Client: Boston Edison

Calculation No. C-002

Title: Soil Properties for the Soil Structure Interaction Analysis for the Pilgrim Site

Project: IPEEE - Seismic / A-46

Method:

INFORMATION ONLY

Acceptance Criteria: ASCE 4-86

Remarks:

REVISIONS

No.	Description	By	Date	Chk.	Date	App.	Date
0	Initial Issue	TMT	1-25-93	MSL	1-26-93	[illegible]	1-26-93

S&A Stevenson and Associates

CALCULATION COVER SHEET

FIGURE 1.3

CONTRACT NO. 91-C2672

For the Reactor Building, the Radwaste Building, the Diesel Generator Building, and the Intake Structure, the compacted fill layer has a depth of 45 ft according to the GEI report. For the Turbine Building, the compacted fill layer has a depth of 35 ft.

For the SSI analysis, the average shear wave velocity across each layer is calculated. The input data for the SSI programs is summarized in the following tables:

Layer No.	Thick (ft)	Shear Wave Velocity (ft/sec)	Density (lb*sec^2/ft)	Damping Ratio (%)	Poisson's Ratio
1	10	535	3.92	0.02	0.33
2	10	745	3.92	0.02	0.33
3	10	860	4.26	0.02	0.4
4	10	925	4.26	0.02	0.4
5	5	963	4.26	0.02	0.4
6	5	1215	4.01	0.02	0.4
7	10	1255	4.01	0.02	0.4
8	10	1310	4.01	0.02	0.4
9	10	1365	4.01	0.02	0.4
10	10	1415	4.01	0.02	0.4
11	10	1465	4.01	0.02	0.4
Rock	-	3000	5.22	0.02	0.4

Table 2 - Reactor Building, Radwaste Building, Diesel Generator Building, Intake Structure

Layer No.	Thick (ft)	Shear Wave Velocity (ft/sec)	Density (lb*sec^2/ft)	Damping Ratio (%)	Poisson's Ratio
1	10	535	3.92	0.02	0.33
2	10	745	3.92	0.02	0.33
3	10	860	4.26	0.02	0.4
4	5	913	4.26	0.02	0.4
5	5	1153	4.01	0.02	0.4
6	10	1200	4.01	0.02	0.4
7	10	1255	4.01	0.02	0.4
8	10	1310	4.01	0.02	0.4
9	10	1365	4.01	0.02	0.4
10	10	1415	4.01	0.02	0.4
11	10	1465	4.01	0.02	0.4
Rock	-	3000	5.22	0.02	0.4

Table 3 - Turbine Building

The soil material damping ratio is assumed to be 2 percent. The final soil damping values, however, were calculated from LAYSOL iterated properties. The Poisson ratio is assumed to be 0.33 for soil above the water table, and 0.40 for saturated soil. The soil densities are given in GEI report (Appendix A-1).

Water Table

As reported by GEI, the water table ranges from +1 to +6 feet above the mean sea level for the Reactor Building and varies from +2 to +7 feet for the Turbine Building. The location of the water table is not critical for the SSI analysis, it affects only the unit weight and the Poisson ratio. The effect will be much less significant than the variation of the shear modulus.

In the SSI analysis, the water table is assumed to be located at +1 feet for all buildings. The level of water will the subject of a parameter study in a separate calculation.

Variation of the Soil Shear Wave Velocities

For the PRA analysis, the variation of soil properties must be taken into account. Among the soil properties, the shear wave velocity or shear modulus has the highest uncertainty. Each analysis in this study is based on three representative runs, namely, the best estimate, the low bound, and the high bound soil properties.

The best estimate properties are the values recommended in previous sections. The low bound and the high bound properties are taken at the plus and minus one standard deviation estimates. According to the recommendations by Professor Whitman, the standard deviation of the shear wave velocity is 15% of the best estimate at the base of the stratum increasing to 35% at ground surface to reflect the greater uncertainty concerning wave velocity at shallow depth in cohesionless soil. The standard deviation of the outwash is 35% of the best estimate considering the wide spread between the available data.

In this study the standard deviation of the shear wave velocity is taken as 35% of the best estimate. This variation in shear velocity corresponds to 82% (1.35 * 1.35 - 1) variation in the shear modulus, which is greater than the minimum of 50% required by the ASCE Standard, but lower than the 100% required by the Standard Review Plan.

In the SSI high bound analyses, the shear wave velocities in tables 2 and 3 are multiplied by a factor of 1.35. In the low bound analyses, the shear wave velocities are divided by 1.35.

Foundation Depth

According to the design drawings, the foundation base level are approximately

Building	Common Z	Reference
Reactor Building	-23 ft	GEI Report
Turbine Building	-3 ft	GEI Report
Radwaste Building	-3 ft	Drawing 6498M-26 Rev.E4
Diesel Generator Building	23 ft	Drawing 6498M-26 Rev.E4
Intake Structure	-24 ft	Drawing 6498C-47 Rev.E2

The closest soil layer elevation is selected for the foundation embeddment depth in the LAYSOL analyses. The grade level is approximately 22 ft for all buildings. These foundation depths are used in the EKSSI input as Common Z which ties the model fixed-base to the foundation impedance matrix at this level.

Appendix A

A-1 Letter from Eugene A. Marciano, GEI Consultants, Inc., February 28, 1992, 91C2672-LRS2-002

A-2 Letter from Eugene A. Marciano, GEI Consultants, Inc., February 28, 1992, 91C2672-LRS2-003

A-3 Letter from Robert V. Whitman, Massachusetts Institute of Technology, November 30, 1992, 91C2672-LRS6-001

A-4 Letter from Robert V. Whitman, Massachusetts Institute of Technology, December 18, 1992, 91C2672-LRS6-002

GEI Consultants, Inc.

1021 Main Street
Winchester, MA 01890-1943
617•721•4000

February 28, 1992
Project 92012

Mr. Thomas J. Tracy
Vice President
Stevenson & Associates
Ten State Street
Woburn, MA 01801

Dear Mr. Tracy:

Re: **Shear Wave Velocities, Unit Weights, and Ground Water Table**
Pilgrim IPEEE, Pilgrim Station, Plymouth, Massachusetts

This letter provides a description of the stratigraphy, unit weights, and shear wave velocities for the soils beneath and surrounding the reactor and turbine buildings of Pilgrim 1. In addition, the ground water fluctuation in this area is provided.

Stratigraphy

The stratigraphy in the area of the reactor and turbine buildings is shown in the attached Fig. 1. It consists of approximately 35 to 45 feet of compacted fill materials, designated as type A and type B fills on Bechtel Drawing C8, above approximately 45 to 35 feet of glacial outwash deposits, which are underlain by bedrock at a depth of approximately 80 feet. The type A and B fills are specified to have been compacted to a minimum of 98% and 96%, respectively, of the maximum dry density as determined by ASTM D1557 and have similar ranges of values for unit weight and shear wave velocity. The outwash deposits are very dense as a result of loading due to glaciation subsequent to their deposition. The outwash deposits are granular, consisting predominately of poor- to well-graded sands. The limits of the compacted fill areas beyond the area of the reactor and turbine buildings are also shown on Drawing C8.

Sections F and H of Drawing C8 indicate that the reactor building is founded on the outwash material. Section A indicates that at least a portion of the turbine building foundation is underlain by type A fill. The elevations of the building foundations and thicknesses of fill are approximate and should be verified when a complete set of drawings becomes available from BECO.

Groundwater Table

The elevation of the ground water table in this area can be expected to experience the following fluctuations due to tidal effects and normal rainfall:

Reactor Building	+1 to +6 feet above mean sea level (depths of 21 to 16 feet)
Turbine Building	+2 to +7 feet above mean sea level (depths of 20 to 15 feet)

This is based on observation well readings conducted by GEI[1] over nearly a 3-year period within and surrounding the Pilgrim 1 area. This does not include the potential effects of flooding, storm surges, or other extreme events on the ground water table.

Total Unit Weights

Based on the data available in the soils report[2] for Pilgrim 2, the average total unit weights for the soil strata are 126 pcf for the compacted fill above the water table, 137 pcf for the compacted fill below the water table, and 129 pcf for the outwash deposits. Bechtel indicates in the soils report a unit weight of 168 pcf for the bedrock.

Shear Wave Velocities

The results of seismic crosshole testing conducted by Weston Geophysical for the site of Pilgrim 2 in 1972 and 1976 is available in the soils report[2]. The results are plotted in Fig. 2 and range from 1,700 to 2,700 fps. There is no compacted fill in this area. Therefore, only the cross-hole results below a depth of about 35 feet are relevant to the Pilgrim 1 site. For the outwash deposits, the following shear wave velocities were recommended for design by Bechtel[2] based on the cross-hole results.

---

[1]GEI (1983). "Analysis of Groundwater Levels, Pilgrim Station Unit 1, Plymouth, Massachusetts," February 28.

[2]Soils Report prepared by Bechtel as part of Pilgrim 2 PSAR, dated August 31, 1976, Amendment 26 (contains GEI soils data reports).

Depth (ft)	Elevation (ft)	Shear Wave Velocity (fps)
35 to 51	-13 to -29	1,950
51 to 71	-29 to -49	2,300
71 to 80	-49 to -58	2,650
>80	<-58	5,900

In addition, we have estimated the shear wave velocities of the outwash soils and compacted fills based on field exploration data and laboratory testing data from the soils report for Pilgrim 2[2]. The outwash deposits of the Pilgrim 1 and Pilgrim 2 sites have similar soil descriptions and ranges of blowcounts and are part of the same depositional history and were both subjected to glacial loading. This information indicates that the characteristics of the outwash materials at Pilgrim 1 and Pilgrim 2 can be expected to be similar.

The results of our estimates of the shear wave velocities are shown in Fig. 2. They are based on blowcount data and laboratory testing on samples obtained from the same area as Weston Geophysical's cross-hole tests for Pilgrim 2. All of the plotted points and curves in this figure are based on a ground water table elevation of +5 feet, i.e., a depth of 17 feet below the ground surface.

Values of shear wave velocity versus depth were calculated and plotted using the following field and laboratory soils data, which were obtained for the outwash deposits in the vicinity of the Pilgrim 2 cross-hole tests:

1) Blowcount data within the glacial outwash corrected for the influence of gravel content.

2) Impulse shear wave velocity tests on undisturbed samples of glacial outwash.

3) Resonant column test results on specimens prepared by compaction of materials from bulk samples obtained from the glacial outwash. The bulk samples were obtained from borings in the vicinity of the Pilgrim 2 cross-hole tests.

In addition, Hardin and Drnevich's relationship for granular materials was used to calculate curves of shear wave velocity versus depth using ranges of measured values for the unit weight and of estimated values of the at-rest coefficient of lateral earth pressure, K_o. This was done for both the compacted fill and the outwash deposits, which have different unit weights and different values of K_o. The range of unit weights of the outwash deposits were determined from *in situ* field density test results. The range of unit weights of the compacted fills were estimated using the results of compaction tests on samples of the outwash materials. The gradation of these compaction samples meets that specified by Bechtel[2] for the compacted fill.

For the compacted fills, upper and lower bound estimate curves for the shear wave velocity are plotted from depths of 10 to 50 feet. For the outwash deposits, upper and lower bound estimate curves are plotted from depths of 35 to 80 feet. The best estimate curve for the fill and the outwash materials is plotted from 0 to 80 feet, passing midway between the upper and lower bound curves.

The plotted results based on the three sources of data listed above generally fall within the range of values indicated by the curves based on Hardin and Drnevich's expression with the fourth source of data, the unit weights and estimated values of K_o, as input.

The estimated values of shear wave velocity are considerably lower than the results of the cross-hole tests. This may be the result of the specific procedures used to perform the cross-hole tests for Pilgrim 2 including the use of explosives for the signal source and the large spacings between the source and receiver holes. The use of explosives for the source generates a much larger percentage of compressive wave (P wave) energy than shear wave (S wave) energy. The velocity of the S wave is typically about half of that of the P wave, and thus the P wave always arrives before the S wave. The result of this is that the P wave tends to obscure the arrival time of the S wave recorded at the receiver holes. In addition, the large spacings (approximately 150 feet) between the source and receiver holes may have resulted in refraction of the wave through deeper, denser layers, which tends to overestimate the shear wave velocity.

It is not possible from the information available to conclusively determine if the cross-hole results are in error. Nevertheless, the similarity of the estimates obtained using four independent sources of field and laboratory data indicates that these estimates should not be ruled out either.

For the outwash materials, we recommend that whichever of the two shear wave velocity profiles will result in the more severe loading, i.e., either the best estimate curve shown in the figure or Bechtel's recommended values, which are given above, be used. In either case, the best estimate curve passing midway between the upper and lower bound curves in Fig. 2 should be used for the fills. Alternatively, cross-hole determinations of shear wave velocity could be made. These measurements should be made using closely spaced (10 to 15 feet) boreholes with signal generation that enhances shear wave propagation.

If you have any questions, please contact me or Dr. Gonzalo Castro.

Sincerely yours,

GEI CONSULTANTS, INC.

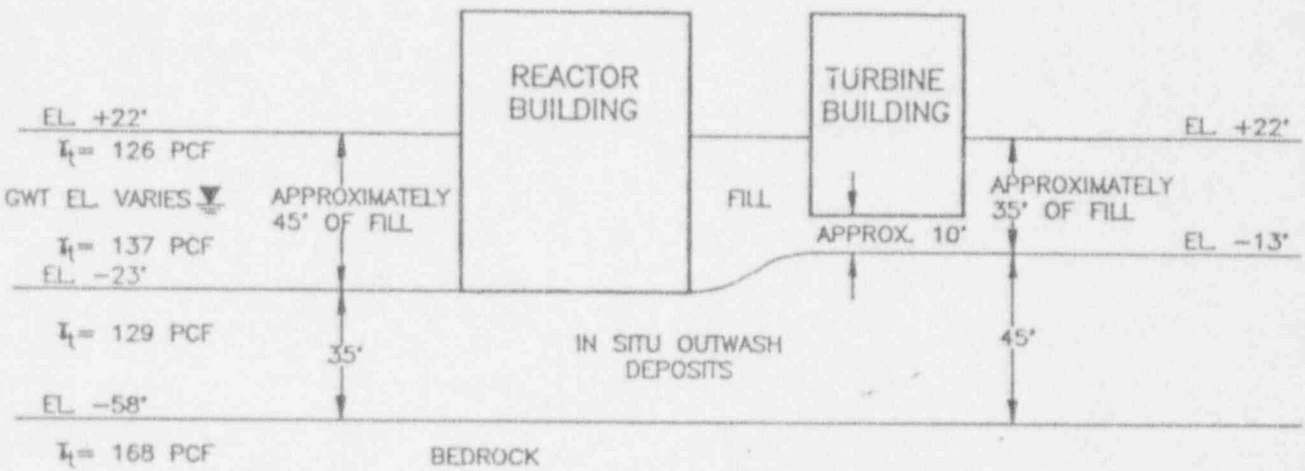
Eugene Marciano

Eugene A. Marciano, Ph.D.
Project Manager

EAM:ms

Enclosures

9/C2672-C002-A1 Sh 6 of 7



NOT TO SCALE

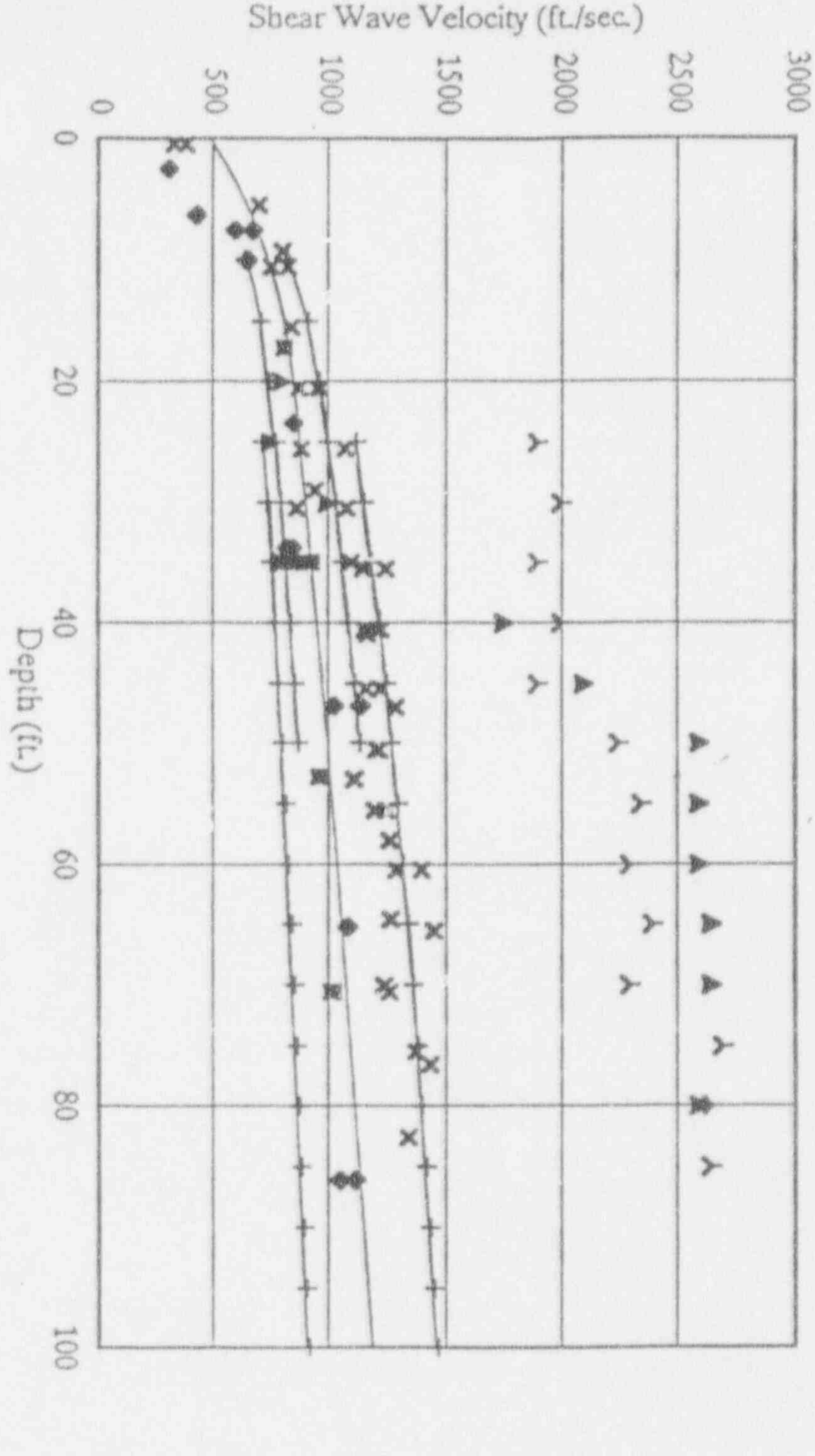
NOTE

THE FOUNDATION ELEVATIONS OF THE REACTOR AND TURBINE BUILDINGS AND THE THICKNESS OF FILL ARE ESTIMATED FROM SECTIONS A, F AND H OF BECHTEL DRAWING C8 AND SHOULD BE VERIFIED FROM THE DESIGN AND AS-BUILT DRAWINGS FOR PILGRIM 1.

Stevenson & Associates Woburn, Massachusetts	Pilgram 1 IPEEE	SOIL PROFILE SCHEMATIC DRAWING
GEI Consultants, Inc.	Project 92012	February 1992 Fig. 1

92012.DWG 2/26/92 AJS

91C2672-C002-A1 Sh 7 of 7



× Correlation with Blow Counts
■ Impulse Test Results
▶ Crossbole Test Results 1976
+ Based on Range of γ_{dry} and K_o
◆ Resonant Column Results
⤙ Crossbole Test Results 1972

91C2672-C002-A2 S418

GEI Consultants, Inc.

1021 Main Street
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617-721-4000

March 23, 1992
Proi 92012

91C2672S2-LRS2-003

Mr. Thomas J. Tracy
Vice President
Stevenson & Associates
Ten State Street
Woburn, MA 01801

Dear Mr. Tracy:

Re: **Poisson's Ratio and Small Strain Damping Values**
Pilgrim IPEEE, Pilgrim Station, Plymouth, Massachusetts

This letter is in response to Dr. Tsiming Tseng's request for recommended values of Poisson's ratio and the small strain damping ratio for the soil-structure-interaction analyses.

The outwash deposits and the compacted fills at the Pilgrim 1 site are very dense granular materials. These materials are relatively free draining and so can be expected to experience at least partial drainage during a seismic event. For this type of material, a Poisson's ratio of about 0.33 to 0.40 is reasonable. The damping ratio at small strains can be taken as 1/2 to 1% based on the range of values reported in the literature for granular materials.

If you have any questions, please contact me.

Sincerely yours,

GEI CONSULTANTS, INC.

Eugene Marciano

Eugene A. Marciano, Ph.D.
Project Manager

EAM:ms

91C2672 C002-A3 Sh1 of 3

ROBERT V. WHITMAN
MASSACHUSETTS INSTITUTE OF TECHNOLOGY, CAMBRIDGE, MA 02139

91C2672-LRS6-001

Room 1-342
Tel: 617-253-7127
FAX: 617-253-6044
email: rwhitman@eagle.mit.edu

November 30, 1992

Stevenson & Associates
Attn: Thomas J. Tracy
10 State Street
Woburn MA 01801

Dear Mr. Tracy:

In response to your letter of 19 October, I have reviewed the information concerning shear wave velocities for the soils at the site of the Pilgrim Nuclear Station. In particular, I have studied the data provided in a report "Pilgrim IPEEE, Plymouth, Massachusetts", dated July 9, 1992 and prepared by GEI Consultants, Inc.

My recommendations for shear wave velocities are given on the attached figure. There are separate sets of curves for compacted fill and for glacial outwash. For each set, there is a best estimate curve plus curves for this best estimate plus and minus one standard deviation. The best estimate values may be tabulated as follows:

	Shear Wave Velocity - ft/sec	
Depth - ft	Fill	Outwash
0	400	
10	670	
20	820	
30	900	1100
40	950	1170
50	1000	1230
60	1050	1280
70		1340
80		1390
90		1440
100		1490

The standard deviation for the fill is 15% of the best estimate, increasing (above 10 foot depth) to 35% at ground surface to reflect the greater uncertainty concerning wave velocity at shallow depths in cohesionless soils. The standard deviation for the glacial outwash is 35%. This number reflects the apparent discrepancies among the reported data. I do not believe that the very large reported velocities are realistic, and - as noted in the GEI report - there are reasons for doubting these data. On the other hand, it does

C002-A3 Sh 2 of 3

seem possible, or even likely, that in-situ velocities exceed those measured in laboratory tests.

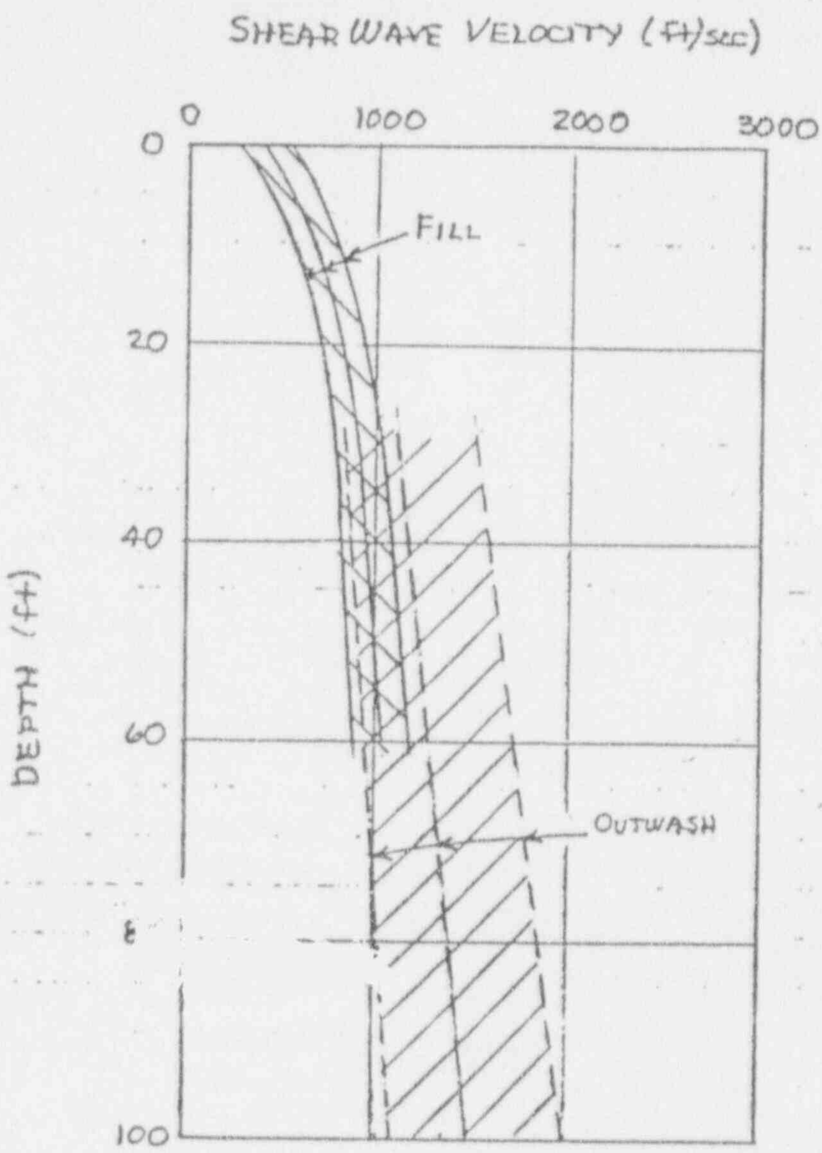
Use of the original Seed-Idriss curves for modulus degradation and damping still represents the state-of-the-art. Their continuing validity has been confirmed by a recent study, in which all data pertaining to soils with near-zero plasticity were reviewed (see Vucetic and Dobry, "Effect of soil plasticity on cyclic response", J. Geotechnical Engineering, ASCE, Vol. 117, GT1, January, 1991.) While the data on which these curves are based come from laboratory tests upon reconstituted samples, these curves apply to in-situ conditions provided that cementation is not a significant factor - which it is not for the Pilgrim site.

Please do not hesitate to contact me if you need clarifications concerning these recommendations.

Best regards,

Robert V. Whitman

Robert V. Whitman


SHEAR WAVE VELOCITY (ft/sec)
0
1000
2000
3000
0
20
40
60
100
DEPTH (ft)
FILL
OUTWASH

C002 Ry Sh. 1 of 2

91C2672-LRS6-002

ROBERT V. WHITMAN

MASSACHUSETTS INSTITUTE OF TECHNOLOGY, CAMBRIDGE, MA 02139

Room 1-342
December 18, 1992

Tel: 617-253-7127
FAX: 617-253-6044
email: rwhitman@eagle.mit.edu

Stevenson & Associates
Attn: Thomas J. Tracy
10 State Street
Woburn MA 01801

Dear Mr. Tracy:

You have asked me to document the basis for the recommendations, concerning shear wave velocities for the Pilgrim site, made in my letter to you dated 30 November 1992.

As regards the compacted fill, I selected as most reasonable the resonant column test results in Figure 6 of the report by GEI Consultants. This is a well-developed test procedure that has been found to give results comparing well to those measured in situ. My best estimate curve is the same as the GEI recommended curve, except near the ground surface where I reduced the velocities to accord better with the results from the resonant column tests. I then made a calculation for the standard deviation of the scattered data points in this figure, with respect to the mean curve. This resulted in the recommended standard deviation of 15%, except that I rather arbitrarily increased the standard deviation near ground surface to account for the greater scatter of data in this zone.

As regards the outwash deposit, I rejected as unreasonable the large values reported from the *in situ* measurements. General experience indicates tha such large values are quite unlikely unless sands are cemented, and the record contains no such description for the outwash deposits at Pilgrim. I am aware of instances where more recent measurements of *in situ* shear velocities , using modern methods, have resulted in values substantially lower than those measured some years ago by Weston Geophysical.

At the same time, it is credible that a deposit in place for several millenia might have a velocity larger than measured in the laboratory using samples that have had at least some disturbance. I hypothesized a 50% proabability that the velocities might be 1.5 times those measured in the laboratory. This implies mean values 1.25 times those measured in the laboratory, with a 35% standard deviation. I felt quite comfortable with

Coo2 A4 Sh 2-82

this result. The -1σ curve for outwash fell somewhat above that for compacted fill, while the +1σ curve for outwash was credible to me as giving possible although unlikely values. Hence I felt very comfortable with the expectation that computations would be made using such a range of values.

Please let me know if I can provide any further clarifications.

Sincerely yours,

Robert V. Whitman

Robert V. Whitman

ATTACHMENT 3

SPECTRA COMPARISON PLOTS

This attachment consists of excerpts from EQE Document No. 42103-0-009 entitled "Seismic Reanalysis of Reactor Building, Pilgrim Nuclear Power Station", dated July 1993 (BECo document reference SUDDS/RF 93-142) which compare Reactor Building Safe Shutdown Earthquake (SSE) spectra at 5% damping for the following cases:

- Pilgrim design floor spectra: These spectra were developed by Bechtel using a 0.15g SSE Taft time history to conservatively envelope the Housner requirement shown in FSAR Figure 2.5-6. These spectra are incorporated in BECo Specification C-114-ER-Q-E0 and are the in-structure floor spectra being used for USI A-46.

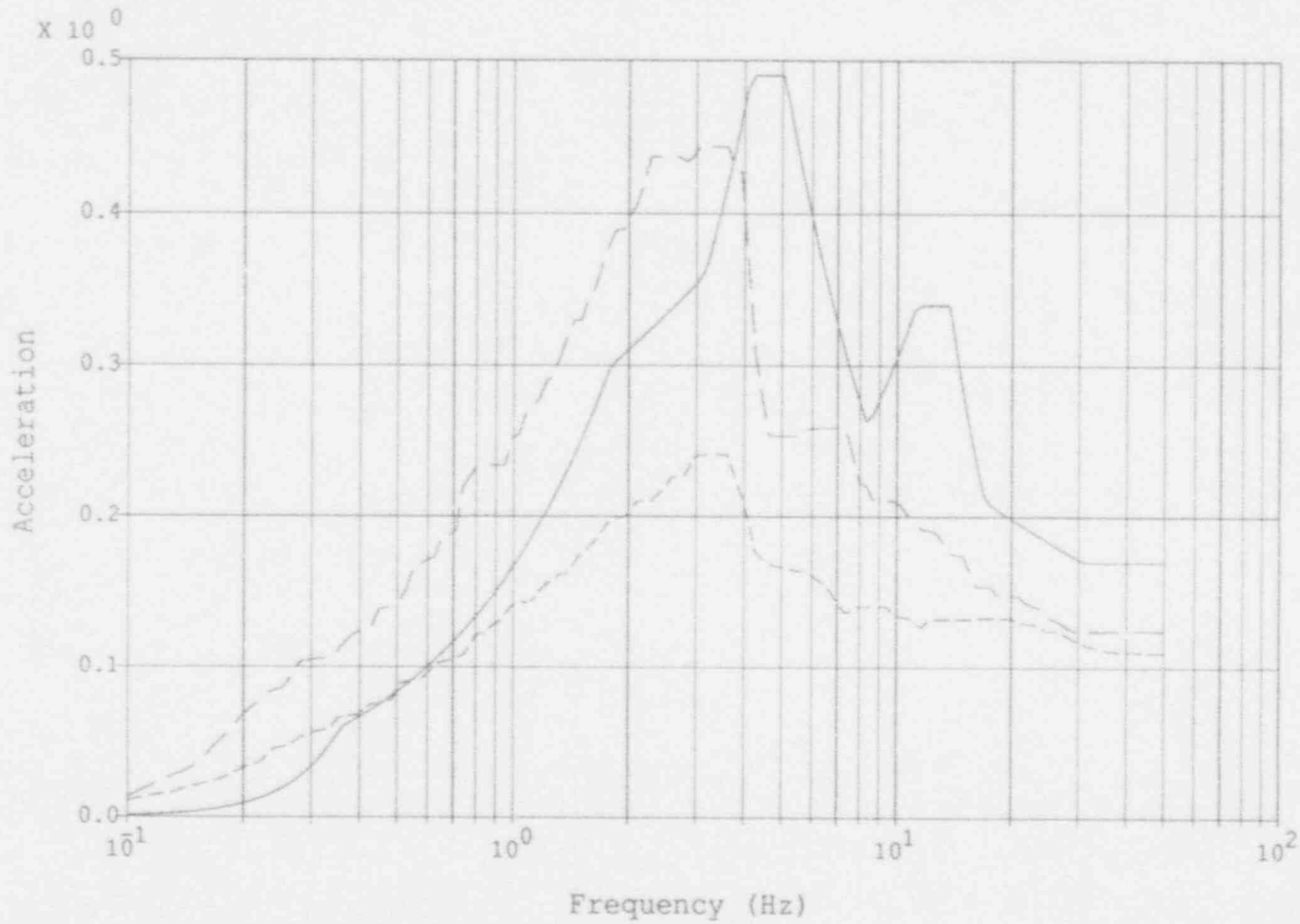
- R. G. 1.60 based floor spectra: These spectra were developed by EQE Engineering using a 0.15g SSE time history to closely fit R. G. 1.60, and meeting current standard Review Plan criteria.

- Housner based floor spectra: These spectra were developed by EQE Engineering using a 0.15g SSE time history to closely fit the Housner requirement shown in FSAR Figure 2.5-6. These are for information only to demonstrate the conservatism introduced by approaches and techniques used in the late 1960's which include the use of the Taft time history to meet the Housner requirement.

nas/GL8702

SPECTRA COMPARISON

C-114 @ 5% DAMPING
R.G. 1.60 @ 5% DAMPING
EQE HOUSNER @ 5% DAMPING



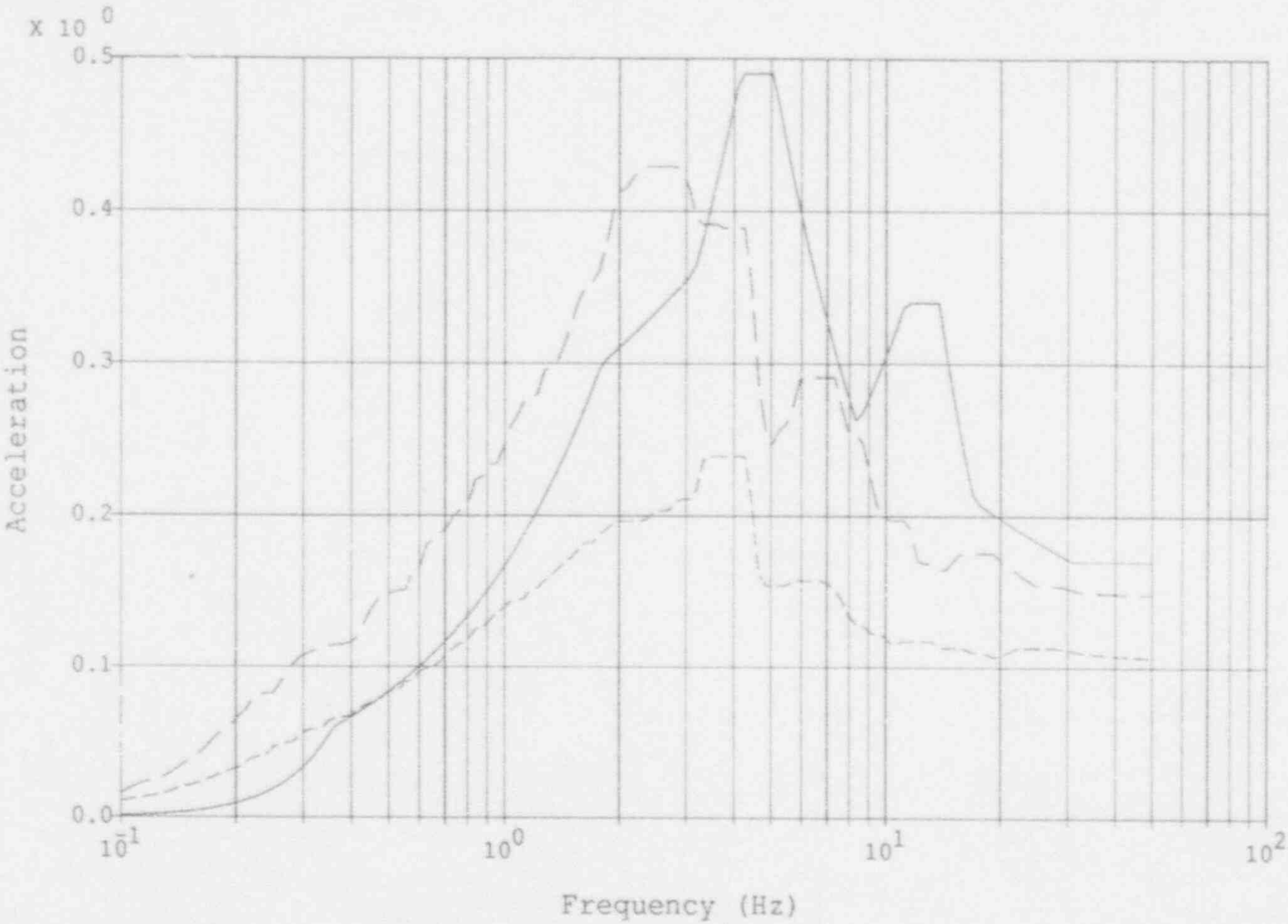
Legend:

BECO C114 Horizontal Design Spectrum	________
E` RG 1.60 In- tructure Spectrum	_ _ _ _ _
EC. Housner In- Structure Spectrum	---------

Notes:

5% Spectral Damping
North-South Direction
Accelerations in g's
1 SSE Level = 0.15g
Five Locations Enveloped

BECO: Pilgrim Reactor Building, Basemat El.-17.5'
Comparison of RG 1.60 and Housner Spectra vs. C114 Design Spectrum



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Legend:

BECO C114 Horizontal Design Spectrum ___________

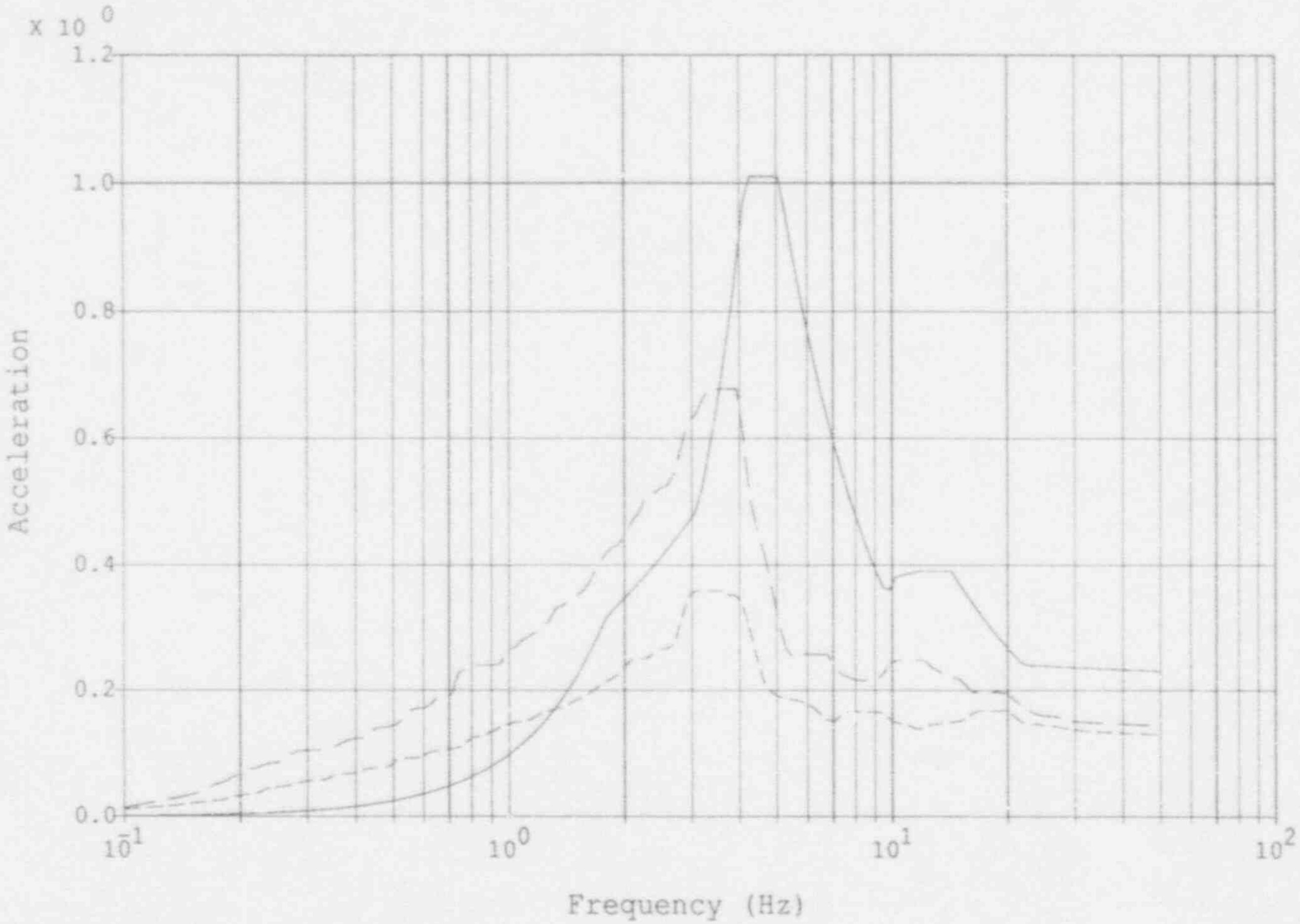
EQE RG 1.60 In-Structure Spectrum _ _ _ _ _ _

EQE Housner In-Structure Spectrum _ _ _ _ _ _ _ _ _ _

Notes:

5% Spectral Damping
East-West Direction
Accelerations in g's
1 SSE Level = 0.15g
Five Locations Enveloped

BECO: Pilgrim Reactor Building, Basemat El.-17.5'
Comparison of RG 1.60 and Housner Spectra vs. C114 Design Spectrum



Legend:

BECO C114 Horizontal Design Spectrum ————

EQE RG 1.60 In-Structure Spectrum — — — —

EQE Housner In-Structure Spectrum ---------

Notes:

5% Spectral Damping

North-South Direction

Accelerations in g's

1 SSE Level = 0.15g

Five Locations Enveloped

BECO: Pilgrim Reactor Building, Elevation 23.0'
Comparison of RG 1.60 and Housner Spectra vs. C114 Design Spectrum

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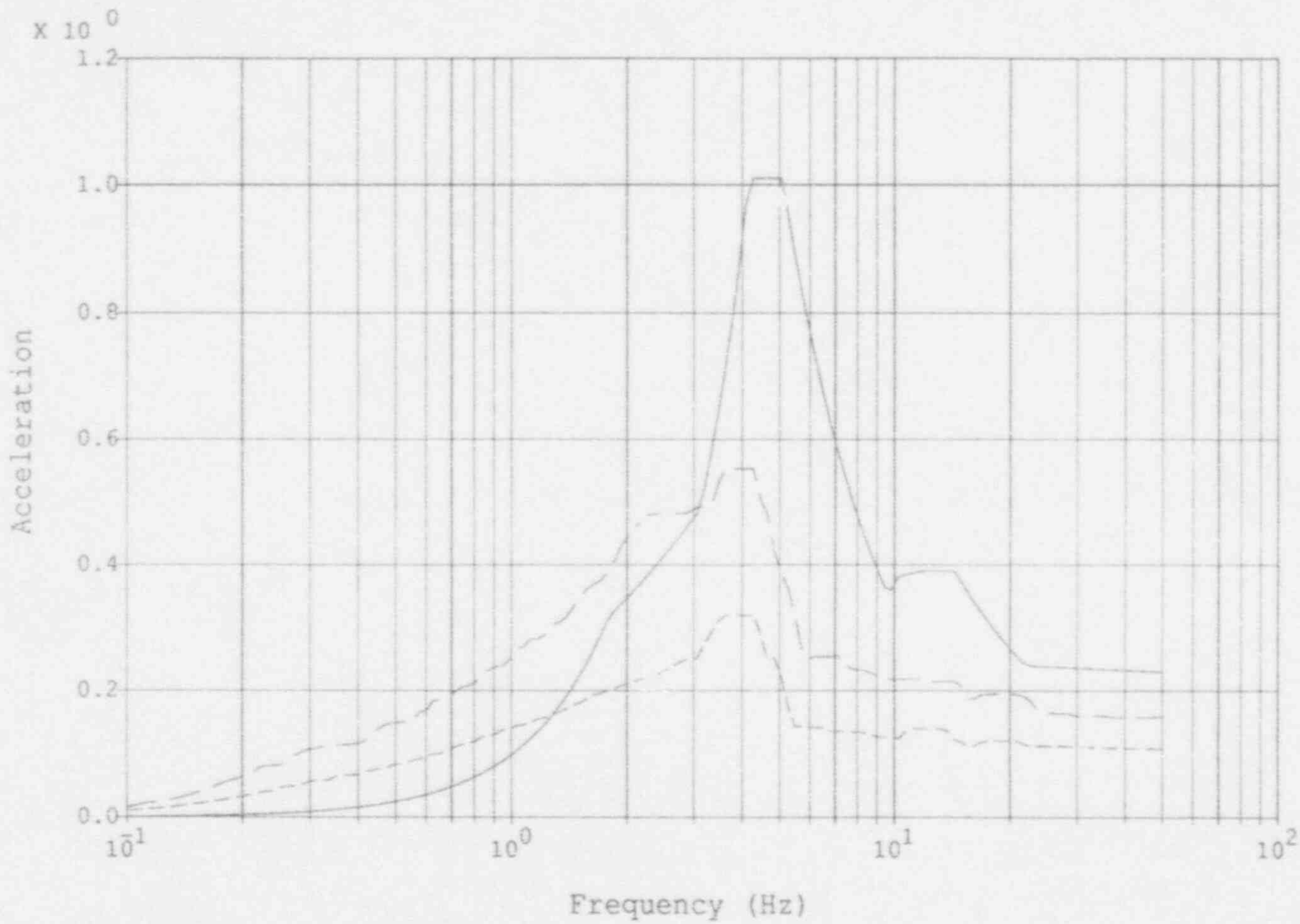


Legend:

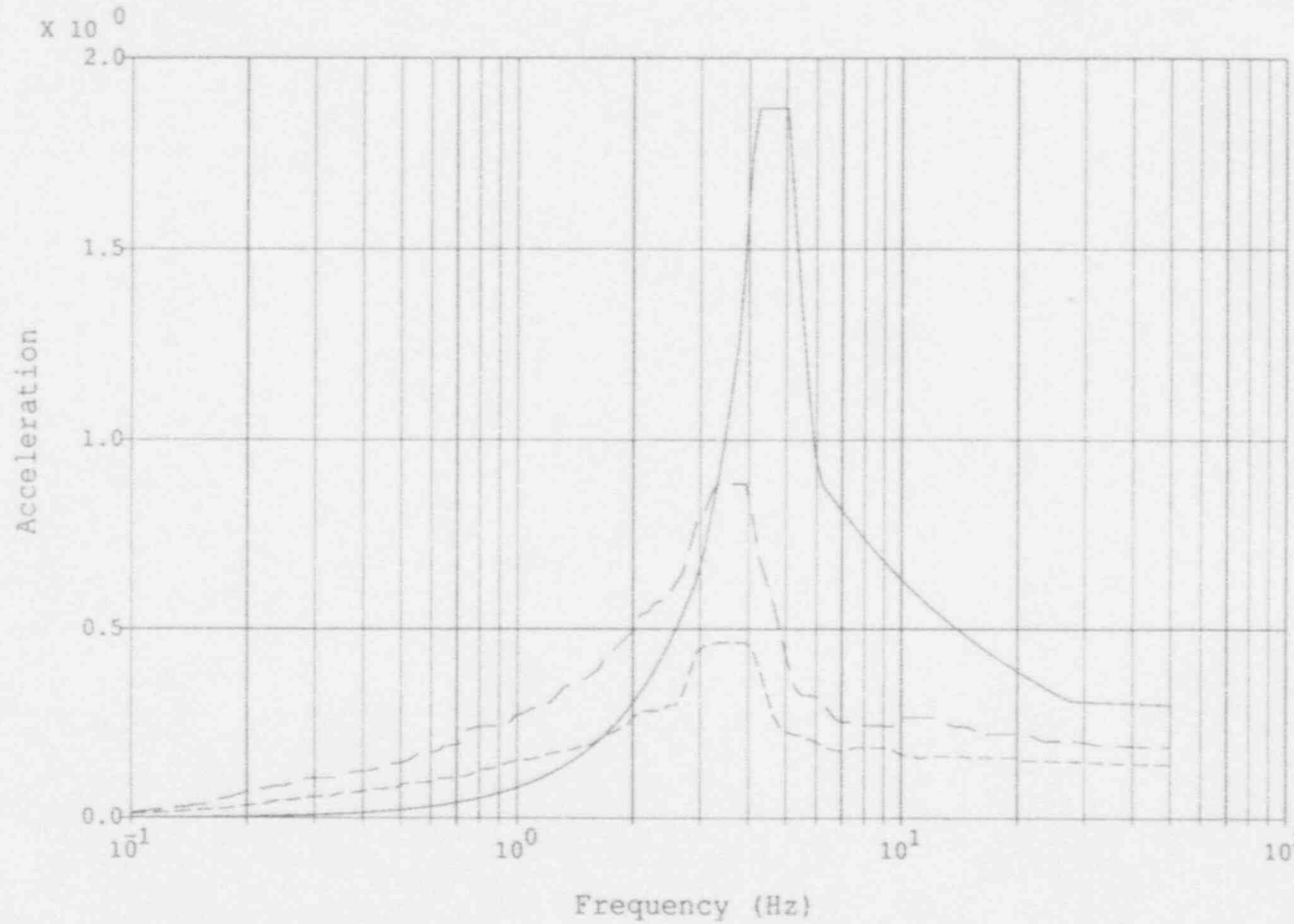
BECO C114 Horizontal Design Spectrum	――――――――
EQE RG 1.60 In-Structure Spectrum	— — — — —
EQE Housner In-Structure Spectrum	- - - - - - - - -

Notes:

5% Spectral Damping
East-West Direction
Accelerations in g's
1 SSE Level = 0.15g
Five Locations Enveloped

BECO: Pilgrim Reactor Building, Elevation 23.0'
Comparison of RG 1.60 and Housner Spectra vs. C114 Design Spectrum

RSPLT SUN V1.2 rb2y.plt 10:25:14 07/15/93



Legend:

BECO C114 Horizontal Design Spectrum ————————

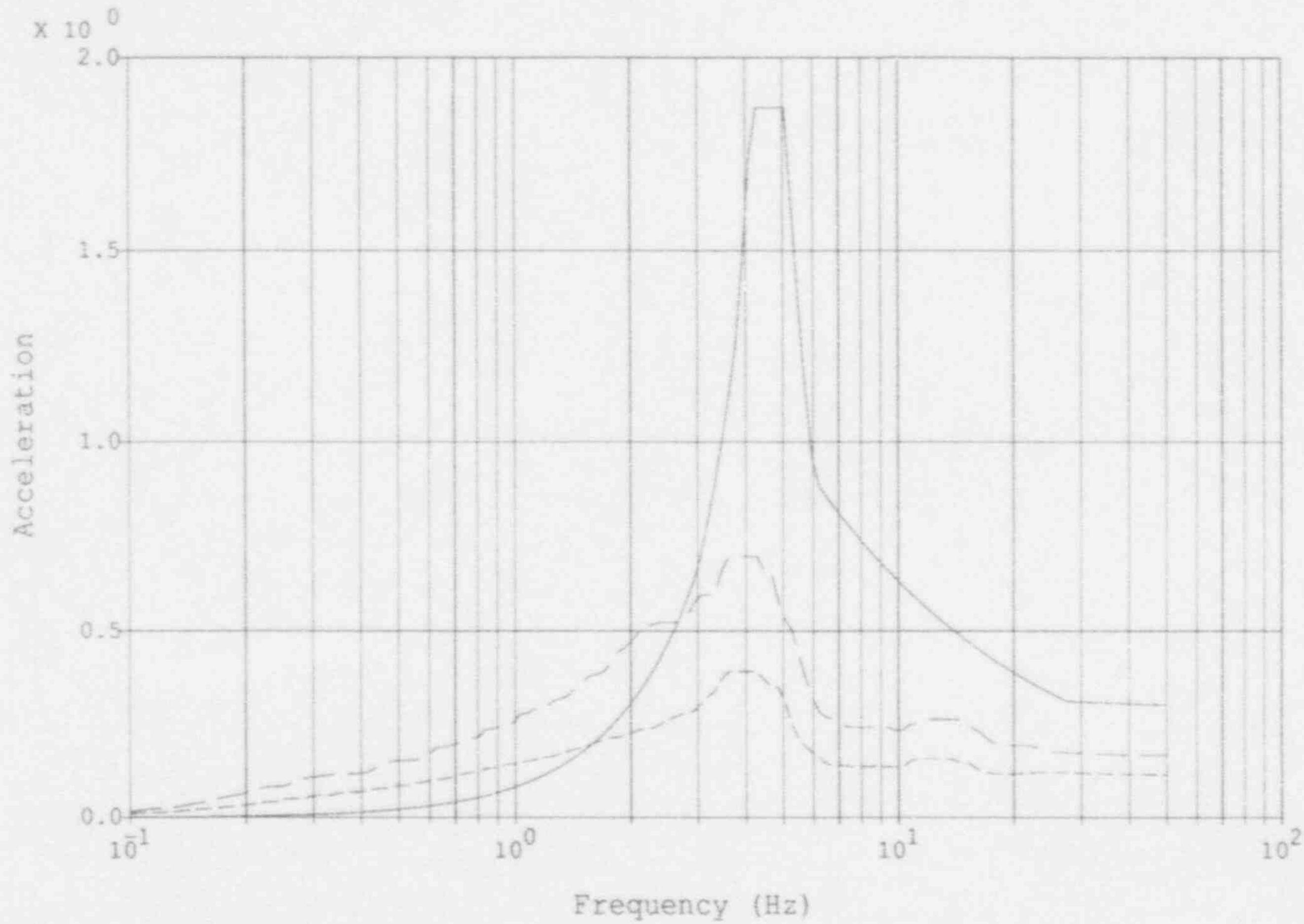
EQE RG 1.60 In-Structure Spectrum — — — — —

EQE Housner In-Structure Spectrum -----------

Notes:

5% Spectral Damping
North-South Direction
Accelerations in g's
1 SSE Level = 0.15g
Five Locations Enveloped

BECO: Pilgrim Reactor Building, Elevation 51.0'
Comparison of RG 1.60 and Housner Spectra vs. C114 Design Spectrum



Legend:

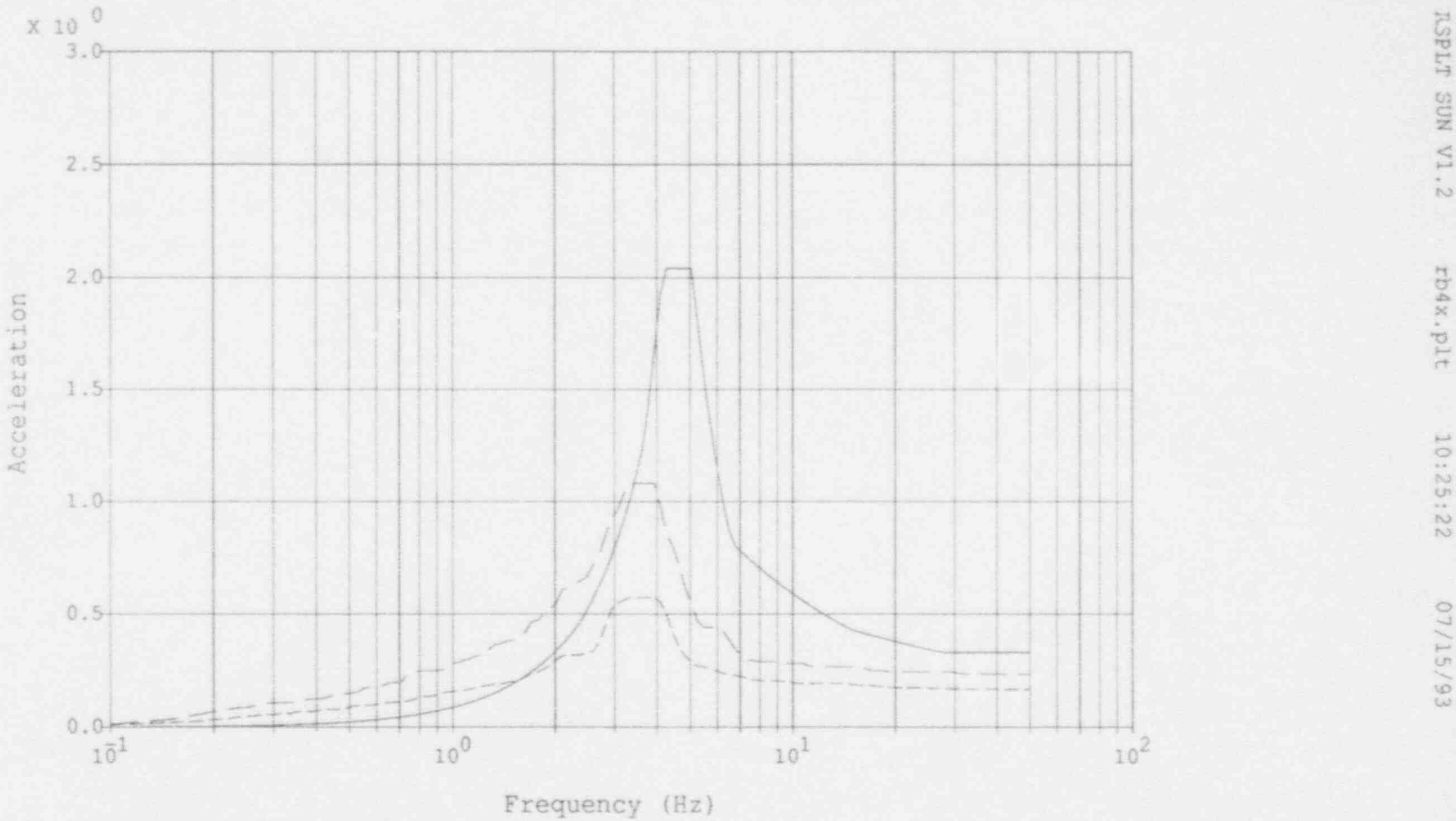
BECO C114 Horizontal Design Spectrum	___________
EQE RG 1.60 In-Structure Spectrum	__ __ __ __ _
EQE Housner In-Structure Spectrum	_________

Notes:

5% Spectral Damping
East-West Direction
Accelerations in g's
1 SSE Level = 0.15g
Five Locations Enveloped

BECO: Pilgrim Reactor Building, Elevation 51.0'
Comparison of RG 1.60 and Housner Spectra vs. C114 Design Spectrum

RSPLT SUN V1.2 rb3y.plt 10:25:17 07/15/93



Legend:

BECO C114 Horizontal Design Spectrum ————————

EQE RG 1.60 In-Structure Spectrum — — — — —

EQE Housner In-Structure Spectrum ----------

Notes:

5% Spectral Damping

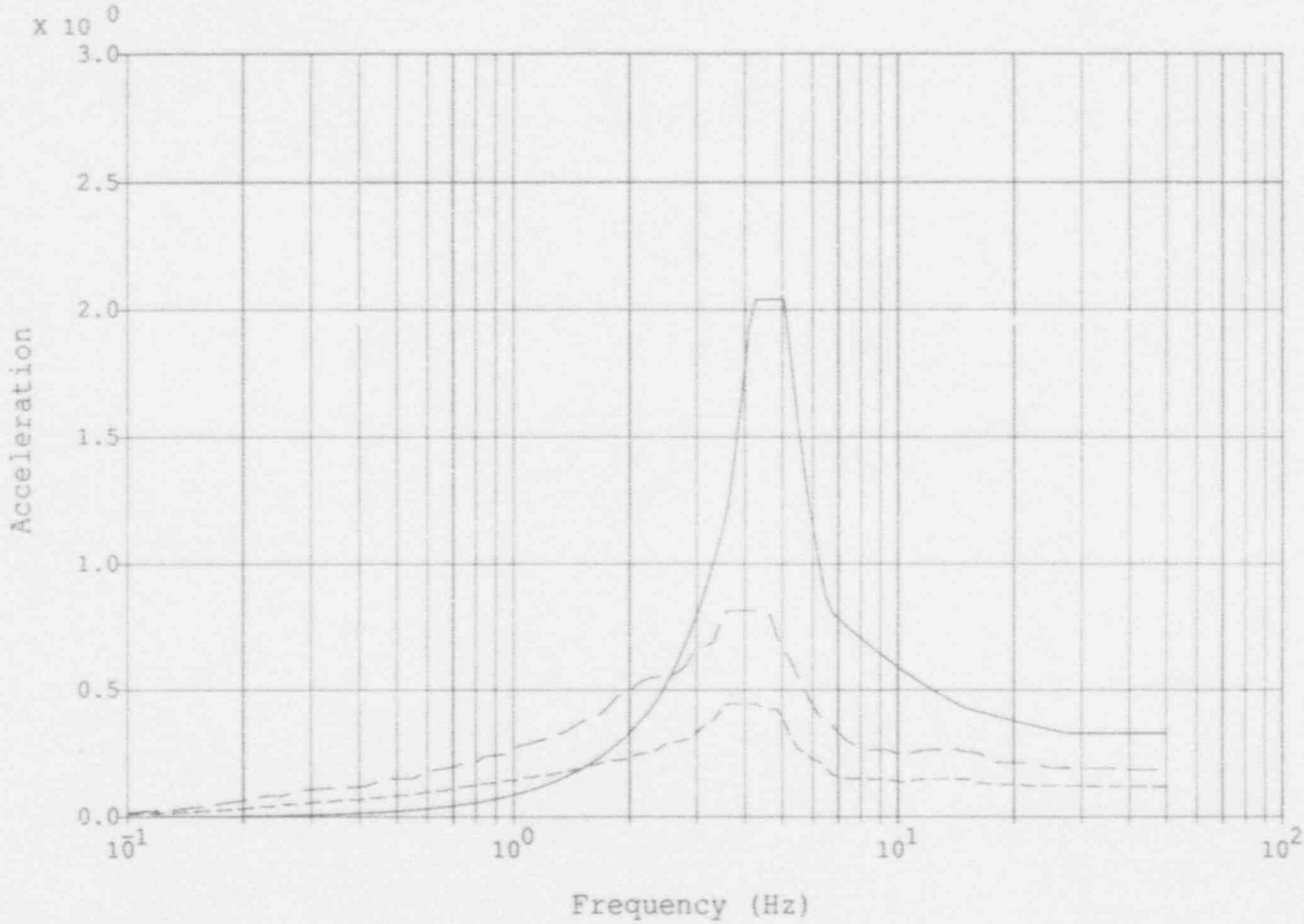
North-South Direction

Accelerations in g's

1 SSE Level = 0.15g

Five Locations Enveloped

BECO: Pi..grim Reactor Building, Elevation 74.25'
Comparison of RG 1.60 and Housner Spectra vs. C114 Design Spectrum



RSPLT SUN V1.2 rb4y.plt 10:25:22 07/15/93

Legend:

BECO C114 Horizontal Design Spectrum

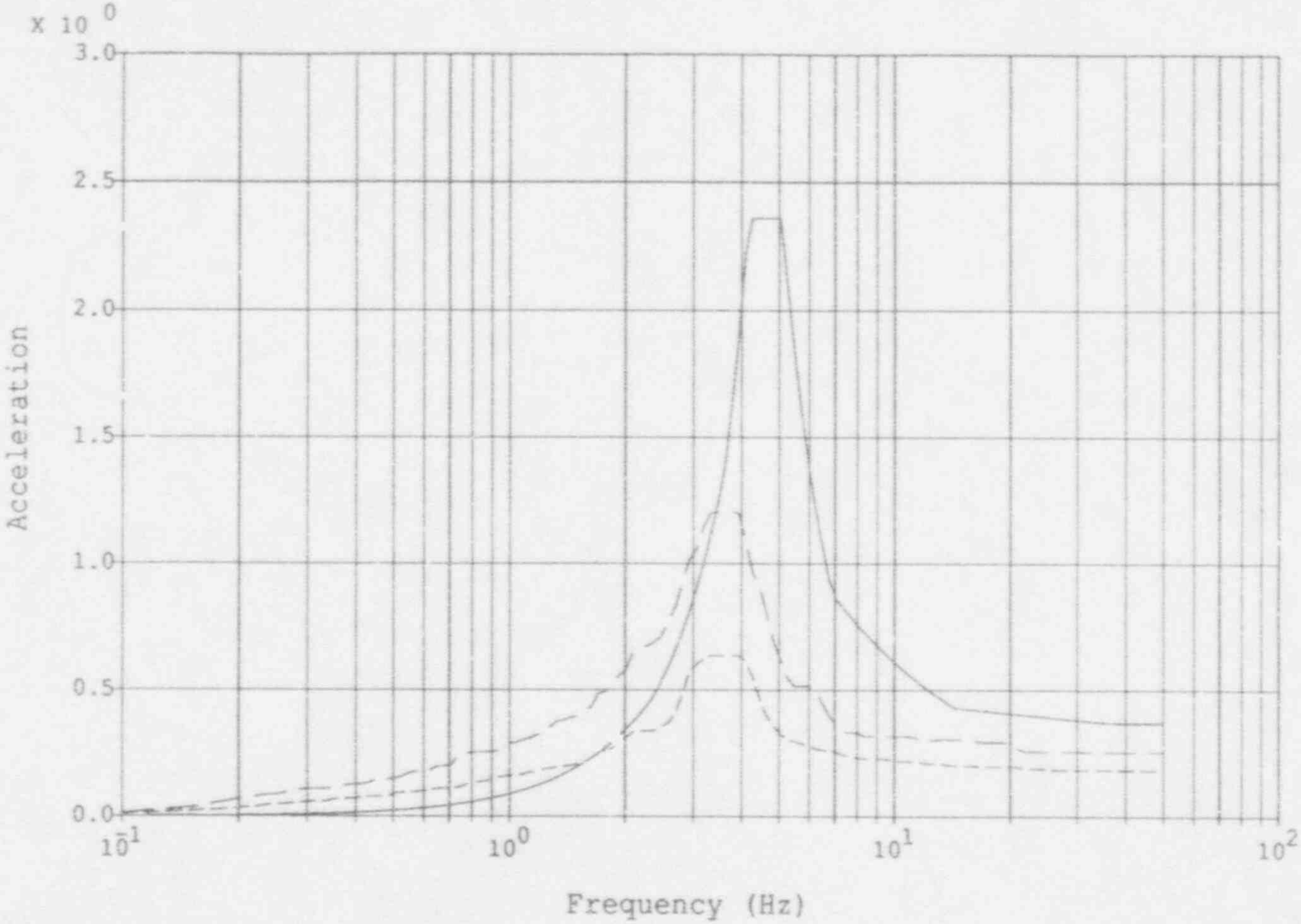
EQE RG 1.60 In-Structure Spectrum

EQE Housner In-Structure Spectrum

Notes:

5% Spectral Damping
East-West Direction
Accelerations in g's
1 SSE Level = 0.15g
Five Locations Enveloped

BECO: Pilgrim Reactor Building, Elevation 74.25'
Comparison of RG 1.60 and Housner Spectra vs. C114 Design Spectrum



RSPLT SUN V1.2 rb6x.plt 10:25:26 07/15/93

Legend:

BECO C114 Horizontal Design Spectrum

EQE RG 1.60 In-Structure Spectrum

EQE Housner In-Structure Spectrum

Notes:

5% Spectral Damping
North-South Direction
Accelerations in g's
1 SSE Level = 0.15g
Five Locations Enveloped

BECO: Pilgrim Reactor Building, Elevation 91.25'
Comparison of RG 1.60 and Housner Spectra vs. C114 Design Spectrum