Georgia Power Company
333 Piedmont Avenue
Atlanta, Georgia 30308
Telephone 404 526-6526

Mailing Address:
Post Office Box 4545
Atlanta, Georgia 30302

Georgia Power
the southern electric system

L. T. Gucwa
Manager Nuclear Engineering
and Chief Nuclear Engineer

NED-85-444
1766N

May 30, 1985

Director of Nuclear Reactor Regulation
Attention: Mr. John F. Stolz, Chief
Operating Reactors Branch No. 4
Division of Licensing
U. S. Nuclear Regulatory Commission
Washington, D. C. 20555

NRC DOCKETS 50-321, 50-366
OPERATING LICENSES DPR-57, NPF-5
EDWIN I. HATCH NUCLEAR PLANT UNITS 1, 2
SUPPLEMENTAL INFORMATION ON SEISMIC DESIGN ISSUES

Gentlemen:

Pursuant to our commitment in the response to NRC question "Q4" attached to letter NED-85-391, dated May 13, 1985, a draft of FSAR revisions concerning seismic design and analysis is enclosed for your information. These revisions are expected to be sent to you in final form, along with other revisions being made in the next FSAR update, by July 22, 1985.

If you have any questions concerning the enclosed information please contact this office.

Yours truly,

RJBaker

for L. T. Gucwa

WEB/mb

Enclosure

xc: J. T. Beckham, Jr.
H. C. Nix, Jr.
J. N. Grace (NRC- Region II)
Senior Resident Inspector

A001
1/1

3.2 CLASSIFICATION OF STRUCTURES, COMPONENTS, AND SYSTEMS

3.2.1 SEISMIC CLASSIFICATION

A two-level system is used for the seismic classification of the structures, components, and systems of the facility:

- Seismic Category I structures, components, and systems
- Category II structures, components, and systems

3.2.1.1 Definitions

Seismic Category I structures, components, and systems are those that must function for safe shutdown, immediate or long-term core cooling, or for activity confinement following a loss-of-coolant accident to ensure that the public is protected in accordance with 10 CFR 100 guidelines.

Seismic Category I structures, components, and systems are designed to withstand the effects of the design basis earthquake and operating basis earthquake as discussed in section 3.7.

When a system as a whole is referred to as Seismic Category I, portions not associated with loss of function of the system may be designated as Category II.

Category II structures, components, and systems are those whose failure would not result in the release of significant radioactivity and would not prevent reactor shutdown. All equipment not specifically listed as Seismic Category I is included as Category II. The failure of Category II structures, components, and systems may interrupt power generation.

All Category II structures are designed to conform to paragraph 2.3.1.4 of the 1970 edition of the Uniform Building Code.

None of the structures in the Hatch Nuclear Plant (HNP) have classifications that are partially Seismic Category I and partially Category II; however, portions of nonseismic Category II systems are seismically supported if their failure could cause damage to Seismic Category I components.

Seismic classification of structures, systems, and components is in accordance with Regulatory Guide 1.29, (August 1973).

DRAFT

3.2.1.2 Seismic Category I Structures

Reactor building

Primary containment structure

Spent-fuel pool

New-fuel storage vault

Diesel generator building

Control building

Intake structure

Main stack

Structures supporting or housing Seismic Category I equipment

Wall around condensate storage tank (CST)

Liquid nitrogen storage tank and foundation

Diesel generator fuel oil storage tanks

3.2.1.3 Seismic Category I Mechanical Components and Systems

Seismic Category I mechanical components and systems are listed in table 3.2-1.

3.2.1.4 Seismic Category I (Class 1E) Electrical Equipment

Switchgear and buses

4160-V buses 2E, 2F, and 2G

600-V load centers 2C and 2D

250 V-dc buses 2A and 2B

4160-V recirculation pump trip (RPT) switchgear

Transformers

1400-1610-kVA, 4160-600-V essential transformers

112.5 kVA, 600-208/120-V essential transformers

DRAFT

225-kVA, 75-kVA 4160-600-V transformers 2F1 and 2F2

600-V bus duct associated with 600-V load centers 2C and 2D and 4160-600-V transformer 2CD

Motors (4kV)

Residual heat removal service water (RHRSW) pump motors (4)

Plant service water (PSW) pump motors (4)

Residual heat removal (RHR) pump motors (4)

Core spray (CS) pump motors (2)

Ac and dc lighting and miscellaneous power cabinets - control and diesel generator buildings

Ac and dc motor control centers

600 V-ac essential motor control center (MCC) - reactor building (3)

250 V-dc essential MCC - reactor building (2)

600-208 V-ac essential MCC - diesel generator building (3)

600 V-ac essential MCC - intake structure (2)

Batteries and chargers

125-250-V station batteries 2A and 2B

125-V diesel batteries 2A and 2C

125-V battery chargers 2A-2F (station batteries)

125-V battery chargers 2G, 2J (diesel batteries)

Diesel generator sets 2A, 2C, and 1B (1B is shared with HNP-1

Neutral grounding resistors for diesels

Primary electrical penetrations (drywell)

Power and control cable for essential equipment and instruments

Raceway supports associated with essential systems and equipment

Pull boxes and junction boxes

Underground ducts, fittings, and encasement

Reactor protection system (RPS) breaker protection panels (2C71-P003A through F)

3.2.1.5 Seismic Category I Instrumentation and Control Systems Equipment

RPS (except distribution cabinets and motor-generator sets)

Primary containment and reactor vessel isolation control system

Power range monitors in nuclear boiler system

Emergency core cooling systems (ECCSs) initiating channels and logic and automatic depressurization system initiating channels and logic

Essential instrumentation and controls on the following systems

Nuclear boiler

Control rod drive (CRD)

RHR and RHRSW

CS

High-pressure coolant injection (HPCI) system

Reactor core isolation cooling (RCIC) system

Standby gas treatment system (SGTS)

PSW

Instrumentation and controls for the following:

Standby liquid control system (SLCS)

Safeguard equipment emergency room coolers and control room air-handling and condensing units

DRAFT

SUPPLEMENT 3.7A

SEISMIC DESIGN

This section describes the seismic design requirements and methods used for Hatch Nuclear Plant-Unit 2 (HNP-2), and the seismic design and analysis of nonnuclear steam supply system equipment. Seismic design of nuclear steam supply system (NSSS) equipment is described in supplement 3.7B.

3.7A.1 SEISMIC INPUT

The two types of seismic inputs used in the seismic analyses were the ground design spectra and the associated synthetic accelerogram.

3.7A.1.1 DESIGN RESPONSE SPECTRA

Ground design spectra were established through extensive investigations on the geological conditions of the plant site and past seismological history of the neighborhood areas. The details of these investigations and the resulting recommendations are presented in section 2.5. The recommendations were given in the form of maximum horizontal acceleration values of the ground, 0.08 g and 0.15 g for operating basis earthquake (OBE) and design basis earthquake (DBE), respectively. The modified Newmark design spectra associated with these acceleration levels were adopted and are shown in figures 3.7A-1 and 3.7A-2. They are characterized by a maximum amplification factor of 3.5 for 2 percent of critical damping and no amplification for frequencies beyond 30 Hz.

3.7A.1.2 SYNTHETIC TIME HISTORIES

3.7A.1.2.1 Modified TAFT Time History

The synthetic acceleration time history shown in figure 3.7A-3 was developed for use as input to the time history analyses that resulted in the generation of the floor response spectra (FRS) used to seismically qualify subsystems until April 14, 1985.

In developing this synthetic accelerogram, the first 20 s of the TAFT 1952 horizontal earthquake component was selected as the input motion. It was then modified using spectrum suppressing and spectrum raising techniques[1] such that its response spectra enveloped the corresponding design spectra at all but a few frequencies. At the few points where the design spectra were not enveloped, the calculated response spectra were within 10 percent of the design spectra. Figures 3.7A-4 and 3.7A-5 show comparisons of response spectra for the modified TAFT earthquake time history with the ground design spectra.

The spectra of the time history were computed at the following 71 frequencies (in Hz):

> 0.2 . . . (increment = 0.1 Hz) . . . 3.0, 3.15, 3.3, 3.45, 3.6, 3.8, 4.0, 4.2, 4.4, 4.7, 5.0, 5.25, 5.5, 5.75, 6.0, 6.25, 6.5, 6.75, 7.0, 7.3, 7.6, 8.0, 8.5, 9.0, 9.5, 10.0, 10.5, 11.0, 11.5, 12.0, 12.5, 13.0, 13.5, 14.0, 14.5, 15.0, 16.5, 18.0, 20.0, 22.0, 25.0, 28.0, and 33.0.

These frequencies were chosen so that most of the increments do not exceed 5 percent within the range of 1 to 15 Hz.

3.7A.1.2.2 Synthetic Time Histories (1984)

A review was performed in 1984 to address the FSAR peak-broadening requirements for FRS, and it was concluded that no significant safety issue exists with the subsystems that were seismically qualified using the existing FRS. As a part of the review, two updated (1984) acceleration time histories were developed for use in generating new FRS. The two time histories developed (one for use in OBE analyses and one for use in DBE analyses) are shown in figures 3.7A-18 and 3.7A-19. Figure 3.7A-20 presents a plot comparing the 3-percent damped spectrum for the OBE time history with the corresponding design spectrum. Similarly, figure 3.7A-21 presents a plot comparing the 5-percent damped response spectrum for the DBE time history with the corresponding design spectrum. The calculated spectra shown in both figures were computed at the 71 frequencies defined in paragraph 3.7A.1.2.1. Comparison of these two figures with the corresponding figures for the original time history (i.e., figures 3.7A-4 and 3.7A-5) demonstrates that the two updated time histories provide a more realistic representation of the design response spectra than does the original time history discussed in paragraph 3.7A.1.2.1.

The new (1984) FRS were developed during the seismic review to reflect the as-built conditions of the structures and to provide a more realistic representation of the specified seismic design environment. These new spectra, which were developed using the updated time histories in conjunction with the applicable methodology defined in the balance of this section, are used, as of April 4, 1985, to seismically qualify subsystems.

3.7A.1.3 DAMPING VALUES

Energy dissipation in structures is generally represented by equivalent viscous damping. Evaluation of damping coefficients is based on the material, the predicted stress and strain level, and the type of connections used in the structural system. Table 3.7A-1 summarizes the damping values used in the seismic analyses. The values listed for structures, assemblies, and piping were adopted from Newmark's paper.[2] As noted in table 3.7A-1, in lieu of using the soil damping values presented in the table, the equations in table 3.7A-2 could be used to calculate the soil damping coefficients. As of April 4, 1985, damping per figures 3.7A-22 and 3.7A-23 for piping systems and cable tray supports, respectively, is used for all new and replacement systems and load reconciliation work.

3.7A.1.4 BASES FOR SITE DEPENDENT ANALYSIS

Site dependent analysis is not used. Subsection 2.5.2 describes the bases for specifying the vibratory ground motion for design use.

3.7A.1.5 SOIL-SUPPORTED SEISMIC CATEGORY I STRUCTURES

Except for the main stack, which is supported on piles, the Seismic Category I structures are supported on soil. The soil underlying the structures extends to a depth of at least 4000 ft before bedrock is encountered.

3.7A.1.6 SOIL-STRUCTURE INTERACTION

The lumped representation and equivalent soil springs and dampers were used to account for soil-structure interaction in the mathematical model for all Seismic Category I structures. The lumped representation is derived from analyzing a model composed of a rigid plate resting on the surface of an elastic half-space. The resulting foundation compliance is frequency dependent but can be approximated by a constant compliance for engineering application.[3] The foundation compliance is a function of the mass and dimensions of the foundation mat and the properties of the foundation medium. As a mechanical analog this compliance function can be represented by equivalent springs and dampers. Expressions for the equivalent soil spring constants used in the seismic analyses are defined in table 3.7A-2. The soil-damping values used are described in subsection 3.7A.1.3.

DRAFT

(This page intentionally left blank)

3.7A.2 SEISMIC SYSTEM ANALYSIS

3.7A.2.1 SEISMIC ANALYSIS METHODS

Response of Seismic Category I structures, systems, and components was determined analytically using the methods described in the following sections. Where the analytical method of analysis cannot ensure the functional integrity of a structure, system, or component, dynamic testing was employed. The procedures of dynamic testing are described in paragraph 3.7A.2.1.2.

3.7A.2.1.1 Modal Superposition

The method of modal superposition was used for the seismic analysis of all Seismic Category I structures, systems, and components. The mathematical model of each of the structures, systems, and components consists of lumped masses and weightless members and was represented by natural frequencies and the associated natural modes. A typical modal equation is given as follows:

$$\ddot{q}_j + 2\beta_j \omega_j \dot{q}_j + \omega_j^2 q_j = - \Gamma_j \ddot{u} \qquad (1)$$

where:

q_j = jth displacement coordinate

ω_j = jth circular natural frequency

β_j = jth modal damping ratio

Γ_j = jth modal participation factor

$\ddot{u}$ = base motion expressed in terms of acceleration

For engineering purposes, all those modes with frequencies lower than 33 Hz were considered in the analysis. The mathematical models of the Seismic Category I structures are shown in figures 3.7A-8 through 3.7A-17.

Depending on the form of earthquake inputs and the information required, two different techniques, response spectrum technique and time-history analysis, were engaged in the computation.

A. Response Spectrum Technique

With the input given in terms of design spectra, the modal displacement response is determined by:

$$q_{j\ max} = \Gamma_j\ (SA)_j/\omega_j^2 \qquad (2)$$

where:

(SA) = the value of spectral acceleration at the frequency f_j ($f_j = \omega_j / 2\pi$) and for damping β_j. The displacement response per mode at any mass point, i, is:

$$x_{ij',\ max} = \phi_{ij}\ q_{j',\ max}$$

where: ϕ_{ij} = modal coordinate

Other structural response quantities per mode, such as shears and moments, can be obtained from $x_{ij',\ max}$ by making use of the stiffness properties of the structural members.

With the modal responses determined, the total response is computed according to the criterion of "the square root of the sum of the squares (SRSSs) of individual modal responses." When modes are closely spaced, they are first divided into groups such that in each group the deviation in frequency between the first and the last modes does not exceed 10 percent of the lower frequency. The criterion of "the sum of absolute values" is then applied to each group and the results from all the groups and the remaining modal responses are combined according to the criterion of SRSSs.

B. Time History Analysis

With the input given in the form of an acceleration time-history, the modal responses are evaluated by a step-by-step integration process using equation 1. The total response of interest is then determined by directly superimposing the modal responses in the time domain.

3.7A.2.1.2 Testing Procedures

For certain Seismic Category I equipment and components where dynamic testing was required to demonstrate functional integrity during and after specified seismic conditions, one of the following approaches was used to satisfy the requirements:

A. Performance data of equipment which, under the specified conditions, has been subjected to equal or greater dynamic loads than those to be experienced under the specified seismic conditions.

B. Test data from previously tested comparable equipment which, under similar conditions, has been subjected to equal or greater dynamic loads than those specified.

C. Actual dynamic testing was in accordance with supplement 3.7A.A, subsection 3.7A.A.3.2.

D. Alternate test procedures that satisfied the requirements are specified in supplement 3.7A.A, subsection 3.7A.A.3.2.

3.7A.2.2 NATURAL FREQUENCIES AND RESPONSE LOADS

Table 3.7A-3 presents the first five frequencies

for Seismic Category I structures. In addition, the SRSSs response loads for the reactor building are also presented. The mathematical models of the major Seismic Category I structures whose natural frequencies appear in table 3.7A-3 are shown in figures 3.7A-8 through 3.7A-17.

For Seismic Category I structures, the response spectra for different damping values were generated at all mass points in the mathematical model on which the equipment is supported.

3.7A.2.3 PROCEDURES USED TO LUMP MASSES

A structure is modeled as a discrete mass system by lumping the mass of the structure, equipment, and components at various locations of high-mass concentration such as floors and/or locations of Seismic Category I equipment. In general, the weight of any one member together with the loads acting on it were equally lumped at two adjacent points where the member was connected. An equipment, component, or system was usually lumped into the supporting structure mass if its estimated weight was less than one-tenth that of the supporting mass;

otherwise, the equipment, component, or system would be itself a mass point. In any case, the number of lumped masses was at least twice the number of the highest mode used in the analysis unless the seismic behavior of the structure was adequately described using a lesser number.

3.7A.2.4 ROCKING AND TRANSLATIONAL RESPONSE SUMMARY

A lumped representation to account for the soil-structure interaction effect was assumed for all Seismic Category I structures and is described in subsection 3.7A.1.6.

3.7A.2.5 METHODS USED TO COUPLE SOIL WITH SEISMIC-SYSTEM STRUCTURES

Finite element analyses were not used for HNP-2.

3.7A.2.6 DEVELOPMENT OF FLOOR RESPONSE SPECTRA

The multi-mass time history method was used to develop the FRS. The spectra were generated at various floors or other locations of concern based on the time history motions obtained from the time history analysis of the structures as described in paragraph 3.7A.2.1.1B. The spectra were calculated at the structural frequencies as well as at additional selected frequencies such that the frequency interval between consecutive frequencies typically did not exceed 10 percent and in no case exceeded 13 percent of the lower frequency for the frequency range from 1 to 22 Hz. For example, the 1984 spectra were calculated at the following 124 frequencies (Hz) in addition to the structural frequencies:

0.1, 0.15, . . . (increments = 0.05 Hz) . . . 1.0, 1.1, (increments = 0.1 Hz) . . . 10.0, 11.0, 12.0, . . . ~~(increments = 1.0 Hz) . . . 25.0.~~

3.7A.2.7 DIFFERENTIAL SEISMIC MOVEMENT OF INTERCONNECTED COMPONENTS

The method of analysis discussed in paragraph 3.7A.2.1.1A was used to compute stresses for any interconnected components between floors. The input floor response spectrum is the envelope of the spectra for the floors to which the components are connected.

DRAFT

3.7A.2.8 EFFECTS OF VARIATIONS ON FLOOR RESPONSE SPECTRA

To account for the effect of possible variations in structural frequencies and subsequently the FRS due to the uncertainties in the material properties of the structure and soil, the computed FRS were smoothed, and peaks associated with the structural frequencies were widened by ±10 percent.

3.7A.2.9 USE OF CONSTANT VERTICAL LOAD FACTORS

No constant vertical load factors were used for Seismic Category I structures. The same method of analysis described in subsection 3.7A.2.1 was also used for the vertical direction. Two-thirds of the horizontal ground spectrum and the horizontal modified accelerogram were used as the minimum vertical input for analysis.

3.7A.2.10 METHODS USED TO ACCOUNT FOR TORSIONAL EFFECTS

For those Seismic Category I structures which are nearly symmetric and have torsional frequencies much higher than the corresponding translational frequencies, the slight eccentricity between the center of mass and rigidity is unlikely to cause any significant effect on the total response. Therefore, the torsional coupling was neglected in the mathematical model of these structures. Static torsional moments were computed, however, to ensure the adequacy of the design.

For those Seismic Category I structures and components which are unsymmetric in nature, including all Class 1 piping systems, torsional coupling was included in the multimass model for computing coupled dynamic response.

3.7A.2.11 COMPARISON OF RESPONSES

Table 3.7A-4 shows the comparison of responses at selected points in the Seismic Category I structures.

3.7A.2.12 METHODS FOR SEISMIC ANALYSIS OF DAMS

Dams were not constructed to impound bodies of water to serve as heat sinks.

3.7A.2.13 METHODS TO DETERMINE SEISMIC CATEGORY I STRUCTURE OVERTURNING MOMENT

The overturning moments of the Seismic Category I structures were calculated by the response spectrum method. The stability of the structures is checked by combining the overturning moment, dead load of the structure, and vertical acceleration. The soil reaction under the containment is obtained by considering the linear stress distribution under a rigid base mat subjected to the worst combined effects of overturning moment, dead load, and vertical acceleration.

3.7A.2.14 ANALYSIS PROCEDURE FOR DAMPING

For structures composed of major subsystems that are made of different materials, the composite modal damping was computed using either the mass proportional, stiffness proportional, modal weighting, or Tsai method. A description of the mass proportional method is illustrated below; the first step involves the formation of the following matrix:

$$\left[C\right] = \left[\phi\right]^T \left[^{\diagdown}\beta_{\diagdown}\right] [M] [\phi] \tag{3}$$

where:

$[\phi]$ = the modal matrix

$[M]$ = the mass matrix

$\left[^{\diagdown}\beta_{\diagdown}\right]$ = a diagonal matrix made up of the damping value specified for the subsystems

The composite damping is then obtained from [C] by using the diagonal terms after they are divided by the generalized mass of the corresponding mode where the generalized mass is defined by $\overline{M}_j$ as follows:

$$\left[^{\diagdown}\overline{M}j_{\diagdown}\right] = \left[\phi\right]^T [M] [\phi] \tag{4}$$

3.7A.3 SEISMIC SUBSYSTEM ANALYSIS

3.7A.3.1 DETERMINATION OF NUMBER OF EARTHQUAKE CYCLES

3.7A.3.1.1 Seismic Category I Structures

The number of maximum amplitude cycles is not a consideration for Seismic Category I structures.

3.7A.3.1.2 Piping and Other Systems and Components

During the 20- to 30-s duration of an earthquake event, strong motion is typically experienced for 4 to 6 s. Frequencies of vibration for which the response is significant are mostly in the range from 1 to 20 Hz with the highest responses occuring within a more narrow range, usually 3 to 8 Hz. One DBE and two OBEs are considered in the design.

The number of cycles for the DBE then can be estimated by multiplying 20 Hz by 6 s by one earthquake which yields 120 cycles. Similarly, the number of cycles for the OBE can be estimated by multiplying 20 Hz by 6 s by two earthquakes which yields 240 cycles. To be conservative, the following total number of loading cycles have been used in the design:

- DBE 300 cycles
- OBE 600 cycles

3.7A.3.2 BASIS FOR SELECTION OF FORCING FREQUENCIES

The methods used to analyze subsystems for dynamic loadings can be either the time history method or the response-spectrum technique. In general, these loadings are in the form of acceleration, velocity, or displacement time histories, or they may be in the form of FRS.

In both of these methods of describing the seismic environment, the structural amplifications are reflected. Therefore, when these loads are used as inputs to the subsystems, each mode responds according to the amplification which has been predetermined in the time history analysis of the supporting structure.

It is considered good practice to avoid the regions of load amplification with any system being designed. This is easily identified by observing the frequencies of all predominant modes which lie near the region of spectral amplification; however, it is sometimes found to be impractical or impossible. In these cases, the subsystem is analyzed and designed for the amplified loadings.

3.7A.3.3 ROOT MEAN SQUARE BASIS

The term "root mean square basis" is not used in describing the procedure for the combination of modal responses for HNP-2.

3.7A.3.4 PROCEDURES FOR COMBINING MODAL RESPONSES

The discussion of the procedures for combining modal responses is referred to in paragraph 3.7A.2.1.1A.

3.7A.3.5 SIGNIFICANT DYNAMIC RESPONSE MODES

Supplement 3.7A.A describes the analysis techniques to be used if the peak of the spectra is used by equipment suppliers. Such considerations are included in Institute of Electrical and Electronics Engineers Standard 344-1971, as defined and referenced in supplement 3.7A.A. The design and analysis of instrumentation and electrical equipment are described in section 3.10.

3.7A.3.6 DESIGN CRITERIA AND ANALYTICAL PROCEDURES FOR PIPING

Piping systems are anchored and restrained to floors and walls of buildings. The relative seismic displacements between buildings, between floors in buildings, and between major components are applied to the piping, anchors, and restraints in a rational and conservative manner. Seismic movements are always considered to be out of phase between independent structures so that maximum relative displacements are used. The resulting stresses are classified as secondary and are combined with other secondary stresses. The sum of secondary stresses are held within the limits of the applicable piping code.

The seismic inputs to the original OBE and DBE piping systems analyses were defined using the 0.5-percent and 1.0-percent damped FRS, respectively. As of April 4, 1985, damping per figure 3.7A-22 is used in response spectrum analyses performed for all new and replacement systems and load reconciliation work. If as a result of using these damping valves, piping supports are removed, modified, or eliminated, the expected increased piping displacements due to greater piping flexibility will be checked to assure that they can be accommodated and that there will be no adverse interaction with adjacent structures, components, and equipment. The damping criteria established by this figure are consistent with the frequency-dependent approach established by the Pressure Vessel Research Council Technical Committee on Piping Systems.[5]

3.7A.3.7 BASES FOR COMPUTING COMBINED RESPONSE

The basis for combining the modal responses, i.e., displacements, effective inertia forces and accelerations, internal forces and moments, and support reactions, is the SRSS method. To obtain conservative results, the three directional (one vertical and two horizontal) responses obtained by the modal combination of each direction are combined by the SRSSs method, or by the absolute sum of the worst horizontal with the vertical.

Having the total internal moments computed by either of the above procedures, stresses were then calculated and combined with the stresses due to other loadings. The combined stresses are held within the stress limits of the applicable code.

3.7A.3.8 AMPLIFIED SEISMIC RESPONSES

A constant vertical load factor is not used for seismic design of Seismic Category I structures, components, or equipment.

3.7A.3.9 USE OF SIMPLIFIED DYNAMIC ANALYSIS

Simplified dynamic analysis is not used for Seismic Category I structures and is normally applied only to field-routed, 2-in. and under piping and some subsystems.

To perform a simplified dynamic analysis on a system, it must have a first mode natural frequency in the rigid range of the response spectrum. The rigid range of the response spectrum curve is defined as that portion in which there is no significant change in spectral acceleration with increasing frequencies. (See point "A" on figure 3.7A-6.) 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 direction is combined on a SRSSs basis. The stresses are then computed in accordance with American Society for Mechanical Engineers (ASME) Boiler and Pressure Vessel Code, Section III, Nuclear Power Plant Components, including 1971 Winter Addenda. The rigid range is dependent on building response and as such is determined on a case basis. The rigid range of floor spectrum typically begins at approximately 20 Hz.

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

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

B. A dynamic analysis is 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.

A summary of typical results comparing the simplified dynamic methods and the response spectrum modal analysis method is contained in Appendix D of BP-TOP-1, Revision 1.[4]

When piping is analyzed by simplified methods all supports and components attached to the piping are required to be in the rigid range so that no amplification of seismic motion exists.

3.7A.3.10 MODAL PERIOD VARIATION

The procedures used to account for modal period variation in models of Seismic Category I structures are discussed in subsection 3.7A.2.9.

3.7A.3.11 TORSIONAL EFFECTS OF ECCENTRIC PIPING

The seismic mass model accounts for the effect of masses that are offset from the pipe centerline. Components with eccentric masses are modeled by placing the component's mass at the component's calculated center of gravity and connecting this mass to the pipe centerline with a rigid connection thereby accounting for its torsional effects.

3.7A.3.12 PIPING OUTSIDE CONTAINMENT

Applicable subsections of sections 3.7A.2 and 3.7A.3 are used for design and analysis of Seismic Category I piping inside and outside containment.

The techniques and criteria used to analyze structural stresses in buried Seismic Category I piping and electrical ducts are presented in supplement 3.7A.B.

3.7A.3.13 INTERACTION OF OTHER PIPING WITH SEISMIC CATEGORY I PIPING

The interface between Seismic Category I piping and non-Category I piping is always an anchor. The anchor is designed to prevent interaction between seismic and nonseismic piping under the most conservative combination of thermal, weight, and seismic loads.

3.7A.3.14 LOCATION OF SUPPORTS AND RESTRAINTS

Seismic supports and restraints for Seismic Category I piping are located so that the stresses, as determined by the dynamic analysis, are less than the appropriate code allowable limits. When rigid seismic supports result in excessive thermal loads on piping or equipment, snubbers or dampers are used.

The pipe support contractors' pipe restraint locations and detailed support drawings are reviewed by pipe stress engineers to ensure that they conform to requirements. In addition, a field inspection of the pipe supports is made by stress engineers to ensure that supports have been installed properly and meet design requirements.

For 2-in. and under Seismic Category I piping, a Bechtel field installation manual is provided so that field engineers can properly design and locate pipe supports and restraints. When the field engineers have completed their designs, they are reviewed by pipe stress engineers.

3.7A.3.15 SEISMIC ANALYSIS FOR FUEL ELEMENTS, CONTROL ROD ASSEMBLIES, AND CONTROL ROD DRIVES

The seismic analysis for fuel elements, control rod assemblies, and control rod drives is discussed in paragraph 3.7B.2.1.6.3

3.7A.3.16 SEISMIC ANALYSIS OF CABLE TRAY SUPPORTS

Cable tray supports are designed to withstand the calculated seismic loads using the FRS corresponding to the locations where the supports are attached. The simultaneous application of the horizontal and vertical earthquake components, which create the highest stresses, is used to design the cable tray supports. Stresses are limited to the allowables specified in paragraph 3.10.2.1.1.

In the original cable tray support analyses, the applicable damping values were established, based upon the supports type of construction, using the values specified in table 3.7A-1. As of April 4, 1985, damping per figure 3.7A-23 is used for all new and replacement systems and load reconciliation work. The damping criteria specified in figure 3.7A-23 provide a conservative estimate of damping for cable tray supports based upon a test program.(6)

(This page intentionally left blank)

3.7A.5.3 SOUTHERN COMPANY SERVICES, INC. SPECIFIED EQUIPMENT AND COMPONENTS

Equipment and Component Specifications

Seismic-response spectra for each location in the plant is developed for use by the design engineer. The design engineer is responsible for including the appropriate seismic-response spectra in the equipment purchase specification in a form that is meaningful to the vendor. All purchase specifications are reviewed by engineers competent in seismic analysis and testing for verification acknowledging complete and correct seismic requirements have been included.

Vendor analyses and/or test data are submitted to the responsible design engineer as agreed upon as part of the purchase specification. The responsible design engineer agrees with the submitted material in writing only after he is satisfied that it meets the design specification requirements. Guidance and counsel of engineers competent in the applicable discipline are made available to the responsible design engineer in the course of such reviews.

The quality assurance program is described in chapter 17 and provides a description of the review and approval of purchase specifications and vendor documents by competent engineering personnel.

REFERENCES

1. Tsai, N.C., "Spectrum - Compatible Motions for Design Purposes," Journal of Engineering Mechanics Division, ASCE, Vol. 98, No. EM2, April 1972.

2. Newmark, N.M., Design Criteria for Nuclear Reactors Subjected to Earthquake Hazards, Proc. I AEA Panel on Aseismic Design and Testing of Nuclear Facilities, Japan Earthquake Engineering Promotion Society, Tokyo, Japan, 1967.

3. Richart, Jr., F.E., Hall, Jr., J.R., and Woods, R.D., Vibrations of Soil and Foundations, Prentice Hall, Inc., New Jersey, 1970.

4. "Seismic Analysis Piping System," BP-TOP-1, Revision 1, February 1974.

5. Welding Research Council Bulletin Number 300: Technical Position on Criteria Establishment; Technical Position on Damping Values for Piping-Interim Summary Report; Technical Position on Response Spectra Broadening; and Technical Position on Industry Practice, 1984.

6. Cable Tray and Conduit Raceway Seismic Test Program, Release 4, Report 1053-21.1-4, ANCO Engineers, Inc., December 15, 1978.

DRAFT

TABLE 3.7A-1

DAMPING FACTORS FOR SEISMIC ANALYSIS IN PERCENT OF CRITICAL DAMPING[a]

	Operating Basis Earthquake	Design Basis Earthquake
Reinforced concrete structures	3.0	5.0
Steel frame structures	3.0	5.0
Bolted and riveted assemblies	3.0	5.0
Welded assemblies	2.0	3.0
Vital piping	0.5	1.0
Translation and rotation of foundation soil[b]	4.0	5.0

a. As of April 4, 1985, damping per figures 3.7A-22 and 3.7A-23 for piping systems and cable tray supports, respectively, is used for all new and replacement systems and load reconciliation work.

b. In lieu of using the soil damping values specified in this table, the equations in table 3.7A-2 may be used to calculate soil damping coefficients.

DRAFT

TABLE 3.7A-2 (SHEET 1 OF 2)

FORMULAS FOR EQUIVALENT FOUNDATION SPRING CONSTANTS AND DAMPING COEFFICIENTS (RECTANGULAR BASE)

Motion	Equivalent Spring Constant	Equivalent Damping Coefficient
Horizontal	$K_x' = 2(1 + \nu)\, G\beta_x \sqrt{BL}$	$c_x' = 0.576\, k_x R \sqrt{\rho/G}$
Rocking	$K_\psi' = \frac{G}{1 - \nu}\, \beta_\psi B^2 L$	$c_\psi' = \frac{0.30}{1 + B_\psi}\, k_\psi R \sqrt{\rho/G}$
Vertical	$K_z' = \frac{G}{1 - \nu}\, \beta_z \sqrt{BL}$	$c_z' = 0.85\, k_z\, R \sqrt{\rho/G}$

where:

B = width of the base mat in the plane of horizontal excitation

L = length of the base mat perpendicular to the plane of horizontal excitation

ν = Poisson's ratio of foundation medium

G = shear modulus of foundation medium

ρ = density of foundation medium

R = equivalent radius of the base mat as defined below

$\beta_x, \beta_\psi, \beta_z$ = constants that are functions of the dimensional ratio, B/L (from figure 10-16 in reference 3)

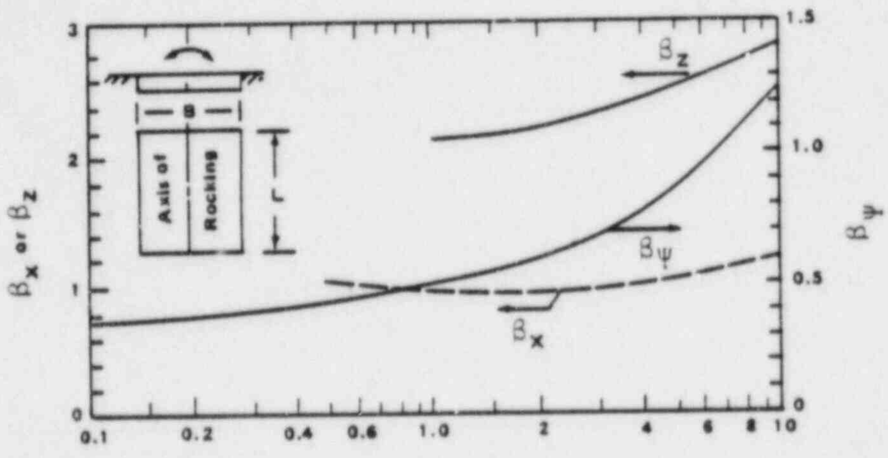
$$B_\psi = \frac{3\,(1-\nu)\, I_o}{8\rho R^5}$$

where:

I_o = total mass moment of inertia of structure and base mat about the rocking axis at the base

REV 2 7/84
REV 3 7/85

TABLE 3.7A-2 (SHEET 2 of 2)



Constants β_x, β_ψ, and β_z for Rectangular Bases

(EQUIVALENT RADIUS FOR RECTANGULAR BASE)

For a rectangular base having a dimension of B x L (B = width of base in the plane of horizontal vibration), the equivalent radius, R, is taken to be the smallest of the parameters, R_x, R_ψ, and R_z, defined below:

$$R_x = \frac{(1 + \nu)(7 - 8\nu)\, \beta_x \sqrt{BL}}{16(1 - \nu)}$$

$$R_\psi = \sqrt[3]{3\beta_\psi B^2 L/8}$$

$$R_z = \beta_z \sqrt{BL/4}$$

TABLE 3.7A-3 (SHEET 1 OF 2)

SUMMARY OF FREQUENCY AND RESPONSE LOADS

	Reactor Bldg		Control Bldg		Diesel Generator Bldg[a]		Intake Structure		Main Stack	
	E-W (Hz)	Vert (Hz)	E-W (Hz)	Vert (Hz)	E-W (Hz)	Vert (Hz)	E-W (Hz)	Vert (Hz)	E-W (Hz)	Vert (Hz)
Freq No. 1	1.61	6.45	1.01	2.37	4.12	4.59	7.04	14.60	0.60	8.64
Freq No. 2	3.73	20.41	5.38	9.44	7.76	83.25	21.13	66.27	2.24	18.02
Freq No. 3	8.37	26.26	7.00	13.71	36.20	NA	35.32	106.73	4.88	24.60
Freq No. 4	9.39	29.00	11.07	37.87	NA	NA	44.41	136.26	8.14	34.74
Freq No. 5	9.74	NA	15.27	49.11	NA	NA	53.74	178.87	11.66	43.98

a. The diesel generator building natural frequencies specified are those associated with the mean soil properties for this building.

DRAFT

DRAFT

TABLE 3.7A-3 (SHEET 2 OF 2)

REACTOR BUILDING SRSSs RESPONSES[(a)]

Elevation	87 ft		130 ft		158 ft		185 ft	
	E-W DBE	Vert DBE	E-W DBE	Vert DBE	E-W DBE	Vert DBE	E-W DBE	Vert DBE
Accel (g)	0.15	0.10	0.19	0.10	0.26	0.11	0.33	0.12
Disp (ft 10^{-4})	48.9	17.4	99.5	19.3	146.6	21.4	190.8	22.6
Force (kips 10^3)	42.1	16.4	40.3	15.7	34.2	12.2	27.5	9.2
Moment (K-ft 10^4)	414.6	NA	228.7	NA	133.5	NA	60.1	NA

Elevation	203 ft		228 ft		256 ft		280 ft	
	E-W DBE	Vert DBE	E-W DBE	Vert DBE	E-W DBE	Vert DBE	E-W DBE	Vert DBE
Accel (g)	0.38	0.12	0.45	0.12	0.60	0.14	0.67	0.14
Disp (ft 10^{-4})	217.2	23.1	253.6	23.5	621.4	25.0	1692.3	25.4
Force (kips 10^3)	19.1	6.2	10.2	3.4	2.0	0.7	1.4	0.3
Moment (K-ft 10^4)	26.9	NA	7.6	NA	3.3	NA	0.0	NA

a. These responses were not updated to reflect the 1984 analysis discussed in paragraph 3.7A.1.2.2

DRAFT

TABLE 3.7A-4 (SHEET 1 OF 2)

COMPARISON OF RESPONSES[a]

Reactor Building (OBE)

Mass Point	Acceleration (g) E-W SRSS	E-W TH	Vert SRSS	Vert TH	Displacement (ft 10^{-4}) E-W SRSS	E-W TH	Vert SRSS	Vert TH
1	0.08	0.09	0.05	0.06	26.6	29.3	9.4	8.8
2	0.10	0.12	0.05	0.06	54.3	56.0	10.5	9.8
3	0.14	0.15	0.06	0.06	80.0	80.2	11.6	10.7
4	0.18	0.18	0.06	0.06	104.2	101.9	12.2	11.2
5	0.21	0.19	0.06	0.06	118.6	115.9	12.5	11.5

Control Building (OBE)

Mass Point	Acceleration (g) E-W SRSS	E-W TH	Vert SRSS	Vert TH	Displacement (ft 10^{-4}) E-W SRSS	E-W TH	Vert SRSS	Vert TH
1	0.11	0.13	0.08	0.10	16.9	20.0	7.2	9.0
2	0.16	0.19	0.09	0.11	26.8	30.0	8.1	10.0
3	0.20	0.22	0.09	0.12	32.8	35.0	8.7	11.0
4	0.23	0.24	0.10	0.12	37.6	40.0	9.1	11.0
5	0.19	0.19	0.18	0.21	721.3	770.0	15.7	19.0

a. These responses were not updated to reflect the 1984 analysis discussed in paragraph 3.7A.1.2.2

TABLE 3.7A-4 (SHEET 2 OF 2)

Diesel Generator Building (OBE)[a]

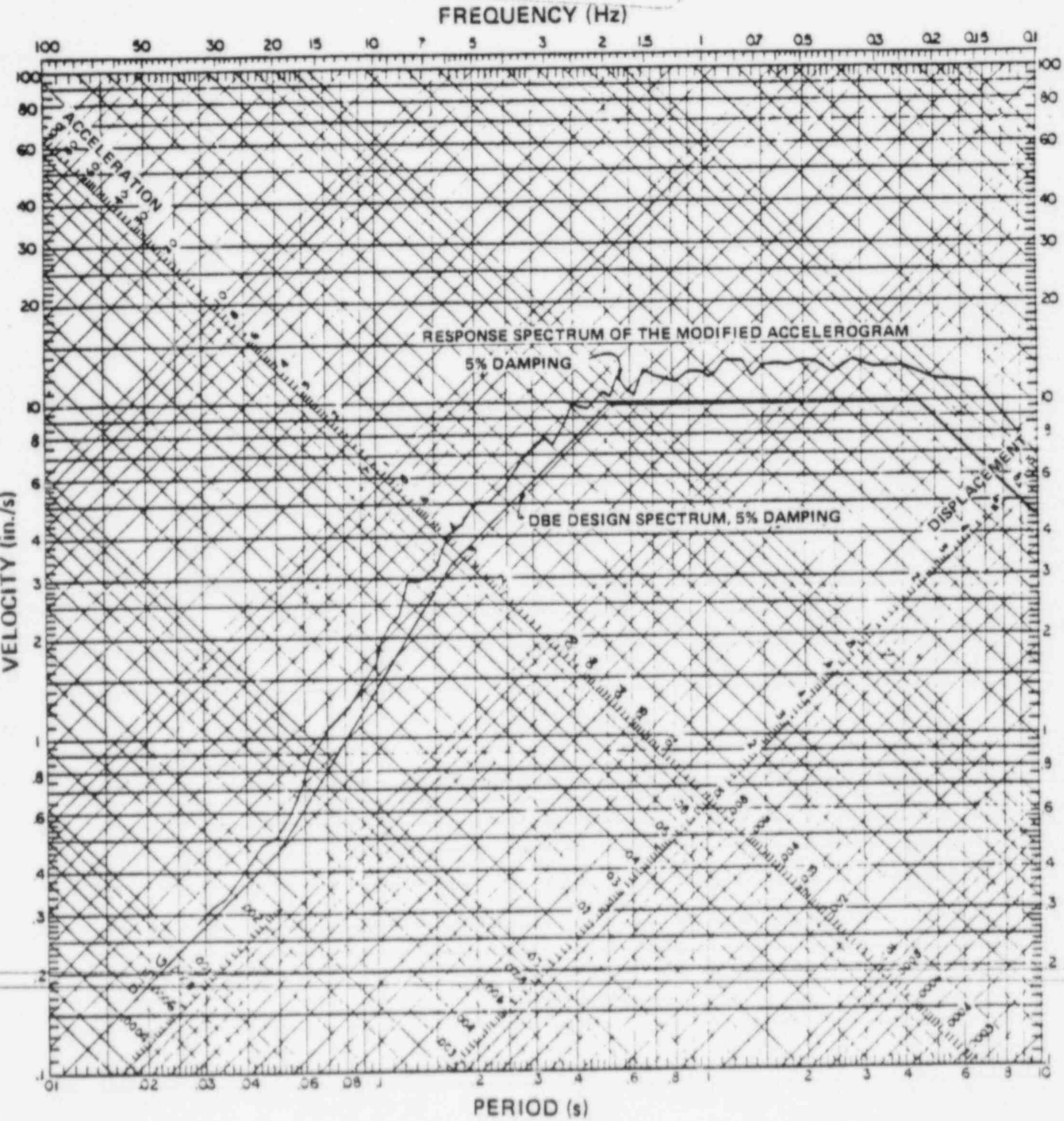
Mass Point	Acceleration (g) E-W SRSS	TH	Vert SRSS	TH	Displacement (ft 10^{-4}) E-W SRSS	TH	Vert SRSS	TH
1	0.20	0.23	0.13	0.15	94.6	110.0	49.9	58.1
2	0.21	0.24	0.13	0.16	100.7	117.0	50.2	58.4

Intake Structure (OBE)

Mass Point	Acceleration (g) E-W SRSS	TH	Vert SRSS	TH	Displacement (ft 10^{-4}) E-W SRSS	TH	Vert SRSS	TH
1	0.06	0.09	0.06	0.08	7.0	7.0	2.0	3.0
2	0.10	0.12	0.06	0.08	15.0	16.0	2.0	3.0
3	0.16	0.17	0.07	0.09	25.0	26.0	2.0	3.0
4	0.23	0.23	0.07	0.09	36.0	38.0	3.0	3.0
5	0.29	0.29	0.07	0.09	44.0	46.0	3.0	3.0

a. Responses are those associated with the mean soil properties for this building.

DRAFT



REV 3 7/85

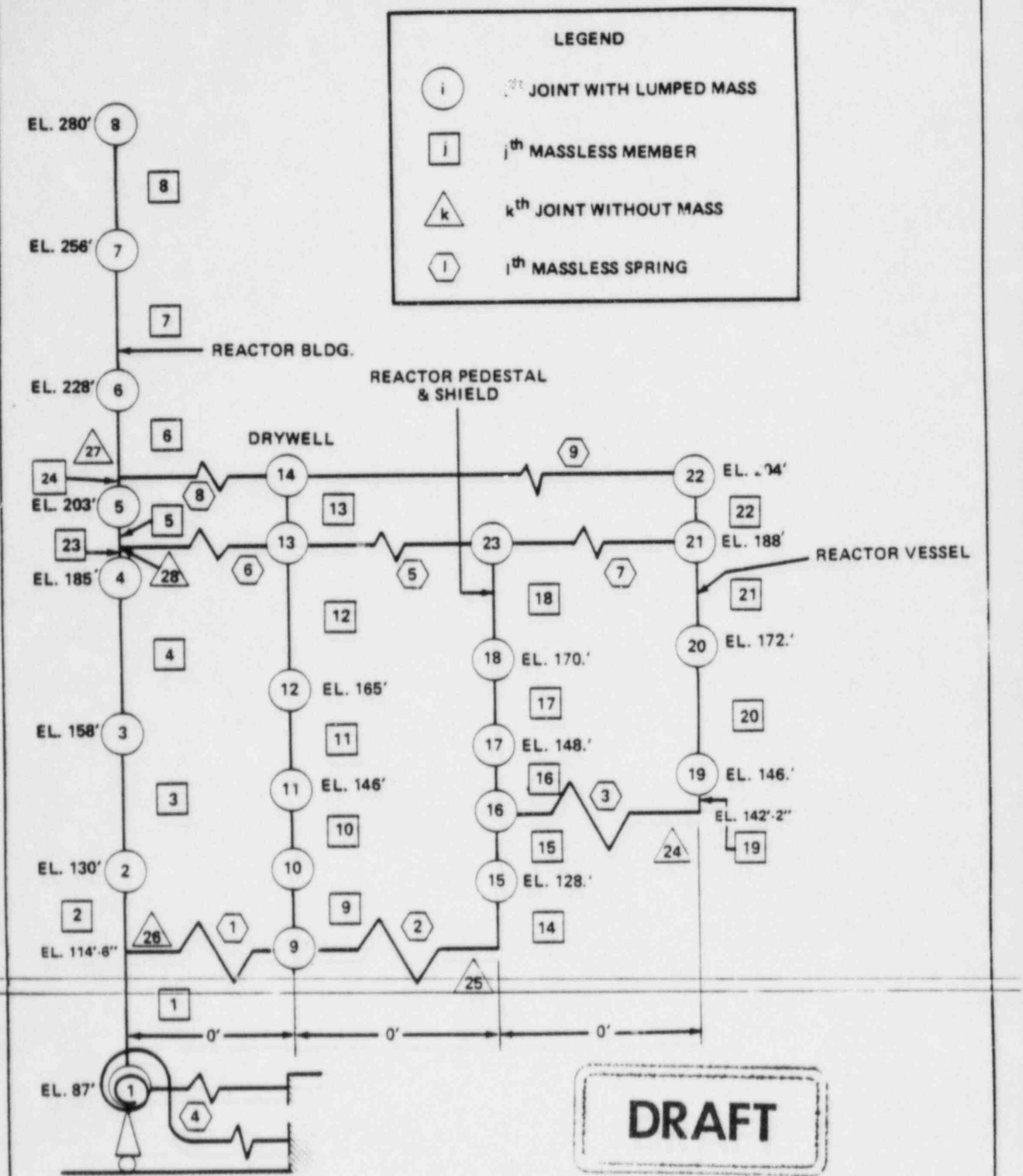
Georgia Power

EDWIN I. HATCH
NUCLEAR PLANT – UNIT 2

DESIGN SPECTRUM COMPARED WITH THE RESPONSE SPECTRUM OF THE MODIFIED ACCELEROGRAM FOR THE DBE WITH 5-PERCENT DAMPING

FIGURE 3.7A-5

3604-0



DRAFT

REV 3 7/85

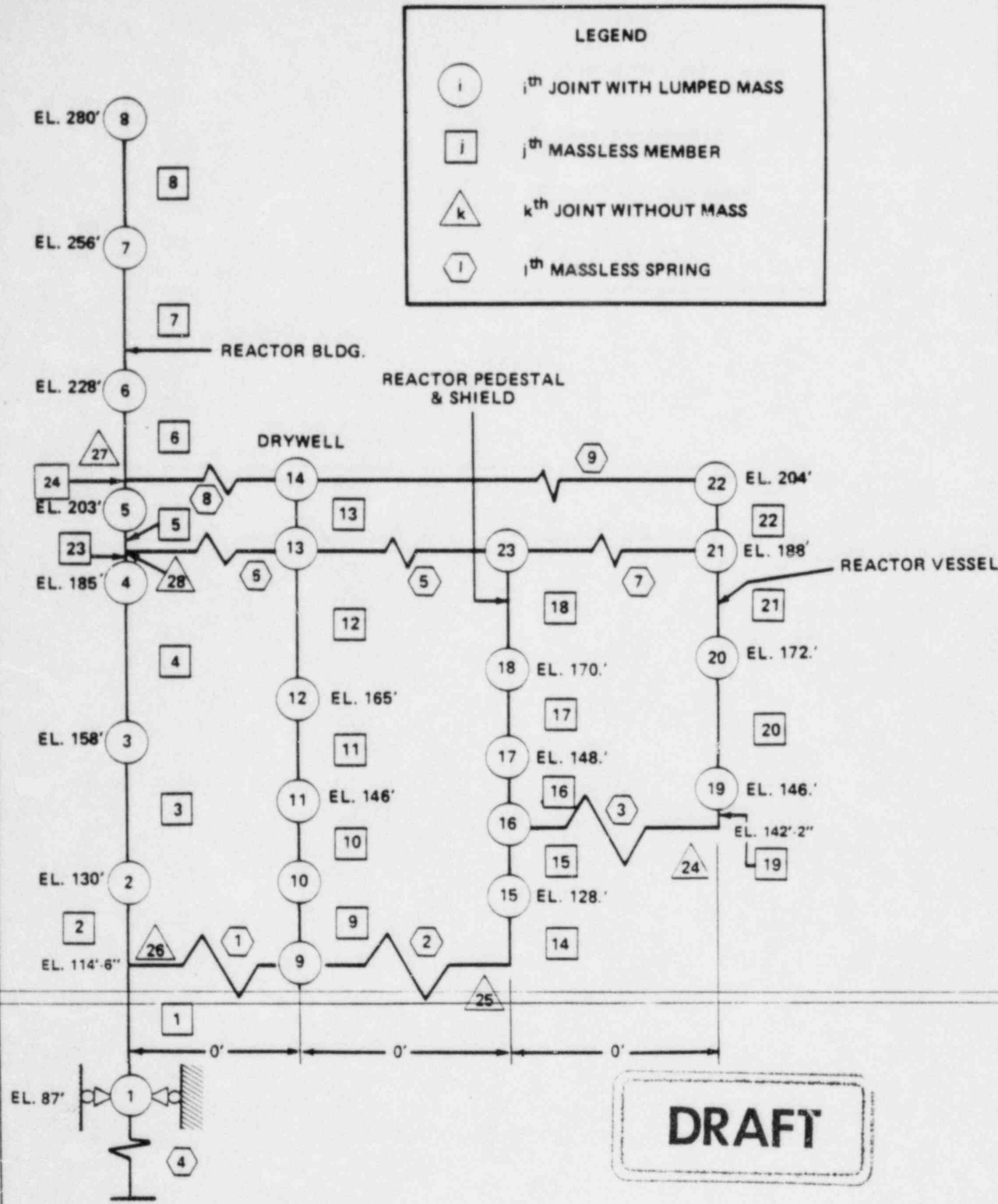
Georgia Power

EDWIN I. HATCH
NUCLEAR PLANT – UNIT 2

SEISMIC MODEL
REACTOR BLDG: LATERAL

FIGURE 3.7A-8

3604-0



DRAFT

REV 3 7/85

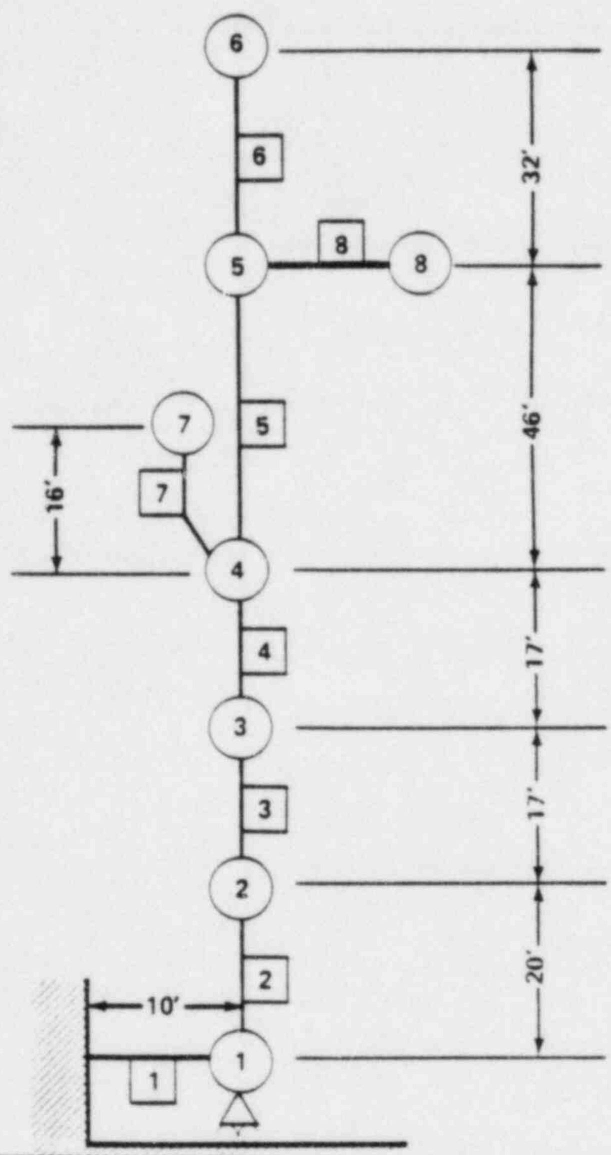
Georgia Power

EDWIN I. HATCH
NUCLEAR PLANT – UNIT 2

SEISMIC MODEL
REACTOR BLDG: VERTICAL

FIGURE 3.7A-9

3604-0



DRAFT

REV 3 7/85

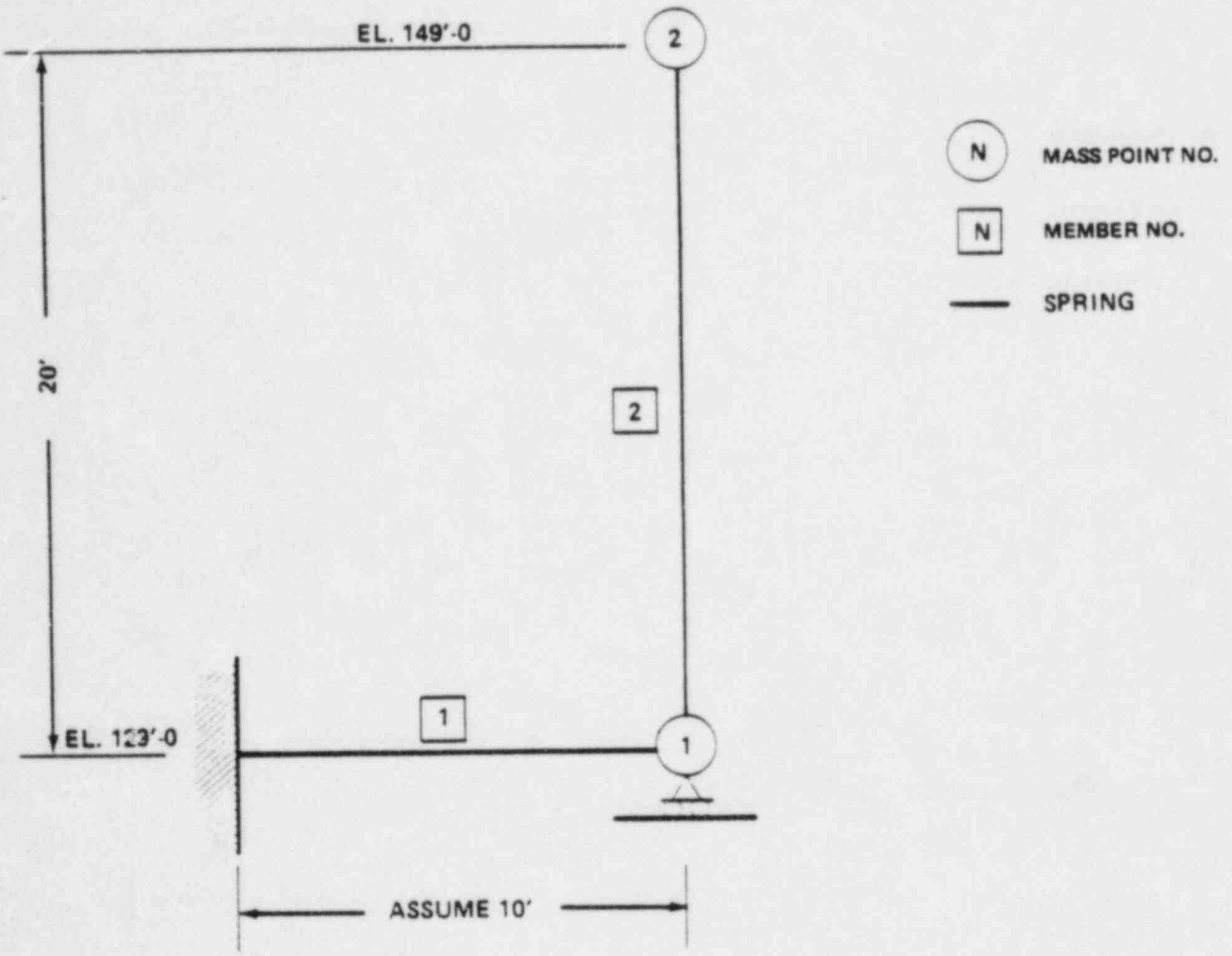
Georgia Power

EDWIN I. HATCH
NUCLEAR PLANT – UNIT 2

SEISMIC MODEL
CONTROL BLDG: LATERAL

FIGURE 3.7A-10

3604-0



DRAFT

REV 3 7/85

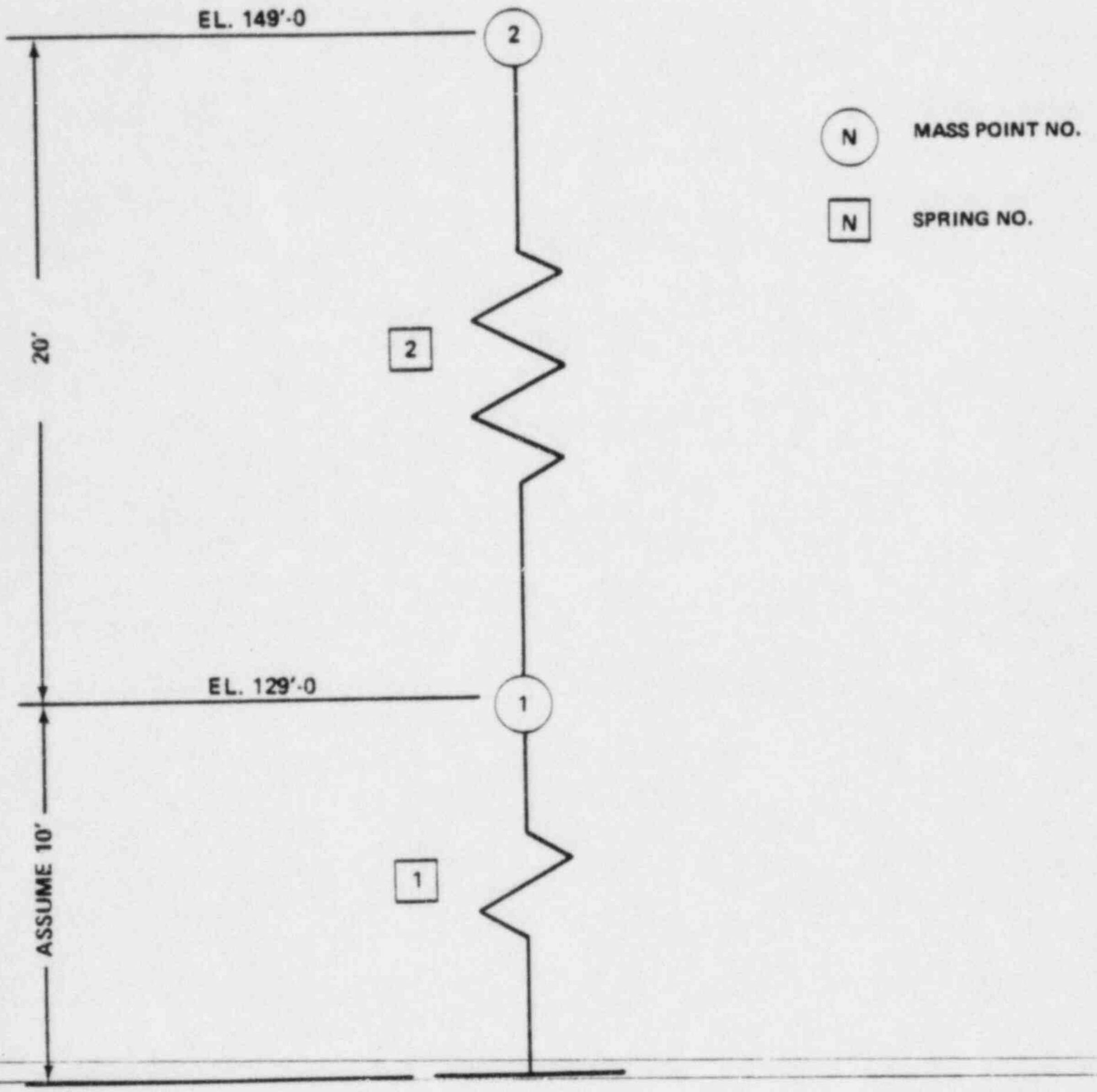
Georgia Power

EDWIN I. HATCH
NUCLEAR PLANT – UNIT 2

SEISMIC MODEL
DIESEL GENERATOR BLDG: LATERAL

FIGURE 3.7A-12

3604-0



DRAFT

REV 3 7/85

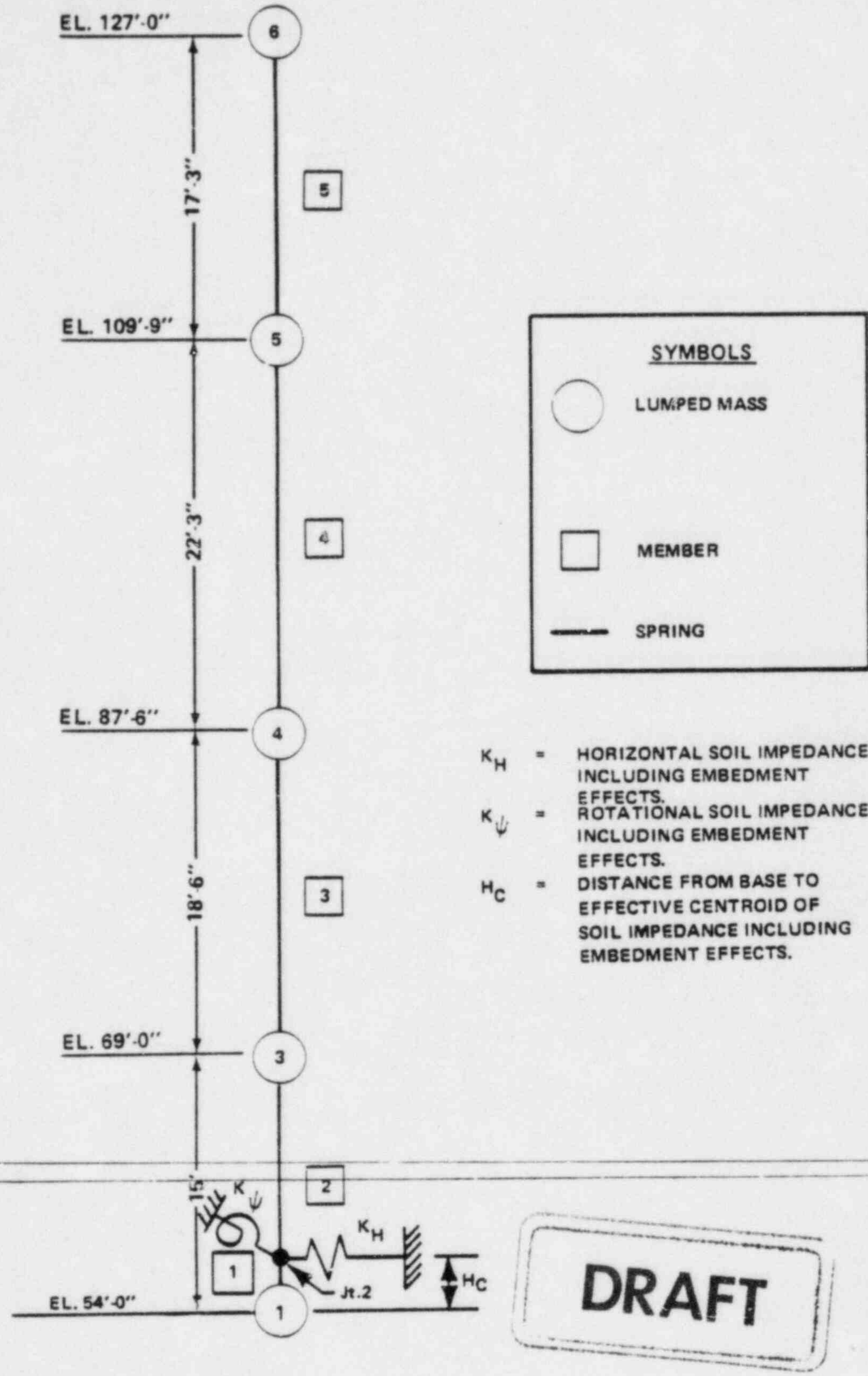
Georgia Power

EDWIN I. HATCH
NUCLEAR PLANT – UNIT 2

SEISMIC MODEL
DIESEL GENERATOR BLDG: VERTICAL

FIGURE 3.7A-13

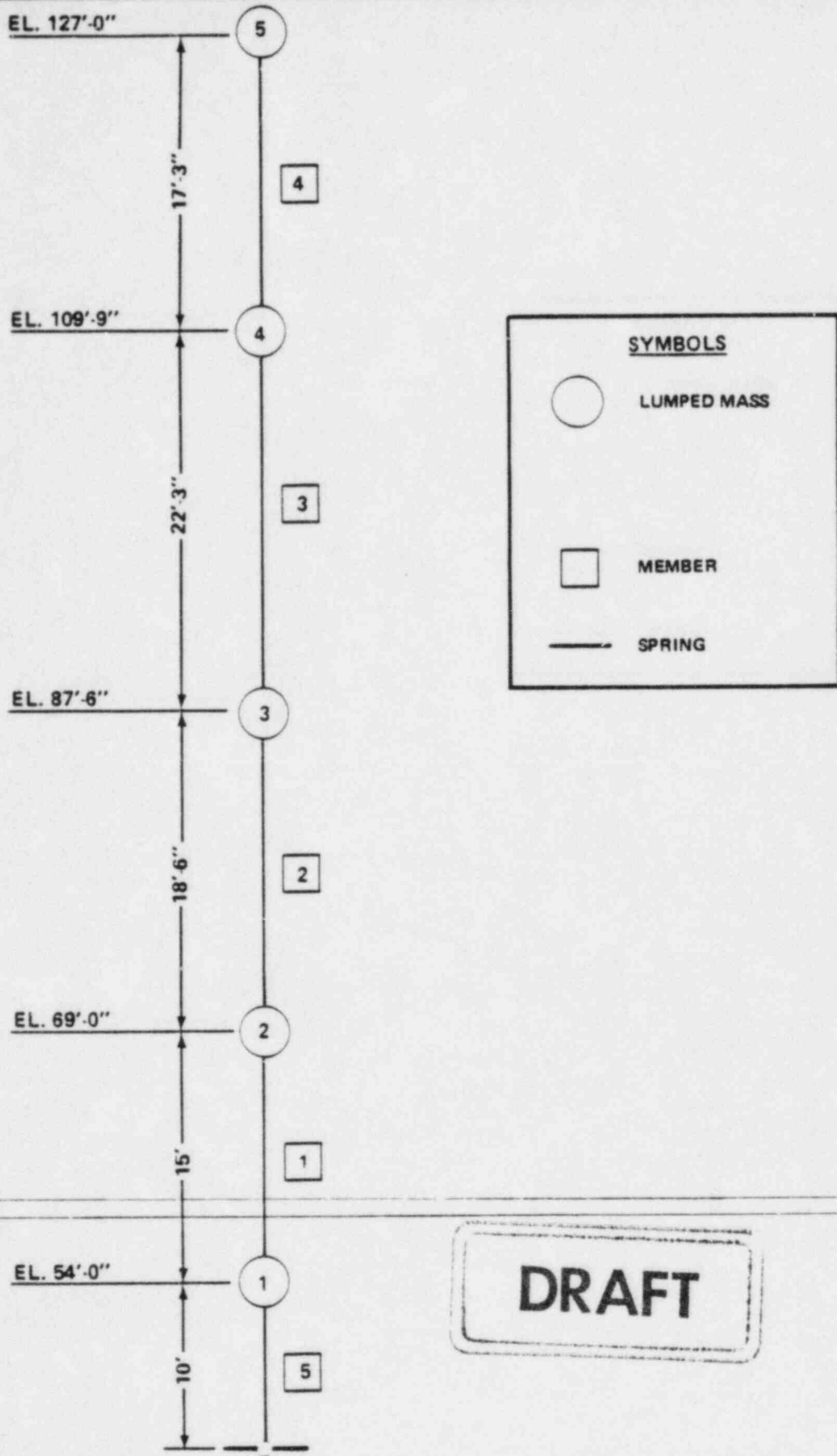
3604-0



REV 3 7/85

Georgia Power	EDWIN I. HATCH NUCLEAR PLANT – UNIT 2	SEISMIC MODEL INTAKE STRUCTURE: LATERAL
		FIGURE 3.7A-14

3604-0



DRAFT

REV 3 7/85

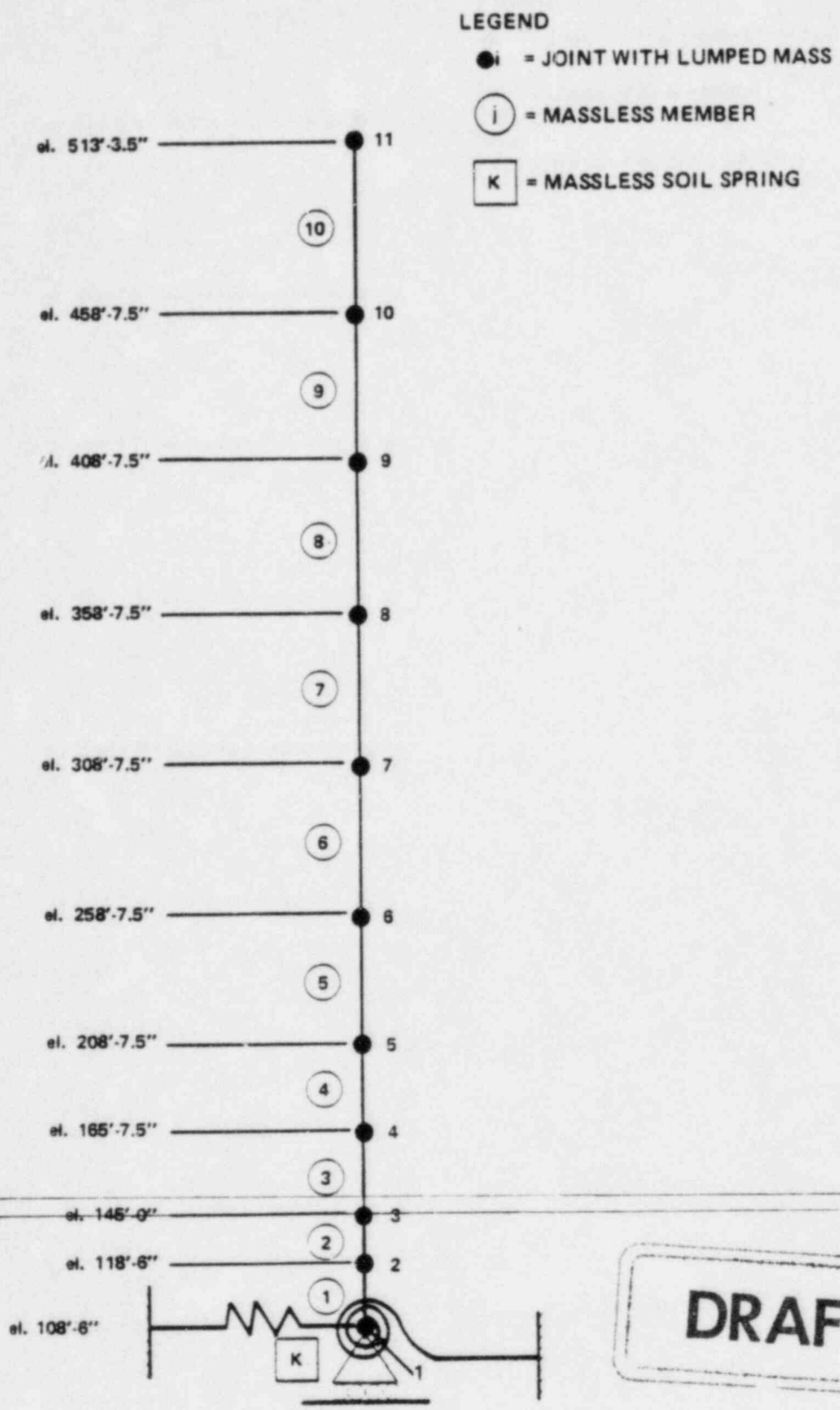
Georgia Power

EDWIN I. HATCH
NUCLEAR PLANT – UNIT 2

SEISMIC MODEL
INTAKE STRUCTURE: VERTICAL

FIGURE 3.7A-15

3604-0



DRAFT

REV 3 7/85

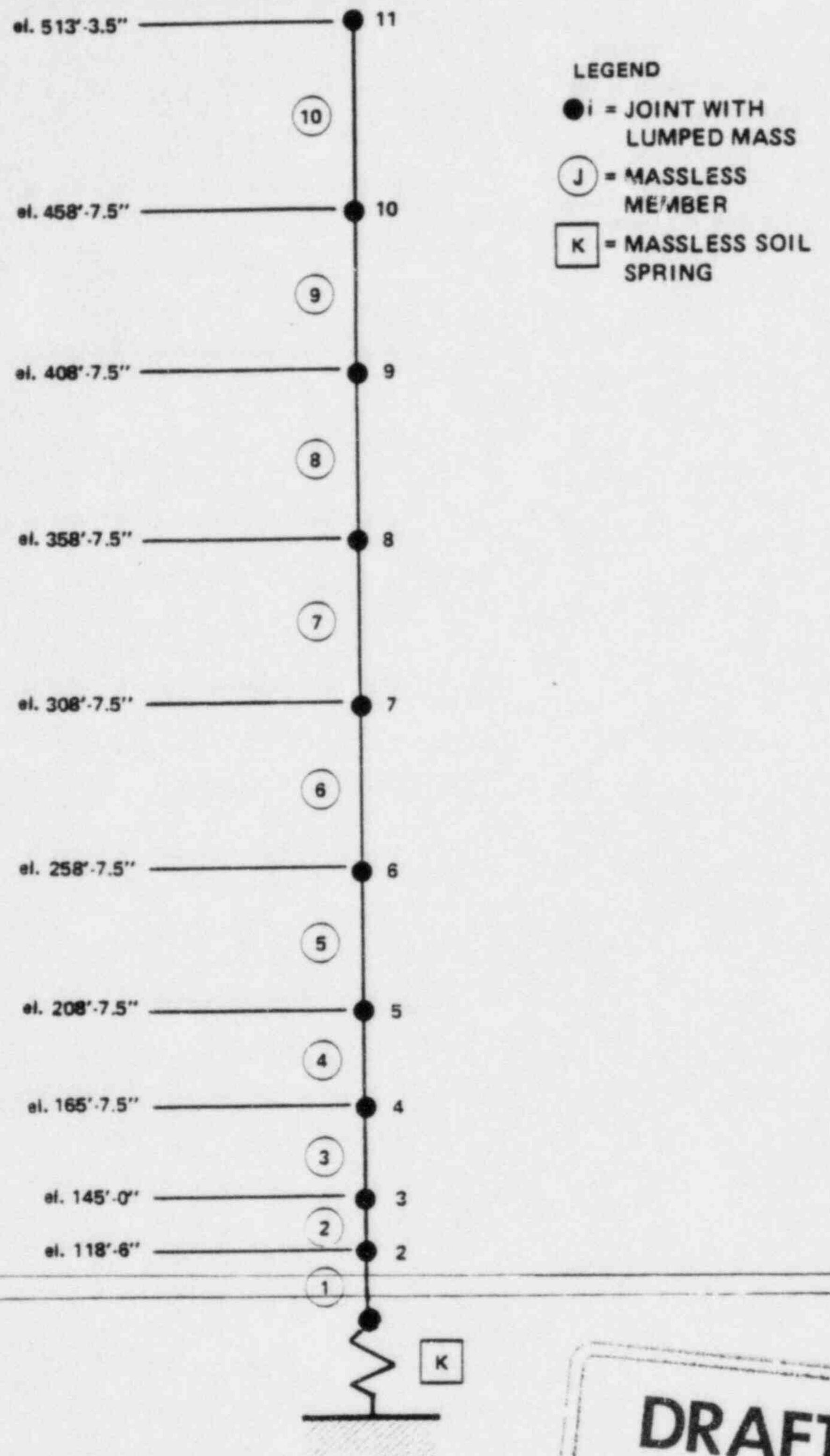
Georgia Power

EDWIN I. HATCH
NUCLEAR PLANT – UNIT 2

SEISMIC MODEL
MAIN STACK: LATERAL

FIGURE 3.7A-16

3604-0



DRAFT

REV 3 7/85

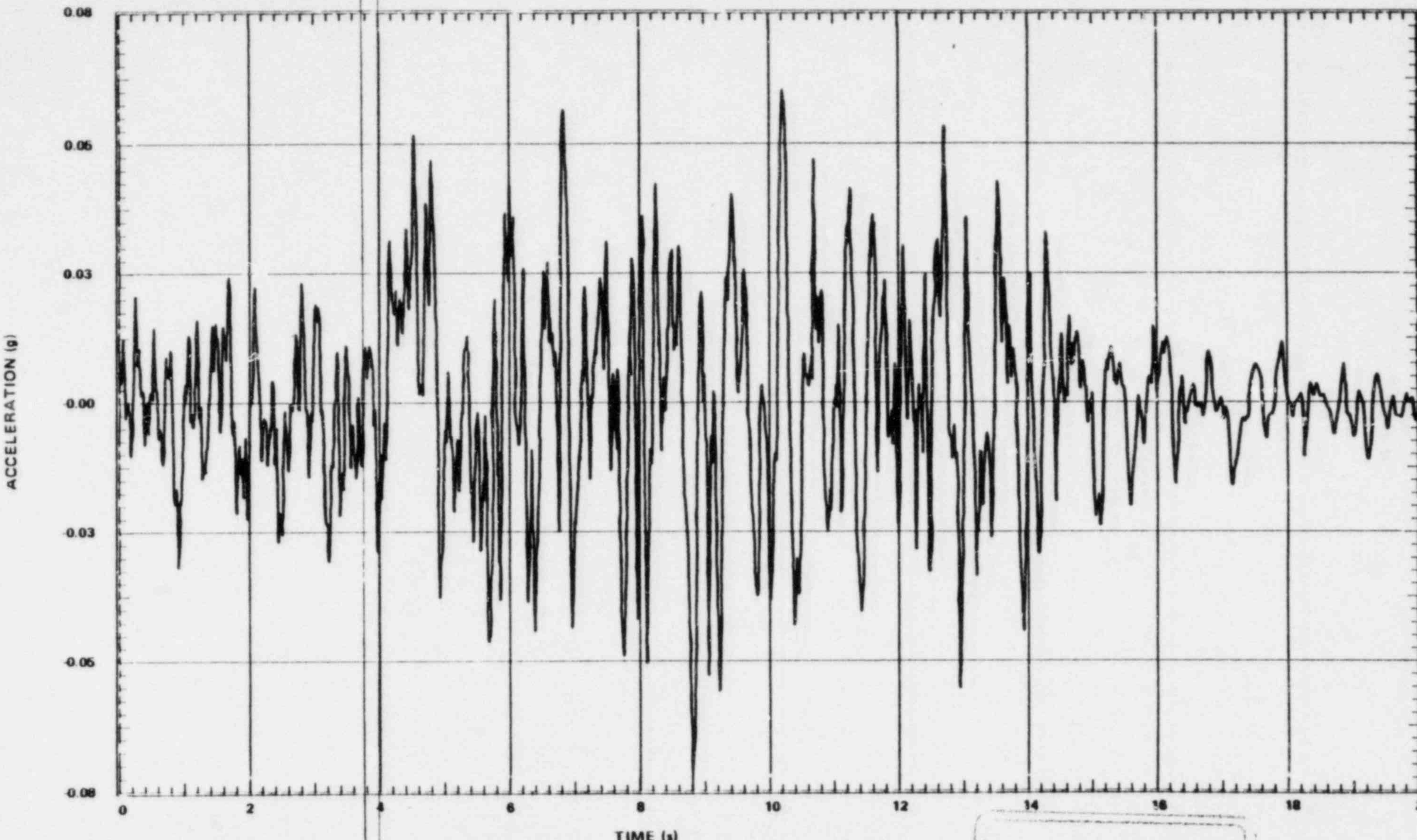
Georgia Power

EDWIN I. HATCH
NUCLEAR PLANT – UNIT 2

SEISMIC MODEL
MAIN STACK: VERTICAL

FIGURE 3.7A-17

3604-0



DRAFT

REV 3 7/85

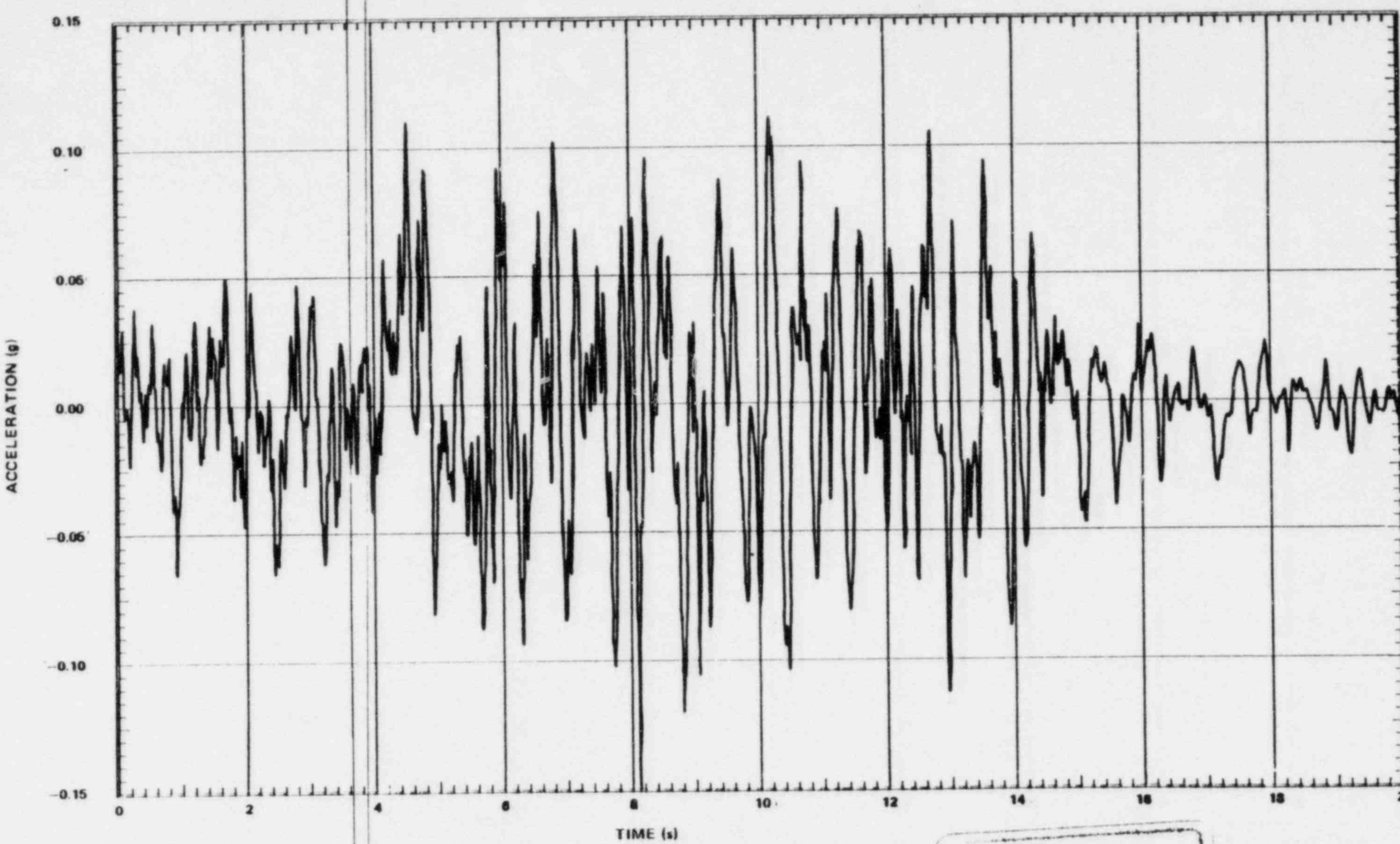
Georgia Power

EDWIN I. HATCH
NUCLEAR PLANT – UNIT 2

OBE SYNTHETIC ACCELEROGRAM (1984)

FIGURE 3.7A-18

3604-0



DRAFT

REV 3 7/85

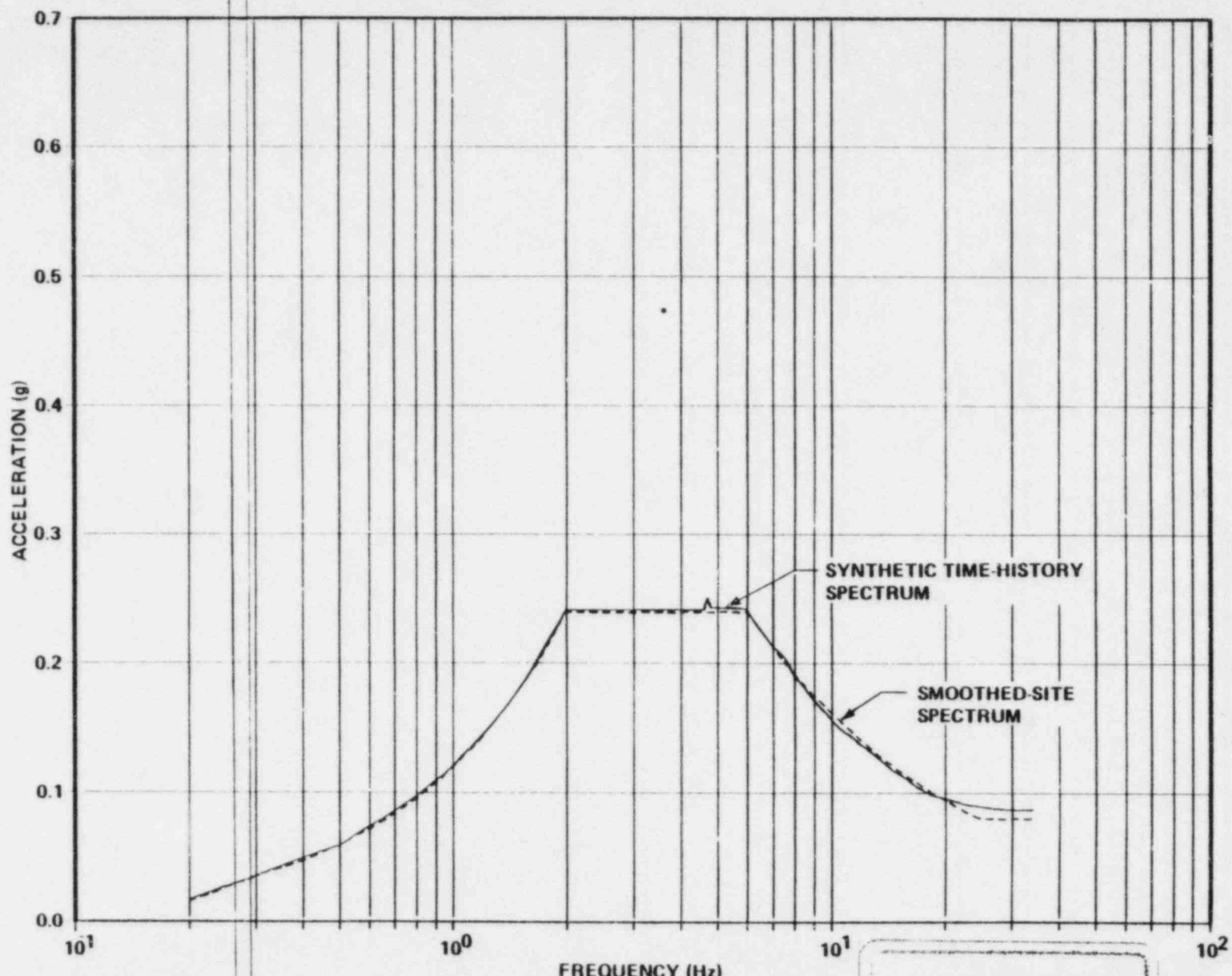
Georgia Power

EDWIN I. HATCH
NUCLEAR PLANT – UNIT 2

DBE SYNTHETIC ACCELEROGRAM (1984)

FIGURE 3.7A-19

3604-0



REV 3 7/85

DRAFT

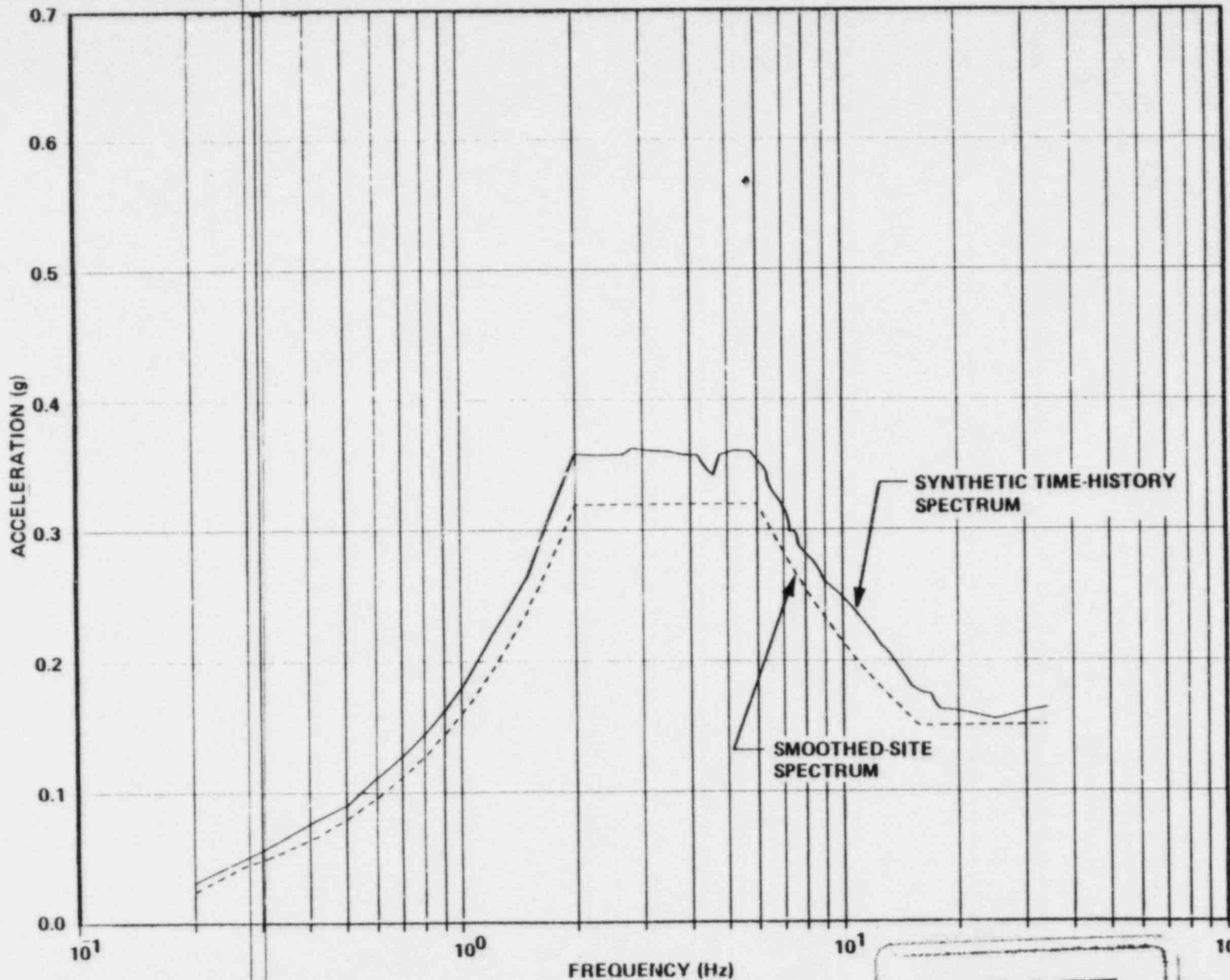
Georgia Power

EDWIN I. HATCH
NUCLEAR PLANT – UNIT 2

DESIGN SPECTRUM COMPARED WITH THE RESPONSE SPECTRUM OF THE OBE SYNTHETIC ACCELEROGRAM (1984) FOR 3% DAMPING

FIGURE 3.7A-20

3604-0



DRAFT

REV 3 7/85

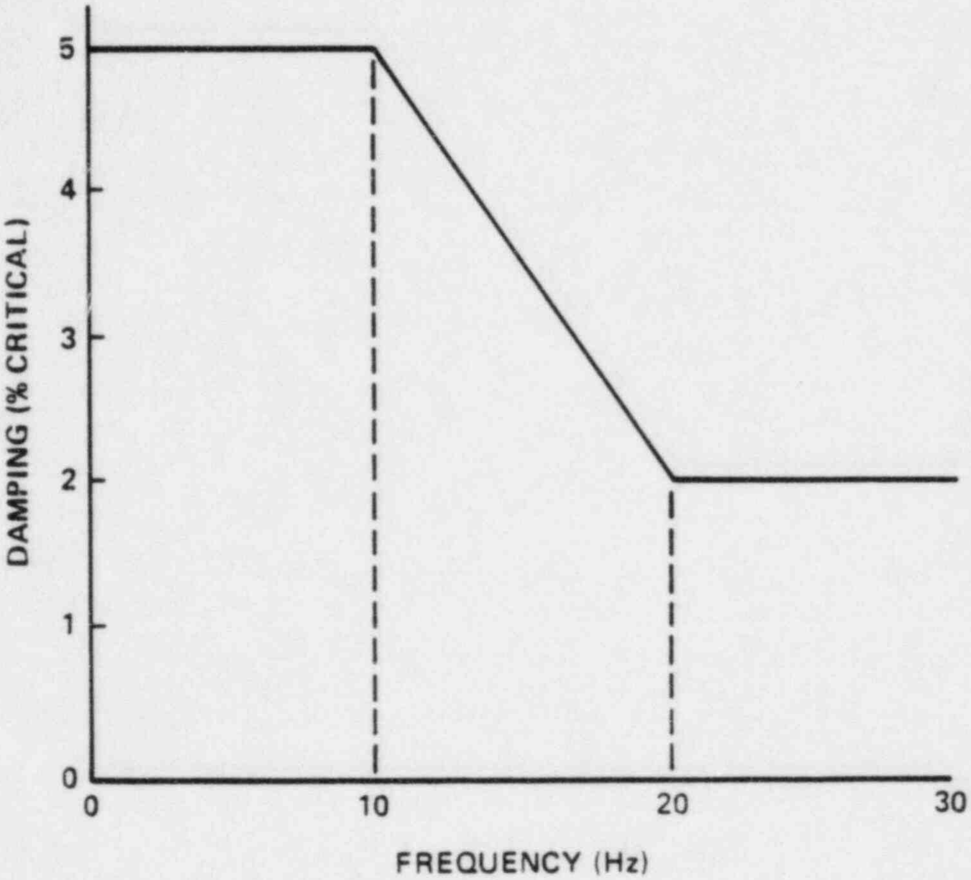
Georgia Power

EDWIN I. HATCH
NUCLEAR PLANT – UNIT 2

DESIGN SPECTRUM COMPARED WITH THE RESPONSE SPECTRUM OF THE DBE SYNTHETIC ACCELEROGRAM (1984) FOR 5% DAMPING

FIGURE 3.7A-21

3604-0



NOTES

1. Applicable to both OBE and DBE, independent of pipe diameter.
2. As of April 4, 1985, damping per this figure is used for all new and replacement systems and load reconciliation work.

DRAFT

REV 3 7/85

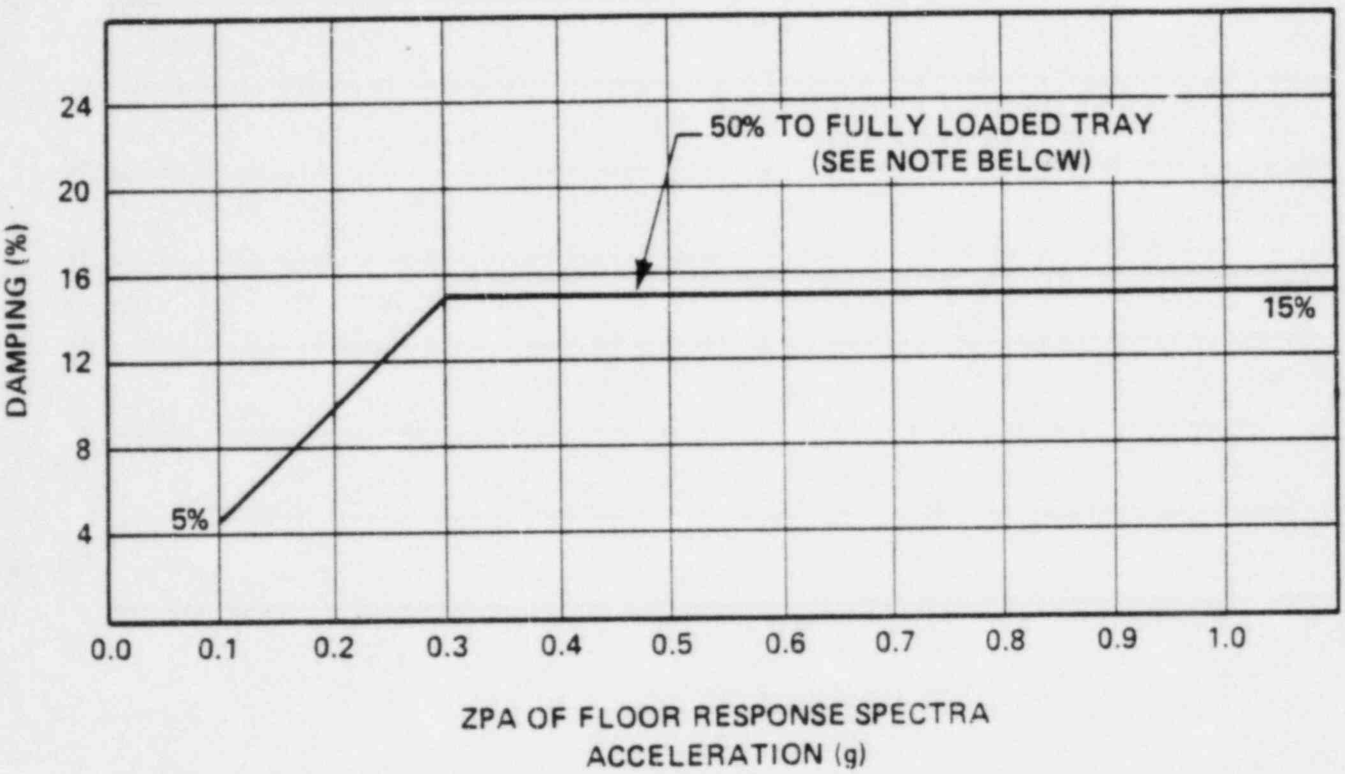
Georgia Power

EDWIN I. HATCH
NUCLEAR PLANT – UNIT 2

DAMPING CRITERIA FOR SEISMIC ANALYSIS OF PIPING SYSTEMS

FIGURE 3.7A-22

3604-0



NOTES:

1. For unloaded tray, use damping values specified in Table 3.7A-1 for steel structures. For tray loaded less than 50% linear, interpolation is used to determine the applicable design damping value.

2. As of April 4, 1985, damping per this figure is used for all new and replacement systems and load reconciliation work.

DRAFT

REV 3 7/85

Georgia Power

EDWIN I. HATCH
NUCLEAR PLANT – UNIT 2

DAMPING CRITERIA FOR SEISMIC
ANALYSIS OF CABLE TRAY SUPPORTS

FIGURE 3.7A-23

3604-0

DRAFT

TABLE 3.7B-1

CRITICAL DAMPING RATIOS FOR DIFFERENT MATERIALS

Item	Percent Critical Damping OBE Condition	Percent Critical Damping DBE Condition
Reinforced concrete structures	3.0	5.0
Welded structural assemblies (equipment and supports)	2.0	3.0
Bolted or riveted structural assemblies	3.0	5.0
Vital piping systems	0.5	1.0
Drywell - building (coupled)	3.0	5.0
Suppression chamber	2.0	3.0
RPV, support, skirt, shroud head, separator, and guide tubes	2.0	3.0
Fuel	7.0	7.0
Steel frame structures	3.0	5.0
Translation and rotation of soil	4.0	5.0

NOTES:

1. Other values may be used if they are indicated to be reliable by experiement or study.

2. As of April 4, 1985, damping per figures 3.7A-22 and 3.7A-23 for piping systems and cable tray supports, respectively, is used for all new and replacement systems and load reconciliation work.

D. State and Local Building Codes

Southern Standard Building Code (SSBC), 1969 Edition

E. Nuclear Regulatory Commission (NRC), Regulatory Guides, General Design Criteria (GDC), Industry Standards and Specifications.

1. NRC Regulatory Guides (Compliance is discussed in Appendix A.)

   Regulatory Guide 1.11 - "Instrument Lines Penetrating Primary Reactor Containment," (March 1971)

   Regulatory Guide 1.29 - "Seismic Design Classification," (August 1973)

   Regulatory Guide 1.46 - "Protection Against Pipe Whip Inside Containment," (May 1973)

   Regulatory Guide 1.54 - "Quality Assurance Requirements for Protective Coatings Applied to Water-Cooled Nuclear Power Plants," (June 1973)

   Regulatory Guide 1.57 - "Design Limits and Loading Combinations for Metal Primary Reactor Containment System Components," (June 1973)

   Regulatory Guide 1.59 - "Design Basis Floods for Nuclear Power Plants," (August 1973)

   Regulatory Guide 1.63 - "Electric Penetration Assemblies in Containment Structures for Water-Cooled Nuclear Power Plants," (October 1973)

   Regulatory Guide 1.64 - "Quality Assurance Program Requirements for the Design of Nuclear Power Plants," (October 1973)

2. General Design Criteria of 10 CFR 50 (Compliance is discussed in section 3.1.)

US Army Corps of Engineers Regulations with Respect to Dredging and Construction

American Society of Civil Engineers Paper 3269 for Wind Design Requirements [20]

American Iron and Steel Institute Specification for the Design of Light Gauge Cold-Formed Structural Members, 1968

Code of Federal Regulations, Title 29, Chapter XVII, Occupational Safety and Health Standards

NRC Regulatory Guides; compliance is discussed in appendix A.

Regulatory Guide 1.10 - "Mechanical (Cadweld) Splices in Reinforcing Concrete Structures," (January 1973)

Regulatory Guide 1.29 - "Seismic Design Classification" (August 1973)

Regulatory Guide 1.54 - "Quality Assurance requirements for Protective Coatings Applied to Water-Cooled Nuclear Power Plants," (June 1973)

Regulatory Guide 1.55 - "Concrete Placement in Category I Structures," (June 1973)

Regulatory Guide 1.59 - "Design Basis Floods for Nuclear Power Plants," (August 1973)

Regulatory Guide 1.64 - "Quality Assurance Program Requirements for the Design of Nuclear Power Plants," (October 1973)

Regulatory Guide 1.69 - "Concrete Radiation Shields for Nuclear Power Plants," (December 1973)

Regulatory Guide 1.76 - "Design Basis Tornado for Nuclear Power Plants," (April 1974)

General Design Criteria of 10 CFR 50

CMAA Specifications for Electric Overhead Traveling Crane No. 70, 1970 Edition

- Indicating and alarm instruments - testing

Input frequency	Single
Input acceleration (g)	See tables 3.10-18 through 3.10-20.
Input motion	Sine beat 10 to 15 cycles/beat 2 beats/frequency 96 beats/axis
Single-axis tests	Three axes independently
TRS versus RRS	See figures 3.10-12 and 3.10-13.

3. Functional and structural verifications

   The calculated stresses in all structural elements for the panel were very low. These results indicated that the panels are capable of withstanding the prescribed seismic environment. All the indicating and alarm instruments were monitored at their normal operating mode, and no malfunctions were indicated during the tests.

<u>Recirculation Pump Trip (RPT) Breakers</u>

1. Method of qualification - testing in accordance with IEEE-344-1975

2. Summary of results

   Input motion - multifrequency sine beats spaced at one-third-octave intervals over the seismic range of 1 to 33 Hz

   Axis of test - front to back and vertical, left to right and vertical (simultaneous horizontal and vertical)

   Damping - 5 percent

   TRS versus RRS (See figure 3.10-14.)

3. Functional verifications

   The equipment was subjected to an excessive number of tests. The switchgear maintained its structural integrity, and there was no physical equipment failure. This equipment performed its intended Class 1E functions during and after the specified seismic events.

3.10.2.1.1 Seismic Design Adequacy of Supports

Analyses or tests are performed for all supports of electrical equipment and instrumentation to ensure their structural capability to withstand seismic excitation. The following bases are used in the seismic design and analysis of cable tray supports and instrument tubing supports:

A. All cable tray supports and instrument tubing supports are designed by the response spectrum method.

B. Analysis and seismic restraint measures for tray supports and tubing supports are based on combined limiting values for static load, span length, and computed seismic response.

C. All Class 1E cable tray supports are designed to meet the requirement by dynamic analysis using the appropriate seismic response spectra.

D. Maximum stress is limited to 90 percent of minimum yield.

E. The Seismic Category I instrument tubing systems are such that the allowable stresses permitted by Section III of the American Society of Mechanical Engineers (ASME) Boiler and Pressure Vessel Code are not exceeded when the tubing is subjected to the loads specified in Section 3.9 for Classes 2 and 3 piping.

For field-mounted instruments, the following is applicable:

A. The mounting structures for Seismic Category I instruments have a fundamental frequency of 20 Hz or greater.

B. The stress level in the mounting structure does not exceed the material allowable stress when subjected to the maximum acceleration level of the mounting location.

Supports are tested with equipment installed. If the equipment is inoperative during the support test, the response at the equipment-mounting location is monitored. In such a case, equipment is tested separately, and the actual input to the equipment is more conservative in amplitude and frequency content than the monitored response.

3.10.2.2 NSSS Equipment

3.10.2.2.1 Seismic Analysis

Very few of the GE-supplied Class 1E devices were completely qualified by analysis alone (table 3.10-1). Sometimes, however, besides being used for passive mechanical devices, analysis was used in combination with testing for larger assemblies containing Class 1E devices. For instance, a test might have been run to determine whether there were natural frequencies in the equipment within the critical seismic frequency range. (See IEEE 344-1971, Paragraph 3.2.2.3.1.) If the equipment was determined to be free of natural frequencies in this range, it was assumed to be rigid, and a static analysis was performed as shown in Appendix C of NEDO-10678. (See IEEE 344-1971, Paragraph 3.2.3.4.) If the equipment had natural frequencies in the critical frequency range, calculations of transmissibility and responses to varying input accelerations were performed to determine whether Class 1E devices mounted in the assembly would operate without malfunctioning.

In addition, analyses or tests have been performed for all supports of electrical and mechanical equipment and instrumentation. The requirements of the applicable paragraphs of IEEE 344-1971 are applied when conducting tests on equipment supports. In all cases, the combined stresses of the support structures are within the limits of the ASME Code Section III, Appendix XVII-2000.

The analog transmitter trip system (ATTS) instrumentation is discussed in paragraph 3.10.2.2.3.

3.10.2.2.2 Testing Procedures

Since the GE-supplied Class 1E equipment was and is used in numerous systems in many different plants under widely varying seismic requirements, the seismic qualifications tests were performed using an expected worst-case envelope of 1.5-g horizontal and 0.5-g vertical at all frequencies from 5 to 33 Hz. (The actual qualification range was 0.25 to 33 Hz, but since test-facility capability usually limited the lower frequency test to 5 Hz, a combination of test and analysis was used to assure that there were no untested resonances. A sample analysis is shown in Appendix B of NEDO-10678.) Based upon experience obtained from seismic test experience conducted on devices of various designs, sizes, and types of construction, none of these devices has a resonant frequency in the 1- to 5-Hz region, and

- To sustain the heaviest load for which the roadways have been designed

- To withstand the effects of the DBE and remain functional during normal and accident conditions

- To meet the requirements for duct banks of American Association of State Highway Officials (AASHO) H.20 truck loading. HNP-2 Class 1E underground electrical duct banks were considered to act as continuous beams on an elastic foundation.

- To withstand the effects of the design basis earthquake (DBE). The safety-related cable ducts were analyzed in accordance with supplement 3.7A.B

Safety-related cable ducts that leave one Seismic Category I structure and enter another Seismic Category I structure are designed to the following criteria:

- To withstand the effects of the DBE and remain functional during normal and accident conditions.

- To meet the design provision that the cable ducts are able to withstand the interactions between the ducts and the Seismic Category I structures. Expansion joints have been installed in the cable ducts.

Each pull box has a 4-in. drain, which is connected to a gravel pit area located beneath the pull-box structure. In addition, gaskets and covers are provided to limit rainwater from entering.

D. Primary Containment Penetration Areas

The primary containment penetration assemblies of one division are separated from the assemblies of the other division as shown in figure 8.3-11 (sheets 1 through 9). Each assembly has been assigned with different system functions as shown on figure 8.3-11 (sheet 2).

E. Intake Structure

All HNP-2 cables in the intake structure are routed in conduits. Cables of different divisions are routed in separate conduits and do not mix with any HNP-1 cables in conduit or tray.

The Seismic Category I raceway supports are designed in accordance with the requirements specified in section 3.10.2.1.1.

8.3.1.4.1.2 Cable Installation. Class 1E cables of one division are routed in a raceway system of the same division.

Non-Class 1E cables associated with Class 1E cables of a division are routed in a raceway system of the same division. The associated cables are subject to requirements placed on Class 1E cables, such as cable derating, environmental qualification, flame retardance, splicing restriction, and raceway fill.

A. Cable Derating

Ampacity rating of cables is established as published in Insulated Power Cable Engineers Association (IPCEA) P-46-426 and in accordance with the manufacturer's standards. To this basic rating, a grouping derating factor, also in accordance with IPCEA P-46-426, was applied. Whenever applicable, a load-diversity factor was taken into consideration. As a minimum, all power cables were selected using a 100-percent-load factor and continuously rated at 125 percent of the full-load current.

B. Cable Tray Fill

As a minimum requirement, cable trays for power cables are limited to a 40-percent fill by cross section. The trays for control and instrumentation cables are limited to 50-percent fill by cross section.

C. Conduit Fill

Cables are installed in conduit in accordance with the allowable percentage of conduit fill listed below.

- Conduit containing one cable - 53 percent
- Conduit containing two cables - 31 percent
- Conduit containing three or more cables - 40 percent

A.60 REGULATORY GUIDE 1.60 - DESIGN RESPONSE SPECTRA FOR SEISMIC DESIGN OF NUCLEAR POWER PLANTS (REVISION 1, DECEMBER 1973)

Conformance

The design response spectra for seismic design of HNP-2 are discussed and provided in section 3.7. The seismic design criteria for HNP-2 were established well before the advent of this guide and thus the requirements of this guide were not utilized in the design of HNP-2.

A.61 REGULATORY GUIDE 1.61 - DAMPING VALUES FOR SEISMIC DESIGN OF NUCLEAR POWER PLANTS (OCTOBER 1973)

Conformance

The design damping values used for the seismic design of HNP-2 are discussed and provided in section 3.7. The seismic design criteria for HNP-2 were established well before the advent of this guide, thus the requirements of this guide were not utilized in the design of HNP-2.

600-V cables 1/0 and above have as a minimum one cable diameter spacing between all cables in the same tray. Where smaller, 600-V cables share the same tray with those 1/0 and above, barriers are installed in the tray to ensure the spacing of the larger cables.

Power cables are secured by ty-wrap at intervals not to exceed 8 ft in horizontal trays and 4 ft in vertical trays.

8.8.3.7 Circuit Protection

All ac power feeders and ac control power feeder cables are protected by circuit breakers. (No fused protection is used for the protection of any power or control of power feeder.)

8.8.3.8 Cable Tray Supports

Cable tray supports are designed to withstand dead loads plus seismic loads. Paragraph 12.3.3.2.1.4 discusses the seismic design bases for cable tray supports.

8.8.3.9 Instrument Racks and Control Consoles

The seismic design criteria to assure the adequacy of Class 1 instrument racks and control consoles were accomplished by static analytical procedures and/or vibration testing.

Static Analysis

The static analysis included the following combination of equivalent seismic coefficients acting at the center of moment applied simultaneously in the most disadvantageous direction.

	Horizontal	Vertical
Operating basis earthquate (OBE)	0.75 g	0.07 g
Design basis earthquate (DBE)	1.50 g	0.14 g

DRAFT

Vibration Testing

The acceleration used in vibration testing of critical instrumentation to assure no loss of safeguards function exceeded the maximum accelerations expected from building motions.

The values used for vibration testing at the points of attachment are equivalent to 1.50-g horizontal and 0.50-g vertical over the frequency range of 5 to 33 Hz.

Seismic Restraints

The methods of seismic restraint include the design of the anchorage systems, welded stiffners, cross bracing, and lateral supports to the building. Stresses due to seismic forces in combination with other design stresses do not exceed the allowable design stresses and/or stiffness required.

8.8.3.10 Fire Protection and Detection Systems

In addition to the fire protection and detection measure mentioned in paragraphs 8.8.3.3 and 8.8.3.5, the Hatch Nuclear Plant Fire Hazards Analysis contains a description of fire protection and detection system.

8.8.4 SAFETY EVALUATION

All cables have adequate flame resistant properties and are designed to resist radiation, high temperature, and high-humidity levels in the area in which they are installed. Power and control cables to safeguard equipment within the primary containment are designed to withstand the environmental conditions caused by an accident. The current-carrying capacity of all power cables is conservatively calculated to preclude thermal overload. Intermixing of power, control, and instrumentation cables in raceways or in penetrations is not permitted. Cables of redundant circuits are physically separated by means of space, fire barriers, concrete walls or floors to assure maximum independence of redundant channels. Cables are installed in either conduits or cable trays.

8.8.5 INSPECTION AND TESTING

Inspection and testing at the vendor factories and initial system tests were conducted to ensure that all cable material and completed cables are operational within their design rating. Periodic tests are conducted on the cables after they

12.3 STRUCTURAL DESIGN BASES

12.3.1 GENERAL

Certain plant structures must remain functional and/or protect vital equipment and systems, both during and following the most severe natural phenomena. These conditions are considered in the design and are investigated and defined in chapter 2, Site and Environment. Required combinations of environmental events, normal operating loads, and design accident loads for the structures are given in section 12.4.

Structures are designed in accordance with applicable codes for dead loads, live loads, seismic loads, and wind loads. Loading conditions and combinations thereof are determined by the function of the structure and its importance in meeting the plant safety and power generation objectives.

12.3.2 DEAD AND LIVE LOADS

All structures in the power plant are designed for the dead loads and live loads to which the structures are subjected. The live loads that have been used in the design of structures are given in table 12.3-1.

12.3.3 SEISMIC LOADS

12.3.3.1 Seismic Classification of Structures

12.3.3.1.1 Class 1 Structures

Class 1 structures are those whose failure might cause or increase the severity of a design basis accident (DBA) which would endanger the public health and safety. This category includes the structures and equipment required for safe shutdown and isolation of the reactor.

The following are Class 1 structures (Class 1 systems and equipment are listed in appendix A.):

- Primary containment structure
- Reactor building
- Spent-fuel pool

- New-fuel storage vault
- Diesel generator building
- Control building
- Intake structure
- Main stack
- Structures supporting or housing Class 1 equipment:
  - Wall around condensate storage tank (CST)
  - Liquid nitrogen storage tank and foundation
  - Diesel generator fuel oil storage tanks

12.3.3.1.2 Class 2 Structures

This class includes those structures which are important for reactor operation but are not essential in mitigation of the consequences of accidents. The failure of Class 2 structures may interrupt power generation.

A Class 2 designated structure does not degrade the integrity of any structure designated as Class 1. Although a structure, as a whole, may be Class 1, less essential portions may be considered Class 2 if they are not associated with loss of function, and their failure does not render the Class 1 portion inoperable.

The following are Class 2 structures:

- Turbine building
- Radwaste building and radwaste building addition
- Circulating water system including cooling towers
- Service building
- Water treatment building
- Off-gas recombiner building
- Waste gas treatment building
- All other structures not listed as Class 1

12.3.3.2 Seismic Design Bases

Design of Class 1 structures to withstand seismic loads is based on a dynamic analysis, using ground response spectrum curves developed for the plant site and described in chapter 2. Class 1 structures are analyzed for the following magnitudes of ground accelerations:

A. Operating basis earthquake (OBE) considers a maximum horizontal ground acceleration of 0.08 g.

B. Design basis earthquake (DBE) considers a maximum horizontal ground acceleration of 0.15 g.

C. The vertical acceleration assumed with the OBE and DBE is equal to two-thirds of the horizontal ground acceleration.

Table 12.3-2 and figures 12.3-2 and 12.3-3 define the damping factors which are used to perform the seismic analysis.

Seismic design of Class 2 structures is based on design criteria established by the Uniform Building Code (UBC) 1967 edition. The plant is designed in accordance with the UBC Zone 1 requirements. Class 2 structures are not subjected to analysis consideration of a DBE loading since safe plant shutdown is not involved.

Class 1 to Class 2 structure interfaces are designed so that there is no functional failure of the Class 1 structures due to possible failures of Class 2 structures.

12.3.3.2.1 Seismic Design Bases for Structures, Piping, Equipment and Cable Tray Supports

Any one or a combination of the following methods have been used in the seismic analysis of Class 1 structures, piping, equipment, and cable tray supports:

- Modal analysis using either lumped- or distributed-mass models and acceleration response spectra for the points of support[a]

a. All Class 1 structures, most cable tray supports, most piping, and some equipment, including the reactor vessel, were analyzed by this method.

- Shaker table testing of prototype components with input consisting of harmonic sine beat, or similar motions compatible with the appropriate support motion[a]

- Use of conservative static coefficients in lieu of dynamic analysis[b]

During the 20- to 30-s duration of an earthquake event, strong motion is typically experienced for 4 to 6 s. Frequencies of vibration for which the response is significant are mostly in the range from 1 to 20 Hz, with the highest responses occuring within a more narrow range, usually 3 to 8 Hz. One DBE and two OBEs are considered in the design.

The number of cycles for the DBE can be estimated by multiplying 20 Hz by 6 s by one earthquake which yields 120 cycles. Similarly, the number of cycles for the OBE can be estimated by multiplying 20 Hz by 6 s by two earthquakes which yields 240 cycles. To be conservative, the following total number of loading cycles have been used in the design:

- DBE 300 cycles

- OBE 600 cycles

There is no significant dynamic coupling between the vertical and horizontal response of buildings and floor slabs; therefore, each was computed independently. The design is based on the maximum effect of vertical and horizontal responses acting concurrently.

The horizontal amplified response loadings, which are used in the seismic design of subsystems, are obtained from time-history analyses of Class 1 buildings, the drywell, the reactor pedestal and shield, and the reactor vessel to which the subsystems are attached. The results of the time-history analysis are presented in the form of acceleration response spectra for the various elevations of the structures. These horizontal accelerations, in combination with vertical acceleration spectra equal to two-thirds of horizontal ground response spectra, are used as the seismic design input for the seismic analysis of subsystems.

---

a. Some equipment, primarily electrical, was analyzed by this method.

b. Some cable tray supports, piping, and equipment were analyzed by this method.

12.3.3.2.1.1 Seismic Design Bases for Structures. See paragraph 12.6.2.1 for seismic design bases for structures.

12.3.3.2.1.2 Seismic Design Bases for Piping. Piping systems which are classified as flexible are analyzed dynamically by the use of a computer program which provides for the calculation of probable maximum stress, resulting forces, and probable maximum displacement in the piping system due to earthquake ground motion effect.

The piping system is described to the program by geometrical and physical characteristics. The earthquake effect is introduced by the applicable response spectrum curves and coded for direction. To obtain the absolute maximum effect of the earthquake, two major directions of motion are considered in the analysis. An X-Y and a Z-Y earthquake are considered separately, X and Z being the two mutually perpendicular horizontal directions and Y being the vertical direction. Although the earthquake input is two dimensional for each earthquake considered, the three-dimensional effects are obtained. Figure 12.3-1 shows an example of a lumped mass model of a piping system for seismic analysis.

The piping structure system is replaced by a lumped-mass model, and the inertia forces are induced in each mass particle. Free vibration of such a model occurs in a finite number of frequencies with particular modal shapes. The modal analysis and later synthesis allows the determination of the maximum response quantities produced in each mode and the probable maximum stress and displacement in the complex structure.

The seismic inputs to the original OBE and DBE piping system analyses were defined using the 0.5-percent and 1.0-percent damped floor response spectra, respectively. As of April 4, 1985, damping per figure 12.3-2 is used in response spectrum analyses performed for all new and replacement systems and load reconciliation work. If, as a result of using these damping values, piping supports are moved, modified, or eliminated, the expected increased piping displacements due to greater piping flexibility will be checked to assure that they can be accommodated and that there will be no adverse interaction with adjacent structures, components, and equipment. The damping criteria established by this figure are consistent with the frequency-dependent approach established by the Pressure Vessel Research Council Technical Committee on Piping Systems.[4]

The principle assumptions made in the theory of analysis are:

- Linearly elastic structure
- Simultaneous displacement of all supports, described by single time dependent function
- Lumped-mass model satisfactorily represents the structure.
- Modal synthesis is applicable.
- Rotational inertias of the masses have negligible effect on the deformation of the piping.

A dynamic analysis has been performed on each of the 2 1/2-in. and larger Seismic Class 1 pipes for which a static analysis was previously performed. This verifies that all significant dynamic modes of response have been included.

Two-in. and smaller Seismic Class 1 pipes are restrained for earthquake by installing vertical and horizontal restraints at precalculated standard spacings which have been developed to result in a piping system natural frequency which is higher than the significant frequencies in the building response.

For certain piping systems where the seismic response of the building or other structure to which the piping is attached is small, a simpler but more conservative static analysis was performed.

This method of analysis uses that portion of the computer program used for the dynamic analysis which computes the mass of the pipeline and the distribution of loads. Conservatism is obtained by assuming that the piping system is subject to an acceleration at all segments and at all frequencies equal to the maximum acceleration from the peak of the seismic response curve.

The determination of which systems would be analyzed statically was actually based on the magnitude of the seismic response of the building or other structure to which the piping is attached.

Valves which have extended operators are analyzed by applying a static coefficient in the most unfavorable direction to the mass of the operator and calculating the stresses in the structure of the valve considering the top works of the valve as a cantilevered beam. The stresses are required to be within the normal code allowable stress for the material without the usual increase for earthquake loads.

Valves with extended operators are modeled as two masses. One is on the centerline of the pipe. The other is at the center of mass of the operator so that the torsional effect of the eccentric mass is taken into account in the seismic analysis.

For Class 1 systems which are connected to Class 2 systems, the interface between the Class 1 portion and the Class 2 portion always occurs at a valve. The analysis of the Class 1 portion includes a part of the Class 2 portion to the next anchor. The integrity of all of the piping which is analyzed with the Class 1 portion is assured by the analysis. Any failure in the unanalyzed Class 2 portion will not affect the piping on the Class 1 side of the anchor. Closure of the valve which separates the Class 1 from the Class 2 prevents the escape of process fluid through the failed Class 2 piping.

Class 1 systems which are not connected to Class 2 systems are investigated to assure that they are protected from damage by failure of a Class 2 system by one or a combination of the following:

- Physical separation
- Physical barriers
- Insufficient pressure in the Class 2 system to cause pipe motion or jet impingement or flooding which would damage the Class 1 system
- The Class 2 system is analyzed and restrained to prevent earthquake from overstressing the Class 2 system (treated as Class 1).

For Seismic Class 1 buried piping, the pipe was assumed fixed at the end entering a structure and extending infinitely into the soil. The horizontal and vertical movements at the entry point, resulting from the seismic analysis of the structure, were then taken as end displacements in computing the stresses.

For any Seismic Class 1 piping extending from one structure to another, the differential movements at support points of the two structures were assumed to be completely out of phase; and thus, the piping and structural stresses were computed based on the absolute sum of the two movements. Resulting stresses when combined with other operating stresses are within allowable values given in American National Standards Institute (ANSI) B31.1.0.

The locations of seismic supports and restraints for Seismic Class 1 piping 2 1/2 in. and larger, piping system components, and equipment including snubbers and sway braces are not determined in the field. The locations are determined in the engineering office and included in the seismic analysis of the piping system.

Seismic Class 1 piping 2 in. and smaller is restrained by the field according to the design guide. The as-built drawings are then reviewed by the engineering office. Where necessary, these piping systems are dynamically analyzed using the as-built condition and modifications are made as required. The evaluations, required by Nuclear Regulatory Commission (NRC) IE Bulletin 79-14, have documented the as-built locations of Seismic Class 1 supports; and reanalyses are performed as required.

A field surveillance is conducted to ensure that the supports, restraints, etc., have been installed in the designated locations. If the as-built locations, are different from the design locations, either the locations are corrected, or the piping is re-analyzed using the as-built locations. If this analysis shows the piping to be overstressed then the restraints are relocated or restraints are added to bring the stresses within allowable stress.

12.3.3.2.1.3 Seismic Design Bases for Equipment. The Class 1 equipment is analyzed by applying a static seismic coefficient and calculating the resultant stresses in the equipment structure. Stresses are required to be within normal allowables.

Paragraph C.3.2.3, C.3.3.2, and subsection C.3.4 provide information concerning nuclear steam supply equipment. The method used to assure the adequacy for earthquake loading of Class 1 mechanical components such as pumps and heat exchangers is described in paragraph C.3.4. The components in general are required to be adequate for the specified earthquake loadings without requiring additional seismic restraint.

In one case, the residual heat removal heat exchangers, a dynamic analysis of the exchanger and its support system indicated that seismic restraint was required to prevent overloading the basic supporting steel. These restraints as well as the supporting steel were designed to resist the OBE loading without exceeding the normal allowable stresses per the American Institute of Steel Construction Code and the DBE loadings without exceeding the yield strength of the structural steel.

The static coefficients of 1.5 g and 0.14 g given in subsection C.3.4 are the values used for the design of equipment listed in table C.3-1. The actual equipment capability (which is usually considerably greater than these values) is compared with the floor response spectra. When any equipment is identified as seismically inadequate, it is modified until adequate.

All natural modes with significant seismic response are considered when evaluating equipment capability.

12.3.3.2.1.4 Seismic Design Bases for Cable Tray Supports. Cable tray supports are designed to withstand the seismic loads calculated using the floor response spectra corresponding to the locations where the supports are attached. The simultaneous application of the horizontal and vertical earthquake components creating the highest stresses are used to design the cable tray supports. Stresses are limited to allowables given in section 12.4.

In the original cable tray support analyses, the applicable damping values were established, based upon the supports' type of construction, using the values specified in table 12.3-2. As of April 4, 1985, damping per figure 12.3-3 is used for all new and replacement systems and load reconciliation work. The damping criteria specified in figure 12.3-3 provide a conservative estimate of damping for cable tray supports based on a test program.(5)

12.3.3.2.2 Dynamic Testing Procedures

A. General Electric (GE)-Supplied Equipment

Two types of tests are used in the dynamic testing of equipment. They are discussed separately below:

1. Free Vibration Test

This test is performed on equipment whose response is dominated by the fundamental mode. The critical damping ratio and fundamental frequency are determined from this test and are used to verify or supplement calculated values used in dynamic analysis of this equipment. This test is not used alone to demonstrate dynamic capability.

In this test, an initial displacement or initial velocity is imparted to the equipment. The initial displacement is introduced by forcibly displacing the equipment and then suddenly releasing the force. The initial velocity is obtained by applying an impulse. Accelerometers or strain gauges are mounted on the equipment. After first assuring that the equipment is vibrating in its primary mode, the critical damping ratio is calculated from the logartithmic decrement.

2. Forced Vibration Test

The equipment is mounted on a shake table or driven by an eccentric shaker. The critical damping ratios, resonant frequencies, and the equipment's functional capability are determined.

The critical damping ratio of the equipment is determined by applying a sinusoidal acceleration and measuring the forced response curve (amplitude vs. forcing frequency). The critical damping ratio is then calculated by using the half-power method, fitting a theoretical forced response curve through the data points, or direct reading of the resonant amplification. The vibratory motion used is such that the vibratory loads equal or exceed the seismic loads represented by the applicable floor spectra. When testing is the only method used to demonstrate functional capability of equipment, the mounting conditions are simulated; and the equipment is operated during and after the tests.

When the seismic testing is supplemented by analysis, the seismic stresses are added to those from normal and accident conditions in the appropriate loading combinations as described in appendix C in order to assure that the equipment will perform its required safety functions. Each type of equipment is examined individually to provide this assurance.

As an example of the approach required for extremely complicated geometrical configurations, the tests and analysis performed on the high-pressure coolant injection turbine are summarized below.

The major structure of the turbine was qualified by dynamic analysis. The turbine control unit components were qualified by dynamic testing on a shake table with electrical and hydraulic systems functional. The actual mounting brackets were simulated in the test mounting. Vibration in all three perpendicular axes (two horizontal and one vertical) was accomplished by orienting the equipment in three directions on a horizontal shake table. A resonant search was made from 1 to 200 Hz, and the components with substantial resonances below 33 Hz were modified before performing the functional qualification test. These modifications were applied to the standard design. This equipment was then tested with a sinusoidal input of 1.5 g and then 3.0 g for at least 30 s at each of the arbitrary frequencies of 10, 15, and 23 Hz in each of the 3 perpendicular directions, with all systems operational. Since there were no functional failures, the equipment was deemed qualified for up to 3.0-g horizontal or vertical maximum floor acceleration for all frequencies 33 Hz and below.

All Seismic Class 1 equipment has been qualified by either test or analysis.

All tests conducted, when required, used methods and procedures comparable to those in the foregoing examples.

B. Equipment Procured by Bechtel

The dynamic testing of Class 1 mechanical equipment is accomplished by any one of the following methods:

1. The equipment is subjected to a sinusoidal excitation, sweeping through input frequencies of 1 to 50 Hz. The input acceleration amplitudes for the forcing function is scaled from the appropriate response spectrum by a factor of 1/2 β where β is the estimated damping coefficient expressed as a fraction of the critical damping. The duration of the excitation is such that the equipment may be adequately excited to the accelerations shown on the response spectra.

2. The equipment is subjected to a transient sinusoidal motion synthesized by pulse exciting a group of appropriate octave filters such that the response of the shaking table and the duration of loading is a realistic and scaled response spectrum curve for the particular direction. The

(This page intentionally left blank)

REFERENCES

1. "Torus Support System and Attached Piping Evaluations for E. I. Hatch Nuclear Plant Unit 1, Mark I Containment," Docket No. 50-321, Bechtel Power Corporation August 1976; "Torus Support System and Nuclear Plant Unit 1, Mark I Containment," Addendum 1, Docket No. 50-321, Bechtel Power Corporation, April 1977.

2. "Mark I Containment Short-Term Program Safety Evaluation Report," NUREG-0408, Nuclear Regulatory Commission, December 1977.

3. "Plant Unique Analysis Report for E. I. Hatch Nuclear Plant Unit 1, Mark I Containment Long-Term Program," Revision 2, Docket No. 50-321, Bechtel Power Corporation, December 1983.

4. Welding Research Council Bulletin Number 300: Technical Position on Criteria Establishment; Technical Position on Damping Values for Piping - Interim Summary Report; Technical Position on Response Spectra Broadening, and Technical Position on Industry Practice, 1984.

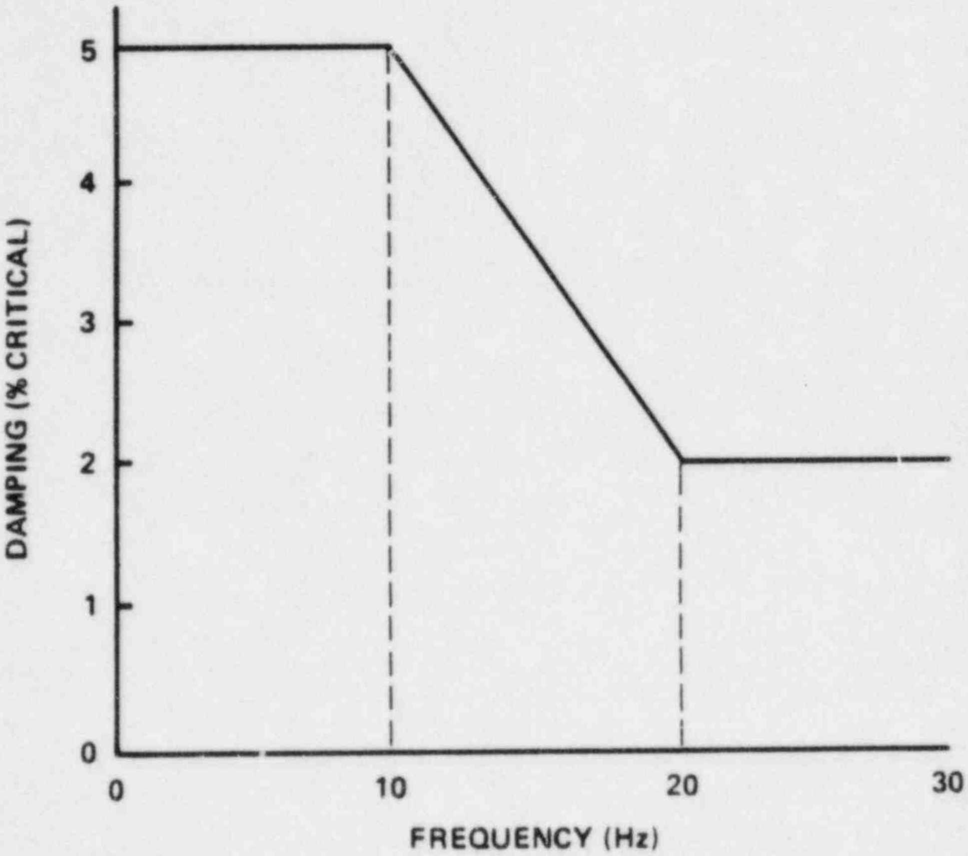
5. Cable Tray and Conduit Raceway Seismic Test Program, Release 4, Report 1053-21.1-4, ANCO Engineers, Inc., December 15, 1978.

TABLE 12.3-2

DAMPING FACTORS FOR SEISMIC ANALYSIS
IN PERCENT OF CRITICAL DAMPING[a]

	Operating Basis Earthquake	Design Basis Earthquake
Reinforced concrete structures	3.0	5.0
Steel frame structures	3.0	5.0
Bolted and riveted assemblies	3.0	5.0
Welded assemblies	2.0	3.0
Vital piping	0.5	1.0
Translation and rotation of foundation soil	4.5	5.5

a. As of April 4, 1985, damping per figure 12.3-2 for piping systems and figure 12.3-3 for cable tray supports is used for all new and replacement systems and load reconciliation work.



NOTES: 1. Applicable to both OBE and DBE, independent of pipe diameter.

2. As of April 4, 1985, damping per this figure is used for all new and replacement systems and load reconciliation work.

DRAFT

REV 3 7/85

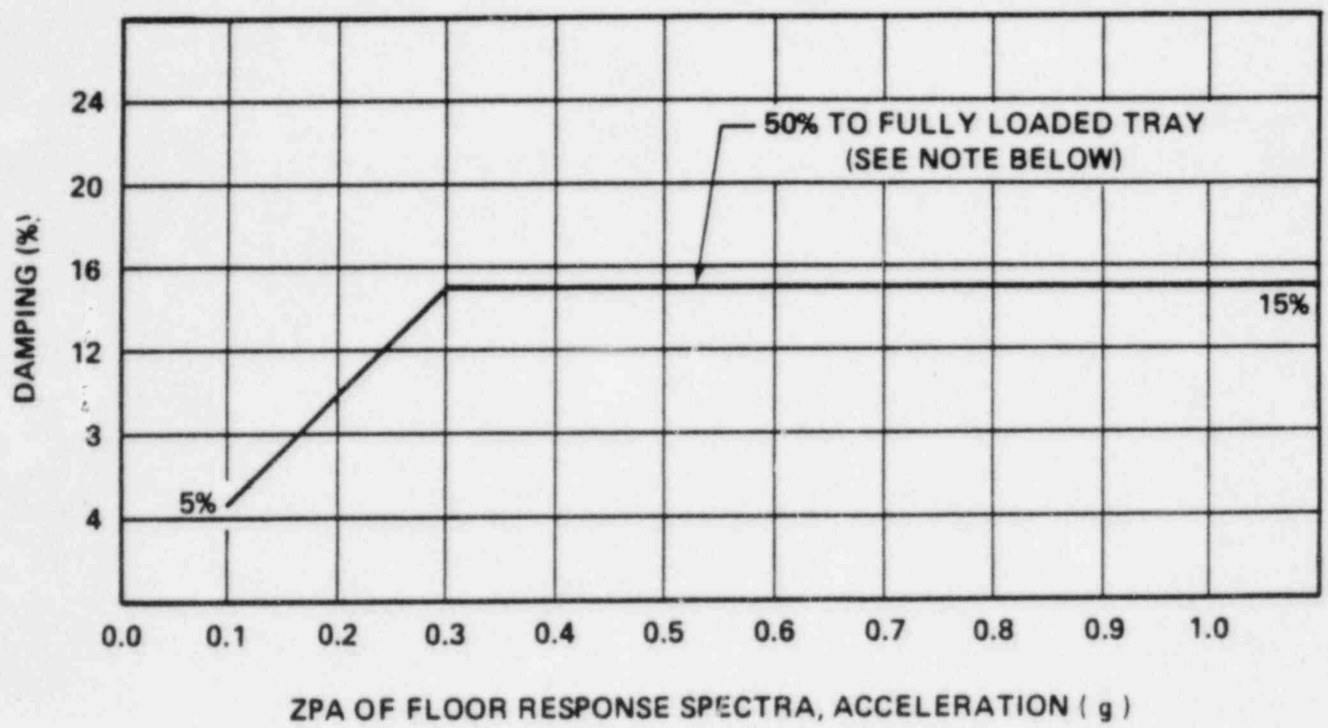
Georgia Power

EDWIN I. HATCH
NUCLEAR PLANT – UNIT 1

DAMPING CRITERIA FOR SEISMIC ANALYSIS OF PIPING SYSTEMS

FIGURE 12.3-2

3605-0



NOTES:
1. For unloaded tray, use damping values specified in Table 12.3-2 for steel structures. For tray loaded less than 50%, linear interpolation is used to determine the applicable design damping value.
2. As of April 4, 1985, damping per this figure is used for all new and replacement systems and load reconciliation work.

DRAFT

REV 3 7/85

Georgia Power

EDWIN I. HATCH
NUCLEAR PLANT – UNIT 1

DAMPING CRITERIA FOR SEISMIC ANALYSIS OF CABLE TRAY SUPPORTS

FIGURE 12.3-3

12.6 ANALYSIS OF SEISMIC CLASS 1 STRUCTURES

12.6.1 SCOPE

The loads, loading combinations, and allowable limits described here apply only to Seismic Class 1 structures. The criteria are intended to supplement applicable industry design codes where necessary to provide design safety margins for rare events like postulated loss-of-coolant accident or earthquakes or tornadoes.

The Seismic Class 1 concrete and steel structures are designed considering 3 inter-related primary functions for the design loading combinations described in subsection 12.4. The first consideration is to provide structural strength equal to or greater than that required to sustain the combination of design loads and provide protection to other Seismic Class 1 structures and conponents. The second consideration is to maintain structural deformations within such limits that Seismic Class 1 components and/or systems will not experience a loss of function. The third consideration is to limit excessive containment leakage by preventing excessive deformation and cracking where containment integrity is required.

12.6.2 STRUCTURAL ANALYSIS

In general, the structural analysis is performed utilizing the "working stress design" method as defined in American Concrete Institute (ACI) Standard Building Code Requirements for Reinforced Concrete (ACI 318-63), and in the American Institute of Steel Construction (AISC) Manual of Steel Construction (1963). Finite element stress analysis and other techniques are also used where applicable or necessary.

Load combinations and allowable limits on stresses are discussed in section 12.4. The maximum permissible calculated concrete compression is limited to 0.75 f'_c, and the maximum permissible calculated main reinforcing steel tension is limited to 0.9 F_y. The maximum permissible calculated concrete shear is as given in ACI 318-63, Chapter 17, for loadings involving R and E'.

Bond and anchorage for reinforcing steel is treated as required by ACI 318-63.

12.6.2.1 Seismic Analysis of Structures

12.6.2.1.1 Generation of Seismic Responses for Design

The method used in the seismic dynamic analysis consists of the following four steps:

- Formulation of the mathematical model of the structure or structures to be analyzed
- Determination of natural frequencies, mode shapes, and damping values
- Finding the spectral acceleration (g) levels from the ground response spectra curves
- Determination of the response of the structure to the earthquake in terms of acceleration, moments, shears, and displacements

The mathematical model of the structure consists of lumped masses and weightless springs. At appropriate locations within the building, points are chosen to lump the mass of the structure. Between these locations, properties are calculated for moments of inertia, cross-sectional areas, and effective shear areas. These properties of the model are used in a computer program to obtain either the flexibility or inversely the stiffness properties of the building.

The mass of the structure is equally distributed to any two adjacent mass points. The masses lumped at any particular location include the mass of the building, the mass of the floor, and the masses of the equipment which are considered to be large enough to affect the response of the coupled system.

Soil and structural material properties and the bases for selection of these properties are listed in table 12.6-3. The dynamic analysis of all Class 1 structures includes the effects of the elasticity of the foundation material. Soil-structure interaction was based upon the elastic half-space theory.

The natural frequencies and mode shapes of the structures are obtained by computer programs. For example, some of the computer programs use the flexibility coefficients and lumped mass of the model. The flexibility coefficients are formulated into a matrix and inverted to form a stiffness matrix. The technique of diagonalization by successive rotations is used to obtain the natural frequencies and mode shapes. Appropriate damping values of individual materials used are presented in table 12.3-2.

The basic description of the earthquake is provided by spectrum response curves. Separate curves are used for the operating basis earthquake (OBE) of 0.08-g horizontal acceleration and the design basis earthquake (DBE) of 0.15-g horizontal acceleration. These curves are presented in figures 2.5-2 and 2.5-3. The response of the structure to the earthquake is obtained by using the spectrum response technique. Appropriate acceleration levels are read from the earthquake spectrum curve corresponding to the natural frequencies of the structure.

The mode shapes, lumped weights, and associated earthquake ground response spectrum acceleration levels are used to calculate modal responses for a given mode using standard spectrum response techniques. The total seismic response value R of interest (i.e., inertia forces, shears, moments, displacements, or accelerations) for a given earthquake component is obtained by combining the individual modal responses at a given location by the square-root-of-the-sum-of-the-squares (SRSSs) method. For example, the total response value R at mass point i is calculated using the following equation:

$$R_i = \left[\sum_{j=i}^{m} R_{ij}^2 \right]^{1/2}$$

where:

R_i = total response value of interest acting at mass point i, for a given earthquake component where the response value can be either force, shear, moment displacement, or acceleration

m = number of modes considered

R_{ij} = response value for mass point i due to mode j

All significant modes, including closely spaced modes, of the structural system are used for obtaining the total response. The only closely spaced modes (i.e., successive modes within 10 percent of each other) identified in the various analyses were the eighth and ninth modes calculated in the north-south analysis of the reactor building and internals, and the fourth and fifth and eight and ninth modes calculated in the east-west analysis of the same structure. Since these closely spaced modes contributed little to the total response of the structure, the use of the SRSSs approach to calculate total structural response was acceptable.

Table 12.6-1 lists frequency values obtained from the dynamic analyses of the reactor building, the control building, the diesel generator building, the intake structure, and the main stack.

Figure 12.6-1 shows the mathematical model used for the seismic analysis of the coupled system of the reactor building, reactor vessel pedestal with reactor shield, and the reactor vessel. The seismic moments and shears obtained from the analysis were used for the structural design of the buildings with particular emphasis on the seismic overturning, connections of the members, and arrangement of the reinforcing in the concrete. Figure 12.6-2 shows moments, shears, displacements, and accelerations for the reactor building which were used in the original design. These values were checked from time to time to evaluate the effects of the changes associated with the design development of the project, and to assure that the design values used were always conservative.

The torsional effect induced by the rotational component of the ground motion and/or the unsymmetric nature of the building was compensated for by considering a static torsional moment acting at the elevation under consideration. The magnitude of this moment is taken as the sum of the individual products of the inertia force and the eccentricity between the center of rigidity at the level of interest and the center of gravity of the mass points above that elevation.

Where uncertainties in the applicability of the elastic half-space theory or in the interpretation of the geophysical test data indicated the possibilities of significant variations from calculated frequencies, parametric analyses were made to encompass a ±50-percent range of expected values, and the worst cases were used for design. Also, the floor response spectra (FRS) for analysis of Class 1 equipment were conservatively plotted as smoothed upper envelopes of the calculated raw curve with peaks widened at least ±10 percent on each side of the expected peak frequency. Finally, the use of the smoothed-response spectra given in figures 2.5-2 and 2.5-3 preclude the possibility of serious errors resulting from expected variations between the true and calculated building frequencies.

The seismic loading on Class 1 equipment is computed in the same manner as that for the buildings except that the response spectrum for the appropriate floor or support is used instead of the ground response spectrum.

The seismic analysis of each Class 1 structure is documented in a report, including the time-history analysis and floor response spectra for analysis of Class 1 equipment. These reports are made available to all design organizations who prepare specifications for Class 1 equipment.

12.6.2.1.2 Generation of Original Floor Response Spectra

This section provides a discussion of the methodology used to develop the FRS that were used for seismic qualification of subsystems until April 4, 1985. Paragraph 12.6.2.1.3 provides a similar discussion for the development of a new set of FRS that are used, as of April 4, 1985, for subsystem seismic qualification.

The FRS were generated for inclusion in the appropriate equipment specifications and for use in subsystem design. Figure 12.6-3 shows the FRS for the reactor building floor at el 228 ft.

The FRS were generated for the OBE; FRS for the DBE were obtained by scaling up the OBE spectra in proportion to the DBE versus OBE results obtained by the response spectrum analysis. For example, the scalars for the 22 mass point reactor building model varied from 1.58 to 1.81, with an average of 1.65. A uniform scalar of 1.7 was used for all reactor building floors and all damping values.

Figure 12.6-8 shows a comparison of the smoothed-site spectra with the raw spectra developed at a maximum frequency interval of 1 Hz for the scaled 1940 north-south El Centro record. The curves are for 3 percent and 5 percent of critical damping which were generally used for the OBE and DBE analyses. Table 12.6-2 shows a comparison of maximum seismic accelerations at the 22 mass points of the reactor building model as computed by the response spectrum and time-history methods. The time-history method shows higher accelerations, because the El Centro ground spectrum is substantially above the smoothed-site spectrum. As is evident from these comparisons, the time-history analysis resulted in a substantially higher building response, as compared to the response calculated from the site spectrum.

Since there is no requirement for designing the equipment to higher seismic loads than used for the supporting buildings themselves, and since reliable methods of modifying the accelerogram were not originally available, results from the time-history analysis were scaled to acceleration levels compatible with those from the spectral analysis. Let A_i be the acceleration response at the i^{th} mass point from the spectral analysis and A_i^* be the acceleration response at the same point from the time-history analysis. Then the scaling factor is defined by:

$$S_f = \frac{A_i}{A_i^*}$$

Before giving any justification for this procedure, it is noted that any motion $\ddot{Y}$ (t) may be expressed in the following form:

$$\ddot{Y}(t) = \ddot{Y}_o f(t)$$

where:

$\ddot{Y}_o$ = maximum amplitude of the motion

f(t) = time-wise variation of the motion

Since the modal superposition method is adopted in the seismic analysis of the structure, the general equation of motion of any mode i may be expressed as follows:

$$\ddot{X}_i + 2\beta_i\omega_i \dot{X}_i + \omega_i^2 X_i = - \left[\frac{\{\phi\}_i^T [M] \{\ \}}{\{\phi\}_i^T [M] \{\phi\}_i} \right] \ddot{Y}_o f(t)$$

where:

X_i = displacement at i^{th} floor relative to the ground

β_i = i^{th} modal damping coefficient

ω_i = i^{th} undamped circular natural frequency

$\{\phi\}_i$ - i^{th} natural mode

$\{\phi\}_i^T$ = transpose of $\{\phi\}_i$

{e} = unit vector

[M] = mass matrix of the structure

It is apparent that the value defined by the bracket on the right side of the equation is independent of the input motion. Let X_i^* be the response of the same system corresponding to an input motion of S Y(t) where S_f is an arbitrary scalar. Then, for a fixed damping value, there exists a linear relationship between the responses such that

$$X_i^* = S_f X_i$$

This relationship holds for every mode considered. Now consider the total response of the system. At any time instant, the total response of a multi-degree-of-freedom system may be expressed as follows:

$$\{X\} = \ddot{Y}_o \sum_{i=1}^{n} C_i \left\{\emptyset_i\right\} \frac{1}{\omega_i^2} (I.A.F.)_i$$

where:

$\{X\}$ = displacement vector

n = number of masses of the structure

C_i = i^{th} modal participation factor

$(I.A.F.)_i$ = i^{th} instantaneous amplification factor and is defined by:

$$(I.A.F.)_i = \omega_i \int_o^t f(\tau) \ \mathrm{Sin}\omega_i \ (t-\tau)d\tau$$

It is obvious that for a given system and a prescribed time-wise variation of the input motion, the terms inside of the summation sign will not be altered by varying the maximum amplitude of the input motion. Therefore, if the amplitude Y is multiplied by a factor of S_f, the response of the system is simply

$$\{X^*\} = S_f \ddot{Y}_o \sum_{i=1}^{n} C_i \ \{\emptyset_i\} \frac{1}{\omega_i^2} (I.A.F.)_i \quad \text{or}$$

$$\{X^*\} = S_f \ \{X\}$$

The important thing to note is that the amplitude of the vector X is modified by a factor of S_f, but the time-wise variation of the response is not changed. This leads to the fact that for a fixed damping value, the FRS generated from any floor time-history before and after the multiplication of the scalar factor S_f will have the same linear relationship S to each other. This indicates that the FRS generated by the scaling procedure meet the basic seismic criteria implied by the smoothed-site spectra.

Additional justification for the scaling factor (S_f) procedure, used to assure that the maximum floor accelerations from the time-history analyses were compatable with the results of the response spectrum analyses, was provided by a later evaluation. In this evaluation, the El Centro 1940 earthquake, north-south component was modified such that its resulting response spectrum envelops the smoothed-site response spectrum as shown in figure 12.6-4. This modified El Centro accelerogram was then used to generate the FRS for the identical reactor building mathematical model and member properties on which the previous time-history analysis was based. The comparison of three representative FRS for 3 percent of critical damping at the basement floor (el 87 ft), intermediate floor (el 158 ft), and top floor (el 228 ft) of the reactor building is shown in figures 12.6-5, 12.6-6, and 12.6-7, respectively. It is seen that all three FRS originally developed and used in the procurement of equipment enveloped at most frequencies the corresponding FRS generated from the modified El Centro accelerogram. The small portion of the curves generated using the modified El Centro accelerogram, which exceeds the FRS used to procure equipment, was insignificant. Therefore, it was concluded that the scaling factor procedure previously used is a justifiable one and that no further work needs to be done.

In plotting FRS, the effect of possible errors in building frequencies was accounted for by broadening the peaks, enveloping data points, or by using parametric analysis, as discussed in paragraph 12.6.2.1.1.

12.6.2.1.3 Generation of 1984 Floor Response Spectra

A review was performed in 1984 that addressed the FSAR peak-broadening requirements of the FRS, and it was concluded that no significant safety issue exists with the subsystems that were seismically qualified using the original FRS discussed in paragraph 12.6.2.1.2. In the process of performing the review, new (1984) FRS were developed to reflect the as-built condition of the structures and provide a more realistic representation of the specified seismic design environment (i.e., ground design response spectra, as given in figures 2.5-2 and 2.5-3).

The 1984 FRS are used, as of April 4, 1985, to seismically qualify subsystems. The following is a discussion of the techniques used to develop these FRS.

The time-history approach was used to generate the new FRS. Instead of increasing the OBE spectra by a factor to obtain the DBE spectra, FRS were developed separately for the OBE and the DBE. Separate synthetic time histories were developed for use in generating the OBE and DBE spectra. Figure 12.6-9 is a plot of the response spectrum of the OBE synthetic time history compared with the OBE ground design response spectrum for 3-percent critcial damping. Similarly, figure 12.6-10 is a plot of the response spectrum of the DBE synthetic time history compared with the DBE ground design response spectrum for 5-percent critical damping. Comparison of these figures with figure 12.6-8 demonstrate that the two new synthetic time histories provide a more realistic representation of the seismic ground design response spectra than does the El Centro time history used to develop the original FRS.

Since the new synthetic time histories provide a more realistic representation of the seismic ground design response spectra than does the El Centro time history, no scaling factor (S), as discussed in paragraph 12.6.2.1.2, was used.

The 1984 FRS were developed at the same mass points as the original FRS and were plotted separately for the north-south and east-west directions. In plotting FRS, the effect of possible errors in building frequencies was accounted for by broadening the peaks, enveloping data points, or by using parametric analysis, as discussed in paragraph 12.6.2.1.1. Examples of the 1984 FRS are shown in figures 12.6-11 through 12.6-14.

12.6.2.2 Tornado Analysis of Structures

Appropriate portions of the plant are designed to withstand the effects of a tornado as defined in section 12.3.

The exterior walls of the reactor building are selected as representative of the design procedure. Using a model of the building and normalized Hoecker pressure profile, suctions and airflows within the building were computed using the principles of compressible fluid flow. A maximum transient crushing and bursting pressure of 292 lb/ft^2 and 136 lb/ft^2 was computed. These were applied to the walls as uniform loads to develop moment and shear diagrams. Additionally, the exterior walls were designed for dynamic concentrated loads representing the tornado missile impacts. These loads were obtained from dynamic analysis of the walls subjected to a pulse loading. The pulse was fitted to each case (i.e., span length, thickness and missile energy) by trial and correction to satisfy energy and momentum principles. The moments and shears due to missiles were combined with those from crushing. The bursting moments and shears, or carryover moments from missile impact, if larger, were used to design the opposite face reinforcement.

In most cases, practical wall designs required a portion of the missile impact energy to be dissipated in the plastic range in the struck span. The ductility ratio as a general rule was limited to 10. This ratio in no case exceeds 20.

12.6.3 IMPLEMENTATION OF STRUCTURAL CRITERIA

This subsection illustrates the loads and load combinations and structural static and dynamic analysis used in the structural design of Seismic Class 1 structures and briefly discusses typical structural elements of the reactor building and summarizes the actual stresses in these elements.

Design procedures used for the reactor building were also used for the other Seismic Class 1 structures, such as the diesel generator building, the control building, and intake structure. The main stack is designed to meet design criteria of Seismic Class 1 structures except for tornado loading.

12.6.3.1 Reactor Building Floor System

The reactor building floor system consists of variable slab thickness on permanent cold formed steel decking supported by composite steel beams girders and columns. Interior and exterior walls above grade are generally nonload bearing.

The floor system is designed to support dead loads, equipment loads, laydown loads, piping loads, live loads (table 12.3-1), and vertical seismic loads based on DL + .25 LL. Additionally, exterior panels are designed to resist moments due to the tornado load on adjacent exterior walls and the horizontal shear due to lateral seismic forces. Slabs are thickened locally to provide radiation shielding.

The slabs range in thickness from 12 in. to 54 in. and are generally designed as one-way continuous slabs in accordance with ACI 318-63. In thick slabs, where temperature reinforcement exceeded that for stresses, 0.18-percent rebar was placed each way in each face.

Most of the steel beams and girders are designed as composite sections in accordance with American Institute of Steel Construction using 7/8-in.-diameter shear studs. While most of the supporting columns are encased in concrete, they were conservatively designed as unencased. Also, the customary live-load reduction for lower story columns was neglected.

The structural steel frame was generally designed only for vertical loads since the concrete shear walls provide lateral resistance. However, exterior columns were checked for stability in the deflected configuration that would result from

DRAFT

TABLE 12.6-1

NATURAL FREQUENCIES OF STRUCTURES
(Hz) FOR EAST-WEST DIRECTION

Mode No.	Reactor Building	Control Building	Diesel Generator Building[a]	Intake Structure	Main Stack
1	0.67	1.01	4.12	7.04	0.60
2	3.21	5.38	7.76	21.13	2.24
3	4.25	7.00	36.20	35.32	4.88
4	6.66	11.07		44.41	8.14
5	7.54	15.27	-	53.74	11.66
6	10.47	22.19	-	69.32	15.22
7	14.95	30.70	-	-	18.57
8	20.07	40.06	-	-	21.26
9	21.57	-	-	-	24.52
10	27.44	-	-	-	26.63
11	-	-	-	-	31.36

a. The diesel generator building natural frequencies are those associated with the mean soil properties for this building.

TABLE 12.6-3 (SHEET 1 OF 2)

SOIL AND STRUCTURAL MATERIALS PROPERTIES

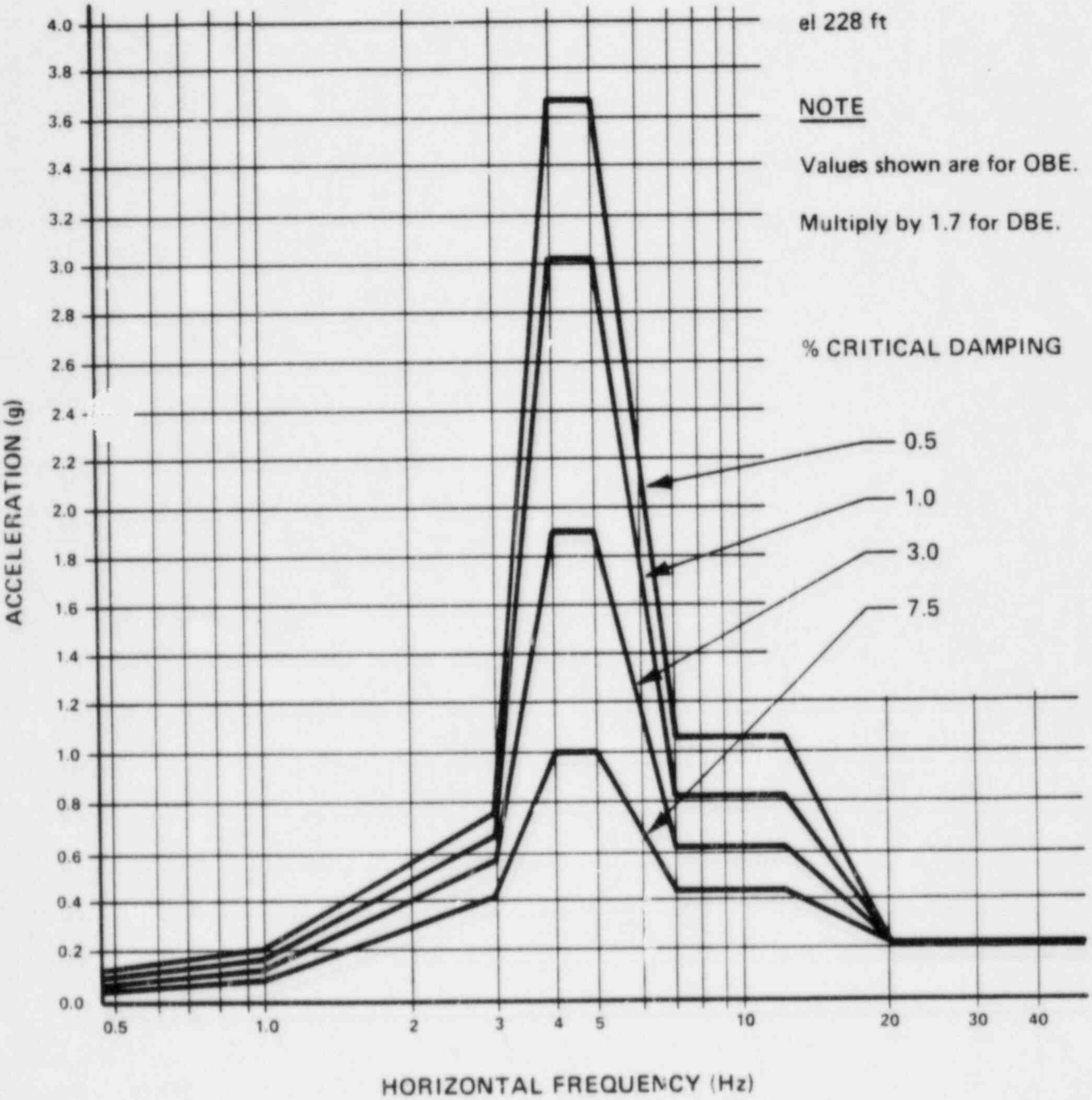
Material	Use or Location	Properties	Bases
Foundation soils	Reactor building	Shear wave velocity 2450 ft/s Compression wave velocity 6600 ft/s	From site geophysical testing by Law Engineering Company
		Poissons ratio 0.42 Young's modulus 66,200 ksf Shear modulus 23,300 ksf	Computed by elastic theory from Vs and Vc
		Unit mass 0.0039 k-s²/f	Assumed
		Horizontal spring 1.04×10^{7} kpf	Computed from elastic half-space theory with equations presented by Barkan and Parmalee
		Rocking spring 7.2×10^{10} kf/rad Damping	Table 12.3-2

The properties listed above were also computed for all other Class 1 and adjacent Class 2 structures. Properties varied due to location depth and geometry of the foundation.

DRAFT

HNP-1-FSAR-12

REV 1 7/83
REV 3 - '85



DRAFT

REV 3 7/85

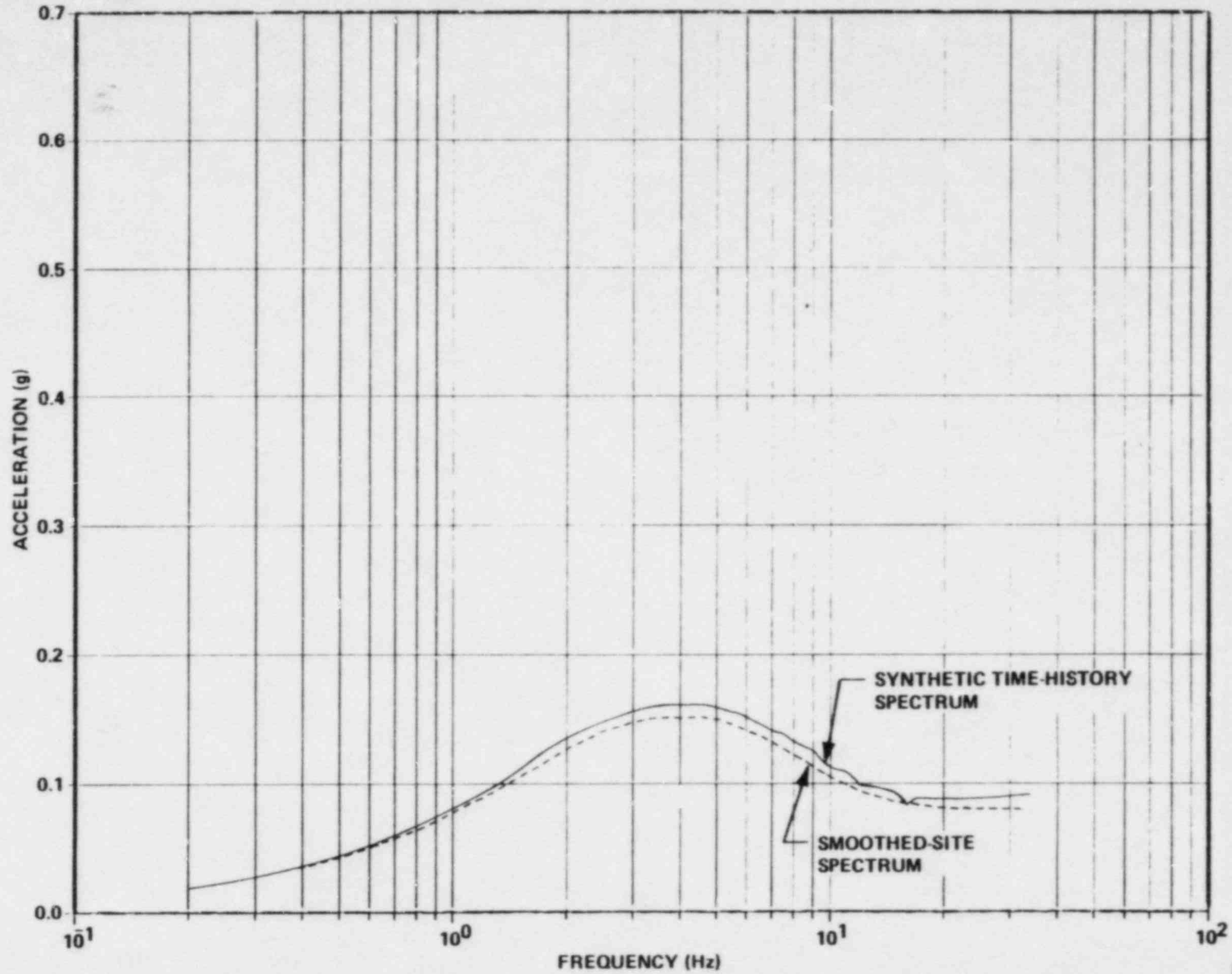
Georgia Power

EDWIN I. HATCH
NUCLEAR PLANT – UNIT 1

REACTOR BUILDING RESPONSE SPECTRA
FOR FLOOR AT el 228 ft

FIGURE 12.6-3

3604-0



DRAFT

REV 3 7/85

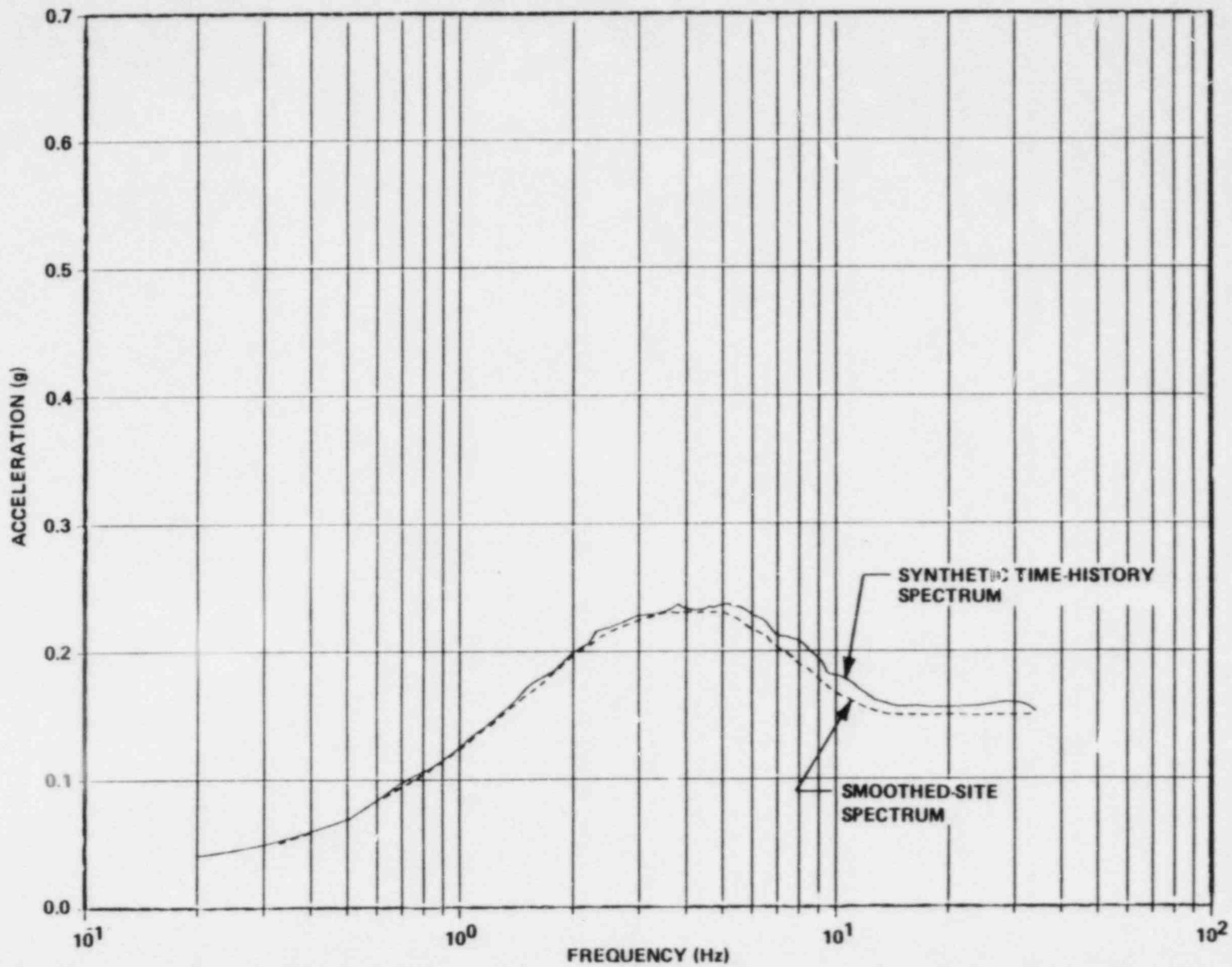
Georgia Power

EDWIN I. HATCH
NUCLEAR PLANT – UNIT 1

COMPARISON OF OBE SYNTHETIC TIME-HISTORY RESPONSE SPECTRUM WITH SMOOTHED-SITE RESPONSE SPECTRUM AT 3% CRITICAL DAMPING

FIGURE 12.6-9

3604-0

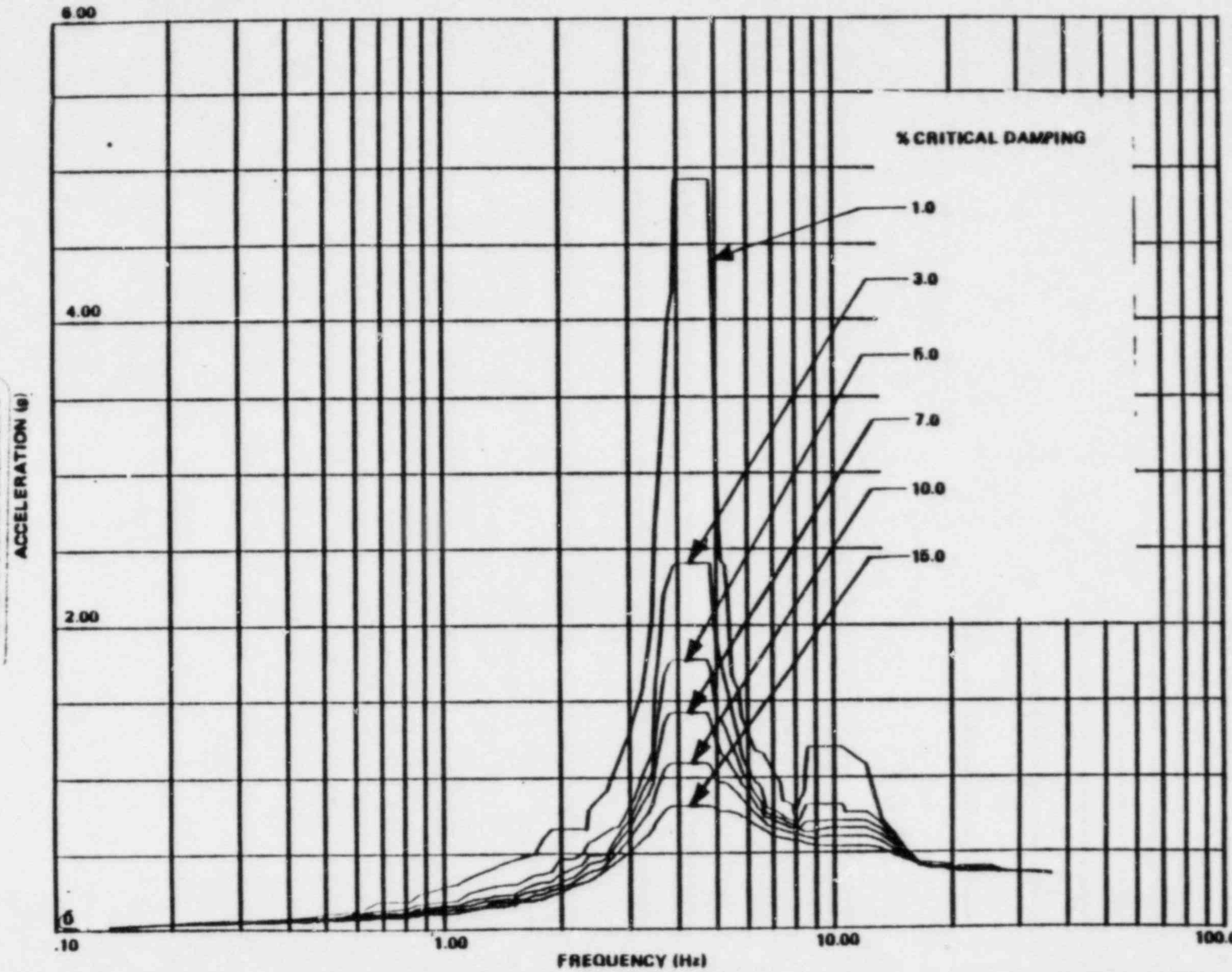


DRAFT

REV 3 7/85

Georgia Power EDWIN I. HATCH NUCLEAR PLANT -- UNIT 1	COMPARISON OF DBE SYNTHETIC TIME-HISTORY RESPONSE SPECTRUM WITH SMOOTHED-SITE RESPONSE SPECTRUM AT 5% CRITICAL DAMPING FIGURE 12.6-10

3604-0



DRAFT

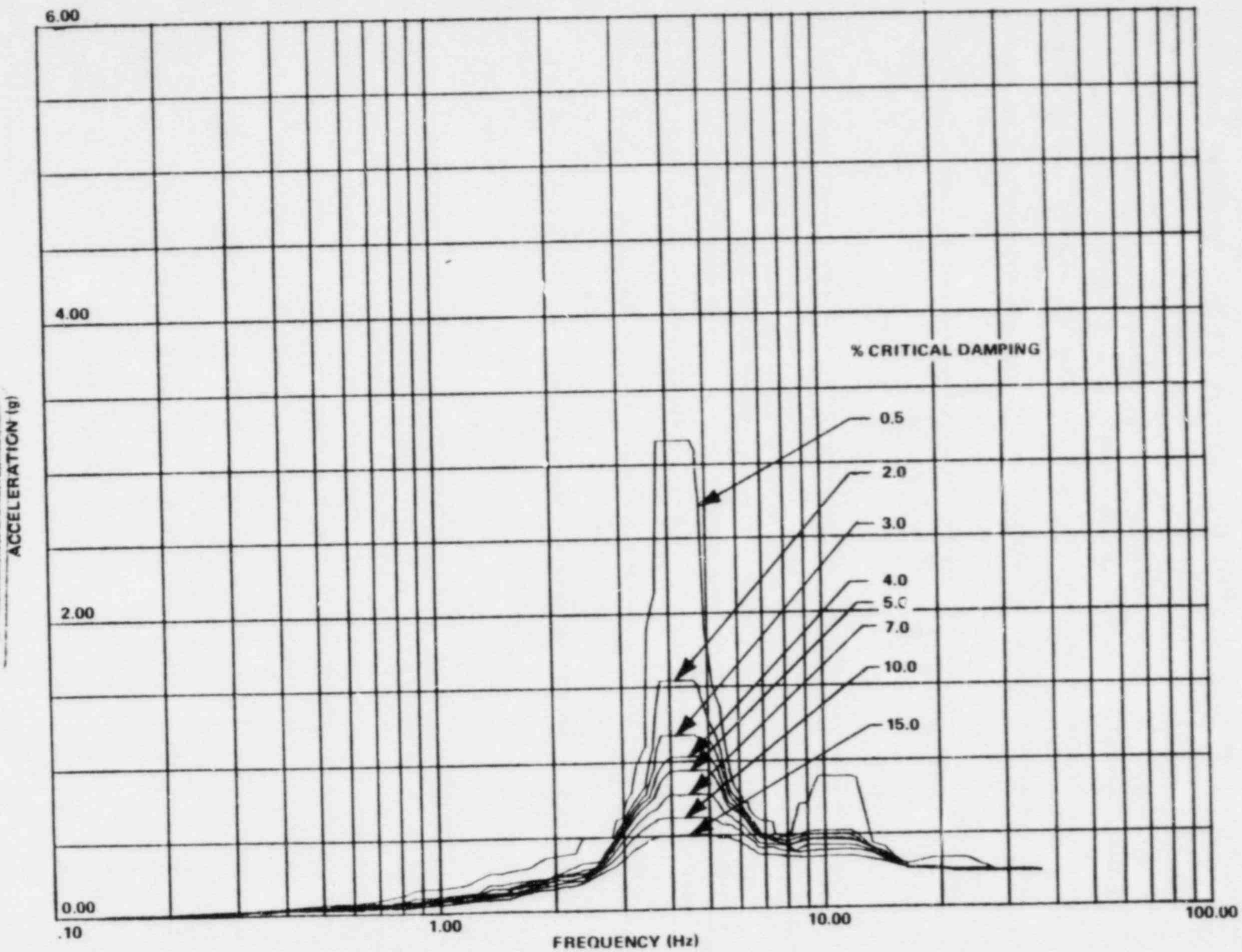
REV 3 7/85

Georgia Power

EDWIN I. HATCH
NUCLEAR PLANT – UNIT 1

REACTOR BUILDING 1984 RESPONSE SPECTRA DBE (N-S) FOR FLOOR AT el 228 ft

FIGURE 12.6-11



DRAFT

REV 3 7/85

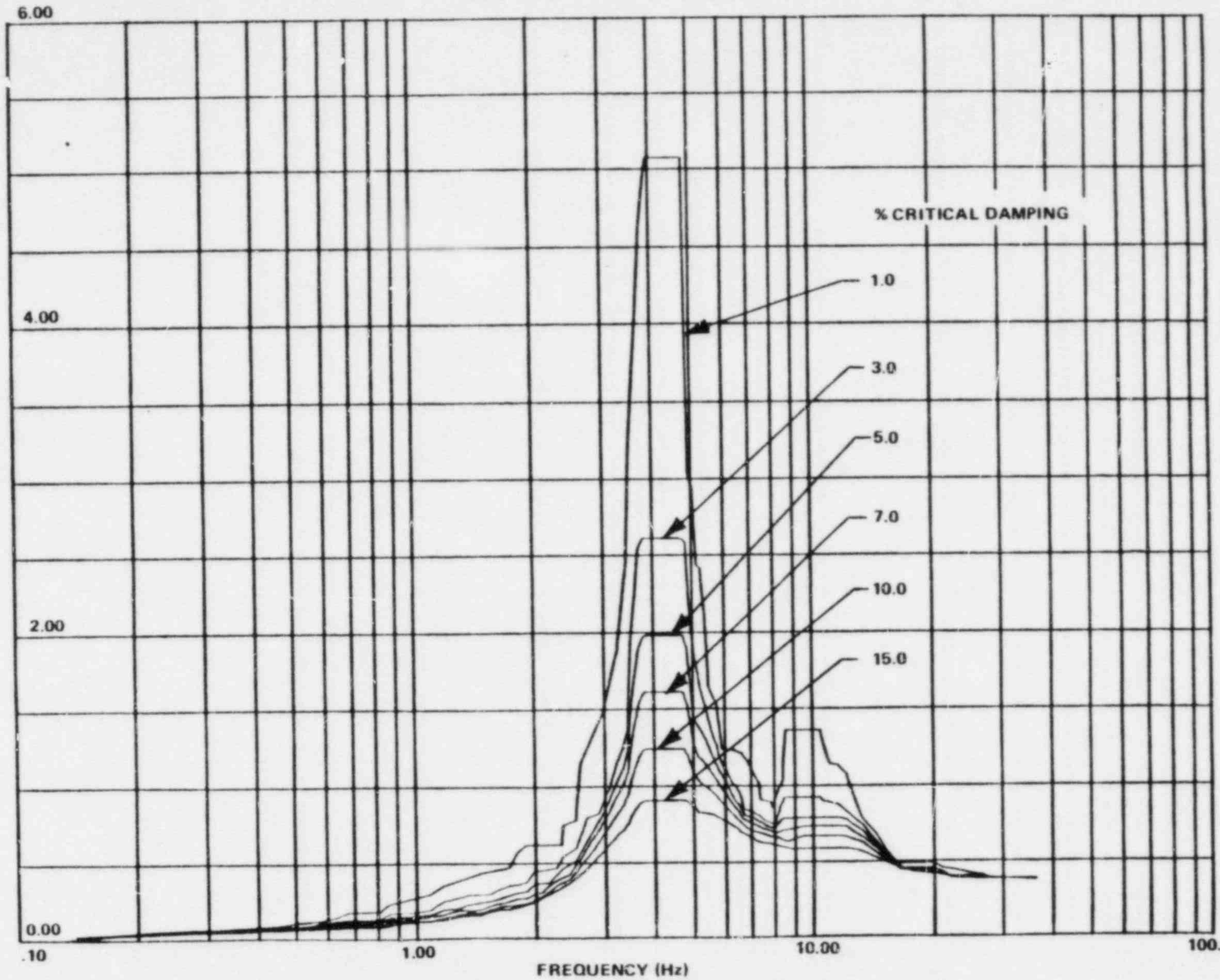
Georgia Power

EDWIN I. HATCH
NUCLEAR PLANT – UNIT 1

REACTOR BUILDING 1984 RESPONSE SPECTRA OBE (N-S) FOR FLOOR AT el 228 ft

FIGURE 12.6-12

3604-0



DRAFT

REV 3 7/85

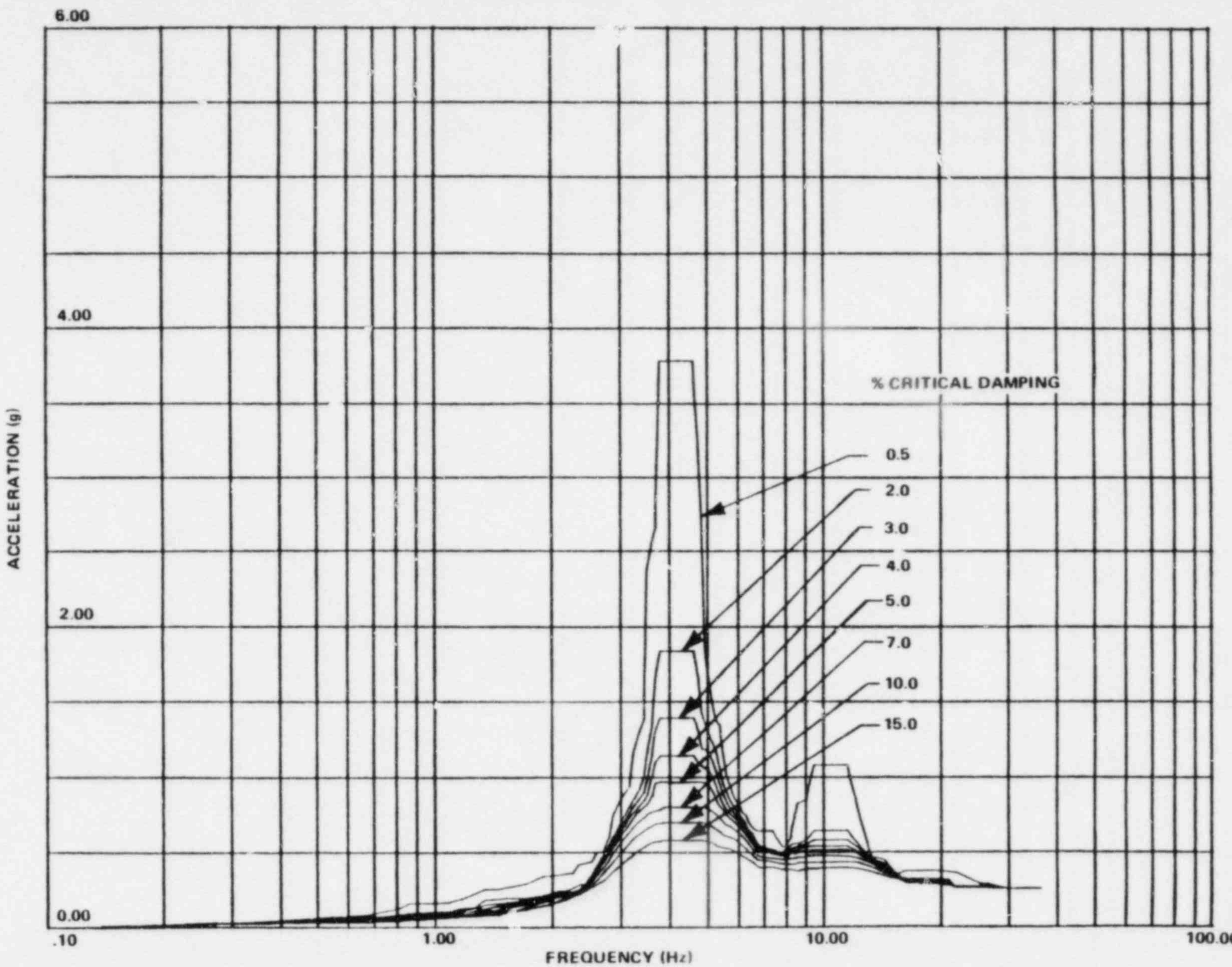
Georgia Power

EDWIN I. HATCH
NUCLEAR PLANT – UNIT 1

REACTOF BUILDING 1984 RESPONSE SPECTRA DBE (E-W) FOR FLOOR AT el 228 ft

FIGURE 12.6-13

3604-0



DRAFT

REV 3 7/85

Georgia Power

EDWIN I. HATCH
NUCLEAR PLANT – UNIT 1

REACTOR BUILDING 1984 RESPONSE SPECTRA OBE (E-W) FOR FLOOR AT el 228 ft

FIGURE 12.6-14

3604-0

A.3 DESIGN REQUIREMENTS

A.3.1 PIPING DESIGN

Pressure and temperature conditions to which piping pressure components are subjected are described in the appropriate system design section of the final safety analysis report (FSAR). All piping systems within the scope of this appendix including pipe, flanges, valves, and fitting meet the requirements of American National Standards Institute (ANSI) B31.1, or ANSI B31.7 as indicated in tables A.2-2 and A.2-3, including requirements for design, erection, supports, tests, inspection, and special additional supplementary requirements specified in this appendix.

A.3.1.1 Allowable Stresses

The allowable stress values of the applicable piping code are used. For materials not covered by the piping codes, the stress values of the American Society of Mechanical Engineers (ASME) Boiler and Pressure Vessel Code are used.

A.3.1.2 Wall Thickness

Pipe wall thickness, fittings, and flange ratings are in accordance with the applicable code, including adequate allowances for corrosion and erosion according to individual system requirements for a design life of 40 years.

A.3.1.3 Reactor Vessel Nozzle Load

All piping including instrument piping connecting to the reactor pressure vessel (RPV) nozzles is designed so that the nozzle to pipe interface load does not result in stresses in excess of the allowable material stresses. Thermal sleeves are used where nozzles are subjected to high thermal stresses.

A.3.1.4 Seismic Design

For the pu[illegible]e of sei[illegible]c design, equipment and piping is categorized according [illegible] the following definitions:

Seismic Class 1

This class includes equipment and piping systems whose failure or malfunction could cause, or increase, the severity of the design basis accident (DBA), cause release of radioactivity in excess of 10 CFR 100 limits, or those essential for safe shutdown and immediate or long-term operation following a loss-of-coolant accident (LOCA).

Seismic Class 2

This class includes equipment and piping system whose failure would not result in the release of significant radioactivity and would not prevent reactor shutdown. The failure of Seismic Class 2 equipment and piping systems may interrupt power generation.

The equipment and piping considered as Seismic Class 1 are shown in table A.3-1. Seismic Class 1 equipment and piping systems are supported and restrained to meet the seismic design analysis criteria in compliance with applicable codes.

The dynamic analysis of Seismic Class 1 piping systems for seismic loads was performed using the spectrum response method, as applied to a lumped mass mathematical model of the piping systems. The maximum responses of each mode were calculated and combined by the square-root-of-the-sum-of-the-squares method to give the maximum response quantities resulting from all modes.

The response thus obtained was combined with the results produced by other loading conditions to compute the resultant stresses. All modes having frequencies less than 30 Hz are used. The percentage of critical damping used in the seismic analysis is defined in paragraph 12.3.3.2.1.2. The horizontal acceleration spectrum curves applied to the piping systems are developed as part of the seismic analysis for the building in which the piping is located.

A.3.1.5 <u>Analysis of Piping</u>

A.3.1.5.1 Primary Stresses (S_P)

Primary stresses are as follows:

A. Circumferential primary stress (S_R) - Circumferential primary stresses are below the allowable stress (S_h) at the design pressure and temperature.

B. Longitudinal primary stresses (S_L) - The following loads are considered as producing longitudinal primary stresses: internal or external pressures; weight loads including valves, insulation, fluids, and equipment hanger loads; static external loads and reactions; the inertia load portion of the seismic loads; and dynamic loads due to a rapid valve closure or opening.

When the seismic load is due to the OBE (maximum horizontal ground acceleration of 0.08 g), the vectorial combination of all longitudinal primary stresses (S_L) does not exceed 1.2 times the allowable stresses (S_h).

When the seismic load is due to the DBE (0.15-g horizontal), the vectorial combination of all longitudinal primary stresses generally does not exceed material yield stress at temperature. Specific cases where higher allowable limits are used for main steam piping are discussed in appendix C.

A.3.1.5.2 Secondary Stresses (S_E)

Secondary stresses are determined by use of the maximum shear stress theory:

$$T_{max} = \frac{1}{2}\sqrt{S_b^2 + 4S_t^2} = \frac{1}{2} S_E$$

therefore,

$$S_E = \sqrt{S_b^2 + 4S_t^2}$$

(See ANSI B31.1.)

The following loads are considered in determining longitudinal secondary stresses:

- Thermal expansion of piping
- Movement of attachments due to thermal expansion
- Forces applied by other piping systems as a result of their expansion
- Any variation in pipe hanger loads resulting from expansion of the system
- Anchor point movement portion of seismic loads

The vectorial combination of longitudinal secondary stresses (S_E) does not exceed the allowable stress range S_A, i.e., $S_E \leq S_A$, where:

$$\left[S_A = f \quad 1.25 \ (S_C + S_H) - S_L \right]$$

(This is equation 1 from paragraph 102.3.2 of ANSI B31.1 modified to include the additional stress allowance permitted when S_L is less than S_h).

A.3.1.6 Special Requirements For Main Steam Piping

The main steam pipe supports and restraints are designed and constructed to assure that the second isolation valve functions, particularly in the event of a pipe failure downstream of the valve. All main steam pipe failure stops within the reactor building are designed to Seismic Class 1.

The main steam lines downstream of the second isolation valves are designed to ANSI B31.1 as a minimum with the use of Code Case 74, B31.1. In addition, the following requirements apply down to but not including the next valve, including all branch lines larger than 2 1/2-in. diameter:

A. Design and Analysis

1. The design includes consideration of earthquake effects. Earthquake loading for the OBE (0.08-g horizontal acceleration) is treated as occasional load as provided for in ANSI B31.1, using suitable static loading corresponding to the pertinent terminal structure response spectrum.

2. In order to determine the end displacements and seismic forces on the main steam piping, sufficient dynamic analyses have been performed to determine needed response spectra at the pipe terminal points.

B. Materials

1. Seamless pipe is ASTM-A106 Grade B. Plate pipe is ASTM-A155 Class I, Grade KC 70.

2. Certification in writing is required from the manufacturer that all pipe, fittings, flanges, bolting materials, valves, and welding wire meet applicable material specifications.

pressures for the core support are calculated to be very small as compared to 1/2 in. The guide tubes, therefore, are not lifted off, although even if they were, this would not be of concern because bypass leakage at this time is not important.

C.3.2.2 Internals Fatigue Analysis

Fatigue analysis was performed using as a guide the ASME Boiler and Pressure Vessel Code, Section III. The method of analysis used to determine the cumulative fatigue usage is described in APED-5460, "Design and Performance of GE-BWR Jet Pumps," September 1968. The most significant fatigue loading occurs in the jet pump - shroud - shroud support area of the internals. The analysis was performed for Unit 1 of the Millstone Nuclear Power Station, a plant where the configuration (gusset-type shroud support) was almost identical to HNP-1. Therefore, the calculated fatigue usage is expected to be a reasonable approximation for this plant.

Loading combination and transients considered are:

- Normal startup and shutdown
- OBE and DBE
- Ten-min blowdown from a stuck relief valve
- High-pressure coolant injection (HPCI) operation
- Low-pressure coolant injection (LPCI) operation (DBA)
- Improper start of a recirculation loop

Cumulative fatigue usage is:

$U_{allowable} = 1.0$

$U_{calculated} = 0.65$

The location of maximum fatigue usage is at the inside diameter (ID) of the jet pump diffuser adapter at the thin end of the tapered transition section.

C.3.2.3 Internals Seismic Analysis

The seismic loads on the reactor vessel and internals are based on a dynamic analysis of the coupled model consisting of reactor building, reactor vessel, and internals. The natural frequencies and mode shapes for the system were determined.

The relative displacement, acceleration, and load response of the reactor vessel and internals were then determined using the response spectrum method of analysis. The dynamic responses were determined for each mode of interest and combined by square root of the sum of the squares of modal responses. The resulting values of displacements, accelerations, shears, and moments were used for design calculations. These results were combined with the results of other loads for the various loading conditions. The combined results for the critical components are presented in table C.3-1.

C.3.3 PIPING

C.3.3.1 Piping Flexibility Analysis

The piping has been analyzed for the effects of dead loads, external loads, and thermal loads. Stresses calculated were combined bending and torsional stresses in accordance with American National Standards Institute B31.1. Power piping and intensification factors were applied in accordance with B31.1. Several pressure temperature cycles were evaluated and the cycle representing the worst for thermal expansion stresses was selected for the design case. All critical points were evaluated to the stress limits of B31.1 and, in addition, events with very low probability of occurrence were analyzed and stresses at all critical points compared with the limits defined in this load criteria. The load combination, allowable stresses, identification of points of highest stress, and highest stress values are summarized in table C.3-1.

C.3.3.2 Piping Seismic Analysis

The piping systems were dynamically analyzed using the response spectrum method of analysis. For each of the piping systems, a mathematical model consisting of lumped masses at discrete joints connected together by weightless elastic elements was constructed. Valves were also considered as lumped masses in the pipe, and valve operators as lumped masses acting through the operator center of gravity. Where practical, a support is located on the pipe at or near each valve. Stiffness matrix and mass matrix were generated and natural periods of vibration and corresponding mode shapes were determined. Input to the dynamic analyses were the appropriately damped acceleration response spectra for the applicable floor elevation. The increased flexibility of the curved segments of the piping systems was also considered. The results for earthquakes acting in the X and Y (vertical) directions simultaneously, and Z and Y directions simultaneously were computed separately.