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9/8/80

Director of Nuclear Reactor Regulation
Attention: D. M. Crutchfield, Chief
Operating Reactors Branch No. 5
Division of Licensing
U.S. Nuclear Regulatory Commission
Washington, D. C. 20555

Gentlemen:

Subject: Docket No. 50-206
Systematic Evaluation Program
San Onofre Nuclear Generating Station
Unit 1

The information requested in your letter dated August 16, 1979 is provided as an enclosure to the letter. The enclosure is entitled, "Additional Information, San Onofre Nuclear Generating Station Unit 1, Structural Topics".

If you have any questions regarding this matter, please let me know.

Very truly yours,

K P Baskin

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Enclosure

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ENCLOSURE

ADDITIONAL INFORMATION

SAN ONOFRE NUCLEAR GENERATING STATION UNIT 1

STRUCTURAL TOPICS

III-2 Wind and Tornado Loads

For each safety-related structure, provide the original design basis of the wind and tornado loadings including,

(1) for wind load, the procedures to transform wind data to design pressure and gust factor,

(2) for tornado load, the maximum rotational wind speed, translational wind speed, pressure drop, radius of maximum rotational wind speed and procedures to transform tornado data into design pressure.

Response

(1) As stated in Section 4.3.2.3 of the SONGS Unit 1 FSA (Reference A), a design wind velocity of 100 miles per hour was used for the containment sphere.

Typical wind loads of 15 or 20 psf, corresponding to wind velocities of approximately 80 and 90 mph respectively, were considered for the remaining structures which were constructed as part of the original plant design, where applicable. The 90 mph wind criteria was applied above a height of 30 feet from the ground. Wind loads were not combined with earthquake loads in accordance with accepted practice. In general the wind load combinations, with the exception of the vent stack, did not govern the design. The vent stack is a light gage steel stack and as such the aerodynamic forces were more significant than the seismic forces. Wind forces on the vent stack were calculated in accordance with "The Design of Self Supported Steel Stacks" by Kaiser Steel Corp., printed in "Modern Designing with Steel," Vol. 6, No. 1, May 1960.

As stated in paragraph 3.1.1.3.2 of Reference B, and paragraph B.2 of the Reference C response to item 2, the design wind load for the sphere enclosure building and the diesel generator building are based on a design wind velocity (defined as the fastest mi/h of wind at 30 feet above ground level) of 100 mi/h, and the procedures of "Building Code Requirements for Minimum Design Loads in Buildings and Other Structures, ANSI A58.1-1972" were used to convert the wind velocity into applied forces for these structures.

(2) As stated in paragraph 3.1.1.3.2 of Reference B, and paragraph C.2 of the Reference C response to item 2, the sphere enclosure building and the diesel generator building were designed to withstand the following postulated tornado effects:

- o Tornado wind having a maximum total horizontal velocity of 260 mi/h, corresponding to 220 mi/h rotational wind with translational velocity of 40 mi/h.

- o Atmospheric pressure drop of 1.5 lb/in.^{2} in 4.5 seconds followed by a constant pressure for 3 seconds and a repressurization.

An evaluation of these design basis tornado characteristics was provided in Reference D.

The methods employed to convert the tornado loadings into forces to the sphere enclosure and diesel generator building structures are contained in Tornado and Extreme Wind Design Criteria for Nuclear Power Plants, Bechtel Topical Report BC-TOP-3. A radius of maximum rotational wind speed of 185 ft. was used.

The sphere enclosure building provides substantial protection for the containment sphere from tornado effects, including tornado missiles. The largest opening exists on the south side of the plant from grade to elevation + 54 to accommodate, in part, the turbine deck equipment hatch.

Tornado effects were not considered in the design of the original San Onofre Unit 1 structures.

III-3.A Effects of High Water Level on Structures

For each safety-related structure,

(1) provide the original design basis water loads including any dynamic effects considered,

(2) clarify the water loads considered in the wall design for each load combination discussed in Topic III-7.B,

(3) explain how the ground water pressure on the embedded part of the steel containment was considered.

Response:

(1) and (2): The design of the Unit 1 structures did not include water loads resulting from the effects of high water levels. However, in 1977 the capacity of the San Onofre Unit 1 storm drain system was increased such that precipitation of probable maximum intensity[a] will not result in flooding of safety-related structures, systems, and components. Potential offsite flood water is prevented from entering the plant site; and potential onsite flood water is directed to the yard catch basin where it enters the intake structure. The installed storm drain system will maintain Unit 1 flooding below elevation +14 feet.

(3) The embedded portion of the steel containment is supported by a concrete cradle several feet thick. In addition, the concrete internal structure (reactor building) foundation is supported from the embedded portion of the containment, thus effectively encapsulating the embedded containment sphere in 6 to 8 feet of concrete. The embedded portion extends from approximately elevation +20' to elevation -20' (MLLW datum), forming a spherical segment.

The average groundwater elevation is +5' MLLW datum. The resulting ground water pressure causes very small compressive stresses in the concrete foundation, therefore, it was not necessary to employ this load condition. Since tensile stresses govern the foundation design (see Table 3.8.5-1 in Reference E.), neglecting the ground water pressure is conservative.

a. Precipitation of probable maximum intensity as determined by Hydrometeorological Report No. 36 is discussed on page 2.7-2 of San Onofre Nuclear Generating Station Units 2 and 3 Preliminary Safety Analysis Report.

III-7.B Design Codes, Design Criteria and Load Combination

For each safety-related structure,

(1) list the codes and standards (including edition date) used for the design and construction of concrete and steel elements,

(2) provide the loads, load combinations and acceptance criteria employed for the design,

(3) provide the design and/or actual material properties (fc and fy) used for the concrete and steel elements. For concrete, provide the age specified and any admixtures used,

(4) provide a copy of the design specification used for construction,

(5) describe the buckling criteria used for design of the steel containment,

(6) provide representative stress levels (compression, tension, and shear) at the critical locations of each structure (e.g., base of the internal structures) for each of the load combinations provided in response to (2) above.

Response: Information is provided herein for safety related Unit 1 buildings, as defined in the San Onofre Unit 1 "Q" List included in Station Order S-A-112, as well as the intake structure, refueling water storage tank and condensate storage tank.

A. Containment Sphere and Reactor Building

(1) The codes and standards used in the design and construction of the containment sphere are stated in paragraph 4.3.2.5 of Reference A and paragraph 3.8.2.2 of Reference E. The codes and standards used in the design and construction of the reactor building are stated in paragraph 3.8.3.2 and 3.8.5.2 of Reference E.

The codes and standards which were utilized in the reevaluation of the containment sphere and the reactor building are discussed in paragraphs 3.8.2.2, 3.8.2.5, 3.8.3.2, and 3.8.5.2. of Reference E.

(2) As discussed in Section 4.3.2.2 of Reference A, the containment sphere was orginally designed for an internal pressure of 46.4 psig and an internal vacuum of 2 psig. As discussed in References E and F, the containment sphere has subsequently been reevaluated considering a containment peak pressure of 49.4 psig (as well as an internal vacuum) in combination with earthquake loads and gravity loads.

The loads, load combinations, and acceptance criteria employed in the reevaluation of the containment sphere and reactor building is provided in Tables 3.8.2-2, 3.8.2-4, and paragraphs 3.8.2.3, 3.8.2.5, 3.8.3.3, 3.8.3.5, 3.8.5.3, and 3.8.5.5 of Reference E.

(3) The material properties of containment sphere steel shell are provided in paragraph 4.3.2.5 of Reference A and paragraph 3.8.2.2 of Reference E.

All concrete within the containment sphere (including the reactor building) has a strength (f'c) of 3,000 psi at 28 days. Grade 40 (fy=40,000 psi) reinforcing steel was used. Miscellaneous support structural steel types and properties are discussed in sections 3.8.3.6 and 3.8.3.5 of Reference E.

(4) A design specification for the construction of the containment sphere and reactor building is not available. The following material procurement specifications are available, however, and will be provided if desired:

BSO-280	Containment Sphere
BSO-252	Concrete
BSO-253	Testing Lab Services
BSO-254	Turbine Gantry and Reactor Bridge Cranes
BSO-255	Platform and Walkway Grating
BSO-260	Miscellaneous Steel
BSO-261	Reinforcing Steel
BSO-264	Main Structural Steel
BSO-267	Unloading and Installing Heavy Equipment
BSO-110	Hanger Inserts

(5) The buckling stability of the containment sphere has been reevaluated as discussed in Reference E, and the results are summarized in Table 3.8.2-4 therein.

(6) Representative stress values for the containment sphere, based on the original design are provided in Table 9.6 and section 9.2.5.2 of Reference A.

Representative stress values for the containment sphere and reactor building, resulting from the subsequent reevaluation are provided in Tables 3.8.2-3, 3.8.2-4 and 3.8.3-2 of Reference A, as well as Reference F.

B. Sphere Enclosure Building

(1) The codes and standards used for the design and construction of the Sphere Enclosure Building are provided in paragraph 3.1.1.2 and Table 3.4-1 of Reference B (as revised by the responses to Items A.12 and A.13 contained in Reference G).

(2) The loads, load combinations, and acceptance criteria employed for the design of the sphere enclosure building are stated in section 3.1.1.3 and Table 3.1-1 of Reference B.

(3) Concrete used for the sphere enclosure building had a compressive strength, f'c, of 6,000 psi at 90 days. Reinforcing steel was grade 60 (fy=60 ksi). Structural Steel for the roof framing was ASTM A 572 Grade 50. Supplemental information pertaining to the properties of materials used in the construction of the sphere enclosure building is provided in the response to A.15 contained in Reference G.

(4) Specification SEP-211 (including Addenda 1 and 2), entitled "Civil/Structural Construction Specification for the Sphere Enclosure Building," was used for the construction of the sphere enclosure building. A copy of this specification is provided as Attachment 1 herein.

(5) Not Applicable.

(6) The maximum shear stress at the base of the building is approximately 22 psi. Typical bending moments vary from 30 to 120 ft-kips/ft. Allowable moment capacity exceeds 500 ft-kips/ft in most cross sections.

The governing load condition for the design of the roof occurred during construction. The remaining loading conditions do not produce large stresses in the roof.

C. Diesel Generator Building

(1) A listing of the codes and standards used for the design and construction of the diesel generator building is provided in the response to Item 1 contained in Reference H. In addition to the listing therein, the provisions of the following NRC Regulatory Guides were used:

a. NRC Regulatory Guide 1.10 Revision 1 - Mechanical (Cadweld) Splices in Reinforcing Bars of Category I Concrete Structures.

b. NRC Regulatory Guide 1.15 Revision 1 - Testing Reinforcing Bars for Category I Concrete Structures, 1972.

c. NRC Regulatory Guide 1.55 - Concrete Placement in Category I Structures, 1973.

d. NRC Regulatory Guide 1.92 - Combination of Modes and Spatial Components in Seismic Response Analysis, 1974.

(2) The loads, load combinations, and acceptance criteria employed for the design of the diesel generator building are provided in the response to Item 2 contained in Reference H.

(3) Concrete used for the diesel generator building had a compressive strength, f'c, of 4,000 psi of 28 days. Concrete additives included pozzolan, air-entraining agent (Darex AEA), and a water reducing agent (Zee-con R-40.). Grade 40 (Fy = 40 ksi) reinforcing steel was used.

A514, type M (Fy = 100 ksi) steel was used for the building's missile protection louver plates. The remaining structural steel for the building is A36 (Fy = 36 ksi).

(4) Section 2, entitled "Scope of Work, Construction and Home Office Support, Specification 82-6220, Standby Power Addition and Modification of Emergency Core Cooling System, SONGS Unit 1" (including Addenda No. 1 through 7), was, in part, used for the construction of the diesel generator building. A copy of Sections 2.1, 2.2, and 2.3 (entitled Sitework, Structural Concrete, and Steel and Metalwork, respectively) of this specification is provided as Attachment 2 herein.

(5) Not Applicable.

(6) The maximum analysis/design stresses at various locations of the diesel generator building are as follows:

Structural Element	Loading Combination	Maximum In-Plane Stress, ksi: σ_x	σ_y
Roof	b	0.264	0.537
West wall	a	0.218	--
	b	--	0.572
Center wall	c	0.336	--
	a	--	0.531
South wall	a	0.226	--
	b	--	0.545
North wall	b	0.243	0.532
East wall	a	0.304	--
	b	--	0.447

a. 1.4 gravity + 1.7 live load +1.9 OBE load.

b. 1.4 gravity + 1.7 live load - 1.9 OBE load.

c. gravity load + live load + pipe load + DBE load.

D. Control and Administration Building

(1) The control and administration building was designed in accordance with ACI 318-63 (1963 edition) and the AISC Specification (1963 edition).

(2) The control and administration building was designed for dead load and live load in combination with the seismic load. Stresses were limited to working stress according to the applicable codes. Seismic inputs of 0.25g and 0.5g were considered.

(3) The design concrete strength, f'c for the control and administration building was 3,000 psi at 28 days. The reinforcing steel was grade 40 (Fy=40 Ksi) and structural steel was A 36 (Fy=36 Ksi). Grade A concrete masonry block, ASTM C-90, with a design f'm of 1350 psi was also used.

(4) A design specification for the construction of the control and administration building is not available. The following material procurement specifications are available, however, and will be provided if desired:

BSO-260	Miscellaneous Steel
BSO-261	Reinforcing Steel
BSO-264	Main Structural Steel
BSO-252	Concrete
BSO-253	Testing Lab Services
BSO-112	Concrete Block Masonry
BSO-116	Roofing
BSO-120	Sheet Metal Work

(5) Not Applicable.

(6) The maximum shear stress at the base of the structure is 61 psi for the OBE condition and 172 psi for the DBE case. The maximum OBE tensile stress corresponds to 167 psi in the concrete or 22,000 psi in the vertical reinforcement.

E. Reactor Auxiliary Building

(1) The reactor auxiliary building was designed in accordance with ACI 318-63 (1963 edition) and the AISC Specification for the Design Fabrication and Erection of Structural Steel Buildings (1963 edition).

(2) The reactor auxiliary building was designed by the working stress method for dead load and live load combined with seismic load, wind load or earth pressure. Hydrostatic loads due to ground water and vehicle surcharge loadings were also considered as appropriate and combined with the above loadings. Stresses were limited to working stress levels for the 0.25g Housner spectrum. Since the building was assumed rigid, a 0.25g static force was applied. Wind loads corresponding to 80 mph (15 psf) were used for this structure since it is less than 30 feet above ground.

(3) The reactor auxiliary building was constructed utilizing concrete with a 28 day strength of 3000 psi and grade 40 reinforcing steel which has a yield point of 40 Ksi. The structural steel used was A36 (Fy=36 Ksi). Grade A concrete masonry block, ASTM C-90, with a design f'm of 1350 psi was also used.

(4) Same as the Control and Administration Building response to item (4).

(5) Not Applicable.

(6) The Reactor Auxiliary building is a single story partially embedded structure with a masonry structure on one corner of the roof, and the seismic stresses are quite small. The thick walls required for shielding provide significant shear resistance and substantial flexural capability (even with minimum reinforcement). The maximum rebar stress was 18,000 psi due to the vehicle surcharge load on an embedded wall. The allowable rebar stress was 28,000 psi. Shear stresses were calculated to be less than 30 psi.

E. Fuel Storage Building

(1) The fuel storage building was designed in accordance with AISC Specification for the Design, Fabrication, and Erection of Structural Steel building (1963 edition) and ACI 318-63 (1963 edition).

(2) The fuel storage building was designed for seismic loads (0.25g) in combination with dead load and live load. In addition, wind loads were considered in combination with dead and live loads. Wind loads of 20 psf, corresponding to a 90 mph wind, were utilized.

The fuel storage building was designed utilizing A36 steel with Fy=36 Ksi. The concrete utilized had a specified 28 day strength of 3,000 psi. Grade 40 (Fy=40 Ksi) reinforcing steel was used. Grade A concrete masonry block, ASTM C-90, with a design f'm of 1350 psi was also used.

(4) Same as the Control and Administration Building response to item (4).

(5) Not Applicable.

(6) A wind load of 20 lbs/ft^2 was used in the design of the fuel storage building. This corresponds to a velocity of approximately 90 mph. Since the seismic load was approximately twice the wind load, the seismic load governed the design. Therefore, no stress calculations were performed for wind loads.

The seismic load combination produced maximum shear stresses at the base of the masonry walls (top of fuel pool) of approximately 7 psi. Stresses in the fuel pool walls were small and therefore minimum reinforcing steel requirements governed the design.

F. Turbine Pedestal

(1) The turbine pedestal was designed in accordance with the Uniform Building Code (1961 edition), the AISC Specification for the Design Fabrication and Erection of Structural Steel for Buildings (1963 edition), and Westinghouse Electric Corporation Booklet, AO 1001, "General Design Notes for Turbine Generator Foundation Design."

The pedestal was designed for dead load plus live load plus condenser vacuum with no increase in allowable stresses. Machine weights were multiplied by 1.35 to account for impact.

The pedestal was also designed for a seismic load of 0.2 times the weight of the pedestal (neglecting gantry crane weight) applied in any direction in combination with dead load, live load, and condenser vacuum. In this case, a 1/3 increase in allowable stresses was permitted.

The pedestal was also designed for seismic (0.2g) plus dead load plus live load plus vacuum load plus shrinkage and thermal effects. Stresses for this condition were limited to 3 times the allowable stresses.

Allowable stresses in the base mat were limited to working stress levels. Allowable stresses in the pedestal were fc=550 psi and fs=10,000 psi, which are less than 1/2 the allowable code working stresses.

(2) The 28 day design strength for the basemat and pedestal concrete is 3,000 psi and 4,000 psi, respectively. Grade 40 reinforcing steel (Fy=40 Ksi) was used.

(3) Same as the Control and Administration Building response to item (4).

(4) Not Applicable.

(5) Maximum vertical stresses in the pedestal piers due to combined seismic and vertical forces are 350 psi compression and 40 psi tension. The maximum shear force in the east west direction is 2420 kips. This results in an average shear stress of 26 psi.

G. Turbine Building (North and South Pedestal extensions plus east and west heater decks)

(1) The turbine building was designed in accordance with the provisions of the Uniform Building Code (1961 edition) and the AISC Specification for the Design Fabrication and Erection of Structural Steel for Buildings (1963 edition).

(2) The design seismic load was 0.2 times the dead load (including the gantry crane) applied in any direction. A wind load of 15 psf was considered, however, the seismic load case governed the design.

Two load cases were considered. Dead load plus live load plus full crane load, and dead load plus live load plus crane dead load plus seismic. In the case of the seismic loading condition a 1/3 increase in allowable stresses was permitted. Allowable stresses were based upon working strength design. The probability of the simultaneous occurrence of full crane load and the maximum seismic load was considered too low to merit consideration.

(3) The turbine building was constructed utilizing concrete with a 28 day strength of 3,000 psi and ASTM A 36 structural steel (Fy=36 Ksi). Grade 40 reinforcing (Fy=40 Ksi) was used. Anchor bolts were ASTM A 307 and ASTM A 193, Grade B. Grade A concrete block masonry, ASTM C-90, with a design f'm of 1350 psi was also used.

(4) Same as the Control Administration response to item (4).

(5) Not Applicable.

(6) The turbine building is a series of rigid frame structures which includes the north and south pedestal extensions and the east and west heater decks. The vertical loads in columns are nearly always compressive. One column is subject to a 5k tensile force under the combined seismic loading case. Typical compressive loads range from 20k to 120 k per column. Base shears range from 20 to 90k per column. Moments varied from 200 foot kips to 600 foot kips. Column sizes range from 24WF76 to 24WF160.

H. Circulating Water System Intake Structure

(1) The intake structure is an embedded concrete structure. It was designed in accordance with ACI 318-63 (1963 edition).

(2) The intake structure was designed for dead load plus live load plus earth pressure plus an H20 vehicle surcharge load. In addition, it was designed to resist a tsunami in combination with dead load and earth pressure. A seismic lateral load based upon a 0.25g Housner spectrum was also considered in combination with dead load. Uplift stability was also checked. Allowable stresses based upon working stress limits constituted the acceptance criteria. Typical values were fs=20,000 psi and fc=1,350 psi.

(3) The intake structure was constructed utilizing concrete with a 28 day strength of 3,000 psi. Grade 40 (fy=40 Ksi) reinforcing steel was also utilized.

(4) Same as the Control and Administration Building response to item (4).

(5) Not Applicable.

(6) Concrete stresses were limited to 1,350 psi and reinforcing stresses were limited to 20,000 psi. Shear in beams without web reinforcement was limited to 60 psi. Shear in members with web reinforcement was limited to 274 psi. The safety factor against uplift is 1.51.

I. <u>Refueling Water Storage Tank and Condensate Storage Tank</u>

(1) The refueling water storage tank (RWST) and condensate storage tank (CST) were designed and fabricated in accordance with API Specification 650 and Appendix D for welded oil storage tanks, including all revisions up to May 1964.

(2) The RWST was designed for seismic load including hydrodynamic effects in combination with normal loads. Two seismic cases were considered. The 0.25g Housner spectrum and 0.50g Housner spectrum. For the case of the 0.25g Housner spectrum, stresses were limited to working stress allowables. For the 0.50g case safe operability had to be assured. In addition to seismic loads, a wind load of 20 psf was considered when the tank is empty. This load was combined with normal loads but not with seismic loads. For this case, stresses were required to be within working stress allowables.

The CST was designed to withstand a seismic force of 0.2g applied in any direction. In addition, the tank was designed to withstand a wind load of 20 psf when empty. Stresses were required to be within working stress limits.

(3) The RWST and CST were fabricated from ASTM A36 steel with fy=36 Ksi.

(4) The only applicable specification which is available is the purchase specification BSO-433. A copy of the specification is enclosed.

(5) Not Applicable.

(6) No calculations are available. The above information was obtained fom the purchase specification BSO-433.

References

A. Final Safety Analysis, San Onofre Nuclear Generating Station, Unit 1, Docket 50-206.

B. Amendment 52 to Final Safety Analysis, San Onofre Nuclear Generating Station, Unit 1, Docket 50-206.

C. Addition of Standby Power and ECCS Modifications (Preliminary Engineering and Safety Analysis Report), dated February, 1975, Amendment 38, San Onofre Nuclear Generating Station, Unit 1, Docket 50-206.

D. Safety Evaluation by the Office of Nuclear Reactor Regulation Supporting Amendment No. 20 to Provisional Operating License No. DRR-13, San Onofre Nuclear Generating Station, Unit 1, Docket 50-206.

E. Seismic Reevaluation and Modification, San Onofre Nuclear Generating Station, Unit 1, dated April 29, 1977, NRC Docket 50-206.

F. Southern California Edison letter to Director of Nuclear Reactor Regulation dated February 4, 1977, Subject: Containment Post Accident Pressure Reanalysis, Docket No. 50-206.

G. Supplement to Sphere Enclosure Project Report, San Onofre Nuclear Generating Station, Unit 1, dated March, 1976, Docket No. 50-206.

H. Supplement 1, Addition of Standby Power and ECCS Modifications, San Onofre Nuclear Generating Station, Unit 1, dated May, 1975, Docket No. 50-206.