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SOUTHERN COMPANY
Energy to Serve Your World℠

July 16, 2007

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50-366

NL-07-1308

U. S. Nuclear Regulatory Commission
ATTN: Document Control Desk
Washington, D. C. 20555-0001

Edwin I. Hatch Nuclear Plant
Request to Implement an Alternative Source Term
Response to Request for Additional Information Regarding the Unit 1
Main Steam Isolation Valve Alternate Leakage Treatment Path Seismic Evaluation

Ladies and Gentlemen:

On August 29, 2006, Southern Nuclear Operating Company (SNC) submitted a request to revise the Edwin I. Hatch Nuclear Plant (HNP) licensing/design basis with a full scope implementation of an alternative source term (AST). By letters dated November 6, 2006, November 27, 2006, January 30, 2007, and June 22, 2007, SNC has submitted further information to support the NRC review of the HNP AST submittal. By letter dated April 10, 2007, the NRC requested additional information concerning the seismic evaluation of the Unit 1 main steam isolation valve alternate leakage treatment path, described in enclosure 8 of the referenced August 29, 2006 submittal, which is credited in the AST loss-of-coolant accident (LOCA) analysis.

The enclosure to this letter contains the SNC response to the referenced NRC request for additional information (RAI).

The 10 CFR 50.92 evaluation and the justification for the categorical exclusion from performing an environmental assessment that were included in the August 29, 2006 submittal continue to remain valid.

(Signature and affirmation are provided on the following page.)

Mr. L. M. Stinson states he is a Vice President of Southern Nuclear Operating Company, is authorized to execute this oath on behalf of Southern Nuclear Operating Company and to the best of his knowledge and belief, the facts set forth in this letter are true.

This letter contains no NRC commitments. If you have any questions, please advise.

Respectfully submitted,

SOUTHERN NUCLEAR OPERATING COMPANY

L. M. Stinson
Vice President Fleet Operations Support

Sworn to and subscribed before me this 16th *day of* July, *2007.*

Notary Public

My commission expires: July 5, 2010

LMS/CLT/daj

Enclosure: Response to Request for Additional Information Regarding the Unit 1 Main Steam Isolation Valve Alternate Leakage Treatment Path Seismic Evaluation

cc: Southern Nuclear Operating Company
Mr. J. T. Gasser, Executive Vice President
Mr. D. R. Madison, Vice President – Hatch
Mr. D. H. Jones, Vice President – Engineering
RType: CHA02.004

U. S. Nuclear Regulatory Commission
Dr. W. D. Travers, Regional Administrator
Mr. R. E. Martin, NRR Project Manager – Hatch
Mr. J. A. Hickey, Senior Resident Inspector – Hatch

State of Georgia
Mr. N. Holcomb, Commissioner – Department of Natural Resources

Enclosure

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Enclosure

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NRC QUESTION 1

It is stated on page 10 of Enclosure 8 of the August 29, 2006, application, that "While the Turbine Building was designed as a category II structure, to withstand the effects of the (Uniform Building Code) UBC earthquake, critical portions of the turbine building were also evaluated for (Operating Basis Earthquake) OBE and (Design Basis Earthquake) DBE seismic loads to ensure no collapse on category I structures or components." The Nuclear Regulatory Commission (NRC) staff requests the licensee to identify the critical portions of the turbine building, which were evaluated for OBE and DBE seismic loads, and describe how the OBE and DBE seismic loads were calculated and applied to these critical portions of the turbine building.

SNC RESPONSE

The purpose of section 1.1.6 of Enclosure 8 is to provide the Turbine Building load combinations used for design. These load combinations include UBC lateral seismic loads as one load combination and tornado loads as another load combination. Please note that for the Turbine Building tornado load combination the allowable stress discussion is provided in section 1.1.5.b(5). In addition, supplemental information was provided on load combinations that include OBE and DBE to address potential seismic II/I issues on a limited portion of the turbine building. As stated in section 1.1.6 these load cases were used for "evaluation." These evaluations were performed in the early phases of the turbine building design.

Evaluations were performed on two east-west (E-W) reinforced concrete frames that formed portions of the southeast portion of the turbine building. One frame is along column line T8 from column line TA to TD. The other frame is along column line T6 from column line TA to TD. The frames extend from the base slab up to the turbine generator operating deck. Above the turbine generator operating deck is the steel superstructure.

The purpose was to evaluate the reinforced concrete structural elements of that portion of the Turbine Building to provide assurance that no seismic Category I load combinations produced localized II/I issues if safety related cables were later routed through the Turbine Building east cable way. The cable way is the east hallway of the Turbine Building located at grade elevation. The hallway has reinforced concrete walls on both sides and a reinforced concrete floor 17 feet above.

Two-dimensional frame models of each of the two frames were developed for static and dynamic analysis. Based on these simple fixed base models, OBE and DBE seismic loads and displacements were calculated. The loads applied to the frames included dead load, equipment loads, live loads, loads from the superstructure steel columns supported by each frame, OBE and DBE loads from the frame models, and crane loads. Forces and moments were calculated for each frame for different load combinations. Then each structural element of both frames was evaluated for these load combinations to determine if the individual structural element satisfied the acceptance criteria for a given load case.

The structural evaluation of those portions of the turbine building was generated to provide preliminary structural assessment based on seismic Category I loading criteria. Later an additional simple seismic evaluation of a typical turbine building reinforced concrete pilaster was performed based on lateral DBE seismic loading representative of the total building.

These early evaluations are not presented as a basis to demonstrate the seismic adequacy of the Hatch Unit 1 Turbine Building per the guidance of the applicable BWROG topical report NEDC-31858P-A Revision 2 and applicable NRC safety evaluation dated March 3, 1999. As stated in Enclosure 8 section 1.0 "Turbine Building" and in section 5.0 "Summary and Conclusions" item 1, the Turbine Building is seismically acceptable based on 1) it being of similar construction to industrial structures that structurally survived earthquake levels significantly larger that the Hatch DBE (Page D-93 to D-95 of Reference 1 of Enclosure 8) and 2) the previous NRC reviewed evaluation of the Hatch Unit 1 Turbine Building in the Hatch Unit 1 Seismic Margin Assessment demonstrating a high-confidence-of-low-probability of failure (HCLPF) of 0.3g peak ground acceleration (PGA) which is at least twice the amplitude of the Hatch Unit 1 DBE (Enclosure 8 References 4, 5, and 6).

NRC QUESTION 2

It is stated on page 10 of Enclosure 8 that the allowable stress for load combinations involving tornado loads can be found in Section 2.1.5b(5). Since the NRC staff could not find the specified information, please indicate the allowable stress values as a percentage of the yield stress for steel and the compressive strength for concrete.

SNC RESPONSE

The intended reference for the allowable stress for load combinations involving tornado loads is section 1.1.5b(5) on page 10 of the referenced Enclosure 8. As stated in section 1.1.5b(5), the turbine building structural elements and their supports were designed to withstand the combined tornado loads discussed in previous Enclosure 8 sections 1.1.5b(1), (2), and (3) without exceeding permissible stresses of 90% f_y for reinforcing steel, 75% f'c for concrete per applicable working stress design formulae, and 100% f_y for structural steel.

NRC QUESTION 3

Enclosure 8 indicates that the original design of the turbine building involved tornado loads, but did not involve a dynamic analysis of DBE loads. During the telephone conference on March 22, 2007, the licensee stated that it had performed a dynamic analysis for the turbine building. The NRC staff requests the licensee to list and confirm that for representative floors, the maximum story shear (lateral) forces resulting from tornado and DBE satisfy the acceptance criterion for structural adequacy of the turbine building.

SNC RESPONSE

Table 1 below provides the maximum story shear forces (summation of lateral forces) resulting from tornado and seismic forces. The NRC question requested DBE lateral loads. The dynamic analysis mentioned in the conference call on March 22, 2007 was not performed specifically for the Hatch Unit 1 DBE. That seismic analysis was performed to obtain Seismic Margin Earthquake (SME) in-structure response spectra similar to that previously calculated for the Hatch Seismic Category I structures as part of the Hatch Unit 1 Seismic Margin Assessment (SMA). The SME soil-structure interaction (SSI) analysis used a median NUREG/CR 0098 0.3g PGA earthquake ground motion. The Hatch Unit 1 DBE PGA is 0.15g. Therefore to obtain an equivalent PGA the SME ground motion was factored by ½. A comparison of the ½ SME ground response spectra to the Hatch Unit 1 DBE (both at 5% damping) is provided in Figure 1. This figure shows that the ½ SME ground motion response spectrum has significantly higher spectral acceleration amplitudes over a wide frequency range than that of the Hatch Unit 1 DBE ground motion response spectrum. In order to conservatively approximate the Hatch Unit 1 DBE lateral inertial loads, the maximum absolute SME accelerations based on the highest values from the lower bound, best estimate, and upper bound soil profiles (whichever produced the highest) were used and then multiplied by one-half to obtain an equivalent maximum response for a ground PGA of 0.15g. The maximum seismic story shears were calculated based on multiplying the Unit 1 lumped masses for each of the representative floors and roof by ½ of the maximum absolute SME accelerations at those locations.

Table 1: Comparison of Maximum Story Shear (lateral forces)
Seismic Loads to Tornado Loads

	North-South Story Shear (kips)		East-West Story Shear (kips)	
Location	Seismic	Tornado	Seismic	Tornado
Roof	1482	1409	763	2974
Floor @ El. 164'	5839	3122	5760	6588
Floor @ El. 147'	6832	3728	6896	7868
Floor @ El. 130'	9214	4032	9240	8508

As presented in Enclosure 8, the structural adequacy of the Hatch Unit 1 Turbine Building in regards to seismic loads was based on screening criteria provided in the BWROG topical report NEDC-31858P-A Revision 2 (Reference 1 of Enclosure 8) and further documented by referencing the seismic margin assessment of the Turbine Building for the Hatch Unit 1 SMA (References 4, 5, and 6 of Enclosure 8). That SMA evaluation documented that the Hatch Unit 1 Turbine Building has at least a HCLPF of 0.3g PGA. As mentioned in Enclosure 8, the Hatch Unit 1 SMA was reviewed by the NRC.

But this NRC question requests that, in addition, confirmation of the structural adequacy be made based on the DBE to tornado comparison. This confirmation was only done where the

seismic story shears were notably larger than the tornado loads. These occur at floor elevations at and below El. 164' in the north-south (N-S) direction, and at and below El.130' in the E-W direction. These portions of the Turbine Building form the concrete portions of the building and have exterior and interior reinforced concrete shear walls. For these locations seismic shear stresses were calculated using the effective shear area of the reinforced concrete walls between floors. In all cases, the calculated shear stresses caused by these seismic story shears were significantly less than the allowable shear stress for concrete. The margin of allowable capacity to demand identified was at least 2.2.

NRC QUESTION 4

Regarding the condenser anchorage, page 19 of Enclosure 8 states that two of the four sets of the cast-in-place anchor bolts (four 2-1/4 inch diameter bolts per set) for one load case exceeded their allowable capacity by 11 percent, but was judged to be acceptable because (1) the bolt material has probably higher yield strength than the specified minimum yield strength, and (2) the total load from all four piers for that load case is less than the total capacity of the anchor bolts at those four piers. With respect to the licensee's argument concerning bolt material strength, the NRC staff cannot accept a statement of probable higher yield strength as a basis for acceptability of the anchor bolts. The NRC staff requests the licensee to verify that such higher yield strength does exist based on Certified Material Test Report (CMTR) data for the bolt material or other means acceptable to the NRC staff. With respect to the argument concerning applied loads vs. anchorage capacity comparison, the NRC staff requests the licensee to describe the type of loads, such as axial tension, axial compression, flexural, shear, and torsion, and their magnitudes of "the total load from all four piers," identify the loading combinations (case) that caused the actual bolt stress to be greater than the allowable capacity by 11 percent, demonstrate that concrete failure strength of the anchorage is greater than that of the anchor bolts, and justify the logic that the capacity of the anchor bolts at all four piers can be summed up to compare with the total load for that load case.

SNC RESPONSE

As summarized in section 5.0 of Enclosure 8, the seismic adequacy of the main condenser and condenser anchorage have been evaluated per the guidance of the applicable BWROG topical report NEDC-31858P-A Revision 2 and applicable NRC safety evaluation dated March 3, 1999. Based on this evaluation and as further supported below, the condenser anchorage will perform its function of maintaining the condenser in place with the necessary structural integrity to serve as part of the Unit 1 main steam isolation valve alternate leakage treatment path.

Consistent with the original design of the main condenser and condenser anchorage, the subject anchor bolts were not required to be procured safety related. It is noted that, for Hatch, the Certified Material Test Report (CMTR) data was not required to be obtained for material, such as anchor bolts, that was not required to be procured safety related. However, a review of relevant design basis information was performed to locate any applicable CMTR. No applicable CMTR was found. Therefore, SNC provides an alternate approach

below to respond to the four specific parts of this question.

The below alternate approach demonstrates the anchor bolt system never exceeds the minimum yield stress in shear and there is an adequate load path; therefore the anchor bolts are considered acceptable without the need to consider load redistribution.

NRC Question 4 Part 1: ...verify that such higher yield strength does exist based on Certified Material Test Report (CMTR) data for the bolt material or other means acceptable to the NRC staff.

Part 1 Response:

The bill of bolts and original shipping documentation from the bolt vendor states the anchor bolts are fabricated from ASTM A36 steel. ASTM A36 has a minimum specified yield stress (Fy) of 36 ksi.

USS Steel Design Manual (see below references 4.1 and 4.2) Chapter 1, titled "The Structural Steels and Mechanical Properties," states the following:

> "The yield stress in shear is approximately equal to its theoretical value of $1/\sqrt{3}$ times the yield stress in tension. Results of shear tests indicate that the ultimate shear strength range from 2/3 to 3/4 of the tensile strength."

Therefore, the minimum yield stress in shear equals $1/\sqrt{3}$ x 36 ksi = 20.8 ksi and the minimum ultimate shear strength is 2/3 (58 ksi) = 38.7 ksi.

In the one load case where there is a slight exceedance in the anchorage allowables for two of the four piers, the anchors are loaded in pure shear; there is basically no tensile load on the anchors. For that one load case the maximum nominal shear stress for an anchor bolt was calculated to be 20.4 ksi. Therefore, the actual maximum nominal anchor bolt stress is less than minimum yield stress for shear and significantly below the minimum ultimate shear strength.

Therefore, the anchorage system is judged acceptable for the conservative load case where the one-half of the Hatch Unit 1 Seismic Margin Earthquake (½ SME) demand was increased by a factor of 1.25, i.e. 1.25 x ½ SME. In Enclosure 8 the purpose for providing the rational (minimum material properties and ability of load redistribution) for the acceptability of the condenser anchorage when one of these conservative load case combinations produced a slight exceedance of an allowable stress in some of the anchor bolts, was to simply indicate this slight exceedance would not result in the condenser losing the ability to remain in place and performing its intended function.

References:

4.1 USS Steel Design Manual; R. L. Brockenbrough and B. G. Johnson, May 1974.
4.2 Structural Steel Design, Second Edition; Lambert Tall, Editor; The Ronald Press Company, 1974.

NRC Question 4 Part 2: ...describe the type of loads, such as axial tension, axial compression, flexural, shear, and torsion, and their magnitudes of "the total load from all four piers," identify the loading combinations (case) that caused the actual bolt stress to be greater than the allowable capacity by 11 percent.

Part 2 Response:

There are four piers that support the condenser. The following table provides the loads on the anchors for each pier for each load combination.

	DW + Live + ½ SME	DW + Live – ½ SME	DW + Live + 1.25 (½ SME)	DW + Live – 1.25 (½ SME)
South-east pier				
Axial (Kips) *	7	-145	26	-164
Shear (Kips)	281	76	325	120
North-east pier				
Axial (Kips) *	143	-84	172	-172
Shear (Kips)	204	114	243	153
South-west pier				
Axial (Kips) *	-21	-166	-3	-184
Shear (Kips)	280	76	324	120
North-west pier				
Axial (Kips) *	166	-53	193	-81
Shear (Kips)	202	113	242	153

* Minus sign indicates tension.

"DW" is dead weight. "Live" is the live load which includes vacuum pressure plus nozzle loads. "½ SME" is the seismic loading based on one-half of the Hatch Unit 1 SME ground motion response spectra. The ½ SME exceeds the required Hatch Unit 1 Design Basis Earthquake (DBE). This exceedance of the seismic spectral acceleration of the Hatch Unit 1 DBE and the resulting conservatism is described in the previous response to NRC question 3 and graphically depicted in Figure 1. The three orthogonal components of earthquake motion were combined by the square root of the sum-of-the-squares (SRSS) rule.

One load combination "DW + Live + 1.25 (½ SME)" produced loads slightly exceeding allowables for two of the four sets of anchors. These were at the south-east and south-west piers.

NRC Question 4 Part 3: ...demonstrate that concrete failure strength of the anchorage is greater than that of the anchor bolts...

Part 3 Response:

There are four 2 ¼" diameter cast-in-place anchor bolts in each pier.

Each cast-in-place bolt is embedded in the reinforced concrete pier the full 4' height of the pier. Each anchor bolt is in a 3 ½" diameter sleeve. The end of the cast-in-place bolt has a hold down plate 7" x 1 ½" x 10" with a nut on the bottom side to fully transmit anchor bolt tensile loads to the concrete pier.

Tensile loads on the anchors are transferred to the pier through concrete bearing of the bolt hold down plates. Shear loads are transferred from the anchor bolts through bolt bearing on the sides of the bolt holes in a thick top sole plate that has welded shear bars on the bottom side of the plate. The top sole plate is set in shear keys formed in the top of the reinforced concrete pier. The top sole plate with shear bars is grouted in place. Therefore shear loads from the anchor bolts are carried by the top sole plate which distributes the shear uniformly to the reinforced concrete pier through bearing of the shear bars on the sides of the shear keys in the top of the pier. The anchor bolts do not directly transfer shear to the concrete pier.

Each pier is approximately 4' high, and has plan dimensions of 4'-10" by 3'-2". Each pier is heavily reinforced with 30 #9 vertical rebar spaced at about 5 ¾" spacing. These vertical rebars were placed as part of the turbine building basemat pour. The embedment in the basemat fully develops the strength of the rebar. These vertical bars have 4 sets of horizontal #5 ties spaced every 12" vertically. Each of these shear ties is fully enclosed with 90 degree hooks. The ties provide 90 degree bends around every corner and alternate vertical #9 rebar.

As part of the condenser anchorage evaluations the axial and shear loads and resulting moments were calculated at the base of the piers to check the structural adequacy of the reinforced concrete piers. The piers were shown to have significant code capacity compared to the calculated demand.

Even though the maximum tensile loads on the anchor bolts are relatively small, the concrete bearing capacity provided by the anchor bolt embedded plates with the tensile capacity of the #9 vertical rebars provide a significantly larger capacity to resist tension in the bolts than the bolts themselves. In regards to shear, the concrete bearing capacity at the shear key/shear bar interfaces greatly exceed the shear yield strength of the anchor bolts. Due to the amount of reinforcement and confinement provided by #9 vertical rebars and the #5 ties, and the direct shear transfer to the pier from the top sole plate with shear bars, concrete failure does not control the failure mode of the anchorage.

NRC Question 4 Part 4: ...justify the logic that the capacity of the anchor bolts at all four piers can be summed up to compare with the total load for that load case.

Part 4 Response:

The condenser is a structurally robust component that is capable of redistributing loads to accommodate slight changes in stiffness of a set of anchors caused by any slight yielding of the bolt material. Slightly exceeding yield stress does not mean loss of that anchor's capacity to carry load but does limit it to a given capacity. Any excess load beyond that capacity can be redistributed through the redundant structural system of the condenser. The intent of the statement in Enclosure 8 was to point out that redistribution to other sets of anchors would not exceed the sum total of the capacity of all the anchorage. Therefore, it is ensured the condenser would remain in place.

But as discussed earlier in this response, the anchor bolt system never exceeds the minimum yield stress in shear and there is an adequate load path; therefore the anchor bolts are considered acceptable without the need to consider load redistribution.

NRC QUESTION 5

It is stated on page 26 of Enclosure 8 that "The minimum design margin for the pipe supports and pipe support anchorage is over three (3.0) times the 0.15g plant design basis." As stated on page 31 of Enclosure 8, the method used to obtain the above design margin appears to not be the anchorage verification method specified in the Generic Implementation Procedure (GIP). However, it is stated on page 24 of Enclosure 8 that "Evaluation of bolted anchorage to concrete follows the procedures established in the Generic Implementation Procedure (GIP) for Seismic Verification of Nuclear Plant Equipment." The NRC staff requests the licensee to address the apparent discrepancy between the statements on pages 31 and 24. Furthermore, the NRC staff requests the licensee to specify and confirm that the anchorage with the minimum margin still satisfies the GIP criterion.

SNC RESPONSE

Pipe support anchorage is evaluated and confirmed to satisfy the GIP criterion. The margin values listed are defined as follows: the available seismic capacity provided divided by the seismic demand on the pipe support and its anchorage. The total capacity provided is defined as the lower of the following: the anchorage capacity per the GIP criterion or the pipe support allowable capacity as described in section 3.2.2.3 of Enclosure 8. The available seismic capacity provided is equal to the total capacity provided minus the demand of the non-seismic portion of the load case.

Information provided on page 24 relates to the "Main Steam Drain to Condenser" pipe supports and their anchorage. As stated in section 3.2.2.3 on page 24, the anchorage evaluation is based on the GIP anchorage criterion. The evaluations of pipe support anchorage were included in determining the margin of over four as described in section 3.2.2.4 on page 24.

Information provided on page 26 relates to the "Interconnected Systems." Just as with the main steam drain to condenser evaluations, the anchorage evaluation is based on the GIP anchorage criterion and is included in the determination of the minimum margin of over three.

The "Summary and Conclusions" section on page 31 repeats the margin values previously listed for the different piping systems supports and their anchorage. For clarity, the summary could have stated these margins included both the supports and the support anchorage, and that the anchorage allowable capacity is determined based on the GIP criterion.

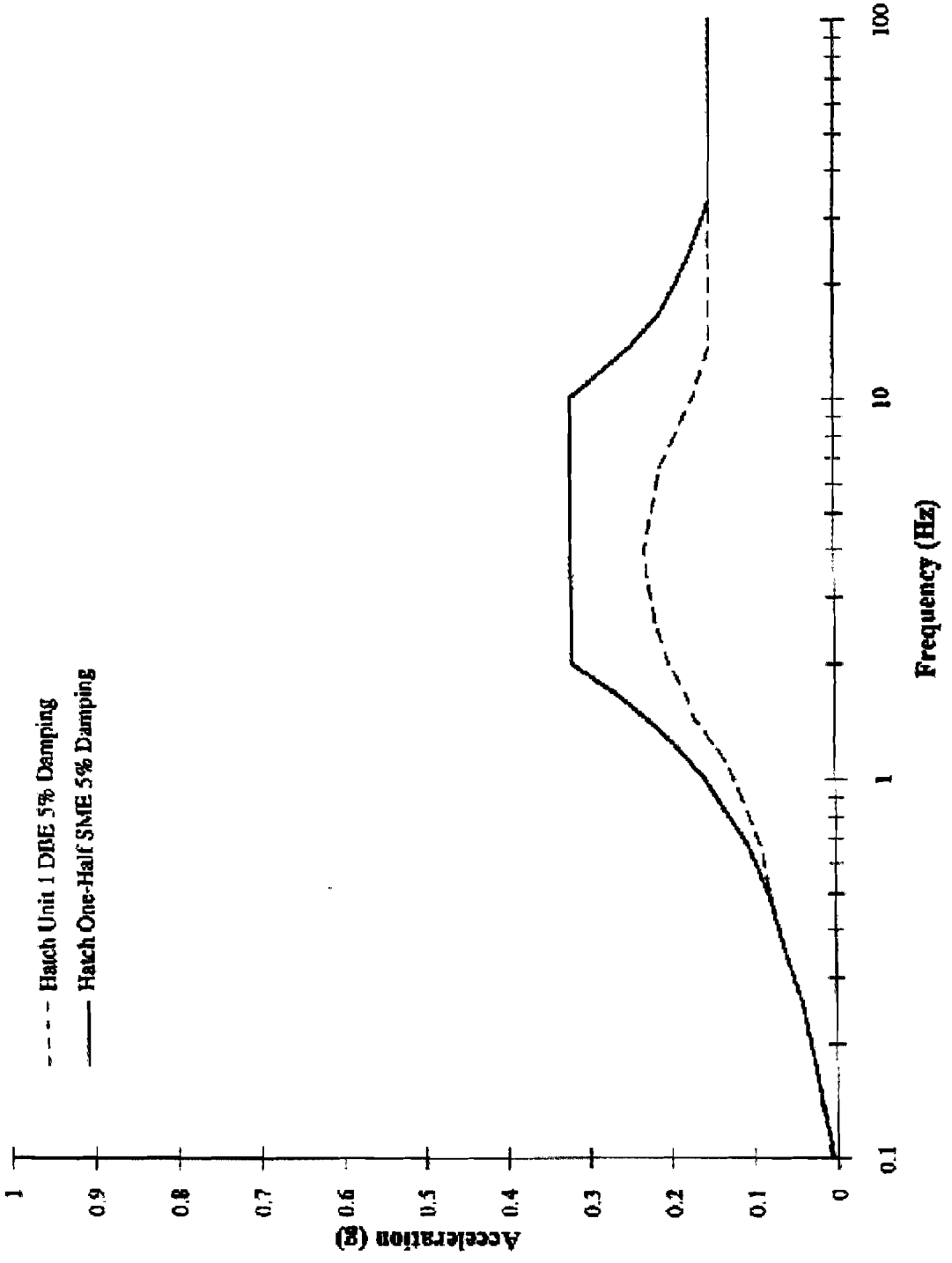

Figure 1 Plant Hatch Unit 1 Design Basis Earthquake Ground Response Spectrum vs. One-Half Seismic Margin Earthquake Ground Response Spectrum