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REPORT TO AEC REGULATORY STAFF

ADEQUACY OF THE STRUCTURAL CRITERIA FOR THE DIABLO CANYON SITE NUCLEAR PLANT

Pacific Gas and Electric Company

(Docket 50-275)

by

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and

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INTRODUCTION

This report concerns the adequacy of the containment structures and components, reactor piping and reactor internals, for the Diablo Canyon Site Nuclear Plant, for which application for a construction permit and operating license has been made to the U. S. Atomic Energy Commission (Docket No. 50-275) by the Pacific Gas and Electric Company. The facility is to be located in San Luis Obispo County, California, 12 miles west southwest of the city of San Luis Obispo, and adjacent to the Pacific Ocean and Diablo Canyon Creek. The site is about 190 miles south of San Francisco and 150 miles northwest of Los Angeles.

Specifically this report is concerned with the evaluation of the design criteria that determine the ability of the containment system, piping and reactor internals to withstand a design earthquake acting simultaneously with other applicable loads forming the basis of the 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 and containment. 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, and the applicant and its consultants, in which many of the design criteria were discussed in detail.

DESCRIPTION OF THE FACILITY

The Diablo Canyon Nuclear Plant is described in the PSAR as a pressurized water reactor nuclear steam supply system furnished by the Westinghouse Electric Corporation and designed for an initial power output of 3250 MWt (1060 MWe net). The reactor cooling system consists of four closed reactor coolant loops connected in parallel to the reactor vessel, each provided with a reactor coolant pump and a steam generator. The reactor vessel will have an inside diameter of about 14.5 ft., a height of 42.3 ft., will operate with a design pressure of 2485 psig, a design temperature of 650°F, and is made of SA-302 grade B low alloy steel internally clad with type 304 austenitic stainless steel.

The reactor containment structure which encloses the reactor and steam generators, consists of a steel lined concrete shell in the form of a reinforced concrete vertical cylinder with a flat base and hemispherical dome. The cylindrical structure of 140 ft. inside diameter has side walls rising 142 ft. from the liner at the base to the spring line of the dome. The concrete side walls of the cylinder and the dome will be approximately 3 ft. 6 and 2 ft. 6 in. in thickness, respectively. The concrete reinforcing steel pattern is described conceptually in Supplement 1 and consists of bars oriented at 30° from the vertical in such a manner that the pattern does not require termination of any bars in the dome. These diagonal bars are designed to carry both the lateral shear as well as vertical tensile forces. In addition there is hoop reinforcing in the cylindrical portion of the structure. For radial shear reinforcing the applicant proposes to use a system of vertical wide flange beams spaced four feet on centers. The beams are attached by hinge connection to the base slab at the lower end and are terminated about 20 ft. above the

top of the base slab. The function of the beams is to provide resistance to the moments and shears created by the discontinuity at the base and to provide a gradual transition of load carrying elements between the base and the cylinder wall. These beams do not participate in resisting either uplift due to pressure or shear and tension due to earthquake loading; these forces are to be resisted by the diagonal steel reinforcing just described. The concrete wall in this lower zone is divided into three zones. The inner zone, about 1 ft. thick, consists of reinforced concrete and is the element to which the liner is attached. The middle zone contains the vertical steel I-beams which in turn act as supports for the 16 in. thick reinforced concrete slab spanning the space between the beams. The outer zone consists of about 14 in. of concrete in which the diagonal and hoop reinforcement are embedded. The three zones are provided with bond-breaking material to insure that the elements will act separately. The reinforcing steel for the dome, cylindrical walls and base mat will be high strength reinforcing conforming to the ASTM A432 specification. The A432 reinforcing bars of size larger than No. 11 are to be spliced with Cadweld splices except in cases where accessibility makes welding mandatory.

The liner, as described in Supplement 2, will be a minimum of 3/8 in. thick for the dome and cylindrical walls and 1/4 in. thick for the base slab. The anchor stubs are to be L shaped and will be fusion welded to the liner plate. The studs will be spaced at 20 in. on centers, and the design is made to preclude major affects arising from buckling of the liner.

Personnel and equipment access hatches are provided for access to the containment vessel. In addition there are other penetrations for piping and electrical conduits.

The facility includes a sea water intake structure located at sea level at the base of the cliff with circulating water conduits and auxiliary salt water conduits leading up to the nuclear plant.

The information on the geology at the site is described in the PSAR and the several supplements. The bedrock at the site area is of tertiary age and comprises marine shales, sandstone and fine-grained tuffaceous sediments, along with a considerable variety of tuffs of submarine volcanic origin. All these rocks are firm and compact, and are exposed in the seaward edge of the terrace on which the plant is to be built, which ranges in elevation from 60 to 100 ft. above sea level, and is approximately 1,000 ft. wide. The bedrock is overlain by marine and non-marine deposits of Pleistocene age. The major components of the power plant are to be founded in bedrock in all cases. The site has been well explored and there is no evidence of any fault offsets of recent origin of significance. The report by the consulting geologist on the project, Dr. Richard H. Jahns, presented as Appendix A of the third supplement, concludes that the possibility of fault-induced permanent ground displacement within the plant area during the useful life of the power plant is sufficiently remote to be safely disregarded.

SOURCES OF STRESSES IN CONTAINMENT STRUCTURE AND TYPE I COMPONENTS

The containment structure is to be designed for the following loadings: dead load of the structures; live loads (including construction loads and equipment loads); internal pressure due to a loss-of-coolant accident of about 47 psig; test pressure of 54 psig; negative internal pressure of 3.5 psig; stresses arising from thermal expansion; wind loading corresponding to the Uniform Building Code - 1964 edition and corresponding to 87 to 100 mph winds; and earthquake loading as described next.

The earthquake loading will be based on two earthquakes, which for the design earthquake condition correspond to maximum horizontal ground accelerations of 0.20g and 0.15g. The containment design also will be reviewed for no loss of function using response spectra corresponding to earthquakes of twice the maximum acceleration noted above, namely 0.40g and 0.03g, but with the latter earthquake having a maximum ground velocity corresponding roughly to a value of 0.40g ground acceleration. The U. S. Coast and Geodetic Survey report (Ref. 3) concurs in 0.20g and 0.40g values of maximum ground acceleration for design and maximum conditions.

Class I piping and equipment, as discussed in answer to Question II.G of Supplement 2 will be designed to the USA S.I.B31.1 Code for pressure piping which includes consideration of internal pressure, dead load, and other appropriate loads such as thermal expansion. It does not contain provision for earthquake loading. However, the applicant indicates that they will combine earthquake loadings with the loadings just noted and further elaboration on this point is given in Appendix A of Supplement 1.

The reactor internals are to be designed for combined earthquake, blowdown loadings and other applicable loadings.

COMMENTS ON ADEQUACY OF DESIGN

Seismic Design

For this facility the containment design is to be made for two earthquakes corresponding to maximum horizontal ground accelerations of 0.20g (Earthquake D) and 0.15g (Earthquake B). For the maximum earthquake loading, the two earthquakes are characterized by horizontal ground accelerations of twice the values just cited, namely 0.40g and 0.30g. Spectra corresponding to these earthquakes are presented as Figs. 2-11 through 2-14 of the PSAR and

again in Supplement No. 3 beginning on page 22, along with an envelope of the spectra for the no-loss-of-function condition (Fig. III.A.12-5, Supplement 3). We concur with the response spectra for the earthquakes when they are used in the following manner.

Since the response spectrum values for earthquake D gives values that control for high frequencies, and for earthquake B, values that control for intermediate and low frequencies, both earthquakes must be used and the maximum response in either must be considered to apply to the design for safe shutdown of single degree of freedom elements. This is permissible in view of the fact that earthquake B gives response values for low and intermediate frequencies that lie above the response spectrum values from TID-7024 when normalized to an acceleration of 0.40g. Hence, this earthquake may be considered to correspond to a 0.40g earthquake for low and intermediate frequencies.

However, for safe shutdown of multi-degree-of-freedom system, we take the position that the combined or envelope spectrum for the two earthquakes must be used in order to avoid a possible deficiency in the provision for safe shutdown. This envelope spectrum is consistent with an El Centro type response spectrum for a maximum ground acceleration of 0.40g.

Vertical acceleration values in all cases will be taken as two-thirds the corresponding maximum horizontal ground acceleration, and the effects of the horizontal and vertical earthquake loadings will be combined, and considered to act simultaneously. In addition, in the elastic analysis, the usual fractional increase in stress for short term loading will not be used. We concur in these criteria.

The damping values to be used in the design are given on page 2-29 (revised 7/31/67) of the PSAR and we concur with the values given therein.

With regard to the method of analysis of the containment structure, it is noted on page 2-29 of the PSAR that all modes having a period greater than 0.08 secs. will be included in the analysis and that in addition for components or structures having multiple degrees of freedom, all significant modes, and in no case less than 3 modes, will be considered. It is further stated that for a single degree of freedom systems, the fundamental mode of vibration will be used in the analysis. Our interpretation of these statements is that for a single degree of freedom system, no matter what the period, whether it is above or below 0.08 secs., the appropriate period and spectral acceleration will be employed in the design, and further that for multiple degree of freedom systems, all modes will be considered. On the basis of this interpretation, as interpreted in the second paragraph of this section, we concur with the approach.

The method of dynamic analysis is described in Section 2 and 5 of the PSAR and again in answer to Question III.A.15 of Supplement 1. It is noted that the dynamic analysis to be followed for the Class I components and structures is the modal participation factor method. It is our understanding further that the modal analysis may be carried out either through the use directly of the smoothed spectra, or employing a time history of ground motion, employing earthquake records with amplitude values scaled which lead to essentially the same smoothed spectra. Discussion of this point is presented by the applicant in answer to Question III.A.13 in Supplement 3. We concur in the use of the modal participation method in the analysis and design, as well as the use of either the smoothed spectra or the time history input

method provided that the time history input yields the same response spectra as given in the report without any major deviations below those smoothed response spectrum values presented in the PSAR.

As a further point on the dynamic analysis, it is our understanding that for the safe shutdown conditions particularly, for Class I components and structures, the design will be made for the envelope of the combined spectra of the two earthquakes for the appropriate damping level. On the assumption that this approach is the one being followed, we concur in the design approach adopted.

General Design Provisions

We have reviewed the design stress criteria presented on page 5-9 of the PSAR and the load factor expressions to be employed in the design and find these reasonable. Further, we note on page 5-12 of the PSAR that no steel reinforcement will experience average stress beyond the yield point at the factored load, and a statement on page 5-13 that the liner will be designed to assure that stresses will not exceed the yield point at the factored loads. Further amplification on these points is given in answer to Question III.A.5 of Supplement 2. We interpret these statements to mean that the average stress in the reinforcement and liners will not exceed yield and that the deformations will be limited to that of general yeilding under the maximum earthquake loading conditions. On the assumption that this interpretation is correct, we concur in the approach.

The detail for carrying the radial shear, namely through the use of a vertical I-beam, as described in the PSAR and in more particular beginning on page 30 of Supplement 1, is novel and appears acceptable to us. We recommend that careful attention be given to the detail at the base of the I

section where it is keyed into the foundation, to insure that no distress can occur in either the liner or the diagonal reinforcing bars through any rotation that might occur at this point under earthquake loadings or other types of accident loadings.

It is noted in answer to Question III.A.9 of Supplement 1 that the diagonal reinforcing will be carried over the top of the cylindrical shell and form a more or less completely tied unit through the containment structure with tie-down into and through the foundation as described in answer to Question III.A.10. It is further noted that the splices for the ASTM A-432 bars, which comprise the diagonal reinforcing in the side walls and carry the lateral shears and vertical loadings in the containment structure, will be spliced by the Cadweld process and that less than 1 percent of them will be welded by virtue of inaccessibility for Cadweld splice units. The proposed approach appears acceptable to us.

The design of the intake structure located at sea level is described in detail in the PSAR and the various supplements. This will be designed as a Class I structure, with due regard for expected tsunami water heights. Although it appears that some protection has been provided against the possibility of rock masses from the cliff falling onto, or into, the pump house, we recommend that careful attention be given to any impossible impairment of the controls or the pumping system through any possible rock falls or slides.

Cranes

The containment crane is listed on page 2-27 (revised 7/31/67) of the PSAR as a Class I structure. We wish to call attention to the design of the cranes to insure that these cranes cannot be displaced from the rails during

the design or maximum earthquake, or otherwise to have damage result from the movement of items supported by them which could cause impairment of the containment or the ability for safe shutdown.

Penetrations

A discussion of the design of the penetrations is given in answer to Question III.A.2 of Supplement 1. It is noted there that for the large penetrations the diagonal rebars will be welded directly to a heavy structural steel ring through use of Cadweld sleeves. This approach appears satisfactory to us.

The applicant further notes in the same section that the stress concentration in the vicinity of the opening will be considered in the analysis. Although this approach may well be satisfactory, we believe that the penetration design should take account of any secondary effects arising from local bending, thermal effects, and so on, to insure that the penetration-door detail behaves satisfactorily, and secondly, that there is no distress in the containment structure in the transition zone from the penetration into the remainder of the shell structure. Partial proof of the integrity of the penetration will be provided by the measurement program to be made concurrently with the proof testing of the containment vessel. We recommend that penetration deformation calculations be made prior to the proof testing to provide demonstrated evidence that the design does indeed meet the criteria set forth for both the large and small penetrations.

Piping, Valves, and Reactor Internals

The design of the piping is described in Section 2 of the PSAR, and in further detail in Supplements 1 and 2. On page 1-22 of the PSAR a statement

is made that all piping will be designed to withstand any seismic disturbance predictable for the site. On page 2-30 of the PSAR it is indicated that there are regions of local bending where the stresses will be equivalent to 120 percent of the yield stress based on elastic analysis for the no-loss-of-function criteria. Further elaboration on the piping design is given in answer to Question II.F and Appendix A of Supplement 1 and again in answer to Question II.G of Supplement 2. The discussion presented in Supplements 1 and 2 indicates that the earthquake loadings will be combined directly with the other applicable loadings for the piping and that the design limits will be established in terms of code allowable stresses, which in cases can be as large as 1.2 to 1.8 times the code allowable stresses. The matter of concern to us is that of the possible impairment of the serviceability of the piping through rupture or buckling if excessive deformations occur. As the result of discussions with the applicant, we believe that for the specific materials used, and under the conditions cited, the deformations generally will be limited to acceptable values. However, we urge that this matter receive further consideration by the applicant during the design process.

The isolation valve design is discussed in several places but particularly in answer to Question II.A.14 of Supplement 1. The approach outline there appears acceptable to us.

The design of the reactor internals has been reviewed in some detail with the applicant. The internals are to be designed to withstand the combined maximum earthquake spectrum concurrent with blowdown in such a manner that moderate yielding would not impair the capability of safe shutdown. It is our understanding that this matter is under detailed study and further documentation and review of the design criteria for the internals is required.

CONCLUSIONS

In line with the design goal of providing serviceable structures and components with a reserve in strength and ductility, and on the basis of the information presented, we believe the design criteria outlined for the primary containment, secondary containment and Type I piping can provide an adequate margin of safety for seismic resistance. Still remaining for review is a detailed evaluation of the criteria to be employed in the design of the reactor internals.

REFERENCES

1. "Preliminary Safety Analysis Report, Volumes 1 and 2," Nuclear Plant, Diablo Canyon Site, Pacific Gas and Electric Company, 1967.

2. "Preliminary Safety Analysis Report, Supplements 1, 2 and 3," Nuclear Plant, Diablo Canyon Site, Pacific Gas and Electric Company, 1967.

3. "Report on the Seismicity of the Diablo Canyon Site," U.S. Coast and Geodetic Survey, Rockville, Maryland, _____.