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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July 12, 1985

Docket No. 50-423
B11579

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

Reference: 1. B. J. Youngblood to J. F. Opeka, Request for Additional Information for Millstone Nuclear Power Station, Unit No. 3, dated May 20, 1985.

Gentlemen:

Millstone Nuclear Power Station, Unit No. 3
Response to Requests for Additional Information
Structural and Geotechnical Engineering Branch (SGEB)

Enclosure 1 to Reference 1 transmitted requests for additional information resulting from the SGEB review of the Millstone Nuclear Power Station, Unit No. 3 FSAR. Attached are Northeast Nuclear Energy Company's (NNECO) responses to Questions 241.23 through 241.26. NNECO met with the NRC Staff on May 14, 1985 to discuss these questions and it was agreed that the information provided herein would resolve their concerns regarding these items.

If you have any questions regarding this submittal, please contact our licensing representative directly.

Very truly yours,

NORTHEAST NUCLEAR ENERGY COMPANY
et. al.

BY NORTHEAST NUCLEAR ENERGY COMPANY
Their Agent

J. F. Opeka

J. F. Opeka
Senior Vice President

3001
1/40

cc: Mr. John Chen
SGEB

Mr. Nilesh Chokshi
SGEB

Mr. Lyman Heller, Section Leader
SGEB

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

Then personally appeared before me J. F. Opeka 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.

Jennifer J. Powers
Notary Public
My Commission Expires March 31, 1989

Millstone Nuclear Power Station, Unit No. 3

Response to Request for Additional Information
Structural and Geotechnical Engineering Branch
Questions 241.23 through 241.26

Question 241.23 The staff must review the calculations associated with the fragility analysis of the retaining wall for better understanding. Therefore, provide these calculations with adequate explanations of assumptions, calculational approach, and the results.

Response: Enclosed are the calculations for the retaining wall. The assumptions and calculational approach are documented in the calculations and are summarized together with the results in the response to the Staff's Question 720.81 (Reference 1).

In addition, as a result of discussions with the staff, calculations to assess the effects of unsymmetrical sliding resistances have been performed for the retaining wall.

Newmark (Reference 2) presents equations to predict the sliding displacement of rigid bodies having both symmetrical and unsymmetrical resistances. Various predictive equations for unsymmetrical resistance are compared to data from time-history analyses using natural earthquake records in Figure 22 of Reference 2. This figure indicates that in the range of interest ($N/A > 0.2$), the most accurate equation is:

$$\Delta = \frac{V^2}{2gN} \left(1 - \frac{N}{A}\right) \frac{A}{N}$$

where N is the total resistance to sliding, and A is the horizontal acceleration capacity.

Inspection of Figure 22 of Reference 2 indicates that this equation is still somewhat conservative for use as a median-centered formulation for the prediction of sliding displacements.

The sliding capacity of the retaining wall was reevaluated using the equation presented above. The following fragility values were calculated:

$\breve{A} = 0.9g$

$\beta_R = 0.17$

$\beta_U = 0.45$

This median capacity is reduced from the value of 1.2g previously estimated. However, sensitivity studies performed for the plant damage state fragilities and occurrence frequencies using the 0.9g capacity together with a modified sliding capacity for the EGE building using a v/a ratio of 36 in/sec/g (Question 241.25c) indicate only minor changes occur. The results of these sensitivity studies are shown in the attached appendix.

References

1. Letter from W. G. Counsil to B. J. Youngblood, "Millstone Nuclear Power Station, Unit 3, Probabilistic Safety Study", dated July 31, 1984.

2. Newmark, N. M., "The Effects of Earthquakes on Dams and Embankments", Geotechnique, Vol. 15, No. 2, 1965.

MILLSTONE UNIT NO. 3

SEISMIC FRAGILITY EVALUATION

WEST RETAINING WALL

ORIGINAL RESPONSE TO QUESTION 720.81

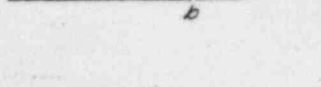
TITLE Millstone 3 | PAGE RW-1 OF | Job No. 20601
BY DSH DATE 7/19/84 | COMMENTS Retaining Wall
CHKD. BY DW DATE 7/25/84

STRUCTURAL MECHANICS ASSOCIATES A Calif. Corp.

The west retaining wall on Dr. EC-14V will be evaluated in response to NRC Question 720.81. Failure of this wall would imply failure of the service water lines buried in the soil behind the wall. The portion of wall having the top of footing at EL (-) 22'-0 obviously governs.

Static Load

If the wall fails due to soil loading, it will deform outward and develop active earth pressure. The soil failure wedge will develop from the heel of the footing as shown in Fig. 28.1 of Ref. A.



Active earth pressure is applied by the failure wedge on plane ab with no shearing stress. Backfill behind the walls consisted of well-graded sands and gravels compacted to 90% of

Ref. A - Terzaghi, K. and R.B. Peck, Soil Mechanics in Engineering Practice, 2nd ed., John Wiley & Sons, 1967.

maximum density (FSAR Sect. 2.5.4.10.3). Angle of internal friction values of about 34° for -#4 fraction are noted in FSAR Sect. 2.5.4.5.2, but discussion indicates that testing on the -#4 fraction results in shear strength more conservative than the whole soil sample. The following values are estimated:

$\phi = 40°$

$\gamma_s = 140$ pcf

$K_A = \tan^2(45 - \frac{\phi}{2})$

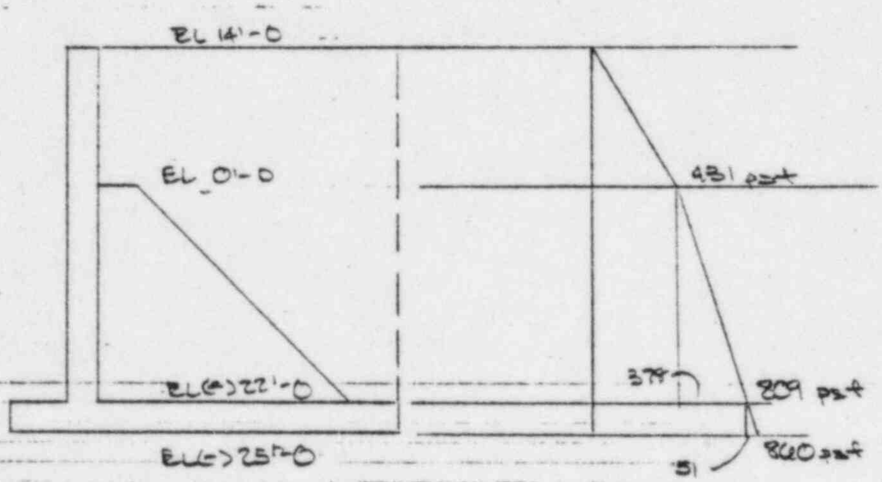
$= \tan^2(45 - \frac{40}{2})$

$= 0.22$

$\gamma_s - \gamma_w = 140 - 62.4$

$= 78$ pcf

Hydrostatic pressure acts on both wall faces below EL 0'-0. Grade is at EL 14'-0.



$p(\text{EL } 0'\text{-}0) = 0.22(140)(14)$

$= 431 \text{ psf}$

$p(\text{EL } (-)22'\text{-}0) = 431 + 0.22(78)(22)$

$= 431 + 378$

$= 809 \text{ psf}$

$p(\text{EL } (-)25'\text{-}0) = 809 + 0.22(78)(3)$

$= 809 - 51$

$= 860 \text{ psf}$

Dynamic Loads

The M-O approach described in Ref. B will be used. Discussion in Ref. B indicates that use of the M-O approach with the buoyant soil weight and use of dynamic water pressures determined by Westergaard theory is adequate. The resultant of the dynamic earth pressure will be assumed to act at a height 0.6H above the base of the wall. Vertical acceleration will be neglected for simplicity.

Dynamic Active Earth Pressure

$$K_{AE} = \frac{\cos^2(\phi - \theta - \beta)}{\cos\theta \cos^2\beta \cos(\delta + \beta - \theta)\left[1 + \sqrt{\dfrac{\sin(\phi + \delta)\sin(\phi - \theta - i)}{\cos(\delta + \beta + \theta)\cos(i - \beta)}}\right]^2}$$

$$\Delta K_{AE} = K_{AE} - K_A$$

Ref. B - Seed, H. B. and R.V. Whitman, "Design of Earth Retaining Structures For Dynamic Loads", 1970 ASCE Specialty Conference on Lateral Stresses in the Ground and Design of Earth Retaining Structures,

TITLE Millstone 3 | PAGE RW-4 OF | Job No. 20001.0

BY ZSU | DATE 7/20/86 | COMMENTS Retaining Walls

CHKD. BY DW | DATE 7/25/86

STRUCTURAL MECHANICS ASSOCIATES
A Calif. Corp.

$\theta = \tan^{-1} k_h$

$\phi = 40°$

$\delta = 0$

$i = 0$

$\beta = 0$

k_h	K_{AE}	ΔK_{AE}
0	0.217	0
0.05	0.242	0.02
0.10	0.268	0.05
0.15	0.297	0.08
0.20	0.328	0.11
0.25	0.363	0.15
0.30	0.401	0.18
0.35	0.442	0.22
0.40	0.488	0.27
0.45	0.539	0.32
0.50	0.596	0.38
0.55	0.661	0.44
0.60	0.735	0.52
0.65	0.823	0.61
0.70	0.929	0.71
0.75	1.065	0.85
0.80	1.204	1.05

1.0

1.15

1.31

TITLE Millstone 3 BY PSH DATE 7/20/84 CHKD. BY DLJ DATE 7/25/84

STRUCTURAL MECHANICS ASSOCIATES A Calif. Corp.

COMMENTS Retaining Walls

In the acceleration range of interest, 0.6g to 0.8g, the approximation $\Delta K_{AE} = 1.0\,k_h$ is adequate

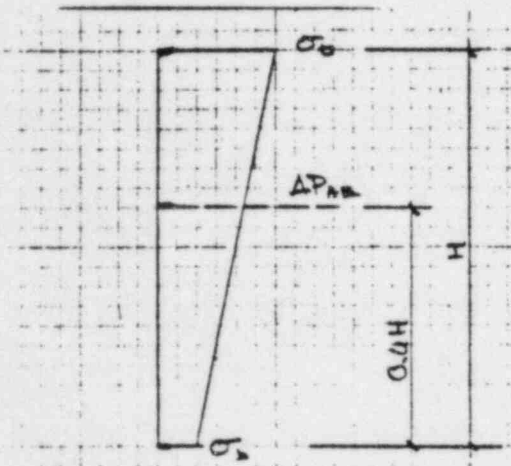
(Note - This is somewhat conservative for sliding which initiates at about 0.4g)

$$\Delta K_{AE} = 1.0\,k_h$$

$$\Delta P_{AE} = \tfrac{1}{2}\gamma H^2 \Delta K_{AE}$$

$$= \left[\tfrac{1}{2}(140)(39)^2 - \tfrac{1}{2}(62.4)(25)^2\right](1.0\,k_h)$$

$$= 87{,}000\,k_h$$



$$\tfrac{1}{2}(\sigma_t + \sigma_b)H = \Delta P_{AE}$$

$$\sigma_t + \sigma_b = \frac{2\Delta P_{AE}}{H}$$

$$\sigma_t = \frac{2\Delta P_{AE}}{H} - \sigma_b$$

$$\sigma_b H\left(\tfrac{H}{2}\right) + \tfrac{1}{2}(\sigma_t - \sigma_b)H\left(\tfrac{2}{3}H\right) = \Delta P_{AE}(0.6H)$$

$$0.5\sigma_b + \tfrac{1}{3}(\sigma_t - \sigma_b) = \frac{0.6\Delta P_{AE}}{H}$$

$$0.5\sigma_b + \tfrac{1}{3}\left(\frac{2\Delta P_{AE}}{H} - 2\sigma_b\right) = \frac{0.6\Delta P_{AE}}{H}$$

$$\tfrac{1}{6}\sigma_b = \frac{\Delta P_{AE}}{15H}$$

$$\sigma_b = \frac{6\Delta P_{AE}}{15H}$$

$$\sigma_t = \frac{8\Delta P_{AE}}{5H}$$

$H = 14 + 25$

$= 39$ ft

$\sigma_b = \frac{6}{15} \frac{87{,}000\ k_h}{39}$

$= 892\ k_h$ psf

$\sigma_t = \frac{8}{5} \frac{87{,}000\ k_h}{39}$

$= 3569\ k_h$ psf

Dynamic Water Pressure

From Ref. B, the dynamic pressure on the seaward side of the wall is equal to the Westergaard pressure (acting towards the sea) and the pressure on the landward side is equal to 70% of the Westergaard pressure. The total dynamic water pressure is therefore 1.7 times the Westergaard pressure.

$p_y = 1.7 \left[\frac{7}{8} k_h \gamma_w (hy)^{1/2} \right]$ $\quad H = 25$ ft

$= 1.7 \left(\frac{7}{8}\right) k_h (62.4) \sqrt{25y}$ $\quad y$ = Depth below water level

$= 464\ k_h \sqrt{y}$

P = total force

$= 1.7 \left(\frac{7}{12}\right) h^2 \gamma_w k_h$

$= 1.7 \left(\frac{7}{12}\right) (25)^2 (62.4)\ k_h$

$= 38{,}680\ k_h$

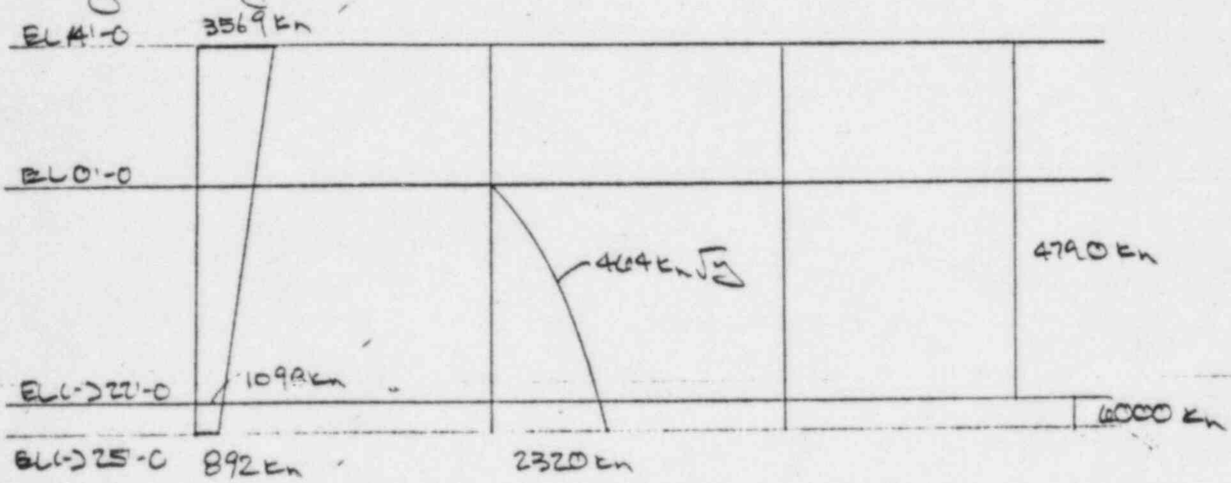
Inertial Load From Soil Above Footing and Structure.

As shown on p. RW-1, soil failure wedge for which the dynamic active earth pressure has been calculated is behind plane ab. There is additional inertial load associated with the mass of the wall and the soil whose weight bears on the footing.

$$p\,(\text{above EL}(-)22'\text{-}0) = [150(3) + 140(31)]\ k_h$$
$$= 4790\ k_h$$

$$p\,(\text{below EL}(-)22'\text{-}0) = 150(40)\ k_h$$
$$\approx 6000\ k_h$$

Summary of Dynamic Load



Moment Capacity of Counterfort

Evaluate as a T-beam using ACI effective flange width

Effective flange width

$\frac{L}{4}$ span = 0.25 (22)

= 5.5' ← Governs

$2 \times (8t_f) + t_w = 2(8)(3) + 3$

= 51'

℄ to ℄ spacing = 14.5'

$b_f = 5.5$ ft

$t_f = 3$ ft

stem reinf. - #9 @ 12" EF vert

$A_{s1} \approx 2(21-3)(1.0)$

$= 36 \text{ in}^2$

$d_1 = 3 + \frac{21-3}{2}$

= 12'

Concentrated reinf.

Have 18 - #11

$\theta = \tan^{-1} \frac{17}{22}$

= 37.7° from vert

$A_{s2} = 18(1.56) \cos 37.7$

$= 22.2 \text{ in}^2$

$d_2 \approx 28$ ft

Material props. same as for pumphouse

$f'_c = 5900$ psi

$f_y = 54$ ksi Grade 40

= 68 ksi Grade 60 - Used for footings & dowels, p. MS-25

$$a = \frac{(36 + 22.2)(54)}{0.85(5.8)(66)}$$

$= 9.66"$ $< 36'$ ok

$= 0.81$ ft

$$M_u = 36(54)\left(12 - \frac{0.81}{2}\right) + 22.2(54)\left(28 - \frac{0.81}{2}\right)$$

$= 55,600$ k-ft

Static Active Earth Pressure

$$M = \left[\tfrac{1}{2}(0.431)(14)\left(\tfrac{14}{3} + 22\right) + 0.431(22)\left(\tfrac{22}{2}\right) + \tfrac{1}{2}(0.378)(22)\left(\tfrac{22}{3}\right)\right](14.5)$$

$= 3120$ k-ft

Dynamic Active Earth Pressure

$$M = \left[0.891(36)\left(\tfrac{36}{2}\right) + \tfrac{1}{2}(3.569 - 0.891)(36)\left(\tfrac{2}{3}\right)(36)\right](14.5)$$ k

$= 25,150$ kn

Dynamic Water Pressure

$$M = \int p(22 - y)\, b\, dy$$

$$= \int_0^{22} 0.464\ \text{kn}\sqrt{y}\,(22 - y)(14.5)\,dy$$

$$= 6.728\ \text{k} \int_0^{22} (22\sqrt{y} - y^{3/2})\,dy$$

$$= 6.728\ \text{kn}\left[\frac{22}{1.5}y^{3/2} - \frac{2}{5}y^{5/2}\right]_0^{22}$$

$= 4070$ kn

Additional Inertial Load

$M = 4.790\,k_h\,(36)\left(\frac{36}{2}\right)(4.5)$

$= 45{,}010\,k_h$

Total Moment

$M = 3120 + (25{,}150 + 4070 + 45{,}010)\,k_h$

$= 3120 + 74{,}230\,k_h$

$3120 + 74{,}230\,k_h = 55{,}600$

$k_h = \dfrac{55{,}600 - 3120}{74{,}230}$

$= 0.71$

$\tilde{A} = 0.71g$

<u>Shear Capacity of Counterfort</u>

$t = 36"$

$L = 29$ ft

reinf. – #9 @ 12" EFEW

$d \approx \dfrac{36(12) + 22.2(28)}{36 + 22.2}$ p. RW-8

$= 18.1$ ft $> 0.6 L_w$

~~Estimate $h_w = \frac{1}{2}$(total wall ht)~~

$= 18$ ft

$\frac{h_{we}}{l_w} = \frac{18}{29}$

$= 0.621$

$v_c = 10\sqrt{5900} - 3.4\sqrt{5900}\,(0.621)$

$= 610$ psi ✓

$\rho = \frac{2(1.0)}{12(36)}$

$= 0.00463$

$v_s = 0.00463(54,000)$

$= 250$ psi ✓

$v_u = 610 + 250$

$= 860$ psi ✓

$V_u = 0.860(36)(18.1)(12)$

$= 6730^{k}$ ✓

Static Active Earth Pressure

$V = \left[\frac{1}{2}(0.431)(14) + \frac{1}{2}(0.431 + 0.809)(22)\right](14.5)$

$= 242^{k}$ ✓

Dynamic Active Earth Pressure

$V = \left[\frac{1}{2}(3.569 + 0.892)(36)\right](14.5)\,k_h$

$= 1164\,k_h$ ✓

Dynamic Water Pressure

$V = \int_0^{22} p\,b\,dy$

$= \int_0^{22} 0.464\,k_h\sqrt{y}\,(14.5)\,dy$

$= \frac{0.464(14.5)}{1.5}(22)^{1.5}\,k_h$

$= 463\,k_h$ ✓

TITLE Millstone 3 | BY ____ DATE 7/20/84 | CHKD. BY ____ DATE 7/25/84

STRUCTURAL MECHANICS ASSOCIATES A Calif. Corp.

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COMMENTS Retaining Wall

Additional Inertial Load

$V = 4.790\, k_n (36)(14.5)$

$= 2500\, k_n$

Total Shear

$V = 242 + (1164 + 463 + 2500)\, k_n$

$= 242 + 4127\, k_n$

$242 + 4127\, k_n = 6730$

$k_n = \frac{6730 - 242}{4127}$

$= 1.6$

$\check{A} = 1.6g$

Load Transfer Into Counterfort

Check tensile capacity at vertical construction joint with wall.

reinf. - 4-#11, 9-#9 @ 9" EF, #9 @ 12" EF (approx. 28 ton)

$A_s = 4(1.56) + (18 + 28)(1.0)$

$= 52.2\, in^2$

$T_y = 52.2(54)$

$= 2819\, k$

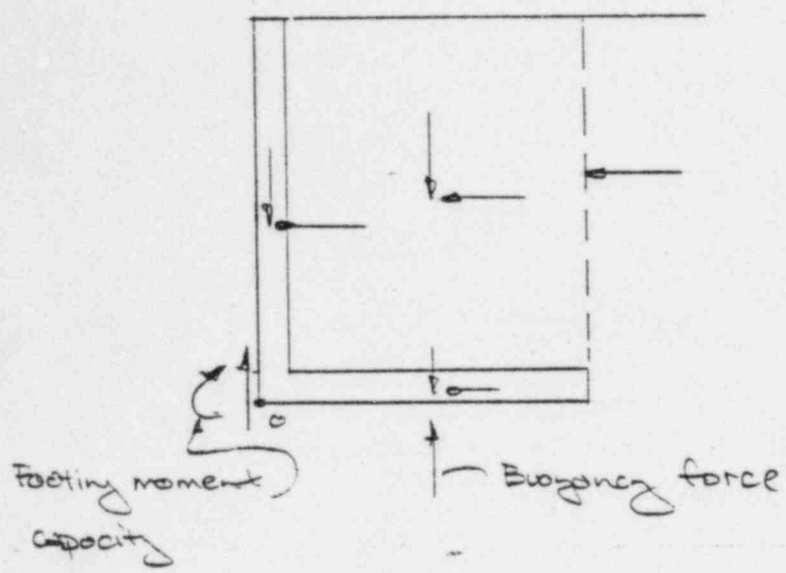
$T = 242 + 4127\, k_n$ same as wall shear

$242 + 4127\, k_n = 2819$

$k_n = 0.62$

Overturning

Rather than pivot about the end of the footing due to overturning, the wall will pivot about the intersection of the vertical plane through the outside wall face and the horizontal plane through the bottom of the footing. This occurs since the toe of the footing has insufficient moment capacity to transmit a highly concentrated load to the end of the footing. The moment capacity of the footing will be developed. The acceleration to initiate overturning will be based on a comparison of applied and resisting moments about Point O



Resistances

Soil Weight

$W = 0.14(36)(31)$

$= 156.2\ ^{k/ft}$

$M_U = 156.2\,(3 + \frac{31}{2})$

$= 2890\ ^{k\text{-}ft/ft}$

Wall Weight

$W = 0.15(36.5)(3)$

$= 16.4\ ^{k/ft}$

$M_U = 16.4\,(\frac{3}{2})$

$= 25\ ^{k\text{-}ft/ft}$

Footing Weight

$W = 0.15(3)(34)$

$= 15.3\ ^{k/ft}$

$M_U = 15.3\,(\frac{34}{2})$

$= 260\ ^{k\text{-}ft/ft}$

Buoyancy Force

$W = -0.0624(34)(22)$

$= -46.7\ ^{k/ft}$

$M_U = -46.7\,(\frac{34}{2})$

$= -794\ ^{k\text{-}ft/ft}$

Footing Moment Capacity

reinf. — #9 @ 4½" bottom

$A_s = 1.0\,\frac{12}{4.5}$

$= 2.67\ in^2/ft$

$$a = \frac{2.67(68)}{0.85(5.9)(12)}$$

Grade 60 rebar for footing

$$= 3.02"$$

$$d \approx 32"$$

$$M_u = 2.67(68)(32 - \frac{3.02}{2})$$

$$= 5540 \text{ "k/ft}$$

$$= 461 \text{ k-ft/ft}$$

Total Resistance

Include reduction in resistance due to vertical seismic

$$M_u = (2890 + 25 + 260)\left[1 - 0.4(\tfrac{2}{3})k_n\right] - 794 + 461$$

$$= 3175(1 - 0.267k_n) - 794 + 461$$

$$= 2842 - 848k_n$$

Applied Load

Static Active Earth Pressure

$$M_o = \tfrac{1}{2}(0.431)(14)(25 + \tfrac{14}{3}) + 0.431(25)(\tfrac{25}{2})$$

$$+ \tfrac{1}{2}(0.378 + 0.051)(25)(\tfrac{25}{3})$$

$$= 269 \text{ k-ft/ft}$$

Dynamic Active Earth Pressure

$$M_o = 0.892k_n(39)(\tfrac{39}{2}) + \tfrac{1}{2}(3.569 - 0.892)(39)(\tfrac{2}{3})(39)k_n$$

$$= 2036k_n$$

Dynamic Water Pressure

$$M_o = \int_0^{25}(0.464k_n\sqrt{y})(25 - y)\,dy$$

$$= 0.464k_n\int_0^{25}(25\sqrt{y} - y^{3/2})\,dy$$

$$= 0.464k_n\left[\frac{25}{1.5}y^{3/2} - \frac{2}{5}y^{5/2}\right]_0^{25}$$

$$= 387k_n$$

Additional Inertial Load

$$M_o = 4.790 k_n (36)(3 + \frac{36}{2}) + 6.0 k_n (3)(\frac{3}{2})$$

$$= 3648 k_n$$

Total Moment

$$M_o = 269 + (2036 + 387 + 3648) k_n$$

$$= 269 + \cancel{[illegible]}\ 6071 k_n$$

$$M_u = M_o$$

$$2842 - 848 k_n = 269 + 6071 k_n$$

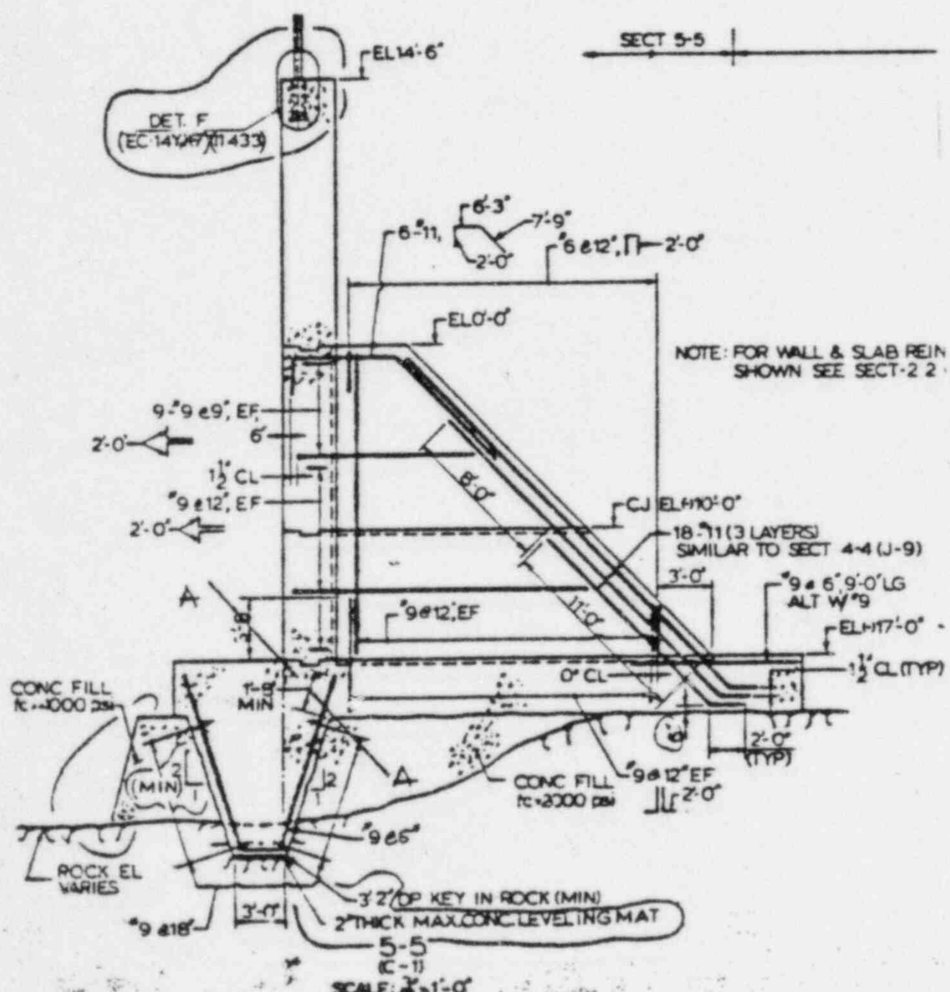
$$k_n = \frac{2842 - 269}{6071 + 848}$$

$$= 0.37$$

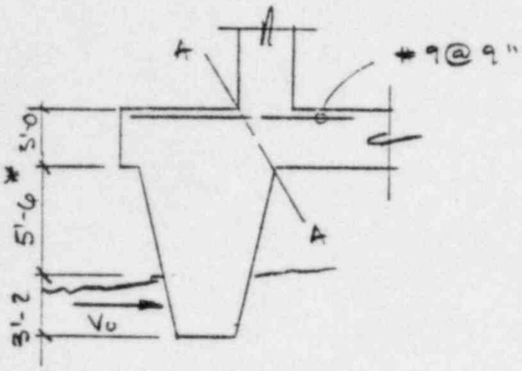
Overturning of the retaining wall will initiate at a peak ground acceleration of 0.37g. The acceleration to cause sufficient displacement to fail the service water lines is greater than this value. Also, this capacity assumes that sliding does not occur. Overturning cannot occur if the wall slides.

Sliding

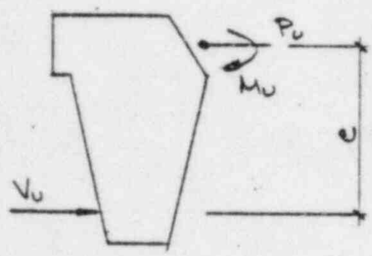
Sliding will occur along the footing - fill concrete interface since the fill concrete - rock interface has a higher coefficient of friction. Sliding capacity is expected to be governed by the portion of wall constructed to Sect. 3-3 on Dr. EC-14V. The same key detail as shown in Sect. 5-5 is used. If the key proportions are approximately as shown in this detail, rather than develop the key capacity, the load transmitted by the key will be limited by the moment capacity of the footing at Sect. A-A below.



Key Capacity



* - 5'-6 max from ~~bottom of footing~~ to top of rock
for ~~Sect~~. 3-3
(Sect. 1-1, Dr. EC-14V)



At Section A-A

$P_u = V_u$

$M_u = V_u e$

$e = \frac{3}{2} + 5.5 + \frac{3.17}{2}$

$= 8.59'$

$A_s = 1.0 \frac{12}{9}$

$= 1.33 \text{ in}^2/\text{ft}$

Try $V_u = 32.5$ k/ft

$P_u = 32.5$ k/ft

$a = \frac{1.33(60) + 32.5}{0.85(5.9)(12)}$ Grade 60 rebar for footing

$= 2.04"$

$d \approx 32"$

$\frac{t}{2} = \frac{36}{2}$

$= 18"$

TITLE Millstone 3
BY PSH DATE 7/25/84
CHKD. BY [illegible] DATE 7/29/84

STRUCTURAL MECHANICS ASSOCIATES A Calif. Corp.

PAGE RW-18A OF ____ Job No. 20601.0
COMMENTS Retaining Wall

$$M_U = 1.33(48)(32 - 18) + 0.85(5.9)(12)(2.04)\left(18 - \frac{2.04}{2}\right)$$

$$= 3351 \text{ ''k}$$

$$= 279.2 \text{ k-ft}$$

$$V_U = \frac{M_U}{e}$$

$$= \frac{279.2}{8.59}$$

$$= 32.5 \text{ k/ft} \quad \text{OK}$$

Friction

For rough concrete interfaces, $\mu = 1.0$ (see DUST calcs)

Soil

$W = 0.14(36)(31)$

$= 156.2^{k}/ft$

Weight of Water on Footing Toe

$W = 0.0624(6)(22)$

$= 8.2^{k}/ft$

Wall

$W = 0.15(36.5)(3)$

$= 16.4^{k}/ft$

Footing

$W = 0.15(3)(40)$

$= 18.0^{k}/ft$

Buoyancy Force

$W = -0.0624(25)(40)$

$= -62.4^{k}/ft$

Total Friction

Include reduction in resistance due to vertical seismic

$V_u = 1.0\{(156.2 + 16.4 + 18.0)[1 - 0.4(\tfrac{2}{3})k_n] - 62.4 + 8.2$

$= 136.4 - 50.8\,k_n$

Total Resistance

$V_u = (136.4 - 50.8\,k_n) + 32.5$

$= 168.9 - 50.8\,k_n$

Static Active Earth Pressure

$V = \frac{1}{2}(0.431)(14) + \frac{1}{2}(0.860 + 0.431)(25)$

$= 19.2\ ^{k}/ft$

Dynamic Active Earth Pressure

$V = \frac{1}{2}(3.569 + 0.892)\,k_n\,(39)$

$= 87.\,k_n$

Dynamic Water Pressure

$V = \int_0^{25} 0.464\,k_n \sqrt{y}\,dy$

$= \frac{0.464\,k_n}{1.5}(25)^{3/2}$

$= 38.7\,k_n$

Additional Inertial Load

$V = 4.790\,k_n\,(36) + 6.0\,k_n\,(3)$

$= 190.4\,k_n$

Total Base Shear

$V = 19.2 + (87 + 38.7 + 190.4)\,k_n$

$= 19.2 + 316.1\,k_n$

$19.2 + 316.1\,k_n = 168.9 - 50.8\,k_n$

$k_n = \frac{168.9 - 19.2}{316.1 + 50.8}$

$= 0.41$

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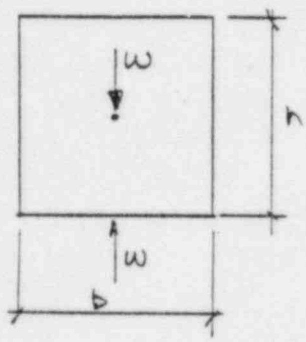
Sliding will initiate at a peak ground acceleration of about 0.41 g. Overturning was previously shown to initiate at about 0.37g. However, a PGA in excess of 0.37g is necessary to cause significant uplift. Sliding and overturning therefore occur at about the same acceleration level. However, the wall cannot overturn if sliding occurs. If the heel of the footing is uplifted when sliding initiates, gravity will cause the heel to drop back in contact with the ground. This is illustrated in the simple example below.

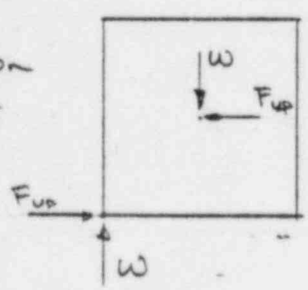
Initial state

$A = 0$



At initiation of uplift

$A = A_{up}$



$F_{up} = W A_{up}$

$F_{up}\left(\frac{h}{2}\right) = W\left(\frac{b}{2}\right)$

$F_{up} = W \frac{b}{h} = W A_{up}$

$A_{up} = \frac{b}{h}$

For retaining wall, $A_{up} \approx A_{slide}$

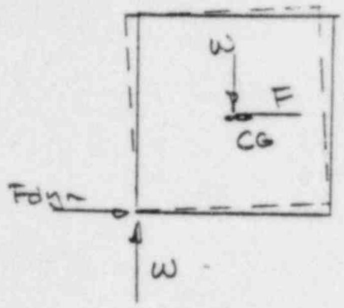
$F_{up} \approx F_{slide}$

$W A_{up} \approx W \mu_{static}$

$\mu_{static} \approx A_{up}$

~~A~~

$A = A_{up} + \Delta A$



$= W(A_{up} + \Delta A)$

$F_{dyn} = W\mu_{dynamic}$

$\mu_{dynamic} = \mu_{static} - \Delta\mu$

$F_{dyn} = W(\mu_{static} - \Delta\mu)$

$$\Sigma M_{CG} = W\left(\frac{b}{2}\right) - F_{dyn}\left(\frac{h}{2}\right)$$

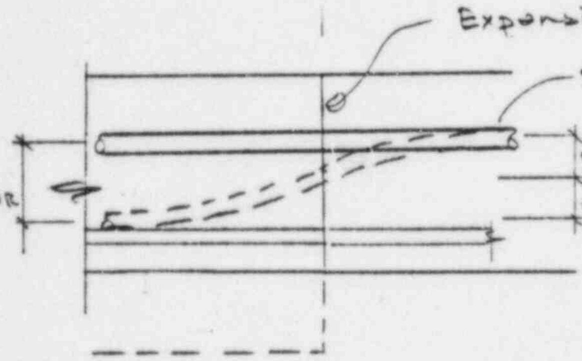
$$= \frac{h}{2}\left[W\frac{b}{h} - F_{dyn}\right]$$

$$= \frac{h}{2}\left[W A_{up} - W\mu_{static} + W\Delta\mu\right]$$

$$= \frac{h}{2}\left[W A_{up} - W A_{p} + W\Delta\mu\right]$$

$$= \frac{h}{2}(W\Delta\mu) \neq 0$$ — Will drop back down.

Even if the retaining wall begins to slide, failure will not occur until sufficient lateral displacement occurs to cause failure of the service water lines. This failure is expected to occur when the wall north of the expansion joint slides relative to the wall south of the expansion joint. The line is still embedded in soil on both sides of the expansion joint. If the wall slides relative to the adjacent wall, the deformed shape of the S.W. lines will be as shown below.



Because of symmetry, no moment acts on the S.W. lines at the joint. The S.W. line can be represented as a semi-infinite buried pipe with a hinged support at the joint and subjected to a relative end displacement of $\delta_R/2$.

This type of buried pipe condition was considered in the evaluation of piping failure due to structure sliding. As noted in the response to NRC question 720.80, a typical relative end displacement of 2" was found to be necessary to cause pipe buckling. The sliding displacement of the retaining

Wall necessary to cause pipe buckling is therefore:

$$\frac{\delta_R}{2} = 2''$$

$$\delta_R = 4'' \quad \text{total}$$

Newmark's method will be used to estimate the peak ground acceleration causing this displacement.

Note – In response to NRC concerns, additional analysis of the service water lines was performed. See pp. 12 to 32, Calcs for response to Question 241.0

U_m = Allowable sliding displacement
= 4"

Total inertial force on wall due to PGA of $K_h g$ = 316.1 K_h p. RW-20
Effective horizontal mass = 316.1 k/ft

Net horiz. resistance = μ[(Total weight) – (Buoyancy force) – (vertical seismic force)] + (key capacity) – (static active E.P. force)

$$= (136.4 - 50.8\,K_h) + (32.5) - (19.2)$$

$$= 149.7 - 50.8\,K_h$$

$$N = \frac{149.7 - 50.8A}{316.1} \qquad A = PGA$$

$$= 0.474 - 0.161A$$

$V = 28 \text{ in/sec}/g$

$= 28A$

$$0 = \frac{V^2}{2gN} - \frac{V^2}{2gA} - U_m$$

$$= \frac{(28A)^2}{2(386.4)(0.474 - 0.16/A)} - \frac{(28A)^2}{2(386.4)A} - 4$$

$$= \frac{1.014A^2}{0.474 - 0.161A} - 1.014A - 4$$

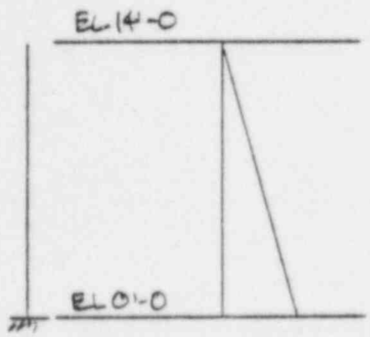
$A = 1.20$

$\check{A} = 1.2g$

Out-of-Plane Moment Failure of Wall At EL 0'-0

Out-of-plane moment failure of the wall at EL 0'-0 (top of counterforts) would occur at a relatively low acceleration if the pressures on p. RW-7 were applied. However, these pressures are based on development of the soil failure wedge from the heel of the footing. Failure of the wall at EL 0'-0 would imply development of the failure wedge from the wall at EL 0'-0. The ability of the wall to resist the forces occurring at the initiation of sliding will be checked.

Static Active Earth Pressure



$K_A = 0.22$

$p(EL\,0'\text{-}0) = 0.22(0.140)(14)$
$= 0.431$ ksf

$M = \frac{1}{2}(0.431)(14)(\frac{14}{3})$
$= 14.1$ k-ft/ft

Dynemic Active Earth Pressure At 0.41 g

$\Delta K_{AE} = 0.27$ p. RW-4

$\Delta P_{AE} = \frac{1}{2}\gamma H^2 \Delta K_{AE}$

$= \frac{1}{2}(0.14)(14)^2(0.27)$

$= 3.71\ ^{k}/_{ft}$

$M = \Delta P_{AE}(0.6H)$

$= 3.71(0.6)(14)$

$= 31.2\ ^{k\text{-}ft}/_{ft}$

Wall Inertial Force

$M = 0.15(3)(0.41)\ \frac{14.5^2}{2}$

$= 19.4\ ^{k\text{-}ft}/_{ft}$

Total

$M = 14.1 + 31.2 + 19.4$

$= 64.7\ ^{k\text{-}ft}/_{ft}$

$t = 36"$

reinf - #9 @ 6" vert EF

$A_s = 1.0\ \frac{12}{6}$

$= 2.0\ in^2/ft$

$d = 32"$

$a = \frac{2.0(54)}{0.85(5.9)(12)}$

$= 1.80"$

$M_u = 2.0(54)\cdot(32 - \frac{1.80}{2})$

$= 3359\ ^{"k}/_{ft}$

$= 280\ ^{k\text{-}ft}/_{ft} > 64.7\ ^{k\text{-}ft}/_{ft} \longrightarrow ok$

Failure of the retaining wall will occur due to sliding. Sliding will initiate at a peak ground acceleration of 0.39g. Failure of the service water lines due to a total sliding displacement of 4" will occur at a median acceleration capacity of 1.2g. Overturning of the wall can occur only if the wall does not slide since sufficient restraint must exist to provide a point about which the wall can pivot. Other failure modes cannot occur since they have acceleration capacities in excess of 0.39g and inertial forces will be limited to those occurring at the initiation of sliding.

Strength

$\check{A} = 1.2g$

$\check{F}_S = \frac{1.2}{0.17}$

$= 7.1$

Uncertainty

Uncertainty in Method

$\beta_U = 0.40$

Uncertainty in the Displacement Corresponding to Failure

A worst case would occur if the soil backfilled in the pumphouse were to be highly restrained by the pumphouse walls. If fixity of the pipe were to occur, the maximum pipe moment would be.

$$\left.\begin{aligned} \sigma_b &= \frac{k\,\delta_R}{2\lambda^2}\,\frac{R}{I} \\ M &= \frac{k\,\delta_R}{2\lambda^2} \end{aligned}\right\} \text{Iqbal + Goodling}$$

For a hinged penetration, the moment is

$$M = 0.1612 \frac{tR}{\lambda^2}$$

An allowable relative end displacement of about 2" was found for the hinged penetrations. The lower bound allowable sliding displacement is then.

$$u_m = 2.0 \frac{0.1612}{0.5}$$

$$= 0.64" \quad \text{say } 0.6"$$

$$0 = \frac{1.014 A^2}{0.474 - 0.161A} - 1.014A - 0.6$$

$A = 0.68$

This capacity is an extreme lower bound estimated to be 3σ below the mean

$$\beta_U = -\frac{1}{3} \ln \frac{0.68}{1.2}$$

$$= 0.19$$

Uncertainty in Resistance

Nearly all of the resistance is associated with friction. Uncertainty in the resistance should be about the same as for the DWST.

β_U on resistance = 0.20 p. DT-30

$$N_{LO} = (0.474 - 0.161A)e^{-0.20}$$

$$= 0.388 - 0.132A$$

$$0 = \frac{1.014A^2}{0.388-0.132A} - 1.014A - 4$$

$A = 1.11$

$$\beta_U = -\ln \frac{1.11}{1.2}$$

$= 0.08$

$$\beta_U = \sqrt{0.40^2+0.19^2+0.08^2}$$

$= 0.45$

Spectral Shape

$V_{+1\sigma} = 38.2$ in/sec /g p. DT-42

$= 38.2\ A$

$$0 = \frac{1.014A^2}{0.474-0.161A}\left(\frac{38.2}{28}\right)^2 - 1.014A\left(\frac{38.2}{28}\right)^2 - 4$$

$$= \frac{1.887A^2}{0.474-0.161A} - 1.887A - 4$$

$A = 0.99$

$$\beta_R = -\ln \frac{0.99}{1.2}$$

$= 0.19$

$\beta_U \approx \frac{2}{3}(0.19)$

$= 0.13$

Combination of EQ Components:

Simultaneous occurrence of the peak horizontal and vertical responses is an upper bound, so above the mean.

$$N = 0.474 - \frac{0.161A}{0.4}$$

$$= 0.474 - 0.403A$$

$$0 = \frac{1.014A^2}{0.474 - 0.403A} - 1.014A - 4$$

$A \approx 0.82$

$$\beta_R = -\frac{1}{3}\ln\frac{0.82}{1.2}$$

$$= 0.13$$

Total

$\check{A} = 1.2g$

$$\beta_R = \sqrt{0.19^2 + 0.13^2}$$

$$= 0.23$$

$$\beta_U = \sqrt{0.45^2 + 0.13^2}$$

$$= 0.47$$

MILLSTONE UNIT NO. 3

SEISMIC FRAGILITY EVALUATION

SENSITIVITY STUDY

WEST RETAINING WALL

RESPONSE TO QUESTION 241.23

Retaining Wall Sliding Capacity

The sliding capacity of the retaining wall will be recalculated using Newmark's unsymmetrical resistance equation. See pp. RW-17 to 25, 28 to 31.

Newmark's Unsymmetrical Resistance Equation

Newmark, N.M., "Effects of Earthquakes on Dams and Embankments", Geotechnique, Vol. 15, No. 2, 1965.

Eqn. 29 (Conservative upper bound)

$$\delta = \frac{V^2}{2gN}\,\frac{A}{N}$$

$$0 = \frac{V^2}{2gN^2}A - \delta$$

$$= \frac{(28A)^2}{2(386.4)(0.474-0.161A)^2}A - 4$$

$$= \frac{1.014A^3}{(0.474-0.161A)^2} - 4$$

A = 0.78

Slightly less conservative

$$\delta = \frac{V^2}{2gN}\,\frac{A}{N}\left(1 - \frac{N}{A}\right) \qquad \text{Fig. 22}$$

$$0 = \frac{V^2A}{2gN^2}\left(1 - \frac{N}{A}\right) - \delta$$

$$= \frac{1.014A^3}{(0.474-0.161A)^2}\left(1 - \frac{0.474-0.161A}{A}\right) - 4$$

A = 0.885

Strength

The capacity can be based upon Newmark's less conservative equation although some conservatism is still retained (Fig. 22).

$$F_s^v = \frac{0.885}{0.17}$$
$$= 5.21$$

Uncertainty

Method

$$\beta = 0.40$$

Displacement Corresponding to Failure

$$0 = \frac{1.014A^3}{(0.474-0.161A)^2}\left(1-\frac{0.474-0.161A}{A}\right) - 0.6 \qquad \text{p. RW-29}$$

$$A = 0.59$$

$$\beta = -\frac{1}{3}\ln\frac{0.59}{0.885}$$
$$= 0.14$$

Resistance

$$0 = \frac{1.014A^3}{(0.388-0.132A)^2}\left(1-\frac{0.388-0.132A}{A}\right) - 4 \qquad \text{p. RW-30}$$

$$A = 0.79$$

$$\beta = -\ln\frac{0.79}{0.885}$$
$$\approx 0.11$$

$$\beta_U = \sqrt{0.40^2+0.14^2+0.11^2}$$
$$= 0.44$$

V/a Ratio

$$0 = \frac{1.887A^3}{(0.474-0.161A)^2}\left(1 - \frac{0.474-0.161A}{A}\right) - 4 \qquad \text{p. RW-30}$$

$A = 0.777$

$\beta_R = -\ln \frac{0.777}{0.885}$

$= 0.13$

$\beta_U = \frac{2}{3}(0.13)$

$= 0.09$

Combination of EQ Components

$$0 = \frac{1.014A^3}{(0.474-0.403A)^2}\left(1 - \frac{0.474-0.403A}{A}\right) - 4 \qquad \text{p. RW-31}$$

$A = 0.646$

$\beta_R = -\frac{1}{3}\ln \frac{0.646}{0.885}$

$= 0.11$

Total

$\breve{A} = 0.898$

$\beta_R = \sqrt{0.13^2 + 0.11^2}$

$= 0.17$

$\beta_U = \sqrt{0.44^2 + 0.09^2}$

$= 0.45$

Failure of a Typical Service Water Pipe Length

The capacity of the service water line against free field ground strains will be checked. The typical line cross-section is shown in Sect. 6-6 on Dr. EC-24B

Maximum Ground Strains

Per ASCE Standard Provision Sect. 3610, it is more appropriate to base the ground strains on the apparent seismic wave speeds associated with the bedrock rather than the shallow overburden. Bending stresses due to free-field ground strains are small and can be neglected.

C_p = Compression wave velocity
≈ 13,000 fps

C_s = Shear wave velocity
≈ 6500 fps

ν = 0.33 } FSAR Sect. 2.5.4.4.3

γ = 165 pcf   FSAR Sect. 2.5.4.2.4

C_R = Rayleigh wave velocity
≈ 0.93 C_s $\nu = 0.33$
= 0.93 (6500)
≈ 6000 fps

Maximum Axial Strain

Per ASCE S.P. Sect. 3610, it is reasonable to assume that half of the peak ground velocity is due to shear waves and half is from Rayleigh waves

$V_m = 28$ in/sec/g

Shear Wave

$$\epsilon_{a,max} = \frac{V_{s\theta}}{2C_s} \qquad \theta = 45^\circ$$

$$= \frac{\frac{1}{2}(28)}{2(6500)(12)}$$

$$= 8.974 \times 10^{-5} \text{ in/in/g}$$

Rayleigh Wave

$$\epsilon_{a,max} = \frac{V_{R\theta}}{C_R} \qquad \theta = 0^\circ$$

$$= \frac{\frac{1}{2}(28)}{6000(12)}$$

$$= 1.944 \times 10^{-4} \text{ in/in/g}$$

Maximum Axial Strain

$$\epsilon_{a,max} = 8.974 \times 10^{-5} + 1.944 \times 10^{-4}$$

$$= 2.842 \times 10^{-4} \text{ in/in/g}$$

Maximum Shear Strain

Shear Wave

$$\gamma_{max} = \frac{V_{S\theta}}{C_S} \qquad \theta = 0^\circ$$

$$= \frac{\frac{1}{2}(28)}{6500(12)}$$

$$= 1.795 \times 10^{-4} \text{ in/in/g}$$

Rayleigh Wave

$$\gamma_{max} = \frac{V_{R\theta}}{C_R} \qquad \theta = 0^\circ$$

$$= \frac{\frac{1}{2}(28)}{6000(12)}$$

$$= 1.944 \times 10^{-4} \text{ in/in/g}$$

Maximum Strain

$$\gamma_{max} = 1.795 \times 10^{-4} + 1.944 \times 10^{-4}$$

$$= 3.739 \times 10^{-4} \text{ in/in/g}$$

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BY: PSH DATE: 6/3/85

CHKD. BY: DW DATE: 6/6/85

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COMMENTS: Q.241.0

Tension Capacity

Conservatively neglect concrete encasement capacity. Typical pipe section is 30"Φ, 0.313" wall thickness fabricated from ASTM B466 or B467, No. 706 copper nickel alloy.

$f_{y,min} = 13$ ksi

$f_{u,min} = 38$ ksi

$\check{f}_y \approx 1.25(13)$

$= 16$ ksi

$\check{f}_u \approx 1.2(38)$

$= 46$ ksi

$E = 22{,}000$ ksi

$\sigma_{max} = 22{,}000\,(2.842 \times 10^{-4})$

$= 6.25$ ksi/g

$\check{A} = \dfrac{38}{6.25}\,(1g)$

$= 6.1g$

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Sheer Capacity

Conservatively neglect concrete encasement capacity.

$$\tau_y \approx \frac{\sigma_y}{\sqrt{3}}$$

$$= \frac{16}{\sqrt{3}}$$

$$= 9.2 \text{ ksi}$$

$$G = \frac{22{,}000}{2(1+0.3)}$$

$$= 8500 \text{ ksi}$$

$$\tau_m = 8500\,(3.739 \times 10^{-4})$$

$$= 3.18 \text{ ksi/g}$$

$$\check{A} = \frac{9.2}{3.18}\,(1g)$$

$$= 2.9g$$

Compression Capacity

The service water pipes will be precluded from buckling by the concrete encasement until the encasement crushes and fractures at its compressive strength. A lower bound compression capacity is the encasement compressive strength.

$f'_{c,min} = 3000$ psi

The minimum median strength for 3000 psi concrete at Millstone was found to be 4500 psi for the control bldg.

$\sigma_u = 0.85(4500)$
$= 3800$ psi

$E_c = 57{,}000\sqrt{4500}$
$= 3.8 \times 10^4$ psi

$\sigma_m = 3.8\times10^4(2.842\times10^{-4})$
$= 1080$ psi/g

$A = \frac{3800}{1080}(1g)$
$= 3.5g$

Displacement Capacity of the Service Water Line

The service water lines may fail if sufficient sliding displacement of the retaining wall occurs. This failure will occur at the wall expansion joint shown on Dr. EC-14V. The portion of wall north of the joint will slide while the portion to the south will be anchored by the shear key shown in Fig. 2-2.

Pipe Buckling Moment

From Walt Briggs

30" ϕ, 0.313" wall thickness

ASTM B466 or B467, No. 706

$f_{y,min} = 13$ ksi at 0.5% extension
$f_{u,min} = 38$ ksi
$\epsilon_{ult} = 25\%$ in 2"

} B467, No. 706, ASTM Sect. II, Part B

$\check{f}_y \approx 1.25(13)$
$= 16$ ksi

$\check{f}_u \approx 1.2(38)$
$= 46$ ksi

$E = 22{,}000$ ksi

$A = \frac{\pi}{4}\left[30^2 - (30 - 2\times0.313)^2\right]$
$= 29.2 \text{ in}^2$ ✓

$I = \frac{\pi}{64}\left[30^4 - (30 - 2\times0.313)^4\right]$
$= 3216 \text{ in}^4$ ✓

This material exhibits a roundhouse type stress-strain curve that can be fitted with a Ramberg-Osgood-Hill curve.

$$\epsilon = \frac{\sigma}{E} + 0.002\left(\frac{\sigma}{\sigma_2}\right)^n$$

$$n = \frac{0.301}{\log\left(\frac{\sigma_2}{\sigma_1}\right)}$$

From Clark & Rolf.

σ_1, σ_2 = Compressive yield strengths at 0.1% and 0.2% offset

When $\epsilon = 0.005$ in/in, $\sigma = 16$ ksi

" " 0.25 in/in " " 46 ksi

$E = 22{,}000$ ksi

$$0.005 = \frac{16}{22{,}000} + 0.002\left(\frac{16}{\sigma_2}\right)^n$$

$$\left(\frac{16}{\sigma_2}\right)^n = 2.136$$

$$\frac{16}{\sigma_2} = 2.136^{1/n}$$

$$\sigma_2 = \frac{16}{2.136^{1/n}}$$

$$0.25 = \frac{46}{22{,}000} + 0.002\left[\frac{46}{16/2.136^{1/n}}\right]^n$$

$$0 = -124.0 + \left[2.875\,(2.136)^{1/n}\right]^n$$

$$= -124.0 + 2.136\,(2.875)^n$$

Clark, J.W. and R.L. Rolf, "Design of Aluminum Tubular Members", J. Structural Division, ASCE, December, 1964.

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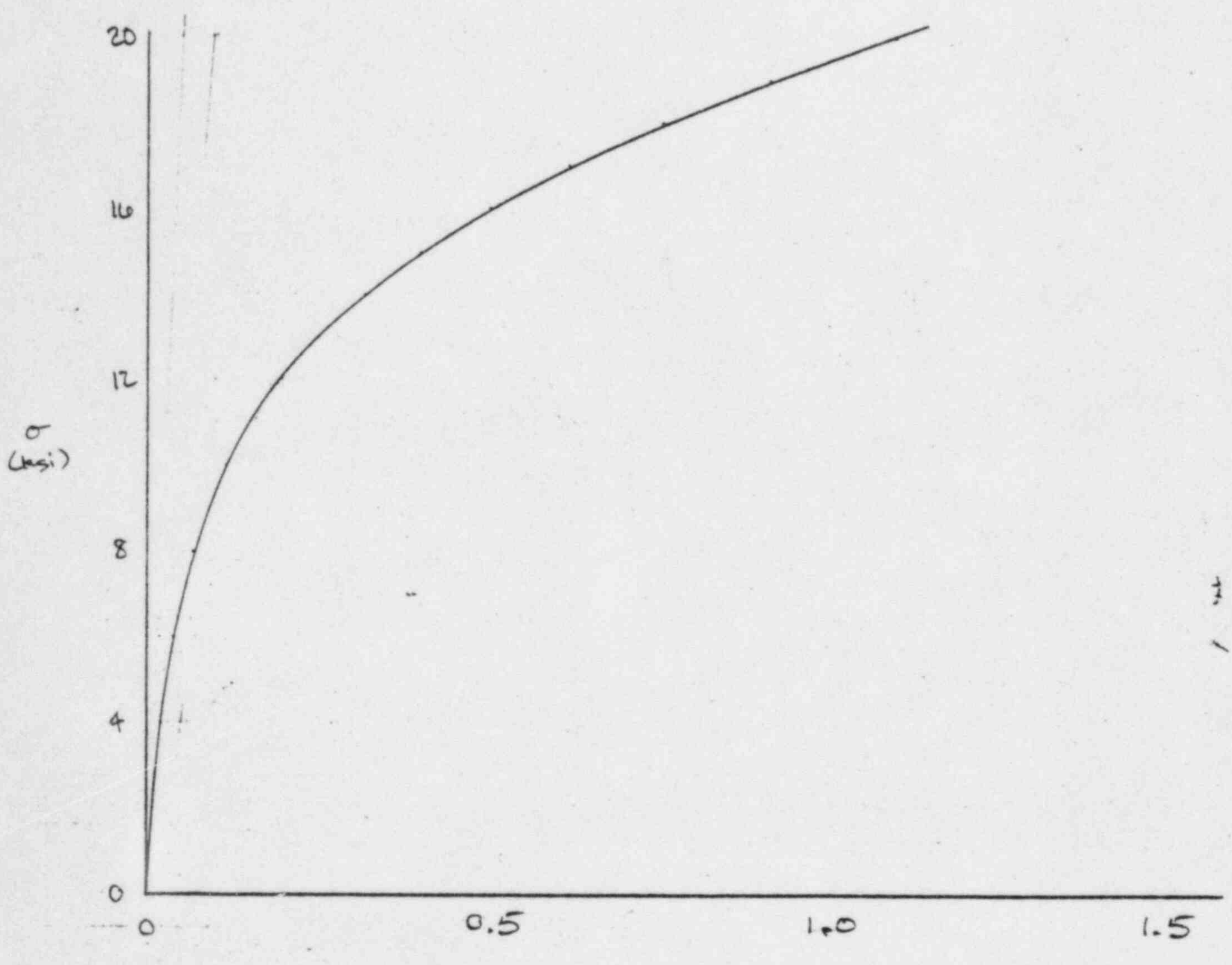
By trial & error

$n = 3.846$

$\sigma_2 = 13.13$ ksi

$\sigma_1 = 10.97$ ksi



Buckling Strength per Clark & Rolf for Aluminum

σ_{tb} = Buckling strength in bending

$$= B_{tb} - D_{tb}\sqrt{\frac{R}{t}}$$

$$B_{tb} = 1.5\sigma_y\left[1 + 4.6\sqrt[5]{\frac{1000\,\sigma_y}{E}}\left(\frac{\sigma_2}{\sigma_1} - 1\right)\right]$$

$$D_{tb} = \frac{B_{tb}}{2.7}\sqrt[3]{\frac{B_{tb}}{E}}$$

σ_y = yield strength (at 0.2% offset for aluminum, typ)

This equation is appropriate for $R/t < \left(\frac{B_{tb} - B_t}{D_{tb} - D_t}\right)^2$

$$B_t = \sigma_2\left[1 + 4.6\sqrt[5]{\frac{1000\,\sigma_2}{E}}\left(\frac{\sigma_2}{\sigma_1} - 1\right)\right]$$

$$D_t = \frac{B_t}{0.9}\sqrt[3]{\frac{B_t}{E}}\sqrt{\frac{\sigma_2}{\sigma_1} - 1}$$

$$\frac{\sigma_2}{\sigma_1} \le 1.06$$

$$\bar{R} = \tfrac{1}{2}(30 - 0.315)$$
$$= 14.8''$$

$$\frac{R}{t} = \frac{14.8}{0.315}$$
$$= 47.4$$

$$B_{tb} = 1.5(13.13)\left[1 + 4.6\sqrt[5]{\frac{1000(13.13)}{22{,}000}}(1.06 - 1)\right]$$
$$= 24.60 \text{ ksi}$$

$$D_{tb} = \frac{24.60}{2.7}\sqrt[3]{\frac{24.60}{22{,}000}}$$
$$= 0.946$$

$$B_t = \frac{B_{tb}}{1.5}$$
$$= \frac{24.60}{1.5}$$
$$= 16.40 \text{ ksi}$$

$$D_t = \frac{16.40}{0.9} = \sqrt{\frac{16.40}{22{,}000}}\sqrt{1.06-1}$$
$$= 0.405$$

$$\left(\frac{B_{tb}-B_t}{D_{tb}-D_t}\right)^2 = \left(\frac{24.60-16.40}{0.946-0.405}\right)^2$$
$$= 230 > \frac{R}{t} \quad \text{ok}$$

$$\sigma_{tb} = 24.60 - 0.946\sqrt{47.4}$$
$$= 18.1 \text{ ksi}$$

$$\epsilon_{tb} = \frac{18.1}{22{,}000} + 0.002\left(\frac{18.1}{13.13}\right)^{3.846}$$
$$\sim 7.687\times10^{-3} \text{ in/in}$$

$$\psi = \frac{7.687\times10^{-3}}{15}$$
$$= 5.125\times10^{-4} \text{ in}^{-1}$$

Buckling By NASA Shell Analysis Manual - see pp. MC-48 to 52A

For $R/t = 47$, $C_x = 0.33$ (p. MC-52)

$\check{\sigma}_{cr,i} = 1.4\, C_x \dfrac{E}{R/t}$ 1.4 median factor of safety

$= 1.4(0.33)\dfrac{22{,}000}{47.4}$

$= 214$ ksi $\longrightarrow$ Inelastic buckling

Estimate $\check{\sigma}_{cr} \approx 16.3$ ksi

$\epsilon_{cr} = \dfrac{16.3}{22{,}000} + 0.002\left(\dfrac{16.3}{13.13}\right)^{3.846}$

$= 5.329 \times 10^{-3}$ in/in

$E_s = \dfrac{16.3}{5.329 \times 10^{-3}}$

$= 3060$ ksi

$E_t \approx 910$ ksi Graphically, p. 14

$\eta = \dfrac{\sqrt{3060(910)}}{22{,}000}$

$= 0.0759$

$\check{\sigma}_{cr} = 0.0759(214)$

$= 16.2$ ksi

This value is reasonably close to the value predicted by Rolf and Clark's equation for aluminum. It is appropriate to add in the increased buckling capacity provided by the pipe internal pressure. Estimate the internal pressure to be about 150 psi (per RDC).

p = 150 psi

$$\left(\frac{p}{E}\right)\left(\frac{R}{t}\right)^2 = \frac{0.150}{22,000}(47.4)^2$$

$$= 0.015$$

$\Delta C_b = 0.16$ p. MC-52A

$$\check{\sigma}_{cr,e} = 1.4(C_b + \Delta C_b)\frac{E}{R/t}$$

$$= 1.4(0.33 + 0.16)\frac{22,000}{47.4}$$

$$= 318 \text{ ksi}$$ inelastic buckling

Estimate $\sigma_{cr} = 18.2$ ksi

$$\epsilon_{cr} = \frac{18.2}{22,000} + 0.002\left(\frac{18.2}{13.13}\right)^{3.846}$$

$$= 7.849 \times 10^{-3} \text{ in/in}$$

$$E_s = \frac{18.2}{7.849 \times 10^{-3}}$$

$$= 2320 \text{ ksi}$$

$E_t = 670$ ksi

$$\eta = \frac{\sqrt{2320(670)}}{22,000}$$

$$= 0.0567$$

$$\sigma_{cr} = 0.0567(318)$$

$$= 18.0 \text{ ksi} \quad \text{ok}$$

$\dot{\sigma}_{cr} = 18.0$ ksi

$\epsilon_{cr} = 7.547 \times 10^{-3}$ in/in

$$\psi_{cr} = \frac{7.547 \times 10^{-3}}{15}$$

$$= 5.032 \times 10^{-4} \text{ in}^{-1}$$

The buckling stress will be taken to be the 18.0 ksi based on the NASA design manual since it includes the effect of internal pressurization. Comparison of the NASA buckling stress without internal pressure with the buckling stress predicted by the Clark and Rolf equation for aluminum indicates that some conservatism may be included.

$\sigma_{cr} = 18$ ksi

$I = 3216 \text{ in}^4$

$S = \frac{3216}{15}$

$= 214 \text{ in}^3$

$M_{cr} = 18\,(214)$

$= 3850$ k-in

Soil Shear Modulus

The backfill soil shear modulus will be estimated using an approximate approach to account for the effects of strain degradation. The depth of soil over the service water line is 8'-0 to the TOC and 12'-6 to the BOC. Strain degradation will be based upon an acceleration level of 0.4g which is the level at which the retaining wall begins to slide.

$\gamma \approx 140$ pcf FSAR Sect. 2.5.4.5.2

$K_o \approx 0.5$ est.

$\bar{H} = \frac{1}{2}(8 + 12.5)$

$= 10$ ft

$\sigma_1 = 140(10)$

$= 1400$ psf

$\bar{\sigma}_o = \dfrac{\sigma_1 + 2K_o\sigma_1}{3}$

$= \dfrac{\sigma_1 + 2(0.5)\sigma_1}{3}$

$= \frac{2}{3}\sigma_1$

$= \frac{2}{3}(1400)$

$= 933$ psf

$= 6.48$ psi

For $\bar{\sigma}_o = 933$ psf, $G_{max} \approx 14{,}000$ psi (Fig 6, p. 21, Millstone 3 Design Criteria)

"Structural Design Criteria For Northeast Utilities Service Company, Millstone - Unit 3", Stone and Webster Engineering Corp., January, 1983.

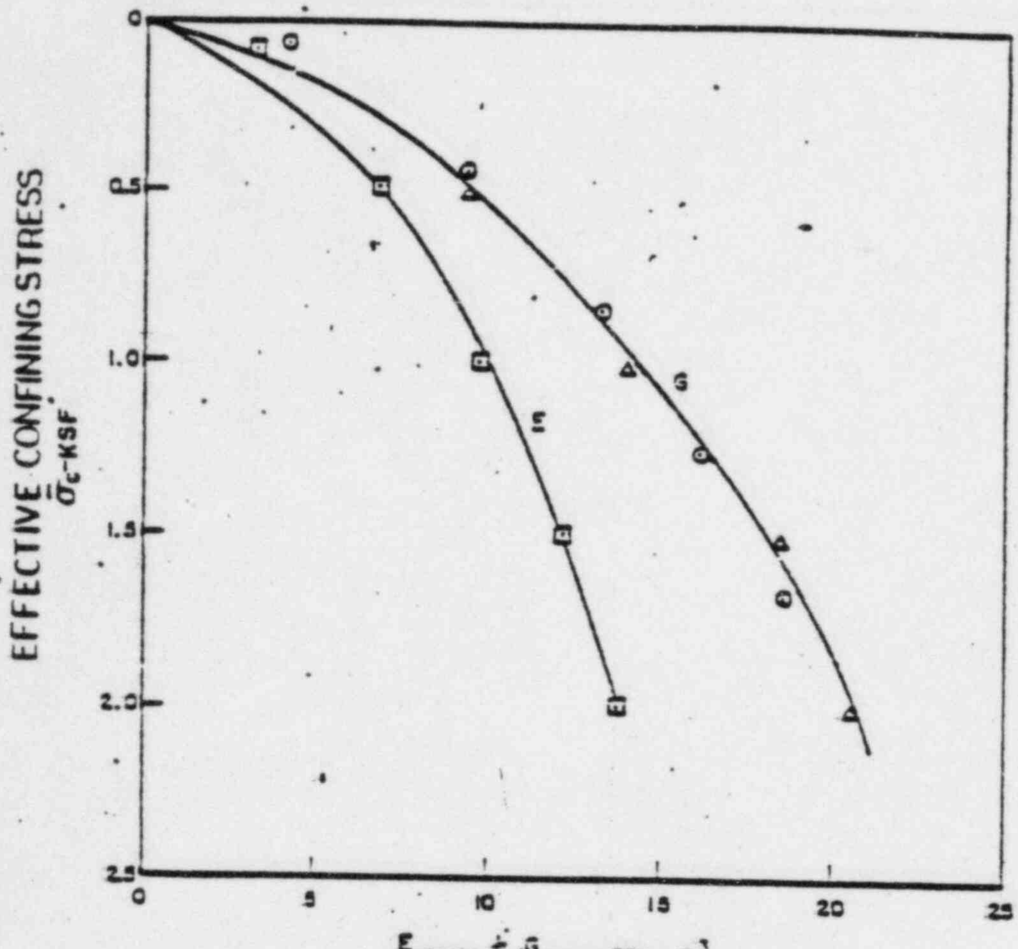


FIGURE 6
MODULUS VS DEPTH -
STRUCTURAL FILL
MILLSTONE 3

Millstone 3 Design Criteria

$A = 0.4g$

$V = 28(0.4g)$

$= 11.2$ in/sec

Strain Degradation - See Midland Vol. I.

1. Assume a value of G/G_{max}
2. Calculate shear wave velocity, $V_s = \sqrt{\frac{G}{\rho}}$
3. Calculate shear strain, $\gamma = \frac{V}{V_s}$
4. Determine G/G_{max} from Seed & Idriss curve for sand (see p. 24)
5. Iterate on G/G_{max}

Assume $G/G_{max} = 0.1$

$G = 0.1(14{,}000)$

$= 1400$ psi

$\rho = \frac{140}{386.4(1728)}$

$= 2.10 \times 10^{-4}$ lb-sec^2/in^4

$V_s = \sqrt{\frac{1400}{2.10 \times 10^{-4}}}$

$= 2580$ in/sec

$\gamma = \frac{11.2}{2580}$

$= 0.00434$

$= 0.43\%$

$G/G_{max} = 0.12$ Seed & Idriss sand, Midland Vol. I, Fig I-3-4

$G = 0.12\,(14{,}000)$

$= 1680$ psi

$V_s = \sqrt{\dfrac{1680}{2.1\times10^{-4}}}$

$= 2830$ in/sec

$\gamma = \dfrac{11.2}{2830}\times10^{2}$

$= 0.40\%$

$G/G_{max} = 0.12$ ok

$G = \cancel{1400}\ 1700$ psi

Note – This value may be somewhat conservative in that it is based on soil stresses and strains at the PGA when the wall starts to slide. The PGA when the pipe fails will be in-excess of this level. Stresses and strains in the soil behind the portion of wall that does not slide will actually be higher, and the effective soil stiffness lower. Furthermore, additional strains will be imposed by the service water line bearing against the soil and strains introduced by development of the active state.

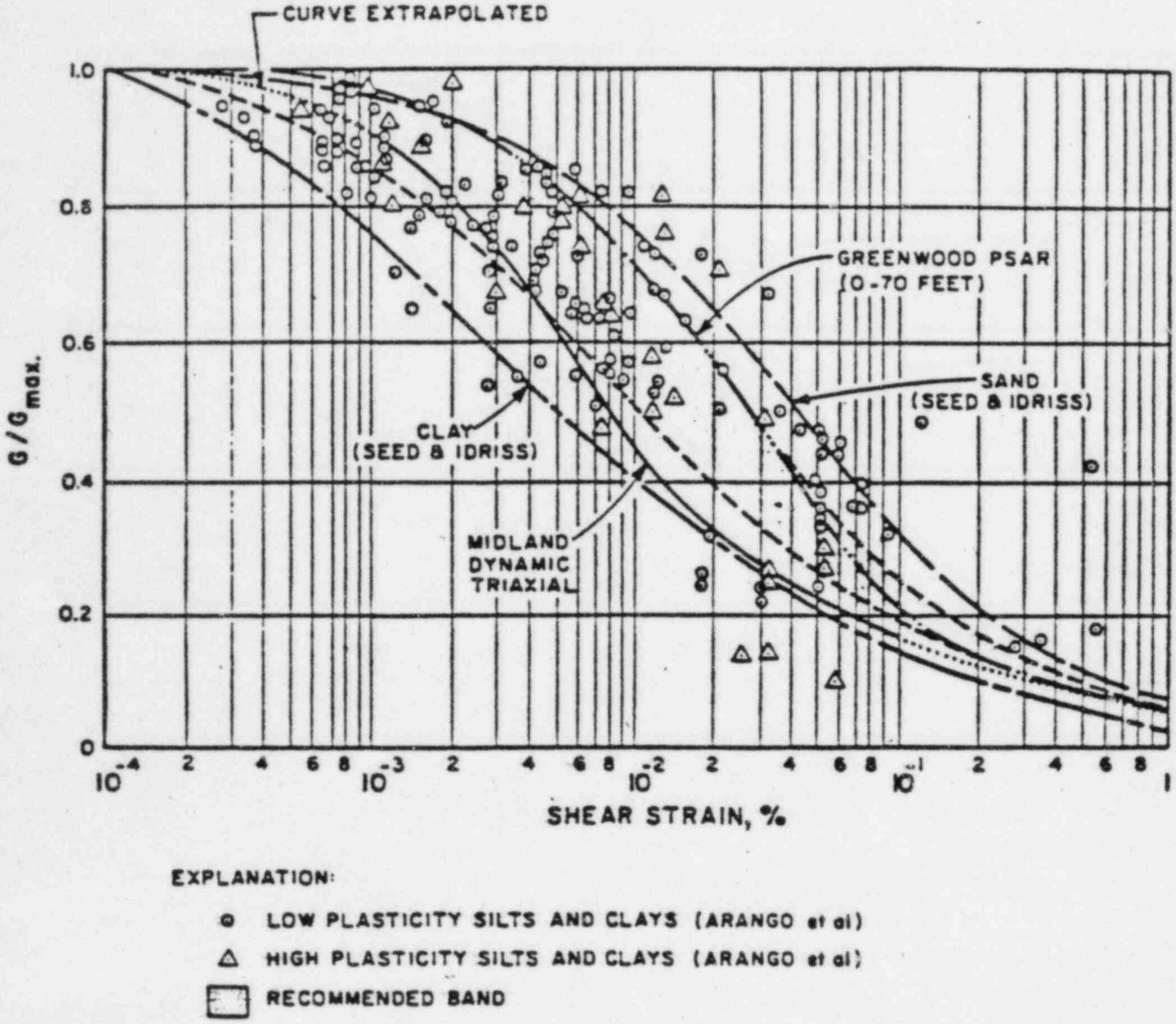
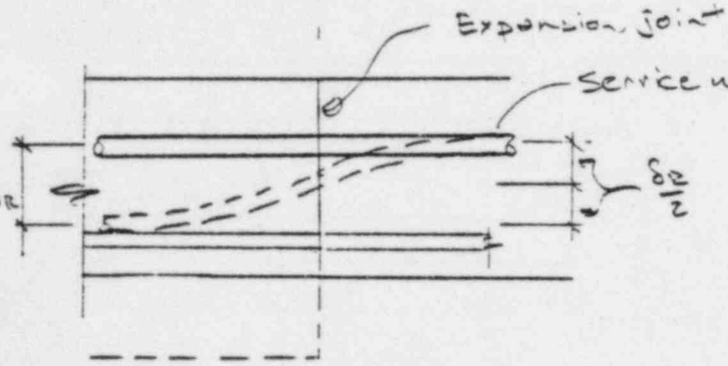


FIGURE I-3-4. STRAIN DEGRADATION RELATIONSHIPS FOR MIDLAND SITE (after Ref. 8)

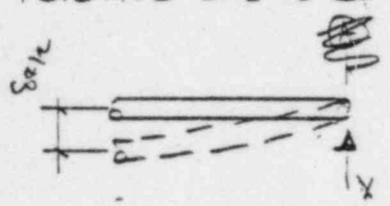
"Seismic Margin Review - Midland Energy Center Project - Volume I, Methodology and Criteria", Structural Mechanics Associates, SMA 13701.05R005 (Volume I), February 1983.

Buried Pipe Loads

As previously noted, the portion of wall north of the expansion joint will slide while the portion of the wall to the south may not. Assuming that the soil has sufficient capacity to drag the S.W. line with the wall, the displaced shape of the line will be as shown below.



Because of symmetry, no moment acts on the S.W. lines at the joint. The S.W. line can be represented as a semi-infinite buried pipe with a hinged support at the joint and subjected to a relative end displacement of $\delta_R/2$.



TITLE Millstone 3 | BY PSH | DATE [illegible] | CHKD. BY [illegible] | DATE [illegible]

STRUCTURAL MECHANICS ASSOCIATES A Calif. Corp.

COMMENTS [illegible]

Sliding-induced loads on the pipes can be determined using the beam on elastic foundation solution. Preliminary calculations indicate that all of the load will be initially resisted by the stiff concrete encasement, but that the encasement will crack and yield at relatively low levels of displacement. Yielding of the encasement does not imply failure since flow of the S.W. line will not be interrupted. Further resistance against soil loading is provided by the S.W. pipes themselves.

The sliding-induced failure displacement will be taken as that causing buckling of the S.W. pipes. This is somewhat conservative since displacement in excess of that causing buckling is necessary to significantly block flow. Even if the pipes crack due to buckling, the concrete encasement will prevent significant leakage. It will also be assumed, also with some conservatism, that the concrete encasement is significantly cracked such that it does not provide any additional resistance against soil loads or pipe buckling.

For a hinged penetration (concentrated end load, no moment)

σ_b = Bending stress

$= 0.1612 \dfrac{kR}{\lambda^2 I} \delta_R$

$k = b k_o$

k_o = Coefficient of subgrade reaction

b = Pipe diameter

$\lambda = \sqrt[4]{\dfrac{k}{4EI}}$

R, I = Pipe radius, moment of inertia

δ_R = Imposed displacement

Iqbal & Goodling

$M = \sigma_b \dfrac{I}{R}$

$= 0.1612 \dfrac{k}{\lambda^2} \delta_R$

$k_o = 0.65 \sqrt[12]{\dfrac{E_s D^4}{E_p I_p}} \dfrac{E_s}{1-\nu_s^2} \cdot \dfrac{1}{D}$ Vesic's eqn. Eqn. 10, Iqbal & Goodling

E_s = Soil elastic modulus

D = Pipe diameter

E_p = Pipe elastic modulus

I_p = Pipe moment of inertia

Iqbal, M. A. and E. C. Goodling, "Seismic Design of Buried Piping", 2nd ASCE Specialty Conference on Structural Design of Nuclear Plant Facilities, New Orleans, LA, December 8-10, 1975.

$\nu_s = 0.2$ to 0.3 Millstone 3 Design Criteria, for backfill

Use $\nu_s = 0.25$

$G_s = 1700$ psi

$E_s = 2(1+0.25)(1700)$

$= 4250$ psi

$I_p = 3216$ in^4 one pipe only

$E_p = 22 \times 10^6$ psi Initial modulus

$D = 54"$ Depth of concrete encasement

For the pipe buckling stress of 18 ksi, inspection of the material stress-strain curve indicates that the material is stressed into the strain-hardening portion of the curve at buckling. For the calculated curvature and moment at buckling, the effective pipe stiffness is:

$M_{cr} = 3850$ k-in

$\psi_{cr} = 5.032 \times 10^{-4}$ in^{-1}

$$EI_{cr} = \frac{3850(1000)}{5.032 \times 10^{-4}}$$

$= 7.651 \times 10^9$ lb-in^2

This value is less than the elastic stiffness of

$EI_e = 22 \times 10^6 (3216)$

$= 7.075 \times 10^{10}$ lb-in^2

The stiffness varies from a maximum of 7.075×10^{10} lb-in^2 to a minimum of 7.651×10^9 lb-in^2 over the length of the pipe. In lieu of a nonlinear analysis, the effective stiffness will be taken as the average value.

$(E_p I_p)_{eff.} = \frac{1}{2}(7.651\times10^{9} + 7.075\times10^{10})(2)$ ← 2 pipes total

$= 7.840\times10^{10}$ lb-in^2

$k = k_o D$

$= 0.65 \sqrt[12]{\frac{4250\,(54)^4}{7.840\times10^{10}}}\ \frac{4250}{1-0.25^2}$

$= 2762$ psi

$= 2.762$ ksi

$\lambda = \sqrt[4]{\frac{2762}{4\,(7.840\times10^{10})}}$

$= 0.009688$

$M = 0.1612 \frac{2.762}{0.009688^2}\delta_R$

$= 4745\,\delta_R$ Total - 2 pipes

$= 2372\,\delta_R$ k-in/pipe

Allowable Sliding Displacement

$M = M_{cr}$

$2372\, \delta_R = 3850$

$\delta_R = 1.62"$

$\Delta_{all} = 2\delta_R$

$= 3.25"$

In light of the conservatisms introduced into these calculations, it is reasonable to expect that the pipe can accomodate a 4" sliding displacement of the wall.

Check Shear

Conservatively neglect shear strength of the concrete encasement.

$$\tau_{max} = \frac{\alpha k}{2\lambda A}\,\delta_R$$ Eqn. 48, Iqbal and Goodling

A = Cross-sectional area

α = Shape factor
= 2

A = 2(29.2) total, 2 pipes
= 58.4 in²

$$\tau_{max} = \frac{2(2.762)}{2(0.009688)(58.4)}(1.62)$$

= 7.9 ksi

$$\tau_y = \frac{f_y}{\sqrt{3}}$$

$$= \frac{16}{\sqrt{3}}$$

= 9.2 ksi > 7.9 ksi ok

Richards & Elms

Richards, R. and D.G. Elms, "Seismic Behavior of Gravity Retaining Walls", Journal of the Geotechnical Engineering Division, ASCE, April, 1979.

$$d = 0.087 \frac{V^2}{Ag}\left(\frac{N}{A}\right)^{-4} \qquad \text{Eqn. 26}$$

$$0 = 0.087 \frac{(28A)^2}{A(386.4)}\left(\frac{0.474-0.161A}{A}\right)^{-4} - 4$$

$$= 0.1765\,A\left(\frac{0.474-0.161A}{A}\right)^{-4} - 4$$

$A = 0.798$

Check Sliding Using Nadim & Whitman

At 0.4g ZPA

At middepth of 35' deep layer

$\sigma_1 = 140(17.5)$
$= 2450$ psf

$\overline{\sigma_0} = \frac{2}{3}(2450)$
$= 1630$ psf
$= 11.3$ psi

$G_{max} \approx 19{,}000$ psi

$V = 0.4(28)$
$= 11.2$ ips

Try $G/G_{max} = 0.2$

$G = 0.2(19{,}000)$
$= 3800$ psi

$v_s = \sqrt{\frac{3800}{2.1\times10^{-4}}}$
$= 4250$ ips

$\gamma = \frac{11.2}{4250}(100)$
$= 0.26\%$

Try $G/G_{max} = 0.16$

$G = 0.16(19{,}000)$
$= 3040$ psi

$v_s = \sqrt{\frac{3040}{2.1\times10^{-4}}}$
$= 3805$ ips

$\gamma = \frac{11.2}{3805}(100)$
$= 0.29\%$

$G/G_{max} = 0.16$ or

$G = 3000$ psi

$V_s = 3780$ in/sec

$= 315$ fps

$f1 = \frac{315}{4(35)}$

$= 2.25$ Hz

For f from about 4 Hz to 10 Hz, f/f1 varies from about 1.2 to 4. N+W do not address this range, but inspection of Fig 7 for sinusoidal input would indicate little effect.

Worst Case

Increase A and V by 50%.

V/A doesn't change

$A = \frac{0.80g}{1.50}$ RoE

$= 0.53g$

Question 241.24

The consequence of seismically induced liquefaction of beach sands is not specifically addressed in the Millstone seismic capability study (Reference 1). The beach sands could be liquefied under seismic events greater than SSE and could flow toward the intake structure, thus preventing and intake structure from conveying the cooling water for the safe shut down. Therefore, provide the analyses for the stability of the beach sand slope and assess its consequences under seismic events greater than SSE.

Response

The response to this question was discussed in NNECO's May 14, 1985 presentation to the NRC Staff and it was agreed at that time that this item is resolved. The following pages detail the presentation given to the Staff.

MILLSTONE NUCLEAR POWER STATION - UNIT 3

SUMMARY OF ANALYSIS OF POSTULATED LIQUEFACTION OF BEACH SAND FOR EARTHQUAKES LARGER THAN THE SSE

The circulating and service water pumphouse is located on the shorefront of Long Island Sound, as shown in Figure 2. A liquefaction analysis of the sands underlying the slope protection zone adjacent to the pumphouse was performed using a 2-dimensional dynamic response analysis in response to question 241.7 and is discussed in FSAR Section 2.5.4.8.3.3. Figure 1 summarizes the results of the analysis and presents the factors of safety against liquefaction for the sand zone under SSE conditions. The analysis indicated that an adequate margin of safety against liquefaction existed for the sand zone at the pumphouse. The east slope was not analyzed since it consists of a sloping rock surface with no sand or till layers.

A fragility analysis of the shorefront slope was performed to assess the effects of earthquakes larger than the SSE. The assumption was made that the sand liquefies under a large earthquake of unspecified magnitude. The shorefront consists of a slope protection zone, sand, basal till, and bedrock as shown in Figure 3. This analysis conservatively assumes that the entire zone of sand liquefies simultaneously, causing the sand and overlying slope protection zone to move as a unit towards the intake channel. From the results of the liquefaction analysis under SSE conditions presented in Figure 1, it is expected that liquefaction during larger events would initially occur in the sand at the toe of the slope. The exact location corresponding to initiation and extent of liquefaction could vary depending on the earthquake magnitude. However, it is unlikely that the entire sand zone would completely liquefy; rather, progressive slumping failures would occur as unliquefied sand behind a liquefied zone slumps into the void left by the sliding sand. It is unlikely that sand beyond the top of slope at el 14 ft would be mobilized and able to move sufficiently to block the intake channel because of the orientation of the top of the slope to the intake channel. Therefore, the assumption used in the analysis of massive flow slides initiating at the toe of the slope and moving large distances is very conservative, even under large magnitude earthquakes.

A liquefaction-induced flow slide of the entire sand and slope protection zones was postulated for the fragility analysis. A post-flow final slope of 20:1 was assumed to occur based on observations by Seed (Ref. 1) from the 1964 Alaska earthquake. The magnitude of the Alaskan earthquake was 8.3, compared to the SSE magnitude of 5.3 at the Millstone site. The existing and post-flow shorefront contours at the pumphouse are shown on Figure 2. Sections A-A, B-B, and C-C (Figure 3) were constructed through the slope and dredged channel.

From these assumptions, the final configuration of the slope was checked for each section on Figure 3 by a conservation of soils method, where the final slope is drawn so that area A1 is approximately equal to A2. This method was applied to Sections A-A and B-B; the intake channel side-slope at Section C-C is too shallow for any significant flow slide to occur. The final slope configurations postulated on these three cross-sections resulted

in a revised topography of the channel bottom as plotted on Figures 2, 4, and 5.

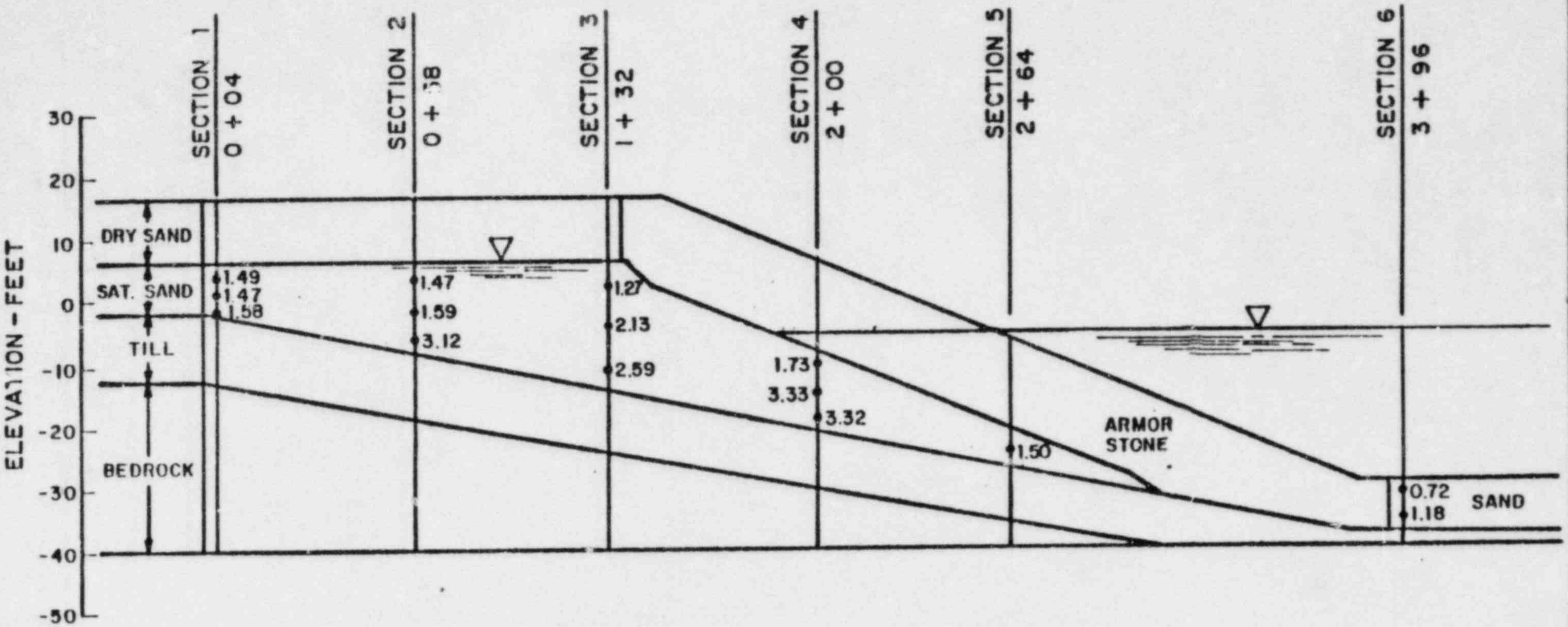
The post-flow channel elevation in front of the pumphouse would be elevation -20 ft, compared to the original elevation of -30 ft under normal conditions. The service water pump inlet, shown on Figure 5, extends to elevation -13 ft; therefore, 7 ft of water would remain available for the pumps after the slide occurs. Therefore, a liquefaction-induced flow slide into the intake channel will not adversely affect the supply of water required for cooling of safety-related systems, even for earthquakes in excess of the SSE magnitude.

The sensitivity analysis summarized above illustrates that liquefaction of the beach sand would not be a dominant seismically induced failure mode in the seismic margins evaluation (Ref. 2) and need not be considered further.

References

1. Seed, H.B. 1968. Landslides during Earthquakes due to Liquefaction. Journal of the Soil Mechanics and Foundation Engineering Division, ASCE, Vol 94, No. SM5.

2. "A Program to Determine the Capability of the Millstone 3 Nuclear Power Plant to Withstand Seismic Excitation above the Design SSE," prepared by NTS/Structural Mechanics Associates for Northeast Utilities, November 1984.



NOTES

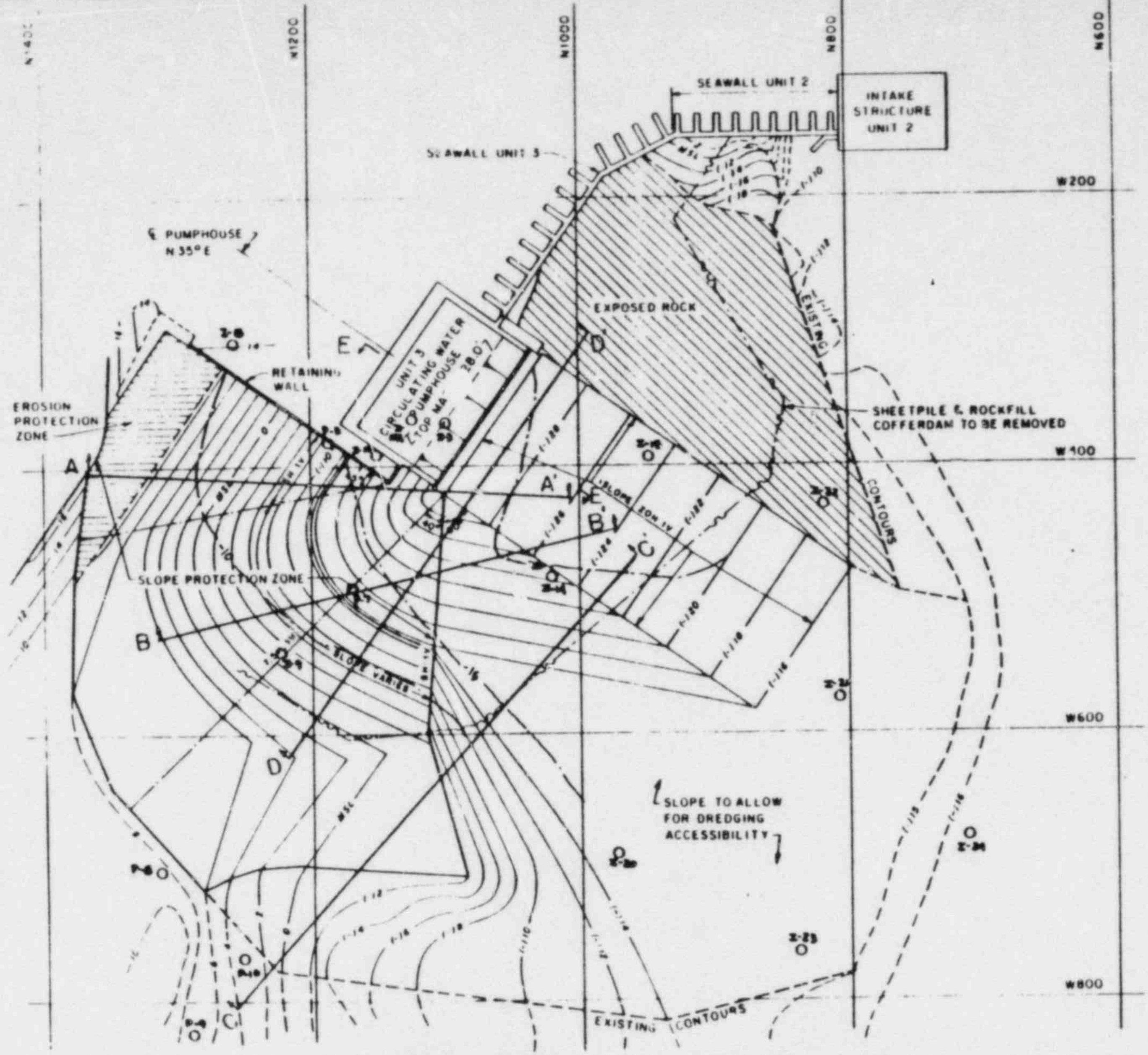
●1.49= FACTOR OF SAFETY AGAINST LIQUEFACTION

REFER TO FSAR SECTION 2.5.4.8.3.3 FOR DISCUSSION

FIGURE 1

SHOREFRONT PROFILE USED IN LIQUEFACTION ANALYSES

MILLSTONE NUCLEAR POWER STATION UNIT 3



Notes:

O Boring Locations

— - — Contours of Postulated Post-flow Slope

Ref. Fig. 2.5.4-41

0 50 100 150
SCALE FEET

FIGURE 2

SHOREFRONT & DREDGING PLAN
MILLSTONE NUCLEAR POWER STATION
UNIT 3

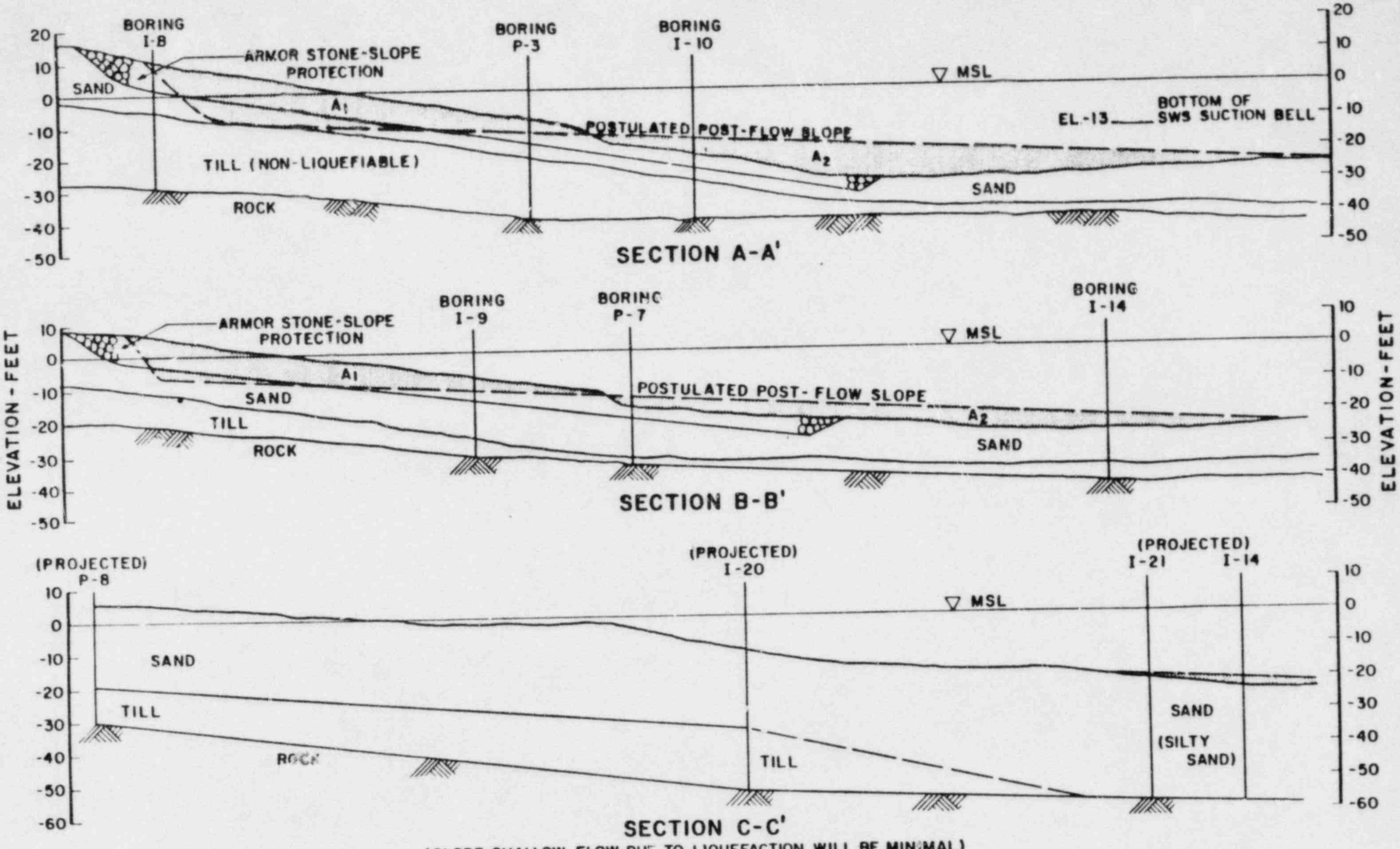
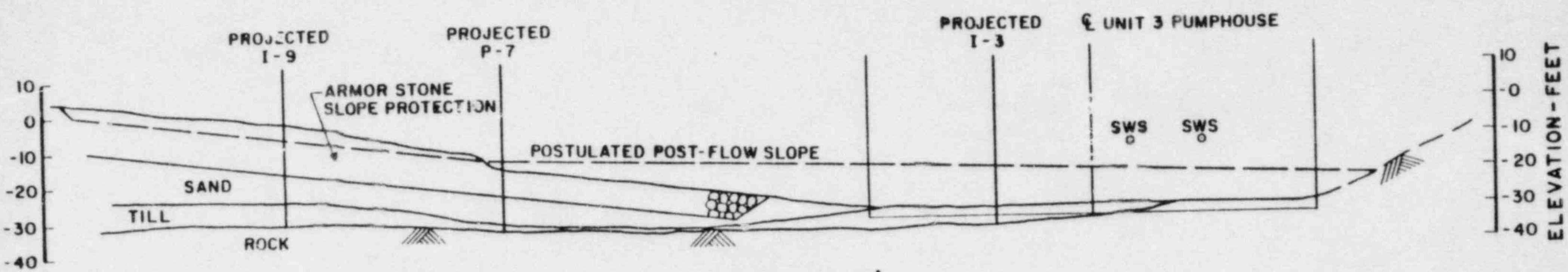

BORING I-8
BORING P-3
BORING I-10
ARMOR STONE-SLOPE PROTECTION
MSL
SAND
A1
EL-13
BOTTOM OF SWS SUCTION BELL
POSTULATED POST-FLOW SLOPE
TILL (NON-LIQUEFIABLE)
A2
SAND
ROCK
SECTION A-A'
BORING I-9
BORING P-7
BORING I-14
ARMOR STONE-SLOPE PROTECTION
MSL
A1
POSTULATED POST-FLOW SLOPE
SAND
A2
TILL
ROCK
SAND
ELEVATION - FEET
SECTION B-B'
(PROJECTED) P-8
(PROJECTED) I-20
(PROJECTED) I-21 I-14
MSL
SAND
TILL
SAND
(SILTY SAND)
TILL
SECTION C-C'
(SLOPE SHALLOW-FLOW DUE TO LIQUEFACTION WILL BE MINIMAL)

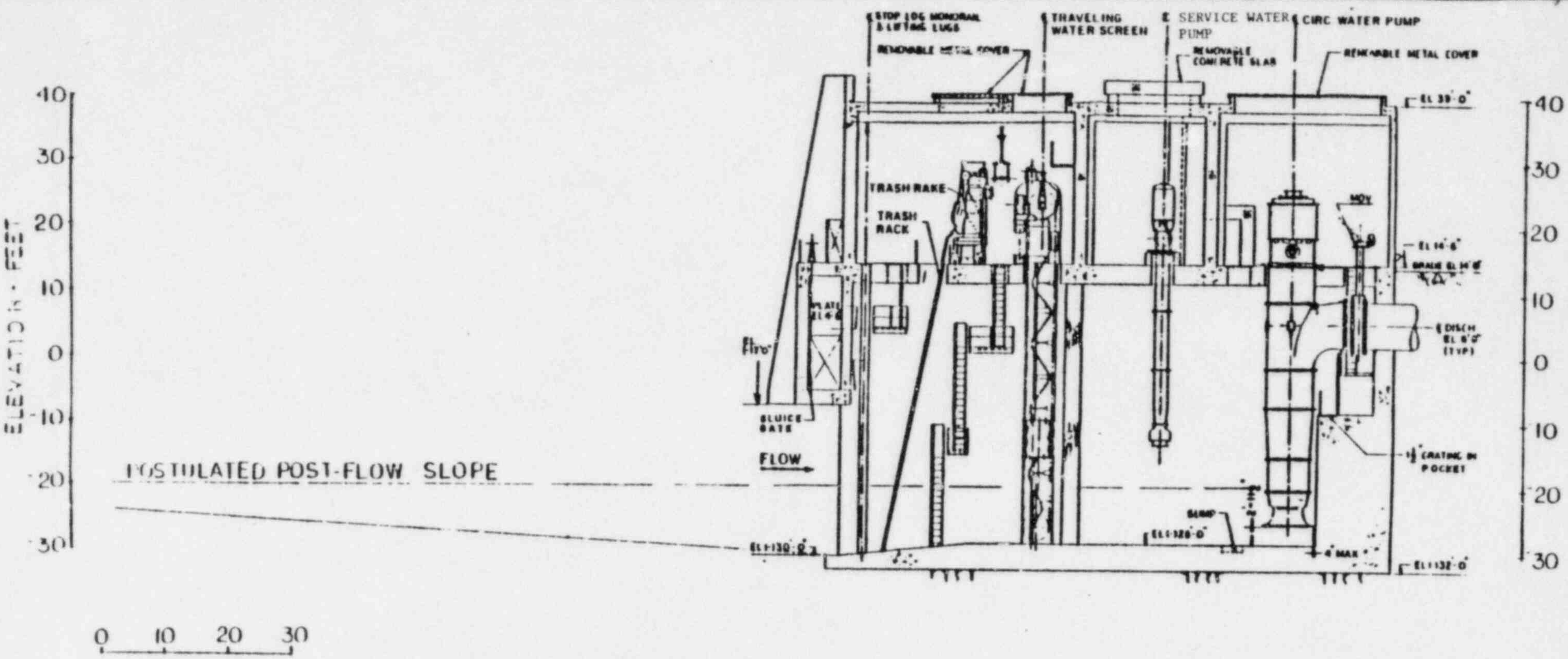
FIGURE 3



SECTION D-D'

FIGURE 4

STONE & WEBSTER ENGINEERING CORPORATION



SECTION E-E'

FIGURE 5

CIRCULATING AND SERVICE WATER PUMPHOUSE

MILLSTONE NUCLEAR POWER STATION UNIT 3

FROM FSAR FIGS. 2.5.4-41 and 3.8-69

Question 241.25a

The staff review of the fragility analysis for the emergency enclosure (EGE) building has produced several concerns. Therefore, provide a sensitivity study taking the following into consideration:

a) As-built foundation support conditions should be used; and specifically, a 2-D model should be used to represent EGE building.

Response

The response to this question was discussed in NNECO's May 14, 1985 presentation to the NRC Staff and it was agreed at that time that this item is resolved. The following is a summary of the information presented to the staff.

In response to concerns raised by the NRC, studies were performed to evaluate the potential affect of structural fill adjacent to the control building and to evaluate what appeared to be structural fill under the center footing of the EGE building.

Finite element soil/structure interaction analysis was performed to evaluate the seismic response of the structure by using the computer code "PLAXLY" as stated in Section 3.7B.2.4 (Reference 5). In this analysis, a 24 ft.-0 in. depth of basal till was considered between the rock and the foundation level. The material properties were as given in Table Q241.3-2.

Concerns were raised by the NRC in the June 13, 1984 meeting about the possible compacted structural fill under the center footing of the EGE building. In response to this meeting an additional sensitivity study was performed by using the computer code "SHAKE" to compare the results of (1) 24 ft.-0 in., basal till, overlying the bedrock and (2) 5 ft.-0 in. structural fill on 19 ft.-0 in. of basal till overlying the bedrock. The results of this study have been submitted to NRC in Reference 7.

The NRC Staff requested that the as-built foundation support conditions be used in determining the response of the EGE building. In this respect, an effort was undertaken to verify and assess the as-built conditions of the EGE foundation excavation. This effort resulted in a revision of the FSAR Figure 2.5.4-54 (Amendment 14) and the density test data in Table 2.5.4-20. The as-built information indicates that there was no over excavation under the center footing.

The foundation excavation drawing shows that the lowest point of excavation is at elevation 7 ft.-9 in. under the center footing. Field concrete pour cards indicate that a concrete mat was placed between the bottom of the footing and the basal till. These as-built conditions have been shown in attached Section J-J, R-R and S-S. By looking at these sections, it can be seen that the north footing is on basal till, the center footing is on concrete fill founded on till and the south footing is on confined structural fill. These conditions reflect the original analysis of basal till up to the footing evaluation, thereby incorporating the as-built conditions.

The NRC Staff raised another concern about independent action of each footing. The applicant believes that the actual configuration of the foundation will not result in independent action. In fact, the sections noted above indicate that all

walls are tied together by a continuous footing and are also tied into the fuel oil tank wall. Furthermore, the slab at elevation 24 ft.-6 in., which ties all EGE walls and the tank vault structure together, makes the structure behave rigidly between elevation 24 ft.-6 in. to elevation 9 ft.-0 in. and all these interfaces have been designed for the proper transfer of the seismic forces.

The gross seismic forces of the structure have been evaluated at the center of mass (CM) and the moments due to the excentricity between the CM to center of resistance (CR) have been included in the design. Thus, the rigid building response precludes any independent action as perceived by the Staff.

The computer code "PLAXLY" used in the original analysis represents a 2-D model. This takes into account the flexibility at each node and has three degrees of freedom representing one horizontal, vertical and rotational response. The structural response includes the effect of coupling action between the horizontal and the vertical directions.

Although it was stated by the staff that the south footing (Figure Number Q241.3-4) appears to be acting independently along with the structure fill, in reality it cannot respond independently due to the rigid box configuration of the structure. It is also noted that the structural fill is confined between the stiff boundaries of control building, the south wall footing and the basal till. In order to maintain the strain compatibilities of the elastic behavior between the boundaries, it forces the confined soil to behave in a manner similar to the behavior of surrounding material. Moreover, since the majority of the structure is founded on basal till, the local fill has no effect on the calculated responses of the structure.

The effect of structural fill adjacent to the control building area was assessed by taking the average values of material properties for the case of 24 ft.-0 in. basal till with and without structural fill. The method of evaluation was as suggested by Professor J. M. Roesset, Reference 1.0. Table Number Q241.3-2 delineates the comparison of material properties with and without structural fill and the fundamental frequency of the soil layers. The frequency with and without the structural fill shows an insignificant change in the values.

In summary, the EGE building will respond as a rigid body. Also, as noted in the attached figures, the average depth of basal till underlaid by bedrock is estimated to be between 12 ft.-0 in. and 14. ft.-0 in., which is substantially less than the 24 ft.-0 in. depth assumed in the original seismic analysis. Thus, the design response of the EGE building, provided in Table 3.78-18 of Reference 5, is very conservative for the following reasons:

- The as-built average depth of basal till is substantially less than the depth assumed in the seismic analysis.
- The orthogonal components of mass accelerations of the structure have been added algebraically rather than utilizing the SRSS method recommended by Regulatory Guide 1.94 to account for 3-D earthquake effect.
- The structure's displacement responses were increased by 10 percent for an additional safety margin.

- o The "PLAXLY" results have been verified by an alternate method which assumes the subgrade is represented by static soil springs developed based on half-space theory. A 3-D stick model is used to represent the structure (Reference 3).

Therefore, by reviewing the as-built conditions of the EGE buildings foundation and its subgrade material, it has been found that the original analysis is very conservative and more than adequate.

References

1. Seismic Design of Nuclear Power Plants, by R. J. Hansen, Editor, M.I.T. Press June 1982; Section on "Fundamentals of Soil Amplification," J. M. Roesset.

2. Introduction to Structural Dynamics, John M. Biggs; McGraw-Hill Book Company.

3. Vertical and Torsional Stiffness of Cylindrical Footings, E. Kausel and R. Uskijima, M.I.T. Department of Civil Engineering, Publication Number R79-6, February 1979.

4. Plane Strain Dynamic Finite Element Analysis of Soil Structure Systems (PLAXLY), by E. Kausel, User's Manual, December 1976.

5. FSAR, Millstone Nuclear Power Station Unit 3, Volume No. 3.

6. Stone & Webster Engineering Corporation Specification No. 2199.142-999 "Placing Concrete and Reinforcing Steel" for the Millstone 3 Project.

7. Northeast Utilities Response to NRC as given in Docket No. 50-423 (B11345) dated October 18, 1984.

Table Q241.3-2

SUMMARY OF THE AVERAGE

CHARACTERISTICS OF THE SOIL BETWEEN THE ROCK

AND THE EGE BUILDING FOOTING

Category	Γ_s (ave)	G (ave)	D (ave)	C_s (ave)	f (n)	% Change
24 ft-0 in. of Basal Till	0.145	16,279.	0.0344	1901.	19.8	-
Basal Till With the Effect of Structural Fill.	0.1448	14,731.	0.0331	1810.	19.67	0.66

NOTE: All values are from Fig. No. Q241.3-2

$$C_s = \sqrt{G/\rho}\ ; \qquad f(n) = \frac{C_s}{4 \cdot H}; \qquad \rho = \frac{\Gamma}{32.2}$$

$$\Gamma_{s\ (ave)} = \frac{\Sigma\ \Gamma j\ \ H_j}{\Sigma\ \ Hj} \quad ; \quad G_{(ave)} = \frac{\Sigma\ Gj\ Aj}{\Sigma\ Aj}$$

; where $Aj = HjBj$
Bj = Length of the Soil Layer

$$C_s\ (ave) = \frac{\Sigma\ C_s\ Hj}{\Sigma Hj} \quad ; \quad D_{(ave)} = \frac{\Sigma\ Dj\ Aj}{\Sigma\ Aj}\ ;$$

H = Height of soil layer

G = Shear modulus (kips/□')

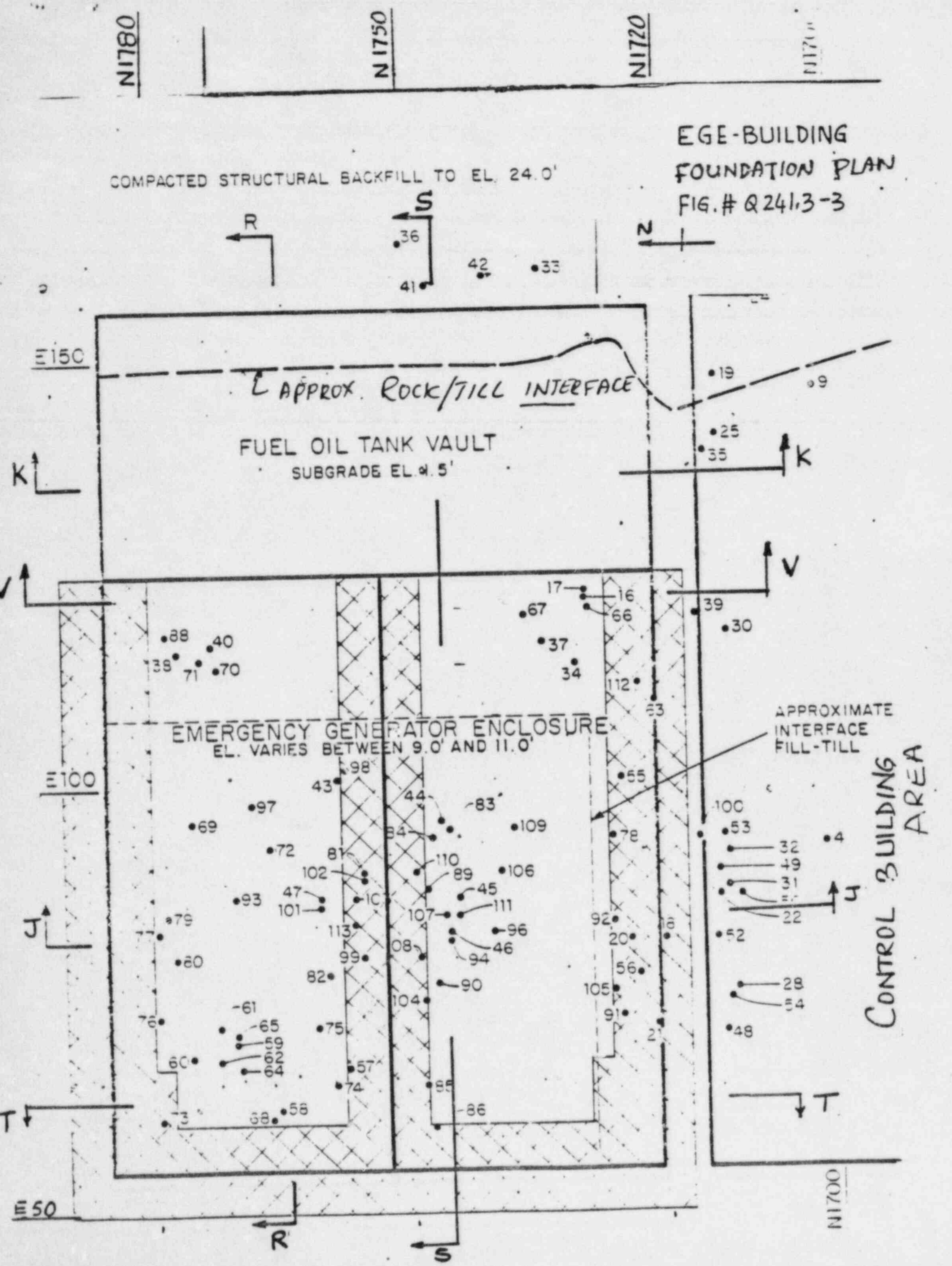
C_s = Shear wave velocity (ft/sec)

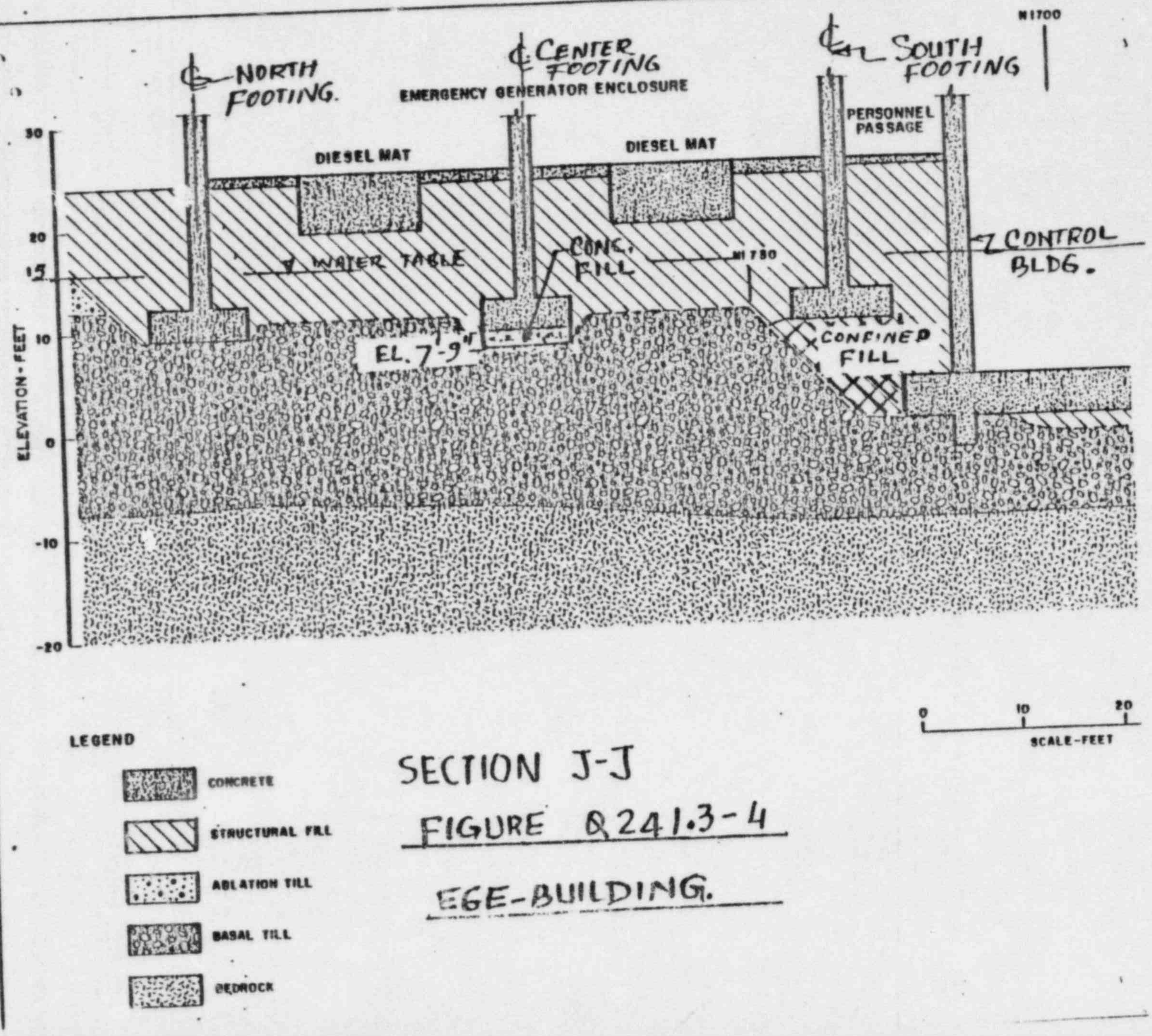
Γ_s = Unit weight of soil (K/CFT)

D = Critical damping

f(n) = Natural frequency of soil layer (cps)

P = Mass density


EGE-BUILDING
FOUNDATION PLAN
FIG. # Q241.3-3
N1780
N1750
N1720
N1700
COMPACTED STRUCTURAL BACKFILL TO EL. 24.0'
APPROX. ROCK/TILL INTERFACE
FUEL OIL TANK VAULT
SUBGRADE EL. 4.5'
EMERGENCY GENERATOR ENCLOSURE
EL. VARIES BETWEEN 9.0' AND 11.0'
APPROXIMATE INTERFACE FILL-TILL
CONTROL BUILDING AREA
E150
E100
E50

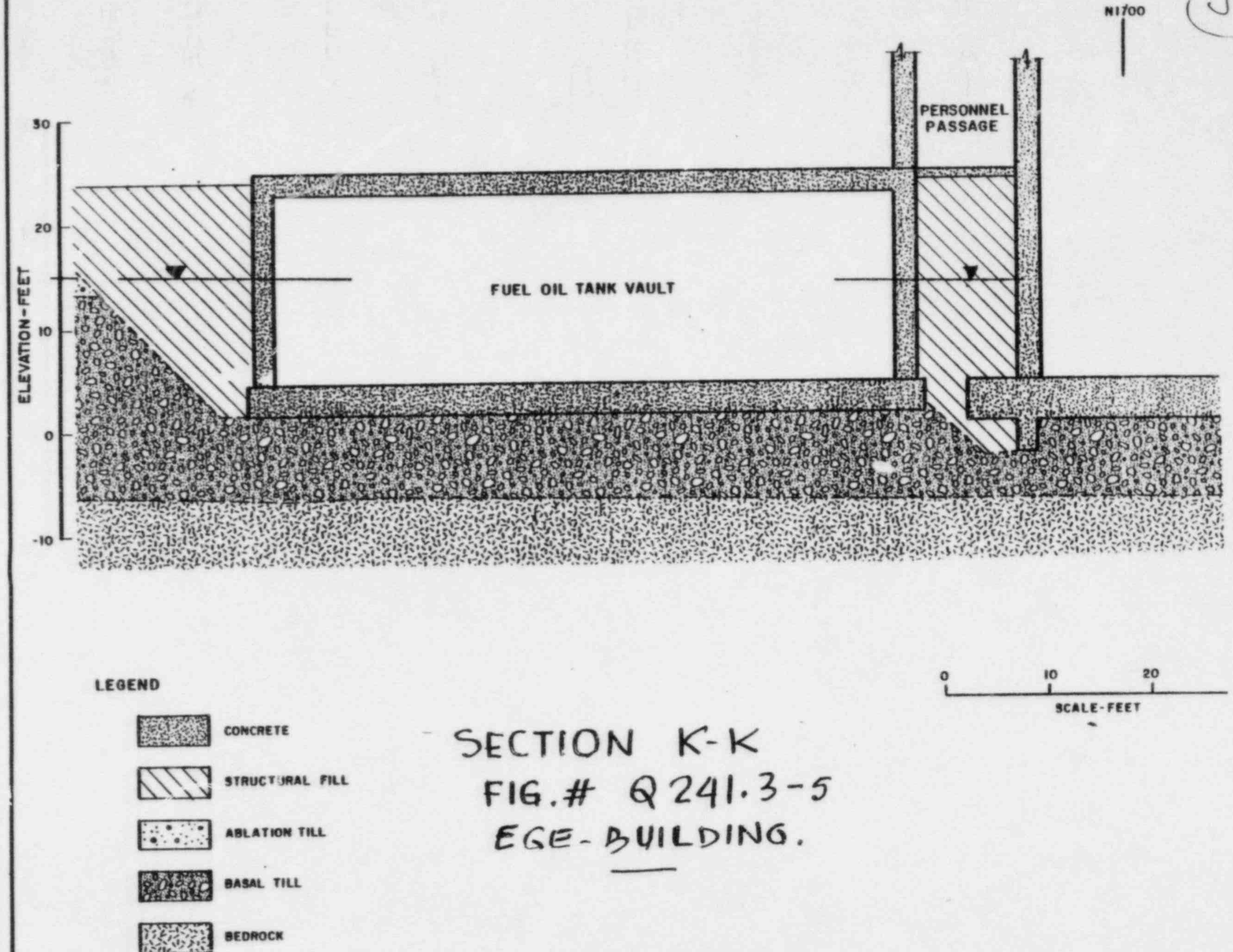


SECTION J-J

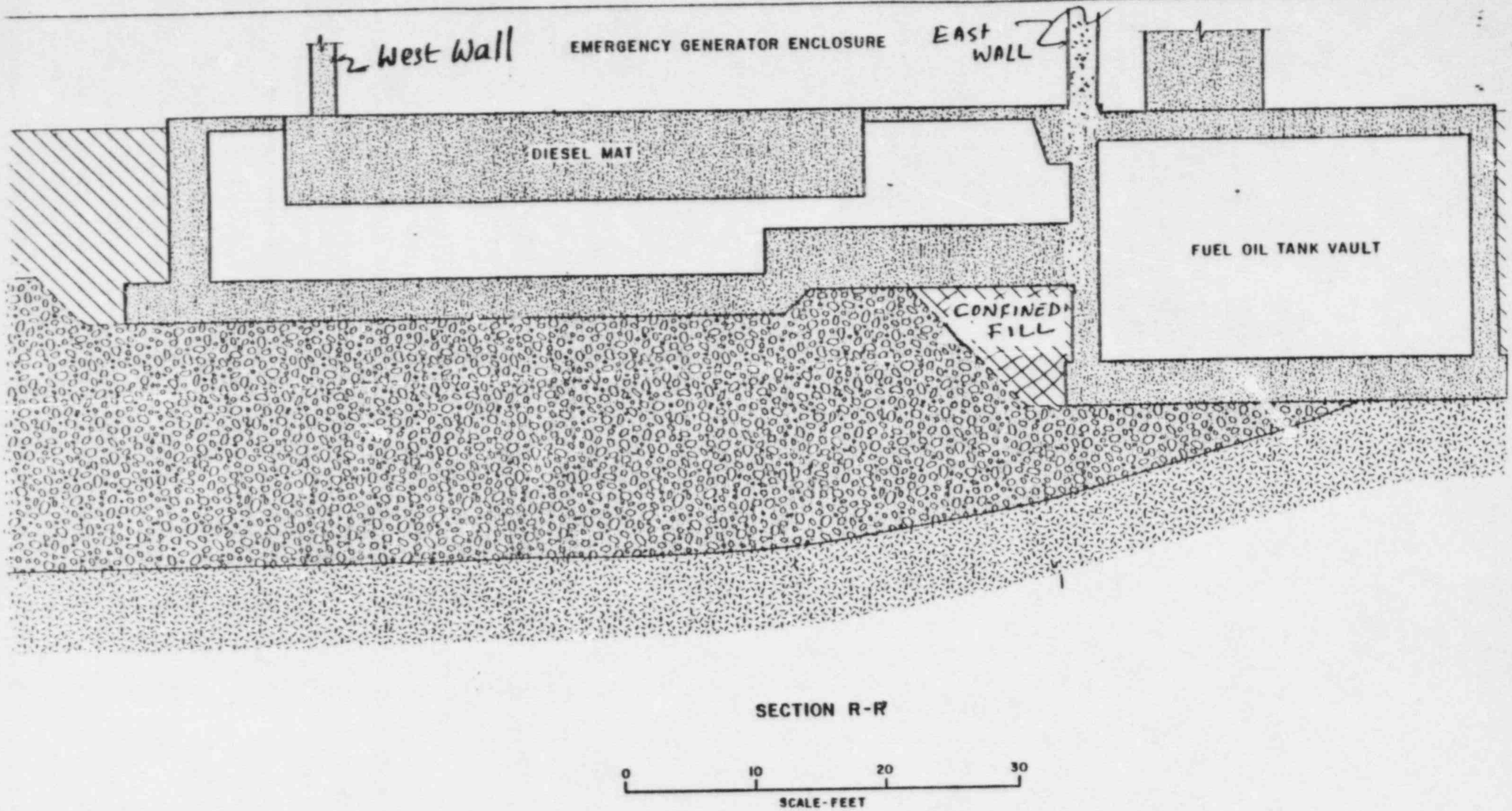
FIGURE Q241.3-4

EGE-BUILDING.

(5)

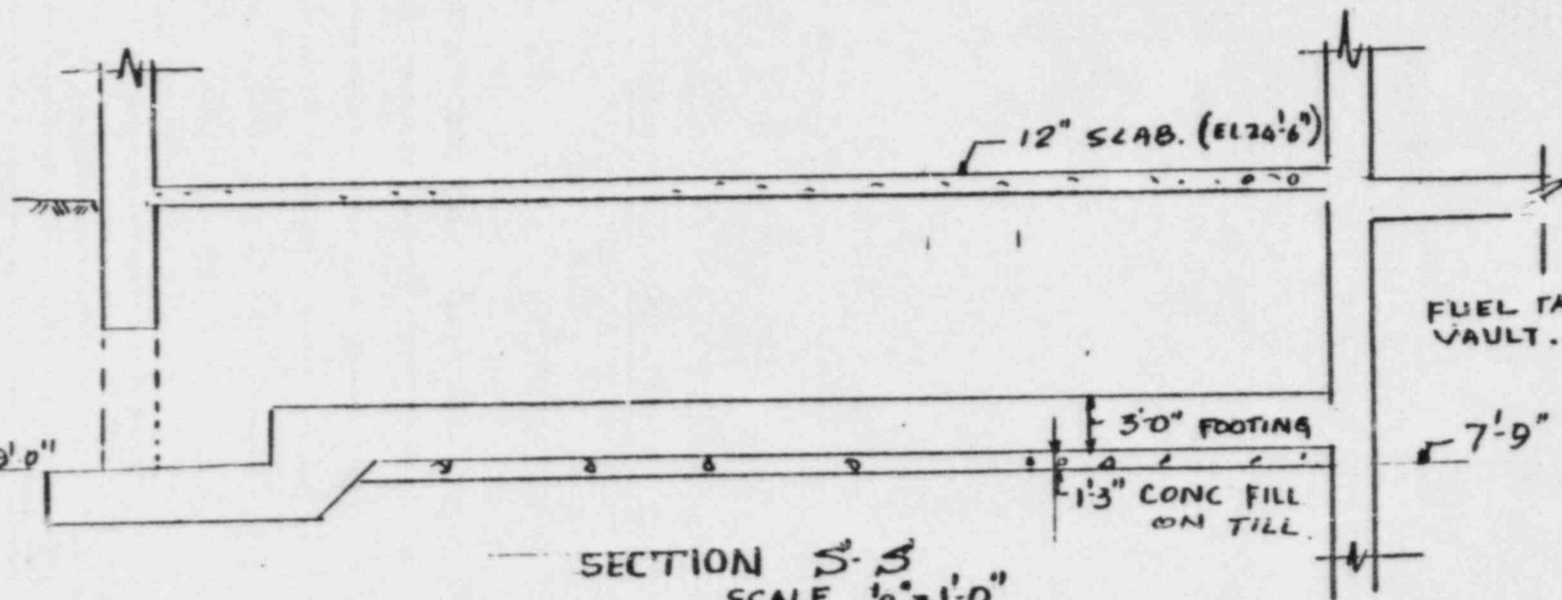


SECTION K-K
FIG.# Q241.3-5
EGE-BUILDING.



N TE
ICTURAL FILL
ATION TILL
AL TILL
ROCK

FIG. Q 241.3-6
EGE - BUILDING.



SECTION S-S
SCALE 1/8" = 1'-0"

FIG. # Q 241.3-7
EGE-BUILDING.
(EXTRACT FROM DWG. 12179-EC-29A-9 & -29F-8)

(8)

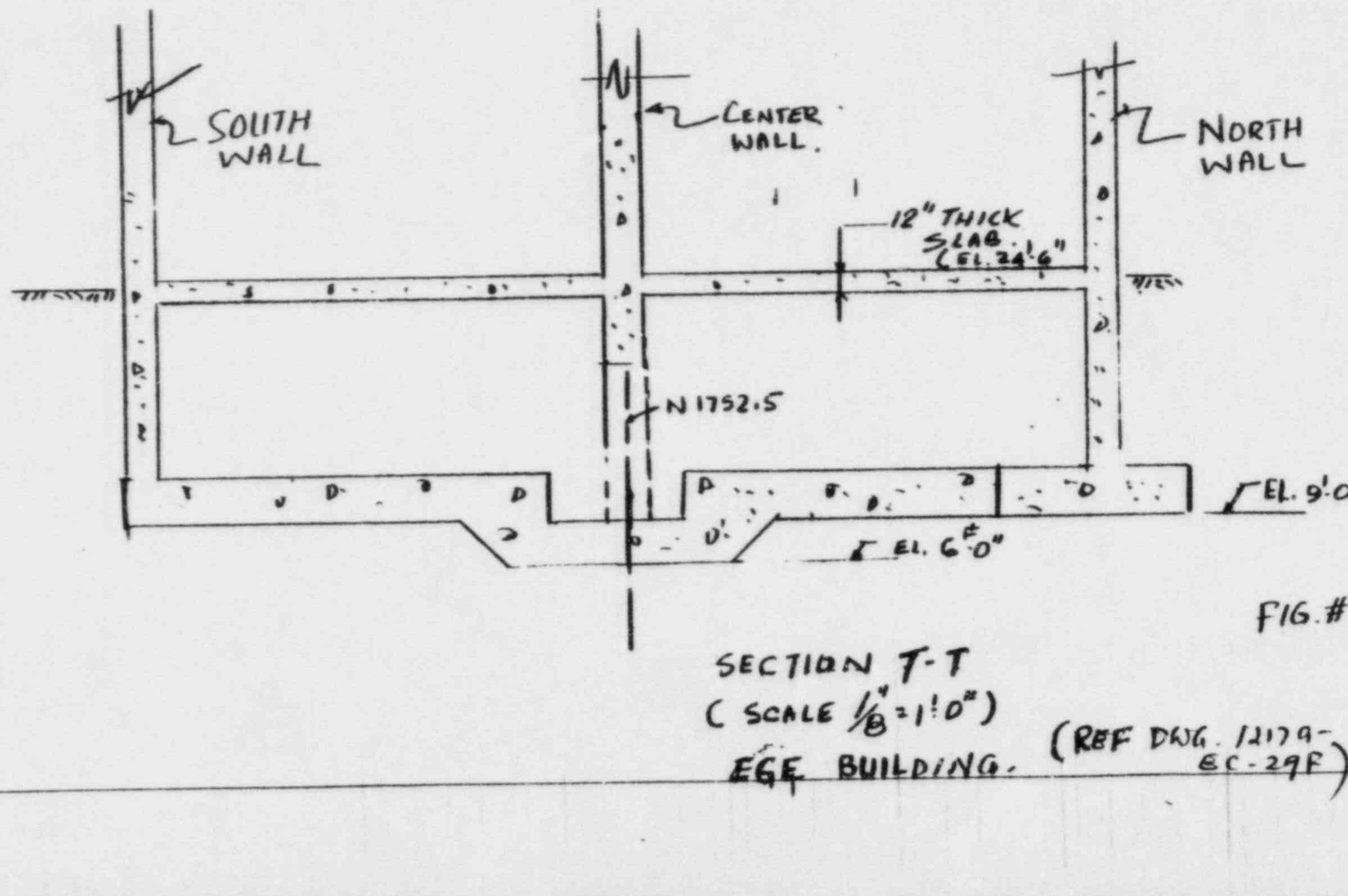
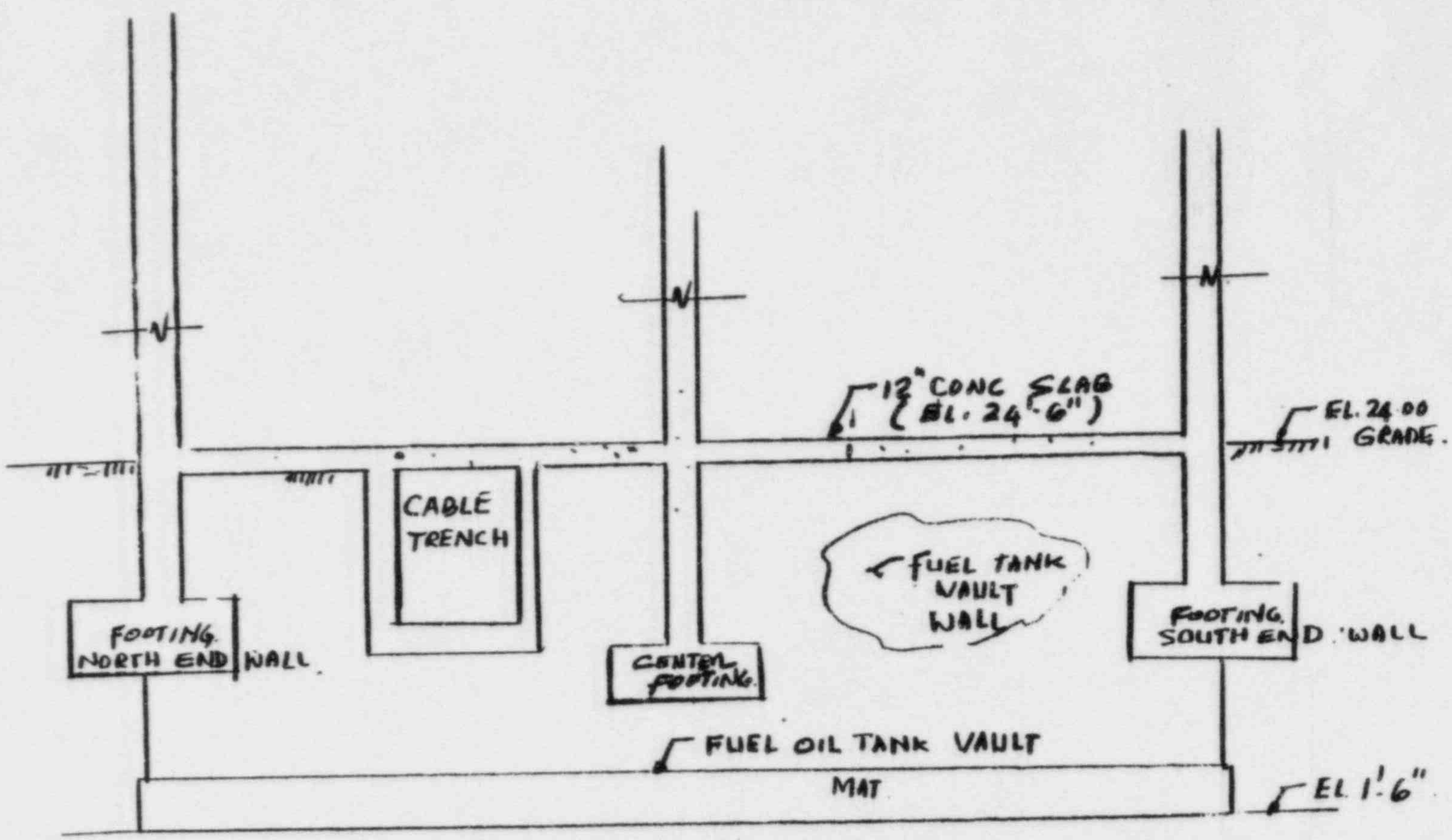


FIG. # Q 241.3-8

SECTION T-T
(SCALE 1/8" = 1'-0")
EGE BUILDING. (REF DWG. 12179-EC-29F)

7	6	5

9



SECTION V-V'
SHOW: FUEL TANK WALL &
EGE EAST-WALLS INTERFACE.

FIG # Q 241.3-9
(REF. DWG 12179-
EC-29J)

Question 241.25b and d

b) Newmark's nonsymmetric formula should be used to calculate the seismic induced movements of the EGE building which is essentially nonsymmetric with respect to sliding.

d) The impact of longer duration associated with larger seismic events should be considered in the study.

Response

Questions 241.25b and d were discussed with the NRC Staff at the May 14, 1985 meeting and it was agreed that these approaches were not appropriate for the EGE building.

Question 241.25c The variation in the assumed ratio of peak ground velocity to peak ground acceleration needs to be considered. For example, 36 inch/sec per peak ground acceleration of 1g is recommended value to be used for soil-supported facilities.

Response: The sliding capacity of the EGE was reevaluated for a ratio of peak ground velocity to peak ground acceleration (v/a ratio) of 36 in/sec/g. Based upon Newmark's symmetrical resistance formula presented in Section 4.1.1.7 of the Millstone 3 fragilities report (Reference 1), the following values were determined:

$\check{A} = 1.1g$

$\beta_R = 0.18$

$\beta_U = 0.45$

The use of Newmark's symmetrical resistance formula is appropriate since the sliding resistance of the EGE is essentially the same in all directions. The median acceleration capacity is reduced slightly from the capacity of 1.3g based upon a v/a ratio of 28 in/sec/g. As noted in the response to NRC Staff Question 720.78 (Reference 2), the use of the v/a ratio of 36 in/sec/g as a median is not appropriate for the fragilities evaluation. The resulting values are therefore considered to be conservative. Nevertheless, sensitivity studies using a v/a ratio of 36 in/sec/g for the EGE and the nonsymmetric resistance sliding capacity for the retaining wall indicate only small variation in the plant damage state capacities as shown in the attached appendix.

References

1. Wesley, D. A., et al, "Seismic Fragilities of Structures and Components at the Millstone 3 Nuclear Power Station", prepared by Structural Mechanics Associates for Northeast Utilities, SMA 20601.01-R1-0, March, 1984.

2. Letter from W. G. Counsil to B. J. Youngblood, Subject: "Millstone Nuclear Power Station Unit 3, Probabilistic Safety Study, dated July 31, 1984.

MILLSTONE UNIT NO. 3

SEISMIC FRAGILITY EVALUATION

EGE BUILDING

SENSITIVITY STUDY IN RESPONSE TO
QUESTION 241.25c

TITLE Millstone 3

BY DSH DATE 5/31/85

CHKD. BY [illegible] DATE 6/6/85

STRUCTURAL MECHANICS ASSOCIATES
A Calif. Corp.

PAGE 1 OF ___ Job No. 2560.0

COMMENTS Q.241.0

EGE Sliding Fragility For v/a = 36 in/sec/g

In response to Q.241.25c, the EGE fragility will be evaluated using a v/a of 36 in/sec/g. Per DAW, Newmark's symmetric resistance formula will be used. See pp. EGE-40 to 66.

$$0 = \frac{(36)^2 A^2}{2(386.4)(0.6662-0.2934A)} - \frac{36^2 A}{2(386.4)} - 4 \qquad \text{p. EGE-62}$$

$$= \frac{1.677A^2}{0.6662-0.2934A} - 1.677A - 4$$

$A = 1.095g$

$\approx 1.1g$

$$F_S = \frac{1.1}{0.17}$$

$= 6.47$

≈ 6.5

Uncertainty due to the v/a ratio will be determined in the same manner as the sensitivity study. See p. SS-18.

$v/a_{+2\sigma} = 45$ in/sec/g

$$0 = \frac{45^2 A^2}{2(386.4)(0.6662-0.2934A)} - \frac{45^2 A}{2(386.4)} - 4$$

$$= \frac{2.620\ A^2}{0.6662-0.2934A} - 2.620A - 4$$

$A_{-2\sigma} = 0.975$

$$\beta_R = -\frac{1}{2}\ln\frac{0.975}{1.1}$$

$= 0.06$

$\beta_U \approx 0.06\left(\frac{2}{3}\right)$

$= 0.04$

All other variabilities should remain about the same

$\breve{A} = 1.1g$ vs. 1.3g

$\beta_R = \sqrt{0.06^2 + 0.17^2 + 0.04^2}$

$= 0.18$ vs. 0.29

$\beta_U = \sqrt{0.42^2 + 0.04^2 + 0.15^2}$

$= 0.45$ vs. 0.46

Question 241.26 For the buried service water piping system, confirm that buckling is the most critical failure mode as compared to shear caused cracking or breakage and to bending caused fracture. Newmark's non-symmetric displacement formula, appropriate velocity - acceleration ratio and seismic duration should be considered in the analysis.

Response: A more detailed evaluation of the service water line was performed. This line typically consists of two pipes lying side-by-side in a concrete encasement. The pipes are typically 30 inches in diameter, 0.313 inches thick, and fabricated from ASTM B466 or B467, No. 706 copper/nickel alloy. The concrete encasement is nominally reinforced by #5 bars at 12 inches in the longitudinal and transverse directions. The depth of burial of the concrete encasement varies along the length of the line.

A number of potential failure modes of the service water line were considered in this analysis. The line was evaluated for its ability to resist both free-field earthquake strains and local loads due to structure displacements. The effects of free-field earthquake strains were considered using the somewhat conservative requirements of ASCE Standard Provision Section 3610. In accordance with these provisions, the free-field strains were derived from the apparent seismic wave speeds associated with the bedrock rather than the shallow overburden or backfill. Based upon the recommendations of the commentary to Section 3610, half of the peak ground velocity was estimated to result from shear waves and the other half from Rayleigh waves. These free-field ground strains

introduce axial and shear stresses in the service water line. Bending stresses are typically very small and were neglected. The service water line was checked for the maximum possible tension, compression, and shear forces. The resistance against axial compression was conservatively based upon the capacity of the concrete encasement alone. The resistances against axial tension and shear were conservatively based upon the service water pipes alone without the additional benefit of the concrete encasement strength. The service water line capacities against these potential failure modes were found to be very large (about 3g or greater).

Local stresses are introduced into buried piping by structure displacements. For the service water line, the greatest structure displacement-induced stresses are expected to result from sliding of the retaining wall adjacent to the pumphouse. The service water line is buried in the soil behind this retaining wall. When a portion of the retaining wall slides, the service water line is constrained to displace along with the wall by the soil backfill behind the wall. Restraint against lateral displacement imposed by adjacent soil or structures causes local bending and shear forces to be developed in the service water line.

The forces introduced into the service water line by sliding of the retaining wall were determined using essentially the same approach as was used for evaluating typical buried piping at structure penetrations. This approach is described in the response to Question 720.80 of Reference 1. Appropriate modifications to this approach were incorporated to account for the specific conditions of the service water line. The sliding capacity of the wall was developed using the unsymmetric resistance approach described in the response to Question 241.23.

The displacement capacity of the service water line is expected to be controlled by buckling. In the initial stages of sliding, much of the resistance is provided by the concrete encasement. However, as the pipes approach buckling, the concrete encasement is expected to be significantly cracked. Consequently, the line stiffness was based upon that of the pipes alone. Similarly, the line capacity was determined by conservatively neglecting the encasement resistance. The pipe buckling moment was determined in a manner similar to that used for typical buried piping as described in the response to Question 720.80 of Reference 1.

Using this approach, it was determined that a total displacement of four inches is necessary to cause buckling of the service water pipes. Shear failure of the pipes was checked and found to be non-controlling. At the displacement corresponding to buckling, the maximum shear stress in the pipe is still less than the yield stress in shear by a factor of about 1.2 so that buckling will control. The four inches of allowable displacement was the value used in calculating the fragility for sliding-induced failure of the retaining wall.

This capacity is expected to be somewhat conservative since the v/a ratios and earthquake durations used in Reference 2 are higher than expected for the Millstone site. Also, the buckling capacity of the pipes was found neglecting any restraint against buckling from the concrete encasement. In addition, the displacement capacity used corresponds to the initiation of pipe buckling whereas additional displacement is probably necessary to cause fracture or significant

flow blockage. Even if fracture occurs, the encasement, although cracked, will prevent significant leakage. Furthermore, the pipe loads were based upon beam on elastic foundation solutions. It is possible that the elastically calculated soil loads may exceed values that the backfill can physically transmit. Should this be the case, the resulting loads on the pipe would be conservative.

References

1. Letter from W. G. Counsil to B. J. Youngblood, "Millstone Nuclear Power Station Unit 3, Probabilistic Safety Study", dated July 31, 1984.

3. Newmark, N. M., "The Effects of Earthquakes on Dams and Embankments", Geotechnique, Vol. 15, No. 2, 1965.

Appendix To

Questions 241.23 and 241.25c

Questions 241.23 and 241.25c

Appendix

Sensitivity Study

To account for the modified fragilities presented in the responses to Questions 241.23 and 241.25c, an additional study to determine the sensitivity of the plant damage state fragilities and occurrence frequencies to these modified fragilities was performed. These modifications were implemented at the NRC request and are quite conservative in several instances. There are three important plant damage states which are potentially affected by the two modified component fragilities, TE - transient (loss of offsite power) with early core melt, SE - small LOCA or ATWS with early core melt, and AE - large LOCA with early core melt. The primary effect of the retaining wall sliding failure is to cause loss of service water and, therefore, loss of emergency power. The EGE sliding failure is taken to directly cause loss of emergency power. Plant damage state fragility curves and annual seismically-induced occurrence frequencies were recalculated using the modified component frequencies. The results are shown in Tables 1 and 2 with the original values (Reference 1) also shown for comparison. As can be seen, the differences are minor. The high confidence, low frequency of failure level for all damage states remains above 0.25g and the median occurrence frequencies remain below $2x10^{-6}$ per year.

References

1. Ravindra, M. K., R. H. Sues, R. P. Kennedy, and D. A. Wesley, "A Program to Determine the Capability of the Millstone 3 Nuclear Power Plant to Withstand Seismic Excitation Above the Design SSE", NTS/SMA 20601.02-R?, November, 1984.

TABLE 1

SEISMICALLY-INDUCED ANNUAL PLANT DAMAGE STATE OCCURRENCE FREQUENCIES USING ORIGINAL AND MODIFIED* COMPONENT FRAGILITIES

Plant Damage State	Annual Frequency		
	Median	Mean	5% - 95% Confidence Bounds
TE - Original	2×10^{-6}	6×10^{-6}	2×10^{-8} - 2×10^{-5}
Modified	2×10^{-6}	7×10^{-6}	5×10^{-8} - 3×10^{-5}
SE - Original	4×10^{-7}	2×10^{-6}	2×10^{-9} - 8×10^{-6}
Modified	4×10^{-7}	2×10^{-6}	3×10^{-9} - 9×10^{-6}
AE - Original	8×10^{-8}	7×10^{-7}	1×10^{-10} - 3×10^{-6}
Modified	9×10^{-8}	8×10^{-7}	4×10^{-10} - 4×10^{-6}

* EGE sliding fragility changed to $\check{A} = 1.1g$, $\beta_R = 0.18$, $\beta_U = 0.45$

Retaining wall sliding fragility changed to $\check{A} = 0.90g$, $\beta_R = 0.17$, $\beta_U = 0.45$

TABLE 2

PLANT DAMAGE STATE FRAGILITIES USING ORIGINAL AND MODIFIED* COMPONENT FRAGILITIES

Plant Damage State	$\check{A}$ (g's)	5%-95% Confidence Bounds on Median (g's)	High Confidence, Low Frequency of Failure Levels (g's)
TE Transient (loss of offsite power) with Early Core Melt			
Original	0.61	0.39-0.84	0.26
Modified	0.56	0.37-0.77	0.26
SE Small LOCA or ATWS with Early Core Melt			
Original	0.77	0.58-1.04	0.40
Modified	0.76	0.57-1.03	0.39
AE Large LOCA with Early Core Melt			
Original	1.22	0.75-1.91	0.45
Modified	1.21	0.73-1.91	0.44

* EGE sliding fragility changed to $\check{A}$ = 1.1g, β_R = 0.18, β_U = 0.45

Retaining wall sliding fragility changed to $\check{A}$ = 0.90g, β_R = 0.17, β_U = 0.45