REPORT NO. 553.13

APPENDIX A

CONCRETE BLOCK AND PRISM TESTING

POINT BEACH NUCLEAR PLANT

STS Job 11297

SOIL TESTING SERVICES

GEOTECHNICAL AND MATERIALS ENGINEERS

8110130455 64pp.

SOIL TESTING SERVICES
OF WISCONSIN, INC.
540 LAMBEAU ST. GREEN BAY, WIS. 54303

September 15, 1981

Computech Engineering Services, Inc.
2855 Telegraph Avenue
Berkeley, California 94705

Attention: Mr. Ron Mayes

STS Job 11297

RE: I.E. Bulletin 80-11, Masonry Wall Upgrade, Concrete Block and Prism Testing, Point Beach Nuclear Plant.

Gentlemen:

The tests on the concrete masonry block and prisms taken from walls at the Point Beach Nuclear Plant have been completed. Enclosed are two copies of the above referenced report. A copy of this report has also been forwarded to Mr. Dave Zabransky and Mr. T. R. Branam of Wisconsin Electric Power Company.

Tests were conducted on concrete masonry block and prisms in general accordance with procedures submitted to us by Mr. Mayes. Compressive strength tests were performed on three prisms and two block samples taken from each of three separate walls. The average compressive strength, f'm, of the nine prisms tested was determined to be 2270 psi, based on the net mortar bearing area. The average net compressive strength of the six single block samples tested was determined to be 3080 psi.

Included in this report are measurements taken of all test samples, compressive strength data, photographs of the prisms at failure, and calibration documentation pertaining to the compression test machine used.

If you should have any further comments regarding this report, please do not hesitate to contact us.

Yours very truly,

SOIL TESTING SERVICES OF WISCONSIN, INC.

Mark C. Spielbauer (JDM)
Mark C. Spielbauer
Assistant Project Engineer

Jack J. Amar (JDM)
Jack J. Amar, P. E.
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cc: Wisconsin Electric Power Company
231 West Michigan, P. O. Box 2046
Milwaukee, Wisconsin 53201
Attn: Dave K. Zabransky

Wisconsin Electric Power Company
Point Beach Nuclear Power Plant
Two Rivers, Wisconsin
Attn: T. R. Branam

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INTRODUCTION

Soil Testing Services of Wisconsin, Inc. (STS) was engaged by Wisconsin Electric Power Company to evaluate the compressive strength of concrete masonry prisms and block taken from walls at the Point Beach Nuclear Power Plant, Two Creeks, Wisconsin. The general scope of the work was described in our proposal to Wisconsin Electric Power Company dated July 23, 1981.

This report includes all work completed through September 11, 1981. The field sampling program was performed on August 24 through 26, 1981 under the direction of Mr. Branam and Mr. Zabransky of Wisconsin Electric Power Company and Mr. Mayes of Computech Engineering. Mr. Mayes directed the initial compression tests on August 26, 1981.

Test sample locations were located by Wisconsin Electric Power Company (WEPCO). Soil Testing Services' personnel extracted concrete masonry prisms and blocks, transported them to our laboratory, and tested them for compressive strength.

FIELD TESTING

We retained a subcontractor to cut test samples under our direction with a diamond saw using procedures previously described in our proposal. To facilitate easier cutting, larger units were removed from the wall. They were then trimmed to specified sizes using a smaller saw. Vibration was reduced by placement of wood wedges into the saw cut. This also lessened the possibility of cracking the mortar at the block mortar interface.

Masonry prisms were compressed in clamping devices as described in our proposal to reduce the potential of cracking the mortar joints. The units were then placed in containers for shipment to our laboratory.

LABORATORY TESTING

Measurements

The block and prism samples were measured in general accordance with ASTM E 447-74, "Compressive Strength of Masonry Prisms". At the direction of Mr. Mayes, measurements were taken at a distance of approximately 0.5 inches from the top and bottom bearing faces of the block samples. The average of these two measurements was used to determine the net block area. A diagram of each prism and block was made on which all measurements were plotted. Irregularities in the samples were also recorded. The mortar bedding of the prisms was measured to determine the net mortar area. Measurements of all prisms and block samples are included in the Appendix of this report.

Trimming of Samples

Before the samples were capped and tested for compressive strength, all mortar was removed from the top and bottom bearing surfaces. This entailed hand chipping both faces to remove any mortar which may have interfered with the capping or compressive strength testing. Also, any mortar that was found in the interior of the hollow block cores was also removed.

While chipping the mortar from Prism No.68-3, the block mortar interface between the bottom and second block cracked.

Capping

After removal of mortar from the bearing surfaces, the prisms and blocks were capped per ASTM Specification C 140-75. Due to awkward handling conditions, the [illegible] of the caps were checked with a feeler gage to verify that the caps were plane within specified tolerances.

Compressive Strength Testing

The test specimens were placed on the lower bearing block with the centroid of the bearing surface vertically aligned with the center of thrust of the spherically seated bearing block. As the spherically seated block was brought to bear on a specimen, it was rotated so that uniform seating was obtained. The loading was continued at a constant rate until failure occurred in the unit. The maximum load indicated on the dial was recorded and photographs were taken of the failed unit. Maximum loads and compressive strengths are included in the Appendix of this report. Photographs of failures have also been included in the Appendix of this report.

The cracked block mortar interface of Prism No. 68-3 did not appear to affect the test results.

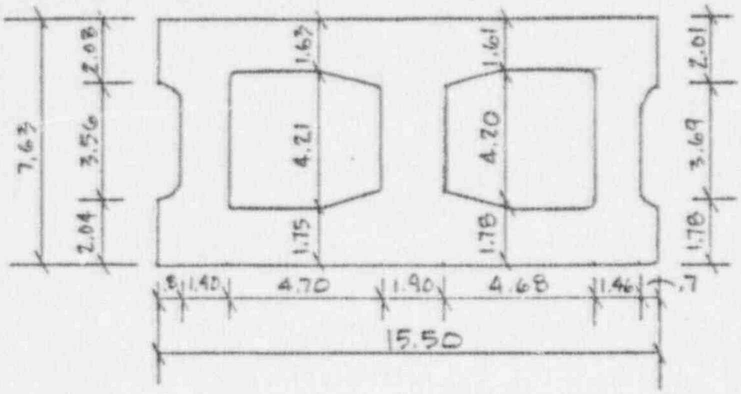
APPENDIX

WALL 68

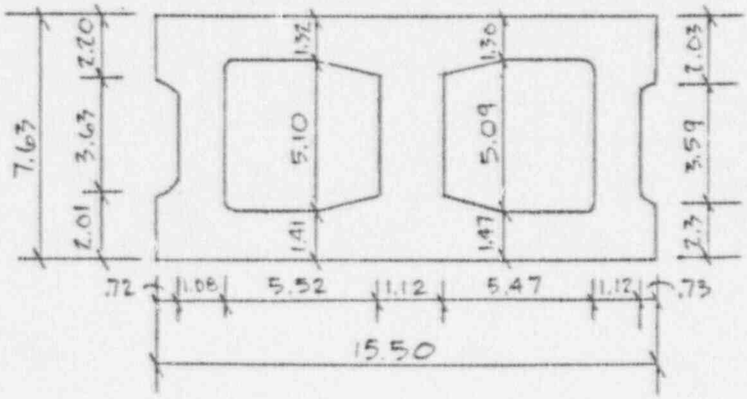
BLOCK 68-1

STS Job 11297
Point Beach Masonry

BOTTOM



TOP

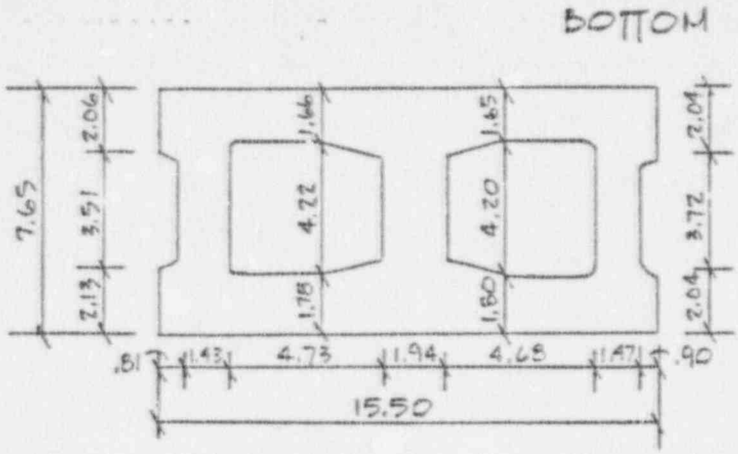


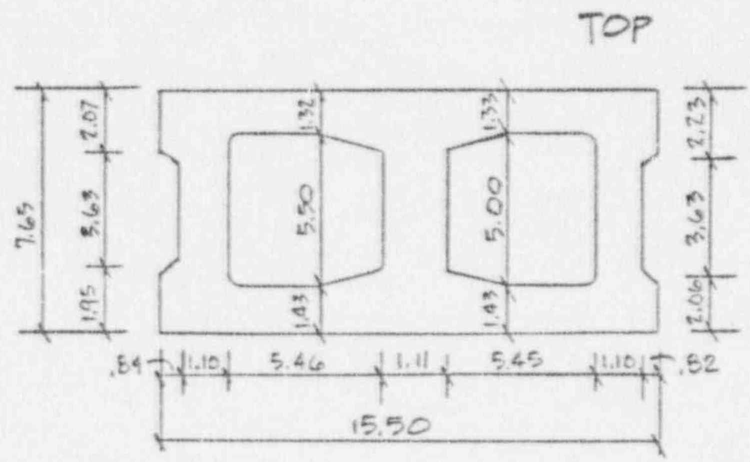
Gross Area-	118.27 in^2
Net Area-	65.52 in^2

Maximum Load -	178,000 lbs.
Gross Compressive Strength-	1510 psi
Net Compressive Strength -	2730 psi

BLOCK 6B-2

STS Job 11297
Point Beach Masonry

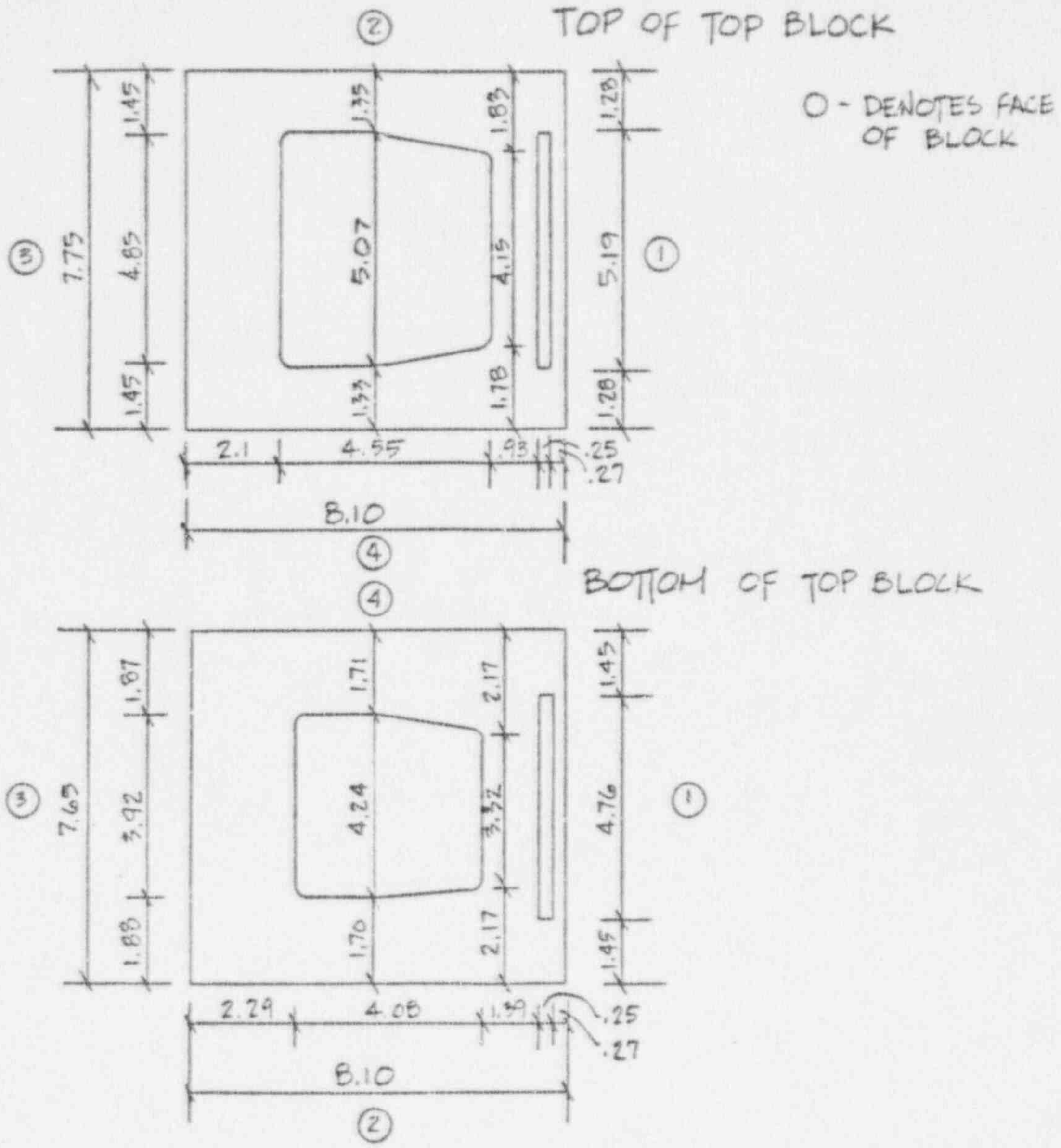


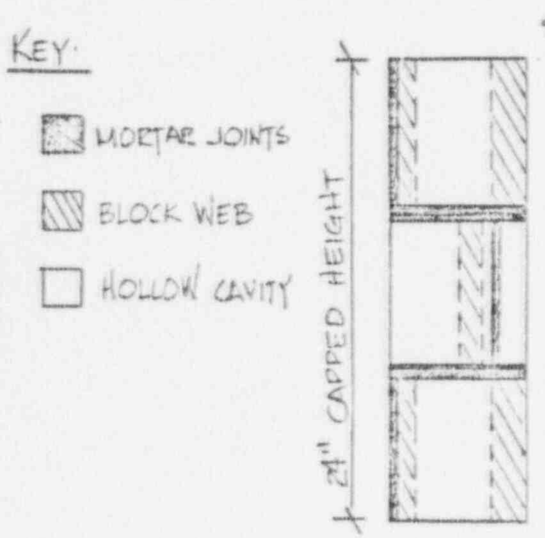


Gross Area	118.58 in^2
Net Area	64.54 in^2

Maximum Load -	146,000 lbs.
Gross Compressive -	1230 psi
Net Compressive Strength -	2260 psi

PRISM 6B-3



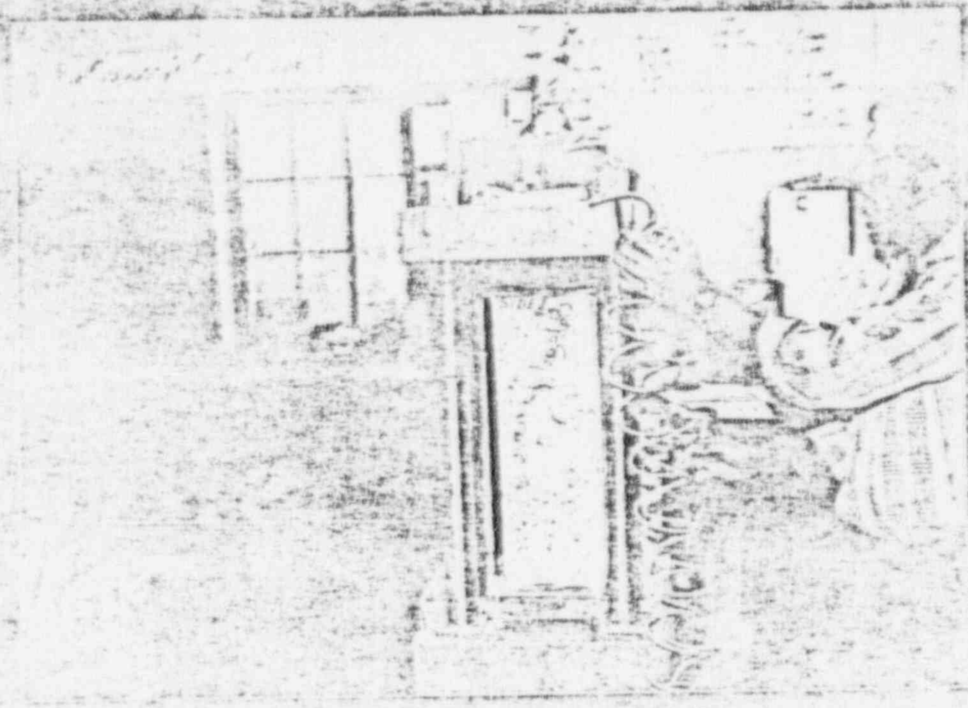


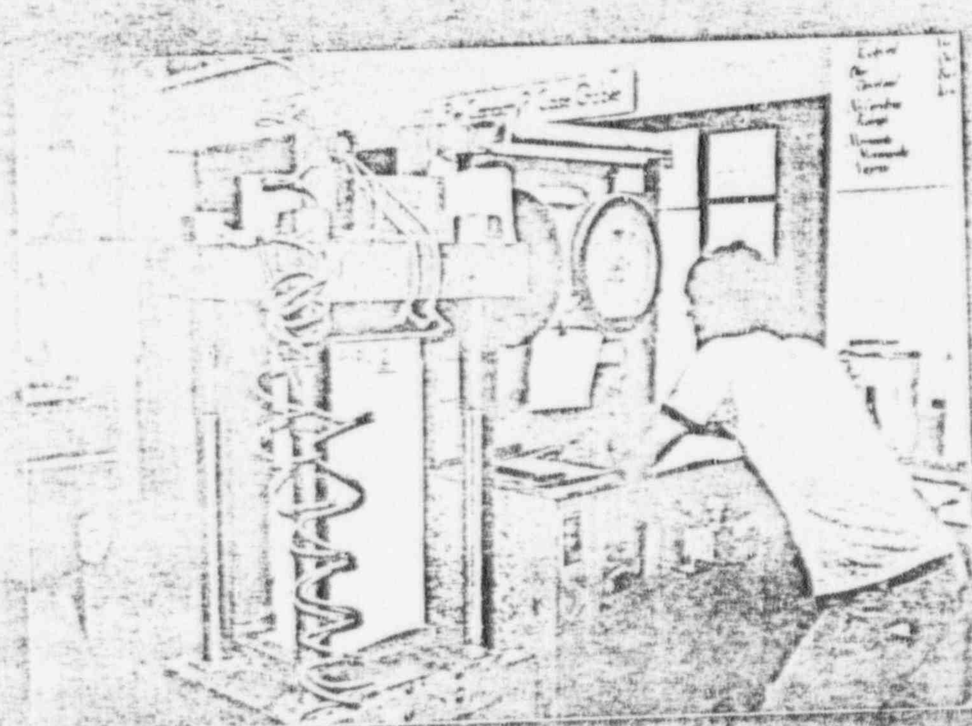
Gross Area	62.37 in^2
Net Area	41.13 in^2
Smallest Mortar Area	24.76 in^2
Maximum Load	51,500 lbs.
Gross Compressive Strength f'm	830 psi
Net Compressive Strength f'm	1250 psi
Compressive Strength f'm based on net mortar bearing area	2080 psi

POINT BEACH MASONRY

STS Job 11297

68-3

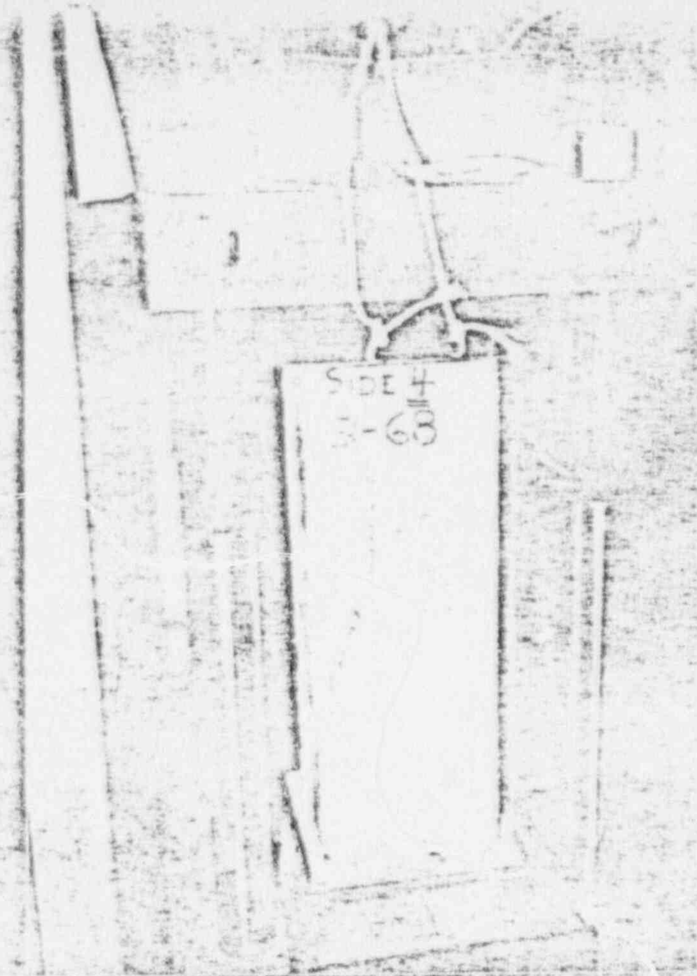




POINT BEACH MASONRY

STS Job 11297

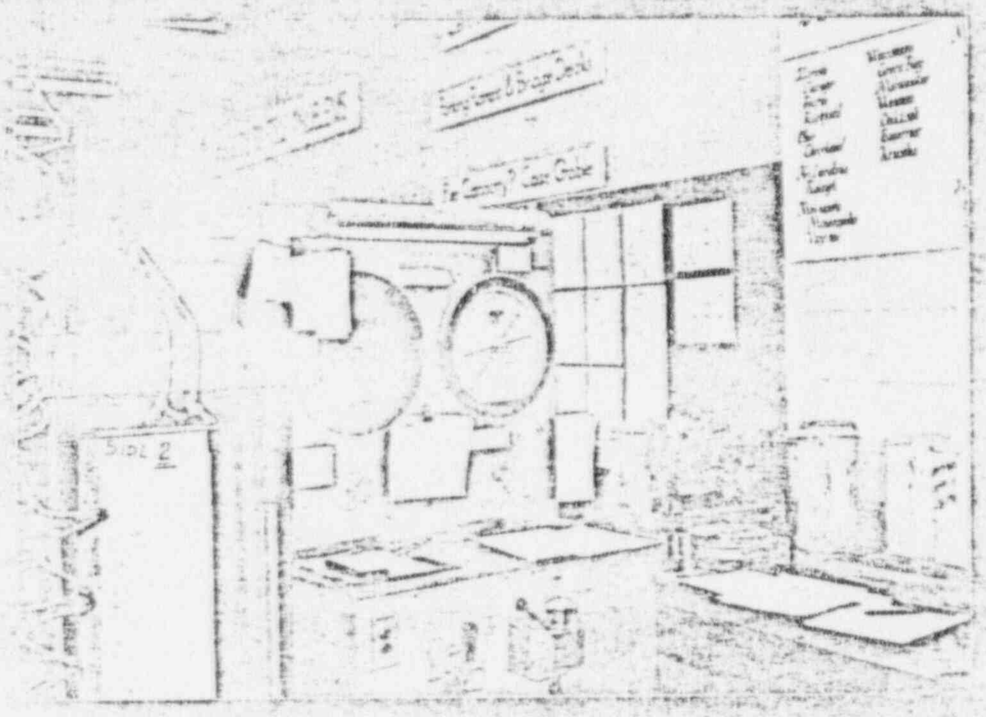
68-3



POINT BEACH MASONRY

STS Job 11297

68-3



POINT BEACH MASONRY

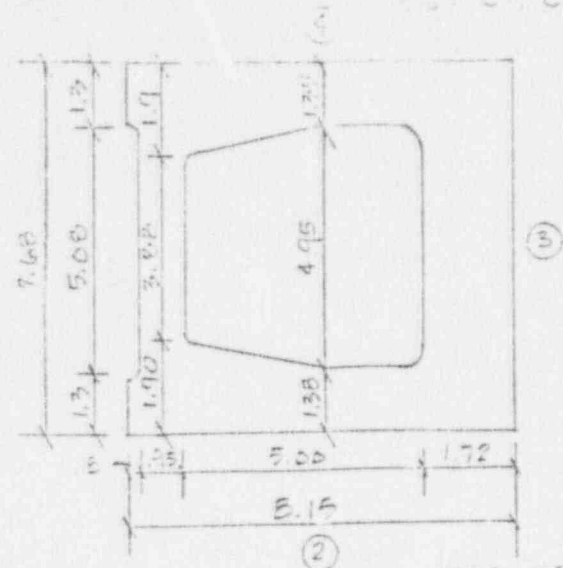
STS Job 11297

68-3

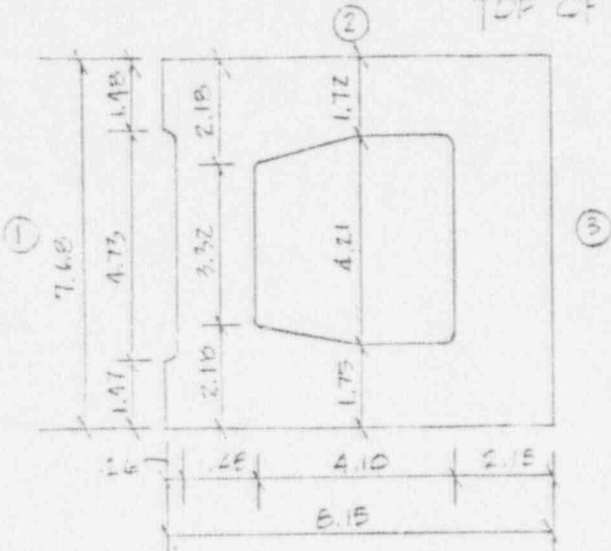
PRISM 18-31

BOTTOM OF TOP BLOCK

O - DENOTES FACE OF BLOCK



TOP OF TOP BLOCK



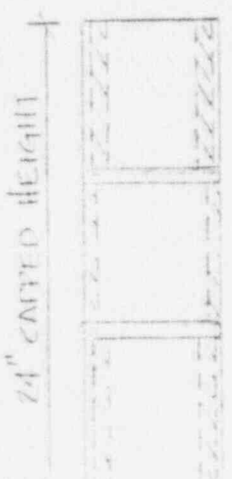
SEC 1-3

KEY

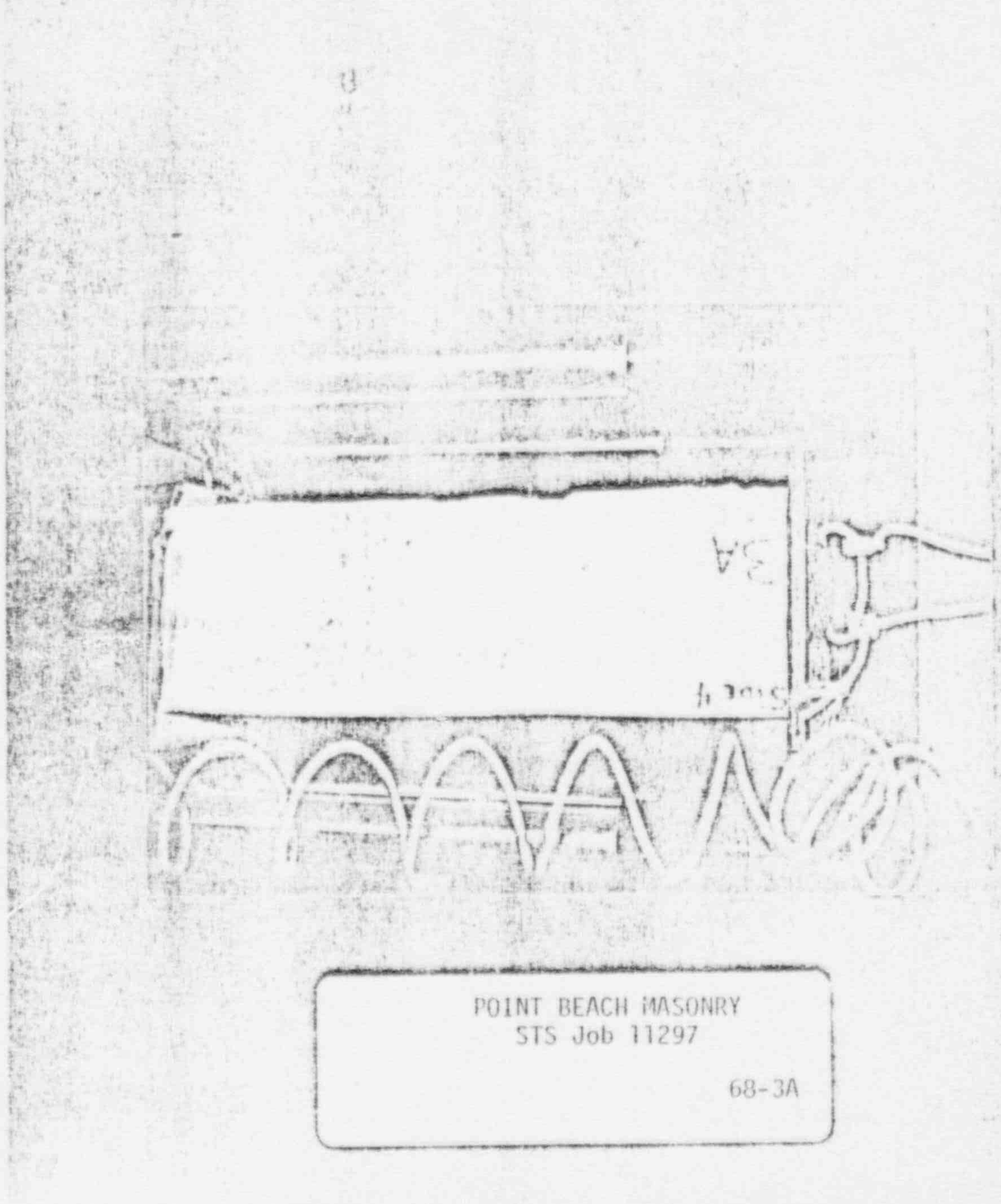
MORTAR JOINTS

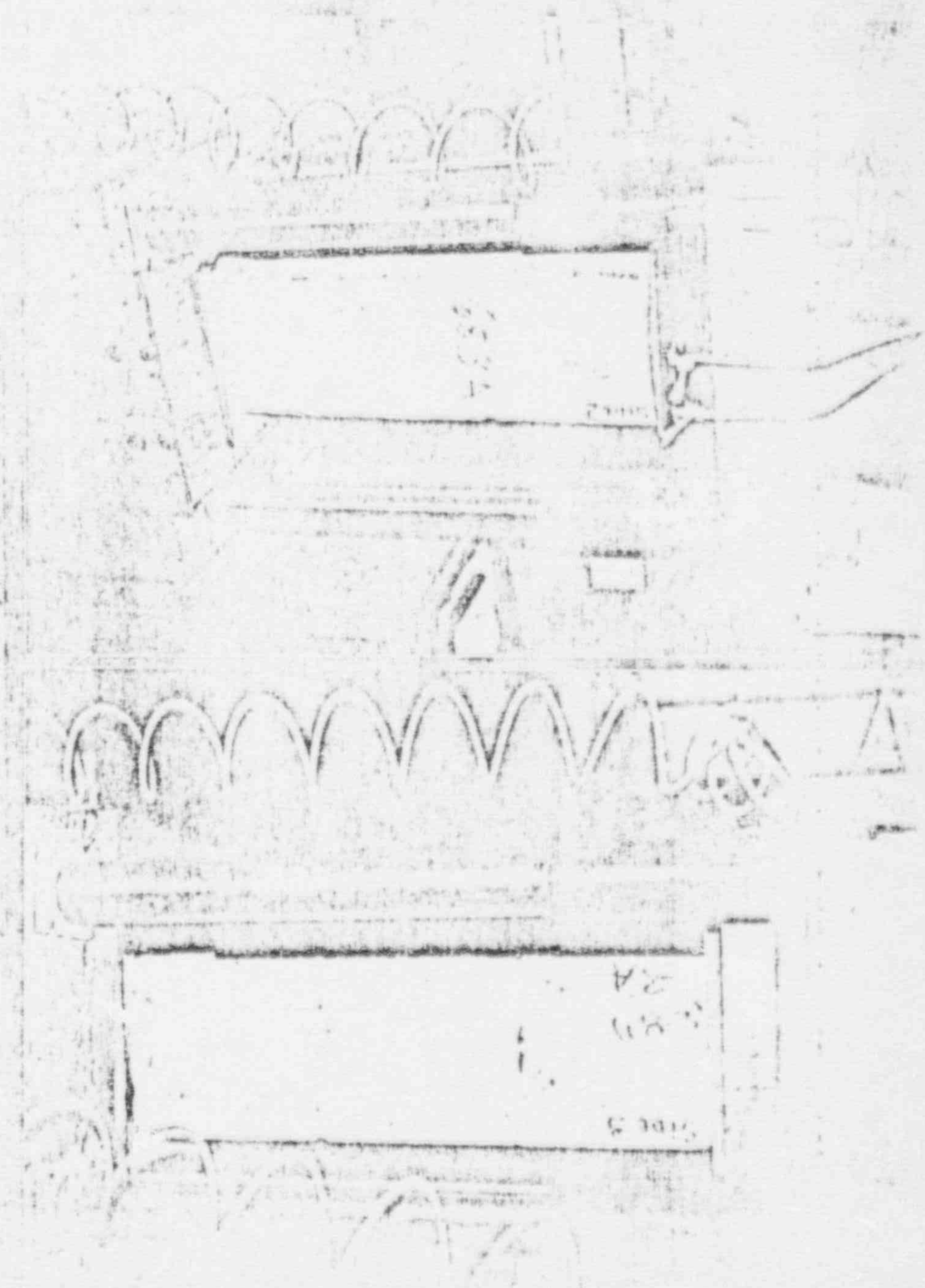
BLOCK WEB

HOLLOW CAVITY



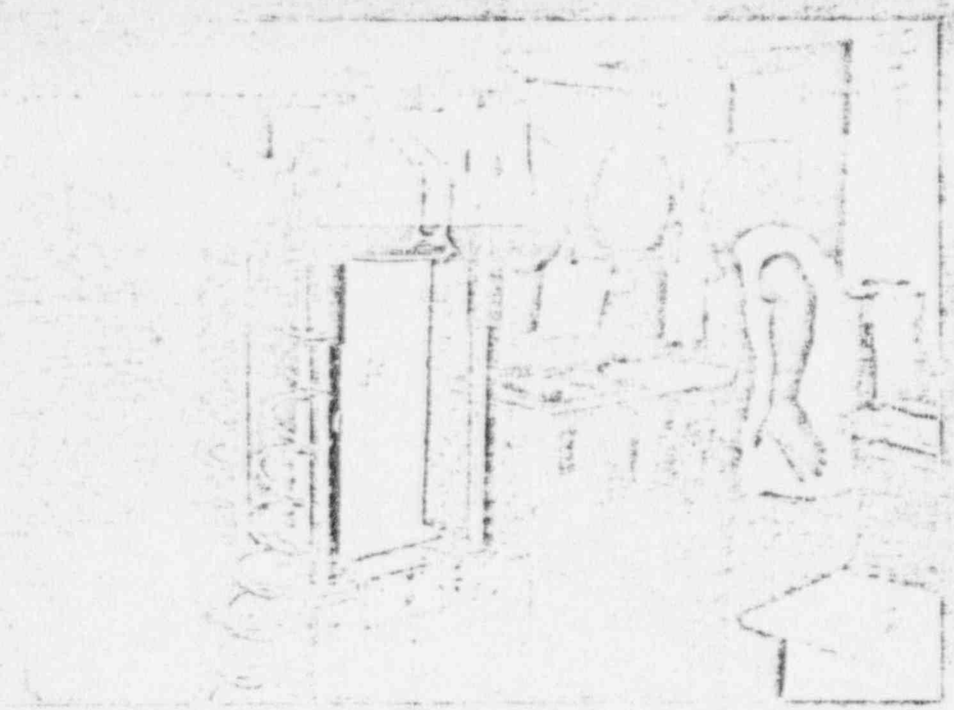
Gross Area	62.55 in^2
Net Area	60.63 in^2
Smallest Mortar Area	32.30 in^2
Maximum Load	53,000 lbs.
Gross Compressive Strength f'm	850 psi
Net Compressive Strength f'm	1300 psi
Compressive Strength f'm based on net mortar bearing area	1640 psi

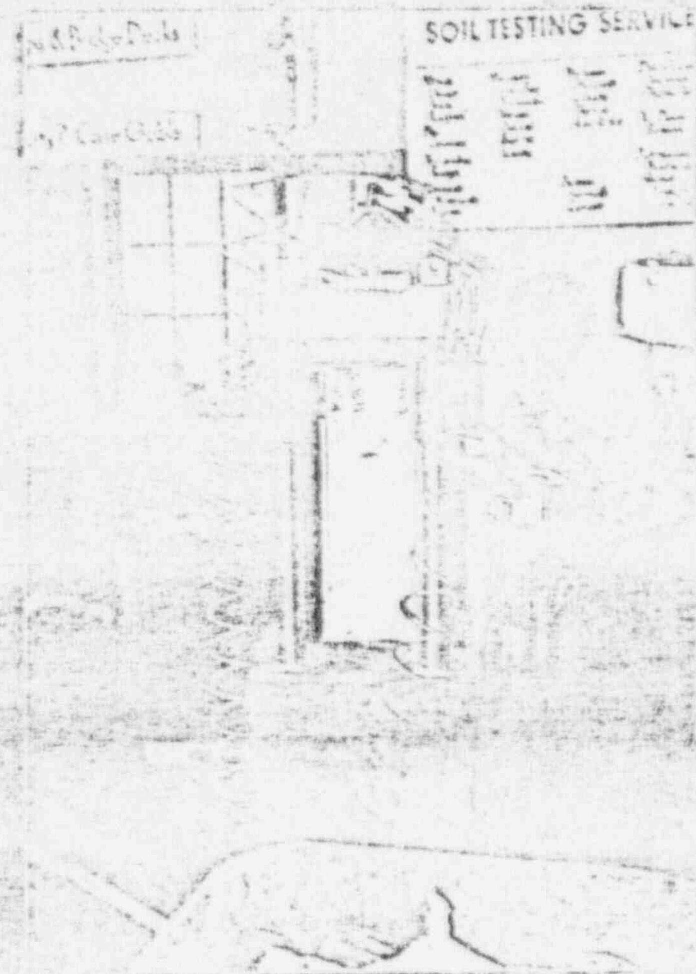




POINT BEACH MASONRY
STS Job 11297

68-3A





POINT BEACH MASONRY
STS Job 11297

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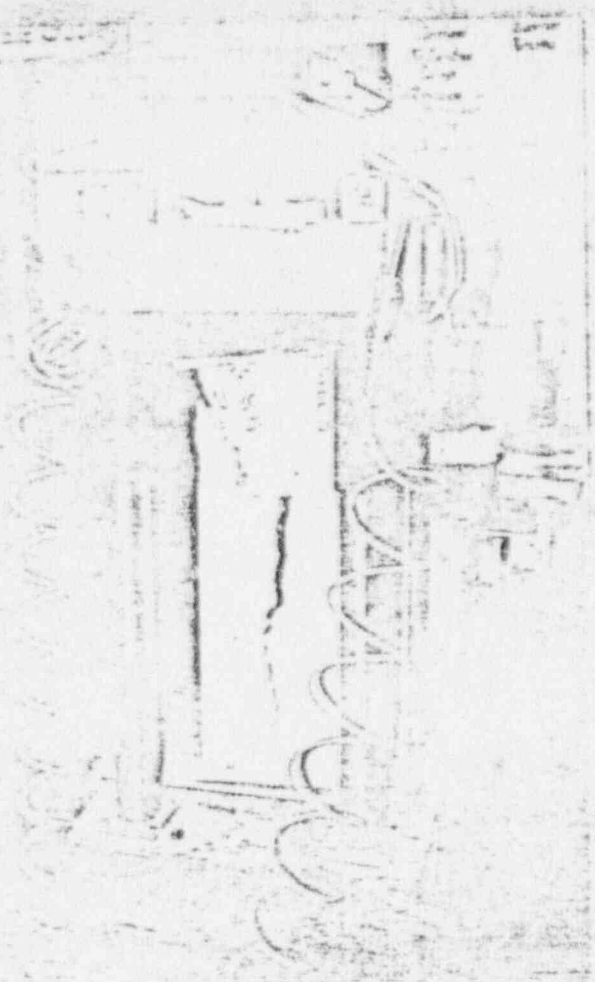
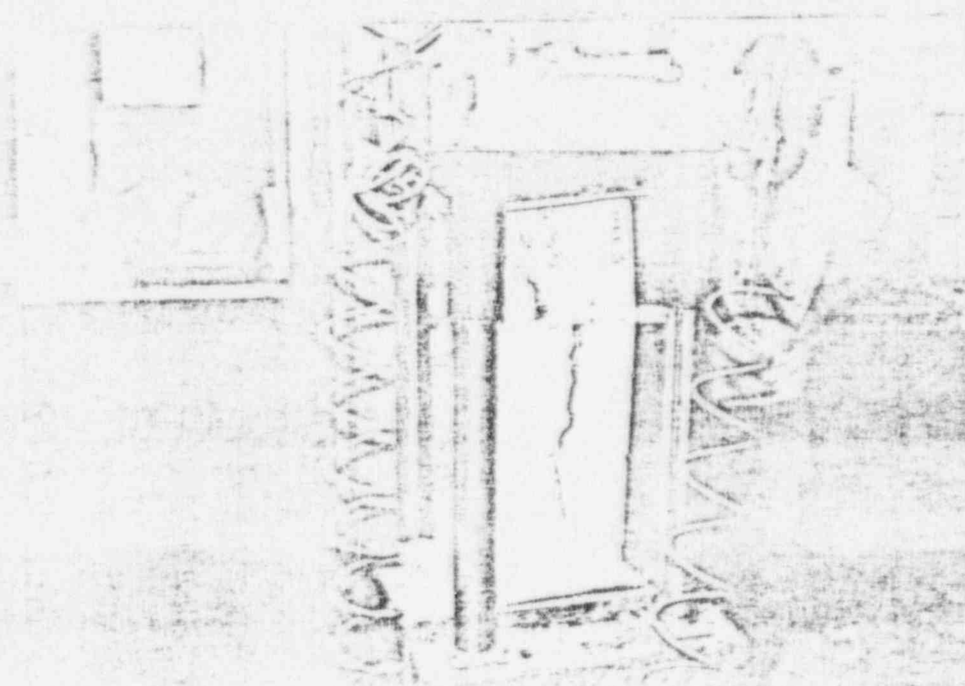
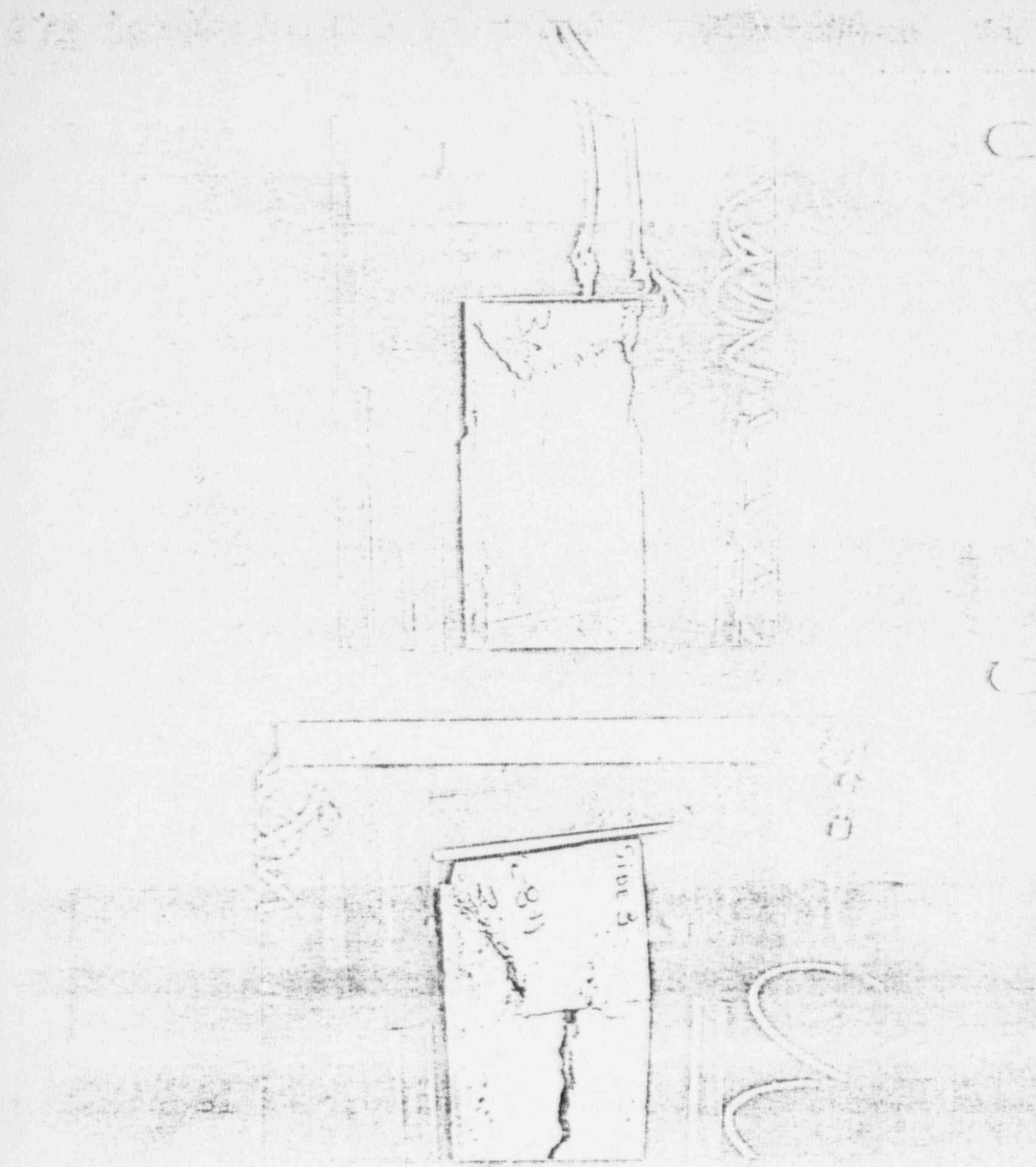




POINT BEACH MASONRY
STS Job 11297

6B-3A



POINT BEACH MASONRY
STS Job 11297

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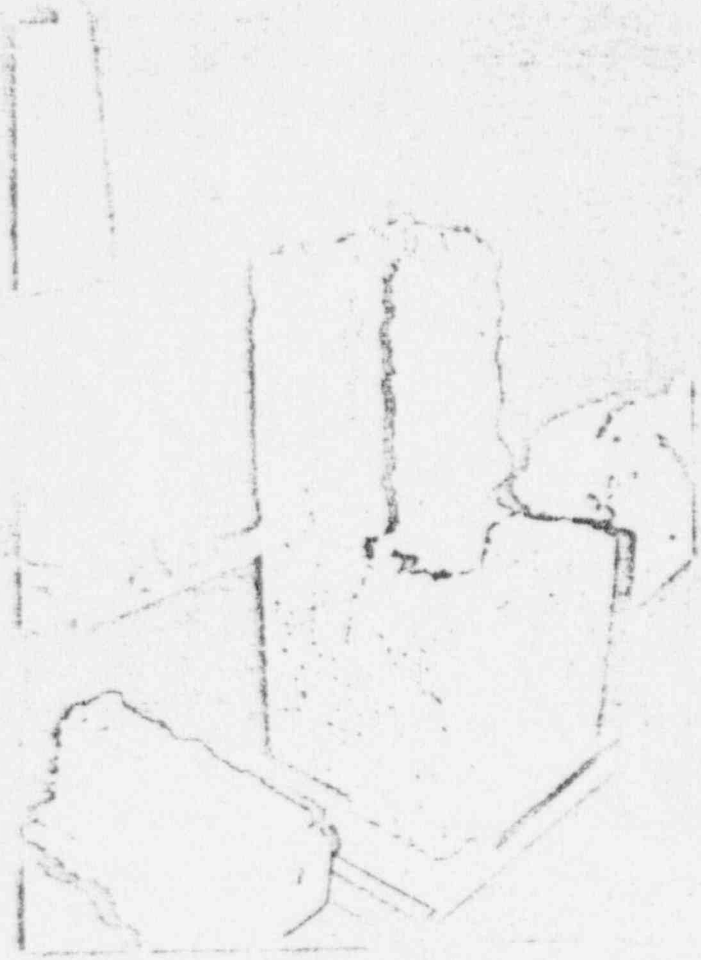

POINT BEACH MASONRY
STS Job 11297

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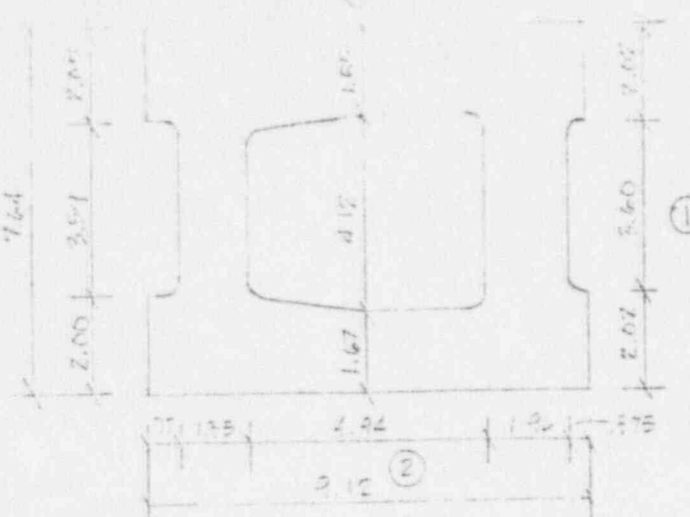
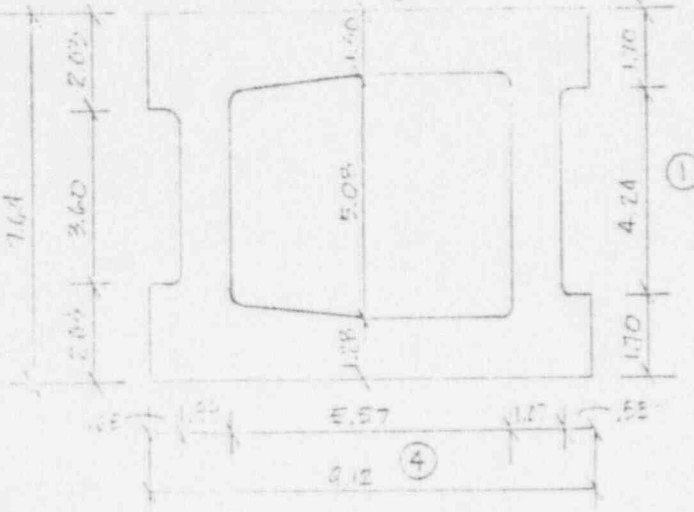
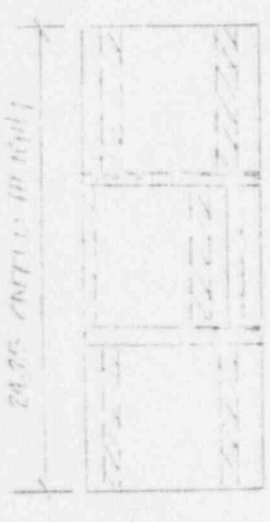


BOTTOM OF TOP BLOCK





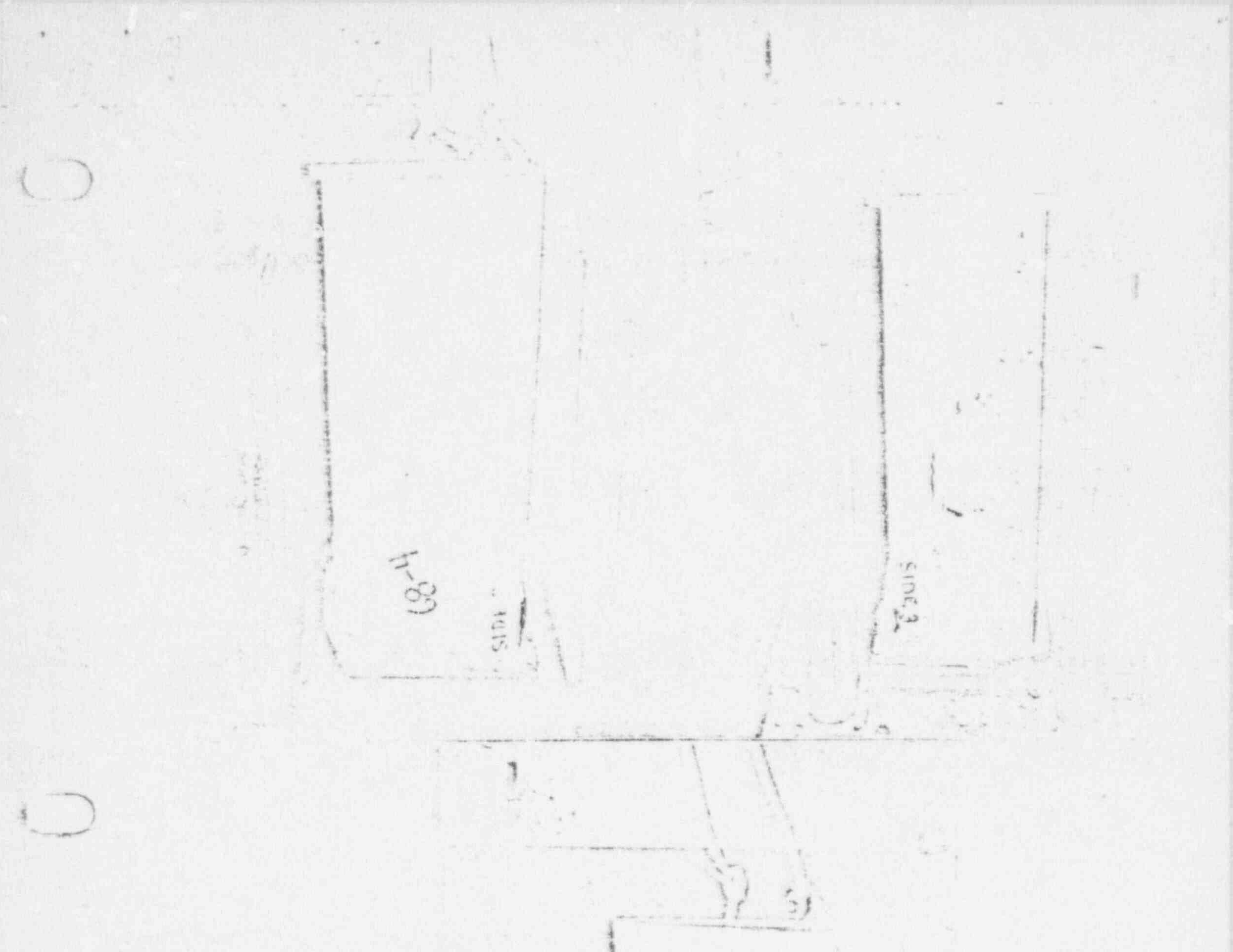
KEY

MORTAR JOINT

BLOCK WEB

HOLLOW CAVITY

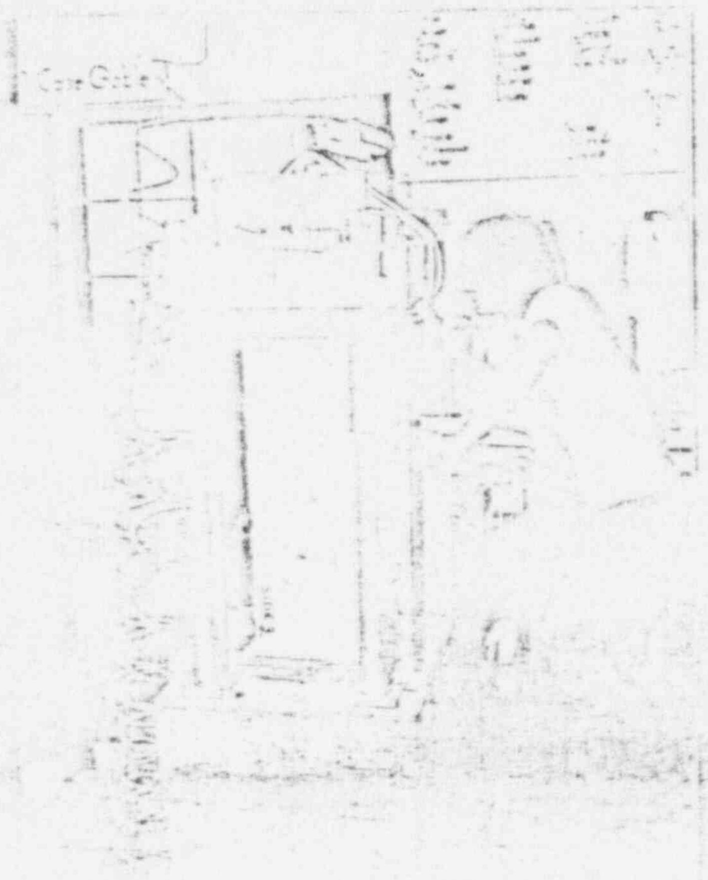
Gross Area	69.68 in^2
Net Area	41.02 in^2
Smallest Mortar Area	29.70 in^2
Maximum Load	66,500 lbs.
Gross Compressive Strength f'm	950 psi
Net Compressive Strength f'm	1620 psi
Compressive Strength f'm based on net mortar bearing area	2240 psi



POINT BEACH MASONRY

STS Job 11297

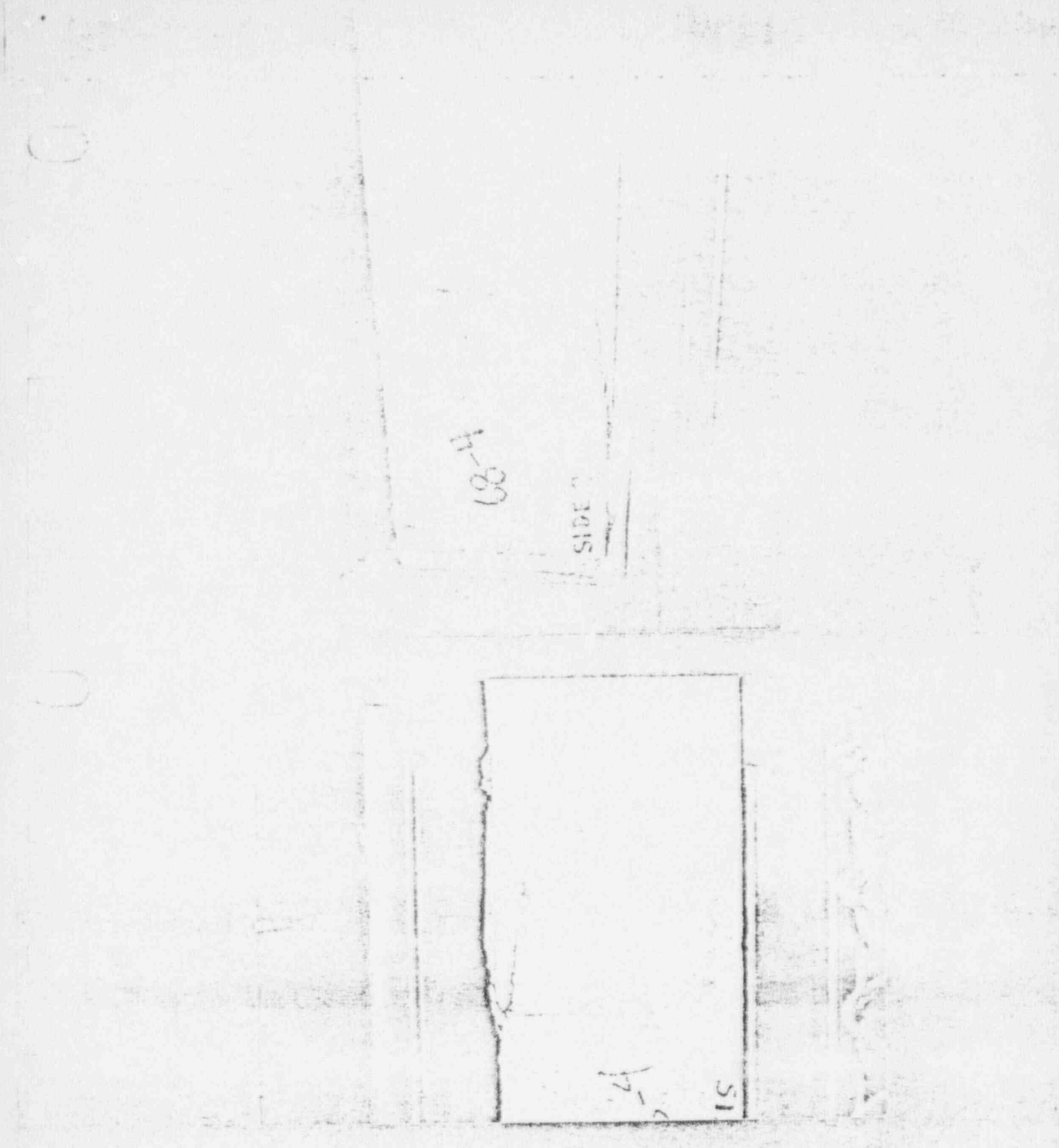
68-4



POINT BEACH MASONRY

STS Job 11297

68-4

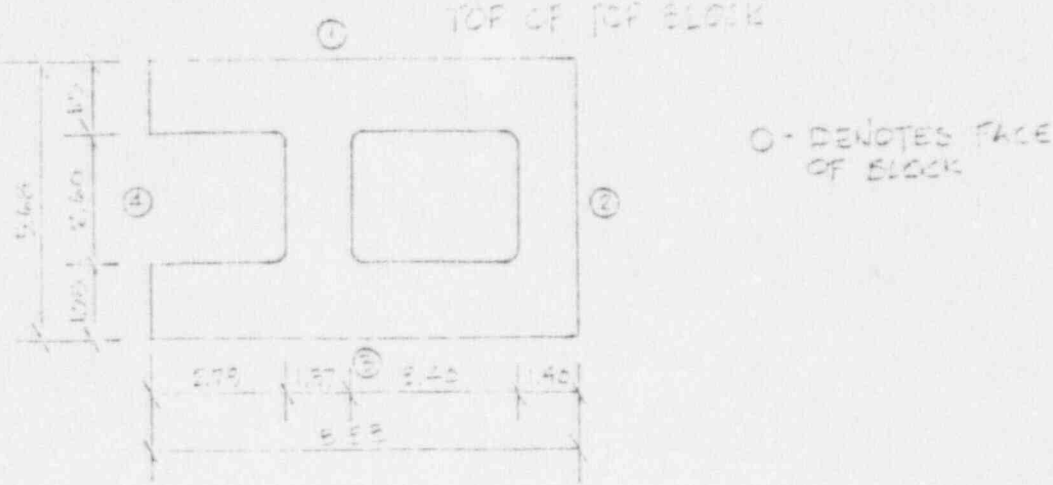


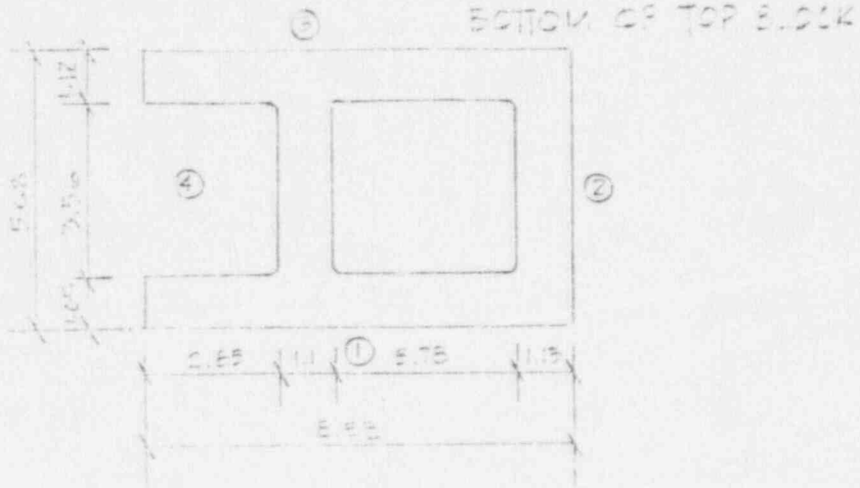
POINT BEACH MASONRY

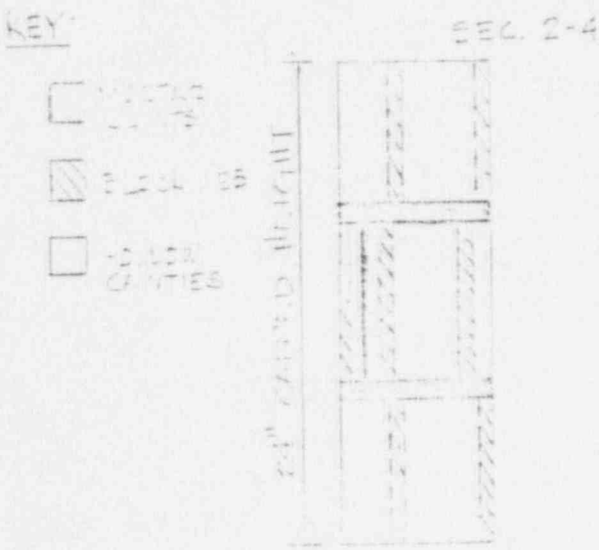
STS Job 11297

68-4

UNIT I - STAIRWELL



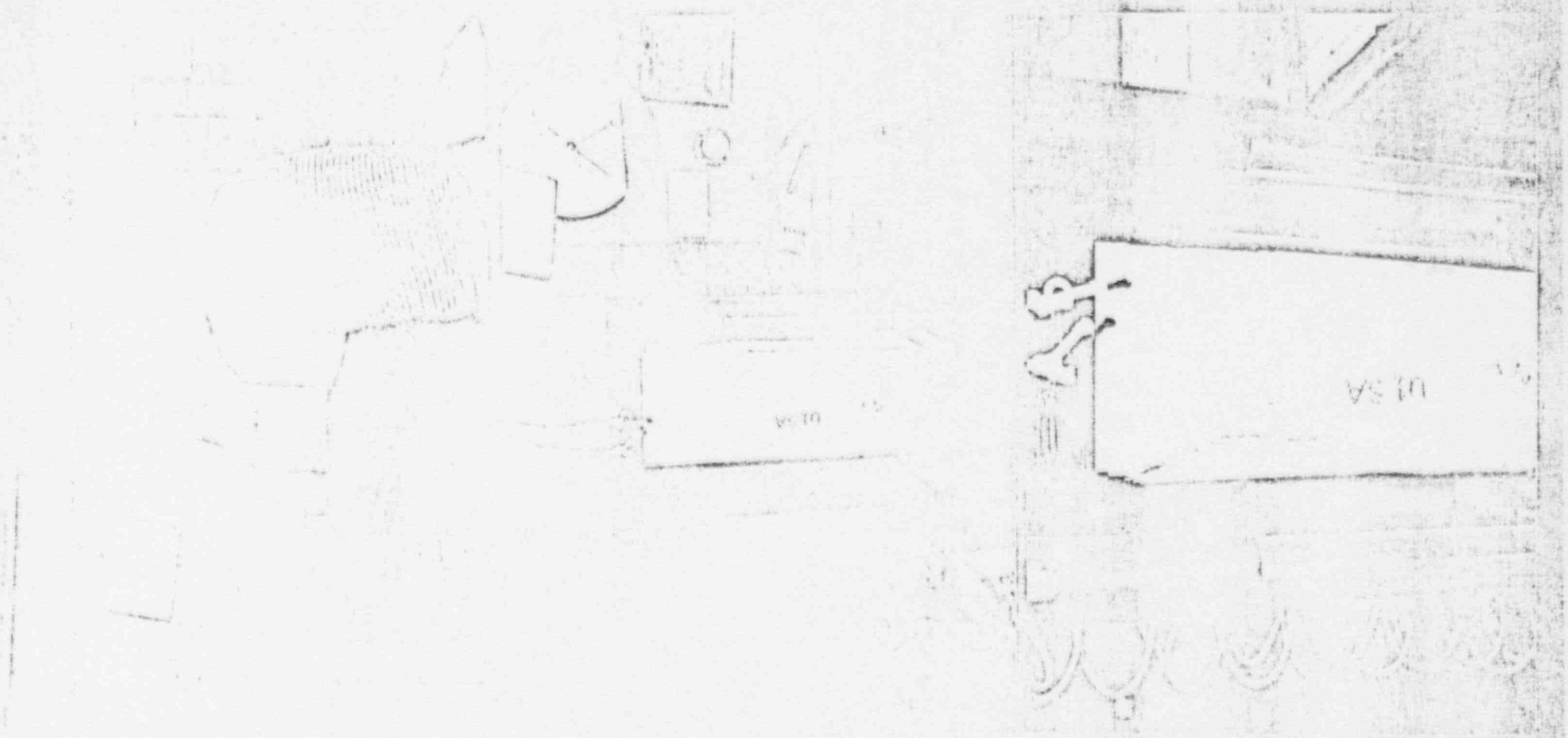




Gross Area	50.37 in^2
Net Area	30.65 in^2
Smallest Mortar Area	22.96 in^2
Maximum Load	48,000 lbs.
Gross Compressive Strength f'm	950 psi
Net Compressive Strength f'm	1570 psi
Compressive Strength f'm based on net mortar bearing area	2090 psi

POINT BEACH MASONRY
STS Job 11297

UI-SA



POINT BEACH MASONRY
STS Job 11297

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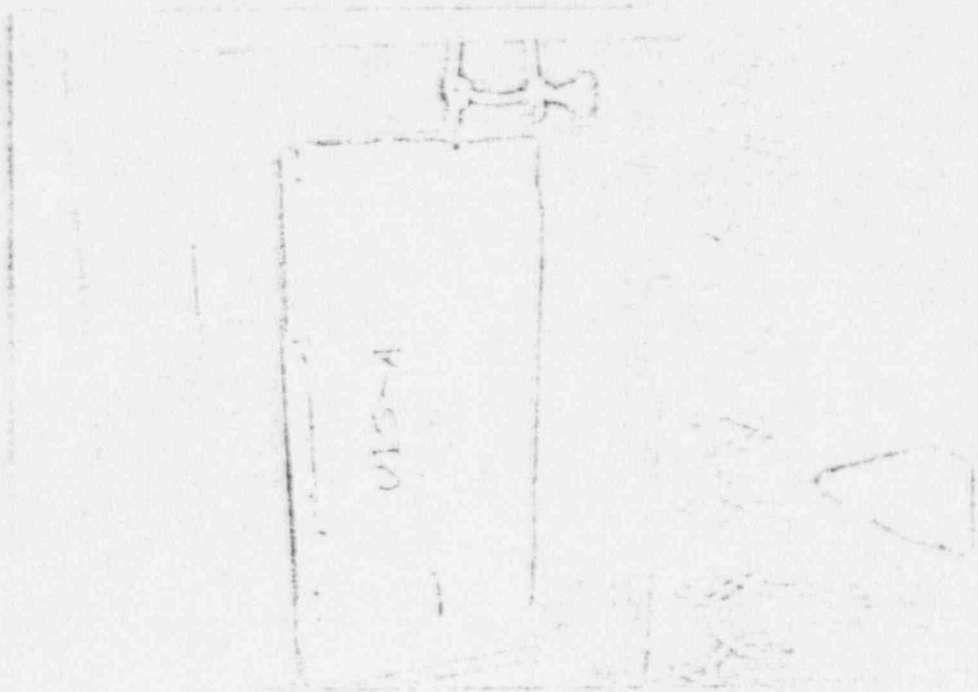
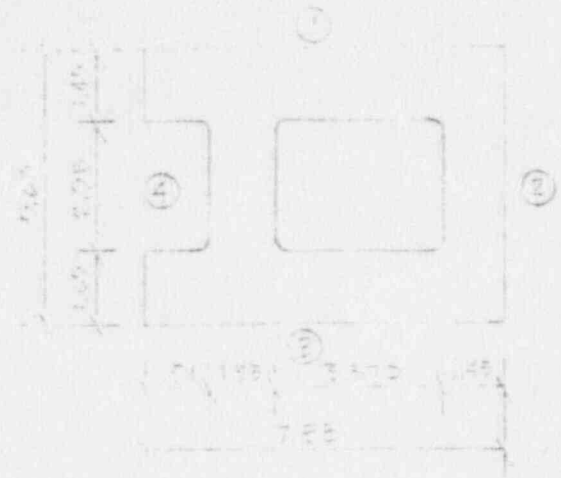
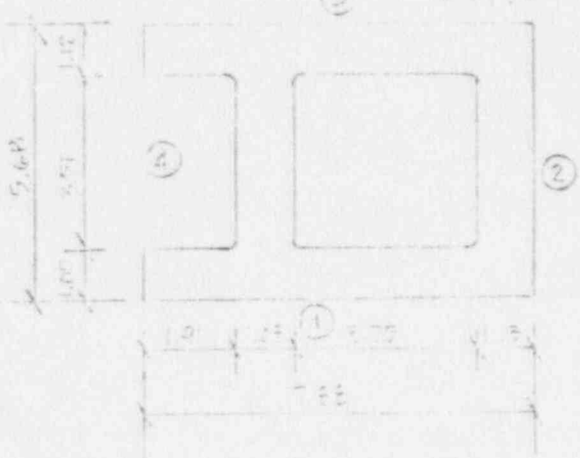


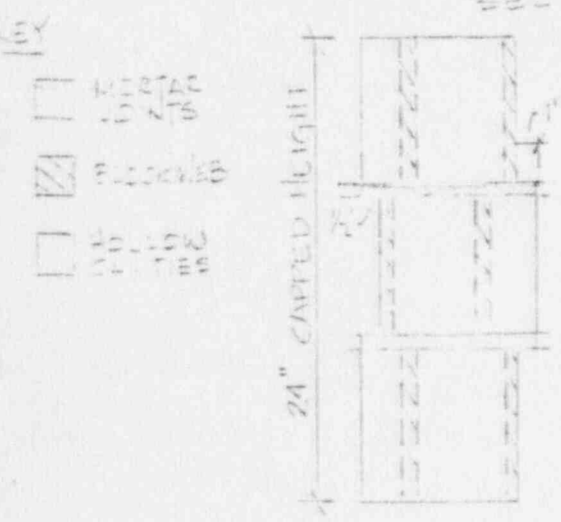
POINT BEACH MASONRY
STS Job 11297

UI-SA



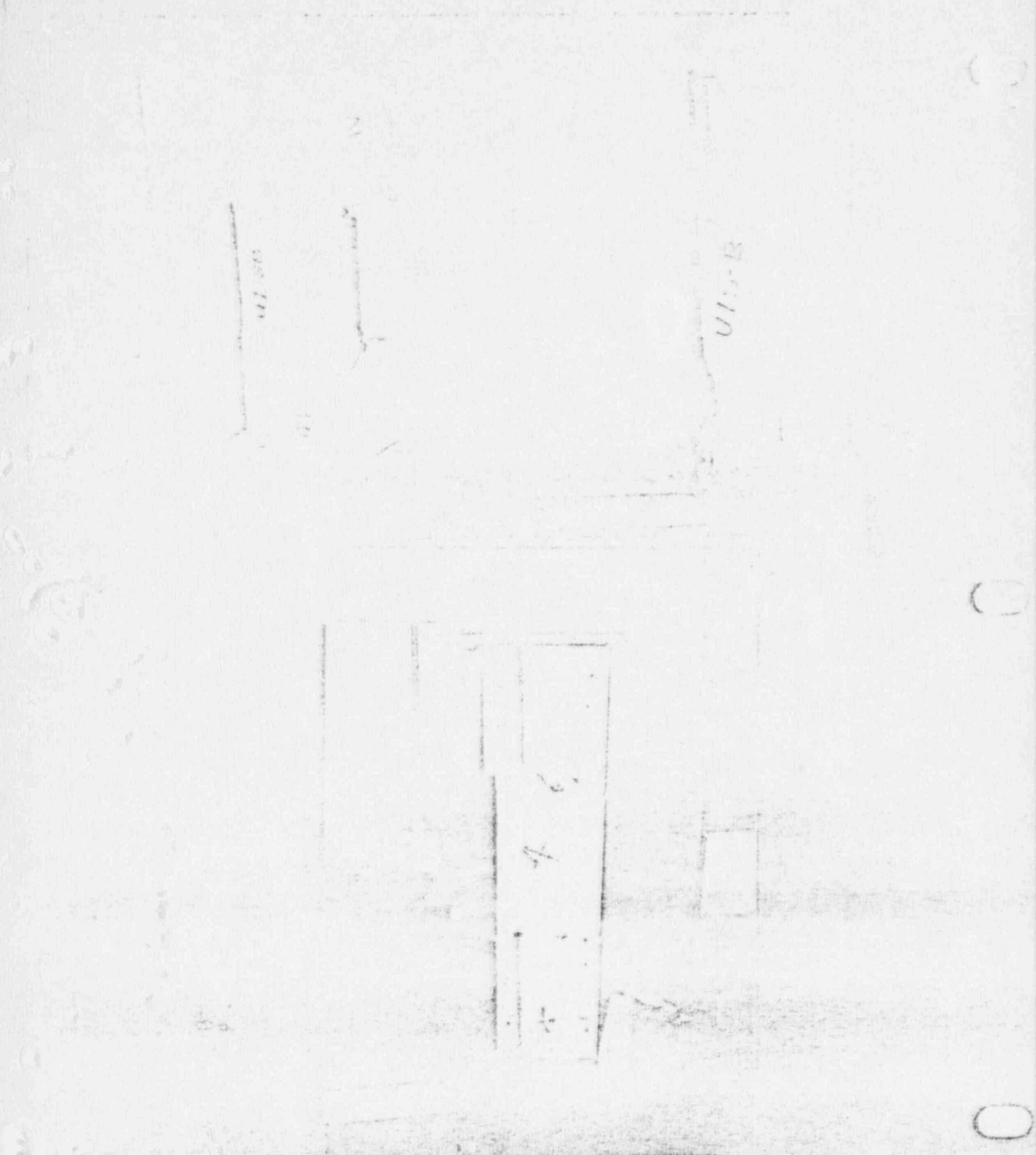
BOTTOM OF TOP BLOCK





Gross Area	44.69 in^2
Net Area	28.67 in^2
Smallest Mortar	21.13 in^2
Maximum Load	44,000 lbs.
Gross Compressive Strength f'm	980 psi
Net Compressive Strength f'm	1530 psi
Compressive Strength f'm based on net mortar bearing area	2080 psi

POINT BEACH MASONRY
STS Job 11297
UI-SB



POINT BEACH MASONRY
STS Job 11297

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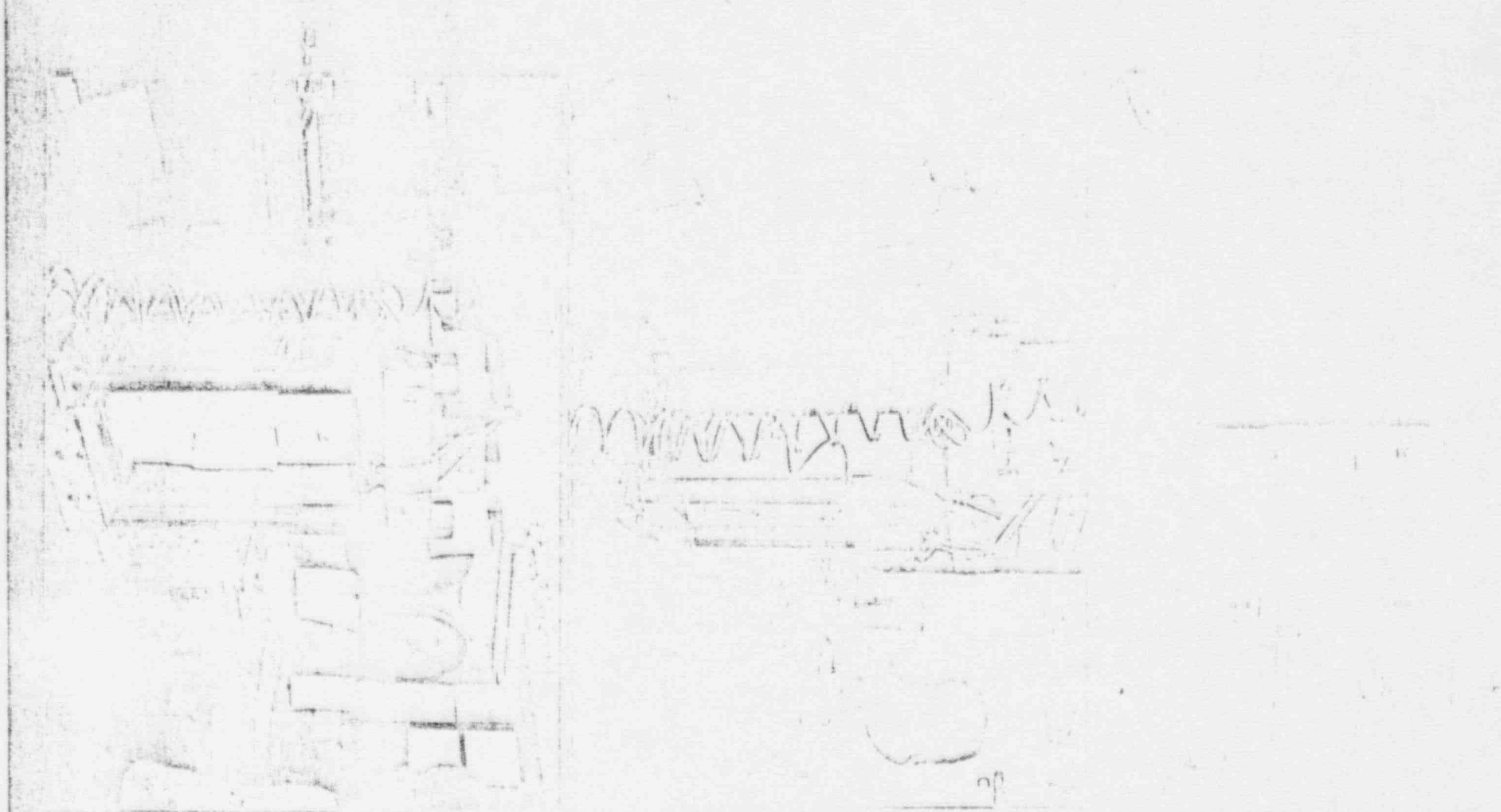
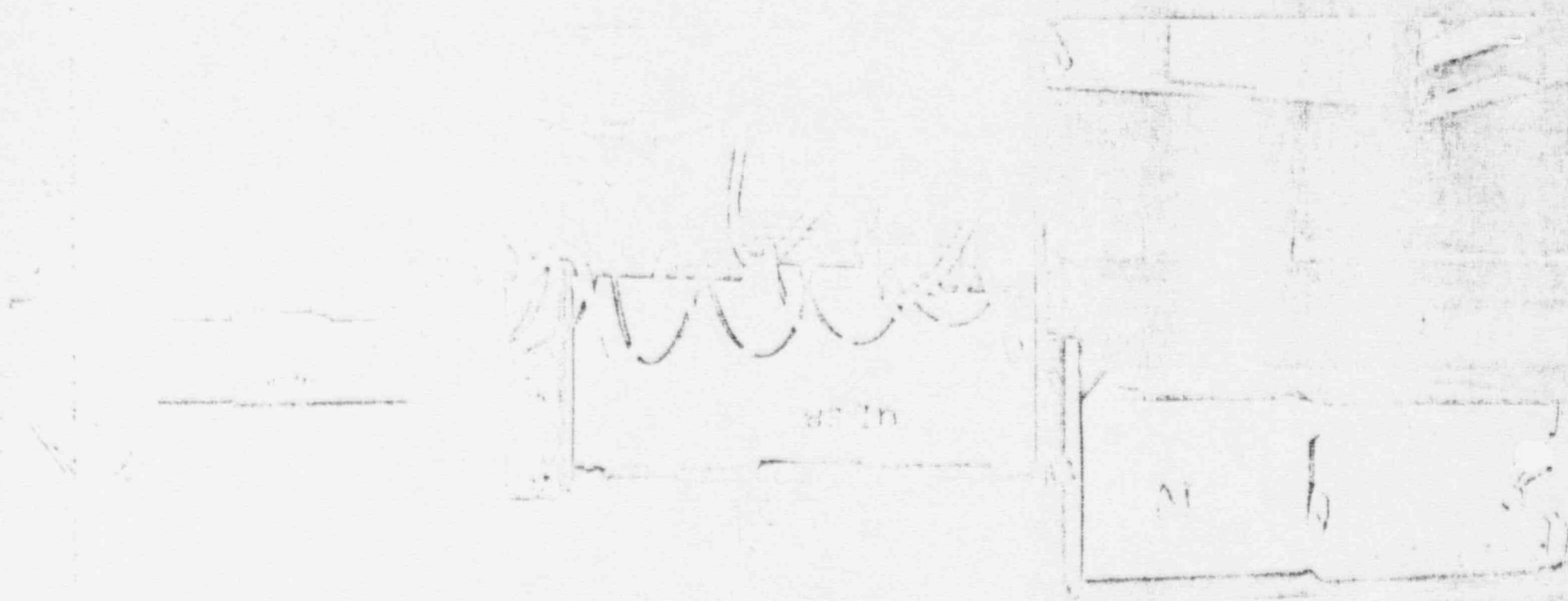
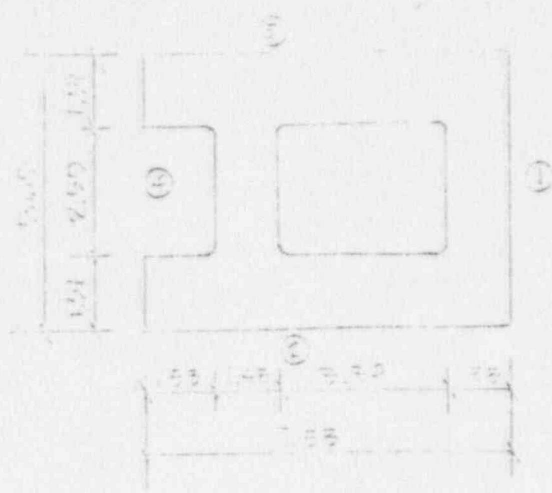


POINT BEACH MASONRY
STS Job 11297
UI-SB



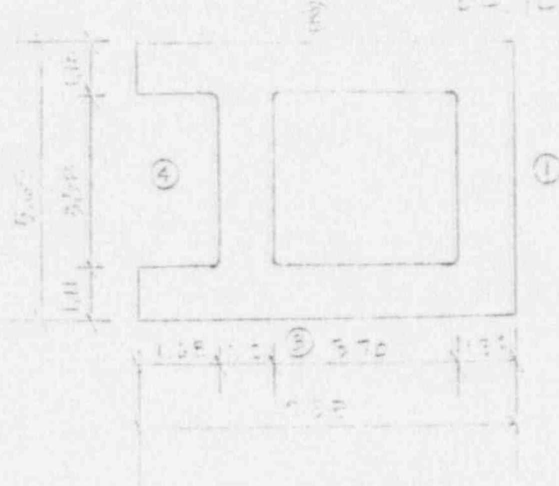
POINT BEACH MASONRY
STS Job 11297

UI-SB



○ DENOTES FACE OF BLOCK

BOTTOM OF TOP BLOCK



KEY:

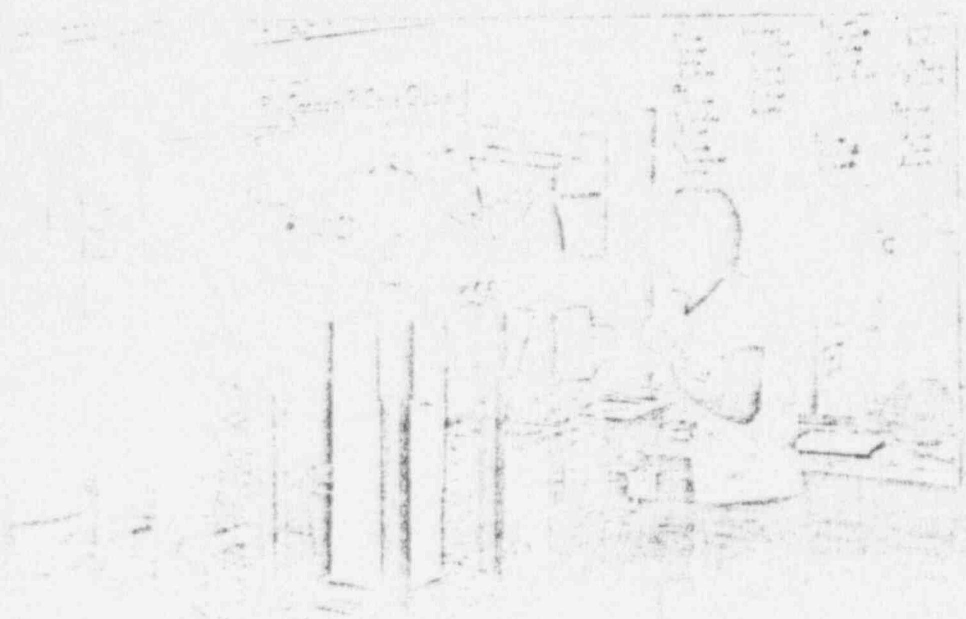
- MORTAR JOINT
- BLOCK WEB
- HOLLOW CAVITIES

Gross Area	42.54 in^2
Net Area	26.69 in^2
Smallest Mortar	18.97 in^2
Maximum Load	41,000 lbs.
Gross Compressive Strength f'm	960 psi
Net Compressive Strength f'm	1540 psi
Compressive Strength f'm based on net mortar bearing area	2160 psi

POINT BEACH MASONRY

STS Job 11297

UI-SC



POINT BEACH MASONRY

STS Job 11297

UI-SC

POINT BEACH MASONRY

STS Job 11297

UI-SC

POINT BEACH MASONRY

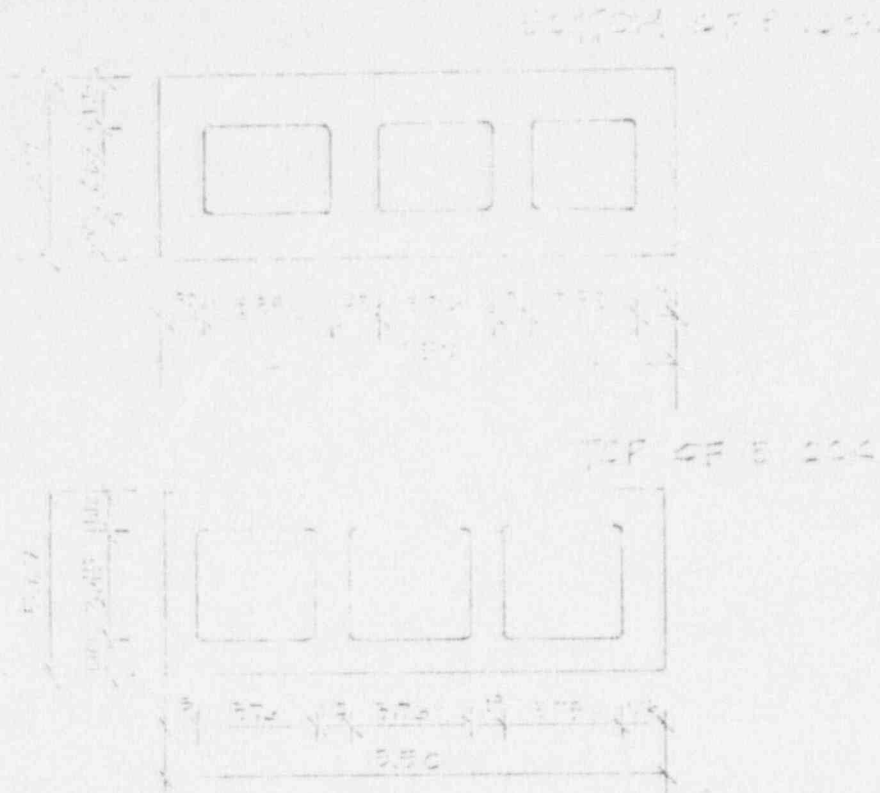
STS Job 11297

UI-SC

POINT BEACH MASONRY

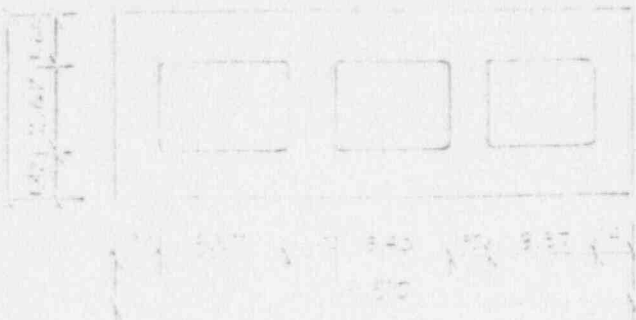
STS Job 11297

U1-SC

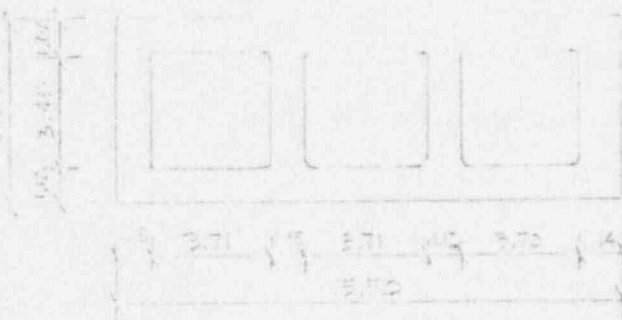


Gross Area	87.89 in^2
Net Area	55.29 in^2
Maximum Load	152,500 lbs.
Gross Compressive Strength	1740 psi
Net Compressive Strength	2760 psi

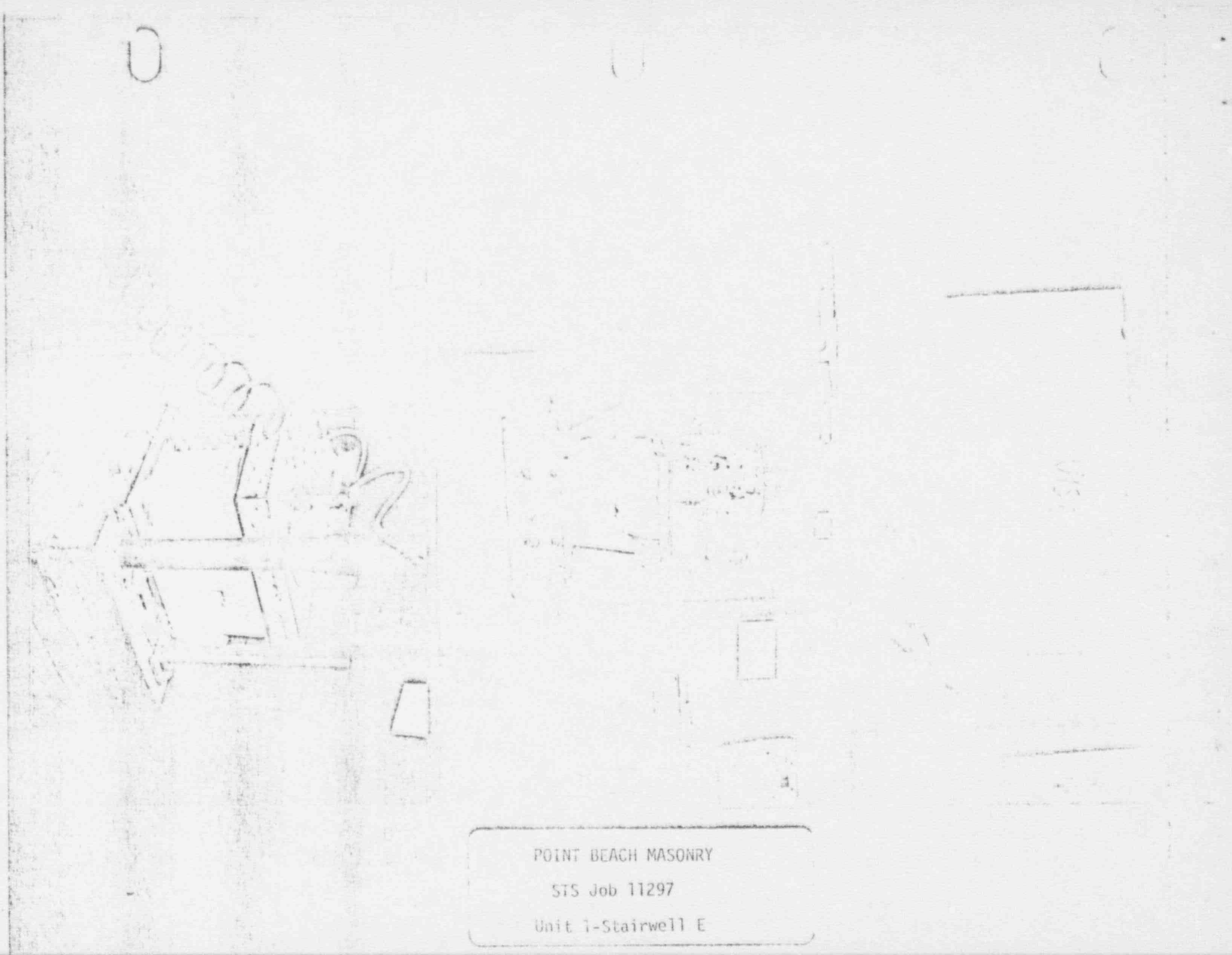
BOTTOM OF BLOCK



TOP OF BLOCK



Gross Area	87.42 in^2
Net Area	55.19 in^2
Maximum Load	104,000 lbs.
Gross Compressive Strength	2100 psi
Net Compressive Strength	3330 psi

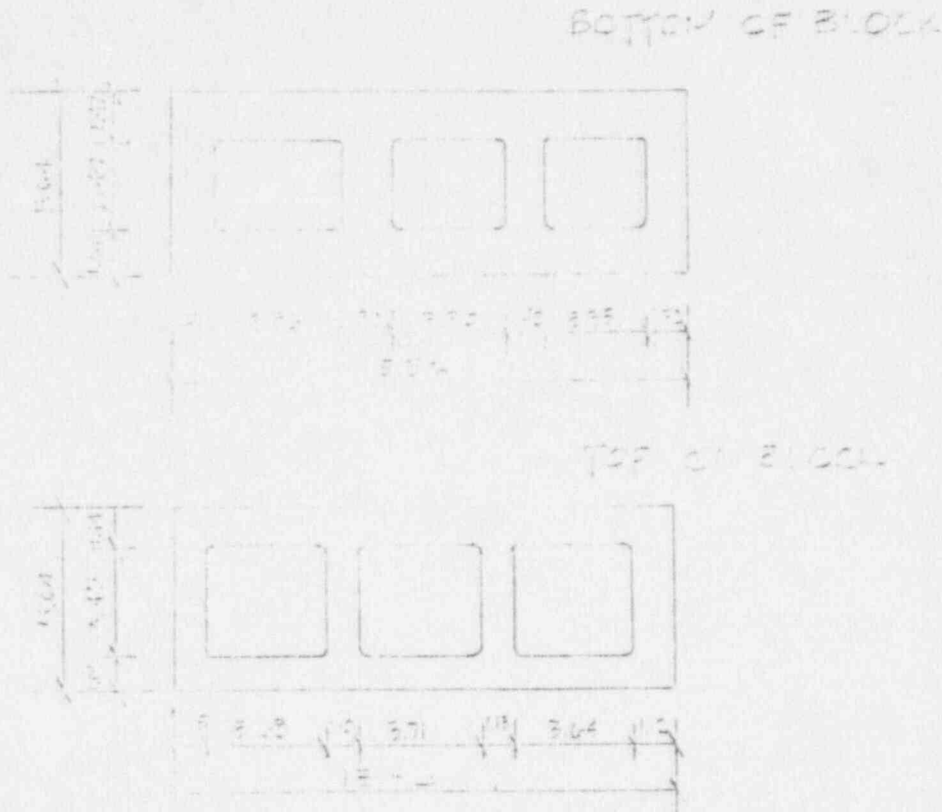

POINT BEACH MASONRY
STS Job 11297
Unit 1-Stairwell E

POINT BEACH MASONRY

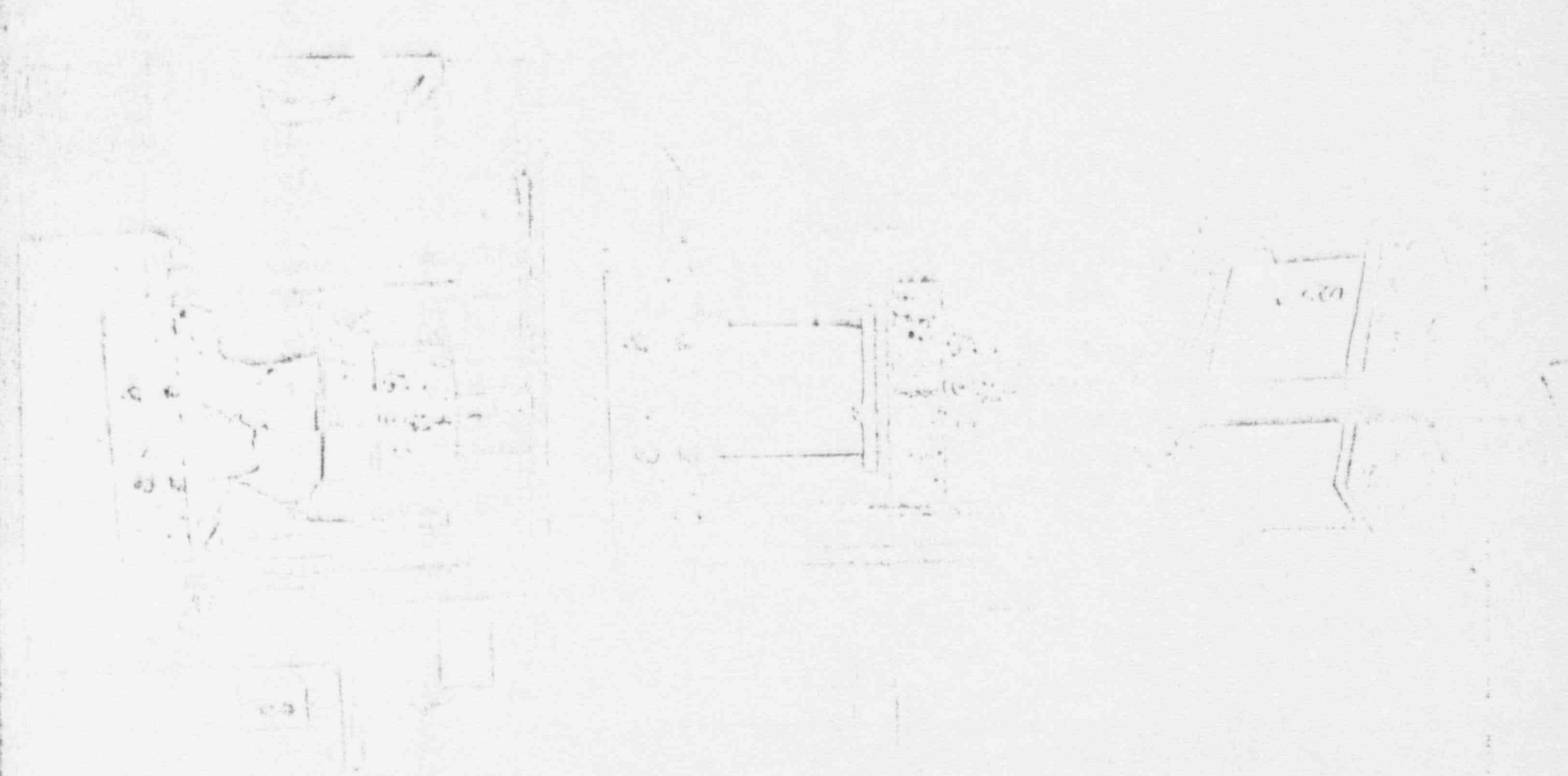
STS Job 11297

Unit 1-Stairwell E

UNIT 2 - STAIRWELL



Gross Area	87.78 in^2
Net Area	56.40 in^2
Maximum Load-	206,000 lbs.
Gross Compressive Strength-	2350 psi
Net Compressive Strength	3650 psi



POINT BEACH MASONRY

STS Job 17297

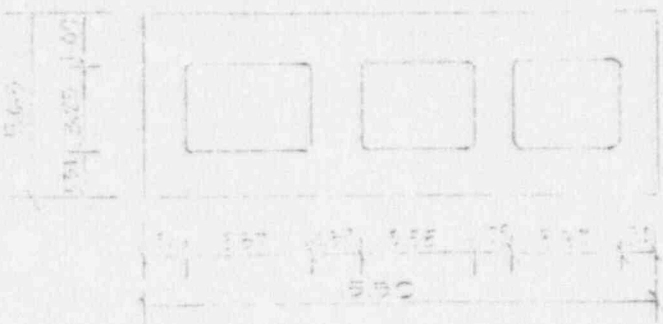
Unit 2-Stairwell A

POINT BEACH MASONRY

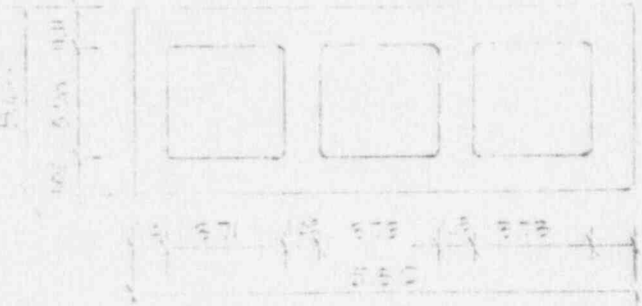
STS Job 11297

Unit 2-Stairwell A

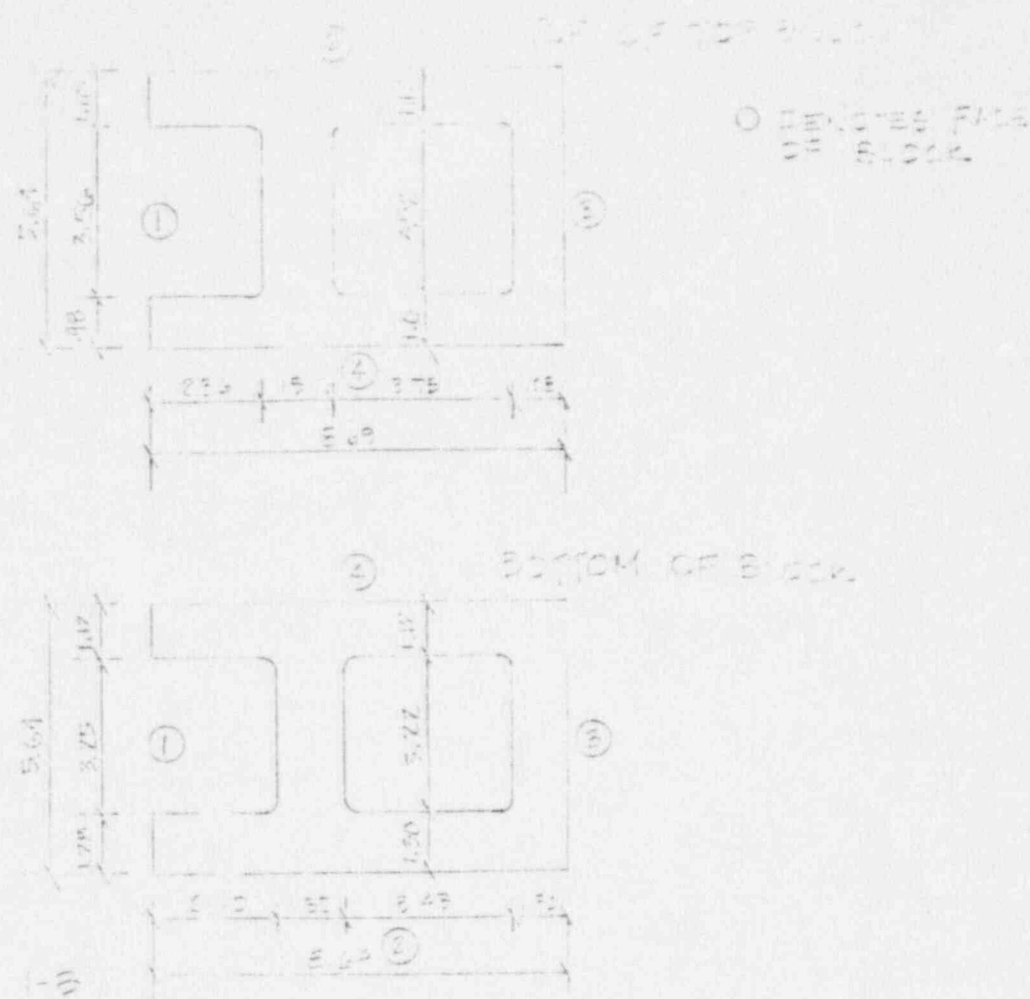
BOTTOM OF BLOCK

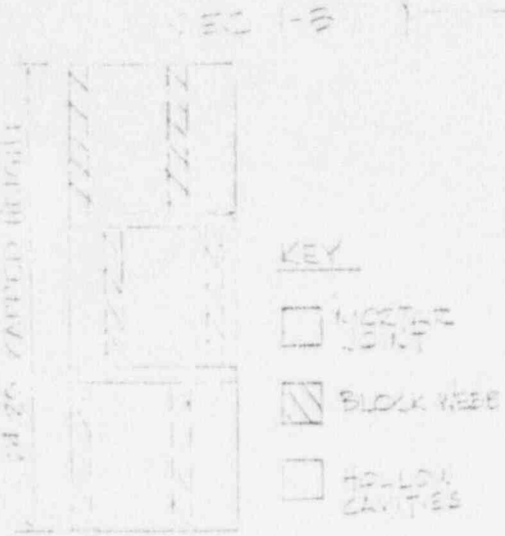


TOP OF BLOCK



Gross Area	87.26 in^2
Net Area	51.43 in^2
Maximum Load-	192,500 lbs.
Gross Compressive Sgrength-	2200 psi
Net Compressive Strength-	3740 psi



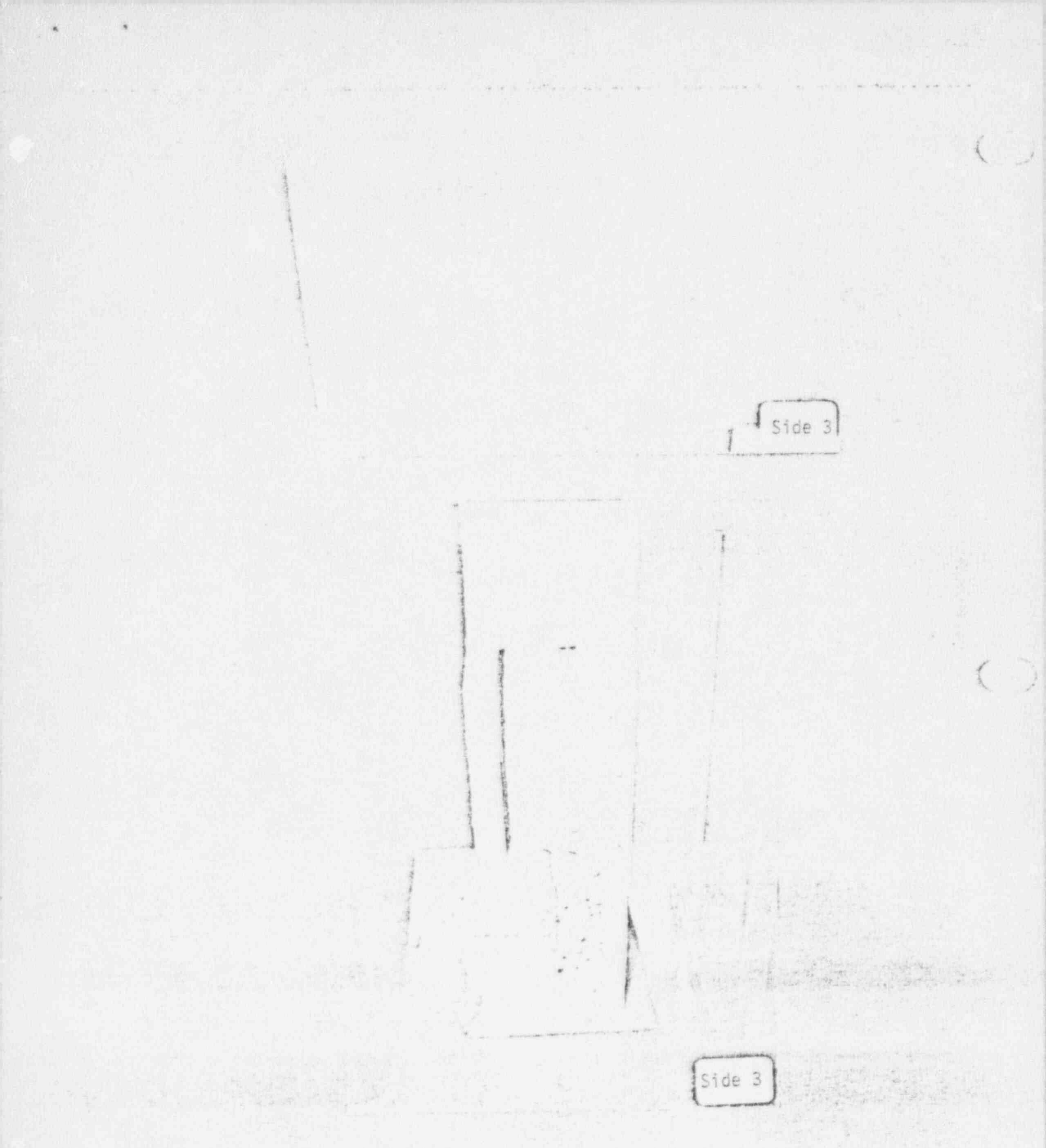


Gross Area	49.01 in^2
Net Area	28.43 in^2
Smallest Mortar Area	19.63 in^2
Maximum Load	49,000 lbs.
Gross Compressive Strength f'm	1000 psi
Net Compressive Strength f'm	1720 psi
Compressive Strength f'm based on net mortar bearing area	2500 psi

POINT BEACH MASONRY

STS Job 11297

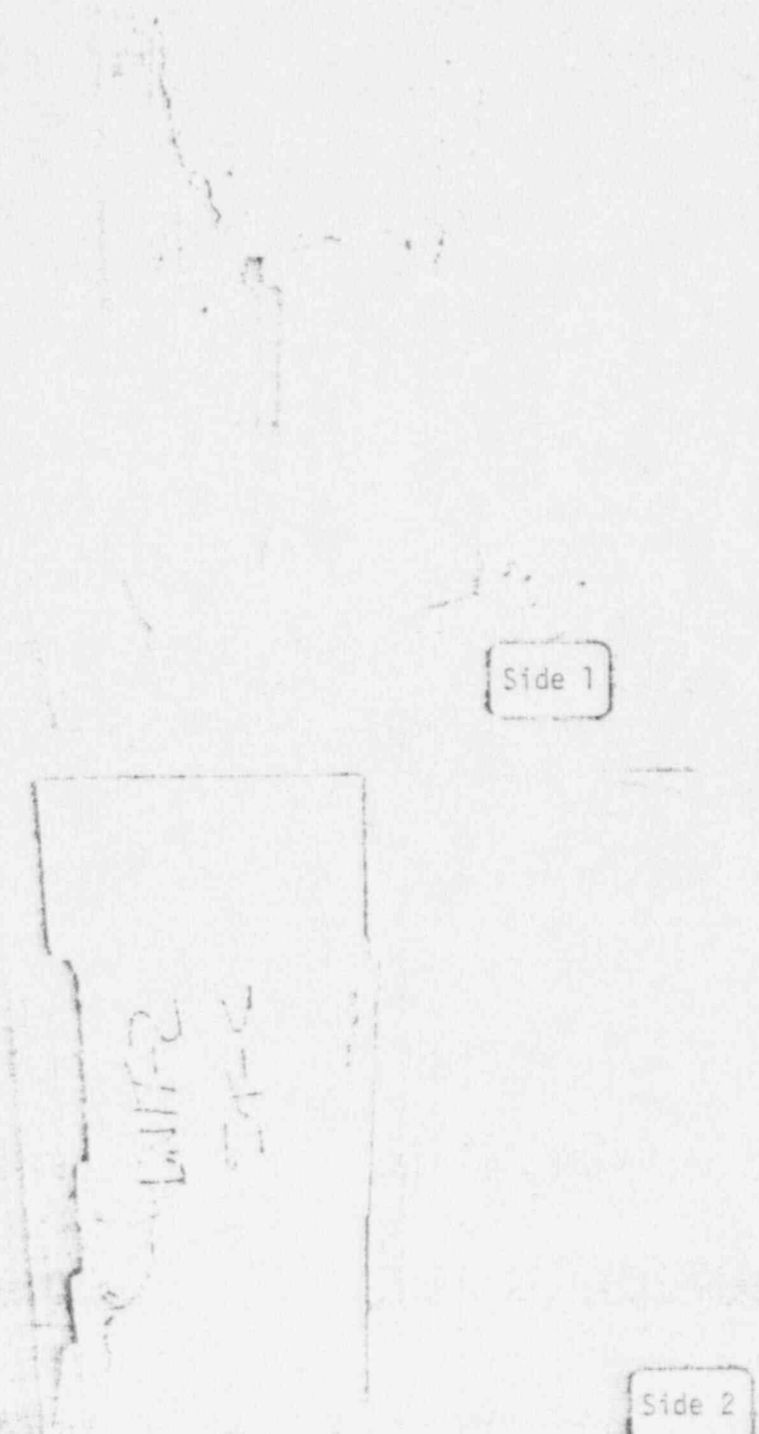
Unit 2-Stairwell C



POINT BEACH MASONRY

STS Job 11297

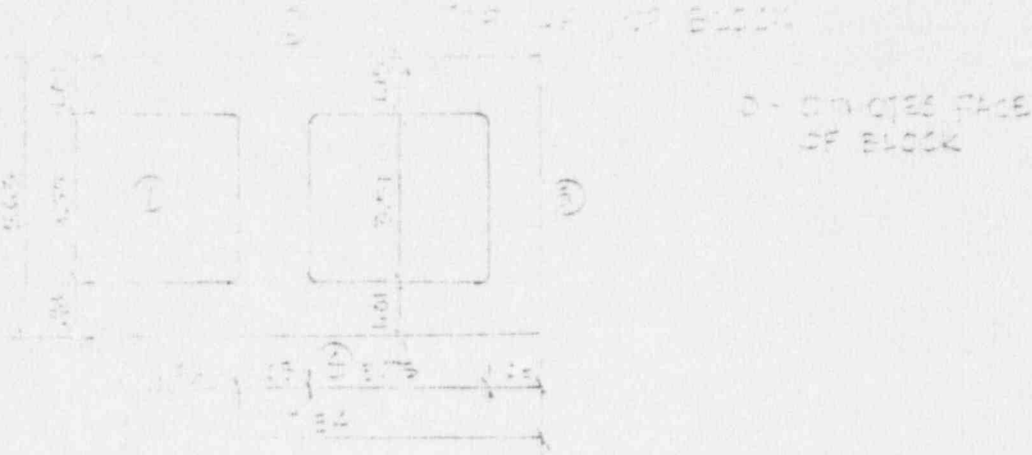
Unit 2 Stairwell C

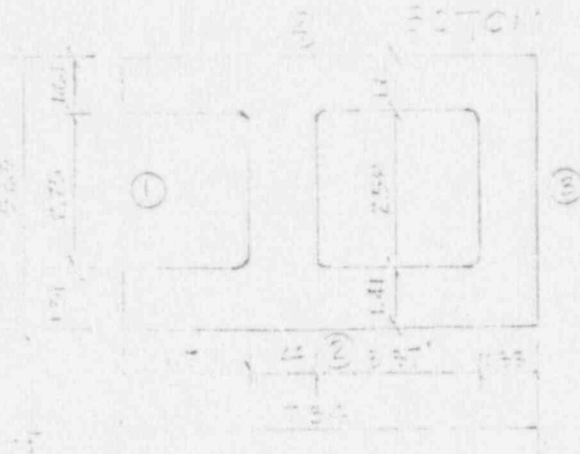


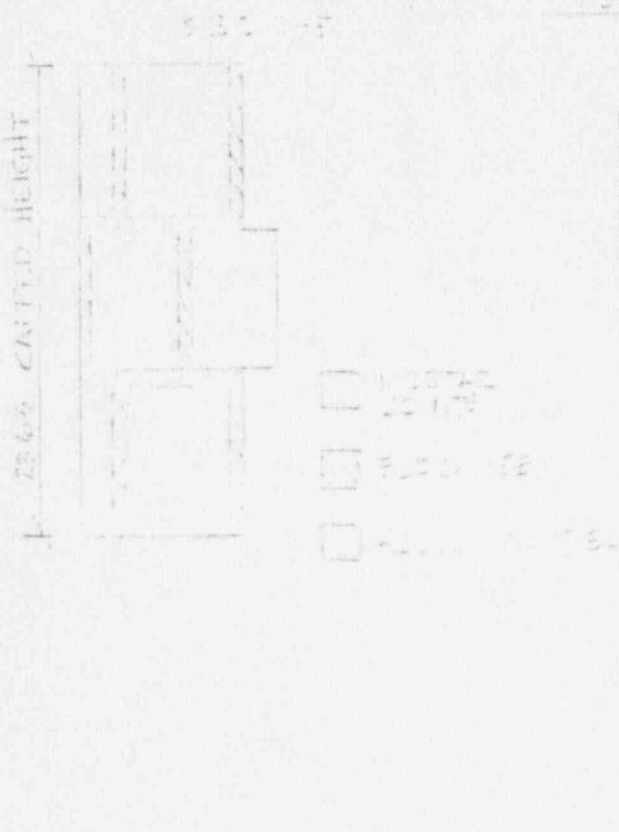
POINT BEACH MASONRY

STS Job 11297

Unit 2-Stairwell C

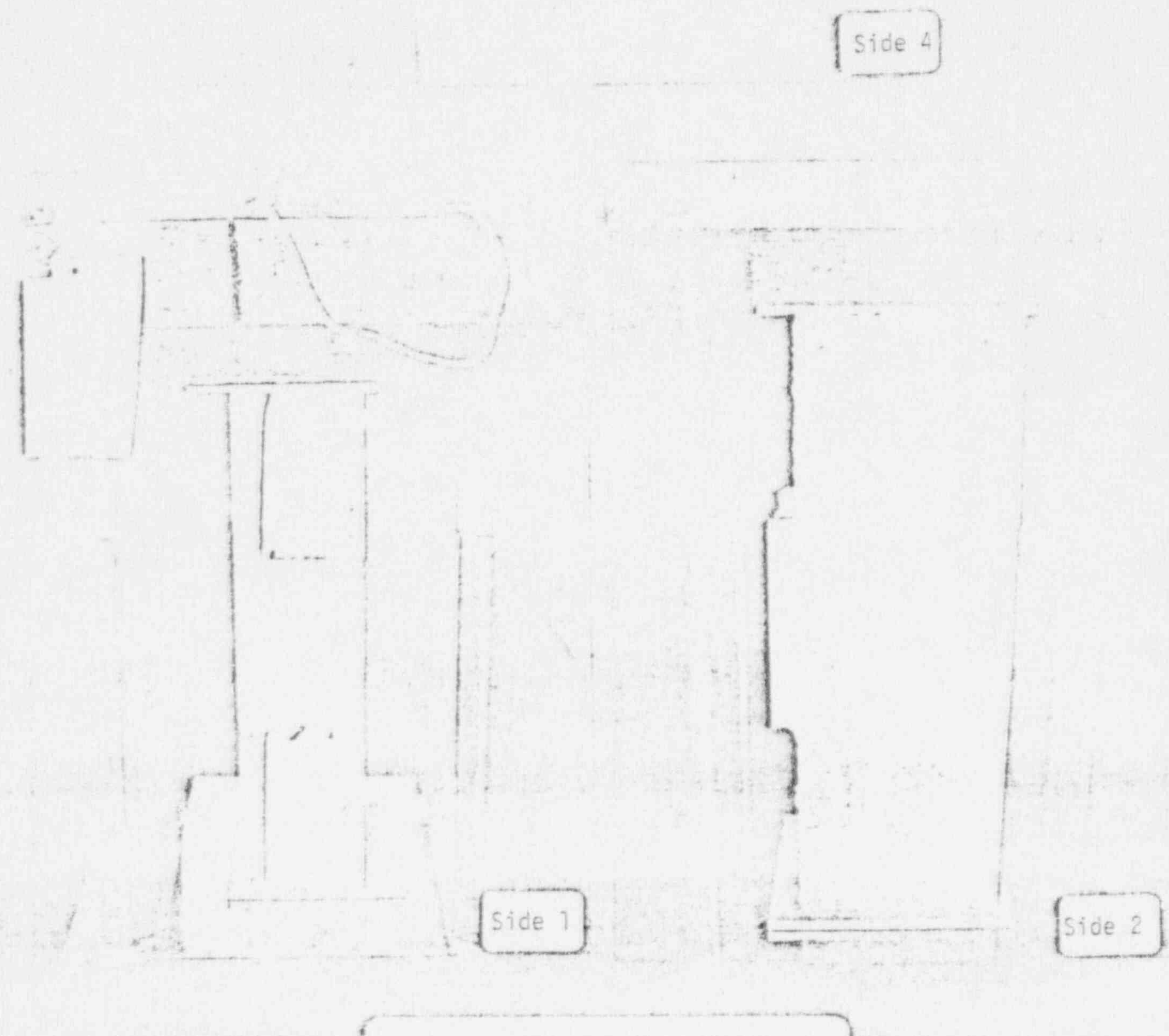






Gross Area	44.14 in^2
Net Area	27.54 in^2
Smallest Mortar Area	19.94 in^2
Maximum Load	54,000 lbs.
Gross Compressive Strength f'm	1220 psi
Net Compressive Strength f'm	1960 psi
Compressive Strength f'm based on net mortar bearing area	2710 psi

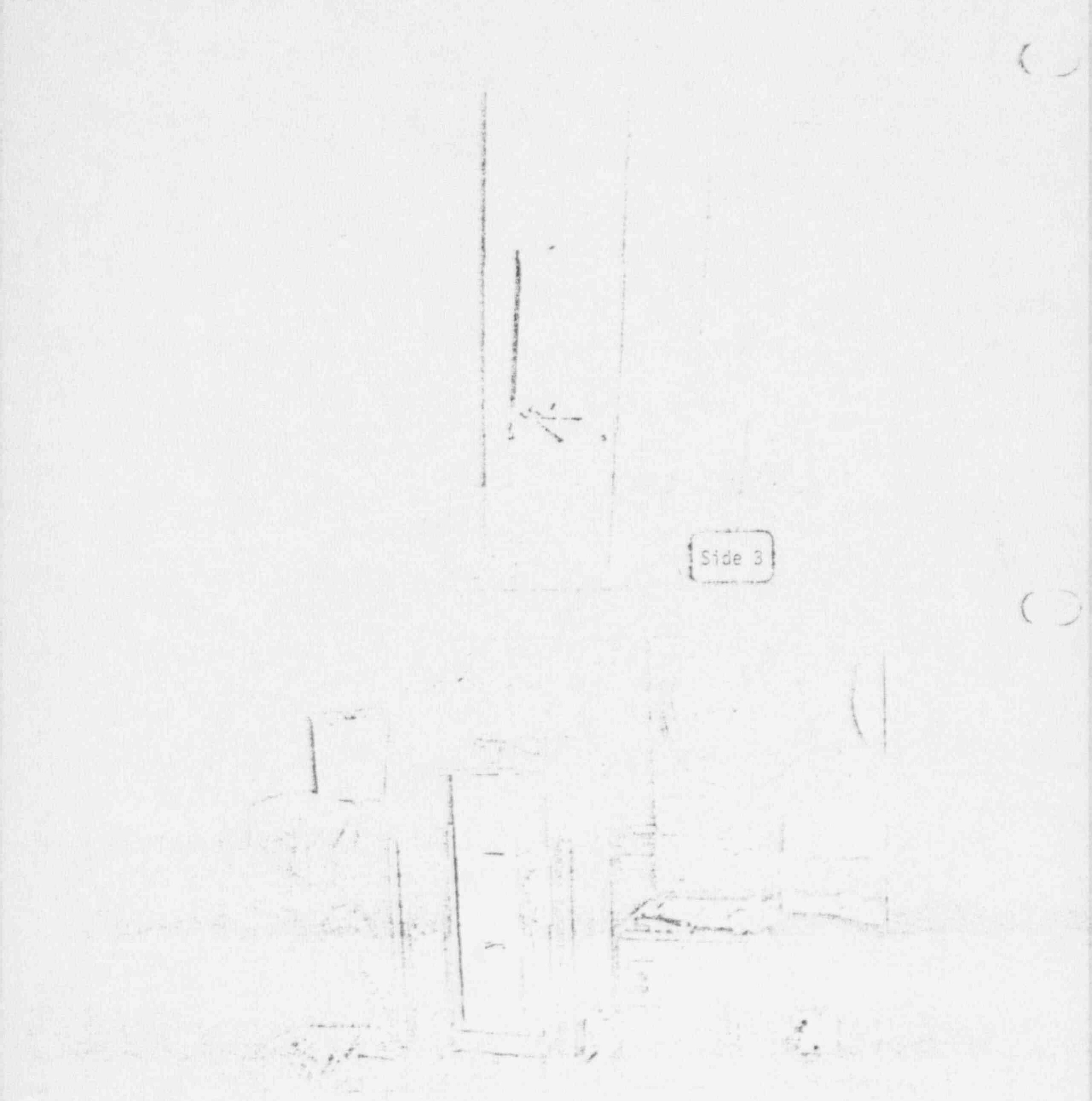
UNIT
2
ST-D



POINT BEACH MASONRY

STS Job 11297

Unit 2-Stairwell D



POINT BEACH MASONRY

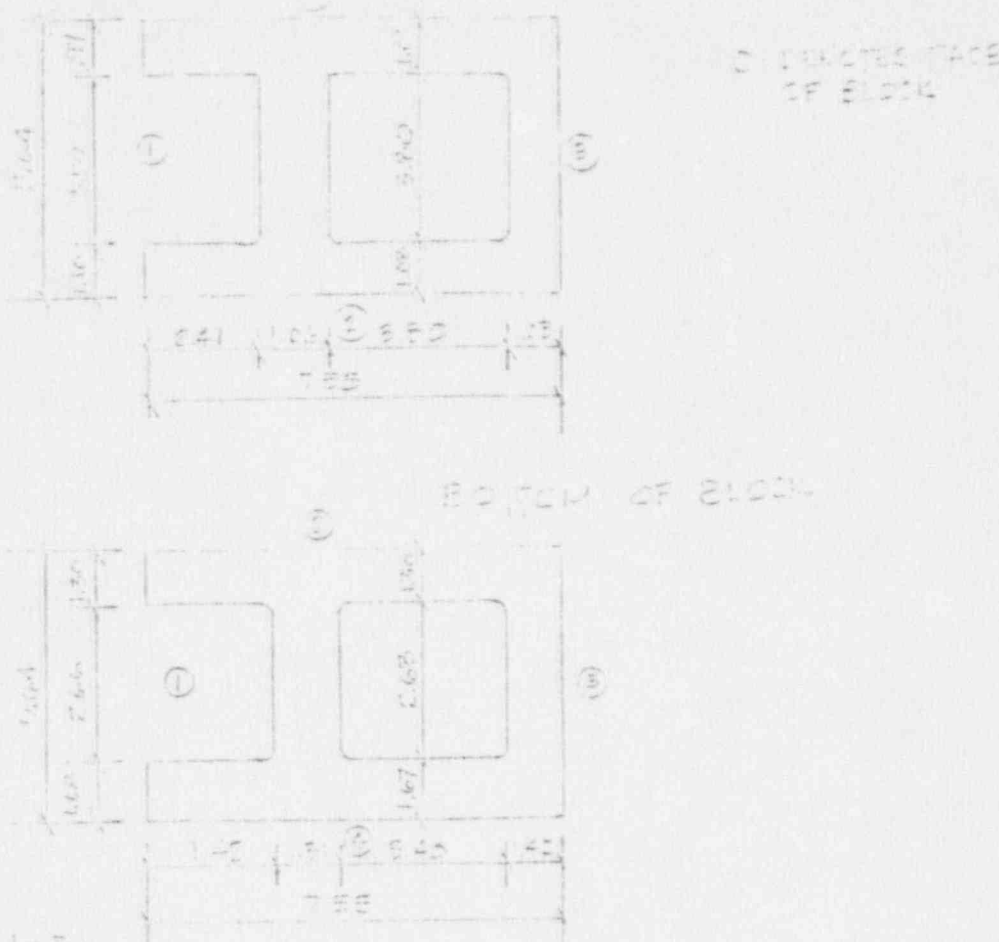
STS Job 11297

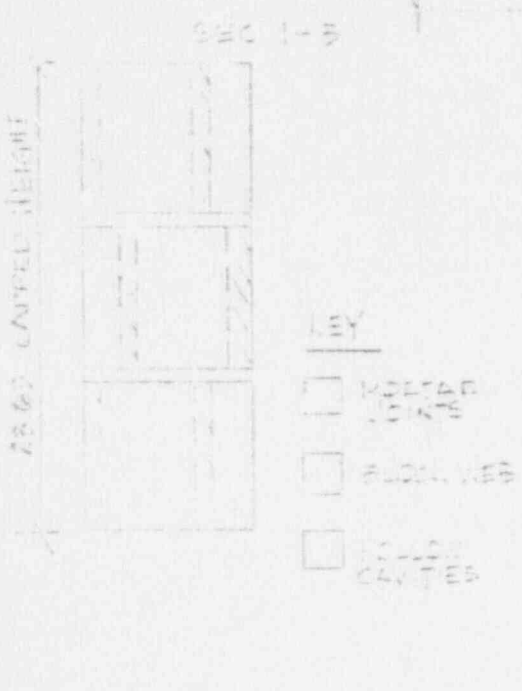
Unit 2-Stairwell D

POINT BEACH MASONRY

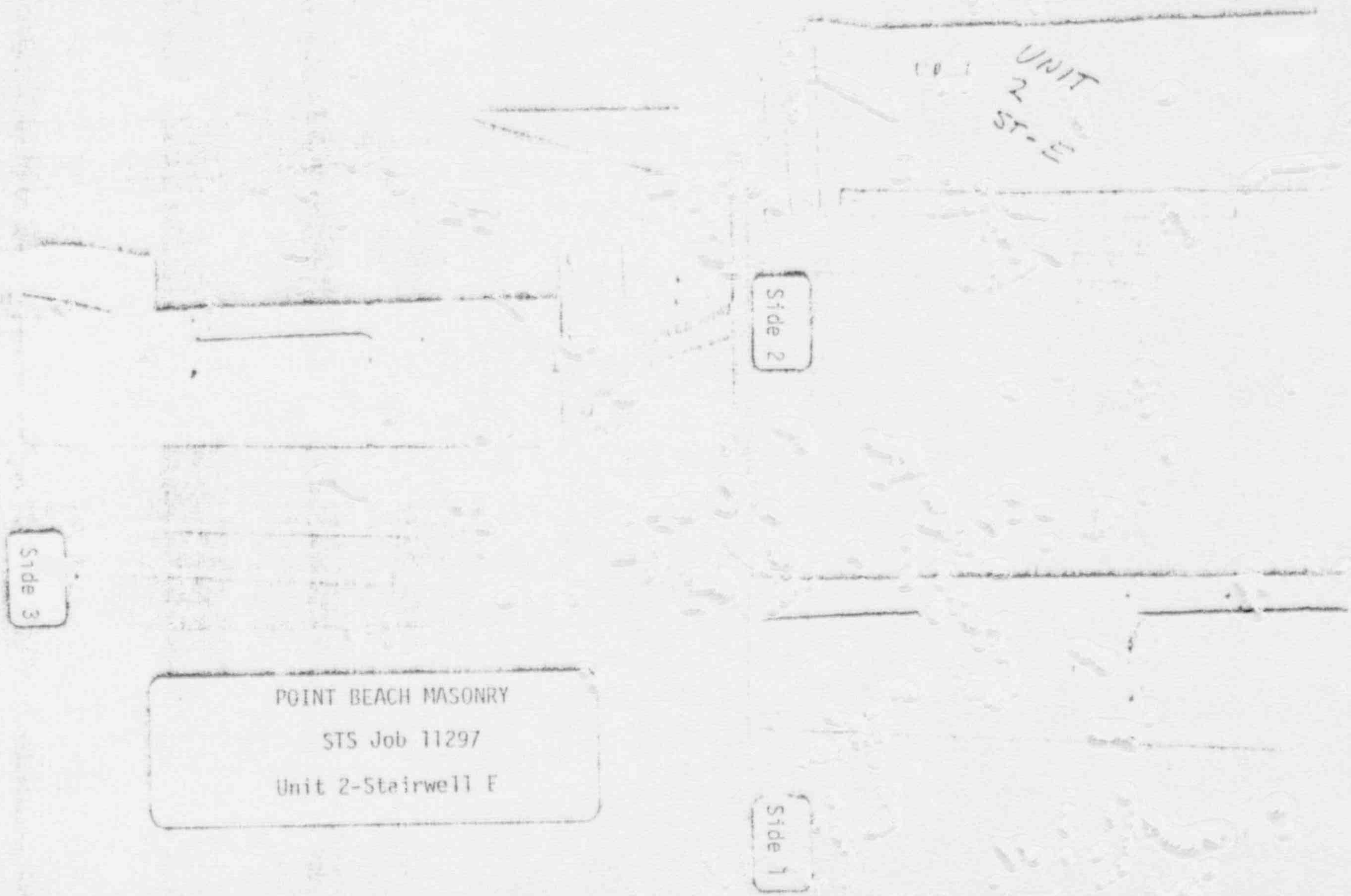
STS Job 11297

Unit 2-Stairwell D





Gross Area	42.58 in^2
Net Area	25.35 in^2
Smallest Mortar Area	17.74 in^2
Maximum Load	52,500 lbs.
Gross Compressive Strength f'm	1230 psi
Net Compressive Strength f'm	2070 psi
Compressive Strength f'm based on net mortar bearing area	2960 psi



POINT BEACH MASONRY

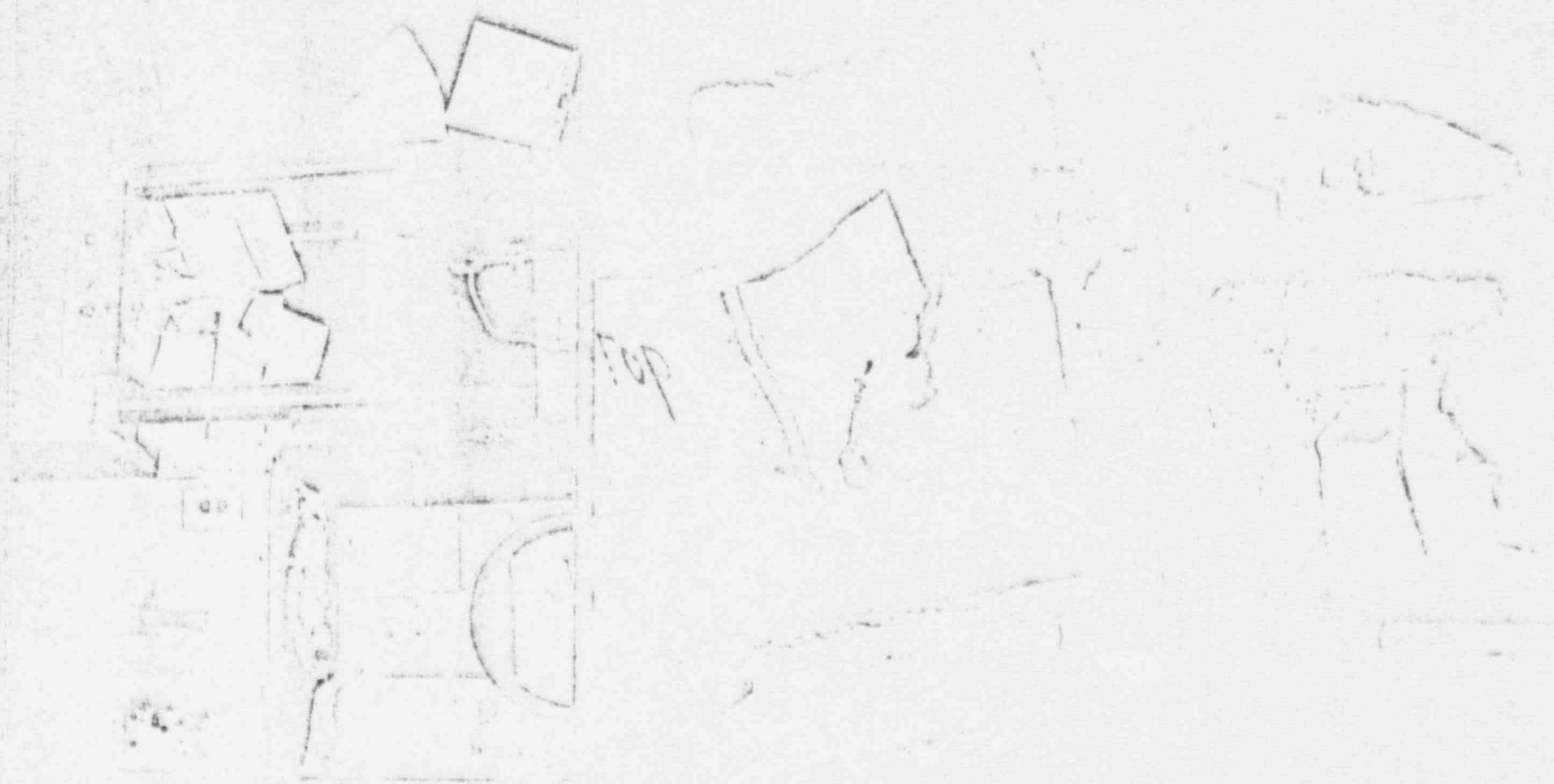
STS Job 11297

Unit 2-Stairwell E

POINT BEACH MASONRY

STS Job 11297

Unit 2-Stairwell E



POINT BEACH MASONRY

STS Job 11297

Unit 2-Stairwell E

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LABQUIP CORPORATION

Machine 350k Forney LT-700
Serial No. 66,173

Verification No.
LQ 27,371

Owner Soil Testing Services of Wisc. Inc.
Location 450 Lambeau St.
Green Bay, WI 54303
Attn: Ken Kujava

Date 8-4-81

TESTING MACHINE VERIFICATION CERTIFICATE

This is to certify that the above testing machine has been calibrated by Labquip personnel. The loading ranges have been found to be within the accuracy tolerance(s) indicated below:

Capacity Range	Loading Range	Accuracy Tolerance
POUNDS	POUNDS	PER CENT
0 to 350,000	35,000 to 325,000	.62
0 to 60,000	6,000 to 55,000	.49

Method of verification and listed data are in accordance with ASTM Designation E 4, or other applicable specification. Accuracy of all Calibration devices traceable to the National Bureau of Standards.

The testing device used for calibration has been certified by the U.S. Bureau of Standards or qualified private agency meeting the conditions set forth in paragraph II-1.

LABQUIP CORPORATION

By Richard D. Besselgeer

LABQUIP CORPORATION

402 Mondamon / P.O. Box 4100 / Minooka, IL 60447 / 815-467-4490

LQ 27371

TESTING MACHINE CALIBRATION DATA AND REPORT

Customer: SOIL TESTING SERVICES OF WISC. INC.

Location: 450 LAMBEALL ST.
GREEN BAY, WI. 54303

Machine: 350K FORNEY LT-700

Serial No. 66173 Previous Calibration Date 6-17-80

Date 8-4-81

Customer Order No.

Order Date

Ring Temp. 72°F

Verification Reading	*	Machine Reading	Error	%	Verification Reading	*	Machine Reading	Error	%
RANGE 350,000 LBS.									
35,000	3	35216	-216	.62					
70,000	3	70268	-268	.38					
140,000	2	140419	-419	.30					
210,000	1	210599	-599	.28					
280,000	1	280686	-686	.24					
325,000	1	325753	-753	.23					
RANGE 60,000 LBS.									
6,000	3	6020	-20	.33					
12,000	3	12032	-32	.27					
24,000	3	23949	+51	.21					
36,000	3	35906	+94	.26					
48,000	3	47802	+198	.41					
55,000	3	54729	+271	.49					

Remarks:

*** CALIBRATION EQUIPMENT USED**

All verification equipment—including dead weights, proving rings (mfg. by Olsen or Morehouse), proving levers, etc. — is calibrated or traceable to the latest procedures stipulated by the U.S. National Bureau of Standards and ASTME74. ALL PROVING CALCULATION CORRECTED FOR TEMPERATURE.

ACCURACY SUMMARY

Capacity Range	Loading Range Within	%	*	Serial No.	Loading Range	Verification Date
350,000	35,000 - 325,000	.62	1	5-5335	400,000	3-12-81
60,000	6,000 - 55,000	.49		732.10/220748		
			2	93	200,000	8-6-79
				STT 01/101550		
			3	809	60,000	5-20-81
				STT 01/101925		

Calibration in accordance with ASTM E4, applicable specification

Service Engineer [signature]

Witnessed by [signature]

EVALUATION

OF

SINGLE WYTHE ASSUMPTION

TO REPRESENT

MULTIPLE WYTHE WALLS

Prepared for

Point Beach Nuclear Power Plant, Units 1 and 2

WISCONSIN ELECTRIC POWER COMPANY

Milwaukee, Wisconsin

Prepared by

COMPUTECH ENGINEERING SERVICES, INC.

Berkeley, California

September, 1981

REPORT NO. R553.07

Revision 1

6pp

TABLE OF CONTENTS

1 INTRODUCTION

The Nuclear Regulatory Commission (NRC) staff on June 9-11, 1981 reviewed the criteria and calculations performed on IE Bulletin 80-11 "Masonry Wall Design" for the Point Beach Nuclear Power Plant. Action Item 5 resulting from the review meeting stated that with regard to out-of-plane loading, the licensee shall demonstrate that the use of the single wythe assumption for multiple wythe walls results in a conservative evaluation with respect to frequency shift and out-of-plane drift consideration.

This short report presents the analyses that were performed, the results of the analyses, a discussion of the results, and the conclusions.

2 ANALYSIS METHODOLOGY

All walls were analyzed in accordance with the procedures given in "Criteria for the Re-evaluation of Concrete Masonry Walls for Point Beach Nuclear Power Plant." Specifically, plate analysis was used to assess the out-of-plane response of the wall. The computer program SAP5A was used to perform a finite element dynamic analysis, utilizing the response spectrum method.

All wythes in a multiwythe wall were assumed to respond as single wythe walls because of the difficulty in verifying the adequacy of the collar joint between the wythes. This was assumed to be conservative when using the re-evaluation criteria and the objective of Action Item 5 is to validate the degree of conservatism.

Two double wythe walls (Wall Nos. 24 and 65-1) were selected to compare the results obtained from the wall acting either as a single or double wythe wall using the re-evaluation criteria. In using the re-evaluation criteria the walls were assumed to have pinned supports at all appropriate boundaries. As a consequence no forces are induced in the wall due to out-of-plane drift. The validity of this assumption is addressed in Computech Engineering Services, Inc. Report No. R553.11. Some of the results of Report No. R553.11 are included in this report for the purpose of addressing Action Item 5. Wall 24 is 164 inches long, 156 inches high and consists of two wythes of twelve inch wide units. Wall 65-1 is 109 inches long, 176 inches high and consists of two wythes of six inch wide units. Each wall was analyzed as both a single and double wythe wall using the same number of nodes, mesh size and boundary conditions. Two boundary conditions were used for each wall: one simply supported on each boundary and the other fixed. For the fixed boundary condition the out-of-plane drift effects were included as reported in Report No. R553.11.

3 RESULTS

The results of the analyses performed using the simply supported boundary conditions are given in Table 1 and 2. Table 1 compares the frequencies of each wall acting either as a single or double wythe wall. Table 2 compares the maximum stress ratios for each wall acting either as a single or double wythe wall. The values given in Table 2 are based on the procedures given in the re-evaluation criteria. That is the walls are assumed to have simply supported boundary conditions.

The results given in Table 3 are extracted from Report No. R553.11 and provide a comparison of the single or double wythe wall with fixed boundary conditions incorporating the effects of out-of-plane drift.

4 DISCUSSION OF RESULTS

From the results presented in Table 1 it is clear that the single wythe assumption is conservative with respect to frequency shift. The fundamental or first mode frequency of the double wythe wall is approximately twice that of the single wythe wall. For walls with more than two wythes the shift in frequency would be even greater. The effect of out-of-plane drift effects depends on how this is incorporated in the analysis of the walls and is further discussed in Report No. R553.11. For the case of simply supported boundary conditions, out-of-plane drift does not induce forces in the walls and the maximum stress ratios given in Table 2 indicate that the maximum stresses in the wall due to other out-of-plane forces for the double wythe wall are approximately one-half of those of the corresponding single wythe wall.

For the case of fixed boundary conditions, incorporating out-of-plane drift effects, the results given in Table 3 indicate that the maximum stress ratios for the double wythe walls are from 1 to 130 percent less than the corresponding single wythe walls. Therefore, for Walls 24 and 65-1 the use of the single wythe assumption to represent double wythe walls results in a conservative evaluation regardless of how out-of-plane drift effects are incorporated.

5 CONCLUSIONS

Two walls were selected to demonstrate that the use of the single wythe assumption for multiple wythe walls results in a conservative evaluation with respect to frequency shift and out-of-plane drift considerations. The results indicate that the frequency of the double wythe walls are almost twice those of the equivalent single wythe wall. Therefore, from frequency shift considerations the use of the single wythe assumption is conservative. The impact on out-of-plane drift considerations depends on how this is incorporated in the analysis. With the method used in the re-evaluation criteria no forces are induced in the wall due to out-of-plane drift effects and the maximum stress ratios from other out-of-plane forces in the double wythe wall are approximately one-half of those in the single wythe wall. If fixed boundary conditions are used and out-of-plane drift effects are included, then the maximum stress ratios in the double wythe walls were 1 to 130 percent less than those in the single wythe wall.

The single wythe assumption is therefore conservative for the procedures specified in the re-evaluation criteria and is reasonably conservative for the procedure of including out-of-plane drift effects specified in Report No. R553.11.

TABLE 1

FREQUENCY OF WALLS WITH SIMPLY SUPPORTED BOUNDARY CONDITIONS

Wall No.	Thickness	Wythes	Frequencies (Hz)
24	12	1	15.39, 37.72
	25	2	32.06, 78.57
65 - 1	6	1	15.28, 24.99, 43.97
	12	2	30.56, 49.98, 87.95

TABLE 2

MAXIMUM STRESS RATIOS OF WALLS WITH SIMPLY SUPPORTED BOUNDARY CONDITIONS

Wall No.	Thickness	Wythes	M_x/M_{xa}	M_y/M_{ya}
24	12	1	.4057	.1699
	25	2	.1809	.0751
65-1	6	1	.4834	.1572
	12	2	.2146	.0697

Note: Subscripts x and y denote stress ratios on horizontal and vertical strips respectively

TABLE 3

MAXIMUM STRESS RATIOS OF WALLS WITH FIXED BOUNDARY CONDITIONS INCLUDING OUT-OF-PLANE DRIFT EFFECTS

Wall No.	Thickness	Wythes	M_x/M_{xa}	M_y/M_{ya}
24	12	1	0.265	0.228
	25	2	0.129	0.226
65-1	6	1	0.264	0.136
	12	2	0.117	0.097

Note: Subscripts x and y denote stress ratios on horizontal and vertical strips respectively

PROCEDURE TO ACCOUNT FOR OUT-OF-PLANE INTERSTORY DRIFT EFFECTS ON THE MASONRY WALL EVALUATION

Prepared for

Point Beach Nuclear Power Plant, Units 1 and 2

WISCONSIN ELECTRIC POWER COMPANY

Milwaukee, Wisconsin

Prepared by

COMPUTECH ENGINEERING SERVICES, INC.

Berkeley, California

September, 1981

REPORT NO. R553.11

Revision 1

TABLE OF CONTENTS

1 INTRODUCTION

The Nuclear Regulatory Commission (NRC) staff on June 9-11, 1981 reviewed the criteria and calculations performed on IE Bulletin 80-11 "Masonry Wall Design" for the Point Beach Nuclear Power Plant. Action Item 11 resulting from the review meeting stated that the licensee shall provide the criteria and procedures to account for the out-of-plane interstory drift effects in seismic analysis.

This short report is in response to that action item and includes a description of the analysis methodology, the results, a discussion of the results, and the conclusions.

2 ANALYSIS METHODOLOGY

All walls were analyzed in accordance with the procedures given in "Criteria for the Re-evaluation of Concrete Masonry Walls for The Point Beach Nuclear Power Plant." Specifically, to assess the out-of-plane response of the walls the boundary conditions at all supports were assumed to be pinned. This assumption was made for the following reasons.

(1) The stresses resulting from out-of-plane seismic load are conservative.

(2) The boundary rotations required for the existence of pinned supports are very small and will exist regardless of what type of fixity is used to prevent it. A field inspection of the walls indicated this was the only reasonable assumption.

(3) The majority of the walls had no support at the top of the wall. In addition no support was assumed around any door or other opening.

In assessing the effect of out-of-plane drift effects on the walls the same assumption of pinned boundary conditions was used for consistency in the analytical procedures. With this assumption no forces are induced in the walls when out-of-plane interstory drift effects are assessed. The validity of this assumption was questioned in Action Item 11 and as a result two walls (Wall Nos. 24 and 65-1) were selected to compare the results that would be obtained if fixed rather than simply supported boundary conditions had been assumed. Wall No. 24 is 164 inches long and 156 inches high and Wall No. 65-1 is 109 inches long and 176 inches high. Each wall is double wythe and was analyzed as a single and double wythe wall with both fixed and simply supported boundary conditions. For the fixed boundary conditions the effects of out-of-plane interstory drift were included in the analysis. The stresses resulting from out-of-plane seismic load were combined absolutely with those resulting from out-of-plane drift effects. Out-of-plane drift effects were calculated by imposing the out-of-plane displacement at the top of the wall with the wall fixed against rotation at both the top corners and the bottom of the wall.

3 RESULTS

A summary of the maximum stress ratios resulting from the eight analyses performed are given in Table 1. Care must be exercised in evaluating the results because the maximum stresses do not fall in the same region of the wall for the different boundary conditions. In the case of the simply supported boundary conditions the maximum stress ratios are towards the center of the wall and for the fixed boundary conditions they are close to or adjacent to the boundaries.

4 DISCUSSION OF RESULTS

The results for Wall 65-1 indicate that the maximum stress ratios of the simply supported boundary conditions for a single wythe wall are conservative when compared with those of the fixed boundary conditions that include out-of-plane interstory drift effects. The maximum stress ratio for this wall occurred on a horizontal strip and was reduced by almost one-half when fixed boundary conditions were used. When the same comparison is made for the double wythe wall, again the maximum stress ratio, which was on a horizontal strip, was reduced by almost one-half. However, the maximum stress ratio on the vertical strip increased from 0.07 to 0.097. In comparing the three analyses: 1) single wythe with fixed boundaries and out-of-plane drift, 2) double wythe with simply supported boundaries and 3) double wythe with fixed boundaries and out-of-plane drift; the maximum stress ratios are all less than the ratios obtained from analyzing the wall with the procedures given in the re-evaluation criteria.

The results for Wall 24 are similar to those of Wall 65-1 in that the maximum stress ratios on a horizontal strip decrease when fixed boundary conditions and out-of-plane drift are considered. However, on a vertical strip the maximum stress ratios increase for Wall 24. The increase for the single wythe wall is from 0.170 to 0.228 and for the double wythe wall it is from 0.075 to 0.226. Although this increase for Wall 24 indicates that for a vertical strip, the simply supported boundary conditions is non-conservative the governing stress ratio on the horizontal strip is not exceeded by the maximum stress ratios in either direction for the other three analyses. Therefore, in this regard the assumption of simply supported boundary conditions used in the re-evaluation criteria can be considered conservative.

5 CONCLUSIONS

The criteria and procedures used to account for out-of-plane interstory drift effects in the re-evaluation criteria have been described and compared with results obtained from an alternate approach. In summary, the procedure used in the re-evaluation criteria assumes the walls have pinned supports at the boundaries because it is our opinion that this is the most realistic representation of field conditions and in addition it results in a conservative estimate of the stresses resulting from out-of-plane seismic load.

Two walls were selected to compare the maximum stress ratios obtained from the assumptions used in the re-evaluation criteria with those obtained from using the assumption that the top and bottom boundaries had fixed supports.

The stresses obtained for the out-of-plane forces acting on the wall with fixed boundary conditions were added absolutely to those resulting from out-of-plane drift effects.

For the two walls that were analyzed, the maximum magnitude of the stress ratio was obtained for the single wythe wall with simply supported boundary conditions. In both cases this was the stress on a horizontal strip. In this regard the assumptions used in the re-evaluation criteria are conservative. For Wall 65-1 the maximum stress ratio on both a horizontal and vertical strip obtained from the single wythe wall with simply supported boundary conditions were conservative. For Wall 24 the same statement is valid for the maximum stress ratio on a horizontal strip. For the maximum stress ratio on a vertical strip there was an increase when fixed boundary conditions were used but this increase did not exceed the maximum stress ratio obtained on the horizontal strip.

From the results presented it is clear that the impact of different boundary conditions varies and is difficult to accurately assess because the region where the maximum stress occurs changes as the boundary condition changes. However, for the two walls analyzed the assumption of a single wythe wall with simply supported boundary conditions (i.e. that used in the re-evaluation criteria) produces the maximum magnitude in the stress ratio on either a horizontal or vertical strip. Furthermore, it is our opinion that these boundary conditions are the most realistic for the conditions that exist in the field.

TABLE 1

MAXIMUM STRESS RATIOS FOR VARYING
BOUNDARY CONDITIONS

Wall No.	Thickness (in)	Condition	M_x/M_{xa}	M_y/M_{ya}
24	12	Simply Supported	0.406	0.170
	12	Fixed, Drift	0.265	0.228
	25	Simply Supported	0.181	0.075
	25	Fixed, Drift	0.129	0.226
65-1	6	Simply Supported	0.483	0.157
	6	Fixed, Drift	0.264	0.136
	12	Simply Supported	0.215	0.070
	12	Fixed, Drift	0.117	0.097

Note: Subscripts x and y denote stress ratios on horizontal and vertical strips respectively

MODAL CONTRIBUTION

TO

DYNAMIC ANALYSIS RESULTS

Prepared for

Point Beach Nuclear Plant, Units 1 and 2

WISCONSIN ELECTRIC POWER COMPANY

Milwaukee, Wisconsin

Prepared by

COMPUTECH ENGINEERING SERVICES, INC.

Berkeley, California

July, 1981

REPORT NO. R553.02

7pp

TABLE OF CONTENTS

1 INTRODUCTION

The Nuclear Regulatory Committee (NRC) staff on June 9-11, 1981 reviewed the criteria and calculations performed on IE Bulletin 80-11 `` Masonry Wall Design '' for the Point Beach Nuclear Power Plant. Action Item 2 resulting from the meeting stated that the licensee will provide documentation including calculation sheets that indicate that the adoption of five (5) modes of seismic response will generally provide 95 % of the total response.

This short report describes the four walls that were selected to document the results, the analyses that were performed, the results of the analyses and a discussion of results and conclusions. Appendices A through D provide summaries of the computer output from which the analysis results were obtained.

2 SELECTION OF WALLS

To provide a cross-section of the boundary conditions and openings of the masonry walls at the Point Beach Nuclear Power Plant, the four walls given in Table 1 were selected to document the results of the analyses.

The four walls include two with door openings and two without openings. Two of the walls are not connected at their top boundary but are pinned on the other three boundaries, one wall is pinned on all four boundaries and the fourth wall is pinned on only two boundaries.

TABLE 1 DESCRIPTION OF WALLS

Wall Number	Wall Thickness (in)	Boundary Conditions	Openings
5-29/2A	8	Pinned on 3 sides Free on top	Door on one side
20/9	8	Pinned on 4 sides	Door on one side
65-1/15	12	Pinned on 3 sides Free on top	None
64-E/15	42	Pinned on 2 sides Top and one side free	None

3 ANALYSIS METHODOLOGY

All walls were analyzed in accordance with the procedures given in `` Criteria for the Re-evaluation of Concrete Masonry Walls for the Point Beach Nuclear Power Plant ''. Specifically, plate analysis was used to assess the out-of-plane response of all walls. The computer program SAP was used to perform a finite element dynamic analysis ultilizing the response spectrum method. The Computech pre-processor program GENIN was used to generate input files for the analyses.

For each wall an eigenanalysis was carried out to extract the first five frequencies and mode shapes and the individual modal responses were combined using the square root of the sum of the squares procedure.
The SAP output was summarized using the post-processor computer program GENOUT. The computer printout of GENOUT lists seperately the values of the moments and reaction forces for first mode dynamic response, the SRSS of the first five modes of dynamic response and the values from static loads. Also given is the absolute sum of the static and SRSS dynamic response values which are the values used to assess the adequacy of the walls.

4 ANALYSIS RESULTS

The detailed results of the analyses of the four walls are given in Appendices A through D. A summary of the results is given in Tables 2 and 3. Table 2 contains a summary of results for walls 5-29/2A, 20/9, and 64-E/15. Table 3 contains a similar summary for wall 65-1/15. For each wall an analysis was performed assuming the wall was either grouted or ungrouted. The maximum moment parallel (Mx) and normal (My) to the bed joints and the maximum boundary shear force (F) are given in Tables 2 and 3. In Table 2 the value of each moment or force resulting from the first mode and the SRSS of the first five modes are given. Table 2 also contains the percentage contribution of the first mode to the SRSS of the first five modes for each maximum moment and force quantity.

Table 3 which contains a summary of results for wall 65-1/15 includes the results from the first mode, the SRSS of the first five modes and the SRSS of the first ten modes. In addition the Table also contains the percentage contribution of the first mode to the SRSS of both the first 5 and 10 modes respectively for each maximum moment and force quantity. Also included is the percentage of the SRSS of the first 5 modes to the first 10 modes for each maximum moment and force quantity.

5 DISCUSSION OF RESULTS

The results of the analyses for walls 5-29/2A. 20/9. 64-1/15 presented in Table 2 indicates that the first mode contributes between 97.9 % and 99.9 % of the SRSS of the first five modes for the maximum moments and boundary forces in the walls. Thus for the openings and boundary conditions of these three walls a first mode analysis would have been sufficient to produce 95 % of the total maximum response quantities. A five mode SRSS analysis is therefore adequate for these three walls.

The results for Wall 65-1/15 presented in Table 3 indicate that the first mode contributes 90 % of the SRSS of the first 5 modes of response and 90 % of the SRSS of the first 10 modes of response. The percentage of SRSS of the first 5 modes to the SRSS of the first 10 modes is 99.9 % for the maximum moments and forces. Thus for a wall pinned on three sides and free at the top a first mode analysis would not have been adequate. However, it is clear that an SRSS of five modes is always equivalent to an SRSS of ten modes.

6 CONCLUSIONS

Four walls were selected to provide documentation that indicates that the adoption of five (5) modes of response will generally provide 95 % of the total response. The four walls selected. covered the full range of boundary conditions and openings found in the masonry walls at the plant.
The results clearly indicate that five modes of SRSS response provide 99.9 % of the total response. In fact for three of the four walls the first mode of response provided 97.9 % or greater of the total response.

As all the masonry walls at the Point Beach plant were analyzed using the first five modes of response it is clear that 99 % of the total response of all the walls has been included in the analytical results.

TABLE 2 SUMMARY OF RESULTS FOR WALLS 5-29/2A, 20/9, AND 64-E/15

Wall Number	Mx1 (Lb-in/in)	Mx5 (Lb-in/in)	Mx1/Mx5 %	My1 (Lb-in/in)	My5 (Lb-in/in)	My1/My5 %	F1 (Lb)	F5 (Lb)	F1/F5 %
5-29/2A									
Grouted	155.4	155.4	100.0	51.66	51.71	99.9	79.93	80.07	99.8
Ungrouted	83.32	83.32	100.0	27.69	27.72	99.9	42.85	42.92	99.8
20/9									
Ungrouted	142.2	142.2	100.0	99.06	100.3	98.8	54.49	55.16	98.8
Part. Grout	173.9	173.9	100.0	121.2	122.7	98.8	66.64	67.46	98.8
64-E/15									
Grouted	84.21	85.96	97.9	101.8	103.2	98.6	229.6	231.1	99.4
Ungrouted	43.61	44.58	97.8	52.75	53.51	98.6	118.9	119.8	99.2

Notations:

M : Maximum moment per linear length
F : Maximum shear force at boundary
x : Parallel to the bed joint
y : Normal to the bed joint
1 : First mode only
5 : SRSS of the first five modes

TABLE 3 SUMMARY OF RESULTS FOR WALL 65-1/15

Wall Number	Moment or Force	1st Mode	SRSS 5 Modes	SRSS 10 Modes	1st/5 %	1st/10 %	5/10 %
65-1/15							
Grouted	Mx (Lb-in/in)	172.6	174.5	174.6	98.9	98.8	99.9
	My (Lb-in/in)	27.43	30.26	30.27	90.6	90.6	99.9
	F (Lb)	77.35	84.87	84.98	91.1	91.0	99.9
Ungrouted	Mx (Lb-in/in)	109.0	110.1	110.2	99.0	98.9	99.9
	My (Lb-in/in)	17.31	19.10	19.10	90.6	90.6	100.0
	F (Lb)	48.83	53.56	53.63	91.2	91.0	99.9

5

Notations:

M : Maximum moment per linear length
F : Maximum shear force at boundary
x : Parallel to the bed joint
y : Normal to the bed joint
1 : First mode only
5 : SRSS of the first five modes
10 : SRSS of the first ten modes

MASONRY TEST PROCEDURES

FOR

POINT BEACH NUCLEAR POWER PLANT

Prepared for

Point Beach Nuclear Plant, Units 1 and 2

WISCONSIN ELECTRIC POWER COMPANY

Milwaukee, Wisconsin

Prepared by

COMPUTECH ENGINEERING SERVICES, INC.

Berkeley, California

June, 1981

REPORT NO. R553.01

34 pp

TABLE OF CONTENTS

1 INTRODUCTION

The Nuclear Regulatory Commission (NRC) staff on June 9-11, 1981 reviewed the criteria and calculations performed on IE Bulletin 80-11 ``Masonry Wall Evaluation'' for the Point Beach Nuclear Power Plant. One of the action itams that resulted from the review meeting was the performance of masonry prism and block tests to validate the use of the special inspection allowable stresses used in the criteria. This document describes the tests to be performed, the number of prism and block samples to be taken from each wall, the method of extracting the prism and block samples and the methods to be used to obtain both the prism and mortar compressive strengths.

2 NUMBER AND TYPE OF PRISM AND BLOCK SAMPLES

For each wall, two prisms at least 8 inches long and 24 inches high will be extracted from the wall at locations to be determined by Wisconsin Electric Power Company (WEPCO). The 8 inch length of the prism must include two of the three webs of the 16 inch long masonry units. Extreme care shall be exercised in extracting the prism sample from the wall to avoid breaking the bond between mortar and masonry unit. In multi-wythe walls it maybe necessary to cut through the full width of the wythes if the single wythe prism sample is well bonded to the adjacent wythe. This decision will be made by WEPCO and/or consultants at the plant.

For each wall, two masonry units 16 inches long and 8 inches high will be extracted from the wall at convenient locations to be determined by WEPCO.

The five walls from which the above samples shall be extracted are as follows.

1)	Stairwell	1 Wythe,	6" Thick
2)	Wall 68	1 Wythe,	8" Thick
3)	Wall 150	3 Wythes,	18" Thick
4)	Wall 65	2 Wythes,	12" Thick
5)	Wall 104	4 Wythes,	33" Thick

3 METHOD OF EXTRACTION

The prism and block samples shall be extracted from each wall using a method of extraction approved by WEPCO and/or consultants. Care shall be exercised to minimize the amount of dust and water resulting from the extraction procedure.

4 TRANSPORTATION OF PRISMS AND MASONRY UNITS

The prisms shall be precompressed by a method approved by WEPCO and/or consultants after they are extracted from the wall. They shall remain upright during all handling and transportation operations. The method of storing the prisms for transportation from the plant to the testing laboratory shall ensure they remain upright and subjected to a minimum amount of vibration. The method to be used shall be approved by WEPCO and/or consultants. Extreme care shall be exercised at all stages to avoid cracking between the mortar and masonry unit.

The masonry units shall be transported in a manner such that the units will not be damaged. The method to be used shall be approved by WEPCO and/or consultants.

5 TEST METHODS

Each prism compressive test shall be performed in accordance with applicable sections of ASTM E-447-74 . The stress-strain curve is not required as part of the test procedure. One undamaged prism sample shall be tested from each wall.

Each masonry unit compressive test shall be performed in accordance with applicable sections of ASTM E-140-75. One undamaged unit shall be tested from each wall.

6 DETERMINATION OF COMPRESSIVE STRENGTHS

The compressive strength f'm of one masonry prism from each wall shall be determined in accordance with Sec. 7.1.6. of ASTM E-447-74. The value of f'm to be used in the criteria for IE Bulletin 80-11 shall be the average value of the five specimens tested or 125% of the minimum value determined by test, whichever is less, but in no case shall it exceed 1000 psi. If the scatter of the five prisms is considered to be excessive by the NRC staff, consideration will be given to conducting two additional tests.

The compressive strength of the mortar shall be determined as follows. The net compressive strength of one masonry unit from each wall shall be determined in accordance with Sec. 10.3 of ASTM C140-75. The compressive strength of the masonry units shall be the average value of the five specimens tested or 125% of the minimum value determined by test. Using this value for masonry units and the value of f'm determined by the same procedure, the type of mortar shall be deduced from Table 1 (i.e. Table 4.3 of ACI 531-79) by interpolation. The compressive strength of the mortar, mo, shall then be determined from Table 2 (i.e. Table 1 of ASTM C270-73) by interpolation. If the compressive strength of the mortar determined by this procedure exceeds 750 psi then 750 psi shall be used as the compressive strength of the mortar in the criteria for IE Bulletin 80-11.

TABLE 1 VALUE OF f'm FOR MASONRY

Compressive test strength of masonry units, psi, on the net cross-sectional area	Compressive strength of masonry f'm, psi	
	Type M and S mortar	Type N mortar
6000 or more	2400	1350
4000	2000	1250
2500	1550	1100
2000	1350	1000
1500	1150	875
1000	900	700

TABLE 2 COMPRESSIVE STRENGTH OF CUBES FOR MORTAR TYPES

Mortar Type†	Average Compressive Strength at 28 Days, psi(Mpa)
M	2500(17.2)
S	1800(12.4)
N	750(5.2)
O	350(2.4)
K	75(0.5)

†Mortar type designations A-1, A-2, B, C, and D are the former type designations in effect prior to 1954.

ACTION ITEM 3

With respect to the containment isolation valves for steam generator blowdown lines outside containment at El. 26', the licensee indicates that additional remote-operated isolation valves within containment will be installed by fall 1982 and spring 1983 for Units 1 and 2, respectively; thus, eliminating the concern originating from the potential failure of the stairwell masonry walls. Licensee is requested to provide a commitment that installation will be completed by the above dates and a technical evaluation that the delay will be acceptable from a probabilistic standpoint.

Licensee will also review the Plant safety aspects associated with failure of the steam generator blowdown isolation valves due to masonry wall failure. If a safety concern exists, it shall be identified and evaluated and submitted with the above technical evaluation.

RESPONSE

Licensee has received and evaluated proposals for the blowdown isolation valves inside containment. A recommended supplier has been selected and it is expected that a purchase order will be issued by August 1, 1981. The drawing and final design review, fabrication, and delivery time requirement for these valves is estimated to be 48 weeks from the date of the purchase order. This schedule will facilitate receipt and installation of these valves in Point Beach Unit 1 during the fall 1982 refueling and in Point Beach Unit 2 during the spring 1983 refueling. We, therefore, expect to install these valves by these dates.

The original Plant design criteria for the blowdown line containment penetration are presented in FFDSAR, Section 5.2.2. The blowdown line penetration is classified as a Class 2 (outgoing lines) penetration and provided wth a single automatic and remotely-operated trip valve located outside containment. Since the blowdown system is considered connected to a closed system inside containment, a second remote isolation valve in the event of a single failure was not provided in the original design.

During normal operation, the failure of these blowdown lines presents no safety concern since adequate makeup water can be provided with the main feedwater pumps and the blowdown lines can be isolated using the manual isolation valves provided. The maximum effective leak diameter of the blowdown system is limited by the minimum drilled passage in the tubesheet for the blowdown connection which is 1.625". This limits the maximum mass flow rate at 1,133 psi steam generator pressure to 4.3×10^5 lbm/hr./steam generator compared to the 3.7×10^6 lbm/hr. capability of each of the main feedwater pumps.

In the event of a design basis accident, such as a loss-of-coolant accident or a main steam line break, together with a loss-of-offsite power and a seismic event, the effect and consequence of the failure of both blowdown lines due to a postulated failure of the facade stairwell wall has not been previously analyzed and could result in safety concerns. These concerns would include maintainability of containment integrity and the ability to maintain long-term cooling of the reactor core through use of the steam generators. It is for these reasons that we have committed to installation of additional blowdown line isolation valves inside containment. Because of the low probability of a blowdown line failure due to a postulated wall collapse concurrent with a design basis accident, this schedule is acceptable.

As discussed in the Point Beach Nuclear Plant Facility Description and Safety Analysis Report (FFDSAR), the northcentral United States, which includes the Plant site location, is a relatively inactive earthquate area. The Coast and Geodetic Survey, Seismic Probability Map of the United States, assigns this area to Zone 0 - No Damage. There is no instrumented or verifiable record of large intensity shocks above Modified Mercalli (MM) intensity VII within 200 miles of the site and no record of damaging earthquakes with epicenters within 100 miles of the site.

During the construction permit licensing stage of the Haven Nuclear Plant, Wisconsin Electric submitted on March 24, 1978, an Appendix 2K to the Haven PSAR Site Addendum. The Appendix presented a seismic risk analysis for the proposed Haven site located approximately thirty miles SSE of the Point Beach Nuclear Plant on the shore of Lake Michigan. Appendix 2K predicted a maximum annual recurrence probability for an intensity level IV (MM) earthquake at the Haven site of 0.00617/year. The observed effects of intensity IV (MM) earthquakes indicate that serious damage to physical structures does not occur. Thus, it is assumed that the facade stairwell walls would remain intact in the event of an earthquake of intensity IV (MM) at the Point Beach Site.

As discussed in the Nuclear Regulatory Commission's Reactor Safety Study, NUREG-75/014, the probability for PWR dominant accident sequences, such as a large LOCA or main steamline break (MSLB), are assessed at 1×10^{-4}/reactor year (Section 5.3.2.1). The report also states in Section 5.4.1 that although it is difficult to predict with precision the probability of potential accidents due to earthquake damage to a nuclear power plant, considering the uncertainties in damage probabilities, it seems reasonable to predict that the risk level lies between 10^{-6} and 10^{-8} per reactor year. At this level of probability, the report concludes that the earthquake-induced accidents should not contribute significantly to reactor accident risks.

We have, therefore, assumed that the probability of a design basis accident together with an earthquake resulting in damage to the blowdown lines from a postulated collapse of the facade stairwell could be conservatively assumed to be the product of the probability of a large pipe rupture, 1×10^{-4}/reactor year, times the recurrence probability for an intensity IV (MM) earthquake which is approximately 6×10^{-3}/reactor year. This results in a predicted probability of less than 6×10^{-7}/reactor year for an unanalyzed failure of the blowdown system coupled with a design basis accident. The actual probability of this combination of events occurring is even smaller since this analysis assumes the probability of a wall collapse in the event of the intensity IV (MM) earthquake and the probability of damage to both blowdown lines in the event the wall collapses are both one. Even at the probability of 6×10^{-7}/reactor year, NUREG 75/014 Figure 2-1 would classify the risk of such an event as negligible and less than the natural hazards mortality rate.

We conclude that this very small risk is acceptable from a probabilistic standpoint over the time period proposed to complete the proposed corrective action. We further conclude that since an acceptable and appropriate course of action has been identified and embarked upon, any additional operability analyses of the facade block wall or analyses of the exact effects or consequences of blowdown isolation valve failures are unwarranted. These activities would serve no purpose other than to confirm or preclude the necessity for the corrective actions already underway.

ACTION ITEM 12

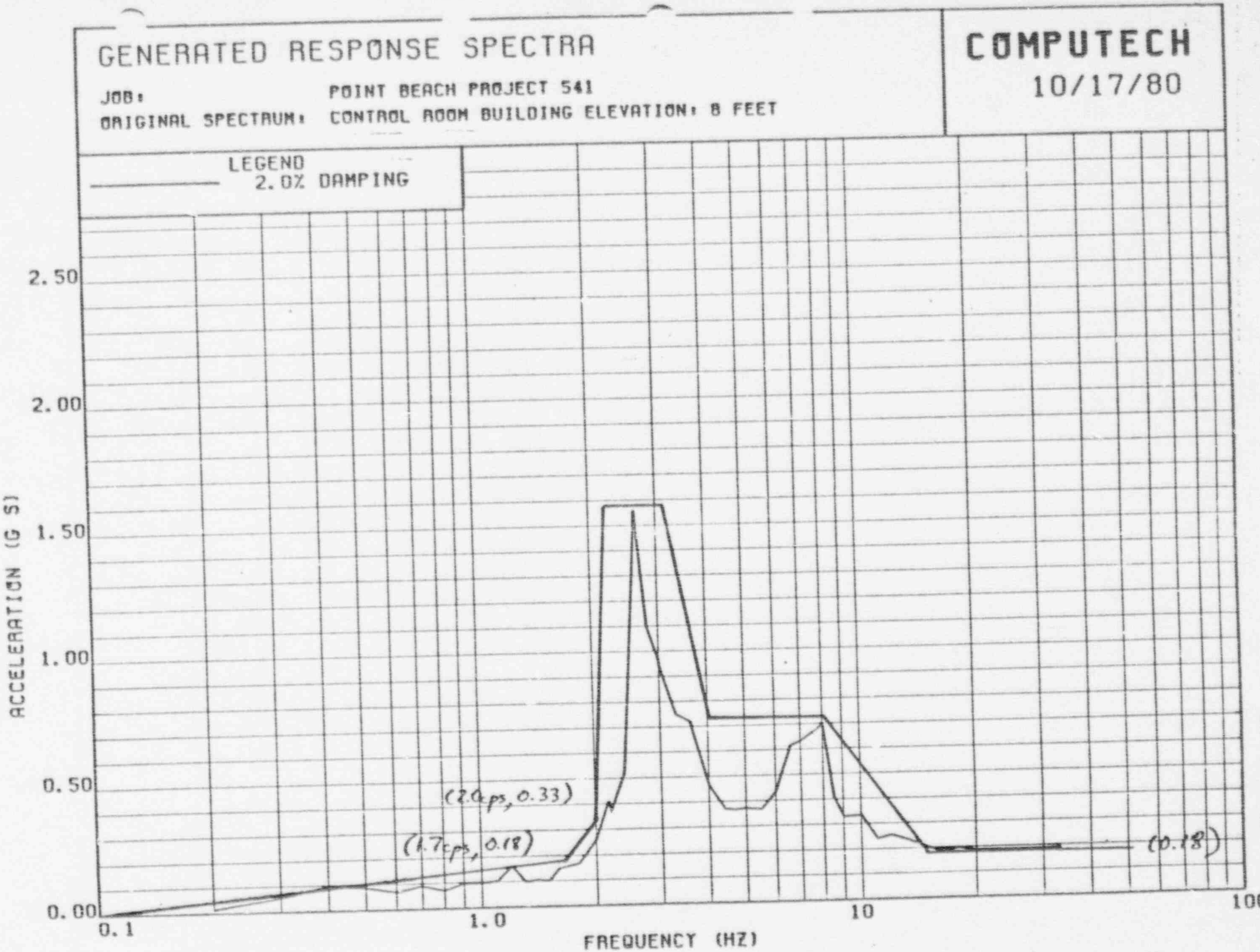
The licensee is required to comply with the requirements of plant operability provisions of the plant Technical Specification as specifically identified in IE Bulletin No. 80-11. The licensee will inform the NRC of its planned action with respect to this item by 07/15/81.

RESPONSE

The Nuclear Engineering Section (NES) Safety Review Committee has conducted an operability analysis of each masonry block wall in which the analyzed SSE induced stress levels exceeded the revised acceptance criteria. Operability has been evaluated against the degree of overstress determined in the analysis of the wall, the consequence of a failure of the wall, and its potential impact on safety related systems and/or components, and the probability of occurrence of conditions resulting in overstressing of the wall prior to reinforcement, modification, or repair of the wall. The NES Safety Review Committee has concluded, for those overstressed walls identified through July 15, 1981, that continued operability of the Point Beach Nuclear Plant pending repairs is justified.

Response to Action Item # 14

2% and 4% floor response spectra used in the masonry wall evaluation.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JOB: POINT BEACH PROJECT 541
ORIGINAL SPECTRUM: CONTROL ROOM BUILDING ELEVATION: 8 FEET
COMPUTECH
10/17/80
LEGEND
2.0% DAMPING
2.50
2.00
1.50
1.00
0.50
0.00
ACCELERATION (G S)
0.1
1.0
10
100
FREQUENCY (HZ)
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(1.7cps, 0.18)
(0.18)

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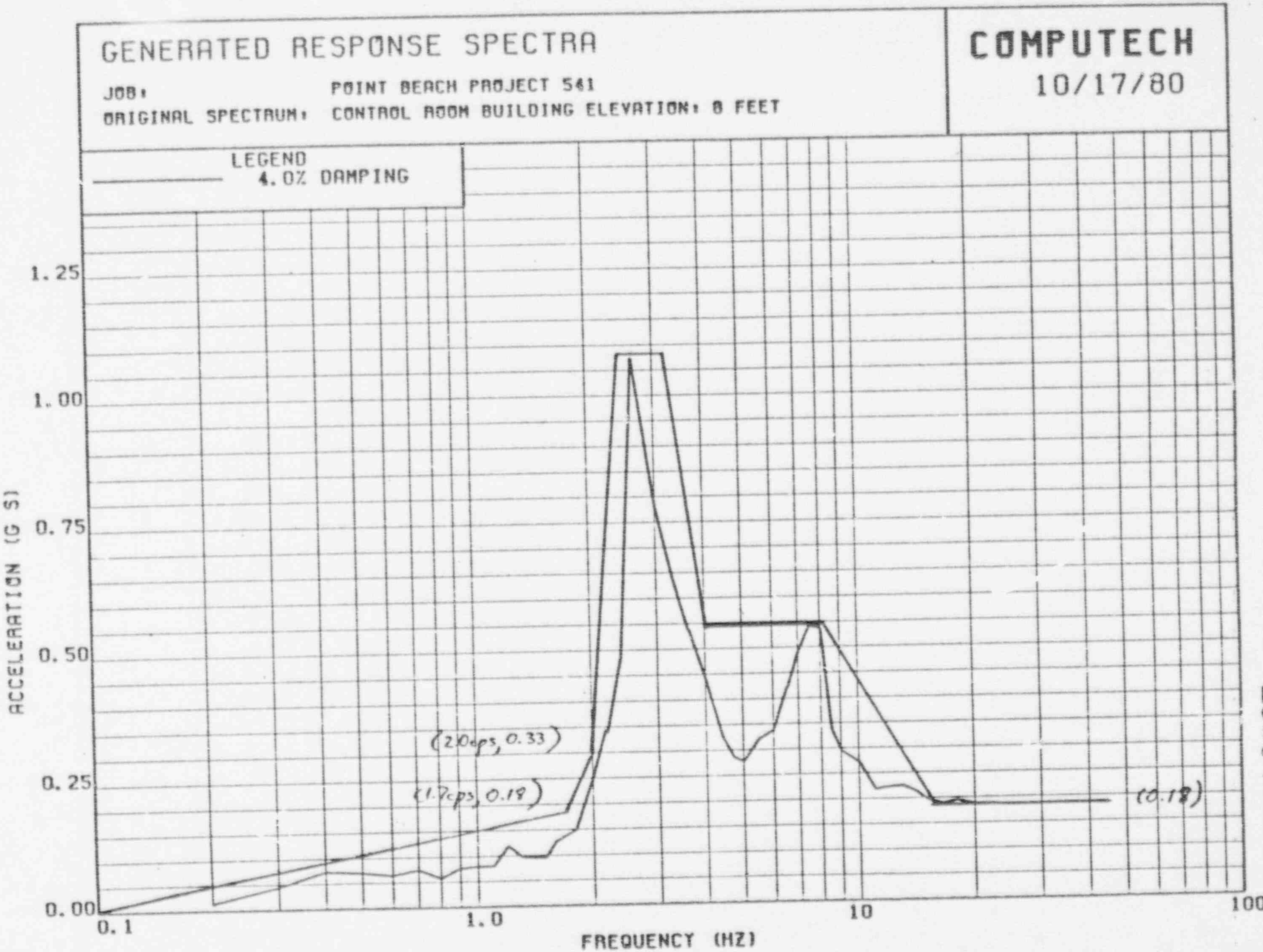
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COMPUTECH
10/17/80
LEGEND
4.0% DAMPING
ACCELERATION (G S)
1.25
1.00
0.75
0.50
0.25
0.00
0.1
1.0
10
100
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(1.7cps, 0.18)
(0.18)

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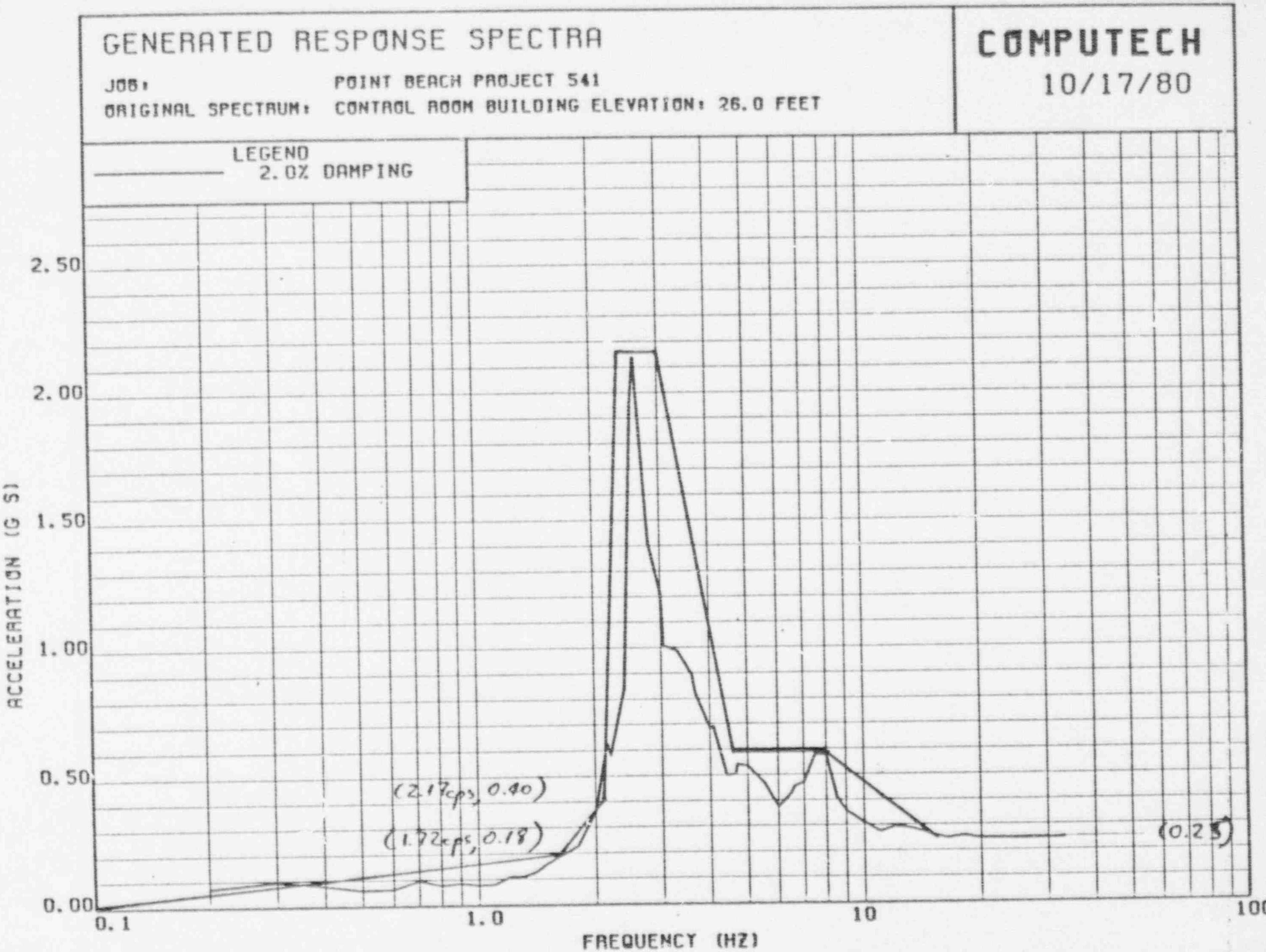

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ORIGINAL SPECTRUM: CONTROL ROOM BUILDING ELEVATION: 26.0 FEET
COMPUTECH
10/17/80
LEGEND
2.0% DAMPING
ACCELERATION (G S)
2.50
2.00
1.50
1.00
0.50
0.00
0.1
1.0
10
100
FREQUENCY (HZ)
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(1.72cps, 0.18)
(0.25)

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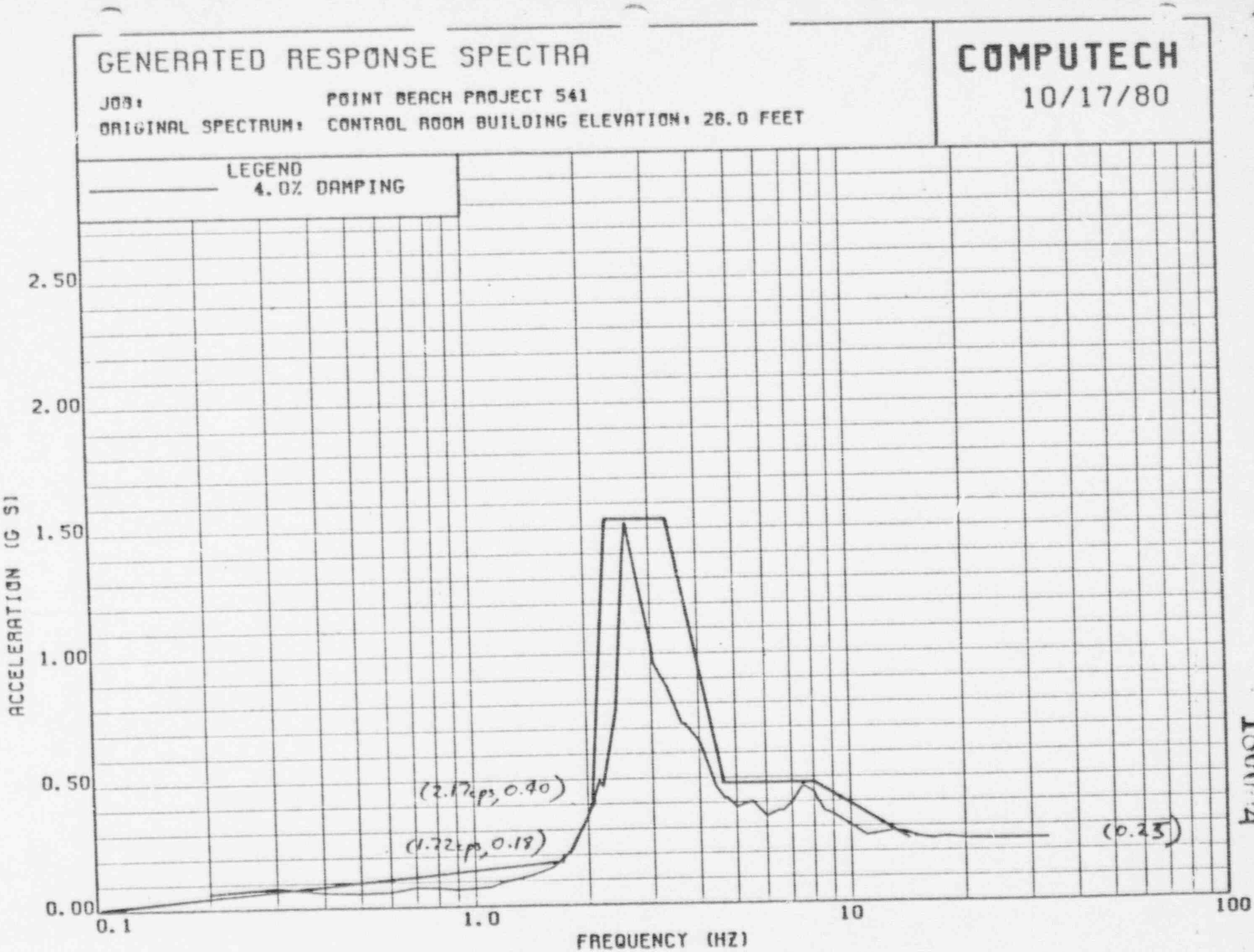
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JOB: POINT BEACH PROJECT 541
ORIGINAL SPECTRUM: CONTROL ROOM BUILDING ELEVATION: 26.0 FEET
COMPUTECH
10/17/80
LEGEND
4.0% DAMPING
ACCELERATION (G S)
0.00
0.50
1.00
1.50
2.00
2.50
FREQUENCY (HZ)
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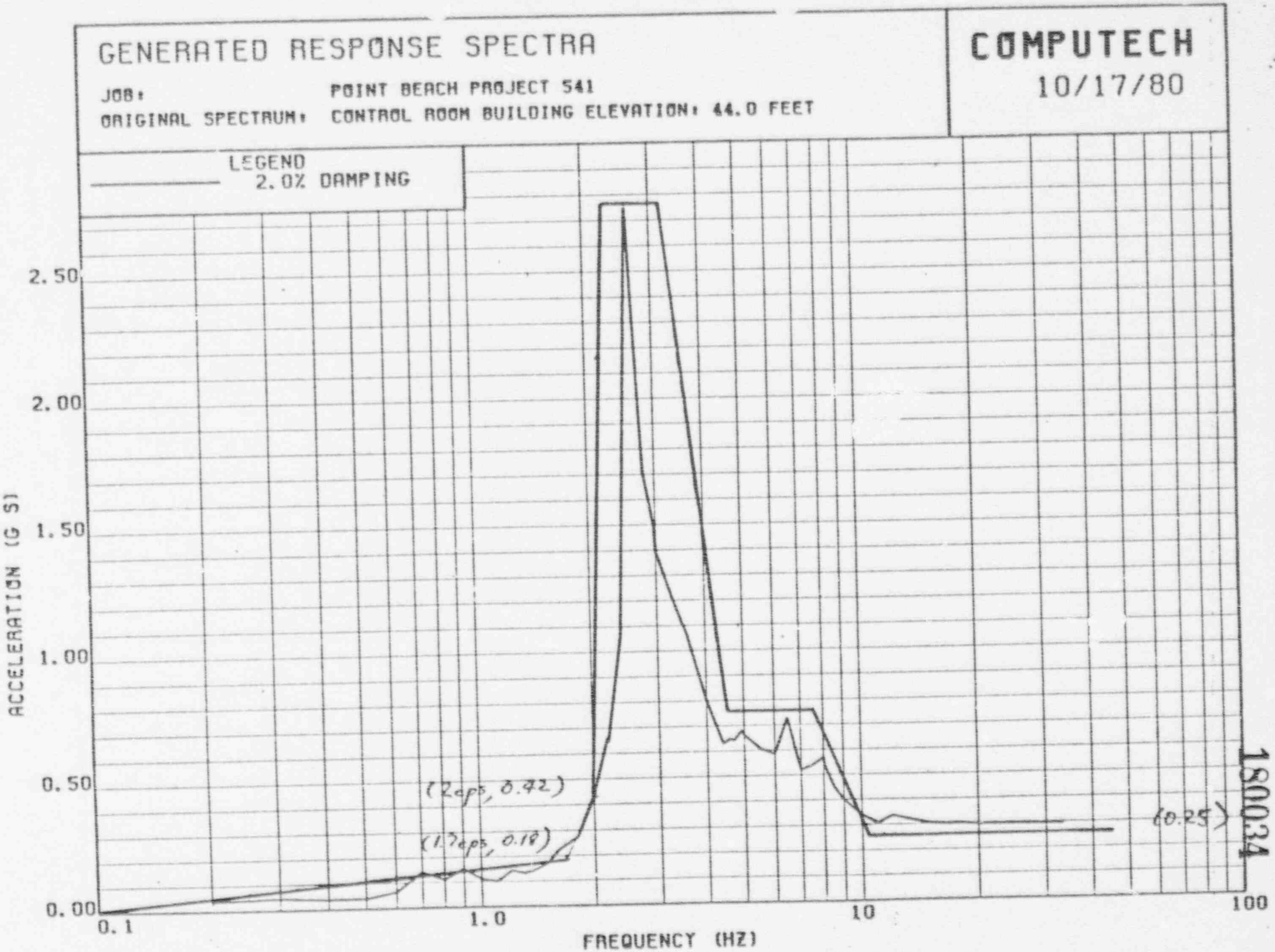
1.0
10
100
(2.17cps, 0.40)
(1.22cps, 0.18)
(0.23)

180034



GENERATED RESPONSE SPECTRA
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ORIGINAL SPECTRUM: CONTROL ROOM BUILDING ELEVATION: 44.0 FEET
COMPUTECH
10/17/80
LEGEND
2.0% DAMPING
ACCELERATION (G S)
0.00
0.50
1.00
1.50
2.00
2.50
FREQUENCY (HZ)
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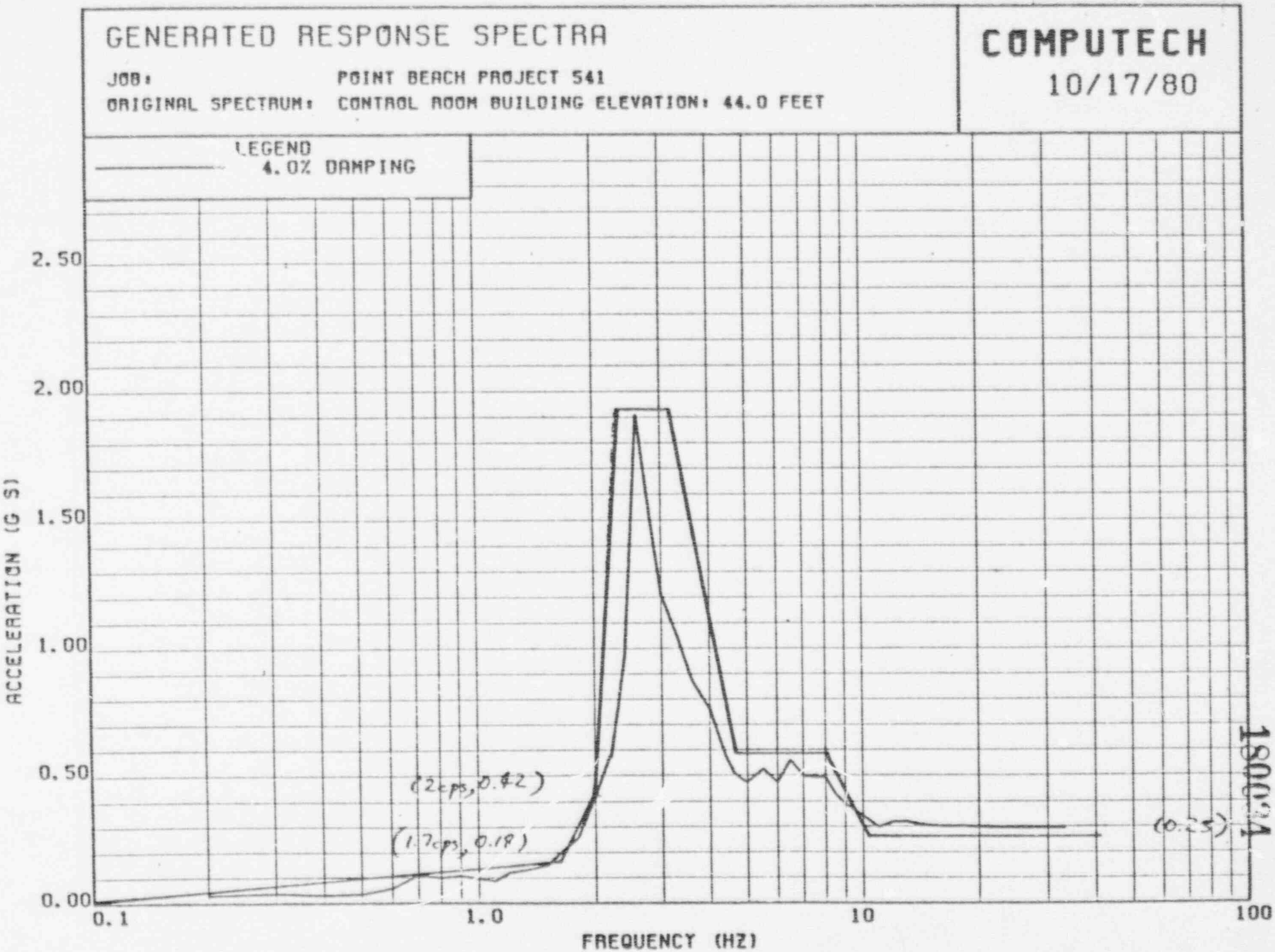
1.0
10
100
(2cps, 0.42)
(1.7cps, 0.18)
(0.25)

1800.34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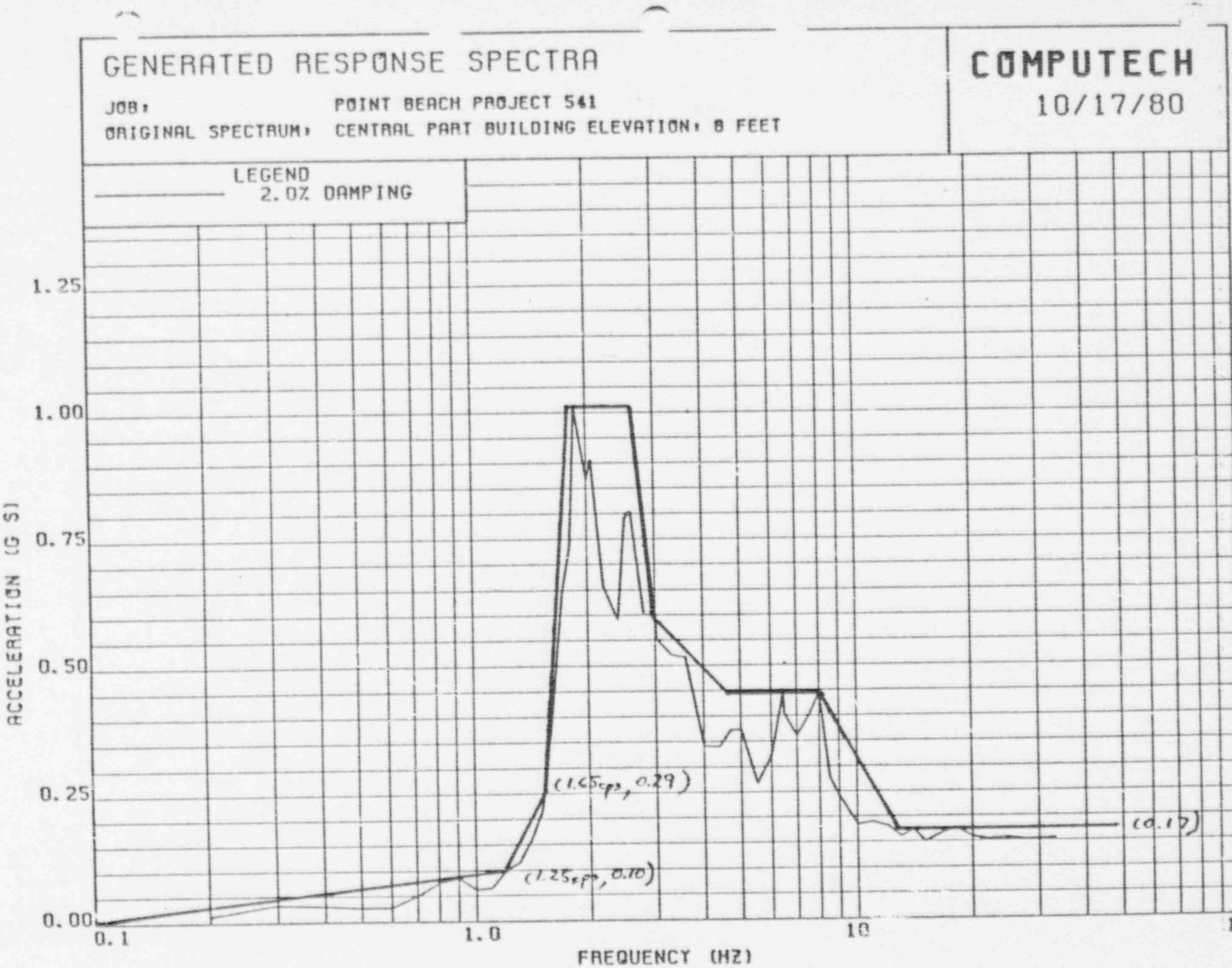
ORIGINAL SPECTRUM: CONTROL ROOM BUILDING ELEVATION: 44.0 FEET
COMPUTECH
10/17/80
LEGEND
4.0% DAMPING
ACCELERATION (G S)
2.50
2.00
1.50
1.00
0.50
0.00
0.1
1.0
10
100
FREQUENCY (HZ)
(2cps, 0.42)
(1.7cps, 0.18)



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COMPUTECH
10/17/80
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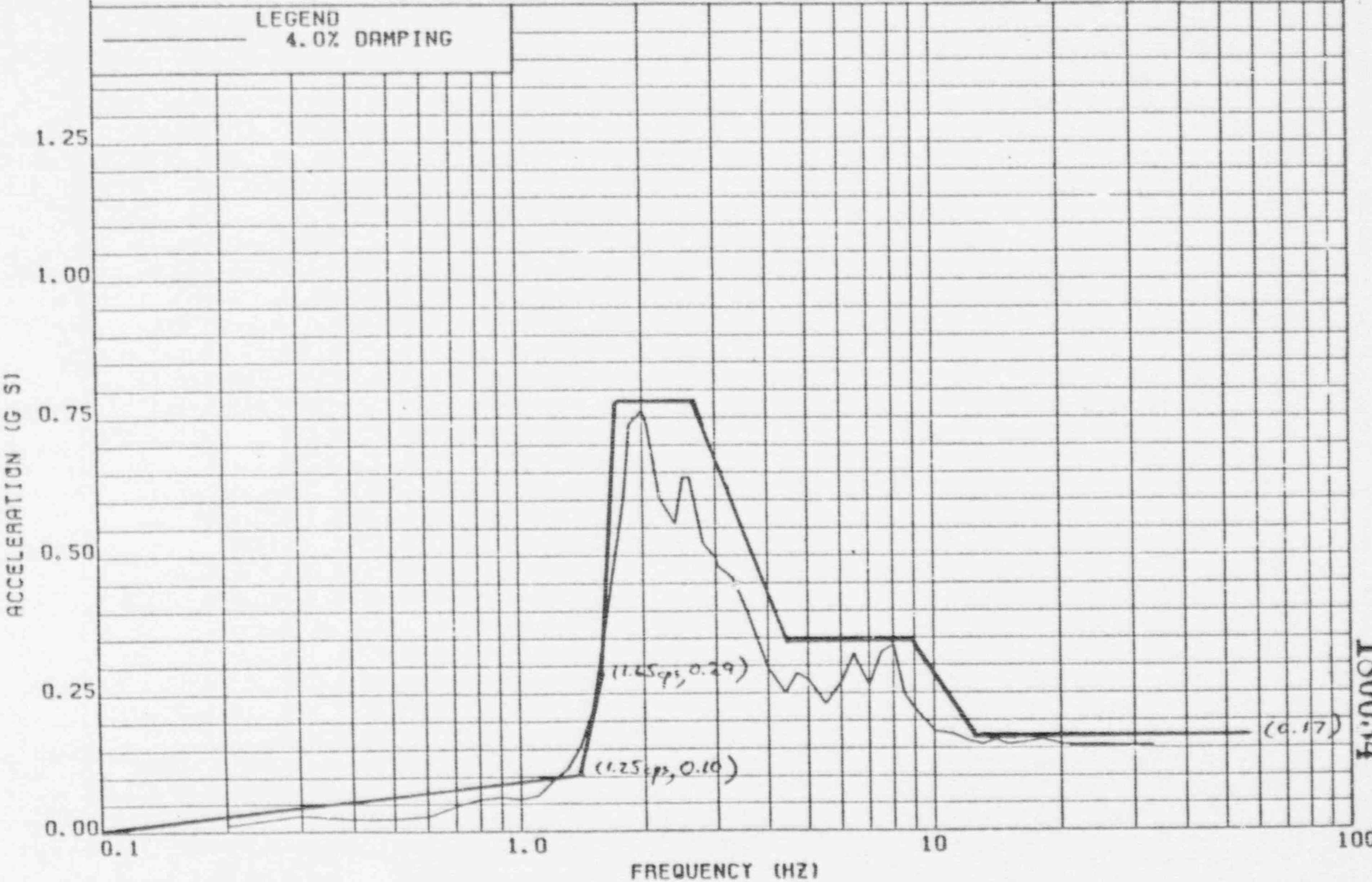
LEGEND
2.0% DAMPING
ACCELERATION (G S)
1.25
1.00
0.75
0.50
0.25
0.00
(1.65cps, 0.29)
(1.25cps, 0.10)
(0.17)
0.1
1.0
10
100
FREQUENCY (HZ)
180034

GENERATED RESPONSE SPECTRA

JOB: POINT BEACH PROJECT 541
ORIGINAL SPECTRUM: CENTRAL PART BUILDING ELEVATION: 8 FEET

COMPUTECH
10/17/80



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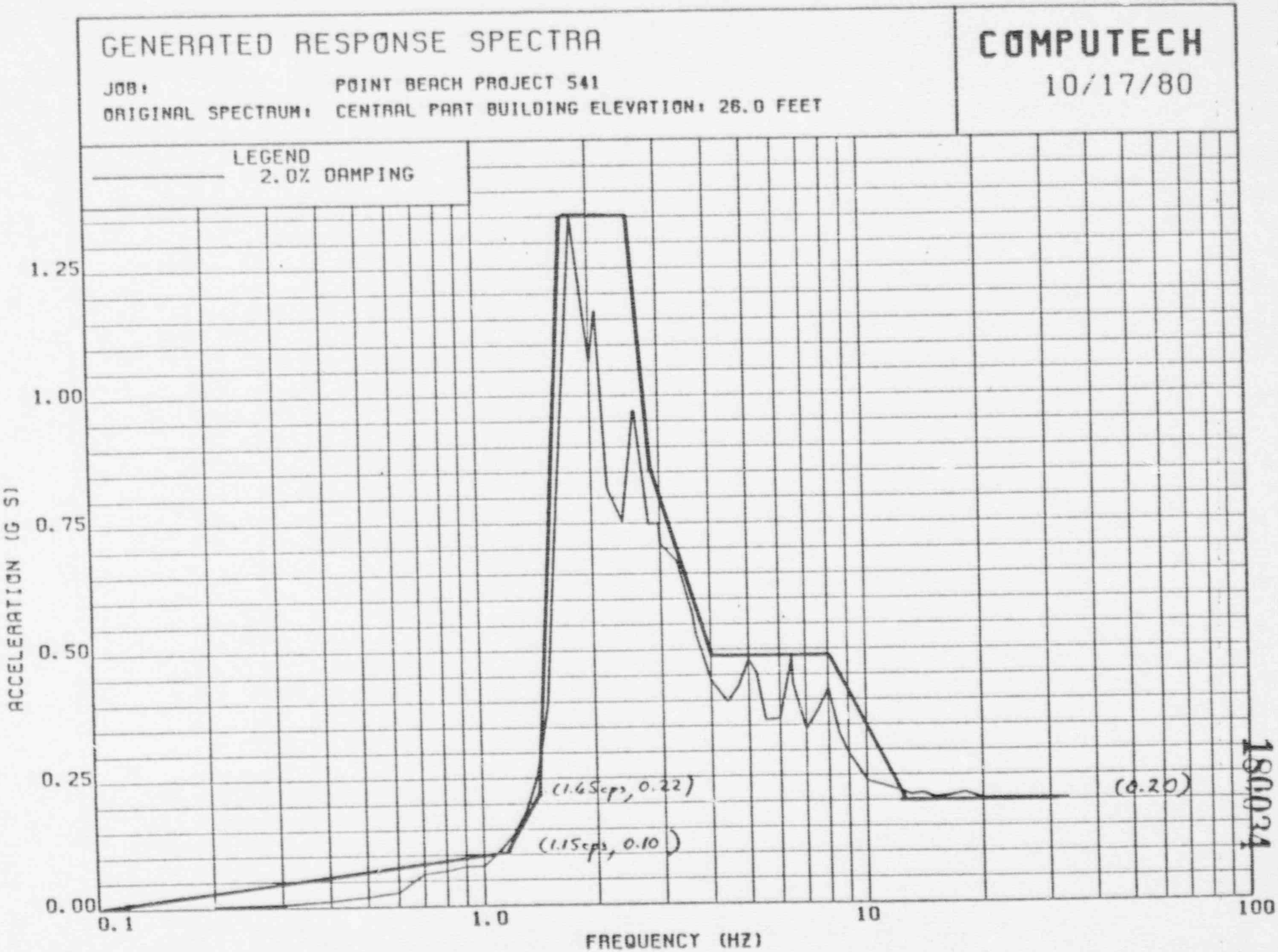
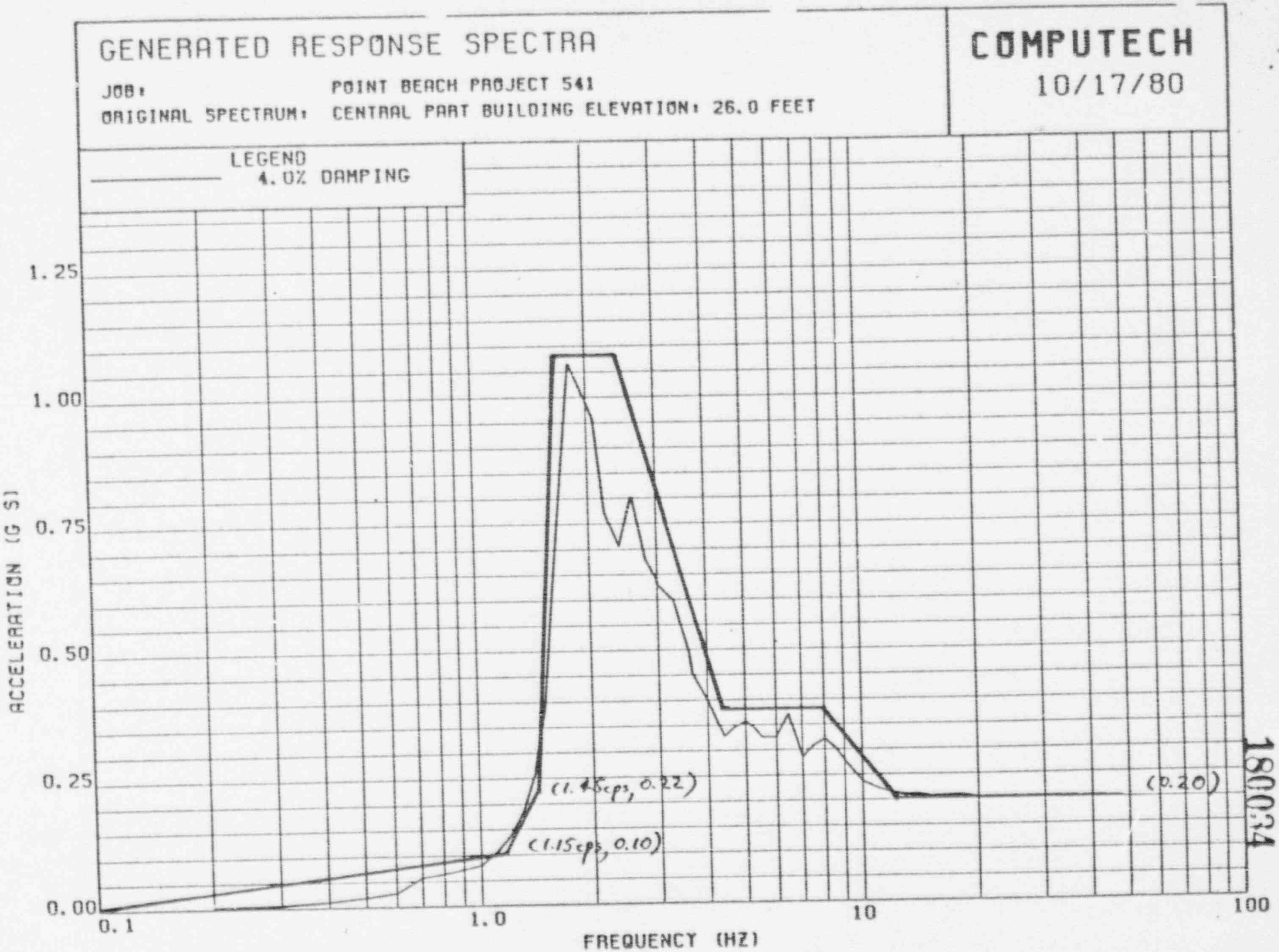


GENERATED RESPONSE SPECTRA
COMPUTECH
10/17/80
JOB: POINT BEACH PROJECT 541
ORIGINAL SPECTRUM: CENTRAL PART BUILDING ELEVATION: 26.0 FEET
LEGEND
2.0% DAMPING
ACCELERATION (G S)
1.25
1.00
0.75
0.50
0.25
0.00
(1.65cps, 0.22)
(1.15cps, 0.10)
(0.20)
180034
0.1
1.0
10
100
FREQUENCY (HZ)



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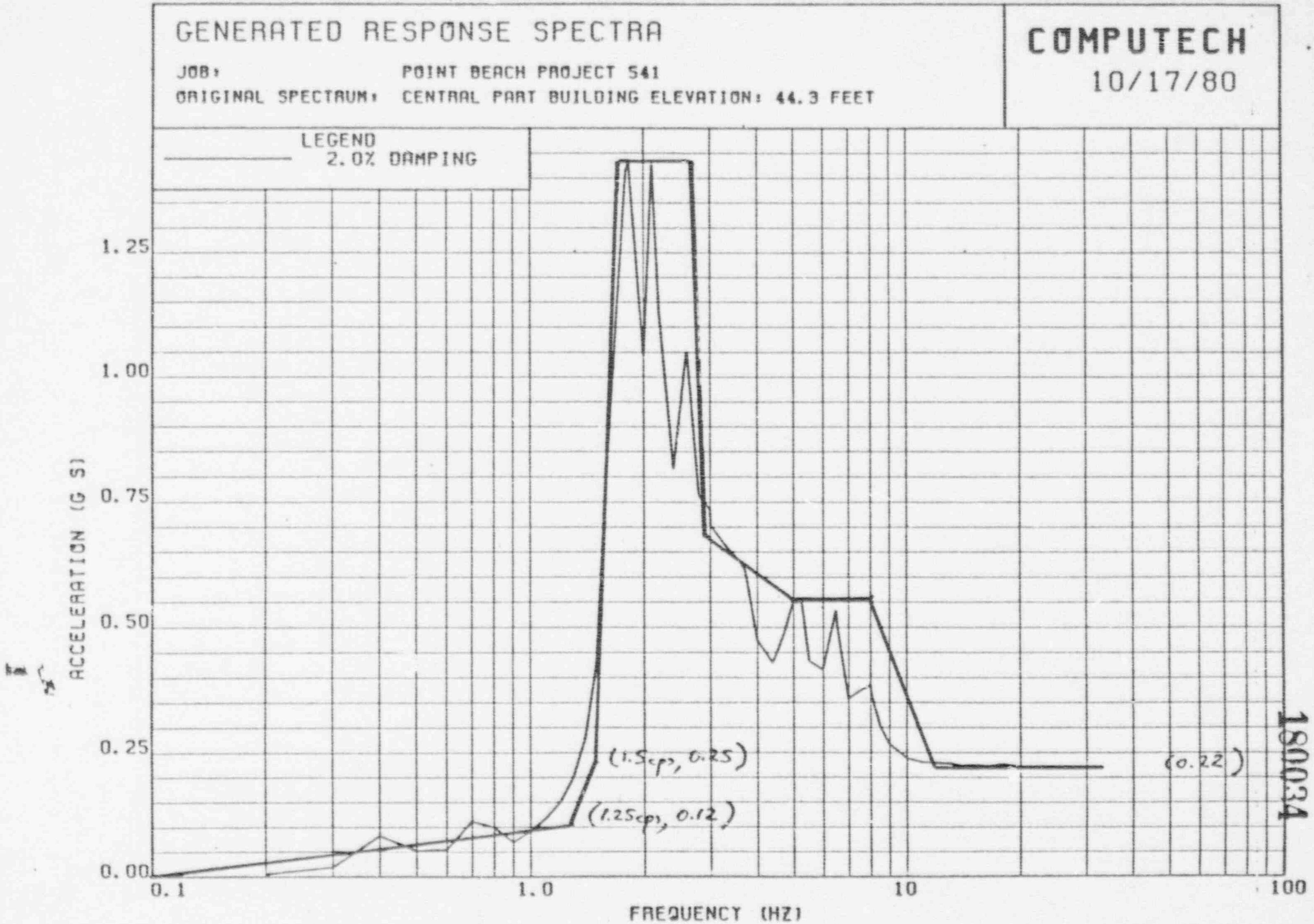
ORIGINAL SPECTRUM: CENTRAL PART BUILDING ELEVATION: 26.0 FEET
COMPUTECH
10/17/80
LEGEND
4.0% DAMPING
ACCELERATION (G S)
1.25
1.00
0.75
0.50
0.25
0.00
(1.45cps, 0.22)
(1.15cps, 0.10)
(0.20)
0.1
1.0
10
100
FREQUENCY (HZ)



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COMPUTECH
10/17/80
LEGEND
2.0% DAMPING
ACCELERATION (G S)
1.25
1.00
0.75
0.50
0.25
0.00
0.1
1.0
10
100
FREQUENCY (HZ)
(1.5cps, 0.25)
(1.25cps, 0.12)
(0.22)
180034

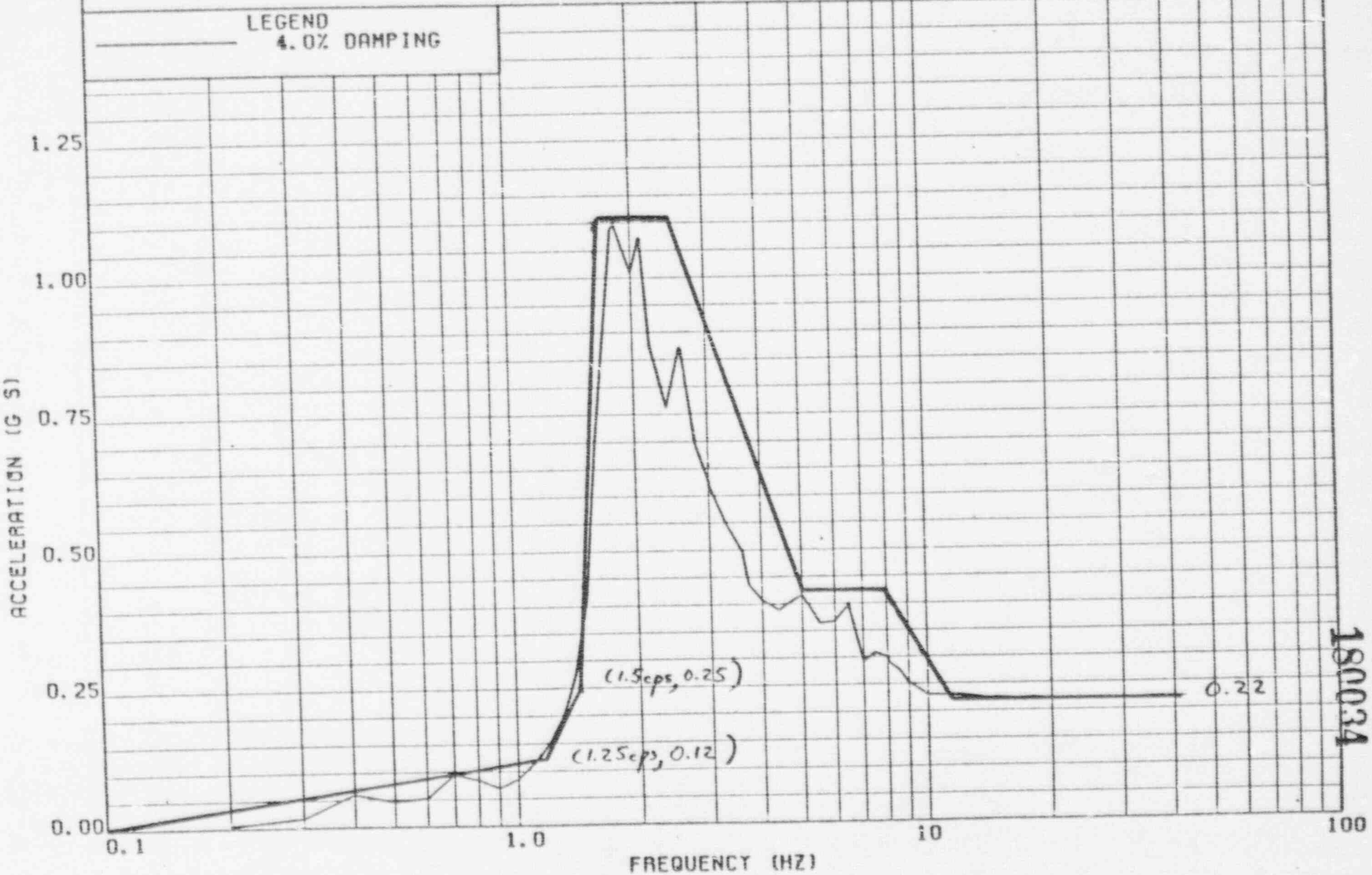
GENERATED RESPONSE SPECTRA

JOB: POINT BEACH PROJECT 541

ORIGINAL SPECTRUM: CENTRAL PART BUILDING ELEVATION: 44.3 FEET

COMPUTECH

10/17/80



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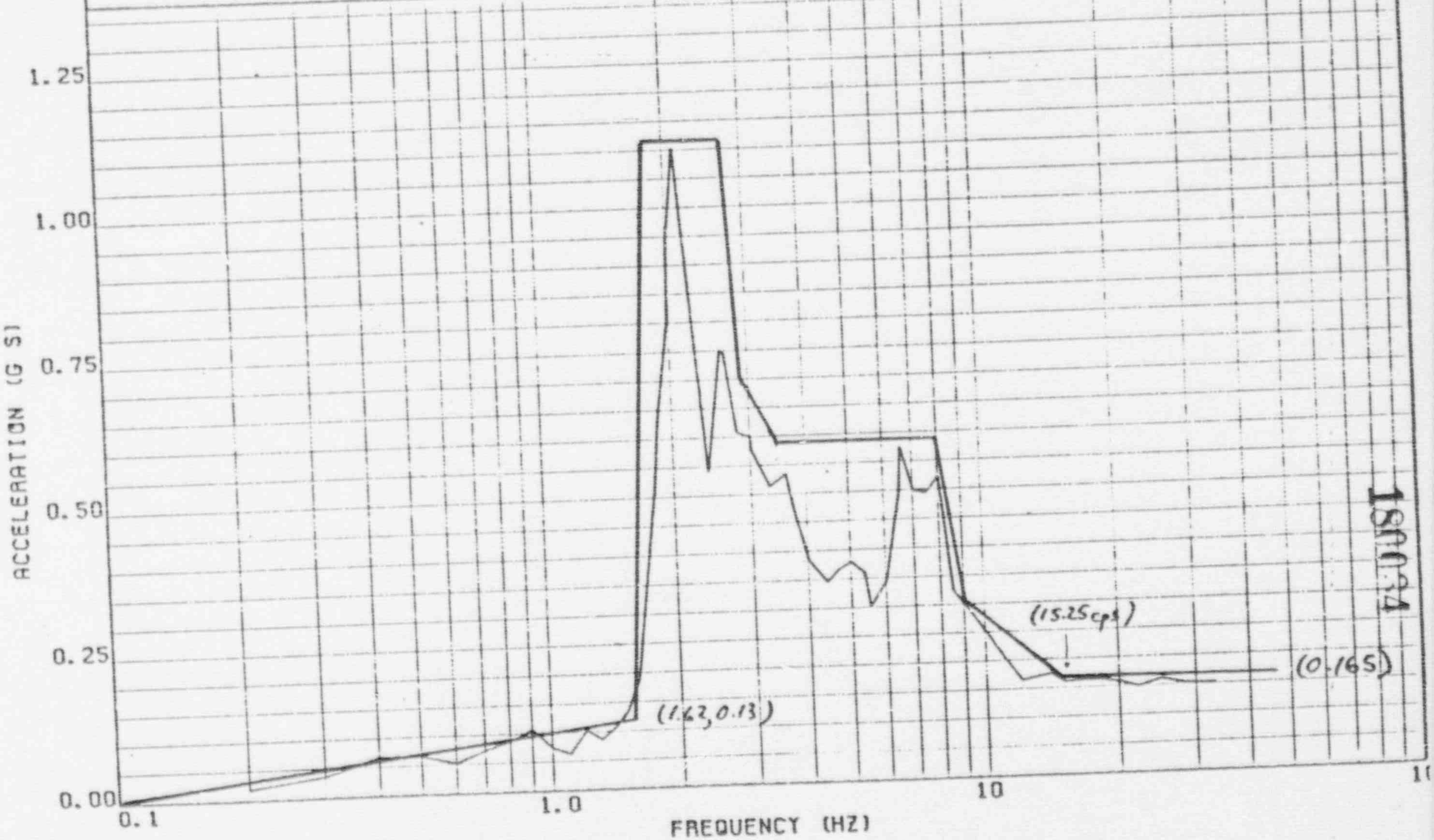
GENERATED RESPONSE SPECTRA

JOB: POINT BEACH PROJECT 541

ORIGINAL SPECTRUM: AUXILIARY SOUTH WING BUILDING ELEVATION: 8 FEET

COMPUTECH
10/19/80

LEGEND
——— 2.0% DAMPING



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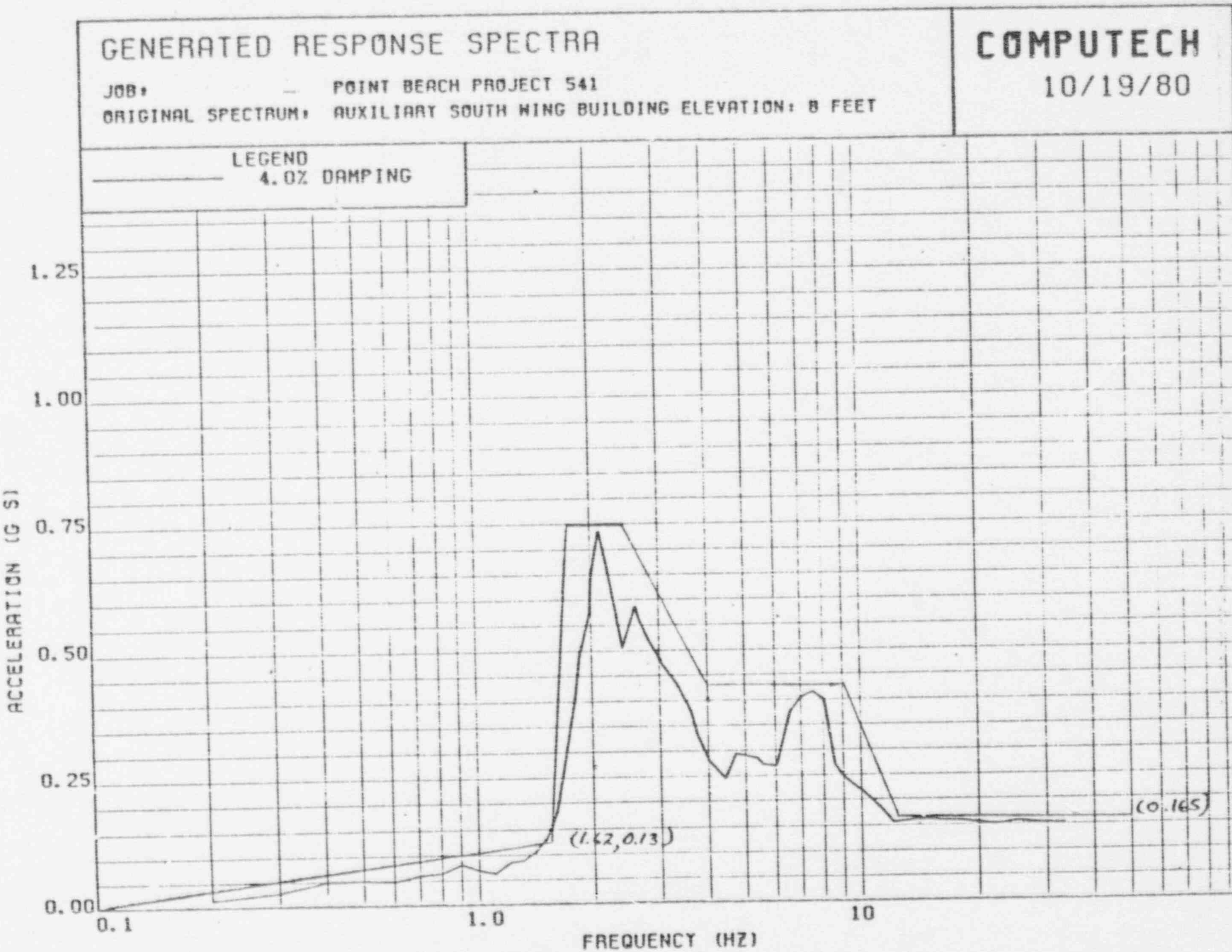

GENERATED RESPONSE SPECTRA
JOB: POINT BEACH PROJECT 541
ORIGINAL SPECTRUM: AUXILIARY SOUTH WING BUILDING ELEVATION: 8 FEET
COMPUTECH
10/19/80
LEGEND
4.0% DAMPING
ACCELERATION (G S)
1.25
1.00
0.75
0.50
0.25
0.00
FREQUENCY (HZ)
0.1
1.0
10
100
(1.42, 0.13)
(0.165)

1800:34

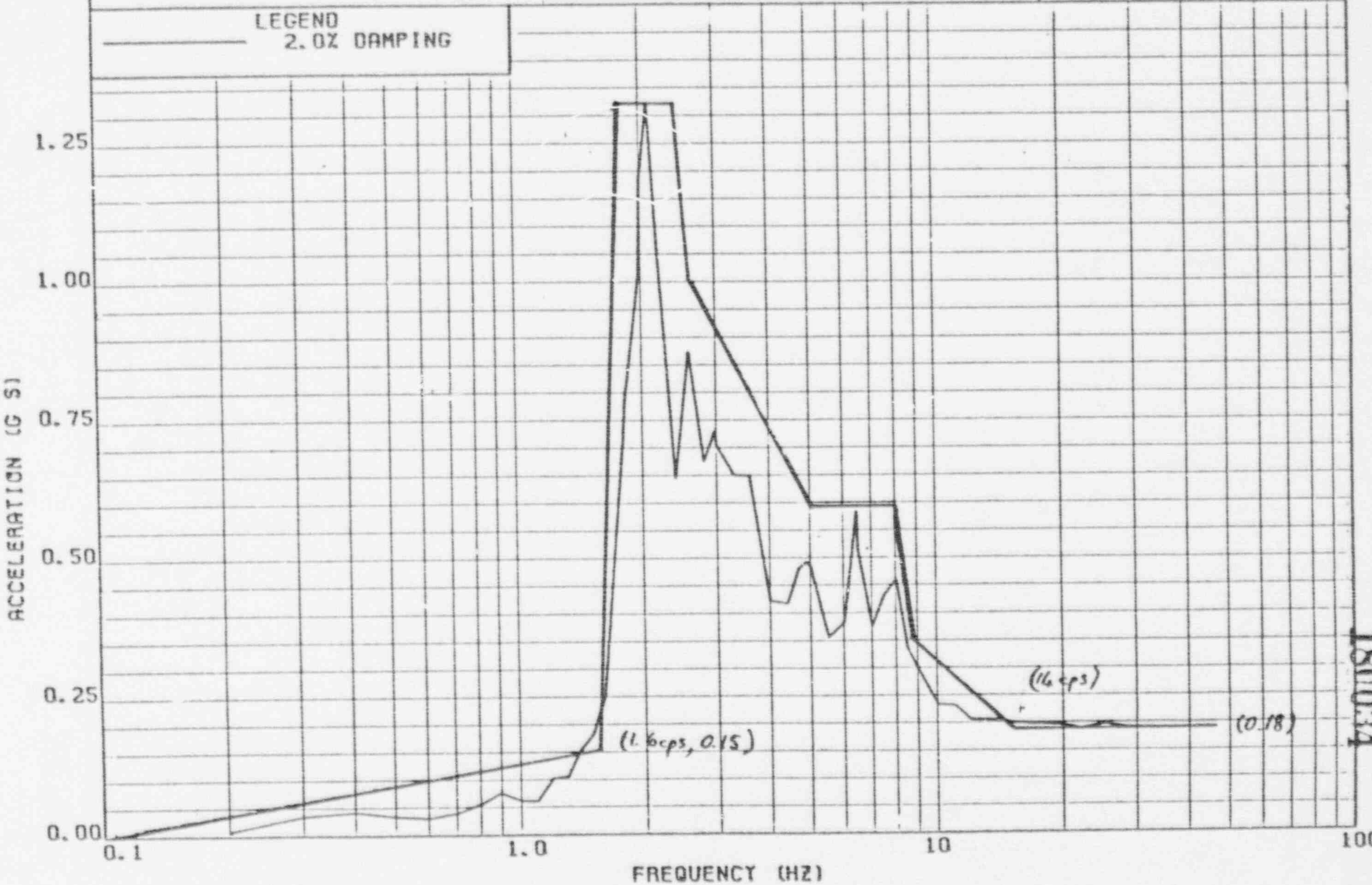
GENERATED RESPONSE SPECTRA

JOB: POINT BEACH PROJECT 541

ORIGINAL SPECTRUM: AUXILIARY SOUTH WING BUILDING ELEVATION: 26 FEET

COMPUTECH

10/19/80



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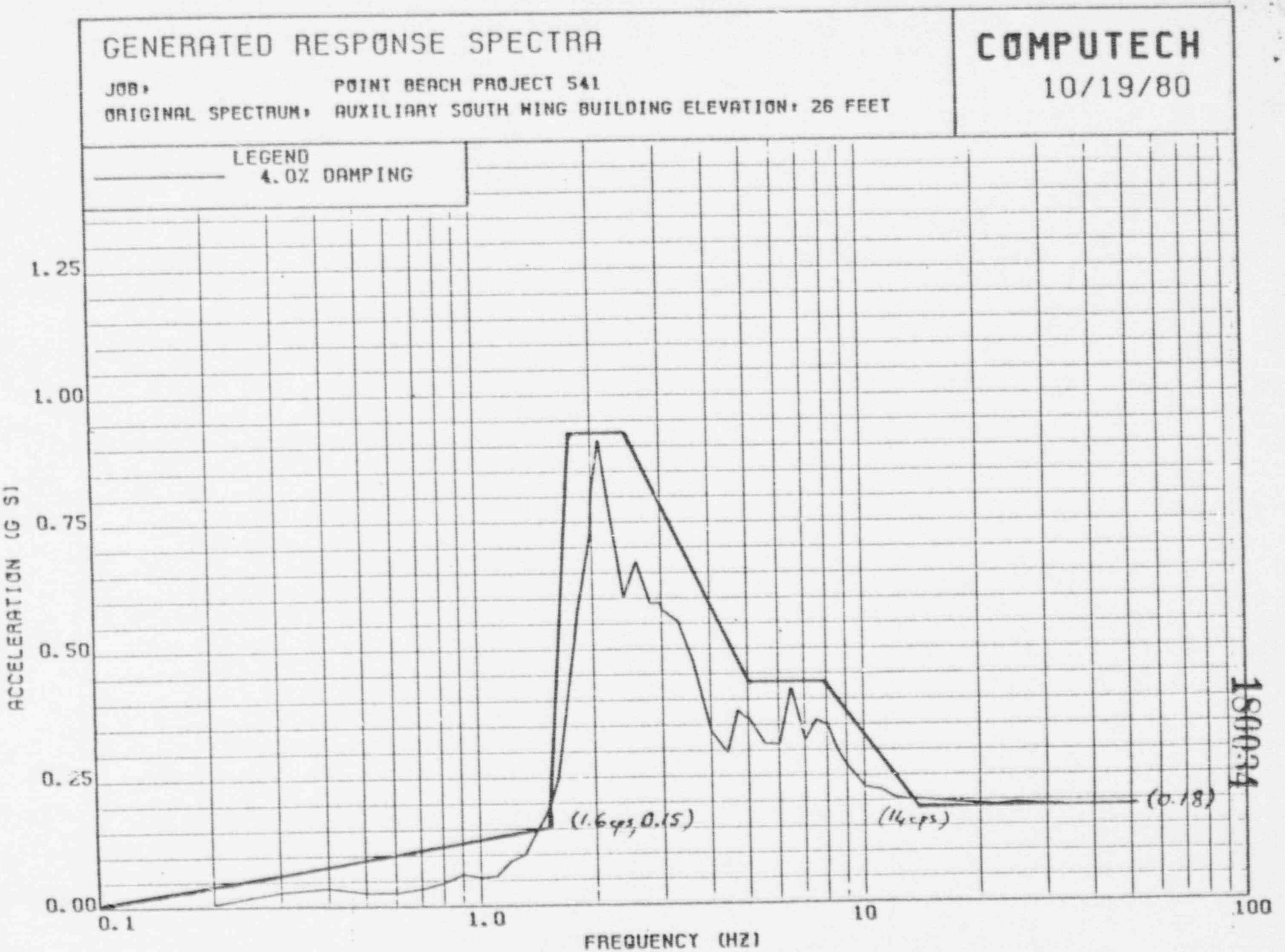
GENERATED RESPONSE SPECTRA
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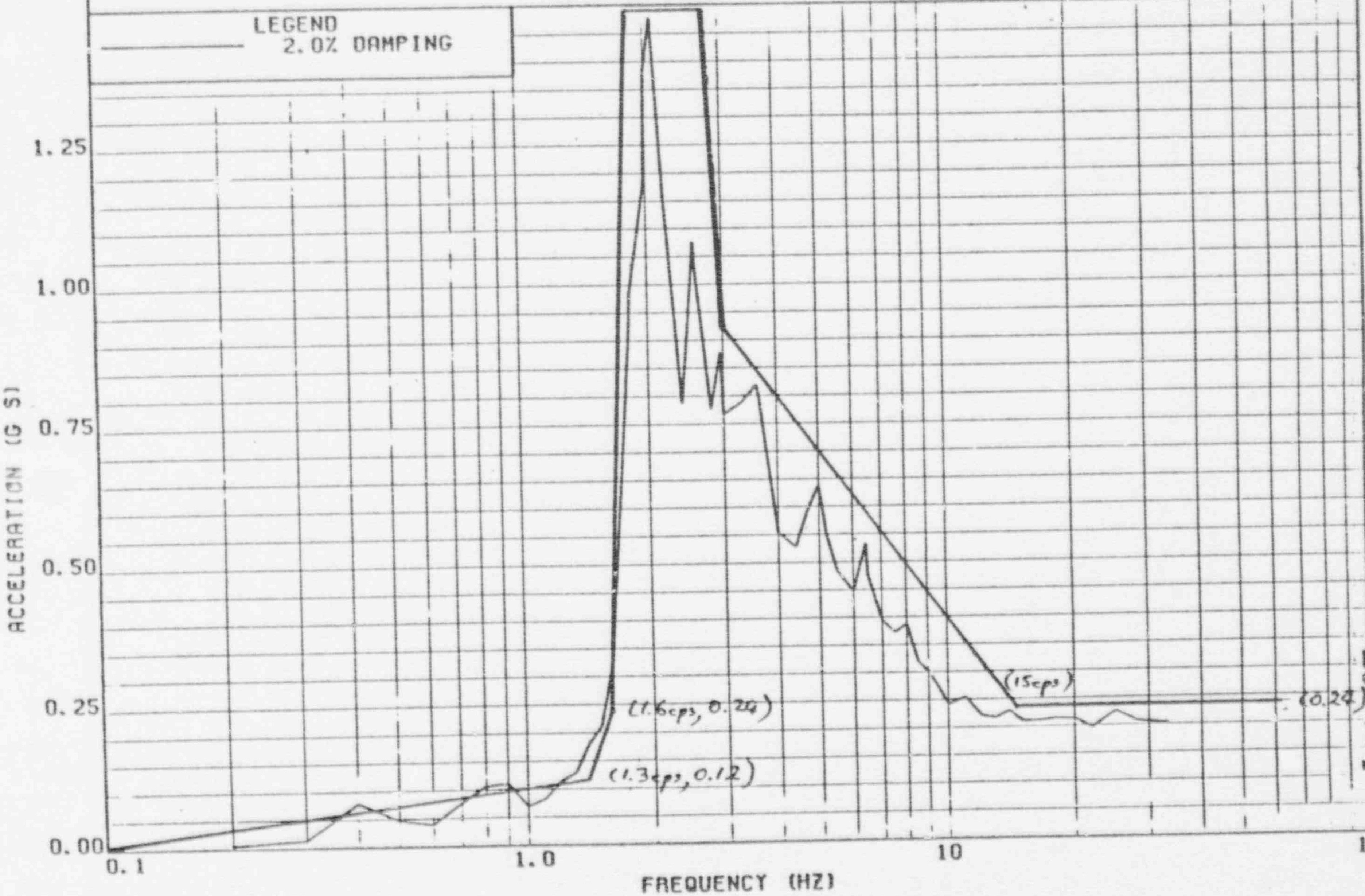
ORIGINAL SPECTRUM: AUXILIARY SOUTH WING BUILDING ELEVATION: 26 FEET
COMPUTECH
10/19/80
LEGEND
4.0% DAMPING
ACCELERATION (G S)
1.25
1.00
0.75
0.50
0.25
0.00
0.1
1.0
10
100
FREQUENCY (HZ)
(1.6cps, 0.15)
(14cps)
(0.18)
1800:34

GENERATED RESPONSE SPECTRA

JOB: POINT BEACH PROJECT 541

ORIGINAL SPECTRUM: AUXILIARY SOUTH WING BUILDING ELEVATION: 48 FEET

COMPUTECH
10/20/80



180034

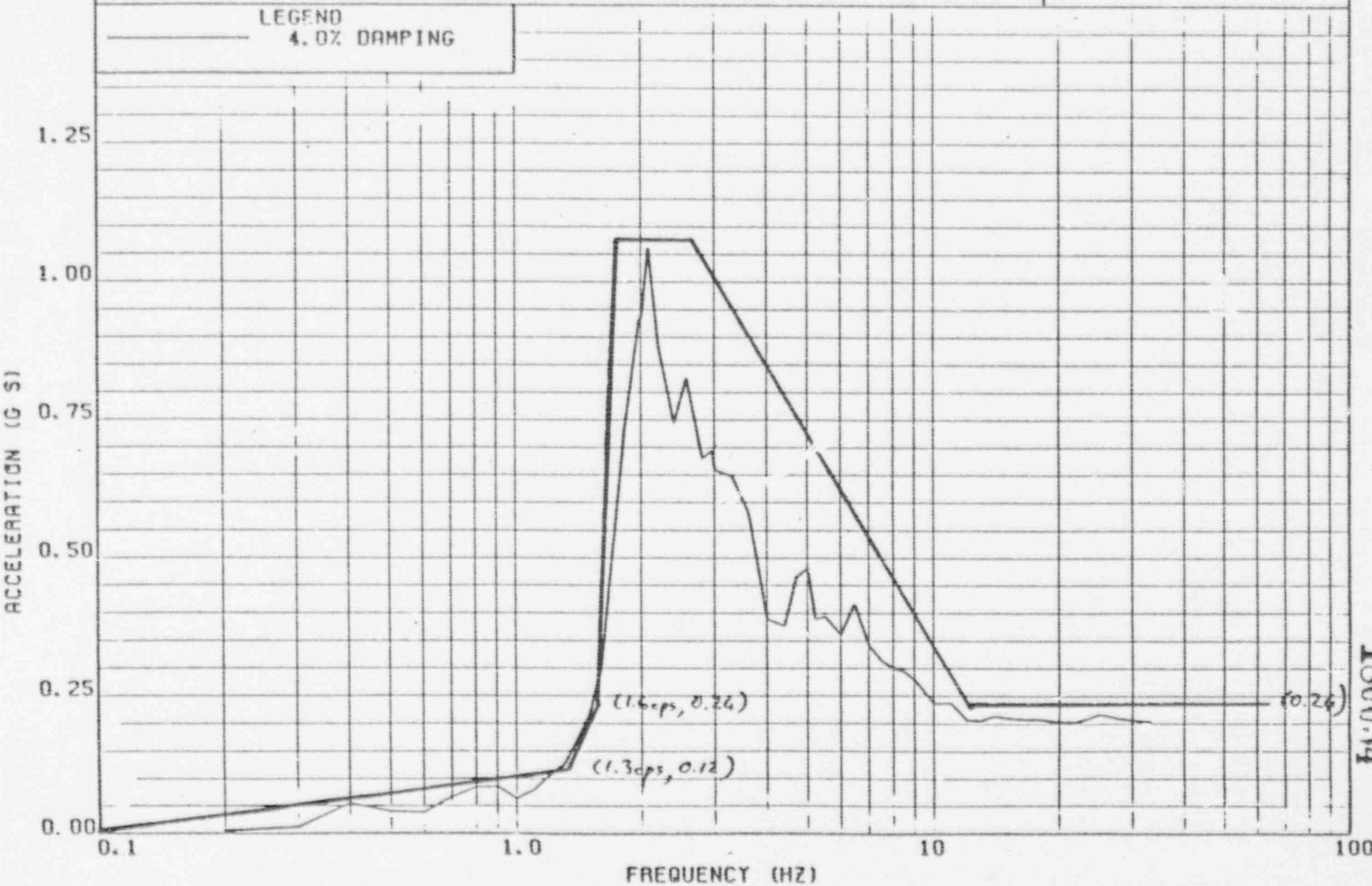
GENERATED RESPONSE SPECTRA

JOB: POINT BEACH PROJECT 541

ORIGINAL SPECTRUM: AUXILIARY SOUTH WING BUILDING ELEVATION: 48 FEET

COMPUTECH

10/20/80



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UPGRADING REQUIREMENTS FOR SAFETY-RELATED MASONRY CONSTRUCTION

MASONRY WALLS

Walls not requiring upgrading (clip angles will be provided along vertical and top edges)

WALL NO.	SKETCH NO.	EXPECTED COMPLETION DATE	WALL NO.	SKETCH NO.	EXPECTED COMPLETION DATE
3	SK-C-176	Complete	86	SK-C-165	Complete
40	SK-C-172	Complete	72	SK-C-169	Complete
5-29	SK-C-174	11-16-81	64	SK-C-164	Complete
5-7	SK-C-177	11-11-81	65	SK-C-166	Complete
24	SK-C-173	Complete	68-1	SK-C-156	Complete
26	SK-C-184	Complete	68-2	SK-C-158	Complete
157-5	SK-C-183	10-23-81	150	SK-C-161	Complete
5-5	SK-C-139	Complete	116	SK-C-115	10-16-81

Walls requiring positive shear transfer mechanisms (clip angles) along boundaries.

WALL NO.	SKETCH NO.	EXPECTED COMPLETION DATE	WALL NO.	SKETCH NO.	EXPECTED COMPLETION DATE
20	SK-C-143	8-28-81	111-2	SK-C-107	10-16-81
151	SK-C-163	12-15-81	111-3S	SK-C-116	10-16-81
104	SK-C-162	Complete	111-4N	SK-C-117	10-16-81
143	SK-C-181	10-16-81	111-4S	SK-C-118	10-16-81
			112-E	SK-C-108	10-16-81
			112-W	SK-C-109	10-16-81
			112-NS	SK-C-110	10-16-81
			115	SK-C-114	10-16-81

Walls requiring upgrading due to flexural tension overstress.

WALL NO.	SKETCH NO.	EXPECTED COMPLETION DATE	WALL NO.	SKETCH NO.	EXPECTED COMPLETION DATE
19	SK-C-131	8-28-81	111-1	SK-C-106	10-16-81
			111-3N	SK-C-119	10-16-81
			113	SK-C-111	10-16-81
			114	SK-C-113	10-16-81

SPECIAL CASES

WALL NO.	SKETCH NO.	DESCRIPTION	EXPECTED COMPLETION DATE
152	SK-C-160	Stack bond wall - this wall will be removed and rebuilt using running bond.	12-4-81
157-N	SK-C-180	Requires installation of pipe hangers to remove pipe loads from this block wall.	7-31-81
39	SK-C-150	Requires new supports for lead-filled pipe chase to remove loads from block wall.	8-7-81
133	---	This wall has been removed from the safety-related category.	
22A	SK-C-125	Requires installation of bracing system to provide additional stability.	10-16-81
146	---		
147	---	Brick walls - these walls will be removed.	Complete
161	---		
162	---		

Facade Stairwell walls: See response to Action Item #3.

MASONRY FILLED BLOCKOUTS

Blockouts not requiring upgrading (clip angles will be provided along vertical and top boundaries.)

WALL NO.	SKETCH NO.	EXPECTED COMPLETION DATE	WALL NO.	SKETCH NO.	EXPECTED COMPLETION DATE
5-31	SK-C-168	Complete	3-6	SK-C-192	Complete
5-22	SK-C-178	11-25-81	3-7	SK-C-179	Complete
5-24	SK-C-199	11-19-81	134-C-A	SK-C-170	11-6-81
45-E	SK-C-154	12-11-81	134-C-B	SK-C-171	11-6-81

Blockouts requiring upgrading (all blockouts requiring upgrading will be removed.)

WALL NO.	EXPECTED REMOVAL DATE	WALL NO.	EXPECTED REMOVAL DATE
30	12-18-81	3-1	12-18-81
5-12	12-18-81	5-51	Complete
14	Complete	71*	12-11-81
22	Complete	3-19	Complete
23	Complete	45-W	Complete

*One blockout in wall #71 will be retained, but it will be covered with steel plate. See SK-C-193 for details.

Response to Action Item #17

List of walls requiring upgrading beyond their present status.

UPGRADING REQUIREMENTS FOR SAFETY-RELATED MASONRY CONSTRUCTION

MASONRY WALLS

Walls not requiring upgrading (clip angles will be provided along vertical and top edges anyway)

wall no.	sketch no.	expected completion date	wall no.	sketch no.	expected completion date
3	SK-C-176		86	SK-C-165	
40	SK-C-172		72	SK-C-169	
5-29	SK-C-174		64	SK-C-164	
5-7	SK-C-177		65	SK-C-166	
24	SK-C-173		68-1	SK-C-156	
26	SK-C-184		68-2	SK-C-158	
157-S	SK-C-183		150	SK-C-161	
		control room wall →	116	SK-C-115	

Walls requiring positive shear transfer mechanisims (clip angles) along boundaries

wall no.	sketch no.	expected completion date	wall no.	sketch no.	expected completion date
20	SK-C-143		111-2	SK-C-107	
151	SK-C-163		111-3S	SK-C-116	
104	SK-C-162		111-4N	SK-C-117	
143	SK-C-181		111-4S	SK-C-118	
			112-E	SK-C-108	
			112-W	SK-C-109	
		control room walls	112-NS	SK-C-110	
			115	SK-C-114	

Walls requiring upgrading due to flexural tension overstress

wall no.	sketch no.	expected completion date	wall no.	sketch no.	expected completion date
19	SK-C-131		111-1	SK-C-106	
22-A	SK-C-125		111-3N	SK-C-119	
5-5	SK-C-139		113	SK-C-111	
		control room walls	114	SK-C-113	

Special Cases

wall no.	sketch no.	description	expected completion date
152	SK-C-100	stack bond wall - this wall will be removed and re-built using running bond	
157-N	SK-C-180	requires installation of pipe hangers to remove pipe loads from this block wall	
39	SK-C-150	requires new supports for lead-filled pipe [illegible] to remove loads from block wall	
133	-	safety-related conduit will be re-routed, so wall will become non-safety-related	
146	-	brick walls - these walls will be removed	
147	-		
161	-		
162	-		

facade stairwell walls: see response to action item # 3.

MASONRY FILLED BLOCKOUTS

Bl. kouts not requiring upgrading (clip angles will be provided along vertical and top boundaries anyway)

wall no.	sketch no.	expected completion date	wall no.	sketch no.	expected completion date
5-31	SK-C-168		3-6	SK-C-192	
5-22	SK-C-178		3-7	SK-C-179	
5-24	SK-C-199		134-C-A	SK-C-170	
45-E	SK-C-156		134-C-B	SK-C-171	

Blockouts requiring upgrading (all blockouts requiring upgrading will be removed)

wall no.	expected removal date	wall no.	expected removal date
30		3-1	
5-12		5-51	
14		71*	
22		3-19	
23		45-W	

* One blockout in wall #71 will be retained, but it will be covered with steel plate. See SK-C-193 for details. The expected completion date is .