BOSTON EDISON COMPANY
GENERAL OFFICES 800 BOYLSTON STREET
BOSTON, MASSACHUSETTS 02199

G. CARL ANDOGNINI
SUPERINTENDENT
NUCLEAR OPERATIONS DEPARTMENT

September 28, 1979

BECo. Ltr. #79-199

Mr. Thomas A. Ippolito, Chief
Operating Reactors Branch #3
Division of Operating Reactors
Office of Nuclear Reactor Regulation
U.S. Nuclear Regulatory Commission
Washington, D. C. 20555

License No. DPR-35
Docket No. 50-293

Information on the Pilgrim Station Breakwater

Dear Sir:

This letter is in response to your letter dated July 31, 1979, in which you requested further information regarding the design and construction of the breakwater at Pilgrim Nuclear Power Station. Our responses to your specific questions are provided as an attachment to this letter.

Should you have any further questions or concerns on the adequacy of the breakwater design, please contact us at your convenience.

Very truly yours,

Attachments

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ENCLOSURE I OF RESPONSE TO NRC LETTER
DATED JULY 31, 1979

a. PHOTOGRAPHIC DOCUMENTATION OF THE EXISTING AND DAMAGED BREAKWATER

Response

Photographs of the 1978 damaged areas are attached as Appendix A. The descriptions of where these photographs were taken are as follows:

Photo #	Description
1	Main breakwater looking north from shorefront area
2, 3, 4	Looking south from seaward side of Main breakwater at larger damaged area.
5	Looking east at smaller damaged area.
6	Looking west at smaller damaged area.
7, 8	Looking south at smaller damaged area.

Photographic evidence of the 1979 damage is not available.

1145 218

b. DETAILED DESCRIPTIONS OF THE DAMAGE WHICH OCCURRED IN THE WINTERS OF 1978 and 1979:

Response

During the 1978 winter storm the main breakwater was damaged in two areas. The easternmost area is approximately seven hundred feet from the center of the east wall of the discharge canal. The width of the area of major rock displacement varied from 12 feet on the seaward side to approximately 40 feet on the leeward side. The width of even slight displacement was about 75 feet on the leeward side. The damage extended to a depth of approximately 10 feet.

The second damaged area resulting from the 1978 storm is approximately four hundred and fifty feet from the center of the east wall of the discharge canal. The total width of the damaged area including the slightly displaced stones was 25 feet. The area of major rock displacement was approximately six feet wide extending approximately one-half the full width of the breakwater, measured from its south side. The break was approximately five feet deep. There was no displacement of armor on the seaward side.

The damaged area which occurred in the winter of 1979 was approximately six hundred feet from the center of the east wall of the discharge canal. This area was sixty-five feet wide including the slightly displaced stones. The area of major rack displacement extended roughly two-thirds of the full width of the breakwater measured from its lee side. The break was approximately four feet deep. The entire seaward side experienced no damage.

All afore mentioned widths were measured along the length of the breakwater.

Due to the difficulty in establishing a precise measurement of these areas the preceding descriptions are based upon a visual assessment.

c. "AS-BUILT" DRAWINGS OF THE BREAKWATER

Response

The following drawings are attached as Appendix B: 6498-C-401, 405, 406 and 416 through C-422. These drawings reflect the as-built conditions.

d. THE RESULTS OF FIELD MEASUREMENTS AND WEIGHINGS TO DETERMINE AVERAGE ROCK SIZE.

Response

The project files have [illegible]n searched and it was determined that this information is not ava[illegible]able.

1145 219

e ADDITIONAL DISCUSSION AND EVALUATIONS OF THE MODEL STUDIES TO SUPPORT THE ADEQUACY OF THE PROTOTYPE BREAKWATER DESIGNS

Response

The two purposes of the Pilgrim Station breakwater are to provide a measure of protection for the plant cooling water intake structure and shorefront and to reduce the rate of sand deposition in the dredged intake channel. Both of these are accomplished by a breakwater which, by design, may be overtopped under severe storm conditions.

To assist with the breakwater design and direct the overall effort Boston Edison agreed to Bechtel's suggestion that Mr. R. O. Eaton be hired as a consultant. Becuase there were few criteria on which to design an overtopped breakwater, it was recognized that a model test program would be of great value in the design effort and arrangements were made with the University of California and Professor J. W. Johnson to provide this service.

Earlier model tests had established the general configuration of the shorefront and the protective breakwaters, so the new tests were expected to provide more specific criteria on which the breakwater armor should be designed. It was recognized, however, that the tests would not provide all the answers and that considerable judgment would be required in the final design.

Because the differences between the manner in which waves "attack" the trunk and the head of a rubble mound breakwater, it was decided that the tests should be done in two parts The first was a test of the typical trunk cross section, in a wave flume or channel, with the wave approaching perpendicular to the breakwater axis. The second was a test of the head of the breakwater performed in a basin in which the angle of attack with respect to the breakwater axis could be varied. These tests were conducted at the University of California; in the first instance on the Berkeley campus and in the second instance at the U.C. Richmond California Field Station.

Breakwaters are often designed on the basis of an "allowable damage" stability criteria. A cover layer displacement of 6% was chosen as being consistent with the function of the breakwater, so this became the design standard. The tests are fully described and documented in the attached report entitled "A Model Study for Design of Armor for the Pilgrim Station Breakwaters" (Appendix C)

It is not practical to discuss the complete model test program herein or to go into the test conclusions and recommendations fully but the following summary can be made. For a complete discussion, please see the report.

1. For the trunk section of 30' height, a stable configuration could be achieved using a 12 and 8 ton nominal armor with 2:1 seaward and leeward slopes. (Nominal weight by definition corresponds to minimum weight -- 50% of the stones were expected to exceed the minimum by 25% or more).

1145 220

2. For the head section of approximately 33' height, a stable configuration could be achieved using 18 and 10 ton armor (nominal) and 3:1 slopes.

3. For the adopted cross section of +16' MLW elevation and 16' crest width, the maximum damage appeared to take place when the still water level was approximately 2' below the crest

4. Maximum damage to the face of the structure appeared to take place with waves breaking about seven wave heights in front of the structure, while maximum damage to the lee side occurred when the wave broke approximately two wave heights in front.

5. Placement of the stone was found to affect the stability greatly, with careful placement and full support assuring a more stable cross section.

6. The lee slope was found to be as important as the sea slope in assuring stability.

7. The breakwater should be expected to sustain some damage and settlement to a more stable cross section under storm action in the first years of service.

f. A SUMMARY REPORT COMPARING THE MODEL TESTING TO THE PROTOTYPE DESIGN AND TO THE CONSTRUCTED BREAKWATER.

Response

A design following the criteria developed from the model test was prepared, and contract drawings were issued for bid. A contract was ultimately signed with Perini Corporation for accomplishing the work. Two or three different quarries were opened to provide rock for the work. It became evident early on that **all of the** stone of the sizes specified could not be obtained from the sources being worked. This problem lead to the necessity of changing the breakwater configuration to reduce the total armor requirement.

A study showed that it would be possible to provide adequate shorefront protection with a shorter main breakwater. The main breakwater was shortened approximately 500' and the east breakwater about 180'. This shortening of the main breakwater moved the head of the structure away from an area where soil borings had identified some foundation material of poor quality. It also made possible the redesign of the end of the main breakwater and the maximum section of the breakwater trunk because these were now in shallower water.

The redesign required a certain amount of judgment and experience which was provided primarily by Mr. Eaton. The design configuration of the main breakwater trunk was changed to show a two-layer capstone course carried over the top of the breakwater with an underlayer below, rather than the combination of capstone, drystone, and wetstone previously shown The head of the breakwater was also modified, and some slopes were steepened to reduce total volume.

1145 221

To check the design being developed, some effort was made to correlate the model test work with the Hudson formula for breakwater rock sizing, and tentative Kd values for the trunk and head of the breakwater were determined. This work is described in a paper entitled "Armor Stability of Overtopped Breakwaters", ASCE Journal of the Waterways Harbors and Coastal Engineering Division, May 1971.

Work on the breakwaters commenced in the spring of 1969. It became apparent as work progressed that the bottom elevation on the sea side between Station 13 and Station 16 was deeper than the contours on the contract drawings indicated. The difference was not great, being from 1' to 2', but it was decided that the armor stone in this area should be increased in size over that shown on the contract drawing. Accordingly, the armor stone from Station 13 to Station 14 was increased from 6 to 8 ton nominal and that from 14 to 16 was increased from 8 to 12 ton nominal.

g. AN EVALUATION, WITH BASES, FOR WHY THE BREAKWATER FAILED

Response

The breakwater was completed in 1970. It has been exposed to nine years of winter storms. Because of the very regular construction, any stone displacement is readily apparent. Prior to 1978, there was only very minor movement of armor stones, which was to be expected. The February 1978 storm, however, did cause measurable damage which exceeded that of the earlier years. It was restricted to two areas and was not indicative of a general failure of the structure. The 1978 storm has been characterized as exceeding the expected 100 year storm and the storm surge and observed still water elevation exceeded those of over 200 years of records in that area.

The damage is described in more detail elsewhere in this report. The failure mechanism is not absolutely clear, but the photographs seem to indicate that a number of capstones were lifted or pushed off the leeward crest of the breakwater and rolled down the leeward slope. This apparently exposed some of the underlayers where additional stones were dislodged leading to further unravelling.

It must be emphasized that the failure was local, and not indicative of a general problem. There is no indication of foundation failure. The fact that the breakwater generally performed as well as it did when subjected to the greatest storm it may face in the useful life of the power plant it is protecting confirms that the general design concept is sound and that the cause of the problem is local in origin. Three possible causes might be considered:

1. The leeside capstones which were apparently displaced may be at the lower end of the weight range specified. The model test did indicate that the leeside armor was potentially an area subject to possible damage. The lee slope in the area in question is 1 ½:1 and a concentration of smaller armorstones in one area could cause a weaker section.

1145 222

2. The leeside capstones might not have been well supported. The model test showed that armor placement was particularly important, with the need for careful positioning of the stones to assure firm three-point support. The type and shape of the armor stone being quarried made this extremely difficult. The stone comes in slab-like peices which are difficult to place in a random interlocking pattern duplicating that of the model test. As a result, some individual capstones, due to their flatter orientation, may have been subjected to wave pressures on a relatively large projected area and have been subject to less restraint from the supporting stones than the design anticipated.

3. Local areas of the breakwater may have slopes which are somewhat steeper than the design requirement. The results of the survey (Appendix D) in which cross sections were taken of the breakwater from Station 14 to Station 19 have been compared with as-built cross sections taken soon after the breakwater was constructed. Considering the fact that the cross sections were taken at slightly different stations and that the breakwater is an irregular structure, the correlation between cross sections is very good with no settlement evident in the crest. The survey also establishes that there has been little change in the bottom contours immediately seaward of this area. There is some evidence, however, that the seaside breakwater slope in some areas is somewhat steeper than constructed. This may be due to consolidation of the smaller material at the toe of the slope, leading to some settlement of the armor stones at the bottom of the slope.

h. DESCRIPTIONS OF PROPOSED SURVEILLANCE AND REPAIR PROCEDURES TO BE IMPLEMENTED FOR FUTURE MAINTENANCE OF THE BREAKWATER

Response

A detailed survey and inspection of the breakwater, particularly the sections of past instability, will be conducted. Sizes of all capstones will be estimated and an assessment will be made of the adequacy of the support provided to the capstones by the surrounding armor. The slopes of the breakwater will be closely checked. Generally speaking, the breakwater will be checked for the three items outlined in the previous section.

If any one of the outlined conditions is obviously present or a combination of the conditions which would suggest possible instability of the section is found, these deviations will be evaluated and an appropriate plan of action will be formulated and enacted to correct the problem.

After the structure has been fully checked and repaired, if necessary, it will be visually surveyed on an annual basis and after any major storm for possible damage. Significant damage exceeding the 6% damage criteria will be repaired as soon as weather and procurement processes will permit. Lesser problems will be evaluated to see if repairs are needed or if they can be delayed until it is practical to mobilize the repair forces.

1145 223

After the detailed survey is made and any repairs are effected, it is expected that annual stone displacement will be minimal. It will not be necessary to make immediate repairs to the structure as a rule, because the breakwater will perform its function even under conditions of extreme damage. Such damage would not take place during a single storm but would be spread over a number of them, permitting repairs to be made if necessary.

i. CROSS SECTIONS OF THE BREAKWATER ESTABLISHED BY SURVEY AT THE TWO LOCATIONS OF PAST INSTABILITY AND FOR A DISTANCE OF AT LEAST 100 FEET ON EITHER SIDE OF THE PROBLEM AREAS. THE SURVEY SHOULD EXTEND AT LEAST 100 FEET BEYOND THE TOES OF THE BREAKWATER SECTION TO ESTABLISH THE EXISTING GROUND PROFILE AND DEPTH OF WATER.

Response

These cross sections and a topographic map of the breakwater and surrounding area are attached as Appendix D.

j. AREA AND DEPTH LIMITS WHERE UNSUITABLE MATERIALS WERE REMOVED WITHIN THE BREAKWATER FOUNDATION ON THE PREVIOUSLY REQUESTED AS BUILT DRAWINGS

Response

No unsuitable material was removed within the breakwater foundation area. The soil exploration program showed no evidence of unsuitable material in the areas in which the structures were placed. There was some indication of poor quality material near the head of the breakwater as originally designed and some plans were formulated for removing and replacing this material. However, the redesigned structure avoided these areas as described in the beginning of Section f. and no removal of foundation material was necessary.

k. ALL BORING LOGS OF OFFSHORE EXPLORATIONS NEAR THE BREAKWATER ALIGNMENT

Response

These boring logs and location drawings are attached as Appendix E.

1145 224

CONCLUSIONS

1. Since the primary function of the breakwater is to dampen wave energy the determination of effectiveness of the breakwater to perform in its intended capacity should be based on the overall breakwater integrity rather than stability of individual armor units.

2. The winter sotrm of February 1978, which caused the most damage to the breakwater is described as the worst winter storm on record. This storm accompanied by spring high tides created water level, wave and wind situations whose likely reoccurrence would be very highly unlikely.

3. Based on our research of this subject including all the items contained in this response, it is apparent to BECo that the Pilgrim Station Unit I main breakwater has more than satisfactorily performed its intended function.

1145 225