UNITED STATES
ATOMIC ENERGY COMMISSION
WASHINGTON, D.C. 20545

December 7, 1967

Docket No. 50-275

Mr. Nunzio Palladino
Chairman, Advisory Committee
on Reactor Safeguards
U. S. Atomic Energy Commission
Washington, D. C.

Dear Mr. Palladino:

Transmitted for the review of the Committee are twenty-four copies of the following:

PACIFIC GAS & ELECTRIC COMPANY
DIABLO CANYON

1. Letter dated December 6, 1967 transmitting Amendment No. 9 which includes Supplement No. 8 to the PSAR.

2. Letter dated November 30, 1967 transmitting Amendment No. 8 which includes Supplement No. 7 to the PSAR.

Sincerely yours,

Peter A. Morris

Peter A. Morris, Director
Division of Reactor Licensing

Enclosures:
As stated above

Distribution:
Suppl.
RPB-5 Reading
R. S. Boyd

C-24

OFFICE ▶	AD/RP					
SURNAME ▶	THill:th					
DATE ▶	12/7/67					

JAN 1 6 1968

Docket No. 50-275

Dr. Nathan M. Newmark
1114 Civil Engineering Building
University of Illinois
Urbana, Illinois 61801

Dear Dr. Newmark:

As you know we are presently preparing our Safety Evaluation regarding Pacific Gas and Electric Company's Diablo Canyon Nuclear Plant. An attachment to our evaluation will include your report on the Adequacy of the Structural Criteria for the Diablo Canyon Site Nuclear Plant. We have placed your draft report of December 1967 in final form as per our telephone conversations. If the report is acceptable to you, we would appreciate your signature on one of the enclosed reports to be included in our record files.

Sincerely yours,

Roger S. Boyd, Assistant Director
for Reactor Projects
Division of Reactor Licensing

Enclosure
Newmark Report
dated 12/67

C-26

OFFICE ▶	RPB#5/DRL	DRL	DRL	OGC		
SURNAME ▶	DKnuth/ds	RSBoyd	PAMorris	RBC		
DATE ▶	1/ /68	1/ /68	1/ /68	1/16/68		

Form AEC-318 (Rev. 9-53)

U.S. GOVERNMENT PRINTING OFFICE : 1966—O-214-629

~~8611040199~~ 1P

NATHAN M. NEWMARK
CONSULTING ENGINEERING SERVICES

1114 CIVIL ENGINEERING BUILDING
URBANA, ILLINOIS 61801

22 January 1968

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

Re: Diablo Canyon Report - Docket No. 50-275

Dear Dr. Morris:

In accordance with the request from Roger S. Boyd, dated 16 January 1968, we have reviewed the report to the AEC Regulatory Staff dated December 1967, prepared by Dr. Hall and myself, and compared the copy with the draft prepared by us. One minor point is called to your attention. At the bottom of the first paragraph on page 7, the original draft and notes (from the end of the first complete paragraph on page 8 of the original draft) indicates that a sentence should be added as follows:

"On this basis, we concur in the design approach adopted."

However, this added statement may not be important, and I believe that the typed copy of the report reflects our views and is in accord with our draft. Therefore I am signing it on page 11 and returning a copy to you. I am also returning the amended draft copy, which is marked for Troy Conner.

Thank you for your cooperation.

Sincerely yours,

N M Newmark

N. M. Newmark

bjw
cc: W. J. Hall
Enclosure

C-27

~~8708130263~~ 1p

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

N. M. Newmark

and

W. J. Hall

December, 1967

241

8708130269 12pp

ADEQUACY OF THE STRUCTURAL CRITERIA FOR

THE DIABLO CANYON SITE NUCLEAR PLANT

by

N. M. Newmark and W. J. Hall

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 resistance to radial shears the applicant proposes to use a system of vertical wide flange beams spaced four feet on centers. The beams are attached by hinge connections 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 èlement 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 studs are to be L shaped and will be fusion welded to the liner plate. The studs will be spaced at the corners of a 20 in. square grid, and the design is intended 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 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 on bedrock in all cases. The site has been well explored and there is no evidence of any significant fault offsets of recent origin. 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 structure; 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 separate earthquakes, which for the design earthquake condition correspond to maximum horizontal ground accelerations of 0.20g or 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 0.30g, 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 Supplements 2 and 5, will be designed for normal loads, (internal pressure, dead load, thermal expansion, etc.) combined with pipe rupture loads and earthquake loading.

The reactor internals are to be designed to resist earthquake combined with blow-down 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 give 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 or safe shut-down 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 shut-down of multi-degree-of-freedom systems, 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 shut-down. This envelope spectrum is consistent with an El Centro type response spectrum for a maximum ground acceleration of 0.40g.

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 single degree of freedom systems, the fundamental mode of vibration will be used in the analysis. The applicant has agreed however 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 significant modes will be considered. On this basis, we concur with the approach.

The method of dynamic analysis is described in Sections 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. Further 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 for the envelope of the two earthquakes considered. The applicant has advised that the time history input used in its analysis yields substantially the same response spectra as the envelope spectra of the two earthquakes considered.

Vertical acceleration values in all cases will be taken as two-thirds the corresponding maximum horizontal ground acceleration, and the effects of horizontal and vertical earthquake loadings will be combined, and considered to act simultaneously. In addition in the elastic analysis, for the containment structure 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.

General Design Provisions for Containment

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. The applicant has confirmed our interpretation that the average stress in the reinforcement and liners will not exceed yield and that the deformations will be limited to that of general yielding under the maximum earthquake loading conditions. On this basis, we concur in this approach.

A discussion of the resistance of the lining to buckling from compressive thermal stress is given in Supplement 2 and also in Supplement 4 in the answers to Question III.A.6. The conditions assumed for buckling of Type I are conservative, and we conclude that the spacing of the stud supports is close enough to give a reasonable margin of safety against buckling of the liner.

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 ingenious and appears acceptable to us.

We recommend that careful attention to 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 the splices will be inaccessible for Cadweld splice units, and will therefore require welding. The proposed approach is 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 consideration be given to 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 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 containment 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.

<u>Piping, Valves, and Reactor Internals</u>

The design of the piping is described in Section 2 of the PSAR, and in further detail in Supplements 1, 2, 4 and 5. 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, Section II of Supplement 4, and in answer to Questions 10 through 13 of Supplement 5. The discussion presented in Supplements 1, 2, 4 and 5 indicates that the earthquake loadings will be combined directly with the other applicable loadings. For the most severe loading condition (involving the maximum earthquake plus normal and pipe rupture loads) oral discussions with the AEC staff have indicated that the limit curves as given in WCAP 5890-1 have been revised such that the strain limits at temperature will consider limited strain hardening no more than 20% of the strain at the maximum stress of the stress-strain curve in simple tension.

The design criteria and design approach as described above are acceptable to us.

The isolation valve design is discussed in several places but particularly in answer to Question II.A.14 of Supplement 1. The approach outlined there is 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 blow down in such a manner that moderate yielding would not impair the capability of safe shutdown. On the basis of our discussion with the applicant, and the material presented in Supplement 5, the design criteria and design approach proposed for the internals are acceptable to us.

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 containment and other Class I components including the reactor internals, piping, vessels, and supports can provide an adequate margin of safety for seismic resistance.

N. M. Newmark

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, 3, 4, 5, and 6," 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, September 21, 1967.