PSEG Public Service Electric and Gas Company

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Robert L. Mittl General Manager
Nuclear Assurance and Regulation

October 12, 1984

Director of Nuclear Reactor Regulation
U.S. Nuclear Regulatory Commission
7920 Norfolk Avenue
Bethesda, MD 20814

Attention: Mr. Albert Schwencer, Chief
Licensing Branch 2
Division of Licensing

Gentlemen:

HOPE CREEK GENERATING STATION
DOCKET NO. 50-354
DRAFT SAFETY EVALUATION REPORT
OPEN ITEM STATUS

Pursuant to discussions with the Environmental and Hydrologic Engineering Branch, enclosed for your review is the revised response to Draft Safety Evaluation Report Open Item Numbers 5a and 5d.

Should you have any questions or require any additional information on these items, please contact us.

Very truly yours,

R. L. Mittl/JCR

Enclosure

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1/1

C D. H. Wagner
USNRC Licensing Project Manager (w/attach.)

W. H. Bateman
USNRC Senior Resident Inspector (w/attach.)

<u>DSER Open Item No. 5 a, b and d</u> (DSER Section 2.4.5)

WAVE IMPACT AND RUNUP ON SERVICE WATER INTAKE STRUCTURE

The applicant has analyzed the wind waves that would traverse plant grade coincident with the PMH surge hydrograph and runup on safety-related facilities. These calculations were based on the assumption that wind waves would be generated in the Delaware Estuary and progress to the site. As the surge level would begin to rise, resulting from the approaching eye of the postulated hurricane, the wind speed would progressively change direction from the southeast clockwise to the west. Waves encroaching on the southern end of the Island would be depth-limited (i.e., the waves would "feel" bottom and thus become shallow water waves) by plant grade elevation on both the Salem and Hope Creek sites. These depth-limited (shallow water) waves will impact and runup on the southern and western faces of the safety-related structures in the power block. The applicant has stated that the southern face of the Reactor Building and the Auxiliary Building are designed for a flood protection level of 38.0 ft msl or 3.2 ft above the maximum calculated wave runup height of 34.8 ft msl and the other exposures of safety-related structures have a flood protection level of 32.0 ft msl or 1 ft above the maximum calculated wave runup height of 31.0 ft msl.

The staff has requested the applicant to provide additional information on the waves that impact on the river face of service water intake structure. The waves impacting on this face of the structure are not reduced in height (depth-limited) as those that traverse plant grade.

As indicated in Section 2.4.1, the applicant states that all accesses to safety-related structures (doors and hatches) are provided with water-tight seals designed to withstand the head of water associated with the flood protection levels. But, the applicant has not indicated whether the water-tight doors are designed to withstand either the combined loading effects of both static water level and the dynamic wave impact or, as cited in Sections 3.4.1 and 3.5.1.4 of this report, the impact of a barge propelled by winds and waves associated with a hydrologic event that floods plant grade.

Based upon its analysis according to SRP 2.4.5, the staff concludes that the flood protection level of El. 38.0 ft msl for the southern face of the Reactor Buidling and Auxiliary Building and El. 32.0 ft msl for the remaining safety-related structures within the power block meets the requirements of Regulatory Guide 1.59. Until additional information and analysis

DSER Open Item No. 5 a, b and d (Cont'd)

are available, the staff cannot conclude that the flood protection level of El. 32.0 ft msl for the Service Water Intake Structure meets the requirements of Regulatory Guide 1.59. Based on its analysis, the staff cannot conclude that the plant meets the requirements of GDC 2 with respect to the hydrologic aspects of Probable Maximum Surges and Seiche Flooding.

Response

The requested information for the service water intake structure has been provided in the responses to the following NRC questions:

QUESTION NO.	INFORMATION PROVIDED
240.8	Wave runup elevations
240.9	Wave impact loads
240.8 & 410.69	Flood protection

As a result of discussions with the NRC staff, the response to Question No. 410.69 has been revised and the following summary calculations have been revised and are attached:

1. Analysis of overtopping of Service Water Intake Structure

2. Runup on the East Face of the Service Water Intake Structure

QUESTION 410.69 (Section 9.2.1)

Provide a figure(s) in the FSAR which shows the protection of the station service water system from the flood water (including wave effects) of the design basis flood.

RESPONSE

The general arrangement of the intake structure is provided in Figures 1.2-40 and 1.2-41. Section AA of Figure 1.2-41 is reproduced here as Figure 410.69-1 which identifies the watertight areas and the walls and slabs designed to accommodate flood loads. As described in Sections 2.4.2 and 2.4.5, the south and west exterior walls of the intake structure are subject to a maximum wave run-up elevation of 134.4 feet due to the probable maximum hurricane (PMH). Such waves could overtop the roof of the western portion of the structure at elevation 128 feet. However, a rigorous analysis has been performed to determine the depth of water in the low area (elevation 122.0 feet) after wave impact and to confirm that water does not enter the building through the air intake control dampers (bottom elevation 128.5 feet). Therefore, flood water will not enter into the dry area of the intake structure. On the north side of the intake structure, the maximum water level will be only slightly higher than the still water elevation (113.8 feet) during the PMH. According to Table 2.4.6, the maximum wave elevation for the north side of the intake structure is 26.3 feet MSL (elevation 115.3 feet) due to a postulated multiple dam break. Therefore, flood protection of the north exterior wall to elevation 121.0 feet is adequate.

On the east side of the intake structure, the maximum wave run-up elevation due to the PMH equals 121.97feet. This elevation is due to a 1% wave traveling in the direction of Fetch "A". Fetch A, which is rotated about 15 degrees from Fetch 1 (as shown in Figures 410.69-2 and 410.69-3), is chosen to maximize the wave run-up elevation. Since this elevation is lower than the bottom of the HVAC exhaust opening, flood water will not enter the intake structure from the east side of the building.

In addition the following assessments have been made to confirm the adequacy of the structure and interior components for the overtopping wave:

a. The exterior walls are designed to withstand the flood loads including the dynamic wave action effects.

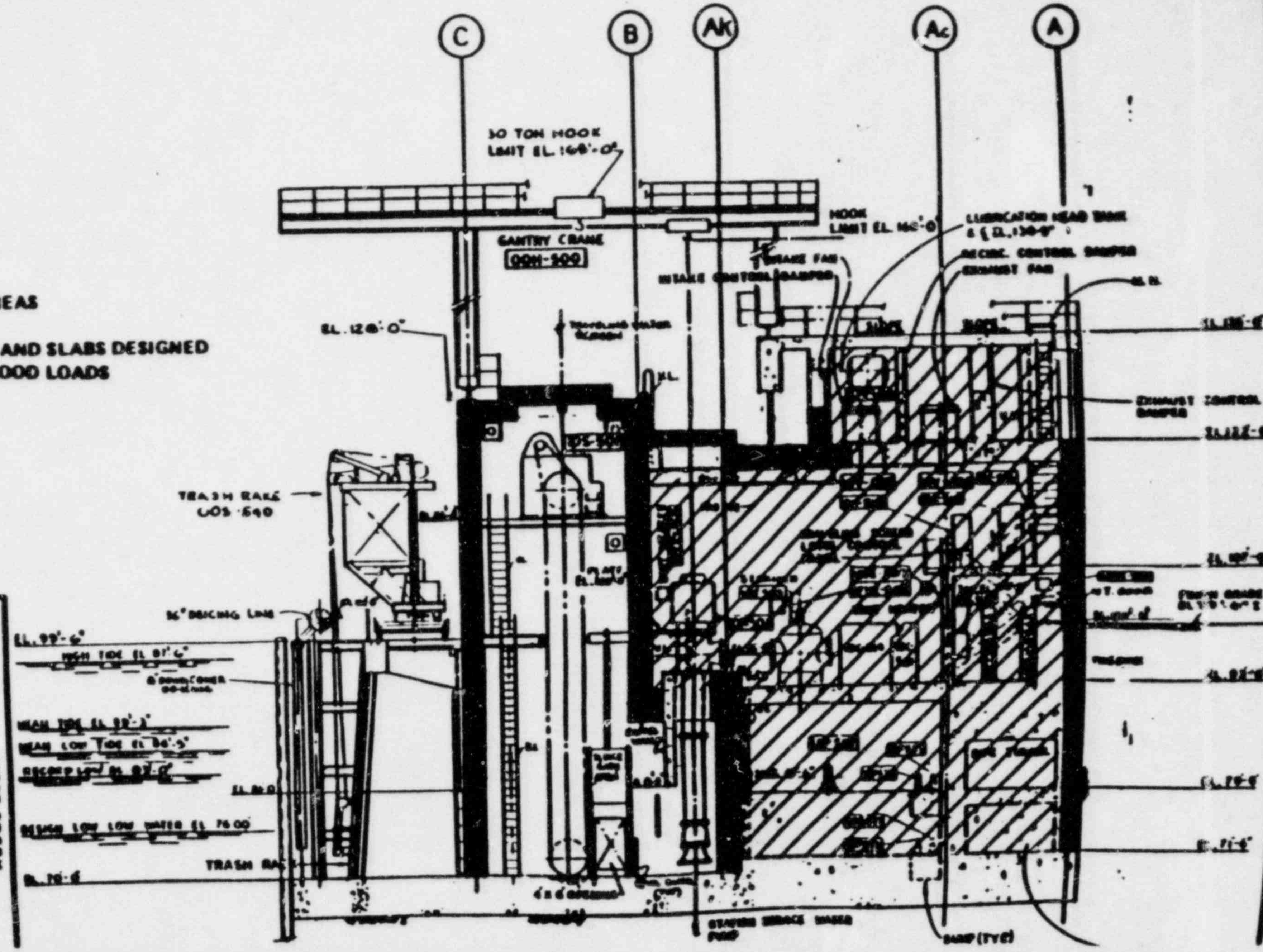
b. The roof hatches at both elevations 122.0 and 128.0 feet have been sealed (caulking, gaskets, etc.) to prevent any intrusion of water. The hatch covers are keyed into the openings to prevent any adverse slippage due to wave induced loadings.

c. All Seismic Category I components except for the traveling water screens are located within the dry areas of the structure.

d. The traveling water-screens, located in the "wet" area between column lines B and C have electric motors which are fully protected against the flood water level.

e. A condition was postulated where suspended moisture enters the dry areas of the structure through the air intake control dampers. It has been assessed that all of the Seismic Category I components subjected to this environment will continue to function as required.

Section 3.4.1 and Table 3.4-1 have been revised for clarification.



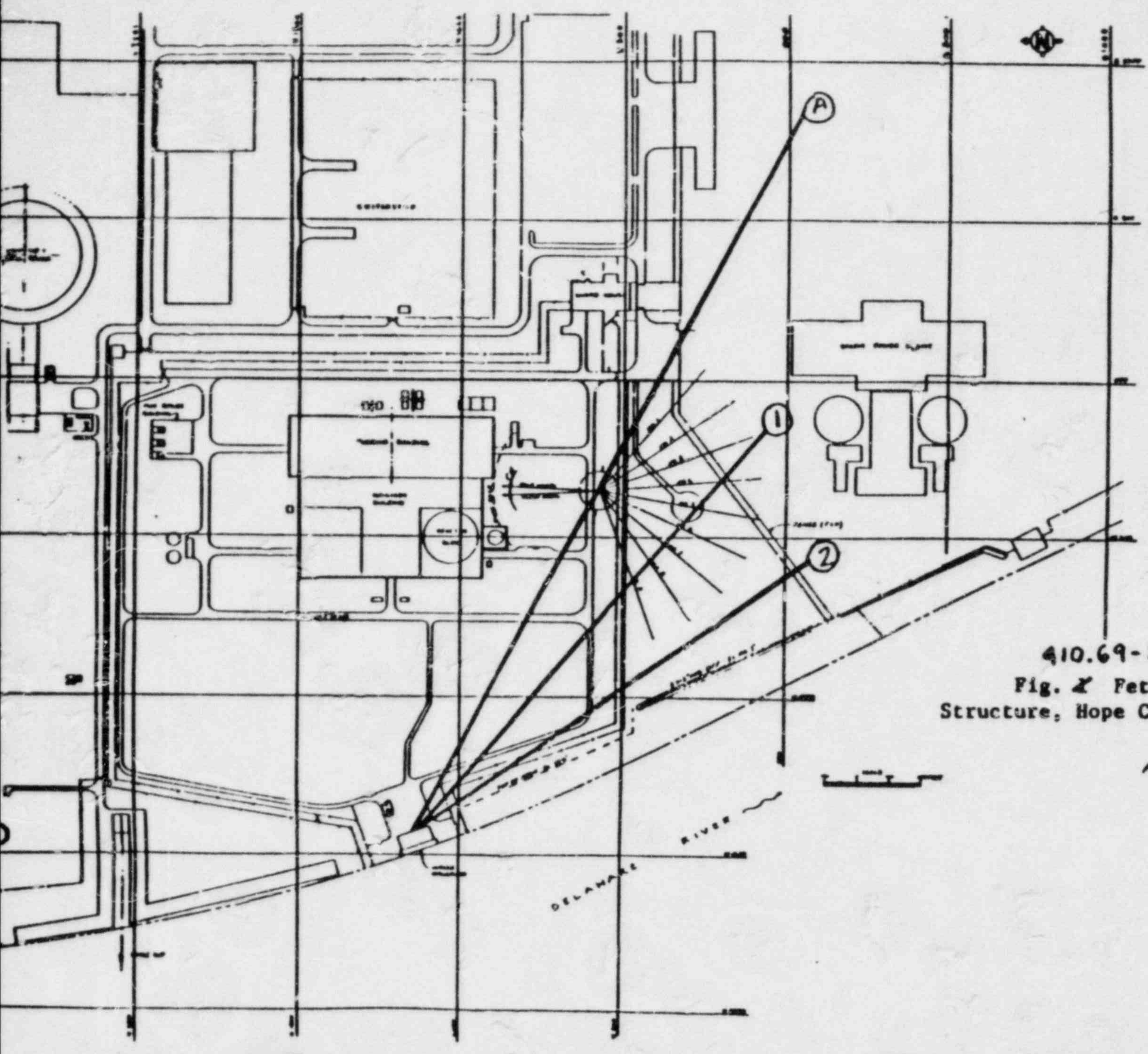
LEGEND:

DRY AREAS

WALLS AND SLABS DESIGNED FOR FLOOD LOADS

HOPE CREEK
GENERATING STATION
FINAL SAFETY ANALYSIS REPORT

SERVICE WATER INTAKE
STRUCTURE — FLOOD
PROTECTION



410.69-2

Fig. 4 Fetches for Service Water Intake Structure, Hope Creek Generating Station.

REV. 4

410.69-3

Fig. 2 Fetches for Service Water Intake Structure, Hope Creek Generating Station

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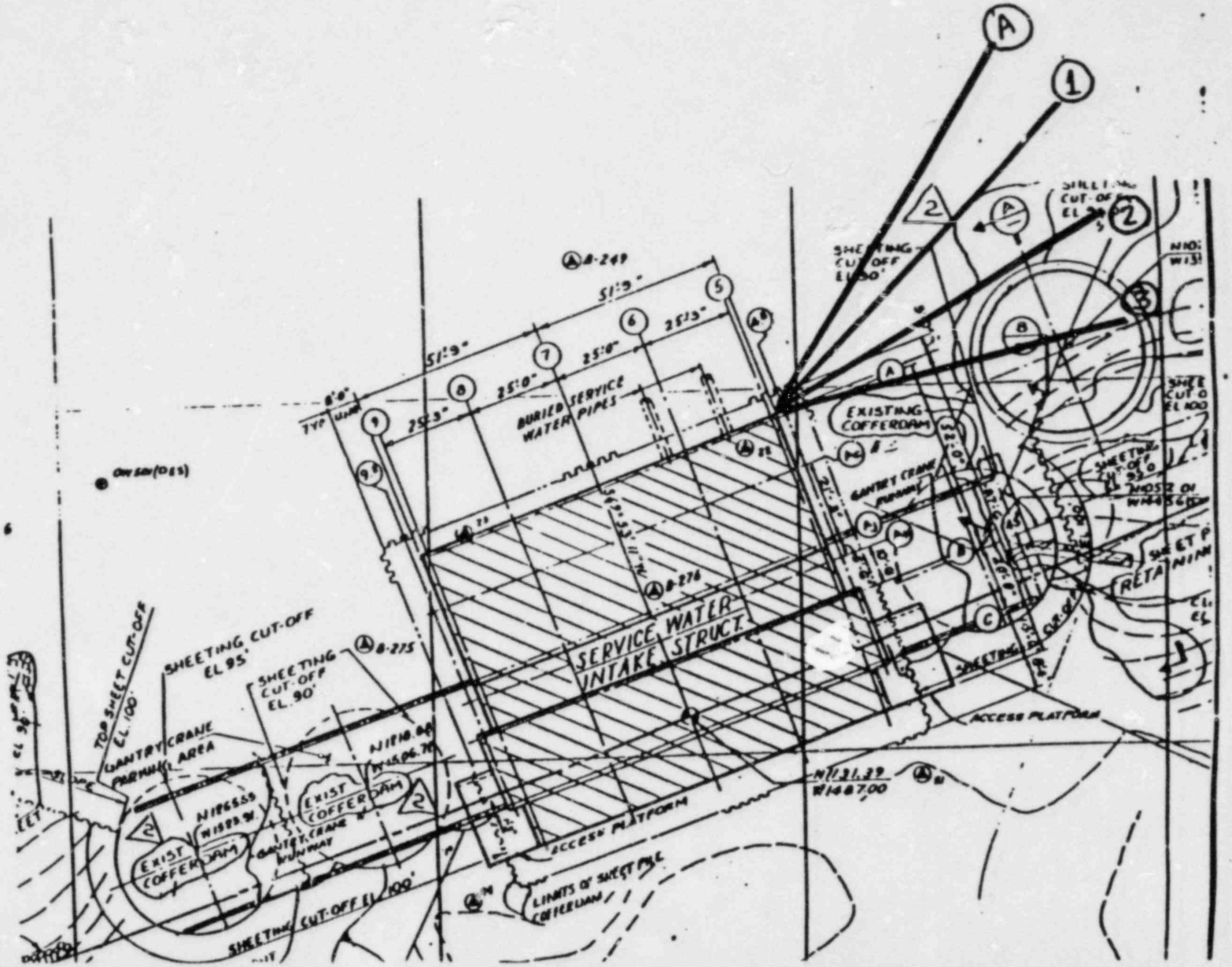


August 6, 1984

Hope Creek Generating Station

Analysis of Overtopping of Service Water Intake Structure

I. **Wave Calculations**

- **Wave heights and periods as well as still-water levels and runup elevations are as given in Table 2.4-10a of FSAR (Amendment 5, April 1984).**

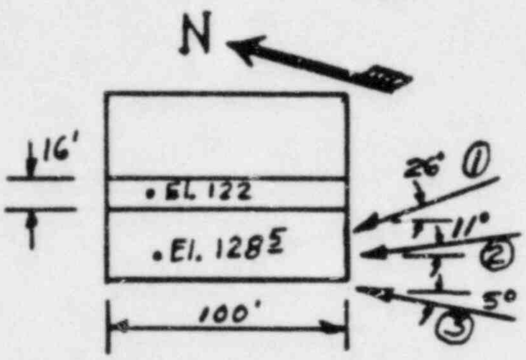
II. Overtopping Calculations

- Overtopping rates were calculated for west face and south face where top of wall elevations are 128.5 and 122.0, respectively.

- Equations from Weggel (1976) were used for the overtopping calculations.

$$Q = \left(g\, Q_o^{*}\, H_o'^{3}\right) \exp\left(-\frac{0.217}{2\alpha} \log_e\left(\frac{R+h-d_s}{R-h+d_s}\right)\right)$$

$$Q_o^{*} = \frac{\left(\frac{\epsilon}{2\pi}\right)^2 \left(\frac{H}{H_o'}\right)^2 \tanh\left(\frac{2\pi d_s}{L}\right)}{\frac{H_o'}{gT^2}}$$



- where ϵ was taken as $1/2\pi$ in order to maximize the value of Q_o^* (see Figure 6 of Weggel's paper)

- α was taken as 0.06 in order to maximize Q (see Equation 4 of Weggel's paper).

- Conservative assumptions in calculating overtopping rates were:

  - It was assumed that waves attacked normal to the wall of the structure.

  - It was assumed that the train of waves was made up of all 1% waves.

  - It was assumed that wave height was constant along the crest.

- Calculated overtopping rate was increased to allow for wind speed using Equation (7-11) of the 1977 edition of the U. S. Army Corps of Engineers Shore Protection Manual.

$$K' = 1.0 + W_f\left(\frac{h-d_s}{R} + 0.1\right)\sin\theta$$

- In making the wind adjustment the factor W_f was assumed to be 2.0 for onshore winds greater than 60 mph. The angle θ was 90°.

o After adjustment for wind the overtopping rates were adjusted for angle of attack by multiplying the overtopping rate by the sin of the angle between the fetch vector and the wall.

III. Maximum water surface elevations were calculated by backwater calculation starting from the north end of the roof.

o The seperate overtopping rates were added and the total was assumed to flow off the top of the structure at the north end.

o Critical depth was assumed to occur at the downstream end of the channel and was calculated as:

$$y_c = \left[\frac{(Q_{TOT}/16)^2}{32.2}\right]^{1/3}$$

where Q is the rate of flow from the west side in cfs/ft.

o The backwater calculation assumes a gradually varied steady flow.

$$y_{x+\Delta x} = \sqrt{\frac{2\Delta Q \cdot \Delta x \cdot Q_x}{16 \cdot 32.2 \cdot y_x} + y_x^2}$$

o Calculations were performed moving upstream starting with the depth at the north end.

o The calculations showed that fetch 3 was the critical case. The total flow rate for fetch 3 was 0.5 cfs/ft from the west and 14.7 cfs/ft from the south end.

o The maximum water surface elevation reached was 126.9 for the fetch 3 condition which is well below the critical 128.5 elevation at which flow could enter the air intakes.

IV. A separate calculation was made considering a surge generated by flow coming over the south end of the building. The depth of flow and velocity of flow ahead of the surge resulting from the previous surge had to be assumed. Velocity ahead of the surge was assumed to be zero, since that condition maximizes the surge height. Depth ahead of the surge was assumed to be 1.0' and does not have a really significant effect on the height of the following surge. The resulting elevation of the crest of the generated surge was 126.9 which is below the 128.5 elevation at which water can flow into the air intake.

V. A check was made to see if flow could surge into the air intakes as a result of plunging from the roof at elevation 128.5.

REV. 4

- Loss coefficients of 0.5 at the entrance to the air intake opening and 0.5 at the bend (see attached sketch were assumed).

- Velocity at the edge of the 128.5 elevation roof section was calculated assuming critical depth there and was increased by 50% for reasons of conservancy.

- The velocity approaching the entrance to the air intake chamber was calculated using the energy equation and neglecting losses.

- Losses incurred by turbulence and impact of the jet entering water ponded on top of elevation 122.0 were neglected.

- Headloss through the screens was neglected.

- The maximum elevation achieved was calculated to be 126.3 or well below the 128.5 elevation at which water could flow into the building.

- A separate analysis was made using a one-dimensional momentum approach. The presence of the louver on top of the outer wall was neglected. A velocity of 26 feet per second was assumed to occur over the top of the lower outer wall whose top elevation is at 124.0. This velocity was calculated assuming that the total potential energy in a wave runup to 134.4 would be converted to kinetic energy at elevation 124 without energy loss. The one-dimensional energy analysis, assuming a flow rate of 5.75 cfs/foot indicates that the water surface within the intake could rise to elevation 127.0 which is below the 128.5 elevation at which water could flow into the service-water intake structure. The assumption of a flow rate of 5.75 cfs/foot is very conservative since that is the total overtopping rate from the west side of the structure for the critical fetch conditions assuming the wave strikes normal to the structure wall.

- The total pressure of the air intake fans equals 4.5 inches of water. The maximum elevations of 126.3 feet and 127.0 feet given above result in margins of 2.2 and 1.5 feet respectively with respect to the 128.5 feet elevation at which water could flow into the building. Therefore, there is sufficient margin to accommodate a rise in water level due to fan suction pressure.

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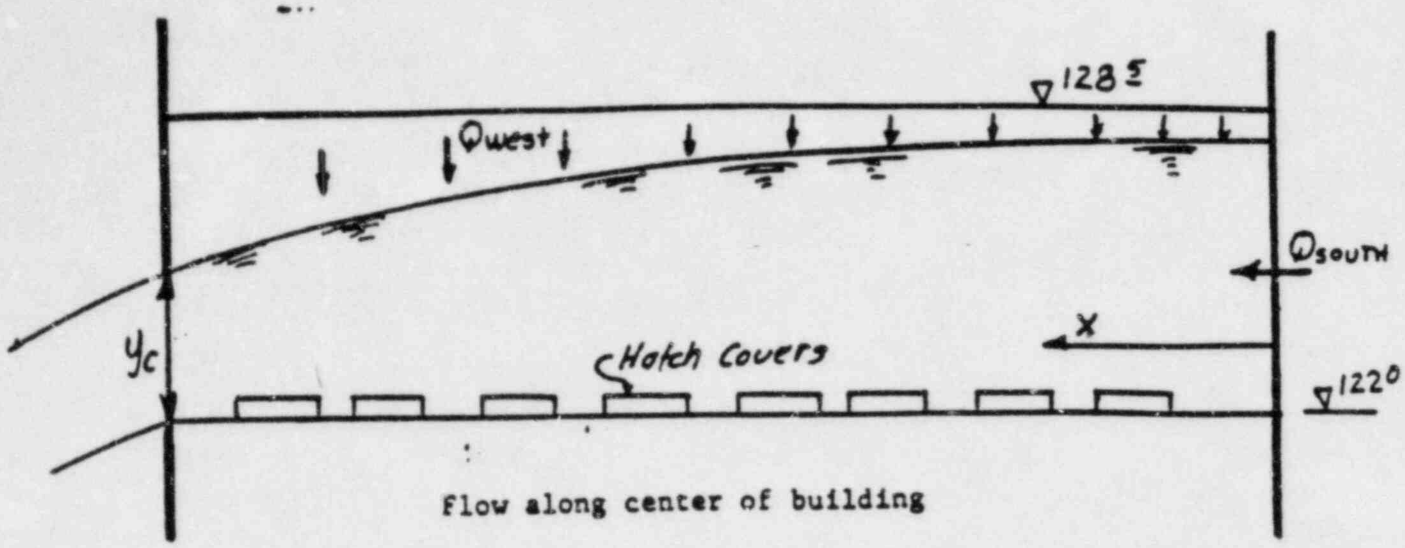
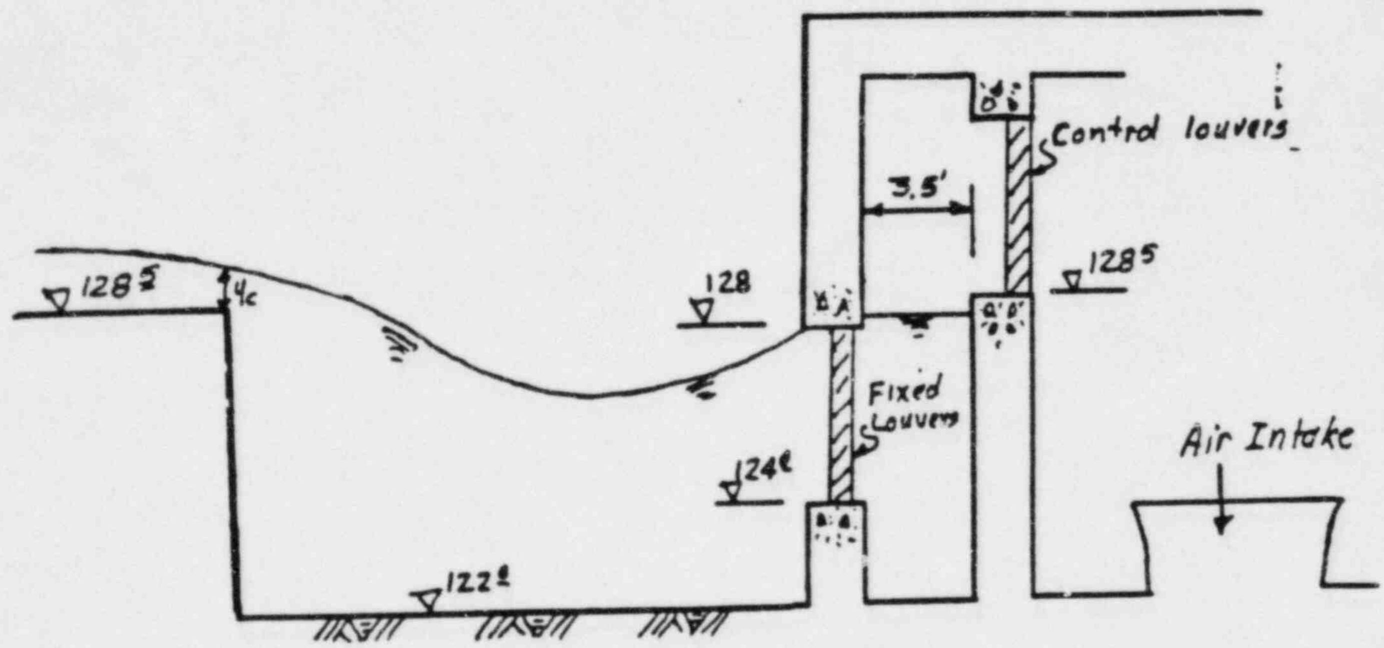


Flow along center of building



Sketch of flow conditions at entrance to air intakes

REV. 4

References

1. Weggel, J. R., "Wave Overtopping Equation" Proceedings of the 1976 Coastal Engineering Conference.

2. Jackowski, R. A. (Editor) Shore Protection Manual, U. S. Army Corps of Engineers, Coastal Engineering Research Center, 1977.

Rev. 4

Calculation Summary
Runup on the East Face of the
Service Water Intake Structure
Hope Creek Generating Station

The attached Figure 1 shows the fetches considered for wave runup on the service water intake structure (SWIS). Fetch A, which has an azimuth of 119°, is 4800 feet long over the island and passes between the Salem Plant and the Hope Creek Generating Station. The wave front from Fetch A approaches the east wall of the service water intake structure at an oblique angle equal to 55° (see Figure 1).

Under design conditions, hurricane generated waves approaching the SWIS would be tripped by passage over the dike at the edge of the island. The top of this dike is at elevation 108 feet (PSE&G Datum).

Incident wave heights, wave lengths, and still water levels are assumed as given in Table 2.4-10A of the FSAR. For Fetch A conditions, we have assumed that the incident wave characteristics, still water level, and wind speed are the same as for Fetch 1. Thus, the incident wave has a significant wave height of 10.8 feet, period of 6.4 seconds, and a length of 180 feet. The corresponding wind speed is 108.6 mph and the still water level is 112.1 feet (PSE&G Datum). The ground elevation of the island is 101 feet (PSE&G Datum), which makes the water depth equal to 11.1 feet (112.1 - 101.0 feet).

Because the dike at the edge of the island would trip all large waves and because the water depth is shallow over the island, it is reasonable to assume that the wave approaching the SWIS along Fetch A would have a significant height equal to the one generated by a 108.6 mph wind over an unlimited fetch and for a water depth of 11.1 feet. Thus, the significant wave height at the east wall of the SWIS would be 4.7 feet according to Figure 3-21 of Reference 1. The one percent wave height is 7.05 feet (1.5 * 4.7 feet). The ratio of maximum (1%) waves to the significant wave height is taken to be 1.5 and was obtained from Reference 2, for shallow water wave generation approaching steady state conditions, including a 30% increase to account for data scatter.

To determine the runup of this wave on the east wall of the SWIS, a wave runup coefficient of 2.0 was chosen in accordance with the results presented in Reference 3 and shown in Figure 2, for a wave approach normal to the structure. This runup coefficient was further modified, taking into consideration the oblique wave approach for the wave propagation along Fetch A. For a wave approach angle of 55°, a wave runup reduction of 30% was estimated based on the results presented in Reference 4 (see Figure 3). This reference was cited by Mr. John Ahrens of the Coastal Engineering Research Center, U.S. Army Corps of Engineers as applicable to the conditions under investigation (Reference 5).

Thus, the 1% wave runup would be 9.87 feet (2.0 * 0.70 * 7.05 feet) and the runup elevation would be 121.97 feet (PSE&G Datum) (112.1 + 9.87 feet).

REV. 4

REFERENCES

1. U. S. Army Corps of Engineers, Shore Protection Manual. Coastal Engineering Research Center, Fort Belvoir, Virginia, 3rd Edition, 1977.

2. Bretschneider, C. L., "Field Investigation of Wave Energy Loss of Shallow Water Ocean Waves", Technical Memorandum No. 46 , Beach Erosion Board, U.S. Army Corps of Engineers, September 1954.

3. Losada, M. A., and L. A. Gimenez - Curto, "Mound Breakwaters Under Wave Attack", Proceedings of the International Seminar on Criteria For Design and Construction of Breakwaters and Coastal Structures, Department of the Oceanographical and Ports Engineering of the University of Santander, Spain, 1980, p. 127-238.

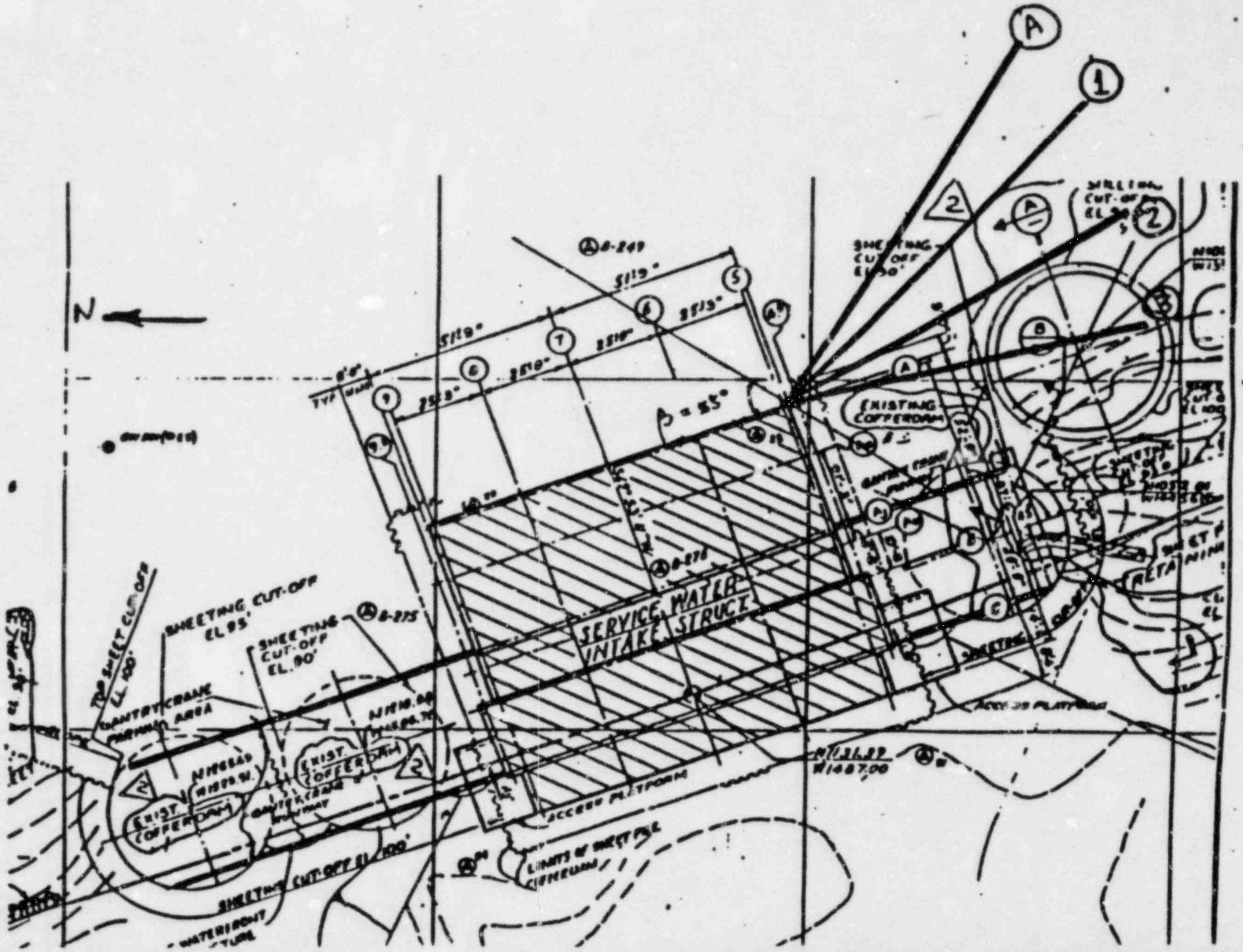
4. Tautenhain, E., S. Kohlhase and H. W. Partenscky, "Wave Run-up at Sea Dike Under Oblique Wave Approach", Proceedings of the Eighteenth Coastal Engineering Conference, Volume I, November 14 to 19, 1982, Cape Town, Republic of South Africa, published by the American Society of Civil Engineers, New York.

5. Personal Communication between J. P. Ahrens of U.S. Army Corps of Engineers, Coastal Engineering Research Center and S. L. Hui of Bechtel Civil and Minerals Incorporated, dated October 9, 1984.

Rev. 4

Fig. 1 Fetches for Service Water Intake Structure, Hope Creek Generating Station

(Revised)
Rev. 4



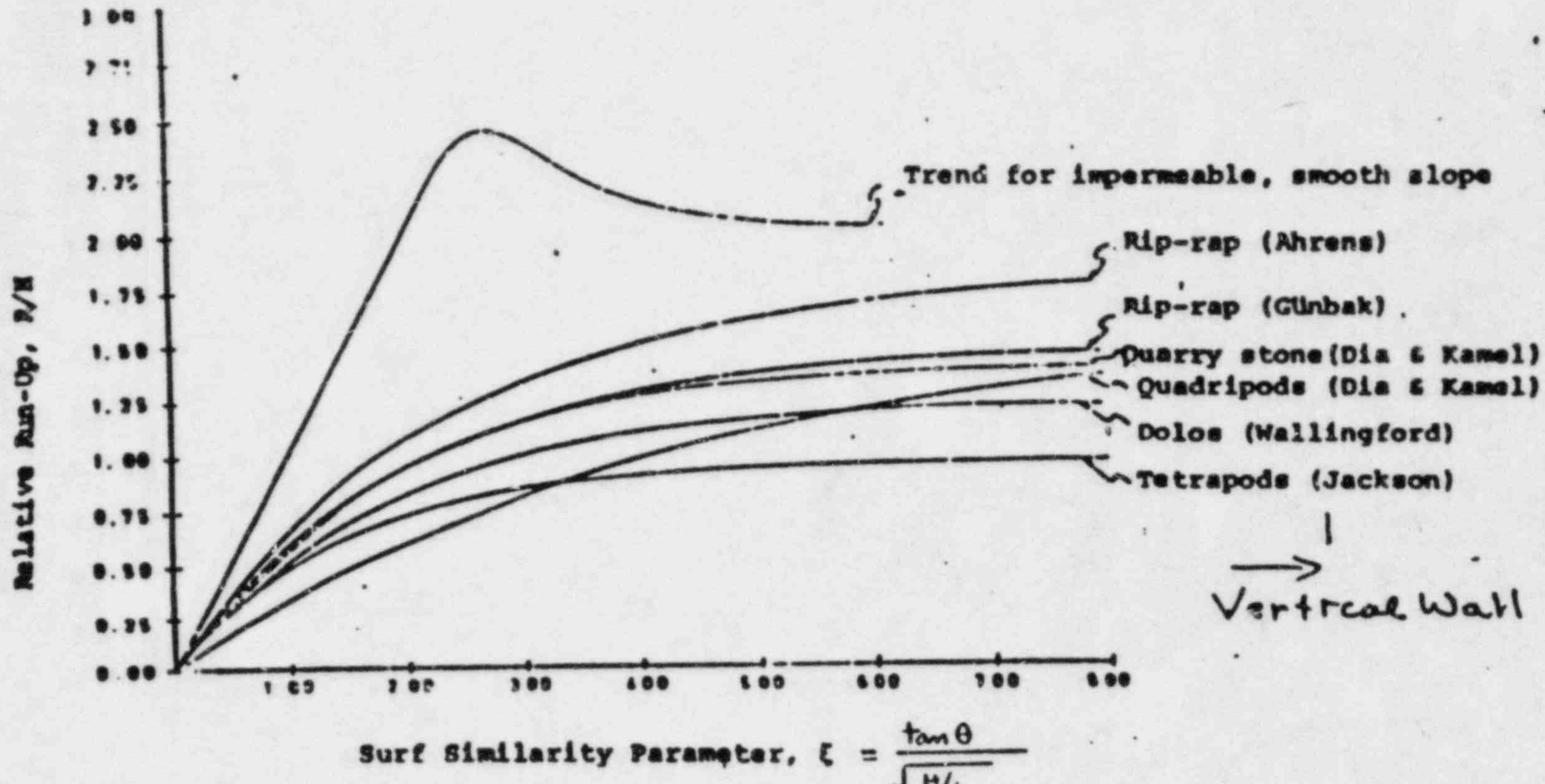


Fig..2 Relative Run-Up Versus ξ For Various Breakwater Armor Units. (From Losada and Gimenez-Curto, 1980).

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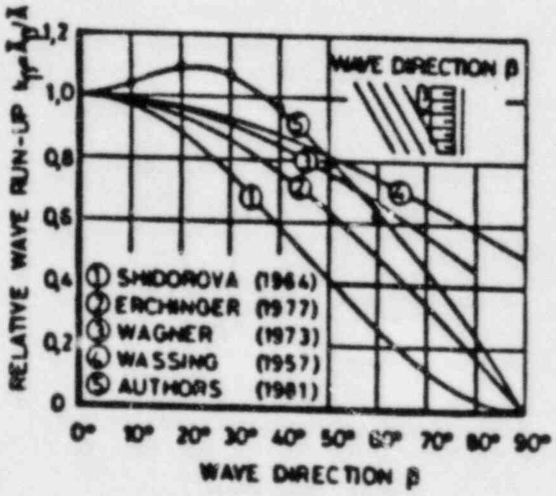

Fig. 3 Relative Wave Run-up For Wave Direction β (From Tautenhain, et al, 1982)

REV. 4