PSNH PUBLIC SERVICE
Company of New Hampshire

SEABROOK STATION
Engineering Office:
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November 10, 1982

SBN-363
T.F. B7.1.2

United States Nuclear Regulatory Commission
Washington, D. C. 20555

Attention: M . George W. Knighton, Chief
Licensing Branch 3
Division of Licensing

References: (a) Construction Permits CPPR-135 and CPPR-136, Docket Nos. 50-443 and 50-444
(b) USNRC Letter, dated July 27, 1982, "Request for Additional Information," F. J. Miraglia to W. C. Tallman
(c) PSNH Letter, dated September 30, 1982, "Response to 240 Series RAIs; (Hydrologic and Geotechnical Engineering Branch)," J. DeVincentis to J. B. Kerrigan

Subject: Response to RAI 240.40 and 240.41(b); (Hydrologic and Geotechnical Engineering Branch)

Dear Sir.

We have enclosed responses to the subject Requests for Additional Information which were forwarded in Reference (b).

It was indicated in Reference (c) that 240.40 and 240.41(b) would be submitted in the "near future."

The enclosed RAI responses will be included in OL Application Amendment 48.

Very truly yours,

YANKEE ATOMIC ELECTRIC COMPANY

J. DeVincentis

J. DeVincentis
Project Manager

ALL/fsf

B001

240.40 (2.4.5)

In your response to Question 240.34 (Hydrologic Engineering Question 240.04) you indicated that wave overtopping will not cause significant erosion because of its short duration. Our analysis indicates that the peak wave overtopping rate of the vertical seawall is in excess of 1600 cfs for a period of about 0.2 hrs. We conclude that this could result in the loss of fill material behind the vertical seawall and adjacent to the two class I electrical manholes (#13/14 and #15/16). Discuss the consequences of this loss of fill material or describe the measures planned to prevent it.

RESPONSE: The area behind the vertical seawall and adjacent to the two class I electrical manholes (#13/14 and #15/16) will be protected to prevent erosion of the fill material due to wave overtopping. To accomplish this, either a stone apron, blacktop, a combination of both, or other suitable methods will be used to prevent detrimental erosion of the fill material. The extent of the area protected will be sufficient to ensure the integrity of both the vertical seawall and the electrical manholes. When the details of this erosion protection are determined they will be forwarded to HGEB of the NRC for review and concurrence.

· 240.41 (2.4.2, 2.4.5)

It is not apparent from our review of the ponding level on plant grade that concurrent intense precipitation was included in your wave overtopping runoff/ponding analysis. Therefore, provide a detailed analysis on the routing of the combined precipitation runoff from Probable Maximum Precipitation and wave overtopping runoff from the PMF/PMH event.

a) If credit is taken for flow through the storm drainage system, provide justification that the storm drainage system cannot become blocked during this event.

b) Identify the maximum water surface levels by location and elevation from the vertical seawall to the overflow weir (seawall).

c) Identify plant access openings and sill elevations that may be affected by the runoff on plant grade.

RESPONSE: a) Credit is not taken for flow through the Storm Drainage System for the PMF/PMH event. However, during a PMF/PMH event, significant blockage of the system is considered an unlikely event and, therefore, discharge through the Storm Drainage System would reduce the maximum standing water elevation on the plant site to less than the 20.6 feet MSL presented in the FSAR.

b) The maximum water surface elevation for structures at Seabrook Station due to the PMP is 20.6 feet MSL.

The one-hour PMP for the site as presented in the FSAR is 8.6 inches. Applying this precipitation over the 2×10^6 ft^2 site area and ignoring all precipitation losses, the average flow rate off the site is 398 cfs. Assuming no credit for the Storm Drainage System, this local intense precipitation would pond on the site until the road perimeter elevation of 20.5 feet MSL was exceeded. Once elevation 20.5 feet MSL is exceeded, water would flow over the roadway and proceed to flow off-site over the flood protection structures. The length of roadway around the perimeter of the plant to the south, east, and north is about 3,800 feet. The depth over the roadway crown can be determined from the weir equation:

$$Q = CLH^{3/2}$$

A conservative weir coefficient, C, of 2.8 is applicable. Solving for H necessary to pass 398 cfs yields 0.1 feet. Therefore, when added to the roadway crown elevation, the maximum water surface elevation for the PMP induced PMF is 20.6 feet MSL. As stated in part a) of this response, complete blockage of the Storm Drainage System is considered unlikely and, therefore, a portion of the 398 cfs would be conveyed off-site through the Drainage System which has a capacity of about 310 cfs. Therefore, the maximum water surface elevation of 20.6 feet MSL is conservative.

The flooding analysis presented in the FSAR conservatively calculated the quantity of wave overtopping waters used in assessing maximum water levels on the site. The analysis maximized the potential overtopping including the allowance for bottom scour, the use of maximum supportable waves, ignoring any reduction affects due to wave approach angles, and increasing wave overtopping quantities a full 70 percent for potential wind affects.

Almost all of the wave overtopping occurs along the seawall section. This overtopping is a direct result of waves breaking against the vertical seawall structure and, therefore, the wave runup and overtopping is heavily concentrated immediately behind and adjacent to the seawall. The calculated wave overtopping quantities are based on the continuous occurrence of maximum supportable waves. In actuality, there will be periods of lower wave activity resulting in reduced rates of wave overtopping and intervals of time with no wave overtopping. During peak overtopping conditions, using the maximum wave height and a 5-second period, the seawall is overtopped about 30 percent of the time.

The seawall and adjacent site flood protection structures are elevated to 20.0 feet MSL, while the crown of the site's perimeter roadway is elevated to 20.5 feet MSL. The crown of this road is located approximately 100 feet from the face of the seawall. The stillwater level site ponding within the perimeter roadway due to the PMP is assumed to be between 20.5 and 20.6 feet MSL, again taking no credit for the Storm Drainage System. A portion of the wave overtopping waters will spread to either side of the seawall and be directed back over the adjacent non-overtopped rock riprap shore barrier due to the 0.5 foot gradient between the perimeter roadway crown and the top of the flood protection barrier, and be directed back across the vertical seawall as wave overtopping is intermittent along the seawall.

Figure 240.41-1 shows the five major potential flow pathways for wave overtopping waters to exit the site. Flow pathways 1, 2 and 3 are assumed bounded to the north by the site perimeter roadway. Flow pathways 4 and 5 are bounded at both their upstream and downstream limits by the site perimeter roadway system with a crown elevation of 20.5 feet MSL.

An analysis was performed to estimate maximum water levels throughout the site, and the relative percentages of flow conveyed by the five previously identified flow pathways. Wave overtopping water was allowed to spread laterally into flow pathways 1 and 3 from 2 and north through 4 and 5 if the necessary hydraulic head was developed. Water was also allowed to return off-site over the vertical seawall, Pathway 2, during times when the seawall was not being overtopped.

The discharge capacity of each flow path was determined by iterating between inflow and outflow until they balanced within acceptable limits. Inflow to flow pathways was calculated using the Manning equation. Outflow was calculated using weir discharge at the overflow points. The slope used in the Manning equation being the ratio of the difference in head between inflow and outflow points to the average path length. To make these calculations, it was assumed that the area between the seawall and the roadway had a uniform head. This is a conservative approach in that no credit is taken for enhanced lateral flow away from the seawall area due to the hydraulic gradients which surely exist during overtopping.

Table 240.41-1 shows the discharge capacity of each flow path for various heads behind the seawall. As can be seen, a water level of about 21.0 feet MSL behind the seawall provides sufficient hydraulic gradient to produce flows which are approximately equivalent to the wave overtopping rate, taking no credit for site drainage. The water elevation within the roadway calculated by this method varies from about 21.0 feet MSL on the south of the site to 20.6 feet MSL on the north of the site, again taking no credit for site drainage. In actuality, there are over 50 catch basins within the roadway area with a drainage capacity well in excess of the combined flow rate of flow Pathways 4 and 5. It is extremely doubtful that all of these catch basins would be blocked and therefore it is concluded that the water level within the perimeter roadway is less than 21.0 feet MSL.

During the time of maximum wave overtopping, the wind is blowing in a northwesterly direction (N 62^{o}W). This wind has already been used to increase overtopping water quantities by 70 percent. The site buildings will cause the wind flow pattern to flow up and over, as well as, around the buildings creating a stagnant area and/or areas of reversal in flow patterns with diminished velocities near ground level and along the buildings. The wind affects on overtopping water should have a minimal net result on any large scale transport of water especially since the water is so shallow.

Wind affects should be more in the form of wind blown spray from site ponding water which would negligibly affect the site flooding levels. The percent of overtopping waters flowing through the site to the north and east sides of the plant have been shown to be minimal in Table 240.41-1.

It is concluded that the maximum sustainable site ponding level at Seabrook Station for the combined PMF/PMH event is less than 21.0 feet MSL.

c) The following is a list of safety-related buildings where ponding water resulting from the PMF/PMH is critical, or approaches being critical to entering the building. Elevations listed represent the sill of the access opening.

	Building	Sill Elevation (Ft.)
1.	Fuel Storage Building	20.5 (a)
2.	Main Steam and Feed Water Pipe Chase-East and West	21.0
3.	Service Water Pump House	21.0
4.	R.H.R. and Containment Spray Equipment Vault	20.67 (b)

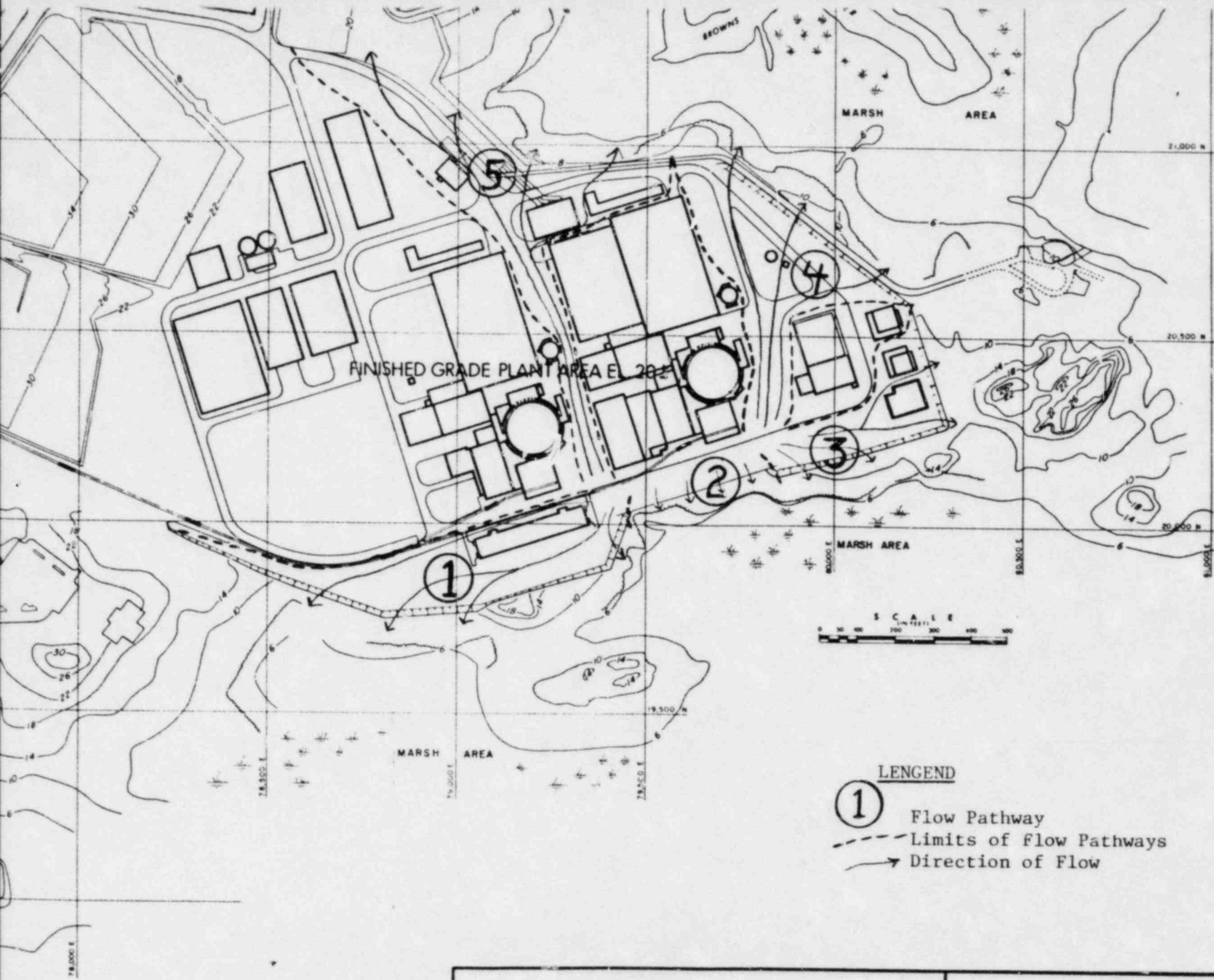
(a) New and spent fuel are locally protected from rising water with a concrete barrier. The barrier heights are 25.5 and 25.0 feet, respectively.

(b) The high point of the sloping floor beyond the sill restricts rising water from penetrating the building. The high point is at El. 21.0 feet.

TABLE 240.41-1

Flow Pathway Discharges - (CFS)

Water Elevation Behind Seawall Feet MSL	Flow Pathways 1	2	[illegible]	4	5	Total
21.0	295	840	325	70	50	1580
20.9	230	710	265	50	35	1290
20.8	175	600	210	30	25	1040



PUBLIC SERVICE COMPANY OF NEW HAMPSHIRE
SEABROOK STATION - UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

FLOW PATHWAYS FOR WAVE OVERTOPPING WATERS

FIGURE 240.41-1