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EVALUATION OF THE EFFECTS OF GROUND WATER TABLE AT PLANT GRADE SAN ONOFRE NUCLEAR GENERATING STATION, UNIT 1

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1.0 INTRODUCTION

Quadrex Corporation has performed an evaluation of the effects of short-term ground water level at grade on the safety-related plant structures in Southern California Edison Company's (SCE) San Onofre Nuclear Generating Station, Unit 1 (SONGS 1). The evaluation was performed to resolve Systematic Evaluation Program (SEP) Topic III-3.A (reference 1). This report presents the methodology, criteria, and results of this evaluation.

1.1 Background

The original design basis ground water elevation for SONGS 1 was +5 feet Mean Lower Low Water (MLLW). While reviewing SEP hydrological topics II-3.A, II-3.B, II-3.B.1, and II-3.C, the U. S. Nuclear Regulatory Commission (NRC) suggested that the design basis ground water level of +10 feet MLLW should be considered (reference 5). In response, SCE furnished additional data (reference 6) and the NRC accepted the +5 feet MLLW as the design basis ground water table (reference 7).

Subsequently, NRC reviewed SCE's evaluation of SEP topic III-3.A, (reference 8), "Effects of High Water Level on Structures", and concluded that "topic III-3.A is complete", except that "the capability of plant structures to withstand a short term hydrostatic load from groundwater at plant grade" was identified as an open issue (reference 1). The present work is to resolve this open issue.

1.2 Summary

The effects of short-term hydrostatic loads on safety-related structures due to ground water table at plant grade were evaluated. This loading phenomenon was categorized as an extreme event, because the probability of such an event was assumed to be of the same order as the DBE. Therefore,

the allowable stresses used for the DBE loading case during the seismic re-evaluation program were used for evaluating the effects of short-term hydrostatic loading. Also, the short-term hydrostatic loading was not considered concurrent with any earthquake event (OBE or DBE). Thus, if short-term hydrostatic loads can be shown to be smaller than the combined loads resulting from the DBE and the design basis hydrostatic loads, the structures can be considered safe. For preliminary assessment, short-term hydrostatic loads were estimated based on the existing calculations of loads resulting from the design basis ground water table (elevation + 5.0 ft). A comparison of the estimated short-term hydrostatic loads with the combined DBE and design basis hydrostatic loads indicated that the short-term hydrostatic loads were significantly smaller. Hence, an evaluation of all plant structures and components for the short-term hydrostatic loads was not considered necessary. Instead, it was concluded that the evaluation of a number of critical buildings and structural components would be sufficient to make general conclusions.

Accordingly, four building structures were evaluated for overall flotation effects, and eleven sub-surface building components were evaluated for structural adequacy. The results of these evaluations showed that the structures have more than adequate margins-of-safety against flotation, and that the stress levels in critical building structural components resulting from short-term hydrostatic loads are significantly less than allowable stresses and are consistently less than those due to DBE. Therefore, it was concluded that all the safety-related buildings and structural components can be considered safe against short-term hydrostatic loads due to water table at plant grade.

2.0 SCOPE OF WORK, LOAD COMBINATIONS, AND EVALUATION CRITERIA

2.1 Scope of Work

The capability of plant structures to withstand short-term hydrostatic loads resulting from ground water at plant grade was evaluated on a sample basis. Thus, a limited number of buildings and structural components were evaluated instead of evaluating every component in each building. The rationale and justification for a sample-basis evaluation are as follows:

a) The assumption of ground water table to be at plant grade is more severe than the normal design basis ground water elevation. Hence, this scenario can be defined as an extreme event. The probability for such an event to occur would be very low (for our purposes, taken to be of the same order as DBE) and the probable duration for the ground water table to remain at plant grade level would be short. Therefore, the occurrence of any seismic or accident event during the period when the ground water table remains at plant grade level is not considered credible. Thus, only normal condition loadings can be assumed concurrent to loads resulting from ground water table at plant grade. Also, the allowable stresses under combined normal and high ground water table loads can be assumed to be equal to those under combined normal and DBE loads.

b) The SONGS 1 safety-related structural components have been evaluated and qualified for seismic loadings resulting from DBE and the hydrostatic loads resulting from the design basis ground water table. A review of structural design calculations for several below-ground structural components showed that, in general, the DBE stresses are significantly higher than the hydrostatic stresses resulting from normal ground water level (elevation +5.0 ft.). Also, it was observed that the DBE stresses alone would be higher than those from ground water table at plant grade (estimated by extrapolating hydrostatic

loads due to design-basis water table). Since the allowable stresses are the same for both loading cases (see item 'a' above), it was judged that the safety-related structural components would have larger margins-of-safety when subjected to ground water table at plant grade. Thus, component-by-component evaluation was not considered necessary to establish the adequacy of the safety-related structures when subjected to hydrostatic loads from ground water table at plant grade. Instead, evaluating representative number of buildings and components was considered sufficient. Accordingly, four structures were evaluated against overall flotation effects, and eleven structural components were evaluated for structural adequacy. The basis of selecting these structures and components are described in section 3.0 of this report.

2.2 Loads and Load Combinations

The following loads and load combinations were considered for evaluating flotation effect and for evaluating the structural adequacy of sample buildings and components (reference 3):

2.2.1 Loads

D = Dead load.

L = Live load.

H = Earth pressure loads.

T_o = Normal operating thermal loads

R_o = Maximum pipe/equipment reaction loads during normal operation.

F = Hydrostatic loads resulting from ground water table at grade.

2.2.2 Load Combinations

The following load combinations were used for evaluating the structural adequacy of the sample buildings and components (reference 3):

(i) $D + L + H + F$ (for evaluating flotation effect)

(ii) $D + L + H + T_o + R_o + F$ (for evaluating structural adequacy of building components)

2.3 Evaluation Criteria

2.3.1 Criteria for Evaluating Flotation Effect

The minimum factor of safety against flotation of buildings subjected to load combination (i) (see section 2.2.2) shall be 1.1 (reference 2).

2.3.2 Criteria for Evaluating Structural Adequacy

The following general criteria was used for evaluating the structural adequacy:

a) The component, when subjected to load combination (ii) (see section 2.2.2), shall maintain its structural integrity.

b) The functionality of safety-related equipment or system shall not be impaired due to the deformation in the structural components evaluated.

The specific structural acceptance criteria used was as follows:

1) When a linear elastic analysis method is used, the moments, shears, and axial forces in concrete members resulting from load combination (ii) shall not exceed U, where U is the section capacity determined by the strength design method of ACI standard 349-76 (reference 4).

2) When an inelastic or yield-line method is used for concrete structural members, the computed ductility demand resulting from load combination (ii) shall not exceed 3 (reference 2).

3.0 SELECTION OF COMPONENTS AND BUILDINGS

3.1 Selection Of Buildings For Evaluating Flotation Effect

A large number of plant design documents and drawings were reviewed to select sample buildings for evaluating flotation effect. Selection was made from the consideration of the following structural parameters:

a) Average embedment depth.

b) Foundation type and configuration.

For the major safety-related buildings at SONGS 1, the average embedment depth and foundation types are listed in table 3-1. From a review of the data on this table, the following structures were selected for flotation effect evaluation:

(1) Containment vessel (including reactor building inside the containment vessel).

(2) Reactor auxiliary building.

(3) Intake structure.

(4) Fuel oil storage tank.

These structures were selected because of their large embedment depths and their box-type foundation which displaces large amount of soil. The fuel oil storage tank was selected primarily considering its light weight and buried configuration.

3.2 Selection of Building Components for Structural Adequacy Evaluation

The effect of high ground water table on the structural adequacy of building components will be more in those components which are below grade. Accordingly, only such components (e.g. footings, base slabs, and external walls below grade) were considered.

A total of eleven structural components were selected from four buildings. These are listed in table 3-2. The selection was based on the following considerations:

- Effect of high ground water level on a wall or a slab is more significant if the water and soil pressure act normal to the face of the wall or the slab. Thus, the walls and slabs which directly resist soil and ground water pressure by bending action are more critical than other walls and slabs.

- Forces induced from ground water level at grade in structural components below grade are larger than those in structural components above grade.

- Structural components with deeper embedment and larger span will be affected more by higher water table elevation.

- Structural components that show smaller design margin under existing design loads are also likely to have smaller margin when higher water table elevation is considered.

TABLE 3-1

EMBEDMENT DEPTH AND FOUNDATION TYPE FOR MAJOR SAFETY-RELATED STRUCTURES AT SONGS 1 SITE

Structure	Dimension	Average Embedment	Foundation Type
Containment Vessel and Reactor Bldg.	140 ft. dia. sphere	20'	Embedded semispherical foundation mat
Enclosure Building	144 ft. dia cylinder	7'	Foundation consists of wall footings
Control Building	110' x 140'	5'	Foundation consist of isolated wall footings
Fuel Storage Building	73' x 48'	12'	Partially embedded foundation mat; also isolated and wall footings
Turbine Building	40' x 50' to 112' x 50'	5'	Foundation consists of column and wall footings
Turbine Pedestal	149' x 47' (irreg. shape)	11'	Pedestal foundation consists of a concrete mat
Ventilation Bldg.	44' x 21'	None	Foundation consists of wall footings
Reactor Auxiliary Building	134' x 56'	24'	Embedded box-type foundation
Circulating Water Intake Structure	85' x 61' (irreg. shape)	44'	Embedded box-type foundation
Diesel Generator Building	80' x 126'	7'	Foundation consists of a concrete mat
F.O. Storage Tank	12.0' x 62.5'	14'	Completely buried structure

TABLE 3-2. BUILDING COMPONENTS SELECTED FOR STRUCTURAL EVALUATION

Building	Component
Reactor Building	Containment vessel foundation mat
Enclosure Building	Building wall footing
Reactor Auxiliary Building	East wall below grade South wall below grade West wall below grade Base slab in the liquid radwaste area Base slab in the charging pump area
Intake Structure	North pump well wall East pump well wall Horizontal beam at the east pumpwell wall Intake box culvert

4.0 EVALUATION OF FLOTATION EFFECTS ON BUILDINGS

The evaluation of flotation effects on the four selected structures due to short-term ground water table at plant grade is described in the following subsections. The results are summarized in section 6.0.

4.1 Flotation Effect on Containment Vessel

The total weight of the sphere and reactor building structure was obtained from the original design calculation. This total weight included dead weight of the structures and equipment weights. Live loads were neglected so that the results are conservative. Ground water level was assumed at plant grade (elevation +20.0 ft). The buoyant force acting on the containment vessel was computed based on the volume of water displaced by spherical segment of the containment vessel below elevation +20.0 ft. The total weight was computed to be larger than the buoyant force; so, no flotation would result.

4.2 Flotation Effect on Reactor Auxiliary Building

The total weight of the reactor auxiliary building was obtained from the original design calculation. This weight included dead weight of the structures and weight of equipment such as tanks, pumps, and piping. Floor live loads were conservatively assumed to be zero. The ground water level was assumed to be at grade (elevation +20.0 ft.), even though actual grade elevation at the south side of the building is +14.0 ft. Thus, the buoyant force acting on the reactor auxiliary building was computed based on the volume of the building below grade elevation +20.0 ft. The buoyant force was less than the weight of the building and the equipment; so no flotation would occur.

4.3 Flotation Effect on Intake Structure

The total weight of intake structure was obtained from the original design calculation, except that the water level inside the intake channel in the pumpwell area was assumed to be at the intake nozzle elevation of the circulating water pump (i.e., elevation - 23.00 ft.). This was based on a conservative assumption that water intake may be completely stopped due to floating debris blocking the screen. The buoyant force acting on the intake structure was computed assuming that ground water level surrounding the intake structure would be at grade (elevation +15.0 ft.). This force was less than the weight of the structure; hence, no flotation would result.

4.4 Flotation Effect on Fuel Oil Storage Tank

The ground water level was assumed to be at grade (elevation +19.5 ft.). Assuming that the fuel oil storage tank is only half full and considering the buried tank as a free body, the total downward force applied to the storage tank was computed based on the dead weight of the tank, weight of the oil inside the tank, buoyant weights of concrete structures and backfill soil directly above the buried tank. The buoyant force acting on the tank was calculated using the volume of water displaced by the entire volume of the tank. The buoyant force was less than the total downward force, and so no flotation would occur.

5.0 EVALUATION OF STRUCTURAL ADEQUACY OF BUILDING COMPONENTS

The components were evaluated using one of the two methods outlined below:

(a) Evaluation Method I.

This evaluation method consisted of the following steps:

- Determine, for the structural component being evaluated, internal forces such as moments and shears due to additional hydrostatic head caused by the higher ground water level. Conventional structural analysis method suitable for hand computation was used.

- Extract from existing design calculations the internal forces (moments and shears) due to other normal loadings.

- Combine the above results according to load combination specified in section 2.2.2 and compare with the component section strength computed according to ACI code requirements for concrete members.

(b) Evaluation Method II.

The evaluation method consisted of the following steps:

- Select the analysis model to be used for the structural component analysis.

- Extract from existing design calculations the normal loadings applicable to elements of the analysis model.

- Determine hydrostatic load for elements of the analysis model due to short-term ground water level at grade.

- Analyze the structure for the specified load combination and determine the internal forces on the selected structural component.

- Compare the resulting internal forces of the structural component with the ACI code allowable values.

The evaluation of selected components in four buildings are described in the following subsections. The results are summarized in section 6.0.

5.1 Reactor Building

The containment vessel foundation mat was selected for evaluation. The mat is spherical in shape and is embedded (grade elevation +20.0 ft.). Below elevation 0.0 ft., the mat is not reinforced (except for nominal temperature and shrinkage reinforcements) and was not treated as a structural element in the original design. Between elevation 0.0 ft. and +20.0 ft. the mat is reinforced and was originally designed as a ring subjected to internal and earth pressure as well as vertical loads. This ring is laterally supported at elevation 0.0 ft. by the foundation mat for the structures inside the containment vessel. The ring has a minimum thickness of 3'-6". This ring was analyzed conservatively assuming that the inside edge is anchored to the interior foundation structure. It was subjected to upward and horizontal pressures due to weight of the superstructure, soil pressure, and hydrostatic pressure. The ground water level was assumed to be at grade (elevation +20.0 ft). Moments and shears at critical sections were determined, and compared with code allowables.

5.2 Enclosure Building

The enclosure building walls are supported by continuous spread footings of three different sizes. The footing, located within the north-west quadrant

of the containment vessel, and next to the dog house and pipe trench was judged to be the most critical and was selected for evaluation. This footing is 3'-3" thick and 11.0 ft. wide.

The ground water level was assumed at grade (elevation +19.75 ft.). The dead load and live load forces transmitted to the footing from the building wall were derived from the original design calculations. A unit-width strip of the footing was analyzed as a one-way slab to determine the internal bending moments and shear due to external loadings. Results of the analysis were compared with the code allowable values.

5.3 Reactor Auxiliary Building

Evaluations of the five reactor auxiliary building sample components are described below. In these evaluations, the ground water level was assumed to be at the elevation of the ground surface adjacent to the building components.

(a) East wall below grade

The part of the east wall near the charging pump room is the thinnest (1'-6" thick) and was selected for evaluation. It was analyzed as a plate. It is supported by the base slab, roof slab (at elevation +20.0 ft.), and the interior walls. It was subjected to soil and hydrostatic pressure loadings.

(b) South wall below grade

The south wall of the reactor auxiliary building below grade is divided into three panels by two interior cross walls. There is no moment connection between this wall and the interior cross walls. At the two ends and at the top and bottom, the south wall reinforcements are such that these edges will develop restraining moments when the wall is subjected to lateral earth and hydrostatic pressure. To determine the lower-bound capacity of the wall with appropriate consideration of the

edge restraining moments, a yield-line analysis was performed. Several failure patterns were investigated to determine the lower-bound loading capacity of the most critical panel of the wall. This capacity was compared with the computed hydrostatic and soil pressure load [resulting from load combination (ii) shown in section 2.2.2] to determine the available margin-of-safety.

The newly-computed load for load combination (ii) was also compared with the load resulting from the equivalent load combination that included DBE (taken from earlier seismic reanalysis calculations). The new load was significantly less than the earlier load for which the ductility demand was computed to be 2.38. Thus, it was concluded that the ductility demand in the wall when subjected to the newly-defined short-term hydrostatic load would be less than 2.38, and hence, would satisfy the evaluation criteria presented in section 2.3.2.

(c) West wall below grade

The west wall at the liquid radwaste hold-up tank area (at the south end of the building) was selected for evaluation because of its longer span and larger loads. The wall was treated as a one-way slab spanning in the vertical direction. Its top is simply supported by the roof slab at elevation +20.0". At the bottom (elevation -2.0') it has moment connection with the base slab. Therefore, the west wall in this area of the building was analyzed together with the base slab and the east wall as a frame-type structure using a unit-width strip of the walls and the slab (see also item 'd' below). The wall was subjected to lateral soil and hydrostatic pressure loads. The effect of back-fill soil condition resulting from the earlier addition of the switchgear enclosure foundation was also taken into consideration. Results of the analysis provided bending moments and shears at critical locations of the wall, which were then compared with code allowable values.

(d) Base slab in the liquid radwaste area

The portion of the base slab located at the south end of the reactor auxiliary building is 2'-4" thick. It is a continuous one-way slab in the east-west direction, and has three equal spans. Two intermediate supports are provided by interior walls. The two ends of the base slab are supported by the west and the east exterior walls. The connection between the base slab and these two walls are moment connections. To determine internal moments, shears, and axial forces, a unit-width strip of the base slab and the walls were modeled as a frame. The base slab was subjected to dead, live, and hydrostatic pressure loads.

(e) Base slab in the charging pump area

The base slab in the charging pump area was selected because of its longer span. In this area, the slab is 2'-4" thick and is 26 ft. by 27 ft. in plan. The slab was idealized as a flat plate with fixed supports at four sides, and was subjected to pressure loads due to dead, live, and hydrostatic pressure loads. The resulting moments and shears were then compared with code allowable values.

5.4 Intake Structure

During the earlier seismic reevaluation program, the circulating water intake structure was investigated for the design basis earthquake (DBE) event. Results of this study showed that for the north, south, and east walls at the pumpwell area, the DBE loads exceeded the allowable values and modification of the walls were proposed. The modification consisted of adding horizontal reinforced concrete beams at elevation 6'-5" along the interior face of the north, south, and the east walls. The intake structure was reanalyzed for this modified configuration in 1983. For the present evaluation, the same configuration was used. However, following the assumption used in the flotation evaluation (see section 4.3), the water

level in the intake channel in the pump well area was conservatively assumed to be at elevation -23.0 ft. The ground water table was assumed at plant grade (elevation +15'-0").

In some locations of the intake structure, sea water has caused corrosion of rebars in the interior face of walls and slabs. Repair works were performed in these areas to restore the capabilities of the wall and slabs to resist the DBE. These conditions of the structure were not explicitly considered in the present evaluation because a comparison of DBE and short-term hydrostatic loads (due to water table at grade) showed that DBE loads are greater.

Evaluation of the four structural components of the intake structure are described in the following paragraphs:

(a) North pumpwell wall

North pumpwell wall was selected because it was one of the critical structural elements identified earlier while performing seismic reanalysis and reevaluation. Presently, for evaluating the effect of ground water at grade (elevation +15.0'), the wall panel above elevation -7.75' was judged to be critical due to its larger span and end support conditions. The loadings acting on this wall panel were divided into two parts: L1 and L2. L1 consisted of the static loads corresponding to normal design loading condition, including hydrostatic loads corresponding to normal ground water level (+5.0 ft.). L2 was the load due to the additional hydrostatic head resulting from ground water level at plant grade. Internal moments and shears induced by loading L1 were obtained directly from earlier analysis results. Internal forces due to loading L2 were determined conservatively by idealizing the wall panel as an one-way slab in the vertical direction. These were added to the forces resulting from L1 to obtain the total design forces.

The north pumpwell wall panel below elevation -7.75' was analyzed using yield-line method and assuming conservatively that there would be no water in the pumpwell because of blocked intake screen.

(b) East pumpwell wall

Like the north walls, the east pumpwell wall was selected because it was also one of the critical structural elements identified in reference 6 seismic reevaluation. For this wall also, the detailed evaluation was performed for only the portion above elevation -7.5' which consists of four panels. Each panel was analyzed to determine internal forces due to applied loads. The evaluation methodology used was similar to that used for the north pumpwell wall (see (a) above).

For the portion of the wall panel between elevations -7.75' and -26.0', explicit analysis and evaluation was not necessary because comparison of the short-term hydrostatic loads (due to water table at grade) with DBE loads showed that the DBE loads are larger.

(c) Horizontal beam at the east pumpwell wall

The horizontal beam in the east pumpwell wall at elevation 5'-3" was evaluated. The beam has three spans and is supported at the intake box culvert walls and the north and south pumpwell walls. The middle span of the beam between intake box culvert walls was the most critical. The beam was evaluated for the reaction loads from adjacent wall panels and lateral pressure applied directly to the beam surface.

(d) Intake box culvert

Intake box culvert is 10.0 x 10.5 ft. in cross-section and is 39'-7" long. The two side walls are 1'-6" thick, and the top and bottom slabs are 1'-3" thick. The out-of-plane bending of the four side walls was evaluated for the short-term hydrostatic loads resulting from ground water table at plant grade (elevation +15'-0"). The culvert was assumed full and a unit-width strip of the four walls was analyzed as a

frame. The walls were subjected to lateral pressure loadings. This analysis provided the internal moments and shears induced in the four walls. These values were then compared with allowables specified in section 2.3.

Following the earlier seismic reevaluation, an additional evaluation of the culvert walls was performed assuming that the rebars on their inside face have lost strength due to corrosion. Based on the criteria used in the earlier seismic reevaluation, this configuration of the culvert walls was evaluated by taking credit of the allowable flexural tensile strength of concrete as

$$f_{bt} = 5\phi\ (f'_i)^{0.5}$$

where, ϕ = 0.65, and

f'_i = ultimate compressive strength of concrete in psi.

6.0 EVALUATION RESULTS

This section summarizes the results of evaluation described in sections 4 and 5. The results of the flotation effect evaluation are given in table 6-1 and the results of the structural component adequacy evaluation are given in table 6-2. Table 6-2 also presents the internal forces resulting from an equivalent load combination that includes DBE when such loads are readily available. The DBE values were taken from earlier seismic reanalysis calculations, and are presented for the purpose of comparison only.

In tables 6-1 and 6-2, minimum margins-of-safety against short-term hydrostatic loads due to ground water table at grade have been presented. The margin-of-safety has been defined as the ratio of the allowable value (per evaluation criteria, see section 2.3) to the computed value.

Table 6-1 shows that the minimum margin-of-safety against flotation is 1.25. This value, as well as other safety margins shown in table 6-1, is conservative, because the frictional resistance between building and soil against flotation was not considered. Table 6-2 shows that the margins-of-safety for structural components are well in excess of unity.

TABLE 6-1. RESULTS OF FLOTATION EVALUATION

Description of Structure	Computed Downward[1] Load (Kips)	Buoyant Force (Kips)	Safety[1] Margins
Reactor Building	46,910	21,886	2.14
Reactor Auxiliary Bldg.	13,421	10,684	1.26
Circulating Water Intake Structure	19,749	15,767	1.25
Fuel Oil Storage Tank	680	441	1.54

Note: (1): These values are conservative since the frictional resistance (between building and soil) was not considered.

TABLE 6-2. RESULTS OF STRUCTURAL COMPONENT EVALUATION

Structural Component	Force[2] Components	Computed[1] Values	Allowable Values	Safety Margin
I. Reactor Building				
(a) Foundation Mat				
(i) At elev. +10'-0"	-Moment	36.5	182.4	5.0
	Shear	10.6	44.7	4.2
(ii) At elev. 0'	-Moment	245.2	530.8	2.2
	Shear	32.5	44.7	1.4
II. Enclosure Building				
(a) Wall Footing	-Moment	163.4 (288.0)	317.0	1.9
	Shear	22.1 (38.9)	45.0	2.0

Notes:

1. Values in the parenthesis are from earlier analyses for the load combination which included DBE loads; these are provided here for the purpose of comparison only.

2. Negative moment produces tension on the exterior face, and positive moment produces tension on the interior face. Moment values are in Kip-ft./ft., and shear values are in Kips/ft.

TABLE 6-2. RESULTS OF STRUCTURAL COMPONENT EVALUATION (continued)

Structural Component	Force[2] Components	Computed[1] Values	Allowable Values	Safety Margin
Reactor Auxiliary Building				
(a) East wall below grade				
(i) Vertical direction	-Moment	8.1 (16.3)	44.4	5.5
	+Moment	2.2 (4.6)	29.5	13.4
	Shear	5.2 (10.0)	17.2	3.3
(ii) Horizontal direction	-Moment	2.9 (6.0)	13.4	4.6
(b) South wall below grade				
Lateral load (per ft.)	Pressure	19.2	116.6[3]	6.1

Notes:

1. Values in the parenthesis are from earlier analyses for the load combination which included DBE loads; these are provided here for the purpose of comparison only.

2. Negative moment produces tension on the exterior face, and positive moment produces tension on the interior face. Moment values are in Kip-ft./ft., and shear and pressure values are in Kips/ft.

3. Ultimate capacity.

TABLE 6-2. RESULTS OF STRUCTURAL COMPONENT EVALUATION (continued)

Structural Component	Force[2] Components	Computed[1] Values	Allowable Values	Safety Margin
Reactor Auxiliary Building (continued)				
(c) West wall below grade	-Moment	46.7 (128.5)	235.3	5.0
	+Moment	49.2 (66.4)	75.4	1.5
	Shear	20.1 (25.3)	29.8	1.5
(d) Base slab in the Liquid Radwaste area	-Moment	46.7 (72.6)	209.3	4.5
	+Moment	5.0 (58.2)	86.6	21.7
	Shear	4.2 (22.6)	26.4	6.3
(e) Base slab in the charging pump area	-Moment	17.5 (56.9)	69.6	4.0
	+Moment	7.5 (24.3)	112.8	15.0
	Shear	5.7 (18.5)	28.8	5.1

Notes:

1. Values in the parenthesis are from earlier analyses for the load combination which included DBE loads; these are provided here for the purpose of comparison only.

2. Negative moment produces tension on the exterior face, and positive moment produces tension on the interior face. Moment values are in Kip-ft./ft., and shear values are in Kips/ft.

TABLE 6-2. RESULTS OF STRUCTURAL COMPONENT EVALUATION (continued)

Structural Component	Force[2] Components	Computed[1] Values	Allowable Values	Safety Margin
IV. Circulating Water Intake Structure				
(a) North pumpwell wall above elev. -7.75'				
(i) Horizontal direction	-Moment	20.7 (20.7)	32.2	1.56
	+Moment	16.9 (16.9)	25.0	1.48
	Shear	21.3 (21.3)	28.7	1.35
(ii) Vertical direction	-Moment	115.1(179.3)	258.0	2.24
	+Moment	4.3 (28.7)	25.6	5.95
	Shear	25.4 (35.7)	37.4	1.47
(b) North pumpwell wall below elev. -7.75'				
Lateral load (per ft)	Pressure	52.8	85.9[3]	1.63
(c) East pumpwell wall				
(i) Horizontal direction	-Moment	9.9 (30.59)	18.8	1.90
	+Moment	4.94	18.8	3.81

Notes:

1. Values in the parenthesis are from earlier analyses for the load combination which included DBE loads; these are provided here for the purpose of comparison only.
2. Negative moment produces tension on the exterior face, and positive moment produces tension on the interior face. Moment values are in Kip-ft./ft., and shear and pressure values are in Kips/ft.
3. Ultimate capacity.

TABLE 6-2. RESULTS OF STRUCTURAL COMPONENT EVALUATION (continued)

Structural Component	Force[2] Components	Computed[1] Values	Allowable Values	Safety Margin
IV. Circulating Water Intake Structure (continued)				
(b) East pumpwell wall (continued)				
(ii) Vertical direction	-Moment	13.8 (42.77)	103.0	7.46
	+Moment	6.8 (21.02)	36.7	5.40
	Shear	9.1 (28.06)	28.7	3.15
(c) Horizontal beam at the east pumpwell wall	-Moment	235.7	834.2	3.54
	+Moment	117.9	662.9	5.62
	Shear	85.7	207.5	2.42
(d) Intake box culvert				
	-Moment	4.1 (8.42)	29.2	7.12
	+Moment	3.0 (7.46)	47.2	15.73
	Shear	3.1 (7.02)	17.0	5.48
	Tension[3]	134.7 psi	218.0 psi	1.62

Notes:

1. Values in the parenthesis are from earlier analyses for the load combination which included DBE loads; these are provided here for the purpose of comparison only.
2. Negative moment produces tension on the exterior face, and positive moment produces tension on the interior face. Moment values are in Kip-ft./ft., and shear values are in Kips/ft.
3. Flexural tensile stress in concrete for the case when inside face reinforcing bars are assumed structurally ineffective.

7.0 CONCLUSIONS

The effects of short term hydrostatic loads due to ground water table at plant grade have been evaluated on a sampling basis. Four structures were evaluated for the overall flotation effects, and eleven safety-related building components were evaluated for structural adequacy. The structures and components which were judged to be more critical than others were selected for evaluation. The evaluation results showed that the structures have more than adequate margins-of-safety against flotation; also, the stress levels in building structural components are significantly less than the allowable values. A comparison of the stress levels showed that the stresses due to short term hydrostatic loadings are consistently lower than those due to DBE.

From the results of this sample-basis evaluation, it is concluded that the safety-related structures in SONGS 1 are adequate to withstand the effects of short-term hydrostatic loads due to ground water table at plant grade.

8.0 REFERENCES

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2. Letter from K. P. Baskin (SCE) to D. M. Crutchfield (NRC), Systematic Evaluation Program, Topic III-6, Seismic Design Considerations, SONGS 1, February 23, 1981.

3. Quadrex Report No. QUAD-7-86-019, "Structural Evaluation Criteria and Procedure for Evaluating the Effects of Ground Water Table at Plant Grade, SONGS 1", May 5, 1986.

4. American Concrete Institute, "Code Requirements for Nuclear Safety-Related Concrete Structures," ACI 349-76.

5. Letter from W. A. Paulson (NRC) to R. Dietch (SCE); Subject: SEP Hydrology Topics 11-3.B, II-3.5.1, and II-3.C, SONGS 1, January 31, 1983.

6. Letter from M. O. Medford (SCE) to D. M. Crutchfield (NRC), Subject: Docket No. 50-206 Systematic Evaluation Program Integrated Assessment, SONGS 1, May 7, 1984.

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8. Letter from R. W. Krieger (SCE) to D. M. Crutchfield (NRC), Docket No. 50-206, Systematic Evaluation Program, Topic III-3.A, SONGS 1, October 20, 1983.