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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October 18, 1984

Docket No. 50-423
B11345

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) W. G. Counsil to B. J. Youngblood, Millstone Nuclear Power Station, Unit No. 3, Technical Review Meeting Summary, Geotechnical Confirmatory Items, dated June 26, 1984.

(2) B. J. Youngblood to W. G. Counsil, Millstone Nuclear Power Station, Unit No. 3, Safety Evaluation Report Related to the Millstone Nuclear Power Station, Unit No. 3, Docket No. 50-423 (NUREG-1031), dated August 2, 1984.

Dear Mr. Youngblood:

Millstone Nuclear Power Station, Unit No. 3
Transmittal of Responses to Safety
Evaluation Report (SER) Confirmatory Items

Enclosed are Northeast Nuclear Energy Company's responses to SER Confirmatory Items 2, 3, and 4 (Reference 2). Confirmatory Items 2, 3, and 4 correspond respectively to Items I, II and III discussed with your Mr. John Chen, Structural and Geotechnical Engineering Branch, on June 13, 1984 (Reference 1). Revisions to the FSAR are provided as they will appear in Amendment 10 to the FSAR, which is scheduled for submittal in October, 1984.

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These responses should fully resolve the Staff's concerns regarding SER Confirmatory Items 2, 3, and 4. If there are any questions related to the information contained herein, please contact our licensing representative, Ms. C. J. Shaffer, at (203) 665-3285.

Very truly yours,

NORTHEAST NUCLEAR ENERGY COMPANY
et. al.

BY NORTHEAST NUCLEAR ENERGY COMPANY
Their Agent

W. G. Counsil

W. G. Counsil
Senior Vice President

W. F. Fee

By: W. F. Fee
Executive Vice President

cc: Mr. John Chen - Structural and Geotechnical Engineering Branch

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

Then personally appeared before me W. F. Fee who being duly sworn, did state that he is Executive 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

ITEM I (From June 13 meeting with NRC)

Soil Structure Interaction (Question 241.3)

B. At the request of the NRC Staff the center footing of the EGE will be assumed to be founded on structural fill to elevation 3.5 ft. As a sensitivity study, the effect of this assumption will be examined to verify that the calculated soil amplification will not be affected.

RESPONSE

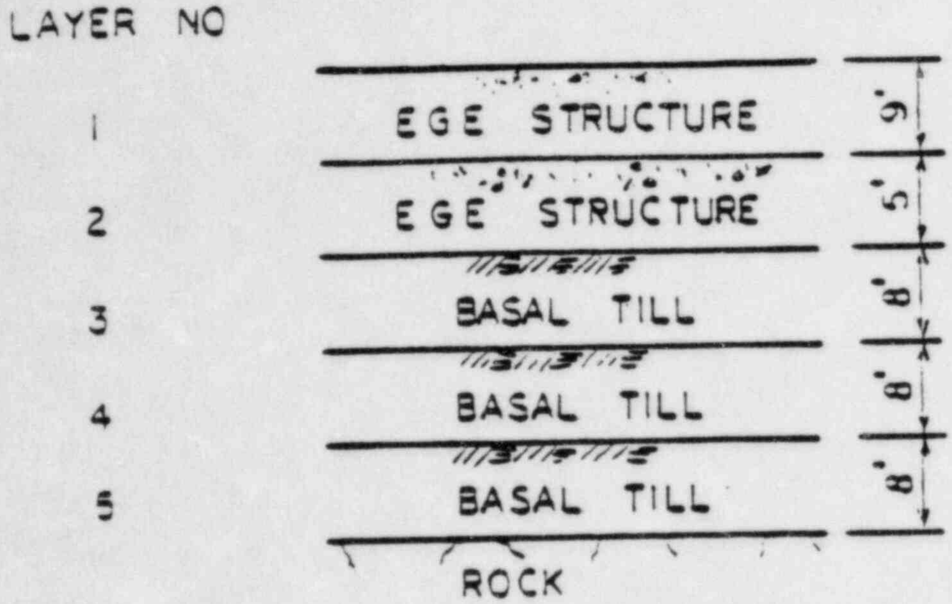
The Emergency Generator Enclosure (EGE) was designed on the basis that the structure footings are founded on 24 ft of dense basal till overlying bedrock. Concerns raised by the NRC (Reference Q241.3) resulted in additional studies on the effects of up to 5 ft of compacted structural fill underlying the footings. This condition does not represent the as-built conditions under the structure, but a comparative analysis was performed and is described below. As-built conditions have a limited depth, averaging less than 5 ft of compacted structural fill used to support footings of the EGE, where they occur adjacent to other structures, and in other local areas where the till was disturbed by construction activity.

The response of the EGE was calculated using the computer program SHAKE, modeling the structure as an equivalent soil, for the case with 24 ft of basal till, and for 19 ft of basal till overlain by 5 ft of structural fill. The comparison showed that the maximum response of the EGE is more limiting when founded on basal till. This sensitivity study shows the EGE was conservatively designed and therefore, Northeast Utilities considers this confirmatory issue as identified in Section 2.5.4.3.2 of the Millstone Unit No. 3 Safety Evaluation Report to be resolved.

The responses from the 'SHAKE' analysis for the two foundation conditions are listed in Table Q241.3-1.

Figure Q241.3-1 shows the models used in the 'SHAKE' analyses.

MODEL FOR EGE
ON 24 FT BASAL TILL



MODEL FOR EGE
ON 5 FT STRUCTURAL FILL & 19 FT BASAL TILL

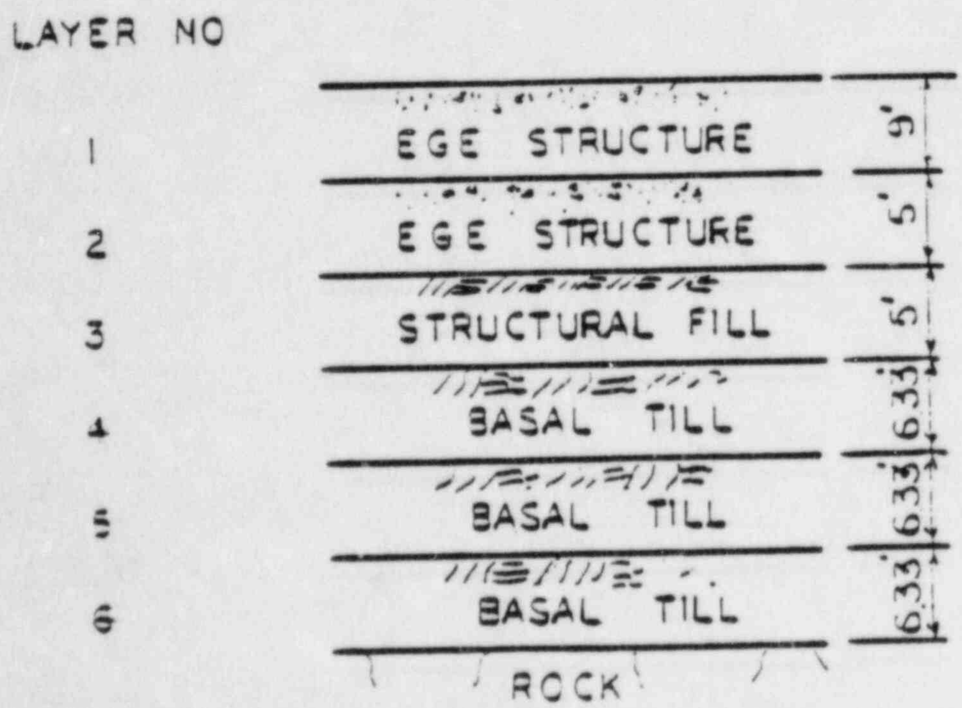


FIGURE Q241.3-1

TABLE Q241.3-1

EGE FOUNDED ON 24' BASAL TILL

Sublayer	Description	Maximum Acceleration (g)
1	EGE Structure	.453
2	EGE Structure	.384
3	Basal Till	.238
4	Basal Till	.208
5	Basal Till	.184

EGE FOUNDED ON 19' BASAL TILL AND 5' STRUCTURAL FILL

Sublayer	Description	Maximum Acceleration (g)
1	EGE Structure	.427
2	EGE Structure	.321
3	Structural Fill	.260
4	Basal Till	.190
5	Basal Till	.183
6	Basal Till	.172

ITEM II (From June 13 meeting with NRC)

West Retaining Wall (Question 241.18)

A. For the existing analysis the following will be provided:

1. The shear capacity and shear stress at the base of the wall.

2. The coefficient of friction between concrete and rock.

3. The factor of safety against overturning and sliding.

B. A sensitivity analysis based on the following will be performed:

1. K_o = 0.7 for soil from the bottom of the wall to the top of counterforts (bottom of wall footing at el -25 ft-0 in. actual condition).

2. $K_o = K_a$ for soil from top of counterforts to grade.

3. Neglecting hydrostatic pressure (both sides).

C. From the above analysis the following information will be provided:

1. The shear capacity and shear stress at the base of the wall.

2. The coefficient of friction used between concrete and rock.

3. The factor of safety against overturning and sliding.

RESPONSE

For the original stability analysis of the west retaining wall, lateral loads were based on an assumed bottom of footing el -30 ft-0 in. Vertical loads were based on a backfill height of 39 ft. In the original retaining wall design, all shear at the base of the wall was assumed to be resisted by the counterforts. Results of the original analysis are provided below:

Shear force at base of counterfort	1474 kip
Shear capacity at base of counterfort	1656 kip
Factor of safety against overturning	1.1
Factor of safety against sliding	1.1
Coefficient of friction	0.7

A sensitivity analysis has been performed on the as-built condition using the requested parameters. Results of the new analysis are provided below:

Shear force at base of wall between counterforts	8.4 kip/ft
Shear capacity at base of wall between counterforts	78 kip/ft
Shear force at base of counterfort	974 kip
Shear capacity at base of counterfort	1656 kip
Factor of safety against overturning	2.91

Factor of safety against sliding	1.29 (neglecting shear key and passive pressure)
Coefficient of friction	0.7

Using a conservative value for the coefficient of friction assumed at the rock and concrete interface of sliding, the retaining wall is stable under the requested sensitivity study analysis, which further supports the design basis analysis.

ITEM III (from June 13 meeting with NRC)

Liquefaction Analysis of Sloping Shorefront (Question 241.7)

A two-dimensional dynamic analysis was performed and submitted to NRC as requested in Q241.8 to confirm the stability of the beach sand deposits. This analysis concludes that liquefaction of the shorefront slopes will not occur and that liquefaction of the intake channel bottom would not affect the integrity of the shorefront slopes adjacent to the circulating and service water pumphouse.

NRC has requested that NUSCO demonstrate that even if liquefaction of the shorefront slopes occurs that this would not impact safe shutdown. NUSCO has committed to provide a sensitivity study demonstrating that even if liquefaction occurred the flow of water into the service water inlet would not be restricted and would not result in a condition that would make the service water system inoperable.

RESPONSE:

See revised FSAR Section 2.5.4.8.3.3.

The profile used in the analysis is shown on Figure 2.5.4-75.

Liquefaction potential was calculated at each element for the six sections shown on this figure. The results of the PLAXLY analysis and the calculated values of safety factor against liquefaction are presented in Table 2.5.4-24.

The blowcount data used in Sections 1 and 5 were obtained from onshore borings in the shorefront area. The blowcount data from boring I21 was used to represent soil conditions in Section 6 because the borings indicate that the sands offshore are denser than the onshore sands. The dynamic shear strength of the sand was calculated by determining the corrected blowcount (N1) in accordance with methods established by Gibbs and Holtz (1957), in which the corrected blowcount data are corrected for an effective overburden stress of 1 tsf. The N1 values are plotted with vertical effective stress on Figures 2.5.4-28 and 2.5.4-29. The mean value of N1 was calculated from these data and used to determine the cyclic stress ratio to resist initial liquefaction from the Seed, et al, (1975) curve presented on Figure 2.5.4-48. The curve for Magnitude 6 earthquakes was used to obtain a nonliquefaction cyclic stress ratio of .27, which was used in the analyses performed on Sections 1 to 5. For Section 6, a mean N1 value of 28 was calculated and a stress ratio of .42 was used in the liquefaction analysis. 241.7

The earthquake-induced shear stresses were computed by averaging the peak shear stress values obtained for each of the four earthquakes at each element in the PLAXLY model. The effective shear stress was obtained by multiplying the average of the four peak values by a factor of two-thirds. Seed and Idriss (1971) recommend multiplying the absolute maximum shear stress value by a factor of .65 to obtain the equivalent uniform cyclic shear stress. This value was compared with the dynamic shear strength of the soil at each element to obtain the safety factor against liquefaction.

The results of the analyses, presented in Table 2.5.4-24, indicate that the safety factor for elements 1 to 5 are all greater than 1.25. Low safety factors were determined for Section 6, mainly because of the low vertical effective stress near the surface of the intake channel at elevation -29 feet. The effective stress increases to the west of this profile location as the side-slopes of the intake channel rise to meet the natural ocean bottom, making these low safety factors a local phenomenon limited to the intake channel only. The post-earthquake slope stability analysis presented in Section 2.5.5.2.1 was reanalyzed to consider the effect of liquefaction of the sand in the intake channel (Soil 7 on Figure 2.5.5-4) on stability of the shorefront slopes. No change in the safety factor of the critical failure circle was calculated, indicating that the shorefront slopes would not fail in the event that the sand in the intake channel would liquefy.

It can be concluded from these analyses that liquefaction of the shorefront slopes will not occur and that liquefaction of the intake channel bottom would not affect the integrity of the shorefront slopes adjacent to the circulating and service water pumphouse or result in a condition that would make the service water system inoperable. The soil underlying the service water pipe encasement adjacent to the pumphouse is not susceptible to liquefaction.

Conservatively postulating that liquefaction could occur during the site SSE, a study was made to determine whether sliding of the slope into the intake channel would cause blockage of the service water intake pumps. Data from slides caused by liquefaction during the Alaskan Earthquake of 1964, (Seed, 1968) indicate that flow slides maintain a slope steeper than 5 percent. Assuming that the saturated
241.7 sand overlying basal till adjacent to the pumphouse liquefies and flows toward the intake channel, with a final slope of 5 percent, then it can be shown that 7 feet of water remains available for suction below the pumps. Therefore, it can be concluded that even in the highly unlikely event that liquefaction of the glacial outwash sands were to occur, the plant would have an adequate supply of water available for cooling of safety-related systems.

2.5.4.8.4 Ablation Till

The circulating water discharge tunnel extends 1,700 feet from the main plant area to the Millstone quarry east of Millstone 1. For approximately 1,200 feet, the tunnel is founded on bedrock. However, in the vicinity of the ventilation stack north of Millstone 1, bedrock drops sharply to a trough. The maximum thickness of the overburden in this trough is approximately 60 feet. Borings 402 through 412 were drilled in this area to determine the subsurface conditions. A cross-section of the trough along the discharge tunnel
241.11 is presented on Figure 2.5.4-51. The location of the section is shown on Figure 2.5.4-31. In this area, which extends for approximately 500 feet, the fill and alluvium overlying the ablation and basal tills were excavated and replaced with crushed stone and concrete fill to the base elevation of the discharge tunnel. Because the ablation till is a sandy material below the groundwater table, the liquefaction potential was analyzed. The analysis described in Section 2.5.4.8.4.1 shows that liquefaction of the ablation till is not possible under the site SSE. The structural fill and basal till have been shown to be nonliquefiable in Sections 2.5.4.8.1 and 2.5.4.8.2, respectively.

2.5.4.8.4.1 Dynamic Response Analysis of Ablation Till

The dynamic response of the ablation till has been evaluated to determine earthquake-induced shear stresses caused by ground motions applied at the bedrock surface and amplified through the soil profile. This evaluation was made using the computer program SHAKE, similar to the analysis in Section 2.5.4.8.3.1.

A horizontally stratified idealized soil profile was selected to model the subsurface conditions input into the SHAKE analysis for the
241.12 discharge tunnel. This profile was based on soil strata encountered in boring 411, which encountered the deepest rock, and represents the most conservative profile in the study area. The generalized soil

profile (Figure 2.5.4-50) used in the analysis of the tunnel consisted of 5 feet of structural fill, 13 feet of ablation till, and 22 feet of basal till. Groundwater level was established at 10 feet below the ground surface, elevation +4 feet, based upon the average groundwater levels measured in borings 407 and 411. (See Figure 2.5.4-31 for locations). The shear moduli values of the soils were obtained from cross-hole tests described in Section 2.5.4.4.3. The values of shear modulus (G) and damping (D) for low strain levels used in the SHAKE analysis for each layer are: 241.12

Layer	Elevation (ft)	Depth (ft)	Soil Type	G_{max} (ksf)	D_{max} (%)
1	+14 to -8	0-22	Discharge Tunnel	-	0.5
2	-8 to -13	22-27	Structural Fill	1.93×10^3	0.5
3	-13 to -26	27-40	Ablation Till	1.30×10^3	0.5
4	-26 to -48	40-62	Basal Till	2.0×10^4	0.5

The reduction of G_{max} with strain was performed through a series of iterations similar to the method described in Section 2.5.4.8.3.1 using the same earthquake records normalized to 0.17g.