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25 May 1967



Dr. Peter A. Morris, Director
Division of Reactor Licensing
U.S. Atomic Energy Commission
Washing, D.C. 20545

Re: Transmittal of Drafts of Reports
Duke Power Company, Dockets 50-269, 50-270, and 50-287
Vermont Yankee Nuclear Power Corporation, Docket 50-271

Dear Dr. Morris:

Transmitted herewith are reports on the above-mentioned applications by Dr. W. J. Hall and myself. These are drafts, and they are submitted for the use of your staff. If there are questions regarding either of these, we shall be glad to attempt to answer them.

Sincerely yours,


N. M. Newmark

bd
cc: Dr. W. J. Hall

21
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DRAFT

Report to AEC Regulatory Staff



ADEQUACY OF THE STRUCTURAL CRITERIA FOR THE
OCONEE NUCLEAR STATION UNITS 1, 2, AND 3

DUKE POWER COMPANY
(Dockets 50-269, 50-270, and 50-287)

by

N. M. Newmark and W. J. Hall

16 May 1967

21

ADEQUACY OF THE STRUCTURAL CRITERIA FOR THE
OCONEE NUCLEAR STATION UNITS 1, 2, AND 3

by

N. M. Newmark and W. J. Hall

INTRODUCTION

This report concerns the adequacy of the containment structures, components, and dams for the three units of 2452 MWt each (874 MWe, net) for which application for a construction permit and operating license has been made to the U.S. Atomic Energy Commission (Dockets No. 50-269, 50-270, and 50-287) by the Duke Power Company. The facility is to be located on the shore of future Lake Keowee in Oconee County, South Carolina, 8 miles NE of Seneca, South Carolina.

The report is concerned specifically with the evaluation of the design criteria that determine the ability of the containment system to withstand a design earthquake acting simultaneously with other applicable loads forming the basis of the containment design. The facility also is to be designed to withstand a maximum earthquake simultaneously with other applicable loads to the extent of insuring safe shutdown as well as containment. The seismic design criteria for Class I equipment and piping are also reviewed herein, along with a review of the analyses of the dams which are required for impounding the required cooling water supplies. This report is based on information and criteria set forth in the Preliminary Safety Analysis Report (PSAR) and Supplements thereto as listed at the end of this report. We have participated in discussions with the AEC regulatory staff, in which many of the design criteria were discussed in detail.

DESCRIPTION OF THE FACILITY

Oconee Nuclear Station Units 1, 2, and 3 are described in the PSAR as pressurized water reactors for which the nuclear steam system and fuel cores are to be supplied by the Babcock and Wilcox Company, each designed for a power output of 874 MWe (net). The reactor coolant system for each unit consists of two closed reactor coolant loops connected in parallel to the reactor vessel, each provided with reactor coolant pumps and a steam generator. The reactor vessel will have an inside diameter of about 14 ft-3 in., a height of about 41 ft-9 in., and is designed for an internal pressure of 2500 psig, a temperature of 650°F, and is made of SA-302 Grade B steel clad with Type 304 austenitic stainless steel.

Each of the reactor units is contained in a fully reinforced concrete structure in the shape of a cylinder with a shallow domed roof and a flat foundation slab. The cylindrical portion is prestressed by a post-tensioning system consisting of horizontal and vertical tendons. The dome has a three-way post-tensioning system. The flat foundation slab is conventionally reinforced with high-strength reinforcing steel, and the entire structure is lined with a 1/4 in. welded steel plate. The cylindrical part of each of the containment structures is approximately 116 ft inside diameter, has an inside height of 216 ft, vertical wall thickness of 3 ft-9 in., and a dome thickness of about 3 ft-3 in. The foundation slab is about 8 1/2 ft thick.

The PSAR on page 5-1 of Vol. I indicates that the design will in many respects be similar to that for the Florida Power and Light Company's Turkey Point Plant, Consumer Power Company's Palisades Plant, and Wisconsin-Michigan Power Company's Point Beach Plant. Although no stated details are

given, we assume, then, that the cylindrical wall is to be provided with a system of hoop tendons which are placed in a 3-120° system using six buttresses as anchorages with the tendons staggered so that half of the tendons at each buttress terminate at that buttress. In Appendix 5B it is noted that the prestressing will be post-tensioned, and unbonded, with the tendons encased in rigid steel conduit and corrosion protection provided by grease injected into the conduit under pressure.

In Appendix 5E it is noted that the welded steel liner will be at least 1/4 in. thick and made up of ASTM A-442 steel with anchors, the spacing of which is not readily identified. It is noted that the liner plate will be thickened in the vicinity of penetrations.

As of this date it appears that several types of prestressing systems are being considered for the post-tensioning of the containment structure.

Appendix 5B indicates that ASTM A-432 reinforcing steel will be used in the base slab, and that ASTM A-15 deformed bars will be employed in the cylinder wall, the domed roof, and around the openings to control shrinkage and tensile cracks. It is further noted in Appendix 5D that for large 14S and 18S reinforcing steel, Cadweld splices will be employed, and the Errata filed with Amendment 3 indicate that the tensile strength of the splices will equal or exceed 125 percent of the minimum yield strength of each grade of reinforcing steel as specified in the appropriate ASTM standard. We recommend that tack welding or other welding not be permitted for the A-432 bars in the foundation slab or elsewhere, to avoid the possibility of fracture or other difficulties in achieving the required ductility of these reinforcing bars.

The geology is summarized in Appendices 2A and 2E; on page 2-9 of Vol. I of the PSAR it is stated that the structure will be founded on the normal Piedmont granite gneisses.

SOURCES OF STRESSES IN CONTAINMENT
STRUCTURE AND TYPE I COMPONENTS

The containment structure is to be designed for the following loads: dead load of the structure; live loads (including roof loads, pipe forces, and reactor service crane loads); accident pressure load associated with loss-of-coolant accident, of 55 psig; test pressure of 63.3 psig; and external-internal pressure differential of about 5 psig, based on information presented in answer to Question 8.2 of Supplement 2, corresponding to a drop of barometric pressure associated with a tornado ranging between 225 and 600 mph as well as wind loading corresponding to 95 mph at 30 ft.

On the basis of the information presented on page 5-5 of Vol. I of the PSAR, Appendix 5B, page 5B-4, and the answer to Question 8.5 of Supplement 1, and in accord with the USC&GS report (Ref. 3), it is our understanding that the design earthquake will be characterized by a maximum horizontal ground acceleration of 0.05g and the maximum earthquake by a 0.10g horizontal ground acceleration. The structure is to be founded on firm basement rock.

COMMENTS ON ADEQUACY OF DESIGN

Seismic Design -- In connection with the selection of the design earthquake and the maximum earthquake, we agree with the values selected, and concurred in by the USC&GS, namely that of a basic design for a design earthquake of 0.05g and design for a maximum earthquake of 0.10g maximum horizontal ground acceleration.

On page 5B-4 of Appendix 5B, for the design earthquake of 0.05g, it is indicated that the horizontal and vertical acceleration will be taken as equal in intensity. We find no mention of this fact for the maximum earthquake but assume that the same situation will obtain there, and assuming that this is the case, we concur in this approach.

The proposed response spectra for various degrees of damping for the maximum earthquake are presented in answer to Question 8.5 of Supplement 1, and for the design earthquake as part of Appendix 2B. On the basis of the information presented in Appendix 2B and in answer to Question 8.5 of Supplement 1, we fail to find any rationale for the selection of the spectra that are presented therein. We find no satisfactory explanation for the basis of the selection of the ground motions, other than for the acceleration values which have already been agreed upon and which control in the high frequency band; moreover we find no rationale for the amplifications of the ground motions which lead to the design spectrum values. On the basis of a recent telephone conversation between Mr. J. Fischer of Dames and Moore, DRL staff, and ourselves we understand that properly scaled spectra corresponding to those presented in Report TID 7024 will be employed in the design.

The damping values to be employed are listed in answer to Question 8.4 of Supplement 1. We are in agreement with the damping values given therein with the further understanding, however, that the 5 percent damping value to be used for the maximum earthquake will be employed in the design in such a way that there will be a limitation on the deformations of the containment structure and its components. The general dynamic design approach outlined in answer to this same question appears acceptable to us both for the containment structure and for the piping.

The loading combinations for the containment design are presented in Appendix 5A. We are in agreement with the load factor expressions stated there for the case of the design and maximum earthquake. In reply to Question 8.1 of Supplement 1, however, it is noted that "the design criteria which will be applied to the above loading is (sic) that the deformation will be limited to values which will permit a safe and orderly shutdown." This statement provides no guide as to what the limitation on deformations will be, but we note that, in connection with the design of the liner, as described in Appendix 5E, a limitation of 0.5 percent strain has been set for the liner. On the assumption that the design will call for a reasonable limitation on ductility, i.e., on the order of not more than two or three times the gross yield deformation, and further that the liner deformation will be restricted as noted, we believe that the design approach for the maximum earthquake will be satisfactory in this respect.

In Appendix 5A it is noted that the polar crane is a Class I structure, and on the assumption that steps will be taken to insure that these cranes cannot be displaced from the rails during a design or maximum earthquake or otherwise topple to create damage which would prevent safe shutdown or impair the containment, we believe that this aspect of the design can be handled properly to make it satisfactory in all respects.

We have reviewed in some detail the design calculations for the dams as given in Supplement 1, and on the basis of the analyses we believe that the safety of the dams is relatively satisfactory for the 0.1g earthquake on the basis that the dams are founded on basement rock as indicated. We call attention to one minor discrepancy in the method of analysis, based on that by N. M. Newmark (1965) given in the Rankine lecture to the

Institution of Civil Engineers, in which the analyses given in Supplement 1 used static slip circles for the dynamic analysis. In general the slip circles for the dynamic analysis will be different from those for a static analysis, but in this case it appears that the results will be only slightly different, and that safety is achieved. It was noted in one case that one of the slices was on the verge of movement for the 0.1g earthquake; however, investigation reveals that such a slice would move only a small distance, and the function of the dam would probably not be impaired in any serious manner by such a minor slippage, should it occur. In summary, we believe that the dams can withstand with relative safety the maximum earthquake stipulated. In any event, we are advised by DRL staff that a natural pool of water will be provided for emergency cooling in the event of unexpected dam failure.

General Design Considerations -- We have reviewed with care and interest the design criteria for the prestressed concrete reactor building as presented in Appendix 5C, and the elaboration on the development for handling the shear at yield loads as given in answer to Question 8.7 of Supplement 2. We are in agreement with the provisions there for handling principal concrete tension and the new recommendations for handling radial shear. In the event that further data become available on this matter prior to completion of the design stages, we trust that such information can be incorporated into the design, if this appears warranted.

Penetrations -- The design of the penetrations is described briefly in Section 5 of Vol. I of the PSAR, and elaboration is given in answer to Question 8.8 of Supplement 2. On the basis of the discussions presented therein, we concur in the approach that is described for this particular design.

Wind Loadings -- With regard to the tornado wind loadings, further amplification on the provisions presented in Vol. I of the PSAR is presented in answer to Question 8.2 of Supplement 2. It is noted that a design tornado wind of 225 mph has been specified; however, in answer to Question 8.2 noted, it is indicated that "the reactor building will be checked, based on ultimate strength with no loss of function, for a 50 percent increase in design tornado wind and negative pressure." On the assumption that this refers to a tornado wind of about 337 mph and a negative pressure differential of 3 to 5 psi, with limited deformation at the "ultimate strength" noted, we concur in this design approach. It is further noted in that same question that the reactor building probably has an ultimate capacity to withstand a tornado in excess of 600 mph, although the meaning of "ultimate capacity" is not defined in terms of deformation limits, leakage, etc.

Surveillance -- We find little detailed information on the planned surveillance program but recommend strongly that a reasonable and sensible surveillance program be maintained throughout the life of the structure.

Piping and Other Type 1 Components -- We find discussion of the design of the piping presented in answer to Question 8.1 of Supplement 1 which refers to Appendix 5A as appropriate for the class of piping involved, with further amplification on the dynamic design provision as given in answer to Question 8.4. We are in general agreement with the approach proposed therein, but are still not sure exactly how the piping analysis will be carried out in the sense that is implied in the last paragraph on page 8.4-3 (4-1-67), which states that the stresses from the horizontal and vertical components acting simultaneously will be combined with the stresses due to weight, thermal and mechanical loads, and internal pressure, and in turn

these stresses will determine the required yield strength of the piping systems. This does not completely answer the question of what limitations will be placed on the piping in terms of behavior under the maximum earthquake, particularly in terms of limitations on deformation. We recommend, for the specific materials used, that the deformations be limited to reasonable values which will preclude any difficulties with fatigue or fracture. Particular attention should be given to the piping at those places where it penetrates the containment, or to that piping which is required for safe shutdown in this regard. The same provisions apply to piping that will run from the intake structures to the plant and which will be required for safe shutdown in the event of an earthquake or an accident.

Conclusions -- On the basis of the information presented, and in accord with the design goal of providing serviceable structures and components with a reserve of strength and ductility and which will provide for containment and/or safe shutdown, we believe that the design criteria outlined for the containment structures and Type 1 piping can provide an adequate margin of safety for seismic resistance, with the following exceptions. We believe that the applicant must provide more rational and reasonable design spectra for use in the design, and also that further attention needs to be given to the provisions for the piping design.

M. M. [unclear]

REFERENCES

1. "Preliminary Safety Analysis Report--Volumes I and II," Oconee Nuclear Station Units 1, 2, and 3, Duke Power Company, 1966.
2. "Preliminary Safety Analysis Report--Supplements 1, 2, and 3," Oconee Nuclear Station Units 1, 2, and 3, Duke Power Company, 1967.
3. "Report on Seismicity of the Oconee Nuclear Station Units 1, 2, and 3," U.S. Coast & Geodetic Survey, Rockville, Maryland, _____.