

NORTHEAST UTILITIES



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

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(203) 666-6911

March 27, 1984

Docket No. 50-423
B11099

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

- References: (1) B. J. Youngblood to W. G. Council, Millstone Nuclear Power Station, Unit No. 3 Draft Safety Evaluation Report (DSER), dated December 20, 1983.
- (2) B. J. Youngblood to W. G. Council, Request for Additional Information for Millstone Nuclear Power Station, Unit No. 3, dated January 16, 1984.

Dear Mr. Youngblood:

Millstone Nuclear Power Station, Unit No. 3
Transmittal of Responses to Requests of Additional
Information and Draft SER Open Items (Geotechnical Issues)

On February 14, 1984 representatives from Northeast Utilities Service Company (NUSCO) and Stone & Webster met with your Mr. John Chen, Structural and Geotechnical Engineering Branch and Ms. E. L. Doolittle, Licensing Project Manager, to discuss the status and proposed resolution of geotechnical issues identified in References (1) and (2). A summary of that meeting, outlining items requiring NUSCO action as requested by Mr. Chen, was submitted on February 22, 1984.

Attachment 1 is a copy of the original meeting summary which has been revised to provide a description of the response enclosed herein (Attachment 2) or a schedule for providing the requested information. This is identified as "RESPONSE (3/23/84)". Revisions to the FSAR are provided as they will appear in Amendment 8, scheduled for submittal in approximately mid-May, 1984.

Attachment 3 is our formal response to Draft SER Open Items regarding Geotechnical issues that NUSCO considers to be resolved by the additional information transmitted herein and/or were resolved at the Structural Audit.

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

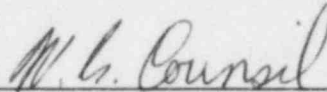
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Ms. Shaffer will contact your Mr. Chen on April 9, 1984 to confirm that his concerns have been addressed and these items/questions are closed.

Very truly yours,

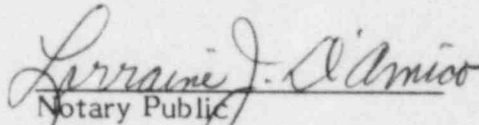
NORTHEAST NUCLEAR ENERGY COMPANY et. al.
By Northeast Nuclear Energy Company,
their agent



W. G. Council
Senior Vice President

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

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


Notary Public

My Commission Expires March 31, 1988

cc: Ms. E. L. Doolittle - NRC Project Manager
Mr. John Chen SGEB

ATTACHMENT 1

RESPONSE OR SCHEDULE FOR
SUBMITTAL OF RESPONSE
TO GEOTECHNICAL MEETING ACTION ITEMS
(241 SERIES QUESTIONS)

NORTHEAST UTILITIES



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

March 27, 1984
NE-34-L-346

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TO: Distribution*

FROM: C. J. Shaffer, Generation Facilities Licensing

SUBJECT: Meeting Summary - February 14, 1984
Discussion of 241 series Questions and
Draft SER Geotechnical Open Items

A meeting was held February 14, 1984 at 9:30 a.m. at NUSCO's Bethesda office to discuss the resolution of Millstone 3 Draft SER Geotechnical Open Items and 241 series questions. Attendees were:

W. R. Rotherforth - NUSCO, Generation Civil Engineering
C. J. Shaffer - NUSCO, Generation Facilities Licensing
F. Vetere - Stone & Webster, Geotechnical Engineer
E. L. Doolittle - NRC Project Manager
J. Chen - NRC, Structural & Geotechnical
Engineering Branch Reviewer

The following is a summary of the discussion of each item and the proposed action to resolve the item. **RESPONSE** (3/23/84) indicates the enclosed response to action items or a schedule for providing the requested information.

Question No.

o 241.1

a) Figure 2.5.4-51

Action: The Figure will be revised to indicate on cross-section H-H" that this was a preconstruction case and to where the fill material overlying the till was removed.

RESPONSE (3/23/84):

Figure 2.5.4-51 has been revised to show where the fill material overlying the till was removed.

b) Figure 2.5.4-67

Mr. Chen asked if the sheet piling shown along the discharge tunnel had been removed. Upon being informed

that the water indication to the left of the discharge tunnel was fill, not water, this question was resolved.

Action: The Figure will be revised to clarify fill to the left of the discharge tunnel.

RESPONSE (3/23/84):

Figure 2.5.4-67 has been revised.

c) Figures 2.5.4-69, -70

Mr. Chen questioned the apparent crossing of the service water line and electrical duct at the same elevation.

Action: NUSCO will check on this and provide clarification.

RESPONSE (3/23/84):

The service water line and electrical duct do cross as shown in revised Figures 2.5.4-69 and -70.

o 241.3

a) Figures 2.5.4-54, -56

Action: Correct elevations of footings and check as built condition of footings.

RESPONSE (3/23/84):

Figures 2.5.4-54, -55 and -56 have been revised to show the correct elevations of the EGE wall footings and the extent of the structural backfill between the EGE and the Control Building (see 241.4(b)).

b) Mr. Chen raised a concern over soil-structure interaction of compacted fill over the till. (a 1 to 10 variance of response is possible).

Action: NUSCO will provide a East - West section through the fuel oil tanks.

RESPONSE (3/23/84):

Figure 2.5.4-57 provides an East-West section through the fuel oil tanks.

o 241.4

- a) Mr. Chen was given a draft of revised Section 3.7.B.1.4 that clarifies the inconsistencies. The revision will be included in the next amendment to the FSAR.
- b) Action: Figure will be revised to show the extent of the structural backfill between the EGE and the Control Building.

RESPONSE (3/23/84):

See response to 241.3(a).

- c) Mr. Chen expressed concern over the variance of shear modulus in the till across the section show (EGE and Control Building) and also the same effect of the compacted fill adjacent to the Control Building.

Action: F. Vetere will do parametric study of varying G_{max} of till for range of ν_s on page 2.5.4-9 of the FSAR and tabulate G_y for various G_{max}

RESPONSE (3/23/84):

Pages 2.5.4-18 and -18a have been revised to discuss the extent of the EGE structural backfill in the SSI analysis.

Table 2.5.4 - 21A provides a tabulation of G_y for various G_{max} for EGE free field soil properties.

Figures 2.5.4-72, -73 and -74 show the EGE Soil Structure Interaction, Shear Modules Curve, and Sampling Curve, respectively.

o 241.6

Sliding Stability calculation does not account for top driving force.

Action: F. Vetere will reanalyze this, adding dynamic driving force from ground surface to bottom of pipe when considering the weight of soil as a benefit.

RESPONSE (3/23/84):

A revised response to Q241.6--Sliding Stability of the Service Water Encasement and corresponding calculations are provided.

o 241.7 and 241.8

- a) F. Vetere discussed the work being done using a sloping rock surface.

Action: F. Vetere will check till slope in Fig. 2.5.4-52 and soil properties as requested by Mr. Chen.

RESPONSE (3/23/84):

Figure 2.5.4-52 has been revised.

- b) What is the basis for the groundwater height at the intake structure? If beach sand liquefaction is not critical the groundwater level does not present a problem.

Action: A discussion of the basis of the groundwater level used will be provided.

RESPONSE (3/23/84):

This information will be supplied on or before April 13, 1984.

- c) Action: NUSCO will provide bedrock spectra for each time history used in the analyses.

RESPONSE (3/23/84):

The Response spectra for the beach area SHAKE Analysis have been provided.

- o 241.16 Mr. Chen feels very strongly that our lateral earth pressure coefficient, $K_0 = .5$, is low and should be near .7 to .8 as a minimum. He said other plants are using .8 to 1.0. He suggested using actual shear wave velocity measurements to determine Poisson ratio. However, no cross-hole tests were performed on structural fill.

Action: F. Vetere will continue sensitivity analysis.

Response (3/23/84)

This information will be supplied on or before April 13, 1984.

- o 241.17 Ring Beam

Action: We will provide a 2-dimensional plan with jointing and foliation for worst case (near Main Steam Valve Building). Also we will supply pictures of rock cut face and top view surface as requested by Mr. Chen.

RESPONSE (3/23/84): Ring Beam

Figure 2.5.5-b shows the Potential Failure Wedges for the West Side of the Containment. Figure 2.5.5-7 is a

photograph showing the rock surface commonly found in the area of the Main Steam Valve Building.

o 241.18 Dynamic Slope Stability

- a) Mr. Chen questioned the use of $\phi = 34^\circ$ for the beach sand, however, agreed that this is not a problem if liquefaction is okay.

RESPONSE (3/23/84):

Pages 2.5.5-5 through -7 of the FSAR have been revised to address Mr. Chen's concerns. Figure 2.5.5-5 provides a summary of the CIU test results for the beach area outwash sands.

- b) Mr. Chen would like to see sections and plans of retaining¹ wall (Cat. I).

Action: We will provide this information.

RESPONSE (3/23/84):

Figure Q241.22-1 provides a plan view of the west retaining wall. Figures Q241.22-2 and -3 provide sections of the west retaining wall.

- c) He would also like to see the seawall design loads - wave loads, seismic loads and dynamic loads.

RESPONSE (3/23/84):

Revision 1 to Question 241.22-1 provides the design loads for the retaining wall.

- o Draft SER Open Item on Ground water level is a EHEB Branch question. NUSCO is developing a response to this item and will discuss it with the EHEB reviewer. Mr. Chen stated that we should consider groundwater monitoring program.
- o 241.20 Mr. Chen has no additional questions or concerns on this item.

During the meeting the following were given to Mr. Chen:

- o Draft Change to 3.7.B.1.4
- o Calculation 123 G
- o Broms paper
- o pages 19-27 of 59G - computer output of sample wedge for ring beam.
- o Draft change to 2.5.5.2.2
- o EC 29A-K

* R. N. Smart
W. R. Rotherforth
W. J. Briggs
R. T. Laudenat
R. L. McGuinness
GFL Memo File
MP3 File

cc: F. Vetere - S&W
W. Emerson - S&W
D. Theodossiou - S&W
E. L. Doolittle - NRC
J. Chen - NRC

ATTACHMENT 2

RESPONSE TO REQUESTS FOR
ADDITIONAL INFORMATION
ACTION ITEMS RESULTING FROM FEBRUARY 14th MEETING

Questions:	241.1	
	241.3	
	241.4	
	241.6	
	241.7 & 8 (partial)	i
	241.16	
	241.17	r
	241.18	
	241.22	t

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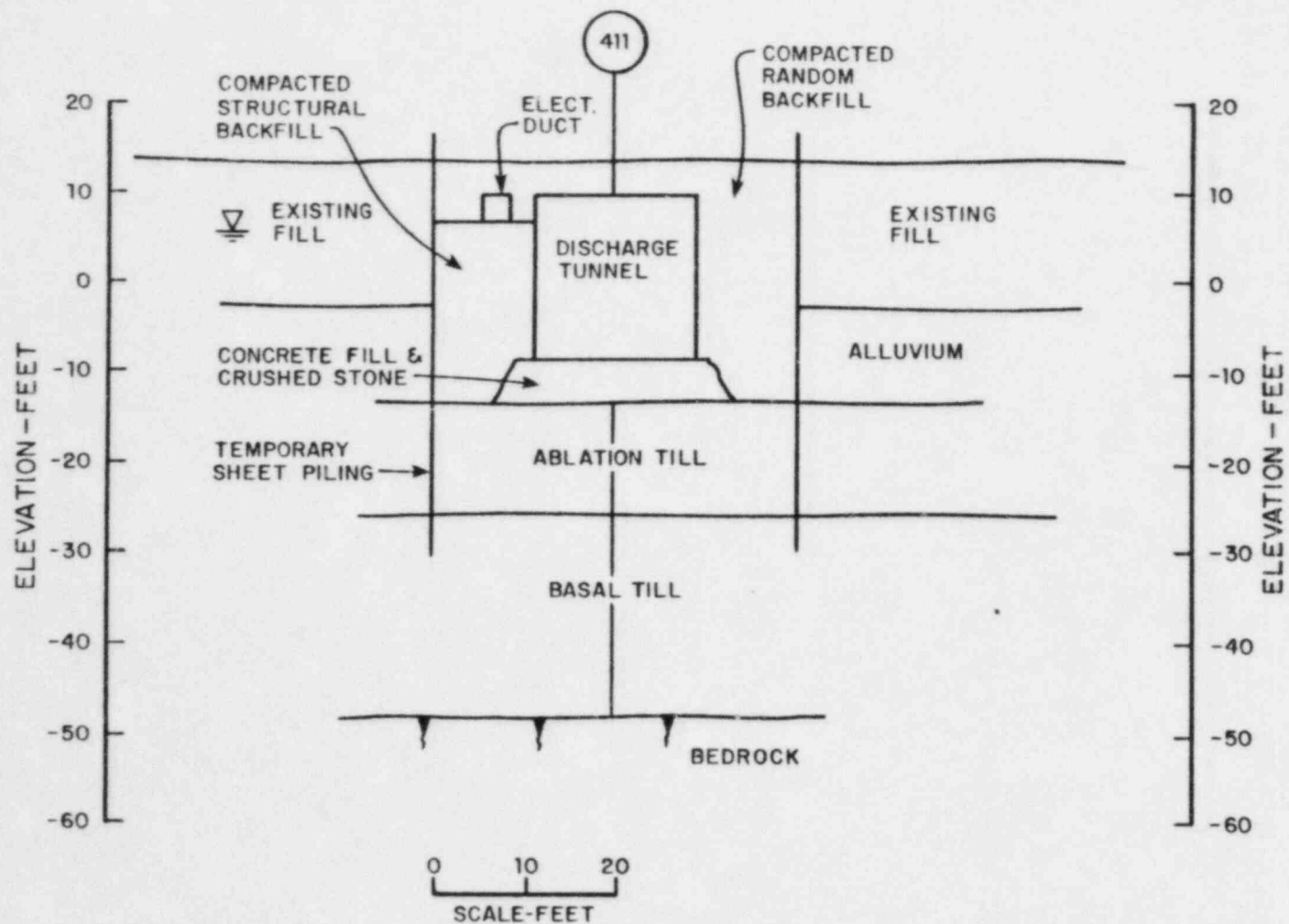
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NOTE

REFER TO FIG. 2.5.4-31 FOR
LOCATION OF SECTION N-N'

▽ DENOTES GROUND WATER

FIGURE 2.5.4-67
GEOLOGIC PROFILE
SECTION N-N'
MILLSTONE NUCLEAR POWER STATION
UNIT 3
FINAL SAFETY ANALYSIS REPORT

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Q241.4

The service water intake pipes, between the circulating and service water pumphouse and the main plant area, are embedded in a rectangular concrete encasement. Soils encountered in the pipeline excavation include beach and outwash sands, unclassified stream deposits, and ablation till. These soils were removed under the pipeline to dense basal till and replaced with Category I structural backfill. The fill was placed at a 1:1 slope from the till surface to the base of the encasement and compacted to the requirements outlined in Section 2.5.4.5.2. The sides of the encasement were backfilled with nonstructural fill similar to the material used to backfill behind retaining walls and described in Section 2.5.4.10.3. The backfill was compacted to 90 percent of maximum dry density as determined by ASTM 1557, Method D.

2.5.4.7.1 Subsurface Material Properties Used in SSI Analysis 1.28

The subsurface profiles used in the soil-structure interaction analyses for the control building and the emergency generator enclosure (EGE) are idealized, horizontal profiles based on subsurface explorations conducted at the site and described in Section 2.5.4.3. Both of these structures are founded on dense basal till overlying bedrock. The computer program SHAKE was used to determine strain corrected values of shear modulus obtained from low strain values previously determined from field testing, laboratory testing, or empirical formulae based on laboratory test data. The program iterates to obtain values of modulus that are compatible with strain levels induced in a particular soil layer by a specific earthquake. The strain levels normally induced by earthquakes of magnitudes similar to the Millstone SSE are several orders of magnitude higher than the low strain levels achieved during laboratory or field testing, resulting in a reduction in shear modulus when these properties are corrected for strain and input into PLAXLY-3.

The soil-structure model used in the EGE analyses is shown on Figure 2.5.4-72. This idealized profile was selected to conservatively model the subsurface conditions under the EGE and in the free-field. The geologic profiles presented in Figures 2.5.4-55, 2.5.4-56, and 2.5.4-71 indicate that the rock surface slopes from approximately elevation 0 feet at the east end of the structure to at least elevation -10 feet at the west end. In the north-south direction, the sloping evacuation face between the control building and the south end of the EGE was backfilled with structural fill over the basal till. The extent of structural fill is shown on Section J-J (Figure 2.5.4-55) and Figure 2.5.4-54. Because the depth and extent of the structural fill under the EGE is limited, it was assumed that the model used in the SHAKE analysis is sufficiently conservative to account for local variations in the subgrade and their effect on structural response.

The soil properties input into the SHAKE calculation are listed in Table 2.5.4-21A for the free-field model and 2.5.4-21B for the structure-effects model. Three earthquake time histories, from the Taft, Parkfield-Temblor, and Pacoima Dam earthquakes, were normalized to the site SSE peak acceleration value of .17g and input at bedrock. Shear modulus and damping iterations were performed within the SHAKE program in accordance with the curves marked "Resonant Column Test" on

Figures 2.5.4-73 and 2.5.4-74. These curves were developed from empirical formulae and resonant column tests performed on samples of compacted structural fill from the Millstone site. These test results are presented on Figure 2.5.4-42. The tests show good correlation with curves present by Seed and Idriss in the SW-AJA report (1972).

The strain corrected values of shear modulus and damping in the free-load are presented in Table 2.5.4-21A. The mean value for each layer was calculated and used to represent the individual soil layer properties used in the PLAXLY model shown on Figures 3.7B-11 and 3.7B-12. The Millstone site artificial earthquake was input at bedrock and the soil was modeled as a finite element mesh. The use of SHAKE to perform shear modulus and damping iterations precludes the need to iterate in the PLAXLY model. A discussion of the soil-structure interaction analysis is presented in Section 3.7B2.4.

For the control building, the soil profile analyzed in SHAKE and used in the soil-structure interaction analysis was the section where rock was the deepest; i.e., top of rock at elevation -15 feet. Shear wave velocities were used to define soil stiffness. The low strain and strain-corrected soil properties for the free field case are listed in Table 2.5.4-22.

2.5.4.8 Liquefaction Potential 2.16

The foundation materials beneath some of the Seismic Category I structures consist of limited depths of dense to very dense basal tills and/or compacted select granular backfill. These materials are not susceptible to liquefaction under earthquake motions as described in the following sections.

2.5.4.8.1 Structural Backfill 2.22

Based on studies of soils where liquefaction has been observed (Seed 1968, Lee and Fitton 1969, Kishida 1969), it is concluded that the

Northeast Nuclear Energy Company (NNECo.) 1977. Fault in Demineralized and Refueling Water Tank Area. Millstone Nuclear Power Station-Unit 3, Docket No. 50-423, Waterford, Conn.	1.10 1.11	
Okabe, S. 1926. General Theory of Earth Pressure. Journal, Japanese Society of Civil Engineers, Vol 12, No. 1.	1.14	
Seed, H. B. 1968. Landslides during Earthquakes Due to Liquefaction. Journal of the Soil Mechanics and Foundation Engineering Division, ASCE, Vol 94, No. SM5.	1.16 1.17	
Seed, H. B.; Arrango, I.; and Chan, C. K. 1975. Evaluation of Soil Liquefaction Potential during Earthquakes. Earthquake Engineering Research Center, Report No. EERC 75-28, University of California, Berkeley, Calif.	1.19 1.20 1.21	
Seed, H. B. and Idriss, I. M. 1967. Analysis of Soil Liquefaction: Niigata Earthquake. Journal of the Soil Mechanics and Foundation Engineering Division, ASCE, Vol 93, No. SM3.	1.23 1.24	
Seed, H. B. and Idriss, I. M. 1971. Simplified Procedure for Evaluating Liquefaction Potential. Journal of the Soil Mechanics and Foundation Engineering Division, ASCE, Vol 97, No. SM9.	1.26 1.27	
Seed, H.B.; Idriss, I.M.; Makdisi, F. and Banjeree, N.R. 1975. Representation of Irregular Stress Time Histories by Equivalent Uniform Stress Series in Liquefaction Analysis. Earthquake Engineering Research Center, Report No. EERC 75-29; University of California, Berkeley, Calif.	1.28 1.29 1.30	241.9
Seed, H. B. and Lee, K. L. 1966. Liquefaction of Saturated Sands during Cyclic Loading. Journal of the Soil Mechanics and Foundation Engineering Division. ASCE, Vol 92, No. SM6.	1.32 1.33 1.34	
Seed, H. B. and Whitman, R. V. 1970. Design of Earth Retaining Structures for Dynamic Loading. Proceedings, ASCE Specialty Conference on Lateral Stresses and Design of Earth Retaining Structures, Cornell University, Ithaca, NY.	1.36 1.37 1.38	
Shannon and Wilson Inc. and Agbabian-Jacobsen Associates. 1972. Soil Behavior Under Earthquake Loading Conditions. Report prepared by Union Carbide Corporation for U.S. Atomic Energy Commission.	1.40 1.41	241.4
Sovinc, I. 1969. Displacements and Inclinations of Rigid Footings Resting on a Limited Elastic Layer of Uniform Thickness. Proceedings of the Seventh International Conference on Soil Mechanics and Foundation Engineering, Vol 1, p 385-389.	1.43 1.45 1.46	
State of Connecticut, Basic Building Code, Seventh Edition, 1978. Building Officials and Code Administrators International, Inc. Homewood, Illinois.	1.47 1.48 1.49	241.9
Teng, W.C. 1962. Foundation Design, Prentice Hall, Inc., Englewood Cliffs, New Jersey.	1.51	

- Terzaghi, K. and Peck, R. 1967. Soil Mechanics in Engineering Practice. 1.53
Second Edition, John Wiley and Sons, Inc., New York, NY. 1.54
- Vesic, A. S. 1975. Bearing Capacity of Shallow Foundations. In: 1.57
Foundation Engineering Handbook. Wenterkon, H. F. and Fang, H. Y. (ed.) 1.58
Van Nostrand-Reinhold, New York, NY.
- Whitman, R. V. and Richart, F. E., Jr. 1967. Design Procedures for 1.60
Dynamically Loaded Foundations. Journal of the Soil Mechanics and 2.1
Foundation Engineering Division, ASCE, Vol 93, No. SM6.

TABLE 2.5.4-21A
EMERGENCY GENERATOR ENCLOSURE
FREE FIELD SOIL PROPERTIES FROM SHAKE ANALYSIS

Layer	Top of Layer Elevation (ft)	Soil Type	γ_r (pcf)	C_{max} (psi)	G_{max} (psi)	Laft G_{max} (ksf)	D _{max}	Packfield G_{max} (ksf)	Pacotaa Dam G_{max} (ksf)	D _{max}	
1	24	Fill	143	1.2×10^5	3810	613	.087	457	576	.101	1.9
2	15	Fill	143	1.63×10^5	5324	778	.104	825	697	.120	1.11
3	10	Basal fill	145	1.4×10^5	1.28×10^5						1.13
4	0	Basal fill	145	1.4×10^5	1.20×10^5	18,800	.014	18,715	17,968	.019	1.23
5	-10	Basal fill	145	1.4×10^5	1.12×10^5	17,850	.018	17,249	16,913	.024	1.24
6	-20	Bedrock	-	1.5×10^5	-	16,981	.022	16,077	15,470	.029	1.25
											1.27
											1.32
											1.34
											1.36
											1.40

NOTES:

1. G_y = The average of the G's for the 3 earthquakes.
2. G = Strain corrected shear modulus (ksf).
3. D = Strain corrected damping ratio.
4. The structure is founded on the basal fill at the top of Layer 3. Soil stiffness was modeled using shear modulus.

N1800

N1700

SERVICE

COMPACTED STRUCTURAL BACKFILL TO EL. 24.0'

R

E150

FUEL OIL TANK VAULT
SUBGRADE EL. 1.5'

K

E100

J

EMERGENCY GENERATOR ENCLOSURE
EL. VARIES BETWEEN 9.0' AND 11.0'

R'

APPROXIMATE EDGE OF EXCAVATION

•51

14••13

23
26
29
27
24

•19

•9

•25

•39

•30

•35

APPROXIMATE
INTERFACE
FILL-TILL

•100

•53

•32

•49

•31

•50

•22

•52

•28

•54

•48

N1700

BUILDING

N1600



•15

•8

•7

APPROXIMATE
INTERFACE
ROCK-TILL

E150

•11

•6



TURBINE BUILDING

•12

•3

CONTROL BUILDING
SUBGRADE EL-0.5'

•5

E100

LEGEND



EGE WALL FOOTINGS

•2

•10



NOTES:

1. FOR TEST RESULTS REFER TO TABLE 2.5.4-20
2. FOR SECTIONS REFER TO FIGURES 2.5.4-55 & 2.5.4-56
3. •- MAP LOCATION NUMBER

0 5 10 15 20 25
SCALE - FEET

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FIGURE 2.5.4-54
LOCATION OF FIELD DENSITY TESTS
EMERGENCY GENERATOR ENCLOSURE
AND CONTROL BUILDING
MILLSTONE NUCLEAR POWER STATION
UNIT-3
FINAL SAFETY ANALYSIS REPORT

AMENDMENT 8

MAY 1984

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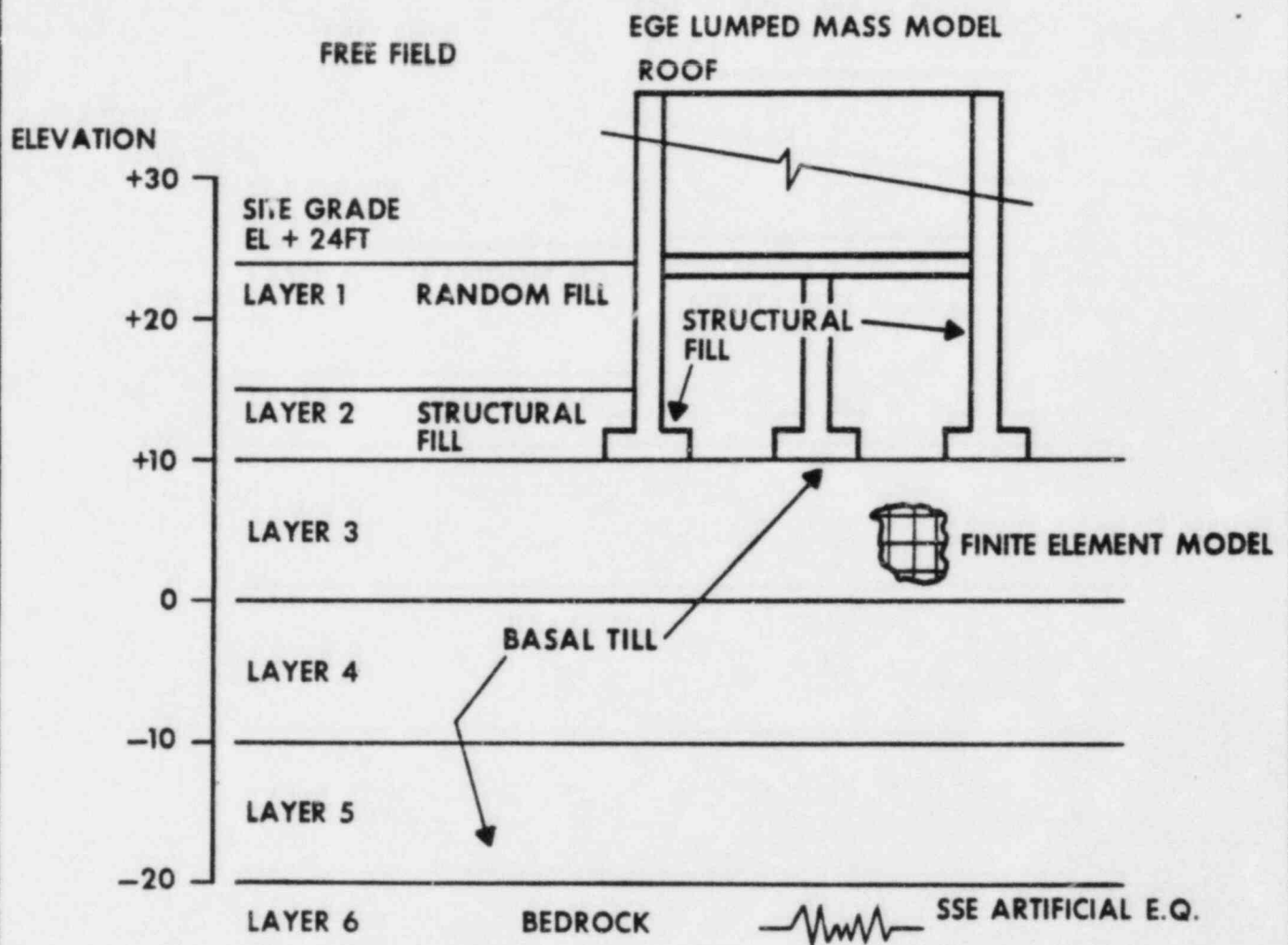


FIGURE 2.5.4-72
 SOIL-STRUCTURE INTERACTION
 EMERGENCY GENERATOR ENCLOSURE
 MILLSTONE NUCLEAR POWER STATION
 UNIT 3
 FINAL SAFETY ANALYSIS REPORT

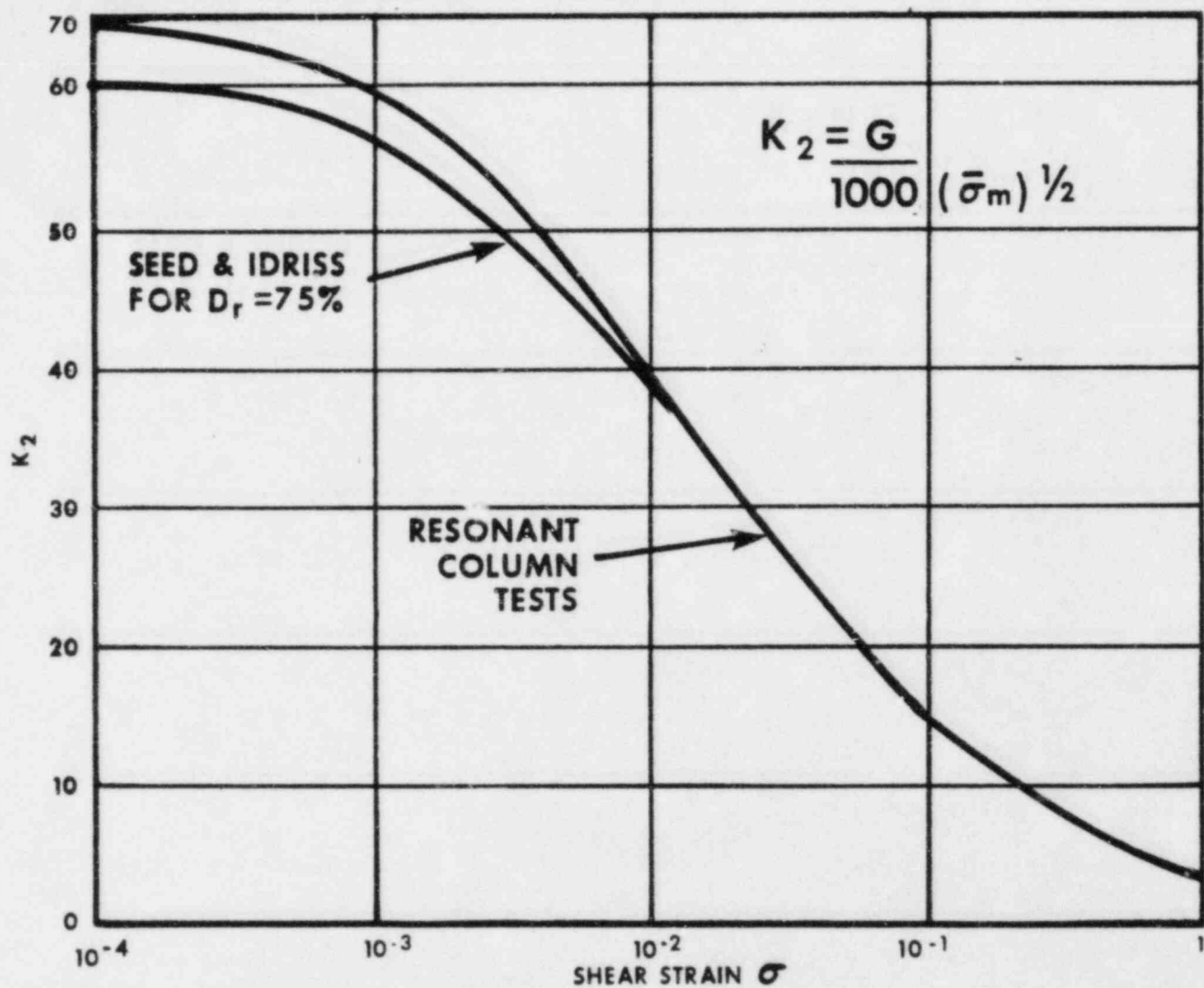


FIGURE 2.5.4-73
SHEAR MODULUS CURVE TYPE 2
SOIL (STRUCTURAL BACKFILL AND
BASAL TILL)
MILLSTONE NUCLEAR POWER STATION
UNIT 3
FINAL SAFETY ANALYSIS REPORT

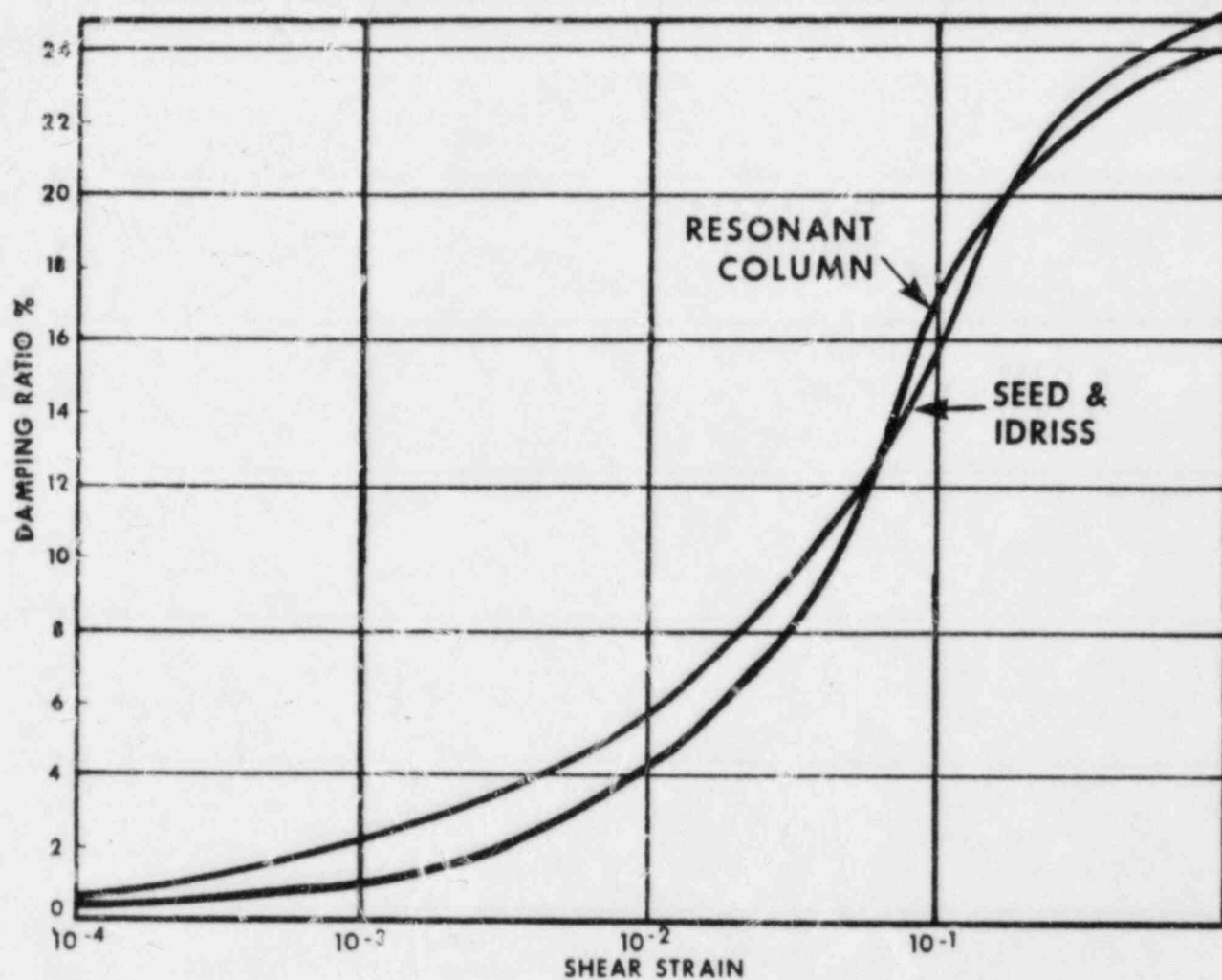


FIGURE 2.5.4-74
 DAMPING CURVE TYPE 2 SOIL
 (STRUCTURAL BACKFILL AND
 BASAL TILL)
 MILLSTONE NUCLEAR POWER STATION
 UNIT 3
 FINAL SAFETY ANALYSIS REPORT

ADDITIONAL INFORMATION ON NRC QUESTION 241.4
FOR MR. JOHN CHEN AT NRC

Q241.4 Variation of shear wave velocity for basal till. (Section 2.5.4.4.3)

The basal till at the Millstone site is a very dense, glacially deposited material consisting of a widely graded mixture of cobble and boulder sized rock fragments, gravel, sand, and some silt binder. Seismic velocity measurements of the basal till were conducted at separate times using the explosive source method and the impact source method.

Data from these tests clearly indicate that the till is very dense with a shear velocity in excess of 2,000 fps. The explosive source crosshole tests were performed on 8 boreholes in the main plant area. The average value of shear wave velocity for the basal till from these tests was 2,200 ft/sec, measured in the vicinity of the turbine building. Impact source crosshole tests were performed in 3 borings in the vicinity of the Unit 1 ventilation stack. Three shear wave velocity measurements were made in the basal till at a depth of 33 feet. The measured values were 1,246 ft/sec, 2,387 ft/sec, and 2,727 ft/sec. The lowest value, from test 408-410-33a, is not considered to be representative of the basal till because of the poor definition of the first shear wave arrival. The shear wave arrival is better defined in tests 408-409-33b and 408-409-33c. The measured shear wave velocities for these tests, 2,387 ft/sec and 2,272 ft/sec respectively, are more indicative of expected shear wave velocities of very dense material, such as the Millstone site basal till.

The raw data for crosshole tests in the basal till are attached for reference. The value of G_{max} used in dynamic analyses is 20,160 KFS, which is calculated from a shear wave velocity of 2,100 ft/sec. Based on test data from the site discussed herein, this value provides a conservative representation of the shear wave velocity of the basal till.

Variations in shear modulus are considered in the soil-structure interaction analysis by means of a 15 percent "peak spreading" of the amplified response spectra calculated at each level of the structure. The techniques used are described in detail in FSAR Section 3.7.2.

IN-SITU VELOCITY MEASUREMENT RECORD
STONE & WEBSTER ENGINEERING CORP.

TEST LOG NO.

408-409-33a

SITE

Millstone 3

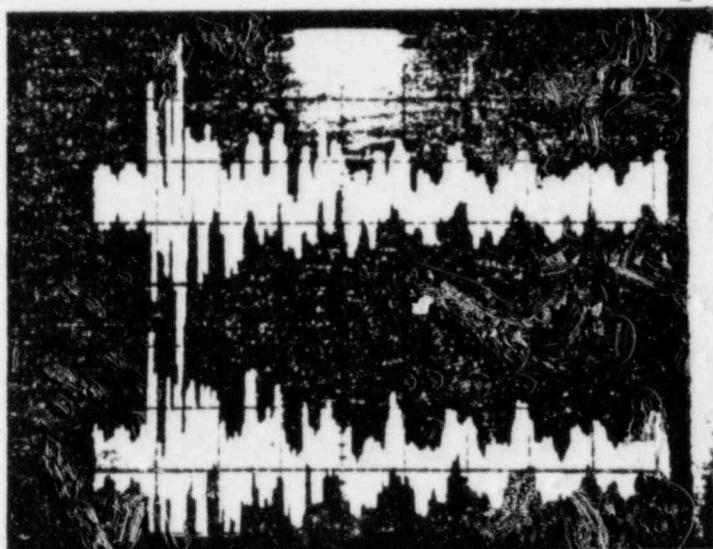
INSPECTORS

Wasker/Vetere

DATE

5/7/80

TEST DESCRIPTION

☒ CROSS HOLE ☐ UPHOLE ☐ DOWN HOLE ☐ DRILL ROD CORRECTION MEASUREMENTIMPACT BORING NO. 408 SURFACE ELEV. 14.36 IMPACT DEPTH 33.5'RECEIVER BORING NO. 407 SURFACE ELEV. 14.54 GEOPHONE DEPTH 33.5'IMPACT-RECEIVER HORIZ. DISTANCE 11.4' BEARING 566WIMPACT PRODUCED WITH hammer - SPT ROD LENGTH 37.7 SPT N VALUE 18

OSCILLOSCOPE SETTINGS

VERTICAL SCALE 2.0 MVOLT/DIVHORIZONTAL SCALE 5.0 MSEC/DIV

REMARKS

VELOCITY CALCULATIONS

P WAVE ARRIVAL 0.8 DIV _____ MSECROD CORRECTION - 2.59 MSEC

P WAVE TRAVEL TIME _____ MSEC

UNCORRECTED DISTANCE 11.4CORRECTED DISTANCE 11.48S WAVE ARRIVAL 2 DIV 10 MSECROD CORRECTION - 2.59 MSECS WAVE TRAVEL TIME 7.41 MSEC $V_p =$ _____ $V_s =$ _____ $V_p =$ $V_s =$

NOTES

IN-SITU VELOCITY MEASUREMENT RECORD
STONE & WEBSTER ENGINEERING CORP.

TEST LOG NO.

408-409-336

SITE

Milestone 3

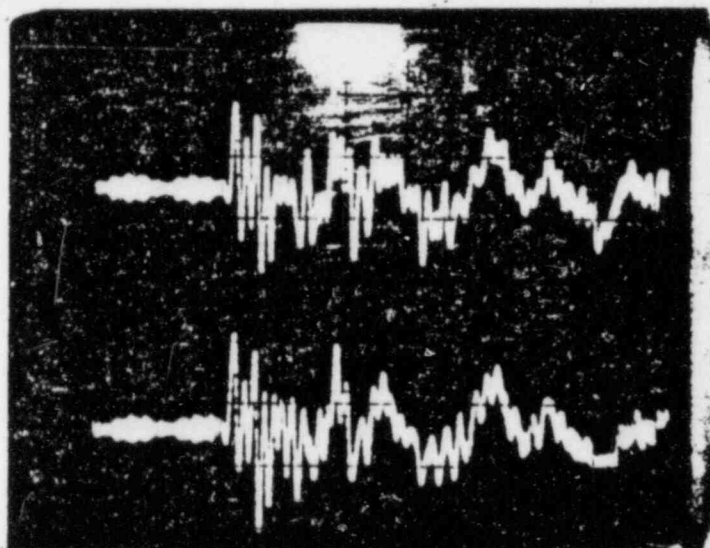
INSPECTORS

Washer/Veter

DATE

5/7/80

TEST DESCRIPTION

☒ CROSS HOLE ☐ UPHOLE ☐ DOWN HOLE ☐ DRILL ROD CORRECTION MEASUREMENTIMPACT BORING NO. 408 SURFACE ELEV. 14.36 IMPACT DEPTH 33.5RECEIVER BORING NO. 409 SURFACE ELEV. 14.54 GEOPHONE DEPTH 33.5IMPACT-RECEIVER HORIZ. DISTANCE 11.4 BEARING S60WIMPACT PRODUCED WITH hammer / SPT ROD LENGTH 37.7 SPT N VALUE 18

OSCILLOSCOPE SETTINGS

VERTICAL SCALE 5.0 MVOLT/DIV
HORIZONTAL SCALE 2.0 MSEC/DIV

REMARKS

S wave 7.4 msec (?) see
bottom trace, or alternatively
at 11.0 msec

VELOCITY CALCULATIONS

P WAVE ARRIVAL 2.05 DIV _____ MSECROD CORRECTION - 2.59 MSEC

P WAVE TRAVEL TIME _____ MSEC

UNCORRECTED DISTANCE 11.4CORRECTED DISTANCE 11.48S WAVE ARRIVAL 3.7 DIV 7.4 MSECROD CORRECTION - 2.59 MSECS WAVE TRAVEL TIME 4.31 MSEC $V_p =$ _____ $V_s =$ 2370 ft/sec $V_p =$ $V_s =$ 2387

NOTES

alternate V_s (11.0 msec) = $\frac{1356 \text{ ft/sec}}{(1365)}$

IN-SITU VELOCITY MEASUREMENT RECORD
STONE & WEBSTER ENGINEERING CORP.

TEST ID. NO.

408-409-33C

SITE

Millstone 3

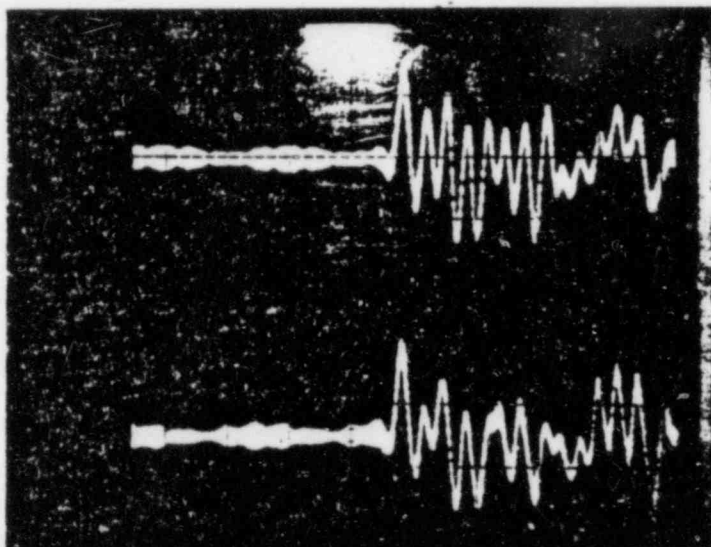
INSPECTORS

Wasker / Vetere

DATE

5/7/80

TEST DESCRIPTION

☒ CROSS HOLE ☐ UP HOLE ☐ DOWN HOLE ☐ DRILL ROD CORRECTION MEASUREMENTIMPACT BORING NO. 408 SURFACE ELEV. 14.36 IMPACT DEPTH 33.5RECEIVER BORING NO. 409 SURFACE ELEV. 14.54 GEOPHONE DEPTH 33.5IMPACT-RECEIVER HORIZ DISTANCE 11.4 BEARING S60WIMPACT PRODUCED WITH hammer / SPT ROD LENGTH 37.7 SPT N VALUE 18

OSCILLOSCOPE SETTINGS

VERTICAL SCALE 5.0 MVOLT/DIVHORIZONTAL SCALE 1.0 MSEC/DIV

REMARKS

appears to be best
record on both P & S
under the 6.8 msec
arrival is just a later P
phase (subverted from top
of wave?)

VELOCITY CALCULATIONS

P WAVE ARRIVAL 4.1 DIV 4.1 MSECROD CORRECTION - 2.59 MSECP WAVE TRAVEL TIME 1.51 MSECUNCORRECTED DISTANCE 11.4CORRECTED DISTANCE 11.48S WAVE ARRIVAL 6.8 DIV 6.8 MSECROD CORRECTION - 2.59 MSECS WAVE TRAVEL TIME 4.21 MSEC $V_p = 7550 \text{ FT/sec}$ $V_s = 2708 \text{ 7/sec}$ $V_p = 7603$ $V_s = 2727$

NOTES P & S arrivals from 408-409-33C

P-wave = 1.51 MSEC
S-wave = 4.21 MSEC

IN-SITU VELOCITY MEASUREMENT RECORD
STONE & WEBSTER ENGINEERING CORP.

TEST LOG NO.

408-410-33a

SITE

Millstone 3

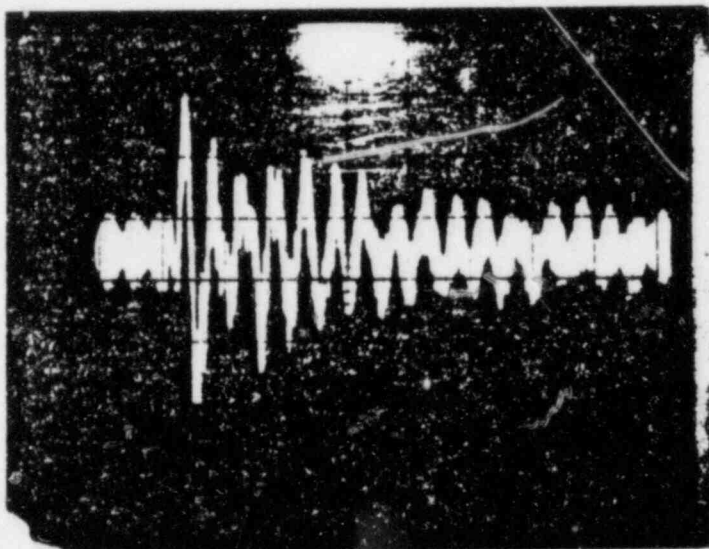
INSPECTORS

Wardner/Victoria

DATE

5/7/80

TEST DESCRIPTION

☒ CROSS HOLE ☐ UP HOLE ☐ DOWN HOLE ☐ DRILL ROD CORRECTION MEASUREMENTIMPACT BORING NO. 408 SURFACE ELEV. 14.36 IMPACT DEPTH 33.5RECEIVER BORING NO. 410 SURFACE ELEV. 14.54 GEOPHONE DEPTH 33.5IMPACT-RECEIVER HORIZ DISTANCE 23.6 BEARING S 61 WIMPACT PRODUCED WITH hammer SPT ROD LENGTH 37.7 SPT N VALUE 18

OSCILLOSCOPE SETTINGS

VERTICAL SCALE 2.0 MVOLT/DIVHORIZONTAL SCALE 5.0 MSEC/DIV

REMARKS

S-wave arrival
difficult to determine

VELOCITY CALCULATIONS

P WAVE ARRIVAL 1.2 DIV 6.0 MSECROD CORRECTION - 2.59 MSEC

P WAVE TRAVEL TIME _____ MSEC

UNCORRECTED DISTANCE 23.6CORRECTED DISTANCE 24.18S WAVE ARRIVAL 4.4 DIV 22 MSECROD CORRECTION - 2.59 MSECS WAVE TRAVEL TIME 19.41 MSEC $V_p =$ _____ $V_s =$ 1216 ft/sec $V_p =$ $V_s =$ 1246

NOTES

Alternate S-wave:
@ 2.6 divisions \Rightarrow 13 MSEC \Rightarrow 2267 ft/sec

408-410-336

5/7/80

IN-SITU VELOCITY MEASUREMENT RECORD
STONE & WEBSTER ENGINEERING CORP.

SITE

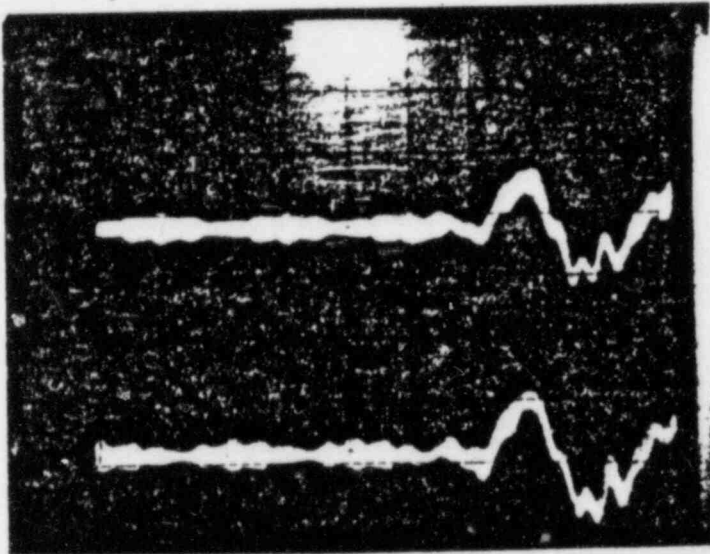
Millstone 3

INSPECTORS

Wasker / Vetere

TEST DESCRIPTION

☒ CROSS HOLE ☐ UPHOLE ☐ DOWN HOLE ☐ DRILL ROD CORRECTION MEASUREMENT
IMPACT BORING NO. 408 SURFACE ELEV. 14.36 IMPACT DEPTH 33.5
RECEIVER BORING NO. 410 SURFACE ELEV. 14.35 GEOPHONE DEPTH 33.5
IMPACT-RECEIVER HORIZ. DISTANCE 23.6 BEARING _____
IMPACT PRODUCED WITH hammer SPT ROD LENGTH 37.7 SPT N VALUE 18



OSCILLOSCOPE SETTINGS

VERTICAL SCALE 5.0 MVOLT/DIV
HORIZONTAL SCALE 1.0 MSEC/DIV

REMARKS

VELOCITY CALCULATIONS

P WAVE ARRIVAL 5.8 DIV 5.8 MSEC

ROD CORRECTION - 2.59 MSEC

P WAVE TRAVEL TIME 3.21 MSEC

UNCORRECTED DISTANCE 23.6

CORRECTED DISTANCE 24.13

S WAVE ARRIVAL _____ DIV _____ MSEC

ROD CORRECTION - 2.59 MSEC

S WAVE TRAVEL TIME _____ MSEC

 $V_p = 7352 \frac{\text{ft}}{\text{sec}}$ $V_s =$
 $V_p = 7539$
 $V_s =$

NOTES

IN-SITU VELOCITY MEASUREMENT RECORD
STONE & WEBSTER ENGINEERING CORP.

TEST LOG NO.

408-410-33c

SITE

millstone 3

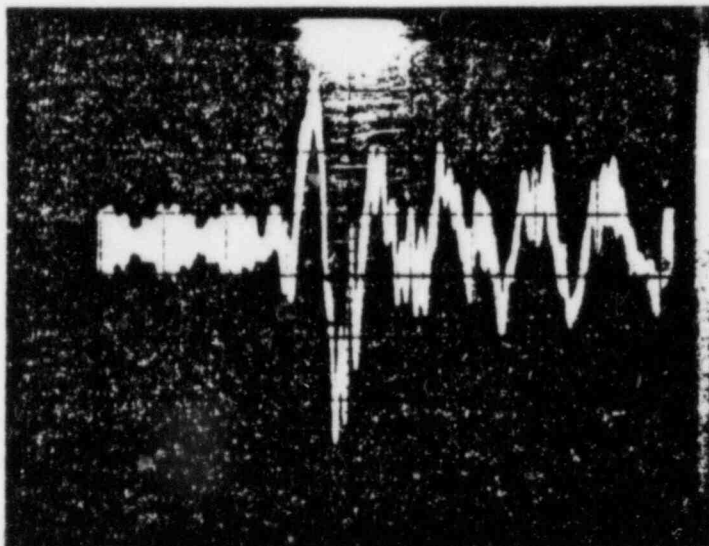
INSPECTORS

Wasker / Vetere

DATE

5/7/80

TEST DESCRIPTION

☒ CROSS HOLE ☐ UPHOLE ☐ DOWN HOLE ☐ DRILL ROD CORRECTION MEASUREMENTIMPACT BORING NO. 408 SURFACE ELEV. 14.36 IMPACT DEPTH 33.5RECEIVER BORING NO. 410 SURFACE ELEV. 14.36 GEOPHONE DEPTH 33.5IMPACT-RECEIVER HORIZ DISTANCE 2.3' BEARING S60WIMPACT PRODUCED WITH hammer SPT ROD LENGTH 37.7 SPT N VALUE 18

OSCILLOSCOPE SETTINGS

VERTICAL SCALE 2.0 MVOLT/DIVHORIZONTAL SCALE 2.0 MSEC/DIV

REMARKS

VELOCITY CALCULATIONS

P WAVE ARRIVAL 2.95 DIV _____ MSECROD CORRECTION - 2.59 MSEC

P WAVE TRAVEL TIME _____ MSEC

UNCORRECTED DISTANCE _____ $V_p =$ _____ $V_s =$ _____

CORRECTED DISTANCE _____

S WAVE ARRIVAL _____ DIV _____ MSEC

ROD CORRECTION - 2.59 MSEC

S WAVE TRAVEL TIME _____ MSEC

 $V_p =$ $V_s =$

NOTES

IN-SITU VELOCITY MEASUREMENT RECORD
STONE & WEBSTER ENGINEERING CORP.

408-410-33d

SITE

Millstone 3

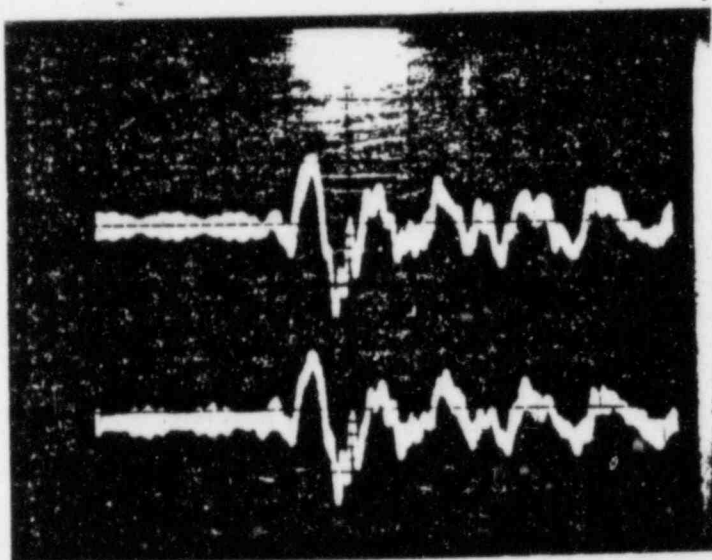
INSPECTORS

Wasker/Vetere

DATE

5/7/80

TEST DESCRIPTION

☒ CROSS HOLE ☐ UPHOLE ☐ DOWN HOLE ☐ DRILL ROD CORRECTION MEASUREMENT
IMPACT BORING NO. 408 SURFACE ELEV. 14.36 IMPACT DEPTH 33.5RECEIVER BORING NO. 410 SURFACE ELEV. 14.36 GEOPHONE DEPTH 33.5IMPACT-RECEIVER HORIZ DISTANCE 23.6 BEARING _____IMPACT PRODUCED WITH hammer SPT ROD LENGTH 37.7 SPT N VALUE 18

OSCILLOSCOPE SETTINGS

VERTICAL SCALE 5.0 MVOLT/DIVHORIZONTAL SCALE 2.0 MSEC/DIV

REMARKS

see log 33c

VELOCITY CALCULATIONS

P WAVE ARRIVAL _____ DIV _____ MSEC

ROD CORRECTION - 2.59 MSEC

P WAVE TRAVEL TIME _____ MSEC

UNCORRECTED DISTANCE _____

CORRECTED DISTANCE _____

S WAVE ARRIVAL _____ DIV _____ MSEC

ROD CORRECTION - _____ MSEC

S WAVE TRAVEL TIME _____ MSEC

 $V_p =$ _____ $V_s =$ _____ $V_p =$ $V_s =$

NOTES

NRC Letter: May 3, 1983 1.8

Question No. Q241.6 (Section 2.5.4.7 and SRP Section 2.5.4) 1.11

Sliding Stability 1.12

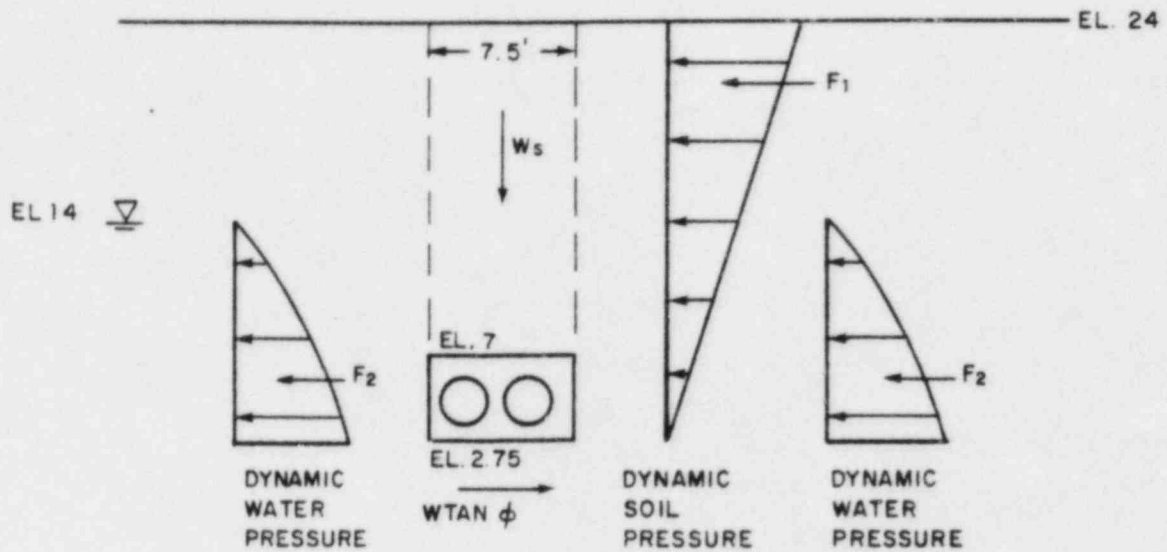
You state that the service water encasement has been analyzed for sliding stability due to seismic loading. Provide the details of the analysis and identify the cross-section used in your analysis. 1.13
1.14

Response: 1.15

Sliding stability of the service water encasement was analyzed for two conditions; i.e., at the circulating and service water pumphouse where site grade is at elevation 14 feet and at the turbine building where site grade is at elevation +24 feet. In both cases, driving forces were assumed to consist of the dynamic lateral soil pressure and the dynamic water pressure. The load diagrams shown on Figure Q241.6-1 were calculated using the equations presented in Figure 2.5.4-43. An additional component of dynamic water pressure was conservatively added as a driving force to account for negative pressures that may result in a suction force being applied to the encasement. The resisting forces consist mainly of the frictional resistance on the base of the encasement due to the weight of the encasement and the soil column above. Static loads were assumed to be balanced and were not considered in the analysis. 1.16
1.17
1.19
1.22
1.23
1.24
1.25
1.27

The factor of safety for the encasement adjacent to the turbine building was calculated to be 1.35. The section analyzed adjacent to the pumphouse has a calculated safety factor of 2.9. 1.28
1.29

CASE 1
SERVICE WATER PIPE ENCASEMENT
SLIDING STABILITY
AT TURBINE BUILDING



CASE 2
SERVICE WATER PIPE ENCASEMENT
SLIDING STABILITY
AT PUMPHOUSE

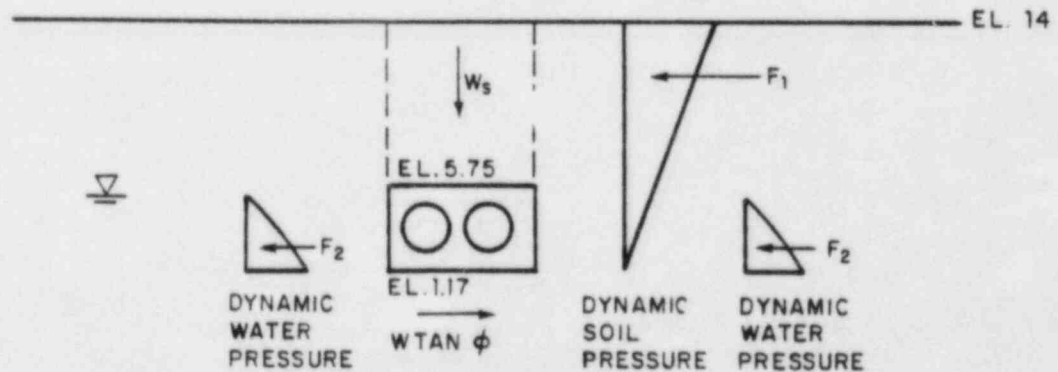
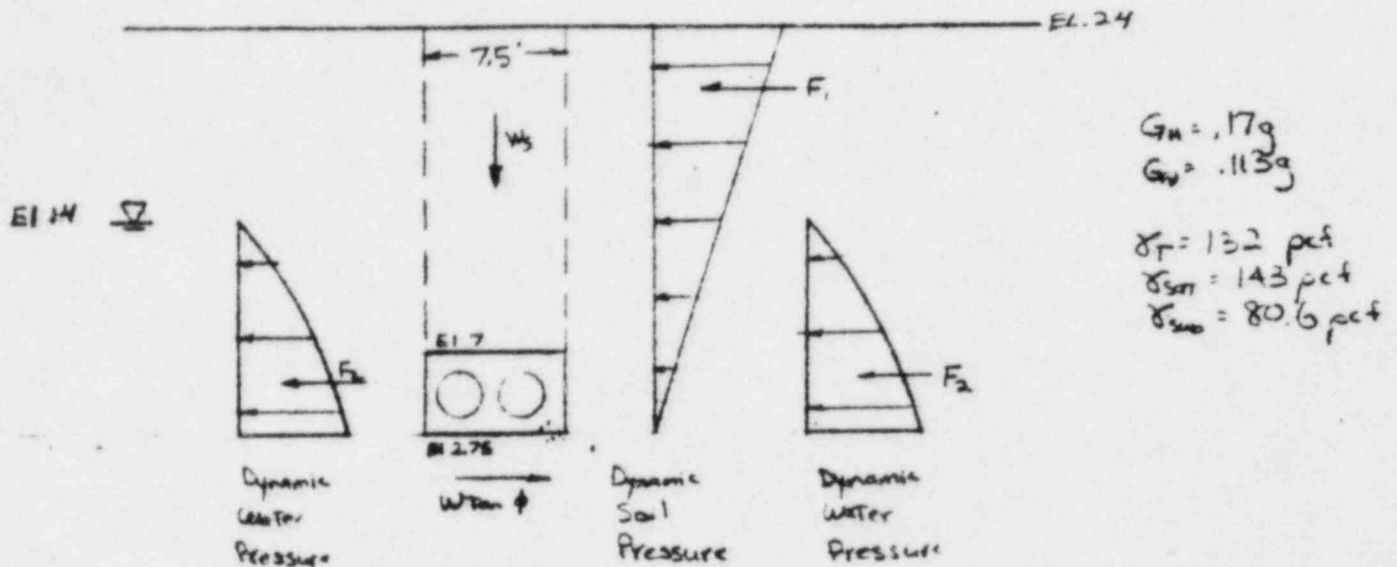


FIGURE Q241.6-1
SLIDING STABILITY OF THE
SERVICE WATER ENCASEMENT
MILLSTONE NUCLEAR POWER STATION
UNIT 3
FINAL SAFETY ANALYSIS REPORT

Service Water Pipe Encasement SLIDING STABILITY at Turbine Building

Q241.6



$$G_H = .17g$$

$$G_W = .113g$$

$$\gamma_T = 132 \text{ pcf}$$

$$\gamma_{sm} = 143 \text{ pcf}$$

$$\gamma_{sw} = 80.6 \text{ pcf}$$

$$F_1 = \frac{3}{4} G_H (H_T \gamma_T - \frac{H_W^2}{H_T} \gamma_W) = 3612.5 \text{ lb/ft}$$

$$F_2 = 0.7 \left[\left(\frac{3}{2} \right) G_W \gamma_W H_W^2 \right] = 548.2 \text{ lb/ft}$$

$$\text{Weight service water pipe encasement} = 3805.9 \text{ lb/ft}$$

$$\text{Buoyant force} = 1987.3 \text{ lb/ft}$$

$$\text{Net weight pipe encasement} = 1816.6 \text{ lb/ft}$$

$$\text{Weight soil} = W_s = 14,131.5 \text{ lb/ft}$$

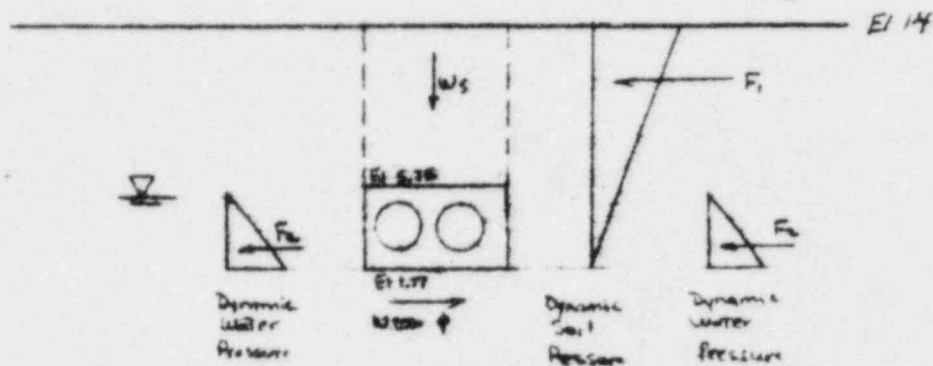
$$\tan \phi = 0.4$$

$$FS = \frac{\text{Resisting Forces}}{\text{Driving Forces}} = \frac{(1816.6 + 14131.5)(0.4)}{3612.5 + 2(548.2)} = 1.35$$

to be send to
Mr. Chen.
Do not incorporate
into FSAR

Service Water Pipe Encasement STABILITY

at Pamphouse



$$F_1 = \frac{3}{4} G_m (H_w \alpha_r - \frac{H_w^2}{2} \alpha_r) = 1326.6 \text{ lb/ft}$$

$$F_2 = 0.7 [(1/12) G_m \alpha_w H^3] = 63.5 \text{ lb/ft}$$

$$\text{Weight service water pipe encasement} = 4166 \text{ lb/ft}$$

$$\text{Buoyant force} = 1792.4 \text{ lb/ft}$$

$$\text{Net weight pipe encasement} = 2373.6 \text{ lb/ft}$$

$$\text{Weight soil} = 8167.5 \text{ lb/ft}$$

$$\tan \phi = 0.4$$

$$FS = \frac{\text{Resisting Forces}}{\text{Driving Forces}} = \frac{(2373.6 + 8167.5) 0.4}{1326.6 + 2(63.5)} = 2.9$$

2 of 2

7	6	5	4	3	2	1	STOP

DOCUMENT/ PAGE PULLED

ANO. 8404240344

NO. OF PAGES 1

REASON

☐ PAGE ILLEGIBLE

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☐ BETTER COPY REQUESTED ON _____

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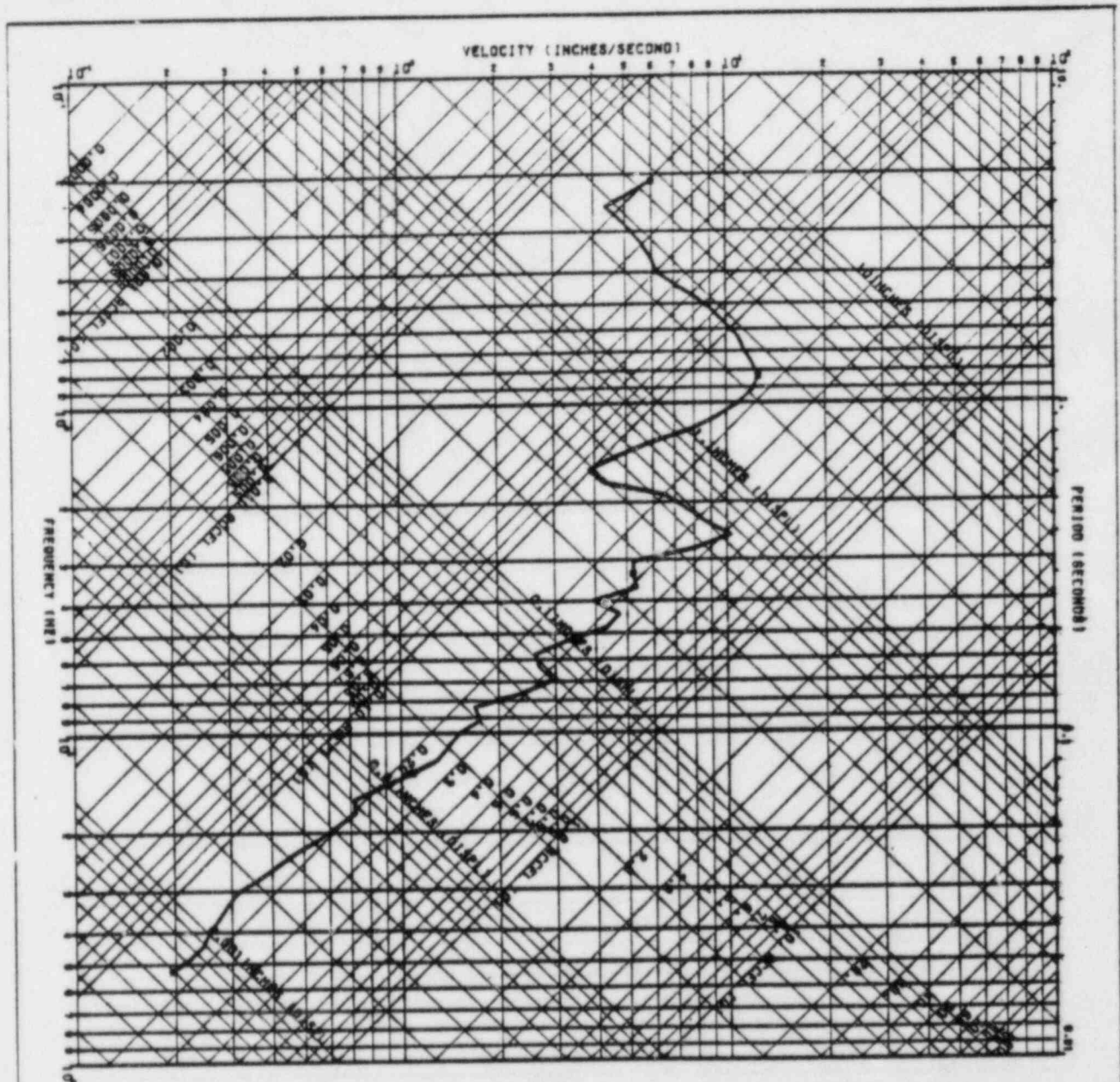
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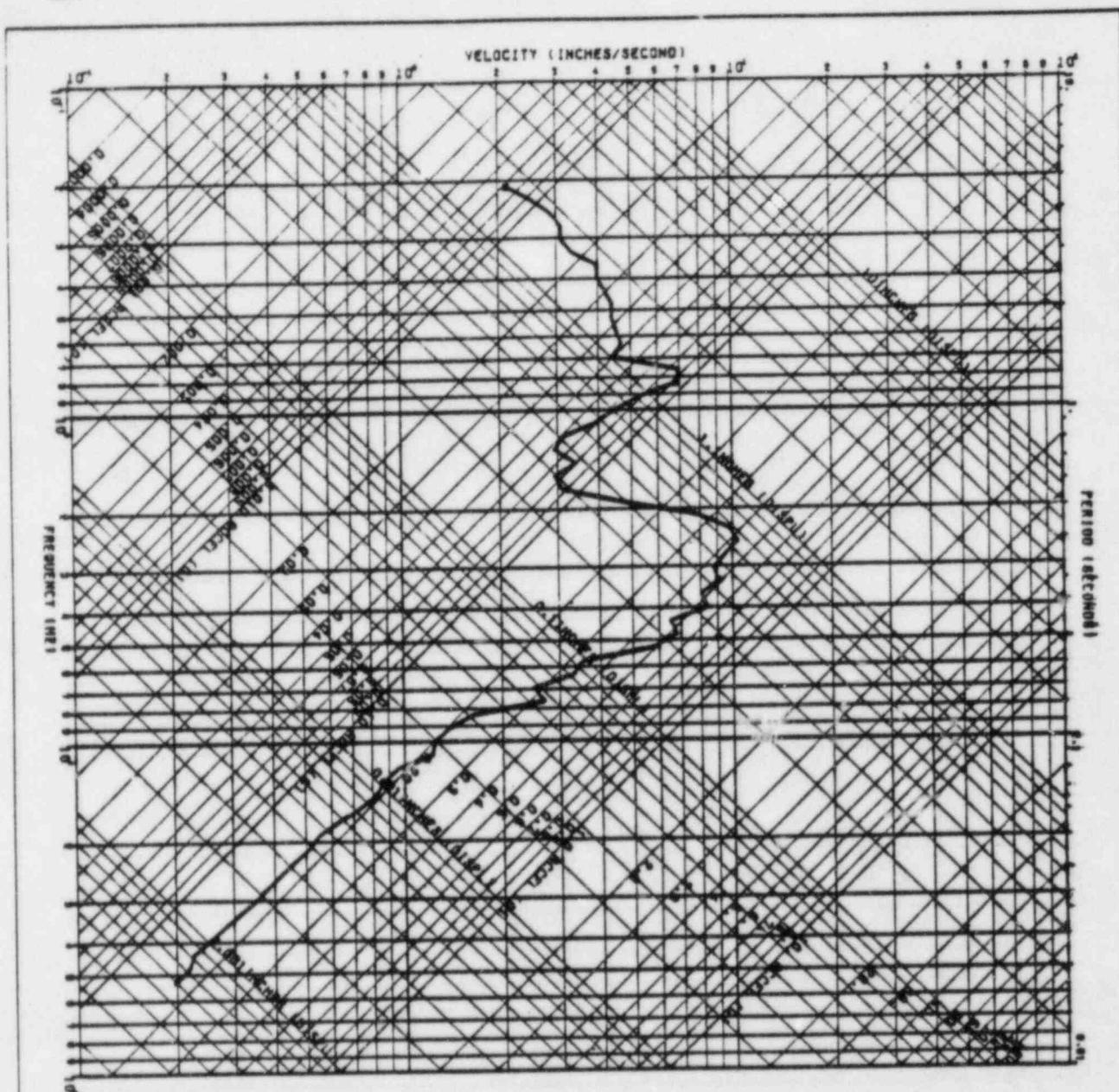
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Q241.8



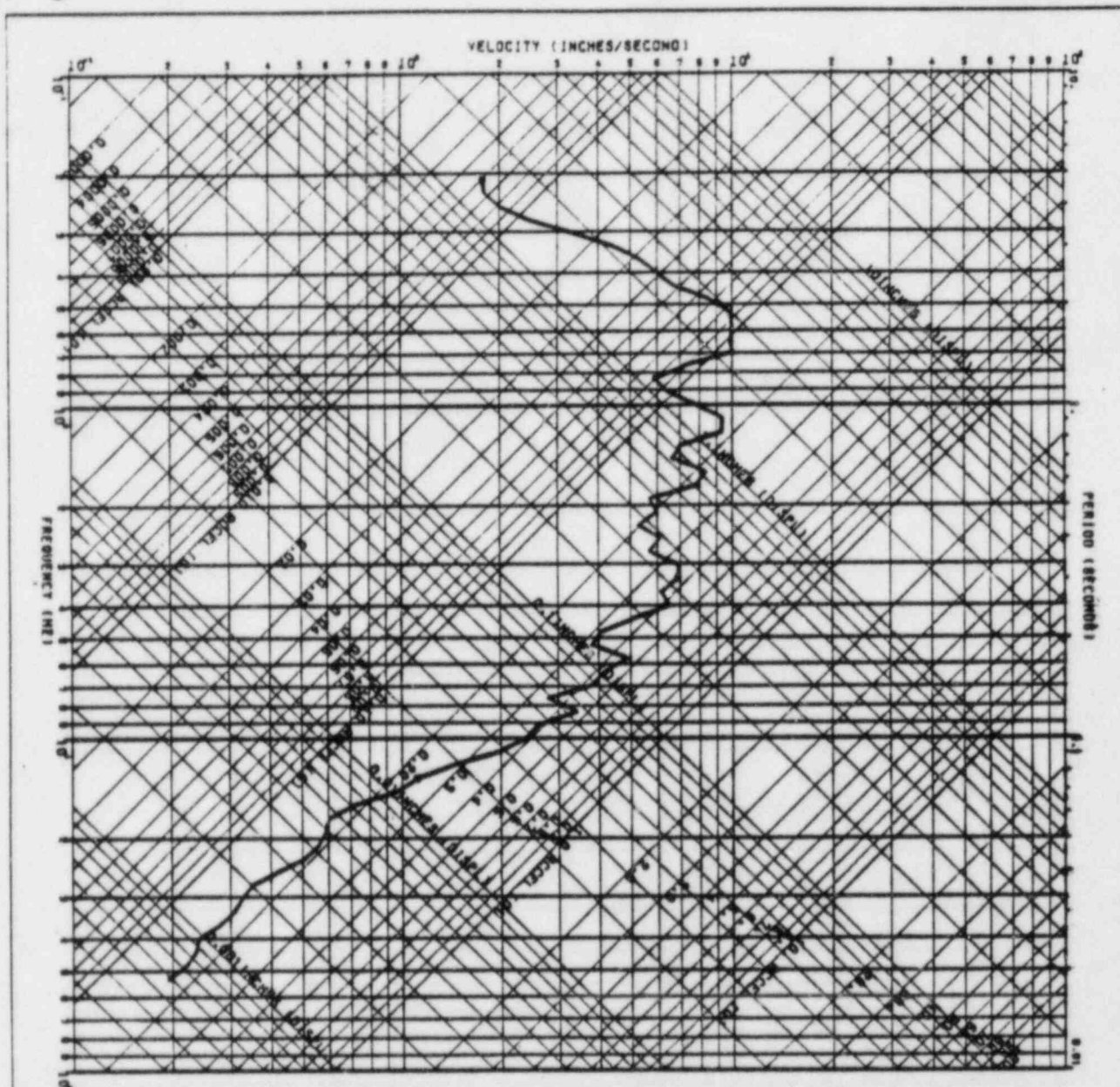
11C041 71.001.0 PACOINA OAR, CAL. COMP 516E
DAMPING RATIOS
@ 0.050

STONE & WEBSTER ENGINEERING CORPORATION		PAGE NO. <u>1</u>
RESPONSE SPECTRA		PRELIMINARY
		ITEM
CLIENT	NUSCO - MILLSTONE NUCLEAR POWER STATION	J.O. NO. CALC 12179-1276
SUBJECT	BEACH AREA SHAKE ANALYSIS	DATE
	BEACH AREA, 40 FT SAND PROFILE	CHECKED
		BY
BASED ON COMPUTER RUN S5664566 ON 02/17/84 AT 14.00.30 BY FS-VETERE		
PROGRAM ST-211 SHAKE - VER 05 LEV 02 - COMPILED ON 83.350 AT 12.21.30		



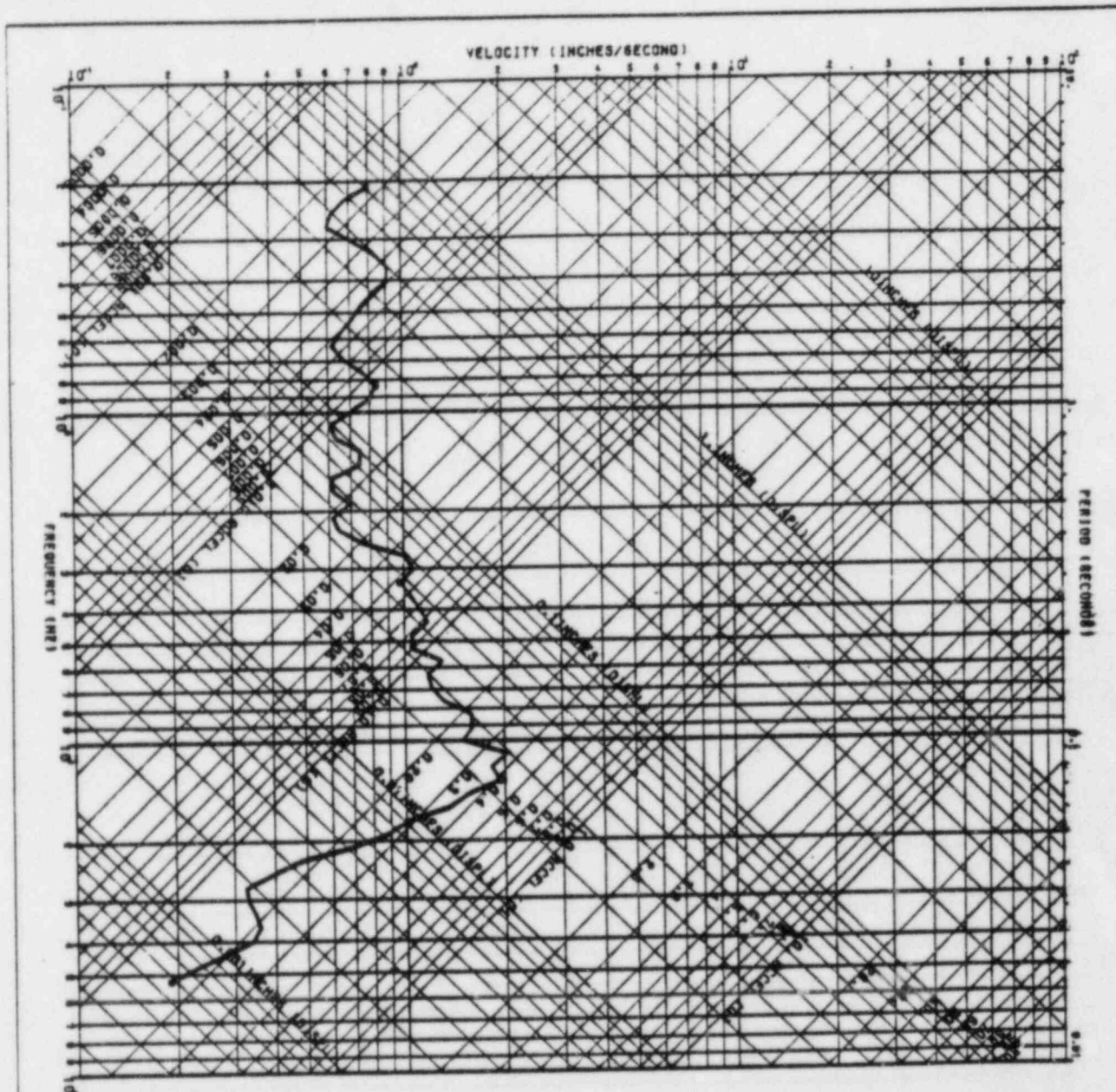
118037 66.006.0 TENBLOR, CALIFORNIA NO. 2 CORP HSBH
DAMPING RATIOS
0.050

STONE & WEBSTER ENGINEERING CORPORATION		PAGE NO. <u>5</u>
RESPONSE SPECTRA		PRELIMINARY <u> </u>
		ITEM <u> </u>
CLIENT	NUSCO - MILLSTONE NUCLEAR POWER STATION	J.O. NO. CALC 12179-1276
SUBJECT	BEACH AREA SHAKE ANALYSIS	DATE BY
	BEACH AREA, 40 FT SAND PROFILE	CHECKED BY
BASED ON COMPUTER RUN S5664569 ON 02/17/84 AT 14.02.50 BY FS-VETERE		
PROGRAM ST-211 SHAKE - VER 05 LEV 02 - COMPILED ON 83.350 AT 12.21.30		



118032 55.001.0 OLYMPIA, WASHINGTON HWY TEST LAB CORP 586H
 DAMPING RATION
 @ 0.050

STONE & WEBSTER ENGINEERING CORPORATION		PAGE NO. <u>2</u>	
RESPONSE SPECTRA		PRELIMINARY <u> </u>	
		ITER <u> </u>	
CLIENT	NUSCO - MILLSTONE NUCLEAR POWER STATION	J.O. NO.	CALC 12179-1276
SUBJECT	BEACH AREA SHAKE ANALYSIS	DATE	BY
	BEACH AREA 40 FT SAND PROFILE	CHECKED	BY
BASED ON COMPUTER RUN 55664563 ON 02/17/84 AT 13.58.22 BY FS-VETERE			
PROGRAM ST-211 SHAKE - VER 05 LEV 02 - COMPILED ON 83.350 AT 12.21.30			



110296 35.002.0 FEDERAL BUILDING, HELENA, MONTANA CORP EAST
DAMPING RATIOS
0.050

STONE & WEBSTER ENGINEERING CORPORATION		PAGE NO. 7	
RESPONSE SPECTRA		PRELIMINARY	
		ITER	
CLIENT	NUSCO - MILLSTONE NUCLEAR POWER STATION	J.O. NO.	CALC 12179-1276
SUBJECT	BEACH AREA SHAKE ANALYSIS	DATE	BY
	BEACH AREA, 40 FT SAND PROFILE	CHECKED	BY
BASED ON	COMPUTER RUN S5864564 ON 02/17/84 AT 13.59.45 BY FS-VETERE		
PROGRAM	ST-211 SHAKE - VER 05 LEV 02 - COMPILED ON 83.350 AT 12.21.30		

sets, and were probably removed during blasting and scaling operations. 3.20

Figure 2.5.5-6 shows the plan view of potential failure wedges that could develop on the west side of the containment excavation. Geologic mapping of this area (see Figure 2.5.4-6) does not exhibit a high density of fractures that could produce these wedges. Figure 2.5.5-7 is a photograph showing the rock surface commonly found in the area of the main steam valve building. 3.21 3.22 3.23 3.25 241.17

The forces applied by the rock wedges on the ring beam are shown on Figure 2.5.5-3. The maximum forces act in the southwest quadrant, due to the effect of the weaker foliation planes which dip into the excavation face in this area. Other areas of instability can be attributed to the high dip angles of the jointing, which are inherently unstable when subjected to seismic and surcharge loadings. The design of the structural support, or ring beam, which transfers this load around the excavation, maintaining the isolation of containment structure from these external loads, is discussed in detail in Section 3.8.1.1. 3.27 3.28 3.29 3.31 3.32 3.33 3.35 3.36 3.37

2.5.5.3 Logs of Borings 3.39

All boring logs are included in Appendix 2.5J. No borings were taken in borrow areas for materials used onsite. 3.41

2.5.5.4 Compacted Fill 3.43

Structural backfill used to raise the shoreline slopes to final design lines meets the requirements outlined in Section 2.5.4.5.2. 3.44 3.45

2.5.5.5 References for Section 2.5.5 3.48

Bishop, A.W., 1955. The use of the Slip Circle in the Stability Analysis of Slopes. Geotechnical, Vol V. 3.50 3.51

Goldsmith, R. 1967. Bedrock Geologic Map of Niantic Quadrangle, Penn. U.S. Geological Survey Quadrangle Map GQ-575, Washington, D.C. 3.54 3.55

Hendron, A.J.; Cording, E.J.; Aiyer, A.K., 1971. Analytical and Graphical Methods for the Analysis of Slopes in Rock Masses. NCG Technical Report No. 36. 3.57 3.58

Northeast Nuclear Energy Co. (NNECo.), 1975. Geologic Mapping of Bedrock Surface. Millstone Nuclear Power Station - Unit 3, Docket No. 50-423. 4.2 4.3

Stone & Webster Engineering Corporation (SWEC), 1974a. Stereographic Projection of Joints (JTPlot). Computer Program ST-212. SWEC, Boston, Mass. 4.5 4.7

MNPS-3 FSAR

Stone & Webster Engineering Corporation (SWEC), 191974b. Analytical	4.9
Method for Analysis of Stability of Rock Slopes; SWARS-2P. Computer	4.10
Program ST-214. SWEC, Boston, Mass.	4.11
Stone & Webster Engineering Corporation (SWEC), 1977. Limiting	4.13
Equilibrium Analysis on Soil Engineering, LEASE II. Computer Program	4.14
GS-018. SWEC, Boston, Mass.	4.15
Stone & Webster Engineering Corporation (SWEC), 1979. Seismically	4.17
Induced Displacements of Embankments and Slopes, Side. Computer	4.18
Program GT-009. SWEC, Boston, Mass.	4.19
U.S. Army Coastal Engineering Research Center, 1975. Shore	4.21
Protection Manual. Department of the Army, Corps of Engineers,	4.22
Washington, D.C. Vol II, Ch. 7.	
Whitman, R.V. and Bailey, W.A., 1967. Use of Computers for Slope	4.24
Stability Analyses. Journal of Soil Mechanics and Foundations	4.25
Division, ASCE. Vol 93, SM4.	4.26
Whitman, R.V. and Moore, P.J., 1963. Thoughts Concerning the	4.28
Mechanics of Slope Stability Analyses. Proceedings, Second Pan	4.29
American Conference on Soil Mechanics and Foundation Engineering.	

241.8

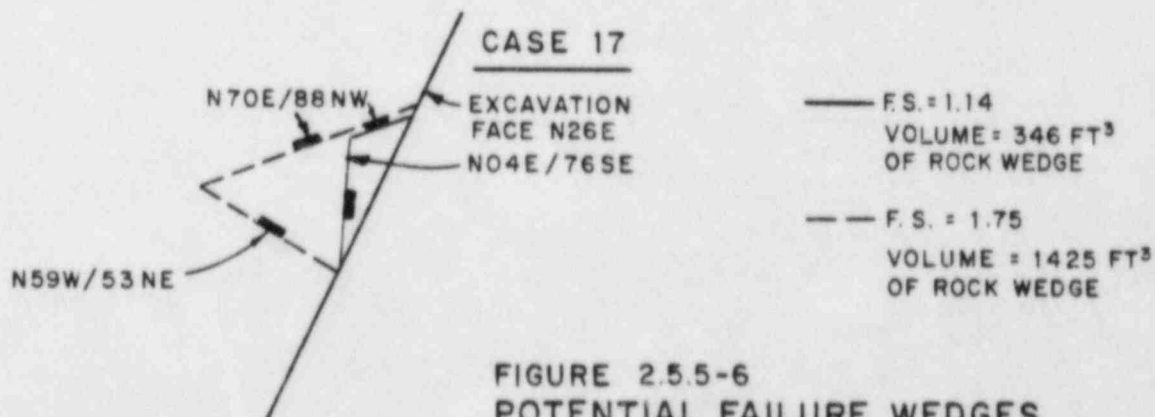
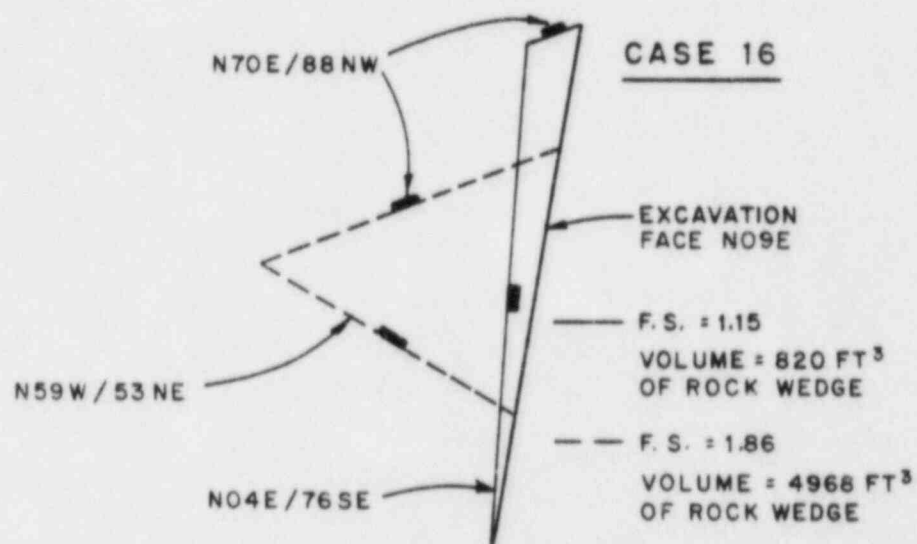
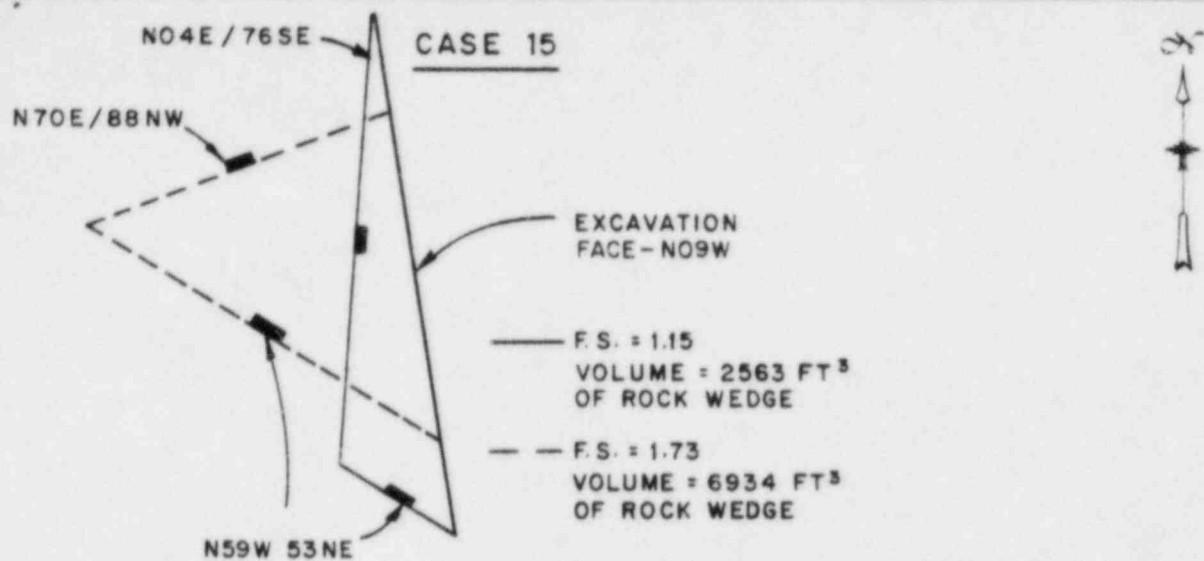


FIGURE 2.5.5-6
POTENTIAL FAILURE WEDGES
WEST SIDE OF
CONTAINMENT EXCAVATION
MILLSTONE NUCLEAR POWER STATION
UNIT 3
FINAL SAFETY ANALYSIS REPORT

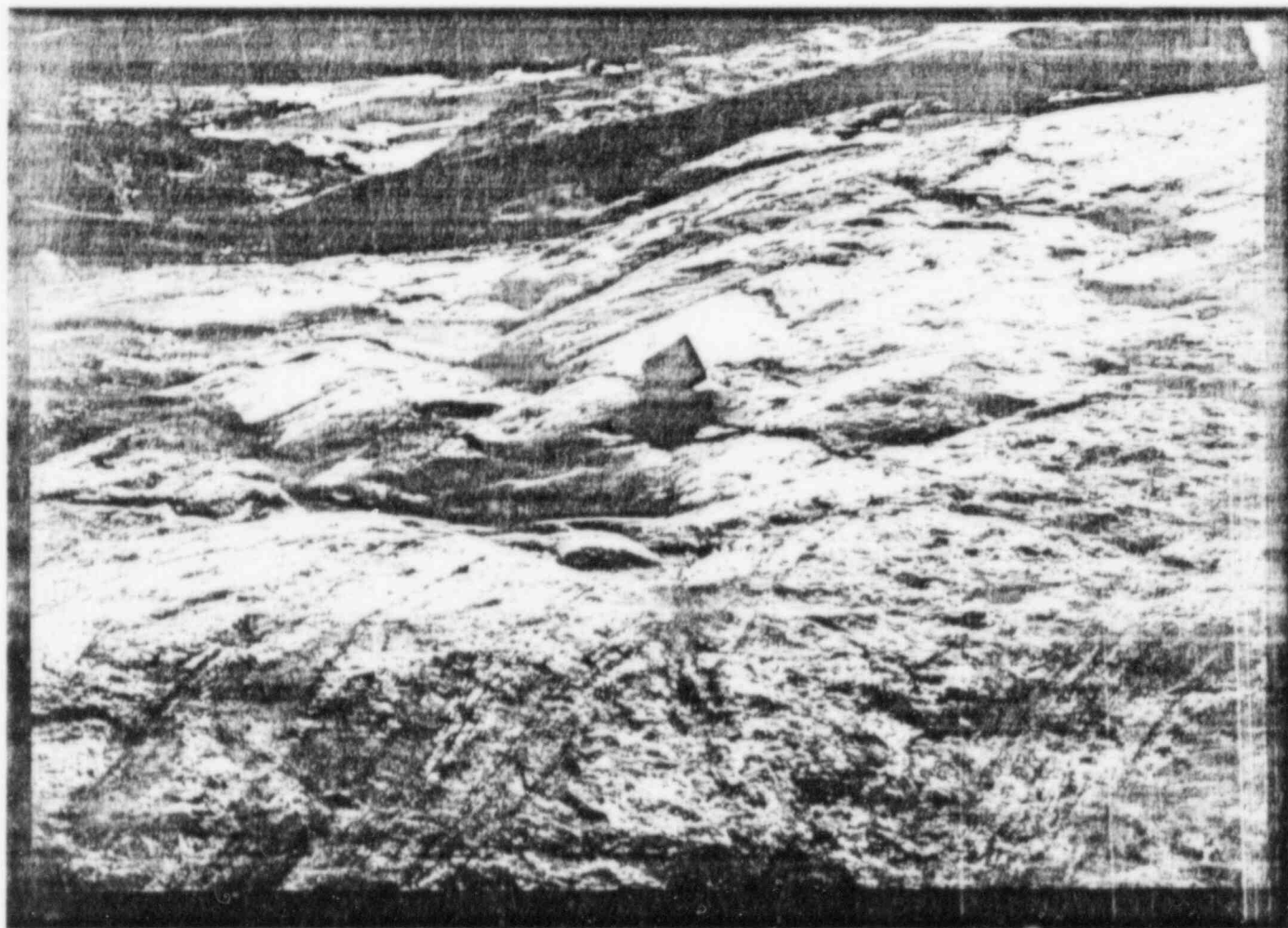


FIGURE 2.3.3-7
ROCK SURFACE NEAR NORTH EDGE
OF MAIN STEAM VALVE BUILDING
Millstone Nuclear Power Station
Unit 3
Final Safety Analysis Report

Soil strength properties used in the stability analysis were selected 1.11
on the basis of standard penetration tests and of cyclic triaxial and 1.12
consolidated undrained (CIU) triaxial tests on undisturbed samples,
as reported in Appendixes 2.5G and 2.5F, respectively. The effect of 1.16
possible pore pressure buildup in the beach and outwash sands was
accounted for in the stability analysis for the post-earthquake 1.17
conditions.

A static slope stability analysis was conducted using the assumptions 1.18
described above together with strengths for the various slope 1.19
materials as shown on Figure 2.5.5-1. The effective internal 1.20
friction angles (ϕ) assigned to the beach and glacial outwash sand —
were selected on the basis of standard penetration tests and of the 1.21
CIU triaxial tests on undisturbed samples. The CIU tests (see 1.22
Appendix 2.5G and Figure 2.5.5-5) revealed effective internal
friction angles of 33 to more than 40 degrees for the samples tested. 1.24
On internal friction, an angle of 34 degrees was used in the 1.25
analysis. The minimum factor of safety against slope failure for the 1.26
static case is 2.9, which is adequate. The dynamic slope stability 1.27
during the SSE was evaluated by using a pseudo-static approach and
undrained shear strengths of the soils. Input horizontal and 1.29
vertical accelerations of 0.25g and 0.17g were based on the average
amplified accelerations described in Sections 2.5.4.8.3.1 and 1.30
2.5.4.9. Acceleration directions were selected to maximize 1.31
instability.

Undrained strength parameters for the beach and glacial outwash sands 1.32
were derived based on undrained triaxial compression test results 1.33
reported in Appendix 2.5G. Stress paths and data from these tests 1.34
indicate that during undrained loading the average A parameter at
maximum obliquity $[(\sigma/\sigma_3)_{MAX}]$ was 0.13. The values of A ranged from 1.36
+0.33 to -0.16. An A' parameter equal to 0.5 and an internal 1.37
friction angle of 34° were used to derive the undrained strengths of
the beach and glacial outwash sands. These values are considered to 1.39
be conservative based on the in situ density and loading conditions.
The derived undrained strengths which are modeled as cohesion and a 1.40
friction angle of zero are shown on Figure 2.5.5-1. The undrained 1.41
strength is calculated as $\tau = \bar{\sigma}_v \tan \phi$ as the mid-level of each layer. —

Results of the pseudo-dynamic stability analysis indicate that the 1.42
minimum factor of safety against slope failure is 0.9 for the assumed 1.43
conditions. This result is very conservative because of the 1.44
assumptions made about slope geometry and end effects. An additional 1.45
analysis was performed on this slope considering only the horizontal
component of seismic loading, and a safety factor of 1.16 was 1.46
calculated for the same failure circle.

Due to the low factor of safety obtained, an analysis was performed 1.47
to estimate the deformations which could theoretically occur along 1.48
the postulated failure surface during earthquake loading. The 1.49
analysis is based on an approach presented by Newmark (1965) using
the computer program SIDES (Seismically Induced Displacement of 1.50
Embankments and Slopes, SWEC, 1979) which calculates the cumulative

mono-directional sliding displacement of a rigid body shaken by an earthquake. An input earthquake accelerogram is represented by a maximum 12,000 point time history of acceleration. No motion is assumed to occur within the slope until the strength of the soil is exceeded; i.e., the limiting acceleration producing a safety factor of 1.0 is exceeded. Analytical equations governing rigid solution are then solved incrementally on the assumption that the input acceleration varies linearly from point to point, and that the displacements are cumulative throughout the duration of the earthquake. Each of the three earthquakes used to compute the dynamic response of the soil were used (Section 2.5.4.8.3). Their time histories were scaled to the appropriate average amplified accelerations (i.e., vertical acceleration of 0.17g and horizontal acceleration of 0.25g) described above. Results from each of these earthquakes indicate maximum cumulative slope movements less than 0.1 inch. The limiting horizontal and vertical acceleration used were 0.2g and 0.12g, respectively. These results indicate that if there is any movement of the slope during the SSE, the movement will be negligible. There will be no adverse effect to any safety related system component or structures.

A post-earthquake stability analysis was performed to quantify the effect of pore pressure generated by the earthquake. The magnitude of pore pressure buildup was estimated from results of cyclic triaxial tests (Appendix 2.5F) considering such factors as the number of equivalent cycles, cyclic shear stress levels, confining pressures, material density, and gradation. The pore pressure buildup for 5 cycles of loading on samples which most closely represent the in situ condition and dynamic loading was between 40 and 60 percent of the effective confining pressure. An estimated pore pressure buildup of 50 percent was used to evaluate the post-earthquake slope stability. Therefore, the soil properties are the same as in the static case but the pore pressures are increased during the LEASE analysis. Results of the post-earthquake analyses reveal that the minimum factor of safety against slope failure is 1.4. This is considered acceptable. The analysis of slope stability indicates that the shoreline slope will be stable under static, dynamic, and post-earthquake conditions.

In addition to the above analysis, where the shorefront slope was considered to consist of a uniform deposit of outwash sand to el -40 feet, the actual subsurface conditions were modeled to determine whether a more critical cross-section existed due to sloping bedrock conditions at the shorefront. The actual soil profiled in this area is shown on Figure 2.5.4-52. This condition was modeled for LEASE and a static slope stability analysis was performed using both the simplified Bishop circular failure method and the Morgenstern-Price wedge failure method. The profile of the sloping rock condition used in the slope stability analysis showing soil properties and slope geometry is shown on Figure 2.5.5-4.

The minimum safety factor for static loading conditions is 3.2 for a circular arc failure surface. This compares to a safety factor of

2.9 for the circular failure surface in the analysis for a uniform depth of sand to el -40 feet indicating that the sloping rock profile is less critical than the uniform sand profile. Also, the dynamic analysis for the sloping rock profile is less critical than the uniform sand profile because the magnitude of the dynamic forces is reduced due to less amplification through the stiffer till and because of the shallower depth to bedrock. The minimum safety factor for dynamic loading conditions is comparable with the uniform sand profile when similar dynamic forces are assumed.

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241.18

Morgenstern-Price wedge failure analyses were also performed to further investigate the sloping rock profile. Safety factors for static loading conditions of 4.14 for shallow wedge and 3.54 for deeper wedge failure surfaces were calculated. These safety factors against slope failure are higher than the circular arc safety factor of 3.2 and confirm the inherent conservatism of the circular arc failure analysis.

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Liquefaction of the shoreline slopes was also investigated. The analyses (Section 2.5.4.8.3) show that the beach sands will not liquefy when subjected to the SSE.

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2.5.5.2.2 Containment Rock Cut

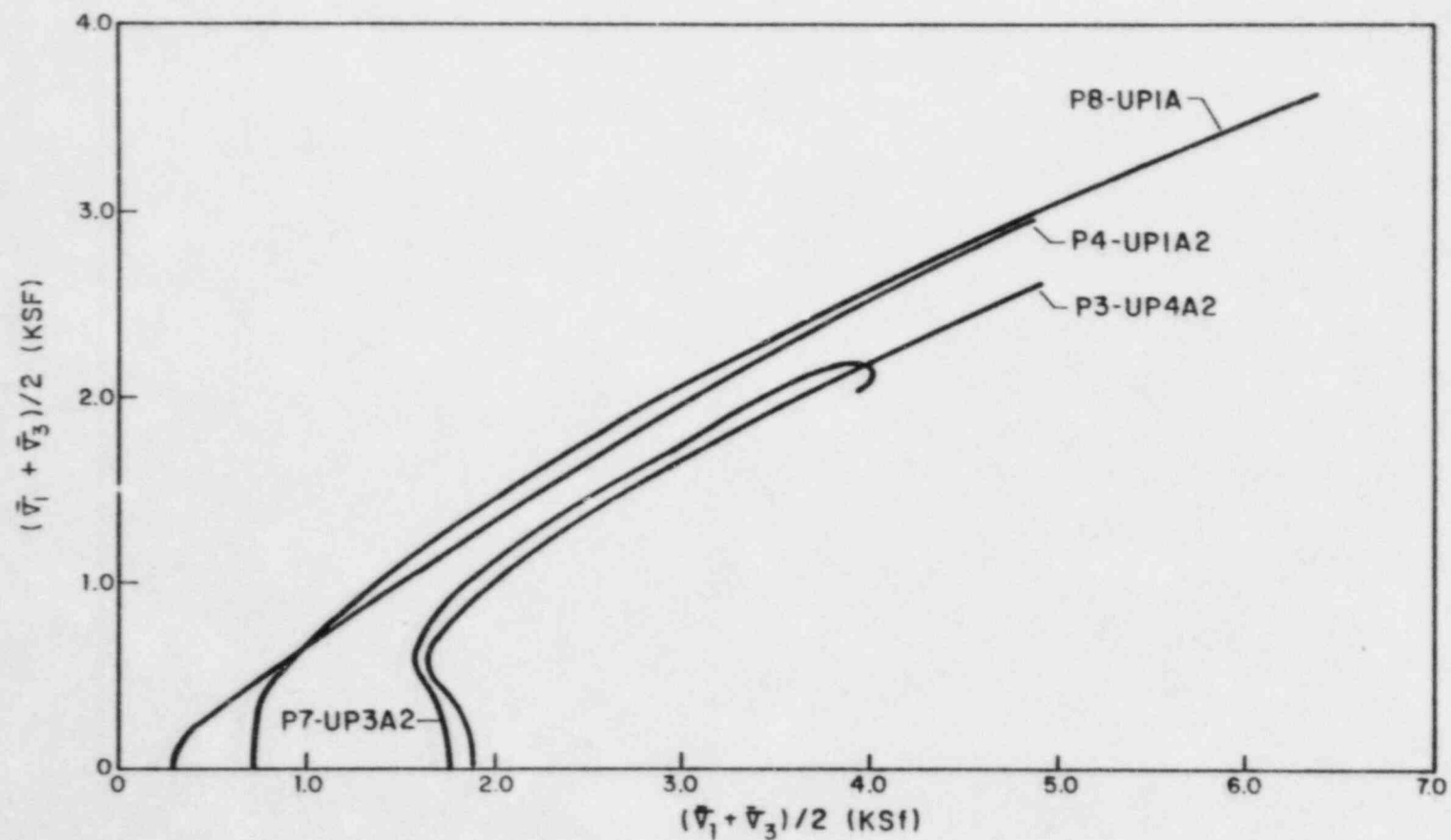
2.47

Two computer programs have been developed to evaluate field data and compute the stability of rock slopes. JTPLOT(ST-212) (SWEC 1974a) is used to reduce data from joint and foliation surveys and to prepare contoured stereographic plots, such as those on Figures 2.5.1-15 and 2.5.1-16. SWARS-2P (SWEC 1974b) is used to analyze the stability of tetrahedral rock wedges formed by the intersections of joint and foliations surfaces with the vertical excavation face. The data are input in geological notation and are converted internally to the format required for rock mechanics calculations. All possible combinations of joints are automatically considered. Effects of seismic loads, rock bolts, surcharges, point loads, and several types of piezometric loads are included in the analysis. In designing a restraining hoop or ring beam, the forces required to stabilize the sliding wedges are input into the program as hypothetical rock bolts, with the load distributed across the projected vertical area of the rock wedge. A minimum safety factor of 1.1 was considered acceptable for determining required stabilizing forces.

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In the analysis, the surcharge loading from adjacent structures was accelerated in the vertical direction, and soil surcharge was accelerated both vertically and horizontally. Water pressure was not applied to the rock wedge surfaces, on the assumption that the differential head acts directly on the containment wall. However, the buoyant weight of the rock was used to account for the presence of groundwater. This assumption is considered conservative because the buoyant weight effectively reduces the resistive forces. Wedges smaller than 100 cubic feet were disregarded, on the assumption that these wedges were formed by the intersection of two high angle joint

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SAMPLE	ϕ
P3-UP4A2	33.3
P4-UIA2	41.9
P7-UP3A2	35.2
P8-UIA	45.5

FIGURE 2.5.5-5
SUMMARY OF CIU TEST RESULTS-
BEACH AREA OUTWASH SANDS
MILLSTONE NUCLEAR POWER STATION
UNIT 3
FINAL SAFETY ANALYSIS REPORT

(FROM APPENDIX 2.5G)

NRC Letter: October 7, 1983 1.9

Question No. Q241.22 1.12

A seawall and a retaining wall adjacent to the Unit 3 pumphouse have 1.13
been identified by the applicant as safety class structures. 1.14
However, the analyses and designs of these two walls were not 1.15
presented in the FSAR. The design information is required for us to 1.16
complete our safety review.

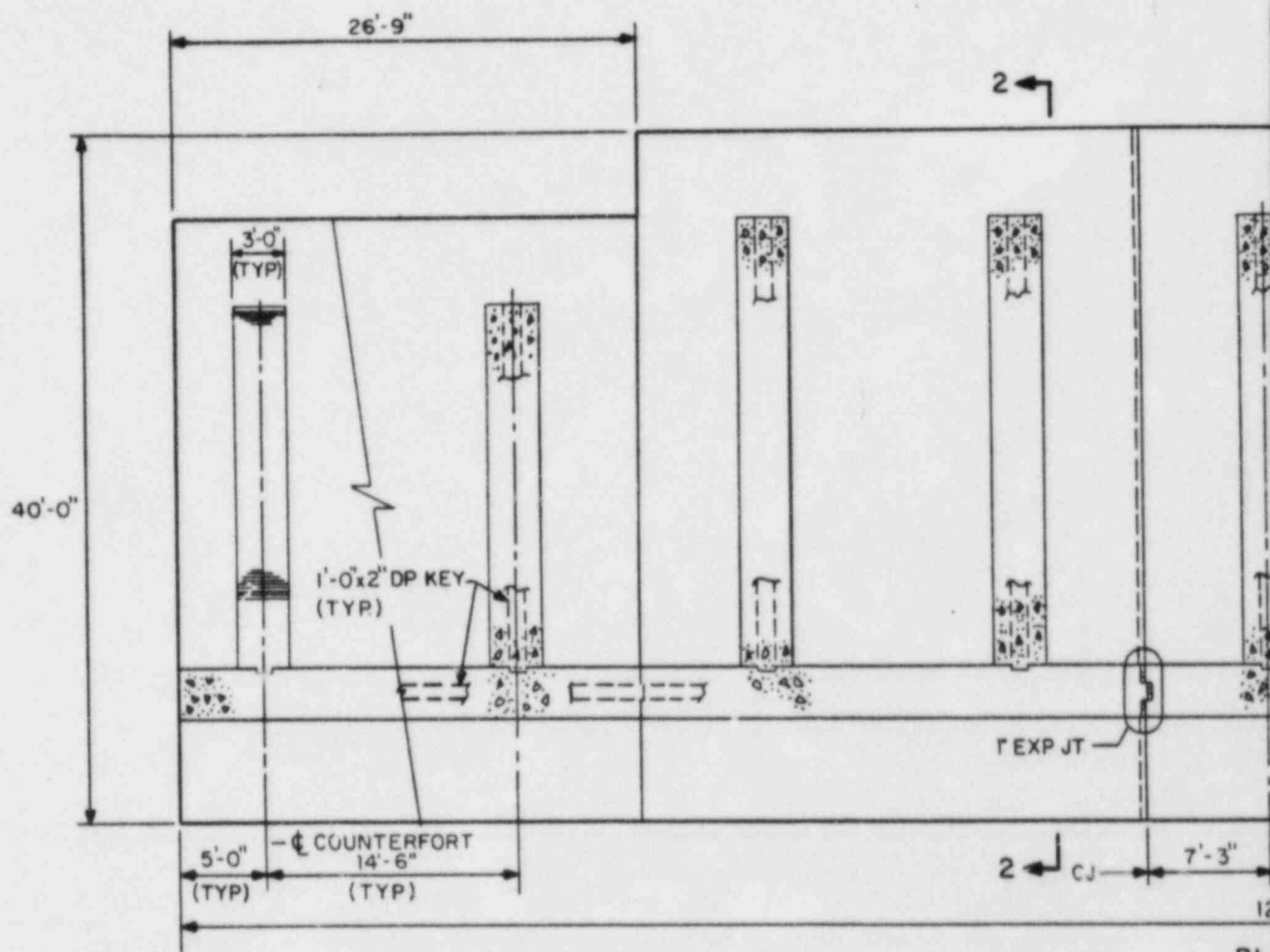
Response: 1.18

The west retaining wall is located on the west side of the 1.19
circulating and service water pumphouse as shown on revised FSAR 1.21
Figure 3.8-69 (sheet 1 of 4), and is designed as a safety-related
counterfort type reinforced concrete retaining wall. It is founded 1.24
on bedrock and is an extension of the west wall of the circulating
and service water pumphouse.

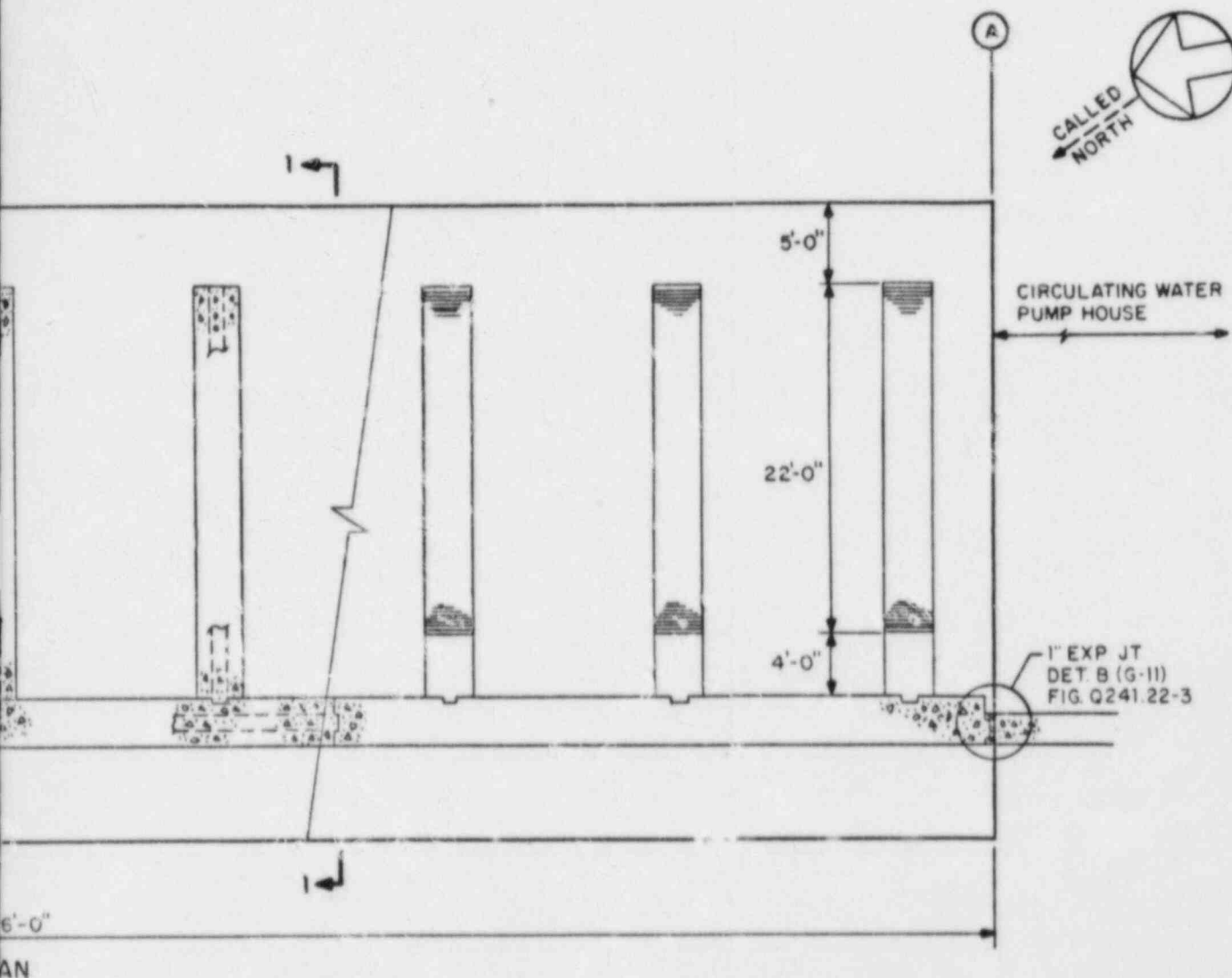
The west retaining wall was designed for static and dynamic lateral 1.25
loads in accordance with Figure 2.5.4-43 for SSE conditions. Soil 1.27
pressures were calculated based on the assumption that site grade on
the landward side of the wall is elevation +14 feet and on the 1.28
seaward side is elevation -6 feet. A groundwater level of +2 feet 1.29
and a seawater level of -6 feet were also used in the analysis. A 1.30
uniform surcharge load of 300 psf was applied as a driving force.
Dynamic forces due to the maximum height wave addressed in FSAR 1.31
Section 2.4 were not considered because the orientation of the wall 1.32
is perpendicular to the wave path. A longitudinal section of the 1.33
west retaining wall looking east is shown on Figure Q241.22-1.
Cross-sections and a detail of the expansion joint of the junction of 1.34
the wall and the pumphouse are shown on Figures Q241.22-2 and 1.35
Q241.22-3.

The function of the west retaining wall is to protect the safety- 1.36
related service water pipelines and electrical duct lines located 1.37
behind the wall from undermining due to wave action scouring. For a 1.39
discussion of the shoreline protection, refer to FSAR Section 2.5.5.
FSAR Section 3.8.4 has been revised to include this wall as a safety- 1.40
related portion of the pumphouse.

The seawall located to the east of the circulating and service water 1.41
pumphouse is not a safety-related structure and therefore is not 1.42
discussed in FSAR Section 3.8.4. The retaining wall is a counterfort 1.43
type reinforced concrete wall with post-tensioned rock anchors. The 1.44
structure is founded on concrete fill to bedrock and extends between
the pumphouse and the existing Unit 2 seawall. The function of the 1.46
seawall is to retain and protect the earth behind it from wave
action; however, were it to fail, there would be no adverse effect on 1.47
any safety-related structure or system.



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FIGURE Q241.22-1
WEST RETAINING WALL
MILLSTONE NUCLEAR POWER STATION
UNIT 3
FINAL SAFETY ANALYSIS REPORT

REVISION 1

MAY 1984

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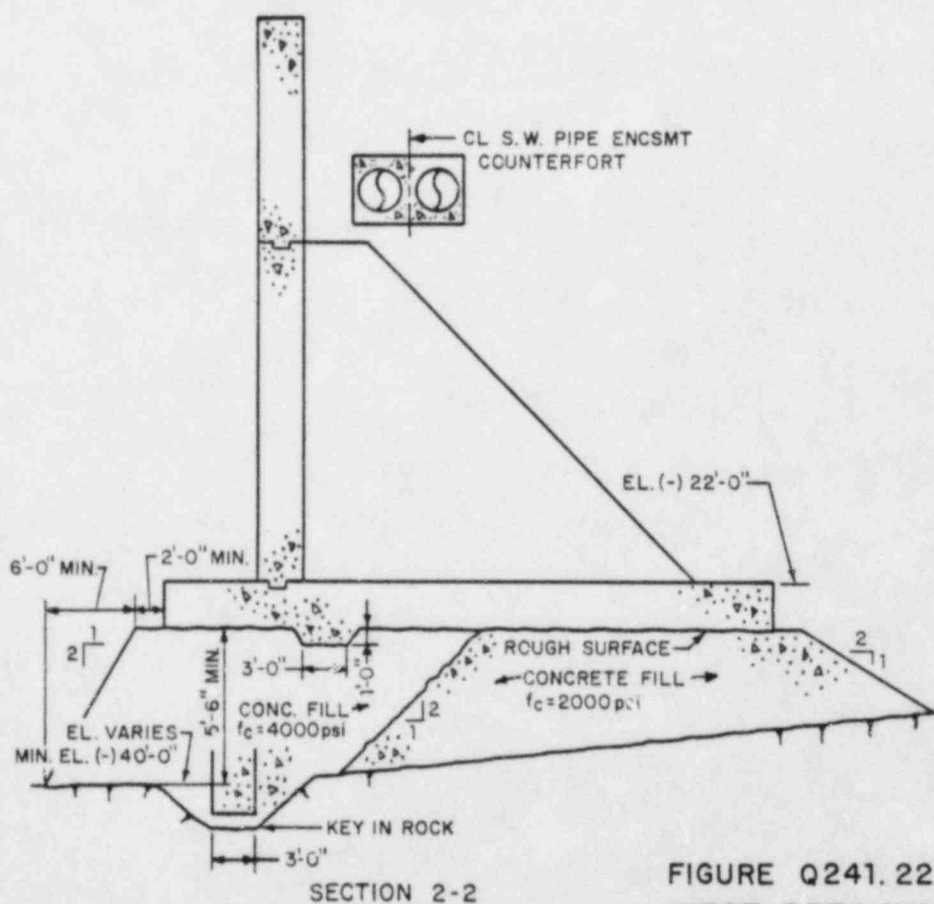
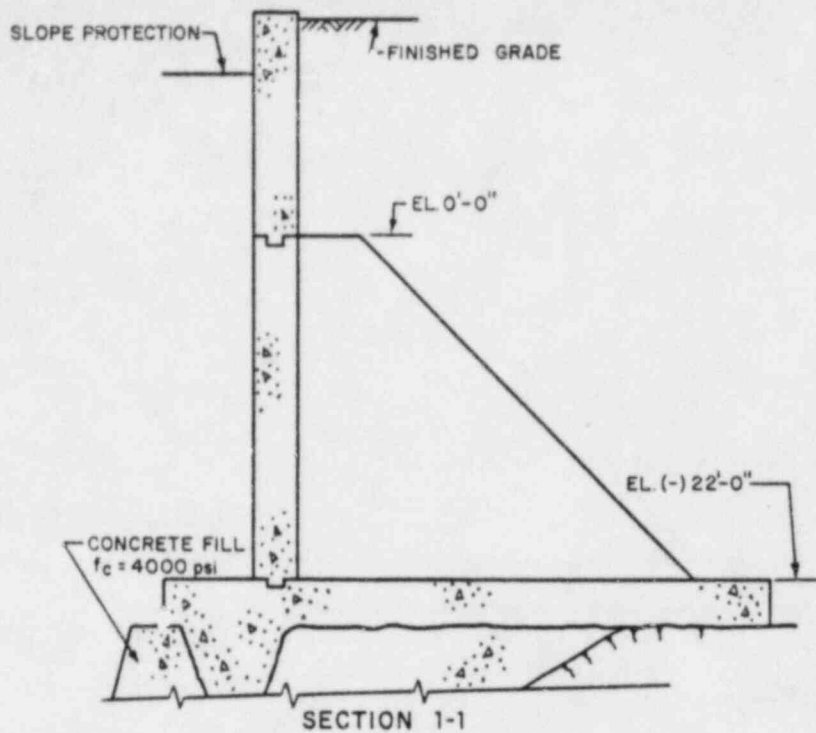
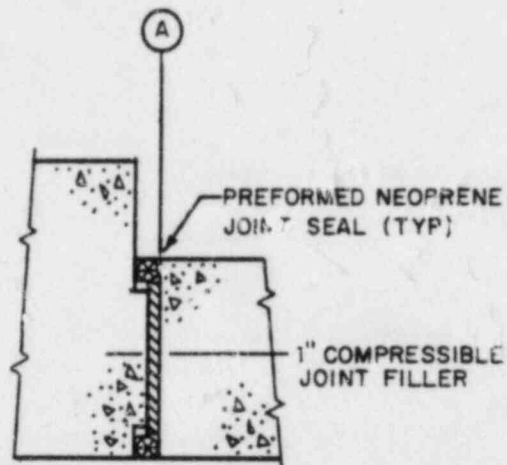


FIGURE Q241.22-2
WEST RETAINING WALL DETAILS
MILLSTONE NUCLEAR POWER STATION
UNIT 3
FINAL SAFETY ANALYSIS REPORT



DETAIL B
(J-4)

JOINT BETWEEN WEST RETAINING WALL
& PUMPHOUSE

FIGURE Q241.22-3
EXPANSION DETAIL
WEST RETAINING WALL
MILLSTONE NUCLEAR POWER STATION
UNIT 3
FINAL SAFETY ANALYSIS REPORT

ATTACHMENT 3

RESPONSE TO DRAFT SER OPEN ITEMS GEOTECHNICAL ISSUES

- SEB-10 Foundation Stability - Dynamic Loading
(Draft SER Section 2.5.4.3)
- SEB-11 Ring Beam Design
(Draft SER Section 2.5.5.2)

Millstone Nuclear Power Station, Unit No. 3

Open Items

Structural and Geotechnical Engineering Branch

SEB-10 Foundation Stability (Draft SER Section 2.5.4.3)

Dynamic Loading

The computer program PLAXLY-3 was used for structures founded on soils to determine the amplification effects of the soil-structure system. The computer program SHAKE was used to determine the strain compatible soil properties. However, the applicant has not provided sufficient information for the staff to confirm the acceptability of the analyses.

Sliding stability resulting from seismic loading was analyzed for the service water pipe encasement in the FSAR. However, the applicant has not provided adequate information for the staff to confirm the acceptability of the analysis.

Additional information is required from the applicant to confirm the acceptability of the dynamic analysis for the control and emergency diesel generator enclosure and to establish the seismic stability of the service water pipe encasement.

Response

This item was closed at the Structural Audit and is listed under Items 11 and 12 of the Structural Audit Meeting Summary.

An example for Item 11 was used comparing the shear moduli and damping from three strong motion records and comparing the results with the Millstone Artificial Earthquake. In general, soil properties obtained from the artificial earthquake analysis were in close agreement with values obtained from the real earthquake analyses.

Similar examples using the plaxly model and half-space model for the Control Building and Emergency Generator Enclosure Building were shown comparing Acceleration and Frequency values for the two buildings.

Additional information submitted to answer Questions 241.3 and 241.4, Figures 2.5.4-54, -55, -56 and -71 and revised FSAR Section 2.5.4.7.1 in Amendment 8 to the FSAR, provides further description of the soil structure interaction methodology.

Sliding stability is addressed in Revision 1 to Question 241.6 and Figure Q241.6-1 in Amendment 8 to the FSAR.

NNECO considers the above Draft SER Open Item (SEB-10) and Questions 241.3, 241.4, and 241.6 to be closed.

Millstone Nuclear Power Station, Unit No. 3

Open Items

Structural and Geotechnical Engineering Branch

SEB-11 Ring Beam Design (Draft SER Section 2.5.5.2)

The containment building is founded on bedrock at approximately el -39 ft. Top of rock varies from approximately el 0 ft to el 20 ft. The excavation walls are vertical, with a 9-in. bench at el -17 ft.

During construction, detailed geologic mapping has uncovered some additional preferred joint sets, shown on FSAR Figure 2.5.5-2. The failure in the bedrock along these joints and foliation surfaces could affect the lateral loading of the containment structure.

Two computer programs have been developed by the applicant to evaluate field data and compute the stability of rock slopes. A ring beam was designed and constructed to transfer the rock load around the excavation, maintaining the isolation of containment structure from these external loads.

The applicant has not provided adequate information for the staff to confirm the acceptability of the ring beam design.

Response

Revised FSAR page 2.5.5-8 and Figures 2.5.5-6 and 2.5.5-7 in Amendment 8 to the FSAR provide the additional information requested by the NRC staff in Question 241.17 and Draft SER Section 2.5.5.2.

NNECO considers this additional information to resolve Question 241.17 and the above Draft SER Open Item (SEB-11).