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April 18, 1983

Mr. Harold R. Denton, Director
Office of Nuclear Reactor Regulation
U.S. Nuclear Regulatory Commission
Washington, DC 20555

Subject: Braidwood Station Units 1 and 2
Additional FSAR Information
NRC Docket Nos. 50-456/457

Reference (a): B. J. Youngblood letter to L. O. DelGeorge
dated January 14, 1983

Dear Mr. Denton:

The above Reference requested that the Commonwealth Edison Company provide certain additional information concerning our FSAR for Braidwood Station Units 1 and 2.

The Attachment to this letter provides our response to Questions 10.61, 130.55, 130.57, 241.4, 241.5, 371.12, 371.16, and 371.18. Our FSAR will be amended to include the information contained in the Attachment to this letter as appropriate.

Please address any questions that you or your staff may have concerning this matter to this office.

One (1) signed original and fifteen (15) copies of this letter with Attachment are provided for your use. For the purposes of clarity, two (2) sets of 11" x 17" figures referenced in our responses to the above questions are being sent directly to Ms. Janice A. Stevens.

Very truly yours,

E. Douglas Swartz

E. Douglas Swartz
Nuclear Licensing Administrator

Attachment

cc: J. G. Keppler - RIII
RIII Inspector - Braidwood

B001

6150N

QUESTION 010.61

(1) Verify that failure of nonseismic Category I structures and equipment in the lake screen house following a SSE will not affect the exposed portions of essential service water supply lines above the lake screen house floor and thereby prevent essential service water flow.

(2) Verify that the essential service water discharge structure is seismic Category I."

RESPONSE

(1) The substructure of the lake screen house, which houses the essential service water intakes, is designed as a seismic structure. Postulated failure of non-seismic portions of the structure and equipment will not affect the intakes due to the location of the intakes away from any such structures and equipment. In addition, the intakes are protected by concrete enclosures protruding above the top of the mat. The three intakes are also separated by 26.5 and 30 feet to prevent a single object from blocking two intakes.

(2) The essential service water discharge structure is seismic Category I.

QUESTION 130.55

"In Section 3.5.1.5 of the FSAR you made a statement which implies that missiles were the only effect of the near-site explosion of a boxcar of TNT. Specify if other loads, such as pressure, have been considered concurrently with the missiles as a result of the explosion. Describe the missiles postulated as a result of the explosion, their energy content, and state the basis for conclusion reached in the FSAR that their effects would be less than those of the missiles associated with the design basis tornado. Also, describe the method and provide the results in sufficient detail to demonstrate that the above conclusion is correct when the missiles are considered in conjunction with the pressure resulting from such an explosion (Refer to the load combinations stated in the Standard Review Plan, Section 3.3.2.II.3)."

RESPONSE

A study has been made to estimate the potential hazard to the Braidwood Station from missiles generated in a postulated accidental explosion of one boxcar of TNT on the railroad 1550 feet from the station.

The typical average dimensions of a boxcar, i.e., car length of 50'-6", width of 9'-6", height of 11'-0" and a conservative wall plate thickness of 1/4" are used in the analysis. A total of 66 tons of TNT explosives is considered. The station has been evaluated for both primary and secondary fragments. Initial velocities for primary fragments have been obtained by using Gurney Energy. This assumes a zero gap between the boxcar wall and the explosives, which results in conservatively high initial velocities of the fragments. The size and velocities of primary fragments and the velocities of secondary missiles have been obtained by using equations given in Reference 1. The impact of primary fragments on the roof is estimated from a high trajectory analysis which uses a terminal velocity approach and a conservatively assumed drag coefficient of 0.1, in determining the final striking velocities.

The evaluation is based on the probability of an explosion of 1.47×10^{-5} per year at the railroad near Braidwood Station, as given in Appendix A to Chapter 2.0 of the Braidwood PSAR (Reference 9 in the Braidwood FSAR, Subsection 2.2.4). As explained in Appendix A, this number is derived based on conservative assumptions.

The results of the study are summarized below:

1. Primary Fragments:

   Primary fragments referred to here denote the fragments from the boxcar. Tables Q130.55-1 and Q130.55-2 summarize the results of primary fragments hitting the wall and the roof respectively. The fragment which is of design concern has a weight of 0.402 lbs (probability of occurrence = 1×10^{-7} per year). The velocity at which the design fragment strikes a wall is 1956 ft/sec. The corresponding kinetic energy is 2.38×10^4 ft-lbs. The penetration depth is 2.8 inches and no spalling will occur on the other side of the wall (24 inch minimum wall thickness). The velocity at which the design fragment strikes the roof is 607 ft/sec and the corresponding kinetic energy is 4.6×10^3 ft-lbs. The penetration depth in the roof is only 0.34 inch and no spalling will occur on the other side of the roof slab.

2. Secondary Fragments Impacting Concrete Wall:

   The blast waves from explosions can interact with objects located near the explosion source and accelerate them to high velocity. these objects are called the "secondary missiles" or "secondary fragments." Three secondary missiles have been postulated and evaluated in the analysis: (1) steel rod, 1-inch diameter x 3-feet long, weight 8 pounds; (2) steel pipe, 3-inch diameter, schedule 40, 10-feet long, weight 78 pounds; and (3) 5/8 inch diameter 2-inch long bolt. These secondary missiles are selected to cover a wide range of shape length, and weight of secondary missiles and also to give a perspective comparison with typical missiles generated by tornados. Depending on the relative orientation of these objects to the railroad track, they may or may not reach the plant after the postulated explosion. Table Q130.55-3 indicates that even if these secondary missiles hit the wall, the impact velocities are very small and no scabbing will occur.

3. Missile in Conjunction with the Pressure Load:

   It is noted that the primary and secondary missiles resulting from the postulated explosion may only cause local partial penetration in the walls and roofs of the safety-related structures. The overall structural response due to these missiles are minimal as compared to the blast pressure wave effect. The evaluation shows that the combined effect of the postulated missiles and pressure load is far less than the resistance capacities of the exterior walls and roofs of the safety-related structures.

Based on the above analysis it is concluded that missiles generated from an accidental explosion will not present a potential hazard at the Braidwood Station.

Reference

1. "A Manual for the Prediction of Blast and Fragment Loadings On Structures," DOE/TIC-11268, U.S. Department of Energy, November 1980.

TABLE Q130.55-1

RESULTS FOR PRIMARY FRAGMENTS HITTING CONCRETE WALL

WEIGHT OF FRAGMENT, W_f lb	STRIKING VELOCITY, V_s ft/sec	STRIKING ENERGY ft-lb	PENETRATION X_f in.	TIME OF IMPULSE* sec	PROBABILITY OF OCCURRENCE per year
0.402	1956	2.38×10^{4}	2.80	2.4×10^{-4}	1.0×10^{-7}
0.454	2084	3.06×10^{4}	3.29	2.6×10^{-4}	7.35×10^{-8}
0.590	2373	5.16×10^{4}	4.61	3.2×10^{-4}	3.50×10^{-8}
0.772	2680	8.61×10^{4}	6.39	4.0×10^{-4}	1.47×10^{-8}
0.934	2905	1.22×10^{5}	7.97	4.5×10^{-4}	7.35×10^{-9}
1.167	3170	1.82×10^{5}	10.20	5.4×10^{-4}	3.00×10^{-9}
1.372	3364	2.41×10^{5}	12.11	6.0×10^{-4}	1.47×10^{-9}

*Time of rectangular impulse for overall response.

TABLE Q130.55-2

RESULTS FOR PRIMARY FRAGMENTS
HITTING CONCRETE ROOF

WEIGHT OF FRAGMENT, W_f lbs	STRIKING VELOCITY, V_s ft/sec	PENETRATION X_f in.	TIME OF IMPULSE (sec)	PROBABILITY OF OCCURRENCE per year
.402	607	0.34	1×10^{-4}	1.3×10^{-8}
.454	619	0.37	1.0×10^{-4}	1.03×10^{-8}
.590	647	0.45	1.2×10^{-4}	4.89×10^{-9}
.772	676	0.54	1.3×10^{-4}	2.05×10^{-9}
.934	698	0.61	1.4×10^{-4}	1.03×10^{-9}
1.167	724	0.72	1.6×10^{-4}	4.19×10^{-10}
1.372	744	0.80	1.8×10^{-4}	2.05×10^{-10}

TABLE Q130.55-3

RESULTS FOR SECONDARY MISSILES

POSTULATED MISSILE	PROJECTED AREA(ft^2)	IMPACT VELOCITY ft./sec.	REMARKS
Steel rod, 1 in. diameter x 3 ft. long, weight 8 lbs	5.45×10^{-3} (minimum)	No Hit	-
	2.5×10^{-1} (maximum)	58.63	No Spalling
Steel pipe, 3 in. diameter, schedule 40, 10 ft. long, weight 78 lbs	1.594×10^{-2} (minimum)	No Hit	-
	2.5 (maximum)	64.14	No Spalling
5/8" diameter bolt, 2 in. length, weight .314 lb.	9.39×10^{-3}	115.0	No Spalling

QUESTION 130.57

"Comparison of Table 3.2-1 (sh.4) and Section 3.8.4.1 reveals an apparent discrepancy regarding the classification of the Essential Service Water Discharge Structure (ESWDS). The ESWDS is classified as Category I in Table 3.2-1 but it is not included in Section 3.8.4, Other Seismic Category I Structures. You are requested to clarify this discrepancy and, assuming that the ESWDS is classified as a Category I structure, provide all the pertinent information regarding its design, location and construction."

RESPONSE

The essential service water discharge structure (ESWDS) is designed as a Seismic Category I structure. It is a reinforced concrete structure designed to provide anchorage for the discharge end of the essential service water pipes in the essential service cooling pond (see Figure 3.8-80). Subsection 3.8.4.1.10 will be revised to include the above description of the ESWDS.

QUESTION 241.4

"The extent or limits of slurry trench cut off shown in FSAR Figure 2.5-74 does not agree with that described in the FSAR text (page 2.5-109). Explain this discrepancy. Also, provide a plan showing slurry trench cut off around the cooling lake and Essential Services Cooling Pond and verify that the slurry trench cut off is continuous and can be relied upon to act as a seepage barrier for the ESCP."

RESPONSE

The slurry cut off trench beneath the perimeter dike system, referred to on page 2.5-109, is the slurry trench cut off for the perimeter dike of the cooling lake. A plan view showing the boundary of the cooling lake which coincides with the perimeter dike slurry trench is given in Figure 2.4-37.

The slurry trenches shown in Figure 2.5-74 are trenches that were installed prior to main plant and lake screenhouse construction to assist in construction dewatering of these facilities. These trenches are not considered permanent installations and should not be confused with the cooling lake perimeter dike slurry trench.

The cooling lake perimeter dike slurry trench is continuous around the perimeter of the cooling lake and, therefore, continuous along the north and west sides of the essential service cooling pond (ESCP). Plan views of the ESCP and perimeter dike along the north and west sides of the ESCP are given in Figures 2.4-26 through 2.4-29. Sections of the perimeter dike and slurry trench are given in Figure 2.4-35. The slurry trench along the ESCP is a soil-bentonite backfilled slurry trench extending from elevation 597 feet to top of till and, in most cases, are keyed into till. As-built profiles are provided in Figures Q241.4-1, Q241.4-2 and Q241.4-3.

The design of the ESCP does not rely on the slurry trench as a seepage barrier. The ESCP seepage has been conservatively determined assuming the slurry trench does not exist. Existance of the slurry trench only makes the seepage analysis more conservative.

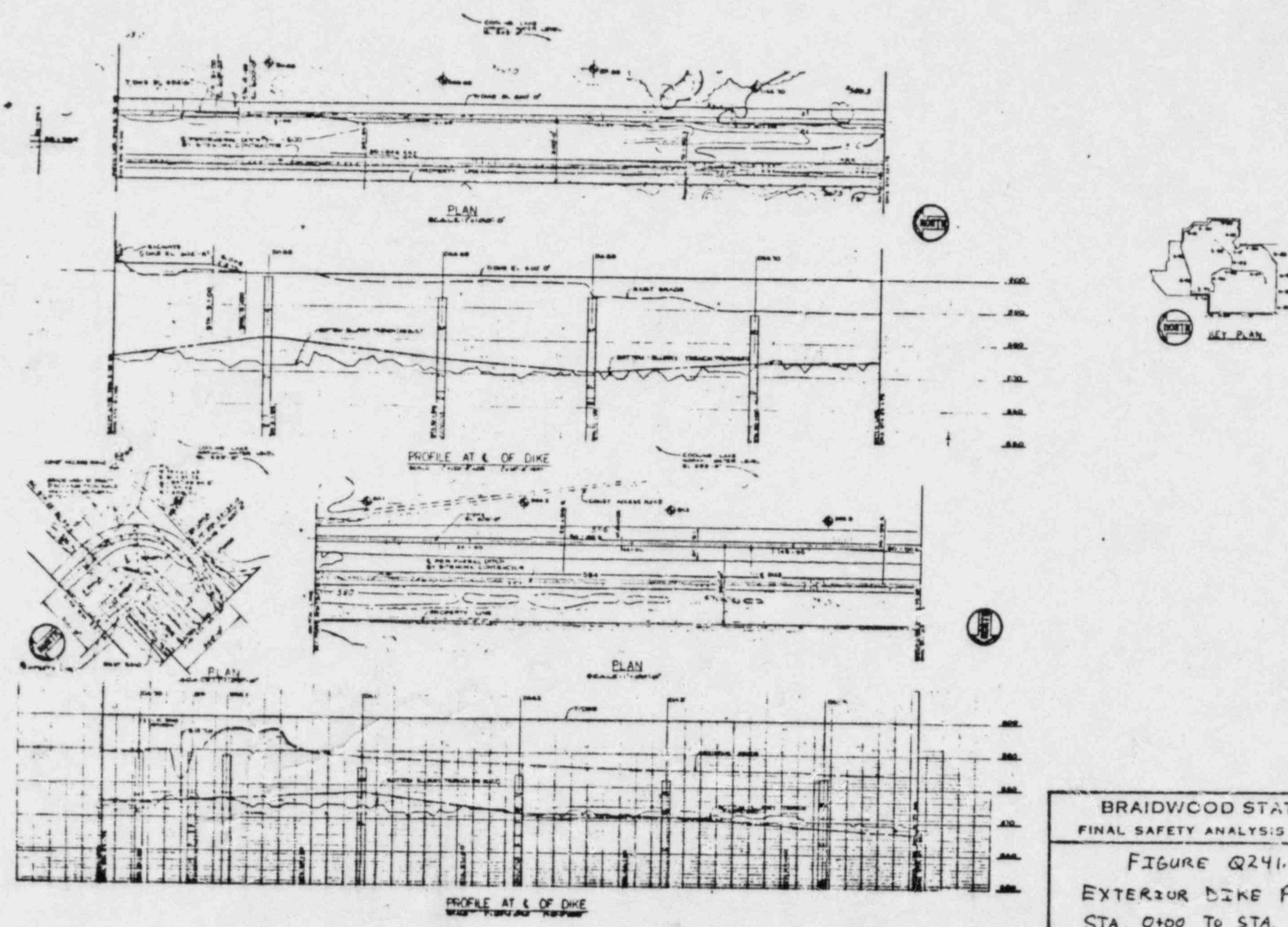

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FIGURE Q241.4-1
EXTERIOR DIKE PROFILE
STA. 0+00 TO STA. 49+00

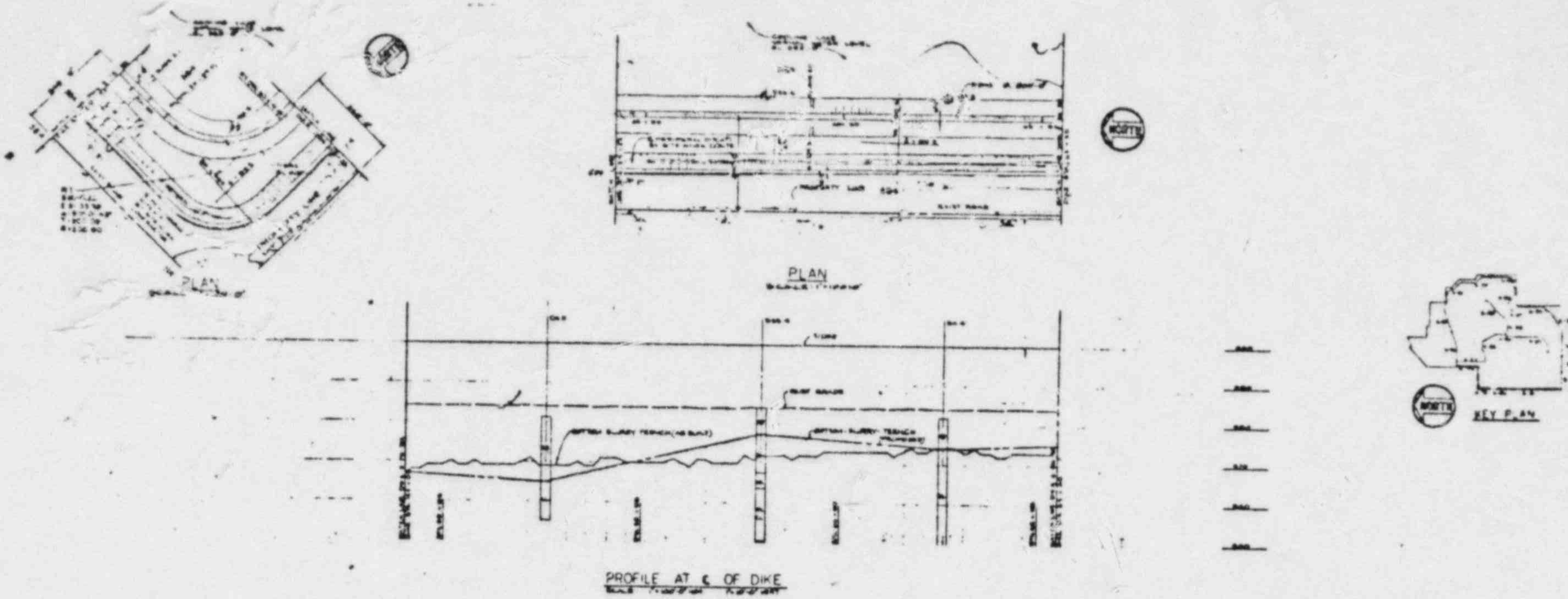

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FIGURE Q241.4-2
EXTERIOR DIKE PROFILE
STA. 49+00 TO STA. 65+50

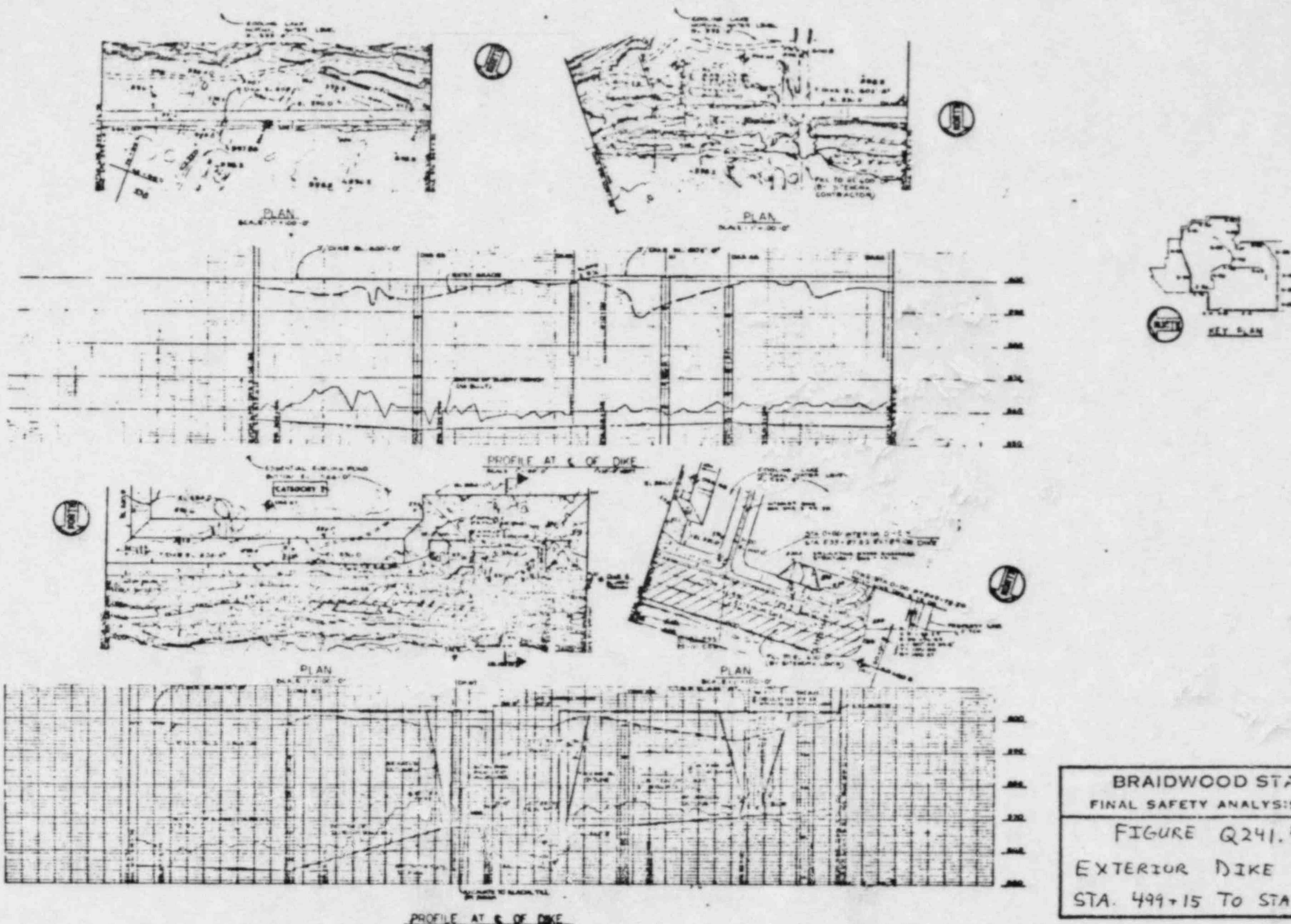

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FIGURE Q241.4-3

EXTERIOR DIKE PROFILE

STA. 499+15 TO STA 540+79.6

QUESTION 241.5

"Provide the following information for seismic Category I Essential Services Water Discharge Structure:

(1) Maximum bearing pressures, and factors of safety against sliding, overturning modes of failure for both static and dynamic loading conditions.

(2) Discuss the liquefaction potential of the material surrounding and beneath the Essential Services Water Discharge Structure.

(3) Furnish a plan showing the location of the Essential Services Water Discharge Structure with reference tc Essential Services Cooling Pond (ESCP). Discuss the potential for the blockage of discharge structure openings in the event of a flow-type failure of the slopes of the ESCP as a result of a Safe Shutdown Earthquake."

RESPONE

(1) The maximum bearing pressures and factors of safety against sliding and overturning modes of failure for both static and dynamic loading conditions are given in Table Q241.5-1.

(2) The essential service water discharge structure is founded on approximately 23 feet of Wedron glacial till deposit overlying the Carbondale bedrock formation. (See boring H-4, Figure 2.5-159, sheet 16.) The glacial till is stiff to hard and not susceptible to liquefaction.

The essential service water discharge structure is backfilled with previously excavated sand compacted to minimum 85% relative density in accordance with ASTM D 2049. Results of eight field density tests indicate relative densities ranging from 90% to 121% and averaging 104%. The average field dry density is 113.4 lb/ft^3. This backfill is not susceptible to liquefaction. The liquefaction potential of the adjacent natural sand deposit forming the ESCP foundation soils is discussed in Subsection 2.5.6.5.2.

(3) A plan showing the location of the essential service water discharge structure with reference to the ESCP is given in Figure 2.4-28. Sections through the structure are given in Figure Q362.8-1. The ESCP slope south of the

structure is 10 horizontal to 1 vertical. The stability of the slope has been analyzed and results presented in Subsection 2.5.6.5.1.2.

In the event that a flow-type failure occurred as a result of the SSE, the discharge pipes would not be blocked with material from the slope. The invert of the discharge pipes is at elevation 591.0 feet. The top of the 10 to 1 slope is greater than 110 feet south of the discharge pipes and has been graded to elevation 590.0 feet. The toe of the interior dike is approximately 215 feet south of the discharge pipes at its closest point. The interior dike has been designed to be stable under OBE conditions and is of sufficient distance away from the discharge pipes to have no potential effect on their operation. It is concluded that the discharge pipes will not become blocked from any flow-type slope failure.

TABLE Q241.5-1

MAXIMUM BEARING PRESSURE AND FACTORS OF SAFETY

FOR ESSENTIAL SERVICE WATER DISCHARGE STRUCTURE

	Static Loading		Dynamic Loading			
			OBE		SSE	
	Case I	Case II	Case I	Case II	Case I	Case II
Maximum Bearing Pressure, KSF	1.22	1.26	1.42	1.46	1.66	1.70
Factors of Safety						
Against Sliding in Direction of Pipes (Section G, Figure Q362.8-1)	5.3	5.0	6.4	6.4	5.9	5.9
Against Sliding in Direction Perpendicular to Pipe (Section F, Figure Q362.8-1)	36.2	28.3	2.8	2.8	1.1	1.1
Against Overturning (Section G, Figure Q362.8-1)	2.4	2.5	2.0	2.1	1.6	1.6
Against Overturning (Section F, Figure Q362.8-1)	2.6	2.8	1.8	1.9	1.2	1.3

Note: Case I - Water Surface at Elevation 598.2 feet (Flood Conditions)
Case II - Water Surface at Elevation 587.0 feet.

QUESTION 371.12

(1) Reference previous Questions 371.2 and 371.4 concerning site drainage. Although the applicant response (Amend 18 and 21) provides some additional information, the FSAR section still does not contain sufficient information for the staff to determine that provisions for site drainage are adequate. We recommend that the applicant be prepared to discuss this subject in detail during the site visit and document the information in the FSAR. We will expect to review with the design engineer(s) the following:

(a) Method used to determine subarea hydrographs or peak discharges, time of concentration, contributing areas and precipitation values.

(b) With appropriate full scale topographic mapping, review the subarea configurations, flow paths, culverts, swales, ditches and road weirs used to convey flow from the site area.

(c) Discuss the potential for partial or full blockage of culverts. You should consider potential for debris, especially during early stages of operation when area construction debris is likely and the effect of relative culvert size (most of yours are relatively small diameter) on potential blockage.

(d) Consideration of security requirements on site drainage. The staff has recently encountered altered drainage as a result of security fencing, berms, etc., at the Grand Gulf site. The applicant should coordinate site drainage requirements with planned security provisions.

(e) You state that area F will pond below plant flood elevations. Since you don't discuss routing or outflow, we must assume that the entire 48 hour PMP volume is stored, which would require about three feet of depth. Since ditch invert elevations are shown at 597.0, we don't think the volume is available. We need to discuss your methodology for this area."

RESPONSE

The analysis for local intense precipitation has been revised. It is assumed that the site drainage system and the culverts will not be functioning at the time of local intense precipitation. The detailed description and results of the analysis are presented in revised Subsection 2.4.2.3.

During the site visit on March 30, 1983, the revised analysis was discussed with Mr. Gary Staley, NRC hydrologist. Full scale site drainage and grading, and other related drawings were presented during the discussion. The general alignment of the security fence as shown on the site drawings was shown, and it was noted that the top of footing of the security fence will be at grade level or below. It was agreed that the revised analysis is acceptable and no further questions on this analysis remain to be answered.

adjusted to the Wilmington site by multiplying the Custer Park discharge by the ratio of the square roots of the drainage areas.

The maximum known discharge near Wilmington, 75,900 cfs, occurred July 13, 1957. Its corresponding gauge height was 11.40 feet above datum. The maximum stage during the period of record was 13.88 feet, caused by ice jams. Ice jam floods in 1883 and 1887 reached a stage of 16.73 feet, for which the discharge is not known. All maximum stages greater than those due to floods were caused by ice jams. Of the 36 years for which maximum water surface elevations are known, 18 maximums were caused by ice jams as high as almost 7 feet above the year's highest flood stage (see Subsection 2.4.7).

2.4.2.2 Flood Design Considerations

The plant main floor is located at elevation 601.0 feet, which is above all flood levels from nearby rivers, streams, and reservoirs. The cooling pond dike system is higher than the calculated flood elevation with coincident wind wave action. The probable maximum precipitation (PMP) water surface elevation of the pond is below safety-related facilities. Floods occurring on the Kankakee River could affect only the river screen house, which is a non-safety-related structure supplying makeup water to the cooling pond. Other streams in the area pose no flood threat to safety-related items. The site drainage system has been designed to pass rainfall without flooding.

There are no dams upstream on nearby rivers whose failure could cause flooding at the site. The general terrain of the area is flat, with no location at which landslides could cause flood waves at the site.

The controlling event for flooding at the site is the probable maximum flood for the cooling pond (see Subsection 2.4.8.2). This event has been analyzed by applying the local probable maximum precipitation (PMP) to the pond watershed following an antecedent storm equivalent to one-half the PMP (see Subsection 2.4.8.2.4).

2.4.2.3 Effects of Local Intense Precipitation

Site grading and drainage are designed to assure that the local PMP will have no effect on safety-related facilities. The layout of roads, tracks, and drainage in the immediate plant area is shown in Figure 2.4-7.

PMP data are taken from Hydrometeorological Report No. 33 (Reference 2) and is estimated to amount to 31.9 inches over a 48-hour period. This is the summer PMP, which is greater than the largest winter PMP coincident with the water equivalent of the 100-year snow pack. The PMP time distribution in 6-hour and 1-hour periods is given in Tables 2.4-2 and 2.4-3. The

$$Q = CIA, \text{ where}$$

Q = peak runoff (cfs)

C = coefficient of runoff

I = intensity of rainfall (inches/hour)

A = drainage area (acres).

The coefficient of runoff was estimated by using a weighted average of the values obtained for the building roofs, roads, graded and other areas (Reference 2a). The time of concentration was computed from Kirpich's formula (Reference 2a). The intensity of rainfall corresponding to a time of concentration was interpolated from Table 2.4-4a. The water surface elevation was estimated for peak flow over the peripheral roads and railroads using a broad crested weir formula with a coefficient of discharge of 2.7.

The plant site area is divided into Zones A, B, C, and D as shown in Figure 2.4-7a.

The storm water, which accumulates in Zone A, flows over two peripheral roads southwest of the plant buildings. The overflow length of the road is approximately 160 feet at elevation 600.0 feet. The area of Zone A is 13.1 acres.

The weighted runoff coefficient is estimated as 0.69 and the time of concentration is 16 minutes. The peak runoff from Zone A is 285 cfs, which will flow over the peripheral road and would produce a water surface elevation near the plant buildings of 600.75 feet. If the value of runoff coefficient is conservatively assumed to be 1, the peak runoff from Zone A would be 413 cfs, with a water surface elevation of 600.97 feet near the plant buildings.

The runoff from Zone B flows over railroad track 1 on the east, and over track 4 on the north. The total overflow length as shown in Figure 2.4-7a is 1,650 feet, and the top of the rail is at elevation 601.0 feet for these two tracks. The area of Zone B is 27.3 acres. An estimated runoff coefficient of 0.63 was used, with a time of concentration of 20 minutes. The peak runoff from Zone B is 496 cfs, which will flow over the peripheral railroad and produce a water surface elevation of 601.23 feet near the plant buildings. If the value of the runoff coefficient is conservatively assumed to be 1, the peak runoff from Zone B would be 787 cfs, with a water surface elevation of 601.32 feet.

TABLE 2.4-4a

MAXIMUM RAINFALL INTENSITY DURING LOCAL PROBABLE MAXIMUM PRECIPITATION

TIME (minutes)	CUMULATIVE RAINFALL (inches)	RAINFALL INTENSITY (inches per hour)
5	4.22	50.7
10	6.51	39.1
15	8.22	39.9
30	11.42	22.8
60	14.61	14.61

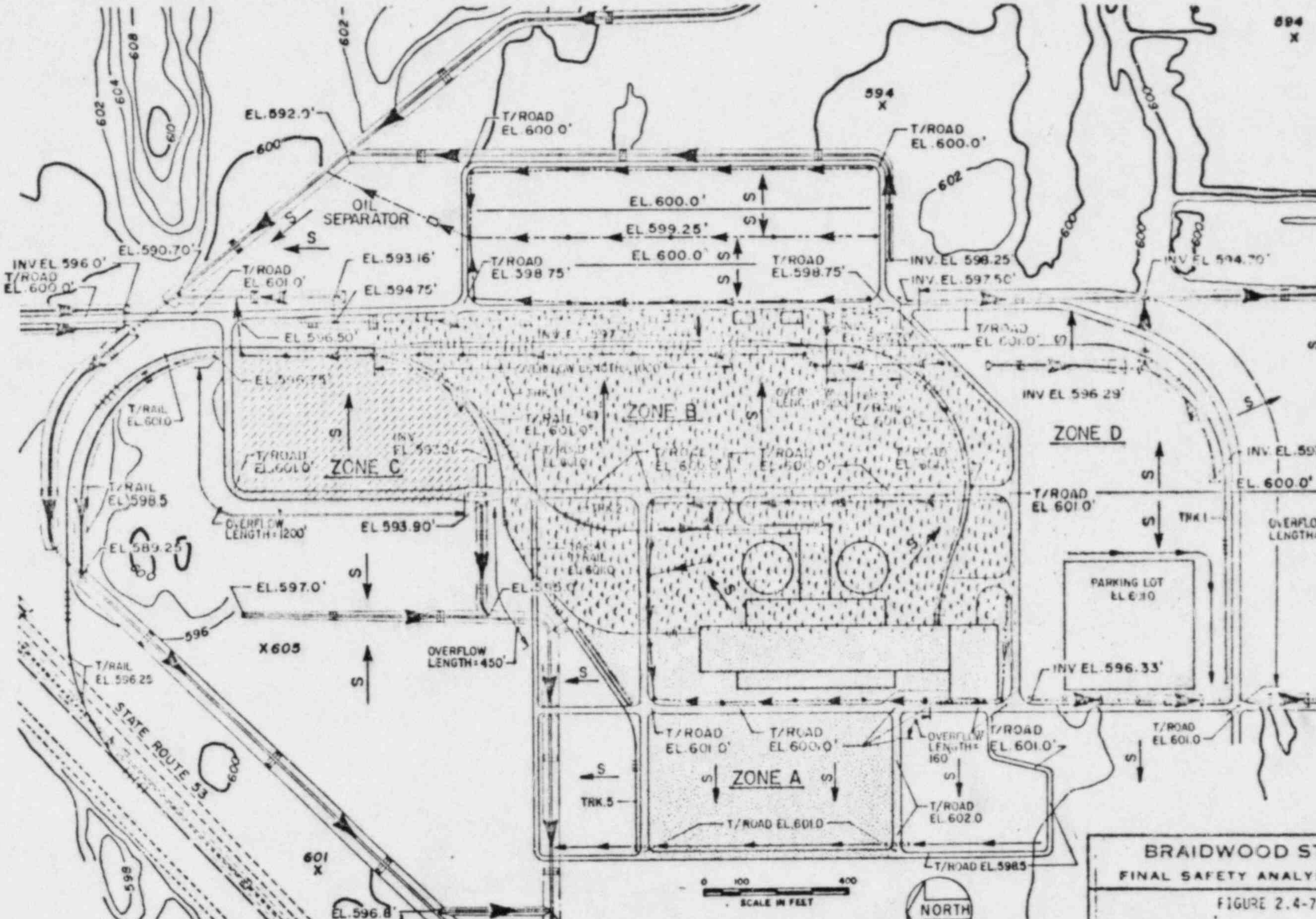

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FIGURE 2.4-7a

SUBDIVISION OF PLANT AREA FOR LOCAL INTENSE PRECIPITATION ANALYSIS

QUESTION 371.16

"The PMF and coincident wind may not be the critical event for structural design of the pond screenhouse. The applicant should consider enough combinations to be assured that the most critical event has been identified. Past experience indicates that in similar situations the maximum gradient wind on the normal pond level with coincident seismic loading may be the controlling design, especially if this scenario yields breaking waves. Provide a table that lists pertinent parameters, including the water induced loads for the scenarios investigated."

RESPONSE

Refer to the response to Question 371.18.

QUESTION 371.18

"The statement to Page 2.4-18, 2nd paragraph, conflicts with the information in the 4th paragraph on page 2.4-4, which indicates different design bases for the pond screen house. Apparently different portions of the screen house have different design bases. Provide additional discussions to clarify this apparent discrepancy. Provide the design bases forces (water induced) for the safety related portion of the pond screen house."

RESPONSE

As described in Subsection 3.8.4.1 the substructure of the lake screen house is designed as a Category I structure. The substructure houses the intakes of the essential service water lines. The essential service water pumps are located in the Auxiliary Building.

The hydrostatic and hydrodynamic forces in combination with seismic events were used in the design of the substructure. Dynamic wave effects resulting from 40 mph wind coincident with PMF pool, and 60 mph wind coincident with normal pool were considered.

The bottom elevation of the essential service cooling pond (ESCP) is at elevation 584 feet 0 inch. Locally the grade around the lake screen house on the lake side is at elevation 570 feet 2 inches. Due to the large depth available in comparison to the wave height (Tables 2.4-11 and 2.4-13) at the lake screen house, the breaking waves will not occur for different lake levels. The forces due to wind generated waves, and the relevant wave parameters are presented in Table Q371.18-1.

TABLE Q371.18-1

WAVE PARAMETERS APPLICABLE
TO THE LAKE SCREENHOUSE

Parameter	Normal Pool	Normal* Pool	PMF Pool
Lake Water Level (feet)	595	595	598.17
Fetch Length (miles)	0.45	1.25	1.25
Overland Wind Speed (mph)	60	60	40
Significant Wave Height (feet)	2.3	3.0	2.35
Wave Period (seconds)	3.4	2.37	2.39
Average Depth (feet)	11.0	11.46	11.46
Depth at the Structure (feet)	24.8	24.8	28.0
Wave Breaking (B) or Non-Breaking (NB)	NB	NB	NB
Hydrostatic Force (lb./ft)	19,190	19,190	24,461
Height of Line of Action Above Grade (feet)	8.33	8.33	9.33
Hydrodynamic Force (lb./ft)	700	2,260	793
Height of Line of Action Above Grade (feet)	13.92	14.25	15.55

*Assumed internal dikes not existing.