AISI 175 lb/yd	
Refueling machine rail	Stainless steel	
CEA change mechanism rail	Stainless steel	
Anchor bolts	ASTM A-36	f_y (lb/in.^{2}) = 36,000 minimum
	ASTM A-307	f_t (lb/in.^{2}) = 60,000 minimum
	ASTM A-325	f_y (lb/in.^{2}) = 81,000 to 92,000 (varies depending on diameter of bolts)
	ASTM A-449	f_y (lb/in.^{2}) = 58,000 to 42,000 (varies depending on diameter of bolts)
	ASTM A-490	f_y (lb/in.^{2}) = 130,000 minimum (varies depending on diameter of bolts)

	ASTM A-540	f_y (lb/in.^{2}) = 105,000 to 150,000 (varies depending on diameter, class, and grade of material)
Bolts	ASTM A-307	f_t (lb/in.^{2}) = 60,000 minimum
High strength bolts	ASTM A-325	f_y (lb/in.^{2}) = 81,000 to 92,000 (varies depending on diameter of bolts)
	ASTM A-490	f_y (lb/in.^{2}) = 130,000 minimum (varies depending on diameter of bolts)
	ASTM A-540	f_y (lb/in.^{2}) = 105,000 to 150,000 (varies depending on diameter, class, and grade of material)
Nelson shear studs	ASTM A-108	f_y (lb/in.^{2}) = f_y = 50,000

D. Containment Steel Liner Plate and Penetration Sleeves

1/4-in. Liner Plate	ASME-SA-285 Grade A	f_y (lb/in.^{2}) = 24,000
Greater than 1/4-in. Liner Plate	ASME-SA-516 Grade 70	f_y (lb/in.^{2}) = 38,000
Embedded Items	ASME-SA-36	f_y (lb/in.^{2}) = 36,000
	ASME-SA-285 Grade A	f_y (lb/in.^{2}) = 24,000
	ASME-SA-516 Grade 70	f_y (lb/in.^{2}) = 38,000
	ASME-SA-106 Grade B	f_y (lb/in.^{2}) = 35,000
Cadweld Connectors	AISI C1026	f_y (lb/in.^{2}) = 72,000
	AISI C1018	f_y (lb/in.^{2}) = 72,000

Nelson Shear Studs	ASTM A-108	f_y (lb/in.^{2}) = 50,000
Penetration Sleeves		
Seamless pipes	ASME-SA-333	
	Grade 1	f_y (lb/in.^{2}) = 30,000
	Grade 6	f_y (lb/in.^{2}) = 35,000
Welded pipes	ASME-SA-516 Grade 70	f_y (lb/in.^{2}) = 38,000
	ASME-SA-155 Grade-KCF-70 Class 1	f_y (lb/in.^{2}) = 38,000
E. Post-Tensioning System		
Prestressing strands	ASTM A-416-74 Grade 270	f'_s (lb/in.^{2}) = 270,000
Bearing plates	ASTM A-537-67a Grade A	f_y (lb/in.^{2}) = 45,000
Sheathing	ASTM A-527	
Anchor heads	AISI 1026	

Materials and their quality control requirements are described in the following paragraphs.

3.8.1.6.1 Reinforced Concrete

3.8.1.6.1.1 Concrete. All concrete work is done in accordance with ACI 318-71, Building Code Requirements for Reinforced Concrete, and ACI-301-66, Specifications for Structural Concrete for Buildings, except as otherwise stated herein.

The concrete is a dense, durable mixture of sound coarse aggregates, fine aggregates, cement, and water. In all areas, pozzolan is substituted for portions of cement used in the concrete. Admixtures are added to improve the quality and workability of the plastic concrete during placement and, in some areas to retard the set of concrete. The sizes of aggregates, water-reducing additives, and slumps are selected to maintain low limits on shrinkage and creep.

3.8.1.6.1.2 Cement. Cement is Type II, low alkali, moderate heat of hydration conforming to the Specification for Portland Cement (ASTM 150-70) including Table 1A for moderate heat of hydration. Certified copies of mill test reports showing the chemical composition and physical properties are obtained for each load of cement delivered. The limitation of the alkali content of the cement may be waived provided that the aggregates pass required laboratory tests and have no history of alkali-aggregate incompatibility.

In addition to the tests required by the cement manufacturers, the following tests are performed:

ASTM C109 - Compressive Strength

ASTM C114 - Chemical Analysis

ASTM C115 or C204 - Fineness of Portland Cement

ASTM C151 - Autoclave Expansion

ASTM C191 or C266 - Time of Set

The purpose of the above tests is to ascertain conformance with ASTM Specification C 150. In addition, tests ASTM C 191 or ASTM C266, ASTM C 109, and ASTM C 451 are repeated periodically during construction to check storage environmental effects on cement characteristics. The tests supplement visual inspection of material storage procedures.

3.8.1.6.1.3 Aggregates. All aggregates conform to the Standard Specifications for Concrete Aggregate (ASTM C 33-69). In addition to the specified gradation, the fine aggregate (sand) has a fineness modulus of not less than 2.3 or more than 3.1 during normal operations; at least 9 of 10 test samples should not vary in fineness modulus more than 0.20 from the average. Coarse aggregate may be rejected, if the loss when subjected to the Los Angeles abrasion test, ASTM C131-69 using grading A, exceeds 40% by weight at 500 revolutions.

Acceptance of aggregates is based on the following tests:

ASTM Test No.	Name of Test
C 131	Los Angeles Abrasion
C 142	Clay Lumps and Friable Particles
C 117	Material Finer than No. 200 Sieve
C 87	Mortar Making Properties
C 40	Organic Impurities

ASTM Test No.	Name of Test
C289	Potential Reactivity (Chemical)
C 136	Sieve Analysis
C 88	Soundness
C 127	Specific Gravity and Absorption
C 128	Specific Gravity and Absorption
C 295	Petrographic

In addition to the foregoing initial tests, a daily inspection control program will be carried on during construction to ascertain consistency in potentially variable characteristics such as gradation and organic content.

3.8.1.6.1.4 Water. Water and ice used in mixing concrete is free of injurious amounts of oil, acid, alkali, organic matter, or other deleterious substances as determined by American Association of State Highway Officials (AASHO) Methods of Sampling and Testing, Designation T26. Water shall not contain impurities in amounts that will cause either a change in the time of setting of Portland cement of more than 25% or, a reduction in the compressive strength of mortar of more than 5% compared with results obtained with distilled water. The water shall not contain more than 250 ppm of chlorides as Cl, nor more than 800 ppm of sulfates as SO_4, nor more than 2000 ppm total dissolved solids. The pH range shall be within 6.0 to 8.5.

3.8.1.6.1.5 Admixtures. The concrete also contains an air entraining admixture, a water reducing admixture, and a pozzolan. The air entraining admixture is in accordance with the Specification for Air Entraining Admixtures for Concrete (ASTM C 260). It is capable of entraining 3 to 6% air, is completely water soluble, and is completely dissolved when it enters the batch. The water reducing admixture may be a type that retards the set of the concrete and that conforms to the Standard Specification for Chemical Admixtures for Concrete (ASTM C 494-68), Types A and D.

Pozzolans conform to Specifications for Fly Ash and Raw or Calcined Natural Pozzolans for Use in Portland Cement Concrete (ASTM C 618) except that ignition loss shall not exceed 6%.

3.8.1.6.1.6 Concrete Mix Design. Concrete mixes are designed in accordance with the American Concrete Institute Standard (ACI) 211.1.70. Only concrete mixes meeting the design requirements specified for the structures are used.

Trial mixes are tested in accordance with the applicable ASTM specifications as indicated below:

ASTM	Test
C-39-66	Compressive strength of molded concrete cylinders
C-143-69	Slump of Portland cement concrete
C-192-69	Making and curing concrete test specimens, in the laboratory
C-231-68	Air content of freshly mixed concrete by the pressure method
C-232-58	Bleeding of concrete

Concrete test cylinders are cast from the mix proportions selected for use in the prestressed concrete for the containment, to determine the following properties:

A. Compressive strength

B. Thermal diffusivity

C. Autogenous shrinkage

D. Thermal coefficient of expansion

E. Modulus of elasticity and Poisson's ratio

F. Uniaxial creep

3.8.1.6.1.7 Concrete Testing. During construction, concrete is sampled and tested for slump, air content, temperature, and unit weight prior to casting compressive strength cylinders.

Compressive strength cylinders are cast from representative samples taken in accordance with Sampling Fresh Concrete (ASTM C 172-68), Paragraph 3.

Cylinders are made, cured, and tested in accordance with the Standard Method for Making and Curing Compression and Flexure Tests in the Field (ASTM C 31-69) and the Standard Test for Compressive Strength of Molded Concrete Cylinders (ASTM C 39-66).

The requirements for taking cylinders are as follows:

A. One set of test specimens is made not less than once a day or less than once for each 100 cubic yards of concrete placed, or fraction thereof, for each mix design. For large concrete placements exceeding 500 cubic yards, one set of test specimens is made not less than once for each 250 cubic yards of concrete placed, or fraction thereof, for each design mix.

B. The procedures for securing strength test samples and molding test specimens comply with the above mentioned standards. A set of test specimens consists of six 6-inch diameter by 12-inch cylinders.

C. Two cylinders from each set of test specimens are tested at the designated test intervals. Tests are performed at 3, 7, 28, and 90 days. The 3-day test is only made on occasion to correlate 3-day strength. When these tests are made, the 90-day tests are not made.

D. When a correlation of test data is established for each mix, the 90-day test cylinders are discontinued except for prestressed concrete.

Concrete cylinders are maintained at a temperature of 60F to 80F prior to stripping, stripped within 24 hours after casting, marked and stored in the curing room until the designated date for testing.

Standards of concrete control are in accordance with the criteria established in ACI 214 for "excellent" concrete. An acceptable correlation is based upon test results from at least the first 10,000 cubic yards of concrete placed for each mix design.

The average of all of the compressive strength tests representing each class of concrete, as well as the average of any five consecutive strength tests representing each class of concrete, is required to be equal to or greater than the specified strength, and no more than one strength test in ten has an average value less than the specified strength. The strength of an individual test is the average of the strengths of the two specimens.

3.8.1.6.1.8 Concrete Placement.

A. General

Conveying and placing of concrete is performed in accordance with ACI 301, ACI 318, ACI 304, ACI SP2, ASTM C-94 and as specified herein. No aluminum pipe or other conveying equipment containing aluminum that would be in contact with the fresh concrete is used for conveying concrete to point of placement. Steel pipe is used for concrete pumps or pneumatic placers. Pipe sizes are limited to 5 inches minimum diameter for 3/4- to 1-1/2-inch maximum size aggregate mixes and may be reduced to smaller pipe sizes for aggregate mix size less than 3/4-inch maximum.

B. Clean-Up Preparation

Before depositing concrete, all equipment is cleaned. Debris is removed from spaces to receive concrete. Reinforcement and other metal to be embedded is thoroughly cleaned of all loose rust, scale, and/or coatings that might impair the bond. All compacted soil, rock, or concrete surfaces to receive concrete are thoroughly wetted before placement.

C. Construction Joint Placement

To the maximum extent possible, concrete is deposited continuously to provide monolithic units in the construction as shown on the approved engineering design drawings. Construction joints are provided in accordance with details as shown on the approved engineering design drawings where the size of large slabs or lengths of continuous strips so dictate. Adjacent vertical placements have a minimum curing time of 3 days. In all cases, concrete is deposited in such a way as to prevent water from collecting at the ends and corners of forms and along form faces during placement.

All contiguous vertical concrete construction joints to receive additional lifts or concrete are moist cured. Moist curing of newly placed concrete is kept wet by continuous application of water for the first 7 days after the concrete has been placed. As soon as unformed surfaces of concrete have hardened sufficiently to prevent surface damage through application of curing procedures, an intermittent fine spray of water is applied as necessary to keep such surfaces continually moist for not less than 7 days.

D. Placement Limitations

Concrete is deposited in horizontal layers between 12 to 24 inches, and is not allowed to flow a distance of more than 5 feet from point of deposition.

E. Segregation

Concrete is not dropped through dense reinforcing steel, which might cause segregation of the coarse aggregate. Concrete is not dropped free from a height of more than 6 feet.

F. Concrete Temperature Control

The target temperature of concrete is as close to 50F as possible and not to exceed 55F for placements which exceed 6 feet in thickness; i.e., the least dimension in any direction.

The target temperature for all other placements is less than 70F for the following conditions:

1. Placement of 6 feet, or less, in thickness.

2. Consecutive placements that do not exceed a total of 6 feet in thickness.

3. Consecutive placements which exceed a total of 6 feet in thickness where the elapsed time between placements is 14 days or more.

For consecutive placements that exceed a total of 6 feet in thickness, and for which the elapsed time between placements is less than 14 days, the target temperature for the contiguous placement shall be as close to 50F as possible and not to exceed 55F.

G. Weather Precautions

During cold weather, if the air temperature drops below freezing at night, or if the mean daily temperature falls below 40F for more than one day during the period when concrete is being placed, concreting is placed in accordance with the Recommended Practice for Cold Weather Concreting, ACI 306. The concrete shall be maintained at a temperature no lower than 50F for at least 72 hours after it is placed. No additional protection from freezing will be required if that temperature is maintained for that length of time by means of insulation in contact with the form or concrete surfaces. Foundation forms can be stripped 24 hours after concrete is placed.

Concrete, when deposited in the forms during cold weather, is required to have a temperature of not less than the following:

Air Temperature (^oF)	Less than 2-1/2 feet in Least Dimension (^oF)	Mass Concrete In excess of 2-1/2 feet Least Dimension (^oF)
30 to 45	60	50
0 to 30	65	55

During hot weather, when the ambient temperature is greater than 80F, concrete is placed in accordance with ACI 305, Recommended Practice for Hot Weather Concreting.

Before depositing concrete in any form or on any surface, cool water is sprinkled on all surfaces and reinforcement steel. Wind breakers are used to prevent wind from blowing over the concrete surface prior to the initiation of curing.

Curing is started as soon as the concrete has hardened to withstand surface damage.

H. Consolidation of Concrete

Concrete is placed with the aid of mechanical vibrating equipment and supplemented by hand spading and tamping. The vibrating equipment is of the internal type. The frequency of vibration is not less than 7000 cycles per minute. Vibration is not allowed to cause segregation. In consolidating each layer of concrete, the vibrator is operated in a near vertical position. The vibrating head is allowed to penetrate under the action of its own weight and to revibrate the concrete in the upper portion of the underlying layer. Neither form nor surface vibrators are used without specific approval from the Bechtel Project Engineer.

Vibrators are not used to move or spread concrete. A ratio of not less than one spare vibrator in good working condition to each three vibrators required for vibration of the concrete being placed is kept available for immediate use at the placement location. Vibration commences within 15 minutes following time of placement.

I. Bonding of Concrete Between Lifts

Horizontal construction joints are prepared for receiving the next lift by either sandblasting or water blasting. Sandblasting or water blasting will be performed before placing forms. The operation shall be continued until all laitance, coatings, stains, and other foreign materials are removed. The surface of the concrete is washed thoroughly to remove all loose materials. The horizontal surface is wet immediately before the concrete is placed.

Surface set retardant compounds are not used.

3.8.1.6.2 Reinforcing Steel

Reinforcing steel is deformed billet steel, conforming to ASTM Designation A-615-68. This steel has a minimum yield strength of 60,000 $lb/in.^2$, a minimum tensile strength of 90,000 $lb/in.^2$, and a minimum elongation of 7% in an 8-inch specimen. Grade 60 is used throughout the project.

Mill test results are obtained from the reinforcing steel supplier for each heat of steel to show proof that the reinforcing steel has the specified composition, strength, and ductility. Splices in reinforcing bar sizes No. 11 and smaller are lapped in accordance with ACI 318-71 and, for bars larger than No. 11, Cadweld splices are made in strict accordance with the manufacturer's instructions as presented in Erico Products Bulletin, RB 10M-670, 1970, Cadweld Rebar Splicing.

3.8.1.6.3 Post-Tensioning System

The prestressed, post-tensioning system selected for the containment is the VSL Post-Tensioning System using ES-55 tendons. Each hoop tendon wraps approximately 240° around the containment. Vertical tendons are placed in layers, consisting of alternates of two tendons and one tendon spaced equally along the containment wall.

3.8.1.6.3.1 Tendon Strands. The strands are of the seven-wire, low-relaxation type having a guaranteed minimum ultimate tensile strength (f'_s) of 270,000 $lb/in.^2$ based on the nominal steel area of strand and the minimum yield strength is required to be not less than 0.90 f'_s. All strands conform to the Standard Specifications for Uncoated Seven-Wire Stress Relieved Strand for Prestressed Concrete, ASTM A 416, Weldless Grade.

Elongation at ultimate failure is not less than that given in ASTM A 416.

3.8.1.6.3.2 Anchorages. The basic performance requirements for the end anchors of the tendons are stated qualitatively by the Seismic Committee of the Prestressed Concrete Institute and published in their Journal of June 1966, as follows:

> "All anchors of unbonded tendons should develop at least 100 percent of the guaranteed ultimate strength of the tendons. The anchorage gripping should function in such a way that no harmful notching would occur on the tendons. Any such anchorage system used in earthquake areas must be capable of maintaining the prestressing force under sustained and fluctuating loads and the effect of shock. Anchors should also possess adequate reserve strength to withstand any overstress to which they may be subjected during the most severe probable earthquake. Particular care should be directed to accurate positioning and alignment of end anchors."

The end anchors used are capable of developing 100% of the minimum tensile strength of the tendons. Furthermore, the end anchors are capable of maintaining integrity for 500 cycles of loads corresponding to an average axial stress variation between 0.7 and 0.75 f'_s, at a repetition rate of one cycle of 0.1 second. This requirement sets the minimum acceptable limits on fatigue effects due to notching by the end anchor, and tendon performance in response to earthquake loads. To provide an adequate factor of safety, the number of cycles has been increased to 500 from the 100 generated from earthquake, wind, and accident loadings. The stress variation has been increased from a conservatively predicted 0.6 f'_s to 0.64 f'_s and the 0.7 f'_s to 0.75 f'_s. Further, the number of cycles caused by earthquake loads is predicted as only 30 of the total of 100, by using all those strong ground motions that exceed one-half of the peak ground motion for the earthquake.

The anchorage assemblies, including the bearing plates, are capable of transmitting the ultimate loads of the tendons into the structure without brittle fracture at an anticipated lowest service temperature of 20F.

3.8.1.6.3.3 Tendon Sheathing and Trumpet Extensions. Trumpet extensions for all tendons are fabricated from rigid tubing. Extensions for the vertical-dome tendons, except in the area of the equipment hatch, extend from the trumpet to a point 1 foot above the top of the basemat; extensions for the vertical dome tendons in the area of the equipment hatch extend to the point of tangency of the vertical transition curve. Extensions for the horizontal-hoop tendons extend from the trumpet to the edge of the buttress. The trumpet extension supplied has a rigid coupler to the trumpet capable of maintaining the required alignment. Material conforms to the Standard Specification for Electric-Resistance-Welded Carbon and Alloy Steel Mechanical Tubing, ASTM A513, and has a wall thickness not less than 0.065 inch.

Sheaths for post-tensioning tendons are galvanized spiral-wrapped, semi-rigid, corrugated tubing and material conforms to the Standard Specification for Steel Sheet, Zinc-coated (Galvanized) by the Hot-Dip Process, Lock-Forming Quality, ASTM A 527, 24-gage cold-rolled carbon steel.

Vertical sheaths are installed in 10-foot lengths.

The hoop sheaths have a field splice at each trumpet extension and at six points in between. This results in hoop sheath lengths of approximately 45-foot lengths.

The dome radial sheaths are spliced at approximately the spring line and at such intermediate points as expedient. The dome hoop sheaths are installed in approximately 45-foot lengths.

Coupling devices are provided at all field splices inherent in the erection. The coupler provides for a field splice that is easily and quickly sealed against leakage and that maintains the alignment of the parent sheath.

The coupler is detailed such that a segment of sheathing may be spliced between two other segments which may be rigidly fixed in concrete.

Field welding of sheathing to form a splice is not permitted.

Protective devices are provided to prevent damage to the ends of the sheaths and couplings during handling operations and to prevent damage or entry of sand, rain, etc., during storage and construction.

3.8.1.6.3.4 Corrosion Protection. Suitable atmospheric corrosion protection is maintained for the tendons from the point of manufacture to the installed locations. The atmospheric corrosion protection provides assurance that the tendon integrity is not impaired due to exposure to the environment.

Prior to shipment, a thin film of rust inhibitor, Visconorust 1601 Amber, manufactured by the Viscosity Oil Company, is applied to the tendons in accordance with the manufacturer's instructions. After the tendons are installed and stressed, the interior of the sheathing is pumped full of a modified, thixotropic, refined petroleum-based product, Visconorust 2090P-4, to provide corrosion protection. The tendon end anchors are also encapsulated by gasketed end caps that are filled with the corrosion protection material and sealed against the bearing plates.

Testing of the permanent corrosion-protection material confirms that there are no significant amounts of chlorides, sulfides, or nitrates present. However, to further verify the chemical composition of the filler material, test samples are obtained from each shipment and analyzed as follows:

A. Water-soluble chlorides (Cl) are determined in accordance with ASTM D512-67 with a limit of accuracy of 0.5 ppm.

B. Water-soluble nitrates (NO_3) are determined by the Water and Sewage Analysis Procedure of the Hach Chemical Company, Ames, Iowa, or by ASTM D-992 Brucine Method with a limit of accuracy of 0.5 ppm.

C. Water-soluble sulfides (S) are determined in accordance with American Public Health Association (APHA) standards with a limit of accuracy of 1 ppm. The APHA Standard methods (Methylene Blue procedure) or the Hach Chemical Company method are used.

Acceptance criteria of the corrosion-protection materials are as follows:

- Chlorides - 5 ppm maximum
- Nitrates - 5 ppm maximum
- Sulfides - 5 ppm maximum

3.8.1.6.3.5 Prestressing Sequences. The criteria of the prestressing sequence are based on the design requirements to limit the membrane tension in concrete to $1.0\sqrt{f'_c}$ and to minimize unbalanced loads or differential stresses in the structure. Prestressing begins after the concrete in the wall and the dome has reached the specified f'_c (6000 lb/in.^{2} at 90 days). The construction opening will be closed prior to prestressing. All tendons are tensioned from both ends.

The procedure for prestressing is carefully worked out with the vendor so that all the tendons proceed in such a manner that the containment structure will not be eccentrically loaded at any phase.

3.8.1.6.4 Containment Liner

The containment structure is lined with 1/4 in. thick welded steel plate, except in limited areas where thickened plate is utilized, conforming to the requirements of ASME-SA-285, Low and Intermediate Tensile Strength Carbon Steel Plates for Pressure Vessels, Grade A, to ensure a leaktight membrane. This steel has a minimum yield strength of 24,000 $lb/in.^2$ and a minimum elongations in an 8-inch specimen of 27%. The ASME-SA-285, material was chosen because it has sufficient strength, low yield point, and ductility to resist the expected stresses from design criteria loading, limit forces due to thermal differentials and, at the same time, to preserve all required leaktightness of the containment. It is readily weldable by all of the commercially available arc and gas welding processes. All thickened steel plate conforms to the requirements of ASME-SA-516, Grade 70.

Design of the liner plate is subject to the provisions of Reference 4. Construction, inspection, and testing of the liner plate were performed using the applicable sections of the ASME Boiler and Pressure Vessel Code Section III, Division 1 as a guide only.

The equipment hatch and personnel and escape locks must resist the full design pressure and are designed in accordance with the ASME Boiler and Pressure Vessel Code, Section III, Subsection NE, Class MC Components.

The interior projections of all penetration assemblies must resist the full design pressure. The design of all penetration assemblies is controlled by the provisions of Reference 4.

The liner plate is designed to function only as a leaktight membrane. It is not designed to resist the tension stresses from internally applied pressure, which may result from any credible accident conditions. Structural integrity of the containment is maintained by the prestressed, post-tensioned concrete. Since the principal applied stress to the liner plate membrane is in compression from shrinkage and creep of the concrete, there is no need to apply special nil ductility transition temperature criteria to the liner plate material. On the other hand, all material for containment parts that resists applied internal pressure stresses, such as penetrations, is impact tested in accordance with the requirements of Article NE-2320 of Section III, Division I of the ASME Boiler and Pressure Vessel Code.

All welding procedures and welding operators are qualified by tests, as specified in Section IX of the ASME Boiler and Pressure Vessel Code. This code requires testing of welded transverse root and face bend samples in order to verify adequate weld metal ductility. Specifically, Section IX of the Code requires that the transverse root and face bend samples be capable of being bent cold 180° to an inside radius equal to twice the thickness of the test sample. Satisfactory completion of these bend tests is accepted as adequate evidence of required weld metal and plate material compatibility.

3.8.1.6.5 Containment Liner Plate Attachments and Associated Hardware

Material for penetration sleeves conforms to the requirements of the three specifications listed below. The lowest service metal temperature is 20F and the maximum test temperature is 0F.

A. Penetration Sleeves - Seamless and Welded Steel Pipe for Low-Temperature Service, ASME SA-333, Grade 1 or 6, ASME SA-155, Grade KCF 70, and ASME SA-516, Grade 70.

B. Penetration Sleeve Reinforcing - ASME SA-516, Grade 70

C. Anchor Rings and Plates - ASME SA-516, Grade 70

Material for bolts, nuts, studs, Cadwelds, and Unistruts conforms to the requirements of the five specifications listed below.

A. Anchor and machine bolts - ASME SA-307, Low-Carbon Steel Externally and Internally Threaded Standard Fasteners

B. Nelson studs - ASTM A 108, Cold-Finished Carbon Steel Bars and Shafting

C. Cadweld connectors - AISI C 1026 or AISI C 1018

D. Unistruts - Flat-Rolled Carbon Steel Sheets of Structural Quality, ASTM A 570-72, Grade E or C.

E. High strength bolts - ASME SA-325, High Strength Bolts for Structural Steel Joints Including Suitable Nuts and Plane Hardened Washers.

3.8.1.6.6 Structural and Miscellaneous Steel

Mill test reports of all structural and miscellaneous steel are obtained for all materials used with the exceptions of hand rails, toe plates, kick plates, stairs and ladders.

Detailing, fabrication, and erection of the structural and miscellaneous steel are in accordance with the AISC Manual of Steel Construction, 1969 edition.

Welding is done in accordance with AWS D 1.1-72, Structural Welding Code.

3.8.1.6.7 Quality Control

Quality control procedures are established and implemented during construction and inspection. The quality control procedures are specified in the technical specifications covering the fabrication, furnishing, and installation of each structural component and provide inspection and documentation to assure that the codes and construction practices are met. Table 3.8-3 provides a listing of all pertinent concrete related tests.

3.8.1.6.7.1 Control Tests for Concrete. Concrete for the containment structure is tested in accordance with ACI 214, Recommended Practice for Evaluation of Compressive Test Results of Field Concrete.

3.8.1.6.7.2 Control Tests for Reinforcing Steel. Full diameter specimens of each size of reinforcing steel are taken for tests. Frequency of tests conform to NRC Regulatory Guide 1.15.

Placement tolerances used for reinforcing steel conforms to the following allowable variances:

A. Concrete cover, No. 3 through No. 11: ±1/2 inch

B. Concrete cover, No. 14 and No. 18: + inch; 1/2 inch

C. Spacing between bars, No. 3 through No. 11: ± inch

D. Spacing between bars, No. 14 and No. 18: ±3 inches

E. Lengthwise of bars: ±2 inches

F. Allowable movement of bars for other embedments were maintained to the following:

   1. Bottoms of beams and elevated slabs: ± 2 inches (horizontally)

   2. All other walls, slabs on grade, columns, etc.: ±4 inches

G. Stirrups, hairpins, and ties were uniformly sloped up to 1 in 4, with the requirement that the spacing on one place be maintained within ±1 1/2 inches of the specified location.

3.8.1.6.7.3 Control of Mechanical Splices of Reinforcing. Control of mechanical splices utilizing filler metal and an enclosing sleeve (Cadweld-type splices) are in accordance with NRC Regulatory Guide 1.10.

3.8.1.6.7.4 Quality Control Procedures for the Liner Plate. The nondestructive examination of the liner plate system meets or exceeds all of the requirements of Guide 1.19. Spot examination by each welder at each position is required at each 25 feet and a minimum of 4% of the total feet of weld is required to be examined.

Table 3.8-3
CONTROL TESTS FOR CONCRETE (Sheet 1 of 3)

Onsite Testing			
Material	Test	Procedure	Frequency
Cement	Time of Setting False Set Compressive Strength	ASTM C-191 ASTM C-451 ASTM C-109	Monthly Monthly Monthly
Coarse aggregate	Gradation Flat and Elongated Particle Clay Lumps	ASTM C-136 CRD C-119 ASTM C-142	1 test per shift 1 test per week If required
Fine aggregate	Gradation Organic Impurities Fineness Modulus Material finer than No. 200 Sieve	ASTM C-136 ASTM C-40 ASTM C-136 ASTM C-117	2 tests per shift 1 test per shift 2 tests per shift 1 test per shift
Concrete	Slump Temperature	ASTM C-143	1 each 100 yd^3 per placement per mix
	Air Content Unit Weight Compressive Strength	ASTM C-138/C-231 ASTM C-138 ASTM C-39	1 each 100 yd^3 or fraction thereof per mix or 1 each 250 yd^3 or fraction thereof per mix for pours over 500 yd^3.
	Capping Cylindrical Concrete Specimens	ASTM C-617	Not applicable
	Accelerated Curing of Concrete Specimens	ASTM C-684	Testing program to be determined upon request of the engineer.
	Standard Method of Sampling Fresh Concrete	ASTM C-172-71	Not applicable

Table 3.8-3
CONTROL TESTS FOR CONCRETE (Sheet 2 of 3)

Onsite Testing			
Material	Test	Procedure	Frequency
Cement	Chemical Analysis	ASTM C-114	1 set of tests per source
	Strength	ASTM C-109	
	False Set	ASTM C-451	
	Fineness	ASTM C-204 or C-115	
	Heat of Hydration	ASTM C-186	
	Soundness by Autoclave Expansion	ASTM C-151	
	Time of Setting	ASTM C-191 or C-266	
	Sulfate Expansion	ASTM C-452	
	Tensile Strength	ASTM C-190	
Pozzolan	Fly Ash and Pozzolan	ASTM C-618	1 test for every 1,000 tons used
Admixtures	Water Reducing	ASTM C-494	1 physical test per source and a chemical test with each 1,000 ton pozzolan test.
	Air Entraining	ASTM C-260	1 physical test per source and a chemical test with each 1,000 ton pozzolan test.
Aggregates	L.A. Abrasion	ASTM C-131	Once every 10,000 yd^3 of concrete.
	Mortar Making Prop.	ASTM C-87	Once every 10,000 yd^3 of concrete.
	Pot. Reactivity (Chem.)	ASTM C-289	Once every 10,000 yd^3 of concrete.
	Pot. Reactivity (Mortar Bar)	ASTM C-227	Once every 40,000 yd^3 of concrete produced.

Table 3.8-3
CONTROL TESTS FOR CONCRETE (Sheet 3 of 3)

Onsite Testing			
Material	Test	Procedure	Frequency
Aggregates (Cont)	Soundness	ASTM C-88	Once every 10,000 yd^3 of concrete
	Sp. Cr. and Absorp. (Coarse)	ASTM C-127	Once every 10,000 yd^3 of concrete
	Sp. Cr. and Absorp. (Fine)	ASTM C-128	Once every 10,000 yd^3 of concrete
	Petrographic	ASTM C-295	When aggregate tests indicate a significant change has occurred in the characteristics of the aggregate
Concrete	Radiation Shielding Properties	ASTM C-637	1 test per design mix
Water and Ice	Chloride Ion Sulfates Time of Set False Set Compressive Strength pH Total Dissolved Solids	ASTM D-512 ASTM D-516 ASTM C-191 ASTM C-451 ASTM C-109 ASTM D-1293 AASHO T-26	Initially and monthly thereafter until reliability is established then semi-annually thereafter, or as directed by the engineer contractor
Concrete Mixes	Design Making and Testing Specimens Air Content Slump Bleeding Compressive Strength Bond Developed with Reinforcing	ACI 211.1 ASTM C-192 ASTM C-231 ASTM C-143 ASTM C-232 ASTM C-39 ASTM C-234	Upon mix adjustment

Erection tolerances used for the liner plate system are as follows:

A. Liner Plate

1. The radial location of any point on the liner plate is not allowed to vary from the design radius by more than ± 3 inches. At any given elevation the maximum diameter minus the minimum diameter was not allowed to exceed 6 inches, with a 3-inch allowance for local out-of-roundness. Measurements were made at 30 degree spacings for each 10 feet of height.

2. Plates to be joined by butt welding are matched accurately and retained in position during the welding operation. Misalignment in completed joints is not allowed to exceed 10% of the plate thickness or 1/16-inch, whichever was greater.

3. A 15-foot long template curved to the required radius is not allowed to show deviations of more than 1-inch when placed against the completed surface of the shell within a single plate section and not closer than 12 inches to a welded seam. When the template is placed across one or more welded seams, the deviation is not allowed to exceed 1-1/2 inches. The effect of change in plate thickness or of weld reinforcement is excluded when determining deviations.

4. A 15-inch long template curved to the required radius is not allowed to show deviations of more than 1/8-inch inward and 3/8-inch outward when placed against the completed surface of the shell within a single plate section. Where the deviation exceeded these limits, remedial action is taken by the Contractor as directed by Bechtel to correct the deficiency.

5. The slope of any 10-foot section of cylindrical liner plate, referred to true vertical, is not allowed to exceed 1:180. The overall out of plumbness of the shell is not allowed to exceed 3 inches.

6. A 10-foot straight edge is not allowed to show deviations greater than $\pm 3/4$-inch in the vertical direction between seam welds.

7. Sharp bends are not permitted unless provision is made for them in the design. A sharp bend is defined as any local bend that deviated from the design radius or a vertical straight edge by an offset of more than 1/2-inch in 1 foot. The template used to measure the local deviations is only 1 to 2 feet longer than the area of the deviation itself.

B. Penetrations

1. Paragraphs 3.8.1.6.1.4 (A-1) through (A-7) also control the tolerance requirements for penetrations.

2. A 30-inch long template curved to the required radius is not allowed to show deviations of more than 3/4-inch when placed against the completed surface of the shell within a single plate section.

3. Alignments of the axis of penetrations (for 12-inch or smaller, nominal pipe size) as erected are not allowed to vary by more than 2 degrees from the alignment shown. Alignments of the axis of penetrations (for larger than 12 inch nominal pipe) as erected are not allowed to vary by more than 1 degree. Individual penetrations (except main steam and feedwater) and penetrations in common reinforcing plates are located within ± 1 inch of their design elevations and circumferential locations. Main steam and feedwater penetrations are located within $\pm 1/2$ inch of their design elevations and circumferential locations.

4. The location of penetrations at the shell in a common reinforcing plate are maintained within $\pm 1/4$-inch of the dimensions shown on the design drawings.

5. Rolled and welded pipe penetration sleeves whose nominal wall thickness is 3/8 inches or less are maintained inside circumferential dimensions equal to pi times the nominal inside diameter, plus or minus pi/32 inches for a minimum distance of 3 inches from the end which extends into the containment structure.

6. Rolled and welded pipe penetration sleeves whose nominal wall thickness is greater than 3/8 inch maintained their ID bored for a minimum distance of 1/2 inch from the end which extends into the containment structure in accordance with the following equations:

$$C = A - 0.041 - 1.75\ t$$

$$C = A - 0.041 - 2.0\ t_m$$

where:

A = nominal pipe OD (inches)

C = machined pipe ID (inches)

t = nominal wall thickness (inches)

t_m = minimum wall thickness (inches)

$t = 8/7\ t_m$

C. Dome Liner Plate

Tolerance allowances for the dome liner plate are maintained in accordance with paragraphs 3.8.1.6.7.4 (A-2 and A-3), and paragraph 3.8.1.6.7.4 (B-2). The radius of curvature of the dome liner plate is maintained within plus 9 inches and minus 13-1/2 inches of the design radius shown.

D. Crane Support and Pipe Support Brackets

Crane support bracket locations shown on the engineering design drawings are maintained within ±1/2 inch of the design elevation and circumferential locations. Pipe support brackets are located within ±1 inch of their design elevation and circumferential locations.

E. Miscellaneous Brackets Located on Penetration Sleeves and Liner Plate Stiffeners

The brackets are located within ±1 inch of their design elevation and circumferential location.

F. Embedded Floor Beams, Thickened Floor Liner Plate, and Miscellaneous Embedments

1. The field placement of the embedded floor beams and thickened floor liner plate within ±1/2 inch of the design location is maintained.

2. The locations of Cadwelds and threaded sleeves on the thickened floor liner plate are maintained ±1/8 inch of the design location. The thickened plate dimensions are maintained within ±1/8 inch of dimensions given on the engineering design drawings.

3.8.1.6.7.5 <u>Control Tests and Inspection of Prestressing System</u>. The following quality control procedures are used:

A. Prestressing Strands

1. Each tendon is individually identified and traceable to the heat numbers of the wire utilized in its buildup. All chemical and physical test reports supporting the integrity of each heat of material are reviewed as a condition of acceptance.

2. Specimens are cut from each reel of strand and tension tested to assure compliance to specifications.

3. Strands are examined for workmanship and quality prior to fabrication of the tendon.

B. Bearing Plates and Trumpets

1. Verify that the bearing plate material complies with that specified on the drawings. Compliance is evidenced by mill test reports traceable to the heat number by serial numbers permanently marked on each bearing plate.

2. Charpy V-notch tests are conducted to provide assurance that the bearing plates are not susceptible to brittle fracture.

3. All plates are examined for workmanship and quality. Cracks, burrs, corrosion, and other defects are not acceptable.

C. Anchor Head

1. All raw material is accompanied by mill certificates and subjected to receiving inspection.

2. After Blanchard grinding, a first article inspection is conducted to check proper surface finish.

3. After drilling of tapered holes, a first article inspection is made and on each 51st piece thereafter.

4. After forming 0.12-inch radius at bottom of strand holes, a first article dimensional inspection is made, followed by a check of every 100th piece.

5. Parts are coated with a preservative prior to shipment.

D. Erection Tolerances

1. Sheathing was wired to mild steel reinforcing or other sheathing with a 16-gage tie wire. These ties were made at intervals of 5 to 7 feet to assure that the duct would not be displaced during concrete placing.

2. Around penetration areas, tolerances were maintained at ± 6 inches in the vertical and ± 4 inches in the horizontal.

3. Alignment of vertical sheathing was maintained within 3/4-inch per 10 feet.

4. Alignment of horizontal and curved sheathing was maintained within ±3/4 inch.

5. The minimum clearance between a penetration edge and the centerline of the sheathing was restricted to 1 foot 0 inches.

6. The minimum radius of bend for all deflected tendons was maintained at 25 feet 0 inches.

3.8.1.7 Testing and Inservice Inspection Requirements

3.8.1.7.1 Structural Integrity Pressure Test

Following construction, the containment is proof-tested at 115% of the design pressure. During this test, deflection measurements and concrete crack inspections are made to determine that the actual structural response is within the limits predicted by the design analyses.

The test procedure complies with the requirements of NRC Regulatory Guide 1.18.

3.8.1.7.2 Long-Term Surveillance

The long-term surveillance program consists of evaluating the general condition of the post-tensioning system. Data on strand corrosion level and tendon lift-off forces are obtained and analyzed. The surveillance tendons and surveillance frequency are designated by the engineer as explained in Section 9.3 of BC-TOP-5.[1] The surveillance program provides assurances of the continuing ability of the structure to meet the design functions as stated in paragraph 3.8.1.5.

3.8.2 STEEL CONTAINMENT

As described in subsection 3.8.1, the containment is a prestressed, reinforced concrete structure; therefore this subsection does not apply.

3.8.3 CONCRETE AND STEEL INTERNAL STRUCTURES OF STEEL OR CONCRETE CONTAINMENTS

3.8.3.1 Description of the Internal Structures

The internal structures located in the containment consist of the reactor supports, steam generator supports, reactor coolant pump supports, reactor coolant pipe restraints, primary shield wall and reactor cavity, secondary shield walls, pressurizer supports, refueling canal walls, and the operating and intermediate floors.

3.8.3.1.1 Reactor Vessel Supports

The reactor vessel is supported by four columns under the cold leg nozzles discussed in subsection 5.4.14 which interface with structural steel built-up members that are almost entirely embedded in the primary shield. The built-up members transmit the vertical loads to the reactor cavity basemat. Lateral supports are provided for the reactor vessel to resist the horizontal loads. These lateral supports transmit the loads to the reactor cavity wall, which houses the reactor. In addition, shear keys at the lower part of the reactor vessel fit into the keyways, located in the base plate, of the column supports. These keyways which are described in subsection 5.4.14 transmit the horizontal loads to the cavity wall through horizontal members attached to the base plate. Both the lateral supports and shear keys are designed to allow movement due to thermal growth, of the reactor vessel in the radial and vertical directions. Details of both the lateral and vertical reactor vessel supports are shown in figure 3.8-20.

3.8.3.1.2 Steam Generator Supports

The steam generator is mounted on a thick, heavily reinforced, concrete pedestal. The loads are transmitted to the pedestal by means of high-strength bolts, bearing plates, and shear keys (Refer to figure 3.8-21, sheet 1). The pedestal, in turn, transmits these loads to the containment base slab. The upper part of the steam generator is restrained by means of shear keys and snubbers that are attached to the refueling canal walls and secondary shield walls. The steam generator pedestal is shown on figure 3.8-21, sheet 2.

3.8.3.1.3 Reactor Coolant Pump Supports and Stops

The reactor coolant pumps are supported by four vertical columns and four horizontal columns. The horizontal columns and a portion of the vertical columns are considered to be part of the NSSS as discussed in subsection 5.4.14. The vertical columns transmit the vertical loads to the containment basemat. The horizontal columns transmit the lateral loads to the refueling canal walls and to the secondary shield walls. All of the columns are hinged to permit radial (defined as an axis passing through the center of the reactor and the pump) movement of the pumps due to thermal growth.

Additionally, three stops are provided for each reactor coolant pump. These stops consist of A-frames designed to resist lateral loads in the radial direction due to a postulated LOCA. The resisted loads are transmitted to the secondary shield walls. Details of the reactor pump supports and stops are shown in figure 3.8-22, sheets 1 and 2, respectively.

3.8.3.1.4 Pressurizer Supports

The pressurizer is supported by a structural steel frame at elevation 45 ft. The main members of the frame are attached to the secondary shield walls by means of moment connections. This frame is supported at one corner by a column which is supported by the basemat. The pressurizer support skirt, which is described in subsection 5.4.14, is attached to the frame using a bolted connection. Both the frame and the connections are designed for vertical and lateral loads as well as for moments due to dead loads, thermal loads, seismic loads, and loads due to surge line and other pipeline breaks.

In addition, the pressurizer is restrained at the top by means of keys which fit into keyways. Four keyway supports are located 90° apart and are designed to transmit the lateral loads due to seismic excitation and due to subcompartment pressures by pipe breaks to the secondary shield walls. These keyways are also designed to permit vertical and radial movement of the pressurizer due to thermal growth. Details of the pressurizer supports are shown in figure 3.8-23.

3.8.3.1.5 Reactor Coolant System Pipe Restraints

The reactor coolant system restraints are provided to restrict the displacement of the reactor coolant piping during a postulated LOCA. These restraints are designed to resist the pipe rupture loads, assuming elastic-plastic behavior. The reactions from the restraints are transmitted to the basemat or various heavily reinforced members which transmit the loads to the basemat.

The restraints are also designed to resist lateral loads during a postulated LOCA. The lateral loads considered are equal to 5% of the axial loads applied laterally.

All the restraints are provided with a gap to permit movement of the pipes due to thermal growth and seismic displacements. Details of the reactor coolant pipe restraints are shown in figure 3.8-24.

3.8.3.1.6 Primary Shield Wall and Reactor Cavity

The primary shield is a heavily reinforced concrete structure that houses the reactor, provides the primary radiation shielding and pressure barrier, and is an integral part of the internal structures.

The massive primary shield walls, which are anchored into the containment base slab, provide a support for the refueling canal walls above the reactor cavity. In plan, the primary shield walls form a monolithical ring, housing the reactor vessel. Large penetrations in the primary shield walls are provided for the primary loop piping and cavity ventilation system.

Details of the primary shield walls are shown in figure 3.8-25.

3.8.3.1.7 Refueling Canal

The refueling canal is a reinforced concrete structure that is flooded during the reactor refueling operation. The canal walls are partially supported by the primary shield and partially by the containment basemat.

The refueling canal is lined with stainless steel plate, and is connected with the spent fuel pool, in the fuel handling building, through the fuel transfer tube.

3.8.3.1.8 Fuel Transfer Tube

A detailed description of the fuel transfer tube is provided in paragraph 3.8.1.1.3.5.

3.8.3.1.9 Secondary Shield Walls

The secondary shield is a heavily reinforced concrete structure enclosing (together with the refueling canal walls) the steam generator compartments. The massive secondary shield walls are anchored into the base slab of the containment in a manner similar to the primary shield walls, in order to allow for load transfer to the foundation. Each of the two enclosed secondary shield compartments houses a steam generator and two reactor coolant pumps. In addition, one of the compartments also houses the pressurizer.

Steel embedments in the secondary shield walls transmit loads from various equipment, pipe supports, platforms to the walls, and from the operating and intermediate floors.

Details of the secondary shield walls are shown in figure 3.8-26.

3.8.3.1.10 Operating and Intermediate Floors

The floors inside the containment consist of both composite construction and steel grating supported by structural steel framing. The steel framing is supported by perimeter steel columns just inside the exterior shell by means of a 6-inch gap between the floors and the shell.

Details of the operating floor are shown in figure 3.8-27.

3.8.3.2 Applicable Codes, Standards, and Specifications

The applicable codes, standards, specifications, regulatory guides, and other documents used in the structural design, fabrication, and construction of the internal structures are provided in paragraph 3.8.1.2.

3.8.3.3 Loads and Load Combinations

3.8.3.3.1 Load Definitions

The internal structures are designed for all credible loading conditions. The design load categories are identified as normal loads, severe environmental loads, extreme environmental loads, and abnormal loads.

3.8.3.3.1.1 Normal Loads. Normal loads are those loads to be encountered during normal plant operation and shutdown. They include the following:

A. Dead Loads

Dead loads are those produced by the weight of the structures, including hydrostatic effects, and permanent equipment loads.

B. Live Loads

Live loads consist of any movable equipment loads and other loads with variable intensity and occurrence.

C. Normal Thermal Loads

Normal thermal loads are produced due to the temperature distribution through the wall during normal operating or shutdown conditions, based on the most critical transient or steady-state condition.

D. Normal Pipe Expansion Loads

Normal pipe expansion loads consist of local forces on the structure caused by thermal expansion of piping during normal operating or shutdown conditions based on the most critical transient or steady-state condition.

3.8.3.3.1.2 Severe Environmental Loads. Severe environmental loads are those loads that could infrequently be encountered during the plant life. Included in this category is the operating basis earthquake (OBE). The OBE loading consists of a static equivalent seismic load for which the dynamic effects have been included in its determination. A more detailed discussion is presented in subsection 3.7.1.

3.8.3.3.1.3 Extreme Environmental Loads. Extreme environmental loads are those loads which are credible, but are highly improbable. They include the design basis earthquake (DBE). The DBE loading consists of a static equivalent seismic load for which the dynamic effects have been included in its determination. A more detailed discussion is presented in subsection 3.7.1.

3.8.3.3.1.4 Abnormal Loads. Abnormal loads are those loads generated by a postulated hypothetical high-energy pipe break accident inside the containment and/or a compartment of the interior structure. Included in this category are the following:

A. Pipe Rupture and Miscellaneous Missile Loads

Pipe rupture loads consist of local loads on a structure which are generated by either the jet impingement from a ruptured high-energy pipe or from the impact of a whipping ruptured pipe generated by a postulated break. These loads can also be generated by the reaction of the ruptured pipe on its supports. All of these loads are applied as a static equivalent load that includes an appropriate dynamic factor. Pipe rupture effects are discussed in section 3.6.

Miscellaneous missile loads are described in detail in section 3.5.

B. Loss-of-Coolant Accident (LOCA) Pressure Load

The design differential pressure of the interior structure compartments is greater than the calculated peak pressure occurring as the result of any postulated LOCA. The steam generator compartment design pressure is given in paragraph 3.8.3.4.3.

C. Abnormal Thermal Loads

Abnormal thermal loads are produced due to the temperature distribution through the wall and expansion of the liner during the loss-of-coolant accident.

D. Abnormal Pipe Expansion Loads

Abnormal pipe expansion loads consist of local forces on the structure caused by thermal expansion of piping during loss-of-coolant accident based on the most critical transient or steady-state condition.

3.8.3.3.2 Load Combinations

The following nomenclature is used in the load combinations.

D = Dead load

L = Appropriate live load

T_o = Normal thermal loads

H_o = Normal pipe expansion load

E = Operating basis earthquake load

E' = Design basis earthquake load

R = Pipe rupture and miscellaneous missile loads

P = LOCA pressure load

T_A = Abnormal thermal load

H_A = Abnormal pipe expansion load

The load combinations and load factors for which the strength method is used are as follows:

A. Concrete

1. Normal Case

   $1.4D + 1.7L$

2. Severe Environmental Case

   $1.25D + 1.25L \pm 1.25E + 1.0\ H_o$

3. Severe Environmental Case

   $1.25D + 1.25L \pm 1.25E + 1.0\ T_o$

4. Abnormal/Severe Environmental Case

   $1.0D + 1.0L \pm 1.25E + 1.0T_A + 1.0P + 1.0R + 1.0H_A$

5. Abnormal/Severe Environmental Case

   $1.0D \pm 1.25E + 1.0T_A + 1.0P + 1.0R + 1.0H_A$

6. Abnormal/Extreme Environmental Case

   $1.0D + 1.0L \pm 1.0E' + 1.0T_o + 1.0P + 1.0R + 1.25H_o$

7. Abnormal/Extreme Environmental Case

   $1.0D + 1.0L \pm 1.0E' + 1.0T_A + 1.0P + 1.0R + 1.0H_A$

8. Abnormal Case

   $1.0D + 1.0L + 1.0T_o + 1.25H_o$

B. Structural Steel

Steel structures shall satisfy the following loading combinations without exceeding the allowable working stress for equations 1 and 3, and 33-1/3% increase in allowable working stress for equation 2, and 90% of the ultimate capacity (with full regard to elastic stability) for equations 4 through 6.

1. Normal Case

$D + L$

2. Severe Environmental Case

$D + L + T_o + H_o + E$

3. Severe Environmental Case

[a] $D + L + H_o + E$

4. Abnormal/Extreme Environmental Case

$D + L + R + P + T_o + H_o + E'$

5. Abnormal/Extreme Environmental Case

$D + L + R + P + T_A + H_A + E'$

6. Abnormal Case

$D + L + R + P + T_o + H_o$

The stress levels reflected in the above loading combinations are based upon the specific nature of the loading condition and the actual function of the structure. In general, for the operating basis seismic design, the structure is designed for elastic behavior using load factors. In the case of concrete, based upon stresses significantly below the ultimate strength capacity, and working stress levels in the case of structural steel. For the accident basis design, stress analysis is based upon stresses at or just below the ultimate strength capacity for concrete, and just below the ultimate elastic capacity for structural steel.

3.8.3.4 Design and Analysis Procedures

The basic techniques of analyzing the internal structures can be classified into two groups: conventional methods involving simplifying assumptions, such as those found in beam theory, and those based on plate and shell

a. For structural elements carrying mainly earthquake forces only; e.g., struts and bracing.

theories of different degrees of approximation. The strength design methods given in the ACI-318-71 code are used for concrete and the AISC code is used for steel. The internal structures are provided with connections capable of transmitting axial and lateral loads to the containment base slab.

The SAP computer program, referenced in appendix 3C, section 3C.5, is used to perform all static and dynamic stress analyses. The RESCOS and OPTCON computer programs, referenced in appendix 3C, section 3C.8, are used in design to proportion principal steel reinforcement.

Seismic analyses for the interior structures conform to the appropriate procedures outlined in section 3.7.

The mathematical model used includes equipment of significant mass values as discrete masses at the appropriate elevation. The seismic loads are determined using the procedures of the design response spectrum technique of analysis. Bending moments and shears resulting from appropriate earthquake loads are combined according to the load combinations described in paragraph 3.8.3.3. The equipment seismic shear is resisted by the anchorage system, anchor bolts, and by additional shear studs.

Design of the interior structure evolves around four basic systems: the reactor coolant system, the main steam system, the engineered safeguards system, and the fuel handling system supply.

The structures that house or support the basic systems are designed to sustain the factored loads described in paragraph 3.8.3.3.

The design bases to be applied are given as follows:

A. All operating loads, seismic loads, and thermal deformations at the levels indicated in paragraph 3.8.3.3.

B. Loads and deformations resulting from a LOCA and its associated effects.

C. Environmental effects resulting from a postulated high-energy line break such as temperature, pressure, humidity, or flooding. The magnitude of thrust forces and pressure buildup resulting from a pipe break is determined from appropriate blowdown values.

D. Jet impingement equivalent static loads on a structure generated by a postulated high energy line break.

E. Missile impact equivalent static loads on a structure generated by or during a postulated high-energy line break, like pipe whipping.

F. Missiles as described in section 3.5.

The containment interior structure is designed to provide structural supporting elements for the entire NSSS, as well as required shielding. Basic supporting components are designed using both reinforced concrete and structural steel as appropriate. All design aspects are integrated with the design criteria of the nuclear steam supply system vendor and include particular attention to the combined thermal and dynamic effects particularly evident during earthquake conditions. Thrusts are taken by rigid members and by shock suppressors. Design loads and loading combinations for the interior structure are listed and described in paragraph 3.8.3.3.

The main considerations in establishing the structural design criteria for the internal structures are to provide a structure that will withstand the differential pressure within the reactor cavity and across the secondary shield walls in the event of an accident, and to minimize the effects of the pipe rupture force and seismic loadings utilizing supports and restraints. Loads and deformations resulting from a LOCA and its associated effects on any one of the basic systems are restricted so that propagation of the failure to any other system is prevented. In addition, a failure in one loop of the NSSS is restricted, so that propagation of the failure to the other loop is prevented. Localized concrete yielding is permitted, when it is demonstrated that the yield capacity of the component is not affected, and that this small localized yielding does not generate missiles that could damage the structure. Full recognition is given to the time increments associated with these postulated failure conditions, and yield capacities are appropriately increased, when a transient analysis demonstrates that the rapid strain rate justifies this approach. The walls are also designed to provide adequate protection for potential missile generation that could damage the containment liner.

The effect of radiation-generated heat on the internal structures was considered in the design of the primary and secondary shield walls. The shield wall thicknesses were determined on the basis of the radiation shielding requirements and, therefore, are greater than those required for structural purposes. This additional thickness provides a reserve strength greater than required to offset minor damages to the structures due to a LOCA. Since high temperatures are damaging to concrete, a thorough ventilation at a constant temperature is maintained within the containment to cool the area surrounding the shield walls and to prevent any appreciable loss of structural strength due to gamma and neutron heating.

The final design of the interior structure and equipment supports are reviewed to assure that they can withstand applicable pressure loads, jet impingement forces, pipe reactions, and earthquake loads without loss of function. The deflections or deformations of the structures and supports are checked to ensure that the functions of the containment and safety feature systems are not impaired.

3.8.3.4.1 Reactor Coolant System Equipment Supports

The steel and concrete supports for the reactor, the reactor coolant pumps, the steam generators, safety injection tanks, the pressurizer and the quench tanks are designed for dead loads, seismic loads, and nozzle reaction loads. These loads include the maximum forces on a support due to accident loads (e.g., pipe rupture) with a dynamic load factor, operating loads, and seismic loads. The directions of the seismic forces are chosen to give the largest load at each support.

The loads are combined using the maximum seismic forces and the maximum accident forces simultaneously. This combination ensures the worst possible design condition that could occur for each support.

All the reactor coolant system equipment supports are designed using conventional design techniques.

A combination of hot gaps, keyways, and snubbers are provided between the above mentioned equipment and their supports to ensure that minimal thermal loads from the expansion of the equipment are transmitted to the supports.

3.8.3.4.2 Primary Shield Wall and Reactor Cavity

For the hypothetical LOCA condition, the cavity wall is designed to withstand jet impingement forces and internal pressurization combined with seismic and LOCA loads on the reactor vessel and coolant pipeline without gross damage to the cavity structure. Local damage to the cavity in the immediate vicinity of the NSSS component failure is inevitable. However, vital parts of the containment are protected from this failure to ensure a post-accident leaktight containment structure.

The reactor cavity is designed to withstand a static equivalent internal pressure of 228.9 lb/in.^{2} due to the LOCA. This pressure acts on the entire cavity for a duration of 1 second. The maximum stress level in the rebar under the worst loading combination is limited to 90% of yield stress of the rebar.

For the normal operating condition, the reactor cavity is designed to withstand the stresses due to dead loads, live loads, and seismic loads. Under this condition, the stresses in the concrete and the reinforcing steel are significantly below working stress levels. In the stress analysis, flexure tensile cracking is permitted but is controlled by the bonded reinforcing steel.

3.8.3.4.3 Secondary Shield Wall and Steam Generator Compartments

The secondary shield wall, the refueling canal walls, and the walls enclosing steam generator compartments are designed for an internal pressure of 28.8 lb/in.^{2} due to a LOCA resulting from any of the postulated pipe breaks listed in table 6.2-1. The compartments are also designed for jet forces on localized areas of the walls resulting from the impingement of escaping fluid. In addition, the affect of pipe rupture loadings at various restraints on the walls has been considered with local analysis of the walls.

3.8.3.4.4 Refueling Canal

For the refueling condition, the walls are designed for the hydrostatic head due to 41.5 feet of water. The steam generator compartment pressure loads due to postulated pipe rupture and hydrostatic head are not considered to occur simultaneously.

3.8.3.5 Structural Acceptance Criteria

The limiting values of stress, strain, and gross deformations are established by the following criteria.

A. To maintain the structural integrity when subjected to the worst load combinations

B. To prevent structural deformations from displacing the equipment to the extent that the equipment suffers a loss of function

The allowable stresses are those specified in the applicable codes. The stress contributions due to earthquakes are included in the load combinations described in paragraph 3.8.3.3.

Table 3.8-4 summarizes the governing load interactions and maximum capacity of principal reinforced concrete members. The ratio of the maximum capacity to the required capacity yields the safety margin. Table 3.8-5 summarizes the governing combined stress ratios from the beam/column interaction equation for principal structural steel members. Table 3.8-6 summarizes the ductility ratios for RCS pipe stops and other pipe whip restraints.

3.8.3.6 Materials, Quality Control, and Special Construction Techniques

The following basic materials are used in the construction of internal structures:

A. Concrete — f'_c (lb/in.^{2}) = 6,000 (at 90 days)

B. Reinforcing Steel

Deformed bars — ASTM A-615 Grade 60 — f_y (lb/in.^{2}) = 60,000 minimum

Table 3.8-4
CONTAINMENT INTERNAL STRUCTURES SUMMARY OF GOVERNING LOAD INTERACTIONS FOR PRINCIPAL REINFORCED CONCRETE MEMBERS (Sheet 1 of 3)

Description of Principal Members	Location of Principal Members	Governing Load Combination Number[a]	Calculated Axial Load (P_u) and Flexural Load (M_u)		Maximum Flexural Interaction Capacity (M_u), Given Axial Load P_u
			P_u[b]	M_u[c]	M_u[c]
Wall Vertical reinforcement	Secondary shield wall From El 15'-0" to El 36'-6"	7	173	465	620
Wall Vertical reinforcement	Secondary shield wall From El 36'-6" to El 48'-0"	6	33.1	72.4	650
Wall Vertical reinforcement	Secondary shield wall From El 48'-0" to El 63'-6"	6	37.6	73.5	379
Wall Vertical reinforcement	Secondary shield wall From El 63'-6" and above	6	94.8	68.5	201
Wall Horizontal reinforcement	Secondary shield wall From El 15'-0" to El 45'-0"	6	92.7	50.3	235

a. Refer to paragraph 3.8.3.3.2.A for description of load combination number.
b. P_u is in kips.
c. M_u is in ft-k/ft.
d. Actual maximum and allowable stresses, respectively.

Table 3.8-4
CONTAINMENT INTERNAL STRUCTURES SUMMARY OF GOVERNING LOAD INTERACTIONS FOR PRINCIPAL REINFORCED CONCRETE MEMBERS (Sheet 2 of 3)

Description of Principal Members	Location of Principal Members	Governing Load Combination Number[a]	Calculated Axial Load (P_u) and Flexural Load (M_u)		Maximum Flexural Interaction Capacity (M_u), Given Axial Load P_u
			P_u[b]	M_u[c]	M_u[c]
Wall Horizontal reinforcement	Secondary shield wall From El 45'-0" to El 63'-6"	6	14.7	4.8	151
Wall Horizontal reinforcement	Secondary shield wall From El 63'-6" and above	6	77.6	154.9	203
Wall Vertical reinforcement	Canal wall From El 15'-0" to El 50'-0"	7	50	401	626
Wall Vertical reinforcement	Canal wall From El 45'-0" to El 63'-6"	6	7.70	437.20	447
Wall Horizontal reinforcement	Canal wall From El 15'-0" to El 45'-0"	6	391	89	748
Wall Horizontal reinforcement	Canal wall From El 45'-0" to El 63'-6"	6	88.2	122.8	259
Slab	1"-3" slab at El 30'-0"	3	0.0	61	61

Table 3.8-4

CONTAINMENT INTERNAL STRUCTURES SUMMARY OF GOVERNING LOAD INTERACTIONS FOR PRINCIPAL REINFORCED CONCRETE MEMBERS (Sheet 3 of 3)

Description of Principal Members	Location of Principal Members	Governing Load Combination Number[a]	Calculated Axial Load (P_u) and Flexural Load (M_u)		Maximum Flexural Interaction Capacity (M_u), Given Axial Load P_u
			P_u[b]	M_u[c]	M_u[c]
Slab	1'-3" slab at El 30'-0"	3	0.0	31.9	37
Slab	3'-0" slab at El 45'-0"	3	0.0	248	304
Slab	1'-3" slab at El 45'-0"	3	0.0	27.4	38
Slab	1'-3" slab at El 45'-0"	3	0.0	54.7	88
Slab	Slab at 63'-6" (reactor head laydown area)	3	0.0	75	99
Slab	Slab at 63'-6" (reactor head laydown area)	3	0.0	160	210
Slab	1'-6" slab at El 63'-6"	3	0.0	17.10	37
Wall	Primary shield hoop bar	4	47 ksi[d]		54 ksi[d]
Column	Steam generator pedestal	6	1,412	65,600	94,700

Table 3.8-5
CONTAINMENT INTERNAL STRUCTURES SUMMARY OF GOVERNING COMBINED STRESS RATIOS FROM THE BEAM/COLUMN INTERACTION EQUATION FOR PRINCIPAL STRUCTURAL STEEL MEMBERS (Sheet 1 of 3)

Description of Principal Members	Location of Principal Members	Governing Load Combination Number[(a)]	Combined Stress Ratio ($\leq$1.0)
Bolts (14" ϕ and 2" ϕ)	Steam generator upper support bolts for snubber and link assembly	5	0.74 and 0.68
Bolts	Steam generator upper support bolts for shear keys	5	0.9
Bolts (3" ϕ)	Reactor coolant pumps vertical column bolts	4	0.57
Bolts (3" ϕ)	Reactor coolant pumps horizontal column bolts	4	0.40
Bolts (3-1/2" ϕ)	Reactor coolant pumps horizontal column bolts	4	0.91
Bolts (3" ϕ)	Reactor coolant pumps snubber bolts	4	0.34
Beam	Reactor coolant pumps stop	4	0.92
Column	Pressurizer vertical support column	4	0.48
Beam	Pressurizer lower support member	4	0.92
Beam	Pressurizer upper support member	(b)	(b)
Bolts	Pressurizer upper support shear key bolts	(b)	(b)
Bolts (1-1/8" ϕ)	Safety injection tank upper lateral support bolts	4	0.92
Beam	Safety injection tank upper lateral support bracket	4	0.95

a. Refer to paragraph 3.8.3.3.2.B for description of load combination number.
b. Values to be provided in an amendment to this FSAR by approximately February 1977.

Table 3.8-5
CONTAINMENT INTERNAL STRUCTURES SUMMARY OF GOVERNING COMBINED STRESS RATIOS FROM THE BEAM/COLUMN INTERACTION EQUATION FOR PRINCIPAL STRUCTURAL STEEL MEMBERS (Sheet 2 of 3)

Description of Principal Members	Location of Principal Members	Governing Load Combination Number[(a)]	Combined Stress Ratio ($\leq$1.0)
Column	Interior structure columns from El 15'-0" to El 47'-0"	3	0.92
Column	Interior structure columns from El 47'-0" and above	3	0.50
Beam	Interior structure main girder at El 30'-0" (composite members)	3	0.85
Beam	Interior structure main girders at El 30'-0" (non-composite members)	3	0.55
Beam	Interior structure main girders at El 45'-0" (composite members)	3	0.97
Beam	Interior structure main girders at El 63'-6" (composite members)	3	0.77
Beam	Interior structure main girders at El 63'-6" (ASTM A-572 Gr.50)	3	0.99
Plate	Polar crane bracket	4	0.98
Girders	Polar crane girders between bracket	4	0.51
Girders	Polar crane main box girders	4	0.93
Column	Reactor vessel vertical support columns	4	0.85
Bolts (3-1/2" ϕ)	Reactor vessel anchor bolts (Bechtel/CE interface)	4	0.58
Forged members	Reactor vessel lateral support axial member	4	0.99

Table 3.8-5
CONTAINMENT INTERNAL STRUCTURES SUMMARY OF GOVERNING COMBINED STRESS RATIOS FROM THE BEAM/COLUMN INTERACTION EQUATION FOR PRINCIPAL STRUCTURAL STEEL MEMBERS (Sheet 3 of 3)

Description of Principal Members	Location of Principal Members	Governing Load Combination Number[(a)]	Combined Stress Ratio ($\leq$1.0)
Bolts (4" ϕ)	Steam generator lower support anchor bolts	4	0.94
Bolts (3-1/2" ϕ)	Steam generator lower support anchor bolts	4	0.77

C. Structural and miscellaneous steel

Rolled shapes, bars, and plates	ASTM A-36	f_y (lb/in.^{2}) =	36,000 minimum
	ASTM A-572 Grade 50	f_y (lb/in.^{2}) =	50,000
Forgings	ASTM A-237 Class C	f_y (lb/in.^{2}) =	58,000 and 60,000 (varies depending on material thickness)
Crane rails	ASCE		175 lb/yd
High-strength bolts	ASTM A-325	f_y (lb/in.^{2}) =	81,000 to 92,000 (varies depending on diameter of bolts)
	ASTM A-490	f_y (lb/in.^{2}) =	130,000 minimum

Table 3.8-6
CONTAINMENT INTERNAL STRUCTURES SUMMARY OF DUCTILITY RATIOS FOR RCS PIPE STOPS AND OTHER PIPE WHIP RESTRAINTS

Description of Principal Members	Location of Principal Members	Governing Load Combination Number[(a)]	Ductility Ratios		Remark
			Actual	Allowable	
RCS pipe stop	Reactor coolant outlet line restraints	4	5.6	6	Compression
RCS pipe stop	Reactor coolant inlet line restraints	4	5.6	6	Compression
RCS pipe stop	Reactor coolant pump suction line restraints (vertical)	4	6	6	Compression
RCS pipe stop	Reactor coolant pump suction line restraints (horizontal)	4	6	6	Compression
Pipe whip restraint	Main steam line restraints	4	27	100	Tension
Pipe whip restraint	Feedwater line restraints	4	86	100	Tension
Pipe whip restraint	Safety injection line restraints (12" ϕ)	4	76	100	Tension
Pipe whip restraint	Surge line restraints	4	41	100	Tension

a. Refer to paragraph 3.8.4.3.2.B for description of load combination number.

	A-449,	f_y (lb/in.^{2}) = 58,000 to 92,000 (varies depending on diameter of bolts)
	A-540	f_y (lb/in.^{2}) = 105,000 to 150,000 (varies depending on the diameter, grade, and class of material)
Other bolts	ASTM A-307	f_t (lb/in.^{2}) = 60,000 minimum
Stainless steel plate, sheet and strip	ASTM A-240, Type 304L	f_y = 25,000
Stainless steel bars and shapes	ASTM A-276, or A-479 Type 304L	f_y (lb/in.^{2}) = 25,000 f_y (lb/in.^{2}) = 25,000
Stainless steel bolts	ASTM A-320 Grade C	f_y (lb/in.^{2}) = 30,000
Unistrut	ASTM A-570 Grade C	f_y (lb/in.^{2}) = 33,000
Shear studs	ASTM A-108	f_y (lb/in.^{2}) = 50,000

D. Interior coating system

Carbon steel surface

Primer - Mobil 78W300 epoxy

Finish coat - Mobil 78W300 epoxy

Concrete and masonry surfaces

First coat - Mobil 46 x 2900

Second coat - Mobil 46 x 2900

Third coat - Mobil 89W900

Concrete floor

First coat - Mobil 84V-200

Second coat - Mobil 89W-900

Third coat - Mobil 89W-900

The materials and the quality control procedures are described in paragraph 3.8.1.6.

3.8.3.7 Testing and Inservice Inspection Requirements

A formal program of testing and inservice surveillance is not planned for the internal structures. The internal structures are not directly related to the functioning of the containment concept, hence, no testing or surveillance is required.

3.8.4 OTHER SEISMIC CATEGORY I STRUCTURES

3.8.4.1 Description of the Structures

Seismic Category I structures other than the containment and its internal structure are listed below:

A. Auxiliary building

B. Fuel handling building

C. Safety equipment building

D. Intake structure

E. Electrical/piping junction structure

F. Diesel generator building

G. Condensate and refueling tank enclosure structure

3.8.4.1.1 Auxiliary Building

The auxiliary building is a conventional reinforced concrete structure containing the control area, radwaste area, primary plant makeup, and radwaste storage tank area, and two pipe-penetration areas. The plan dimension of the structure is approximately 220 x 280 feet with a maximum height of approximately 94 feet. Several typical plans and sections are shown in figures 3.8-28 through 3.8-30.

The diverse functional requirements of the various entities housed within the auxiliary building have resulted in structural systems with correspondingly diverse physical characteristics. The control area is a relatively open, steel-framed, beam-column system supporting the floor slabs with a perimeter shear wall and light interior partition walls. The radwaste area consists of heavy shear walls to satisfy the compartmentalization and biological shielding requirements associated with its functional characteristics. The tankage area also incorporates a shear wall design concept. However, the story heights within this area are greatly increased over those in the adjoining sectors of the building, and the east perimeter wall is partially embedded. Finally, the penetration areas consist of a steel-framed, beam-column system supporting the cantilevered floor slabs and a partial perimeter shear wall.

3.8.4.1.2 Fuel Handling Building

The fuel-handling building is a conventional reinforced concrete structure containing the new- and spent-fuel handling, storage, and shipment facilities, fuel pool water cooling equipment, and decontamination area. The overall plan dimension of the structure is approximately 134 x 86 feet, with a maximum height of 110 feet. Several typical plans and sections are shown in figures 3.8-31 through 318-34. The structure is of heavy shear wall construction with a concrete-slab, steel-frame, composite-action roof system. Partial soil embedment of about 20 feet is present on three sides of the structure with no embedment on the fourth side.

3.8.4.1.3 Safety-Equipment Building

The safety-equipment building is an unsymmetrical, conventionally reinforced concrete structure that houses the safety-injection system, containment spray system, component cooling water system, and engineered safety features (ESF) electrical gallery. The safety-injection area and component cooling water area are located in the below-grade portion of the structure with the ESF electrical gallery occupying the roof elevation. The maximum plan dimension of the structure is 174 x 74 feet with an overall height of 70 feet. The minimum dimension of the structure is 48 feet in width. Several typical plans and sections are shown in figures 3.8-35 and 3.8-36. The safety-injection area consists of a uniform distribution of heavy shear walls to satisfy the separation and shielding requirements. The component cooling water area consists of large, open rooms with a minimum amount of shear walls. The one exception is in the lower elevation, where a more uniform distribution of shear walls is required to satisfy compartmentization and to provide structural support for the component cooling water heat exchangers. The ESF electrical gallery consists of an open tunnelway with longitudinal shear walls and a heavy roof diaphram. The basemat elevations of the safety-injection portion is not coplanar with the component cooling water portion, and the embedment characteristics vary on all four sides of the structure. The safety injection system piping is located in a tunnel attached to, and below, the basemat of the component cooling water area. The interfaces of the safety equipment building with the emergency sump tunnel of the containment, penetration area, and tunnel under the auxiliary building, are connected by a flexible stainless steel and Inconel bellows to allow movement in any direction due to seismic excitation.

3.8.4.1.4 Intake Structure

The intake structure is a conventional reinforced concrete buried structure that houses the major components of the circulating water system and the pumps associated with the saltwater cooling system (component cooling water system). The structure is quite irregular in shape with numerous piers, partition walls, and localized slab elevations (see figures 3.8-38). The maximum plan dimension is approximately 110 by 280 feet with a maximum height of approximately 60 feet. It is situated adjacent to the auxiliary building and the turbine building and is embedded in the soil to a varying degree, with the major portion of the structure completely below grade. Figure 3.8-37 also represents an isometric of the Unit 2 intake structure. The Unit 3 structure is symmetrical about an east-west axis along the south edge of the Unit 2 facility. Plant grade is at elevation +30 feet.

3.8.4.1.5 Electrical and Piping Gallery Structure

The electrical and piping gallery structure is a partially buried conventional reinforced concrete shear-wall structure. The structure provides a transition area for Seismic Category I piping and electrical cable from the underground tunnels and duct runs into the safety-equipment building. The overall plan dimension of the structure is 85 x 67 feet with a maximum height of 54 feet. Several typical plans and sections are shown in figures 3.8-39 through 3.8-40. The interior of the structure consists of numerous partial floor slabs, partition walls and vertical risers. Due to the physical proximity to other structures, the embedment characteristics vary on each side of the structure.

3.8.4.1.6 Diesel Generator Building

The diesel generator buildings are conventional reinforced concrete shear-wall structures with steel-frames and concrete-slabs. Each generator is situated in a separate compartment, and the foundations are located at grade. The resulting structures are regular in shape and exhibit little or no geometric eccentricities.

Typical plans and sections will be furnished in an amendment to this FSAR by approximately November 1977.

3.8.4.1.7 Condensate and Refueling Tank Enclosure Structure

This structure is a conventional reinforced concrete shear-wall structure that contains two steel-plate condensate storage tanks, two steel-plate refueling water storage tanks, one steel plate nuclear service water storage tank, and all of the associated piping valves and pumps. Each tank is installed in a separate compartment, and the foundations are located at grade. The resulting structure is fairly regular in shape but does exhibit some geometric eccentricities between the center of mass and the center of rigidity, due to the difference in size between the condensate storage tanks and the refueling water tanks.

The refueling water storage tanks are each 40 feet in diameter with a height of 38 feet. One condensate storage tank is 50 feet in diameter by 38 feet high, while the second tank is 30 feet in diameter by 31 feet high.

Typical plans and sections will be furnished in an amendment to this FSAR by approximately March 1977.

3.8.4.2 Applicable Codes, Standards and Specifications

The following codes, standards and regulations, specifications, design criteria, and NRC Regulatory Guides constitute the basis for the design, fabrication, and construction of other Seismic Category I structures. Modifications to these codes, standards, etc. are made when necessary to meet the specific requirements of the structure. These modifications are indicated in the sections where references to the codes, standards, etc. are made.

3.8.4.2.1 Codes

A. Uniform Building Code (UBC), 1970 Edition.

B. American Institute of Steel Construction (AISC), Manual of Steel Construction, 1969 Edition.

C. American Concrete Institute (ACI) 318-71, Building Code Requirements for Reinforced Concrete

D. American Welding Society (AWS), AWSD1.1-72, Structural Welding Code

3.8.4.2.2 Standards and Regulations

A. Occupational Safety and Health Act (OSHA)

B. State of California, Division of Industrial Safety General Industry Safety Orders

C. Nuclear Property Insurance Association-Mutual Atomic Energy Reinsurance Pool (NEPIA-MAERP), Basic Fire Protection for Nuclear Plants

D. National Fire Protection Association (NFPA), NFPA No. 24, Outside Protection

E. Hydraulic Institute (HI) Standards

3.8.4.2.3 Specifications

A. Industry Specifications

1. American Society for Testing and Materials (ASTM)

   ASTM standard specifications are used whenever possible to describe material properties, testing procedures, and fabrication and construction methods. The standards used, and the exceptions to these standards, if any, are identified in the applicable sections.

2. American Concrete Institute (ACI), ACI 301, Specification for Structural Concrete for Buildings, May 1972.

3. American Iron and Steel Institute (AISI), Specification for the Design of Light Gage, Cold-Formed Steel Structural Members, 1968 Edition

4. Crane Manufacturers Association of America (CMAA), CMAA Specification No. 70, 1971.

B. Project Design and Construction Specifications

Project design and construction specifications are prepared to cover the areas related to design and construction of other Seismic Category I structures. These specifications, prepared specifically for the San Onofre Nuclear Generating Station, Units 2 and 3, emphasize important points of the industry standards for the design and construction of the Seismic Category I structures, and reduce options that otherwise would be permitted by the industry standards. Unless specifically noted otherwise, these specifications do not deviate from the applicable industry standards. They cover the following subject headings:

1. Excavation and Backfill
2. Concrete Placement
3. Inspection of Concrete Production
4. Reinforcement Steel Placement
5. Structural Steel Erection
6. Miscellaneous Metalwork Installation
7. Stainless Steel Liner Plate System Installation
8. Concrete and Conrete Products
9. Reinforcing Steel and Associated Products
10. Structural Steel

11. Miscellaneous Steel and Embedded Materials

12. Stainless Steel Liner Plate

3.8.4.2.4 Design Criteria

A. Project Design Criteria

Project design criteria are prepared to include comprehensive design requirements of the other Seismic Category I structures, and contain specific references to prescribed Bechtel internal design guides, applicable industry standards, and pertinent technical texts, journals, and reports used in preparing the criteria.

B. Bechtel Topical Reports

1. BC-TOP-4, Seismic Analyses of Structures and Equipment for Nuclear Power Plants, Rev. 1, September 1972

2. BC-TOP-9A, Design of Structures for Missile Impact, Revision 2, September 1974

3. BN-TOP-2, Design for Pipe Break Effects, Revision 1, September 1973

C. Project Reports

1. Seismic and Foundation Studies, Dames and Moore, April 15, 1970.

2. Methods of Direct Application of Element Damping - San Onofre Units 2 and 3, Bechtel Power Corporation, Los Angeles Office, January 1972.

3. Development of Soil-Structure Interaction Parameters, Proposed Units 2 and 3 San Onofre Generating Station. Woodward-McNeil & Associates, Orange, California, January 31, 1974.

4. Elastic and Damping Properties, Laydown Area, San Onofre Nuclear Generating Station, Woodward-McNeill and Associates, Orange, California, October 14, 1971.

5. Preliminary Safety Analysis Report - San Onofre Units 2 and 3.

3.8.4.2.5 NRC Regulatory Guides

A. Regulatory Guide 1.10, Mechanical (Cadweld) Splices in Reinforcing Bars of Category I Concrete Structures

B. Regulatory Guide 1.15, Testing of Reinforcing Bars for Category I Concrete Structures

C. Regulatory Guide 1.55, Concrete Placement in Category I Structures

3.8.4.3 Loads and Load Combinations

3.8.4.3.1 Load Definitions

The other Seismic Category I structures are designed for all credible conditions of loading. The design load categories are identified as normal loads, severe environmental loads, extreme environmental loads, and abnormal loads.

3.8.4.3.1.1 Normal Loads. Normal loads are those loads to be encountered during normal plant operation and shutdown. They include the following:

A. Dead Loads

Dead load consists of the weight of the conrete wall, roof, base slab, steel, and permanently attached equipment, and in addition includes hydrostatic loads which consist of lateral hydrostatic pressure resulting from ground or flood water, as well as buoyant forces resulting from the displacement of ground or flood water by the structure.

B. Live Loads

Live loads consist of any movable equipment loads and other loads which vary with intensity and occurrence, such as floor occupancies and soil pressures.

C. Normal Thermal Loads

Normal thermal loads are produced due to the temperature distribution through the wall during normal operating or shutdown conditions, based on the most critical transient or steady-state condition.

D. Normal Pipe Expansion Loads

Normal pipe expansion loads consist of forces on the structure caused by thermal expansion of piping during normal operating or shutdown conditions based on the most critical transient or steady-state condition.

3.8.4.3.1.2 Severe Environmental Loads. Severe environmental loads are those loads that could infrequently be encountered during the plant life. Included in this category are:

A. Operating Basis Earthquake (OBE)

The OBE consists of a static equivalent seismic load for which the dynamic effects have been included in its determination. A more detailed discussion is presented in subsection 3.7.1.

B. Wind Loads

Refer to subsection 3.3.1 for a detailed description of wind loads.

3.8.4.3.1.3 Extreme Environmental Loads. Extreme environmental loads are those loads that are credible, but are highly improbable. They include:

A. Design Basis Earthquake (DBE)

The DBE consists of a static equivalent seismic load for which the dynamic effects have been included in its determination. A more detailed discussion is presented in subsection 3.7.1.

B. Tornado Loads

Tornado loads consist of the combined effects of tornado wind pressure, pressure differential, and missile impingement. Refer to subsection 3.3.2 for a detailed description.

3.8.4.3.1.4 Abnormal Loads. Abnormal loads are those loads generated by a postulated high-energy pipe break accident within a building and/or compartment thereof. Included in this category are the following:

A. Pipe Rupture and Miscellaneous Missile Loads

Pipe rupture loads consist of local loads on the structure generated by either jet impingement from, or the reaction of, a ruptured high-energy pipe, and missile impact due to or during a postulated pipe break (e.g., pipe whipping), all of which are applied as a static equivalent load that includes an appropriate dynamic factor. Pipe rupture effects are further discussed in section 3.6.

Miscellaneous missile loads are described in detail in section 3.5.

B. Abnormal Thermal Loads

Abnormal thermal loads are produced due to the temperature distribution through the wall and expansion of the liner during the loss-of-coolant accident.

C. Abnormal Pipe Expansion Loads

Abnormal pipe expansion loads consist of local forces on the structure caused by thermal expansion of piping during loss-of-coolant accident based on the most critical transient of steady-state condition.

3.8.4.3.2 Load Combinations

The following nomenclature is used in the load combinations.

D = Dead load

L = Appropriate live load

T_o = Normal thermal loads

H_o = Normal pipe expansion load

E = Operating basis earthquake load

W = Wind loads

E' = Design basis earthquake load

W_t = Tornado loads

R = Pipe rupture and miscellaneous missile loads

T_A = Abnormal thermal load

H_A = Abnormal pipe expansion load

The load combinations and factors for which the strength method is used are as follows:

A. Concrete

1. Normal Case

$$1.4D + 1.7L$$

2. Severe Environmental Case

$$1.25D + 1.25L \pm 1.25E/W + 1.0H_o$$

3. Severe Environmental Case

$$1.25D + 1.25L \pm 1.25E/W + 1.0T_o$$

4. Abnormal/Severe Environmental Case

$$1.0D + 1.0L \pm 1.25E + 1.0T_A + 1.0R + 1.0H_A$$

5. Abnormal/Severe Environmental Case

$$1.0D \pm 1.25E + 1.0T_A + 1.0R + 1.0H_A$$

6. Abnormal/Extreme Environmental Case

$$1.0D + 1.0L \pm 1.0E' + 1.0T_o + 1.0R + 1.25H_o$$

7. Abnormal/Extreme Environmental Case

$$1.0D + 1.0L \pm 1.0E' + 1.0T_A + 1.0R + 1.0H_A$$

8. Abnormal Case

$$1.0D + 1.0L + 1.0W_t + 1.0T_o + 1.25H_o$$

B. Structural Steel

Steel structures shall satisfy the following loading combinations without exceeding the allowable working stress for equations 1 and 3, a 33-1/3% increase in allowable working stress for equation 2, and 90% of the elastic yield capacity with full regard to elastic stability) for equations 4 through 7.

1. Normal Case

$$D + L$$

2. Severe Environmental Case

$$D + L + T_o + H_o + E \text{ or } W$$

3. Severe Environmental Case

(a) $D + L + H_o + E$

4. Abnormal/Extreme Environmental Case

$$D + L + R + T_o + H_o + E'$$

5. Abnormal/Extreme Environmental Case

$$D + L + R + T_A + H_A + E'$$

6. Abnormal Case

$$D + L + R + T_o + H_o$$

a. For structural elements carrying mainly earthquake forces only; e.g., struts and bracing.

7. Extreme Environmental Case

$$D + L + T_o + H_o + W_t$$

The stress levels reflected in the above load combinations are based upon the specific nature of the loading condition and the actual function of the structure. In general, for the operating basis seismic design, the structure is designed for elastic behavior using load factors based upon stresses significantly below the strength capacity in the case of concrete, and using working stress levels in the case of structural steel. For the accident basis design, stress analysis is based upon load levels at or just below the strength capacity for concrete, and just below the elastic capacity for structural steel.

3.8.4.4 Design and Analysis Procedures

The analysis procedures, including assumptions of load distribution and boundary conditions for other Seismic Category I structures, listed in paragraph 3.8.4.1, are based on conventional methods. The basic analytical techniques may be classified in two groups: methods involving simplifying assumptions, such as those found in beam theory, and those based on plate theories of different degree of approximation. The structures are designed to behave under loading as structural units, and are provided with connections capable of transmitting vertical and lateral loads by axial and diaphragm action to their foundations.

The structures are, in general, proportioned to maintain elastic behavior when subject to various combinations of dead, live, thermal, seismic, tornado, and accident loads. The upper limit of elastic behavior is considered to be the yield strength of the effective load-carrying structural material. The yield strength, F_y, for steel (including reinforcing steel) is considered to be the guaranteed minimum in appropriate ASTM specifications. The yield strength for reinforced concrete structures is considered to be the ultimate resisting capacity as calculated from the ACI-318-71 code. Reinforced concrete structures are designed for ductile behavior.

Under seismic loading, no plastic analysis is considered. Local yielding or erosion of barriers is considered permissible due to pipe rupture loading or missile impact, provided there is no general failure.

Structural steel is designed in accordance with basic working stress design methods as outlined in the 1970 AISC manual of steel construction. Increased allowable stresses are used for the accident condition.

The range of design variables that influence the results of the analyses is considered as follows:

A. Accuracy of design loads

B. Variation from assumed load distributions

C. Future changes in type or magnitude of loads

D. Frequency of loading and impact

E. Accuracy of analysis

F. Design accuracy of member sizing and proportioning

G. Reliability of specified material strengths

H. Construction dimensional variations

I. Function of structure

For the working stress design methods, the effects of the design variables are included in the values of the allowable stresses. For the strength design method, the effects of the design variables are accounted through load factors and capacity reduction factors.

Computer programs used in the analysis and design of reinforcing steel of the other seismic Category I structures are as follows:

	Computer Program
A. Auxiliary and Fuel Handling Building	
1. Compute Equivalent Stiffness Matrix	COPK (refer to subsection 3C.7)
2. Dynamic Analysis	SUPER SMIS (refer to subsection 3C.3)
3. Static Stress Analysis	SAP (refer to subsection 3C.5)
4. Design Reinforcing Steel	RESCOS (refer to subsection 3C.8)
B. Safety Equipment Building	
1. Dynamic and Static Stress Analyses	SAP
2. Design Reinforcing Steel	RESCOS
C. Intake Structure	
1. Static Stress Analysis	ICES STRUDL-II (refer to subsection 3C.6)
D. Electrical and Piping Gallery Structure	
1. Dynamic and Static Stress Analyses	SAP
2. Design Reinforcing Steel	OPTCON (refer to subsection 3C.9)

E. Diesel Generator Building and Condensate and Refueling Tank Enclosure Structure

1. Dynamic and Static Stress Analyses	SAP
2. Design Reinforcing Steel	OPTCON

3.8.4.5 Structural Acceptance Criteria

The limiting values of stress, strain, and gross deformations are established by the following criteria:

A. To maintain the structural integrity when subjected to the worst load combinations

B. To prevent structural deformations from disturbing the Seismic Category I equipment to the extent that it suffers a loss of function

The allowable stresses are those specified in the applicable codes. The stress contributions due to earthquake loading are included in the load combinations described in paragraph 3.8.4.3.

Structural deformations were not found to be a controlling criterion in the design of other Seismic Category I structures, listed in paragraph 3.8.4.1.

The tables listed below summarize (1) the governing load interactions and maximum capacity of principal reinforced concrete members (see category A) and, where applicable, (2) the governing combined stress ratios from the beam/column interaction equation for principal structural steel members (see category B).

	Reference Table Number	
Structure	Category[a] A	Category B
Auxiliary building	3.8-7	3.8-8
Fuel handling building	3.8-9	-
Safety-equipment building	3.8-10	3.8-11 3.8-12
Intake structure	3.8-13	-
Electrical and piping gallery structure	3.8-14	-

a. The ratio of the maximum capacity to the required capacity yields the safety margin.

Table 3.8-7
AUXILIARY BUILDING SUMMARY OF GOVERNING LOAD INTERACTIONS FOR PRINCIPAL REINFORCED CONCRETE MEMBERS (Sheet 1 of 5)

Description of Member	Location of Principal Member	Governing Load Combination Number(a)	Calculated Axial Load (P_u) and Flexural Load (M_u)		Maximum Flexural Interaction Capacity (M_u), Given Axial Load (P_u)(b)(c)	Calculated Shear Load (V_u)(b)	Maximum Shear Capacity (V_u)(b)
			P_u(b)	M_u(c)			
	Radwaste Area						
2'-6" thick wall - vertical reinforcement	Exterior south wall @ El 9'-0"	6	-36	161	278	--	--
2'6" x 73'6" wall - vertical reinforcement	Exterior south wall @ El 9'-0"	6	+2,929	8.31×10^4	33.2×10^4	--	--
2'-6" x 73'6" wall - vertical and horizontal reinforcement	Exterior south wall @ El 9'-0"	6	-10,388	9.41×10^4	60.2×10^4	10,975	11,719
3'-0" thick wall - vertical reinforcement	Interior wall @ El 9'-0"	6	-43	260	291	--	--
3'-0" x 28'-6" wall - vertical and horizontal reinforcement	Interior wall @ El 9'-0"	6	-4,477	1.21×10^4	9.7×10^4	5,709	6,387
3'-0" x 28'-6" wall - vertical reinforcement	Interior wall @ El 9'-0"	6	+685	1.07×10^4	5.5×10^4	--	--
1'-0" thick wall - vertical reinforcement	Interior wall @ El 9'-0"	6	-15	12	48	--	--
1'-0" x 11'-2" wall - vertical and horizontal reinforcement	Interior wall @ El 9'-0"	6	-655	161	4,645	408	615
1'-0" x 11'-2" wall - vertical reinforcement	Interior wall @ El 9'-0"	6	+229	142	1,606	--	--

a. Refer to paragraph 3.8.4.3.2.A for description of load combination number.
b. P_u and V_u are in kips; Sign convention for P_u: Compression (-), Tension (+).
c. M_u is in ft-k/ft

Table 3.8-7
AUXILIARY BUILDING SUMMARY OF GOVERNING LOAD INTERACTIONS FOR PRINCIPAL REINFORCED CONCRETE MEMBERS (Sheet 2 of 5)

Description of Member	Location of Principal Member	Governing Load Combination Number(a)	Calculated Axial Load (P_u) and Flexural Load (M_u)		Maximum Flexural Interaction Capacity (M_u), Given Axial Load (P_u)(b)(c)	Calculated Shear Load (V_u)(b)	Maximum Shear Capacity (V_u)(b)
			P_u(b)	M_u(c)			
2'-0" thick wall - vertical reinforcement	Interior wall @ El 9'-0"	6	-29	67	81	--	--
2'-0" thick wall - vertical reinforcement	Exterior south wall @ El 50'-0"	6	-16	93	98	--	--
2'-0" x 153'-6" wall - vertical and horizontal reinforcement	Exterior south wall @ El 50'-0"	6	-7,797	139,937	1,393,770	7,758	20,030
2'-0" x 153'-6" wall - vertical reinforcement	Exterior south wall @ El 50'-0"	6	+962	139,937	969,222	--	--
2'-6" thick wall - vertical reinforcement	Interior wall @ El 50'-0"	6	-19	121	183	--	--
2'-6" x 218'-6" wall - vertical and horizontal reinforcement	Interior wall @ El 50'-0"	6	-10,569	454,826	3,282,980	9,674	29,229
2'-6" x 218'-6" wall - vertical reinforcement	Interior wall @ El 50'-0"	6	-822	454,826	2,605,940	--	--
8'-0" thick basemat - E-W reinforcement	El 9'-0"	3	--	1,936	2,343	--	--
2'-0" thick slab - E-W reinforcement	El 24'-0"	3	--	60	73	--	--
	<u>Radwaste Storage Tank Area</u>						
2'-6" thick wall - vertical reinforcement	Exterior south wall @ El 9'-0"	6	-14	185	217	--	--
2'-6" x 32'-6" wall - vertical and horizontal reinforcement	Exterior south wall @ El 9'-0"	6	-3,802	4,568	11.18×10^4	1,499	4,699

Table 3.8-7
AUXILIARY BUILDING SUMMARY OF GOVERNING LOAD INTERACTIONS FOR PRINCIPAL REINFORCED CONCRETE MEMBERS (Sheet 3 of 5)

Description of Member	Location of Principal Member	Governing Load Combination Number	Calculated Axial Load (P_u) and Flexural Load (M_u)		Maximum Flexural Interaction Capacity (M_u), Given Axial Load (P_u)(b)(c)	Calculated Shear Load (V_u)(b)	Maximum Shear Capacity (V_u)(b)
			P_u(b)	M_u(c)			
2'-6" x 32'-6" wall - vertical reinforcement	Exterior south wall @ El 9'-0"	6	+2,477	4,568	5.18×10^4	--	--
4'-0" thick wall - vertical reinforcement	Exterior east wall @ El 9'-0"	6	--	907	1,044	--	--
2'-0" thick wall - vertical reinforcement	Exterior east wall @ El 37'-0"	6	-10	60	89	--	--
2'-0" x 60'-0" wall - vertical and horizontal reinforcement	Exterior east wall @ El 37'-0"	6	-2,133	14,629	168,267	4,470	6,639
2'-0" x 60'-0" wall - vertical reinforcement	Exterior east wall @ El 37'-0"	6	+863	732	107,052	--	--
2'-6" thick wall - vertical reinforcement	Interior wall @ El 37'-0"	6	-12	103	142	--	--
2'-6" x 42'-0" wall - vertical and horizontal reinforcement	Interior wall @ El 37'-0"	6	-3,427	7,526	110,556	3,072	4,353
2'-6" x 42'-0" wall - vertical reinforcement	Interior wall @ El 37'-0"		+2,002	7,526	30,637	--	--
8'-0" thick basemat - E-W reinforcement	El 9'-0"	3	--	991	1,167	--	--
2'-0" thick slab - N-S reinforcement	El 37'-0"	3	--	75	96	--	--
	<u>Control Area</u>						
2'-6" thick wall - vertical reinforcement	Exterior west wall @ El 9'-0"	6	-239	450	458	--	--
2'-6" x 218'-6" wall - vertical and horizontal reinforcement	Exterior west wall @ El 9'-0"	6	-23,700	901,000	4.13×10^6	27,979	27,973

Table 3.8-7
AUXILIARY BUILDING SUMMARY OF GOVERNING LOAD INTERACTIONS FOR PRINCIPAL REINFORCED CONCRETE MEMBERS (Sheet 4 of 5)

Description of Member	Location of Principal Member	Governing Load Combination Number[(a)]	Calculated Axial Load (P_u) and Flexural Load (M_u)		Maximum Flexural Interaction Capacity (M_u), Given Axial Load (P_u)[(b)(c)]	Calculated Shear Load (V_u)[(b)]	Maximum Shear Capacity (V_u)[(b)]
			P_u[(b)]	M_u[(c)]			
2'-6" x 218'-6" wall - vertical reinforcement	Exterior west wall @ El 9'-0"	6	-1,198	879,750	2.78 x 10^6	--	--
2'-0" x 60'-6" wall - vertical and horizontal reinforcement	Interior wall @ El 9'-0"	6	-7,890	14,730	268,223	3,324	5,072
2'-0" x 60'-6" wall - vertical reinforcement	Interior wall @ El 9'-0"	6	+3,310	14,730	52,982	--	--
2'-0" thick wall - vertical reinforcement	Exterior west wall @ El 50'-0"	6	-24	61	107	--	--
2'-0" x 2'3'-0" wall - vertical and horizontal reinforcement	Exterior west wall @ 50'-0"	6	-13,058	306,431	2.6 x 10^6	13,241	19,852
2'-0" x 219'-0" wall - vertical reinforcement	Exterior west wall @ El 50'-0"	6	-742	306,431	1.73 x 10^6	--	--
2'-0" thick wall - vertical reinforcement	Exterior south wall @ El 50'-0"	6	-24	52	127	--	--
2'-0" x 74'-6" wall - vertical and horizontal reinforcement	Exterior south wall @ El 50'-0"	6	-7,430	26,315	363,666	3,503	6,982
2'-0" x 74'-6" wall - vertical reinforcement	Exterior south wall @ El 50'-0"	6	+2,653	26,315	121,403	--	--
8'-0" thick basemat - E-W reinforcement	El 9'-0"	3	--	5,736	5,765	--	--
1'-0" thick slab - E-W reinforcement	El 30'-0"	3	--	33	38	--	--
1'-0" thick slab - E-W reinforcement	El 50'-0"	3	--	41	41	--	--

Table 3.8-7
AUXILIARY BUILDING SUMMARY OF GOVERNING LOAD INTERACTIONS FOR PRINCIPAL REINFORCED CONCRETE MEMBERS (Sheet 5 of 5)

Description of Member	Location of Principal Member	Governing Load Combination Number[a]	Calculated Axial Load (P_u) and Flexural Load (M_u)		Maximum Flexural Interaction Capacity (M_u), Given Axial Load (P_u)[b][c]	Calculated Shear Load (V_u)[b]	Maximum Shear Capacity (V_u)[b]
			P_u[b]	M_u[c]			
	Penetration Area						
2'-0" x 45'-6" wall - vertical and horizontal reinforcement	Exterior west wall @ El 9'-0"	6	-6,730	9,140	173,522	3,426	6,127
2'-0" x 45'-6" wall - vertical reinforcement	Exterior west wall @ El 9'-0"	6	+2,360	8,924	51,464	--	--
8'-0" thick basemat - N-S reinforcement	El 9'-0"	3	--	3,927	4,181	--	--
1'-0" thick slab - E-W reinforcement	El 30'-0"	3	--	44	48	--	--

Table 3.8-8
AUXILIARY BUILDING SUMMARY OF GOVERNING COMBINED STRESS RATIOS FROM THE BEAM/COLUMN INTERACTION EQUATION FOR PRINCIPAL STRUCTURAL STEEL MEMBERS (Sheet 1 of 2)

Description of Principal Members	Location of Principal Members	Governing Load Combination Number[(a)]	Combined Stress Ratio ($\leq$1.0)
	Penetration Area - Fuel Handling Building Site		
W 24 x 68	Main girder @ El 30'-0"	2	0.89
W 12 x 65 with 3/4 x 10 plate	Knee brace @ El 30'-0"	2	0.89
W 24 x 110	Main girder @ El 45'-0"	2	0.96
W 24 x 84	Main girder @ El 6e'-6"	2	0.83
W 12 x 58	Knee brace @ El 45'-0"	2	0.76
	Column @ El 63'-6"	2	0.74
	Column @ El 9'-0"	2	0.99
	Penetration Area - Radwaste Side		
W 30 x 116 with 1-1/2 x 9 plate	Girder @ El 30'-0"	2	0.83
W 12 x 65	Knee brace @ El 30'-0"	2	0.92
W 30 x 116 with 1-1/2 x 9 plate	Girder @ El 45'-0"	2	0.95
W 12 x 65	Knee brace @ El 45'-0"	2	0.80
W 14 x 127	Column @ El 63'-6"	2	0.82
W 14 x 228	Column @ El 45'-0"	2	0.65
	Radwaste Storage Tank Area		
W 27 x 145 with 12 x 1/2 plate	Girder @ El 37'-0"	1	0.77
W 36 x 194 with 10 x 1-1/2 plate	Girder @ El 63'-6"	2	0.85

a. Refer to paragraph 3.8.3.3.2.B for description of load combination number.

Table 3.8-8
AUXILIARY BUILDING SUMMARY OF GOVERNING COMBINED STRESS RATIOS FROM THE BEAM/COLUMN INTERACTION EQUATION FOR PRINCIPAL STRUCTURAL STEEL MEMBERS (Sheet 2 of 2)

Description of Principal Members	Location of Principal Members	Governing Load Combination Number[(a)]	Combined Stress Ratio ($\leq$1.0)
	Radwaste Area		
W 16 x 64	Girder @ El 24'-0"	2	0.5
C 10 x 15.3	Staircase stringer	2	0.91
	Control Area		
W 30 x 190 with 13 x 3/4 plate	Main girder @ El 30'-0"	2	0.75
W 21 x 55 with 7 x 3/4 plate	Floor beam @ El 30'-0"	2	0.67
W 33 x 220	Main girder @ El 70'-0"	2	0.99
W 36 x 245	Top chord of truss @ El 70'-0"	2	0.90
W 36 x 300	Bottom chord of truss @ El 50'-0"	2	1.00
W 14 x 287	Diagonal of truss @ El 50'-0"	2	0.95
W 14 x 605	Column supporting truss @ El 9'-0"	2	0.86
W 14 x 426	Column supporting truss @ El 70-0"	2	0.91
W 14 x 426	Column @ El 9'-0"	2	0.93
W 14 x 119	Column @ El 85'-0"	2	0.96

Table 3.8-9
FUEL HANDLING BUILDING SUMMARY OF GOVERNING LOAD INTERACTIONS
FOR PRINCIPAL REINFORCED CONCRETE MEMBERS (Sheet 1 of 2)

Description of Member	Location of Principal Member	Governing Load Combination Number(a)	Calculated Axial Load P_u and Flexural Load (M_u)		Maximum Flexural Interaction Capacity (M_u), Given Axial Load (P_u)	Calculated Shear Load	Maximum Shear Capacity
			P_u (b)	M_u (c)	M_u (c)	(V_u)(b)	(V_u)(b)
2'-6" x 103'-0" wall - vertical and horizontal reinforcement	Exterior east wall @ El 17'-6"	6	-7,413	12.44×10^4	83.43×10^4	5,437	12,840
2'-6" x 103'-0" wall - vertical reinforcement	Exterior east wall @ El 17'-6"	6	-1,362	10.08×10^4	66.21×10^4	--	--
5'-0" x 79'-6" wall - vertical and horizontal reinforcement	West wall @ 17'-6"	6	-11,181	19.4×10^4	110.37×10^4	8,743	29,958
5'-0" x 79'-6" wall - vertical reinforcement	West wall @ El 17'-6"	6	-1,463	15.72×10^4	87.9×10^4	--	--
5'-0" x 36'-10" wall - vertical and horizontal reinforcement	Interior wall @ El 17'-6"	6	-6,962	1.61×10^5	2.54×10^5	5,410	16,856
5'-0" x 36'-10" wall - vertical reinforcement	Interior wall @ El 17'-6"	6	-1,247	1.49×10^5	1.95×10^5	--	--
2'-6" x 49'-6" wall - vertical and horizontal reinforcement	Exterior south wall @ El 30'-0"	6	-5,586	45,294	204,673	2,102	5,154
2'-6" x 49'-6" wall - vertical reinforcement	Exterior south wall @ El 30'-0"	6	+3,013	45,294	70,271	--	--
2'-6" x 103'-6" wall - vertical and horizontal reinforcement	West wall @ El 63'-6"	6	-3,457	56,885	602,355	3,043	10,216

a. Refer to paragraph 3.8.4.3.2.A for description of load combination number
b. P_u and V_u are in kips; sign convention for P_u: Compression (-), Tension (+)
c. M_u is in ft-k/ft

Table 3.8-9
FUEL HANDLING BUILDING SUMMARY OF GOVERNING LOAD INTERACTIONS
FOR PRINCIPAL REINFORCED CONCRETE MEMBERS (Sheet 2 of 2)

Description of Member	Location of Principal Member	Governing Load Combination Number(a)	Calculated Axial Load P_u and Flexural Load (M_u)		Maximum Flexural Interaction Capacity (M_u), Given Axial Load (P_u)	Calculated Shear Load	Maximum Shear Capacity
			P_u (b)	M_u (c)	M_u (c)	(V_u) (b)	(V_u) (b)
2'-6" x 103'-6" wall - vertical reinforcement	West wall @ El 63'-6"	6	+75	56,885	473,948	--	--
2'-6" thick wall - vertical reinforcement	Exterior south wall (crane location)	6	-122	158	332		
7'-0" thick basemat - E-W reinforcement	Basemat in pool area	7	-599	2,277	2,716		
7'-0" thick basemat - E-W reinforcement	Basemat in pool area	7	-527	2,604	2,660		
8'-0" thick basemat - E-W reinforcement	Basemat in penetration area	6	--	3,533	5,500	224	304
5'-0" thick wall - vertical reinforcement	Spent fuel pool west wall	7	-589	952	1,322		
4'-0" thick wall - vertical reinforcement	Spent fuel pool south wall	7	-404	445	947		
5'-0" thick wall - vertical reinforcement	Spent fuel pool west wall	7	--	666	674		
2'-6" thick wall - vertical reinforcement	Exterior east wall @ El 17'-6"	6	-24	158	223		

Table 3.8-10

SAFETY EQUIPMENT BUILDING SUMMARY OF GOVERNING LOAD INTERACTIONS FOR PRINCIPAL REINFORCED CONCRETE MEMBERS (Sheet 1 of 2)

Description of Principal Members	Location of Principal Members	Governing Load Combination Number(a)	Calculated Axial Load (P_u) and Flexural Load (M_u)		Maximum Flexural Interaction Capacity (M_u), Given Axial Load P_u
			P_u (b)	M_u (c)	M_u (c)
Basemat slab - E-W reinforcement	Pump rooms El (-) 17'-6: Safety injection area	7	9	307	442
Basemat slab - E-W reinforcement	Pump rooms El (-) 5'-3" Component cooling water area	7	9	196	442
West wall - vertical reinforcement	Pump room Between El (-) 15'-6" & (+) 8'-0"	4	21	131	160
West wall - vertical reinforcement	Piping room Between El (-) 5'-3" & (+) 8'-0"	7	49	70	86
Center wall - vertical reinforcement	Shutdown heat exchanger room walls Between El (-) 5'-6" & (+) 8'-0"	4	10	100	118
East wall - vertical reinforcement	Pump rooms Between El (-) 15'-6" & (+) 8'-0"	7	117	68	95
North wall - vertical reinforcement	Shutdown heat exchanger room Between El (-) 15'-6" & (+) 8'-0"	4	25	83	108

a. Refer to paragraph 3.8.4.3.2.A for description of load combination number.
b. P_u is in kips.
c. M_u is ft-k/ft.

Table 3.8-10
SAFETY EQUIPMENT BUILDING SUMMARY OF GOVERNING LOAD INTERACTIONS FOR PRINCIPAL REINFORCED CONCRETE MEMBERS (Sheet 2 of 2)

Description of Principal Members	Location of Principal Members	Governing Load Combination Number (a)	Calculated Axial Load (P_u) and Flexural Load (M_u)		Maximum Flexural Interaction Capacity (M_u), Given Axial Load P_u
			P_u (b)	M_u (c)	M_u (c)
South wall - vertical reinforcement	Piping room Between El (-) 17'-6" & (+) 8'-0"	7	30	88	101
Slab - E-W reinforcement	Piping room El (+) 8'-0"	4	21	104	115
East wall - vertical reinforcement	Chemical storage tank room Between (+) 8'-0" & (+) 30'-6"	4	117	176	284
Slab - E-W reinforcement	Electrical cable tray tunnel (+) 30'-6"	7	12	88	117
West wall - vertical reinforcement	Cable tray tunnel room Between (+) 30'-6" & (+) 50'-6"	2	19	36	86
Slab - E-W reinforcement	Main steam line & feedwater line support slab (+) 50'-6"	4	25	225	268
Slab - N-S reinforcement	Main steam line and feedwater line support slab (+) 50'-6"	4	12	158	284

Table 3.8-11
SAFETY EQUIPMENT BUILDING SUMMARY OF GOVERNING COMBINED STRESS RATIOS
FROM THE BEAM/COLUMN INTERACTION EQUATION
FOR PRINCIPAL STRUCTURAL STEEL MEMBERS

Description of Principal Members	Location of Principal Members	Governing Load Combination Number[(a)]	Combined Stress Ratio ($\leq$1.0)
Low pressure safety injection pump support columns	El (-) 23'-6" Pump rooms	4	0.245
Containment spray pump support columns	El (-) 17'-6" Pump rooms	4	0.245
Shutdown heat exchanger support beams	El 8'-0" Heat exchanger rooms	4	0.432

a. Refer to paragraph 3.8.4.3.2.B for description of load combination number.

Table 3.8-12
SAFETY EQUIPMENT BUILDING SUMMARY OF DUCTILITY RATIOS
FOR PIPE WHIP RESTRAINTS

Description of Principal Members	Location of Principal Members	Governing Load Combination Number[(a)]	Ductility Ratios		Remark
			Actual	Allowable	
Pipe whip restraint	Main steam line restraint	4	71	100	Tension
Pipe whip restraint	Feedwater line restraint	4	41	100	Tension

a. Refer to paragraph 3.8.4.3.2.B for description of load combination number.

Table 3.8-13
INTAKE STRUCTURE SUMMARY OF GOVERNING LOAD INTERACTIONS FOR
PRINCIPAL REINFORCED CONCRETE MEMBERS (Sheet 1 of 5)

Description of Principal Members	Location of Principal Members	Governing Load Combination Number[a]	Calculated Axial Load (P_u) and Flexural Load (M_u)		Maximum Flexural Interaction Capacity (M_u) Given Axial Load[c]	Remarks
			P_u[b]	M_u[c]		
Base slab	Recirculation system: crossover box	7	16	169	230	
North wall	Recirculation system: crossover box	7	46	131	157	
South wall	Recirculation system: crossover box	7	29	471	548	
Roof slab	Recirculation system: crossover box	7	29	115	151	
Base slab	Recirculation system: seal well	7	97	491	516	
Walls: north above (+)10'-0"	Recirculation system: seal well	7	18	220	257	
Walls: north below (+)10'-0"	Recirculation system: seal well	7	29	471	548	
Walls: south above (-)6'-0"	Recirculation system: seal well	1	26	330	361	
Walls: south below (-)6'-0"	Recirculation system: seal well	1	139	467	651	
Roof slab	Recirculation system: seal well	7	49	248	264	
Base slab	Intake conduit	7	97	491	516	
Walls: north	Intake conduit	1	139	467	651	

a. Refer to paragraph 3.8.4.3.2.A for description of load combination number.
b. P_u is in kips.
c. M_u is in ft-k/ft except as noted.

Table 3.8-13
INTAKE STRUCTURE SUMMARY OF GOVERNING LOAD INTERACTIONS FOR PRINCIPAL REINFORCED CONCRETE MEMBERS (Sheet 2 of 5)

Description of Principal Members	Location of Principal Members	Governing Load Combination Number(a)	Calculated Axial Load (P_u) and Flexural Load (M_u)		Maximum Flexural Interaction Capacity (M_u) Given Axial Load(c)	Remarks
			P_u(b)	M_u(c)		
Walls south	Intake conduit	1	84	262	423	
Roof slab	Intake conduit	7	35	224	257	
Base slab	Discharge conduit	7	97	491	516	
Walls	Discharge conduit	1	84	262	639	
Roof slab	Discharge conduit	7	35	224	257	
Base slab	Transition	7	1	350	389	
Walls: north	Transition	7	47	358	544	
Walls: center	Transition	7	123	78	582	
Walls: south	Transition	7	79	378	592	
Roof slab	Transition	7	69	335	396	
Crane column supports	Salt water tunnel area El (-)9'-0" to El (+)7'-0"	7	1,640K	5,667 Ft-K	5,857 Ft-K	
Crane column supports	Salt water tunnel area El (+)7'-0" to (+)30'-0"	7	513K	2,802 Ft-K	3,240 Ft-K	Axial tension
Crane column supports	Tsunami wall area	7	426K	302 Ft-K	843 Ft-K	Axial tension
Walls	Stop-gate structure	7	0	650	1,049	

Table 3.8-13
INTAKE STRUCTURE SUMMARY OF GOVERNING LOAD INTERACTIONS FOR PRINCIPAL REINFORCED CONCRETE MEMBERS (Sheet 3 of 5)

Description Principal Members	Location of Principal Members	Governing Load Combination Number(a)	Calculated Axial Load (P_u) and Flexural Load (M_u)		Maximum Flexural Interaction Capacity (M_u) Given Axial Load(c)	Remarks
			P_u(b)	M_u(c)		
Walls	Recirculating gate structures	7	14	117	117	
Slab	Slab El (-)26'-0", thickness = 4' screen well area	1	5	310	313	
Slab	Slab El (-)26'-0", thickness = 5' screen well area	7	29	68	333	Axial tension
Slab	Slab El (-)26'-0", thickness = 7'-3" screen well area	7	30	177	1,127	
Slab	Slab El (+)9'-0" screen well area	7	38	30	90	Axial tension
Slab	Slab El (+)35'-0" screen well area	7	11	74	83	
Slab	(+)16'-0" deck screen well area	1	0	43	147	
Beams	(+)16'-0" deck - traveling water screen area	7	49	1,252	2,088	Axial tension
Beams	(+)16'-0" deck - fish elevator area	1	351	661	4,797	Axial tension
Walls	Center wall	1	40	360	421	Axial tension
Walls	Baffle walls	7	38	31	95	

Table 3.8-13

INTAKE STRUCTURE SUMMARY OF GOVERNING LOAD INTERACTIONS FOR PRINCIPAL REINFORCED CONCRETE MEMBERS (Sheet 4 of 5)

Description of Principal Members	Location of Principal Members	Governing Load Combination Number(a)	Calculated Axial Load (P_u) and Flexural Load (M_u)		Maximm Flexural Interaction Capacity (M_u) Given Axial Load(c)	Remarks
			P_u(b)	M_u(c)		
Slab	Pump slab El (-)7'-5"	7	90	253	347	
Slab	Pump well slab El (-)22'-9"	7	14	243	335	Axial tension
Walls - interior	Pump well area	7	52	250	269	
Walls - west exterior	Pump well area	7	24	348	544	
Walls - east exterior	Pump well area	7	36	53	313	
Walls	Tsunami walls	7	0	190	235	
Base slab	Salt-water tunnel area Slab at El (-)9'-0"	7	100	263	446	
Walls	Salt-water tunnel area West walls El (-)9' to El (+)4'	7	23	434	514	
Walls	Salt-water tunnel area East walls El (-)9' to El (+)4'	7	64	228	270	Axial tension
Walls	Salt-water tunnel area West walls El (+)7'-0" to (+)27'-6"	7	2	38	337	Axial tension
Walls	Salt-water tunnel area East walls El (+)7'-0" to (+)27'-6"	7	90	20	238	Axial tension

Table 3.8-13
INTAKE STRUCTURE SUMMARY OF GOVERNING LOAD INTERACTIONS FOR PRINCIPAL REINFORCED CONCRETE MEMBERS (Sheet 5 of 5)

Description of Principal Members	Location of Principal Members	Governing Load Combination Number[(a)]	Calculated Axial Load (P_u) and Flexural Load (M_u)		Maximum Flexural Interaction Capacity (M_u) Given Axial Load[(c)]	Remarks
			P_u[(b)]	M_u[(c)]		
Slab	Salt-water tunnel area Slab El (+)7'-0"	7	25	283	375	
Slab	Salt-water tunnel area Slab El (+)30'-0"	7	17	172	207	

Table 3.8-14
ELECTRICAL AND PIPING GALLERY STRUCTURE SUMMARY OF GOVERNING LOAD INTERACTIONS FOR PRINCIPAL REINFORCED CONCRETE MEMBERS

Description of Principal Members	Location of Principal Members	Governing Load Combination Number[a]	Calculated Axial Load (P_u) and Flexural Load (M_u)		Maximum Flexural Interaction Capacity (M_u), Given Axial Load P_u[c]
			P_u[b]	M_u[c]	
3'-0" thick basemat N-S reinforcement	Basemat at El (-) 2'-6"	4	87.56	138.78	177.56
3'-0" thick basemat E-W reinforcement	Basemat at El (-) 2'-6"	4	70.06	177.18	200.50
2'-0" thick slab E-W reinforcement	Slab at El 9'-6"	2	23.92	61.53	118.62
1'-6" thick slab N-S reinforcement	Floor at El 30'-6"	5	142.01	35.39	46.22
1'-6" thick slab E-W reinforcement	Floor at El 30'-6"	4	51.71	41.00	64.84
1'-8" thick slab N-S reinforcement	Floor at El 48'-6"	5	81.87	43.98	91.11
1'-6" thick wall horizontal reinforcement	North end exterior wall	7	81.41	39.49	74.99
1'-6" thick wall vertical reinforcement	North end exterior wall	6	79.75	38.66	68.57
1'-6" thick wall vertical reinforcement	Mid-north exterior wall	5	115.32	38.20	69.47
1'-6" thick wall vertical reinforcement	Mid-south exterior wall	4	79.77	50.95	60.99
2'-6" thick wall horizontal reinforcement	South end exterior wall	3	77.81	139.11	162.61
2'-6" thick wall vertical reinforcement	South end exterior wall	7	114.18	76.27	153.03
2'-0" thick wall horizontal reinforcement	West end exterior wall	5	115.20	56.17	152.10
2'-0" thick wall vertical reinforcement	West end exterior wall	5	42.43	53.26	152.96
1'-6" thick wall horizontal reinforcement	Mid-east exterior wall	4	58.43	42.59	81.39
1'-6" thick wall vertical reinforcement	Mid-east exterior wall	5	62.79	43.85	77.23
1'-6" thick wall vertical reinforcement	Interior wall running N-S direction	2	19.65	20.58	58.63

a. Refer to paragraph 3.8.4.3.2.A for description of load combination number.
b. P_u is in kips.
c. M_u is in ft-k/ft.

3.8.4.6 Materials, Quality Control, and Special Construction Techniques

The following basic materials are used in the construction of the Seismic Category I structures listed in paragraph 3.8.4.1.

A.	Concrete		$f'c$ (lb/in.^{2})	=	4,000
B.	Reinforcing steel deformed bars	ASTM A-615 Grade 60	f_y (lb/in.^{2})	=	60,000
C.	Structural and miscellaneous steel rolled shapes, bars, and plates	ASTM A-36	f_y (lb/in.^{2})	=	36,000
	High-strength bolts	ASTM A-325	f_y (lb/in.^{2})	=	81,000 to 92,000 (Varies depending on diameter of bolts)
		ASTM A-449	f_y (lb/in.^{2})	=	58,000 to 92,000 (Varies depending on diameter of bolts)
		ASTM A-490	f_y (lb/in.^{2})	=	130,000 minimum
	Anchor bolts	ASTM A-307	f_t (lb/in.^{2})	=	60,000 minimum
	Stainless steel	ASTM A-167 Type 304	f_y (lb/in.2)	=	30,000
		STM A-240 Type 304L	f_y (lb/in.^{2})	=	25,000
	Stainless steel	ASTM A-276	f_y (lb/in.^{2})	=	30,000
	Bars and Shapes	Type 304			
	Stainless steel	ASTM A-554	f_y (lb/in.^{2})	=	30,000
	Tubing	Type 304			
	Stainless steel	ASTM A-193	f_y (lb/in.^{2})	=	30,000
	Bolts, nuts and threaded studs	Grade B8			
	Insert plates	ASTM A-36	f_y (lb/in.^{2})	=	36,000

The materials and quality control procedures are described in paragraph 3.8.1.6.

The other Seismic Category I structures listed in paragraph 3.8.4.1 are built of reinforced concrete and structural steel, using proven methods common to heavy industrial construction. No special construction techniques have been employed in the construction of these structures.

3.8.4.7 Testing and Inservice Inspection Requirements

Testing and inservice surveillance are not required for Seismic Category I structures other than containment, and no formal program of testing and inservice surveillance is planned.

3.8.5 FOUNDATIONS

3.8.5.1 Description of the Foundations

3.8.5.1.1 Containment

The containment foundation is a conventionally reinforced, circular concrete mat, 9 feet thick with a diameter of 184 feet, bearing directly on the San Mateo formation. The reactor cavity is located near the center of the mat and forms an integral part of the foundation. Figure 3.8-1 shows the relative position of the two containment foundations.

Figure 3.8-2 shows cross-sections of the containment base slab.

The internal structures that support the large equipment, such as steam generators and reactor coolant pumps, are anchored to the base slab in order to transfer the loads. Figure 3.8-21 and 3.8-22 show a typical detail of anchorage to the base slab for the steam generator and reactor coolant pumps.

Figure 3.8-2 shows the reinforcing pattern at the junction of the base slab and containment wall.

3.8.5.1.2 Auxiliary Building

The auxiliary building foundation is a reinforced concrete slab 8 feet thick, 280 feet long, and approximately 221 feet wide, bearing directly on the San Mateo formation. A piping gallery extending 11 feet below the bottom of the basemat in the control area is an integral part of the foundation.

Refer to figure 3.8-28 for location of auxiliary building foundation in relation to other Seismic Category I structures.

Figures 3.8-29 and 3.8-30 show typical base slab sections and foundation details.

3.8.5.1.3 Fuel Handling Building

The fuel handling foundation is a reinforced concrete slab which varies in thickness from 4 feet 6 inches to 8 feet, and is 134 feet 6 inches long by 87 feet - 6 inches wide, bearing directly on the San Mateo formation.

Refer to figures 3.8-31, 3.8-32 and 3.8-33 for location of the fuel handling foundation in relation to other Seismic Category I structures.

Figure 3.8-34 shows typical base slab and foundation details.

3.8.5.1.4 Safety Equipment Building

The safety equipment building foundation is a stepped reinforced concrete slab which is 4 feet thick, 174 feet long, and 74 feet wide, bearing directly on the San Mateo formation.

Refer to figure 3.8-35 for location of the safety equipment building in relation to other Seismic Category I structures.

Figure 3.8-36 shows typical base slab and foundation details.

3.8.5.1.5 Intake Structure

The intake structure foundation is a reinforced concrete slab, which is 4 feet thick, 119 feet 6 inches long, and 109 feet 10 inches wide, bearing directly on the San Mateo formation.

Refer to figure 3.8-37 for location of the intake structure foundation in relation to other Seismic Category I structures.

Figure 3.8-38 shows typical base slab and foundation details.

3.8.5.1.6 Electrical and Piping Gallery Structure

The electrical/piping junction structure is a partially buried conventional reinforced concrete shear-wall structure. The structure provides a transition area for Seismic Category I piping and electrical cable from the underground tunnels and duct runs into the safety-equipment building. The overall plan dimension of the structure is 85 x 67 feet with a maximum height of 54 feet. The interior of the structure is a maze of partial floor slabs, partition walls, and vertical risers. Due to the physical proximity to other structures, the embedment characteristics vary on each side of the structure. The resulting structure exhibits geometric eccentricity between the center of mass and the center of rigidity at the various elevations within the structure and at the soil-structure interface.

Refer to figure 3.8-39 for location of the electrical and piping gallery structure foundation in relation to other Seismic Category I structures.

Figure 3.8-40 shows typical base slab and foundation details.

3.8.5.1.7 Condensate and Refueling Tank Enclosure Structure

A description of this foundation will be provided in an Amendment to the FSAR by approximately March 1977.

3.8.5.1.8 Diesel Generator Building

A description of this foundation will be provided in an Amendment to the FSAR by approximately November 1977.

3.8.5.2 Applicable Codes, Standards, and Specifications

The applicable codes, standards, specifications, regulatory guides, and other documents used in the structural design, fabrication, and construction of foundations are covered in the following paragraphs.

Containment, paragraph 3.8.1.2

Internal Structures, paragraph 3.8.3.2

Other Seismic Category I Structures, paragraph 3.8.4.2

3.8.5.3 Loads and Load Combinations

Containment foundation loads and loading combinations are discussed in paragraph 3.8.1.3.

Foundation loads and loading combinations for other Seismic Category I structures are discussed in paragraph 3.8.4.3.

3.8.5.4 Design and Analysis Procedures

Design and analysis procedures used, including the computer programs employed, in the design of foundations are discussed in paragraph 3.8.1.4 for the containment, in paragraph 3.8.3.4 for the internal structures, which include the major concrete support structures, and in paragraph 3.8.4.4 for the other Seismic Category I structures.

3.8.5.5 Structural Acceptance Criteria

The foundations of all Seismic Category I structures are designed to meet the same structural acceptance criteria as the structures themselves. These criteria are discussed in paragraphs 3.8.1.5, 3.8.3.5, and 3.8.4.5.

A minimum factor of safety of 1.5 against overturning is maintained for all structures and supports as shown in table 3.8-15. The procedure used to determine the stability ratio against structural overturning is discussed in detail in section 4.4 of BC-TOP-4-A.[6]

The foundation bearing pressures shown in table 3.8-15, when compared with the allowable bearing pressures, show that the foundation medium can safely support the pressures caused by overturning moments. The ratio of the allowable bearing pressure to the actual bearing pressure yields the safety margin.

3.8.5.6 Materials, Quality Control, and Special Construction Techniques

The foundations and equipment supports are built of reinforced concrete using conventional methods for heavy industrial construction. The description of the materials, and the quality control procedures, as well as special construction techniques for foundations, are the same as those discussed in paragraphs 3.8.1.6, 3.8.3.6, and 3.8.4.6, and chapter 17.0.

3.8.5.7 Testing and Inservice Inspection Requirements

Testing and inservice surveillance are not required and are not planned for foundations of structures or for concrete supports. A discussion of the test program that serves as the basis for the Soils Investigations and Foundation Report may be found in section 2.5, Geology, Seismology and Geotechnical Engineering.

Table 3.8-15
SUMMARY OF ACTUAL AND ALLOWABLE FOUNDATION BEARING PRESSURES, SETTLEMENTS, AND FACTORS OF SAFETY AGAINST OVERTURNING FOR SEISMIC CATEGORY I STRUCTURES

Structure	Foundation Medium	Foundation Bearing Pressure (D+L+Seismic) (k/ft^2)	Foundation Allowable Bearing Pressure (k/ft^2)	Estimated Short-Term Settlements of Structure (in.)	Estimated Long-Term Settlements of Structure (in.)	Minimum Factor of Safety (1.5) Against Overturning	Remarks
Containment	Undistrubed natural San Mateo sand	18	60	0.21	0.3	40	
Auxiliary Building	Undisturbed natural San Mateo sand	15	25 (See Remarks)	See paragraph 2.5.4.10	See paragraph 2.5.4.10	150	Foundation allowable value is lower bound dictated by shear capacity of soil, neglecting actual horizontal extent of basemats.
Fuel Handling Building	Undisturbed natural San Mateo sand	21	(See Remarks)	See paragraph 2.5.4.10	See paragraph 2.5.4.10	10	Foundation allowable value is lower bound dictated by shear capacity of soil, neglecting actual horitontal extent of basemats.
Safety Equipment Building	Undisturbed natural San Mateo sand	(DBE) 9.022 (OBE) 7.326	47.5	0.28	0.40	Not applicable (See Remarks)	More than 2/3 of structure is embedded in soil
Intake Structure	Undisturbed natural San Mateo sand	7.0	25+	-	-	Not applicable	
Electrical and Piping Gallery Structure	Undisturbed natural San Mateo sand	(DBE) 11.154 (OBE) 10.497	56.26	0.28	0.40	Not applicable (See Remarks)	Approximately 2/3 of structure is embedded
Diesel Generator Building							
Condensate Refueling Tank Closure Structure							

REFERENCES

1. "Prestressed Concrete Nuclear Reactor Containment Structures," BC-TOP-5, Revision 1, Bechtel Power Corporation, San Francisco, California, December 1972.

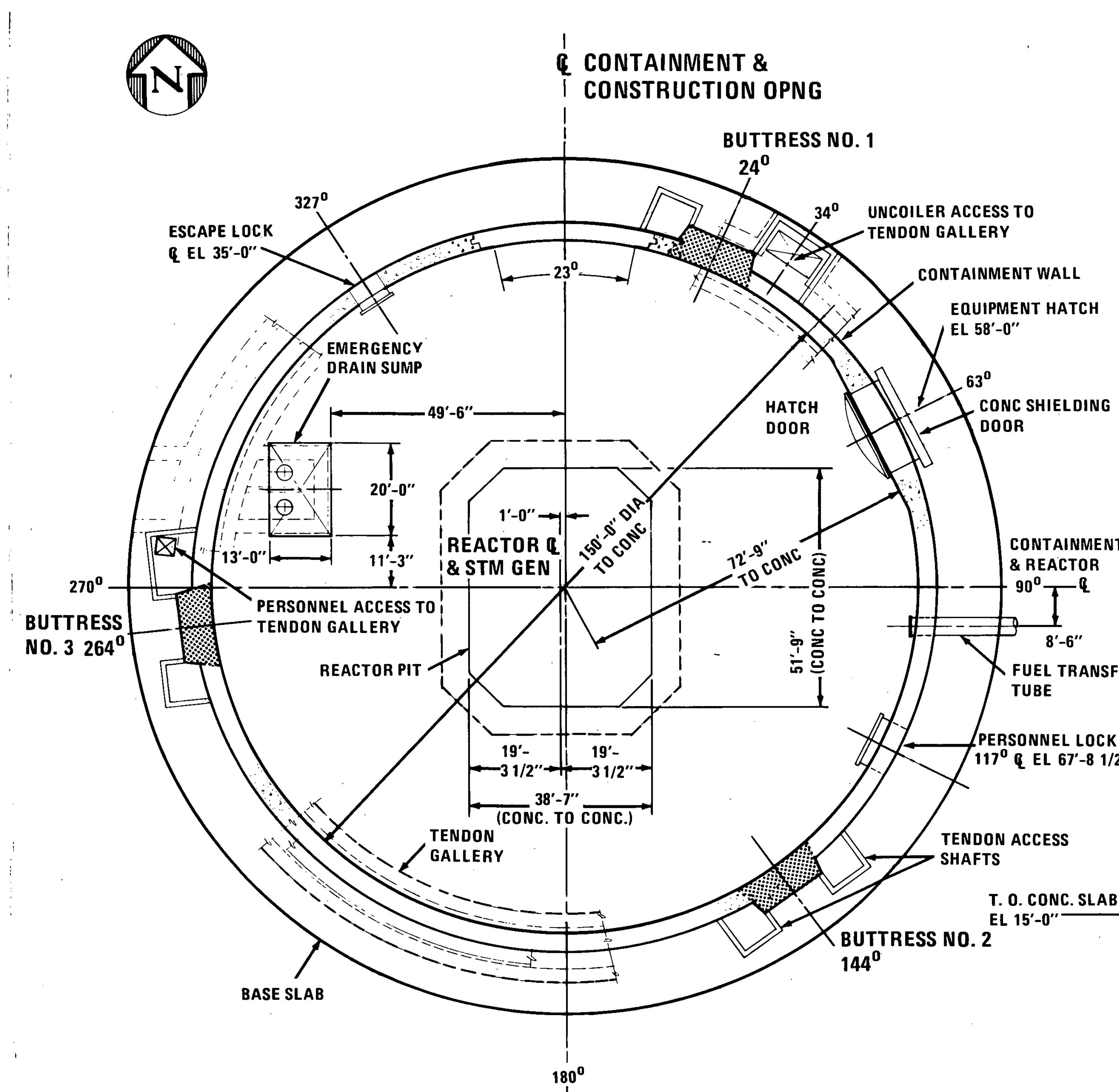
2. "Full Scale Buttress Test for Prestressed Nuclear Containment Structures," BC-TOP-7, Bechtel Corporation, San Francisco, California, August 1971 (Reprinted September 1972).

3. "Tendon End Anchor Reinforcement Test," BC-TOP-8, Bechtel Corporation, San Francisco, California, November 1971.

4. "Containment Building Liner Plate Design Report," BC-TOP-1, Revision 1, Bechtel Power Corporation, San Francisco, California, December 1972.

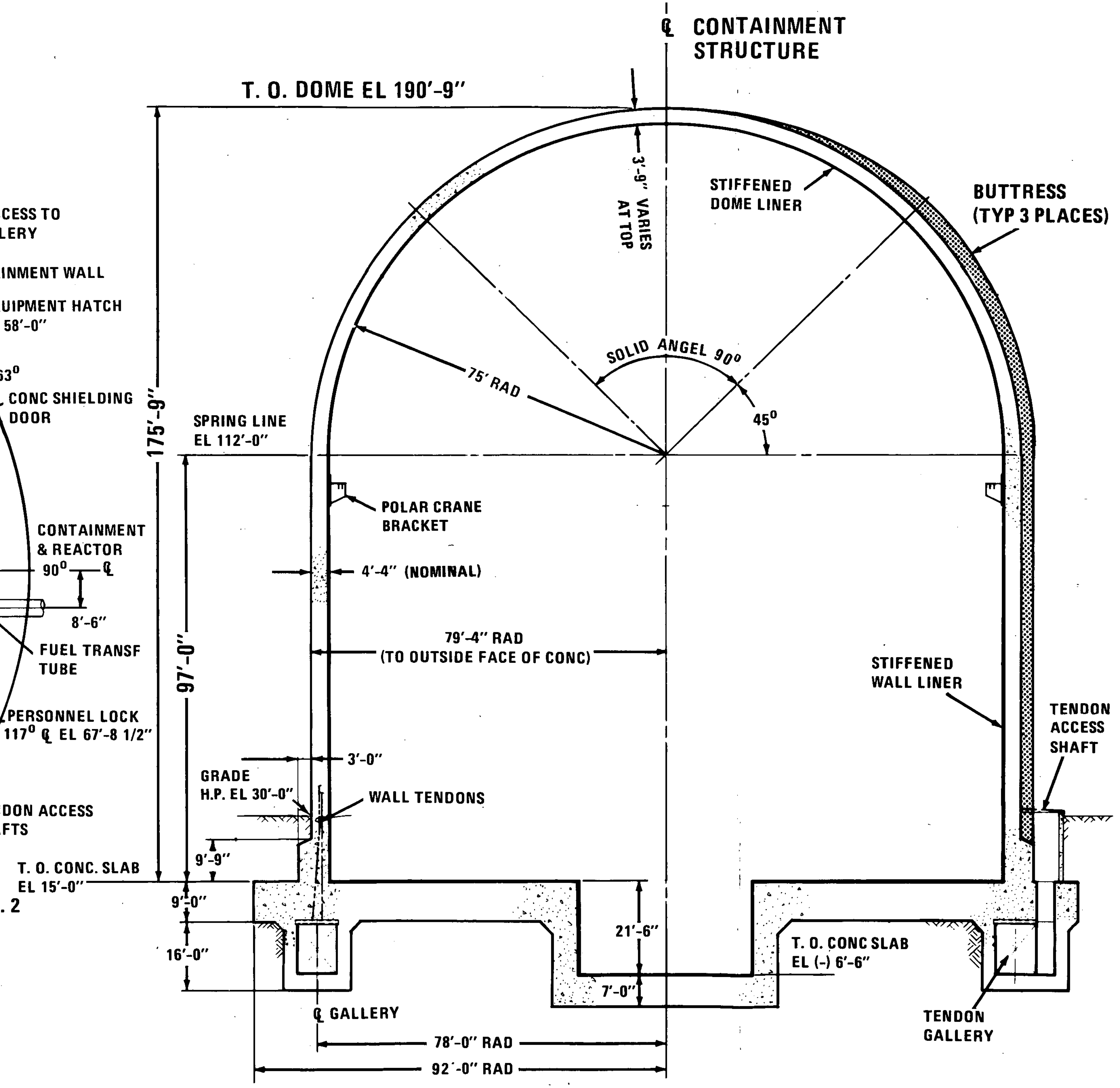
5. Eringer, A. C., Naghdi, A. K., and Thiel, C. C., State of Stress in Circular Cylindrical Shell with a Circular Hole, Welding Research Council Bulletin No. 102, January 1965.

6. "Seismic Analyses of Structures and Equipment for Nuclear Power Plants," BC-TOP-4-A, Revision 3, Bechtel Power Corporation, San Francisco, California, November 1974.



UNIT 2 – SHOWN
UNIT 3 – OPPOSITE HAND ABOUT 90° - 270° AXIS

PLAN



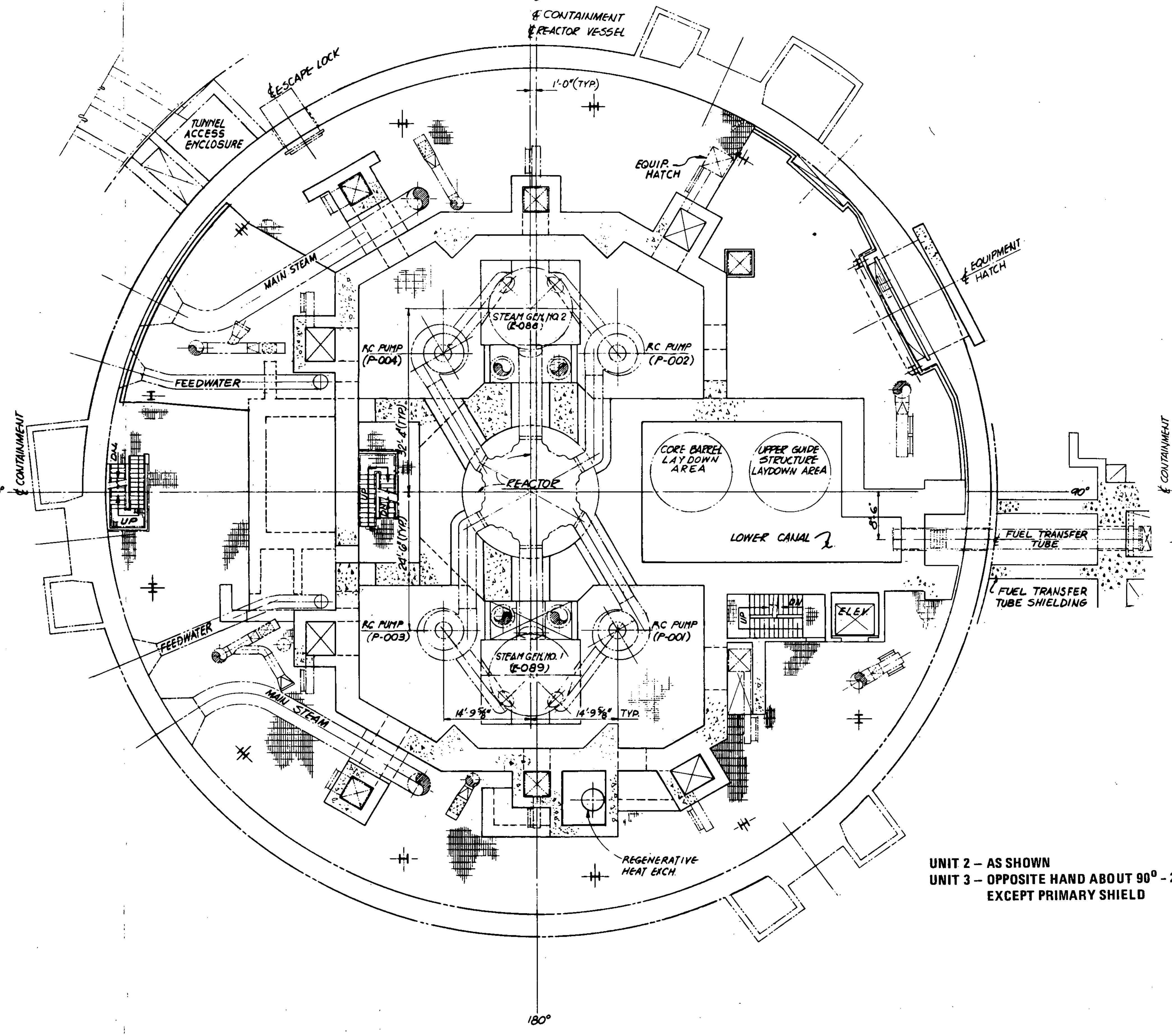
CROSS SECTION

SAN ONOFRE
NUCLEAR GENERATING STATION
Units 2 & 3

CONTAINMENT STRUCTURE
GENERAL ARRANGEMENT
(Sheet 1 of 3)

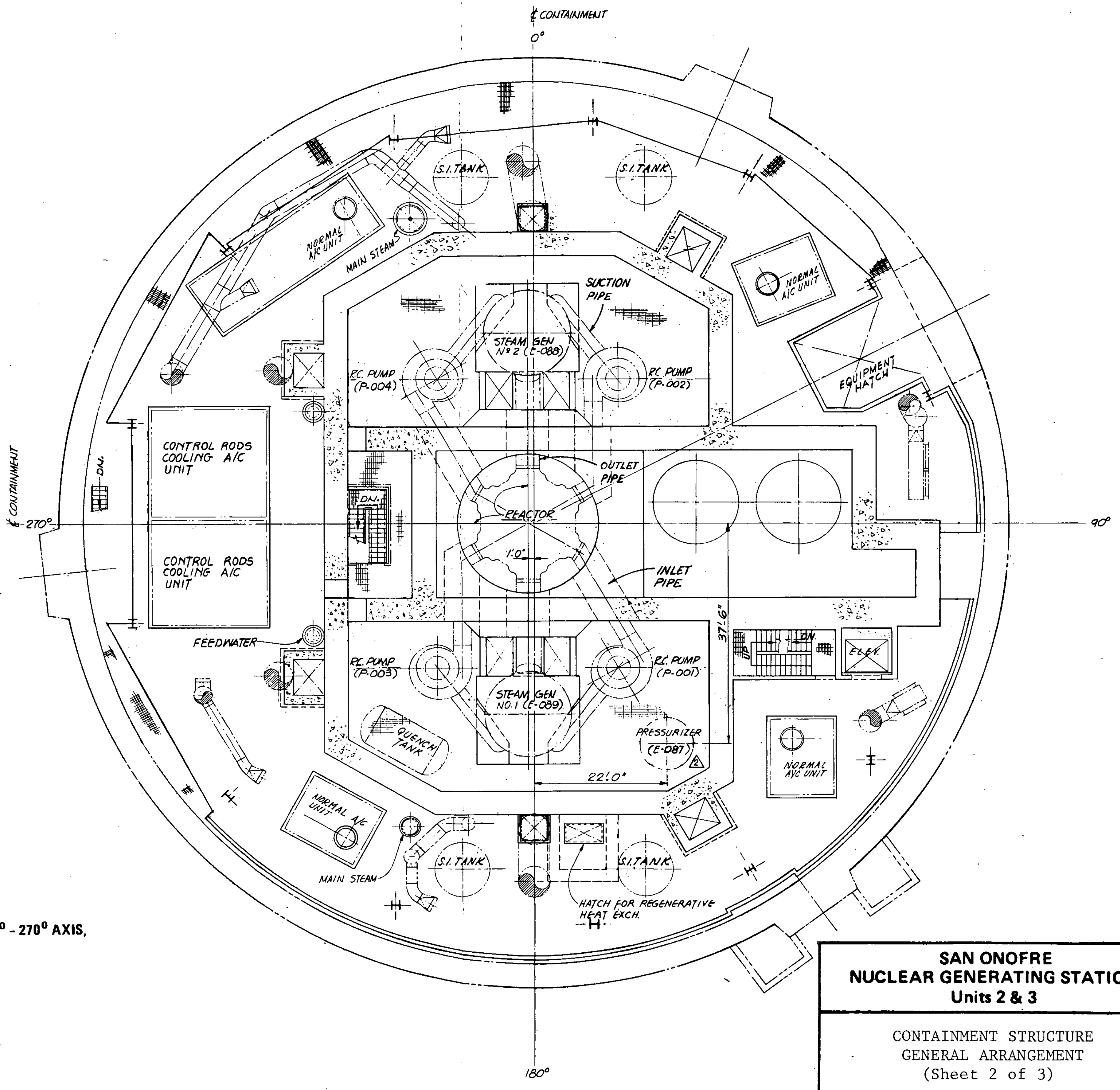
Figure 3.8-1

10079-11D-225
26AU6



PLAN AT ELEVATION 30'-0"

UNIT 2 – AS SHOWN
UNIT 3 – OPPOSITE HAND ABOUT 90° - 270° AXIS, EXCEPT PRIMARY SHIELD



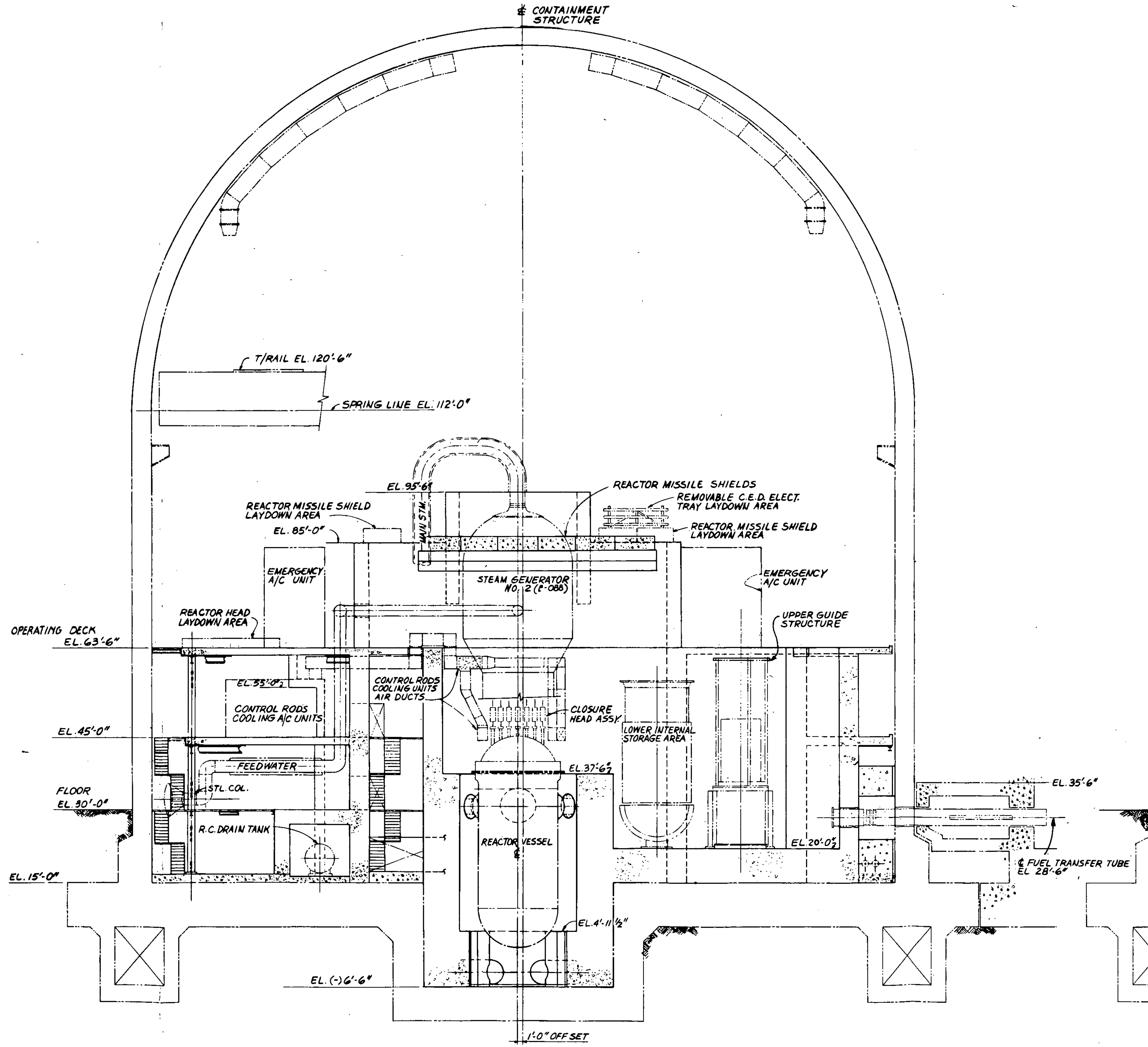
PLAN AT ELEVATION 45'-0"

10079-11D-226

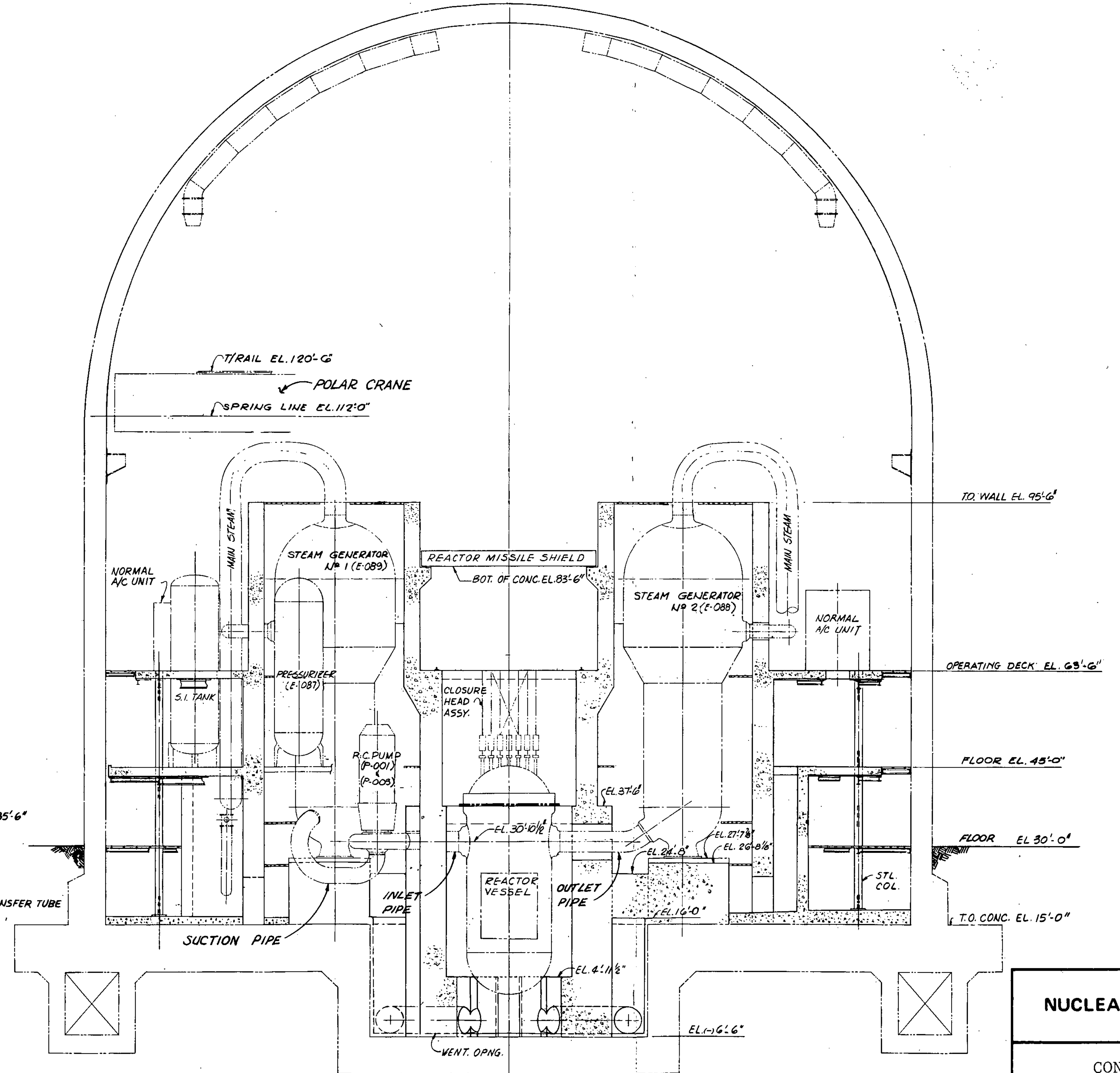
SAN ONOFRE NUCLEAR GENERATING STATION Units 2 & 3

CONTAINMENT STRUCTURE
GENERAL ARRANGEMENT
(Sheet 2 of 3)

Figure 3.8-1



CROSS SECTION FACING NORTH



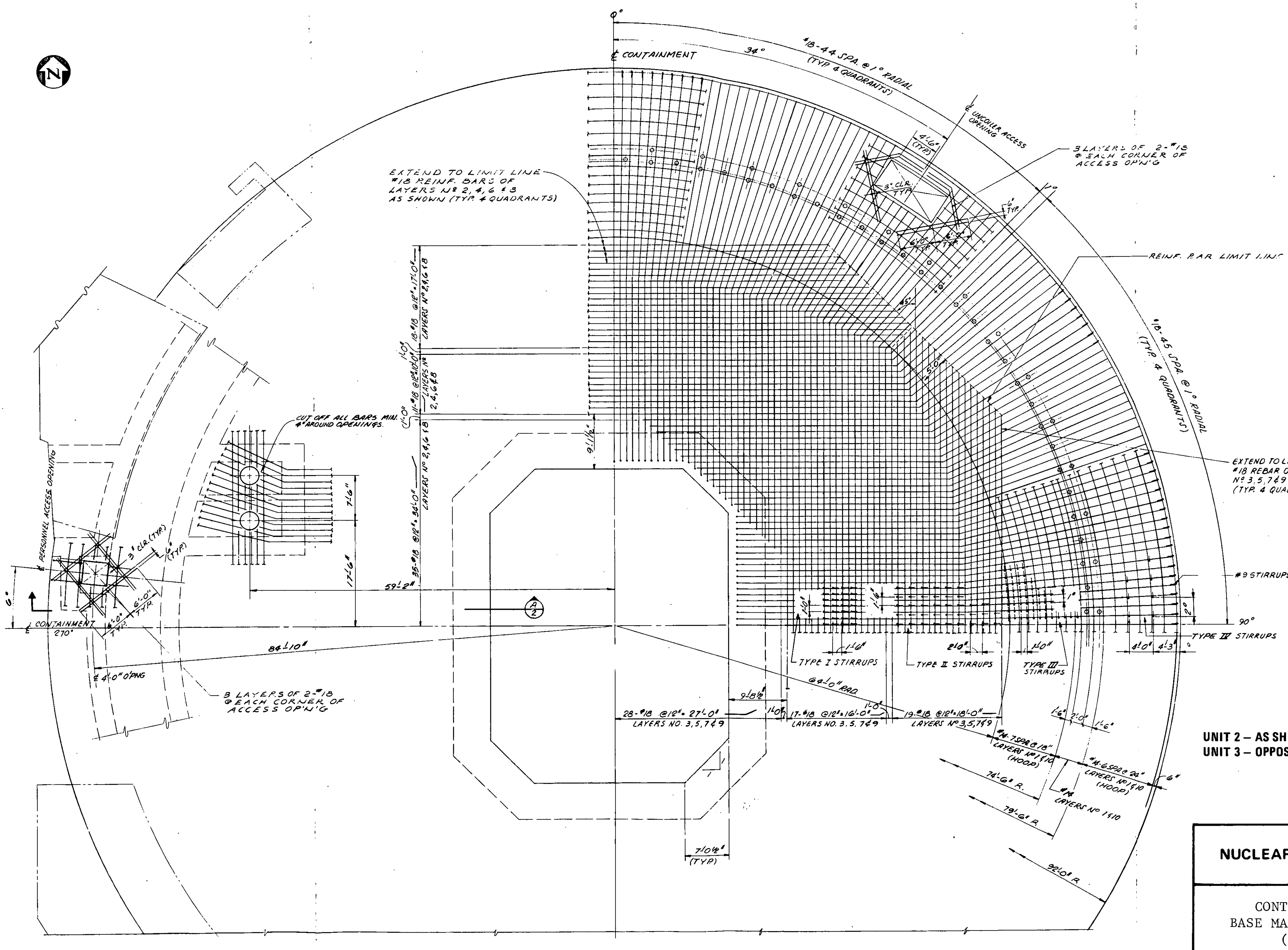
CROSS SECTION FACING WEST

SAN ONOFRE
NUCLEAR GENERATING STATION
Units 2 & 3

CONTAINMENT STRUCTURE
GENERAL ARRANGEMENT
(Sheet 3 of 3)

Figure 3.8-1

10079-11D-227
26AU6



PLAN SECTION AT BOTTOM LAYERS REINF

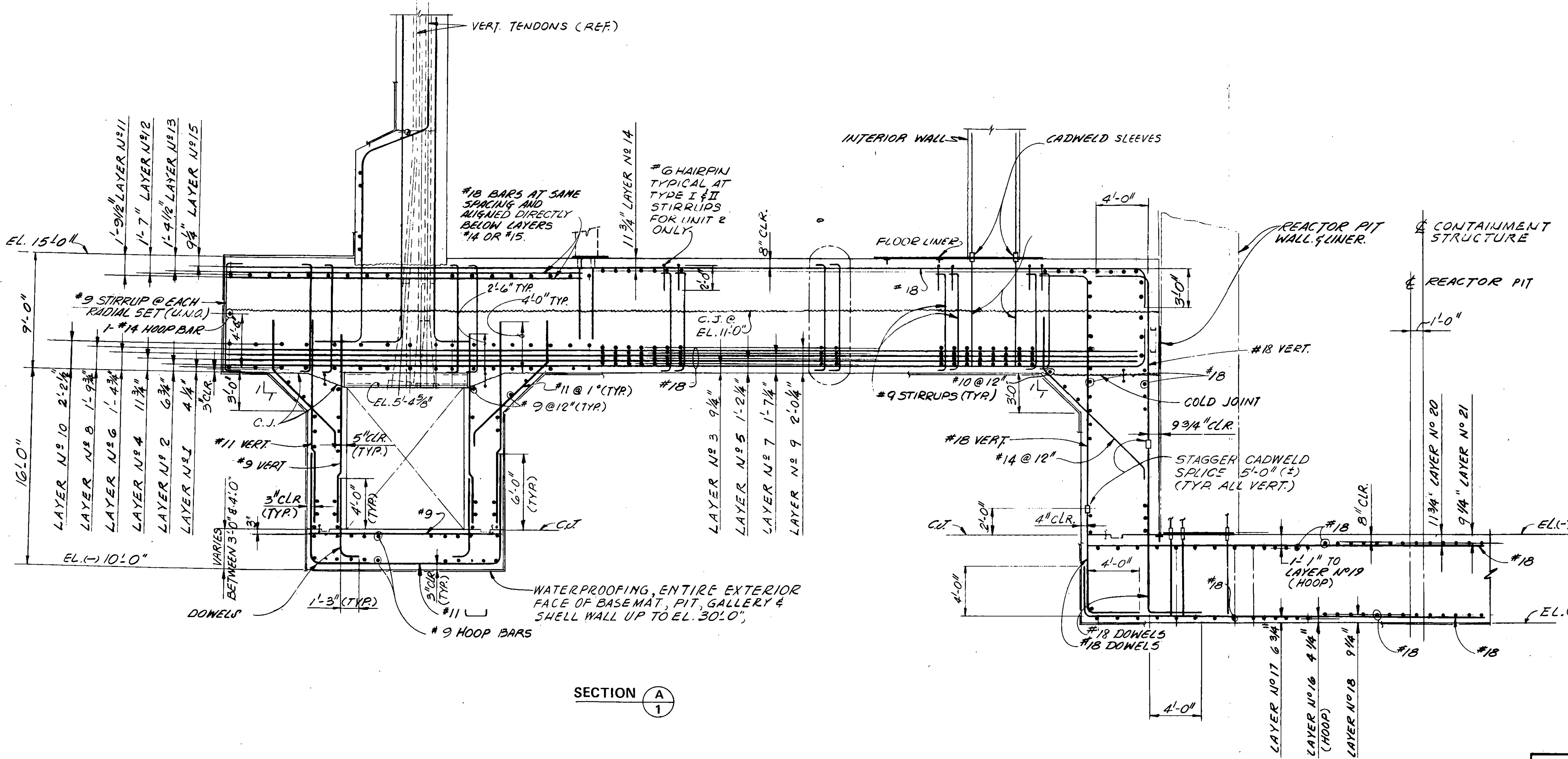
UNIT 2 – AS SHOWN
UNIT 3 – OPPOSITE HAND ABOUT 90° - 270° AXIS

SAN ONOFRE NUCLEAR GENERATING STATION Units 2 & 3

CONTAINMENT STRUCTURE
BASE MAT, REINFORCING STEEL
(Sheet 1 of 2)

Figure 3.8-2

10079-11D-228
26AU6

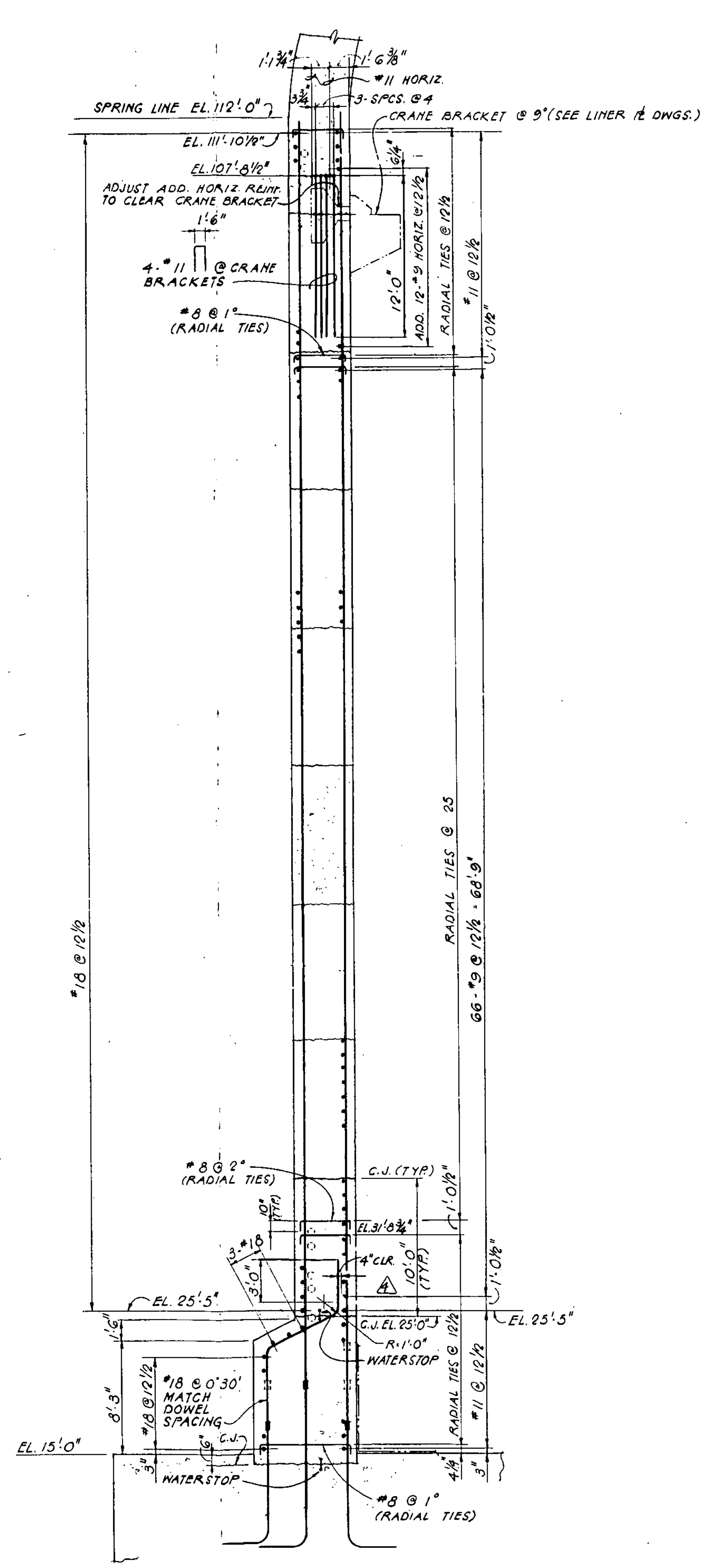


SAN ONOFRE
NUCLEAR GENERATING STATION
Units 2 & 3

CONTAINMENT STRUCTURE
BASE MAT, REINFORCING STEEL
(Sheet 2 of 2)

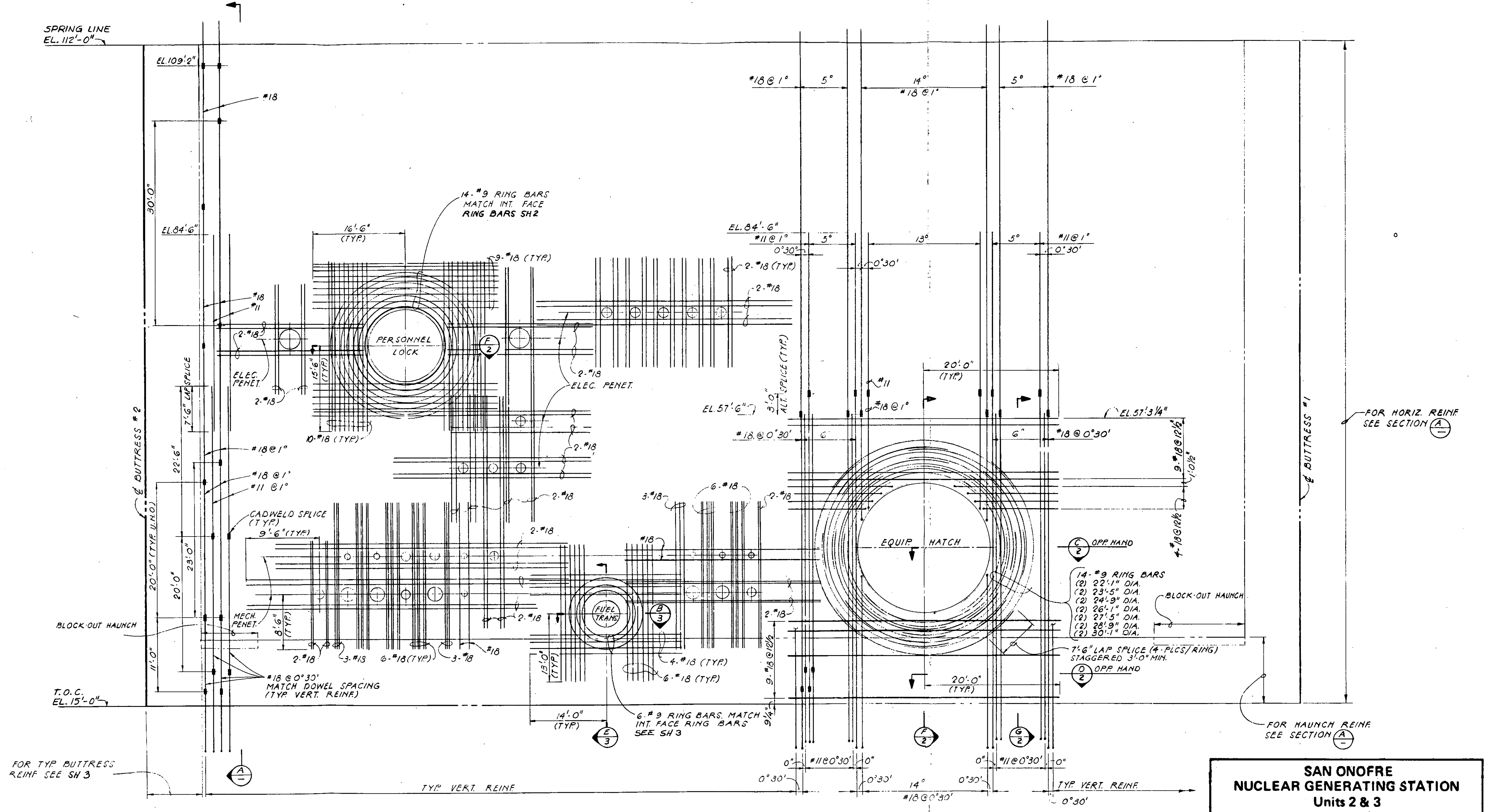
Figure 3.8-2

10079-11D-229
26AU6



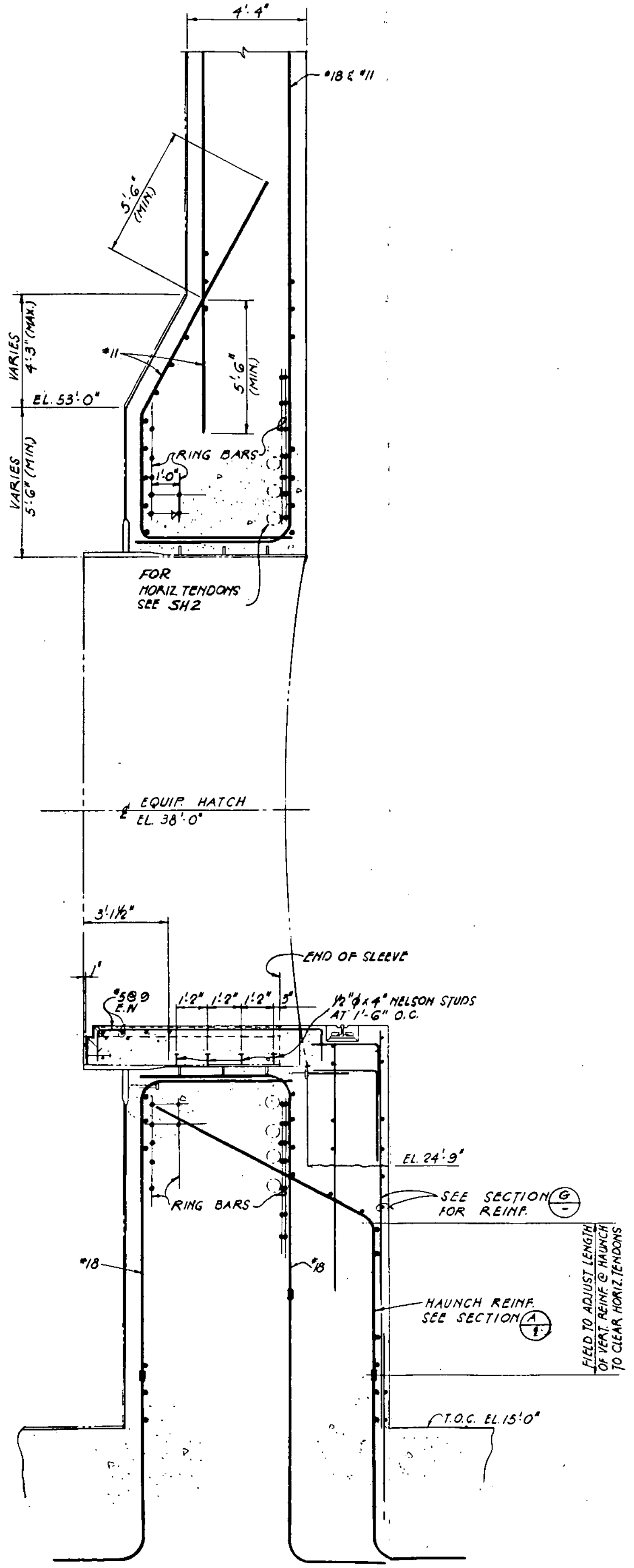
SECTION A

10079-11D-230
28AP6

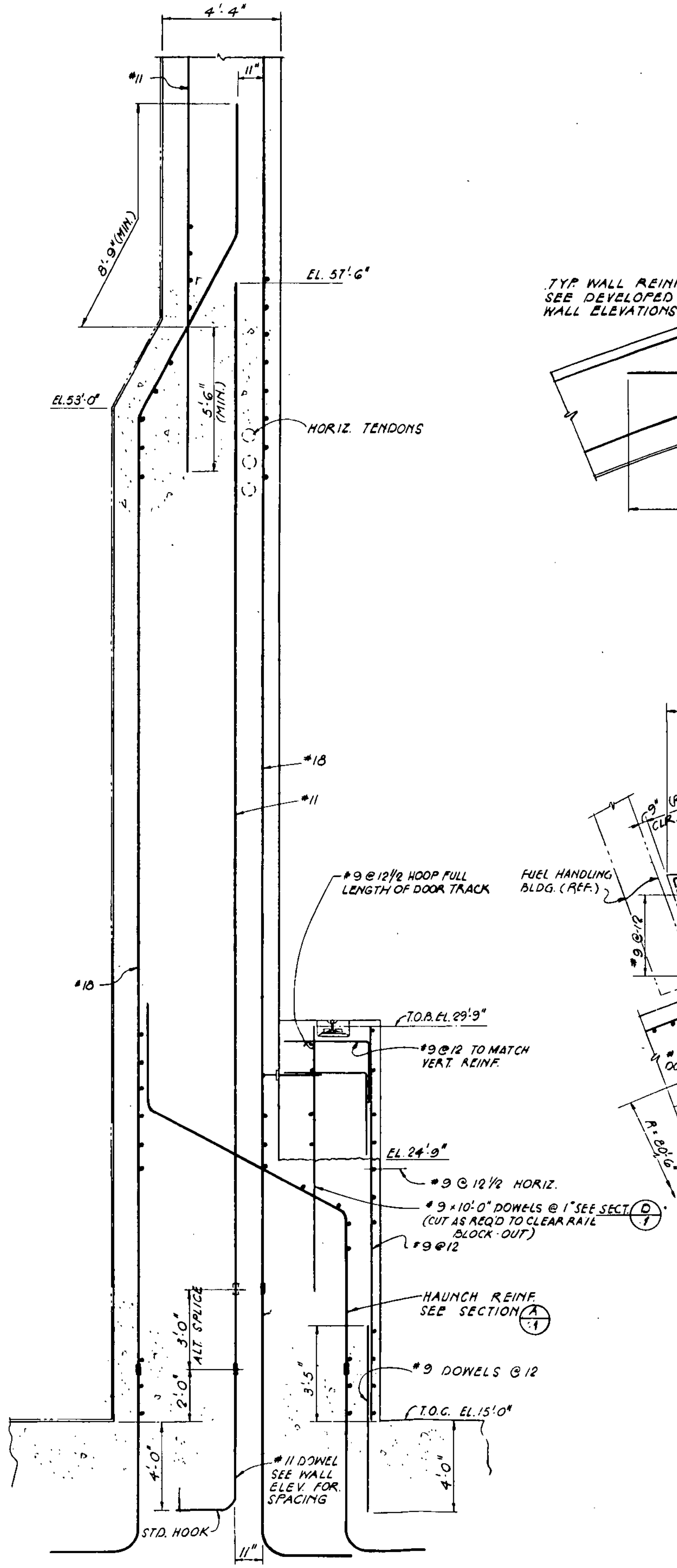


DEVELOPED ELEVATION OF WALL
LOOKING FROM OUTSIDE

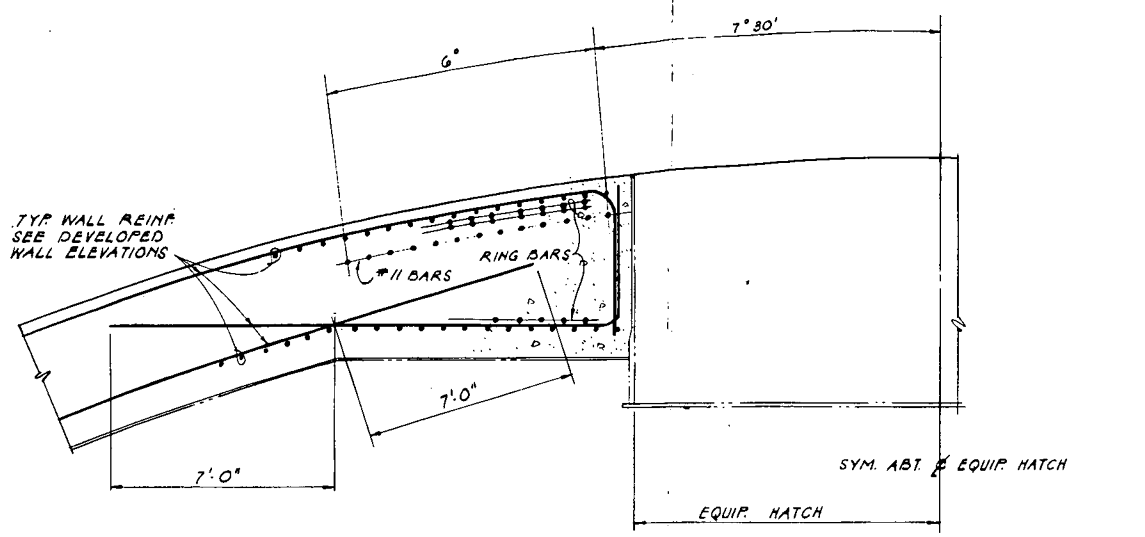
SAN ONOFRE NUCLEAR GENERATING STATION Units 2 & 3
SHELL WALL REINFORCING STEEL (Sheet 1 of 3)
Figure 3.8-3



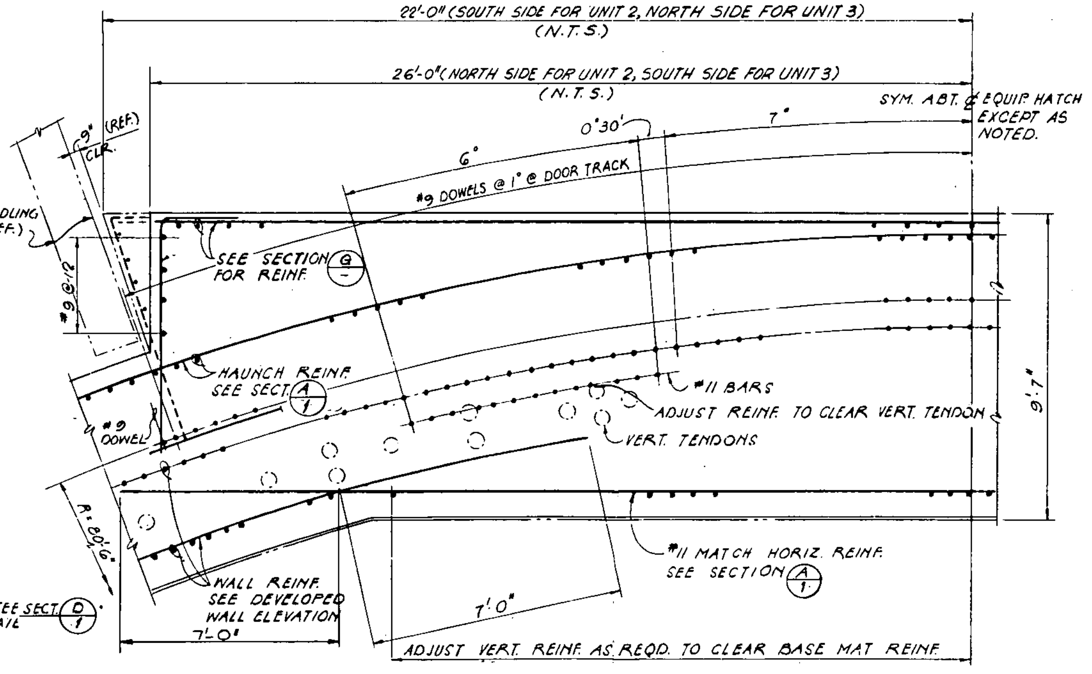
SECTION F/1



SECTION G/1



SECTION C/1



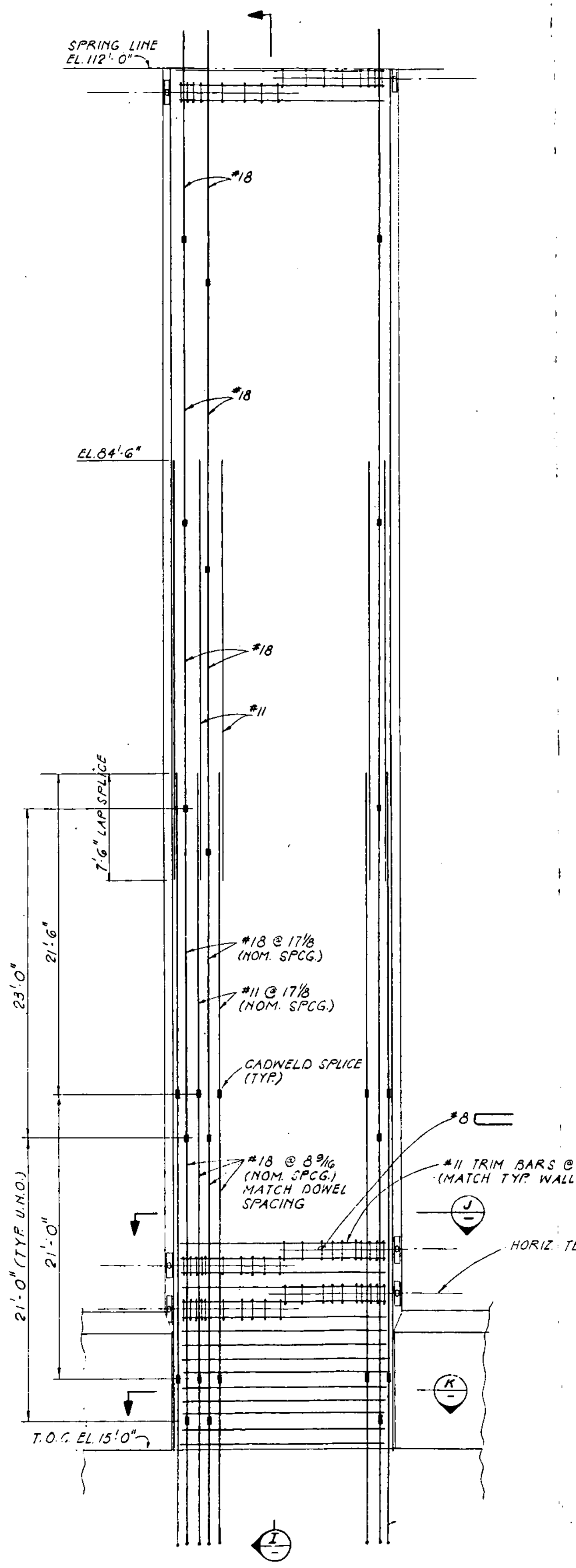
SECTION D/1

10079-11D-231
26AU6

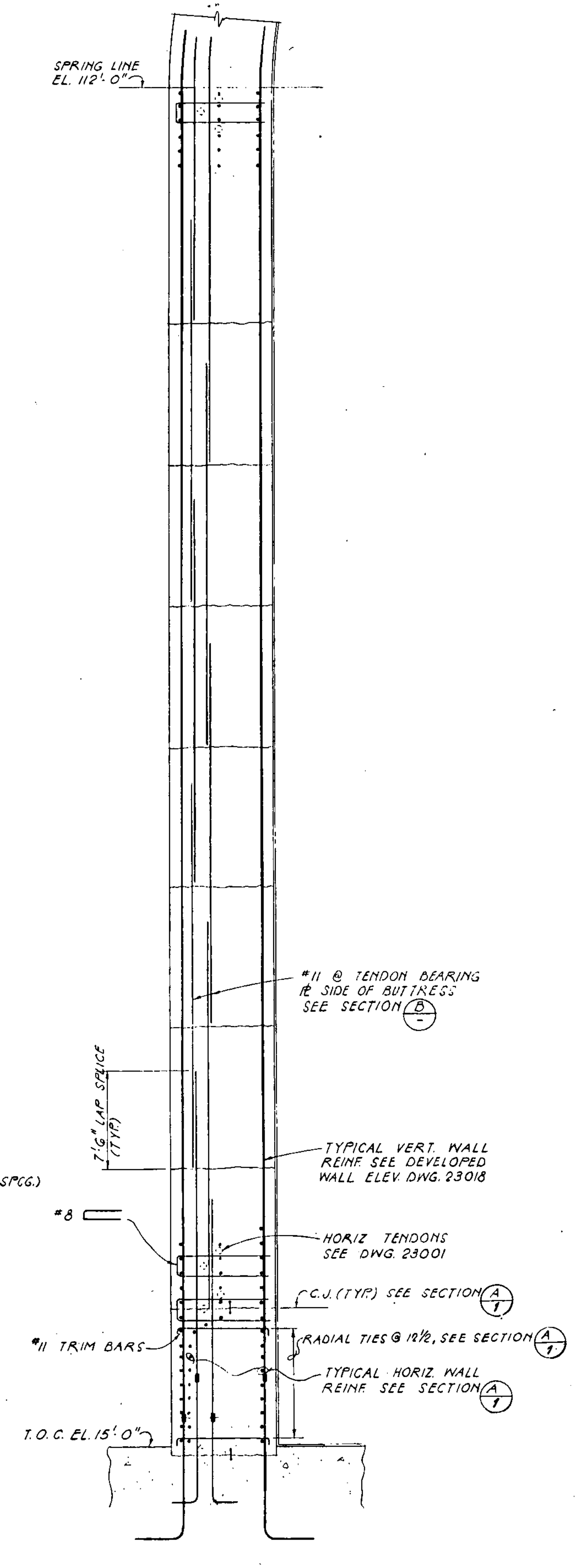
SAN ONOFRE
NUCLEAR GENERATING STATION
Units 2 & 3

SHELL WALL
REINFORCING STEEL
(Sheet 2 of 3)

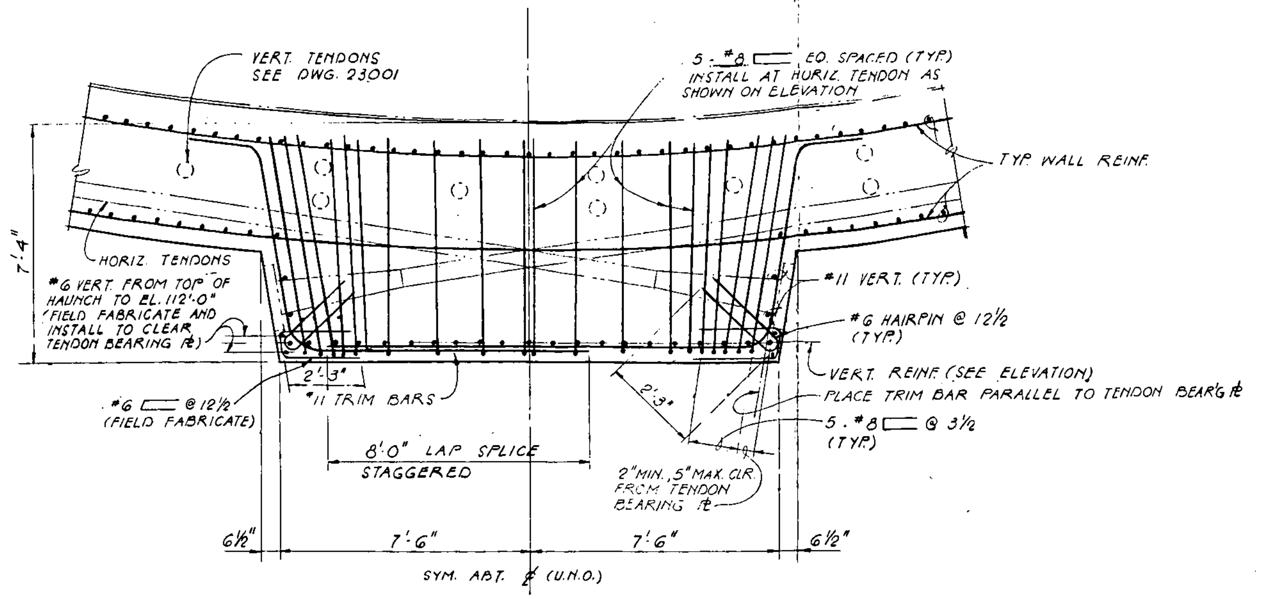
Figure 3.8-3



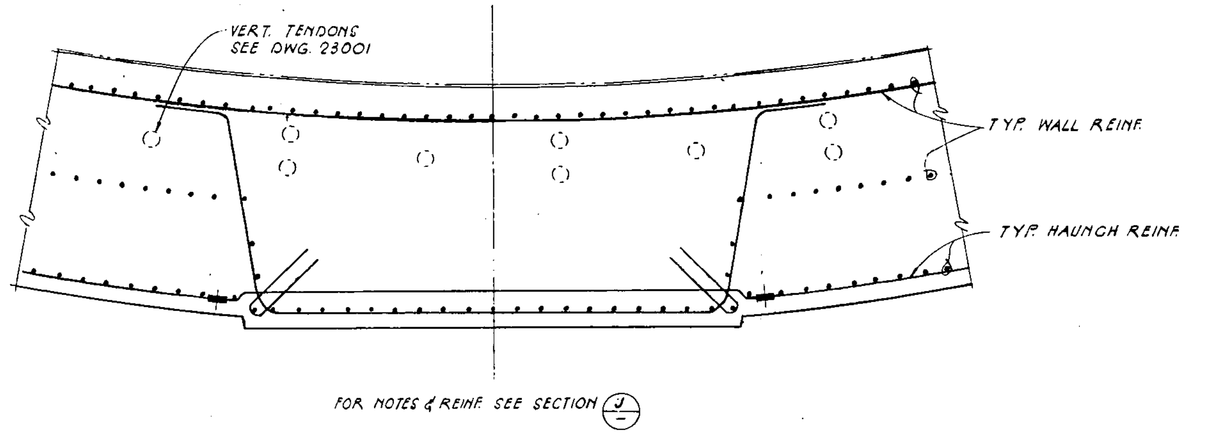
TYPICAL BUTTRESS ELEVATION
LOOKING FROM OUTSIDE



SECTION I

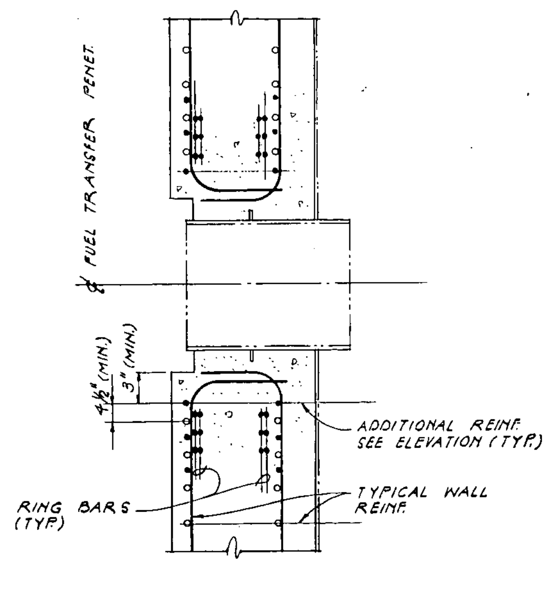


SECTION J

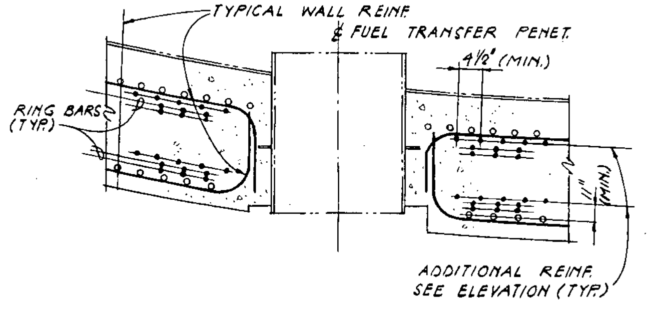


FOR NOTES & REINF. SEE SECTION J

SECTION K



SECTION F 1



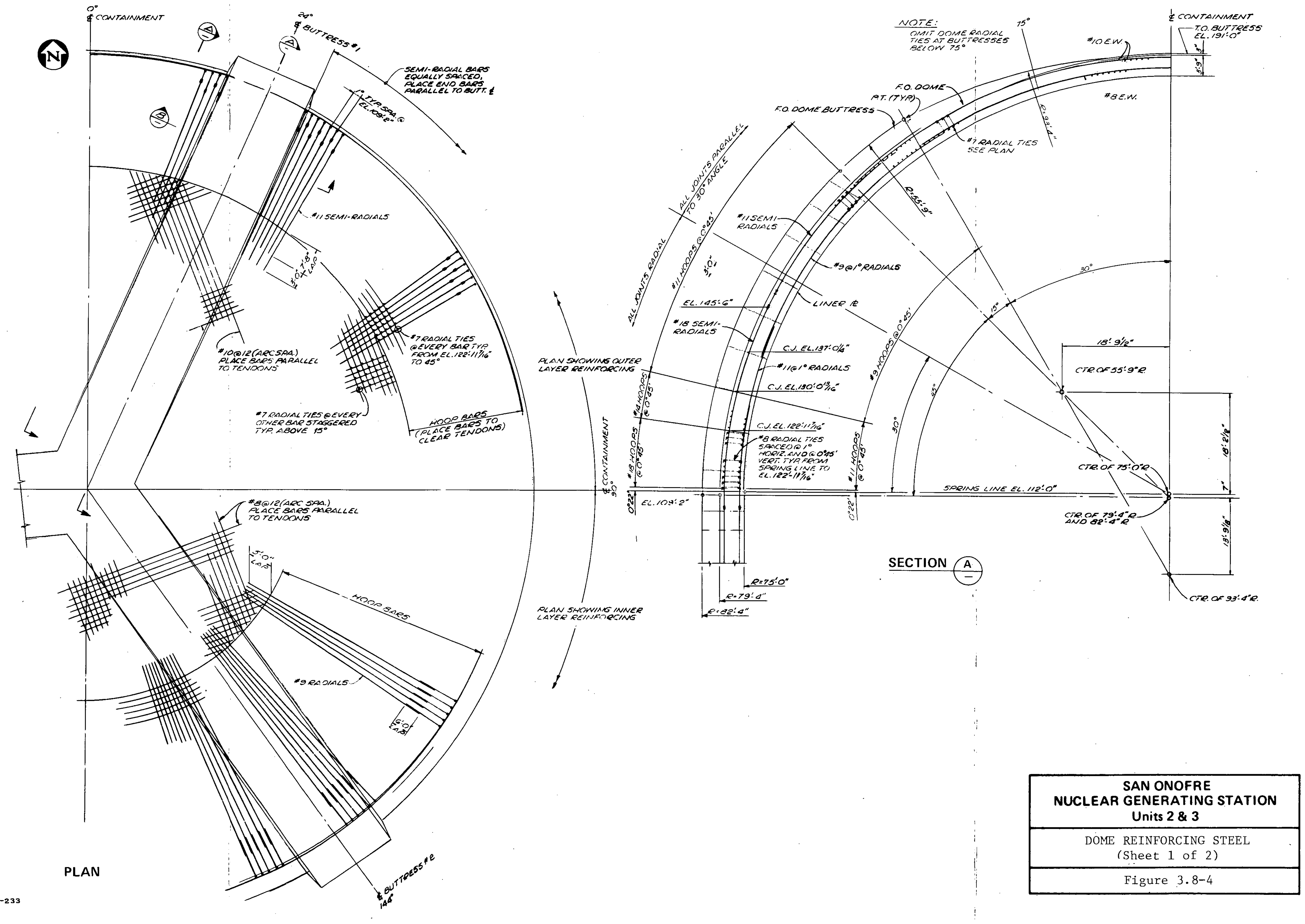
SECTION B 1

SAN ONOFRE NUCLEAR GENERATING STATION Units 2 & 3

SHELL WALL REINFORCING STEEL (Sheet 3 of 3)

Figure 3.8-3

10079-11D-232
26AU6

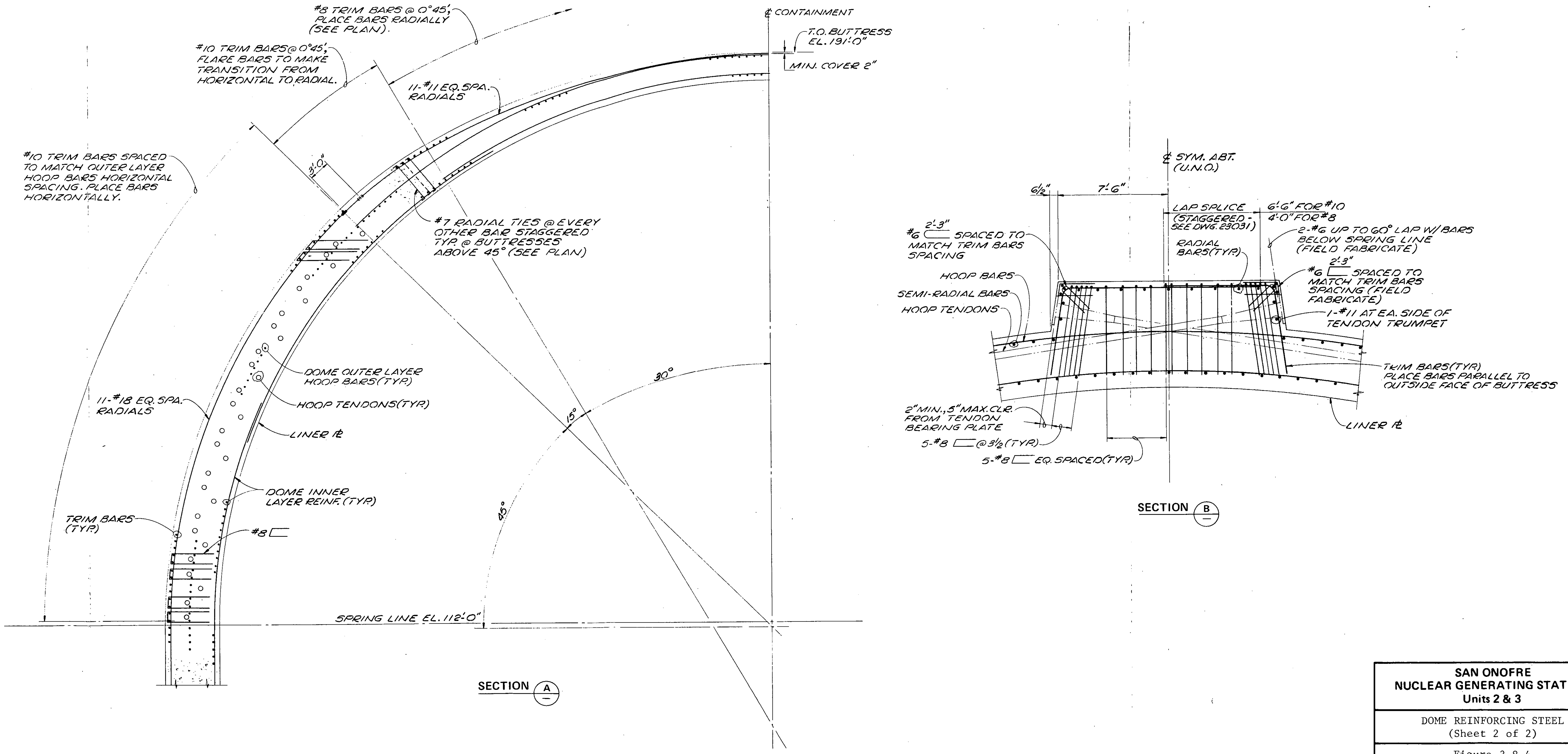


10079-11D-233
28AP6

SAN ONOFRE
NUCLEAR GENERATING STATION
Units 2 & 3

DOME REINFORCING STEEL
(Sheet 1 of 2)

Figure 3.8-4

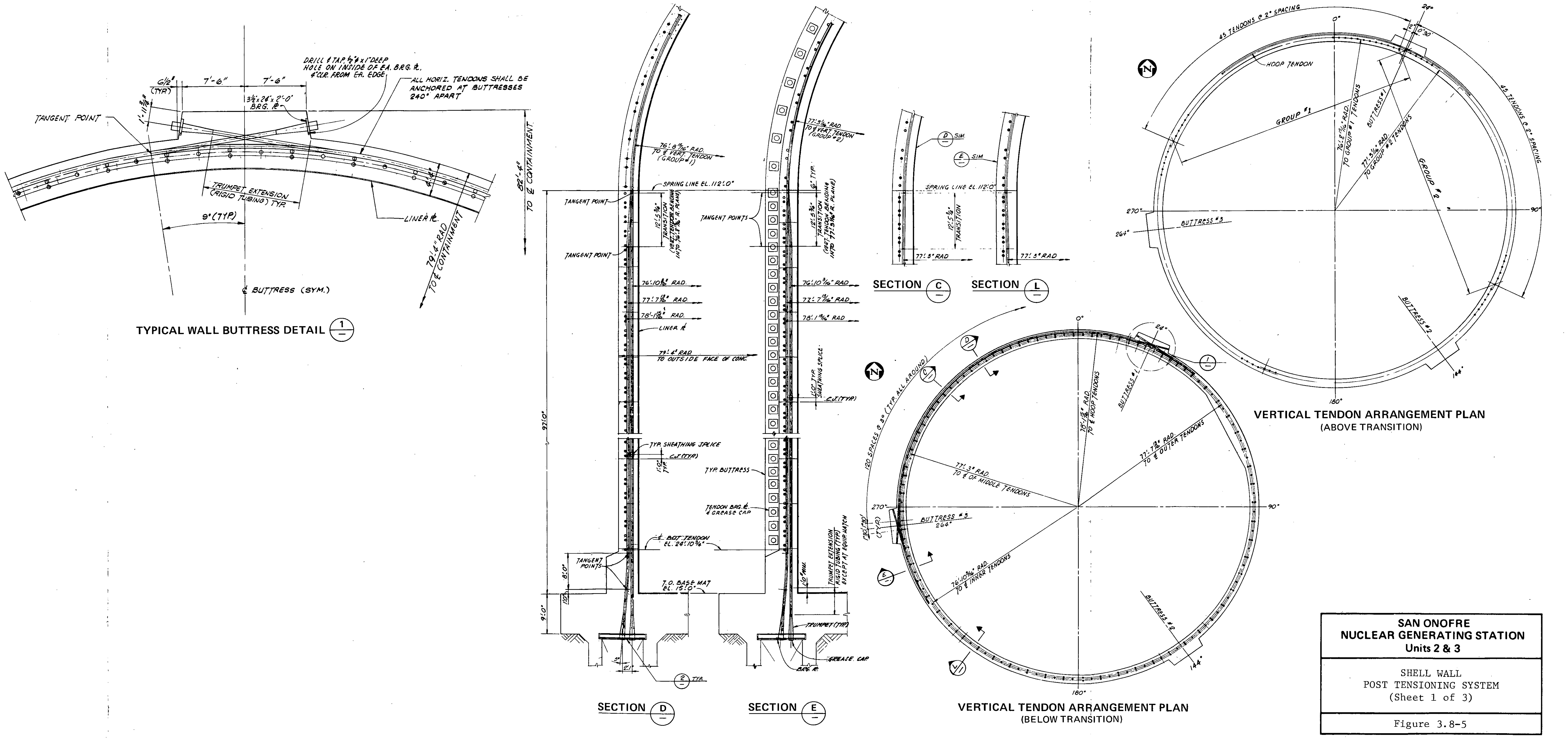


SAN ONOFRE
NUCLEAR GENERATING STATION
Units 2 & 3

DOME REINFORCING STEEL
(Sheet 2 of 2)

Figure 3.8-4

10079-11D-234
28AP6

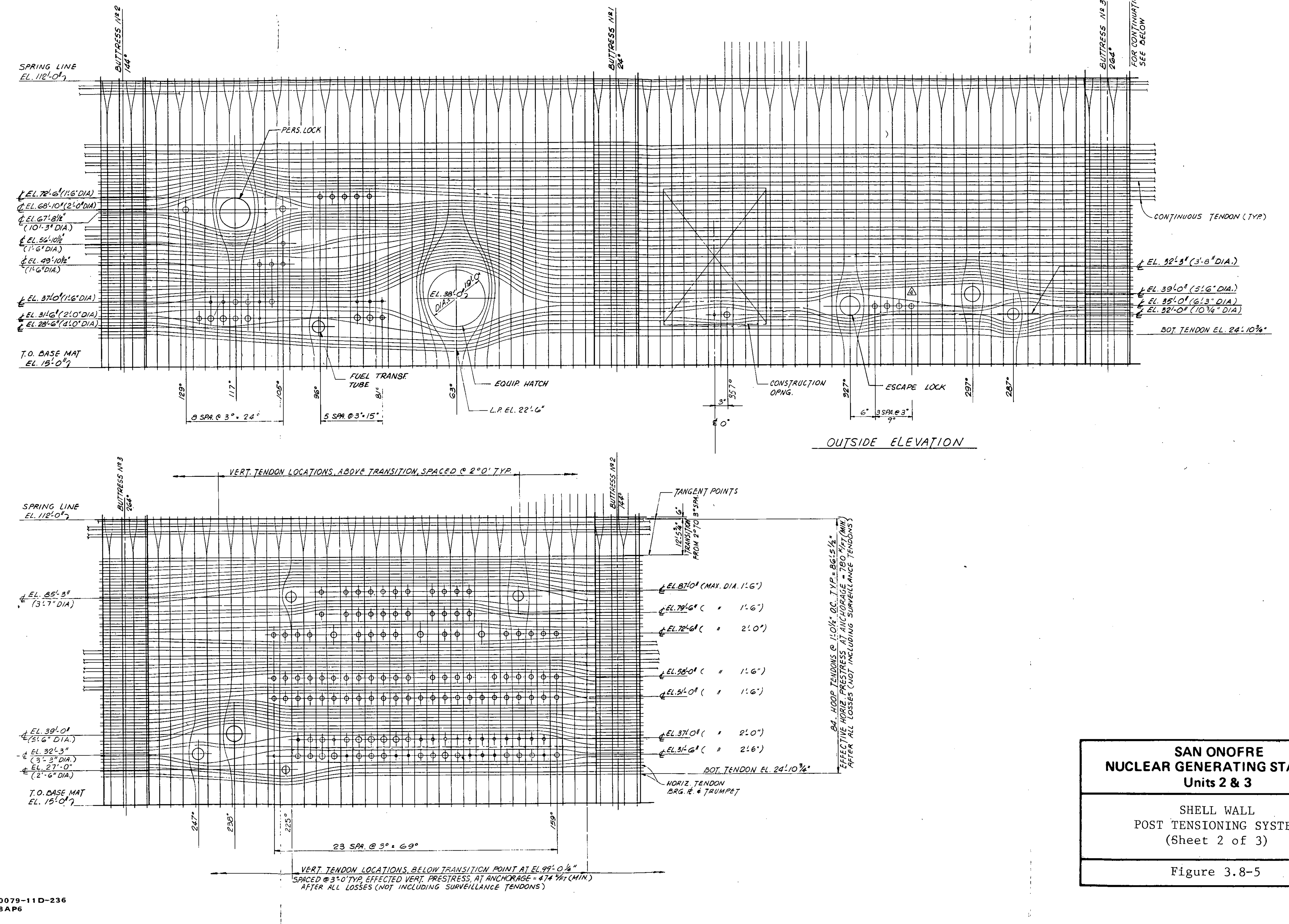


SAN ONOFRE
NUCLEAR GENERATING STATION
Units 2 & 3

SHELL WALL
POST TENSIONING SYSTEM
(Sheet 1 of 3)

Figure 3.8-5

10079-11D-235
28AP6

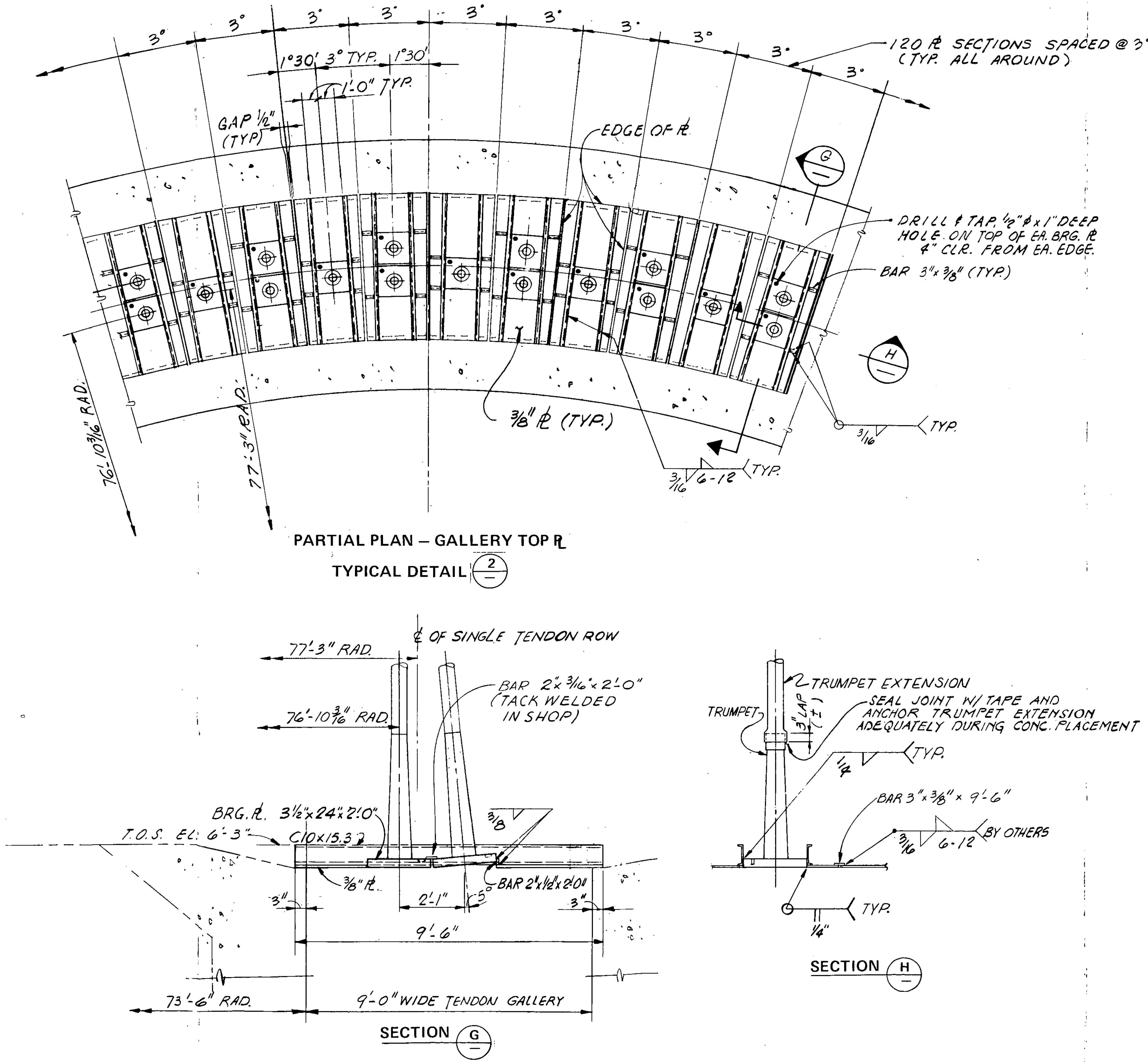


10079-11D-236
28AP6

SAN ONOFRE
NUCLEAR GENERATING STATION
Units 2 & 3

SHELL WALL
POST TENSIONING SYSTEM
(Sheet 2 of 3)

Figure 3.8-5

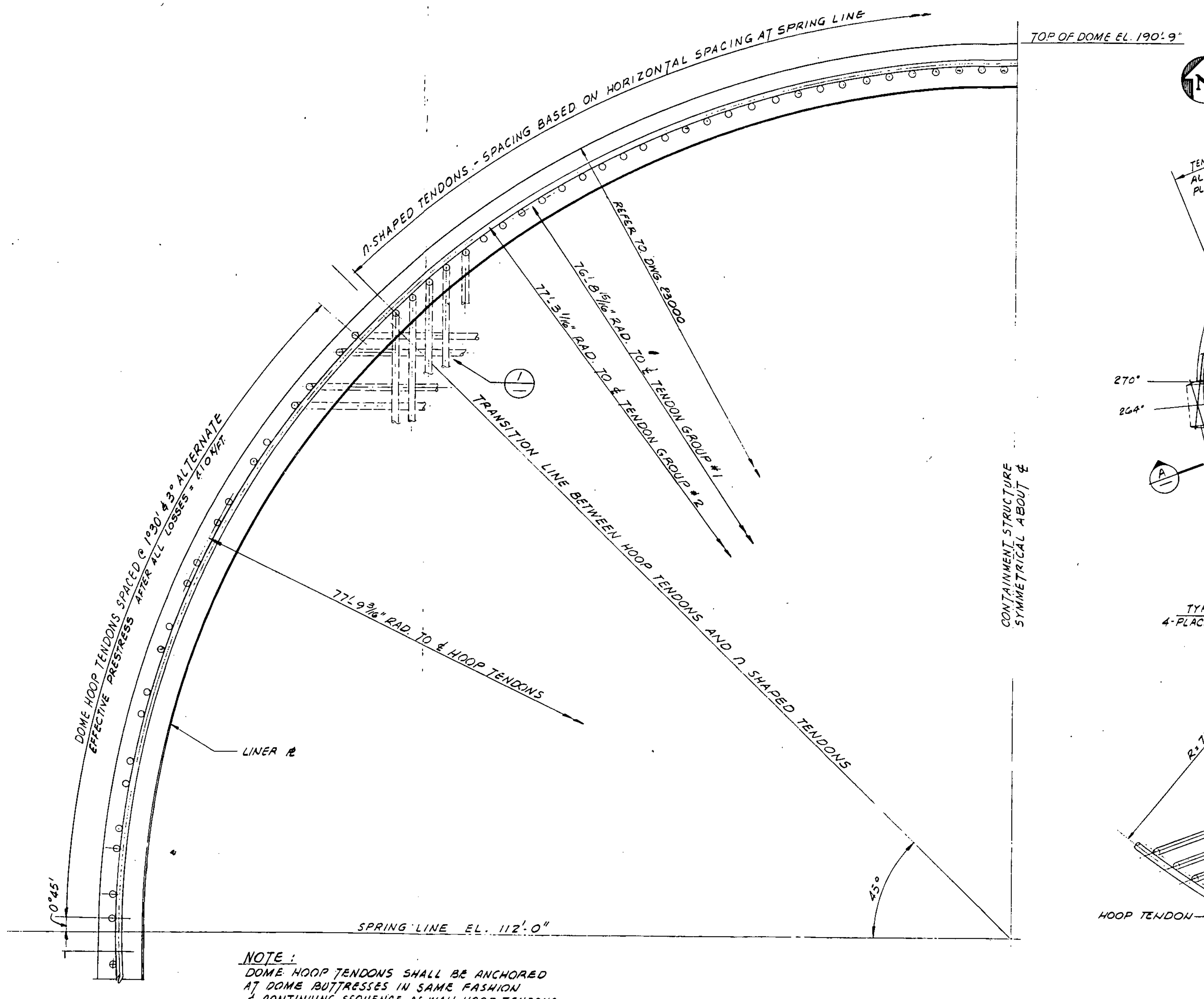


10079-11D-237
28AP6

SAN ONOFRE NUCLEAR GENERATING STATION
Units 2 & 3

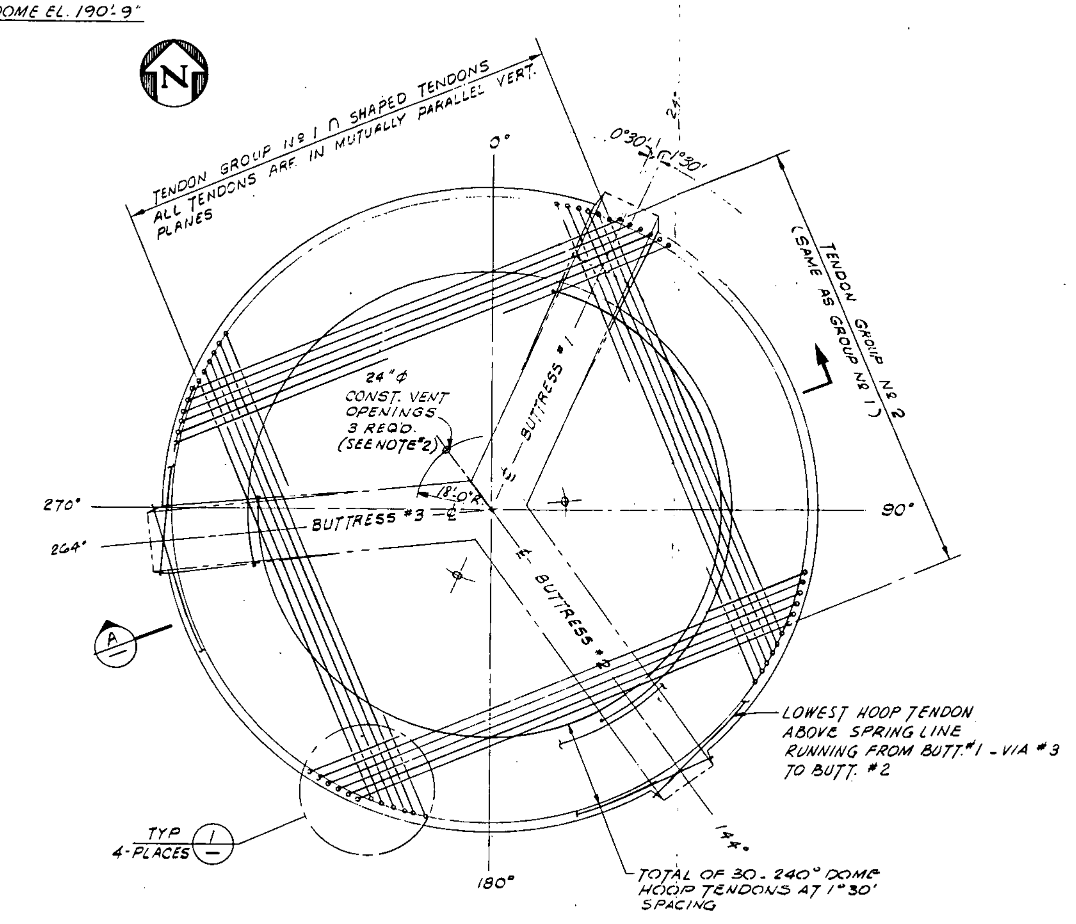
SHELL WALL
POST TENSIONING SYSTEM
(Sheet 3 of 3)

Figure 3.8-5

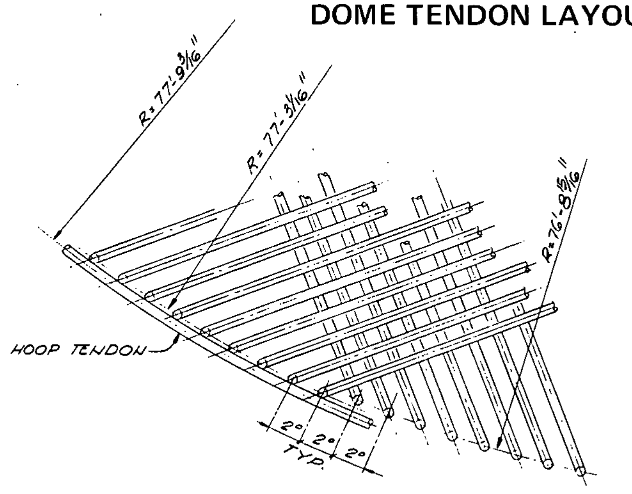


NOTE :
DOME HOOP TENDONS SHALL BE ANCHORED AT DOME BUTTRESSES IN SAME FASHION & CONTINUING SEQUENCE AS WALL HOOP TENDONS.

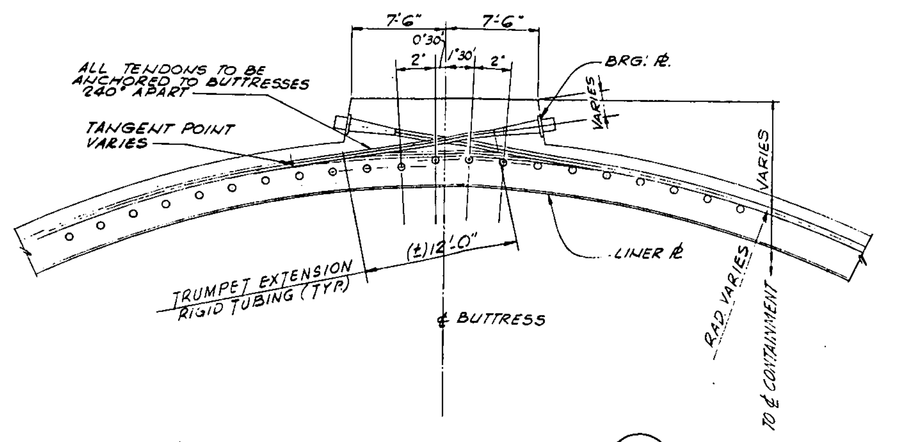
SECTION A



DOME TENDON LAYOUT PLAN



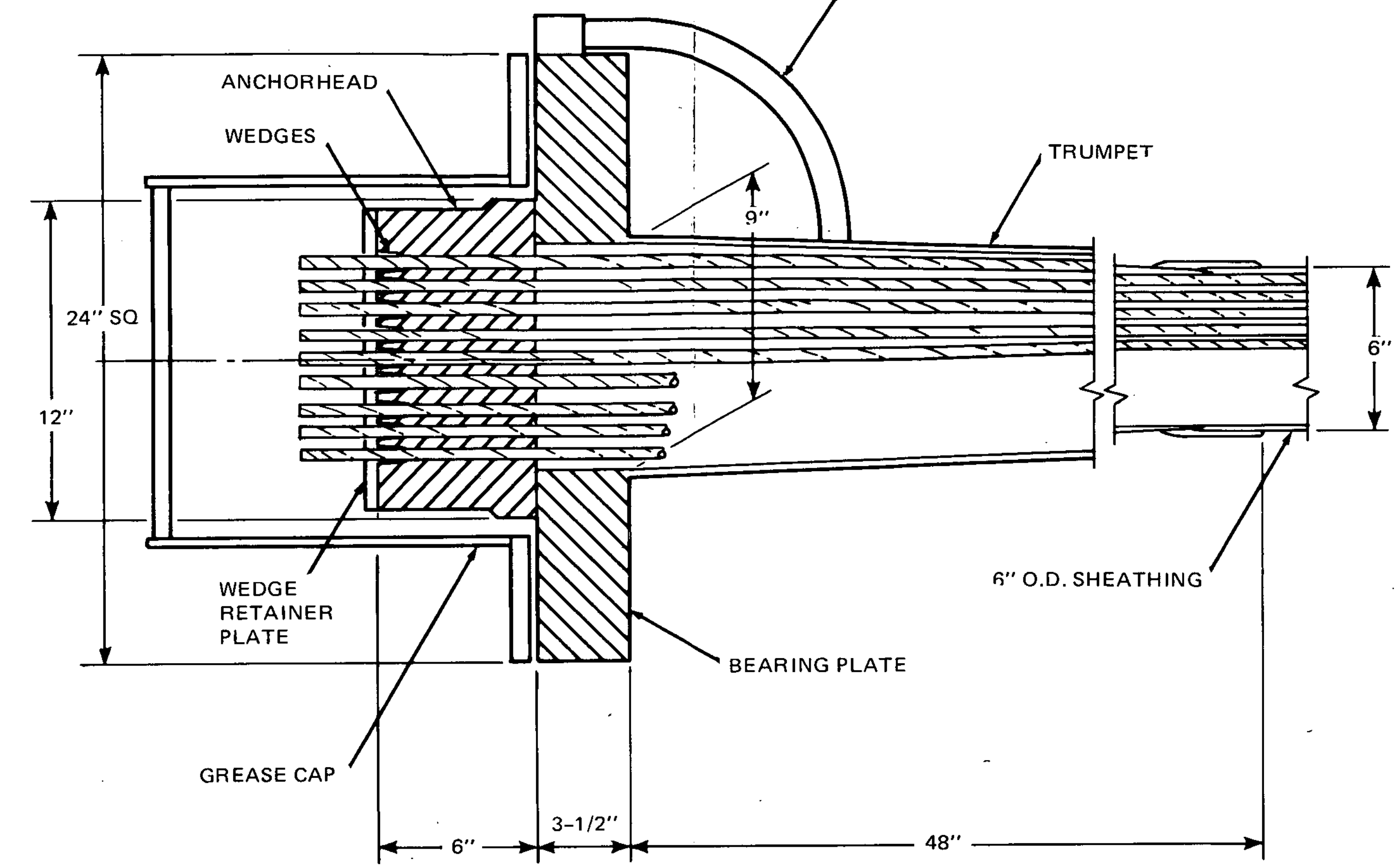
DETAIL 1



BUTTRESS SECTION AT DOME B

10079-11D-238
28AP6

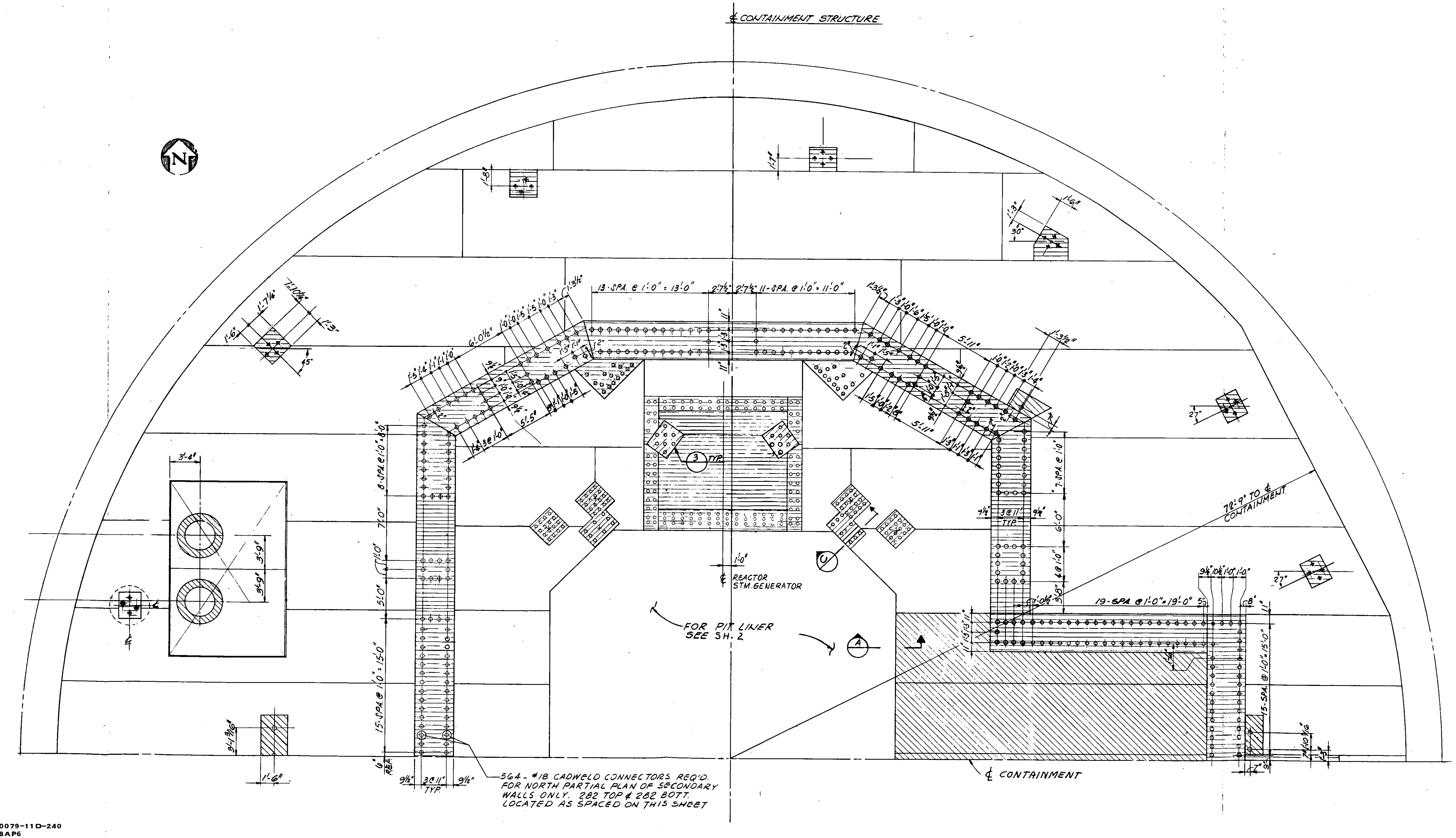
SAN ONOFRE NUCLEAR GENERATING STATION Units 2 & 3
POST TENSIONING SYSTEM
Figure 3.8-6



STRAND AREA = .153 SQ IN.

SAN ONOFRE NUCLEAR GENERATING STATION Units 2 & 3
55 STRAND VSL TENDON END ANCHOR
Figure 3.8-7

10079-11D-239
28AP6



LEGEND

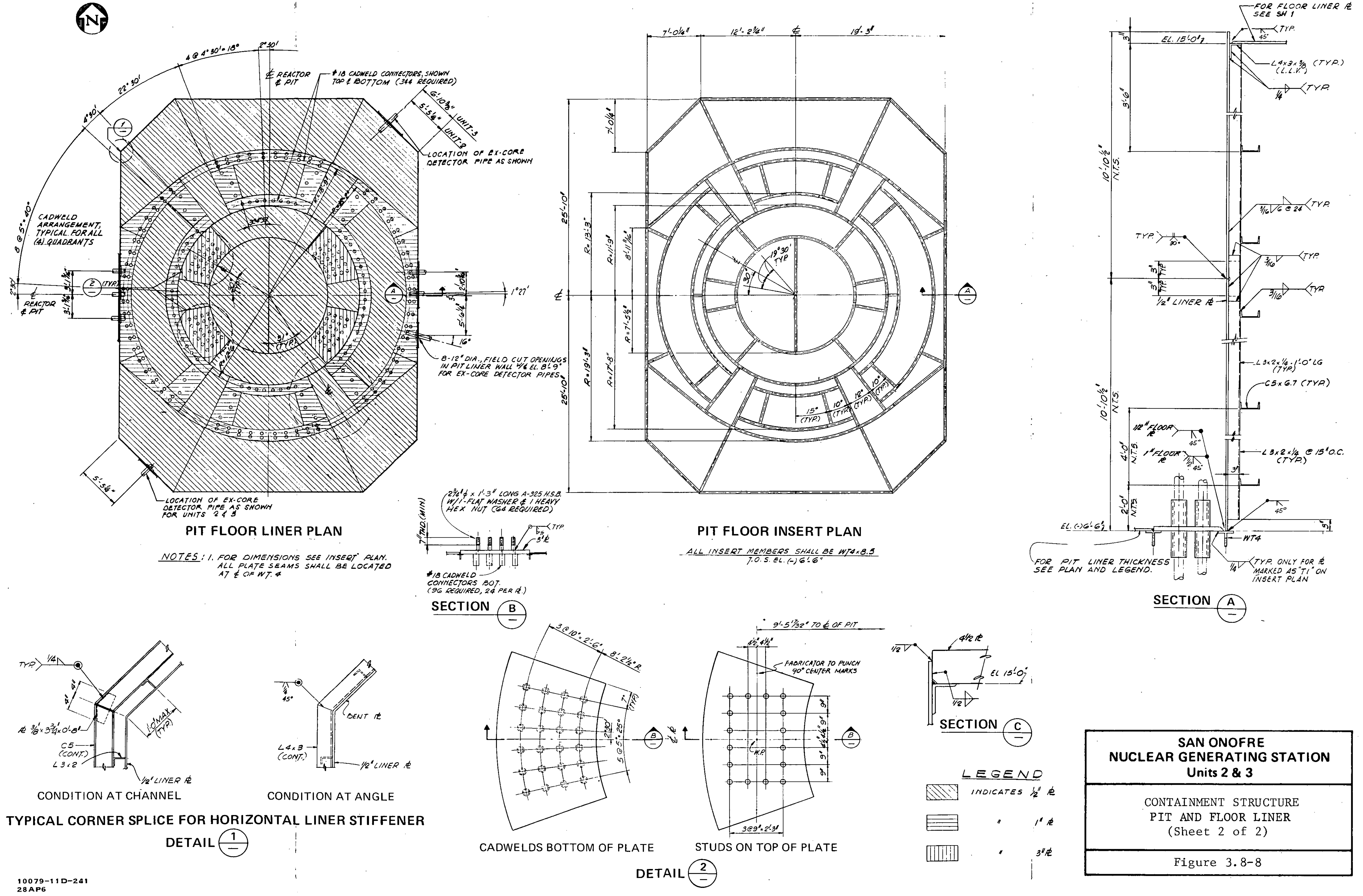
- INDICATES 1/4" LINER PL.
- INDICATES 1/2" LINER PL.
- INDICATES 1" LINER PL.
- INDICATES 3" LINER PL.
- INDICATES 4 1/2" LINER PL.

10079-11D-240
28AP6

SAN ONOFRE NUCLEAR GENERATING STATION
Units 2 & 3

CONTAINMENT STRUCTURE
PIT AND FLOOR LINER
(Sheet 1 of 2)

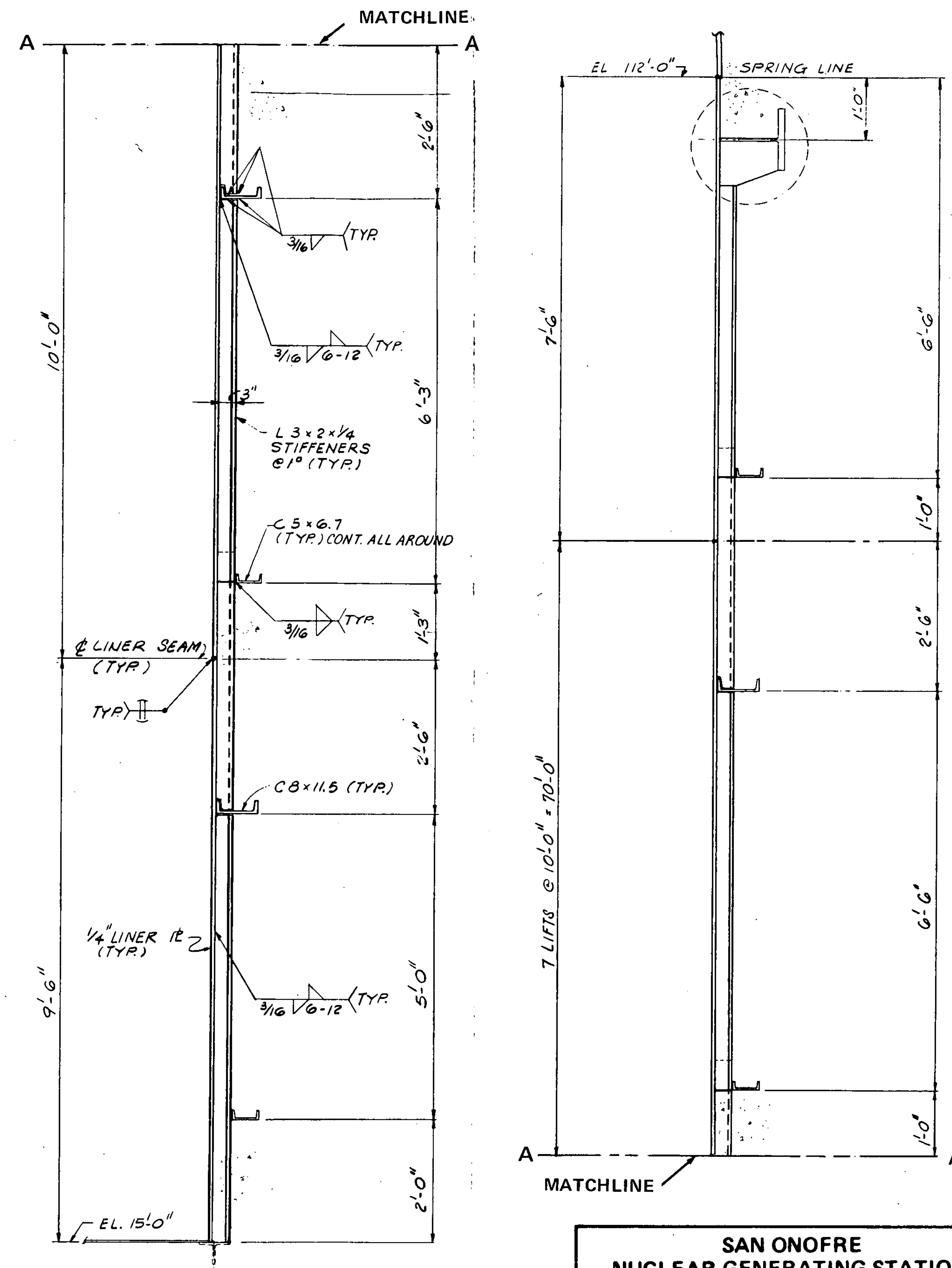
Figure 3.8-8



SAN ONOFRE NUCLEAR GENERATING STATION Units 2 & 3

CONTAINMENT STRUCTURE
PIT AND FLOOR LINER
(Sheet 2 of 2)

Figure 3.8-8

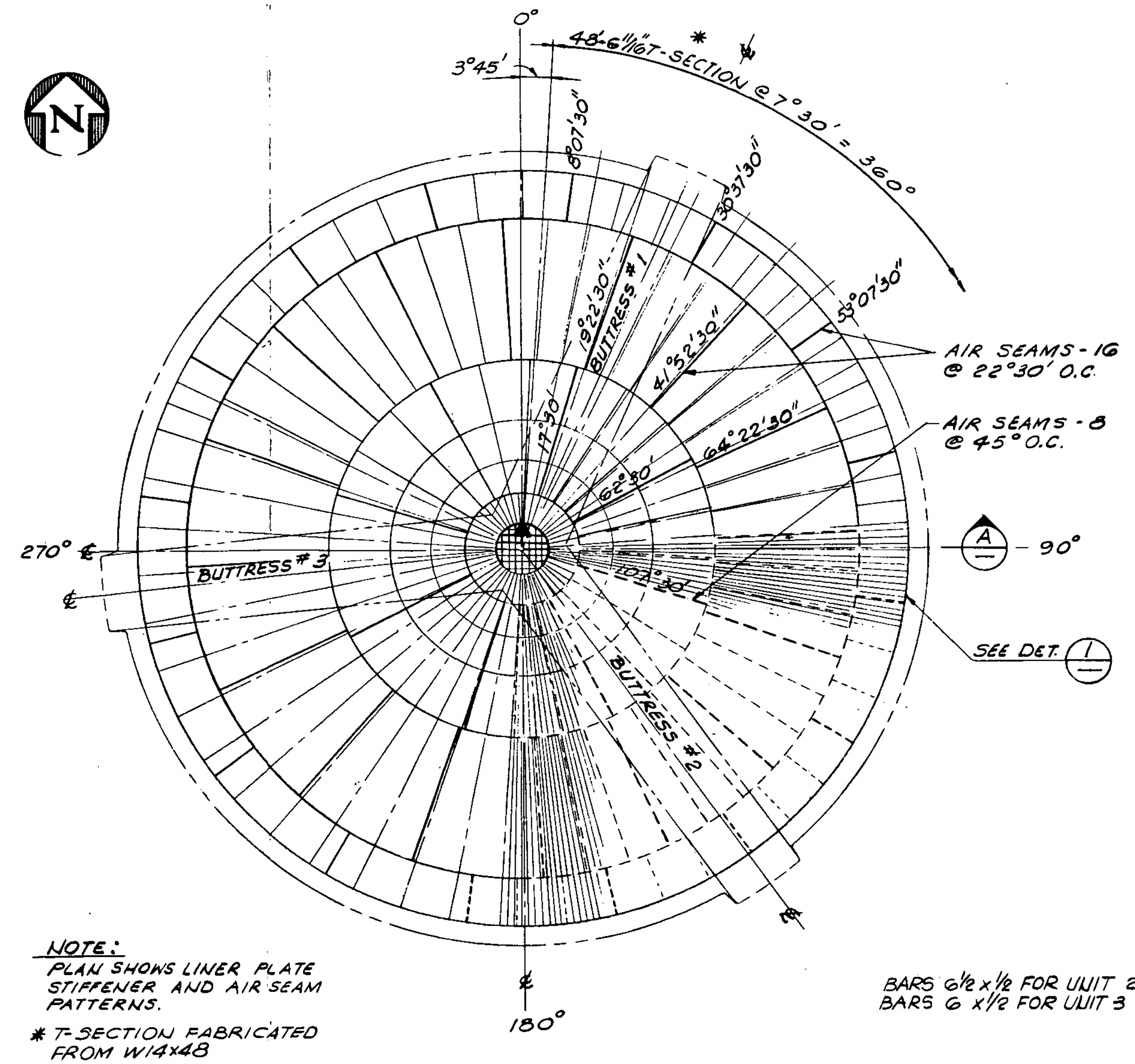


SAN ONOFRE
NUCLEAR GENERATING STATION
Units 2 & 3

SHELL WALL LINER

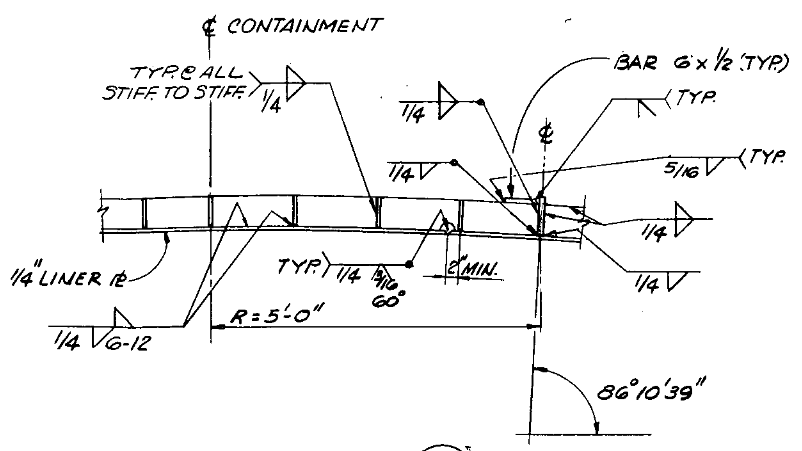
Figure 3.8-9

10079-11D-242
28AP6

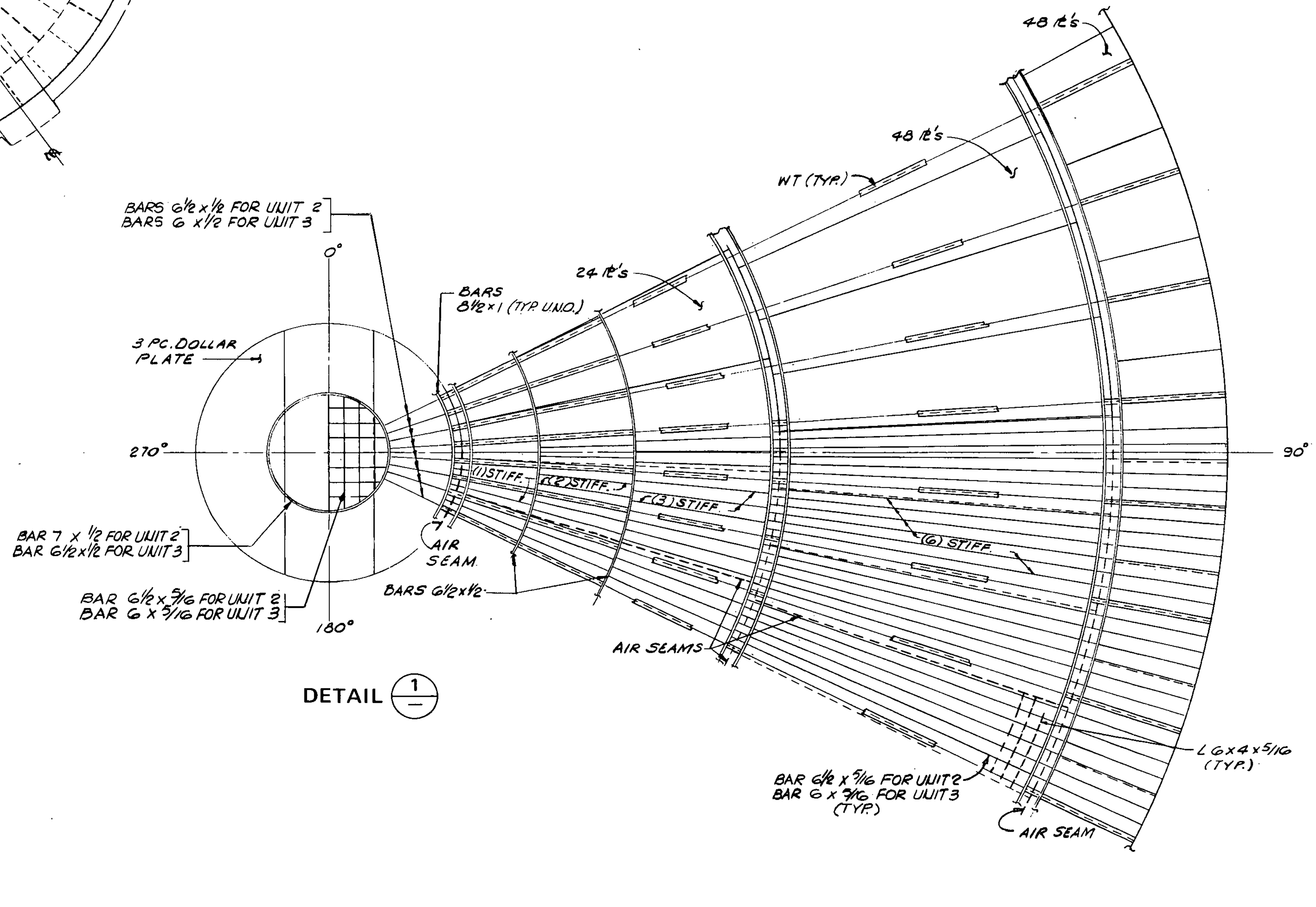


NOTE:
PLAN SHOWS LINER PLATE STIFFENER AND AIR SEAM PATTERNS.
* T-SECTION FABRICATED FROM W14x48

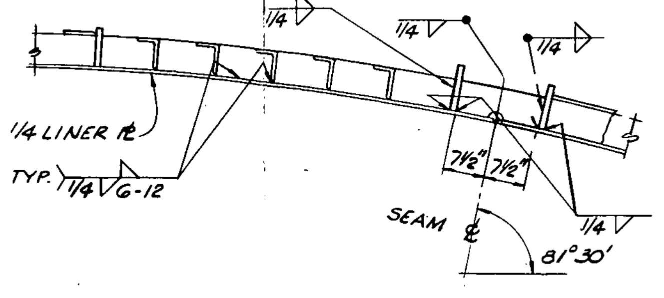
DOME PLAN



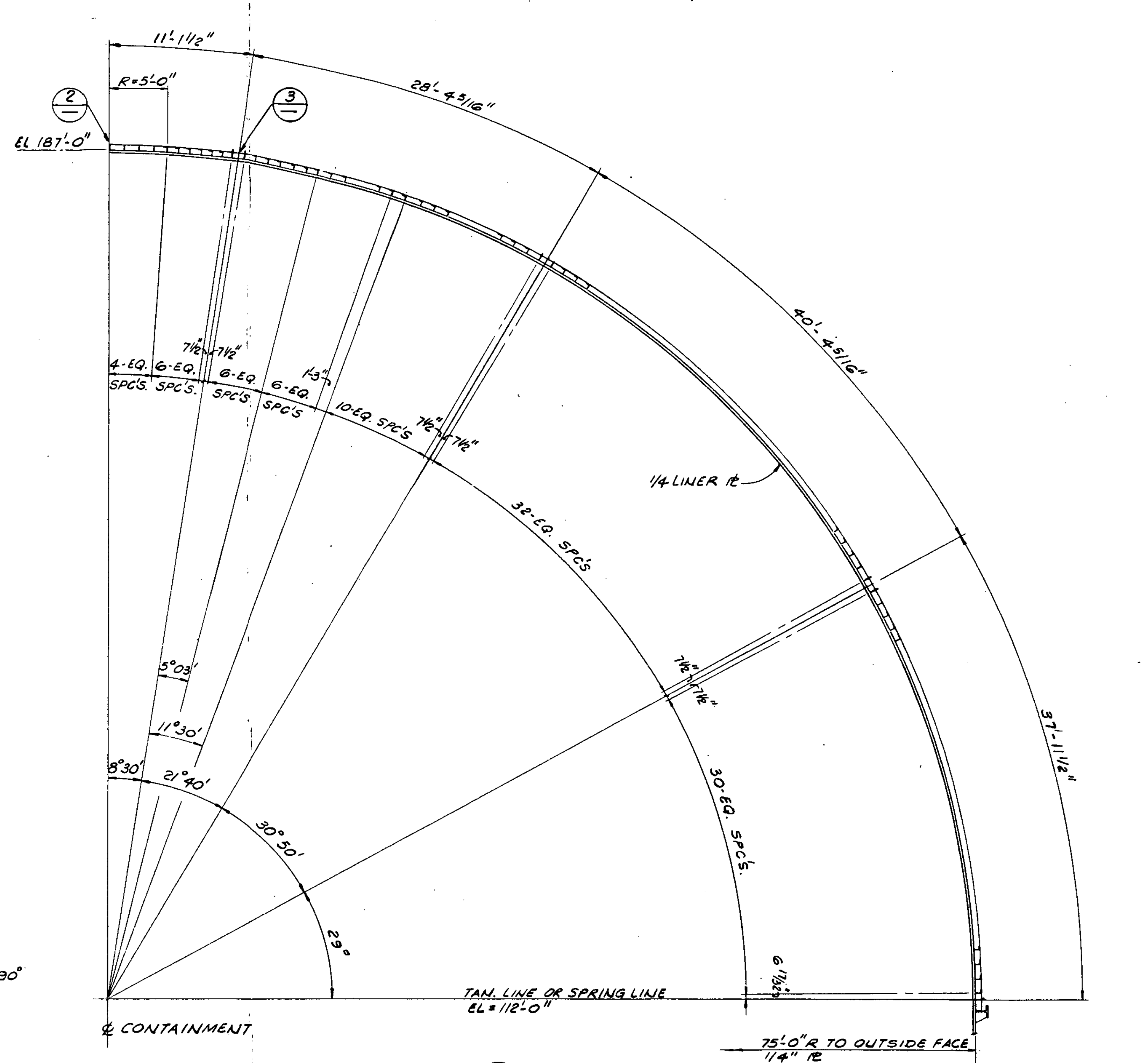
DETAIL 2



DETAIL 1



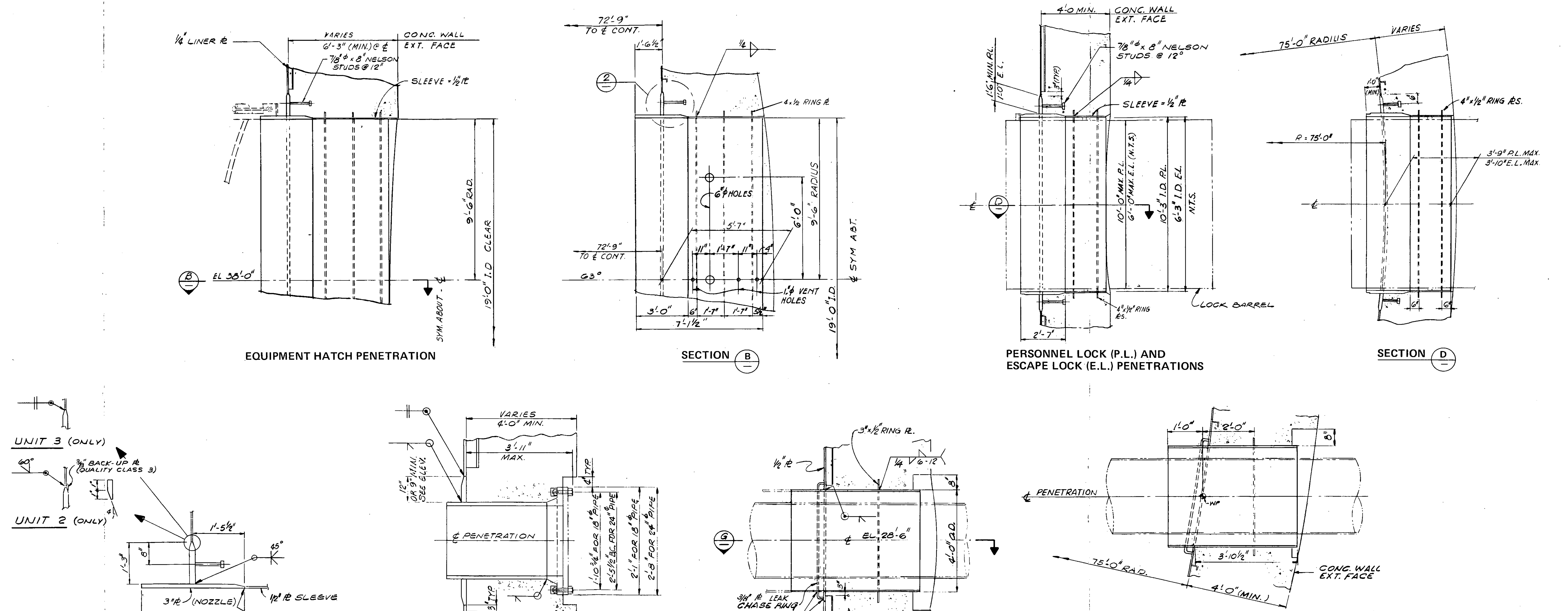
DETAIL 3



SECTION A

SAN ONOFRE NUCLEAR GENERATING STATION Units 2 & 3
DOME LINER
Figure 3.8-10

10079-11D-243
28AP6

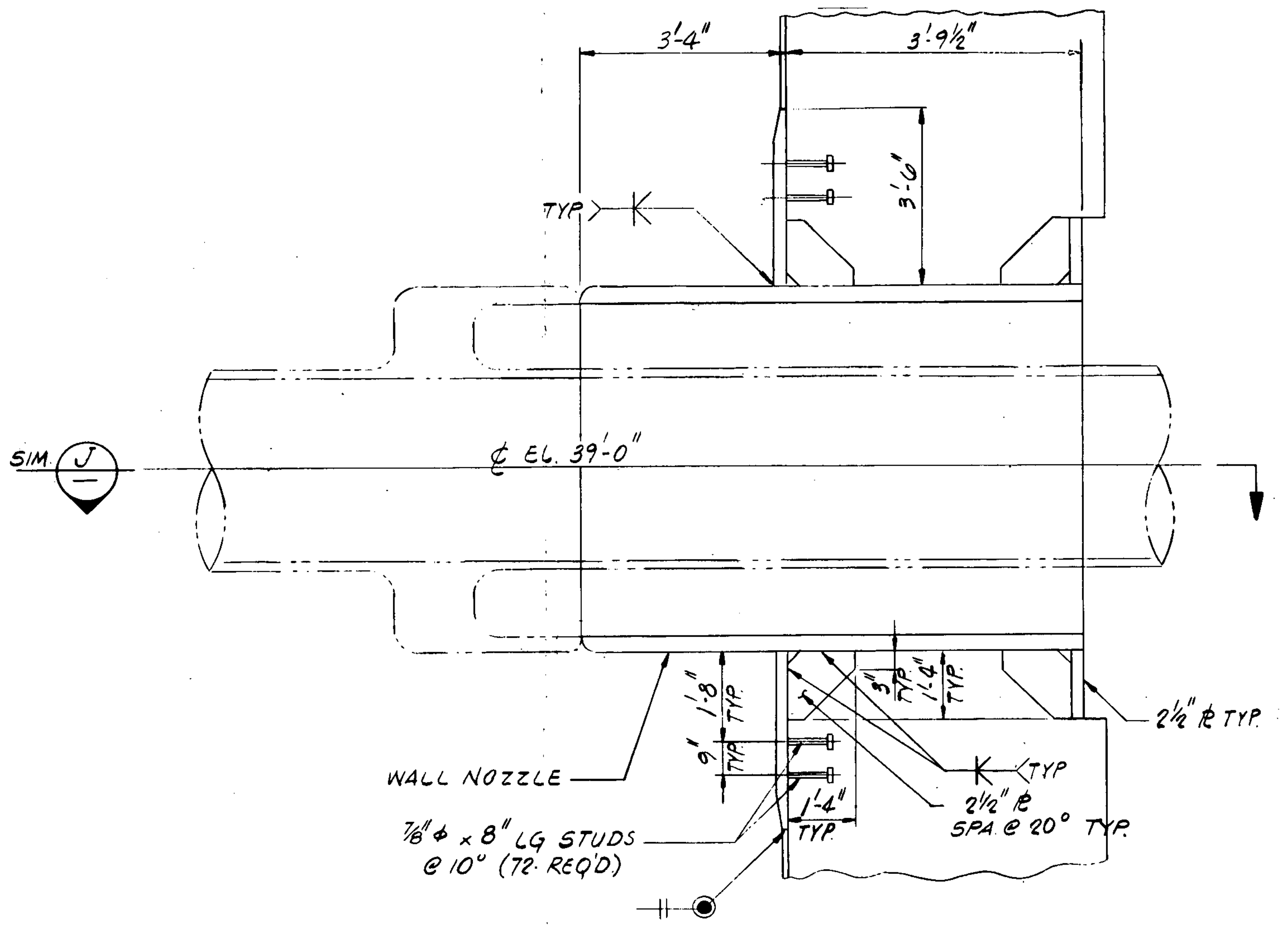


SAN ONOFRE NUCLEAR GENERATING STATION Units 2 & 3

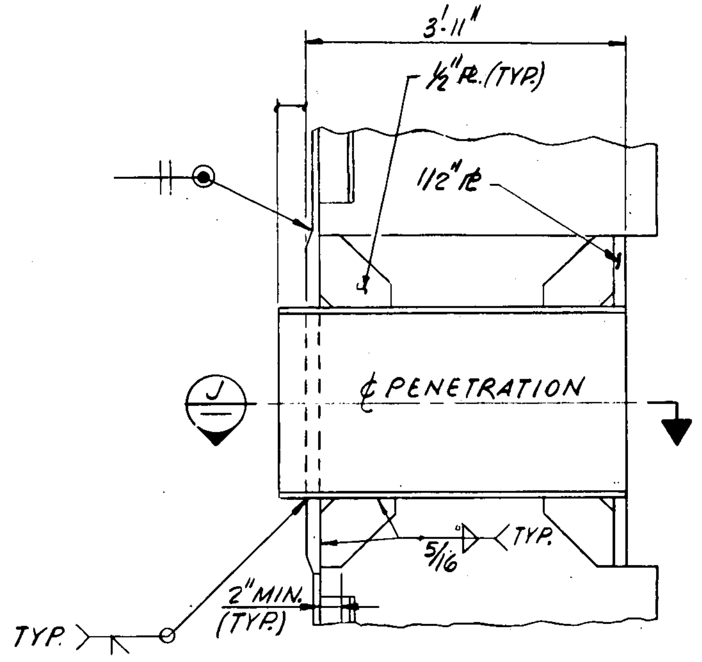
CONTAINMENT STRUCTURE
LINER PENETRATION ASSEMBLIES
(Sheet 1 of 2)

Figure 3.8-11

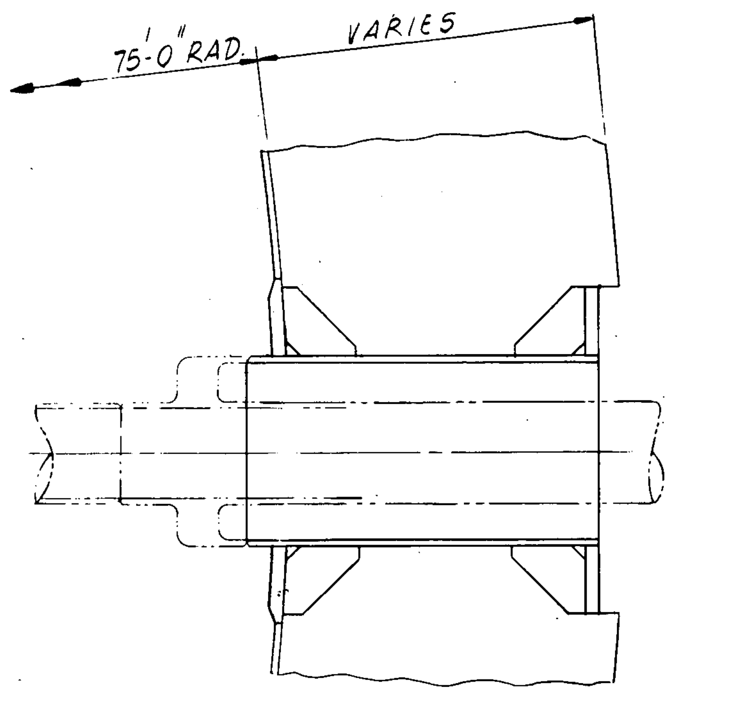
10079-11D-244
28AP6



TYPICAL HOT LINE PENETRATION



TYPICAL COLD PIPE PENETRATION



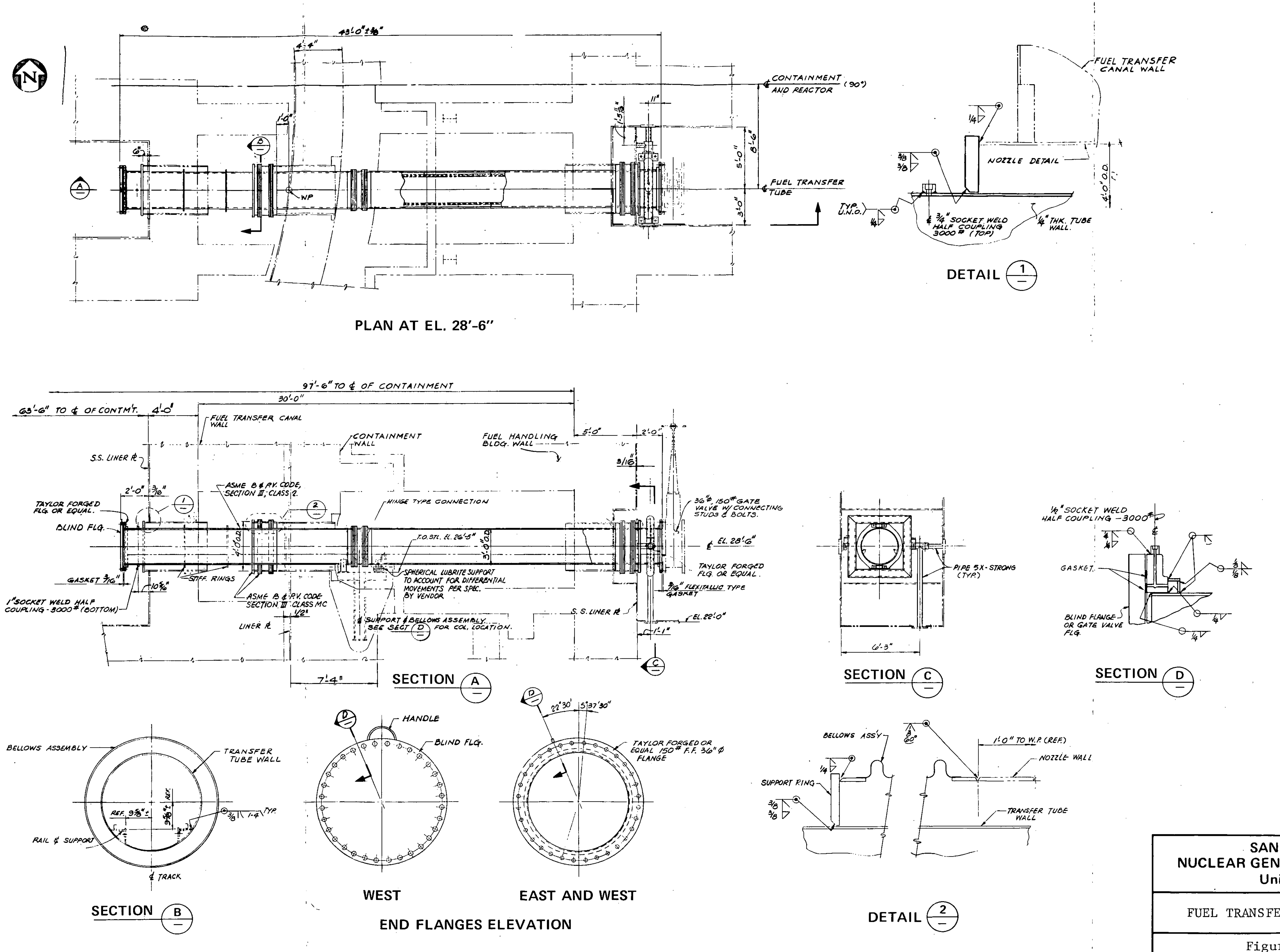
SECTION J

SAN ONOFRE
NUCLEAR GENERATING STATION
Units 2 & 3

CONTAINMENT STRUCTURE
LINER PENETRATION ASSEMBLIES
(Sheet 2 of 2)

Figure 3.8-11

10079-11D-245
26AU6

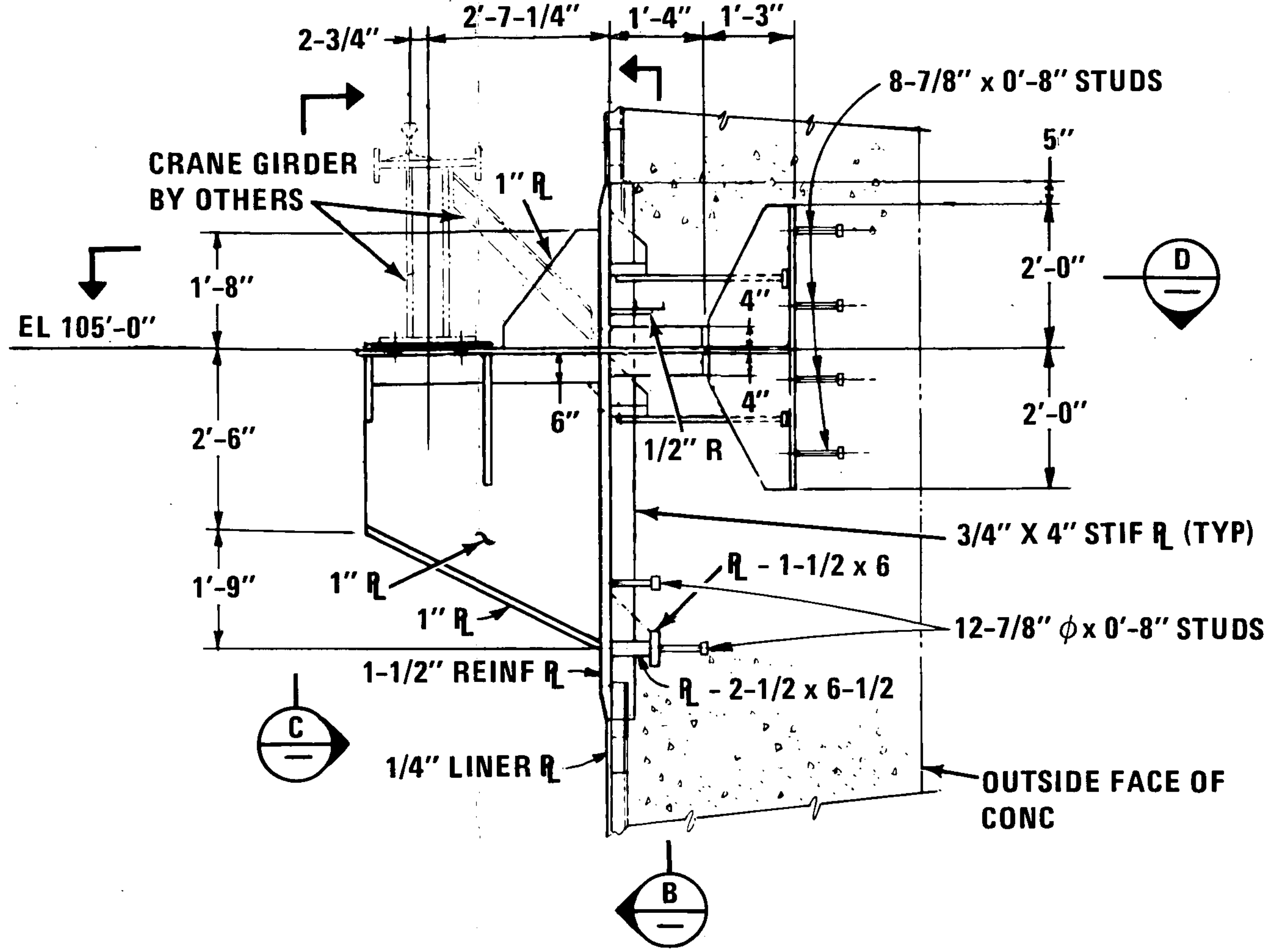


SAN ONOFRE NUCLEAR GENERATING STATION
Units 2 & 3

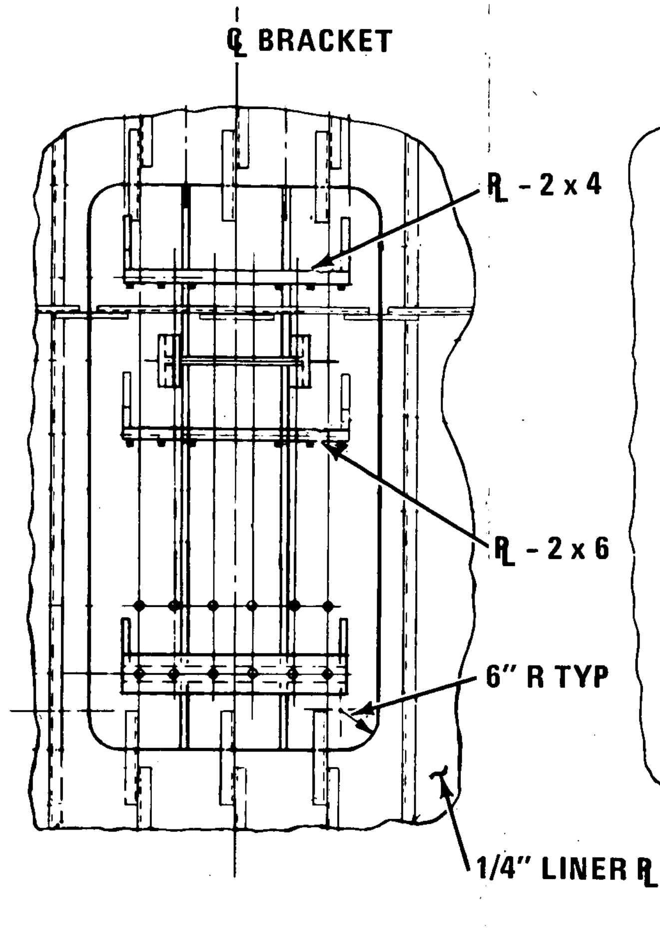
FUEL TRANSFER TUBE ASSEMBLY

Figure 3.8-12

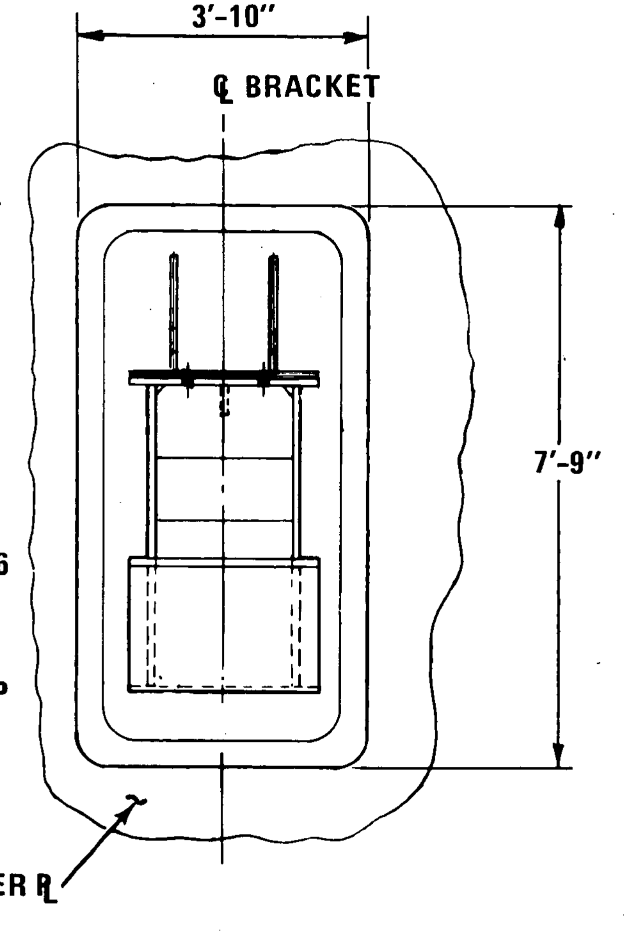
10079-11D-246
28AP6



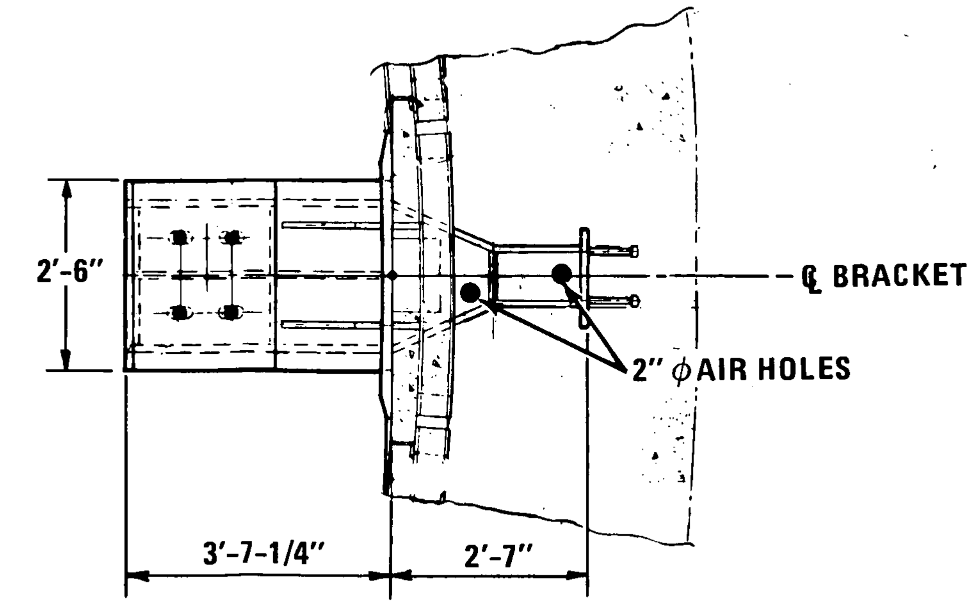
SIDE ELEVATION



SECTION B



SECTION C



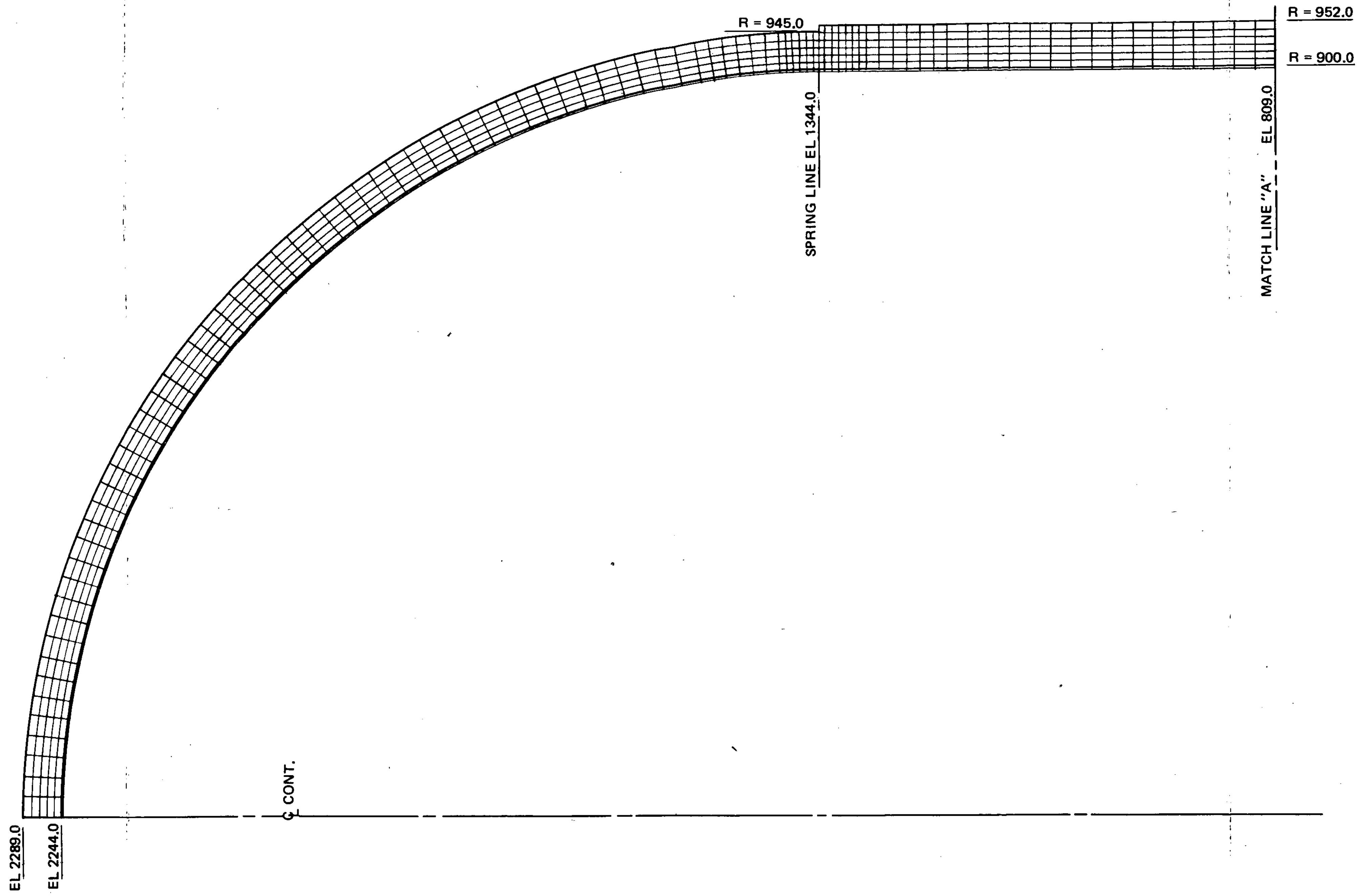
SECTION D

10079-11D-247
26AU6

SAN ONOFRE
NUCLEAR GENERATING STATION
Units 2 & 3

POLAR CRANE BRACKETS

Figure 3.8-13

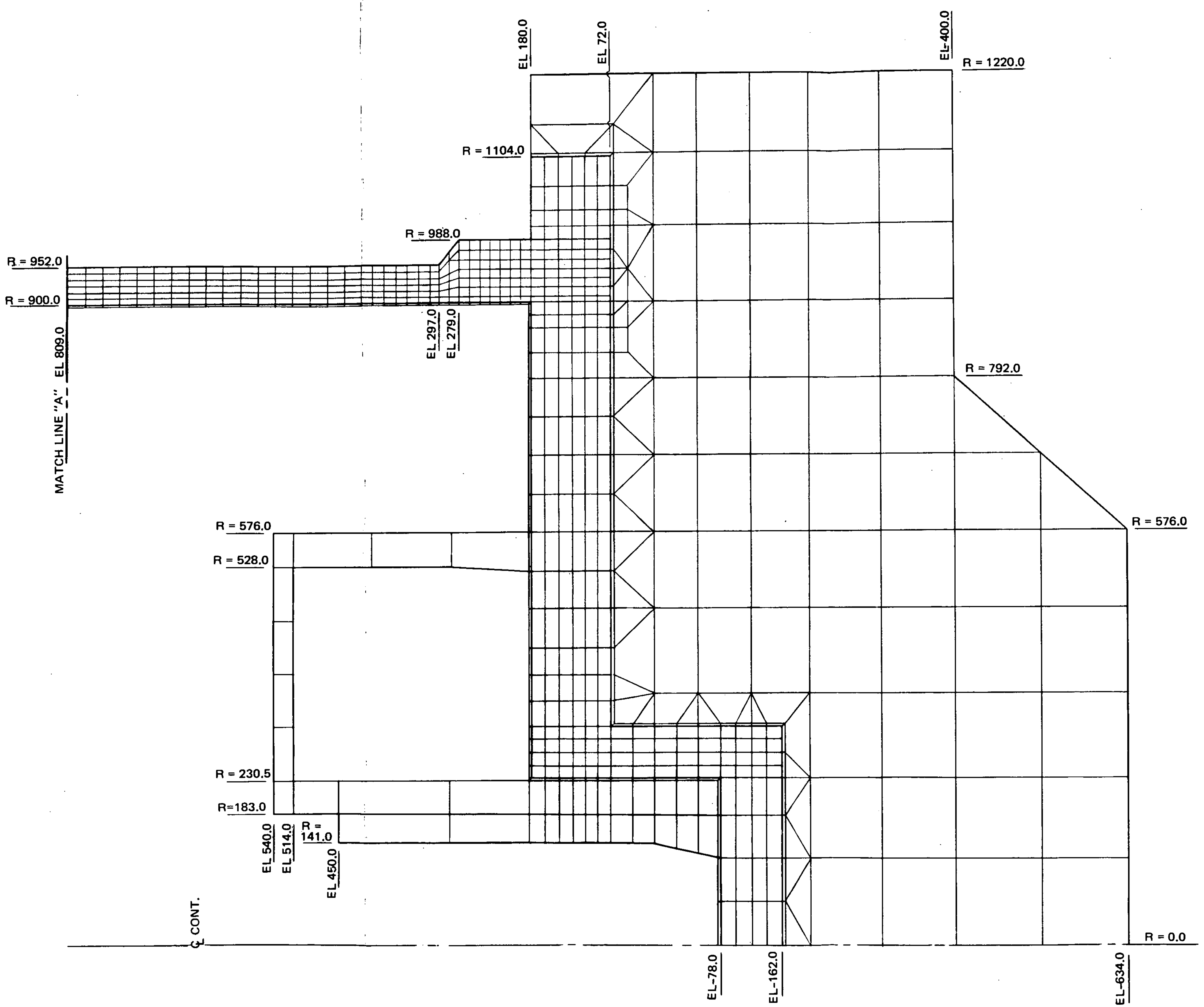


SAN ONOFRE
NUCLEAR GENERATING STATION
Units 2 & 3

CONTAINMENT STRUCTURE
FINITE ELEMENT LAYERED MESH
FOR FINEL ANALYSIS
(Sheet 1 of 4)

Figure 3.8-14

10079-11D-273
10MY6

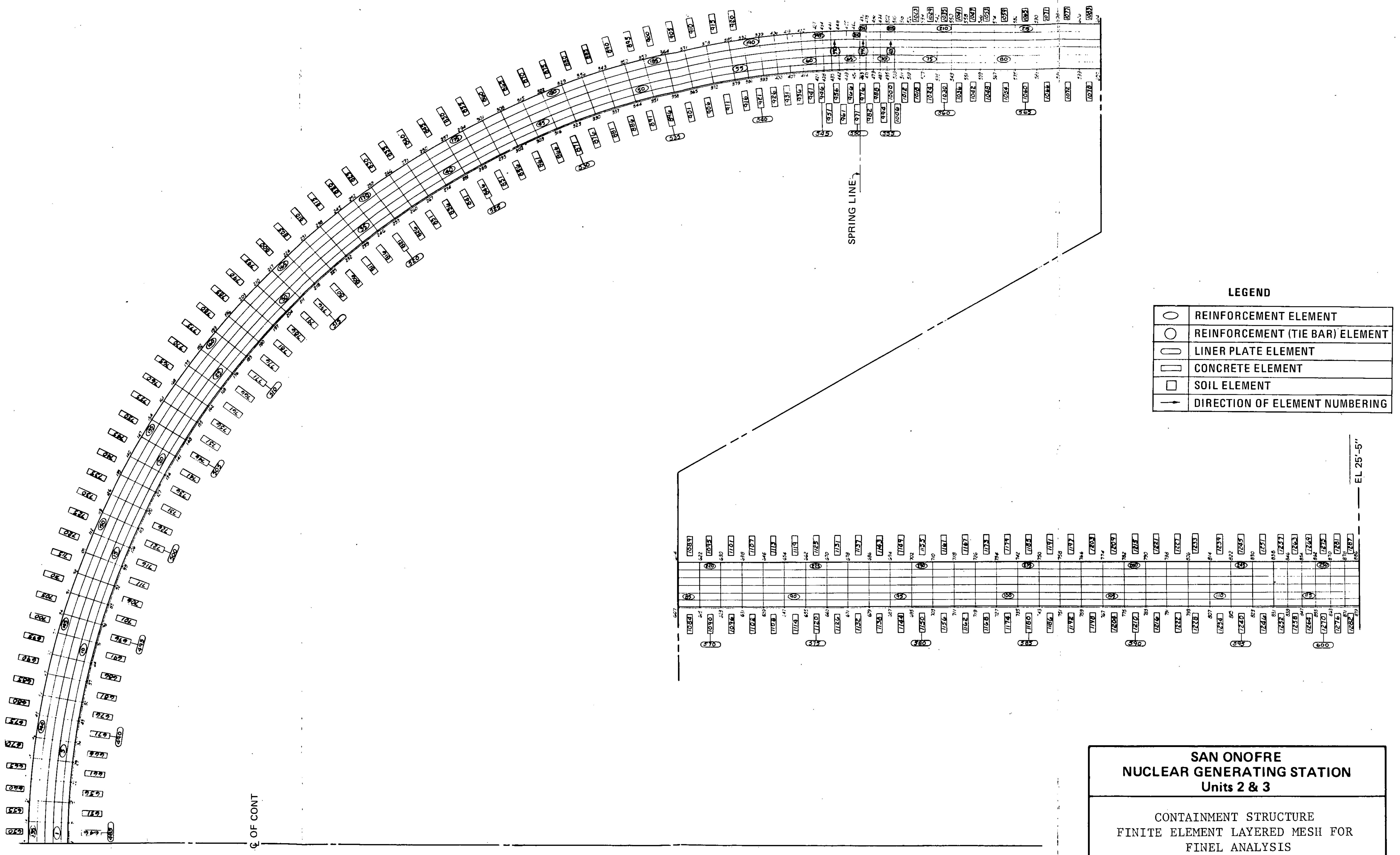


10079-11D-274
10MY6

SAN ONOFRE
NUCLEAR GENERATING STATION
Units 2 & 3

CONTAINMENT STRUCTURE
FINITE ELEMENT LAYERED MESH
FOR FINEL ANALYSIS
(Sheet 2 of 4)

Figure 3.8-14

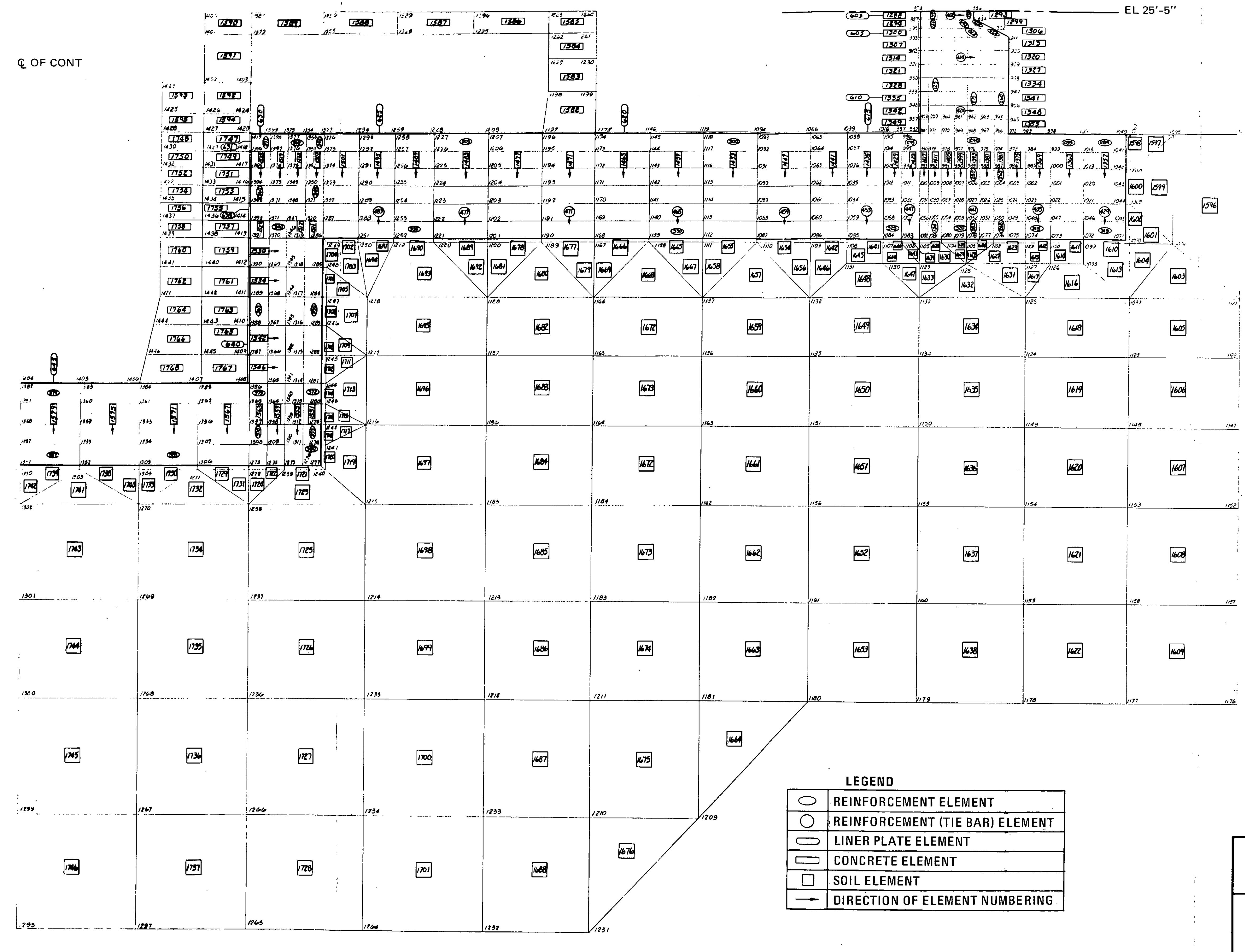


10079-11D-275
01OC6

SAN ONOFRE
NUCLEAR GENERATING STATION
Units 2 & 3

CONTAINMENT STRUCTURE
FINITE ELEMENT LAYERED MESH FOR
FINEL ANALYSIS
(Sheet 3 of 4)

Figure 3.8-14

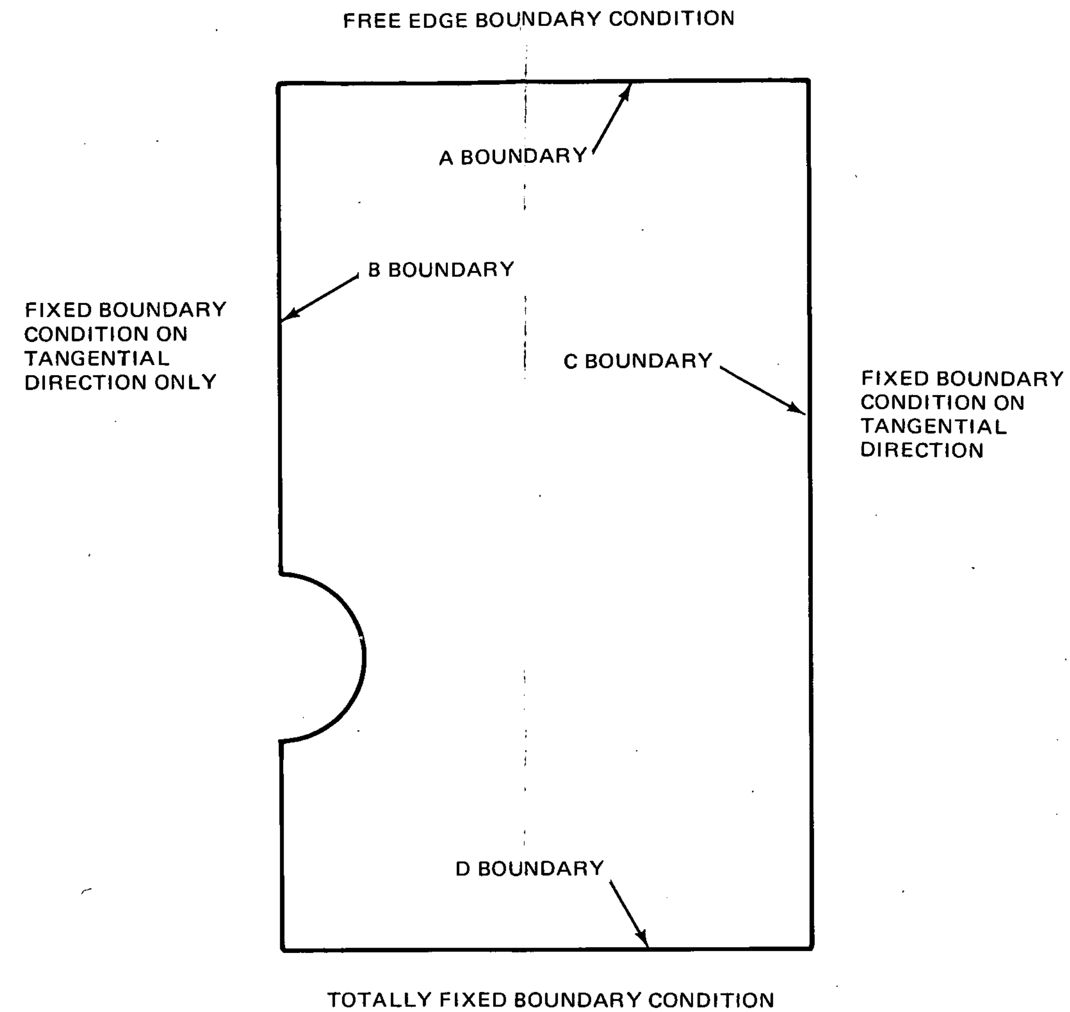


SAN ONOFRE
NUCLEAR GENERATING STATION
Units 2 & 3

CONTAINMENT STRUCTURE
FINITE ELEMENT LAYERED MESH FOR
FINEL ANALYSIS
(Sheet 4 of 4)

Figure 3.8-14

10079-11D-276
01OC6

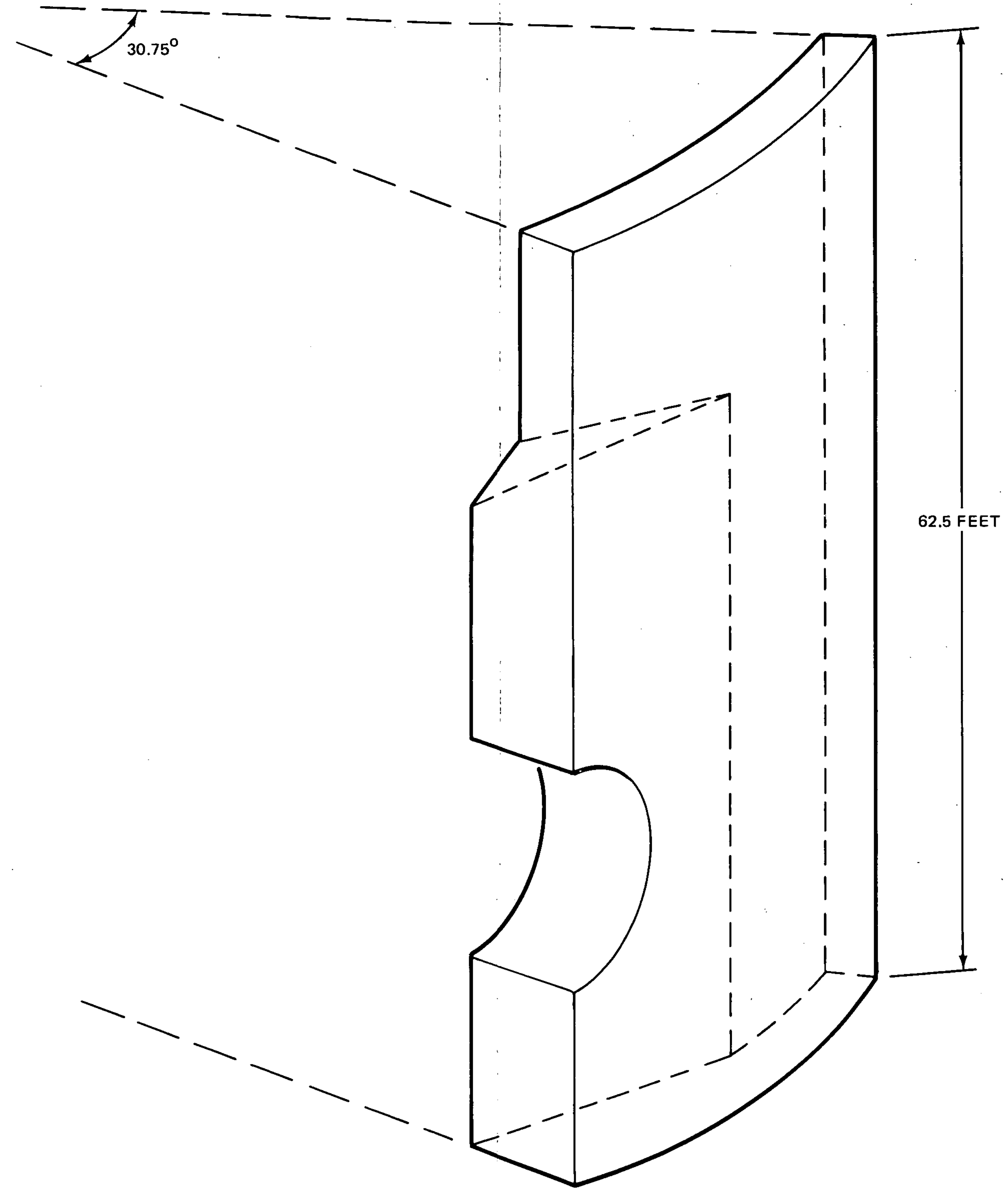


10079-11D-316
21MY6

SAN ONOFRE
NUCLEAR GENERATING STATION
Units 2 & 3

EQUIPMENT HATCH
BOUNDARY CONDITIONS
FOR FINITE ELEMENT ANALYSIS

Figure 3.8-15

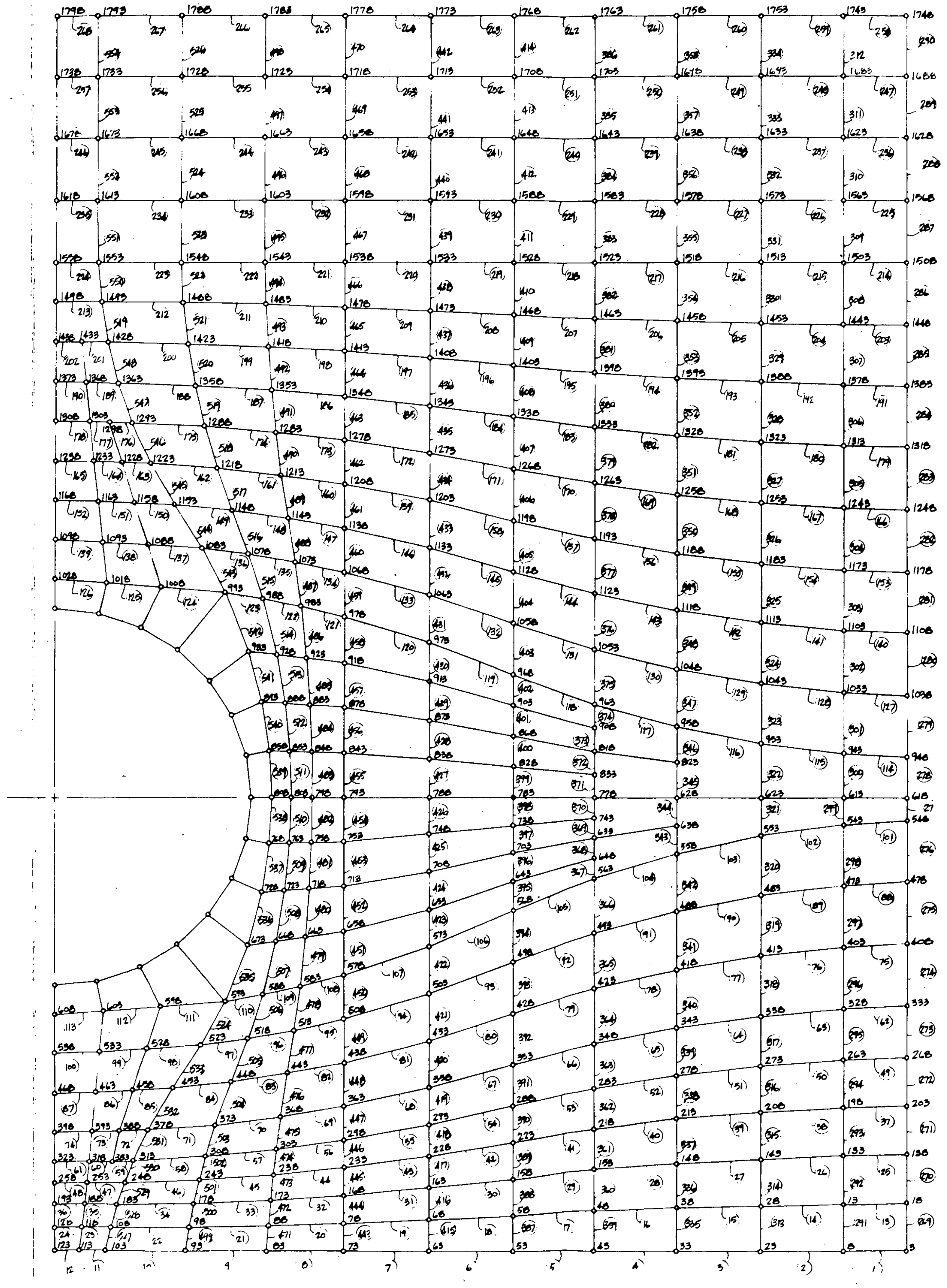


SAN ONOFRE
NUCLEAR GENERATING STATION
Units 2 & 3

EQUIPMENT HATCH
PENETRATION REGION
FINITE ELEMENT MODEL GEOMETRY

Figure 3.8-16

10079-11D-317
21MY6



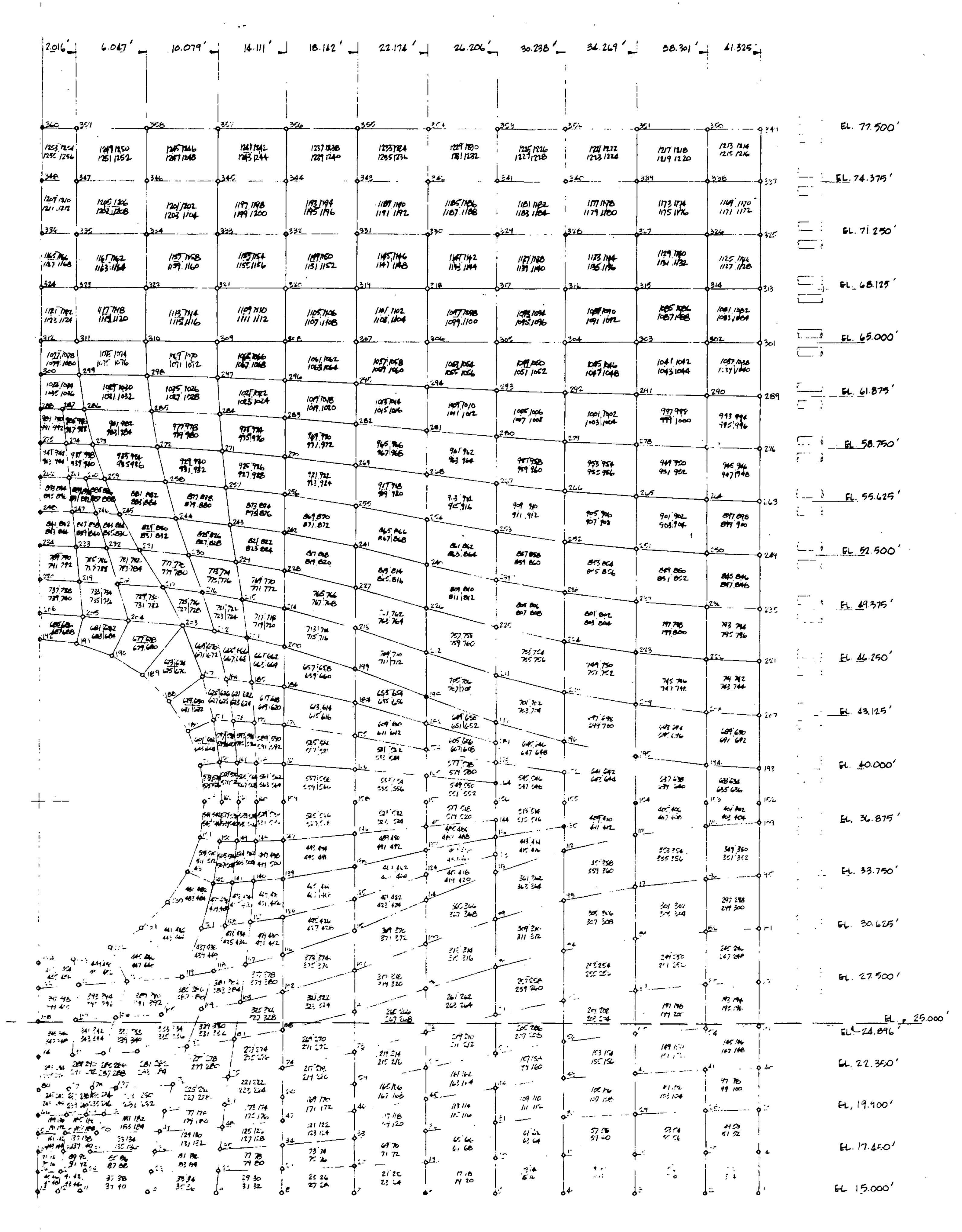
TRUSS ELEMENTS - NODE POINTS

SAN ONOFRE
NUCLEAR GENERATING STATION
Units 2 & 3

EQUIPMENT HATCH
FINITE ELEMENT ANALYSIS
(Sheet 1 of 4)

Figure 3.8-17

10079-11D-318
21MY6



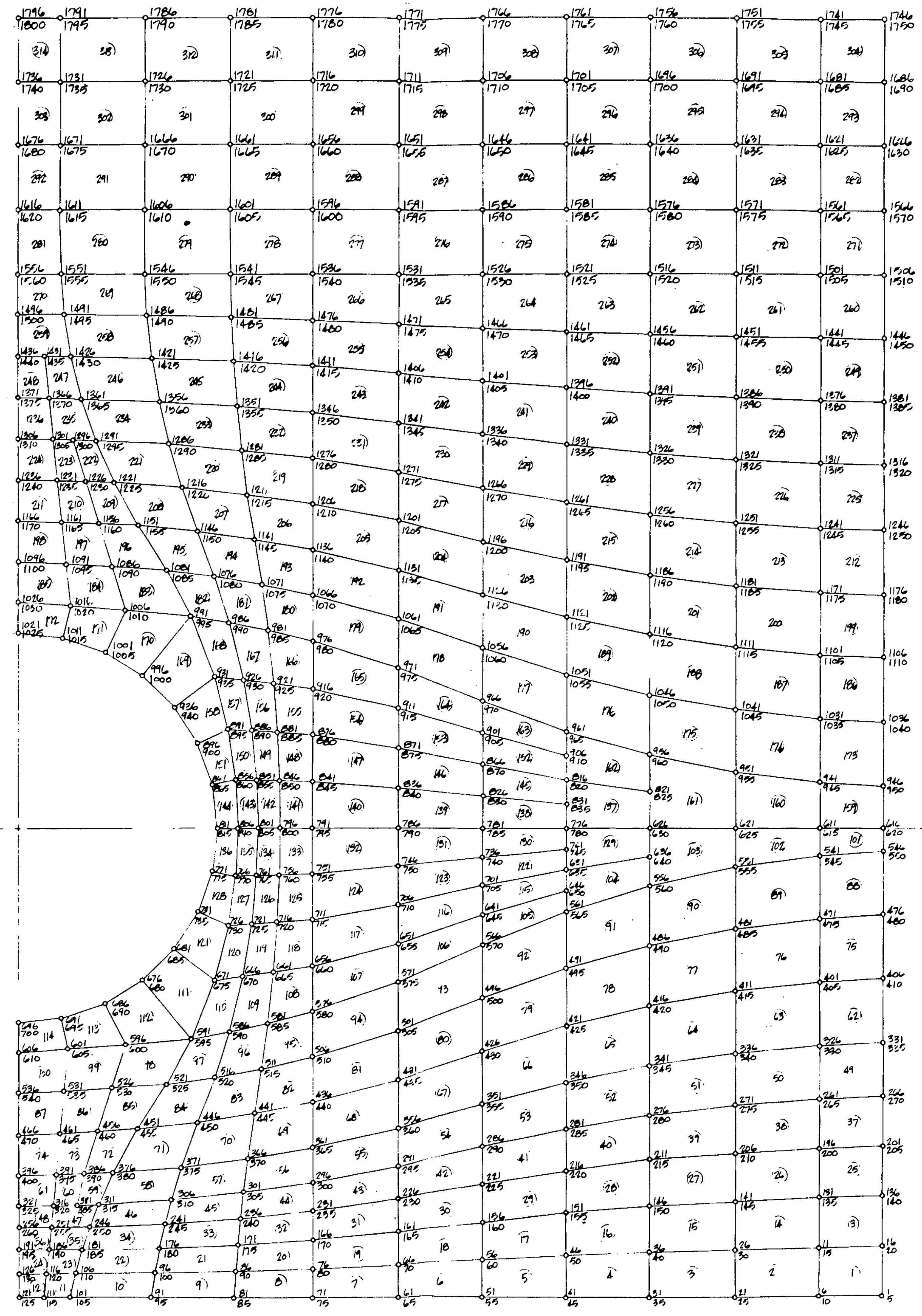
BRICK ELEMENTS - GRID POINTS

10079-11D-319
21MY6

SAN ONOFRE
NUCLEAR GENERATING STATION
Units 2 & 3

EQUIPMENT HATCH
FINITE ELEMENT ANALYSIS
(Sheet 2 of 4)

Figure 3.8-17



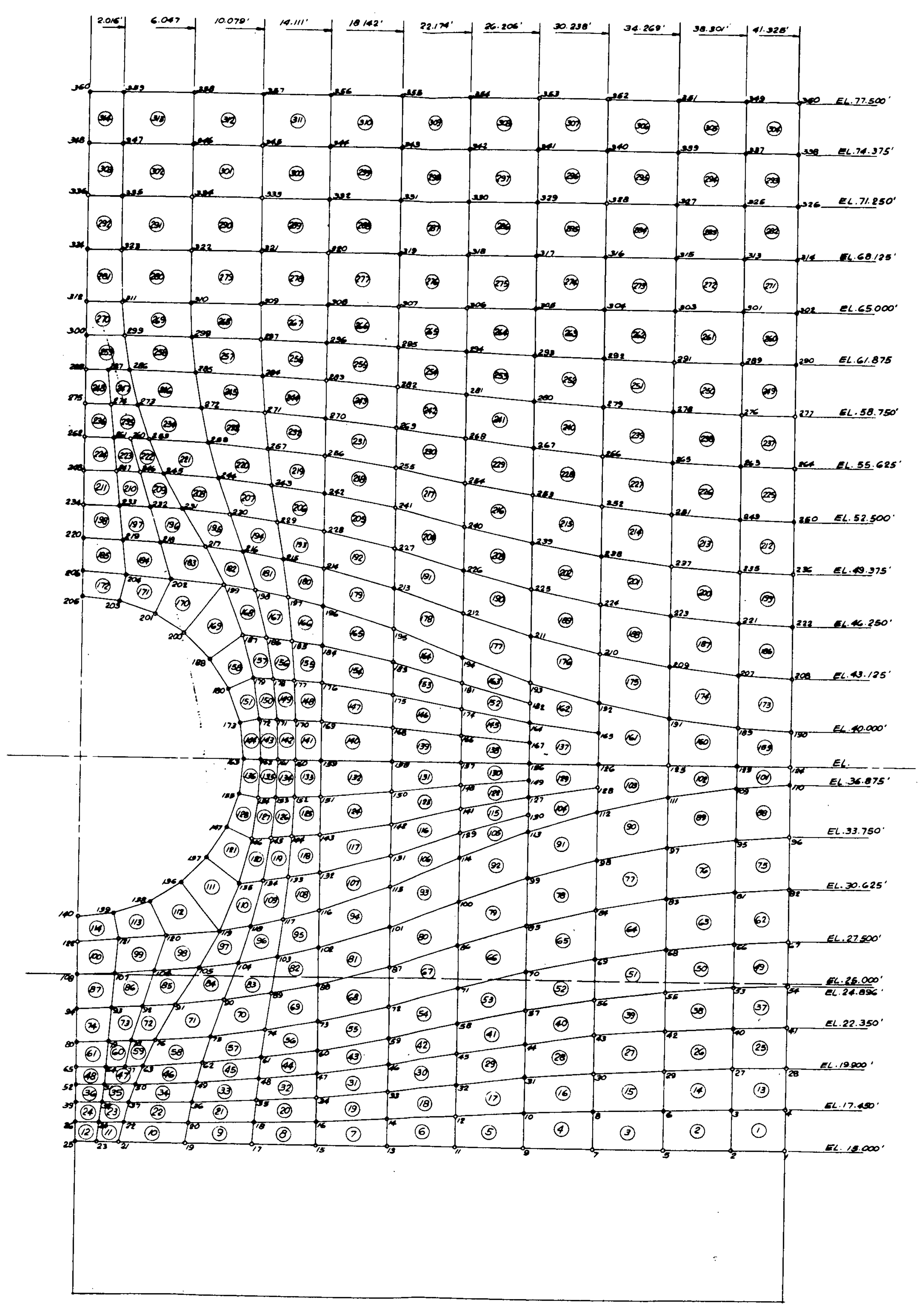
GRID ELEMENTS - THREE DIMENSIONAL NODE POINTS

SAN ONOFRE
NUCLEAR GENERATING STATION
Units 2 & 3

EQUIPMENT HATCH
FINITE ELEMENT ANALYSIS
(Sheet 3 of 4)

Figure 3.8-17

10079-11D-320
21MY6



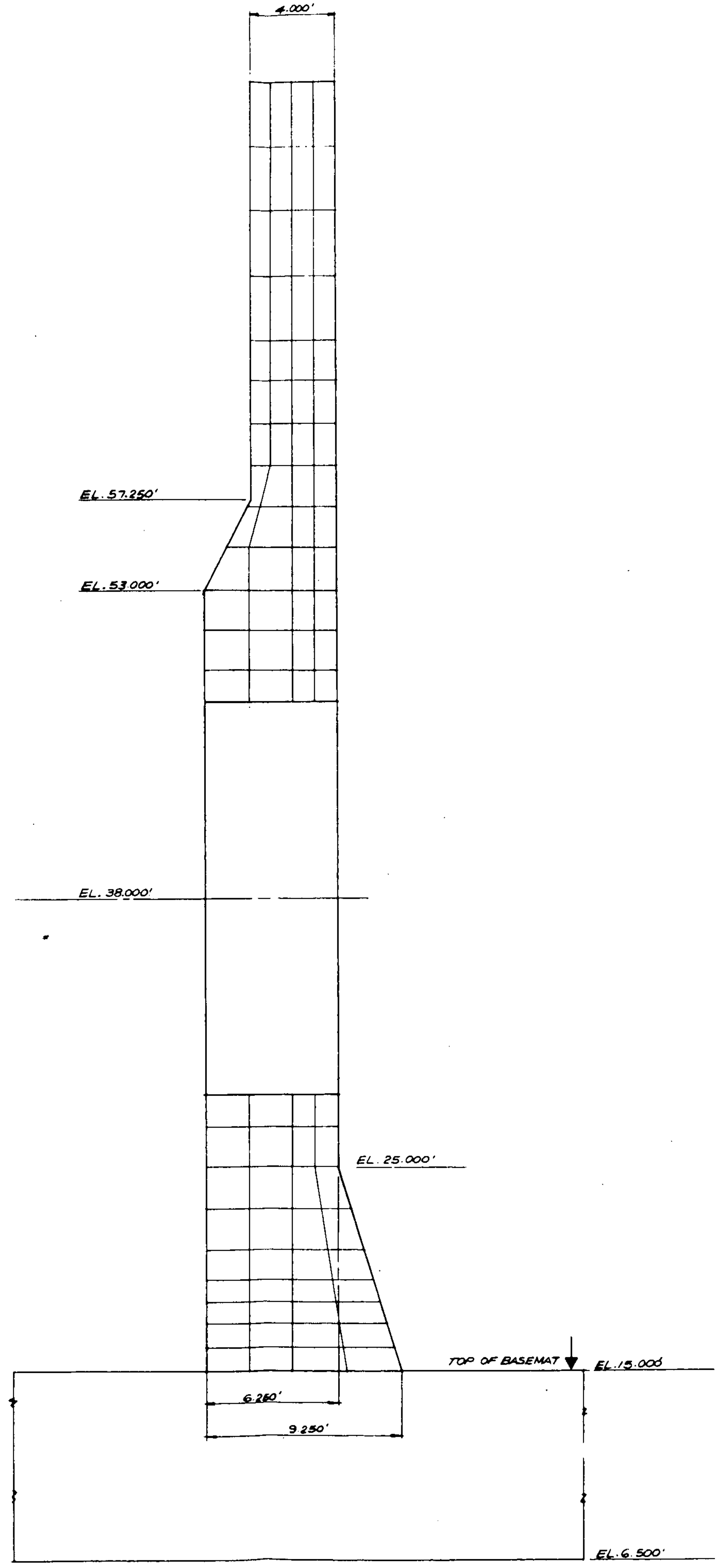
BRICK ELEMENTS - NEW GRID POINTS

SAN ONOFRE NUCLEAR GENERATING STATION Units 2 & 3

EQUIPMENT HATCH
FINITE ELEMENT ANALYSIS
(Sheet 4 of 4)

Figure 3.8-17

10079-11D-321
21MY6



10079-11D-322
21MY6

SAN ONOFRE
NUCLEAR GENERATING STATION
Units 2 & 3

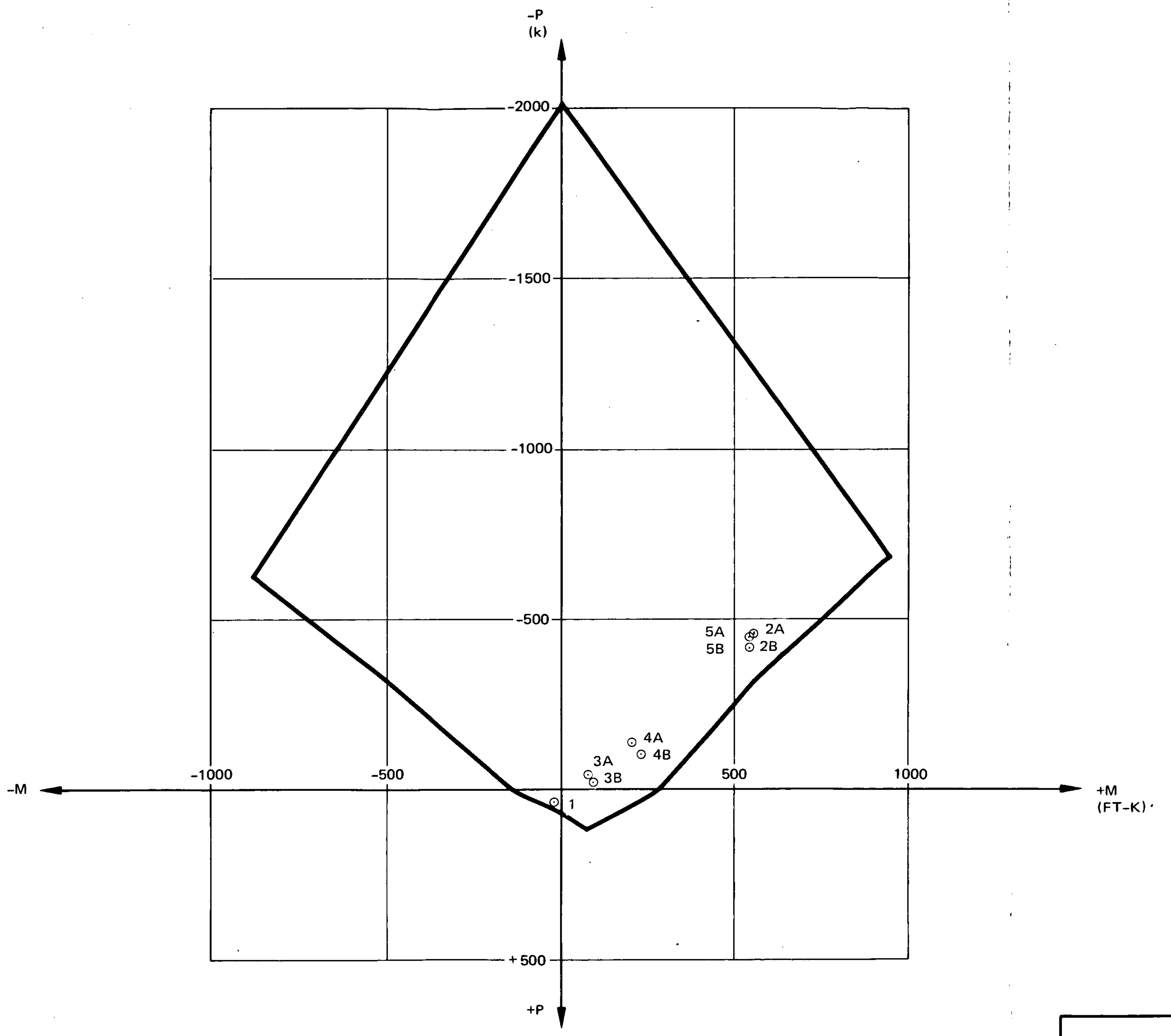
EQUIPMENT HATCH
LAYERED ELEMENTS THROUGH
WALL THICKNESS

Figure 3.8-18

GOVERNING LOADING COMBINATIONS:

1. $D + F + T_A + 1.5P$
2. $D + F + T_A + 1.25P \pm 1.25E$
3. $D + F + T_A \pm 1.25E$
4. $D + F + T_A + P \pm E'$
5. $D + F + T_A \pm E'$

10079-11D-277
11MY6



SECTION 1 – MERIDIONAL

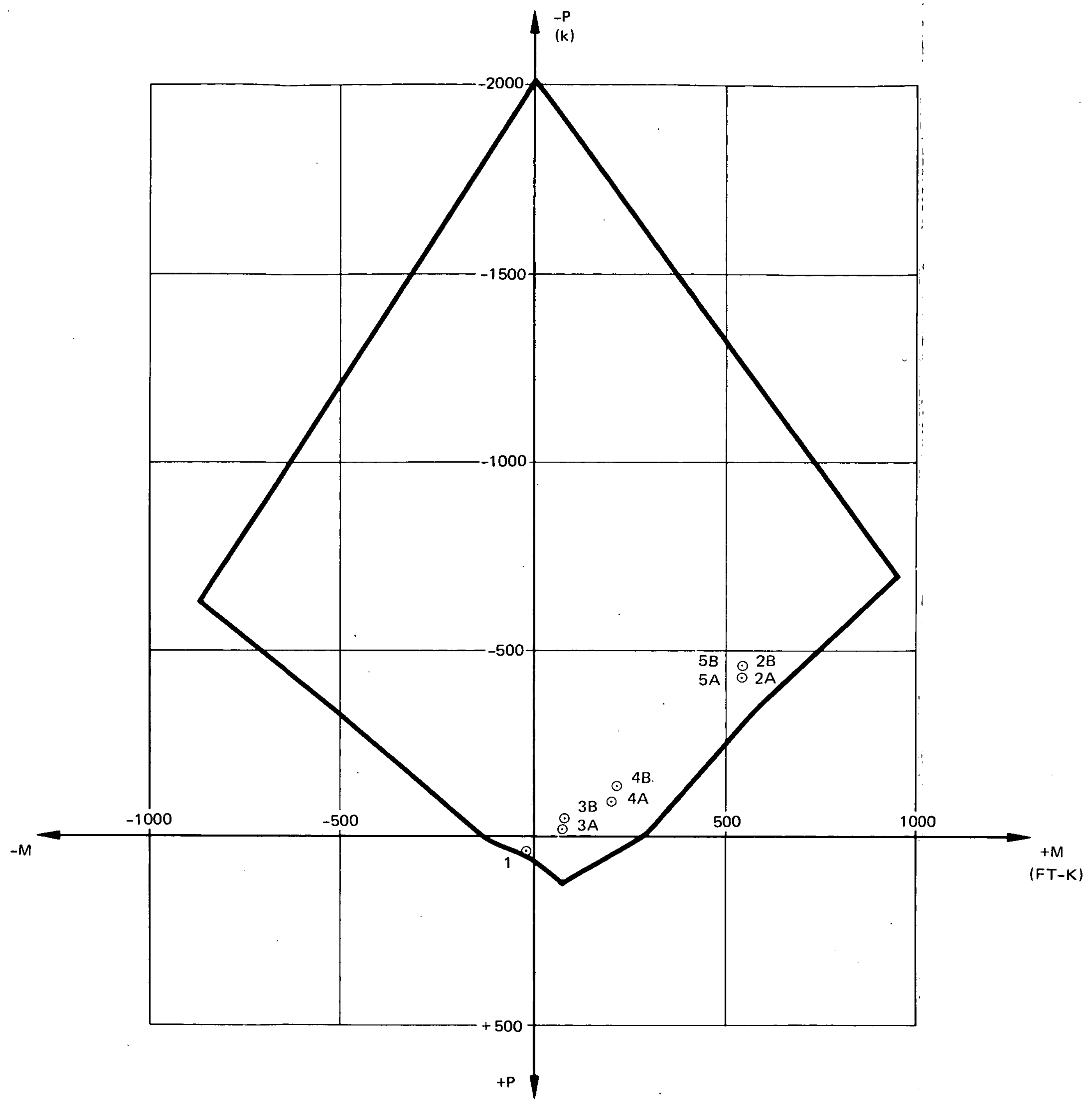
NOTES:

1. REFER TO TABLE 3.8-1, SHEET 1, FOR SECTION NUMBER IDENTIFICATION
2. NOTATIONS "A" AND "B" IDENTIFY "PLUS" AND "MINUS" LOAD COMBINATIONS RESPECTIVELY.

SAN ONOFRE NUCLEAR GENERATING STATION Units 2 & 3
CONTAINMENT STRUCTURE INTERACTION DIAGRAMS WITH ACTUAL RESULTS OF PRINCIPAL LOAD COMBINATIONS (Sheet 1 of 18)
Figure 3.8-19

GOVERNING LOADING COMBINATIONS:

1. $D + F + T_A + 1.5P$
2. $D + F + T_A + 1.25P \pm 1.25E$
3. $D + F + T_A \pm 1.25E$
4. $D + F + T_A + P \pm E'$
5. $D + F + T_A \pm E'$



SECTION 1 – HOOP

NOTES:

1. REFER TO TABLE 3.8-1, SHEET 1, FOR SECTION NUMBER IDENTIFICATION
2. NOTATIONS "A" AND "B" IDENTIFY "PLUS" AND "MINUS" LOAD COMBINATIONS RESPECTIVELY.

SAN ONOFRE
NUCLEAR GENERATING STATION
Units 2 & 3

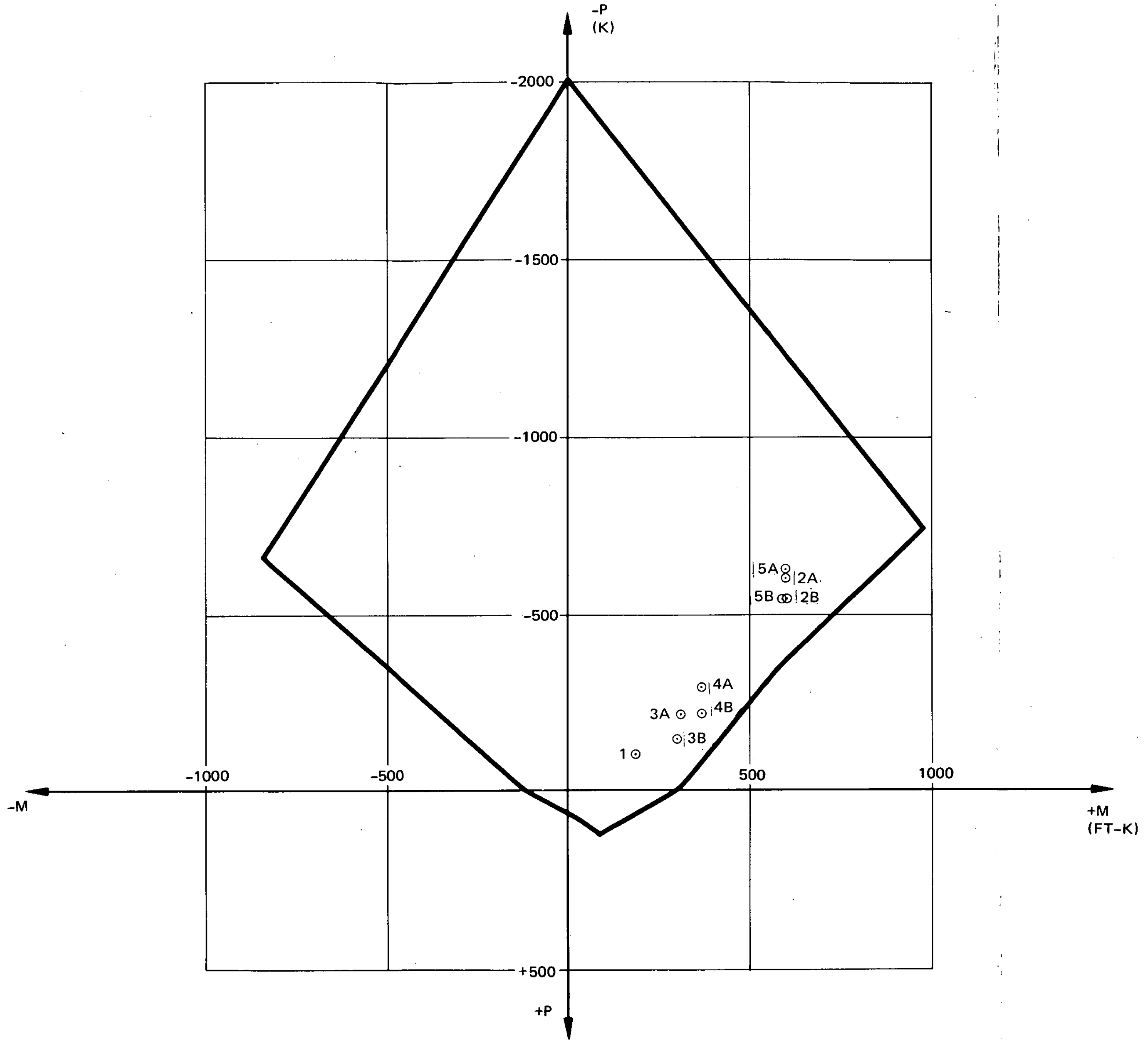
CONTAINMENT STRUCTURE
INTERACTION DIAGRAMS
WITH ACTUAL RESULTS OF
PRINCIPAL LOAD COMBINATIONS
(Sheet 2 of 18)

Figure 3.8-19

10079-11D-278
11MY6

GOVERNING LOADING COMBINATIONS:

1. $D + F + T_A + 1.5P$
2. $D + F + T_A + 1.25P \pm 1.25E$
3. $D + F + T_A \pm 1.25E$
4. $D + F + T_A + P \pm E'$
5. $D + F + T_A \pm E'$



SECTION 2 – MERIDIONAL

NOTES:

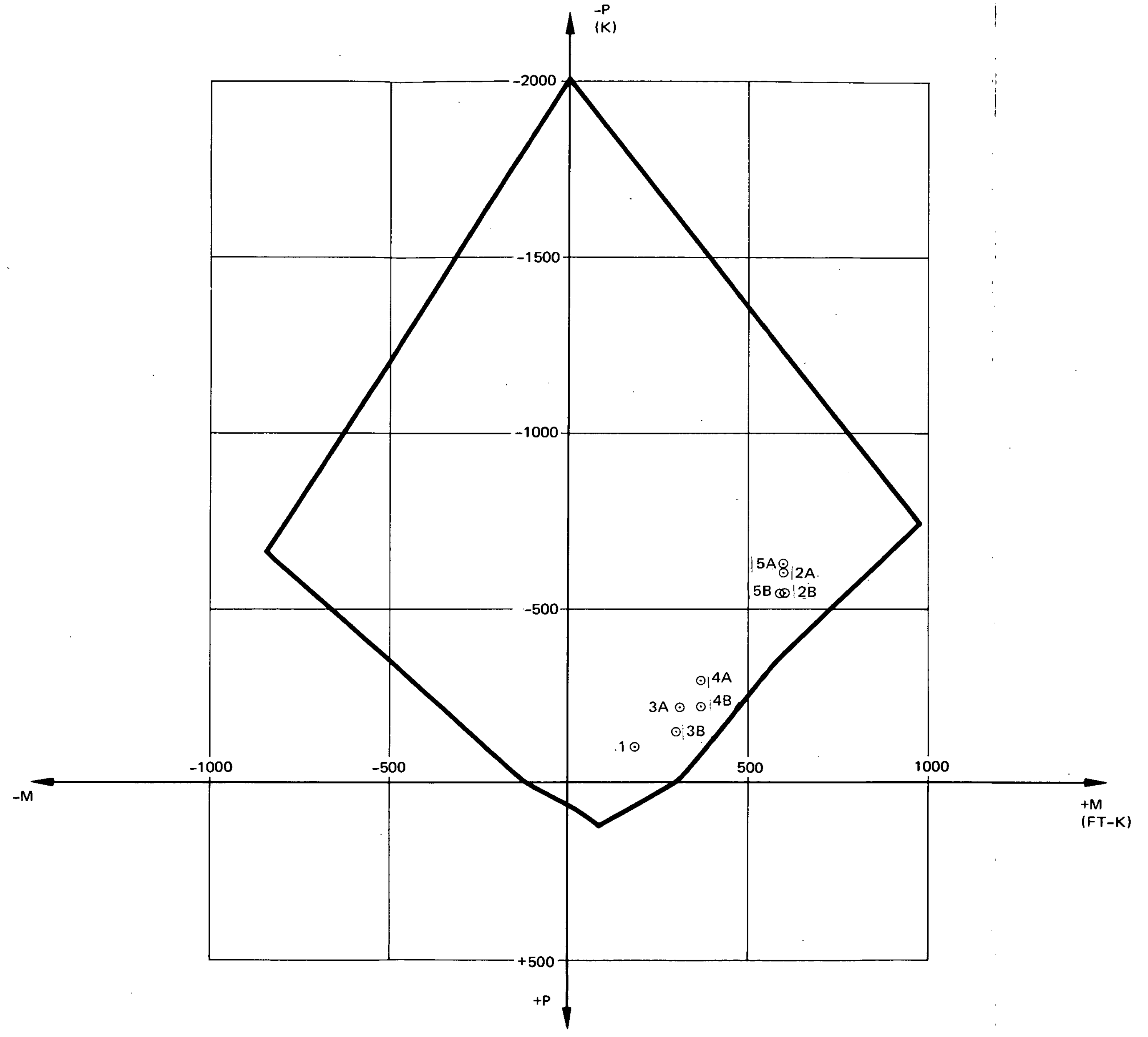
1. REFER TO TABLE 3.8-1, SHEET 1, FOR SECTION NUMBER IDENTIFICATION
2. NOTATIONS "A" AND "B" IDENTIFY "PLUS" AND "MINUS" LOAD COMBINATIONS RESPECTIVELY.

10079-11D-279
11MY6

SAN ONOFRE NUCLEAR GENERATING STATION Units 2 & 3
CONTAINMENT STRUCTURE INTERACTION DIAGRAMS WITH ACTUAL RESULTS OF PRINCIPAL LOAD COMBINATIONS (Sheet 3 of 18)
Figure 3.8-19

GOVERNING LOADING COMBINATIONS:

1. $D + F + T_A + 1.5P$
2. $D + F + T_A + 1.25P \pm 1.25E$
3. $D + F + T_A \pm 1.25E$
4. $D + F + T_A + P \pm E'$
5. $D + F + T_A \pm E'$



SECTION 2 – MERIDIONAL

NOTES:

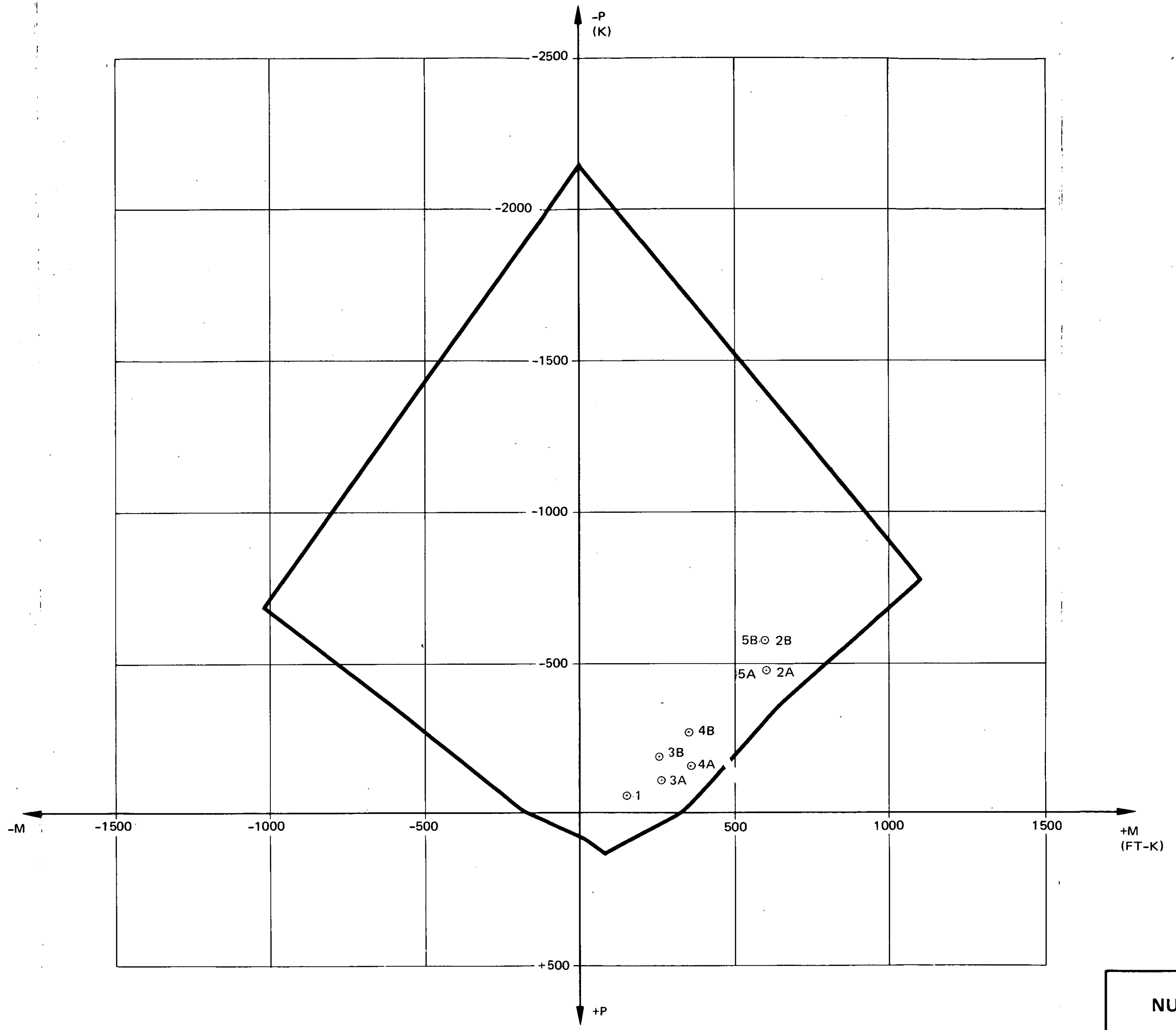
1. REFER TO TABLE 3.8-1, SHEET 1, FOR SECTION NUMBER IDENTIFICATION
2. NOTATIONS "A" AND "B" IDENTIFY "PLUS" AND "MINUS" LOAD COMBINATIONS RESPECTIVELY.

SAN ONOFRE NUCLEAR GENERATING STATION Units 2 & 3
CONTAINMENT STRUCTURE INTERACTION DIAGRAMS WITH ACTUAL RESULTS OF PRINCIPAL LOAD COMBINATIONS (Sheet 3 of 18)
Figure 3.8-19

10079-11D-279
11MY6

GOVERNING LOADING COMBINATIONS:

1. $D + F + T_A + 1.5P$
2. $D + F + T_A + 1.25P \pm 1.25E$
3. $D + F + T_A \pm 1.25E$
4. $D + F + T_A + P \pm E'$
5. $D + F + T_A \pm E'$



SECTION 2 –HOOP

NOTES:

1. REFER TO TABLE 3.8-1, SHEET 1, FOR SECTION NUMBER IDENTIFICATION.
2. NOTATIONS "A" AND "B" IDENTIFY "PLUS" AND "MINUS" LOAD COMBINATIONS RESPECTIVELY.

10079-11D-280
11MY6

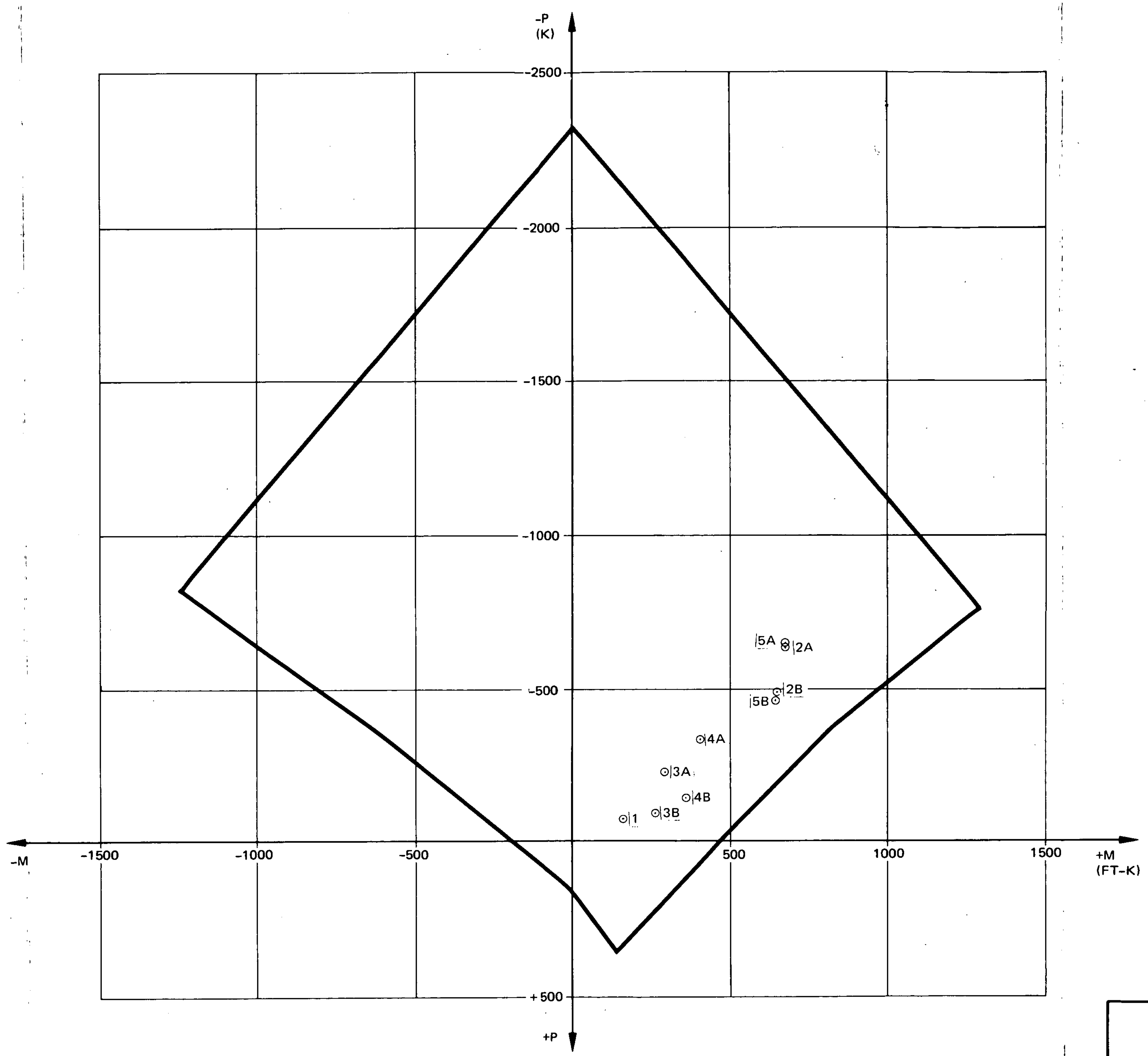
SAN ONOFRE
NUCLEAR GENERATING STATION
Units 2 & 3

CONTAINMENT STRUCTURE
INTERACTION DIAGRAMS
WITH ACTUAL RESULTS OF
PRINCIPAL LOAD COMBINATIONS
(Sheet 4 of 18)

Figure 3.8-19

GOVERNING LOADING COMBINATIONS:

1. | D + F + T_A + 1.5P
2. | D + F + T_A + 1.25P ± 1.25E
3. | D + F + T_A ± 1.25E
4. | D + F + T_A + P ± E'
5. | D + F + T_A ± E'



SECTION 3 – MERIDIONAL

NOTES:

1. REFER TO TABLE 3.8-1, SHEET 1, FOR SECTION NUMBER IDENTIFICATION.
2. NOTATIONS "A" AND "B" IDENTIFY "PLUS" AND "MINUS" LOAD COMBINATIONS RESPECTIVELY.

10079-11D-281
11MY6

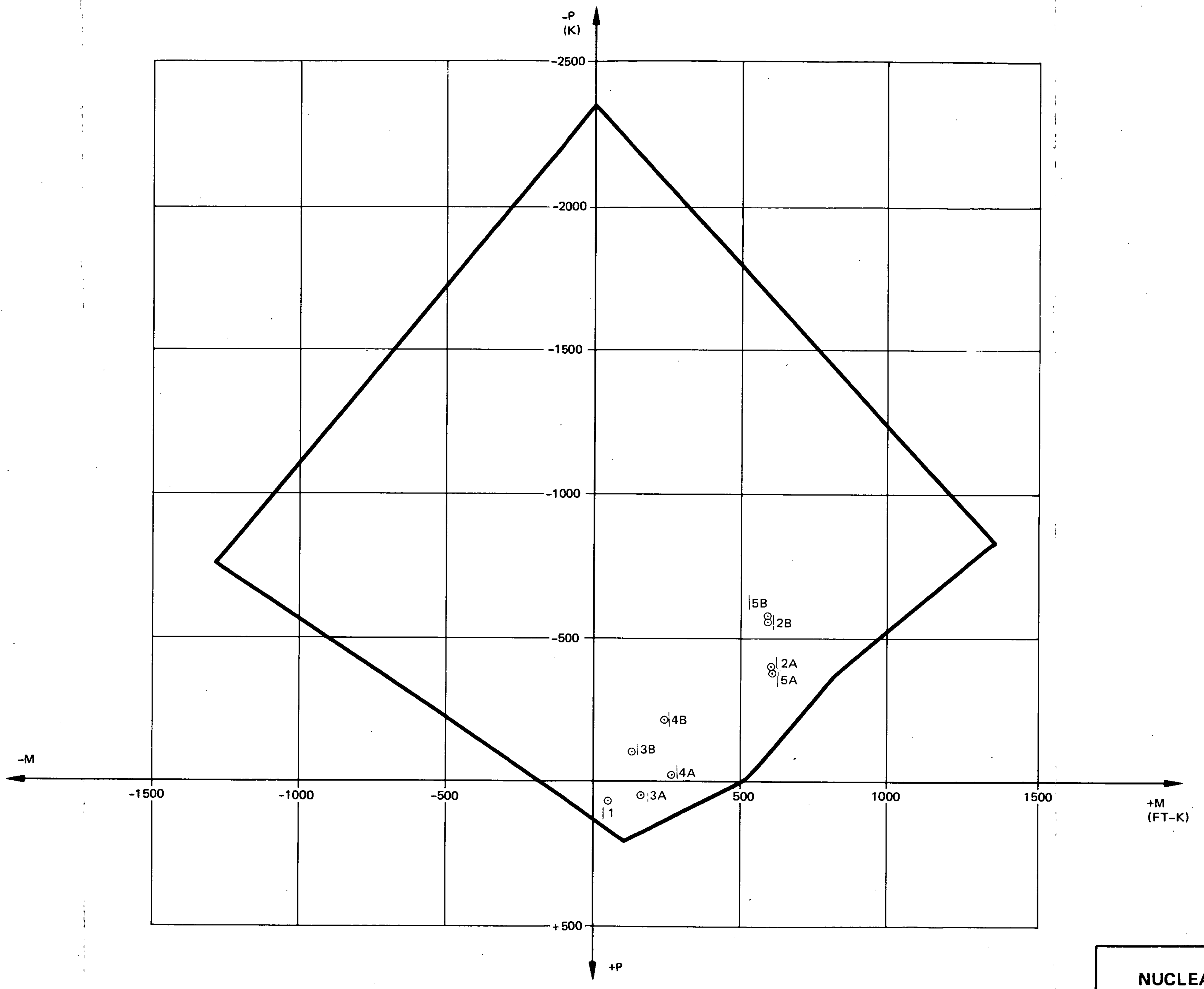
SAN ONOFRE NUCLEAR GENERATING STATION Units 2 & 3

CONTAINMENT STRUCTURE INTERACTION DIAGRAMS WITH ACTUAL RESULTS OF PRINCIPAL LOAD COMBINATIONS (Sheet 5 of 18)

Figure 3.8-19

GOVERNING LOADING COMBINATIONS:

1. $D + F + T_A + 1.5P$
2. $D + F + T_A + 1.25P \pm 1.25E$
3. $D + F + T_A \pm 1.25E$
4. $D + F + T_A + P \pm E'$
5. $D + F + T_A \pm E'$



SECTION 3 – HOOP

NOTES:

1. REFER TO TABLE 3.8-1, SHEET 1, FOR SECTION NUMBER IDENTIFICATION.
2. NOTATIONS "A" AND "B" IDENTIFY "PLUS" AND "MINUS" LOAD COMBINATIONS RESPECTIVELY.

10079-11D-282
11MY6

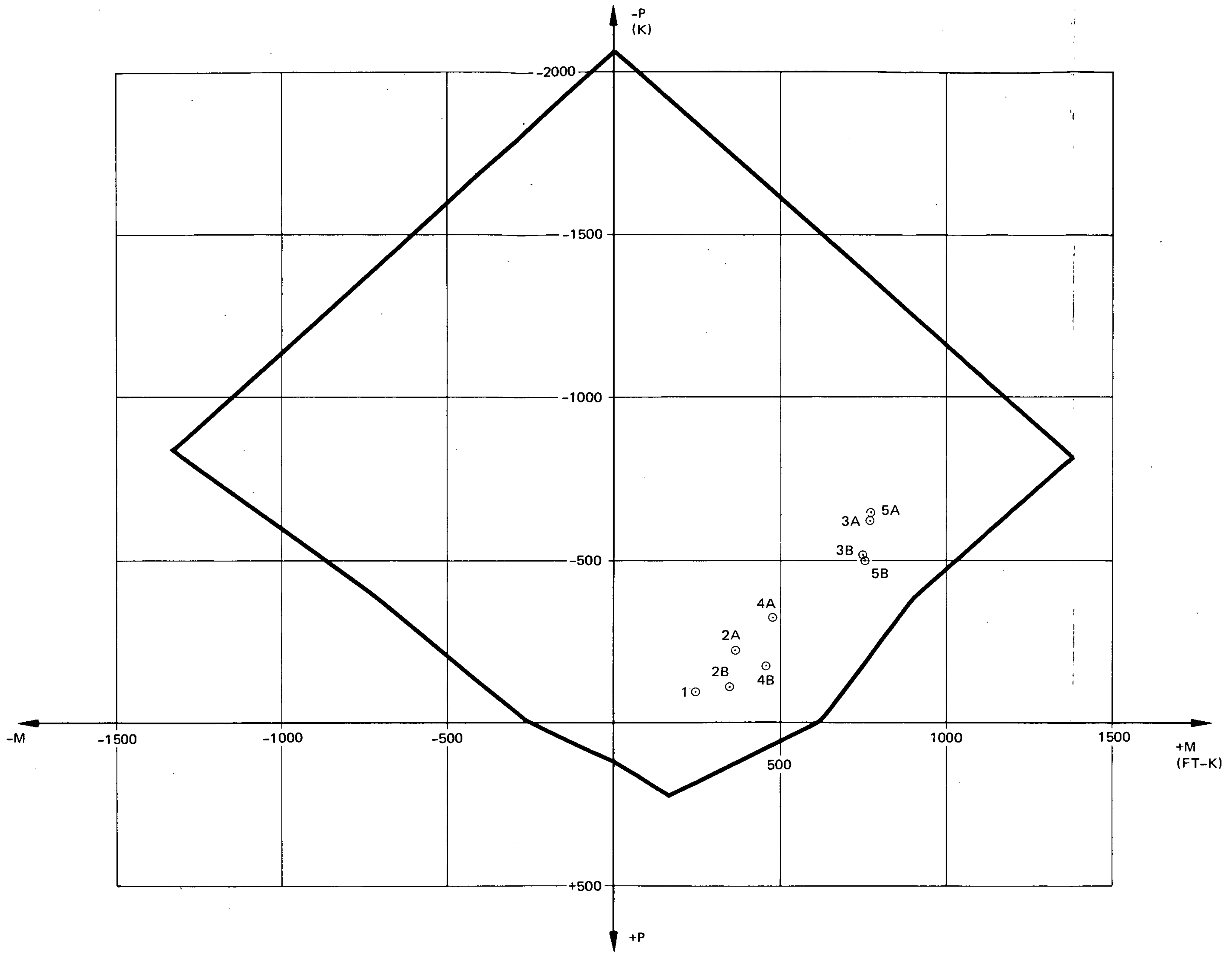
SAN ONOFRE NUCLEAR GENERATING STATION Units 2 & 3

CONTAINMENT STRUCTURE INTERACTION DIAGRAMS WITH ACTUAL RESULTS OF PRINCIPAL LOAD COMBINATIONS (Sheet 6 of 18)

Figure 3.8-19

GOVERNING LOADING COMBINATIONS:

1. $D + F + T_A + 1.5P$
2. $D + F + T_A + 1.25P \pm 1.25E$
3. $D + F + T_A \pm 1.25E$
4. $D + F + T_A + P \pm E'$
5. $D + F + T_A \pm E'$

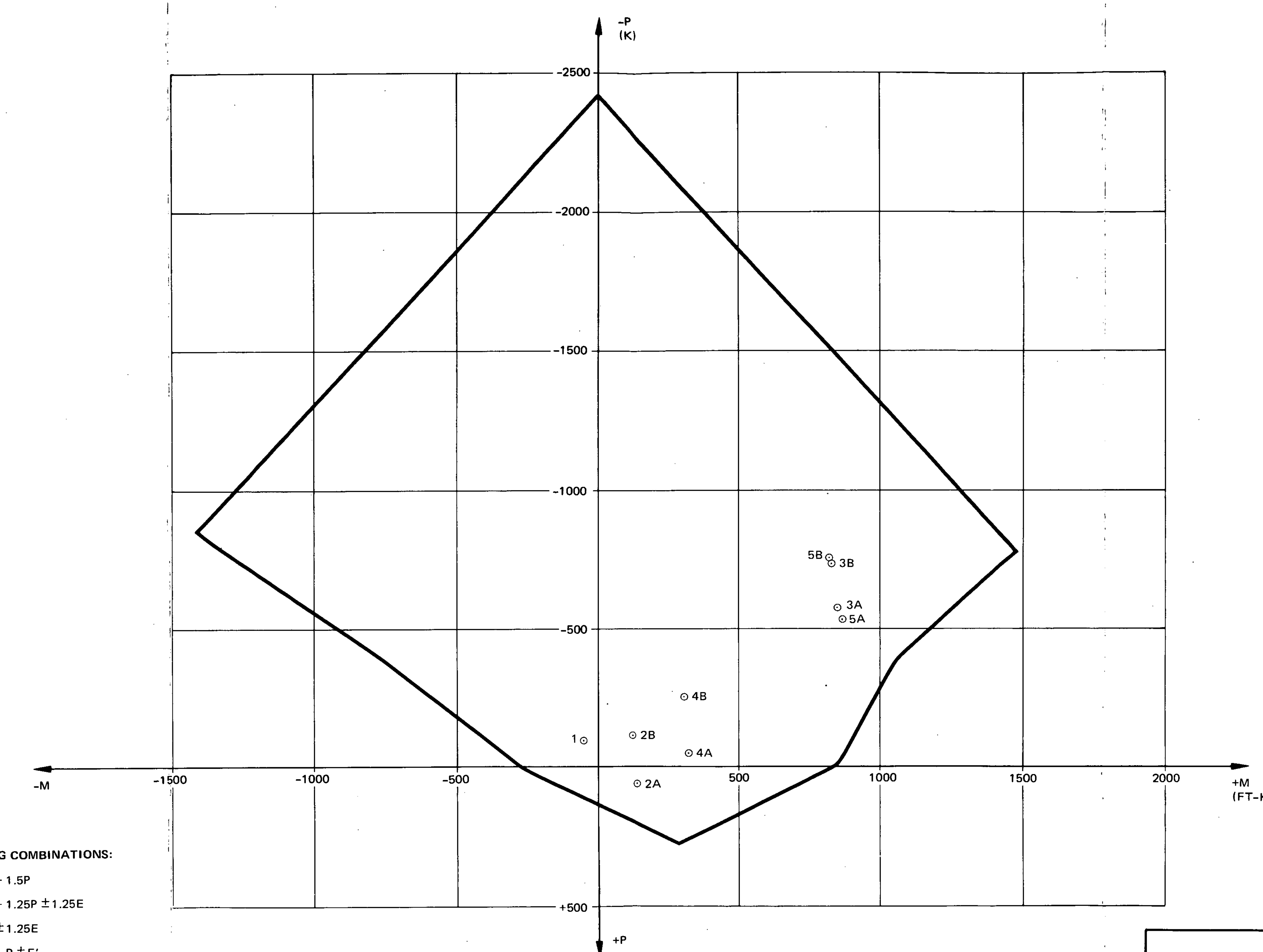


SECTION 4 – MERIDIONAL

NOTES:

1. REFER TO TABLE 3.8-1, SHEET 1, FOR SECTION NUMBER IDENTIFICATION.
2. NOTATIONS "A" AND "B" IDENTIFY "PLUS" AND "MINUS" LOAD COMBINATIONS RESPECTIVELY.

SAN ONOFRE NUCLEAR GENERATING STATION Units 2 & 3
CONTAINMENT STRUCTURE INTERACTION DIAGRAMS WITH ACTUAL RESULTS OF PRINCIPAL LOAD COMBINATIONS (Sheet 7 of 18)
Figure 3.8-19



SECTION 4 – HOOP

GOVERNING LOADING COMBINATIONS:

1. $D + F + T_A + 1.5P$
2. $D + F + T_A + 1.25P \pm 1.25E$
3. $D + F + T_A \pm 1.25E$
4. $D + F + T_A + P \pm E'$
5. $D + F + T_A \pm E'$

NOTES:

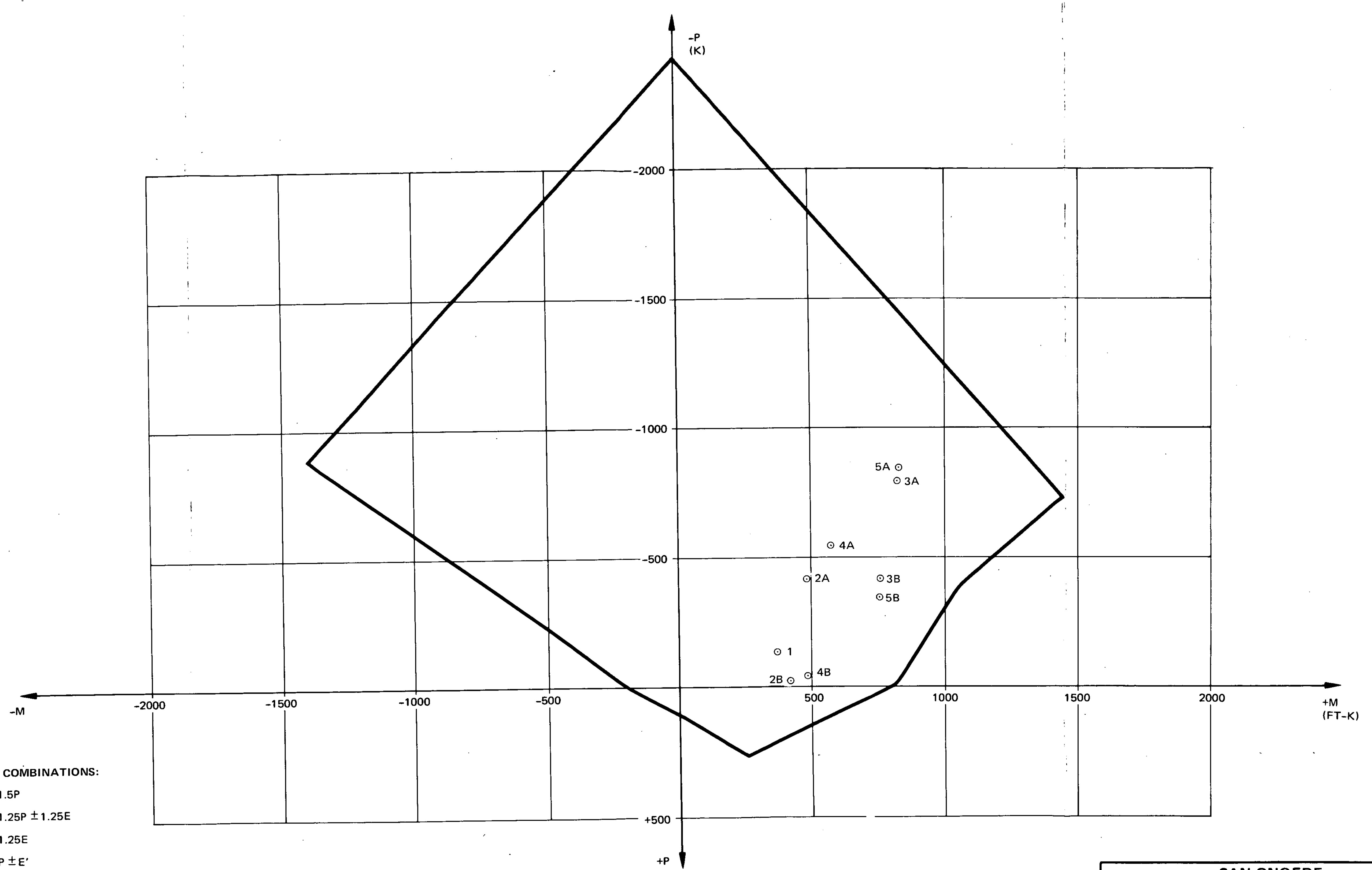
1. REFER TO TABLE 3.8-1, SHEET 1, FOR SECTION NUMBER IDENTIFICATION.
2. NOTATIONS "A" AND "B" IDENTIFY "PLUS" AND "MINUS" LOAD COMBINATIONS RESPECTIVELY.

10079-11D-284
11MY6

SAN ONOFRE
NUCLEAR GENERATING STATION
Units 2 & 3

CONTAINMENT STRUCTURE
INTERACTION DIAGRAMS
WITH ACTUAL RESULTS OF
PRINCIPAL LOAD COMBINATIONS
(Sheet 8 of 18)

Figure 3.8-19



SECTION 5 – MERIDIONAL

GOVERNING LOADING COMBINATIONS:

1. $D + F + T_A + 1.5P$
2. $D + F + T_A + 1.25P \pm 1.25E$
3. $D + F + T_A \pm 1.25E$
4. $D + F + T_A + P \pm E'$
5. $D + F + T_A \pm E'$

NOTES:

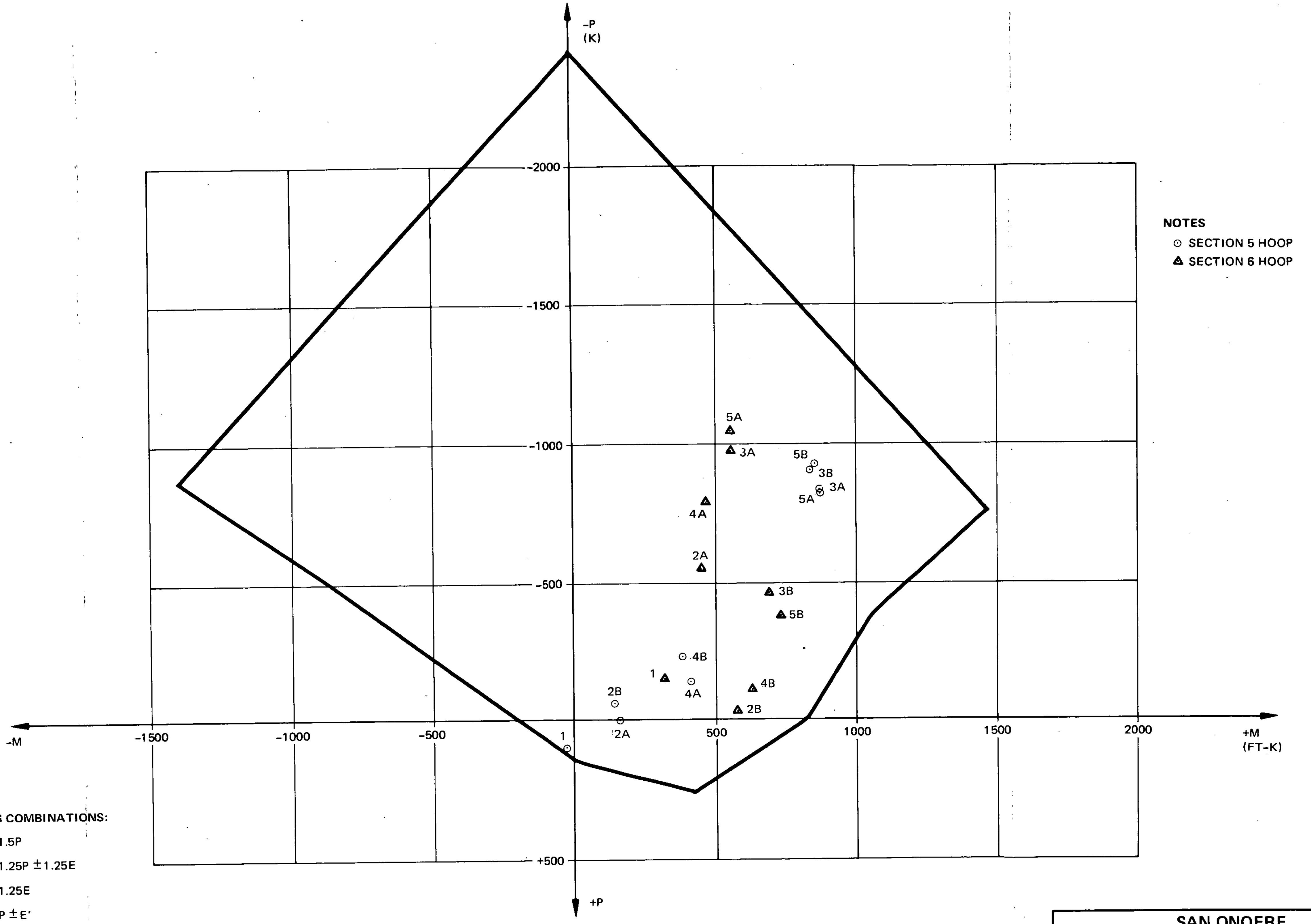
1. REFER TO TABLE 3.8-1, SHEET 1, FOR SECTION NUMBER IDENTIFICATION.
2. NOTATIONS "A" AND "B" IDENTIFY "PLUS" AND "MINUS" LOAD COMBINATIONS RESPECTIVELY.

SAN ONOFRE NUCLEAR GENERATING STATION Units 2 & 3

CONTAINMENT STRUCTURE INTERACTION DIAGRAMS WITH ACTUAL RESULTS OF PRINCIPAL LOAD COMBINATIONS (Sheet 9 of 18)

Figure 3.8-19

10079-11D-285
11MY6



SECTION 5 AND 6 – HOOP

GOVERNING LOADING COMBINATIONS:

1. $D + F + T_A + 1.5P$
2. $D + F + T_A + 1.25P \pm 1.25E$
3. $D + F + T_A \pm 1.25E$
4. $D + F + T_A + P \pm E'$
5. $D + F + T_A \pm E'$

NOTES:

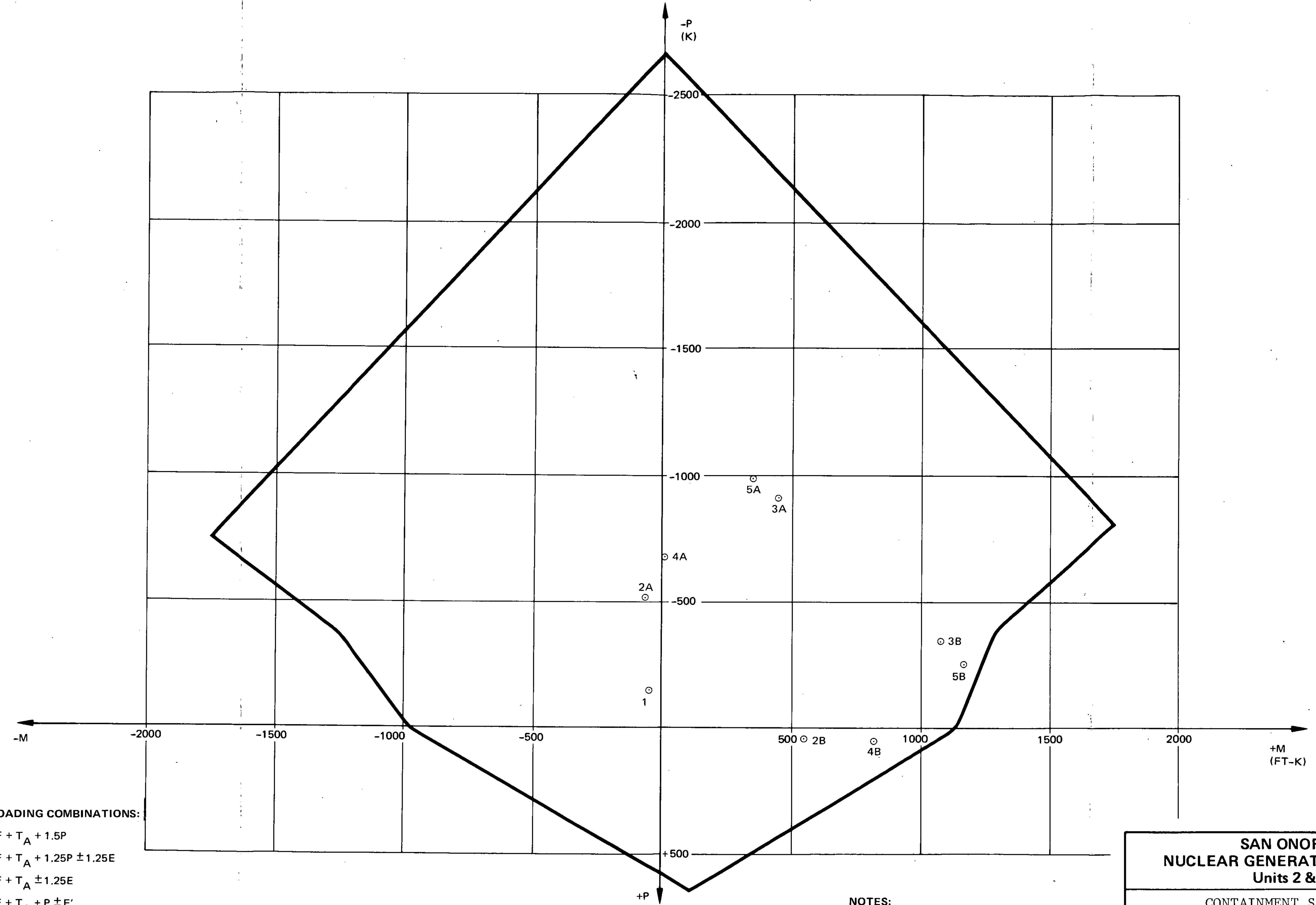
1. REFER TO TABLE 3.8-1, SHEET 1, FOR SECTION NUMBER IDENTIFICATION.
2. NOTATIONS "A" AND "B" IDENTIFY "PLUS" AND "MINUS" LOAD COMBINATIONS RESPECTIVELY.

10079-11D-286
11MY6

SAN ONOFRE NUCLEAR GENERATING STATION Units 2 & 3

CONTAINMENT STRUCTURE INTERACTION DIAGRAMS WITH ACTUAL RESULTS OF PRINCIPAL LOAD COMBINATIONS (Sheet 10 of 18)

Figure 3.8-19



SECTION 6 – MERIDIONAL

GOVERNING LOADING COMBINATIONS:

1. $D + F + T_A + 1.5P$
2. $D + F + T_A + 1.25P \pm 1.25E$
3. $D + F + T_A \pm 1.25E$
4. $D + F + T_A + P \pm E'$
5. $D + F + T_A \pm E'$

NOTES:

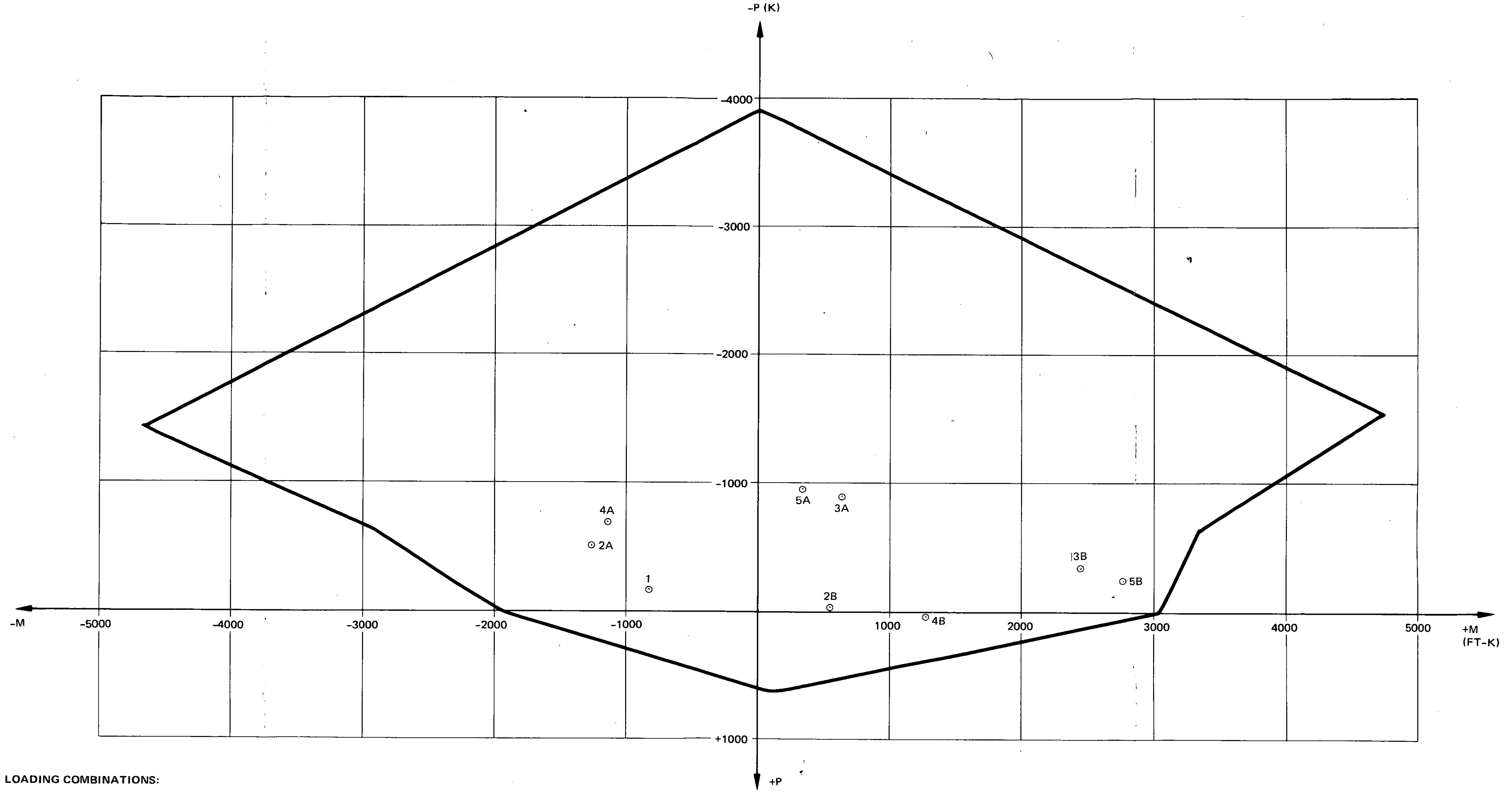
1. REFER TO TABLE 3.8-1, SHEET 1, FOR SECTION NUMBER IDENTIFICATION.
2. NOTATIONS "A" AND "B" IDENTIFY "PLUS" AND "MINUS" LOAD COMBINATIONS RESPECTIVELY.

10079-11D-287
11MY6

SAN ONOFRE NUCLEAR GENERATING STATION Units 2 & 3

CONTAINMENT STRUCTURE INTERACTION DIAGRAMS WITH ACTUAL RESULTS OF PRINCIPAL LOAD COMBINATIONS (Sheet 11 of 18)

Figure 3.8-19



SECTION 7 – MERIDIONAL

GOVERNING LOADING COMBINATIONS:

1. $D + F + T_A + 1.5P$
2. $D + F + T_A + 1.25P \pm 1.25E$
3. $D + F + T_A \pm 1.25E$
4. $D + F + T_A + P \pm E'$
5. $D + F + T_A \pm E'$

NOTES:

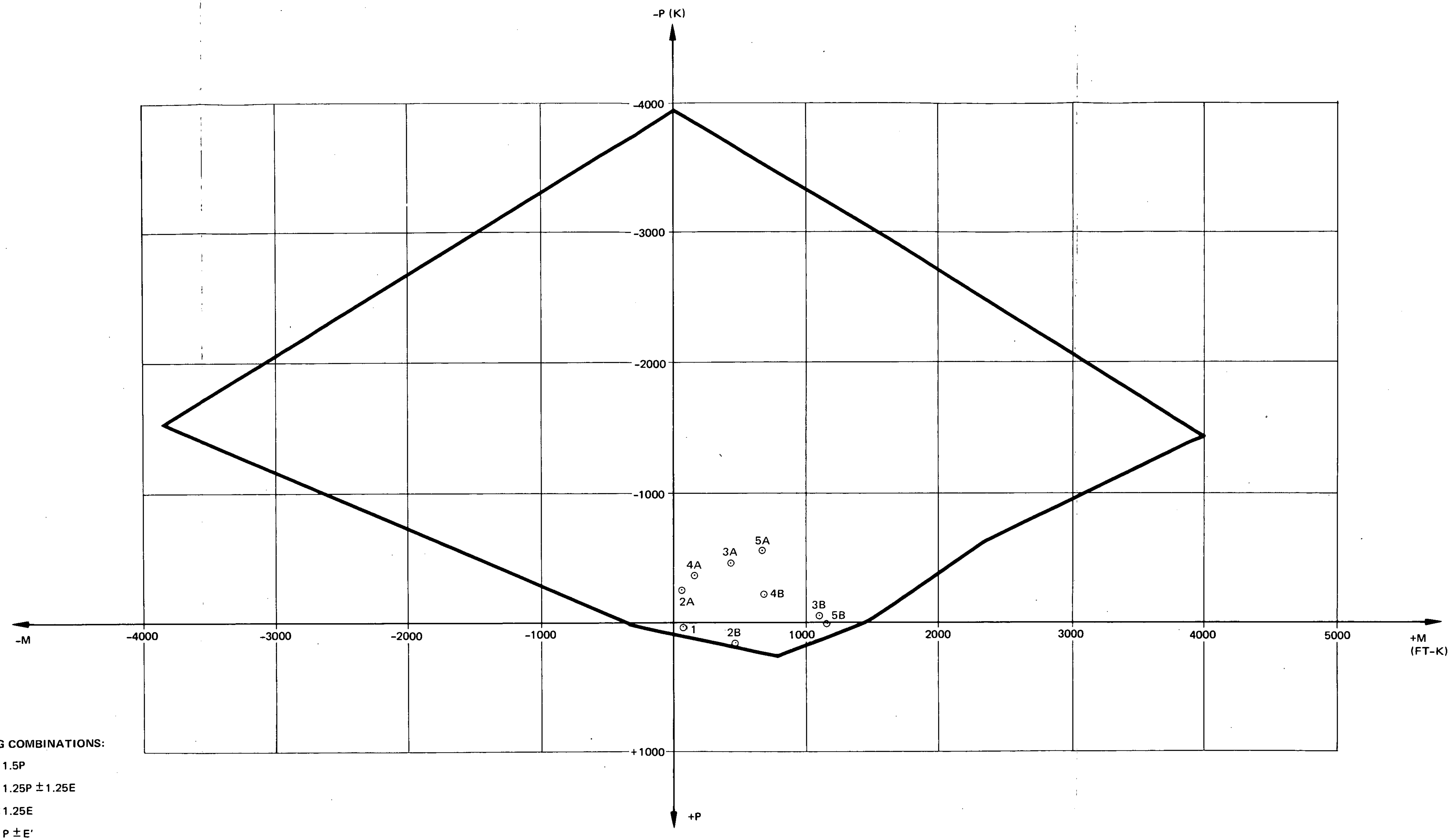
1. REFER TO TABLE 3.8-1, SHEET 1, FOR SECTION NUMBER IDENTIFICATION.
2. NOTATIONS "A" AND "B" IDENTIFY "PLUS" AND "MINUS" LOAD COMBINATIONS RESPECTIVELY.

10079-11D-288
11MY6

SAN ONOFRE NUCLEAR GENERATING STATION Units 2 & 3

CONTAINMENT STRUCTURE INTERACTION DIAGRAMS WITH ACTUAL RESULTS OF PRINCIPAL LOAD COMBINATIONS (Sheet 12 of 18)

Figure 3.8-19



SECTION 7 — HOOP

GOVERNING LOADING COMBINATIONS:

1. $D + F + T_A + 1.5P$
2. $D + F + T_A + 1.25P \pm 1.25E$
3. $D + F + T_A \pm 1.25E$
4. $D + F + T_A + P \pm E'$
5. $D + F + T_A \pm E'$

NOTES:

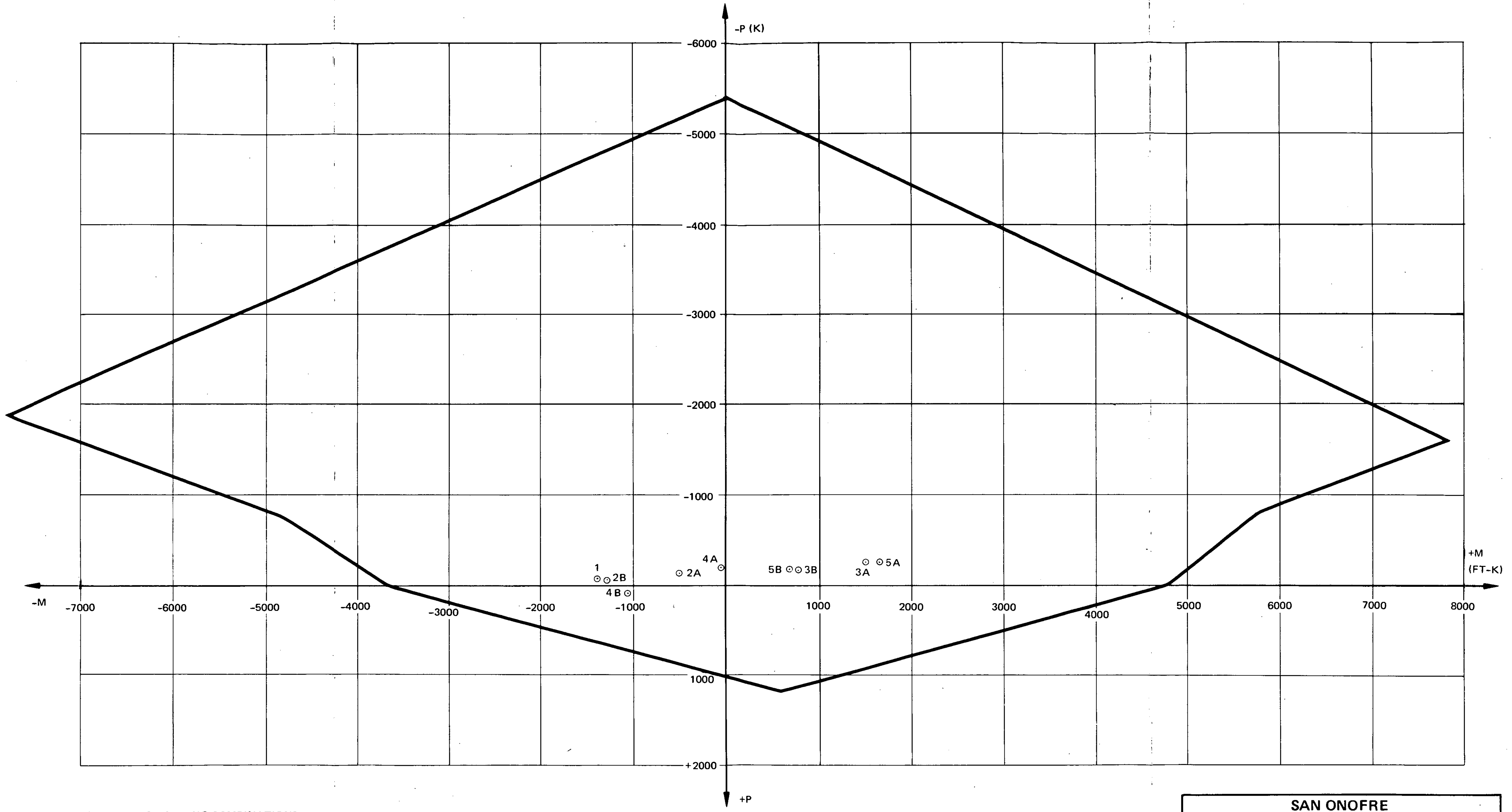
1. REFER TO TABLE 3.8-1, SHEET 1, FOR SECTION NUMBER IDENTIFICATION.
2. NOTATIONS "A" AND "B" IDENTIFY "PLUS" AND "MINUS" LOAD COMBINATIONS RESPECTIVELY.

SAN ONOFRE NUCLEAR GENERATING STATION Units 2 & 3

CONTAINMENT STRUCTURE INTERACTION DIAGRAMS WITH ACTUAL RESULTS OF PRINCIPAL LOAD COMBINATIONS (Sheet 13 of 18)

Figure 3.8-19

10079-11D-289
11MY6



SECTION 8 – MERIDIONAL

GOVERNING LOADING COMBINATIONS:

1. $D + F + T_A + 1.5P$
2. $D + F + T_A + 1.25P \pm 1.25E$
3. $D + F + T_A \pm 1.25E$
4. $D + F + T_A + P \pm E'$
5. $D + F + T_A \pm E'$

NOTES:

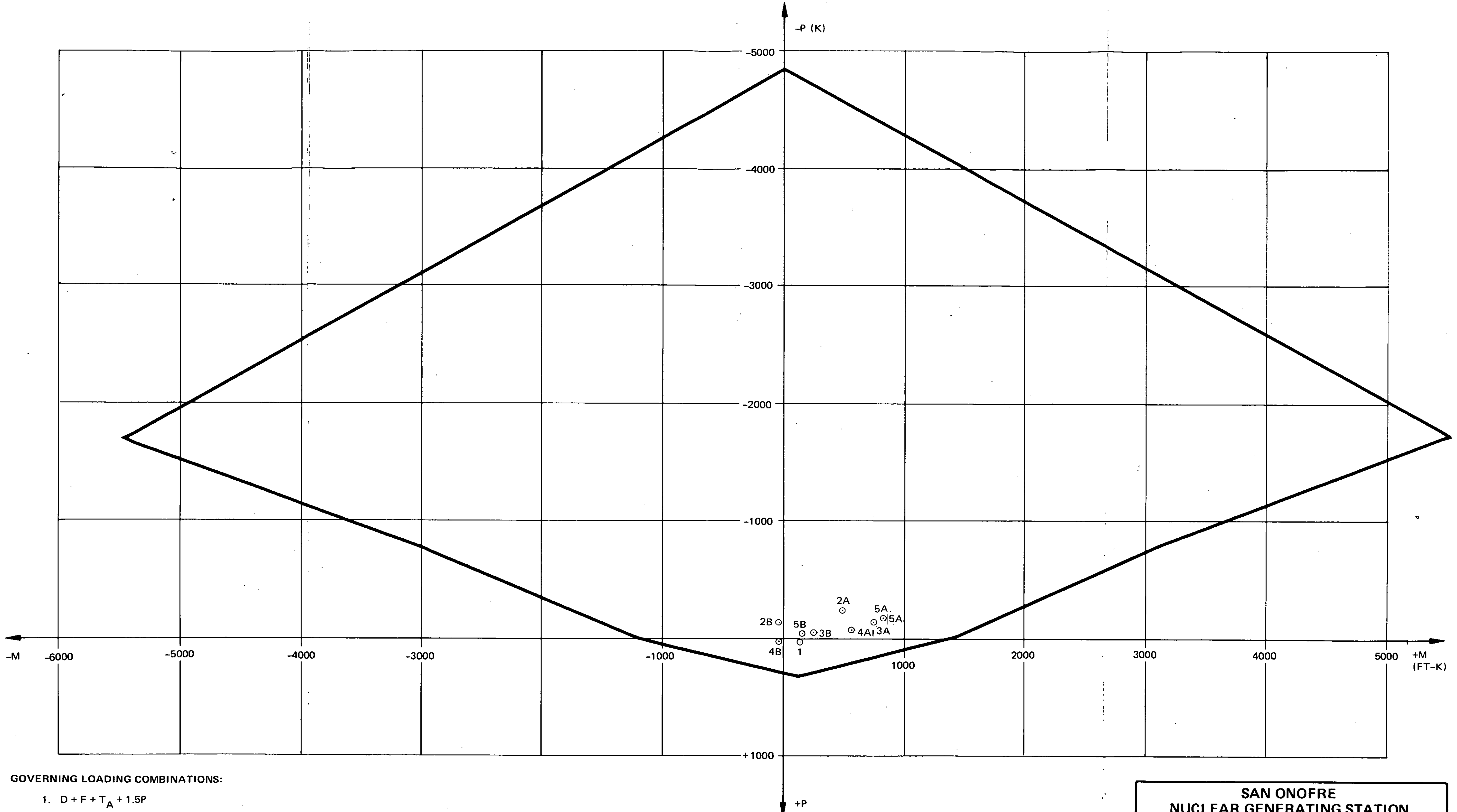
1. REFER TO TABLE 3.8-1, SHEET 1, FOR SECTION NUMBER IDENTIFICATION.
2. NOTATIONS "A" AND "B" IDENTIFY "PLUS" AND "MINUS" LOAD COMBINATIONS RESPECTIVELY.

SAN ONOFRE NUCLEAR GENERATING STATION Units 2 & 3

CONTAINMENT STRUCTURE INTERACTION DIAGRAMS WITH ACTUAL RESULTS OF PRINCIPAL LOAD COMBINATIONS (Sheet 14 of 18)

Figure 3.8-19

10079-11D-290
11MY6



SECTION 8 – HOOP

GOVERNING LOADING COMBINATIONS:

1. $D + F + T_A + 1.5P$
2. $D + F + T_A + 1.25P \pm 1.25E$
3. $D + F + T_A \pm 1.25E$
4. $D + F + T_A + P \pm E'$
5. $D + F + T_A \pm E'$

NOTES:

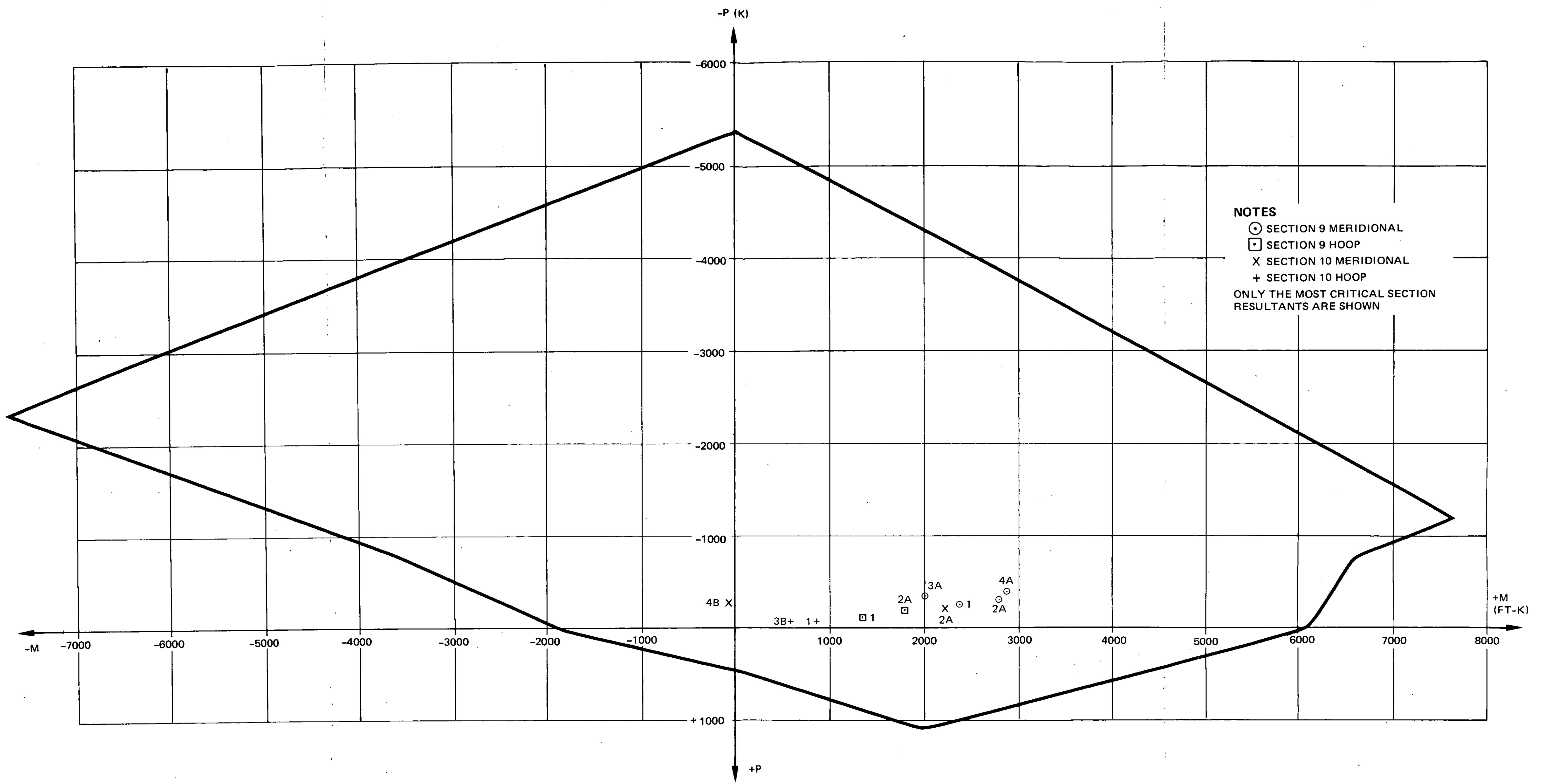
1. REFER TO TABLE 3.8-1, SHEET 1, FOR SECTION NUMBER IDENTIFICATION.
2. NOTATIONS "A" AND "B" IDENTIFY "PLUS" AND "MINUS" LOAD COMBINATIONS RESPECTIVELY.

SAN ONOFRE
NUCLEAR GENERATING STATION
Units 2 & 3

CONTAINMENT STRUCTURE
INTERACTION DIAGRAMS
WITH ACTUAL RESULTS OF
PRINCIPAL LOAD COMBINATIONS
(Sheet 15 of 18)

Figure 3.8-19

10079-11D-291
11MY6



SECTIONS 9 AND I0 – MERIDIONAL AND HOOP

GOVERNING LOADING COMBINATIONS:

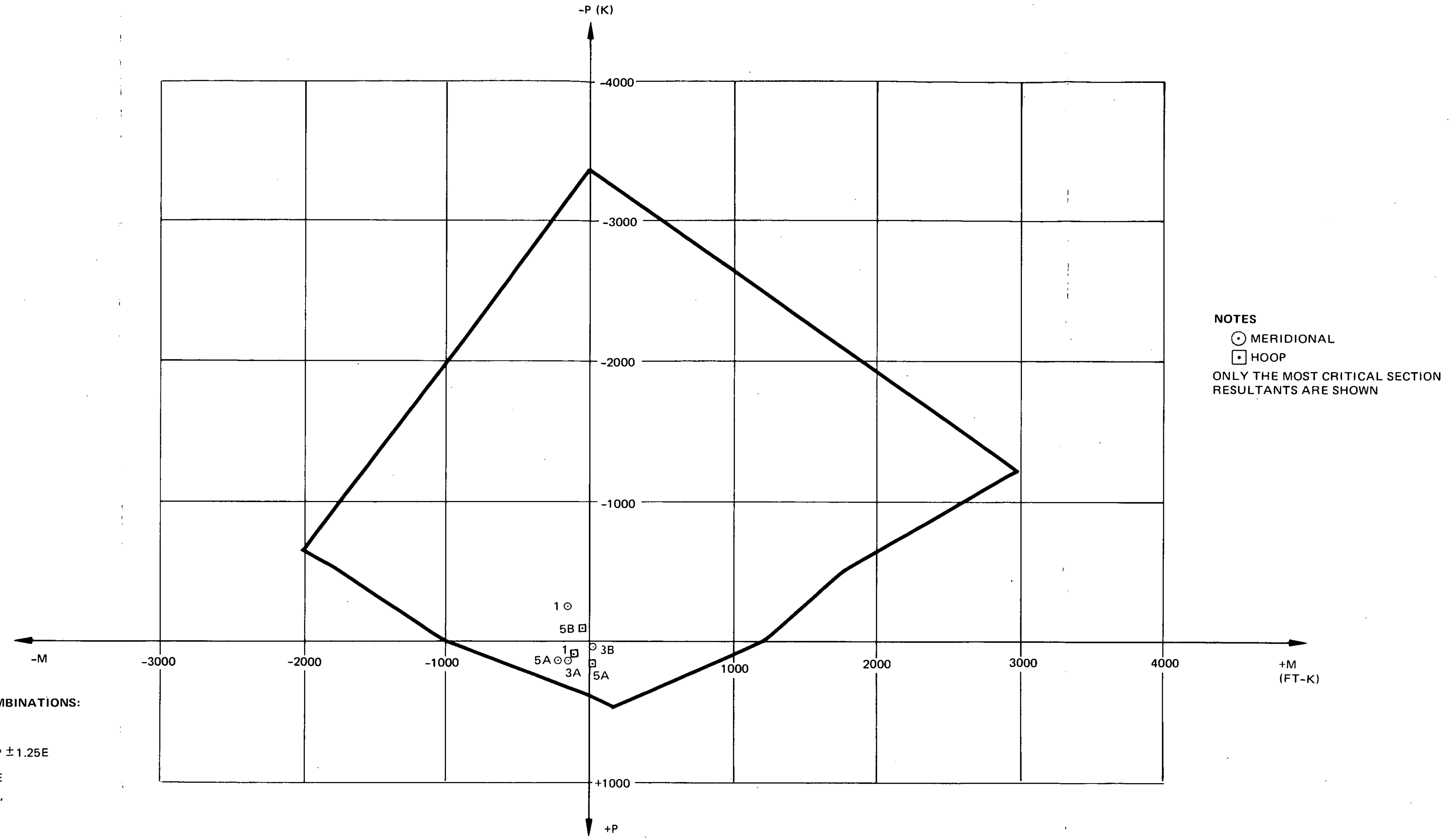
1. $D + F + T_A + 1.5P$
2. $D + F + T_A + 1.25P \pm 1.25E$
3. $D + F + T_A \pm 1.25E$
4. $D + F + T_A + P \pm E'$
5. $D + F + T_A \pm E'$

NOTES:

1. REFER TO TABLE 3.8-1, SHEET 1, FOR SECTION NUMBER IDENTIFICATION.
2. NOTATIONS "A" AND "B" IDENTIFY "PLUS" AND "MINUS" LOAD COMBINATIONS RESPECTIVELY.

SAN ONOFRE NUCLEAR GENERATING STATION Units 2 & 3
CONTAINMENT STRUCTURE INTERACTION DIAGRAMS WITH ACTUAL RESULTS OF PRINCIPAL LOAD COMBINATIONS (Sheet 16 of 18)
Figure 3.8-19

10079-11D-292
11MY6



SECTION II – MERIDIONAL AND HOOP

GOVERNING LOADING COMBINATIONS:

1. $D + F + T_A + 1.5P$
2. $D + F + T_A + 1.25P \pm 1.25E$
3. $D + F + T_A \pm 1.25E$
4. $D + F + T_A + P \pm E'$
5. $D + F + T_A \pm E'$

NOTES:

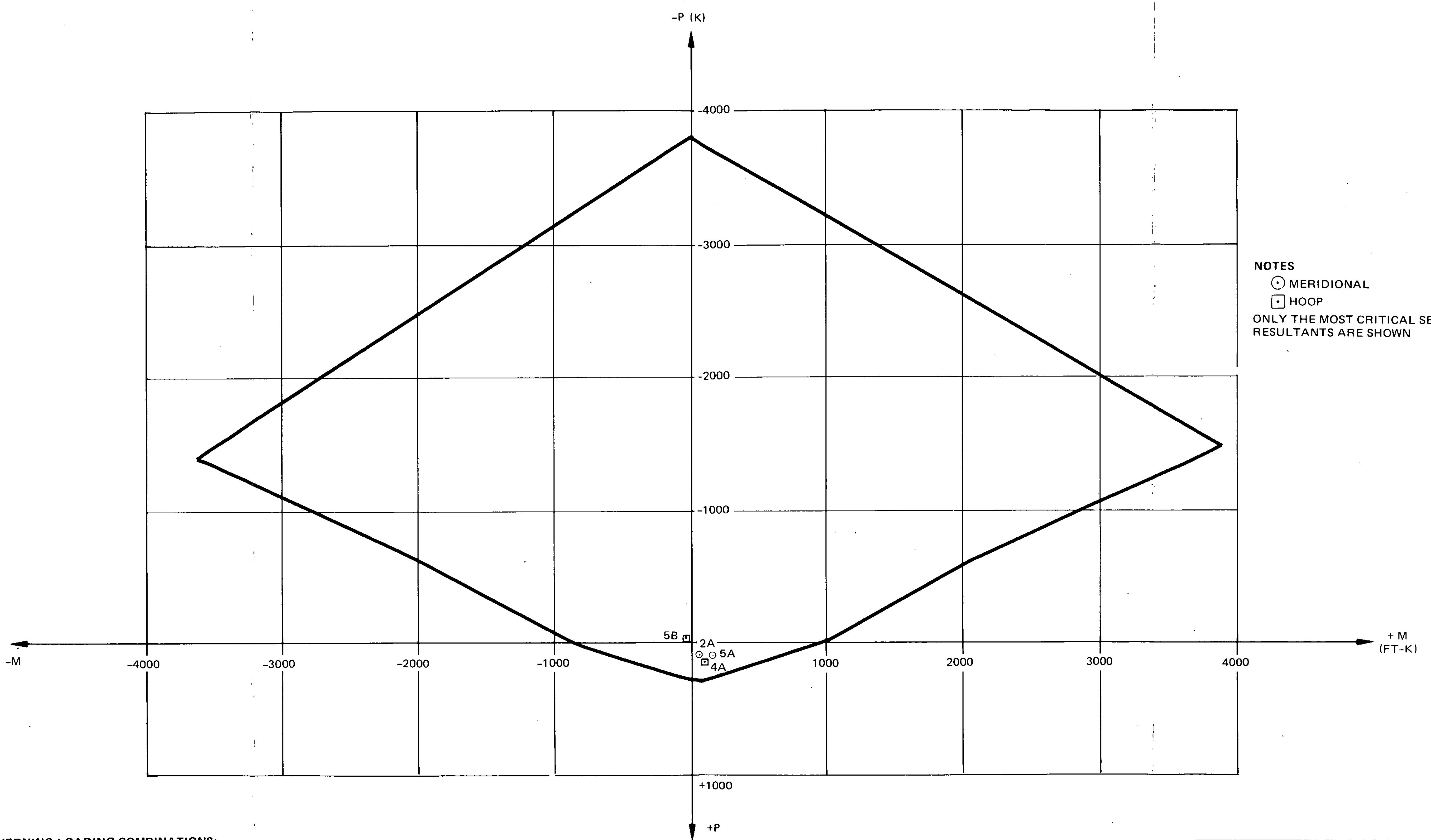
1. REFER TO TABLE 3.8-1, SHEET 1, FOR SECTION NUMBER IDENTIFICATION.
2. NOTATIONS "A" AND "B" IDENTIFY "PLUS" AND "MINUS" LOAD COMBINATIONS RESPECTIVELY.

10079-11D-293
11MY6

SAN ONOFRE NUCLEAR GENERATING STATION Units 2 & 3

CONTAINMENT STRUCTURE INTERACTION DIAGRAMS WITH ACTUAL RESULTS OF PRINCIPAL LOAD COMBINATIONS (Sheet 17 of 18)

Figure 3.8-19



SECTION I2 – MERIDIONAL AND HOOP

NOTES
⊙ MERIDIONAL
⊡ HOOP
ONLY THE MOST CRITICAL SECTION RESULTANTS ARE SHOWN

GOVERNING LOADING COMBINATIONS:

1. $D + F + T_A + 1.5P$
2. $D + F + T_A + 1.25P \pm 1.25E$
3. $D + F + T_A \pm 1.25E$
4. $D + F + T_A + P \pm E'$
5. $D + F + T_A \pm E'$

NOTES:

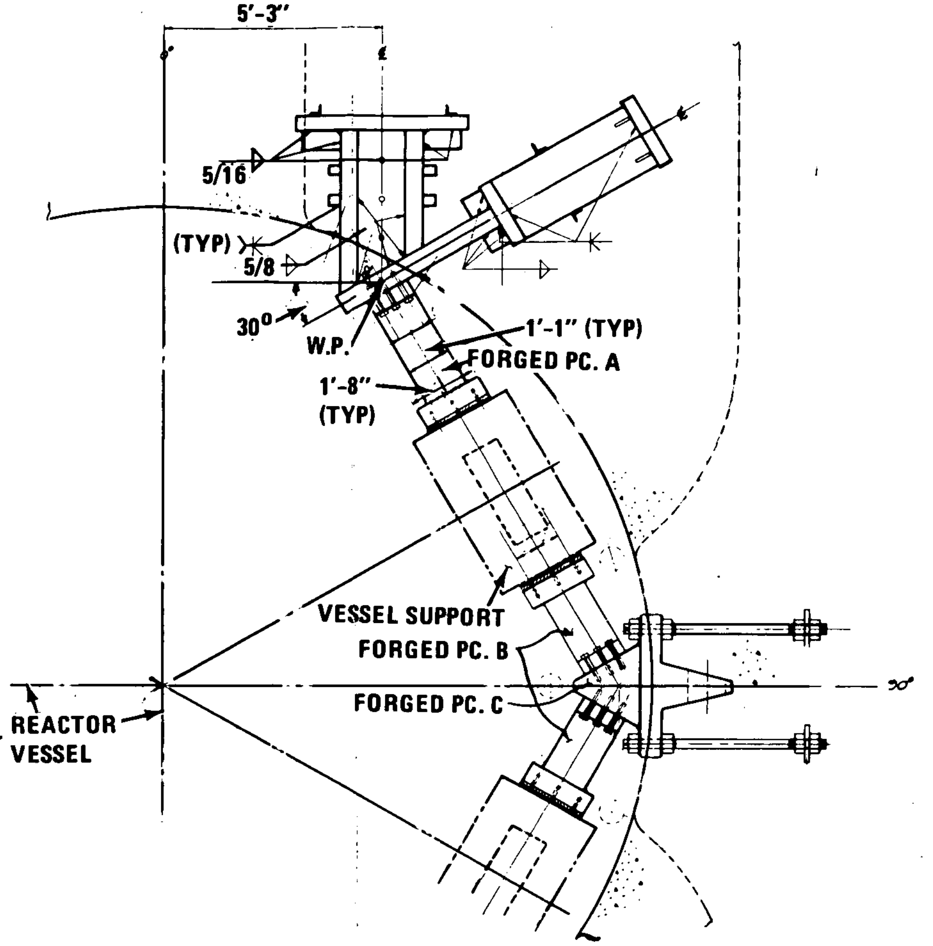
1. REFER TO TABLE 3.8-1, SHEET 1, FOR SECTION NUMBER IDENTIFICATION.
2. NOTATIONS "A" AND "B" IDENTIFY "PLUS" AND "MINUS" LOAD COMBINATIONS RESPECTIVELY.

10079-11D-294
11MY6

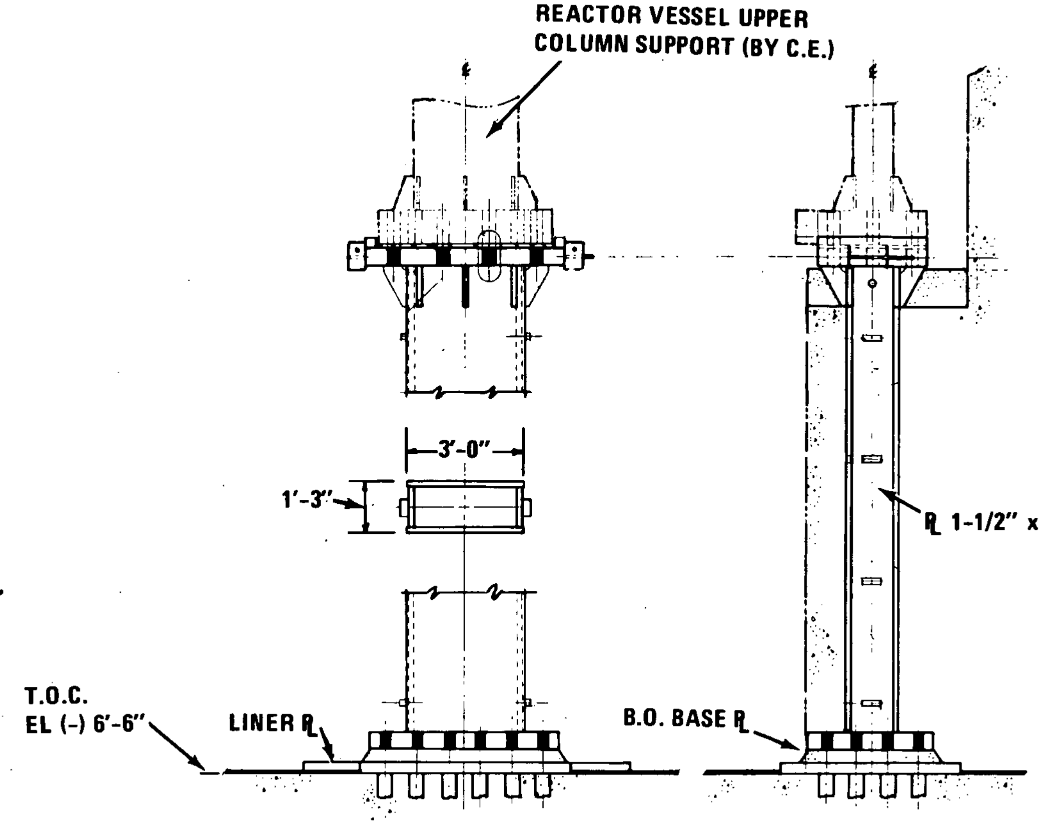
SAN ONOFRE NUCLEAR GENERATING STATION Units 2 & 3

CONTAINMENT STRUCTURE INTERACTION DIAGRAMS WITH ACTUAL RESULTS OF PRINCIPAL LOAD COMBINATIONS (Sheet 18 of 18)

Figure 3.8-19

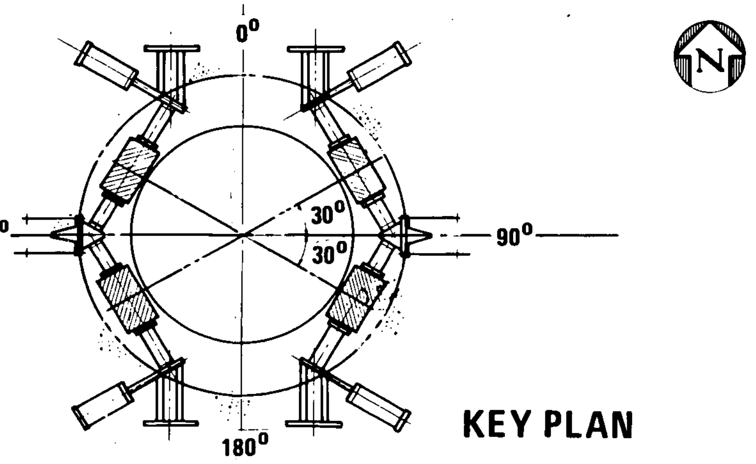


PARTIAL PLAN
EL 27'-10-7/8"

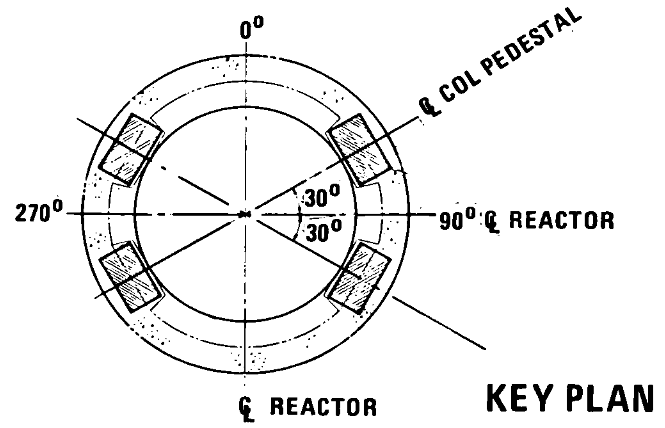


SIDE ELEV

END ELEV



UPPER LATERAL SUPPORTS



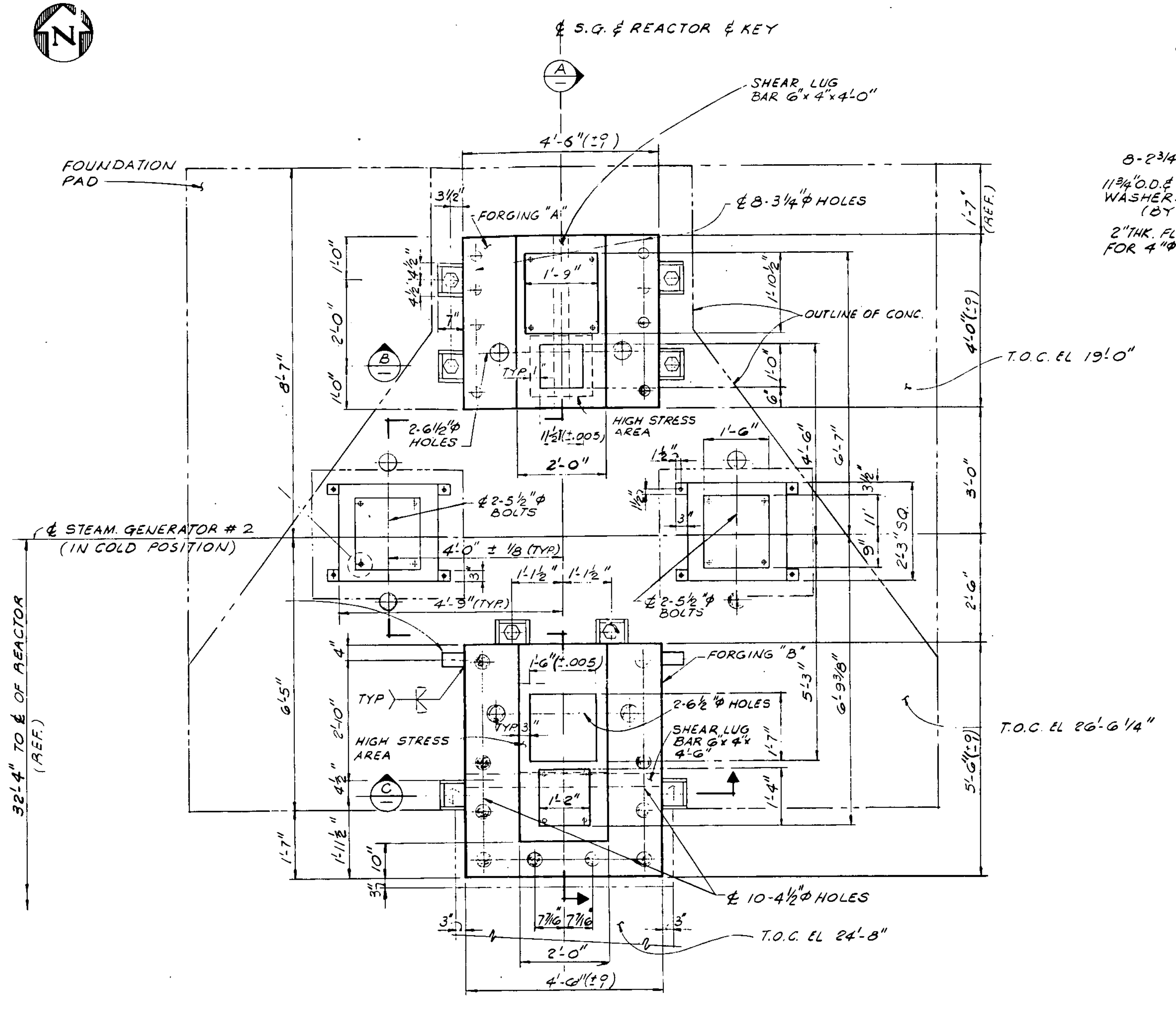
LOWER VERTICAL SUPPORTS

10079-11D-301
26AU6

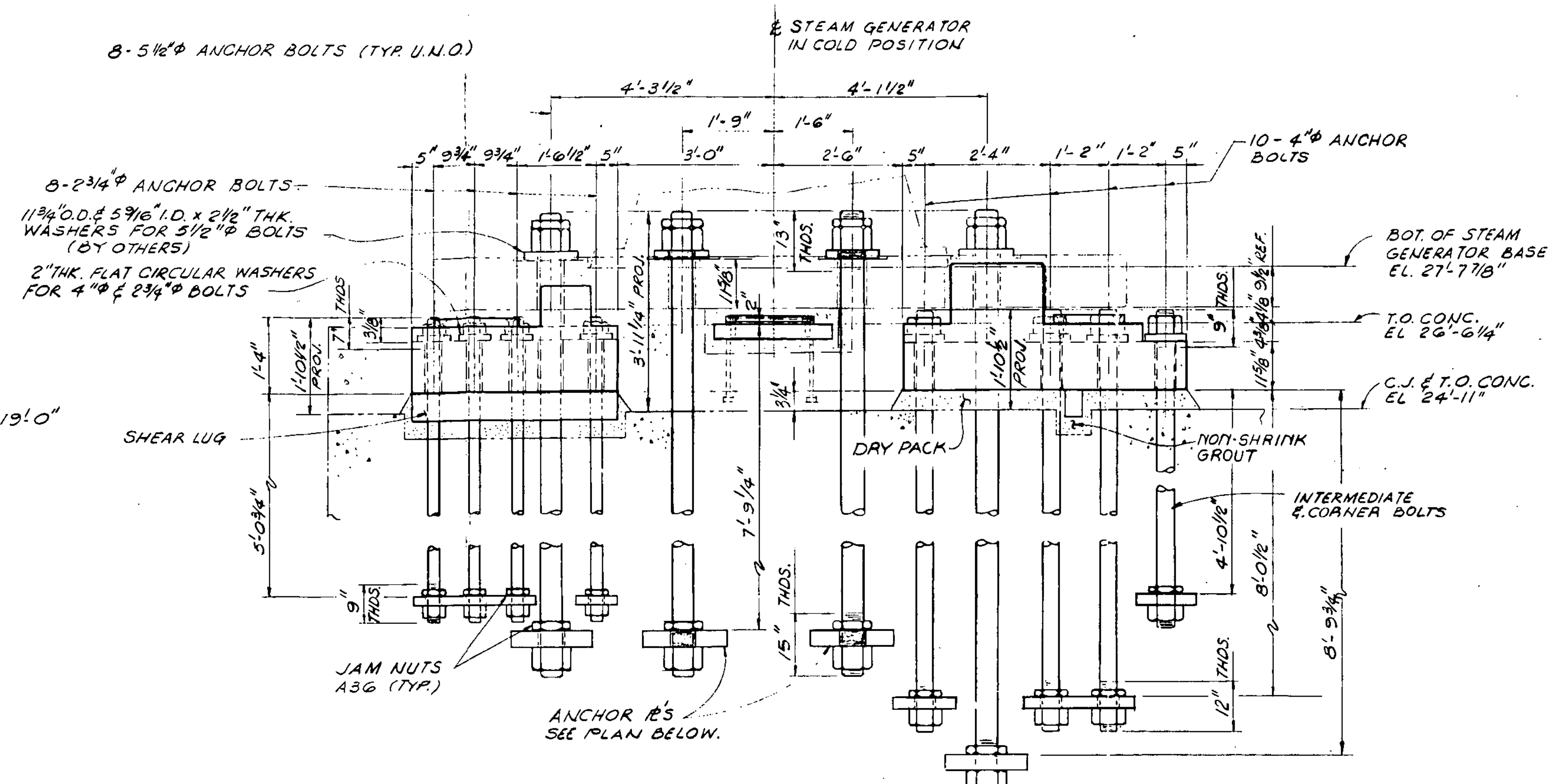
SAN ONOFRE
NUCLEAR GENERATING STATION
Units 2 & 3

REACTOR VESSEL SUPPORTS

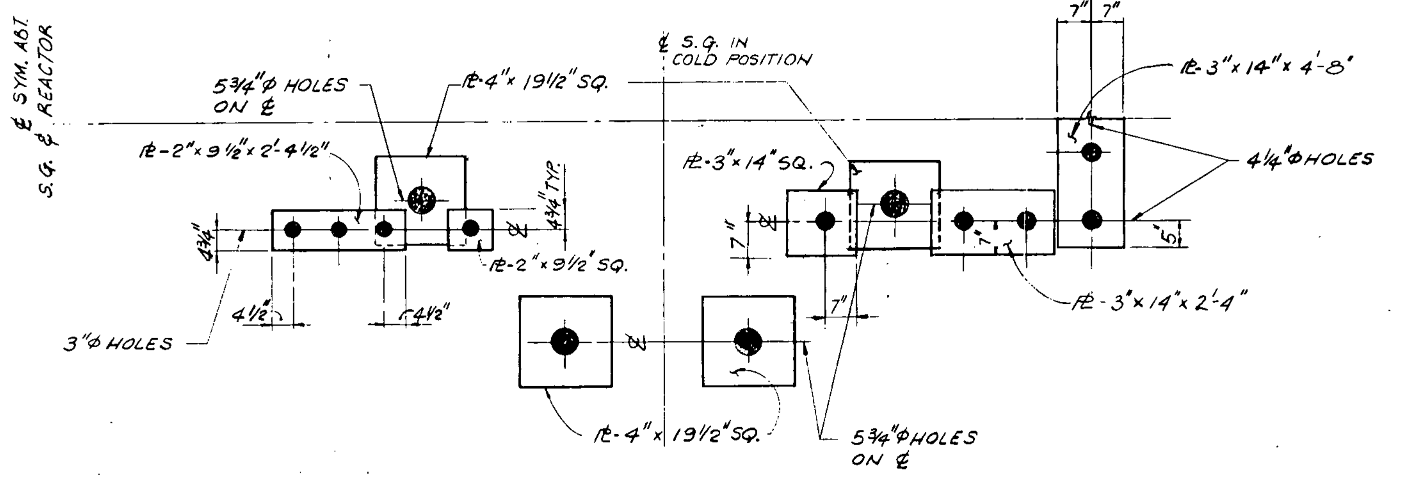
Figure 3.8-20



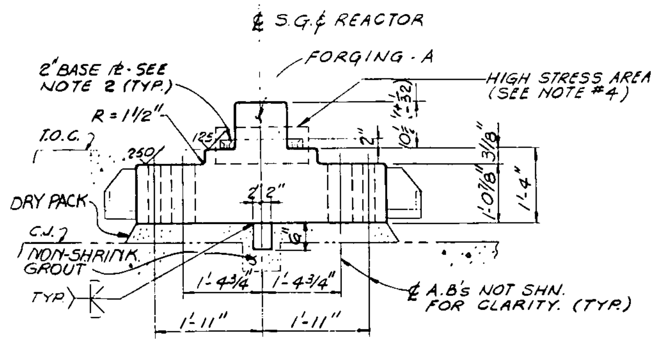
PLAN – STEAM GENERATOR SUPPORT



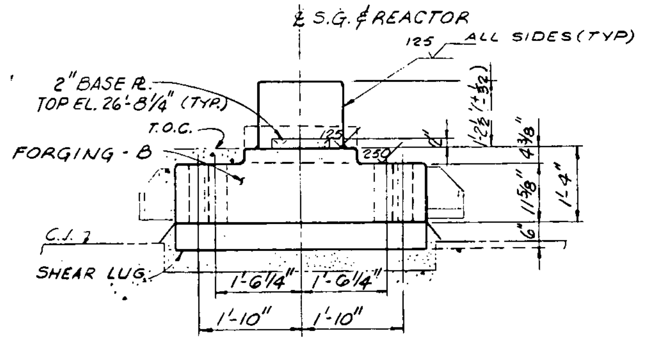
SECTION A



PLAN – ANCHOR ℞'S FOR S.G. SUPPORT A.B.'S



SECTION B



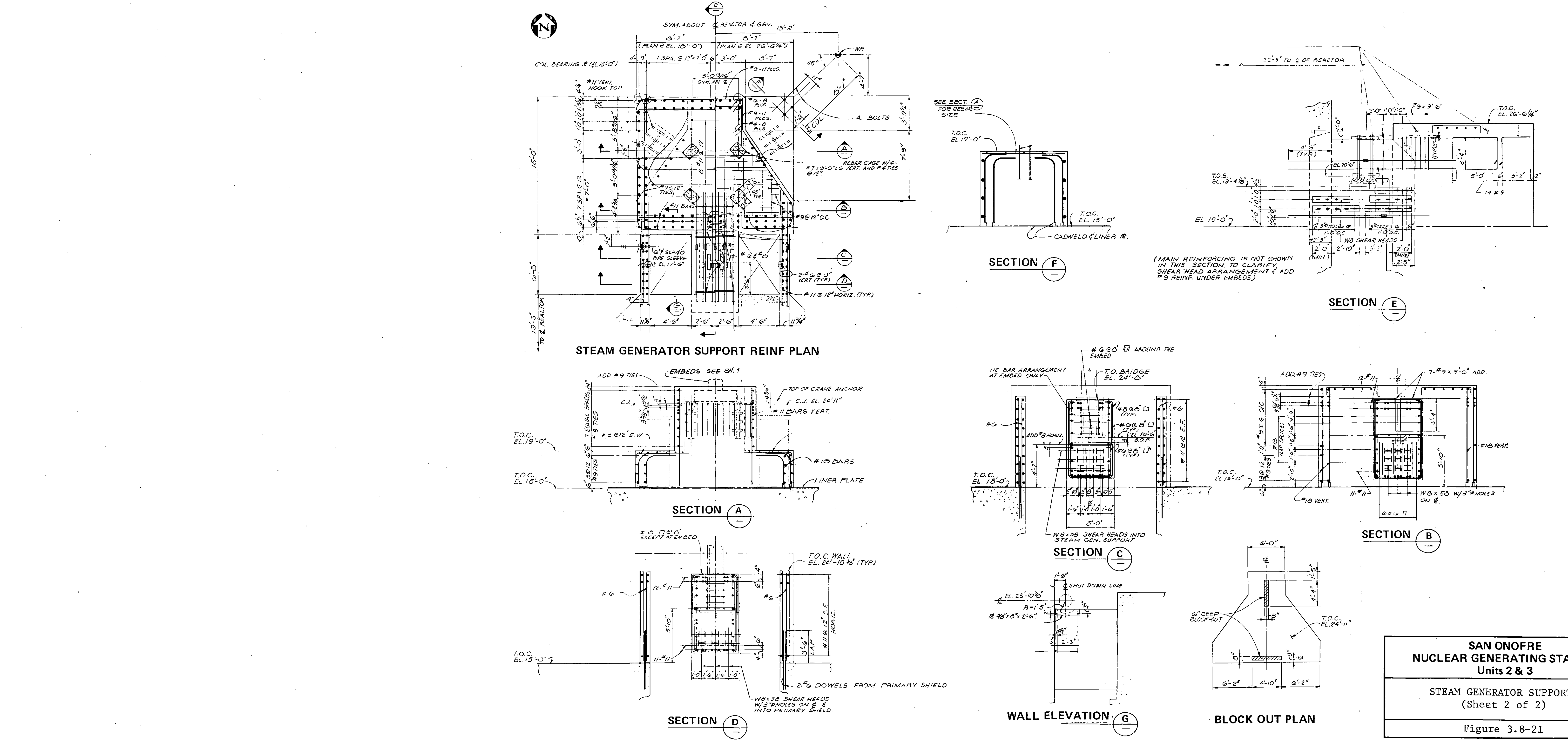
SECTION C

10079-11D-302
17MY6

SAN ONOFRE NUCLEAR GENERATING STATION Units 2 & 3

STEAM GENERATOR SUPPORTS (Sheet 1 of 2)

Figure 3.8-21

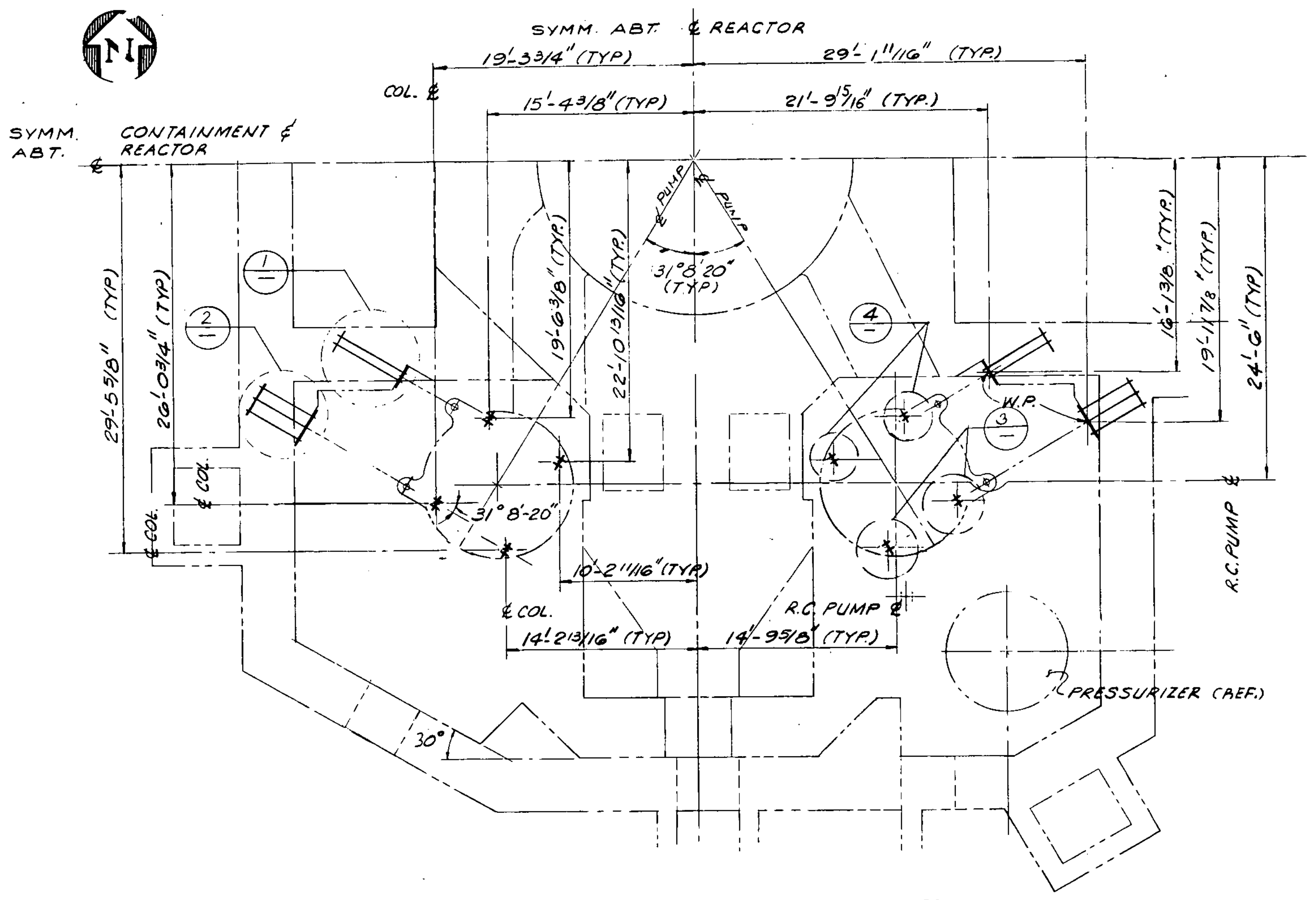


10079-11D-303
17MY6

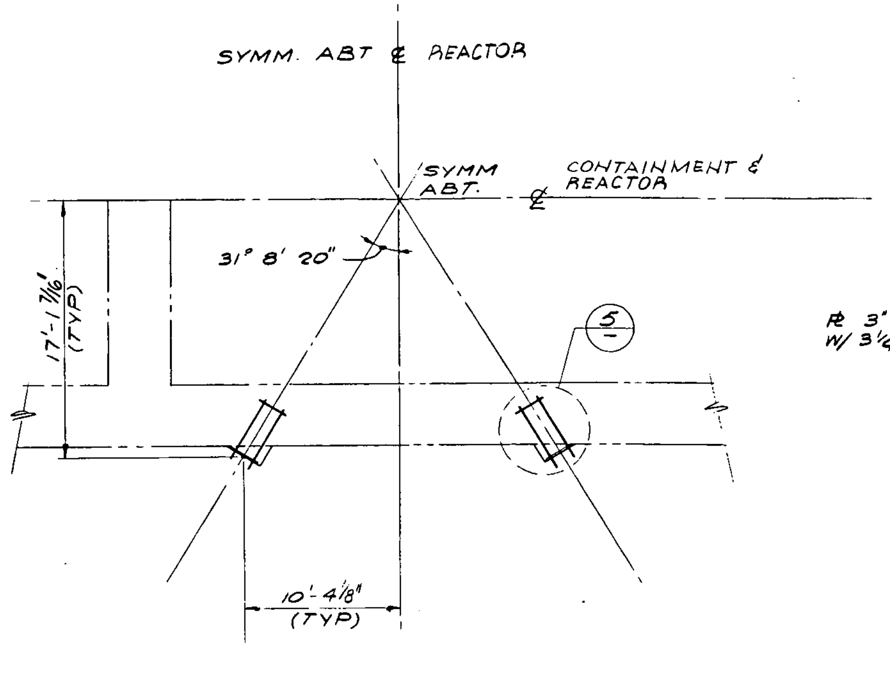
SAN ONOFRE
NUCLEAR GENERATING STATION
Units 2 & 3

STEAM GENERATOR SUPPORTS
(Sheet 2 of 2)

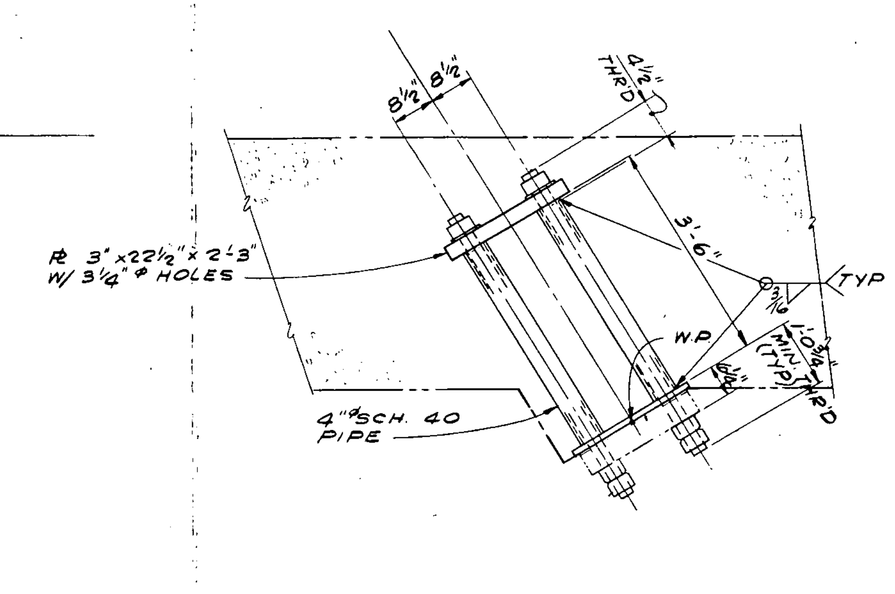
Figure 3.8-21



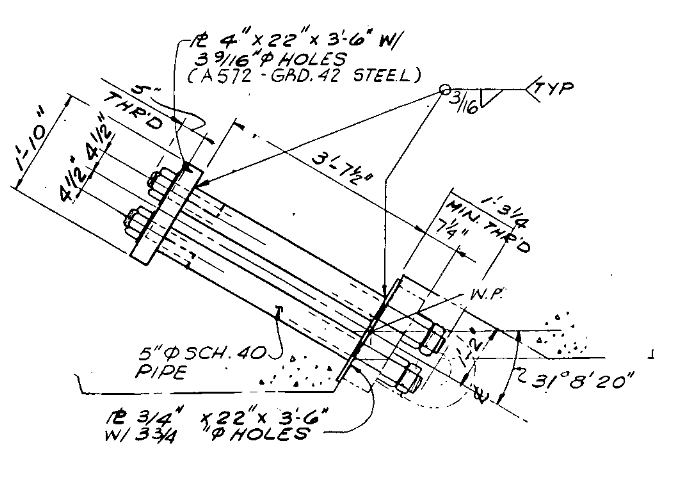
PUMP SUPPORTS PARTIAL PLAN
EL 26'-10-1/2" & EL 39'-11-1/8"



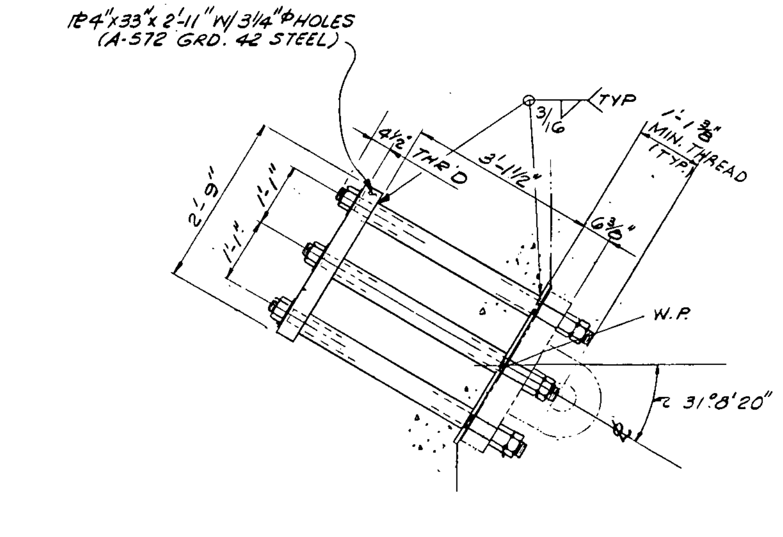
PARTIAL PLAN AT EL 50'-10-1/2"



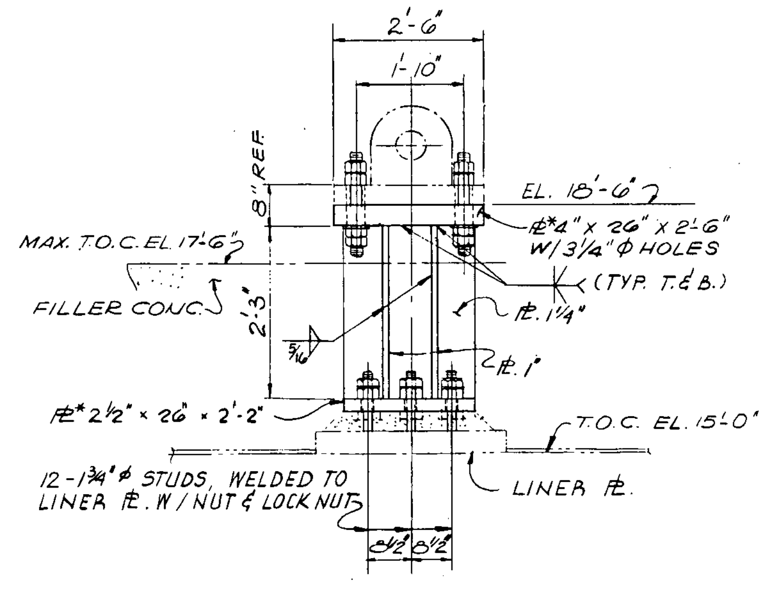
DETAIL 5



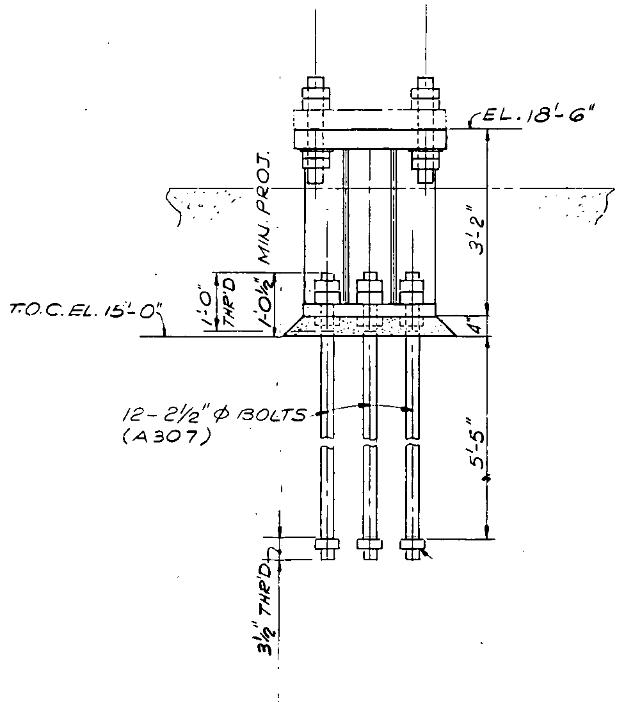
DETAIL 1



DETAIL 2



DETAIL 3



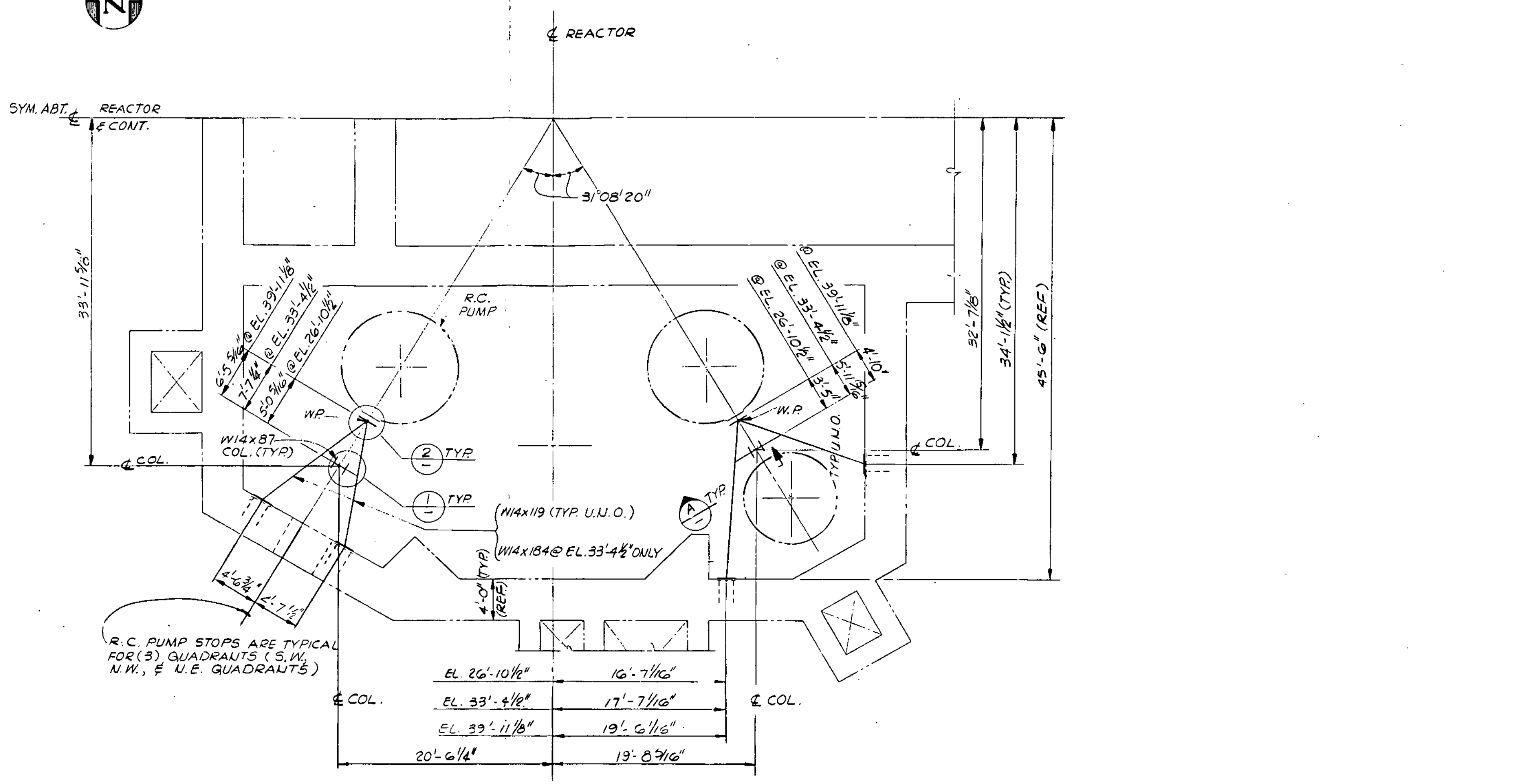
DETAIL 4

SAN ONOFRE
NUCLEAR GENERATING STATION
Units 2 & 3

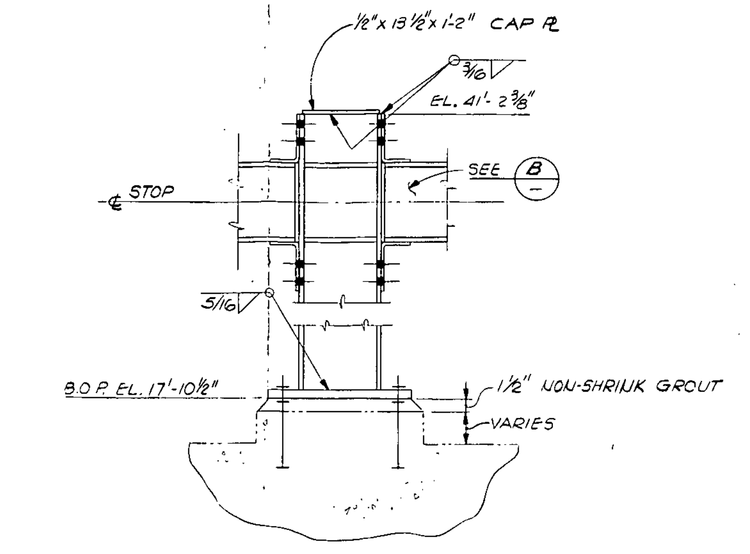
REACTOR COOLANT PUMP STOPS
AND SUPPORTS
(Sheet 1 of 2)

Figure 3.8-22

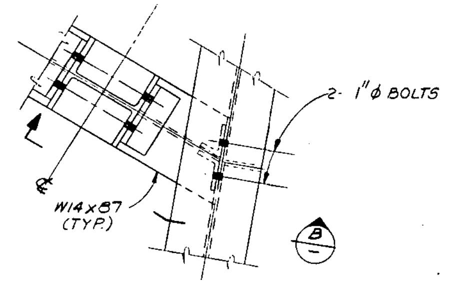
10079-11D-304
26AU6



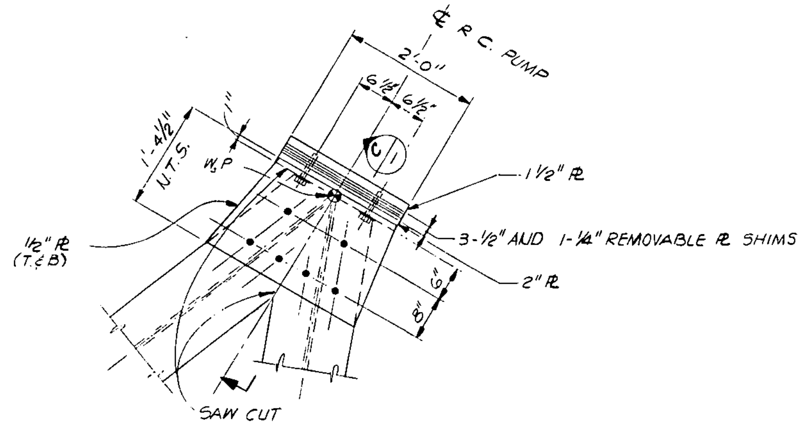
PUMP STOPS PARTIAL PLAN



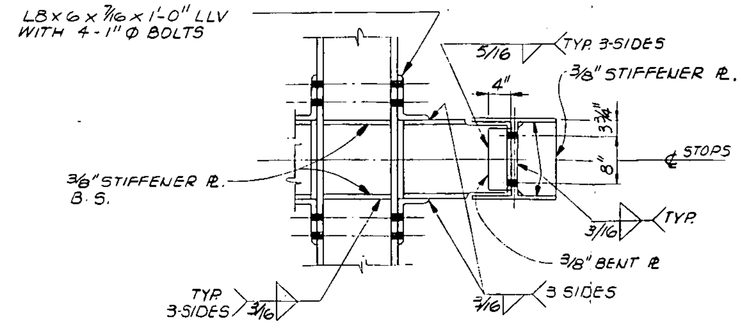
SECTION A



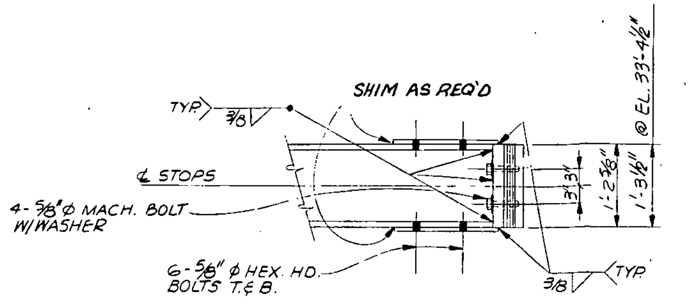
DETAIL 1



DETAIL 2



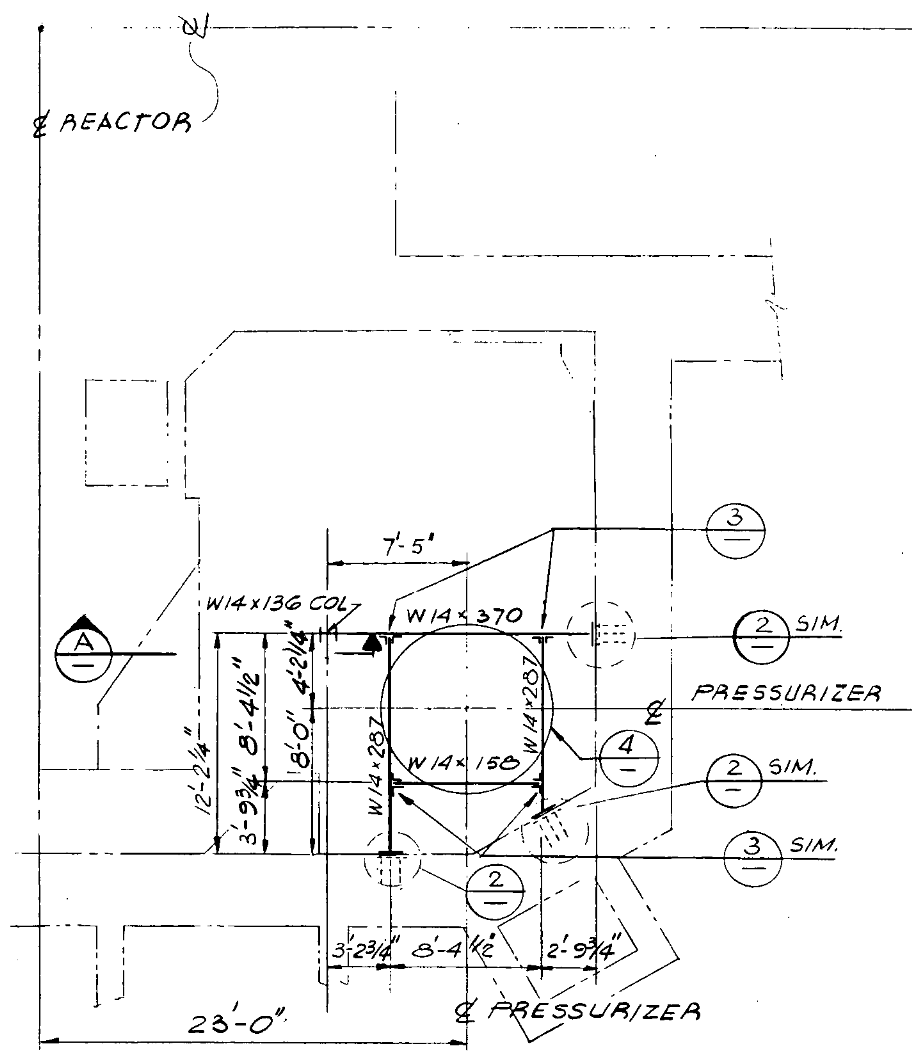
SECTION B



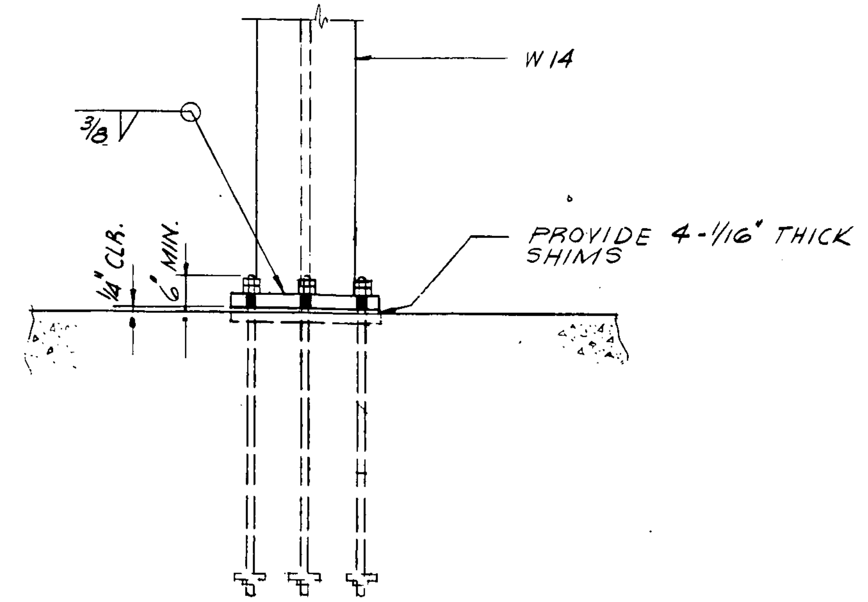
SECTION C

10079-11D-305
01OC6

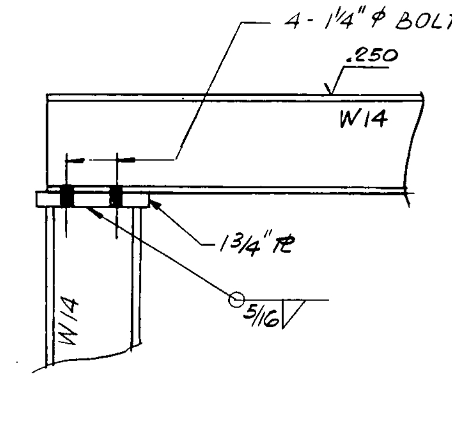
SAN ONOFRE **NUCLEAR GENERATING STATION** **Units 2 & 3**
REACTOR COOLANT PUMP STOPS AND SUPPORTS (Sheet 2 of 2)
Figure 3.8-22



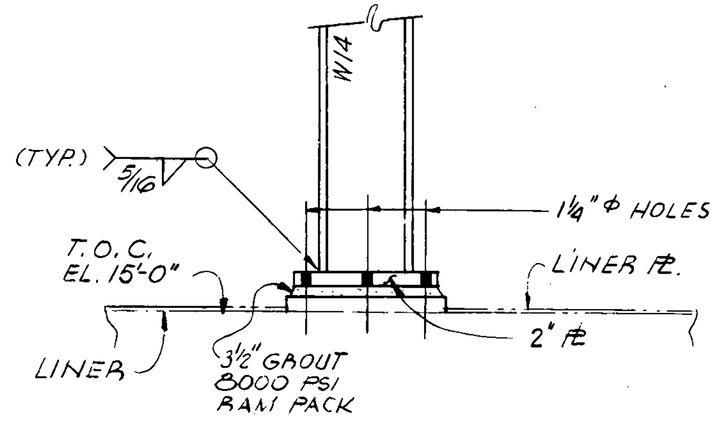
PRESSURIZER SUPPORTS PARTIAL PLAN
EL 45'-0"



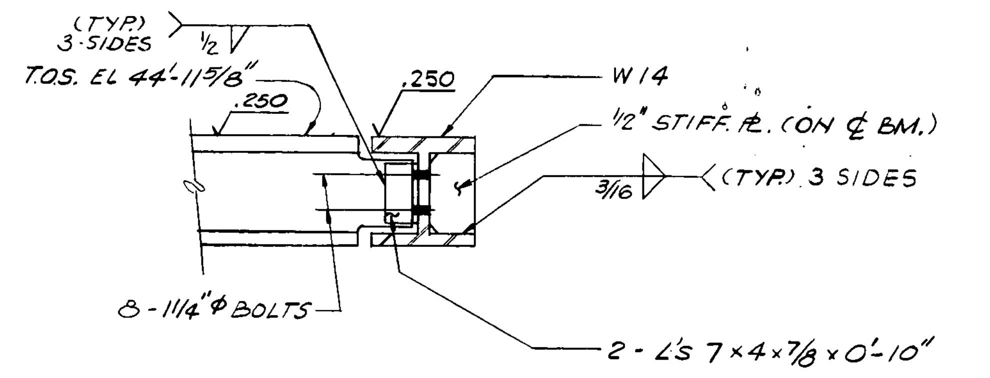
DETAIL 2



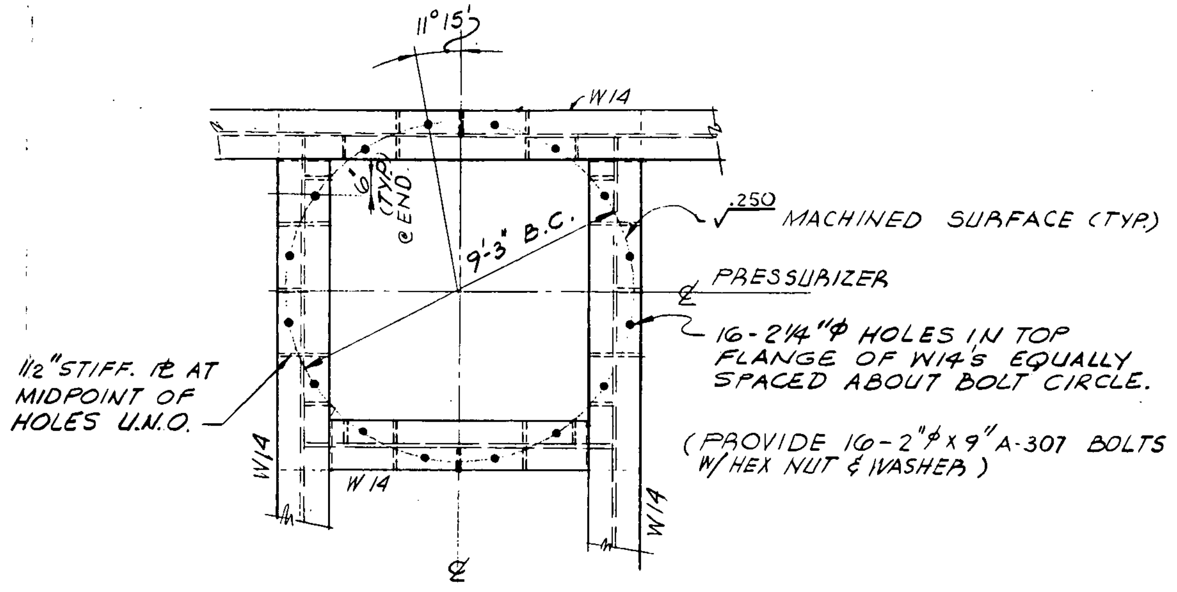
SECTION A



COLUMN BASE
DETAIL



DETAIL 3



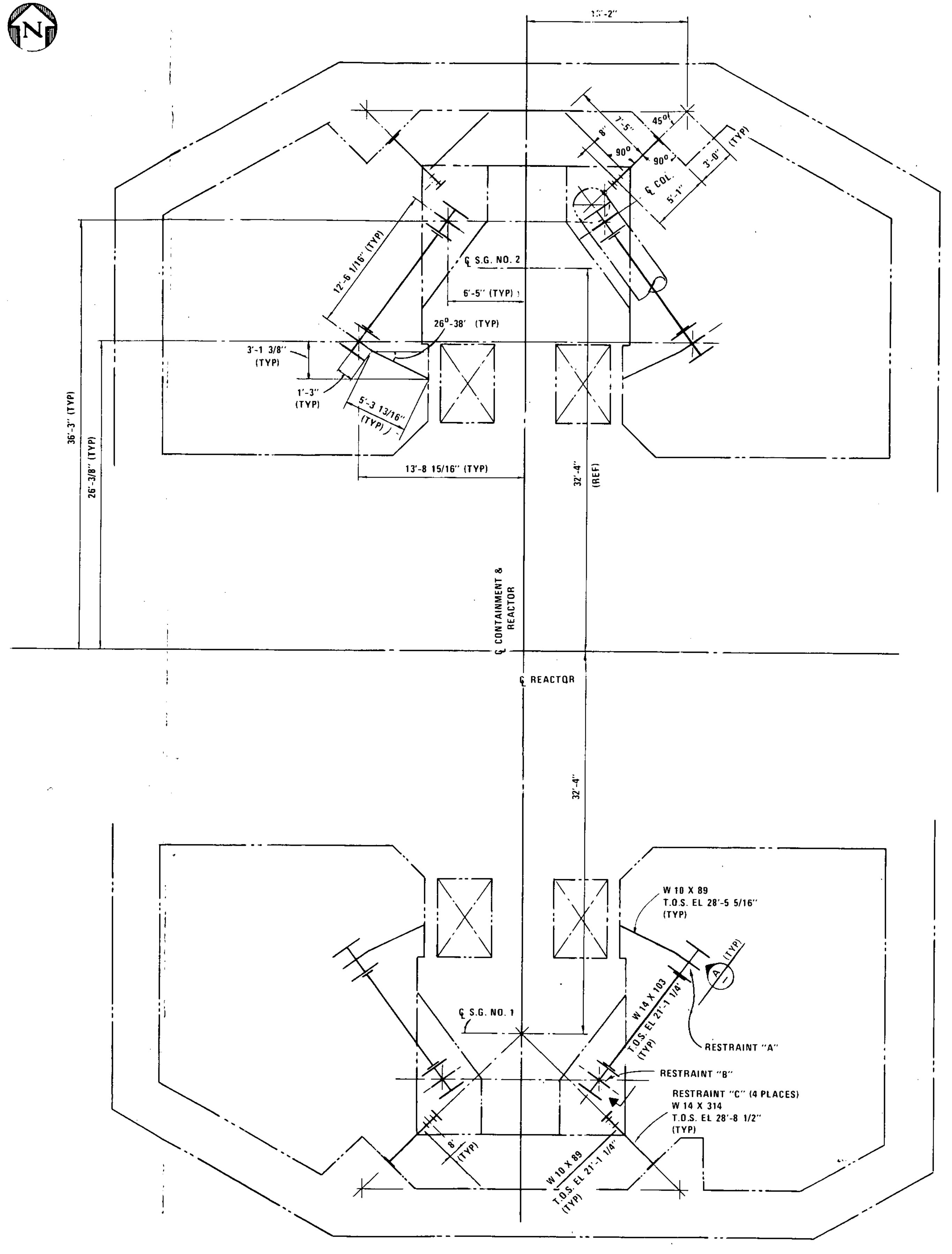
PLAN – DETAIL 4

10079-11D-306
17MY6

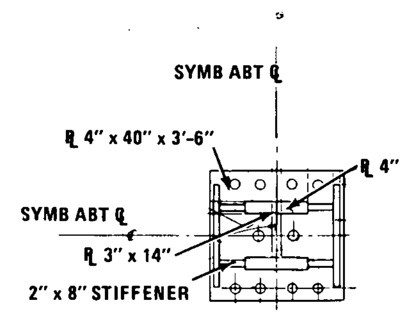
SAN ONOFRE
NUCLEAR GENERATING STATION
Units 2 & 3

PRESSURIZER SUPPORTS

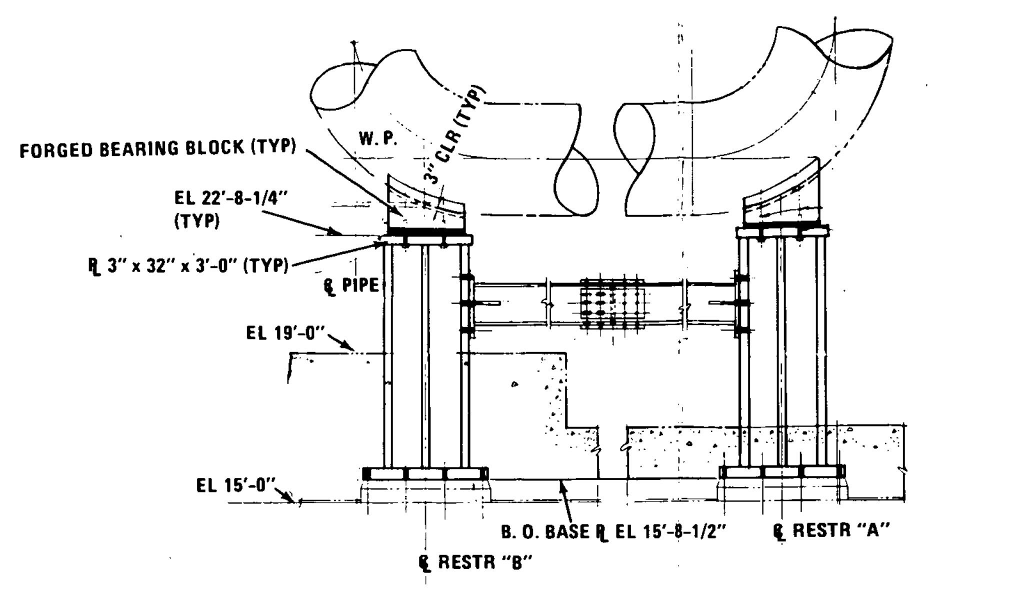
Figure 3.8-23



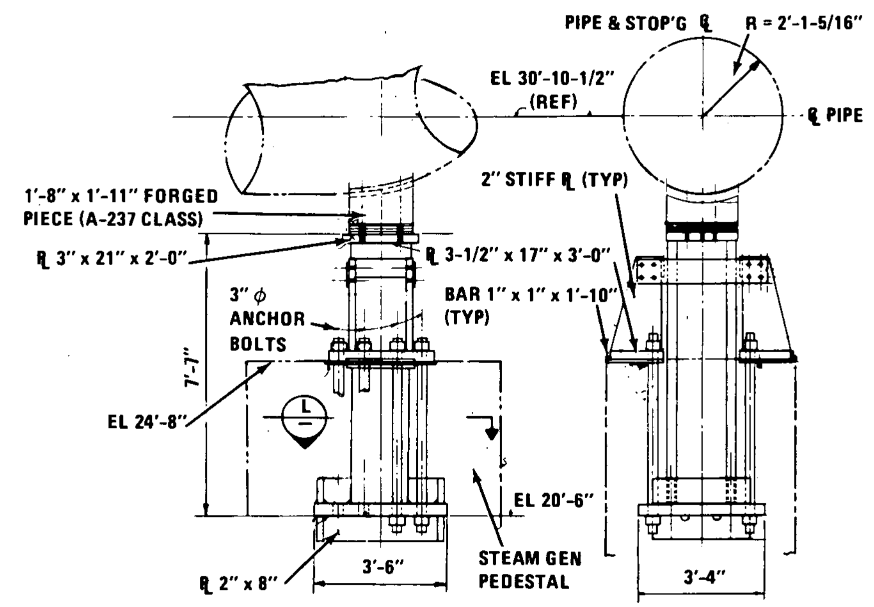
REACTOR COOLANT PIPE RESTRAINTS



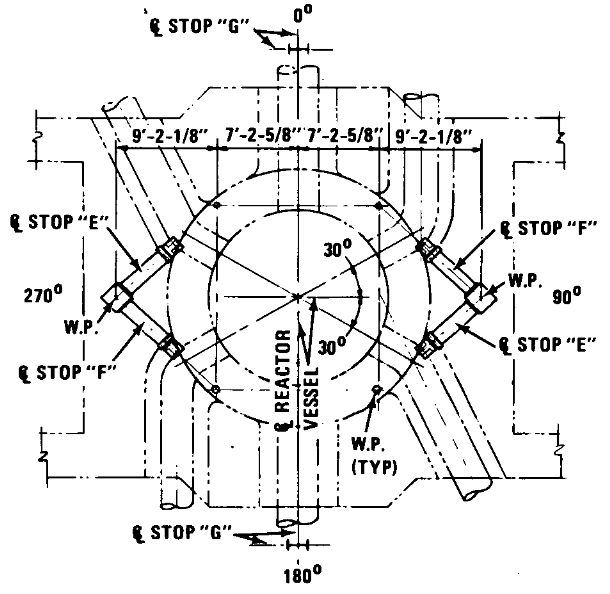
SECTION L



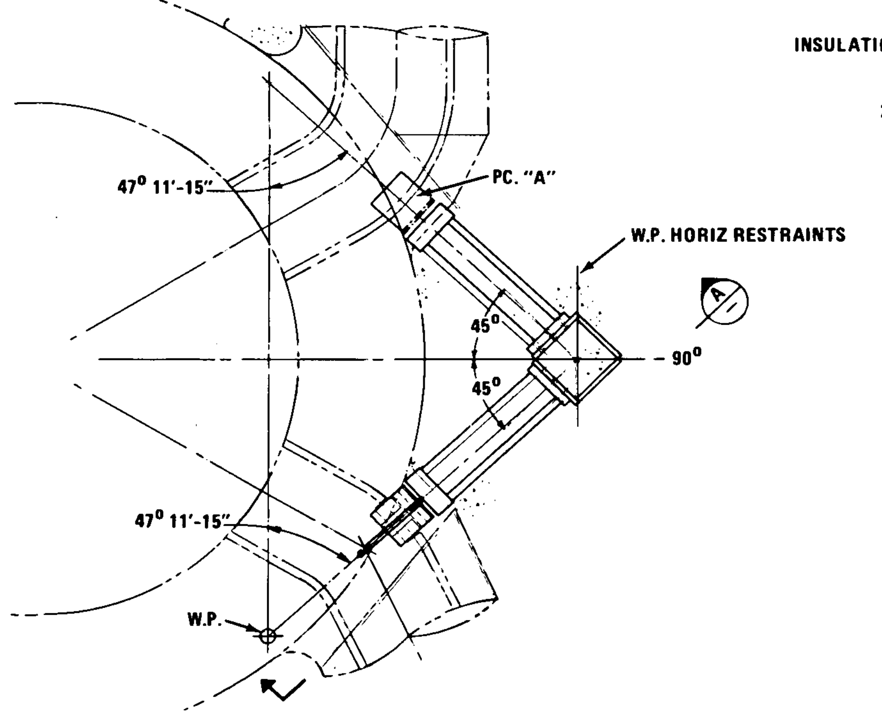
REACTOR COOLANT PUMP SUCTION PIPE RESTRAINTS
ELEVATION A



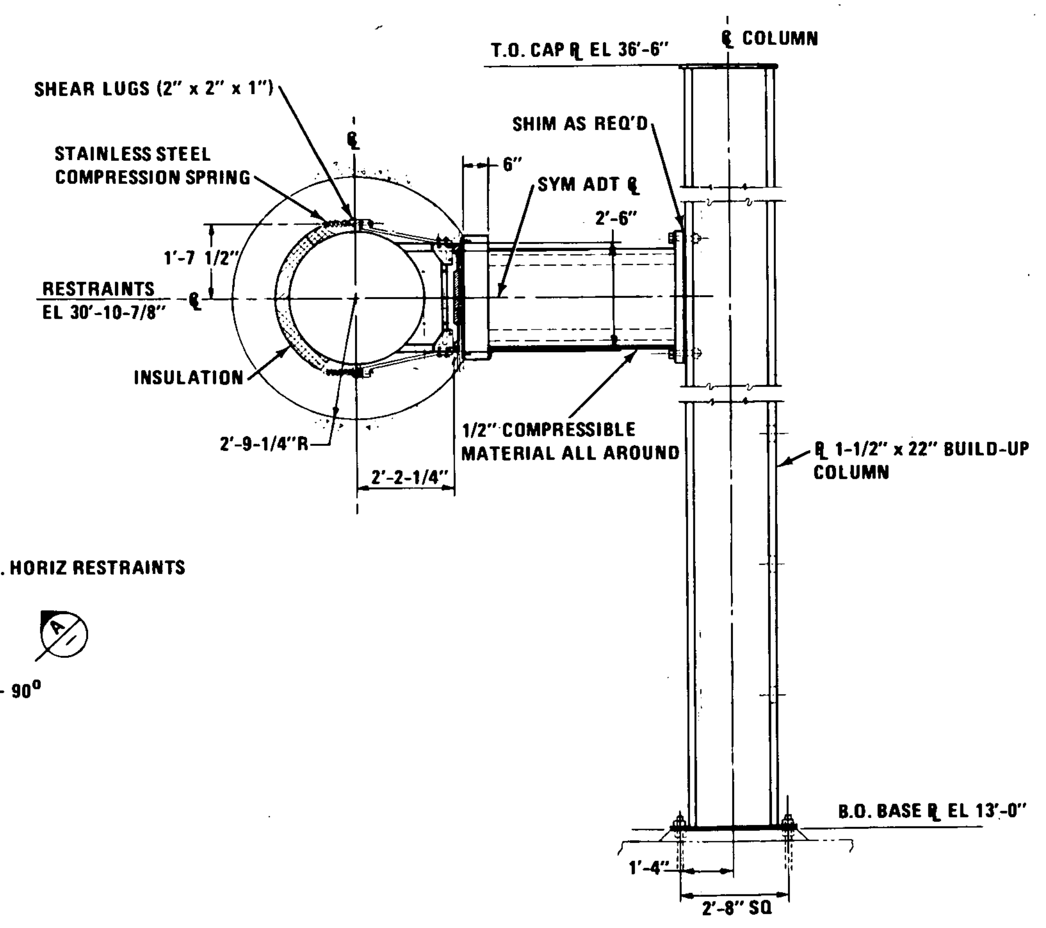
OUTLET REACTOR VESSEL PIPE RESTRAINTS



KEY PLAN



PARTIAL PLAN

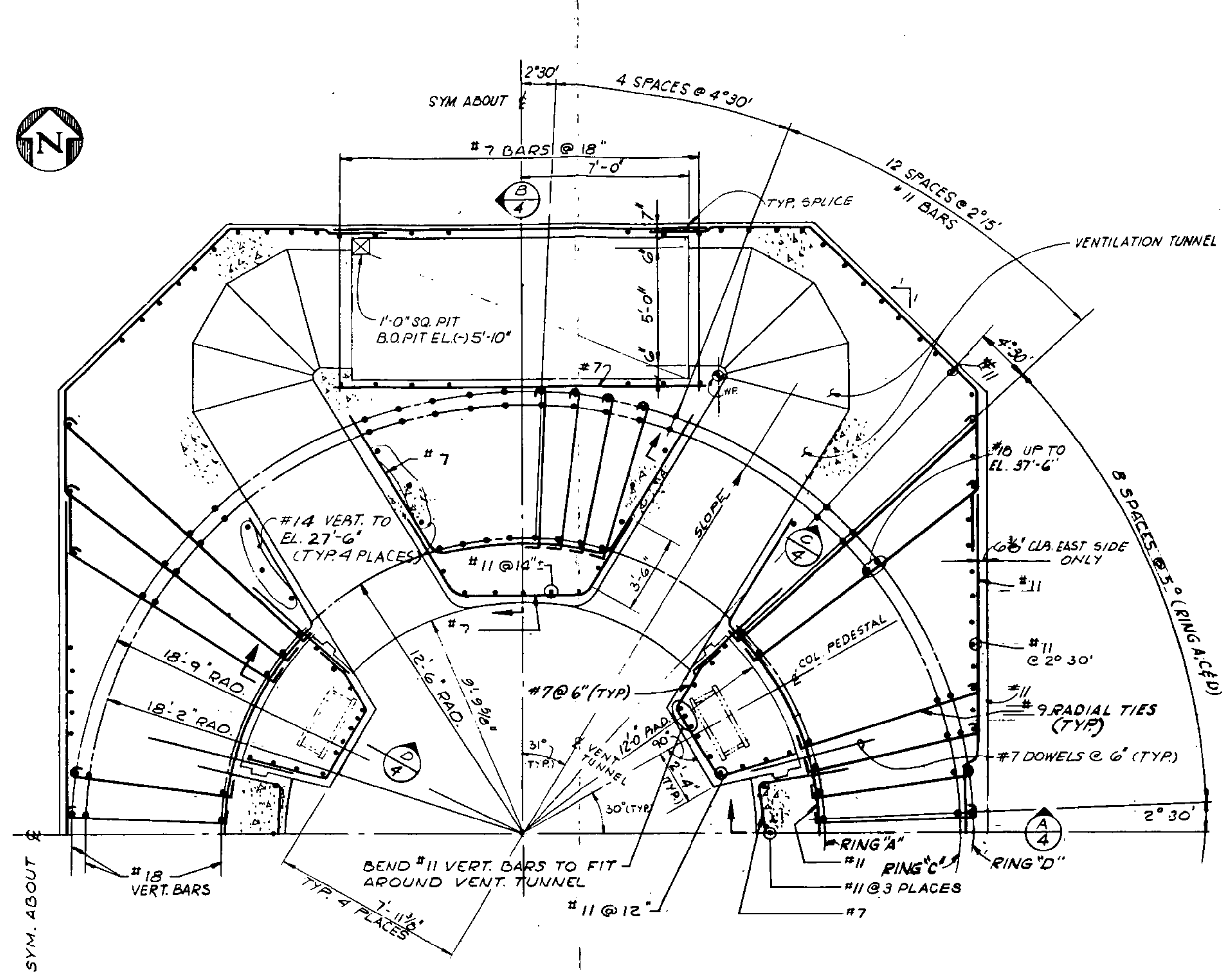


COLUMN ELEVATION A

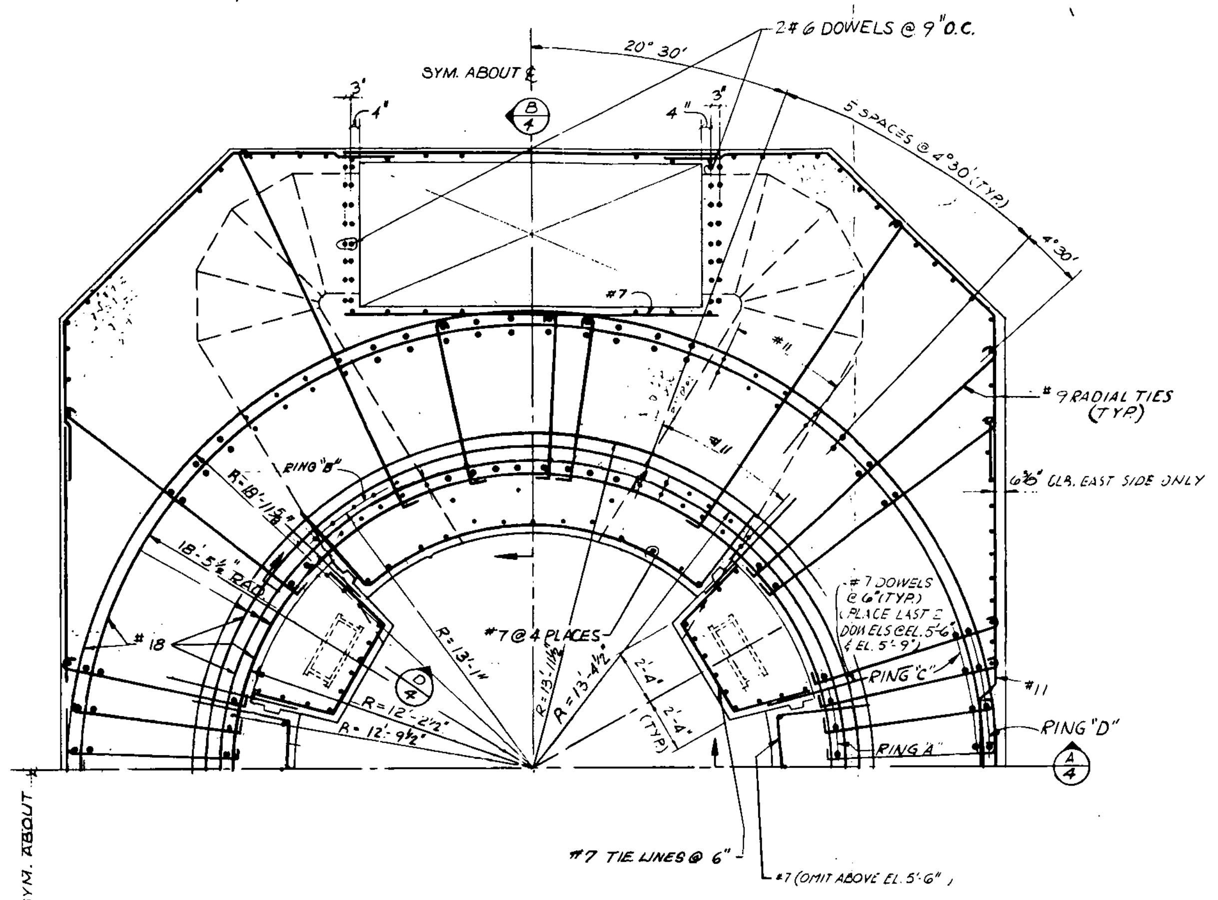
REACTOR VESSEL INLET PIPE RESTRAINTS

10079-11D-307
17MY6

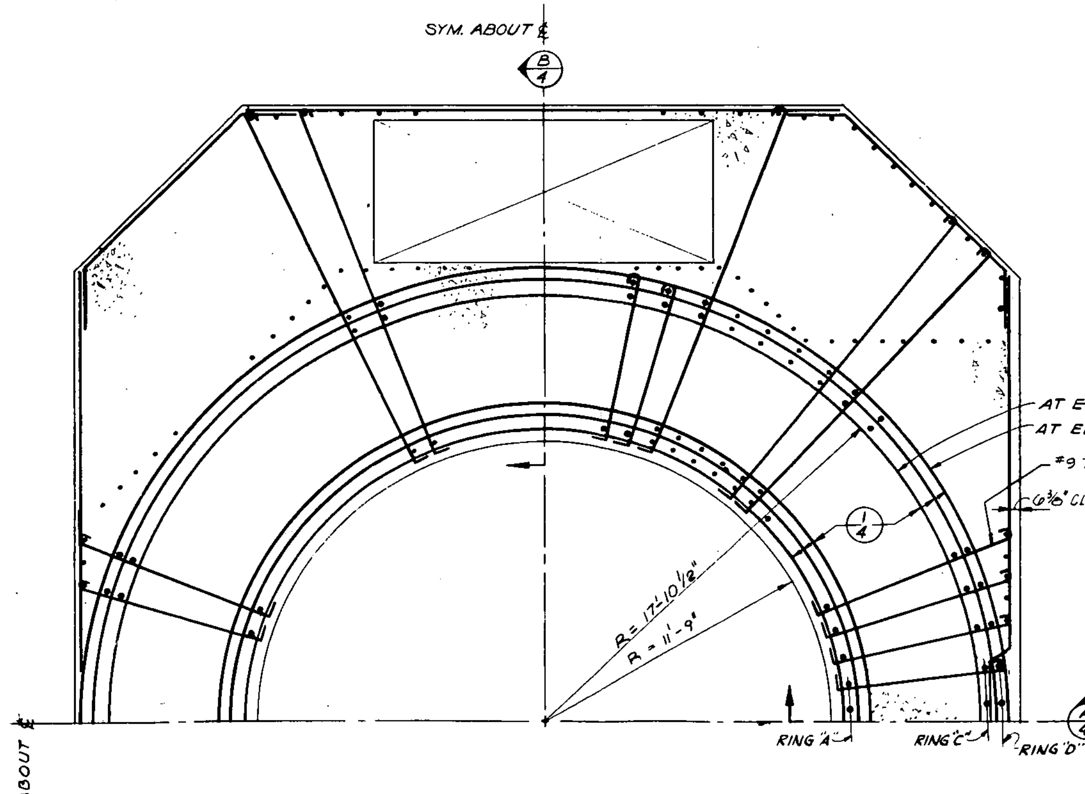
SAN ONOFRE NUCLEAR GENERATING STATION Units 2 & 3
REACTOR COOLANT PIPE RESTRAINTS
Figure 3.8-24



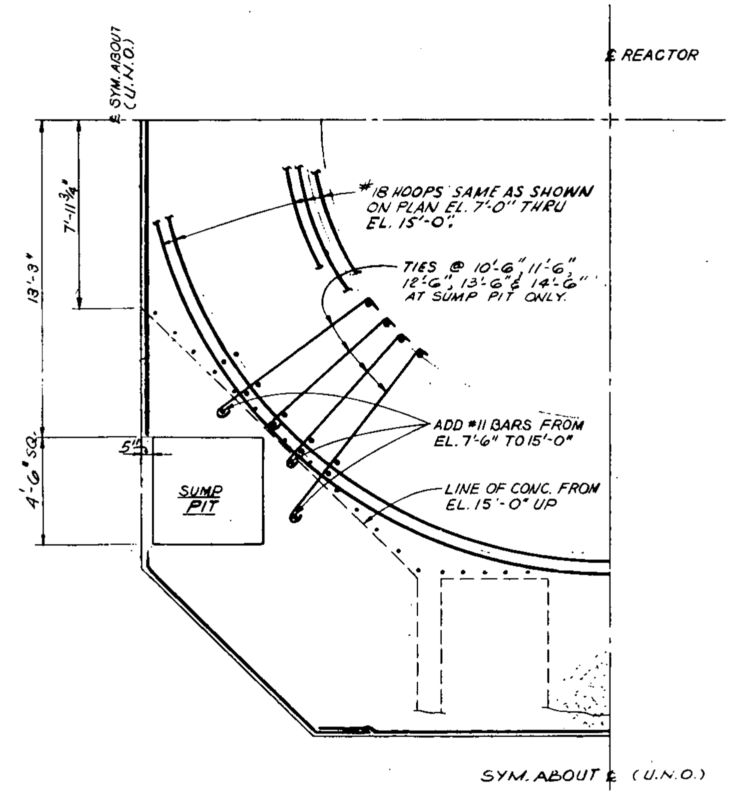
PRIMARY SHIELD—PARTIAL REINFORCING PLAN
AT EL (-) 2'-3" (±)



PRIMARY SHIELD—PARTIAL PLAN ABOVE VENT DUCTS
AT EL 3'-0" (±)



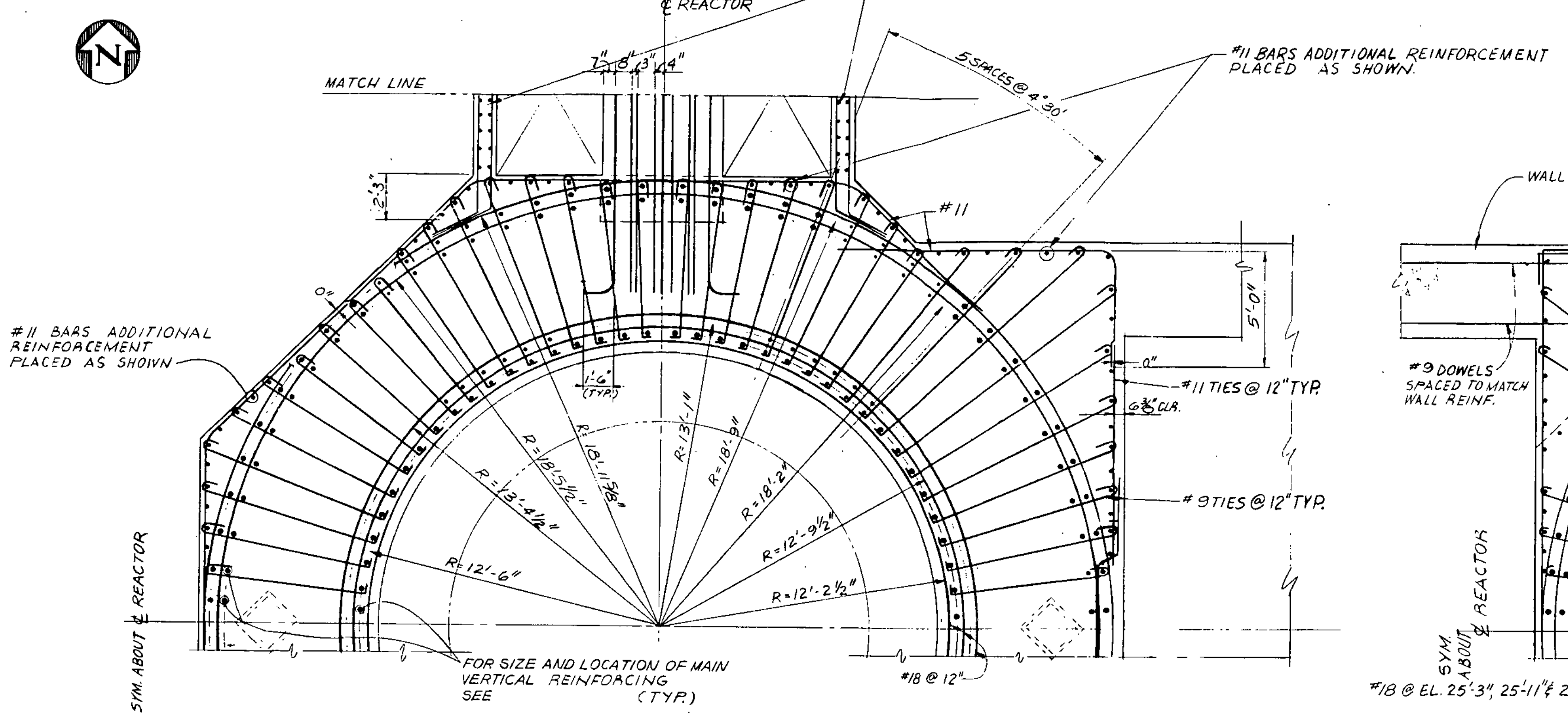
PRIMARY SHIELD—PARTIAL REINFORCING PLAN
FROM EL 7'-0" THRU EL 15'-0"



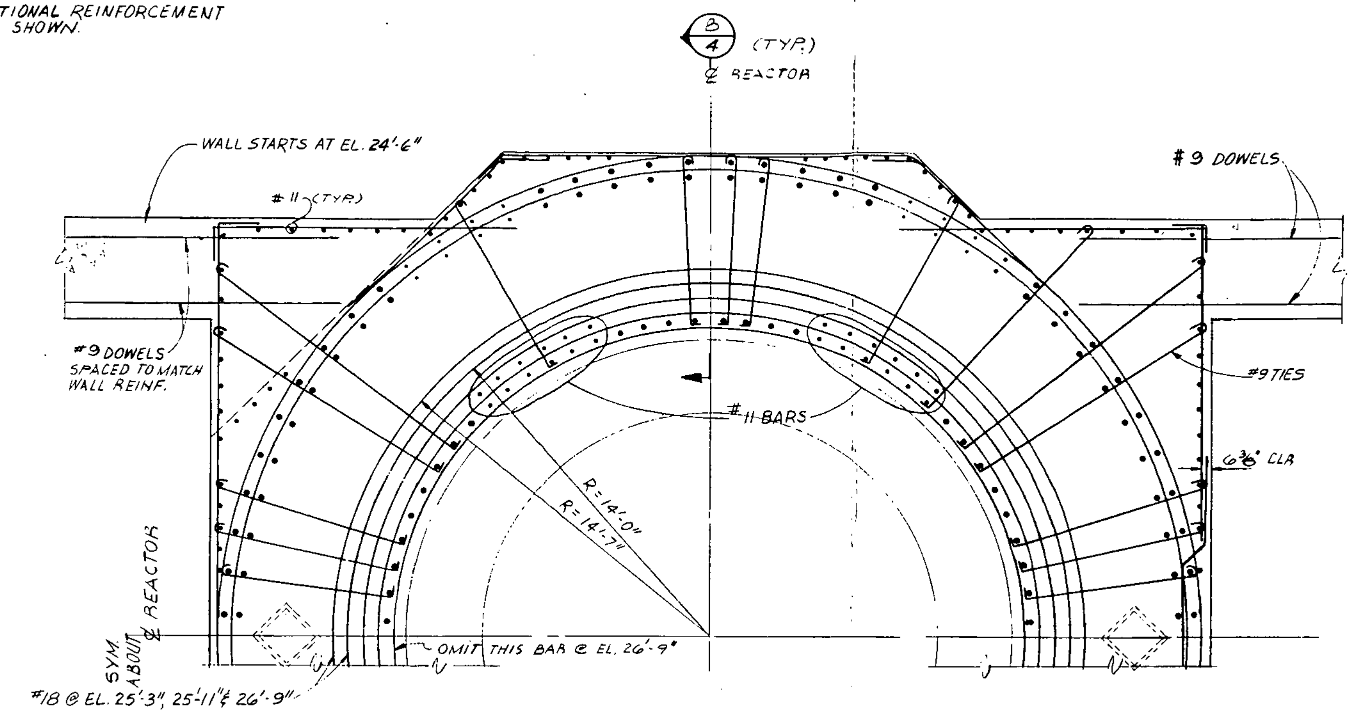
PRIMARY SHIELD PARTIAL REINFORCING PLAN
AT SUMP PIT

10079-11D-308
26AU6

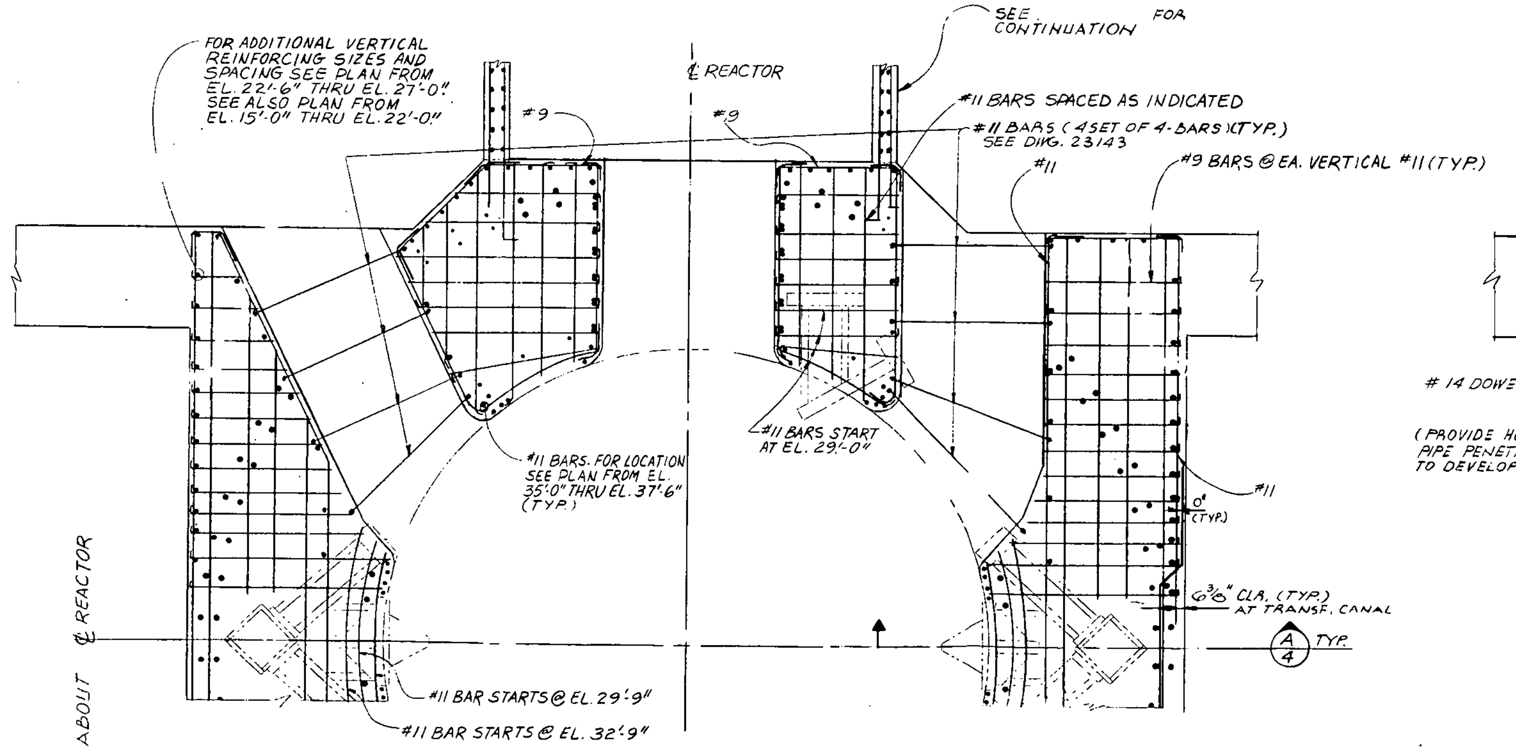
SAN ONOFRE NUCLEAR GENERATING STATION Units 2 & 3
PRIMARY SHIELD WALLS AND REACTOR CAVITY REINFORCING STEEL (Sheet 1 of 4)
Figure 3.8-25



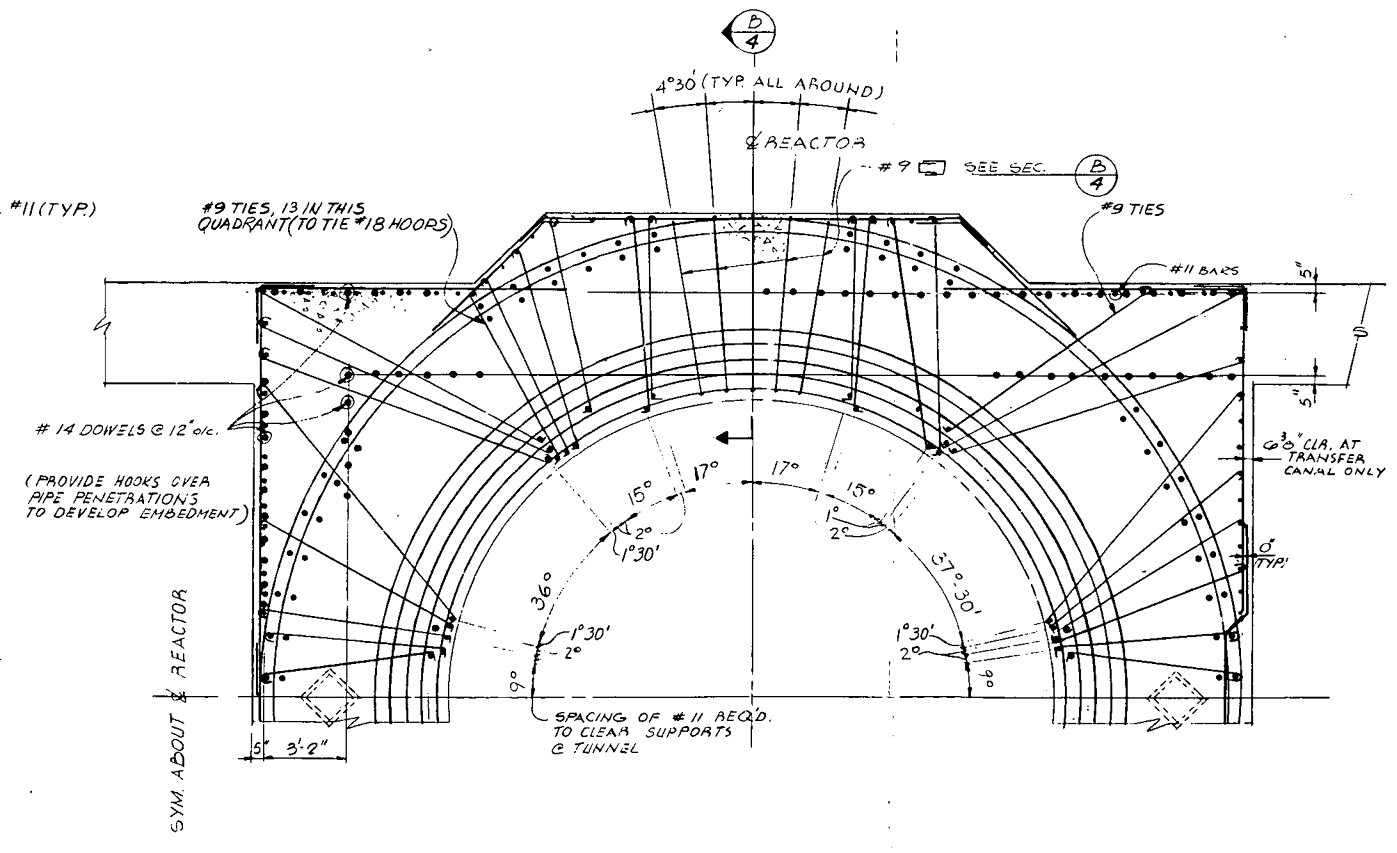
PRIMARY SHIELD REINFORCING
FROM EL 15'-0" THRU 22'-0"



PRIMARY SHIELD REINFORCING
FROM EL 22'-6" THRU 27'-0"



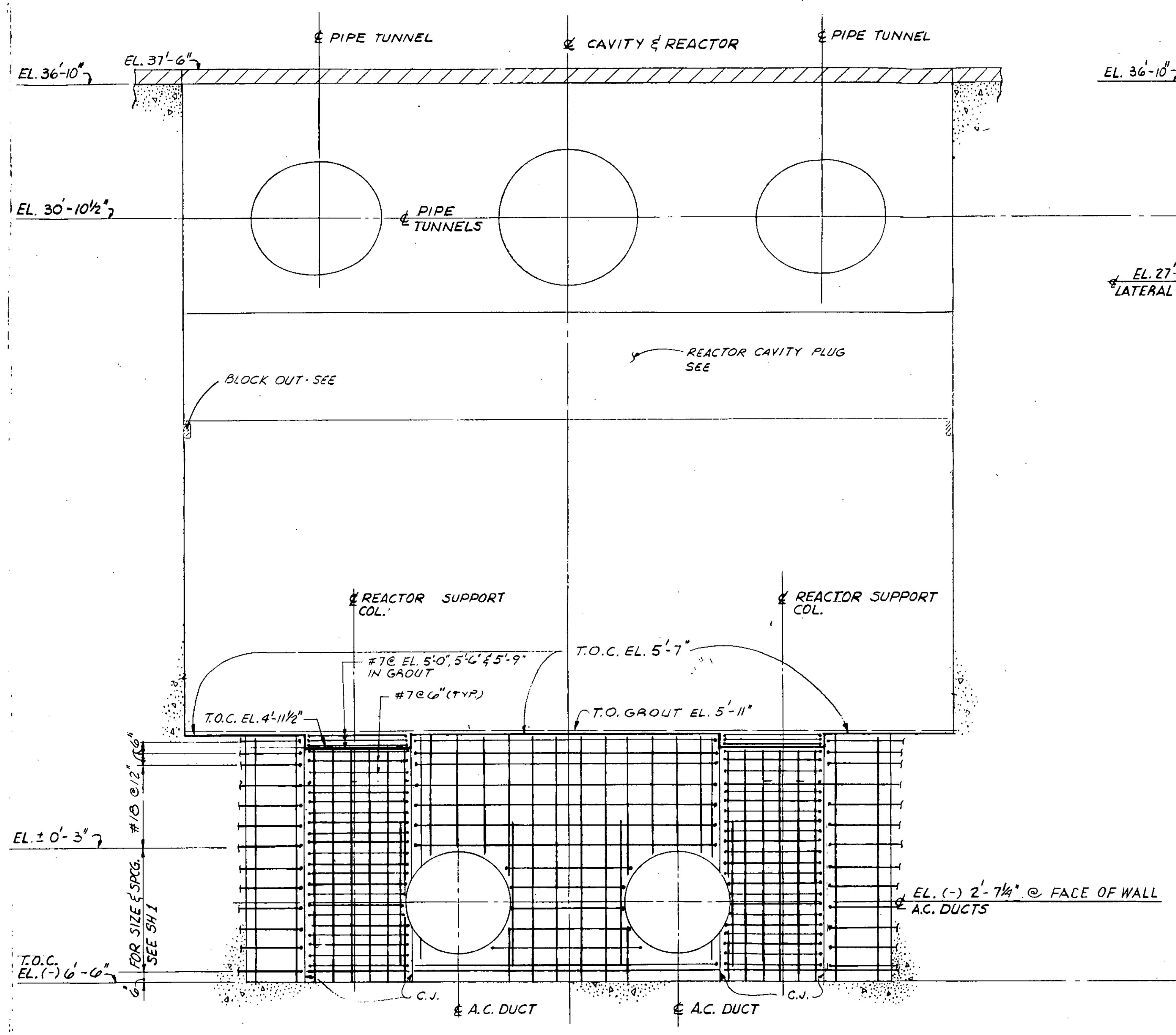
PRIMARY SHIELD REINFORCING
AT EL 30'-10-1/2"



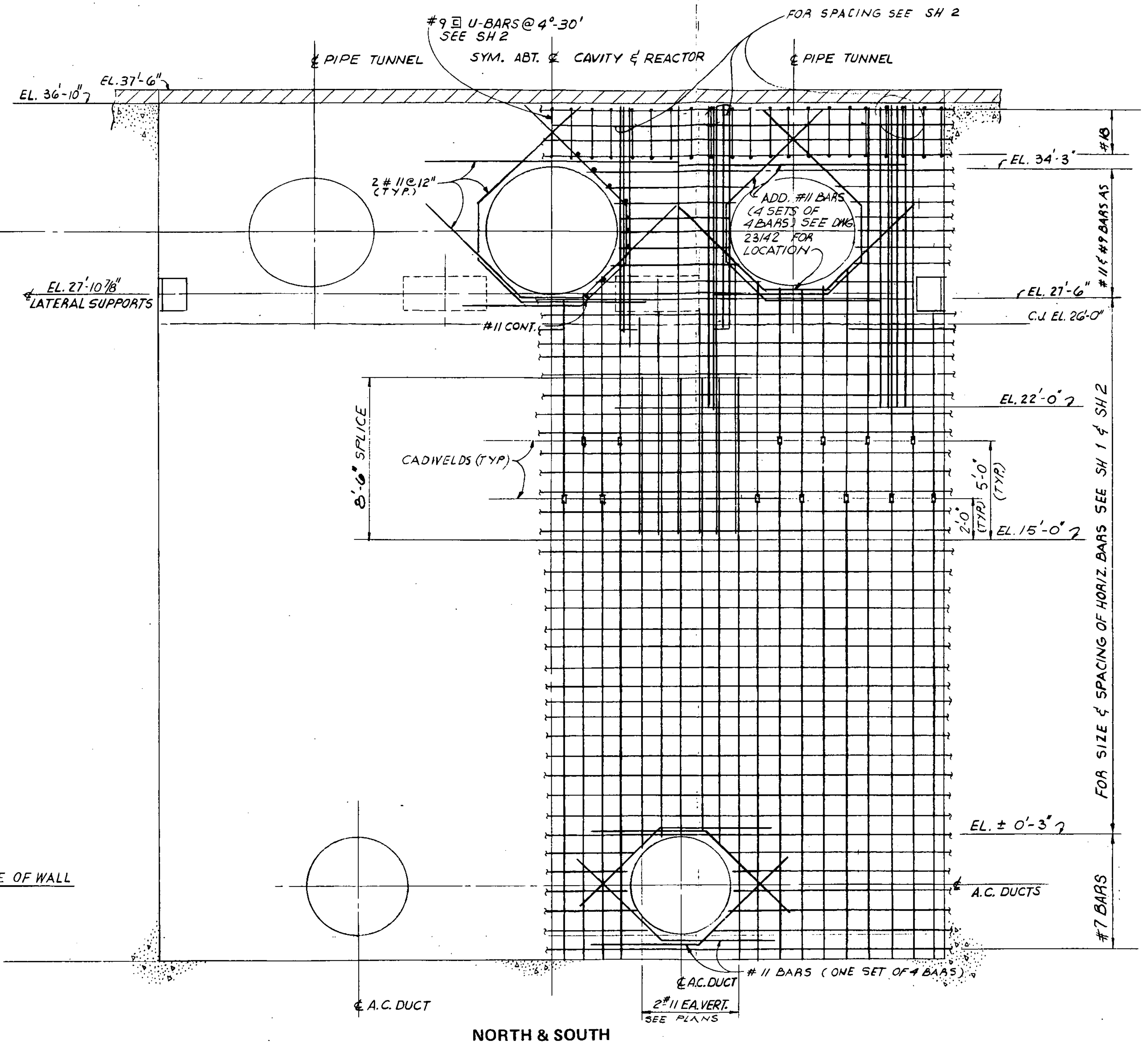
PRIMARY SHIELD REINFORCING
FROM EL 35'-0" THRU EL 37'-6"

SAN ONOFRE NUCLEAR GENERATING STATION Units 2 & 3
PRIMARY SHIELD WALLS AND REACTOR CAVITY REINFORCING STEEL (Sheet 2 of 4)
Figure 3.8-25

10079-11D-309
26AU6



NORTH & SOUTH
PARTIAL DEVELOPED ELEV OF REACTOR CAVITY
LOOKING FROM INSIDE (R = 9'-9 5/8" U.N.O.)
SHOWING REINFORCING STEEL



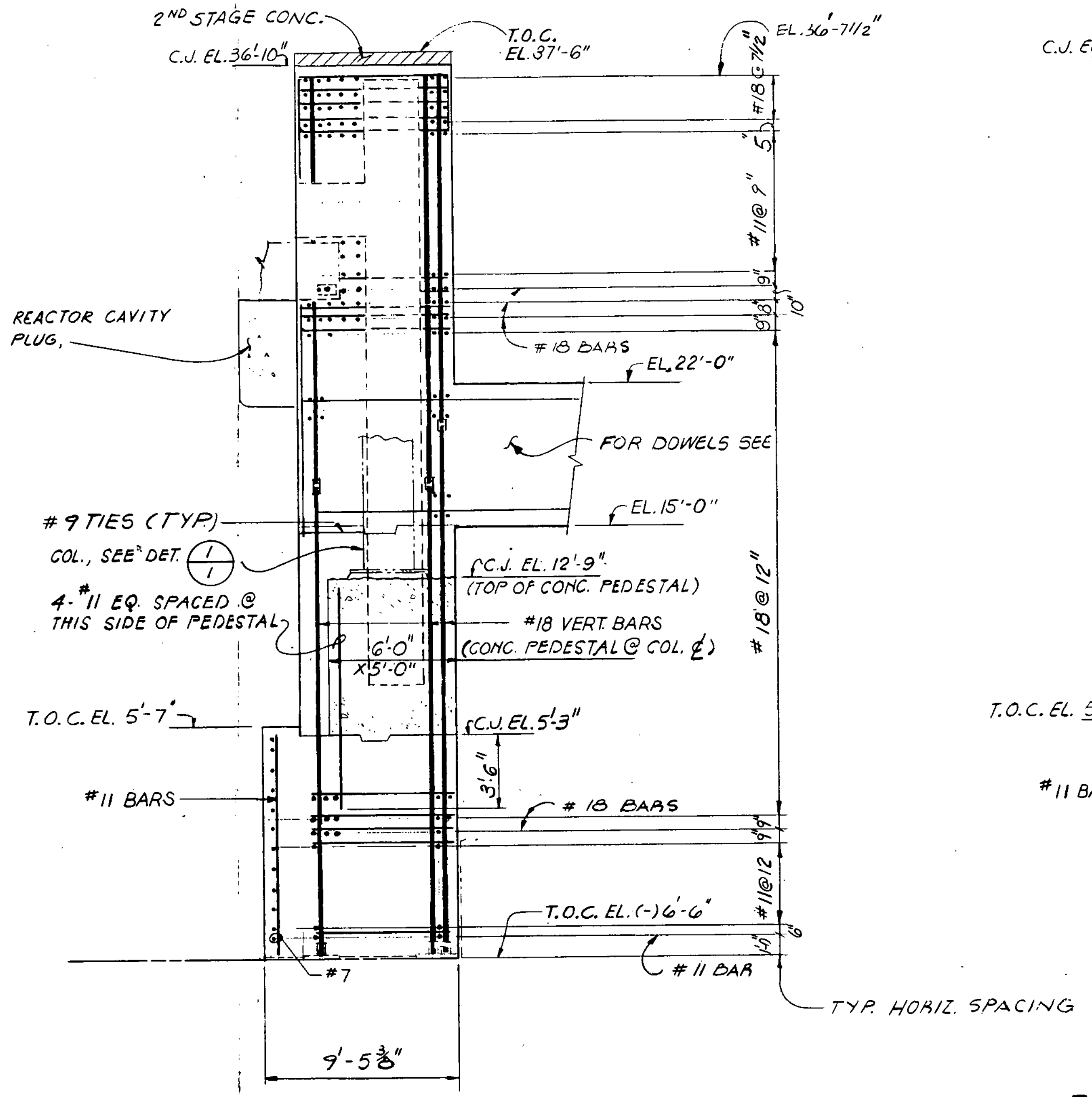
NORTH & SOUTH
PARTIAL DEVELOPED ELEV OF REACTOR CAVITY
LOOKING FROM INSIDE (R = 12'-6" U.N.O.)
SHOWING REINFORCING STEEL

LEGEND

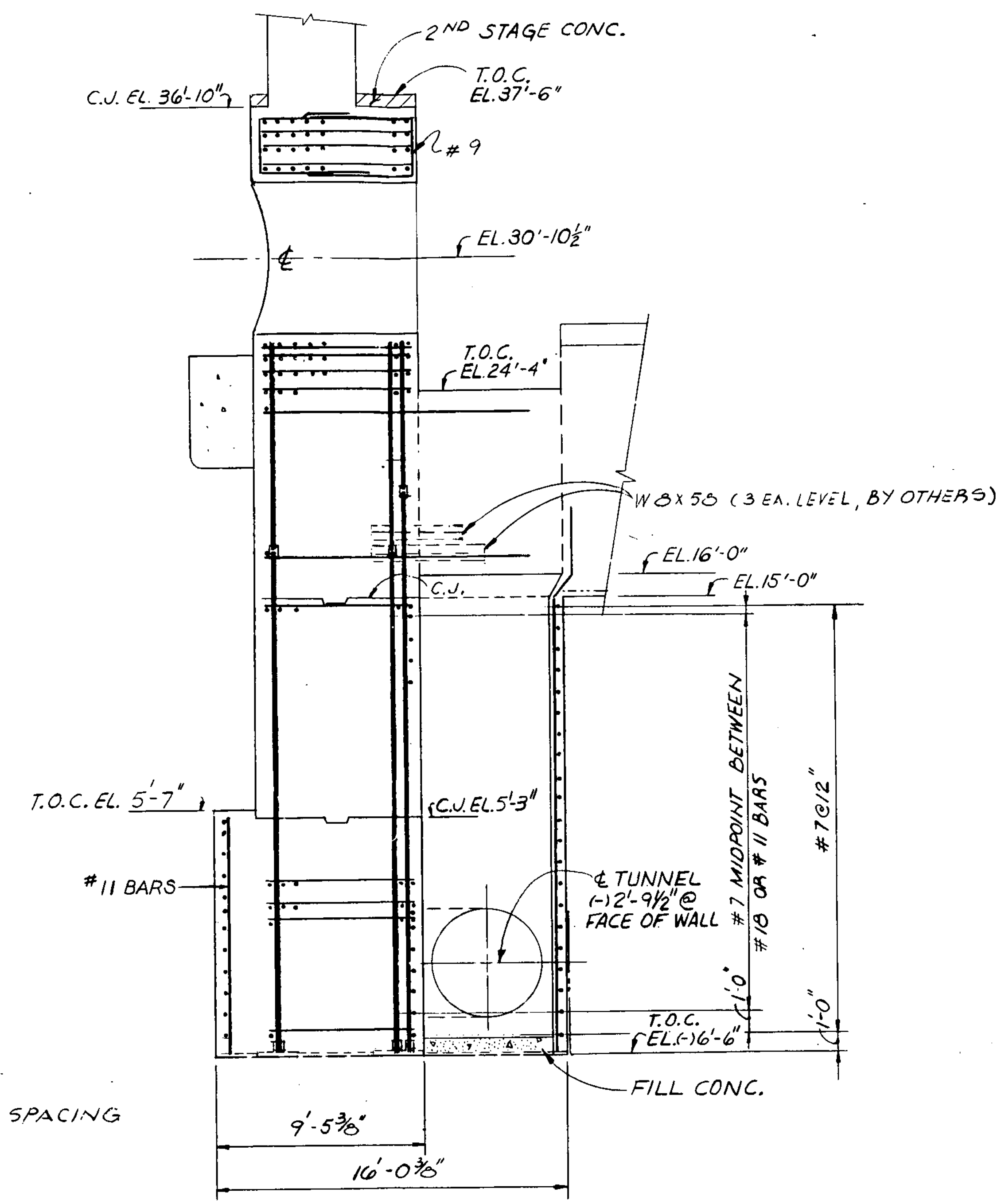
CROSS HATCHED AREA INDICATES SECOND STAGE CONC.

SAN ONOFRE NUCLEAR GENERATING STATION Units 2 & 3
PRIMARY SHIELD WALLS AND REACTOR CAVITY REINFORCING STEEL (Sheet 3 of 4)
Figure 3.8-25

10079-11D-310
26AU6

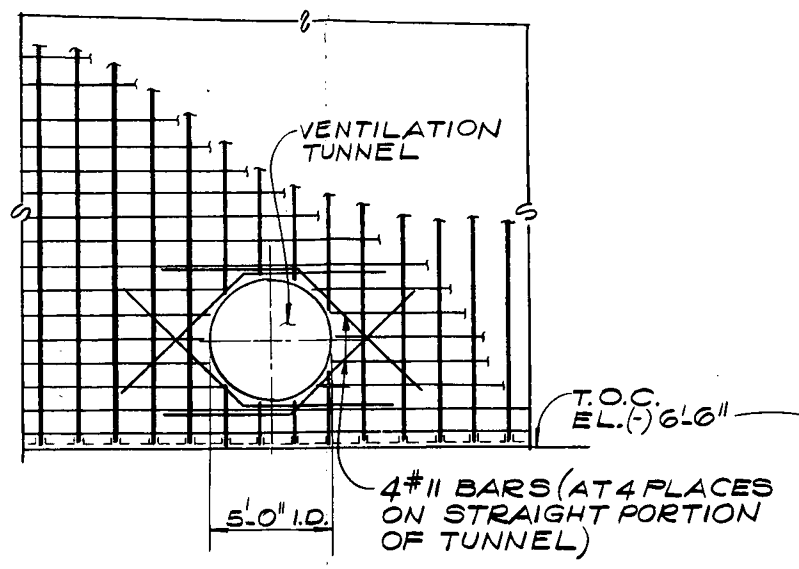


SECTION A/1

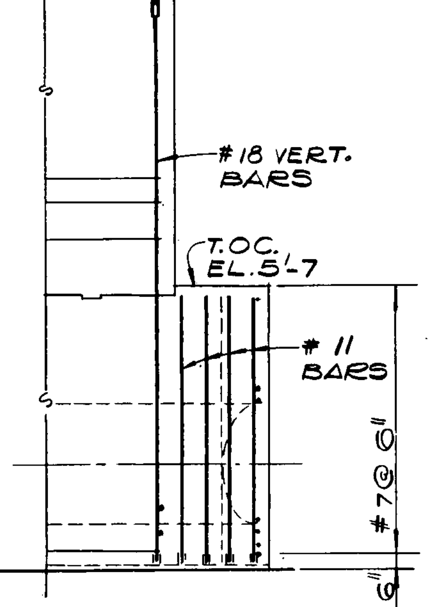


FOR SIZE AND LOCATION OF MAIN HORIZONTAL REINF. SEE SECT. A/1

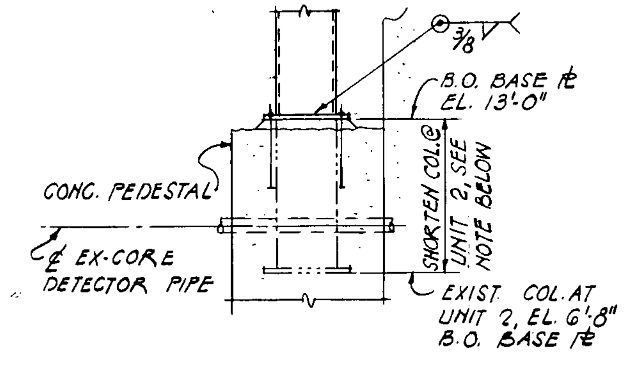
SECTION B/1



SECTION C/1



SECTION D/1



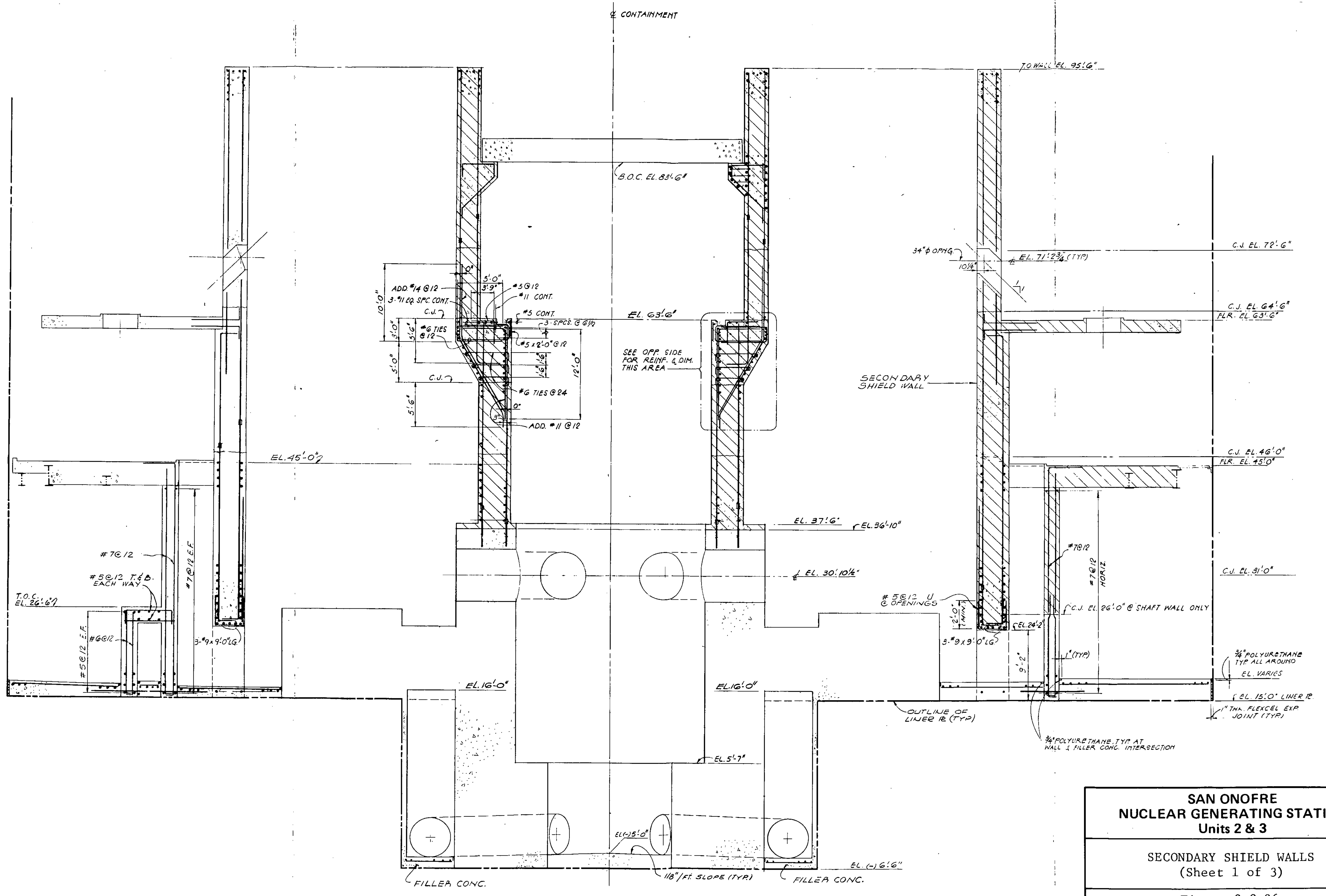
DETAIL 1/—

10079-11D-311
26AU6

SAN ONOFRE
NUCLEAR GENERATING STATION
Units 2 & 3

PRIMARY SHIELD WALLS AND REACTOR CAVITY REINFORCING STEEL
(Sheet 4 of 4)

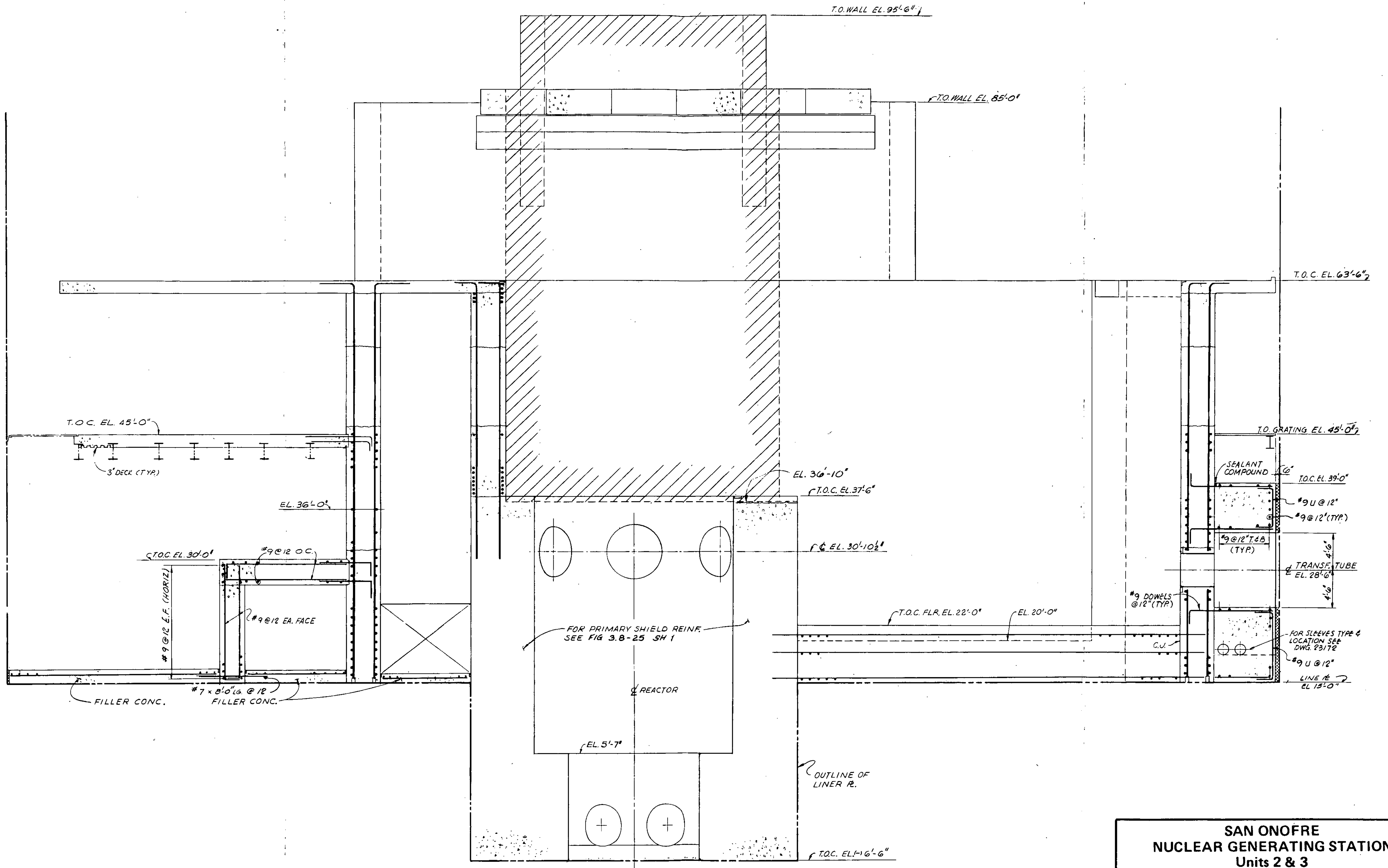
Figure 3.8-25



TYPICAL CROSS SECTION FACING WEST

10079-11D-312
01OC6

SAN ONOFRE NUCLEAR GENERATING STATION Units 2 & 3
SECONDARY SHIELD WALLS (Sheet 1 of 3)
Figure 3.8-26



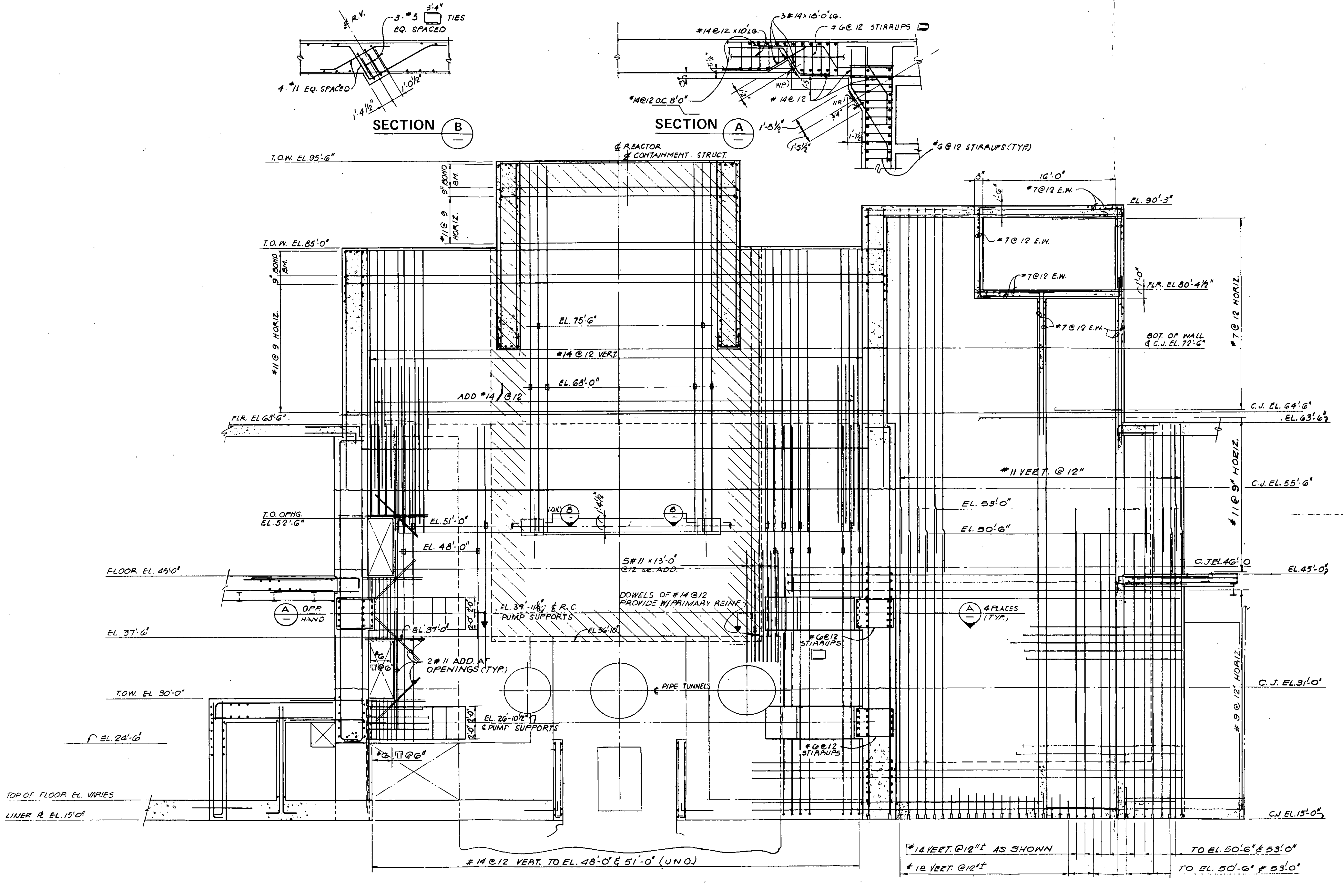
TYPICAL CROSS SECTION FACING NORTH

SAN ONOFRE
NUCLEAR GENERATING STATION
Units 2 & 3

SECONDARY SHIELD WALLS
(Sheet 2 of 3)

Figure 3.8-26

10079-11D-313
01OC6



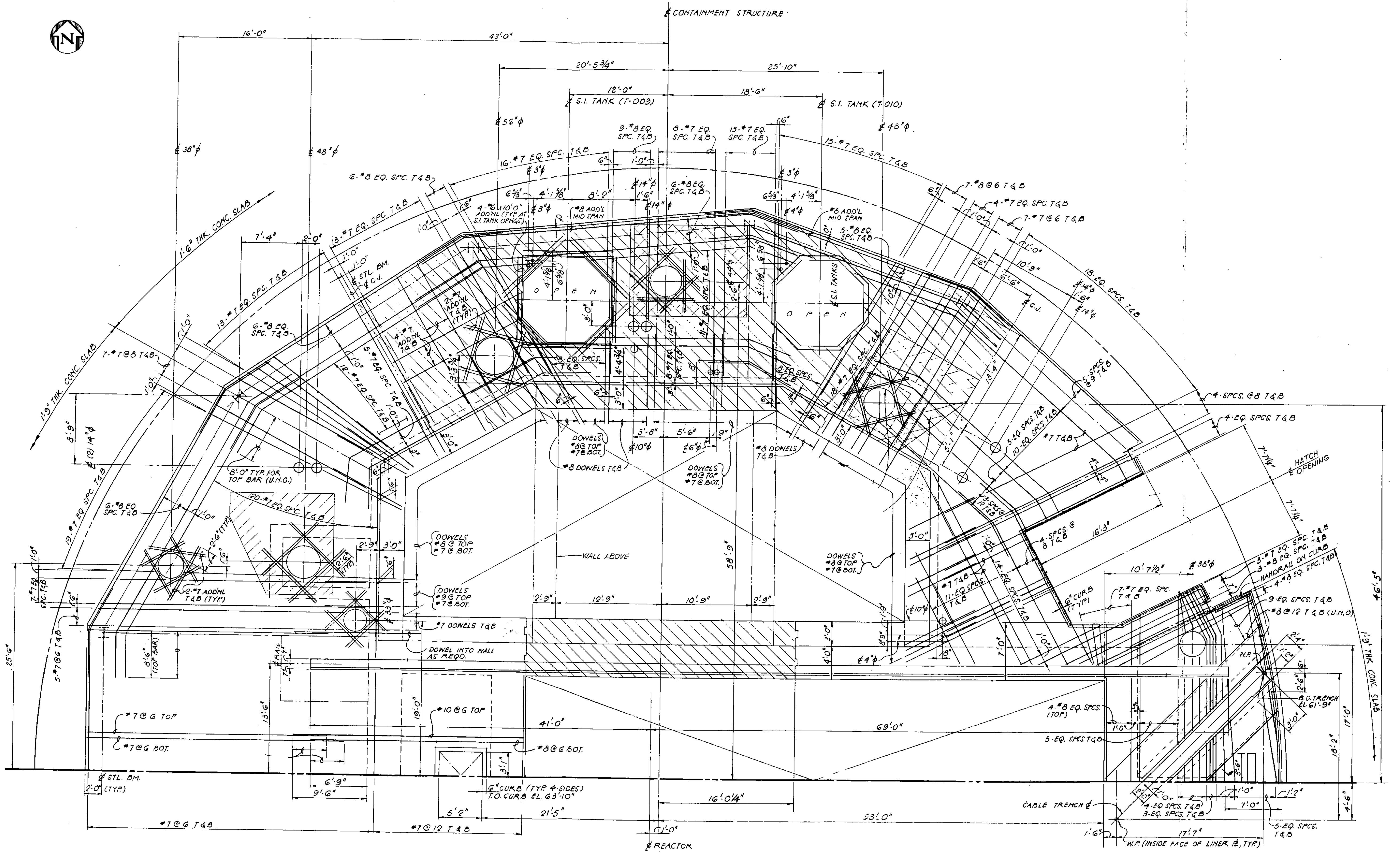
LOOKING FROM OUTSIDE

10079-11D-314
17MY6

SAN ONOFRE NUCLEAR GENERATING STATION Units 2 & 3

SECONDARY SHIELD WALLS
(Sheet 3 of 3)

Figure 3.8-26



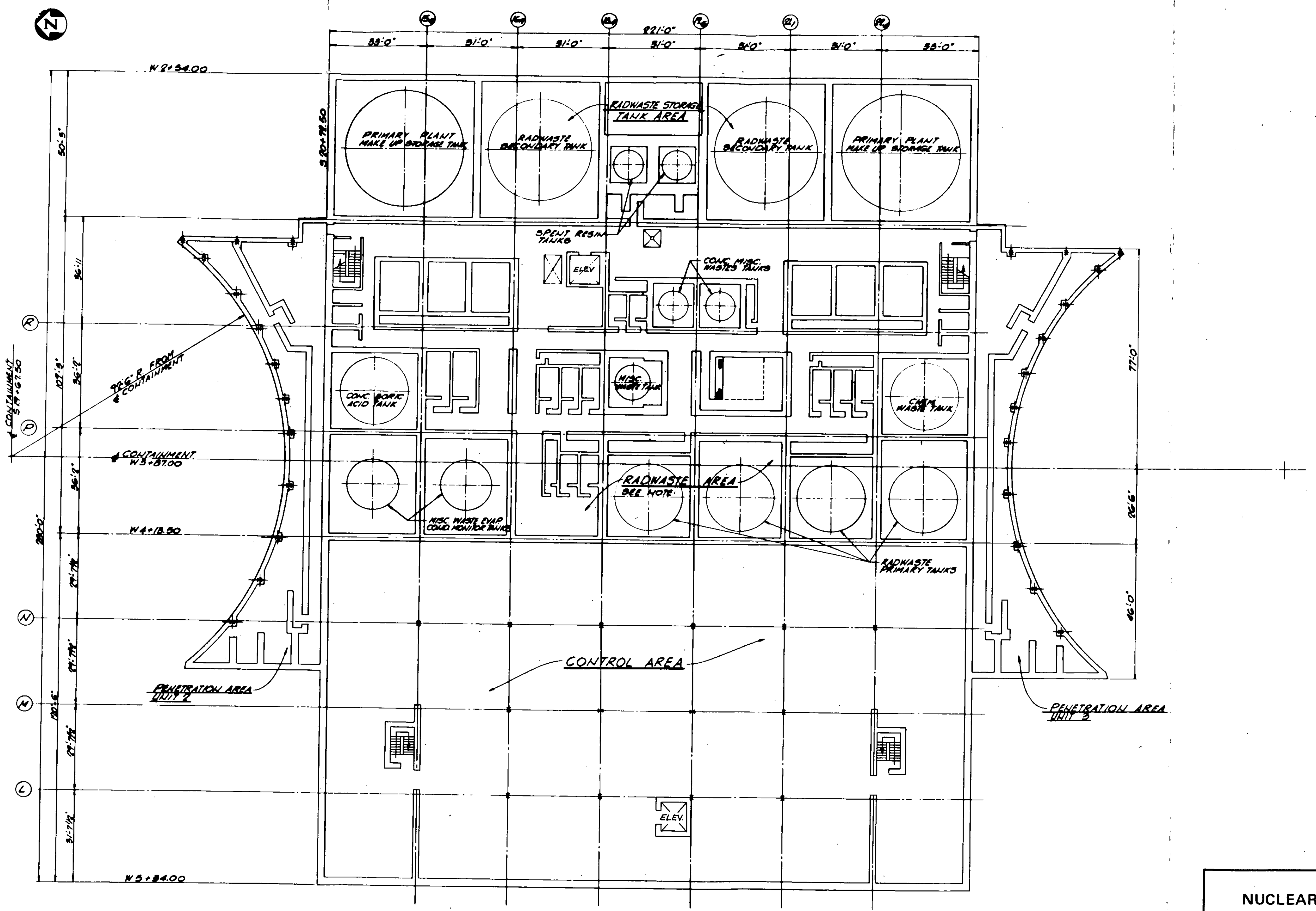
PLAN AT ELEVATION 63'-6"

10079-11D-315
26AU6

SAN ONOFRE
NUCLEAR GENERATING STATION
Units 2 & 3

INTERNAL STRUCTURE OPERATING FLOOR
TYPICAL REINFORCING STEEL

Figure 3.8-27



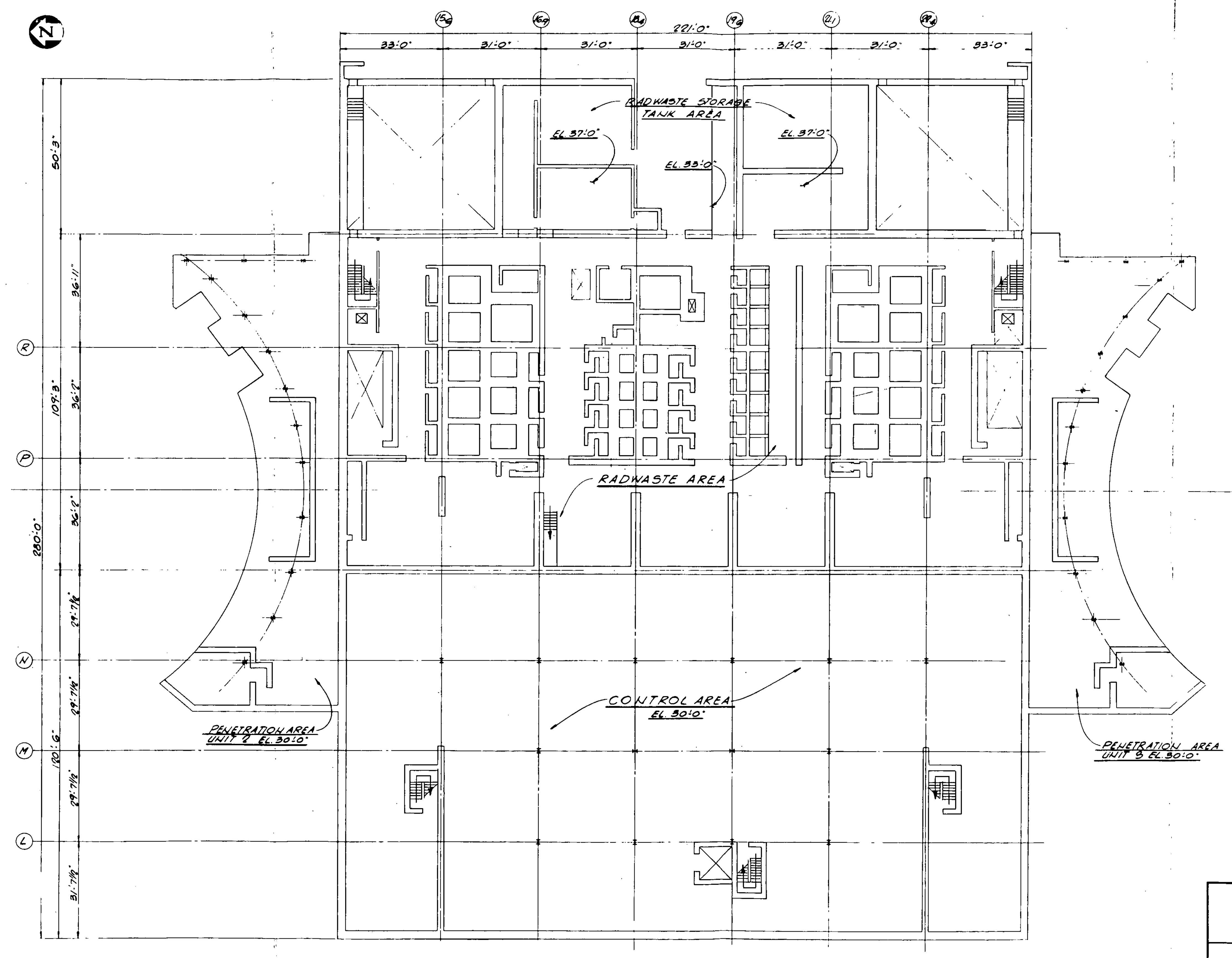
BASE MAT PLAN AT ELEVATION 9'-0"

10079-11D-323
26AU6

SAN ONOFRE
NUCLEAR GENERATING STATION
Units 2 & 3

AUXILIARY BUILDING PLANS
(Sheet 1 of 2)

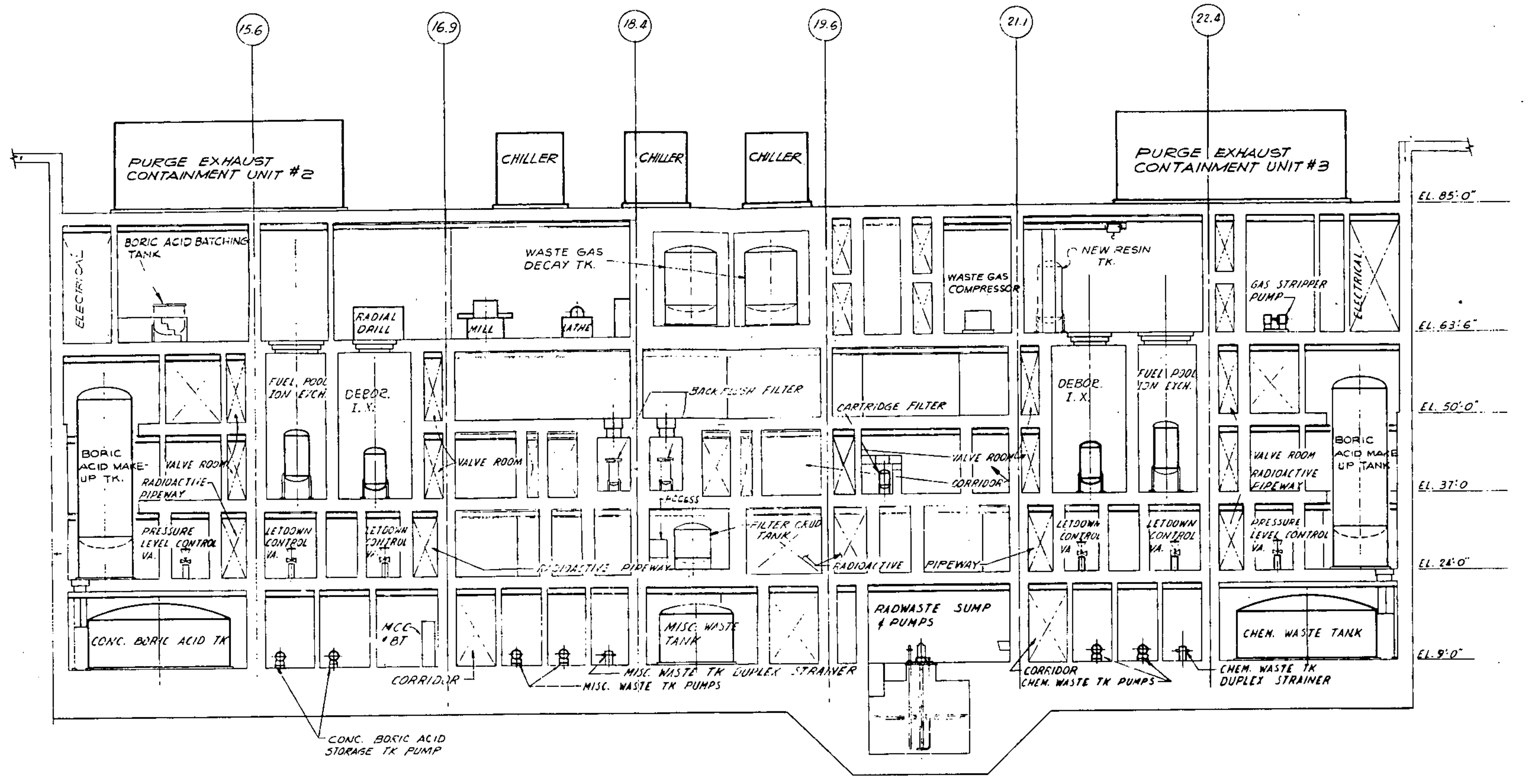
Figure 3.8-28



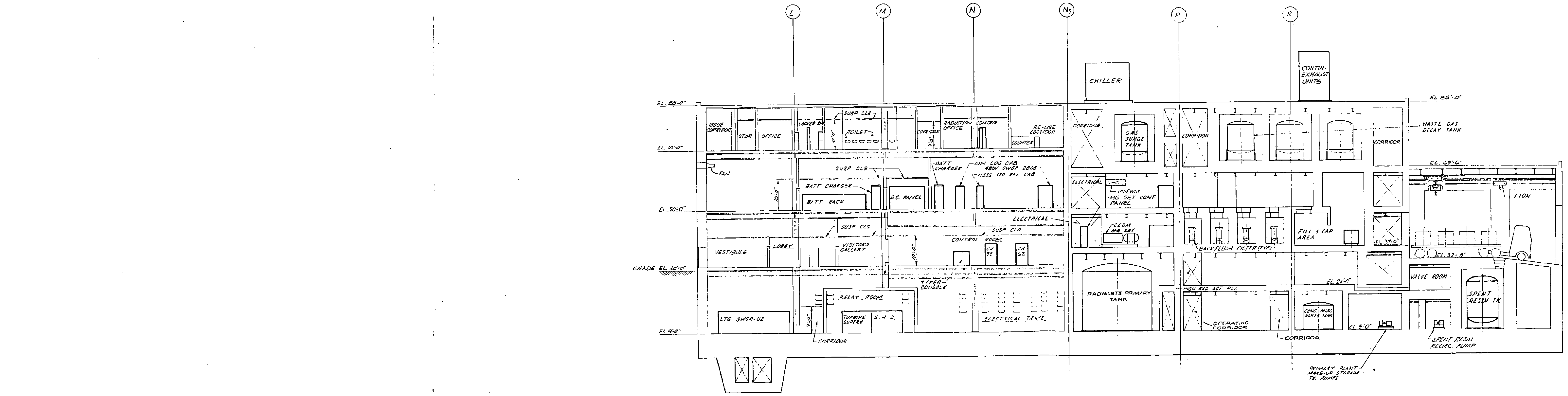
PLAN AT ELEVATION 37'-0" U.N.

SAN ONOFRE NUCLEAR GENERATING STATION Units 2 & 3
AUXILIARY BUILDING PLANS (Sheet 2 of 2)
Figure 3.8-28

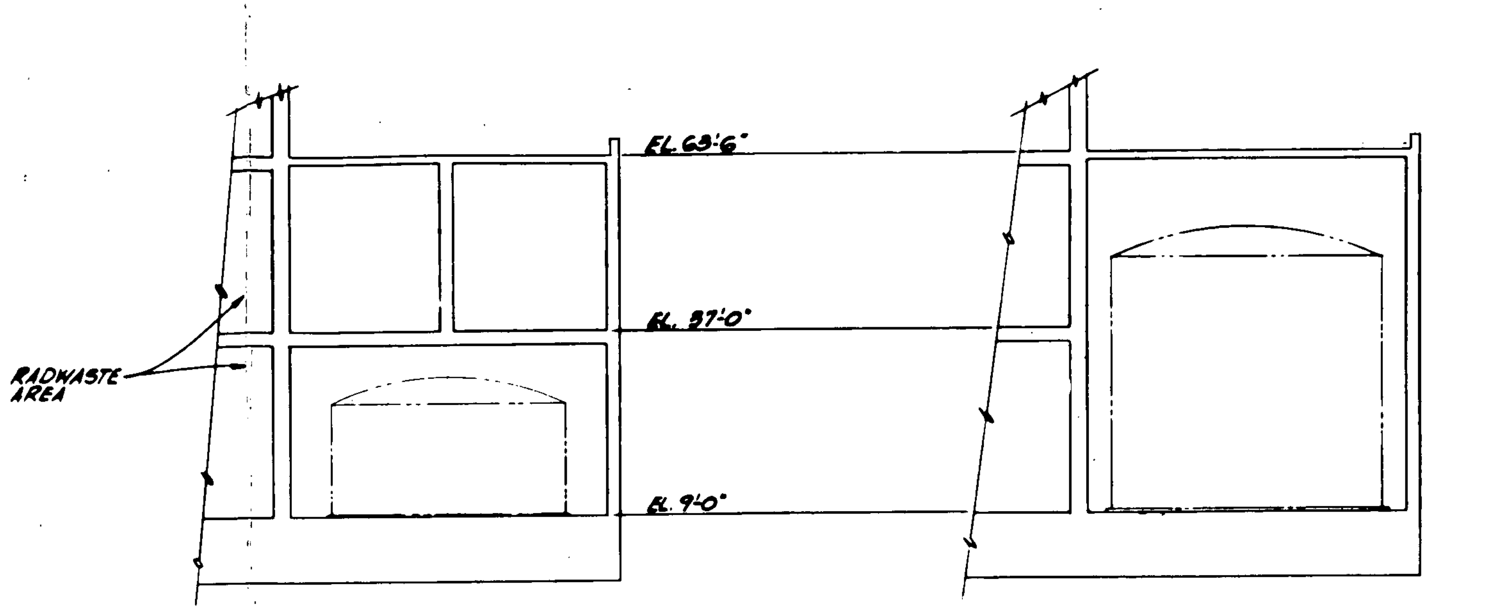
10079-11D-324
26AU6



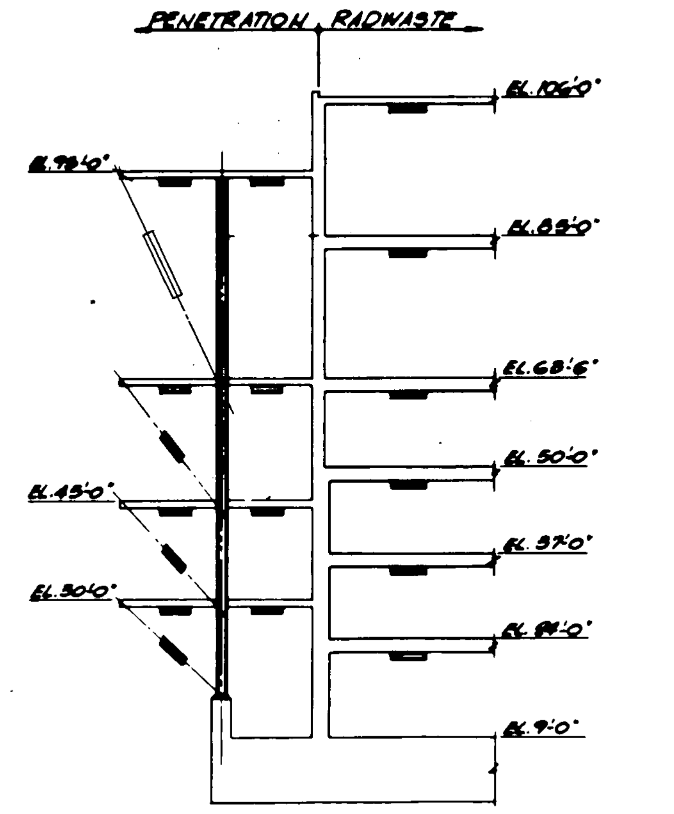
SECTION
LOOKING EAST



SECTION
LOOKING NORTH



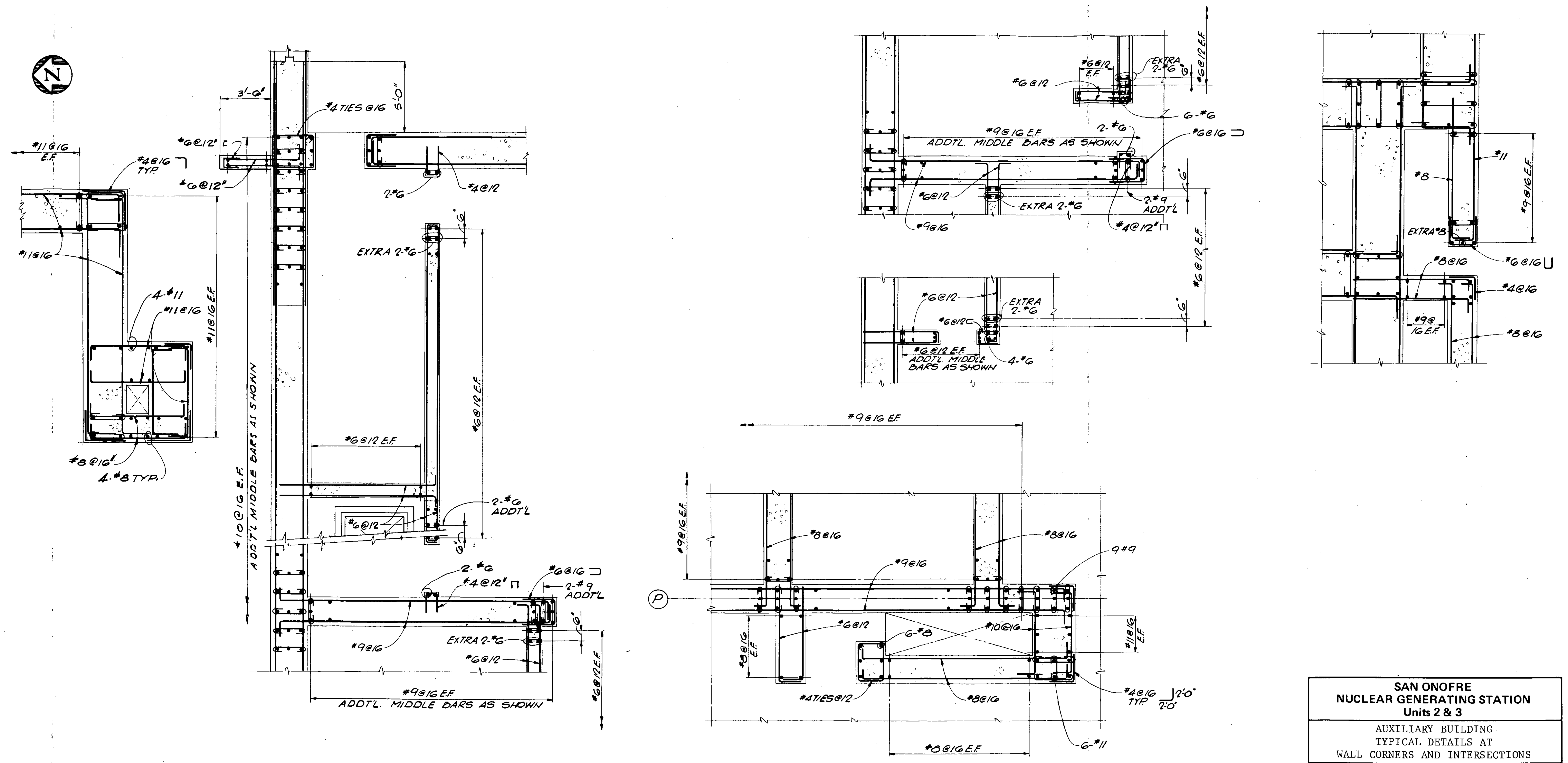
TANKAGE AREA SECTIONS
LOOKING NORTH



PENETRATION AREA
TYP. SECTION

SAN ONOFRE NUCLEAR GENERATING STATION Units 2 & 3
AUXILIARY BUILDING TYPICAL SECTION ELEVATIONS
Figure 3.8-29

10079-11D-325
26AU6

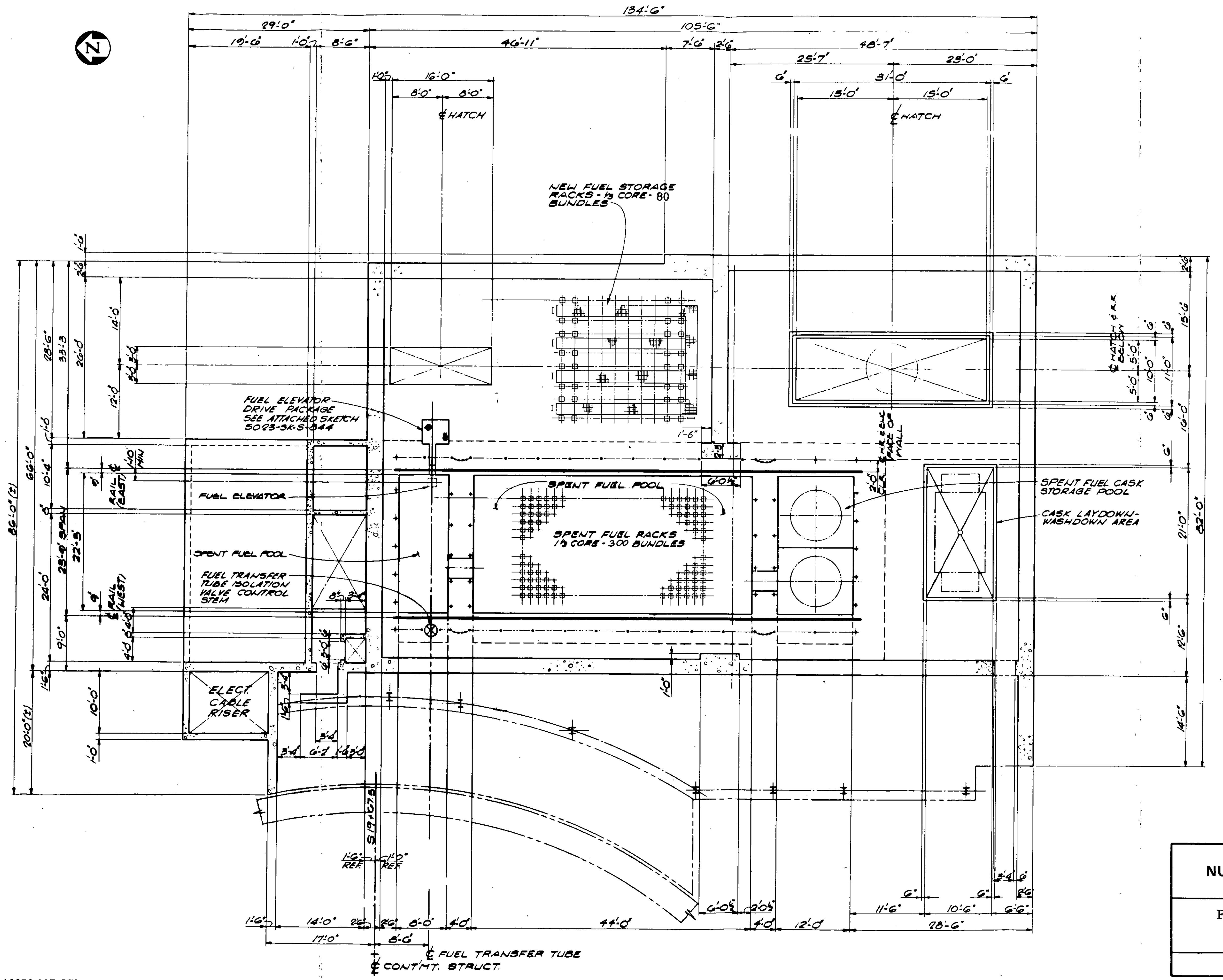


SAN ONOFRE
NUCLEAR GENERATING STATION
Units 2 & 3

AUXILIARY BUILDING
TYPICAL DETAILS AT
WALL CORNERS AND INTERSECTIONS

Figure 3.8-30

10079-11D-326
21MY6

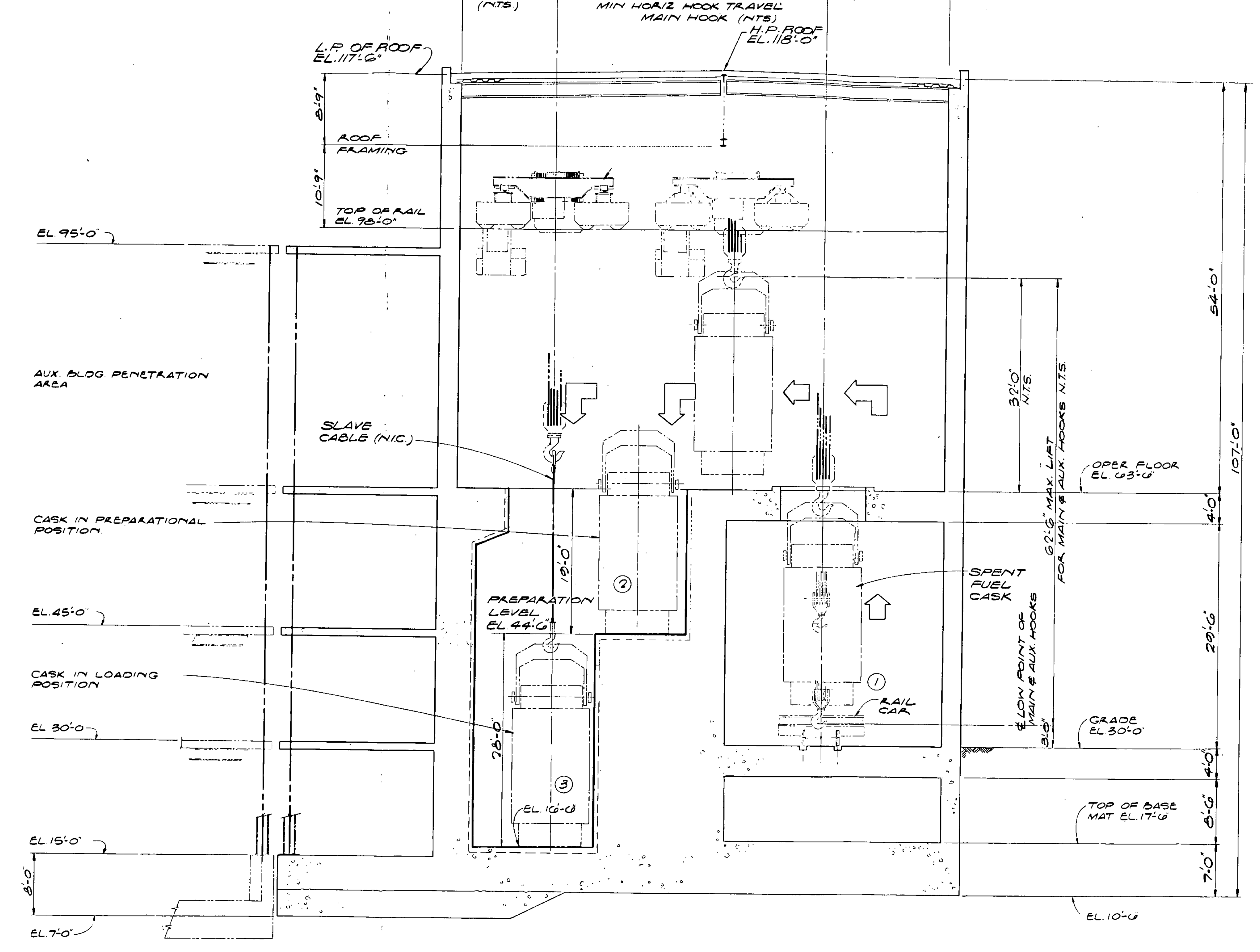


10079-11D-328
26AU6

SAN ONOFRE
NUCLEAR GENERATING STATION
Units 2 & 3

FUEL HANDLING BUILDING PLAN
AT EL 63 FT 6 IN.

Figure 3.8-31



SPENT FUEL CASK HANDLING SCHEME

SECTION LOOKING NORTH

A / 700

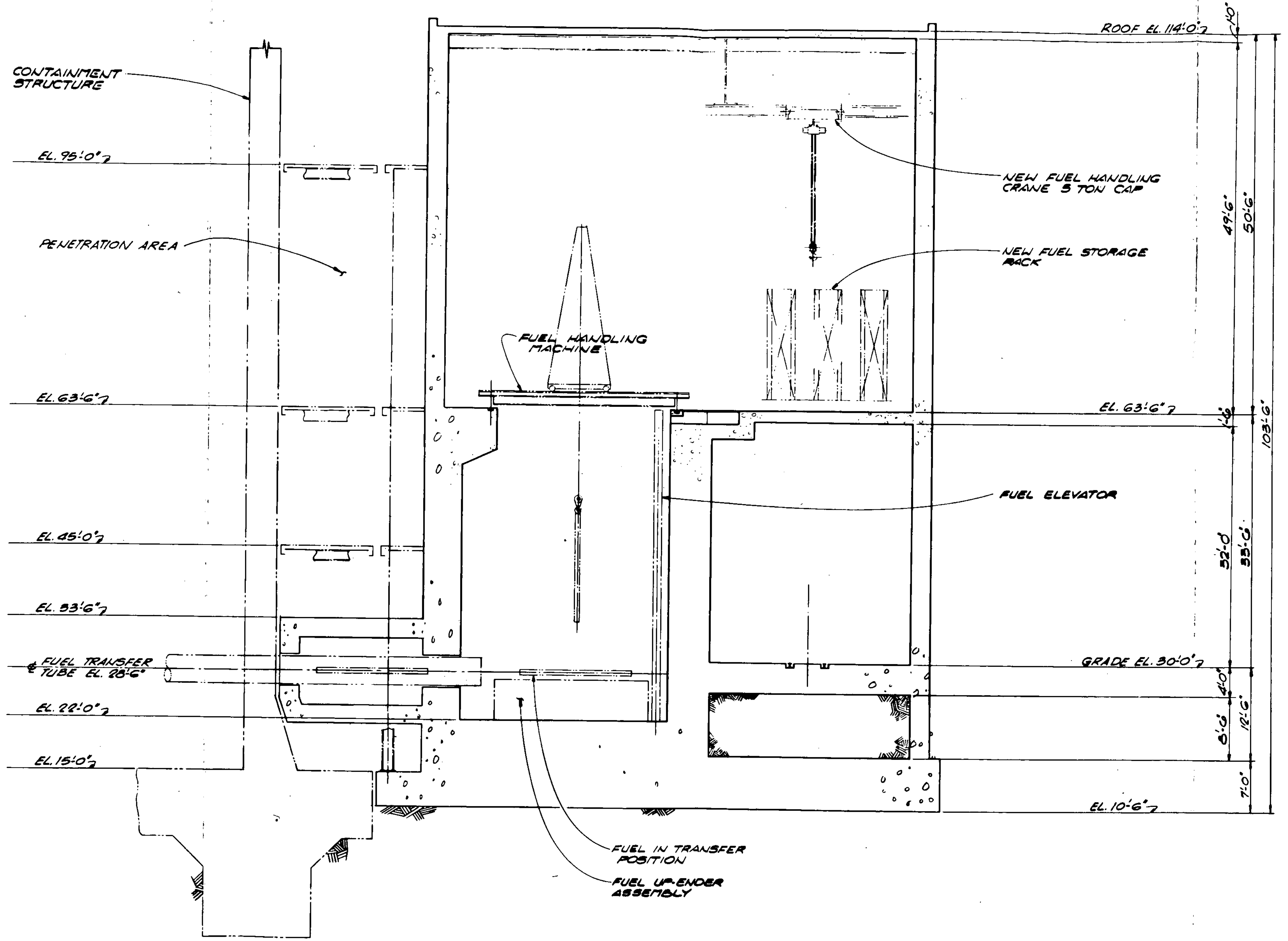
1. CASK-RAISED FROM SHIPPING POSITION TO OPER. FLOOR LEVEL, THEN CASK TRAVELS HORIZONTALLY TO STORAGE POOL AREA.
2. CASK-LOWERED INTO PREPARATION LEVEL OF POOL WHERE CASK HEAD WILL BE LOOSENED AND SLAVE CABLE ATTACHED.
3. CASK-LOWERED INTO LOADING LEVEL OF POOL WHERE HEAD WILL BE REMOVED AND SPENT FUEL LOADED.

10079-11D-327
25AU6

SAN ONOFRE
NUCLEAR GENERATING STATION
Units 2 & 3

FUEL HANDLING BUILDING PLAN
CASK HANDLING CRANE LIFTS

Figure 3.8-32

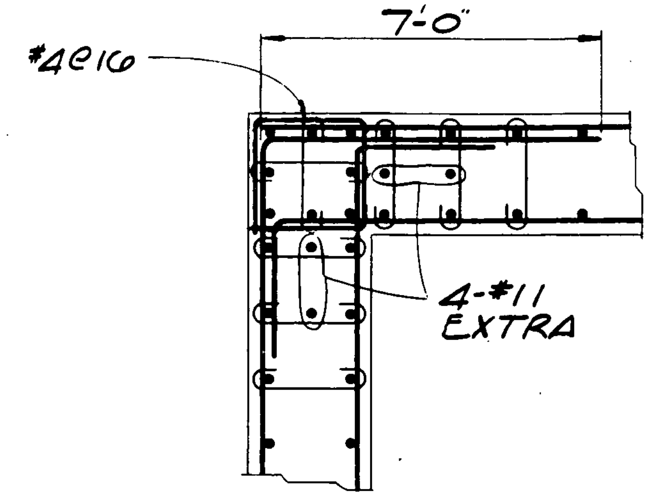


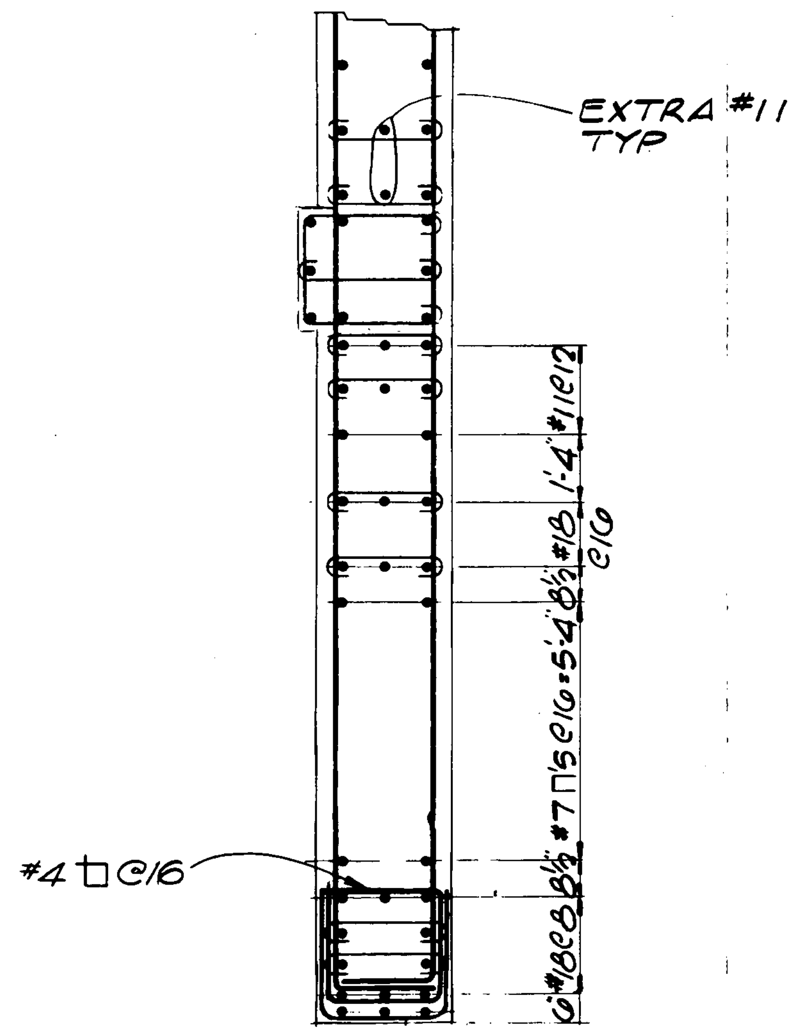
10079-11D-329
26AU6

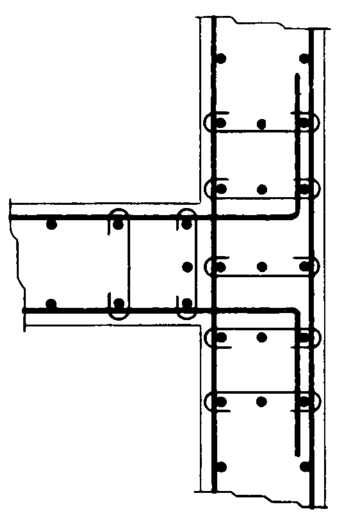
SAN ONOFRE NUCLEAR GENERATING STATION
Units 2 & 3

FUEL HANDLING BUILDING PLAN
TYPICAL SECTION ELEVATION

Figure 3.8-33





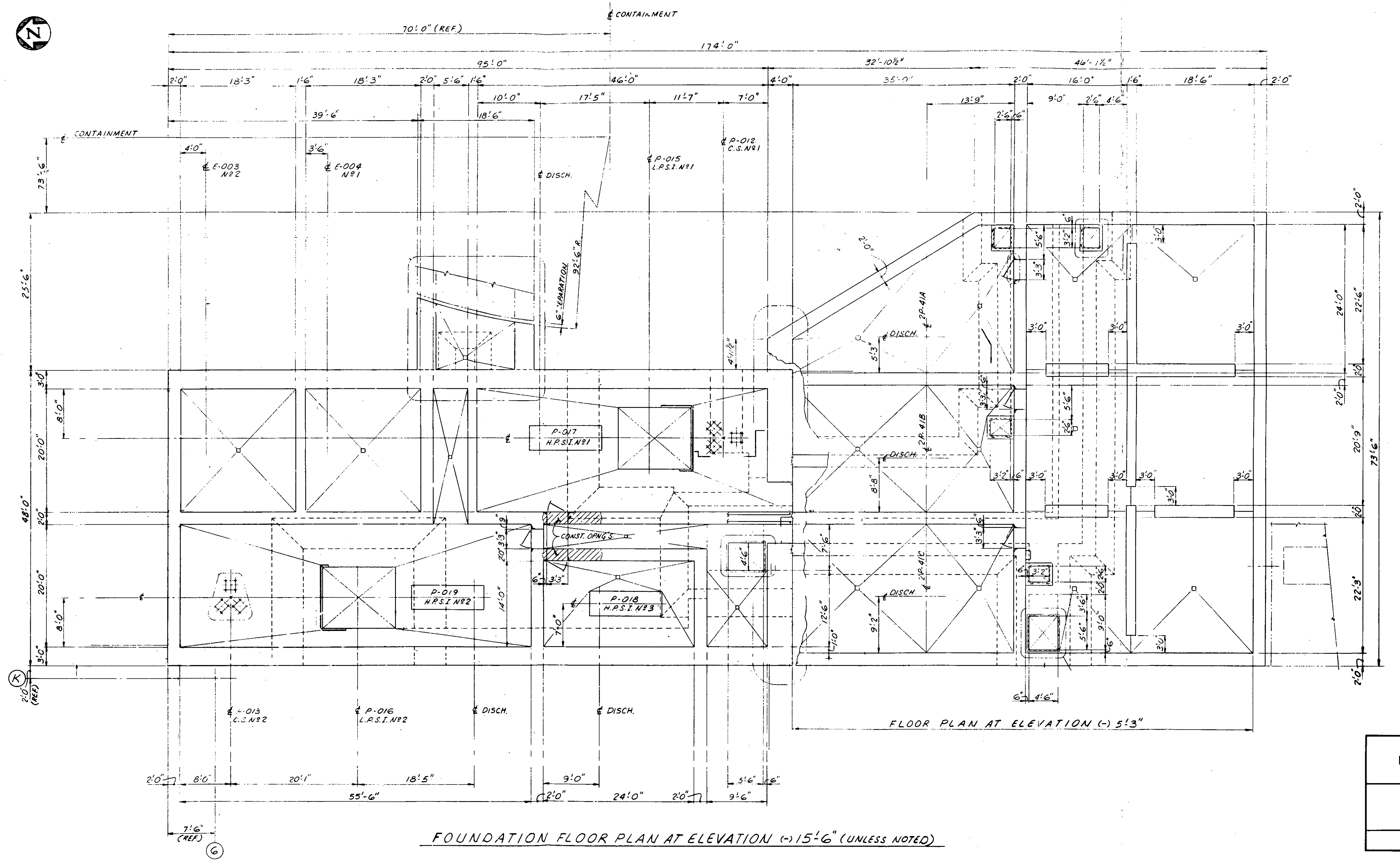


SAN ONOFRE
NUCLEAR GENERATING STATION
Units 2 & 3

FUEL HANDLING BUILDING
TYPICAL DETAILS AT
WALL CORNERS AND INTERSECTIONS

Figure 3.8-34

10079-11D-330
21MY6

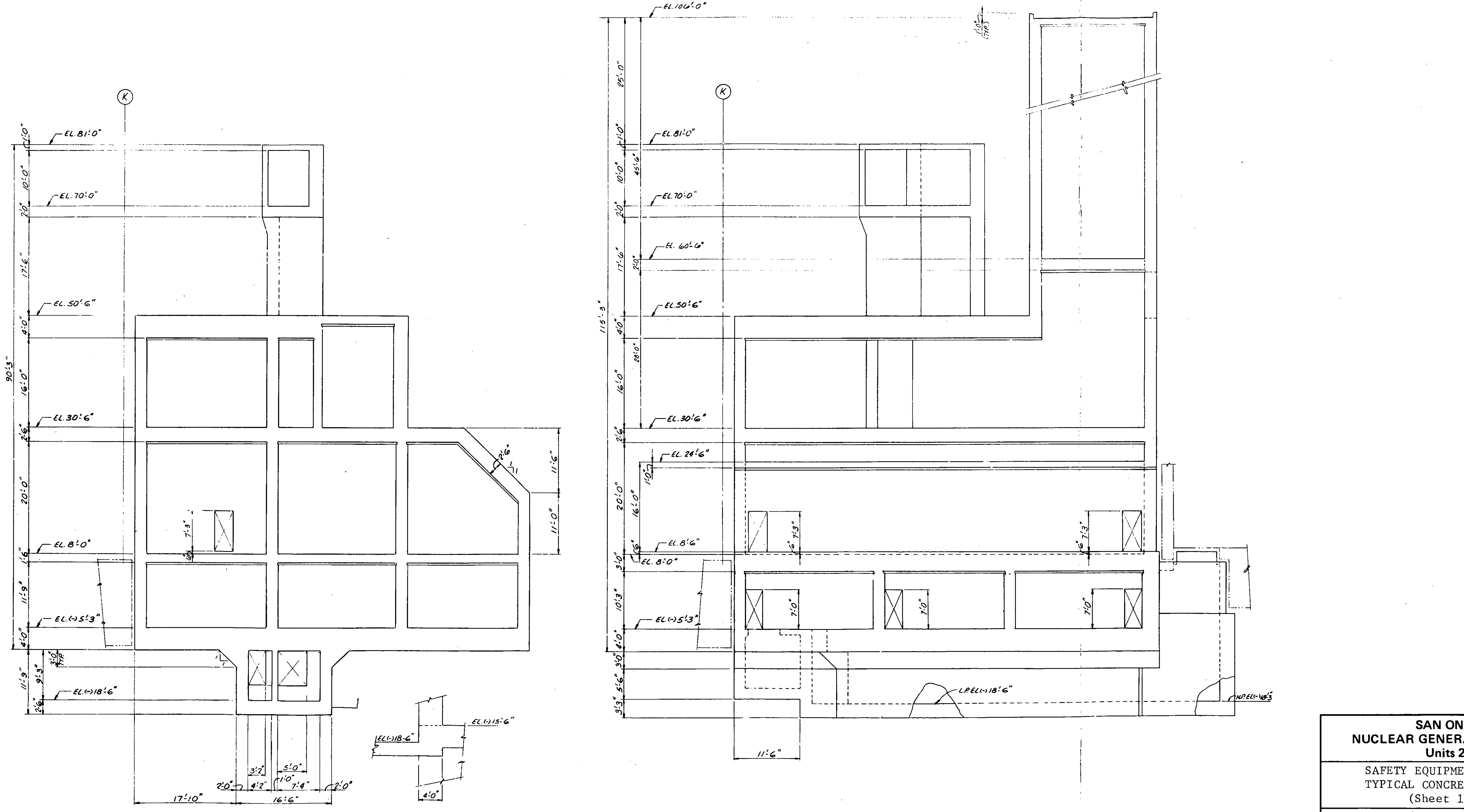


10079-11D-331
21MY6

SAN ONOFRE NUCLEAR GENERATING STATION
Units 2 & 3

SAFETY EQUIPMENT BUILDING
FOUNDATION PLAN

Figure 3.8-35

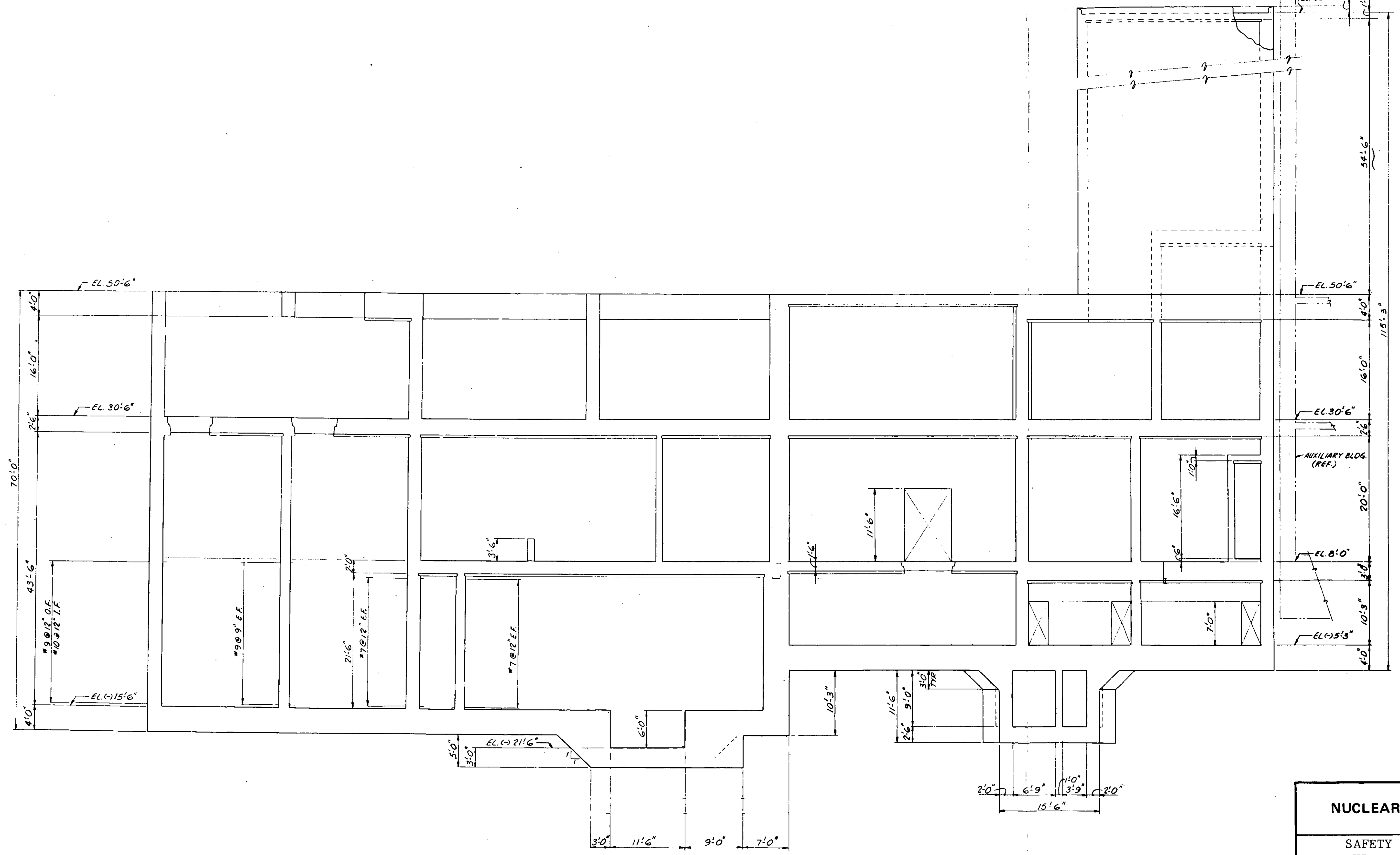


SAN ONOFRE
NUCLEAR GENERATING STATION
Units 2 & 3

SAFETY EQUIPMENT BUILDING
TYPICAL CONCRETE SECTIONS
(Sheet 1 of 2)

Figure 3.8-36

10079-11D-332
21MY6

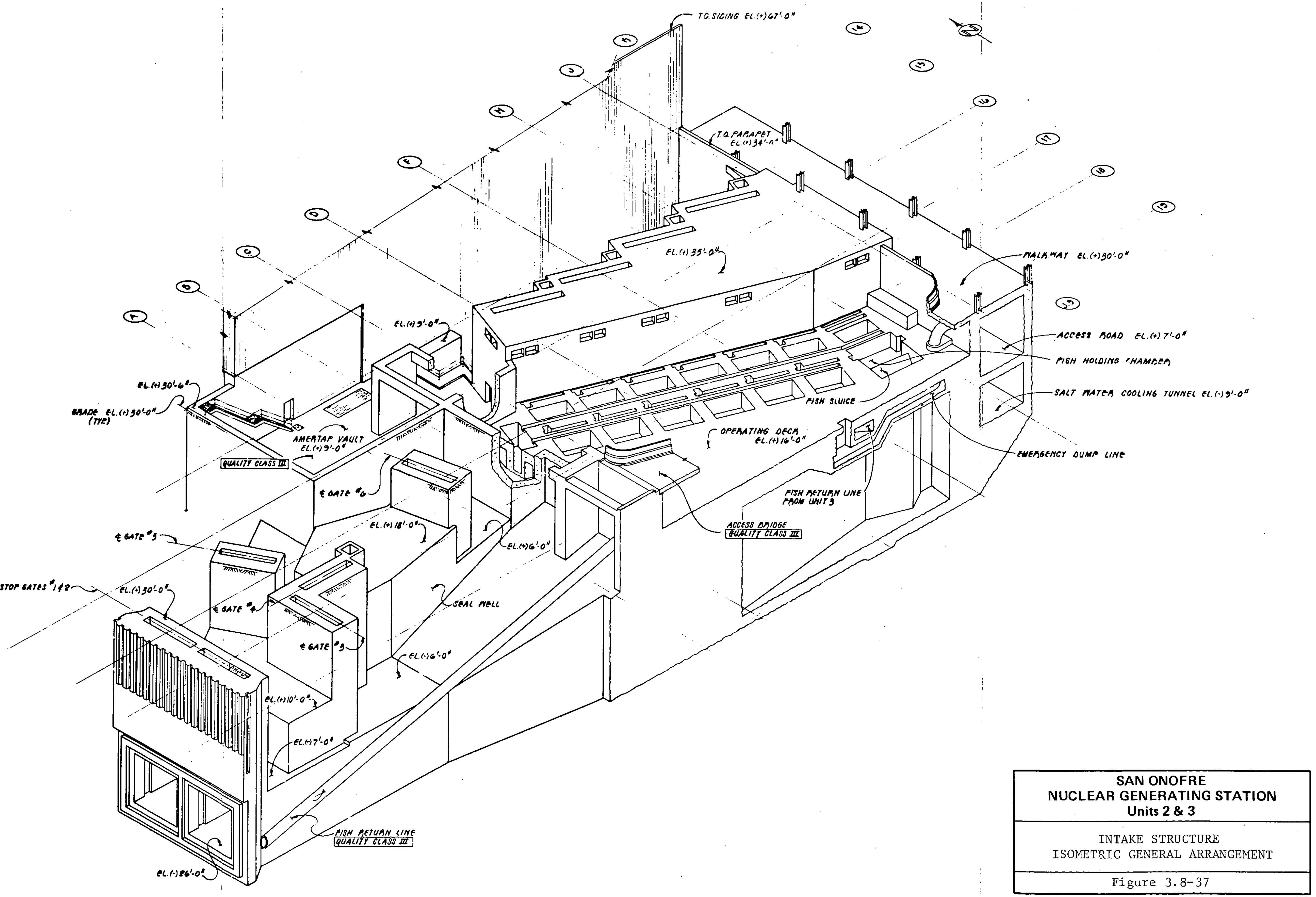


SAN ONOFRE
NUCLEAR GENERATING STATION
Units 2 & 3

SAFETY EQUIPMENT BUILDING
TYPICAL CONCRETE SECTIONS
(Sheet 2 of 2)

Figure 3.8-36

10079-11D-333
21MY6

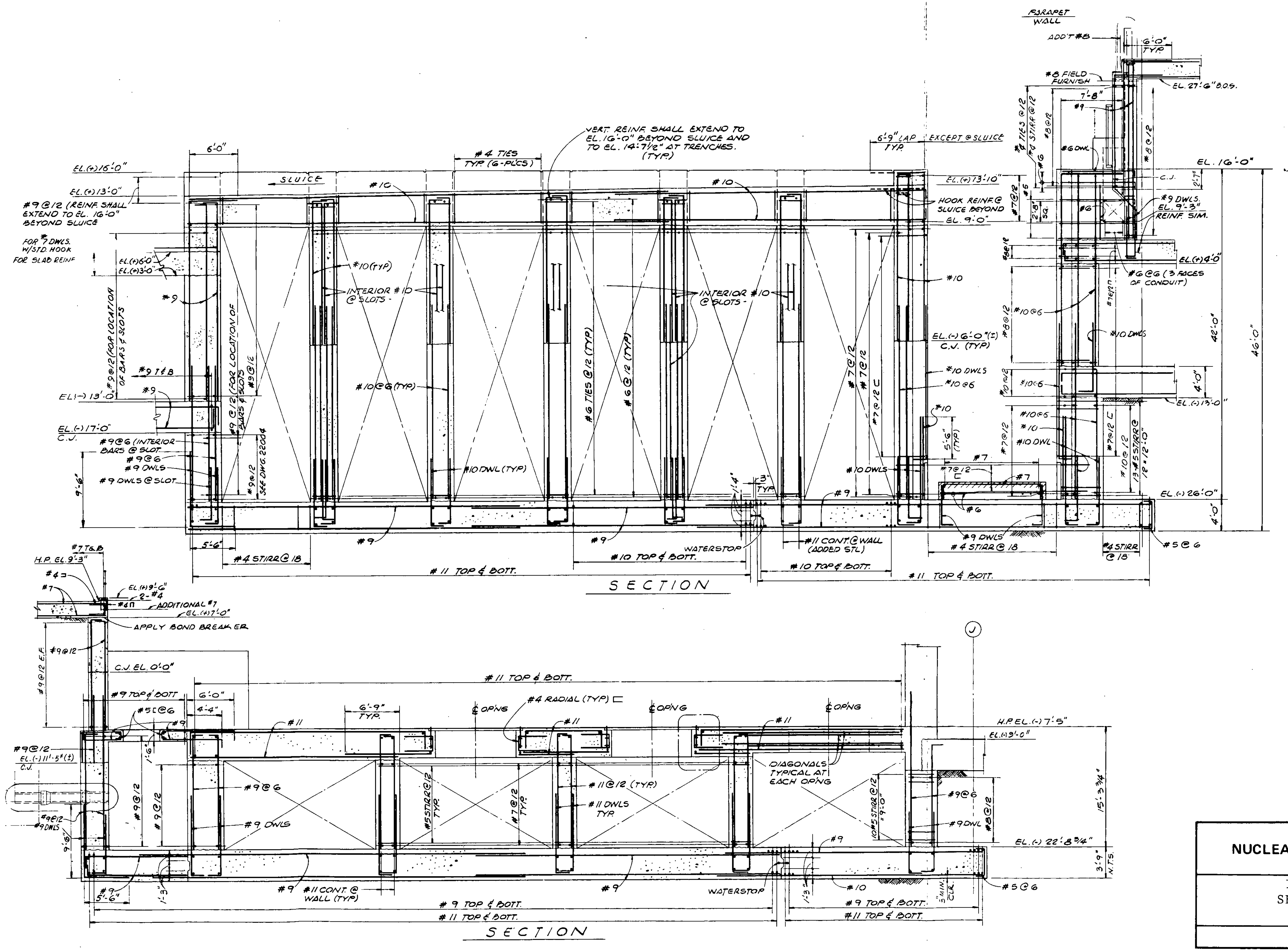


10079-11D-334
21MY6

SAN ONOFRE
NUCLEAR GENERATING STATION
Units 2 & 3

INTAKE STRUCTURE
ISOMETRIC GENERAL ARRANGEMENT

Figure 3.8-37

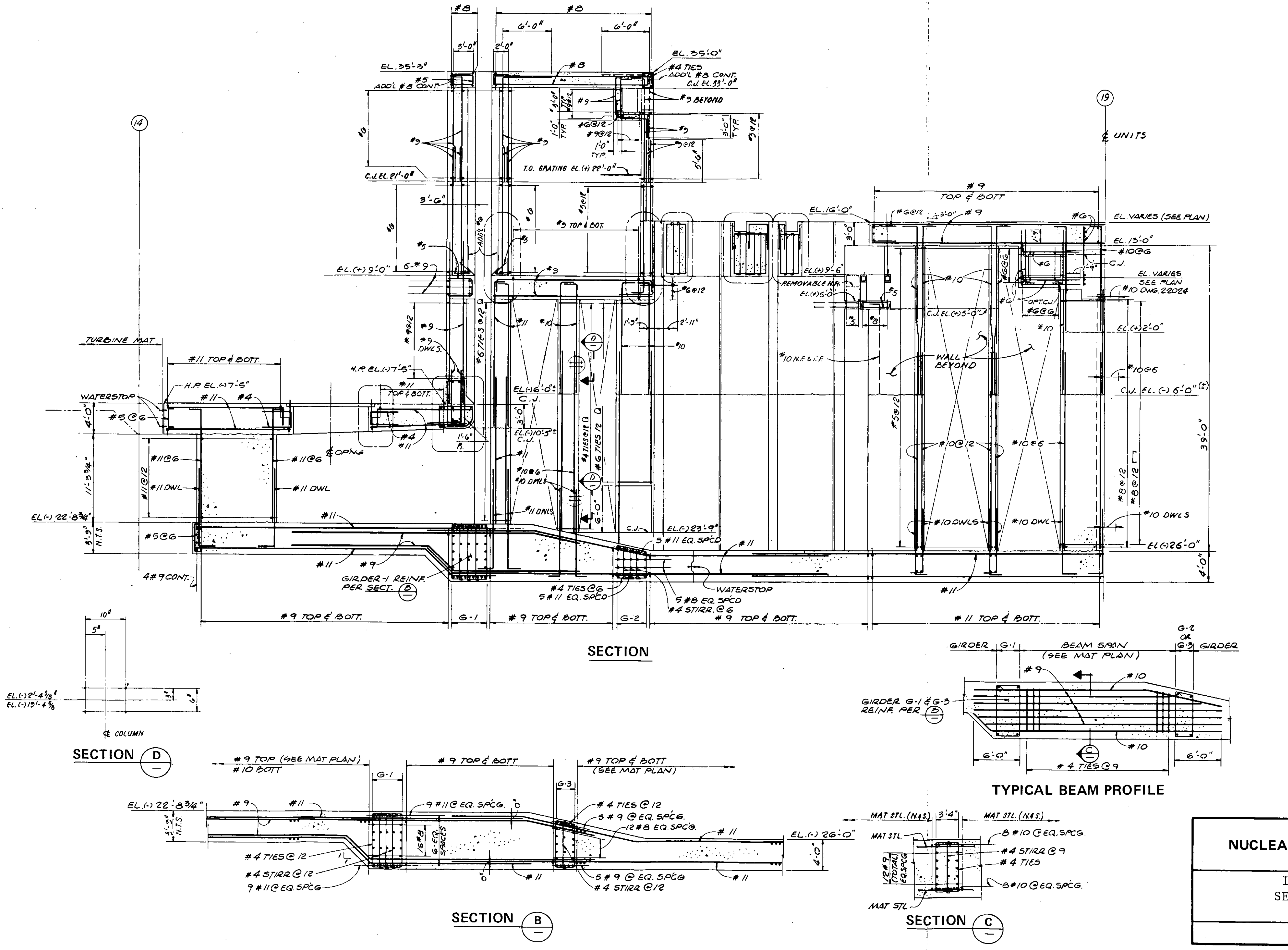


SAN ONOFRE NUCLEAR GENERATING STATION Units 2 & 3

INTAKE STRUCTURE
SECTION ELEVATIONS
(Sheet 1 of 2)

Figure 3.8-38

10079-11D-335
21MY6

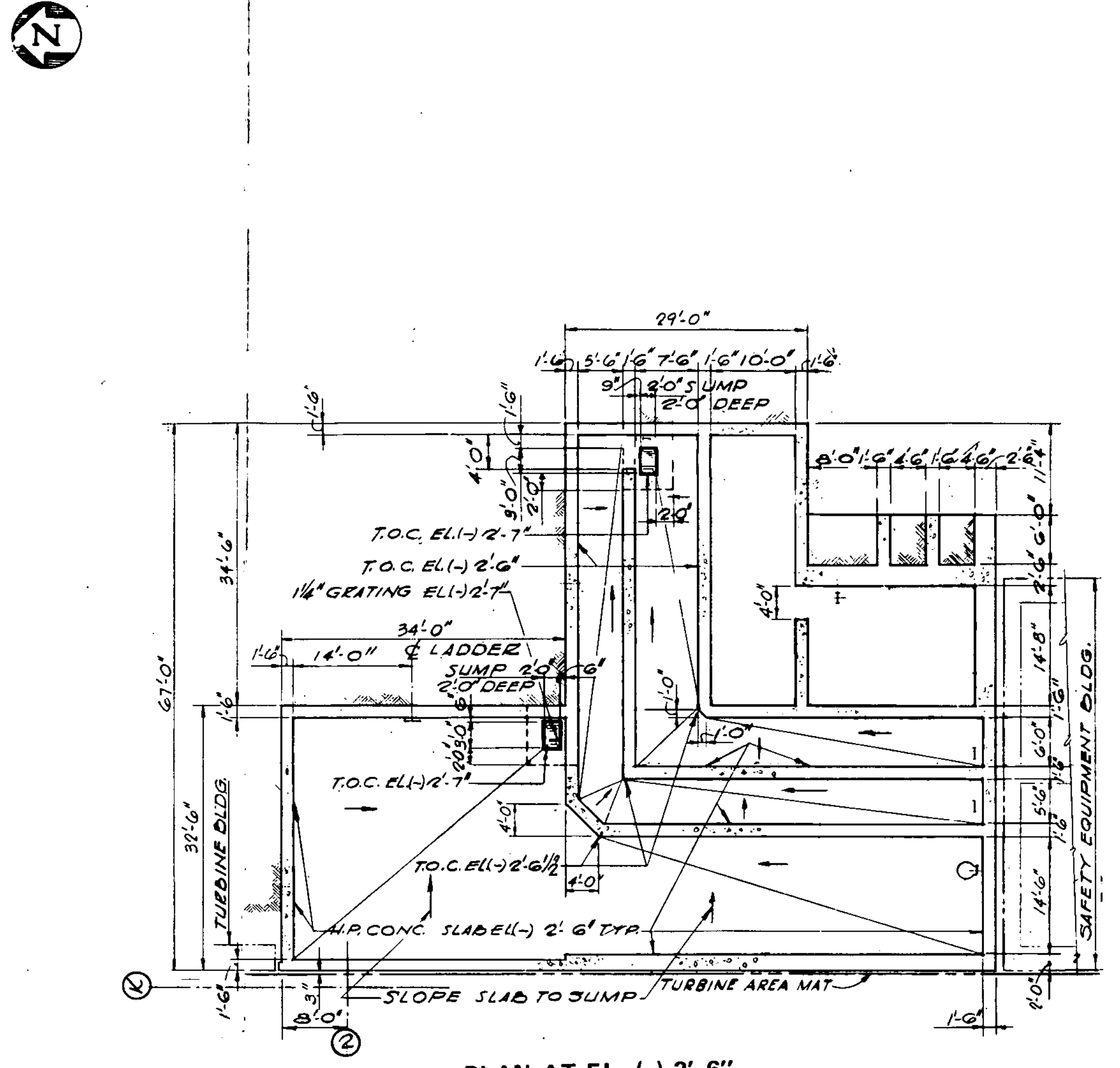


SAN ONOFRE
NUCLEAR GENERATING STATION
Units 2 & 3

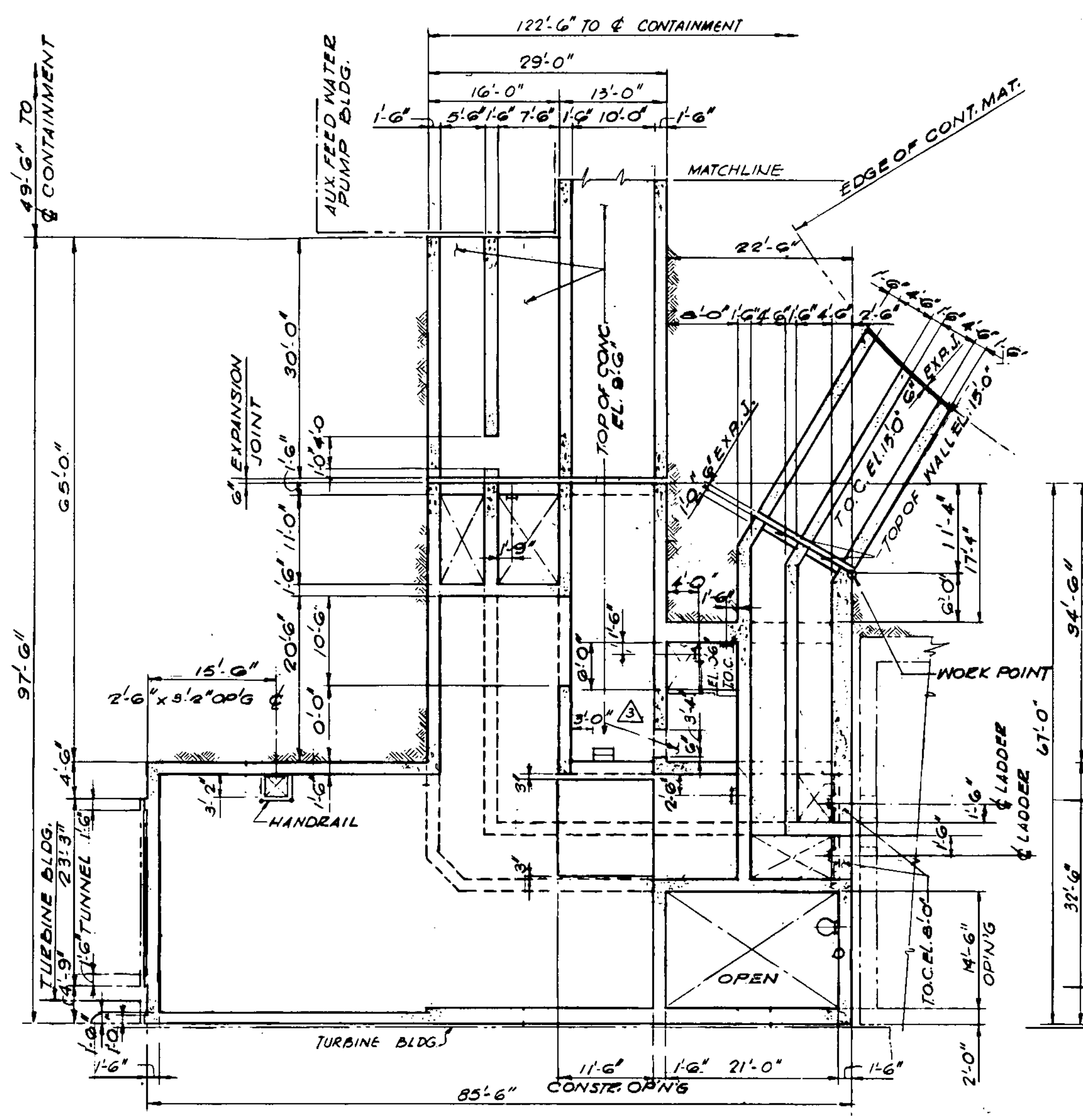
INTAKE STRUCTURE
SECTION ELEVATIONS
(Sheet 2 of 2)

Figure 3.8-38

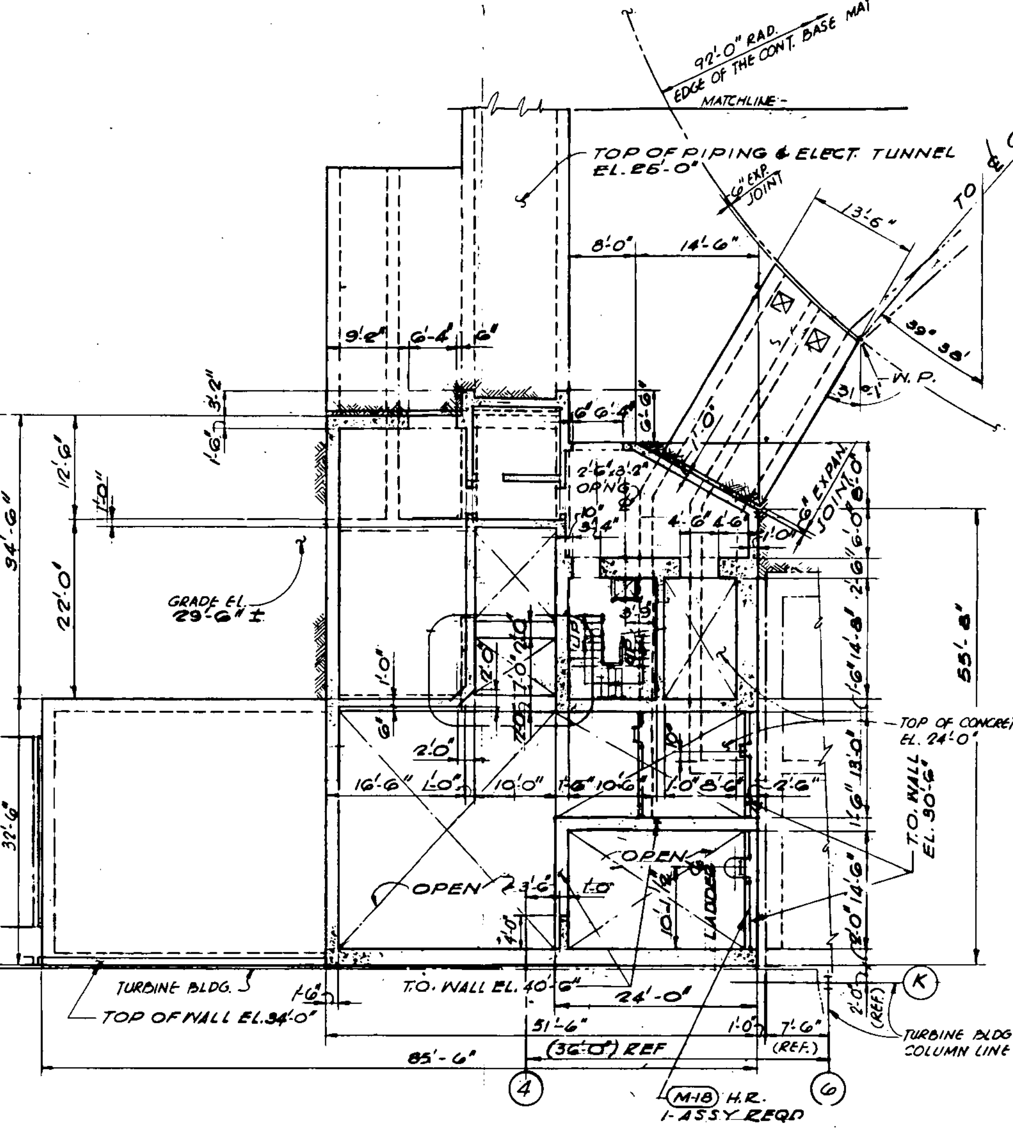
10079-11D-336
21MY6



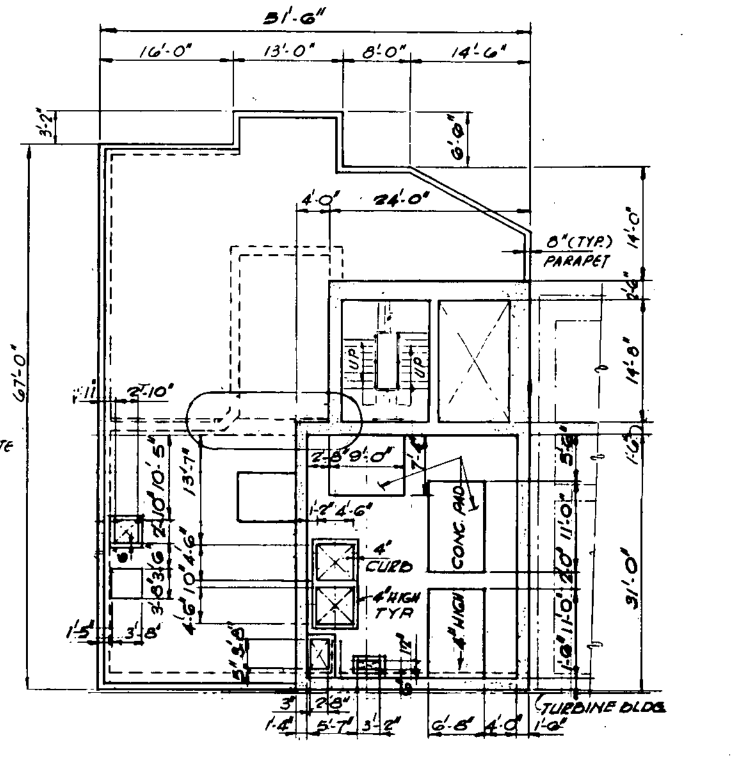
PLAN AT EL. (-) 2'-6"



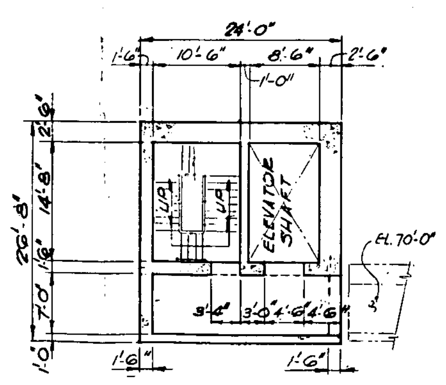
PLAN AT EL. 11'-6"



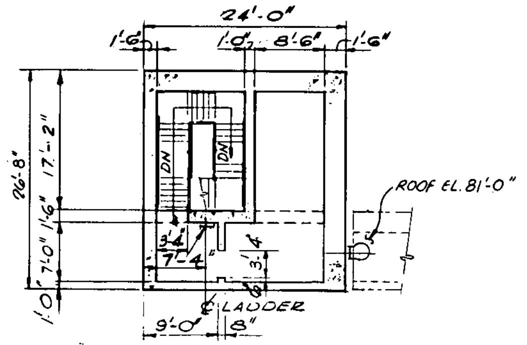
PLAN AT EL. 30'-6"



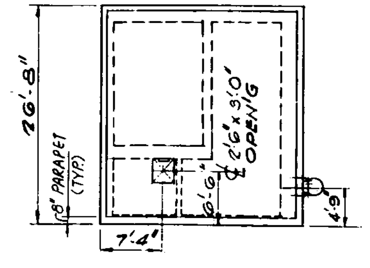
PLAN AT EL. 48'-6"



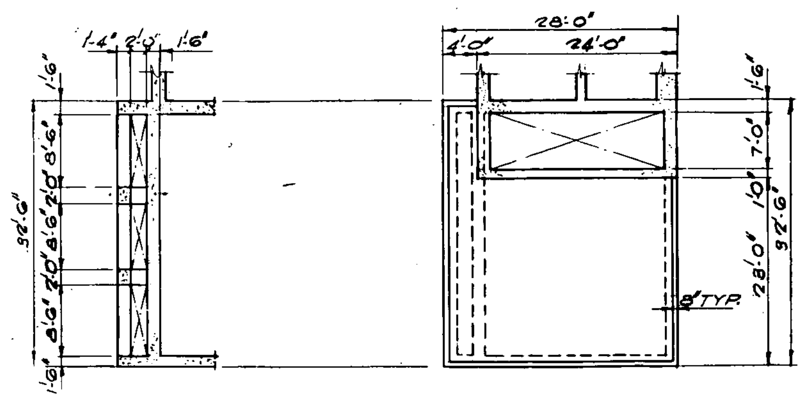
PLAN AT EL. 70'-0"



ELEVATOR MACHINE ROOM
PLAN AT EL. 86'-10½"



ROOF PLAN AT EL. 96'-0"

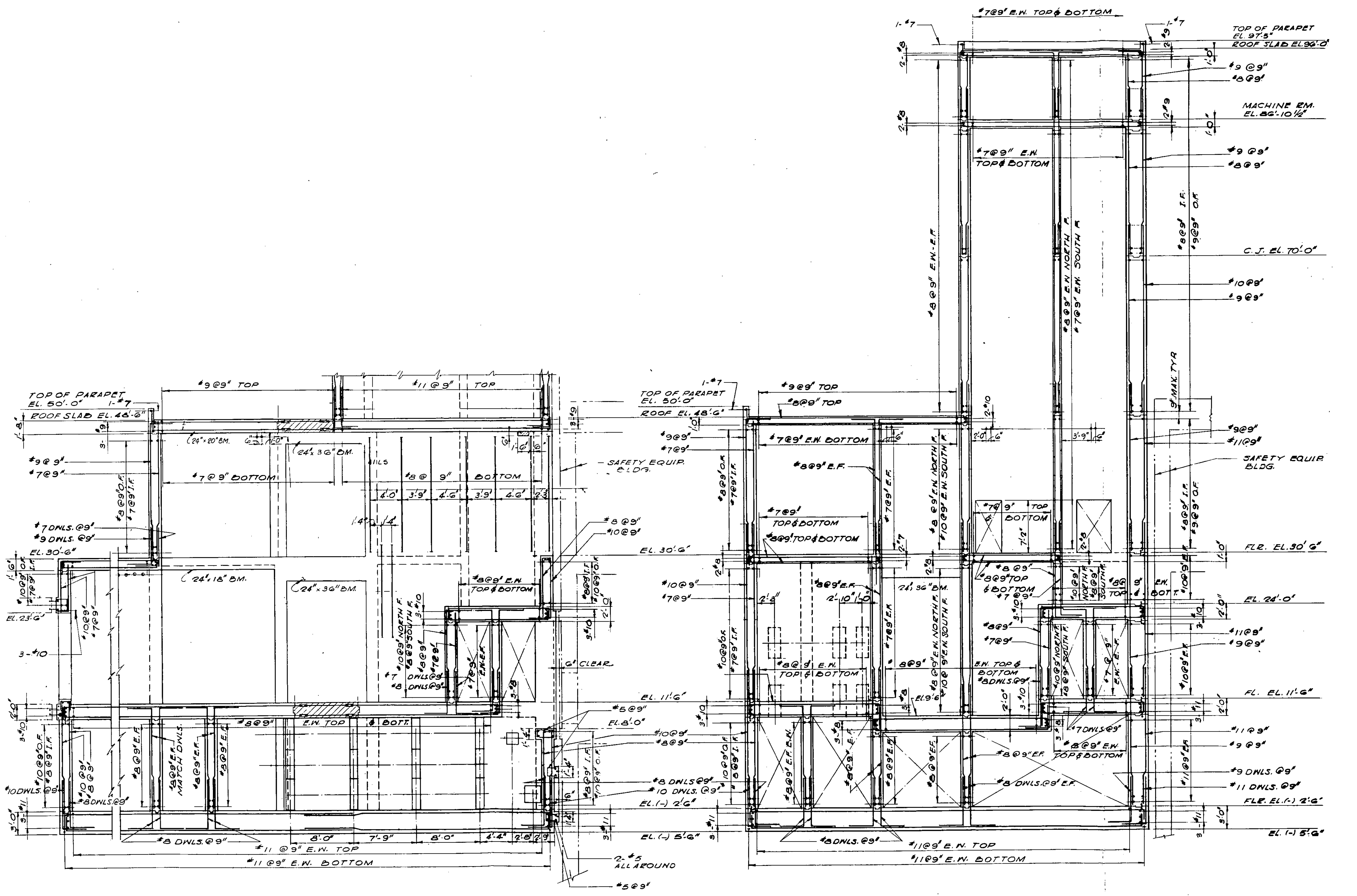


10079-11D-337
21MY6

SAN ONOFRE
NUCLEAR GENERATING STATION
Units 2 & 3

ELECTRICAL AND PIPING
GALLERY STRUCTURE PLANS

Figure 3.8-39



SAN ONOFRE
NUCLEAR GENERATING STATION
Units 2 & 3

ELECTRICAL AND PIPING
GALLERY STRUCTURE
TYPICAL SECTION ELEVATIONS

Figure 3.8-40

10079-11D-338
21MY6