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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April 13, 1984

Docket No. 50-423
B11132

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

References: (1) B. J. Youngblood to W. G. Counsil, Millstone Nuclear Power Station, Unit No. 3 Draft Safety Evaluation Report (DSER), dated December 20, 1983.

(2) B. J. Youngblood to W. G. Counsil, Request for Additional Information for Millstone Nuclear Power Station, Unit No. 3, dated January 16, 1984.

(3) W. G. Counsil to B. J. Youngblood, Millstone Nuclear Power Station, Unit No. 3, Transmittal of Responses to Requests for Additional Information and DSER Open Items (Geotechnical Issues), dated March 27, 1984.

Dear Mr. Youngblood:

Millstone Nuclear Power Station, Unit No.3
Transmittal of Response to Request for Additional Information
and DSER Open Item SEB-23, Lateral Earth Pressure Coefficient

The status and proposed resolution of all Geotechnical Issues identified in References (1) and (2) were discussed with NRC staff reviewer, Mr. John Chen, at a meeting on February 14, 1984.

In Reference (3) Northeast Nuclear Energy Company transmitted additional information to resolve all geotechnical concerns except (1) beach sand liquefaction potential (DSER Open Item SGEB-24 (Section 2.5.4.4) and Questions 241.7 and 241.8,and (2) the lateral earth pressure coefficient at rest (DSER Open Item SGEB-23 (Section 2.5.4.5) and Question 241.16.). In Reference (3) we committed to supply information to resolve these concerns on or before April 13, 1984.

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Attached is the additional information as requested by Mr. Chen to resolve his concerns regarding the lateral earth pressure coefficient. All revisions to the FSAR are provided as they will appear in Amendment 8 to the FSAR, scheduled for submittal in approximately mid-May, 1984. As discussed with Ms. E. L. Doolittle, Licensing Project Manager, the additional analysis required to respond to Mr. Chen's concerns regarding the stability of the beach sand deposits is not yet completed. This information will be provided as it becomes available.

Northeast Nuclear Energy Company trusts that the attached responses to DSER Open Item SEB-23 and Question 241.16 are adequate to resolve Mr. Chen's concerns and close this item/question.

If you have any concerns related to the information contained herein or any questions related to our responses, 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
Senior Vice President

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.

Lorraine J. D'Amico
Notary Public

My Commission Expires March 31, 1988

Millstone Nuclear Power Station, Unit No. 3

Open Items

Structural and Geotechnical Engineering Branch

SGEB-23 Lateral Earth Pressure Coefficient (Draft SER Section 2.5.4.5)

The below-grade walls of structures were designed to resist both the static and dynamic pressure resulting from the surrounding earth and water. The value of the lateral earth pressure coefficient at rest used in the design is 0.5. The dynamic lateral earth pressure on the below-grade walls was determined in accordance with Seed and Whitman's procedure. The procedure used to obtain the dynamic lateral earth pressure is in accordance with the state-of-the-art methods required by the Standard Review Plan (NUREG-0800) and is acceptable. However, the lateral earth pressure at rest is low and requires additional information to confirm the safety design of the below-grade walls.

Response

As requested by the NRC Structural and Geotechnical Branch Reviewer during the January 14, 1984 review meeting, Northeast Nuclear Energy Company has provided additional information to confirm the safety design of the below-grade walls in response to Question 241.16 and Draft SER Open Item SGEB-23 regarding the lateral earth pressure coefficient.

A limited review was made of three representative structures considering the effect of increasing the groundwater level to site grade to evaluate structural stability against flotation, the strength of the containment base mat, and the vertical perimeter walls below grade. In addition, the effect of increasing the coefficient of lateral earth pressure at rest from the design value of 0.5 to a value of 0.7 was evaluated for two representative structures.

All structures were found to be within allowable design criteria for both static and dynamic conditions for all combinations of loading.

A discussion of the plant design basis for groundwater is included in revised FSAR Section 2.5.4.6, and the design basis for lateral earth pressure is discussed in Section 2.5.4.10.3. The response to Question 241.16 has been revised to include the additional information. These revisions are provided herein as they will appear in Amendment 8 to the FSAR scheduled for submittal in May 1984.

Northeast Nuclear Energy Company hereby considers the above Draft SER Open Item (SGEB-23) and Question 241.16 to be closed, pending NRC reviewer's confirmation.

Status (4/84)

DSER Open Item SGEB-23 - CLOSED

Question 241.16 - CLOSED

NRC Letter: May 3, 1983 1.9

Question No. Q241.16 (Section 2.5.4.10.3 and SRP Section 2.5.4) 1.12

Lateral Earth Pressure 1.13

Provide the design values of the lateral earth pressures used in the 1.14
design of rigid, unyielding, foundation walls.

Response: 1.15

FSAR Figure 2.5.4-43 shows the lateral pressure distribution used in 1.16
designing rigid unyielding foundation walls. The figure has been 1.17
revised to correct errors and to add the unit weight of backfill.
Also FSAR Section 2.5.4.10.3 has been changed to correct 1.20
typographical errors.

A limited review of structure stability was performed for the 1.21
emergency generator enclosure tank vault, the service building, and 1.22
the containment structure, considering the effect of increasing the
groundwater level to site grade, elevation +24 feet. The review was 1.24
limited to evaluating structure stability against flotation, the
strength of the base mat, and the strength of the vertical perimeter 1.25
walls below grade. In addition, the effect of increasing the 1.26
coefficient of lateral earth pressure at rest from the design value
of 0.5 to a value of 0.7 was evaluated for the auxiliary building and 1.27
the engineered safety features building. The results were combined 1.28
such that the effect of both the increase in groundwater level and
the increase in K_o were considered on the vertical perimeter walls 1.29
below grade for all five structures listed above. These structures 1.30
were selected because they are representative of all plant structures
for the conditions evaluated.

All structures were found to be within allowable design criteria for 1.31
both static and dynamic conditions for all combinations of loading. 1.32

A discussion of the plant design basis for groundwater is included in 1.33
revised FSAR Section 2.5.4.6, and the design basis for lateral earth 1.34
pressure is discussed in Section 2.5.4.10.3.

2.5.4.5.3 Extent of Dredging

To facilitate the flow of water into the service and circulating water pumphouse, an intake channel has been dredged to the limits shown on Figure 2.5.4-41. Side and longitudinal slopes of the intake channel are designed at 10 and 5 percent, respectively. The beach slope varies from 20 to 10 percent and is protected with heavy armor, as discussed in Section 2.5.5.1.

Borings and laboratory testing in the beach area adjacent to the circulating and service water pumphouse indicate that the beach sands are generally moderately dense, with occasional thin zones of less dense material. Liquefaction analyses of these sands, discussed in Section 2.5.4.8.3.2, indicate that a general liquefaction of the sand adjacent to the pumphouse is highly unlikely. If the looser zones do liquefy, the extent of the failure would be strictly local and would not cause a massive soil movement into the dredged channel.

2.5.4.6 Groundwater Conditions

Groundwater observations have been documented in previous reports (Ebasco 1966; Bechtel Corporation 1969). Water level readings in borehole piezometers were taken for the Millstone 3 site study between 1971 and 1973. In addition, pressure testing of rock in three boreholes and during installation of rock anchors in the turbine and service buildings was conducted to determine the permeability of the rock mass. Also, temporary drains were installed in sections of the containment excavation face and the inflow of water into all excavations was observed throughout construction. These observations form the design bases for groundwater at the site, as discussed below.

2.5.4.6.1 Design Basis for Groundwater

Groundwater observations at the site prior to construction were made in piezometers installed in several borings. Listings of the water elevations and dates of reading are presented in Table 2.5.4-17. Three borings, 303, 310, and 311, were continually monitored over a 2-year period. A plot of elevation vs date for water levels in these boreholes is shown on Figure 2.5.4-38. As a result of these observations, a stabilized groundwater level contour map, based on the water levels measured in January 1972, shown on Figure 2.5.4-37, is used as the basis for determining hydrostatic loadings on structure foundations.

241.16

Localized perched groundwater conditions probably exist because of the irregular distribution of ablation till materials of varying gradation and porosity. It is also likely that shallow, ponded water exists in localized bedrock troughs. The prevalence of bedrock outcrops to the north and northwest of the site indicates that bedrock acts as a groundwater divide, isolating the soils of the tip of Millstone Point from soils further inland. Thus, groundwater recharge would primarily be due to absorption of local precipitation, with probable migration of the waters to the immediately adjacent Long Island Sound. Little groundwater is present in the crystalline bedrock, and virtually all of the groundwater movement is restricted to the soil overburden.

Measurements taken during previous investigations (Bechtel Corporation 1969) showed average influx rates into test pits of about 8 gallons per hour, and it was concluded that both the ablation and basal tills were relatively impervious. The ablation till soils are more pervious than the basal tills and occasionally exhibit partial stratification, including sporadic sand lenses. Thus, the upper portions of the soil will transmit water more readily than the underlying dense basal tills.

All structures are designed for the groundwater levels shown in Table 2.5.4-14 which are based on groundwater contours plotted on Figure 2.5.4-37. No safety-related permanent dewatering system is required to lower groundwater levels. These groundwater contours represent average groundwater elevations of the site prior to the start of construction. A comparison of groundwater contours with the top of basal till contours on Figure 2.5.4-36 verifies that the primary medium for groundwater flow is the permeable surficial soil overlying the basal till. Recharge of the groundwater occurs mainly from precipitation infiltrating through the surficial soils, and flowing toward Long Island Sound and the outwash deposits above the till.

241.16

Construction of the plant results in large changes to the site geohydraulic conditions. Site grade will be lowered to a uniform elevation of +24 feet from the original site grade which varied from elevation 26 feet to 30 feet. The major plant structures are founded at approximately elevation 0 feet on blasted rock excavations and backfilled from subgrade level to the ground surface with fill materials of relatively high permeability. The backfilled zones under and around these structures and the circulating water intake pipelines provide a continuous hydraulic conduit for groundwater flow from the plant area to Long Island Sound. Therefore, the average water levels prior to construction are not necessarily representative of post-construction groundwater conditions. Design groundwater levels used in plant design are shown in Table 2.5.4-14.

A seepage diversion system, consisting of a series of underdrains and porous concrete, has been installed under and around several structures to minimize the amount of seepage into the basement of structures founded below the groundwater table. The quantity of seepage expected to be diverted through the system will be small, due to the low permeability of the basal till and rock at the site. This system is not considered safety related because dewatering is not necessary to ensure the stability of any structure, and failure of the system would not result in a significant inflow of water into the basement levels of any structures. The containment and all other Category I structures are protected from groundwater inflow by a waterproof membrane below the groundwater level.

Water levels measured in borings taken at the site in early 1972 indicate a groundwater piezometric surface with a 3-percent gradient generally sloping from northeast to southwest, as shown on Figure 2.5.4-37.

As discussed in Section 2.4.5.2, Flood Design Considerations, the 2.31 8
controlling event for flooding at the Millstone 3 site is a storm surge 2.33
resulting from the occurrence of the probable maximum hurricane (PMH).
The maximum stillwater level resulting from hurricane surge was 2.34
calculated to be elevation 19.7 feet msl. As shown on Figure 2.4-9, the 2.35
water level drops significantly with time, so that after 2 hours the
flood level is at elevation 17 feet and after 6 hours the surge level 2.36
subsides to elevation 10 feet. A continuous hydraulic connection would 2.37
occur across the site from the main structure area to the shorefront
through the backfill placed around structures and the backfill placed in 2.38
the circulating water pipeline trench. It can be expected that the 2.39
maximum groundwater level due to flooding would not exceed elevation
19.7 feet and would probably be less because of head losses in the soil. 2.40
According to Figure 2.4-9, the water level will drop to 17 feet after 2.41
2 hours.

The design groundwater levels for major safety-related structures shown 2.42
on Table 2.5.4-14 are all equal to or greater than elevation 19 feet 2.43
with the exception of the hydrogen recombiner building, which has a
design groundwater level of 18 feet. However, founding grade is at 2.45
elevation 20 feet for this structure, which is founded on concrete fill
placed directly on bedrock. Design criteria for flood conditions are 2.47
discussed in Section 3.4.

2.5.4.6.2 Groundwater Conditions During Construction 2.49

During construction, the inflow of water into the excavations was 2.50
controlled by pumping from sumps located outside of the building lines 2.52 241.16
adjacent to structures. Most flow through the overburden was 2.53
transported through the sand lenses. All water-softened material was 2.54
removed and replaced with a fill concrete working mat as described in
Section 2.5.4.5.1. The rate of inflow was sufficiently low to allow 2.56
enough time to pour the concrete working mat without further softening
of the till. 2.57

Drainage pipes were installed in the southwest face of the containment 2.58
excavation in order to relieve the hydrostatic pressure on the bedrock 2.59
joint and foliation surfaces. Very little water was observed flowing 2.60
through these pipes, indicating that the quantity of flow through the
bedrock is small and that the permeability of the rock is low. 3.1

Water pressure tests were performed in three boreholes prior to 3.2
construction. These tests indicated that the rock within the site area 3.3
is generally massive with slight to moderate interconnected jointing. A 3.5
summary of the water pressure test data from the boreholes is included
in Table 2.5.4-16. Additional pressure tests were performed prior to 3.6
installation of rock anchors in the turbine and service buildings.
These tests further verified the low permeability of the rock mass. 3.7

These observations suggest that the permeability of the bedrock is 3.8
extremely low, and that little or no groundwater or seawater is expected 3.9
to seep through the fresh rock mass.

2.5.4.7 Response of Soil and Rock to Dynamic Loading 3.11

All Seismic Category I structures and associated piping are founded 3.12
either on bedrock, basal till, or structural backfill. Portions of the 3.15
circulating water discharge tunnel are founded on ablation till in the 241.4
vicinity of the ventilation stack north of Millstone Unit 1. A listing 3.18
of the founding strata for all Category I structures is included in
Table 2.5.4-14.

Hard crystalline bedrock forms the basement complex of the area. The 3.20
overlying dense basal till consists of a hard, compact soil which has
been heavily preloaded by continental ice. Static and dynamic 3.21
properties of the basal till and bedrock are discussed in
Sections 2.5.4.2.5 and 2.5.4.2.6, respectively. Static and dynamic 3.23
properties for the compacted structural backfill are discussed in
Section 2.5.4.5.2.

The bedrock, basal till, ablation till, and structural backfill are 3.24 241.4
stable materials under vibratory motion caused by the SSE. The basal 3.26
till, ablation till, and structural backfill are not susceptible to
liquefaction, as discussed in Section 2.5.4.8. 3.27 241.4

The soil-structure interaction analyses for Seismic Category I 3.28
structures founded on soil were performed using the computer program 3.29
PLAXLY-3. The nonlinear behavior of the subgrade was accounted for by 3.30
use of the computer program SHAKE (LaPlante and Christian 1974) which 3.31
was used to determine the strain-corrected soil properties. The 3.32 241.4
subsurface material properties used in the SSI analysis are discussed in
Section 2.5.4.7.1. The method of SSI analysis and the results are 3.33
discussed in Section 3.7.2.4.

The response of buried piping to seismic loadings is discussed in 3.34
Section 3.7.3.12.

The shorefront west of the circulating and service water pumphouse 3.35
consists of a structural fill and beach and outwash and slope varying 3.37 241.4
from 5H:1V to 10H:1V, protected by graded layers of armor stone. A plan 3.38
showing the extent of the shoreline protection system is presented on 3.39
Figure 2.5.4-41. A typical section is shown on Figure 2.5.5-1. Static 3.41
and dynamic properties of the beach sands are discussed in Section
2.5.4.2.2 and documented in the reports in Appendix 2.5G. The 3.43
liquefaction potential of the beach and outwash sand is discussed in 241.4
Section 2.5.4.8. The stability of the shoreline slopes under static and 3.44
dynamic loading is discussed in Section 2.5.5.2.

The service water intake pipes, between the circulating and service 3.51
water pumphouse and the main plant area, are embedded in a rectangular 3.52
concrete encasement. Soils encountered in the pipeline excavation 3.53
include beach and outwash sands, unclassified stream deposits, and 241.4
ablation till. These soils were removed under the pipeline to dense 3.55
basal till and replaced with Category I structural backfill. The fill 3.56
was placed at a 1:1 slope from the till surface to the base of the 3.57
encasement and compacted to the requirements outlined in 3.58
Section 2.5.4.5.2. The sides of the encasement were backfilled with 3.59

nonstructural fill similar to the material used to backfill behind
retaining walls and described in Section 2.5.4.10.3. The backfill was 4.1_
compacted to 90 percent of maximum dry density as determined by
ASTM 1557, Method D.

241.16

2.5.4.7.1 Subsurface Material Properties Used in SSI Analysis 4.9

The subsurface profiles used in the soil-structure interaction analyses 4.10
for the control building and the emergency generator enclosure (EGE) are 4.12
idealized, horizontal profiles based on subsurface explorations
conducted at the site and described in Section 2.5.4.3. Both of these 4.14
structures are founded on dense basal till overlying bedrock. The 4.15
computer program SHAKE was used to determine strain corrected values of
shear modulus obtained from low strain values previously determined from 4.16
field testing, laboratory testing, or empirical formulae based on
laboratory test data. The program iterates to obtain values of modulus 4.18
that are compatible with strain levels induced in a particular soil
layer by a specific earthquake. The strain levels normally induced by 4.20
earthquakes of magnitudes similar to the Millstone SSE are several
orders of magnitude higher than the low strain levels achieved during 4.21
laboratory or field testing, resulting in a reduction in shear modulus 4.23
when these properties are corrected for strain and input into PLAXLY-3.

241.4

The soil-structure model used in the EGE analyses is shown on 4.24
Figure 2.5.4-72. This idealized profile was selected to conservatively 4.25
model the subsurface conditions under the EGE and in the free-field. 4.26
The geologic profiles presented in Figures 2.5.4-55, 2.5.4-56, and 4.27
2.5.4-71 indicate that the rock surface slopes from approximately 4.28
elevation 0 feet at the east end of the structure to at least elevation
-10 feet at the west end. In the north-south direction, the sloping 4.30
evacuation face between the control building and the south end of the
EGE was backfilled with structural fill over the basal till. The extent 4.32
of structural fill is shown on Section J-J (Figure 2.5.4-55) and
Figure 2.5.4-54. Because the depth and extent of the structural fill 4.33
under the EGE is limited, it was assumed that the model used in the
SHAKE analysis is sufficiently conservative to account for local 4.34 8
variations in the subgrade and their effect on structural response. 4.35

The soil properties input into the SHAKE calculation are listed in 4.36
Table 2.5.4-21A for the free-field model and 2.5.4-21B for the 4.37
structure-effects model. Three earthquake time histories, from the 4.38
Taft, Parkfield-Temblor, and Pacoima Dam earthquakes, were normalized to
the site SSE peak acceleration value of .17g and input at bedrock. 4.39
Shear modulus and damping iterations were performed within the SHAKE 4.40
program in accordance with the curves marked "Resonant Column Test" on 4.41
Figures 2.5.4-73 and 2.5.4-74. These curves were developed from 4.42
empirical formulae and resonant column tests performed on samples of
compacted structural fill from the Millstone site. These test results 4.44
are presented on Figure 2.5.4-42. The tests show good correlation with 4.45
curves present by Seed and Idriss in the SW-AJA report (1972).

The strain corrected values of shear modulus and damping in the free- 4.46
load are presented in Table 2.5.4-21A. The mean value for each layer 4.47
was calculated and used to represent the individual soil layer

properties used in the PLAXLY model shown on Figures 3.7B-11 and 4.48
3.7B-12. The Millstone site artificial earthquake was input at bedrock 4.49
and the soil was modeled as a finite element mesh. The use of SHAKE to 4.50
perform shear modulus and damping iterations precludes the need to 8
iterate in the PLAXLY model. A discussion of the soil-structure 4.51
interaction analysis is presented in Section 3.7B2.4.

241.4

For the control building, the soil profile analyzed in SHAKE and used in 4.52
the soil-structure interaction analysis was the section where rock was 4.53
the deepest; i.e., top of rock at elevation -15 feet. Shear wave 4.54
velocities were used to define soil stiffness. The low strain and 4.55
strain-corrected soil properties for the free field case are listed in
Table 2.5.4-22.

2.5.4.8 Liquefaction Potential 4.57

The foundation materials beneath some of the Seismic Category I 4.58
structures consist of limited depths of dense to very dense basal tills 4.59
and/or compacted select granular backfill. These materials are not 5.1
susceptible to liquefaction under earthquake motions as described in the
following sections.

2.5.4.8.1 Structural Backfill 5.3

Based on studies of soils where liquefaction has been observed (Seed 5.4
1968, Lee and Fitton 1969, Kishida 1969), it is concluded that the 5.6

TABLE 2.5.4-14

FOUNDATION DATA FOR MAJOR STRUCTURES

Structure	Foundation Bearing Load (psf)	Founding Grade (ft)	Founding Material	Average Thickness Till (ft)	Average Thickness Structural Fill (ft)	Dimensions of Foundation (ft)	Design Groundwater Elevation (ft)	Maximum Calculated Static Settlement (in)
Containment	8,250	-38.7	Rock	-	-	158 diameter	21	0.04
Main Steam Valve	5,000	+9.0	Rock	-	-	70 x 60	19	0.01
Auxiliary	4,450	-0.5	Rock	-	-	177 x 164	23	0.02
Engineered Safety Features	2,950	-0.5	Rock	-	-	139 x 47	21	0.01
Control	3,500	-0.5	Till	0 to 10	-	125 x 105	19	0.02 to 0.03
Emergency Generator Enclosure (EGE)	4,000	+9.0	Till	10		10' Strip	19	0.01 to 0.4
Emergency Generator Oil Tank	1,600	+1.5	Till	10	4	65 x 32	19	less than 0.01
Emergency Generator Mats	1,500	+18.50	Structural Backfill	17	9.5	44 x 12	19	0.25
Refueling Water Storage Tank	4,000	+[illegible].0	Rock	-	-	45 diameter	-	less than 0.01
Demineralized Water Storage Tank	4,000	+14.5	Rock	-	-	35 diameter	-	less than 0.01
Fuel	5,500	+3.0	Rock		-	93 x 112	23	less than 0.01
Waste Disposal (Liquid)	4,100	+0.5	Till	2 to 8	-	112 x 48	23	0.02
Waste Disposal (Solid)	3,500	+19.5	Structural Backfill	23	7	114 x 38	23	0.25
Hydrogen Recombiner	4,400	+20.0	Concrete Fill	-	-	39 x 27	18	less than 0.01

NOTE: All foundations are structural mat except EGE which is strip footing and slab on grade.