November 13, 1984

Docket No.: 50-206
LS05-84-11-011

Mr. Kenneth P. Baskin, Vice President
Nuclear Engineering
Licensing and Safety Department
Southern California Edison Company
2244 Walnut Grove Avenue
Post Office Box 800
Rosemead, California 91770

Dear Mr. Baskin:

SUBJECT: SAN ONOFRE NUCLEAR GENERATING STATION, UNIT 1 - SEP TOPIC II-4.F, SETTLEMENT OF STRUCTURES AND BURIED EQUIPMENT

Enclosed is the staff's final Safety Evaluation Report on SEP Topic II-4.F, "Settlement of Structures and Buried Equipment" for San Onofre Generating Station, Unit 1. This evaluation specifically addresses the potential settlement of the sea wall that could result from a safe shutdown earthquake. As discussed in the evaluation, the staff concludes that such settlement would not prevent the sea wall from protecting the plant from flooding during a tsunami. However, our evaluation concludes that cracking of the concrete cover of the sea wall and pavement adjoining the sea wall may occur. Such damage would not prevent the sea wall from fulfilling its safety function during the tsunami but would be expected to require repair.

The staff's review of the geotechnical aspects of the site is complete except for a few structures that rest on uncompacted backfill. Consequently, the evaluation of the potential for settlement of those structures will be conducted in parallel with the evaluation of their seismic integrity under SEP Topic III-6, Seismic Design Considerations.

Sincerely,

Original signed by/

John A. Zwolinski, Chief
Operating Reactors Branch #5
Division of Licensing

Enclosure: As stated

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John Z. Zwolinski, Chief
Operating Reactors Branch #5
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The staff's review of the geotechnical aspects of site as they related to the other Seismic Category I structures is not yet complete because of the status of the evaluation of the seismic capability of those structures. Consequently, the evaluation of the potential for settlement of those structures will be addressed as part of the evaluation of their seismic integrity under SEP Topic III-6, Seismic Design Considerations.

Sincerely,

Walter A. Paulson, Acting Chief
Operating Reactors Branch #5
Division of Licensing

Enclosure: As stated

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10/26/84	10/25/84	10/ 1 /84	10/ /84	10/ /84

Mr. Kenneth P. Baskin

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Region IX Office
ATTN: Regional Radiation Representative
215 Freemont Street
San Francisco, California 94105

John B. Martin, Regional Administrator
Nuclear Regulatory Commission, Region V
1450 Maria Lane
Walnut Creek, California 94596

SYSTEMATIC EVALUATION PROGRAM
SAFETY EVALUATION REPORT
SAN ONOFRE NUCLEAR GENERATING STATION, UNIT 1
TOPIC II-4.F, SETTLEMENT OF STRUCTURES AND BURIED EQUIPMENT

I. INTRODUCTION

Topic II-4.F pertains to the geotechnical engineering review of the properties and stability of subsurface materials and foundations as they influence the static and seismically induced settlement of the plant's critical structures and buried equipment.

II. REVIEW CRITERIA

This Topic was reviewed using the following criteria:

A. 10 CFR Part 50, Appendix A

1. General Design Criterion (GDC)1: "Quality Standards and Records." This criterion requires that structures, systems and components important to safety be designed, fabricated, erected, and tested to quality standards commensurate with the importance of the safety functions to be performed. It also requires that appropriate records of the design, fabrication, erection, and testing of structures, systems, and components important to safety be maintained by or under the control of the licensee throughout the life of the plant.

2. GDC 2: "Design Bases for Protection Against Natural Phenomena." This criterion requires that safety-related portions of the system be designed to withstand the effects of earthquakes, tornadoes, hurricanes, floods, tsunami, and seiches without loosing the capability to perform their safety functions.

B. 10 CFR Part 100, Appendix A

"Seismic and Geologic Siting Criteria for Nuclear Power Plants". These criteria describe the investigations required to obtain the geologic and seismic data needed to determine site suitability and identify the geologic and seismic factors that must be taken into account in the siting and design of nuclear power plants.

III. RELATED SAFETY TOPICS AND INTERFACES

The geotechnical engineering aspects of slope stability are reviewed under Topic II-4.D. Other topics that interface with II-4.F include:

II-3.B Flooding Potential and Protective Requirements
II-3.C Safety-Related Water Supply (Ultimate Heat Sink)
II-4.E Dam Integrity

III-3.A Effects of High Water Level on Structures
III-3.C In-Service Inspection of Water Control Structures
III-6 Seismic Design Considerations

IX-3 Station Service and Cooling Water Systems

XVI Technical Specifications

IV. REVIEW GUIDELINES

In general, the review was conducted in accordance with Section 2.5.4 of the NRC Standard Review Plan (NUREG-0800) (Reference 1). The geotechnical engineering aspects of the design and as-constructed conditions were reviewed and compared to current criteria, and the safety significance of any differences was evaluated.

The following NRC Regulatory Guides (RG) were used in the review.

A. RG 1.132, "Site Investigations for Foundations of Nuclear Power Plants." This guide describes geotechnical engineering site investigations programs that would normally meet the needs for evaluating the safety of site from the standpoint of how the foundation and earthworks would comply with 10 CFR 100 and 10 CFR 100, Appendix A under anticipated loading conditions, including earthquakes. RG 1.132 provides general guidance and recommendations for developing site-specific investigation programs as well as specific guidance for conducting subsurface investigations, the spacing and depth of borings, and sampling.

B. RG 1.138, "Laboratory Investigation of Soils for Engineering Analysis and Design of Nuclear Power Plants." This guide describes laboratory investigations and testing practices acceptable for determining the soil and rock properties and characteristics that are needed for the engineering analysis and design of nuclear power plant foundations and earthwork so they will comply with 10 CFR 100, and 10 CFR 100, Appendix A.

V. EVALUATION

A. Site Description

The San Onofre Nuclear Generating Station Unit 1 (SONGS-1) is located on the Camp Pendleton Marine Reservation on the coast of California in San Diego County about 51 mi northwest of San Diego and about 62 mi southeast of Los Angeles.

The topographic features of the immediate coastal area include a narrow band of beach sand terminating as seacliffs that reach a height of 60 to 80 ft in the vicinity of the site. A gentle coastal plain extends inland to the western foothills of the Santa Margarita Mountain Range approximately 1-1/2 mi to the east. The plant site is on the shoreline. Before the start of plant construction, the plant site elevation at the top of the seacliff bluff ranged from +80.0 to +115.0 ft MLLW. The finished plant grade elevation is +20.0 ft MLLW in the immediate vicinity of the sphere enclosure building, and + 15.0 ft in the vicinity of the sea wall.

The staff's evaluation of SEP Topic II-4.F assumes a ground water level of el + 5.0 ft. This assumption is consistent with the results of the staff's review of SEP Topic II-3.A, "Hydrologic Description."

The subsurface soil exposed on grade level and in excavation for plant facilities include the Quarternary terrace deposits that overlie a Pliocene age sand material named the San Mateo Formation. The terrace deposits consist of tan, buff, and light brown, silty or clayey, fine to coarse sand with some cobbles. These deposits are crudely stratified with a thickness of up to 55 ft. The San Mateo Formation is a cemented, massive, well-graded, yellow-brown, fine to coarse sand with gravel and occasional lenses of thin-bedded gray shale or siltstone and is approximately 1,000 ft thick at the site. At grade, the San Mateo Formation is a poorly cemented but very dense sand.

To accommodate the plant, the seacliff bluff was cut back using a "bench design" approach. Cut slope profiles consist of a 15-ft wide bench at the interface of the terrace deposit and the San Mateo Formation. The San Mateo Formation comprises the lower 25 ft of the cut slope. Above and below the bench, the cuts were excavated to a slope of one horizontal to two vertical (Ref. 3).

The seismic Category I structures at this plant include a reactor containment structure and sphere enclosure building, a turbine building and turbine pedestal, an administration and control building, a circulating water system intake structure (pump well), a diesel generator building, a refueling water storage tank, a condensate storage tank, and a sea wall. Figure 1 shows the location of the SONGS-1 sea wall.

B. Description of the Sea Wall

The sea wall is a steel sheet pile wall intended to protect the plant from flooding during and after a tsunami event. The wall is approximately 600 ft long with a top elevation of +28.2 ft and a bottom elevation of -8.0 ft. The ground surface adjoining the sea wall is at el + 15.0 ft on the plant side and at el + 14.0 ft on the beach side. The wall consists of MZ-27 sheet pile sections driven through granular material into native San Mateo sand.

The wall is covered with 2.5 inches of concrete. The wall was constructed in 1966, and a beach walkway was constructed on the sea side of the wall in 1981. Figures 2 and 3 show plan and cross section views of the sea wall.

C. Foundation Conditions

Reference 4 presents the NRC staff's evaluation of the soil conditions at this site. Figure 2 shows the subsoil profile along the sea wall based on data from borings drilled in connection with the beach walkway. The generalized stratigraphy consists of beach sand from ground surface (el. +15.0 ft) to el +5.0 ft. The beach sand is underlain by a 5-ft (±) thick stratum of residual San Mateo sand that in turn overlies a 2-ft (±) thick stratum of gravel and cobbles. Native San Mateo sand underlies the gravel and cobble layer; all borings were terminated in this stratum. The sheet piles for the sea wall were driven to el. -8.0 ft which generally extended into the native San Mateo sand.

The intake and discharge conduits for the plant cooling water pass beneath the sea wall. Construction of these conduits required excavation to el - 29.0 ft; the trench was backfilled with granular material recovered from excavations. Here, the sheet piles were driven to el. -8.0 ft which resulted in approximately 21 ft of loose granular backfill material between the tip of the sheet pile and bottom of the trench. As part of the beach walkway construction, 30-inch diameter gravel drains were installed in the loose sand beneath the beach walkway in the vicinity of this conduit trench to dissipate excess pore pressure after a seismic event. This is the critical section of the sea wall used in the stability and settlement analyses; it is shown in Figure 3.

The backfill material was not compacted during construction of the SONGS-1. References 5, 6, and 7 present the licensee's evaluation of the condition of the backfill material at this site. During the licensee's evaluation, only limited in situ density tests were performed at the sea wall; therefore, the licensee assumed the density of the granular backfill material near the sea wall to be the lowest measured at this site. This was assumed to be granular backfill with a relative compaction of 85 percent; i.e., the in situ dry density is 85 percent of the maximum dry density determined according to the ASTM D 1557 test method. This method resulted in a dry density of 110 pcf and a relative density of 50 percent. This was the condition assumed for the sand backfill on the plant side of the wall and that below the water table on the beach side of the wall. The compacted sand and gravel fill above the water table on the beach side of the wall is said to be compacted under supervision, and was therefore assumed to be at 95 percent relative compaction or 85 percent relative density.

The sea wall was evaluated for static, design-basis seismic event and post-seismic tsunami conditions. Soil parameters appropriate for these conditions were assigned. These are shown in Figure 2. For the static load condition and tsunami condition, the soil parameters are the same, except high water level and wave forces are present during a tsunami. During an SSE, saturated sand with a relative density of 50 percent would liquefy. Therefore zero shear strength was assumed for the loose sand fill on both the plant side and beach side of the sea wall. For other conditions, soil parameters were assigned on the basis of their assumed insitu relative density.

The staff reviewed and concurs with the licensee's evaluation of the soil conditions. The staff finds the soil parameters and associated earth pressure coefficients which were used to be reasonable and acceptable.

D. Analysis

The stability and settlement of the sea wall were evaluated for static, seismic, and tsunami loading conditions.

1. Static Loading Condition

The sea wall has been in place since 1966; the structure and the adjoining sand, are in a state of equilibrium under present static load conditions. On the basis of a recently completed survey, the licensee stated that the lowest elevation of the top of the sea wall is higher than the design elevation of el.+ 28.2 ft (Reference 8). During a site visit, the staff viewed this sea wall and found it to be in a satisfactory condition (Reference 9). The staff therefore concludes that the sea wall is stable under static loading conditions, and any future settlement as a result of this loading is expected to be insignificant.

2. Seismic Loading Condition

2.1 Stability

Figures 2 and 3 show the cross-section and soil design parameters used in the analysis. The design-basis seismic event, the SSE, has a zero period acceleration of 0.67 g. The sea wall's inertia force and dynamic earth pressures were used in the analysis. The coefficients of dynamic earth pressures were determined in accordance with the recommendations of Seed and Whitman (Reference 10). The required depth of embedment of the sea wall to sustain the lateral earth pressure and inertia force was determined by the pseudo static method of analysis. The actual embedment was 18 to 35 percent more than that required for various load combinations (different directions of inertia force).

Thus, the penetration or embedment of the sheet piles used in the sea wall is satisfactory.

The analysis also determined the maximum stress in the sheet piles during seismic loading. The factor of safety, defined as the ratio of maximum allowable stress in steel to the actual maximum stress, ranged from 2.9 to 4.8 for the various load combinations mentioned above. Hence, the depth of embedment and section modulus of the sheet pile is satisfactory to sustain the SSE loading. The structural response of the sea wall during an SSE is evaluated under SEP Topic III-6.

The actual ground surface at the plant side of the sea wall is only 1 to 3 feet higher than that at the beach side; the water table is at the same elevation on both sides of the sea wall. Therefore, net lateral pressure on the sea wall is minimal. The sand and gravel, above the water table on the beach side of the wall is said to be denser than the sand on the plant side. For the loose sand below the water table on the beach side, the gravel drain and the gravel fill above it will aid in dissipating the excess pore water pressure generated during a seismic event. This dissipation will reduce the potential for liquefaction of this material. The gradation of the gravel in the gravel drain meets filter criterion, and the gravel drains are expected to be functional during a seismic event. Therefore, the material in front of the sea wall is expected to be stable during a seismic event. The loose backfill on the plant side is surrounded by non-liquefiable material, the native San Mateo's sand found in the north and south sides of the conduit trench. Loose backfill will experience excess pore pressure but has no potential for flow or excessive lateral deformation as a result of an SSE event. Dissipation of excess pore pressure will result in volume change or settlement of this sand but not lateral instability. On this basis, the staff concludes that the foundation of the sea wall will remain functional and support the wall.

2.2 Settlement

The settlement of sand during an SSE was evaluated according to the methods proposed by Silver and Seed (Reference 11) and Lee and Albaisa (Reference 12). The settlement of the sea wall is assumed equal to the settlement of the loose saturated sand below el -8.0 ft. (bottom of the sea wall). This saturated sand is located within the conduit trench. There are gravel drains in the sand in front of the sea wall. During a seismic event, these drains will dissipate the excess pore pressure and prevent the build-up of excess pore pressures leading to total liquefaction. This sand will undergo volume change and settlement as the pore pressure generated during a seismic event dissipates. The magnitude of this settlement was estimated to be 3 to 6 inches. This settlement value was estimated by a technical panel consisting of Drs. H. B. Seed, I. M. Idriss, and R. McNeill. Considering the accuracy of the methodology used in estimating the settlement, the staff independently estimated the settlement to range between 4 inches and 6 inches. The maximum settlement will be at the section where the thickness of the loose sand beneath the sea wall is 21 ft. The settlement will decrease to zero near the sides of the conduit trench where the steel sheet pile tip is founded in native San Mateo sand. Steel sheet pile walls are flexible, and vertical movement or settlement is accommodated by slipping along the joints between sheet piles. This sheet pile wall can sustain a settlement of 6 inches over an 80-foot stretch (the width of conduit trench) and still maintain its ability to protect the plant from flooding during a tsunami. The concrete cover on the sheet piles may crack during settlement of the sea wall, but the steel sheet pile wall will be intact and be able to fulfill its safety function.

After an SSE, the licensee must shut down the plant and assess and qualify the safety of all seismic Category I structures before restarting the plant. This requirement will ensure that the sea wall will be repaired to its original condition before the plant is restarted.

The settlement of the loose granular backfill between the ground surface and the bottom of the sea wall (el. -8.0 ft), will cause subsidence of the ground surface on both sides of the sea wall. This settlement is estimated to be in the range of 3 inches to 4 inches, in addition to the settlement of 4 to 6 inches estimated for the sea wall. Thus, the total settlement or subsidence of the ground surface adjoining the sea wall will be between 7 and 10 inches, and will result in cracking of the pavement adjoining the sea wall. Although the sea wall will be in place and protect the plant against flooding, extensive remedial work may be necessary after the tsunami to repair both the sea wall and the adjoining pavement.

On the basis of the above analysis, the staff concludes that the sea wall will be stable and functional during and after an SSE.

3. Tsunami Loading Condition

The probable maximum tsunami expected at SONGS-1 would be generated by local offshore seismic activity. The tsunami will reach the site about 8 minutes after the SSE. If the tsunami occurs simultaneously with an astronomical tide of 7.0 ft, an isostatic anomaly of 0.33 ft, and a surge of 1.98 ft, the total runup elevation of the tsunami will be el + 15.6 ft. Because the tsunami has a long period (approximately 12 minutes), the tsunami runup elevation is the still water level in conjunction with storm waves. During a tsumani, the plant should be protected against a water level of el + 15.6 ft. as well as a 7-foot high wave and wave splash, for a total splash elevation of +27.5 ft. The sea wall was analyzed for a static load condition plus the hydrostatic and hydrodynamic forces from the above water level and wave. For these loads, the actual embedment of the sea wall is 75 percent more than the required embedment. The factor of safety against overstressing the steel sheet pile is 3.04. Therefore, the sea wall will be stable under tsunsmi condition. Because the tsunami loading is less severe than the seismic loading, the foundation of the sea wall will also be stable during a tsunami. The beach walkway and the gravel fill on the beach side of the wall were found to be stable against hydrodynamic forces during tsunami (Reference 2).

On the basis of the above reasoning the staff concludes that the sea wall will be stable during tsunami.

It is estimated that the sea wall will settle a maximum of 6 inches as a result of an SSE; additional settlement due to the tsunami will be insignificant.

VI. CONCLUSIONS

On the basis of its review of information submitted by the licensee, the NRC docket file, and information obtained during a site visit, the staff concurs with the licensee that settlement of the sea wall will not preclude the sea wall from protecting the plant from flooding during a tsunami. The sea wall meets General Design Criterion 2, and 10 CFR Part 100, Appendix A. However, after an SSE and tsunami, damage to the wall and the pavement adjoining the sea wall would be expected.

The staff's review of geotechnical engineering aspects of the San Onofre Unit 1 site is generally complete except for a few structures that rest on uncompacted backfill. Consequently, the potential for settlement of those structures will be evaluated in parallel with the evaluation of their seismic integrity under SEP Topic III-6, "Seismic Design Considerations."

VII. REFERENCES

1. U. S. Nuclear Regulatory Commission. NUREG-0800, "Standard Review Plan for the Safety Review of Nuclear Power Plants," July 1981.

2. Letter from W. Paulson of NRC to R. Dietch of Southern California Edison Company, dated January 31, 1983, Subject: Hydrologic Engineering Safety Evaluation Report - SEP Topic II-3.A, II-3.B, II-3.B.1, and II-3.C, SONGS-1.

3. Southern California Edison Company "Final Safety Analysis Report, San Onofre Nuclear Generating Station, Unit 1, Docket No. 50-206."

4. Letter from W. Paulson of NRC to R. Dietch of Southern California Edison Company, dated December 1, 1982, Subject: SEP Topic II-4.F, Settlement of Structures and Buried Equipment, San Onofre Nuclear Generating Station, Unit 1.

5. "Report of Soil Backfill Conditions - San Onofre Nuclear Generating Station, Unit 1," August 12, 1982, submitted under cover letter from K. P. Baskin, Southern California Edison to D. M. Crutchfield, NRC dated August 17, 1982, Subject: "In-situ Soil Conditions SEP Topic III-6, Seismic Design Considerations San Onofre Nuclear Generating Station, Unit 1."

6. Letter from K. P. Baskin of SCEC to D. M. Crutchfield of NRC, dated April 18, 1983, Subject: SEP Topic II-4.F, San Onofre Nuclear Generating Station, Unit 1. Transmitted revised Sections 1, 2, and 3 and appendices of August 17, 1982 report on Soil Backfill Conditions.

7. Letter from R. W. Krieger of SCEC to D. M. Crutchfield of NRC, dated September 1, 1983, Subject: SEP Topic II-4.F, San Onofre Nuclear Generating Station, Unit 1. Transmitted revisions and addendum to their April 18, 1983, Submittal on Soil Backfill Conditions.

8. Letter from M. O. Medford, SCEC, to D M. Crutchfield, NRC dated May 17, 1984, Subject: "SEP Topic II-4.F, San Onofre Nuclear Generating Station Unit 1."

9. Memo dated April 2, 1984, from B. Jagannath, NRC, to G. Lear, Subject: Report on Site Visit and Technical Review Meeting, February 8, and 9, 1984 - Geotechnical Aspects.

10. Seed, H. B. and R. v. Whitman, "Design of Earth Retaining Structures for Dynamic Loads," ASCE Speciality Conference on Lateral Stresses and Earth Retaining Structures, 1970.

11. M. L. Silver and H. B. Seed - "Volume Changes in Sands During Cyclic Loading," ASCE Journal of the Soil Mechanics and Foundation Division, Vol. 97, No. SM9, September 1971, pp. 1171-1182.

12. K. L. Lee and A. Albaisa - "Earthquake Induced Settlements in Saturated Sands," ASCE Journal of the Geotechnical Engineering Division, Vol. 100, No. GT4, April 1974, pp. 387-406.

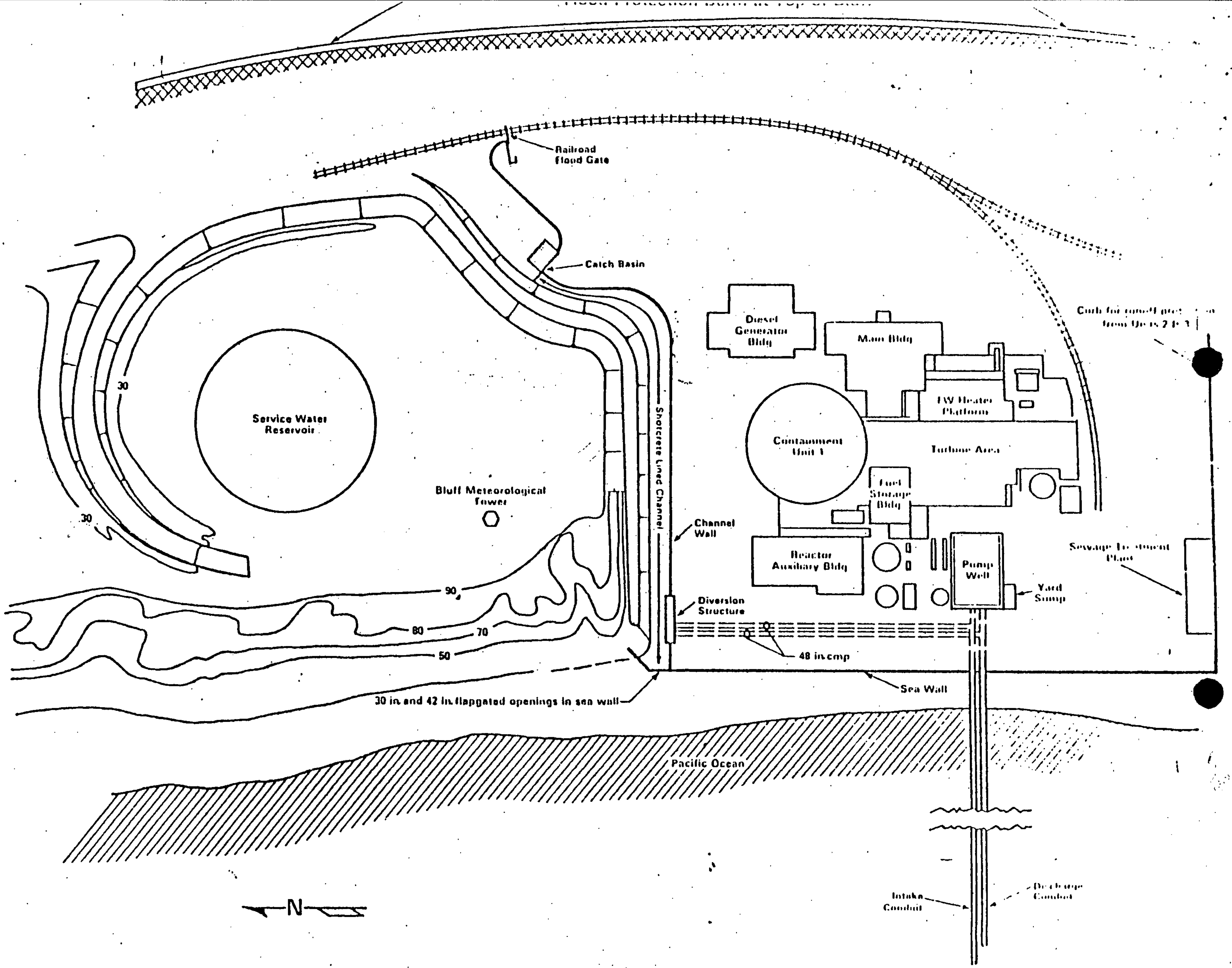
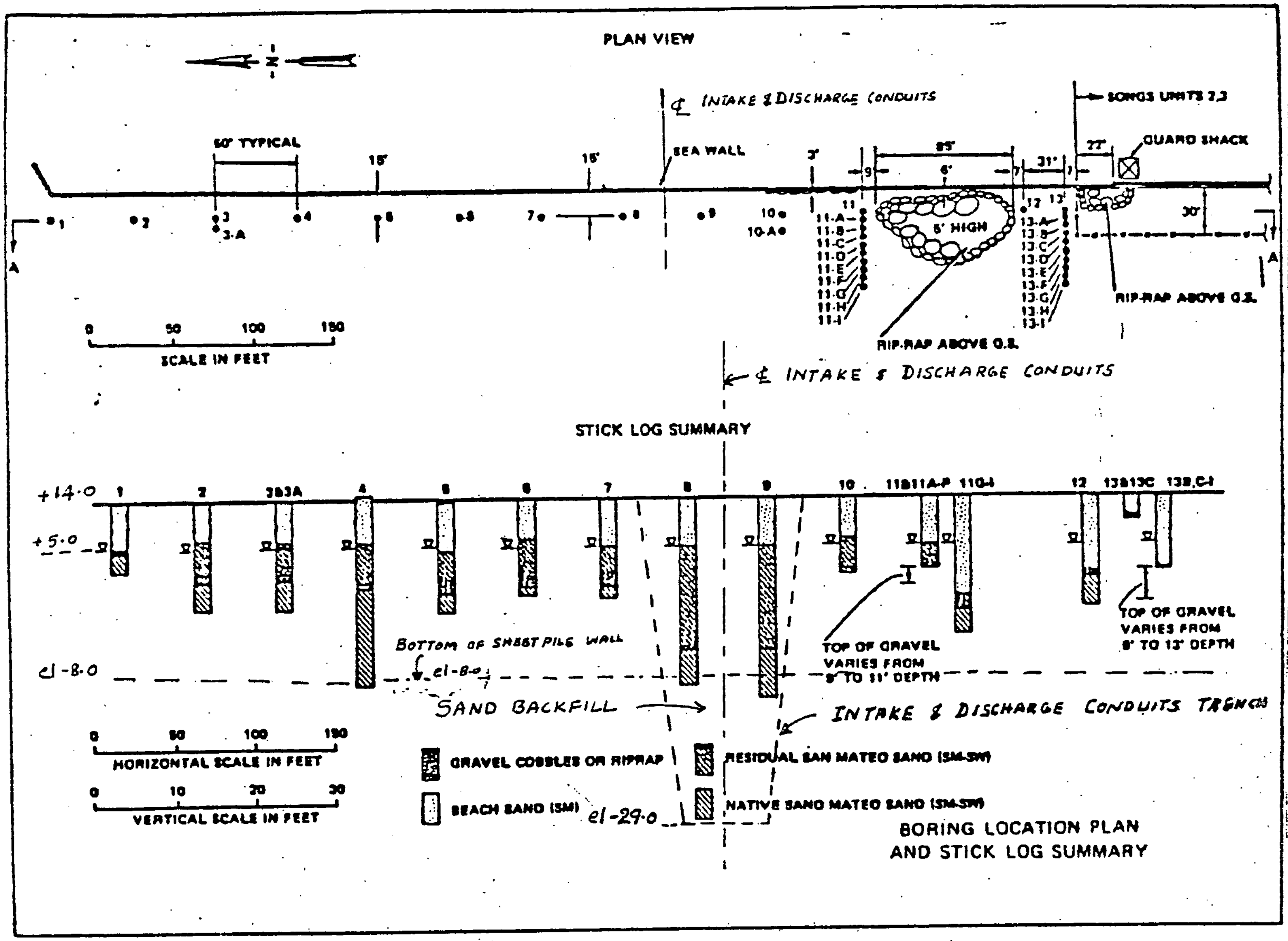
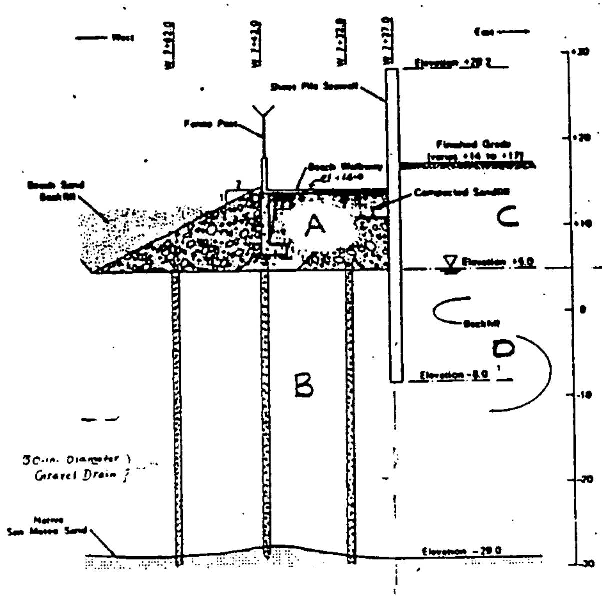


Figure 1 - San Onofre - General Site Plan





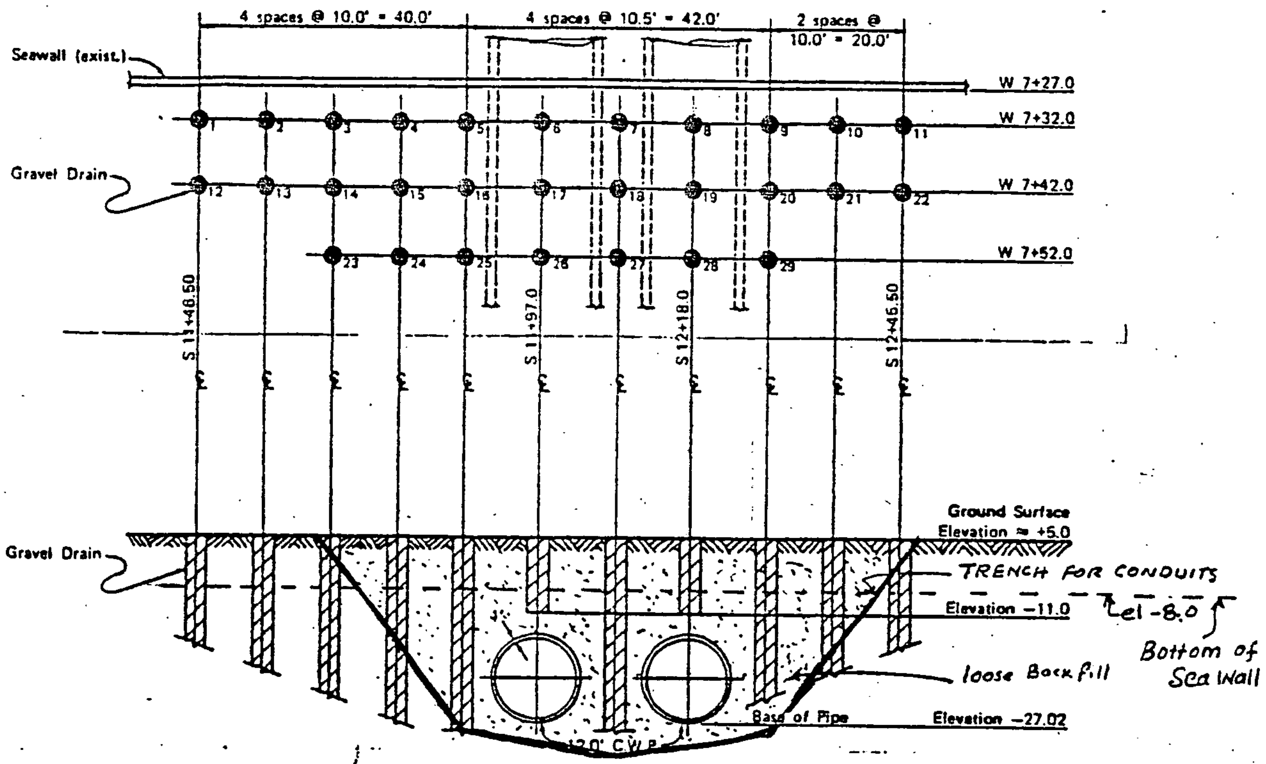
SOIL DESIGN PARAMETERS

LOADING CONDITION	Soil A	Soil B	Soil C	Soil D
SEISMIC (SSE)				
Total Density, pcf	120	-	110	-
Submerged Density, pcf	66	61	-	61
Active Earth Pressure Coefficient Ka	0.57	1.00	1.00	1.00
Passive Earth Pressure Coefficient Kp	2.15	1.00	1.00	1.00
NON-SEISMIC (Static, tsunami)				
Total Density, pcf	120	-	110	-
Submerged Density, pcf	66	61	-	61
Ka	0.21	0.49	0.31	1.00
Kp	4.80	2.03	3.25	1.00

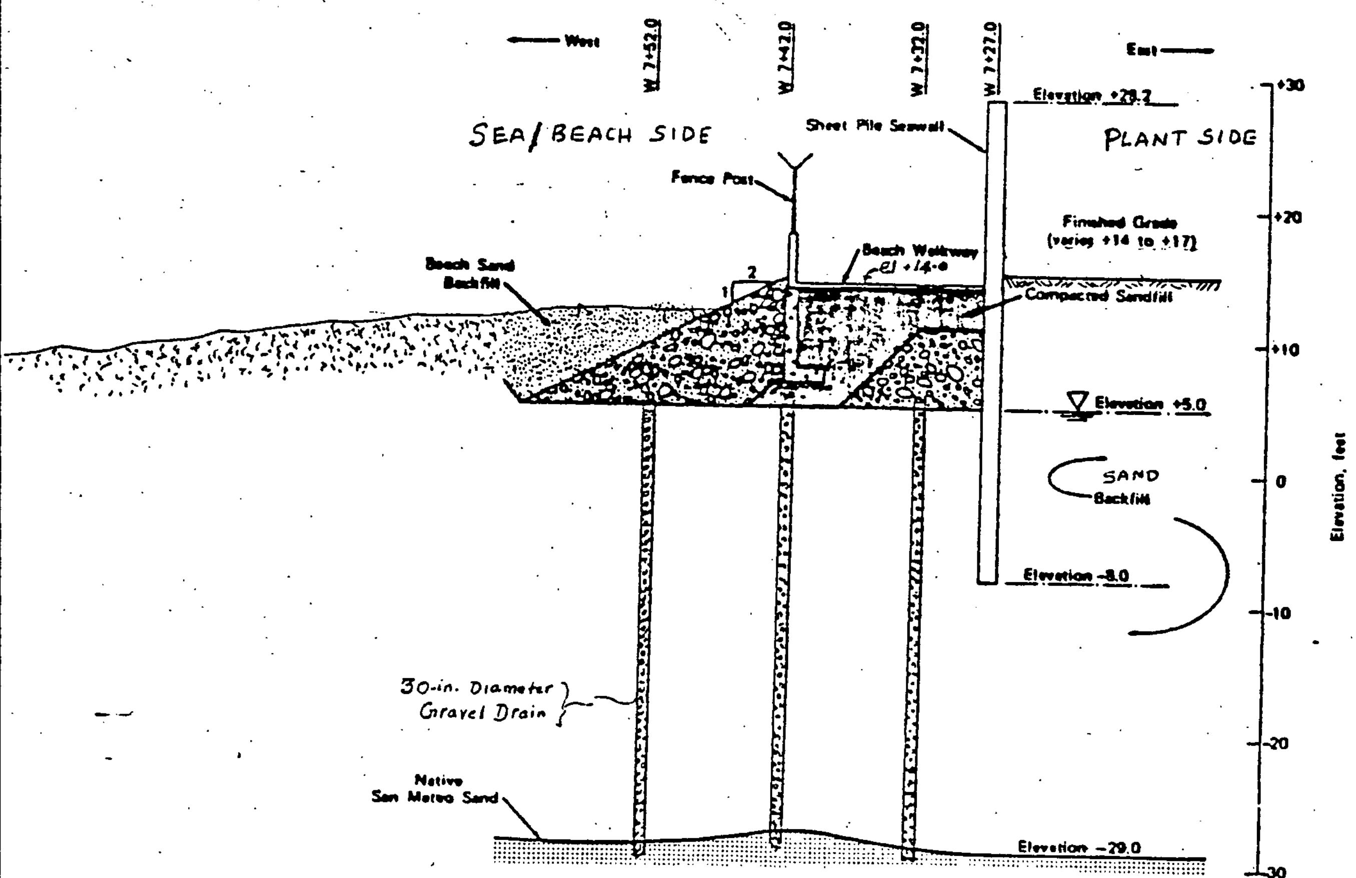
FIGURE - 2

PLAN AND SECTION VIEWS OF GRAVEL DRAINS

PARTIAL PLAN



SECTION VIEW



CROSS SECTION SHOWING SOIL CONDITIONS USED IN THE EVALUATION OF THE SEAWALL

FIGURE - 3