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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May 15, 1984

Docket No. 50-423
B11192

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

Refernces: (1) B. J. Youngblood to W. G. Counsil, Draft SER for Millstone Nuclear Power Station, Unit 3, dated December 20, 1983.

(2) B. J. Youngblood to W. G. Counsil, Request for Additional Information for Millstone Nuclear Power Station, Unit 3, dated January 16, 1984.

Dear Mr. Youngblood:

Millstone Nuclear Power Station, Unit 3
Transmittal of Response to Request for
Additional Information and Draft
SER Open Items
Structural and Geotechnical Engineering Branch (SGEB)

Section 2.5.4.4 of the Draft SER for Millstone Nuclear Power Station, Unit 3 (Reference 1) and Question 241.8 (Reference 2) request that a two-dimensional dynamic analysis be performed to confirm the stability of the beach sand deposits. The attached response (FSAR Section 2.5.4.8.3.3) provides a discussion of the results of this anlaysis and 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 or result in a condition that would make the service water system inoperable.

The revised FSAR Section 2.5.4.8.3.3 is provided as it will appear in Amendment 9 to the FSAR.

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If you have any concerns related to the information contained herein or any questions related to our response, 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

cc: Ms. E. L. Doolitle - NRC Project Manager
Mr. John Chen - SGEB
Mr. Jeff Kimball - GSB

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

Then personally appeared before me W. G. Counsil, 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.

Marjorie J. Bolles
Notary Public

My Commission Expires March 31, 1988

NRC Letter: May 3, 1983

Question No. Q241.7 (Section 2.5.4.8.3 and SRP Section 2.5.4)

Dynamic Response Analysis of Beach Sand

Based on FSAR Figure 2.5.4-35, the thickness of beach sands varies from a few feet to about 50 feet and the thickness of basal till is also variable. In view of this variation of profile, justify that the one dimensional computer program SHAKE is suitable for analyzing the dynamic response of the shorefront sand deposits. Identify the location of the idealized profile used in the analyses. Substantiate the assumption that the groundwater is 10 feet below the ground surface using field monitoring results.

Provide the bases for not using artificial time history conforming to R.G. 1.60 design response spectra in your dynamic response analyses of beach sands.

Response:

The stratigraphic profiles shown in FSAR Figure 2.5.4-35 show the extent and thickness of the beach and outwash sands, which are found in the vicinity of the circulating and service water pumphouse. The idealized profile described in Section 2.5.4.8.3.1, which consists of 40 feet of sand overlying 4 feet of basal till and bedrock, was developed from information from this figure. The criteria used to select the idealized profile were first, that it would lead to a representative but conservative estimate of earthquake-induced shear stresses and second, that it is appropriate for the area where liquefaction effects would be most serious. The idealized profile most closely models the areas south and west of the structure.

The computer program SHAKE was used to analyze the dynamic response of the beach and glacial outwash sands. SHAKE analyses performed using profiles containing 20, 30, and 40 feet of sand confirm that the profile described in FSAR Section 2.5.4.8.3.1 was conservative because it produced the highest shear stresses.

The groundwater level assumed in the liquefaction analyses was based on records of groundwater observations presented in FSAR Figures 2.5.4-37 and 2.5.4-38. Information which supplements this data includes January 1972 readings from Boring 316, located approximately 250 feet north of the pumphouse. Water elevations in the borehole ranged from 6.4 to 8.0 feet (msl). Also, visual observations of seepage at the land side face of the excavation for the circulating and service water pumphouse revealed that seepage emerged at an elevation approximately at sea level.

The ground surface on shore in the area underlain by the 40 feet thickness of outwash and beach sand is at elevation 14 feet. The contours shown in Figure 2.5.4-37 indicate that groundwater levels in this area range from less than elevation +6 to elevation 0 msl. The

water depth of 10 feet assigned to the idealized profile corresponds to elevation 4 feet. This groundwater level is appropriate.

An artificial time history was not used in the liquefaction analyses for the beach and glacial outwash sands. Rather, actual strong motion records from three rock sites were used with the program SHAKE to compute the dynamic response of the soil. Artificial time histories can be inappropriate for liquefaction analyses because they often include inappropriate frequency and energy content, which results in an overestimation of soil moduli reduction with increasing strain levels.

The effect of sloping bedrock and ground surface was incorporated into the dynamic response analysis of beach area outwash sands and is discussed in new FSAR Section 2.5.4.8.3.3.

An additional method of assessing liquefaction potential can be developed by comparing standard penetration resistance data from the vicinity of the pumphouse structure with standard penetration resistance data from sites which have been subjected to earthquakes. This method, described in detail below, also indicates there is no danger of liquefaction in the beach sands at the site.

An empirical approach relating standard penetration resistance data (N values) to liquefaction potential was proposed by Seed, Arango, and Chan (1975), who presented cyclic strengths based on empirical data from sites which did and did not experience liquefaction during earthquakes. Also included were data from large-scale shake table tests by DeAlba, Chan, and Seed (1975) which were corrected to account for effects of stress history and multidirectional shaking. Based on these data, Figure 6-1 of Seed et al (1975) (included herein as Figure 2.5.4-48) presents lower bounds of the cyclic stress ratios causing liquefaction versus the standard penetration resistances of sands for magnitudes 5 to 6 and 7 to 7 1/2 earthquakes, corrected to an effective overburden pressure of 1 ton per square foot (N_1) based on the Gibbs and Holtz (1957) correlation of relative density of sands to blow count and effective stress. A plot of N_1 values vs effective stress used in this method is the SPT blow count for borings P1 through P8 and I2, I3, I8, I9, and I10 is included as Figures 2.5.4-28 and 2.5.4-29. The mean value of corrected blow count for these borings was calculated as 20.0, which corresponds to a cyclic stress ratio of 0.278 for a magnitude 5 to 6 earthquake, using Figure 2.5.4-48. When compared with the earthquake induced shear stresses obtained from the SHAKE analysis described in Section 2.5.4.7, the minimum factor of safety against liquefaction calculated by this method was 1.68 at a depth of 15 feet.

A very conservative factor of safety against liquefaction was also calculated using a cyclic stress ratio based on the mean corrected blow count less one standard deviation. An N_1 value of 13.1 was used to obtain a cyclic stress ratio of 0.185 from Figure 2.5.4-48. The minimum factor of safety calculated for the lower value of N_1 was 1.13 at a depth of 15 feet. This is considered acceptable, considering the fact that the mean value of N_1, less one standard deviation, is well below the mean value originally used by Seed et al in determining the curves in Figure 2.5.4-48. An additional conservatism in the analysis is the use of the magnitude 6.0 relationship for determining the cyclic stress ratio. The SSE at the site is based on an Intensity VI-VII earthquake, which corresponds to a magnitude of approximately 5.3, using relationships developed by Gutenberg and Richter (1942).

The factor of safety against liquefaction at various depths for each analysis is presented on Figure 2.5.4-49. It can be concluded that liquefaction will not occur in the beach and glacial outwash sands adjacent to the circulating and service water pumphouse, and that the shorefront is stable against sliding failures due to liquefaction of the sand. The stability against sliding of the shorefront during the SSE is discussed in Section 2.5.5.2. 241.9

INSERT 2.5.4.8.3.3

Section 2.5.4.8.3.3 Liquefaction Analysis of Beach Area Sands Using 2-Dimensional Dynamic Response Analysis

Liquefaction ~~A~~nalyses performed on the sands at the shorefront and discussed in Sections 2.5.4.8.3.1 and 2.5.4.8.3.2 were based on the assumption that the subsurface conditions in this area could be modeled conservatively as a 40 ft deep uniform sand layer overlying 4 ft of basal till and bedrock. An additional analysis was performed in which the sloping bedrock and ground surfaces to the west of the ~~C~~irculating and ~~S~~ervice ~~W~~ater ~~P~~umphouse were incorporated into a 2-dimensional dynamic response model to determine earthquake induced shear stresses. The section selected for the 2-dimensional dynamic response model is similar to the slope stability profile shown in Figure 2.5.5-4. The liquefaction potential of the saturated glacial outwash sands was determined by comparing the induced effective shear stresses calculated from the dynamic model with the dynamic shear strength of the sand available to resist initial liquefaction previously determined from corrected blow count values obtained from ~~S~~tandard ~~P~~enetration ~~T~~ests performed on beach area borings.

The computer program PLAXLY (Plane Strain Dynamic Finite Element Analysis of Soil-Structure Systems) was used to calculate earthquake induced shear stresses within the soil profile. The initial value of low strain shear modulus and damping, total unit weight, and Poisson's ratio of the elements were assigned in accordance with the table below:

Soil Type	Depth (ft)	Gmax (ksf)	Damping (min)	Unit Wt (pcf)	Poisson's Ratio
Outwash Sand	0-10	600	.02	123	.49
	10-20	1250	.02	123	.49
	20-30	1500	.02	119	.49
	30-40	1800	.02	119	.49
Basal Till		20160	.02	145	.40
Bedrock		216000		165	.40
Armor Stone	0-14	900		150	

Note : A Poisson's ratio of .25 was used for unsaturated sands.

The strain compatible shear moduli and damping ratios of the soil were determined through a series of iterations within the PLAXLY program. The time histories of the 4 earthquakes listed below were normalized to the site SSE peak acceleration of .17g and input at the rigid base of the model. These earthquake records were selected because they were recorded at rock sites or stiff soil sites and therefore would be expected to approximately match dynamic response at the Millstone site.

Taft S69E	1952 Kern County Earthquake
Helena N-S	1935 Montana Earthquake
Pacoima Dam S16E	1971 San Fernando Earthquake
Temblor N65W	Parkfield Earthquake

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

Liquefaction potential was calculated at each element for the 6 sections shown on this figure. The results of the PLAXLY analysis and the calculated values of Safety Factor against liquefaction are presented on Table 2.5.4-24.

The blow count data used in Sections 1 to 5 were obtained from onshore borings in the shorefront area. The blow count 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 blow count data is 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 in Figure 2.5.4-48. The curve for Magnitude 6 earthquakes was used to obtain a non-liquefaction 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.

The earthquake induced shear stresses were computed by averaging the peak shear stress values obtained for each of the 4 earthquakes at each element in the PLAXLY model. The effective shear stress was obtained by multiplying the average peak value by two-thirds. 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 analysis,presented on 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 ft. 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 this analysis that liquefaction of the shorefront slopes will not occur and that liquefaction of the intake channel bottom would not effect the integrity of the shorefront slopes adjacent to the Circulating and Service Water Pumphouse. The soil underlying the service water pipe encasement adjacent to the pumphouse is not susceptible to liquefaction.

or result in a condition that would make the service water system inoperable.

TABLE 2.5.4-24

RESULTS OF TWO-DIMENSIONAL LIQUEFACTION ANALYSIS OF BEACH AREA SANDS

SECTION 1
N1=19.81 (MEAN)
SURFACE ELEVATION = +16 FT
GROUNDWATER ELEVATION = +6 FT

ELEMENT NO.	SOIL TYPE	DEPTH (ft)	VERT. STRESS	CYC.STR. RATIO	TAU AVAIL	SHEAR STRESS (PLAXLY) PKFLD	PACOIMA	TAFT	HELENA	TAU (EFF)	F.S.
1	SAND	2.50	308	NA	NA	138	89	97	95	70	NA
2	SAND	7.50	923	NA	NA	378	214	264	199	177	NA
3	SAND	11.50	1318	0.27	356	510	279	367	269	239	1.49
4	SAND	14.25	1481	0.27	400	574	302	435	311	272	1.47
5	SAND	17.00	1643	0.27	444	583	302	465	325	281	1.58
6	TILL	21.00	NA	NA	NA	715	378	616	400	353	NA
8	ROCK	33.00									

SECTION 2
N1=19.81 (MEAN)
SURFACE ELEVATION = +16 FT
GROUNDWATER ELEVATION = +6 FT

ELEMENT NO.	SOIL TYPE	DEPTH (ft)	VERT. STRESS	CYC.STR. RATIO	TAU AVAIL	SHEAR STRESS (PLAXLY) PKFLD	PACOIMA	TAFT	HELENA	TAU (EFF)	F.S.
87	SAND	2.50	308	NA	NA	143	97	108	90	73	NA
88	SAND	7.50	923	NA	NA	390	241	289	230	193	NA
89	SAND	13.00	1407	0.27	380	523	311	394	317	259	1.47
90	SAND	17.00	1643	0.27	444	549	316	448	349	278	1.59
91	SAND	22.00	1930	0.27	521	313	194	281	210	167	3.12
92	TILL	27.00	NA	NA	NA	745	444	698	456	392	NA
94	ROCK	38.00									

SECTION 3
N1=19.81 (MEAN)
SURFACE ELEVATION = 16 FT
GROUNDWATER ELEVATION = +6 FT

ELEMENT NO.	SOIL TYPE	DEPTH (ft)	VERT. STRESS	CYC.STR. RATIO	TAU AVAIL	SHEAR STRESS (PLAXLY) PKFLD	PACOIMA	TAFT	HELENA	TAU (EFF)	F.S.
167	SAND	2.50	308	NA	NA	83	53	70	60	45	NA
168	SAND	7.50	923	NA	NA	316	187	248	188	157	NA
169	SAND	14.00	1466	0.27	396	638	362	493	374	313	1.27
170	SAND	20.00	1820	0.27	491	476	258	363	281	231	2.13
171	SAND	27.00	2205	0.27	595	485	242	353	290	229	2.59
172	TILL	33.00	NA	NA	NA	1260	607	906	747	590	NA
174	ROCK	43.00									

TABLE 2.5.4-24 (Cont)

SECTION 4
N1=19.81 (MEAN)
SURFACE ELEVATION =+6 FT
GROUNDWATER ELEVATION = -6 FT

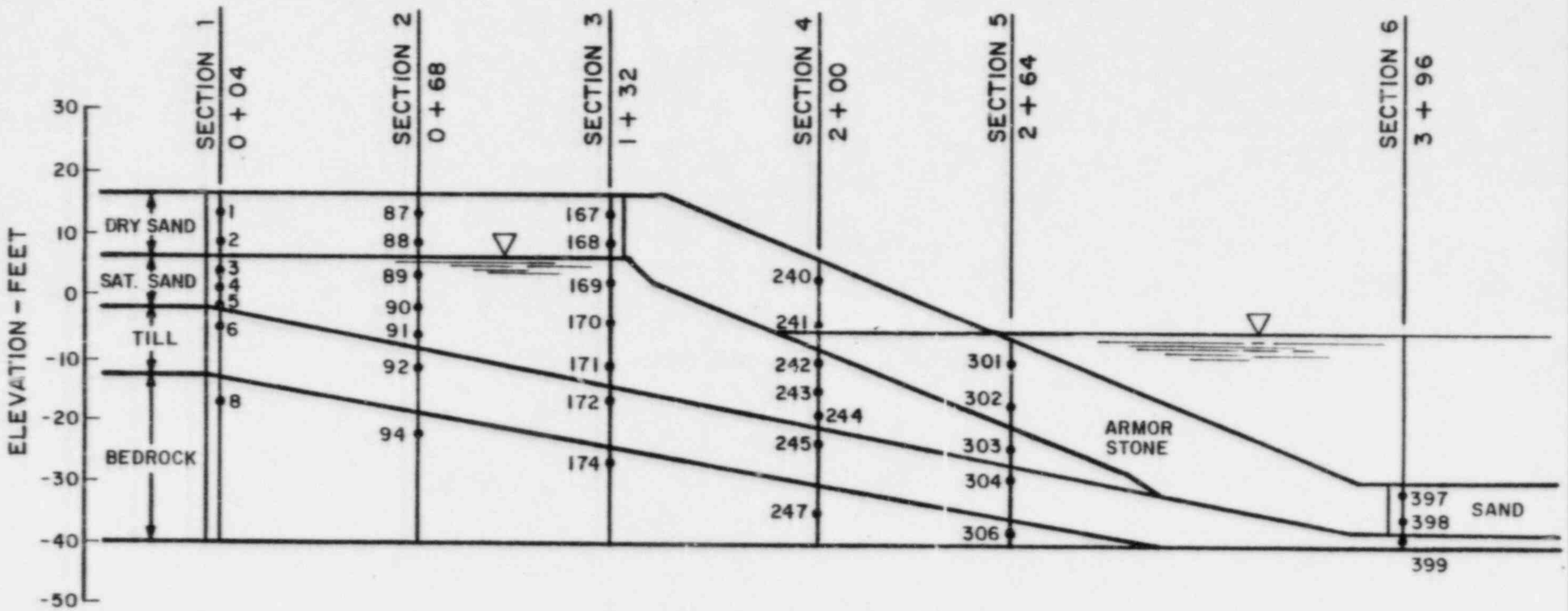
ELEMENT NO.	SOIL TYPE	DEPTH (ft)	VERT. STRESS	CYC.STR. RATIO	TAU AVAIL	SHEAR STRESS (PLAXLY) PKFLD	PACOIMA	TAFT	HELENA	TAU (EFF)	F.S.
240	ARMOR	3.00	450	NA	NA	153	93	11,	92	76	NA
241	ARMOR	10.00	1500	NA	NA	256	164	199	150	129	NA
242	SAND	17.00	2149	0.27	580	678	411	529	381	335	1.73
243	SAND	21.00	2381	0.27	643	384	226	315	227	193	3.33
244	SAND	25.00	2601	0.27	702	422	239	350	250	211	3.32
245	TILL	29.00	NA	NA	NA	1070	575	879	587	521	NA
247	ROCK	39.00									

SECTION 5
N1=19.81 (MEAN)
SURFACE ELEVATION = -6 FT
GROUNDWATER ELEVATION = -6 FT

ELEMENT NO.	SOIL TYPE	DEPTH (ft)	VERT. STRESS	CYC.STR. RATIO	TAU AVAIL	SHEAR STRESS (PLAXLY) PKFLD	PACOIMA	TAFT	HELENA	TAU (EFF)	F.S.
301	ARMOR	3.00	258	NA	NA	128	93	107	99	72	NA
302	ARMOR	10.00	860	NA	NA	208	143	175	147	113	NA
303	SAND	18.00	1440	0.27	389	490	316	405	340	260	1.50
304	TILL	23.00	NA	NA	NA	751	479	609	485	389	NA
306	ROCK	32.00									

SECTION 6
N1=28.0 (MEAN)
SURFACE ELEVATION = -29 FT
GROUNDWATER ELEVATION = -6 FT

ELEMENT NO.	SOIL TYPE	DEPTH (ft)	VERT. STRESS	CYC.STR. RATIO	TAU AVAIL	SHEAR STRESS (PLAXLY) PKFLD	PACOIMA	TAFT	HELENA	TAU (EFF)	F.S.
397	SAND	2.00	118	0.42	50	81	102	76	150	69	0.72
398	SAND	6.00	354	0.42	149	162	183	151	259	126	1.18
399	TILL	9.30	NA	NA	NA	325	347	306	466	242	NA



NOTES

• = ELEMENTS USED IN ANALYSIS

REFER TO FIG. 2.5.5-4 FOR DETAILS OF SUBSURFACE CONDITIONS.

FIGURE 2.5.4-75

SHOREFRONT PROFILE USED IN LIQUEFACTION ANALYSES

MILLSTONE NUCLEAR POWER STATION
UNIT 3
FINAL SAFETY ANALYSIS REPORT