NORTHEAST UTILITIES

THE CONNECTICUT LIGHT AND POWER COMPANY
WESTERN MASSACHUSETTS ELECTRIC COMPANY
HOLYOKE WATER POWER COMPANY
NORTHEAST UTILITIES SERVICE COMPANY
NORTHEAST NUCLEAR ENERGY COMPANY

General Offices • Selden Street, Berlin, Connecticut

P.O. BOX 270
HARTFORD, CONNECTICUT 06141-0270
(203) 666-6911

May 31, 1984

Docket No. 50-423
B11215

Director of Nuclear Reactor Regulation
Mr. B. J. Youngblood, Chief
Licensing Branch No. 1
Division of Licensing
U. S. Nuclear Regulatory Commission
Washington, D. C. 20555

Reference: (1) W. G. Counsil to B. J. Youngblood, NRC Structural & Geotechnical Engineering Branch (SGEB) Review Meeting, May 9, 1984, dated May 15, 1984.

Dear Mr. Youngblood:

Millstone Nuclear Power Station, Unit No. 3
Structural and Geotechnical
Engineering Branch
Additional Information in Response
to Confirmatory Items

On May 9, 1984 Northeast Nuclear Energy Company (NNECO) met with Mr. Nilesh Chokshi and Mr. David Jeng of the Structural and Geotechnical Engineering Branch (SGEB) to discuss the status and resolution of the Draft SER and Structural Audit open and confirmatory items.

Attachment I provides the additional information and clarifications that NNECO committed to provide to resolve the Staff reviewer's concerns and close Items 3, 8, 9, 19, 27, 29, 32, 33, 36, 39 and 42 (see Reference 1). Revisions to the original responses are identified by |1. All remaining structural confirmatory items are expected to be resolved at a meeting scheduled for early June, 1984.

If you have any concerns related to the information contained herein or any questions related to our responses, please contact our Licensing representative, Ms. C. J. Shaffer, at (203) 665-3285.

Very truly yours,

NORTHEAST NUCLEAR ENERGY COMPANY
et. al.
NORTHEAST NUCLEAR ENERGY COMPANY
Their Agent

W G Counsil

W. G. Counsil
Senior Vice President

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STATE OF CONNECTICUT)
) ss. Berlin
COUNTY OF HARTFORD)

Then personally appeared before me W. G. Counsil, who being duly sworn, did state that he is Senior Vice President of Northeast Nuclear Energy Company, an Applicant herein, that he is authorized to execute and file the foregoing information in the name and on behalf of the Applicants herein and that the statements contained in said information are true and correct to the best of his knowledge and belief.

Lorraine J. D'Amico
Notary Public

My Commission Expires March 31, 1988

Attachment I

Revised Responses to Structural Audit Items:

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	ACI 349-76	Millstone 3	Regulatory Guide 1.142	Justification
9	U = 0.75 (1.4 D + 1.7 L + 1.4 To + 1.7 Ro)	U = 0.75 (1.4 D + 1.7 L + 1.4 Ro + 1.4 To)	U = 0.75 (1.4 D + 1.7 L + 1.7 To + 1.7 Ro)	In equations 9, 10, and 11. Millstone 3 meets the intent of ACI 349 and Reg. Guide. See Note 3.
10	U = 0.75 (1.4 D + 1.7 L + 1.7 OBE + 1.4 To + 1.7 Ro)	U = 0.75 (1.4 D + 1.7 L + 1.9 OBE + 1.4 To + 1.4 Ro)	U = 0.75 (1.4 D + 1.7 L + 1.9 OBE + 1.7 To + 1.7 Ro)	
11	U = 0.75 (1.4 D + 1.7 L + 1.7 W + 1.4 To + 1.7 Ro)	U = 0.75 (1.4 D + 1.7 L + 1.7 W + 1.4 To + 1.4 Ro)	U = 0.75 (1.4 D + 1.7 L + 1.7 W + 1.7 To + 1.7 Ro)	

1. Refer to ACI 349 and Millstone 3 FSAR for definition of loads.

2. Lateral and Vertical pressure of liquids (F) and lateral earth pressure (H) are included in all applicable cases, as required by ACI 318, 349 and Reg. Guide 1.142.

3. Since R_o and T_o are loads known fairly accurately, Millstone 3 design provides an adequate margin by using the same load factors as dead load. Equations involving SSE and W_t control the design. 1

Item 3

ATTACHMENT 3

ITEM 8 Provide justification for not considering cracked sections on - containment internals in seismic analysis.

Response:

In the seismic analysis of the containment structure it was assumed that cracked section properties of the internals will not have any significant change in the overall response of the structure. To verify this assumption a parametric study was performed using the cracked section properties of one of the steam generator cubicles. The dynamic model utilized assumed the containment exterior wall to be cracked.

Only one pipe break is postulated at any given time. The design basis for a steam generator cubicle is the cubicle internal overpressurization resulting from a primary coolant loop break. While overpressurization due to this pipe break may cause cracking in one steam geneator cubicle, the amount of overpressurization of areas adjacent to this cubicle is negligible.

The parametric study selected is representative of design conditions and demonstrates the effect of extensive cracking of a specific primary structural element. 1

For loading conditions involving SSE, cracking of most concrete elements can be expected, consistent with the general behavior of reinforced concrete under load. The effect of such cracking is covered by the peak spread criteria.

Therefore, only one steam generator cubicle has been postulated to be cracked.

Figure 8-1 shows the key plan showing the steam generator cubicle for which the cracked section properties were used.

Figure 8-2 shows the dynamic model of the containment structure. In this model members 4, 5, 6, and 7 have stiffness characteristics which are determined from cross sections which include the steam generator cubicle walls. In the verification analysis, members 4, 5, 6, and 7 have been modified to account for cracking of the "B" steam generator cubicle wall. 1

Conclusion:

Table 8-1 shows the comparison of frequencies of uncracked vs cracked case.

Due to cracking of one steam generator cubicle the shift in the fundamental frequency of the internals is insignificant and will have negligible effects on the structural response. Maximum percent shift of the frequencies falls well within the peak spreading criteria of ±15 percent. (Ref. FSAR 3.7). Also shown in Table 8-2 is a comparison of resultant accelerations from the cracked versus uncracked sections and the variations were found to be negligible.

ITEM 9 Provide comparison of development of torsional constants using computer program SECPROP3 versus classical shell theory for crane wall. This issue was resolved. Applicant will provide this response.

Response:

A comparison was made between the torsional constants developed using SECPROP3 vs classical shell theory for the members of the dynamic model of the containment structure representing the internal structure. This comparison is presented in Table 1. Since a large difference is evident between the two methods for members 7 and 8, the containment dynamic model was rerun with revised torsional stiffness for members 4 through 8 based on the torsional constants developed from shell theory. A comparison of resulting frequencies and responses for the original and revised model are presented in Tables 2 and 3. It is clear from this comparison that no significant change in response resulted due to the change in torsional stiffness, therefore demonstrating that the original containment dynamic model is an adequate representation of the actual building.

SECPROP3 develops cross-sectional properties for structural elements which have been input as straight walls. The difference in torsional constant identified above occurs when properties for circular walls are determined with SECPROP3. Since properties for circular walls were determined from SECPROP3 only for the containment internals, the scope of this is limited to the containment. 1

ITEM 19 Provide a comparison of results in internal structure from Finite Element analysis and STRUDL. An example of the results of comparison was presented and the item was resolved. The Applicant will provide sketches and documentation.

Response:

There are four RCS loops and four steam generator cubicles. Each cubicle houses one Reactor Coolant Loop System (RCS). All four cubicles are similar in configuration and are subjected to the same forces.

Figure 1 shows the plan view of cubicle "B" at el 3 ft-8 in. Steam generator supports S_5, S_6, S_7, S_8, and RC pump supports P_4, P_5, and P_6 are shown in relation to hot leg bumper, cross over bumpers I and II, and steam generator column below el 3 ft-8 in. slab. For the frame analysis of slab at el 3 ft-8 in., walls of the cubicle and the slab is divided into frame sections. Frame sections I, II, and III have been selected and are shown on Figure 1. For stiffness analysis of the frame, STRUDL II computer program is used.

As a verification to frame analysis, a finite element analysis has been performed using bending and stretching elements. STRUDL II was used to analyze the model.

For the comparison of results, loading combination $1.0\ D + 1.0\ L + 1.5\ Pa + 1.0\ R_A + 1.0\ T_A$ has been used.

Figures 2 through 5 show the graphical representation of member forces, moments, and shears on frame section III due to frame as well as finite element analysis.

In the design, forces obtained due to frame analysis were used as they governed the design.

By comparing the results from frame analysis and finite element analysis, it is concluded that the frame analysis results are conservative and hence they are used in the design of containment internals. 1

ITEM 27: Provide Justification for Generation of ARS Based Upon One - Component of Input Motion

Response

The method and criteria used in the generation of ARS for Millstone Unit 3 are inherently conservative. Conservative damping values are used both for structures and equipment. The conservatism in those damping values can be seen when they are compared with the recognized conservative values in Regulatory Guide 1.61. In addition, the input time history used to generate the ARS envelopes the design response spectrum as shown on Figures 3.7B-3 through 3.7B-8 of the FSAR.

To evaluate the conservatism of the one-component ARS curves, a study was conducted to provide a comparison between ARS developed for the SSE condition using:

1. One-component response with 5 percent structural damping and 1 percent oscillator damping.

2. Three-component response (SRSS of the three ARS components) with 7 percent structural damping and 2 percent oscillator damping.

To assure a study representative of all Category I buildings, the Containment Structure, Auxiliary Building, and Fuel Building were evaluated. The Auxiliary Building is representative of all Category I buildings with little coupling, and the Fuel Building is representative of buildings with a large degree of coupling. In every case, the ARS were generated for SSE condition using the three-component response and are enveloped by the response spectra that have been used in piping and equipment qualification.

The attached amplified response spectra curves used in the study are provided for the above buildings in the direction of E-W, N-S and vertical excitation are as follows:

Reactor Containment Interior Structure for the following elevations 1

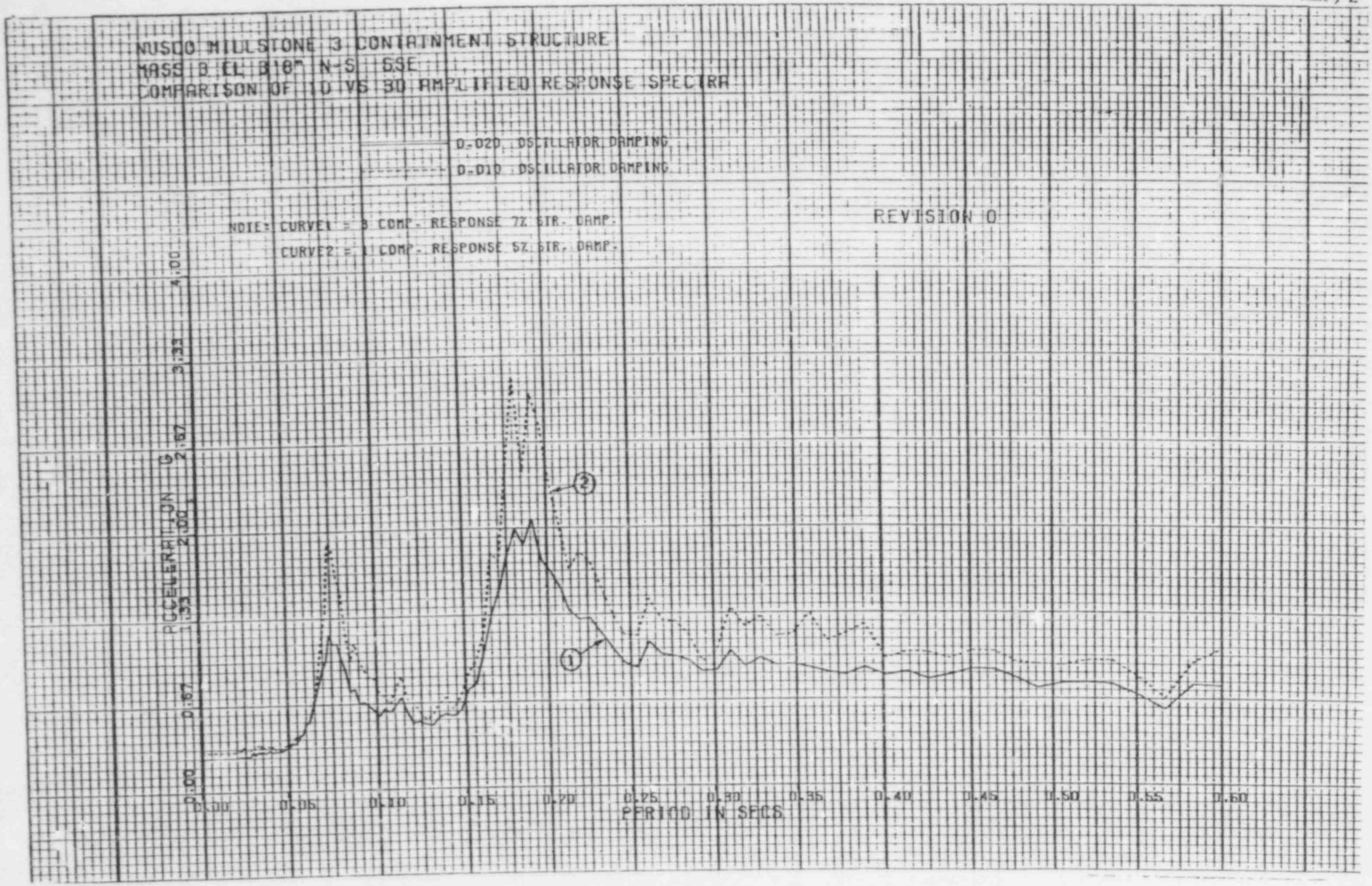
El. 3 ft-8 in.
El. 52 ft-4 in.
El. 109 ft-1 in.

Auxiliary Building at elevation 66'-6"

Fuel Building for the following elevations

El. 52 ft-4 in.
El. 93 ft-10 in.

Therefore, it is concluded that the one-component ARS curves developed for all Millstone Unit 3 Category I buildings are conservative with respect to the three-component ARS curves.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MASS 8 EL 8'0" N-S SSE
COMPARISON OF 1D VS 3D AMPLIFIED RESPONSE SPECTRA
0.020 OSCILLATOR DAMPING
0.010 OSCILLATOR DAMPING
NOTE: CURVE1 = 3 COMP. RESPONSE 7% STR. DAMP.
CURVE2 = 1 COMP. RESPONSE 5% STR. DAMP.
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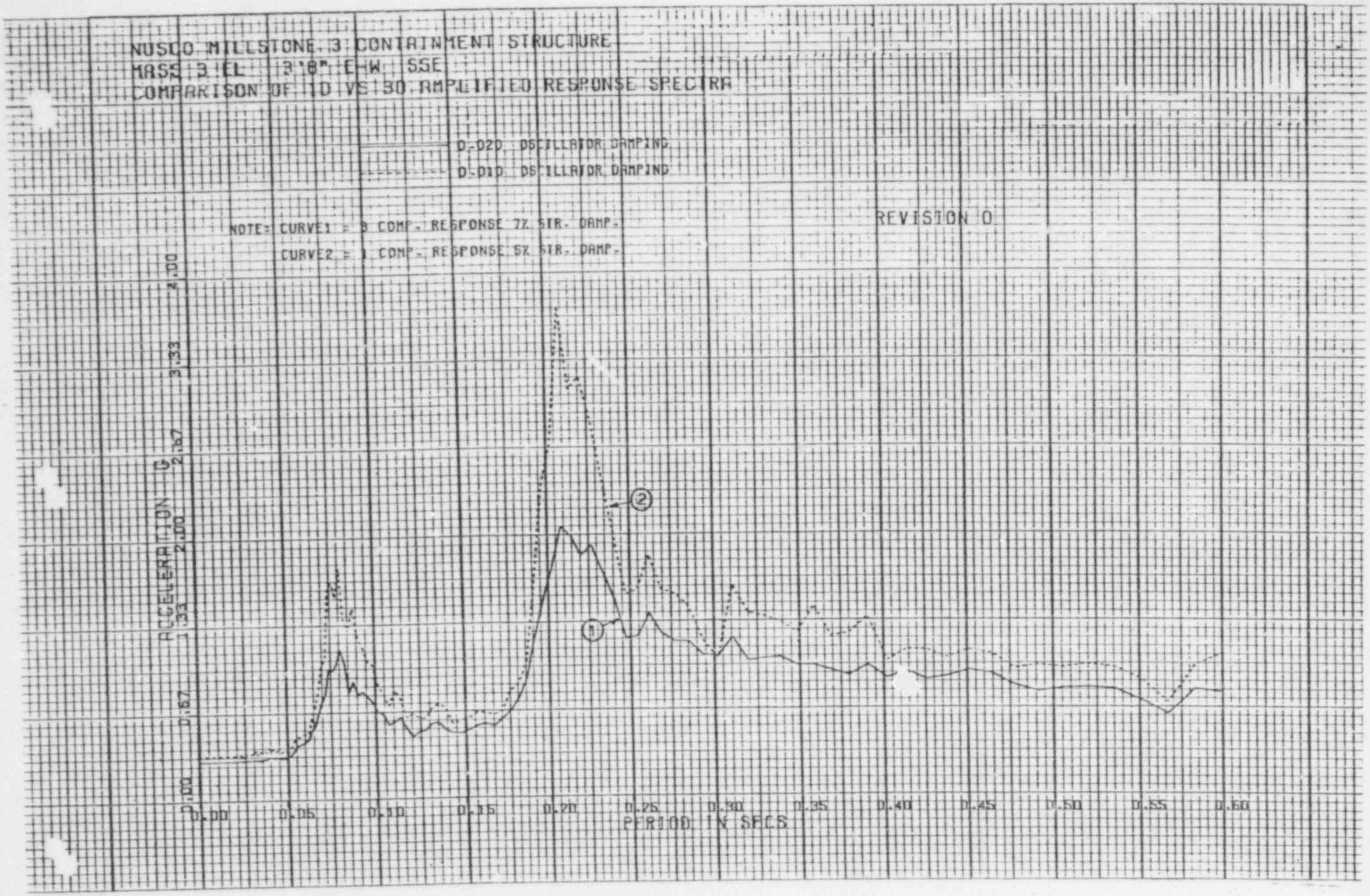
NUSCO MILLSTONE 3 CONTAINMENT STRUCTURE
MASS 3 EL 3'8" E-W SSE
COMPARISON OF 1D VS 3D AMPLIFIED RESPONSE SPECTRA
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0.010 OSCILLATOR DAMPING
NOTE: CURVE1 = 3 COMP. RESPONSE 7% STR. DAMP.
CURVE2 = 1 COMP. RESPONSE 5% STR. DAMP.
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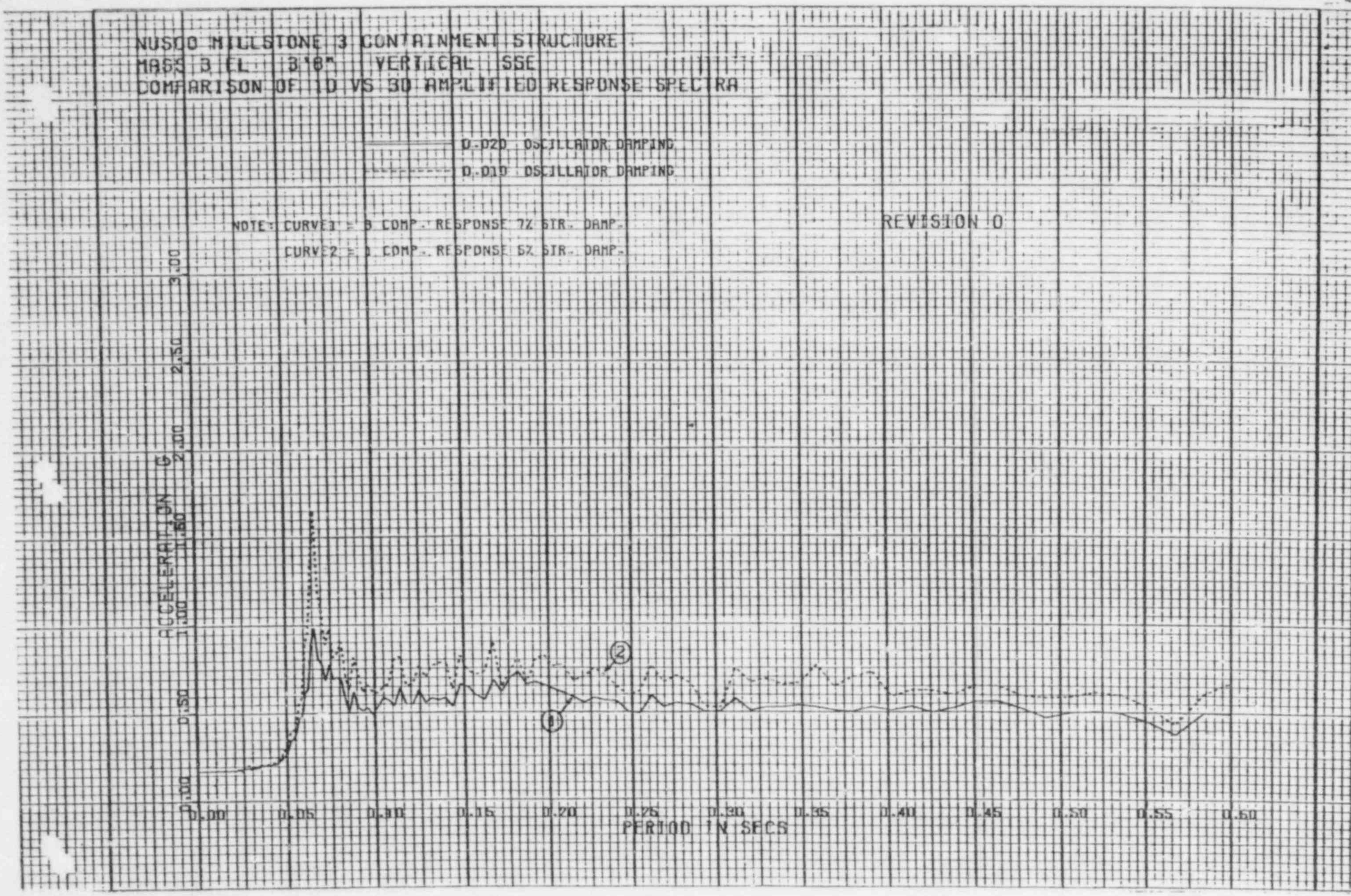
NUSCO MILLSTONE 3 CONTAINMENT STRUCTURE
MASS 3 EL 3'8" VERTICAL SSE
COMPARISON OF 1D VS 3D AMPLIFIED RESPONSE SPECTRA
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0.010 OSCILLATOR DAMPING
NOTE: CURVE1 = 3 COMP. RESPONSE 7% STR. DAMP.
CURVE2 = 1 COMP. RESPONSE 5% STR. DAMP.
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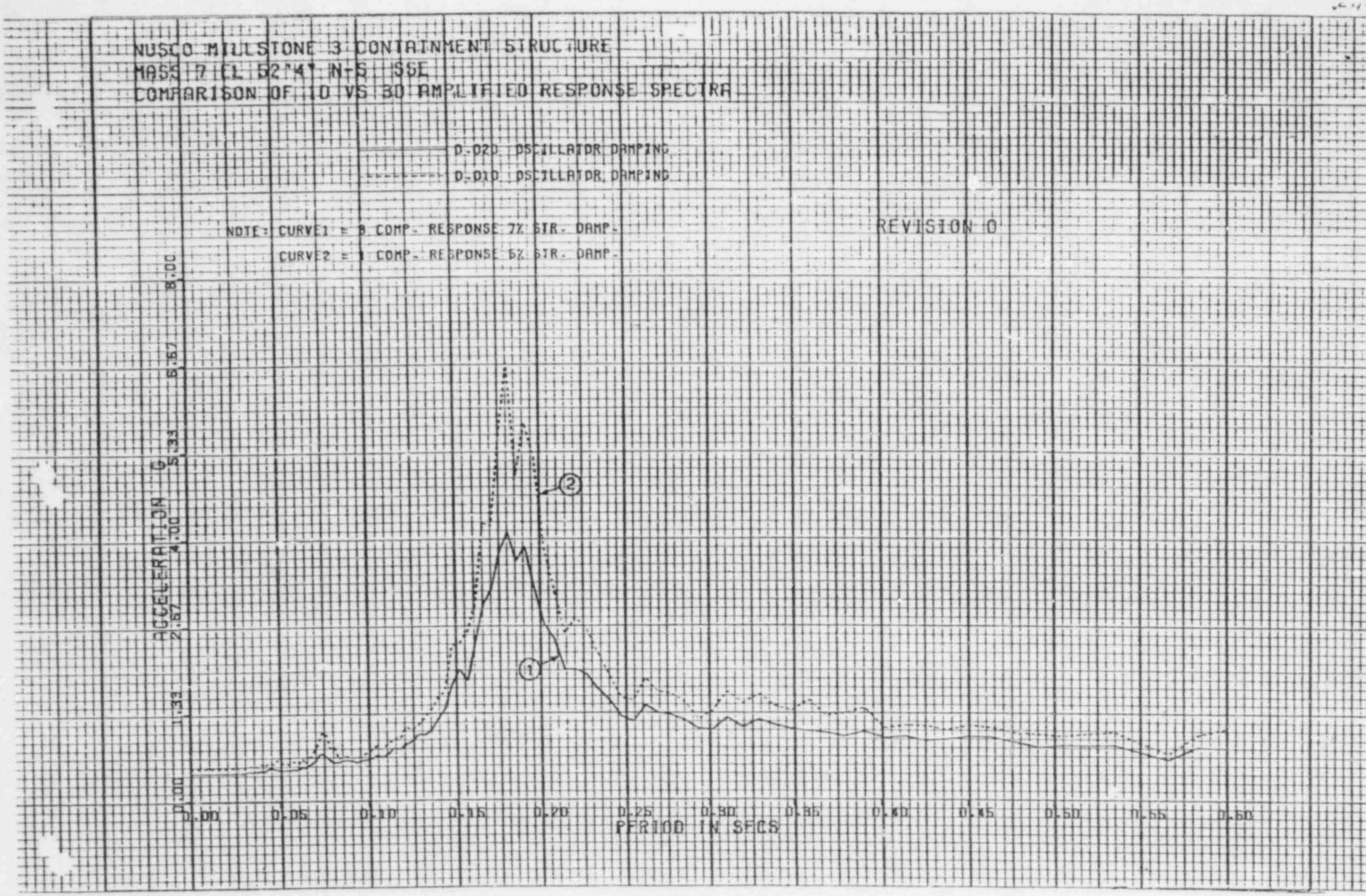
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MASS 7 EL 52'4" N-S SSE
COMPARISON OF 1D VS 3D AMPLIFIED RESPONSE SPECTRA
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0.010 OSCILLATOR DAMPING
NOTE: CURVE1 = 3 COMP. RESPONSE 7% STR. DAMP.
CURVE2 = 1 COMP. RESPONSE 5% STR. DAMP.
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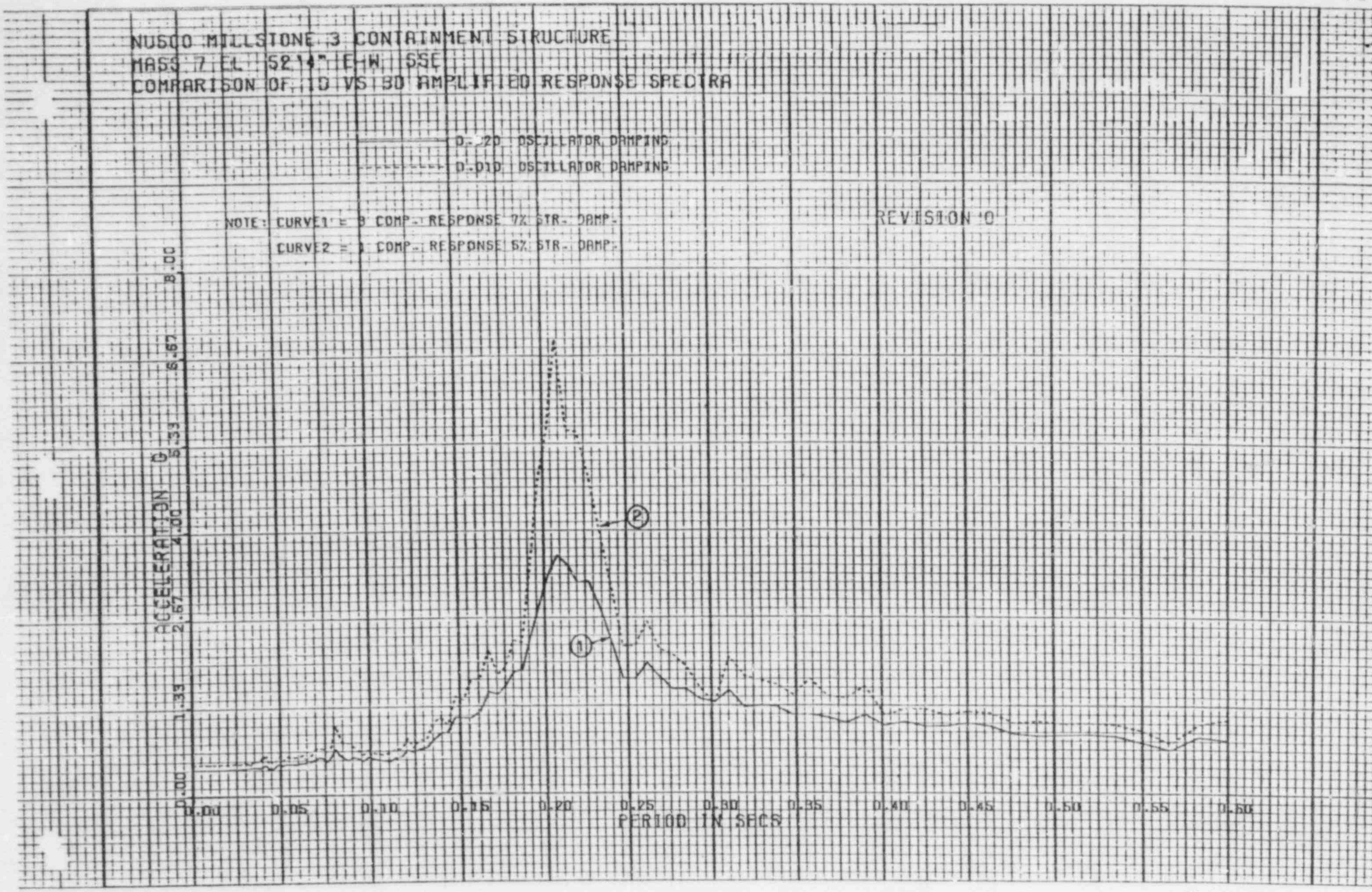
NUSCO MILLSTONE 3 CONTAINMENT STRUCTURE
MASS 7 EL. 52'4" E-W SSE
COMPARISON OF 1D VS 3D AMPLIFIED RESPONSE SPECTRA
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0.010 OSCILLATOR DAMPING
NOTE: CURVE1 = 3 COMP. RESPONSE 7% STR. DAMP.
CURVE2 = 1 COMP. RESPONSE 5% STR. DAMP.
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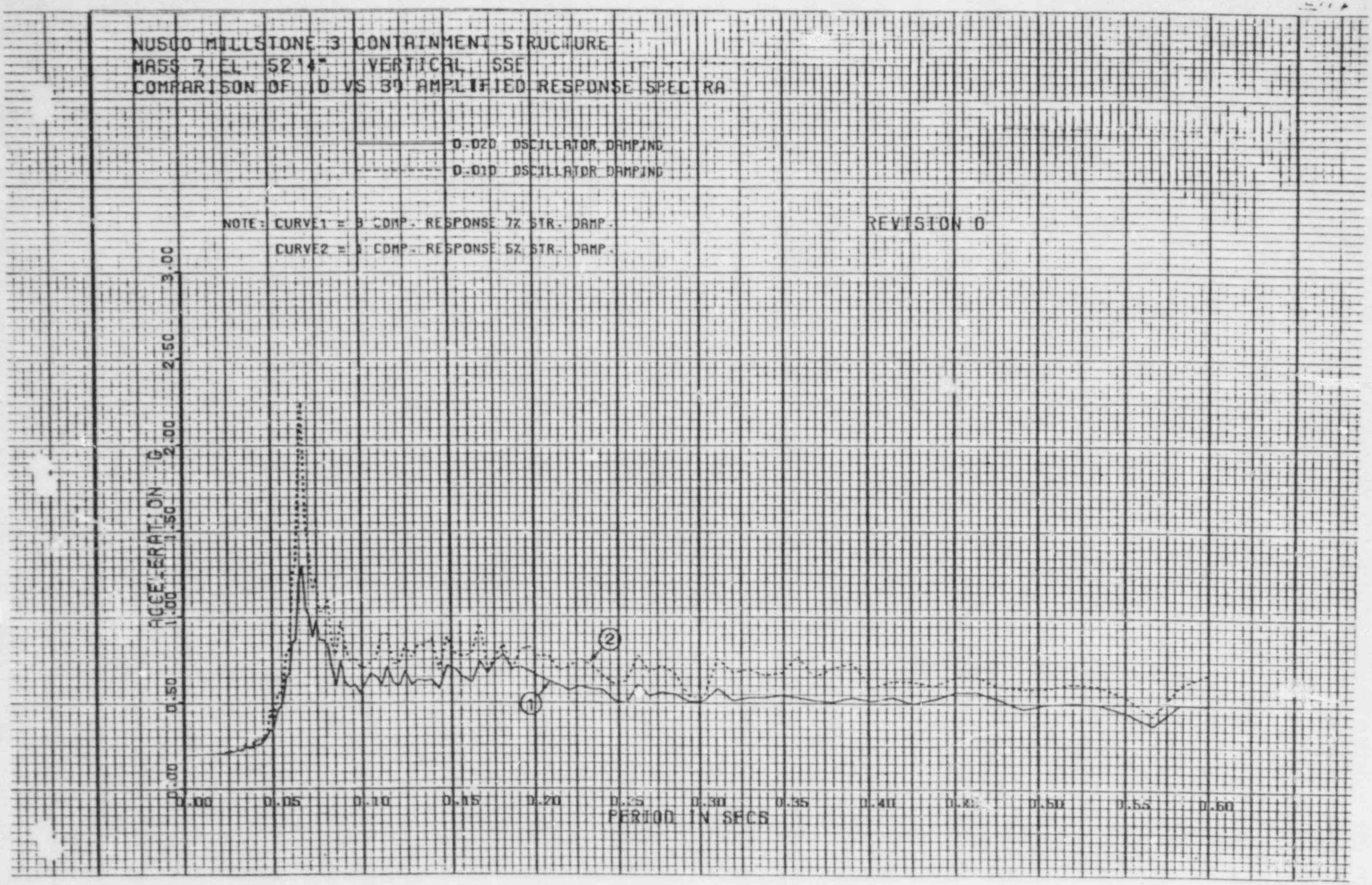
NUSCO MILLSTONE 3 CONTAINMENT STRUCTURE
MASS 7 EL 52'4" VERTICAL SSE
COMPARISON OF 1D VS 3D AMPLIFIED RESPONSE SPECTRA
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NOTE: CURVE1 = 3 COMP. RESPONSE 7% STR. DAMP.
CURVE2 = 1 COMP. RESPONSE 5% STR. DAMP.
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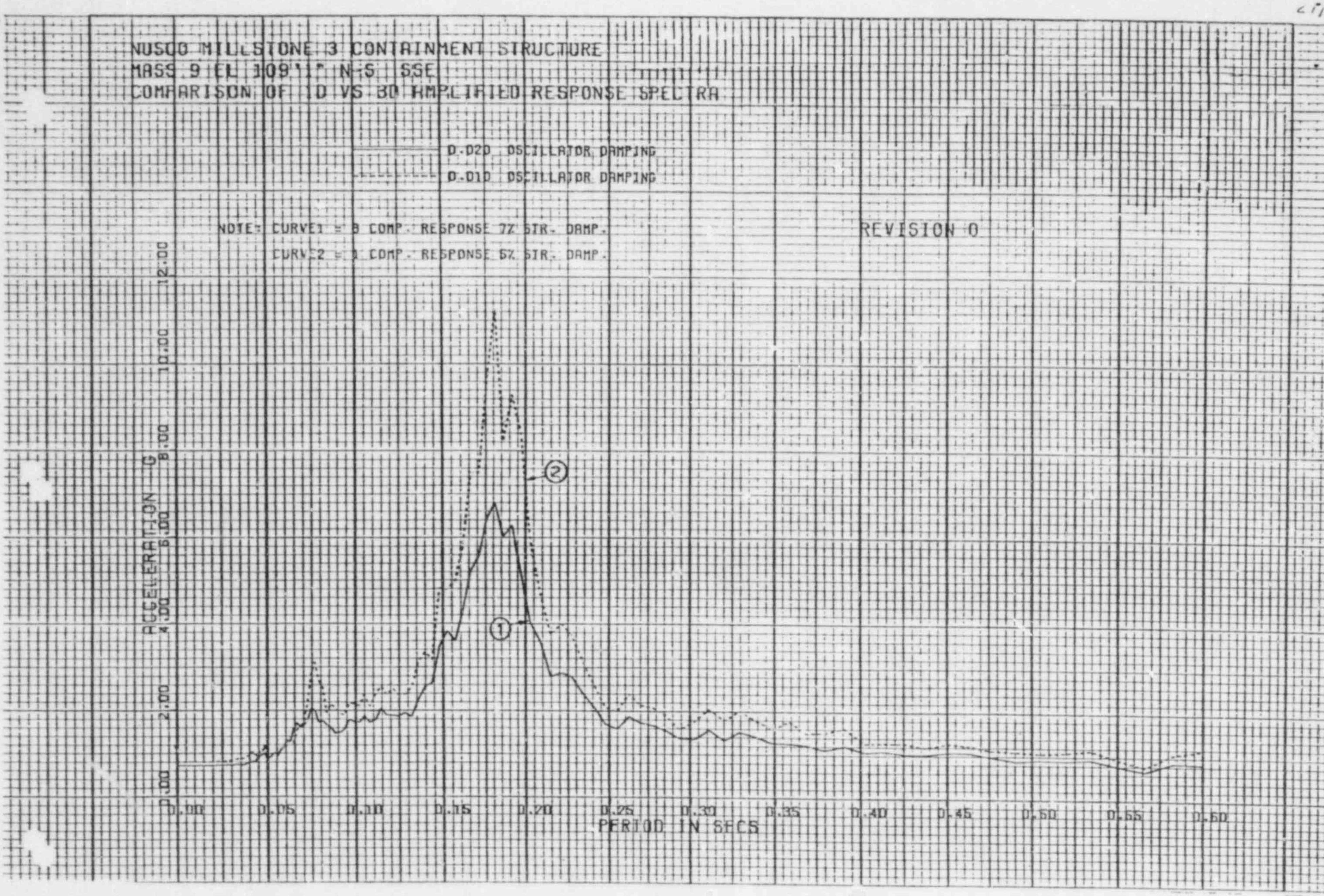
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MASS 9 EL 109'1" N-S SSE
COMPARISON OF 1D VS 3D AMPLIFIED RESPONSE SPECTRA
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0.010 OSCILLATOR DAMPING
NOTE: CURVE 1 = 3 COMP. RESPONSE 7% STR. DAMP.
CURVE 2 = 1 COMP. RESPONSE 5% STR. DAMP.
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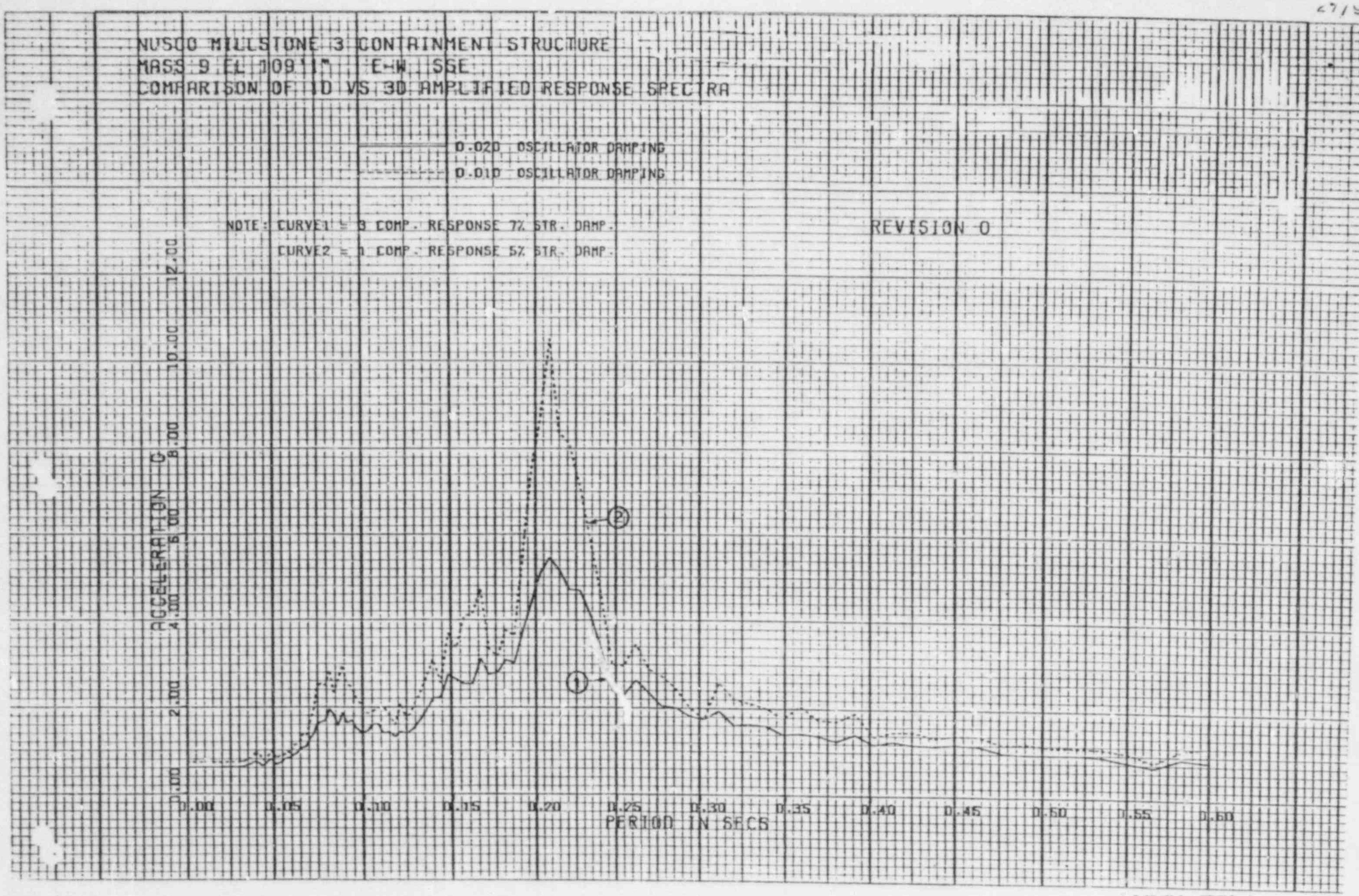
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MASS 9 EL 109'11"   E-W SSE
COMPARISON OF 1D VS 3D AMPLIFIED RESPONSE SPECTRA
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NOTE: CURVE1 = 3 COMP. RESPONSE 7% STR. DAMP.
CURVE2 = 1 COMP. RESPONSE 5% STR. DAMP.
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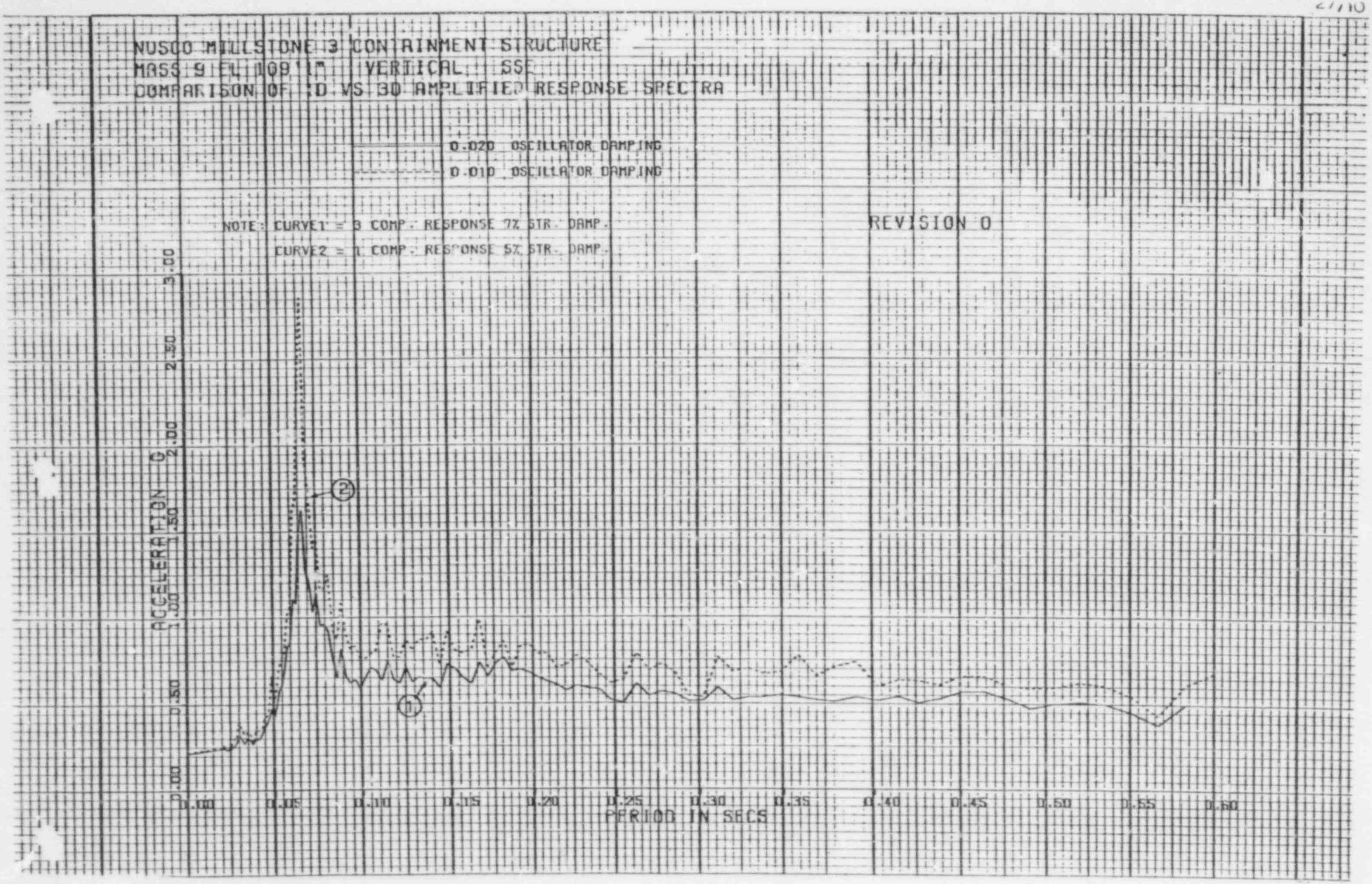

NUSCO MILLSTONE 3 CONTAINMENT STRUCTURE
MASS 9 EL 109'1" VERTICAL SSE
COMPARISON OF 1D VS 3D AMPLIFIED RESPONSE SPECTRA
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NOTE: CURVE1 = 3 COMP. RESPONSE 7% STR. DAMP.
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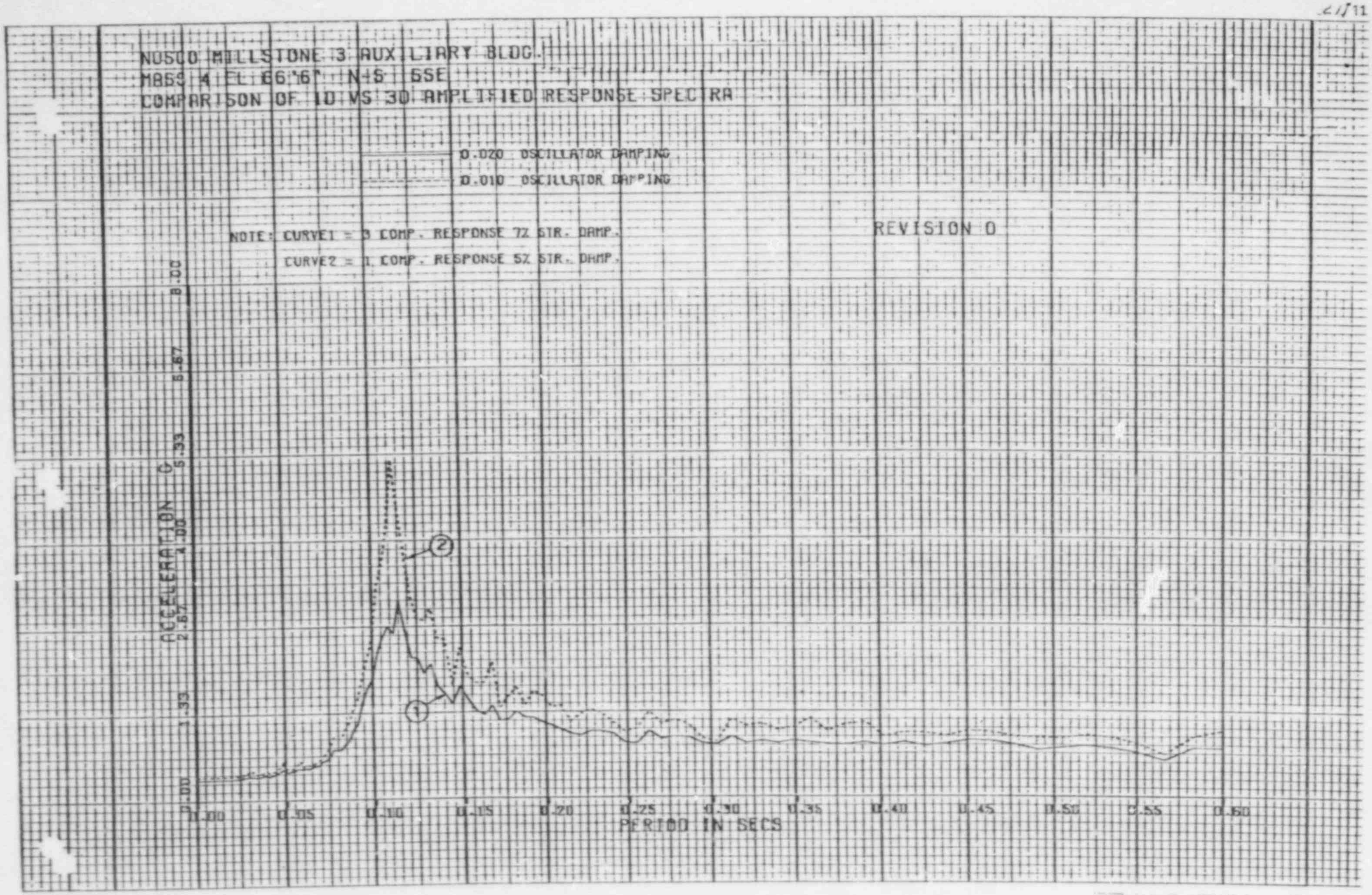

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NUSCO MILLSTONE 3 AUXILIARY BLDG.
MASS 4 EL 66'6" N-S SSE
COMPARISON OF 1D VS 3D AMPLIFIED RESPONSE SPECTRA
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NOTE: CURVE1 = 3 COMP. RESPONSE 7% STR. DAMP.
CURVE2 = 1 COMP. RESPONSE 5% STR. DAMP.
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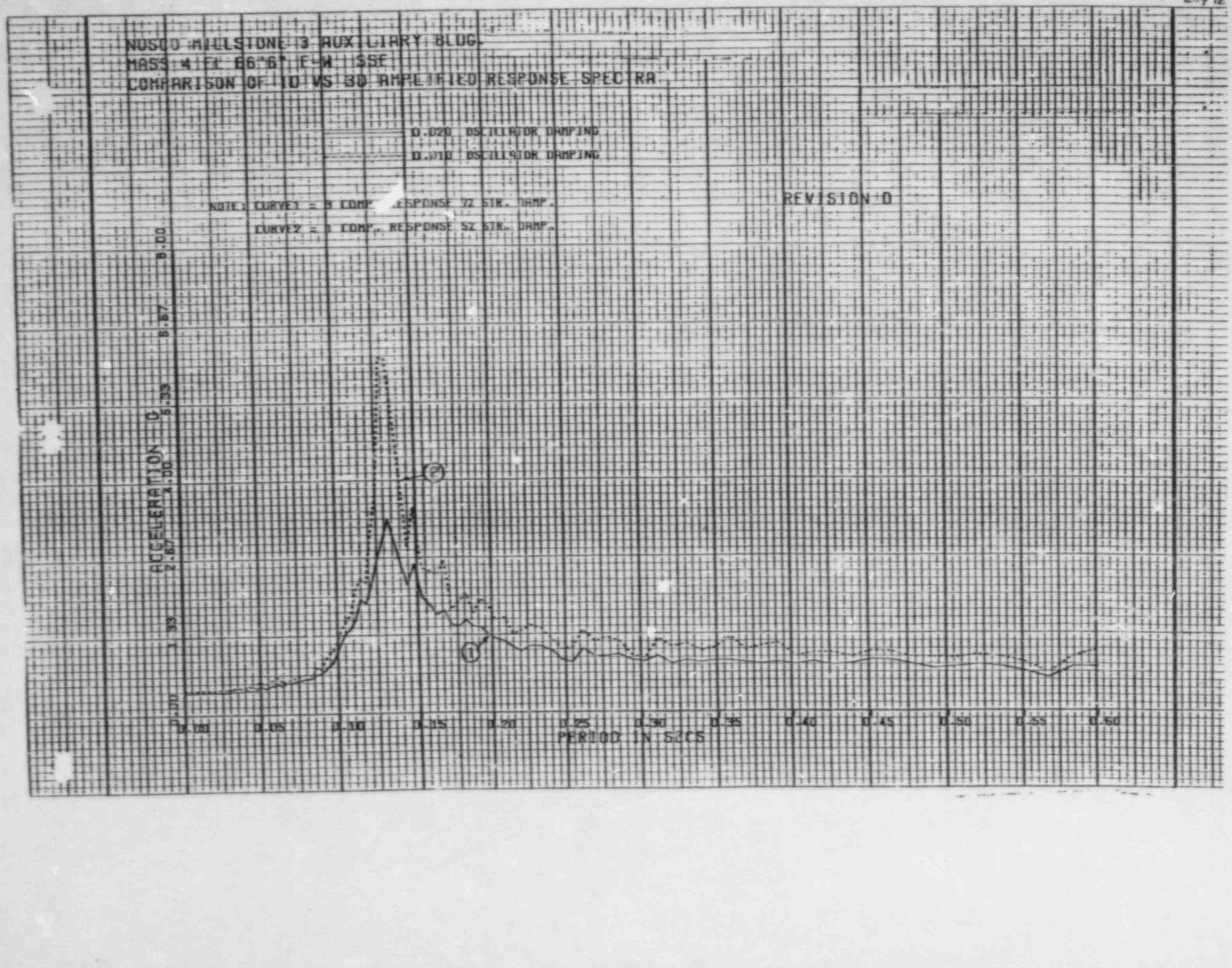

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MASS 4 EL 66'6" E-W SSE
COMPARISON OF 1D VS 3D AMPLIFIED RESPONSE SPECTRA
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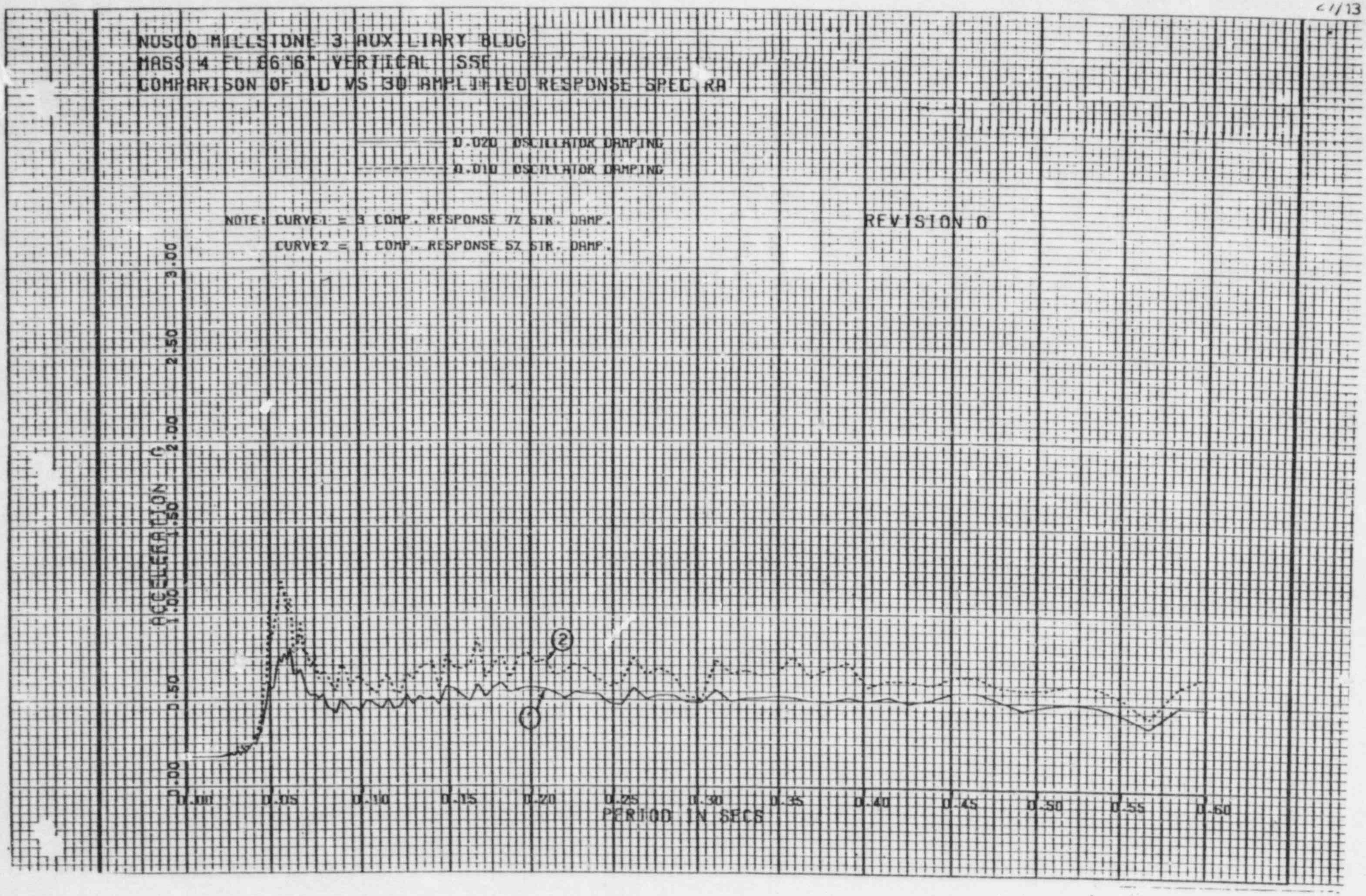

NUSCO MILLSTONE 3 AUXILIARY BLDG
MASS 4 EL 66'6" VERTICAL SSE
COMPARISON OF 1D VS 3D AMPLIFIED RESPONSE SPECTRA
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NUSCO MILLSTONE 3 FUEL BLDG.
MASS 5 EL 52'4" N-S SSE
COMPARISON OF 1D VS 3D AMPLIFIED RESPONSE SPECTRA

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NOTE: CURVE1 = 3 COMP. RESPONSE 7% STR. DAMP.
CURVE2 = 1 COMP. RESPONSE 5% STR. DAMP.

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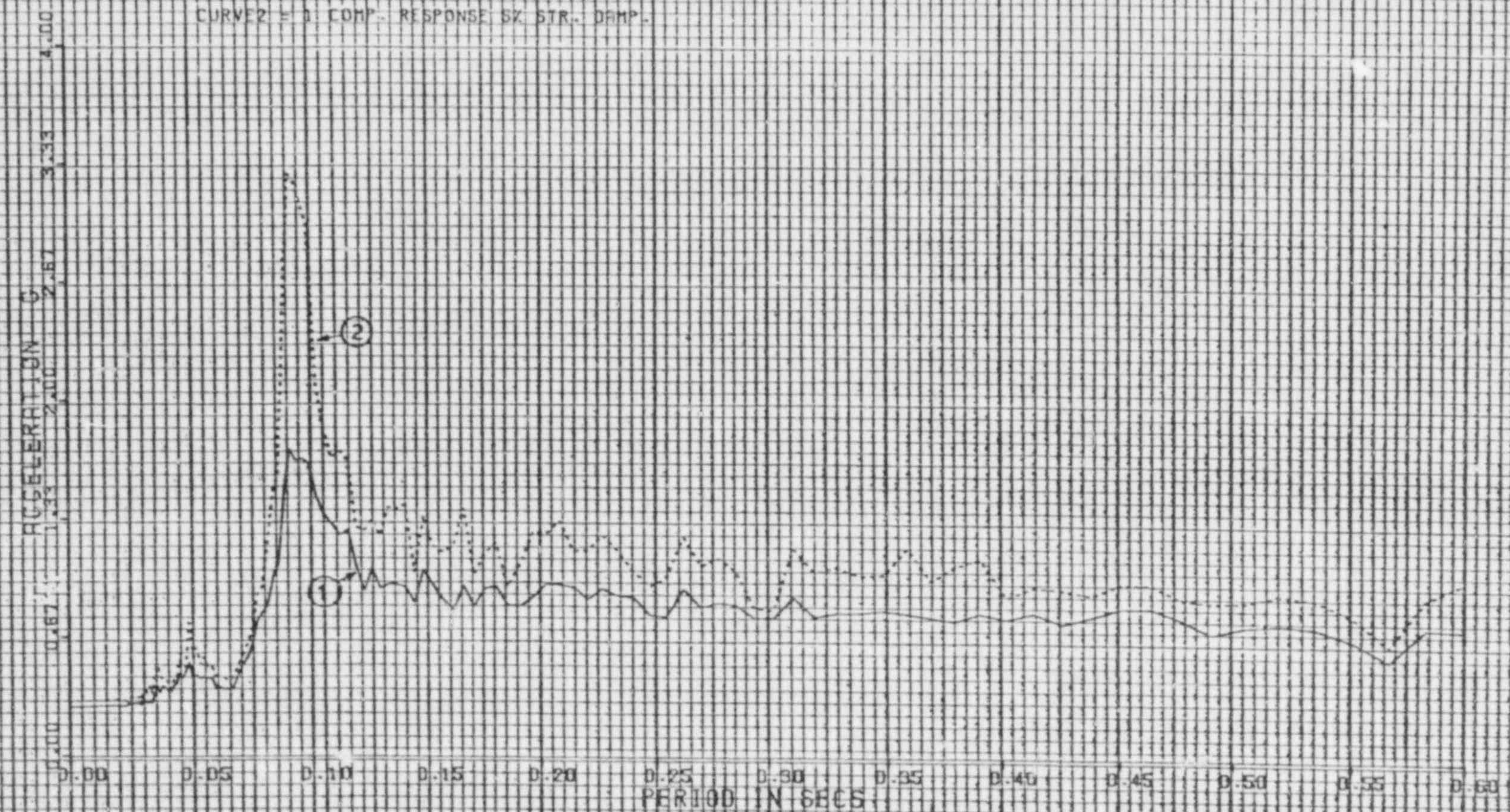

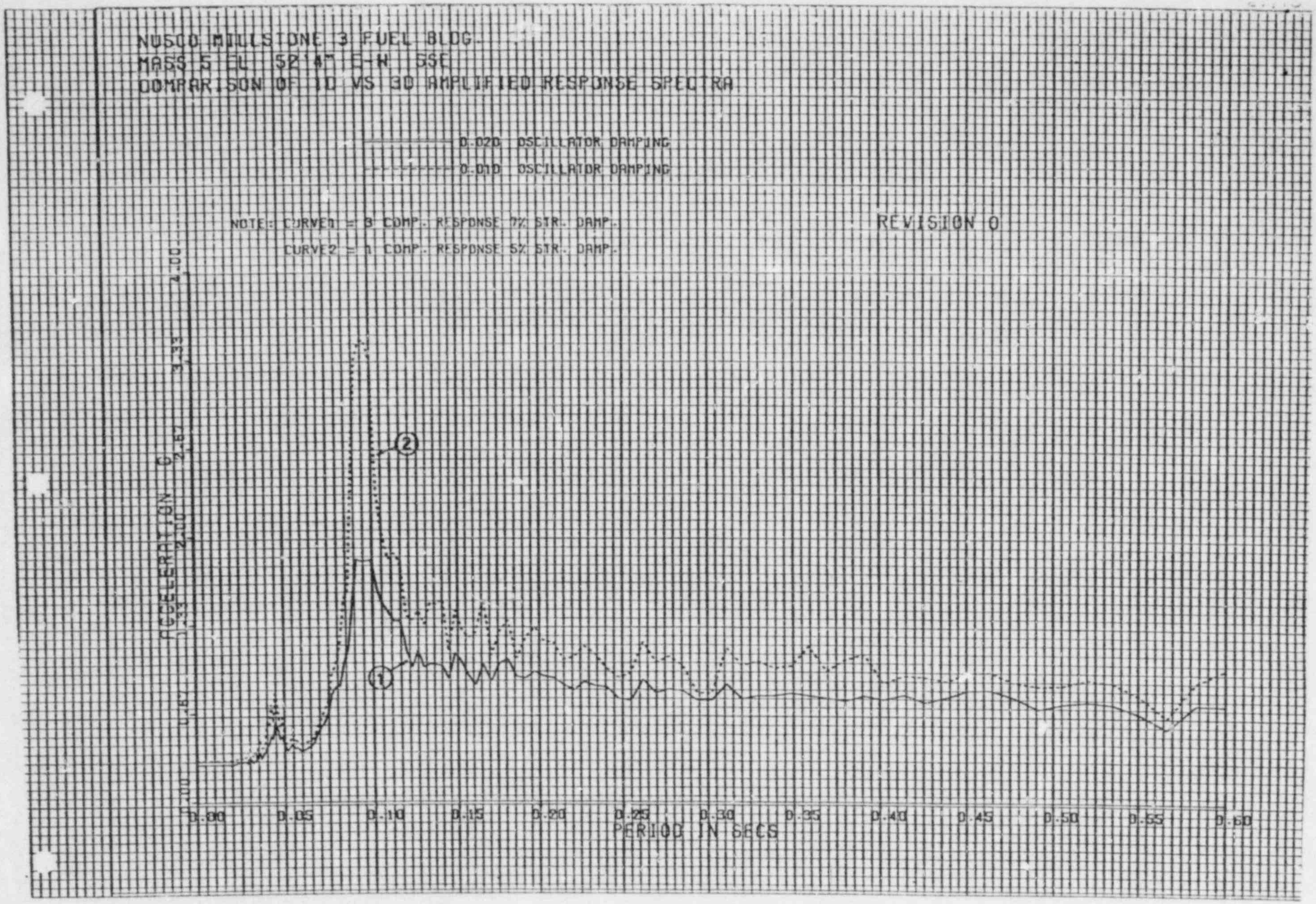
NUSCO MILLSTONE 3 FUEL BLDG.
MASS 5 EL 52'4" E-W SSE
COMPARISON OF 1D VS 3D AMPLIFIED RESPONSE SPECTRA
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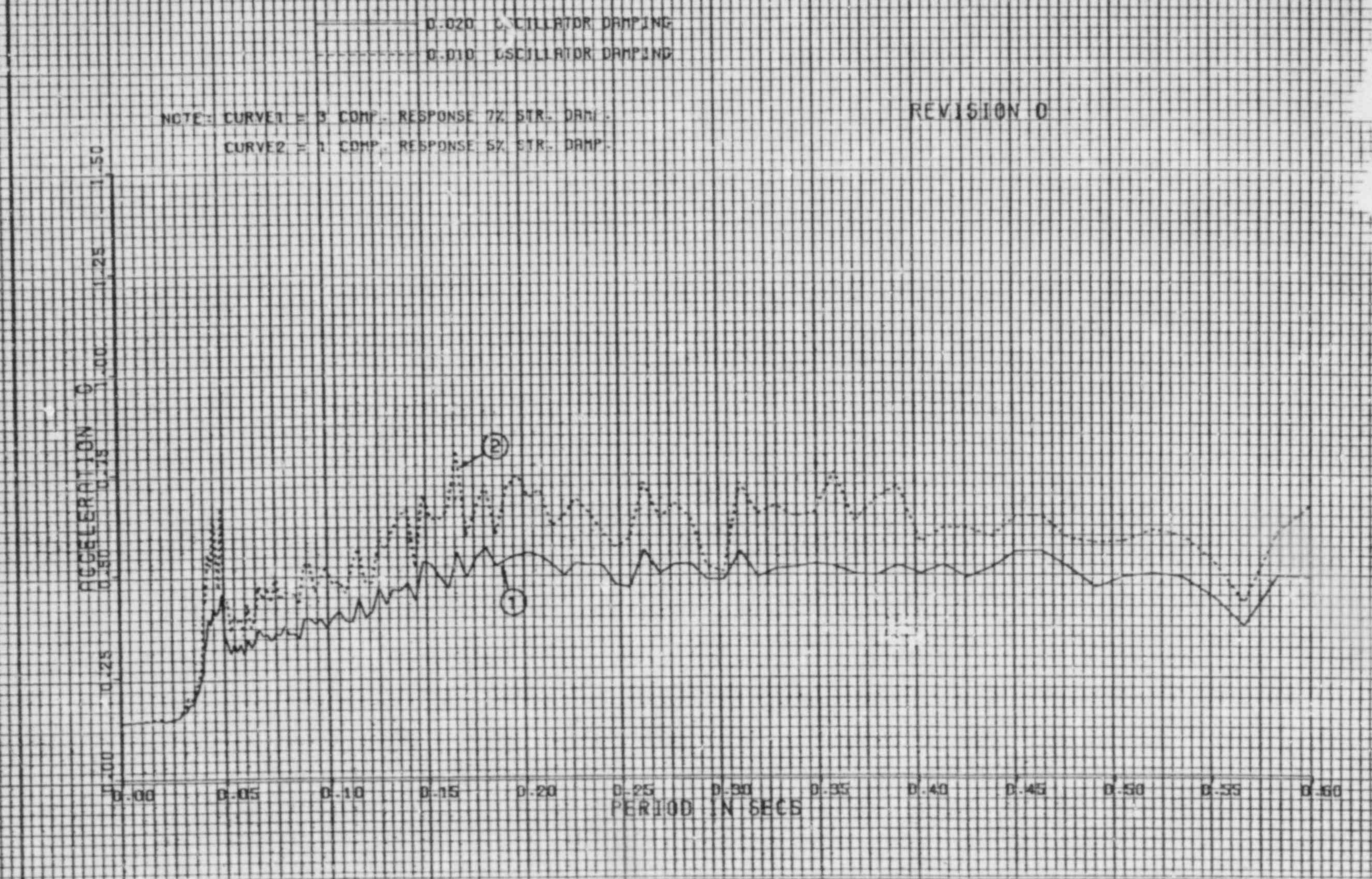
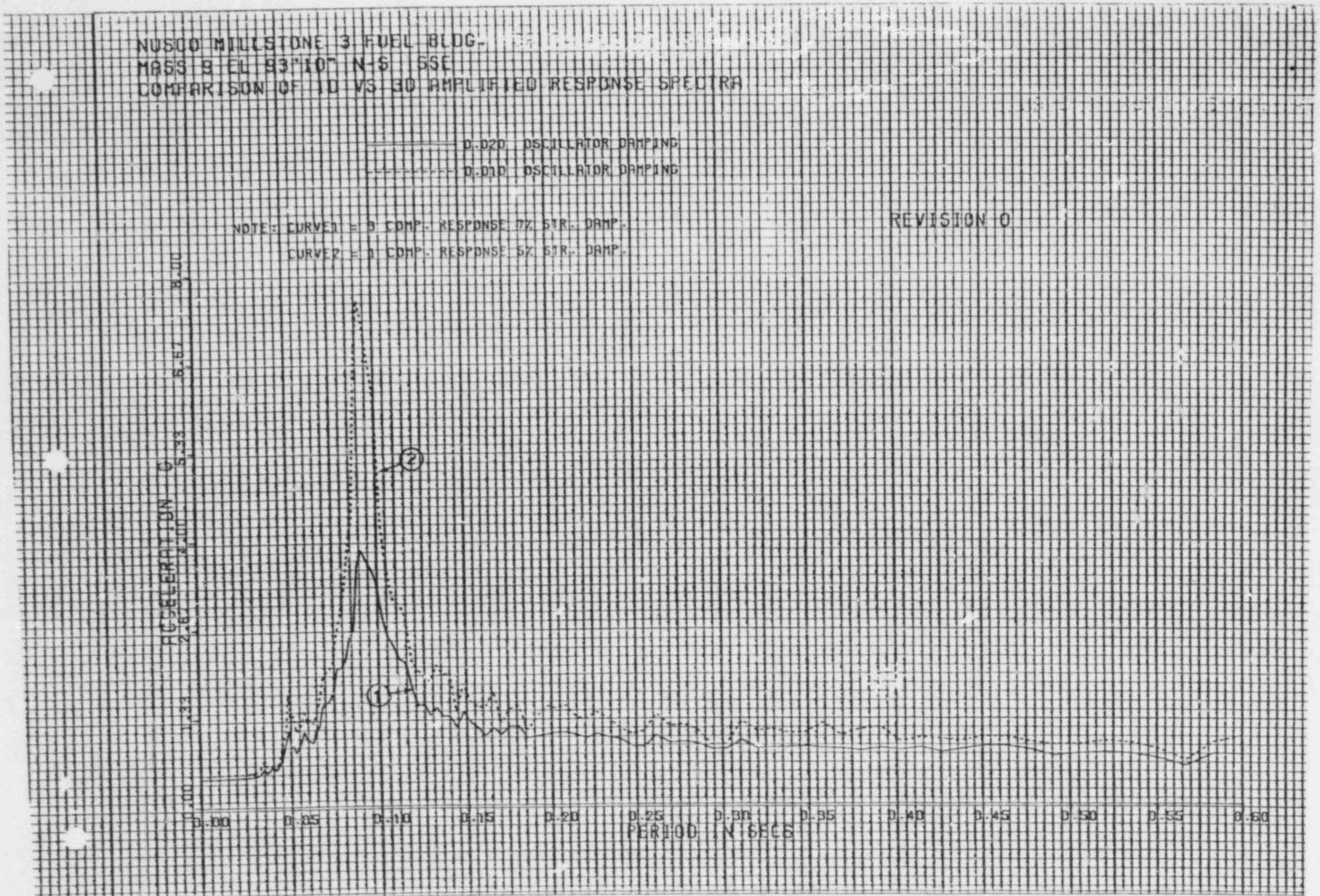
NUSCO MILLSTONE 3 FUEL BLDG.
MASS 5 EL. 52'4" VERTICAL SSE
COMPARISON OF 1D VS 3D AMPLIFIED RESPONSE SPECTRA

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MASS 9 EL 93'10" N-S SSE
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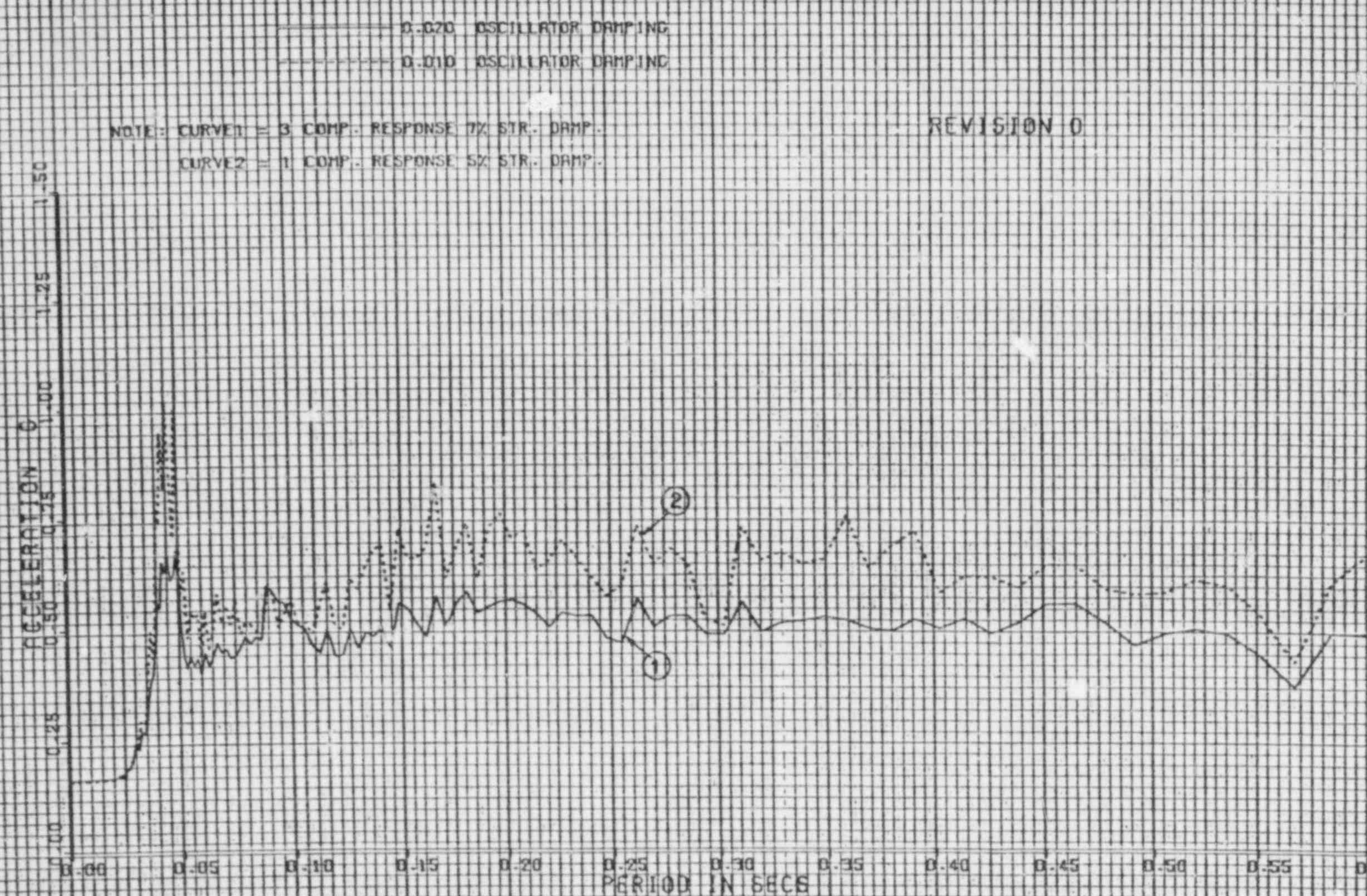
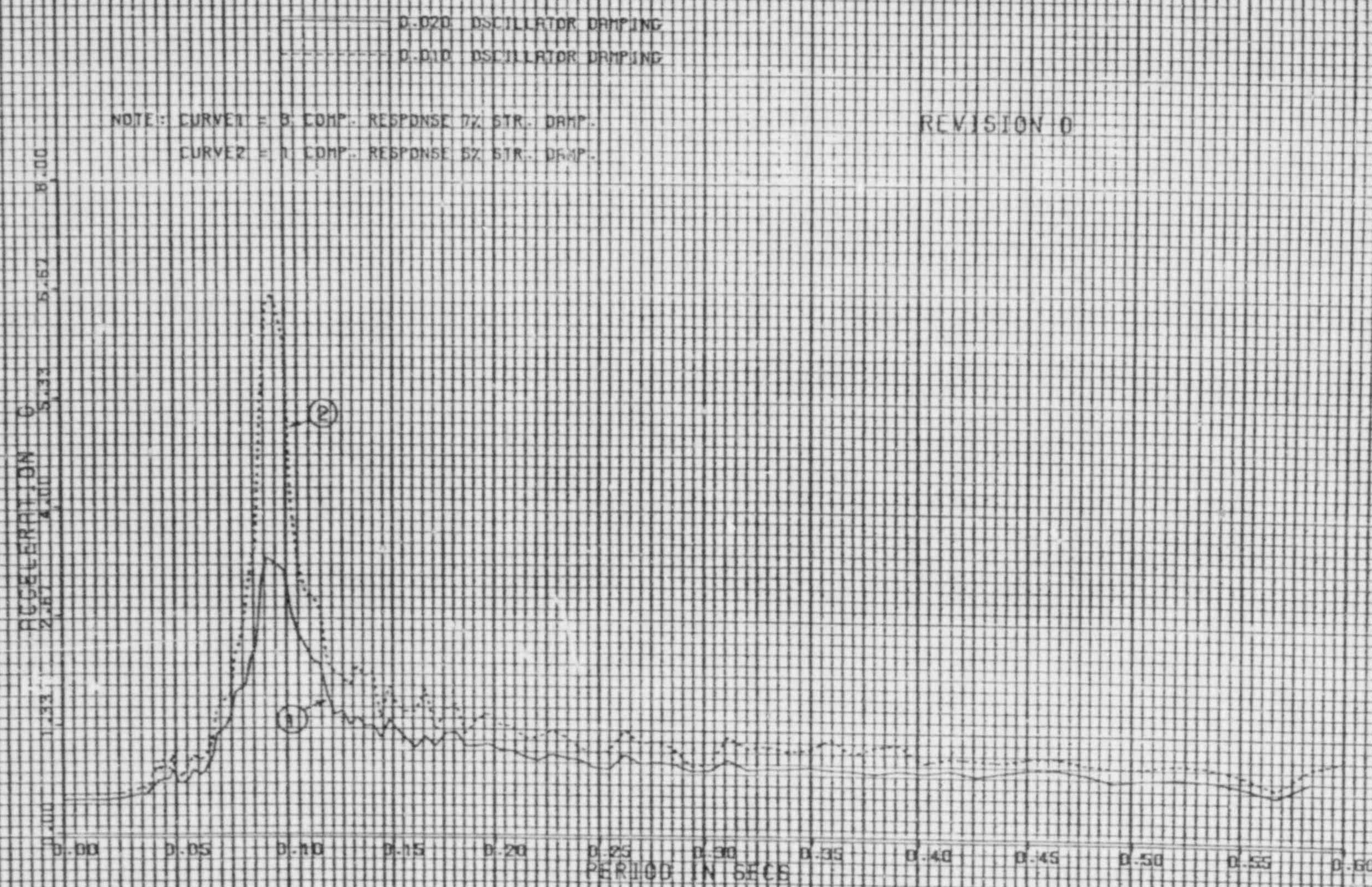
NUSCO MILLSTONE 3 FUEL BLDG.
MASS 9 EL 93'10" VERTICAL SSE
COMPARISON OF 1D VS 3D AMPLIFIED RESPONSE SPECTRA



NUSCO MILLSTONE 3 FUEL BLDG.
MASS 9 EL 93'10" E-W SSE
COMPARISON OF 1D VS 3D AMPLIFIED RESPONSE SPECTRA



DYNAMIC LOAD FACTOR CURVE
FOR TORNADO PRESSURE DROP

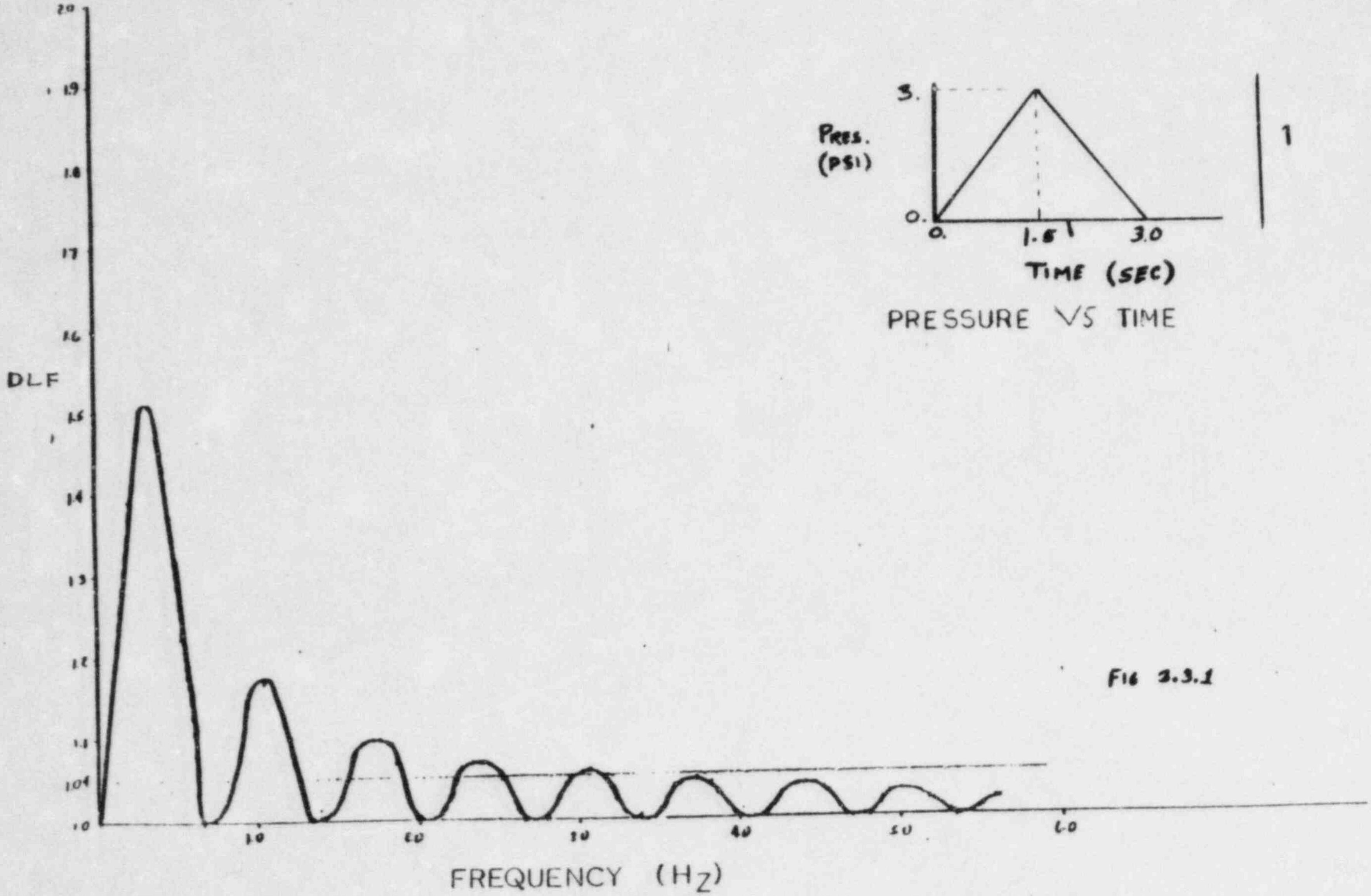


FIG 3.3.1

ITEM 29 Provide the Finite Element-Frame Analysis comparisons for the Spent Fuel Pool Analysis. An example of the results of comparison was presented and the issue was resolved. The Applicant will provide the documentation.

Response:

Spent Fuel Pool Analysis

Results of space frame model were verified by the use of a finite element model. In both models, the same geometry and loadings were used.

Documentation for comparison of results of Finite Element Analysis and Space Frame Analysis for Node 276 on the East wall of the pool are provided in the attached sheets.

Page 2 - Finite element model - element and node identification numbers in circle are the member numbers in the space frame model.

Page 3 - Tabulation of stresses for the finite element model (Syy and Sxx).

Stresses due to space frame analysis are shown in the parenthesis.

Page 4 - Sign convention for finite element and space frame analysis.

Page 5 - Graphic representation of the stresses for both methods.

Pages 6&7 - Excerpt from the computer output of the space frame analysis. Node 276 is the common joint for members 451 and 452.

Pages 8&9 - Excerpt from the computer output for the finite element analysis (stresses at Node 276 are underlined on Page 10).

By comparing the results from frame analysis and finite element analysis, it is concluded that the frame analysis results are conservative and hence they are used in the design of spent fuel pool. 1

ITEM 32 Discuss how venting was considered for internal cubicle pressurization (Sec. FSAR 3.3.3). Item resolved if a sentence to reflect actual venting is included in FSAR, i.e., that all structures except for fuel building were designed for nonventing situation. The Applicant will provide this response.

Response:

Revise the MNPS-3 FSAR p. 3.3-3 as follows:

Tornado Differential Pressure Load (W_p)

For all reinforced concrete structures with the exception of the Fuel Building and the Emergency Generator Enclosure, the exterior walls and roofs were designed for nonvented conditions (full 3 psi pressure drop).

For the Fuel Building, the section of metal roof at el 55 ft-3 in. and between column lines G.5 and H is capable of venting the building. For this reason, the design is based on two conditions. The first condition considers the structure to be vented through the area covered by this metal roof. Those sections of walls and floors of interior cubicles which are affected by the venting of the building through the roof area are designed for the full 3 psi pressure drop. For the second condition, it is assumed that this roof does not vent the building. Therefore, the exterior walls and roofs of the building are designed for the full pressure drop (3 psi).

For the Emergency Generator Enclosure, the diesel generator muffler cubicle above el 51 ft-0 in. is capable of venting. Therefore, the design of this building is also based on two conditions. The first condition considers the diesel generator muffler cubicle above elevation 51 ft-0 in. to be vented by the large openings in the exterior walls of the cubicle. The portions of walls and floors of interior cubicles which are affected by venting of the muffler cubicle are designed for the full 3 psi pressure drop. For the second condition, it is assumed that this cubicle does not vent and the exterior walls and the roof of the structure affected are designed for the full 3 psi pressure drop. 1

ITEM 33 Justification of unity DLF of pressure drop provided description and results as presented. Resolved. The Applicant will provide the response.

Response:

The MNPS-3 FSAR will be revised to add the following:

A dynamic load factor of 1.0 is applied to the pressure drop since all wall and floor panels subject to pressure drop have frequencies greater than 4 Hz as shown in Table 3.3.1. Figure 3.3.1 presents the pressure drop time history and the calculated dynamic load factor curve for the pressure drop. Comparing the two it is evident that the Millstone III structural elements experience no dynamic amplification during the pressure drop condition. 1

DYNAMIC LOAD FACTOR CURVE
FOR TORNADO PRESSURE DROP

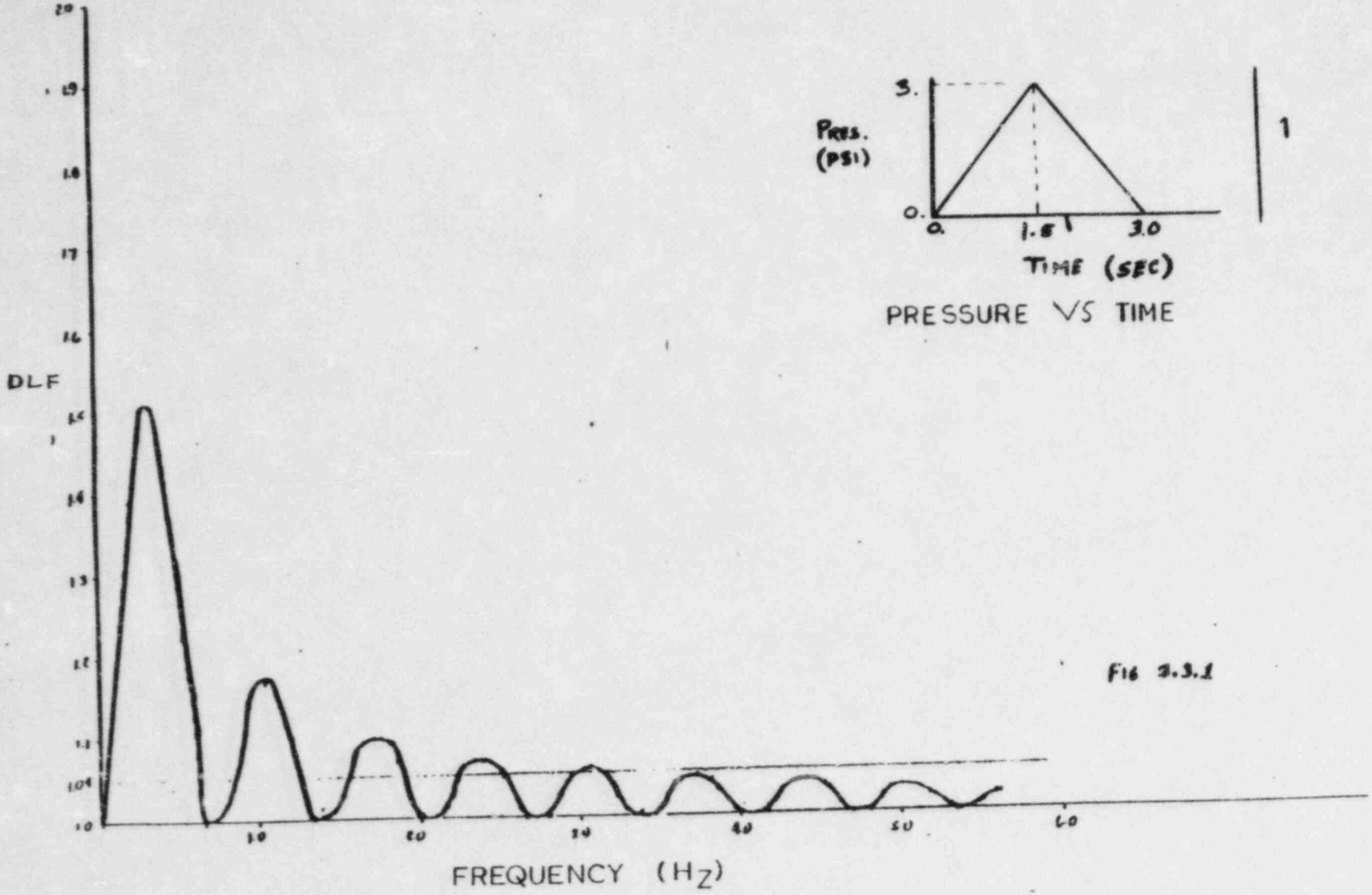


FIG 3.3.1

ITEM 36

Provide assessment of Category I tanks to include consideration of flexibility and demonstrate that the intent of NRC is met. For those tanks which are judged not to require assessment, provide basic reasons for not doing so.

RESPONSE

Category I tanks have been evaluated with consideration of tank wall flexibility. The evaluation was based on the methods and references of NUREG Contractor Report 1161. Where this analysis method is applicable, the tanks were found to respond rigidly for all modes of vibration, with the exception of the refueling water storage tank (RWST).

The RWST was reanalyzed in accordance with NUREG CR 1161 for nonrigid response and was found to be adequate without modification.

ANALYTICAL METHOD

Category I tanks are first evaluated with respect to orientation, contents and structural materials. Tanks outside the scope of NUREG CR 1161 (Reference 1) are eliminated from further consideration.

Next, tank fundamental frequency is determined by the method of References 2, 4, or other appropriate analytical techniques compatible with the tanks geometry and boundary conditions. Tanks that are rigid are eliminated from further consideration.

Flexible tanks are analyzed for seismic loading. Loads and stresses are developed in accordance with Reference 1. This analytical approach is checked against the later published work Reference 4. Structural analysis not addressed by References 1 or 4 is based on elastic behavior using commonly accepted principles of engineering appropriate to the geometry of the structure. Tank structural adequacy is determined by comparison of calculated stress against the applicable ASME Code allowable stress (Reference 3).

SUMMARY OF RESULTS

Table 1 indicates which tanks are outside the scope of NUREG CR 1161. Table 2 indicates that all remaining Category I tanks except the RWST are rigid and not subject to concern with tank wall flexibility. Table 3 indicates the RWST stress analysis results.

Table 1

CATEGORY I TANKS NOT REQUIRING EVALUATION

Identification Equipment Name, Mark No., Design Code, Class, and Location	Reasons Evaluation Not Required (Non-Applicability of Reference 1)
Reactor Plant Component Cooling Surge Tank, 3CCP-TK1, ASME III, CL 3, Auxiliary Building	Tank orientation horizontal
Process Gas Receiver, 3GWS-TK1, ASME III, CL 3, Auxiliary Building	Tank contents: gas
Emergency Generator Fuel Oil Day Tank, 3EGF-TK 2A, 2B, ASME III, CL 3, Emergency Generator Enclosure Building	Tank orientation horizontal
Control Building Charging Water Surge Tank, 3HVK-TK1A, 1B, ASME III, CL 3, Control Building	Tank orientation horizontal
Demineralized Water Storage Tank, 3FWA-TK1; ASME III, CL 3, Yard	Rigid vertical concrete tank

Table 2

CATEGORY I TANKS REQUIRING EVALUATION

Identification: Equipment Name, Mark No., Location	Design Code and Class	Orientation	Frequency	Cut-off Frequency
Refueling Water Storage Tank, 3QSS-TK1, Yard	ASME III, CL 2	Vertical	3.2 Hz	55 Hz
Boric Acid Tanks, 3CHS-TK5A,5B, Auxiliary Building	ASME III, CL 3	Vertical	22.0 Hz	22 Hz
Refueling Water Chemical Addition Tank, 3QSS-TK2, Yard	ASME III, CL 2	Vertical	75.3 Hz	55 Hz
Charging Pumps Seal Cooling Surge Tank, 3CCE-TK1, Auxiliary Building	ASME III, CL 3	Vertical	38.4 Hz	22 Hz
Safety Injection Pumps Cooling Surge Tank, 3CCI-TK1, Engineering Safety Features Building	ASME III, CL 3	Vertical	38.4 Hz	20 Hz

Table 3

RWST STRESS ANALYSIS RESULTS

Tank Component	Applied Stress (PSI)	Allowable Stress (PSI)	Safety Factor
Shell Hoop Stress	22,727	36,000	1.5
Shell Compressive Stress	5,441	6,000	1.1
Anchor Bolt Tension	20,665	46,000	2.2
Top Stiffening Ring	31,671	43,920	1.4
Vertical Stiffener	5,121	10,697	2.0
Base Ring	27,761	43,920	1.6
Skirt Weld	4,340 (lb/in)	5,771 (lb/in)	1.3

Foundation	Applied Stress (PSI)	Allowable Stress (PSI)	Safety Factor
Bearing Stress	90	1,388	High
Concrete Shear Pull-Out at Anchor Bolts	88	220	2.5
Shear at Critical Section of Mat	31	110	3.5

	Required	Actual
Factor of Safety against Overturning	1.5	2.6
Factor of Safety against Sliding	1.1	1.9

REFERENCES

1. U.S. Nuclear Regulatory Commission. NUREG Contractor Report (CR) 1161. Recommended revisions to Nuclear Regulatory Commission Seismic Design Criteria, May 1980.

2. Veletsos, A. S., Yang, J. Y. Earthquake Response of liquid-storage tanks advances in Civil Engineering through Engineering Mechanics. Proceeding of the annual EMD speciality Conference, Raleigh, N.C. ASCE, May 1977.

3. American Society of Mechanical Engineers (ASME) Boiler and Pressure Vessel Code, Section III 1971; "Nuclear Power Plant Components" and addenda through December 31, 1973.

4. Haroun, M. A.; Housner, G. W. Seismic Design of Liquid Storage Tanks. Journal of the technical councils of ASCE. April 1981.

ITEM 39 A review of accidental torsion SEB Item 08 should be performed. A review was performed and the issue was resolved. The Applicant will provide this response.

Response:

The effects of accidental torsion have been evaluated for the fuel building, control building, containment structure internals, and containment structure shell.

Calculations performed for these buildings indicate that they are able to accommodate the 5 percent additional torsion. Reinforcing steel requirements for critical wall sections in these structures are summarized below:

Building	Wall/Elev	Reinf Required (Including 5 Percent Additional Torsion)	Reinf Provided
Fuel	G/ 52 ft-4 in.	2.89 in.^{2}/foot	4.16 in.^{2}/foot
Control	E-1/ 4 ft-6 in.	2.09 in.^{2}/foot	3.74 in.^{2}/foot
Containment Structure	Shell/(-)27 ft-3 in.	2.37 in.^{2}/foot	3.23 in.^{2}/foot
Containment Structure	Cranewall Column/ (-)27 ft-3 in.	25 - #18	28 - #18

The torsional moment from the accidental torsion is assumed to occur at the center of rigidity of all resisting walls between building floors. This moment is converted into shear force in the plane of the resisting walls. 1

The shear force calculated as stated above is distributed to each wall in proportion to its shear stiffness and distance from the center of rigidity.

The accidental torsional shear is added to the normal shear force at critical elevations of a building under consideration.

ITEM 42: Question 220.9 concerns the ability of structural steel frames to withstand tornado pressure prior to siding blowout.

Revised Response:

For the original tornado wind design of the service, turbine, waste disposal, containment structure enclosure (CSE), and fuel buildings (or steel-framed portions thereof) the tornado wind load was calculated based on the projected sail area of the steel framing, with the assumption that all siding and roof deck had blown off. The stress allowables used for this design were based on SRP 3.8.4.

The above buildings have also been evaluated for the overall effects of tornado wind (i.e., stability) assuming that all metal siding and roof deck remain intact. The acceptance criterion for this case was a resultant total wind shear force less than or equal to the sum of the column base shears as originally calculated. To generate tornado wind pressures for this evaluation, the Regulatory Guide 1.76 Tornado (290 mph tangential velocity, 70 mph translational velocity, 150 foot radius) was used as input to a model which considered radial, tangential, translational, and vertical winds. The individual buildings were then evaluated for the total resulting wind pressure. For example, the east-west tornado wind load on the turbine building, assuming all siding intact and including an appropriate drag factor, was 6938^k. This compares with 7112^k, which is the sum of the east-west column base shear forces for tornado wind on exposed steel 1
framing, as originally calculated.

The above buildings have also been evaluated for local tornado wind effects, which were used to check the design of individual structural members. For windward loading, in all cases the siding was assumed to remain intact. For outward acting pressures, however, the limiting load on structural framing was based on the failure capacity of siding and roof deck fasteners. The failure capacities of the fasteners were provided by the siding and roof deck contractor, and were based on test results. The failure load of a turbine building siding liner panel fastener, for example, is 1,200 pounds. Spaced at 12 inches, the limiting load on a turbine building girt due to a local outward acting pressure was 1,200 pounds per foot. For this evaluation, the acceptance criterion was that the buildings did not collapse.

The steel-framed portion of the Fuel Building is not Seismic Category I. The Turbine Building, Waste Disposal Building, and the steel-framed portion of the Service Building are non-safety related. The only FSAR commitment under tornado loading is that the non-seismic/non-safety related portions of these buildings do not collapse (see FSAR Sections 3.3.2.3 and 3.8.4.1). The CSE, which houses the secondary leak collection and release system (SLCRS), is seismic Category I. However, the SLCRS is not required under tornado loading. The only FSAR commitment for the CSE under tornado loading is that the building framing "remain intact". Therefore, SRP design allowables were used only as a guide for evaluating individual structural members in the above buildings. Local cases in which theoretical bending moments exceeded the AISC plastic moment capacity were acceptable provided that the building stability was not affected had the overstressed member been ignored in stability analysis.

The result of these evaluations is that the service, turbine, waste disposal, containment enclosure, and fuel buildings will not collapse under tornado wind loading, whether or not the siding remains in place at very high wind velocities. 1