

## HATCH UNIT 1 AND 2 SPRA RELIEF

### Information to Support Reconsideration of Hatch U1 for Inclusion in SPRA Relief due to Seismic Robustness and Similarities to Hatch U2

#### BACKGROUND

- NRC will be providing SPRA “relief” for 11 ½ sites. Hatch Unit 2 is the “½” site.
- SNC agrees with NRC’s opinion that Hatch Unit 2 should receive SPRA relief.
- This document contains additional information for NRC’s consideration to also provide SPRA relief to Hatch Unit 1.

#### TECHNICAL POINTS AND DISCUSSION

1. **Technical Point: Both DBEs anchored at 0.15g.**

**Discussion:** Unit 1 Design Basis Earthquake (DBE) is a Housner spectral shape, Unit 2 DBE is a Modified Newmark spectral shape. Both are anchored at 0.15g. [See *Reference 1 and Attachment 1*]

2. **Technical Point: Units 1 and 2 are basically the same plant.**

**Discussion:** Both units are GE Boiling Water Reactors Type 4g with Mark I containments and the drywell/pressure suppression concept is used. The Reactor Buildings are reinforced-concrete structures that act as the secondary containment. Hatch Units 1 and 2 are the same site, common Control Building, common Control Room, common Intake Structure, common Diesel Generator Building.

3. **Technical Point: The U1 and U2 In-Structure Response Spectra (ISRS) used to design/qualify equipment are very similar.**

**Discussion:** Though the spectral shapes are different, the U1 and U2 ISRS used to design/qualify equipment are very similar. In many cases, the U1 ISRS are higher than U2 equivalent ISRS (for sister unit buildings, elevations). [See *Reference 2 and Attachment 3*]

4. **Technical Point: The two Units have similar ISRS.**

**Discussion:** For all shared structures, Hatch Unit 1 seismic analysis produces similar ISRS to those calculated for Hatch Unit 2 in peak spectral amplitude and frequency range. This demonstrates that even though the Unit 1 DBE spectral shape accelerations are less than the Unit 2 DBE accelerations, the resulting seismic demand used to design and qualify Unit 1 and Unit 2 safety related Structures, Systems, and Components (SSCs) are essentially the same. The Hatch Unit 1 DBE by itself cannot be used as an indicator of the Unit 1 seismic robustness. Also it can then be expected that the seismic robustness of both units would be similar. Conclusion: No significant difference between the seismic demands calculated and used for design between Hatch Unit 1 and Hatch Unit 2; essentially the seismic designs are the same.

5. **Technical Point: Previous Seismic Margin Assessments**

**Discussion:** An additional demonstration of the Hatch Unit 1 seismic robustness or seismic margin was provided by implementing the full EPRI Seismic Margin Assessment (SMA) methodology as a pilot implementation project. The Hatch Unit 1 SMA demonstrated a High Confidence of a Low Probability of Failure (HCLPF) of at least 0.3g; where the review level earthquake (RLE) ground motion exceeded the Hatch Unit 1 DBE by a factor of 2 or more. It should be noted the Hatch Unit 1 SMA also showed that the Unit 1 NSSS has at least a HCLPF of 0.3g. For IPEEE-seismic a reduced scope SMA of Hatch Unit 2 was performed that demonstrated a plant HCLPF of 0.3g. No seismic issues were identified that were unique just to Hatch Unit 1. Conclusion: Hatch Unit 1 has significant seismic margin as demonstrated by the SMA as was also demonstrated for Hatch Unit 2; and no seismic issues were found that were Hatch Unit 1 unique. [See References 7 and 8, Attachments 2 and 4]

- a. The maximum GMRS/DBE ratio for either unit is less than 2
- b. The maximum HCLPF/U2 DBE ratio is 2.6
- c. The maximum HCLPF/U1 DBE ratio is 3.388

6. **Technical Point: NTTF 2.3 Seismic verified SMA modifications.**

**Discussion:** The NTTF 2.3 seismic walk downs verified modifications resulting from the Hatch Unit 1 SMA, used as the IPEEE-seismic response, are fully implemented. Conclusion: Under the 2.3 seismic it was verified that SMA modifications remain in place for both units which supports the expectation that both the Hatch Unit 1 and Unit 2 seismic margins remain at their previous HCLPF values. [See References 3 and 4]

7. **Technical Point: NTTF 2.1 ESEP documented U1 ESEL components have HCLPFs of at least 0.3g.**

**Discussion:** The Hatch Unit 1 ESEP provided documentation that the Unit 1 ESEL components have HCLPFs of at least 0.3g using the same SMA ground motion used for the Hatch Unit 1 SMA and the Unit 2 SMA. This SMA ground motion is called the review level earthquake ground motion which exceeds the Hatch Unit 1 DBE by a factor of 2 or more. (See Attachments 1 and 2). The only exception is the CST which is surface mounted where the Hatch GMRS was used as the input motion. No modifications were identified. Similar ESEP results were found for Unit 2. Conclusion: The ESEP assessment further demonstrates the significant seismic margin above the Hatch Unit 1 DBE as was also demonstrated for Hatch Unit 2. [See References 5 and 6]

8. **Technical Point: NRC GI-199 SCDF is same for both units.**

**Discussion:** The NRC GI-199 Safety/Risk Assessment indicates that no concern exists regarding adequate protection and that the current seismic design of operating reactors provides a safety margin to withstand potential earthquakes exceeding the original design basis. The site hazard is the same for both units, the HCLPF is the same for both units. Conclusion: The estimated point estimate SCDF would be the same for both Unit 1 and Unit 2.

9. **Technical Point: Enveloping curves are used for design/qualification.**

**Discussion:** Since the mid 90's seismic design and qualification for components for both Units have used an envelope of the Hatch Unit 1 and Unit 2 ISRS of record and 1/2 the SMA ISRS. This is a conservative practice, but it demonstrates the use of later seismic analysis results with design basis results to assure conservative design and qualification. This points out that any new design changes reflect all the latest seismic analyses. Conclusion: Hatch Unit 1 and Unit 2 both

are incorporating the latest seismic results with the original design basis for new design changes; therefore, both Units are being maintained to the same standard

## IN CONCLUSION

SNC agrees with the NRC that Hatch Unit 2 should receive SPRA relief. The Technical Points and Discussion above illustrate that the seismic designs, loads, etc. are essentially the same for Hatch Unit 1 and that the seismic margin of 0.3g for both units has been demonstrated several times. No unique seismic issues have been identified between Unit 1 and Unit 2; and all new design changes for both units incorporate later seismic analyses. It can be concluded based on the above that the seismic robustness of Hatch Unit 1 is similar to Hatch Unit 2; and therefore based on the relief from a SPRA granted to Unit 2, a relief from a SPRA should also be granted to Unit 1.

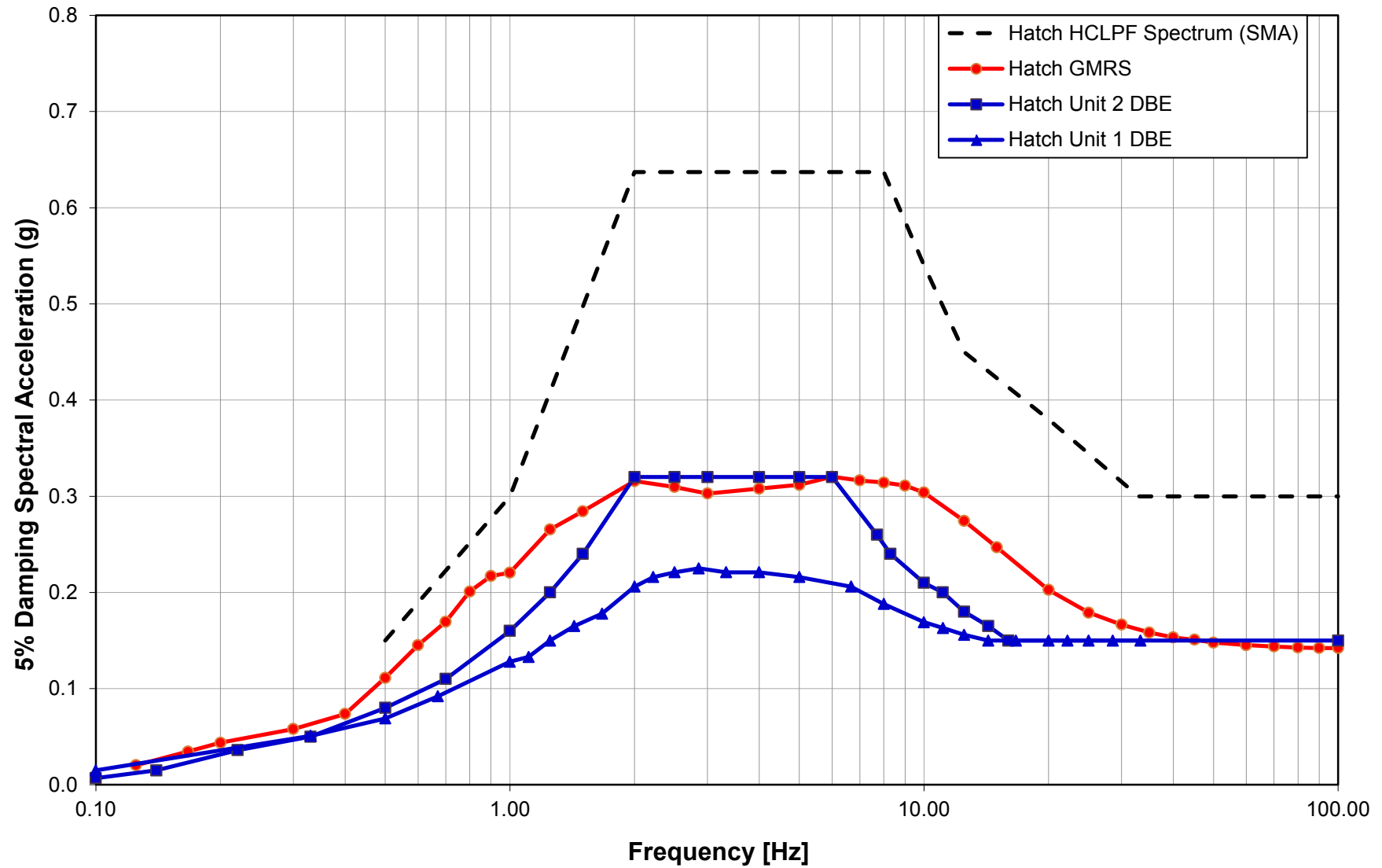
## REFERENCES

1. "Edwin I. Hatch Units 1 and 2, Seismic Hazard and Screening Report for CEUS Sites", March 31, 2014, NL-14-0343, Sections 2.0 and 3.1
2. "Edwin I. Hatch Nuclear Power Plant Units 1 and 2, Seismic Floor Response Spectra of Record", Rev. 1 dated July 31, 1987
3. Hatch Unit 1, Staff Assessment of the Seismic Walkdown Report Supporting Implementation of Near-Term Task Force Recommendation 2.3 Related to the Fukushima Dai-Ichi Nuclear Power Plant Accident (ML14155a361)
4. Hatch Unit 2, Staff Assessment of the Seismic Walkdown Report Supporting Implementation of Near-Term Task Force Recommendation 2.3 Related to the Fukushima Dai-Ichi Nuclear Power Plant Accident (ML14079a355)
5. Memo: Diane Jackson to Mohamed Shams, July 7, 2015, Hatch Units 1 and 2, Technical Review Checklist Related to Interim ESEP Supporting Implementation of NTTF R 2.1, Seismic (ML15190A131)
6. NRC to Mr. C. R Pierce, July 22, 2015, Hatch Units 1 and 2 – Staff Review of Interim Evaluation Associated with Revaluated Seismic Hazard implementation of Near-Term Task Force Recommendation 2.1 (ML15201A474)
7. A Methodology for Assessment of Nuclear Power Plant Seismic Margin (Revision 1), EPRI NP-6041-SL, August 1991, Appendix S "Lessons Learned from Hatch Nuclear Plant Seismic Margin Assessment"
8. Seismic Margin Assessment of the Edwin I. Hatch Nuclear Plant, Unit 2, EPRI NP-7217s-M, October 2008.

## ATTACHMENTS

- Att 1 - Figure/Plot – Hatch U1 DBE, Hatch U2 DBE, Hatch GMRS, Hatch HCLPF Spectrum (SMA)
- Att 2 - Table – Hatch Response Spectra Comparison (Ratios: GMRS to DBE, HCLPF to DBE)
- Att 3 - Table/Word Doc – Hatch Comparison of U1 to U2 ISRS Similar Bldgs/Elevations DBE
- Att 4 – Excerpt from Hatch SMA, Sections 1 through 5
- Att 5 – Excerpt from Hatch IPEEE Response, Section 4.9 Third-Party Audit

## Attachment 1 - Hatch Units 1 and 2



ATTACHMENT 2 - HATCH RESPONSE SPECTRA COMPARISON

Freq (Hz)	GMRS Spectral Acceleration (g)	H1 DBE Spectral Accel (g)	GMRS/U1 DBE Ratio	H2 DBE Spectral Accel (g)	GMRS/U2 DBE Ratio	HCLPF Spectrum Accel (g)	HCLPF/U2 DBE Ratio	HCLPF/U1 DBE Ratio
0.1		0.015		0.007				
0.14				0.015				
0.125	0.0203							
0.167	0.0346							
0.2	0.0437							
0.22				0.036	1.21			
0.3	0.058							
0.33		0.051		0.05				
0.4	0.0737							
0.5	0.1113	0.069	1.61	0.08	1.39	0.15	1.875	2.174
0.6	0.1452							
0.67		0.092						
0.7	0.1696			0.11	1.54			
0.8	0.2009							
0.9	0.2171							
1	0.2206	0.128	1.72	0.16	1.38	0.3	1.875	2.344
1.11		0.133						
1.25	0.2654	0.15	1.77	0.2	1.33			
1.43		0.165						
1.5	0.2844			0.24	1.19			
1.67		0.178						
2	0.3158	0.206	1.53	0.32	0.99	0.637	1.991	3.092
2.22		0.216						
2.5	0.3096	0.221	1.40	0.32	0.97			
2.86		0.225						
3	0.3029			0.32	0.95			
3.33		0.221						
4	0.308	0.221	1.39	0.32	0.96			
5	0.3118	0.216	1.44	0.32	0.97			
6	0.3203			0.32	1.00			
6.67		0.206						
7	0.3164							
7.7				0.26				
8	0.3142	0.188	1.67			0.637	2.600	3.388

ATTACHMENT 2 - HATCH RESPONSE SPECTRA COMPARISON

Freq (Hz)	GMRS Spectral Acceleration (g)	H1 DBE Spectral Accel (g)	GMRS/U1 DBE Ratio	H2 DBE Spectral Accel (g)	GMRS/U2 DBE Ratio	HCLPF Spectrum Accel (g)	HCLPF/U2 DBE Ratio	HCLPF/U1 DBE Ratio
8.3				0.24				
9	0.3111							
10	0.3039	0.169	1.798	0.21	1.447	0.54	2.571	3.195
11.11		0.163		0.2				
12.5	0.2744	0.156	1.76	0.18	1.52	0.45	2.500	2.885
14.29		0.15						
14.3				0.165				
15	0.2469							
16				0.15				
16.67		0.15		0.15				
20	0.2027	0.15	1.35	0.15	1.35	0.3	2.000	2.000
22.22		0.15		0.15				
25	0.179	0.15	1.19	0.15	1.19			
28.67		0.15		0.15				
30	0.166	0.15	1.11	0.15	1.11			
33						0.3	2.000	2.000
33.33		0.15		0.15				
35	0.1583	0.15	1.06	0.15	1.06			
40	0.1532	0.15	1.02	0.15	1.02			
45	0.1508	0.15	1.01	0.15	1.01			
50	0.1478	0.15	0.99	0.15	0.99			
60	0.1452	0.15	0.97	0.15	0.97			
70	0.1438	0.15	0.96	0.15	0.96			
80	0.1427	0.15	0.95	0.15	0.95			
90	0.1422	0.15	0.95	0.15	0.95			
100	0.1422	0.15	0.95	0.15	0.95	0.3	2.000	2.000

Comments:

- 1. Unit 1 max GMRS/U1DBE ratio occurs at 10hz and is less than 2 (ratio is 1.798)
- 2. Unit 2 max GMRS/U2DBE ratio occurs at 12.5hz, and is less than 2 (ratio is 1.52)
- 2. Unit 2 max GMRS/U2DBE ratio between 1 and 10 hz is less than 2 (ratio is 1.447)

### Attachment 3 - Comparison of Hatch Unit 1 and 2, Similar Buildings/elevations/Horizontal ISRS DBE

5% critical damping

	Plant Hatch Unit 1 Peak ( __gs @ __ hz)	Unit 1 zpa(gs)		Plant Hatch Unit 2 Peak ( __gs @ __ hz )	Unit 2 zpa (gs)
Control Bldg 130' N-S	0.781g @ ~6 - 8hz	0.1680g		0.792g @ ~5.5 - 7hz	0.2110g
Control Bldg 130' E-W	1.051g @ ~6 - 8hz	0.1800g		0.745g @ ~5.5 - 7hz	0.2140g
Control Bldg 164' N-S	1.245g @ ~6 - 8hz	0.2560g		1.073g @ ~5.5 - 7hz	0.2850g
Control Bldg 164' E-W	1.458g @ ~6 - 8hz	0.2520g		0.919g @ ~ 5.5 - 8hz	0.2640g
Reactor Bldg 130' N-S	0.660g @ ~ 3.8 - 5hz	0.1610g		0.875g @ ~ 2.9 - 3.8hz	0.1810g
Reactor Bldg 130' E-W	0.773g @ ~3.8 - 5hz	0.1650g		0.799g @ ~ 3.1 - 4hz	0.2070g
Reactor Bldg 158' N-S	0.906g @ ~ 3.8 - 5hz	0.1850g		1.228g @ ~3 - 4hz	0.2300g
Reactor Bldg 158' E-W	1.047g @ ~ 3.8 - 5hz	0.1840g		1.116g @ ~3.5 - 4.8hz	0.2490g
Intake Structure 111' N-S	0.795g @ ~8 to 10hz	0.2000g		0.680g @ ~7.5 – 9.5hz	0.1960g
Intake Structure 111' E-W	1.438g @ ~5.8 – 8hz	0.2550g		1.963g @ ~5.9 – 8hz	0.3440g
Intake Structure 128' N-S	0.888g @ ~8 - 12hz	0.2230g		0.743g @ ~ 7.5 – 9.5hz	0.2090g
Intake Structure 128' E-W	1.753g @ ~6 - 8hz	0.3020g		2.373g @ ~ 6.0 – 8hz	0.4100g
Diesel Gen Bldg 130' N-S	1.241g @ 2.5 – 5 hz	0.2250g		0.876g @ ~2.8 – 5hz	0.2010g
Diesel Gen Bldg 130' E-W	1.235g @ 2.5 – 5 hz	0.2240g		0.889g @ ~2.8 – 5hz	0.2000g

Reference: "Edwin I. Hatch Nuclear Power Plant Units 1 and 2, Seismic Floor Response Spectra of Record", Rev. 1 dated July 31, 1987

NOTE: Highlighted text/values indicate the higher acceleration between the two ISRS

SEISMIC MARGIN ASSESSMENT OF EDWIN I. HATCH NUCLEAR PLANT  
Unit 1

Volume 1: Main Report

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NP- Volume 1  
Research Project 2722-22

Final Report

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## Section 1

### INTRODUCTION

The Electric Power Research Institute (EPRI) has sponsored a seismic margin assessment (SMA) of Plant Hatch Unit 1. This project was funded by EPRI and Georgia Power Company (GPC). GPC owns Plant Hatch jointly with the Oglethorpe Power Corporation, the Municipal Electric Authority of Georgia, and the City of Dalton. The plant, operated by GPC, consists of two General Electric boiling water reactor (BWR) IVs with Mark I containments. The rated output of each unit is approximately 813 MWe. The plant is located on a soil site on the Altamaha River near Baxley, Georgia. Unit 1 was placed in commercial operation in 1975.

The Nuclear Regulatory Commission (NRC) and EPRI have developed similar approaches to performing an SMA (1, 2, 3). The NRC approach was used to perform a trial SMA of the Maine Yankee Atomic Power Station (4). The EPRI methodology was used to perform an SMA of the Catawba Power Station Unit 2 (5). Both of these trial SMAs were of pressurized water reactors on rock sites. EPRI and the NRC identified a need to perform an SMA for a BWR on a soil site to further test the SMA methodology. GPC agreed to use Plant Hatch Unit 1 as this trial BWR. The Hatch SMA was performed using the EPRI SMA methodology.

The NRC has reviewed the EPRI methodology (1) and finds it an acceptable approach. The NRC participated in the Hatch SMA in an independent review capacity through NRC Research and an outside peer review group consisting of five consultants. The NRC also funded a separate study using the event tree/fault tree modeling of front-line and support systems. The results of this study are not described in this report, but will be published as a NUREG. The event tree/fault tree study has no effect on this review. The EPRI methodology applies the success-path modeling approach.

The primary objective of the Hatch SMA was to test the EPRI methodology for performing an SMA of a BWR on a soil site. The purpose of an SMA is to demonstrate sufficient margin over the safe shutdown earthquake (SSE) and identify the "weaker links," if any, which control the level of earthquake for which there is a high confidence that the plant can safely shut down. The Hatch SMA consisted

of identifying two independent paths of systems and their associated components that are capable of establishing and maintaining a safe shutdown condition within 72 hours following the seismic event. The components that make up these systems were evaluated for seismic capability by performing a plant walkdown of the components and using the screening guidelines found in the EPRI methodology. The effects of relay contact chatter were evaluated for each component. Plant soils and structures which could affect the success paths were also evaluated. Subsequent sections of this report describe the assessment process and the results of the Hatch SMA.

Portions of the Seismic Qualification Utility Group (SQUG) Generic Implementation Procedure (GIP) (6) to implement the technical resolution to unresolved safety issue (USI) A-46 were implemented in conjunction with the Hatch Unit 1 SMA. Since these programs are similar, it was possible to perform both programs with one walkdown. The USI A-46 portions of the project were funded exclusively by GPC. The SQUG GIP requires a more formalized, detailed evaluation of equipment, especially anchorage. The areas where the SQUG GIP required additional effort beyond that required by the SMA methodology are identified in this report under the appropriate sections.

#### REFERENCES

1. A Methodology for Assessment of Nuclear Power Plant Seismic Margin. Palo Alto, CA: Electric Power Research Institute, October 1988. NP-6041.
2. R. J. Budnitz, et al. An Approach to the Qualification of Seismic Margins in Nuclear Power Plants. Lawrence Livermore National Laboratory, prepared for U. S. Nuclear Regulatory Commission, August 1985. NUREG/CR-4334.
3. P. G. Prassinis, et al. Recommendations to the Nuclear Regulatory Commission on Trial Guidelines for Seismic Margin Reviews of Nuclear Power Plants. Lawrence Livermore National Laboratory, prepared for U. S. Nuclear Regulatory Commission, March 1986. NUREG/CR-4482.
4. P. Prassinis, R. Murry, G. Cummings. Seismic Margin Review of the Maine Yankee Atomic Power Station. Lawrence Livermore National Laboratory Report UCID-20948, prepared for U. S. Nuclear Regulatory Commission, March 1987. NUREG/CR-4826.
5. Seismic Margin Assessment of the Catawba Nuclear Station. Palo Alto, CA: Electric Power Research Institute, April 1989. NP-6359.
6. Generic Implementation Procedure (GIP) for Seismic Verification of Nuclear Plant Equipment, Revision 1. Seismic Qualification Utility Group, December 1988.

## Section 2

### GENERAL PLANT DESCRIPTION

#### 2.1 SITE LOCATION AND AREA

Plant Hatch is located in Appling and Toombs counties, Georgia, at the intersection of the Altamaha river with U.S. Hwy No. 1. This location is approximately 98 miles southeast of Macon, Georgia, and approximately 73 miles northwest of Brunswick, Georgia (Figure 2-1). The site, which is owned by Georgia Power Company (GPC), consists of about 2244 acres. A plot plan of the plant is shown in Figure 2-2.

#### 2.2 SITE TOPOGRAPHY

The local topography of the site is gently rolling. The present elevation of the plant area is about 125 to 145 ft msl. The average river elevation is about 71 ft msl. The finished grade elevation around the plant is approximately 129 ft msl.

The site is located in a region of relatively flat to rolling terrain. Elevations within an approximate 15-mile radius do not exceed an estimated 310 ft msl. The confluence of the Oconee and Ocmulgee rivers that forms the Altamaha river is about 12 air miles to the west. These rivers lie in an open, shallow valley that broadens downstream from the site.

#### 2.3 SITE GEOLOGY AND SEISMOLOGY

Plant Hatch, located on the south bank of the Altamaha River in Appling County, southeastern Georgia, is in the Coastal Terraces subprovince of the Atlantic Coastal Plain physiographic province. The site is underlain by approximately 4000 feet of relatively unconsolidated Mesozoic and Cenozoic sands, gravels, clays, marls, claystones, sandstones, and limestones. These strata, overlying basaltic basement rock of pre-Cretaceous age, dip and thicken seaward. No structural features affect the material underlying the site. No major or minor fault zones are near the site, nor were any local faults discovered during field mapping, exploratory drilling, and construction.

The site is within a region of infrequent seismic activity. No earthquakes within 200 miles of the site produced Modified Mercalli intensities at the site greater than VI. This includes those in the Charleston area about 150 miles from the site. Historically, reported earthquakes occurring in other areas have not produced intensities greater than VI at the site. The design basis earthquake (DBE) was conservatively selected as Modified Mercalli Intensity VII.

The Hawthorn Formation of Miocene to Pliocene age is the foundation-bearing stratum for the major plant structures. It consists primarily of sand, clay, and cemented sand and clay layers. There are no zones of deformation, alternation, or weakness within the Hawthorn Formation.

The site is underlain by both confined and unconfined aquifers. Local and regional ground water conditions have not been altered by construction and operation of the plant.

#### 2.4 NUCLEAR STEAM SUPPLY SYSTEM

Operating license number DPR-57 was issued for Plant Hatch Unit 1 on August 6, 1974, and commercial operation began December 31, 1974. The gross electrical output is approximately 813 MWe, which corresponds to a net output of approximately 786 MWe. Unit 2 began commercial operation on September 5, 1979.

The Hatch nuclear steam supply system (NSSS) is a General Electric boiling water reactor (BWR) IV. The Hatch containment is a Mark I BWR containment incorporating the drywell/pressure suppression concept. The reactor building is a reinforced-concrete structure that acts as the secondary containment.

#### 2.5 STRUCTURES

Class 1 structures include the primary containment structure, reactor building, spent fuel pool, new-fuel storage vault, diesel generator building, control building, intake structure, and the main stack. Other structures supporting or housing Class 1 equipment include the wall around the condensate storage tank (CST), the liquid nitrogen storage tank and foundation, and the diesel generator fuel oil storage tanks. These items are shown in Figure 2-2.

The following structures were considered as part of the seismic margin analysis (SMA):

#### Intake Structure

The river intake structure constructed for Unit 1 is shared by Unit 2. It is a reinforced-concrete structure housing plant service water and residual heat removal service water pumps.

#### Diesel Generator Building

The diesel generator building is a reinforced-concrete structure designed to house the diesel generators, local control panels, and emergency switchgear for both Units 1 and 2. Each diesel generator and its control panel is physically separated from the other diesel generator units.

#### Control Building

The control building houses the common control room for Units 1 and 2 and associated auxiliaries. The building is a reinforced-concrete structure with steel framing above el 164 ft.

#### Reactor Building

The reactor building encloses the reactor, reactor primary containment, auxiliary cooling systems, new-fuel storage vault, and spent-fuel storage pool. The reactor building provides secondary containment for the reactor and primary containment for auxiliary systems. Primary containment for the reactor consists of the drywell and the pressure suppression chamber.

The reactor building is a reinforced-concrete structure with reinforced-concrete exterior walls up to the refueling floor level. Exterior walls above the refueling floor consist of structural steel columns and prefabricated concrete panels. The reactor buildings for Units 1 and 2 are separated by their own exterior walls except above the refueling floor; the refueling floor is common to both units. There is a 3 in. seismic gap between the reactor buildings.

#### Liquid Nitrogen Storage Tank

The tank is a horizontal steel tank supported by two steel saddles anchored to a common concrete foundation.

#### Diesel Generator Fuel Oil Storage Tanks

The diesel generator fuel oil storage tanks consist of five equal-capacity (40,000 gal.) horizontal-type steel fuel oil storage tanks located underground.





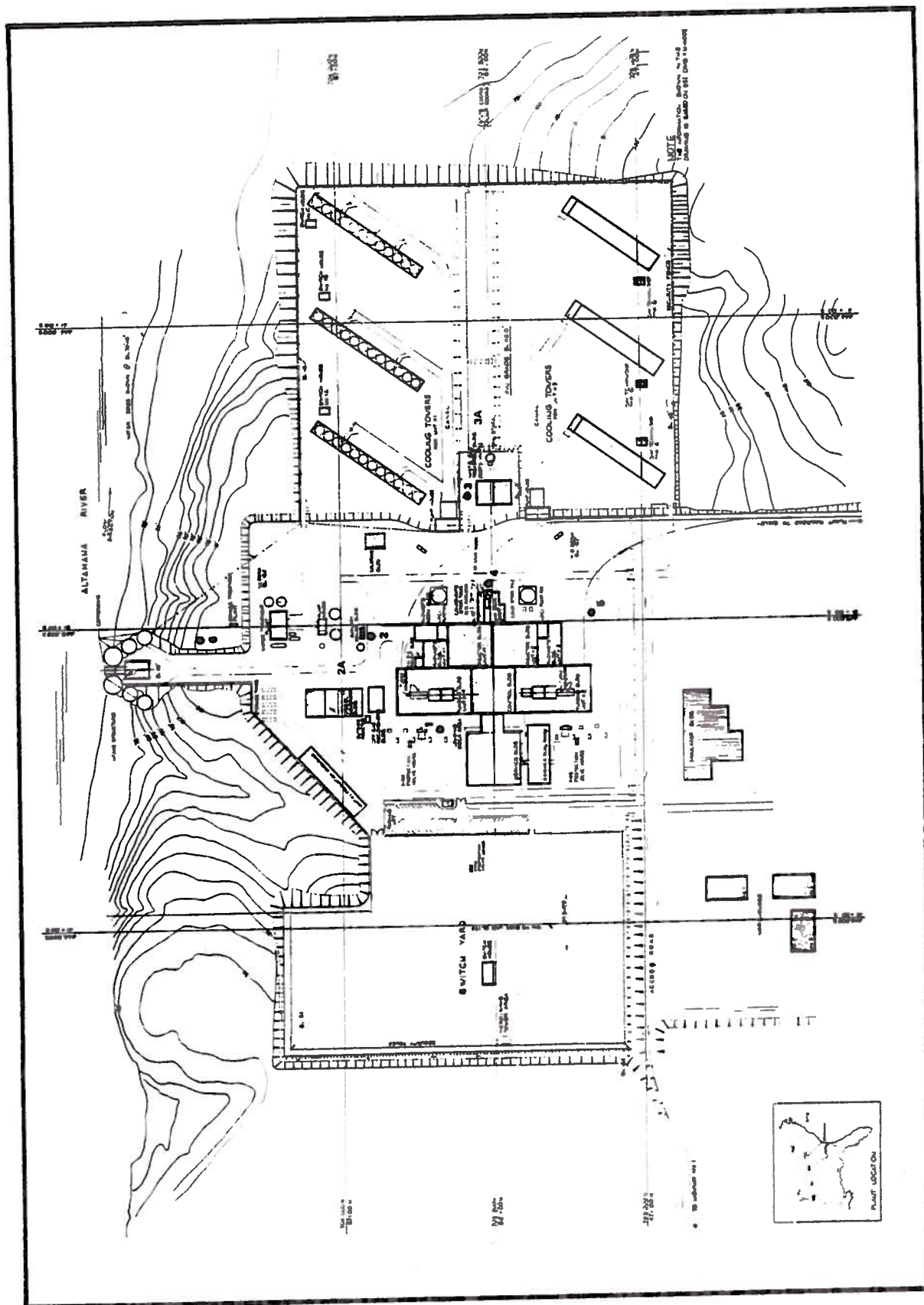


Figure 2-2. Plot Plan (Units 1 and 2)



### Section 3

#### PLANT SEISMIC DESIGN BASIS

Appendix A contains a comprehensive description of the seismic design basis of Plant Hatch, Unit 1. A brief summary description of the seismic design basis for Unit 1 structures and equipment is presented here. Plant Hatch Unit 1 was designed in the late 1960s and early 1970s.

##### 3.1 SEISMIC INPUT

Plant Hatch Unit 1 was designed using a Housner type spectrum with a peak ground acceleration (pga) of 0.08 g and 0.15 g for the operating basis earthquake (OBE) and the design basis earthquake (DBE), respectively. Figures 3-1 and 3-2 are plots of the OBE and DBE design ground response spectra. It should be noted that some seismic class 1 structures are shared with Unit 2 and are also analyzed and designed to meet Unit 2 seismic design bases. The shared structures of interest are the control building, the diesel generator building, and the river intake structure. The Unit 2 design spectrum is a modified Newmark spectrum with the same pga values as the Unit 1 seismic design bases. The Newmark spectra have higher spectral amplification than the Housner spectra. The design spectra are specified as free field ground spectra at grade. The actual input motion for soil-structure interaction (SSI) analyses was conservatively placed at the foundation level of each building.

##### 3.2 DESIGN AND ANALYSIS OF STRUCTURES

Safety-related structures that are part of this SMA are founded on soil. As previously mentioned, the input motion for SSI analyses was conservatively placed at the foundation level of each structure. In 1968, a refraction survey of the site was done from which a single (average) composite value of shear wave velocity of 2450 ft/s was obtained for characterization of the soil used in the original SSI analyses. The only purpose of the refraction survey was to obtain a shear wave velocity to use in the SSI analysis. The refraction survey results to the information from the many soil borings made in the plant area were not

correlated. Later reviews, as discussed in section 8.2, indicated that a lower composite value of shear wave velocity was appropriate. The original SSI approach was based upon the elastic half-space theory which, at the time of these original analyses, was the standard analysis approach to address SSI.

Two-dimensional horizontal lumped-mass stick models with soil springs were developed for each seismic class 1 structure. A vertical dynamic analyses was not performed. Response spectrum analyses were performed to calculate seismic forces on structures. Time history analyses of these building models were performed to obtain horizontal in-structure response spectra (IRS). The vertical IRS were specified as 2/3 the horizontal design ground response spectra. As discussed in Appendix A, the original time history was the scaled 1940 N-S El Centro time history record that conservatively enveloped the plant design ground response spectra. In 1985, the IRS were redone using synthetic time histories that more closely enveloped the plant design response spectra. The response spectra were smoothed and peaks broadened by  $\pm 10$  percent in the frequency range.

Forces resulting from one horizontal earthquake direction were combined absolutely with the forces resulting from the vertical earthquake. The maximum damping used was 3 percent for structures and 4.5 percent for soils for OBE and 5 and 5.5 percent, respectively, for DBE.

A recent review of the Hatch Unit 1 seismic analysis for the NRC (1) has shown these calculated responses to be very conservative. This conservatism was principally the result of the treatment of SSI radiation damping effects.

Concrete structures were designed to meet the requirements of ACI 318-63 although special reinforcement detailing requirements were followed to produce ductile type behavior. Steel structures were designed using the requirements of the AISC specifications, 1963, from the AISC Manual of Steel Construction, sixth edition.

The reactor building and the control building are reinforced concrete shear wall type structures with rigid base mats. Both structures have steel frame superstructures with cross bracing to support bridge cranes and the roof loads. The diesel generator building and the intake structure are concrete shear wall structures with rigid base mats. As stated before, all these structures are supported on soil.

### 3.3 MECHANICAL EQUIPMENT AND PIPING

All safety-related mechanical equipment was qualified by analysis for both OBE and DBE, with a limited number qualified by testing. Pressure boundaries of safety-related systems were evaluated to the criteria of the appropriate ASME code sections. Large-bore piping was dynamically analyzed for seismic response. Two-inch and smaller seismic class 1 piping is supported using standard spacing to produce natural frequencies above the earthquake frequency range of interest. In some cases, a simple conservative static analysis was performed.

### 3.4 ELECTRICAL AND CONTROL EQUIPMENT

Electrical equipment was primarily qualified by shaker table testing of prototype components with input consisting of harmonic sine beat, or similar, motions compatible with the appropriate support motion. Even though Unit 1 did not commit to IEEE 344-1971, since most of the equipment was ordered before this standard was issued, similar Unit 2 equipment was qualified to this standard. Some passive electrical equipment was qualified by either dynamic analysis or the conservative static coefficient method. Most cable tray supports were qualified by dynamic analysis with some supports qualified using the conservative static coefficient method.

### 3.5 SYSTEM INTERACTION

Safety systems are separated by structural barriers to provide redundancy in the event of flood or fire. Non-class 1 seismic equipment whose failure could effect class 1 seismic equipment was designed to meet II/I criteria.

### 3.6 REFERENCES

1. Idriss, I. M., Moriwaki, Y., Smith, M. G. Seismic Margin Assessment Issues Related to Soils and Earthquake Ground Motions, Georgia Power Company's Edwin I. Hatch Nuclear Power Plant, Appling County, Georgia. Woodward-Clyde Consultants for Southern Company Services, Inc., April 28, 1988.

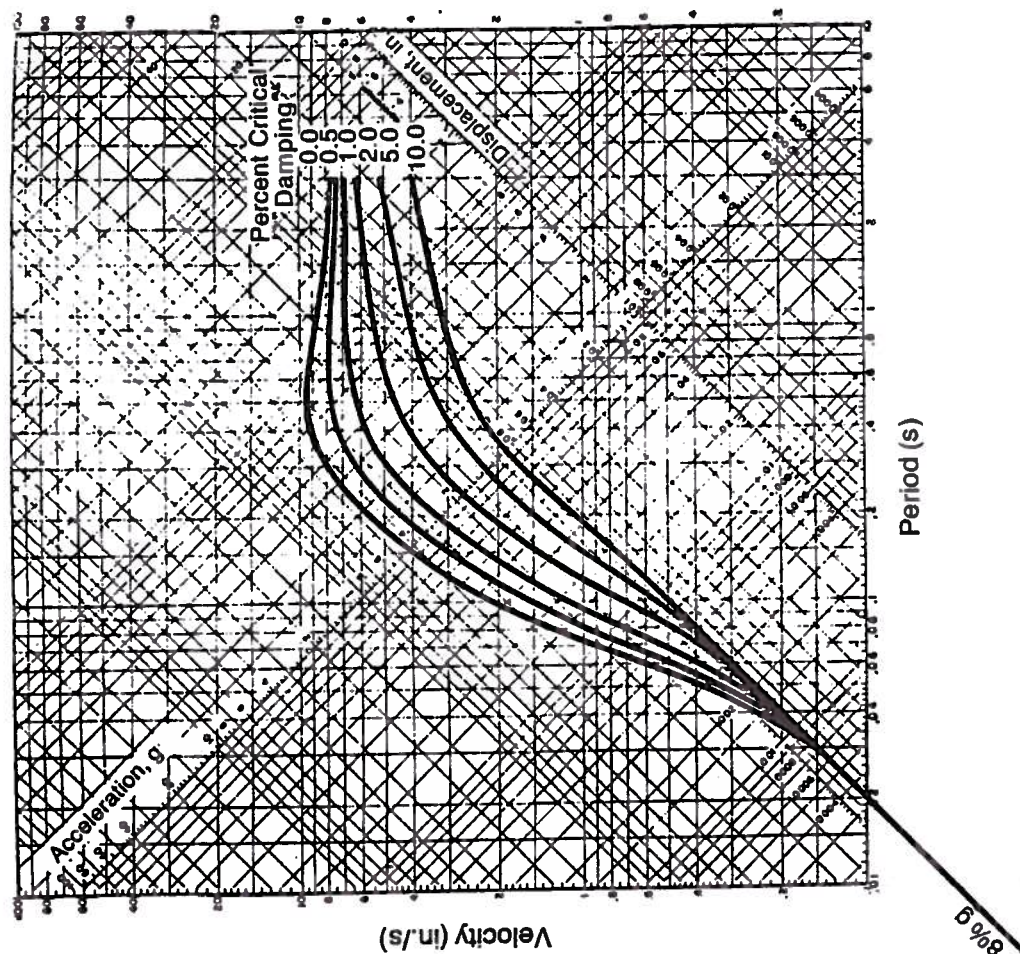


Figure 3-1. Plant Hatch Unit 1 Operating Basis Earthquake  
Design Ground Response Spectra



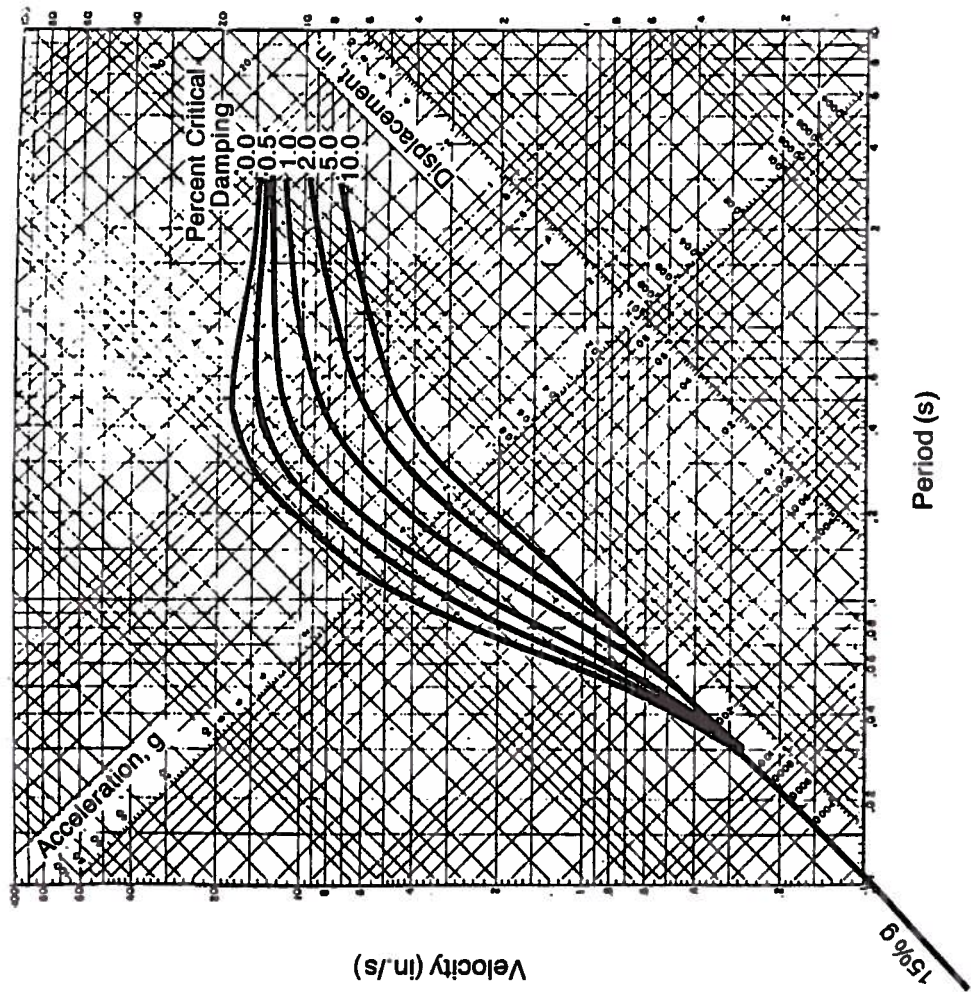


Figure 3-2. Plant Hatch Unit 1 Design Basis Earthquake  
Design Ground Response Spectra

## Section 4

### SEISMIC MARGIN EARTHQUAKE SELECTION

The first step in the seismic margin analysis (SMA) procedures is the selection of the seismic margin earthquake (SME). Basically, there are four alternate approaches given in the methodology (1). The choice of an approach will depend to a great extent on the end purpose of the SMA. The SME level should be set sufficiently high so that some plant components in the success path are found to have high-confidence low-probability-of-failure (HCLPF) capacity levels less than the SME level, thus identifying weaker link components. However, the SME level should not be set so high as to result in a substantial increase in the workload for the SMA. Considering both these goals, the methodology recommends that for plants with safe shutdown earthquake (SSE) levels of about 0.2 g or lower that the SME level should be set at about 0.3 g peak ground acceleration (pga). Typically, the SME is specified in terms of a smooth broad-frequency response spectrum shape.

No matter how the SME is developed, the HCLPF capacities of the components and the plant are estimated based on the following basic assumption.

At each natural frequency, and in each direction, there is approximately a 16 percent probability that the response spectrum ordinate will be exceeded if the SME occurs.

Therefore, the selection of the SME should be consistent with the conservatism relative to this assumption.

The fourth approach of selecting the SME was used for the Plant Hatch SMA. This approach is to select a standard (non-site-specific) SME spectrum that is negotiated with the NRC. Dr. I. M. Idriss of Woodward-Clyde Consultants (WCC) developed the SME for Plant Hatch. The SME was developed using a pga of 0.3 g. The spectrum shape was based on NUREG/CR-0098 (2) median centered spectra but modified for magnitude effects using the procedure proposed by Idriss (3). The adjustments were made to reflect a magnitude of  $m = 6.25$ . The values of  $v/a = 100$  cm/s/g or 39.4 in/s/g and  $ad/V^2 = 5$  were selected. The spectra amplification factors were those specified in NUREG/CR-0098 for a median spectra shape. Figure

4-1 is the 5 percent damped SME ground response spectrum. This spectrum is specified at grade in the free field. The vertical SME ground response is equal to  $2/3$  the horizontal ground response spectrum.

For comparison purposes, Figure 4-1 also shows two additional spectra. The spectrum identified as DBE is the Plant Hatch Unit 1 design basis earthquake ground response spectrum used as the earthquake design basis for the plant. The DBE spectrum is a Housner spectrum with the peak ground acceleration of 0.15 g. The plot shows that the SME spectrum envelopes the DBE spectrum by a factor of 2 or more. This comparison demonstrates that the SME will significantly challenge the plant. Recently, the EPRI Seismic Owners Group (SOG) completed the uniform seismic hazard calculations for plant sites of the member utilities. The Plant Hatch 85 percent nonexceedance probability (NEP) uniform hazard spectra for a  $10^{-4}$  annual frequency of exceedance is plotted for information. Here again, the SME significantly exceeds this hazard curve. The EPRI SMA methodology (1) does provide the application of seismic hazard curves as alternatives for the selection of the SME level. The hazard curves were not considered for the Plant Hatch Unit 1 SME because they were not available when the SME was selected.

Synthetic accelerograms were developed that have a zero period acceleration (ZPA) of 0.3 g and spectral ordinates that provide a reasonable fit to the smooth response spectrum. Figure 4-2 is the accelerogram of one of the two horizontal time histories. Figure 4-3 is a comparison of the spectrum of horizontal time history to the SME spectrum. In addition, the power spectra density (PSD) and the cumulative plots were developed to assure adequate energy over the frequency range of interest for the synthetic time histories. Figure 4-4 is the PSD of one of the horizontal time histories scaled to a pga of 1.0 g. The adequacy of the PSD was demonstrated by comparing it to the average PSD of 14 records of the 1971 San Fernando earthquake. The average PSD of these 14 records scaled to a pga of 1.0 g is given in Figure 4-5.

The PSD of the SME time history is approximately the same or exceeds the average PSD of the 14 records of the 1971 San Fernando earthquake over the frequency range of interest. Of special interest is the low frequency range of the maximum response of the reactor building and the control building, which is around 1 to 3 Hz. For even this low frequency range, the PSD of the SME time history exceeds that of the average PSD of the 14 records of the 1971 San Fernando earthquake. The fact that the PSD of the SME time history doesn't envelope below 0.7 Hz is not

judged to be a deficiency of the SME time history and would not have any significant effect on the reported SMA results since there is no structure, equipment, or subsystem that would have a natural frequency below 0.7 Hz.

In addition, Figure 4-6 shows the cumulative PSD for the synthetic time history compared to the cumulative PSD for the average PSD of the 14 records from the San Fernando earthquake. As can be readily seen in this figure, the cumulative PSD for this time history builds at a faster rate (i.e., greater power) for frequencies higher than about 0.7 Hz. The only reason the cumulative PSD of the average PSD of the San Fernando records exceeds the cumulative PSD of the SME time history below about 1 Hz is that the average PSD of the San Fernando records had more energy below 0.7 Hz, which is below the frequency range of possible structural and equipment response. Moreover, the absolute value of the cumulative PSD for this time history is approximately 50 percent greater than that for the average of the 14 San Fernando records. The San Fernando earthquake had a surface wave magnitude of 6.6 and a moment magnitude of 6.5. The SME for Plant Hatch is postulated to have a moment magnitude of 6-1/4; i.e., about 1/4 magnitude lower than the San Fernando earthquake.

Based on the fact that the PSDs for the selected synthetic time histories are comparable to or exceed those for the average of the 14 San Fernando records, and there is a reasonable fit of the spectra of the synthetic time histories to the SME spectrum, the selected synthetic time histories are judged to be fully adequate.

The SME is specified at the free field ground surface. The effect of the spatial variation of the ground motion is discussed in section 5.1.

Two horizontal and one vertical synthetic time histories were developed to be statistically independent of each other. The absolute value of the correlation coefficient was less than 0.16.

Appendix C should be referred to for more details. This selection of the SME is described in the WCC soils evaluation report dated April 28, 1988, and the development and evaluation of the adequacy of the synthetic time histories given in Enclosure A, Adequacy of Power Spectra Density of Synthetic Time History Used in the Seismic Margin Assessment to Represent the Seismic Margin Earthquake.



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Palo Alto, CA: Electric Power Research Institute, October 1988.  
NP-6041.
2. N. M. Newmark and W. J. Hall. Development of Criteria for Seismic Review of Selected Nuclear Power Plants. Nuclear Regulatory Commission, May 1978. NUREG/CR-0098.
3. I. M. Idriss, Y. Moriwaki, and M. G. Smith. Seismic Margin Assessment Issues Related to Soils and Earthquake Ground Motions, Georgia Power Company's Edwin I. Hatch Nuclear Power Plant, Appling County, Georgia. Woodward-Clyde Consultants for Southern Company Services, Inc., April 28, 1988.

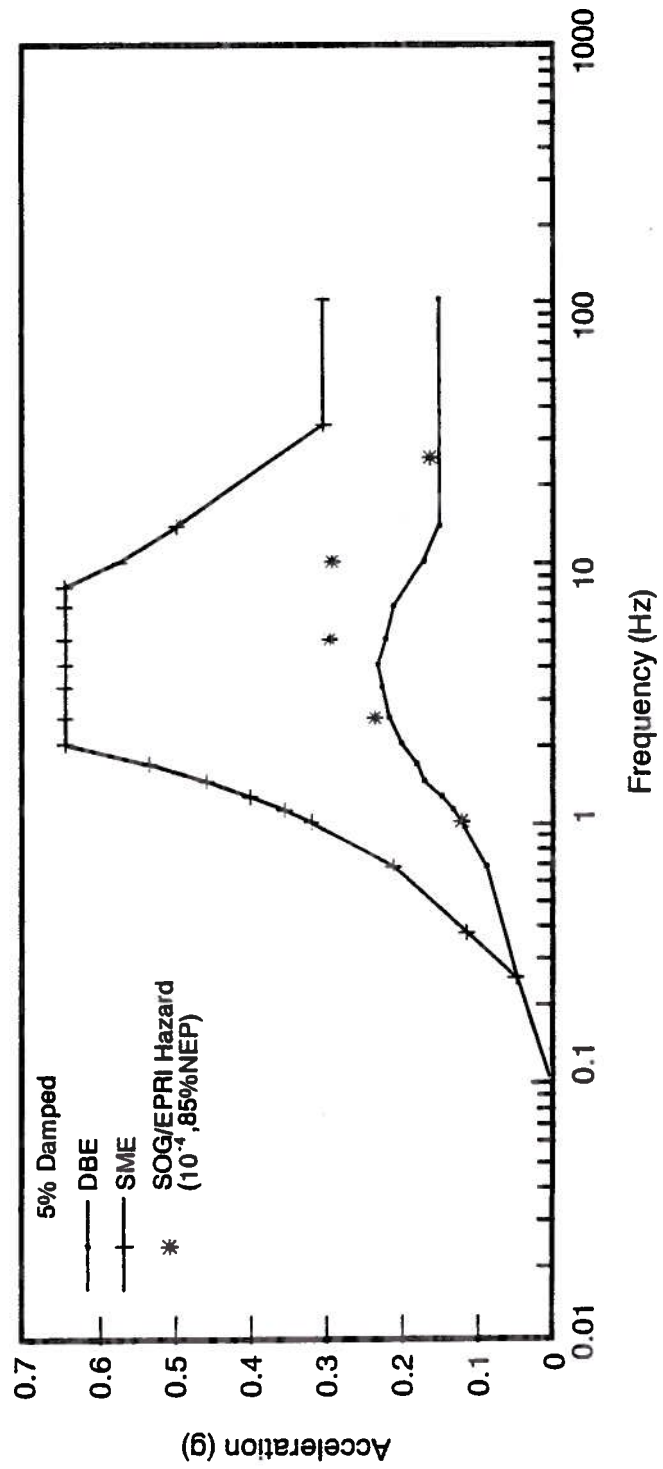


Figure 4-1. Plant Hatch Unit 1 Free Field Ground Response Spectrum

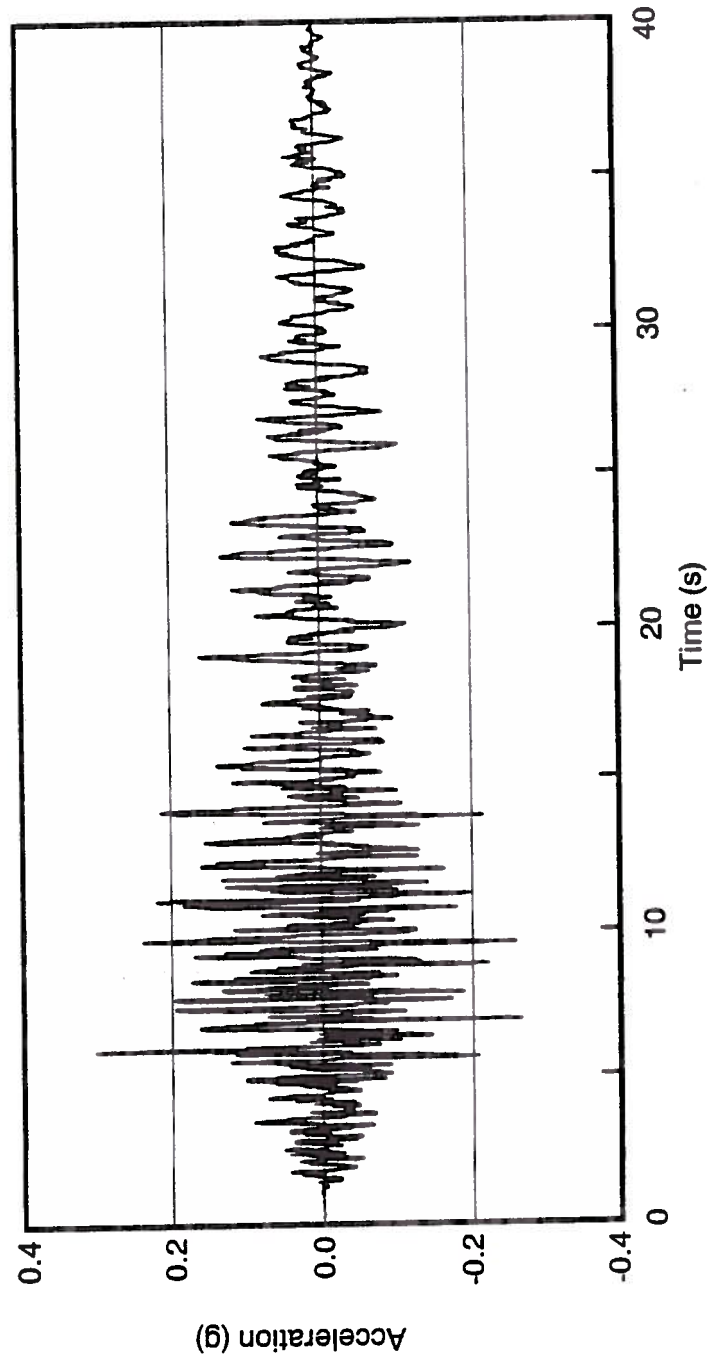


Figure 4-2. Accelerogram of Synthetic Time History H-1

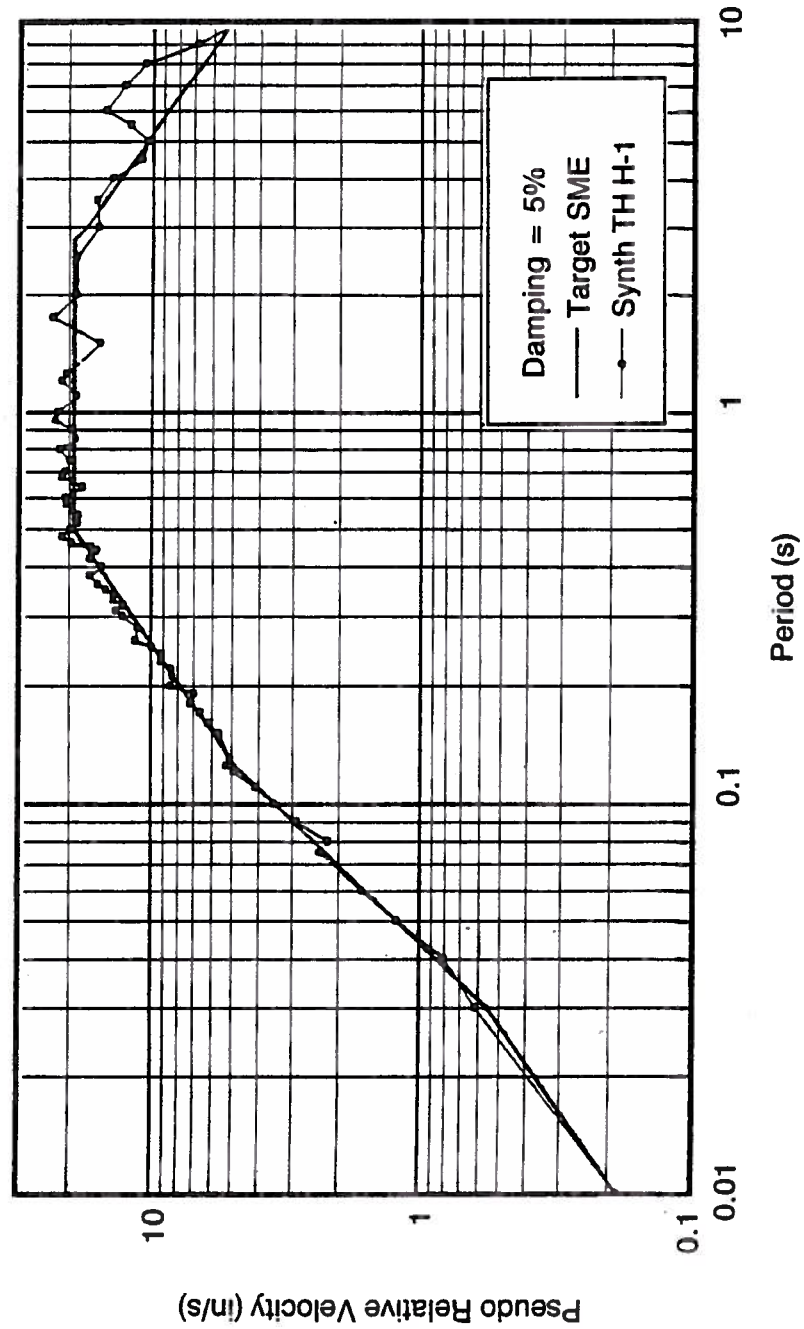


Figure 4-3. Plant Hatch Unit 1 - SMA Comparison of Target Smooth Spectrum and Spectrum for Synthetic Time History H-1

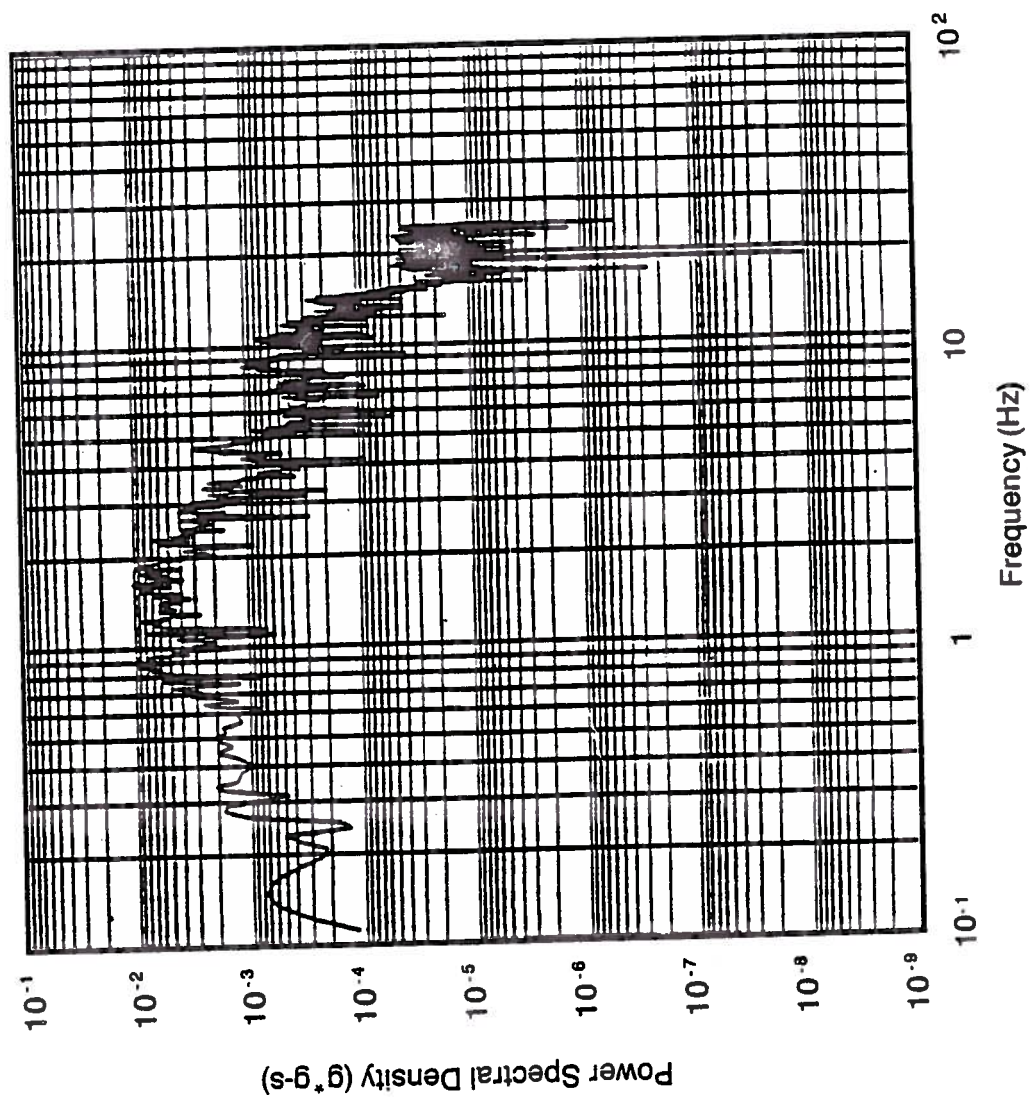


Figure 4-4. Power Spectral Density - Synthetic Time History H-1

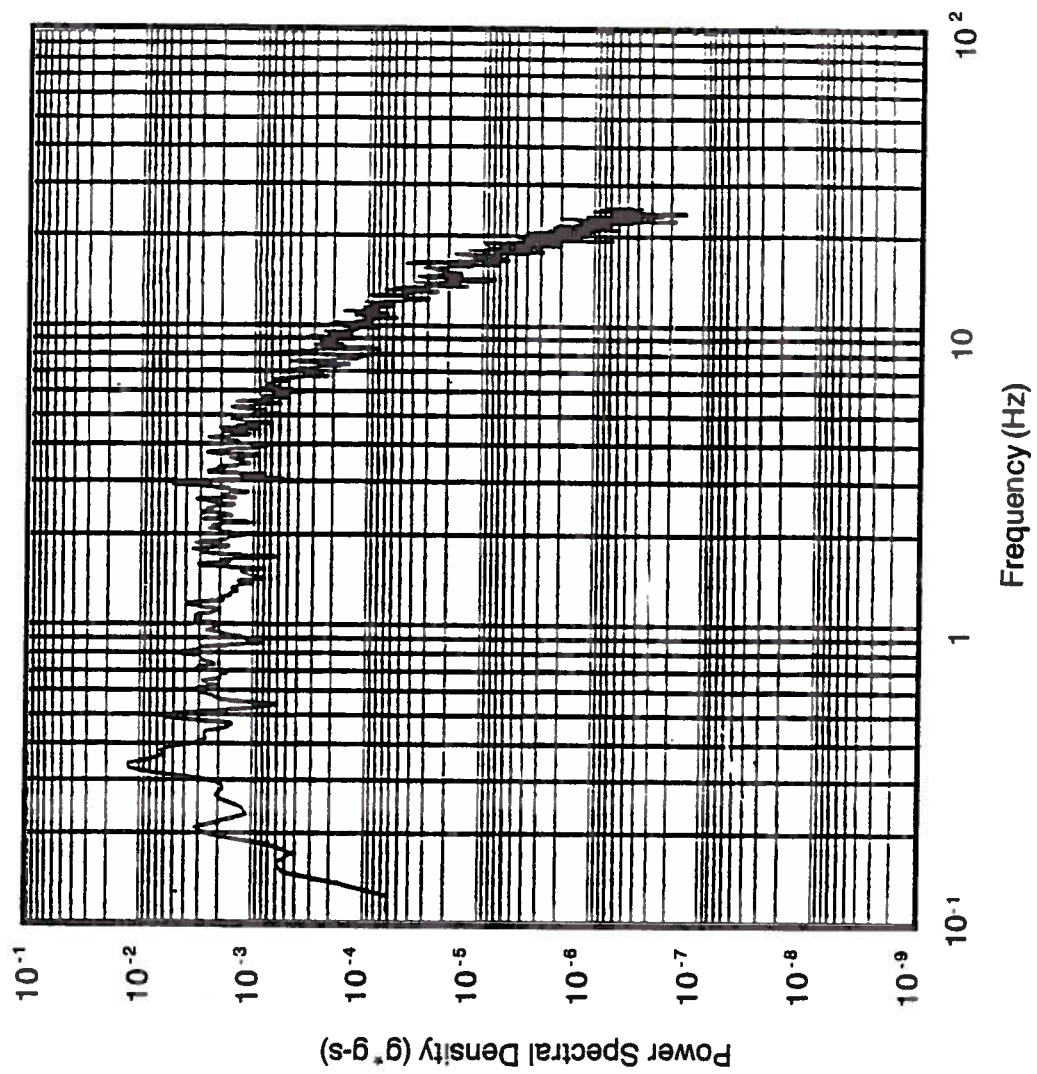


Figure 4-5. PSD - Average of 14 Records, 1971 San Fernando Earthquake

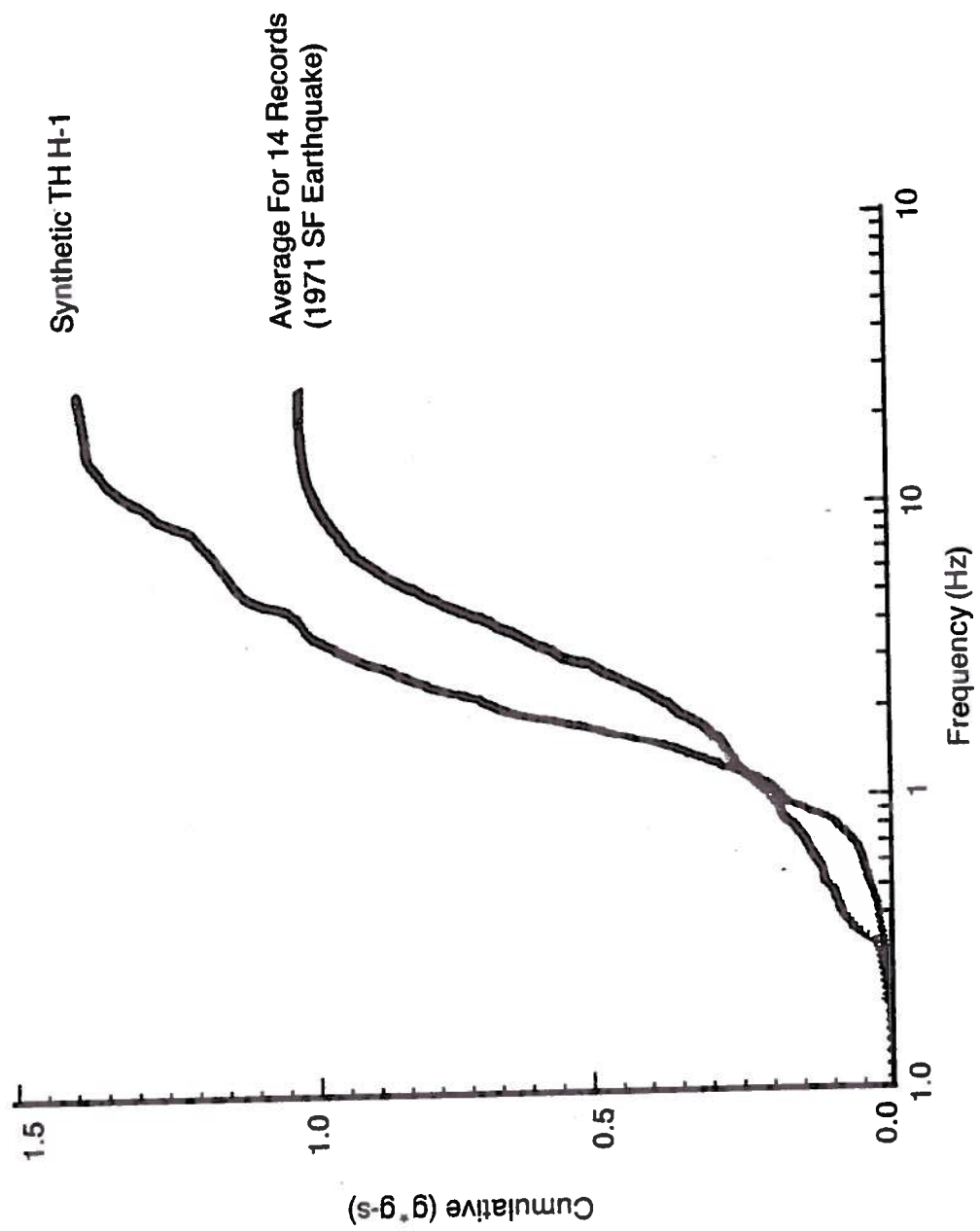


Figure 4-6. Cumulative PSD for Synthetic Time History and Average of 14 Records

## Section 5

### DEVELOPMENT OF IN-STRUCTURE RESPONSE SPECTRA

Plant Hatch buildings are founded on soil and some are significantly embedded in the soil. Both the frequency and the amplitude of response to seismic excitation for these buildings are greatly affected by soil-structure interaction (SSI). To get the predicted structural and equipment response for the seismic margin earthquake (SME) to be median centered, proper consideration of SSI is required. The original SSIs of the Hatch buildings were performed conservatively. For example, spatial variation of the ground motion with depth was not used and the limitation on soil damping, including effects of radiation damping, was set not to exceed 5.5 percent of critical damping. A recent review of the seismic analysis of Plant Hatch Unit 1 by the NRC (1) indicated significant conservatism in the original SSI analysis procedures. Based on the conservatism of the original SSI analysis and other parameters originally used, scaling the original in-structure response spectra (IRS) was not appropriate. Therefore, Dr. J. J. Johnson of EQE, Inc., was contracted to generate the median-centered responses of the major structures for the SME.

The major element in identifying seismic margin for the Hatch SMA is to obtain median-centered response from properly executed SSI analyses. This effort has a significant impact on the results of the Hatch SMA.

The soil profiles and their variation were developed by Dr. I. M. Idriss of Woodward-Clyde Consultants (WCC). SSI analysis follows the SMA methodology (2) that was partially written by Dr. Johnson. Dr. Johnson and O. R. Maslenikov of EQE, Inc., developed the specific SMA SSI analysis approach for Plant Hatch, performed the SSI analyses, and calculated the structural responses, in particular, the IRS. The IRS are being used to assess the high-confidence low-probability-of-failure (HCLPF) level of components and equipment supported in the buildings. To properly calculate the HCLPF levels for the plant, the calculated responses should be median-centered. Therefore, the SSI evaluation, structural models, and parameter values used are median-centered.



The intent of this section is to provide an overview of the SSI analysis and development of the IRS. Appendix B has also been added to provide copies of all the 5 percent damped SME IRS as well as a summary report of the SSI seismic response analyses. This appendix should be referred to for additional information.

## 5.1 SOIL PROFILES AND THEIR VARIATION

The development of the basic plant soil profiles is discussed in section 8.2.1. For this discussion, the soil profiles from section 8.2.1 are referred to as best estimate or average. This section briefly addresses the development of the lower, intermediate, and upper soil profiles for the SSI analyses of each of the Plant Hatch major structures. Appendix C of this report contains a WCC report titled "Dynamic Soil Properties for SSI Analyses," which provides a more detailed discussion and tabulates the soil profiles that were used for the SSI analyses of each building.

The strain-compatible dynamic soil properties were obtained from the response analyses of Profiles I and II using the computer program SHAKE (3). The synthetic time history was applied at the ground surface. The strain-compatible shear moduli were obtained by using the average modulus reduction for sands (4). Strain-compatible values of material soil damping were computed using the average damping curve as well as the lower range damping curve for sands (4).

Three additional sets of moduli were also analyzed. One set was considered a reasonable lower range and was set at 0.75 of the best estimate values. Another set, considered a reasonable upper range, was set at 1.8 times the best estimate values. Finally, an intermediate range was obtained by using 1.2 times the best estimate values. The results of the SHAKE analyses are included in Appendix C.

The results of these analyses of the strain-compatible dynamic soil properties were evaluated. The strain-compatible moduli were not significantly affected by the damping curve used. Also, for the cases in which the same damping curve was used, the strain-compatible damping ratios for different sets of moduli were almost identical.

The results of these analyses were used to develop recommendations for strain-compatible modulus and soil material damping values for use in the SSI analyses for the intermediate, lower, and upper range modulus cases.

The recommendations consisted of an intermediate soil profile (shear modulus and soil material damping percentage for soil layers having depths of 10 ft or more). The lower and upper range modulus cases were specified as 0.6 and 1.6 times the modulus values of the intermediate case. The use of the 0.6 and 1.6 times the strain-compatible intermediate modulus corresponds reasonably well to the use of 0.75 and 1.8 times the low-strain best estimate modulus, respectively. The soil material damping percentages listed for the intermediate case are the average of the strain-compatible damping values obtained using the the lower range damping curve and those using the average damping curve. The intermediate case soil material damping was also recommended as the damping to be used for the lower and upper bound cases. For an example, Table 5-1 is the recommended strain-compatible intermediate case for the control building.

The EPRI SMA methodology recommends that median-centered SSI evaluations, structural models, and parameter values be used. Median-centered SSI evaluations require that full credit be taken for vertical spatial variation of ground motion, kinematic interaction, and radiation of energy from the structure into the soil. The procedures and parameter values are to be median-centered. However, considerable uncertainty exists in soil-structure system frequency estimates and in items such as vertical spatial variation of free-field ground motion. The SMA needs to account for this uncertainty. One mechanism to do so is to shift soil stiffness properties over a range to encompass the effects of approximately plus--and minus--one standard deviation parameter variation. The large variation of the soil dynamic properties discussed above properly addresses these uncertainties.

## 5.2 STRUCTURAL MODELS

The original two-dimensional seismic models were reviewed to ensure they were adequate for determining the responses for the SME. Two of the structures, the control building and the river intake structure, could potentially have significant torsional responses and, therefore, these models were converted to three-dimensional models. The other two structures, the reactor building and the diesel generator building, are essentially symmetrical and, therefore, the use of two-dimensional models was sufficient. In addition, over the past several years, the mass and stiffness characteristics of the seismic models have been reviewed and have been upgraded to reflect as-built conditions. Therefore, no additional reviews were necessary. Also, the

stiffness of concrete was judged to be a second-order effect on response compared to the effect of soil stiffness. The large variation of soil stiffness would largely mask the effects of reduced concrete stiffness caused by cracking. Therefore, consideration for the case with cracked concrete properties was not justified.

Structural damping was also evaluated for use in the structural response analysis to obtain seismic demand estimates that are essentially median centered. A rough estimate of the SME seismic forces and stresses was compared to the original values. Based on this evaluation and the damping values recommended in Table 4-1 of the EPRI Methodology (2), a conservative estimate of the median damping value was selected as 7 percent of critical. This structural damping value was used for all buildings except the roof structure portion of the reactor building. The roof structure is a bolted steel structure that begins at and extends approximately 50 ft above the refueling floor. It supports the reactor building roof, the building crane, and the precast concrete siding. The roof structure will experience inelastic action at the SME level; therefore, 10 percent damping was specified for this portion of the reactor building. This elastic damping value will not be increased to account for the inelastic energy dissipation of the roof structure.

The seismic building models, and their associated fixed base natural frequencies, modal damping values, and percent effective masses can be found in Appendix B.

### 5.3 SOIL-STRUCTURE INTERACTION APPROACH

The SSI and structure response of the Hatch buildings was analyzed using the substructure approach as implemented in the CLASSI system of programs. The substructure approach separates the SSI problem into a series of simpler problems, solves each independently, and superimposes the results. The elements of the substructure approach, as applied to structures with rigid bases subjected to earthquake excitations, are as follows:

- specifying the free-field ground motion
- defining the soil profile
- calculating the foundation input motion

- calculating the foundation impedances
- determining the dynamic characteristics of the structure
- performing the SSI analysis; i.e., combining the previous steps to calculate the response of the completed soil-structure system.

The soil profiles and the structural models are discussed in sections 5.1 and 5.2, respectively. This approach is basically the approach used by Dr. J. J. Johnson of EQE, Inc., for the SSI blind prediction analysis for the NRC/EPRI seismic experiment at Lotung, Taiwan.

The free-field ground motion is specified at the free surface at the top of finished grade. The control motion is specified as three acceleration time histories that are essentially statistically independent. The spatial variation of motion is defined by vertically propagating waves.

Foundation input motion differs from the free-field ground motion except for surface foundations subject to vertically incident waves. For one, the free-field motion varies with soil depth. For another, the soil-foundation interface scatters waves because points on the foundation are constrained to move according to its geometry and stiffness. The foundation input motion is related to the free-field ground motion by means of a transformation defined by a scattering matrix. The reactor building and the intake structure were analyzed as embedded foundations which require the scattering matrixes. The control building is not as deeply embedded and the diesel generator building is even less embedded. Therefore, for these two buildings, the foundation motion was identical to the free-field ground motion. This simplification is somewhat conservative.

Foundation impedances describe the force-displacement characteristics of the soil. They depend on the soil configuration and material behavior, the frequency of the excitation, and the geometry of the foundation. CLASSI was used to generate the foundation impedances for all structures.

Uncertainties in the SSI analyses are documented in the literature (2, 5, 6). For the Hatch SMA, the effect of uncertainties was accounted for in the SSI analyses by varying the soil shear modulus. This method has been acceptable as specified in Reference 5 and in the SMA methodology. The adequacy of this process to account for uncertainty associated with deconvolution is demonstrated by the

fact that the resulting envelope spectrum at the foundation level of the reactor building (Figure 8-20) generally exceeds 60 percent of the surface free-field motion.

In addition to the variation of the soil shear modulus, sensitivity studies were conducted to evaluate the effect of partial embedment of the reactor building and to determine the proper treatment of the free-field control motion at the intake structure. For the reactor building, the SME was specified at the free field ground surface to properly account for spatial variation of ground motion, but no bonding of the side soil to the structure was assumed since the reactor building is in contact with side soil only on its east side and partially on its north side. The modeling approach was judged to be more realistic than assuming all sidewalls are fully bonded to the soil. To account for the uncertainty of the effective embedment, the requirement was imposed that the spectra be broadened an additional plus 10 percent in the frequency domain for subsystems whose fundamental natural frequency is approximately 2 to 3 Hz. This accounts for a possible shift of the major spectral peak of the IRS because of a stiffer foundation caused by side soil contact on a portion of the embedded reactor building. For the intake structure, the input motion was defined at the free field of the berm as opposed to the free field of the riverbed since, in general, the responses of the structure were higher for input motion at the berm.

#### 5.4 SSI RESULTS

As previously mentioned, the purpose of a new SSI analysis is to obtain the median response of the buildings for the SME. These median responses were developed as IRS. They were calculated for the three different soil profiles and the three different directions. The format for plotting was to overplot the response spectra for the three soil profiles while holding all other parameters constant.

The spectra were developed for three different damping values, 3, 5, and 10 percent. This spread of the spectral damping was adequate to perform HCLPF calculations. The IRS used for the seismic margin assessments were the envelope of the three IRS associated with the lower bound, the intermediate, and the upper bound soil modulus profiles. The IRS for the intermediate soil modulus profile was broadened by  $\pm 15$  percent in the frequency domain. Thus the IRS used to evaluate equipment and subsystems for a given damping value, direction, and location is a single envelope of the following three overlaid IRS: the IRS for the



lower bound soil profile, the IRS for the intermediate soil profile broadened by  $\pm 15$  percent in the frequency domain for all frequencies, and the IRS for the upper bound soil profile. This follows the procedures discussed in reference 5.

Figure 5-1 is an example of the reactor building SME IRS. It is a 5 percent damped N-S spectrum at el 158 ft, which is 28 ft above grade. The DBE (or SSE) IRS for the same location are shown for comparison. Figure 5-2 is an example of the control building SME IRS. It is a 5 percent damped N-S spectrum at el 164 ft, which is 34 ft above grade. The DBE IRS for the same location are shown for comparison. An example of the diesel generator building SME IRS is shown in Figure 5-3. It is the 5 percent damped N-S spectrum at el 130 ft, which is grade. Again for comparison purposes, the DBE IRS for the same location are shown. Figure 5-4 is an example of the intake structure SME IRS. It is a 5 percent damped E-W spectrum at el 109.75 ft, which is at the grade or berm adjacent to the intake structure. The DBE (or SSE) IRS for the same location are shown for comparison. The SME IRS at 5 percent damping for all building elevations are given in Appendix B. Note that the  $\pm 15$  percent peak broadening in the frequency domain of the SME IRS for the intermediate soil profile as discussed in the previous paragraph has not been shown in these figures or in Appendix B.

Some comments are in order to better understand these IRS. First, the DBE IRS were based on seismic models that used dynamic soil properties significantly different from those recently estimated for a 1987 NRC study of the seismic design of Plant Hatch (1) and the set of soil properties similar to those in the NRC study developed by WCC for the Hatch SMA (7). The soil properties used to develop the DBE IRS are significantly stiffer. The difference in soil stiffness is the major reason for the downward frequency shift of the IRS peaks for the SME IRS. Second, the peak spectral accelerations (SA) of the SME IRS have less amplitude in general than the peak SA of the DBE IRS, especially when adjusted for the fact that the DBE is a low amplitude Housner type ground spectrum with a pga of 0.15 g.

The comparisons of the peak SAs of the two spectra point out the large conservatism in the original SSI analysis. They also show that it was not possible to estimate median-centered response by scaling the original DBE IRS for Plant Hatch.

## 5.5 REFERENCES

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Table 5-1

Recommended Strain-Compatible Shear Modulus and Damping Parameters,  
Intermediate Range - Control Building

Layer No.	Depth <sup>(1)</sup>		Shear Modulus (2)	Damping Percent of Critical
	From	To		
1	0	10	6,140	1.9
2	10	20	5,360	3.5
3	20	30	4,975	4.4
4	30	40	4,550	5.3
5	40	50	4,280	5.9
6	50	55	4,160	6.2
7	55	60	1,230	10.5
8	60	70	1,345	10.4
9	70	80	1,493	10.3
10	80	90	1,668	10.1
11	90	105	1,928	9.7
12	105	120	2,305	9.1
13	120	130	2,715	8.4
14	130	140	9,190	4.6
15	140	160	11,178	4.2
16	160	180	13,838	3.9
17	180	200	16,708	3.8
18	200	230	18,930	3.9

- Notes: 1. feet below the ground surface  
2. kips/square foot



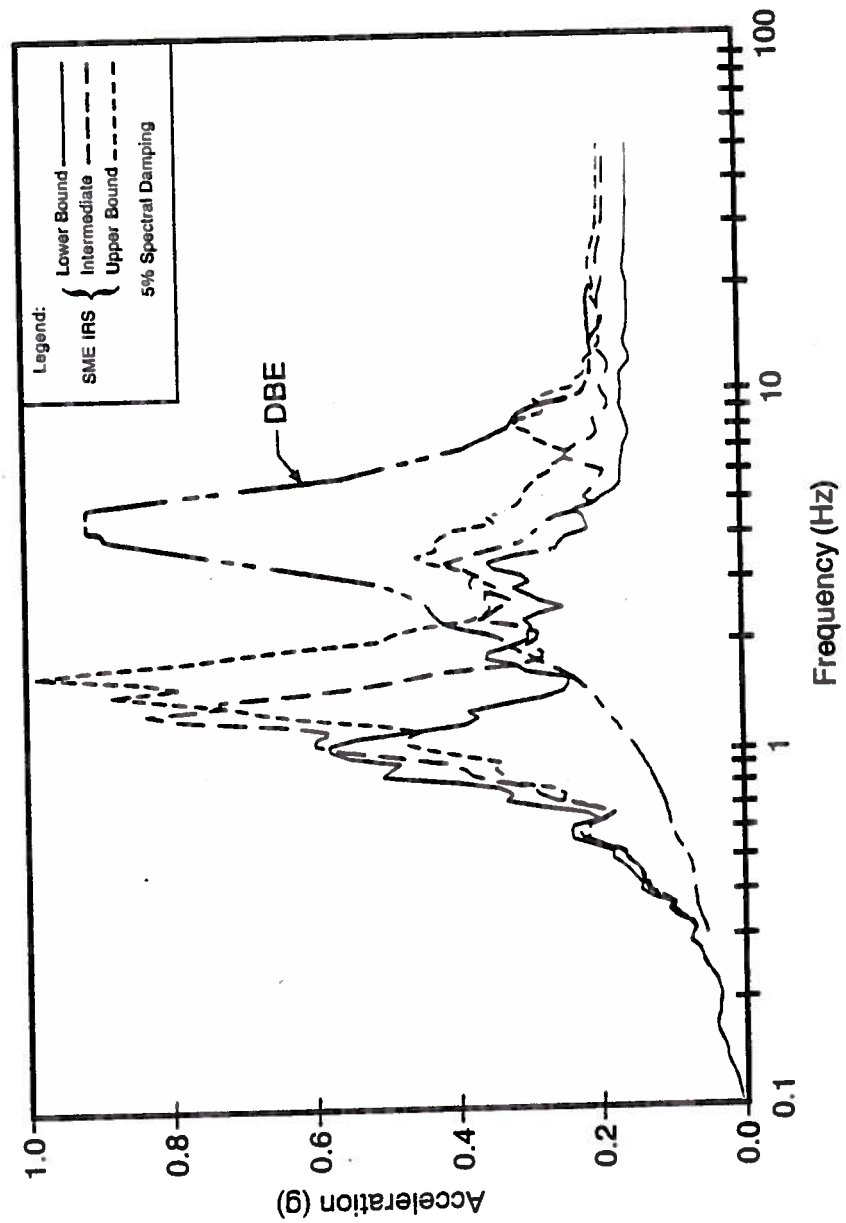


Figure 5-1. Reactor Building SSI Analysis In-Structure Response Spectra at Mass Point 3, el 158.0 ft, North - South Direction

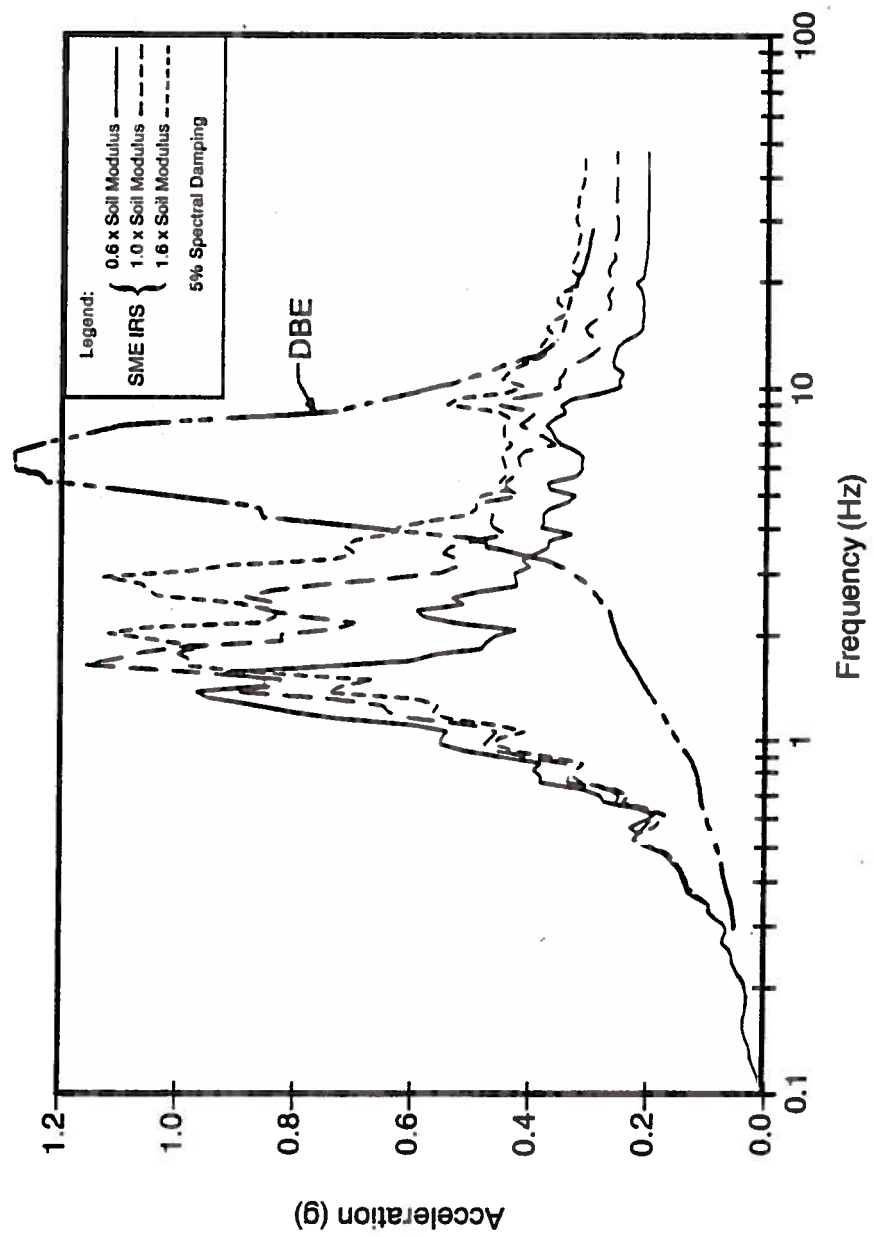


Figure 5-2. Control Building SSI Analysis In-Structure Response Spectra at Mass Point 11, el 164.0 ft, North - South Direction

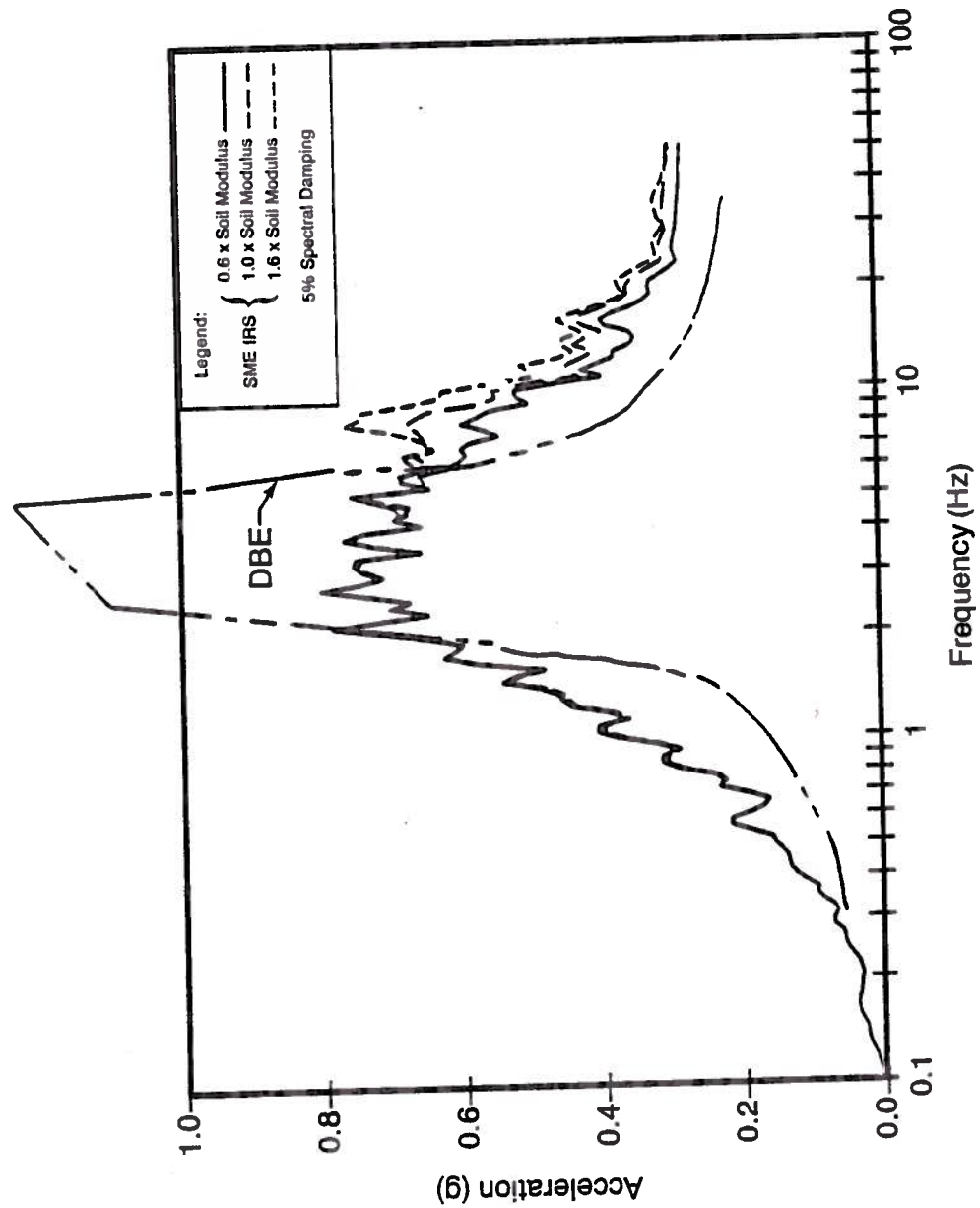


Figure 5-3. Diesel Generator Building SSI Analysis In-Structure Response Spectra at Mass Point 2, el 130.0 ft, North - South Direction

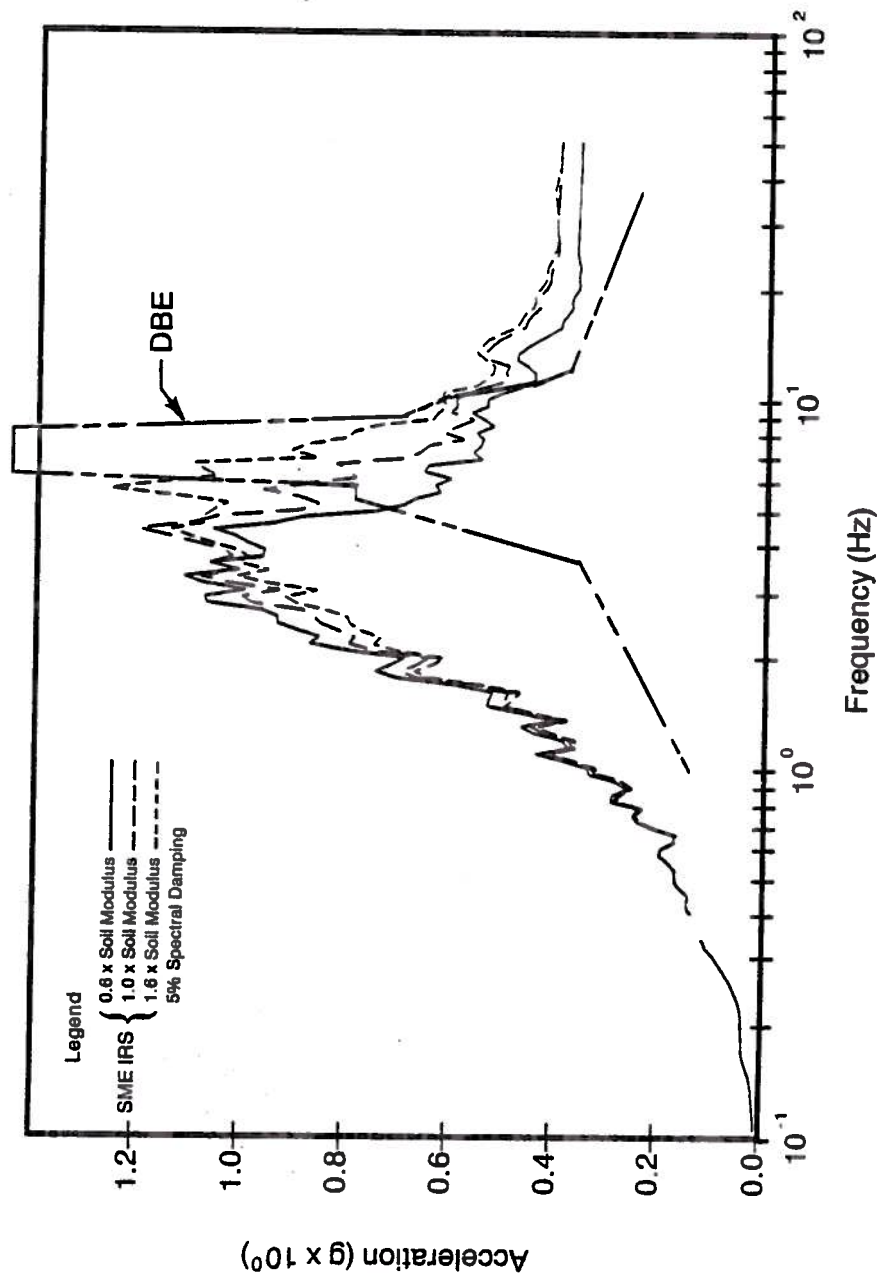


Figure 5-4. Intake Structure SSI Analysis In-Structure Response Spectra at Mass Point 15, el 109.75 ft, East - West Direction.

Hatch IPEEE Response

#### 4.9 THIRD-PARTY AUDIT

The third-party audit for Plant Hatch Unit 1 was originally conducted as part of the pilot plant SMA study from 1988 to 1989. This SMA study was part of a program that included combining compatible portions of the SQUG GIP with the EPRI SMA methodology. The NRC was actively involved in this program, including reviews by the NRC Seismic Design Margins Working Group, an NRC peer review group composed of industry experts, NRC staff, and an NRC consultant involved in the USI A-46 programs. Refer to References 9, 10, 11, 12, and 13 for information on the final report on the Plant Hatch Unit 1 SMA and the review reports from the NRC peer review group and the NRC Seismic Design Margins Working Group.

Due to changes to the SQUG GIP after the completion of the original Hatch Unit 1 USI A-46 program, the original Unit 1 USI A-46 evaluation was updated to comply with these changes. Therefore, an updated third-party audit was also required. This updated third-party audit for Hatch Unit 1 was performed by Dr. R. P. Kennedy of Structural Mechanics Consulting. Dr. Kennedy is a noted expert in the field of seismic design and analysis and served as chairman of the

SSRAP for USI A-46. The Unit 1 third-party audit consisted of a walkdown by Dr. Kennedy which included a representative sampling of SSEL components and electrical raceways. In addition, Dr. Kennedy reviewed the Unit 1 SEWS, sample calculations, outlier resolutions, and proposed modification packages. The results of Dr. Kennedy's third-party audit for Unit 1 are documented in a letter from Dr. Kennedy dated November 26, 1993, which is included in Appendix M. Dr. Kennedy's third-party audit report concludes with the following statement:

My overall conclusion is that the SRTs are conducting the A-46 and Seismic Margin reviews of Plant Hatch in a very professional and conscientious manner. These reviews are excellent and of the highest quality. I believe that the recommended outlier resolutions are sufficient to provide for the future seismic safety of Plant Hatch and that the conscientious efforts of the SRTs have resulted in important seismic improvements.