2. SITE CHARACTERISTICS

2.4 HYDROLOGIC ENGINEERING

The data and information contained in section 2.4 and referenced Appendices are historical information developed during San Onofre's original design to address Hydrologic Engineering. The information was used to determine the plant's design basis. Unless otherwise noted in the text, this information has not been updated to reflect data from later years.

2.4.1 HYDROLOGIC DESCRIPTION

2.4.1.1 Site and Facilities

The site is located on the southern California coast of the Pacific Ocean near the City of San Clemente, California in the SW1/4, NW1/4, section 30, T. 9 S., R. 6 W., as shown in figures 2.4-1 and 2.4-2 taken from the U.S. Geological Survey San Onofre Bluff Quadrangle. Bordering the site to the north is the existing San Onofre Nuclear Generating Station Unit 1, to the south the San Onofre State Beach, and to the east the U.S. Marine Corps Base, Camp Pendleton. Access to the site is by way of Interstate Highway 5, approximately 300 feet to the east and the Atchison, Topeka and Santa Fe Railroad, approximately 200 feet to the east.

The power block finish grade elevation is +30.00 mean lower low water (mllw). A subsurface drainage system will carry normal storm drainage flows to the cooling-water intake structure. For consideration of flooding due to the thunderstorm probable maximum precipitation (PMP), all catch basins for the subsurface drainage system were assumed plugged; surface drainage facilities will transmit all thunderstorm drainage flows in the power block over the seawall to the ocean. Drainage areas contributing to the surface drainage flows in the power block area are shown on figure 2.4-3. Runoff from the coastal hills east of Interstate Highway 5 will be diverted to the San Onofre Creek Basin, as shown in figure 2.4-4.

A description of all exterior accesses, openings, or penetrations of safety-related structures subject to hydrologic considerations are tabulated in table 3.4-1.

2.4.1.2 Hydrosphere

2.4.1.2.1 San Onofre Region

The San Onofre site is situated on a coastal plain at the base of the western foothills of the Santa Margarita Mountain Range. In this area the elevation rises sharply from sea level to a fairly level terrace formation 100-200 feet above sea level. At the terminus of the terrace formation, some 1500 feet inland, the foothills begin, rising with moderate slopes to an elevation of 3000 feet above sea level. The foothills belt extends approximately 28 miles inland and lies in a general northwest-southeast direction.

Natural plant cover in the coastal plain sector typically consists of coastal chaparral and grassland, while the foothill sector is composed mainly of chaparral and open woodland. The

C-H

majority of the soil structure in the coastal plain is a combination of sandstone and shale. Alluvial fans within this area are composed of sandy-clay and clayey-loam material. Soil structures within the foothill sector consist mainly of sandy loam over a layer of clayey subsoil.[1]

The mean annual temperature in the coastal plain region is 61°F, with a mean minimum temperature of 42°F in January. Annual rainfall amount falls between 10 to 16 inches, with 90% of this annual total occurring during the months of November through April. Soils generally remain thoroughly moistened during this period; however, they become dry by summer due mainly to the rapid plant growth period taking place during the months of April and May.

The foothills sector exhibits a climate similar to that of the coastal plains. The mean annual temperature is 60°F with a January mean minimum of 38°F. Annual rainfall ranges from 12 to 20 inches. Heavy rainfall usually occurs during the months of December through April. As in the coastal plains, the soils are usually thoroughly moistened at this time, drying by summer.[1]

2.4.1.2.2 Major Drainage Basins

There are no perennial streams in the general vicinity of the plant site. However, ephemeral streams and water courses do exist. The major streams are San Mateo Creek, located approximately 2 miles to the northwest, and San Onofre Creek located approximately 1 mile to the northwest.

2.4.1.2.2.1 San Mateo Creek

San Mateo Creek has a drainage area of 132 square miles in size. Records from the U.S. Geological Survey recording gage[2] at the mouth are available for water years October 1946 to September 1967, at which time management of and record keeping for the gage was discontinued. From an examination of the topography of the area, it was determined that the drainage divide separating San Mateo and San Onofre Creeks would preclude the plant site being influenced by San Mateo Creek.

2.4.1.2.2.2 San Onofre Creek

San Onofre Creek has a drainage area of 43 square miles. A U.S. Geological Survey recording gage is located at the mouth with available records covering a period from October 1946 to September 1967. The drainage basin is presented in figure 2.4-5. The basin length is approximately 9.7 miles and is 4.7 miles in width.

San Onofre Creek and its watershed lies entirely on U.S. Marine Corps Base, Camp Pendleton. The origin of the basin is in the Santa Margarita Mountains to the northeast of the site. The maximum elevation in the basin is 3187 feet above mllw with the minimum at sea level. Ground slope within the tributary area varies from approximately 3% to 10%.

There are no existing or proposed water control structures within the San Onofre Creek Basin. Camp Pendleton currently uses surface runoff infiltration for purposes of recharging the base well system, otherwise there are no surface water users in the basin.

2.4.1.2.2.3 Foothill Drainage Basin

The foothill drainage basin identified in figure 2.4-6 contributes to the hydrologic factors influencing the plant site. The basin totals 0.86 square mile in area. There are no gaging stations located within the basin and, consequently, stream flow records are not available.

The entire watershed lies within the boundaries of the Marine Corps Base, Camp Pendleton. Elevation in the basin varies between a maximum of 1200 feet and a minimum of 100 feet above msl. Ground slope varies from 8 to 22%. Ground cover is moderate within the basin consisting mainly of chaparral and grassland.

Water control structures consist of the 42-inch and 72-inch diameter concrete culverts under Interstate Highway 5, as shown in figure 2.4-6. The culverts are maintained by the California State Department of Transportation. The capacity of these culverts is 180 and 520 ft^3/s, respectively. In addition to the two culverts identified above, an earthen channel traverses the basin along the east side of Interstate Highway 5 diverting runoff to San Onofre Creek, as shown in figure 2.4-6. The capacity of the channel is 1850 ft^3/s.

2.4.2 FLOODS

2.4.2.1 Flood History

There are four U.S. Geological Survey stream-gaging stations located within the general plant vicinity. Two are located on San Mateo Creek and two on San Onofre Creek. The period of record for these gages varies from 17 to 21 years. Peak recorded discharges are as presented in table 2.4-1. An examination of stream flow data revealed that measurable flows occur only 4 to 5 months of the year, usually during the months of December through April.

Ocean storm surges and tsunami historical data are presented in subsections 2.4.5 and 2.4.6, respectively. Ice jams and dam failures do not apply to the San Onofre site (2.4.7 and 2.4.4 respectively).

2.4.2.2 Flood Design Considerations

For purposes of determining and analyzing potential flood sources, consideration was given to the San Onofre Creek Basin, as shown in figure 2.4-5, and the foothill drainage area east of the site, as defined in figure 2.4-6. In both cases the probable maximum flood (PMF) was defined as the design basis event. Regulatory Guide 1.59, Design Basis Floods for Nuclear Power Plants, and references noted therein were employed as standards in the determination of the PMF.

Results of the PMF analysis concluded that the San Onofre Creek Basin exhibits no flooding potential to the site. The maximum flood stage, as a result of the PMF, was determined to be 24.1 feet at the mouth of the creek. Topographical features of the basin would contain this flow and thereby preclude flooding of the site by this source.

An analysis of the flooding potential of the foothill drainage area was also performed. The results of this analysis produced evidence that the site could be subjected to flooding during the occurrence of the design basis PMF. In order to preclude flooding of the site by this source a diversion structure, as shown in figure 2.4-4, routes the surface runoff from the foothill drainage area to the San Onofre Creek Basin. For purposes of design of the diversion structure, the PMF was used as the design basis event for determining the maximum water elevation.

As discussed in paragraphs 2.4.5.2, 2.4.5.3, and 2.4.6.1, the occurrence of storm surge, storm wave action, and tsunami will not cause flooding of the site.

2.4.2.3 Effects of Local Intense Precipitation

The San Onofre region is susceptible to major storm activity during the months of October through April. U.S. Weather Bureau Hydrometeorological Report 36 was used to calculate the orographic and conveyance components of the frontal probable maximum precipitation (PMP).(5) Evaluation of the thunderstorm PMP was also determined for the San Onofre site based on the methods of the U.S. Weather Bureau.(6) The thunderstorm PMP event causes the highest flood level on the San Onofre site and was therefore used as the design basis event.

The distribution of precipitation in the 6-hour thunderstorm used to design the site drainage structures is presented in table 2.4-2. Arrangement of the incremental values into the critical PMP storm is based on procedures used by the U.S. Army Corps of Engineers.(7)

The drainage area tributary to the Units 2 and 3 power block was divided into sub-basins as presented in figure 2.4-3. Drainage characteristics and peak flows for the sub-basins are presented in table 2.4-3.

The U.S. Soil Conservation Service soil-complex method(8) was used to construct the hydrograph resulting from the PMP. Runoff curves for the given soil types were selected on the basis of Antecedent Moisture Condition III. Due to the relatively short times of concentration and corresponding very short lag times of the sub-basins, the assumption was made that all precipitation excess within any period became runoff during that period. Precipitation intensities for durations less than 15 minutes were interpolated from the thunderstorm PMP data presented in table 2.4-2. The resultant PMF hydrographs for the sub-basins are shown in figures 2.4-7 through 2.4-11.

The subsurface drainage system (figure 2.4-12) is designed to accommodate the runoff from the onsite areas and offsite areas west of Interstate Highway 5, resulting from a precipitation intensity of 3 in./h.

Table 2.4-1
STREAM FLOW STATIONS IN VICINITY OF SAN ONOFRE SITE(2)(3)(4)

Station	USGS Station No.	Location	Period of Record	Drainage Area (mi^2)	Peak Flow Date	Peak Flow Discharge (ft^3/s)	Peak Flow Gage Height (ft)
San Onofre Creek near San Onofre, California	11-0462	Lat 33°23'23" Long. 117°30'50"	Oct 1950 to Sept 1967	34.6	4/1/58	2,680	5.90
San Onofre Creek at San Onofre, California	11-0462.5	Lat 33°23'00" Long. 117°34'22"	Oct 1946 to Sept 1967	42.2	4/1/58	2,600	6.90
San Mateo Creek near San Clemente, California	11-0463	Lat 33°28'15" Long. 117°28'20"	Oct 1952 to Sept 1967	80.8	12/6/66	7,300	10.45
San Mateo Creek at San Onofre, California	11-0463.7	Lat 33°23'46" Long. 117°35'21"	Oct 1946 to Sept 1967	132	12/5/66	10,000	10.42

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Table 2.4-2

6-HOUR, 1-SQUARE-MILE PROBABLE MAXIMUM THUNDERSTORM PRECIPITATION (PMP)

Time (hours)	PMP Total (inches)	PMP Incremental (inches)	PMP Critically Arranged (inches)
0.25	3.01	3.01	0.14
0.50	4.90	1.89	0.14
0.75	6.16	1.26	0.14
1.00	7.00	0.84	0.14
1.25	7.63	0.63	0.14
1.50	8.12	0.49	0.21
1.75	8.54	0.42	0.21
2.00	8.96	0.42	0.21
2.25	9.31	0.35	0.28
2.50	9.59	0.28	0.35
2.75	9.87	0.28	0.42
3.00	10.15	0.28	0.63
3.25	10.36	0.21	1.26
3.50	10.57	0.21	3.01
3.75	10.78	0.21	1.89
4.00	10.99	0.21	0.84
4.25	11.20	0.21	0.49
4.50	11.41	0.21	0.42
4.75	11.55	0.14	0.28
5.00	11.69	0.14	0.28
5.25	11.83	0.14	0.21
5.50	11.97	0.14	0.21
5.75	12.11	0.14	0.21
6.00	12.25	0.14	0.14

Table 2.4-3

SAN ONOFRE SITE SUB-BASIN DRAINAGE PARAMETERS

Sub-Basin[a]	Area (acres)	Time of Concentration (minutes)	Time Increment For Analysis (minutes)	Peak PMP Discharge (ft^3/s)
1	6.4	10/5	10/5	59
2	8.8	5	5	83[c]
3	8.8	5	5	83[c]
4	7.9	5	5	96
5	32.6	30	30	270[b]
6	1.0	15	15	12

[a] See figure 2.4-3

[b] Ten ft^3/s of this contributes to surface drainage in the power block area.

[c] This value only includes discharge over the seawall.

[d] This value does not include the MUD tank area that will not contribute to surface drainage.

All catch basins for the subsurface drainage system (figure 2.4-12), roof drains, and exposed floor drains were assumed plugged for the purpose of determining water surface elevations arising during the thunderstorm PMP event. Two 4-foot by 4-foot box culverts at Highway 101 (figure 2.4-12) were assumed to remain operational. The box culverts are sufficiently large and are in an area that would not supply materials capable of plugging them. Even if flow were restricted through them, the topography along Highway 101 is such that the resulting drainage flows would not impact the plant site.

The switchyard has an upper and lower bench as illustrated in figure 2.4-12. With the normal catch basins plugged, the ponded water on the upper bench will drain to the south access road. A peak flow of 32 ft^3/s would be discharged from the upper bench as a result of the thunderstorm PMP event. This peak was calculated by routing the storm hydrograph through the upper bench by the storage routing procedure specified by Henderson.[9] The cross-section of the access road is sloped to prevent the upper bench drainage from entering the lower bench.

Drainage entering the lower bench includes not only drainage from the thunderstorm PMP event over this area, but also accounts for the possibility that water could enter the normal storm drain system at the upper bench and emerge from the normal catch basins in the lower bench. The resulting peak flow of 54 ft^3/s will overtop the berm and flow into the Unit 2 and 3 power blocks.

For the extreme thunderstorm PMP event, when the normal catch basins are assumed plugged, the upper site area will drain into the barranca to the west without impacting the Area 4 drainage (figure 2.4-3). Area 5 will drain through the two 4-foot by 4-foot box culverts; one enters the subsurface storm drain line leading to the power block and one enters the line to the barranca. The capacity of each box culvert (135 ft^3/s) is less than the approximate 300 ft^3/s capacity of the 54-inch conduit to which each culvert drains.

A runoff flow of 10 ft^3/s from Area 5 is assumed to enter the Unit 3 power block area at the junction of Highway 101 and the Unit 2 and 3 power block access road (figure 2.4-3). The balance of the runoff flow from Area 5 will be diverted from Highway 101 into the upper site area and away from the power block area at station coordinate S44+90 (approximate) (figure 2.4-3). At the south entrance to the Unit 3 power block, the peak discharge to the power block area was determined by routing the hydrographs from the contributing areas, taking into account the time of concentration and lag times for Areas 1, 4, and 6, plus including 10 ft^3/s from Area 5 (figure 2.4-3). The resulting peak flow, at the south entrance to the Unit 3 power block, is 154 ft^3/s. Construction of a diversion channel on the south side of the Service Building allows 80 ft^3/s to be diverted over the seawall outside of the power block resulting in 74 ft^3/s flowing into the Unit 3 power block.

Swales are provided in the asphalt areas around the power block to convey the drainage to the seawall where it will discharge to the ocean. During normal rainfall intensities, the subsurface drainage system, figure 2.4-12, will convey all drainage to the cooling water intake structure. A duration of 5 minutes was used in calculating the peak discharges in the power block areas. The maximum backwater elevations resulting from these peak flows are shown on figure 2.4-13. Contributing roof drainage was derived by routing the PMP hydrograph for each roof through the parapet openings. Pertinent roof discharge points and the corresponding ponded water surface depths are presented in figure 2.4-13.

Penetrations in the auxiliary building control area, at elevations 72 and 30 feet are protected against ponding resulting from roof drainage by openings in the seismic gap which allows the water to fall to elevation 7 in the turbine area. The areas contributing to the turbine buildings are roof drainage from the safety equipment building, the auxiliary building, and the turbine areas. The volume of floodwater entering elevation 7 of the turbine building during the PMP thunderstorm will not impact safety-related equipment.

Surface water will not enter the auxiliary building from the west because the bridge walkway over the intake structure has numerous parapet openings which would allow drainage to flow into the cooling water intake structure. Additionally, a berm on this bridge walkway prevents control room complex flooding due to a possible surge in the circulating water system.

The maximum postulated flood level in the Units 2 and 3 power blocks has been established below elevation +31.0 feet mean lower low water (MLLW) level as shown on figure 2.4-13. Those penetrations below elevation +31.0 are specified in table 3.4-1. In the extreme event that the thunderstorm PMP occurs, no safety-related equipment will be impacted by flooding for the following reasons:

- Watertight barriers prevent water from reaching most safety-related areas.

- Negligible water will enter the exterior doors with sills below the floodwater elevation because they either open outwards, are locked/card coded, have weather stripping, and/or are protected by curbs.

- Drainage water in the structures which entered from other areas (e.g., from roofs, open areas) will not reach safety-related equipment.

The drainage features indicated previously and those presented below will prevent all but approximately 15 cfs of runoff from Units 2 and 3 from reaching the Unit 1 power block area:

- Grade elevations in the switchyard access, Unit 1 access, and the railroad spur access are above the calculated flood elevations.

- A curb is located at the slope interface of the Unit 1 and 2 power block areas, except at the stairway between Unit 1 and Unit 2. The small amount of runoff (approximately 15 cfs) from Unit 2 into Unit 1 is easily within the capacity of the Unit 1 Yard Sump.

Storm drainage in the Unit 1 power block will not flow to Units 2 and 3 because of the elevation differential; Unit 1 is at elevation 20 feet and Units 2 and 3 are at 30 feet. Although the Unit 1 access road connects to the access road to the Unit 2 and 3 switchyard, the Unit 1 access road will be graded to preclude drainage flows from entering the Unit 2 and 3 site.

2.4.3 PROBABLE MAXIMUM FLOOD ON STREAMS AND RIVERS

An analysis of the 0.86 square mile foothill drainage area and the 43 square miles San Onofre Creek Basin was conducted to determine the PMF and subsequent contribution to flooding at the San Onofre site. The recommendations of NRC Regulatory Guide 1.59 were used in conducting the PMF analysis.

2.4.3.1 Probable Maximum Precipitation

The San Onofre area is susceptible to frontal storms, usually occurring during the months of October through April, and local thunderstorms which are predominant during summer and early fall. A comparison of the PMP values associated with both types of storms was made to determine the critical event. HMR 36[5] was used to calculate the frontal storm PMP and the National Weather Service Report[6] was used in determining the thunderstorm PMP. By comparison, it was concluded that the thunderstorm PMP was the more critical and consequently was used as the design basis event. The 6-hour, 1-square mile thunderstorm PMP is presented in table 2.4-2.

Table 2.4-4

6-HOUR PROBABLE MAXIMUM THUNDERSTORM PRECIPITATION
SAN ONOFRE CREEK BASIN (PMP)

Time (hours)	PMP Total (inches)	PMP Incremental (inches)	PMP Critically Arranged (inches)
0.25	1.96	1.96	0.16
0.50	3.19	1.23	0.16
0.75	4.00	0.81	0.17
1.00	4.55	0.55	0.17
1.25	4.94	0.39	0.18
1.50	5.32	0.38	0.19
1.75	5.71	0.39	0.20
2.00	6.09	0.38	0.21
2.25	6.35	0.26	0.25
2.50	6.60	0.25	0.26
2.75	6.86	0.26	0.38
3.00	7.11	0.25	0.39
3.25	7.32	0.21	0.81
3.50	7.51	0.19	1.96
3.75	7.71	0.20	1.23
4.00	7.91	0.20	0.55
4.25	8.10	0.19	0.39
4.50	8.28	0.18	0.38
4.75	8.47	0.19	0.26
5.00	8.65	0.18	0.25
5.25	8.82	0.17	0.20
5.50	8.98	0.16	0.19
5.75	9.15	0.17	0.19
6.00	9.31	0.16	0.18

Table 2.4-4 provides the 6-hour thunderstorm PMP for San Onofre Creek Basin. Because of the size of San Onofre Creek Basin, the 1-square mile PMP was adjusted and applied over the basin in accordance with the procedures outlined in reference 6. As noted in table 2.4-4, the PMP total for the San Onofre Creek Basin is 9.31 inches.

The thunderstorm PMP for the foothill drainage basin was derived in a manner similar to the method used for the San Onofre Creek Basin. The 1-square mile PMP total was calculated at 12.25 inches as presented in table 2.4-2.

Occurrence of snow coincident with the PMP was not considered a probable event at the San Onofre Site.

2.4.3.2 Precipitation Losses

From results of PMP studies noted in the National Weather Service Report,[(6)] it was determined that the occurrence of antecedent precipitation preceding the design basis PMP is highly probable. For this reason, the assumption was made that a storm of significant magnitude occurred immediately prior to the PMP, resulting in complete saturation of the soil with minimum loss to infiltration.

After examining records of major storms in the general vicinity of San Onofre, the decision was made to use the major storms of January and February 1969 for analysis of infiltration capability. The Corps of Engineers Hydrologic Engineering Center HEC-1 computer program,[(10)] Unit Hydrograph and Loss Rate Optimization Subroutine, was used to determine infiltration rates for the above storms. Records available for San Onofre Creek were not precise enough to permit a valid reconstruction of the storm runoff hydrograph. For this reason an analysis was conducted of the Santa Margarita River Basin near Oceanside. The Santa Margarita River and San Onofre Creek Basins were judged similar in regard to soil type and average ground slopes, although the Santa Margarita River Basin is considerably larger in total drainage area. On the basis of results obtained from the analysis a uniform infiltration rate of 0.1 in./h was used. Initial abstraction was not considered since it was assumed that the soil was saturated at the beginning of the PMP.

2.4.3.3 Runoff and Stream Course Models

2.4.3.3.1 San Onofre Creek Basin

The San Onofre Creek Basin was subdivided as shown in figure 2.4-5. Hydrologic parameters for each sub-basin were calculated and are tabulated in table 2.4-5. The sub-basin lag times were calculated on the basis of figure 2.4-15 derived by the Corps of Engineers and presented by Linsley.[(11)] As noted in the referenced publication, this basin lag curve was derived as a result of a study of various drainage basins in southern California conducted by the Corps. For purposes of conservatism, a 10% reduction of all calculated lag times was performed prior to their use in calculations.

Table 2.4-5
SAN ONOFRE CREEK BASIN SUB-BASIN DRAINAGE PARAMETERS

Sub-basin	Area (mi^2)	L[a] (mi)	L_a[b] (mi)	Slope[c] (ft/mi)	Lag Factor $L \cdot L_a / \sqrt{S}$	Lag Time[d] (h)	Adjusted Lag Time[e] (h)
A1	12.80	7.6	3.2	342	1.3	1.3	1.2
A2	12.10	7.6	5.5	380	2.1	1.6	1.4
A3	0.60	1.1	0.8	542	0.04	0.4	0.3
A4	9.10	6.3	2.8	308	1.0	1.2	1.1
A5	8.60	5.7	2.8	155	1.3	1.3	1.2

(a) Length of watercourse from sub-basin divide to sub-basin outlet.

(b) Distance from point on watercourse nearest centroid of sub-basin to outlet of sub-basin.

(c) Slope of sub-basin from divide to outlet.

(d) Time in hours from centroid of effective rainfall to peak of unit hydrograph.

(e) Adjusted lag time reflecting 10% reduction in calculated lag time.

Using the Corps HEC-1 computer program,[10] the PMF hydrograph for each sub-basin was derived. A Snyder peaking coefficient of 0.7 was used for unit hydrograph computations in the program for each sub-basin. This value was determined as a result of the analysis of the major storms of January and February 1969 referenced in paragraph 2.4.3.2. As noted, the analysis was performed for the Santa Margarita River Basin where records permitted a valid reconstruction of the basin runoff hydrograph. After determining the PMF hydrograph for each individual sub-basin they were routed and combined to obtain a PMF hydrograph at the mouth of San Onofre Creek.

Flood routing was conducted using the Muskingum Method. The Muskingum storage coefficient K for each reach was taken as 50% of the basin's lag time. This value was arrived at by assuming low flow and high flow conditions in sub-basin A5, calculating the corresponding velocities and average flow velocity. The average velocity was combined with the reach length to yield the travel time through the reach. From this relationship, a proportionality constant was calculated (i.e., 0.5) and then used in calculating K for the remaining sub-basins. The routing coefficient X was assumed as 0.3. This value is recommended for use in mountainous regions.[11]

2.4.3.3.2 Foothill Drainage Basin

The analysis of the foothill drainage basin was conducted in a manner similar to that of San Onofre Creek Basin described above. The drainage area was subdivided as shown in figure 2.4-6. Sub-basin hydrologic parameters are as defined in table 2.4-6. Figure 2.4-15 was used for calculation of sub-basin lag times.

The HEC-1 computer program was used to develop the unit hydrograph and resultant PMF hydrograph for each sub-basin. As explained in paragraph 2.4.3.3.1, a Snyder peaking coefficient value of 0.7 was used in the derivation of the unit hydrographs. The PMF hydrographs obtained from each sub-basin were routed to San Onofre Creek assuming the future structure as shown in figure 2.4-4. Due to the relatively short distance between the outlets of sub-basins B1 and B2, approximately 0.75 mile, and narrow range of lag times for the sub-basins, it was decided to ignore lag and travel times and combine the individual sub-basin PMF hydrographs directly, yielding a conservative PMF hydrograph at the outlet of sub-basin B2.

2.4.3.4 Probable Maximum Flood Flow

2.4.3.4.1 San Onofre Creek Basin

The PMF hydrograph resulting from the PMP for San Onofre Creek Basin is presented in figure 2.4-16. This hydrograph represents the PMF flow for the basin's tributary area above the mouth, station 8 on figure 2.4-5. As shown in figure 2.4-16, the PMF peak flow is 71,000 ft^3/s.

Table 2.4-6
FOOTHILL DRAINAGE BASIN
SUB-BASIN DRAINAGE PARAMETERS

Sub-basin	Area (mi^2)	L (mi)	L_a (mi)	Slope (ft/mi)	Lag Factor $L \cdot L_a / \sqrt{S}$	Lag Time (hr)	Adjusted Lag Time[a] (h)
B1	0.42	1.58	0.85	743	0.043	0.36	0.32
B2	0.14	0.92	0.26	1,108	0.007	0.18	0.16
B3	0.15	0.78	0.33	833	0.009	0.20	0.18
B4	0.08	0.56	0.17	1,000	0.003	0.13	0.12
B5	0.07	0.57	0.21	1,211	0.0035	0.14	0.13

[a] Adjusted lag time reflecting 10% reduction in calculated lag time.

2.4.3.4.2 Foothill Drainage Basin

The PMF hydrograph for the foothill basin was developed as discussed in paragraph 2.4.3.3.2, assuming a diversion structure. PMF hydrographs for each of the sub-basins are presented in figures 2.4-7 through 2.4-10. Figure 2.4-11 presents the PMF hydrograph for local inflow from sub-basin B2 to San Onofre Creek. The combined hydrograph for the entire area is presented in figure 2.4-17. The PMF peak discharge was calculated to be 5225 ft^3/s for the basin.

An analysis of potential debris yield from the basin was performed in order to determine the amount of debris storage required behind the diversion structure. The debris runoff analysis was based on debris production curves derived by the Los Angeles County Flood Control District given in reference 12. The curve selected for the basis of analysis was curve DPA-6, valid for the Puente Hills area in Los Angeles County. This curve is reproduced in figure 2.4-18. From comparison, the Puente Hills was judged similar to the Foothill Basin on the basis of topograph, soil type, and ground cover. Since Curve DPA-6 was based on a less severe precipitation intensity than that of the PMP for the Foothill Basin an adjustment was made on the basis of the Universal Soil-Loss Equation.

$$A = R\,K\,L\,S\,C\,P$$

where:

A = annual soil loss (tons/acre-yr)

R = rainfall factor

K = soil erodibility factor

L = slope-length factor

S = slope-gradient factor

C = cropping

P = supporting conservation practice index

The factors K, L, S, C and P in the Universal Soil-Loss Equation are all functions of topography, soil type and ground cover. Since the Puente Hills area and the Foothill Basin were judged similar in these respects, the only variable factor is the rainfall factor, R. Reference 13 indicates that R is related to precipitation by the following equation

$$R = 35.1\,P^{1.96}$$

where:

P = 6-hour precipitation (inches)

The Los Angeles County curve was based on a 6-hour precipitation of 3.9 inches with a corresponding R of 505 whereas the PMP of 12.3 inches has an R of 4800. The resultant ratio of 10 was used to adjust the debris production curve as presented in figure 2.4-18. Figures 2.4-7 through 2.4-11 show the calculated debris runoff and the resultant design hydrograph for each sub-basin. Figure 2.4-17 presents the composite PMF hydro-graph for the entire Foothill Basin. As noted, the design peak discharge was calculated to be 7335 ft^3/s. The debris production curve, DPA-6, was derived by Los Angeles County assuming a major watershed burn immediately prior to the debris producing storm. On this basis, the debris runoff calculated for the Foothill Basin is conservative in nature.

2.4.3.5 Water Level Determinations

2.4.3.5.1 San Onofre Creek Basin

The PMF peak discharge of 71,000 ft^3/s was used in determining the maximum flood stage in San Onofre Creek. A water surface profile was constructed using the Corps HEC-2 computer program.(14) Profile computations commencing at the mouth of San Onofre Creek were carried approximately 2.3 miles upstream. Locations of cross-sections used in the analysis are given in figure 2.4-19.

For calculation purposes, a Manning's n value of 0.12 was used throughout the reach being analyzed. This value is representative of a natural channel or flood plain with heavy underbrush or growth of timber. San Onofre Creek and floodplain are scattered with light to medium brush and trees with the majority of the watercourse clean and clear, suggesting a Manning's n on the order of 0.07. The higher value of 0.12 was used for conservatism.

The analysis performed with HEC-2(14) concluded that flow through the bridge structure at the mouth was restricted to subcritical. Yarnell's energy equation was used for the calculation of the change in water surface elevation through the bridge. A shape coefficient of 1.25 for a square nose and tail pier was used. The results of the analysis ascertained that the flow is contained within the limits of the floodplain of San Onofre Creek not presenting any risk of flooding at the site. PMF flood stage values for San Onofre Creek are presented in table 2.4-7.

2.4.3.5.2 Foothill Drainage Basin

As shown in figure 2.4-6 the runoff will be diverted to San Onofre Creek by the diversion structure. The diversion structure consists of an earth filled berm with excavated channel designed to convey the peak discharge associated with the PMF. Alignment and cross-sectional details are presented in figures 2.4-20 and 2.4-21.

Table 2.4-7
PMP WATER SURFACE PROFILE SAN ONOFRE CREEK

Cross-Section[a]	Discharge (ft^3/s)	Depth of Flow (ft)	Water Surface Elevation[b] (ft)	Average Velocity (ft/s)	Critical Depth (ft)	Manning's n
1	71,000	8.3	8.3	16.5	8.3	0.12
2	71,000	18.4	28.4	8.6	10.8	0.12
3	71,000	22.4	32.4	11.4	15.3	0.12
4	71,000	22.6	32.6	11.9	15.3	0.12
5	71,000	24.1	39.1	3.9	9.5	0.12
6	71,000	20.0	40.0	5.1	10.0	0.12
7	71,000	21.4	44.4	3.1	10.8	0.12
8	71,000	14.4	48.4	3.2	6.5	0.12
9	71,000	13.8	53.8	3.4	6.0	0.12
10	71,000	14.0	64.0	5.1	9.6	0.12
11	71,000	12.8	77.8	4.1	6.3	0.12
12	71,000	16.6	86.6	7.0	10.2	0.12
13	71,000	15.2	95.2	6.6	7.9	0.12
14	71,000	15.5	105.5	6.4	7.0	0.12

[a] Cross-sections as located in figure 2.4-19
[b] Elevation based on datum of msl

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2.4.3.5.2.1 Sediment Transport Analysis

An analysis of the berm and channel was performed to determine the degree and location of aggradation resulting from the drainage basin sediment yield. Degradation was not considered since the channel is to be lined with riprap as described in subsection 2.5.6.

The sediment runoff hydrographs presented in figures 2.4-7 through 2.4-11 and 2.4-17 were used to describe the sediment inflow to the channel. Inflowing sediment will be transported through the channel by two hydraulic modes, bed-load and suspended or wash-load. If the transport capacity of a channel region is exceeded by the inflow supply, aggradation occurs in that region. A distinction between bed-load and wash-load was made on the basis of sediment particle size. Sediment finer than 0.08 mm would expect to be transported by the wash or suspended-load, while sediment coarser than 0.08 mm would rely on the bed-load function for transport through a channel reach. From the result of an extensive soil investigation of the drainage basin, it was determined that the mean size of sediment material available for delivery to the channel would be approximately 0.08 mm.

To mathematically model the channel, reaches were defined as presented in table 2.4-8. Hydraulic and physical similitude was used as a basis for defining the reaches. Using the sediment transport theories of Inglis-Lacey, Einstein-Brown and DuBoys a bed-load function was established for each reach.

The sediment inflow into a reach is defined as the combined total of the sediment transported from upstream reaches and local sediment inflow from the portion of the drainage basin tributary to the reach in question. The sediment outflow from a reach is defined as the sediment transport capacity. The following equation was used as the basis for the mathematical model of the channel.

$$\frac{ds}{dt} = I - O$$

where:

$\frac{ds}{dt}$ = change in sediment storage with respect to time

I = sediment inflow

O = sediment outflow

The above equation was used in finite-difference form to calculate the sediment storage rate of each reach. The volume of sediment, deposited in a given reach was converted to an equivalent depth of deposit, based on the physical geometry of the reach. Results of the analysis are presented in table 2.4-8 and figure 2.4-22.

Table 2.4-8

CHANNEL BED AGGRADATION

Reach (Station)	Aggradation At Time of Peak Discharge (ft)
0+00 to 5+00	0.00
7+00 to 28+00	2.40
29+00 to 35+00	2.44
37+00 to 49+00	0.00
51+00 to 57+00	1.70
59+00 to 67+00	8.17
69+00 to 75+00	7.32
77+00 to 88+00	0.00

2.4.3.5.2.2 Water Surface Profile

The PMF water surface profile for the berm and channel is presented in table 2.4-9 and figure 2.4-22. The U.S. Army Corps of Engineers computer program, HEC-2, was used to determine the resultant profile. A Manning's n of 0.035 was used in the computations. The resultant water surface profile was derived from a comparison of water surface profiles generated, assuming subcritical and supercritical flow, as recommended by HEC. Headloss coefficients for expansions and contractions of 0.30 and 0.10, respectively, were assumed. The amount of channel bed aggradation assumed in the analysis is provided in table 2.4-8.

2.4.3.6 Coincident Wind Wave Activity

Coincident wind wave activity for the San Onofre Creek and Foothill Drainage Basins was not considered critical. Due to the relatively short fetch length behind the PMF diversion structure, consideration of coincident wind wave activity was not required.

2.4.4 POTENTIAL DAM FAILURES, SEISMICALLY INDUCED

There are no existing dams located within the vicinity of the plant site, whose seismically induced failure could result in adverse flooding at the site.

Table 2.4-9
PMF WATER SURFACE PROFILE AT PEAK DISCHARGE - FOOTHILL DRAINAGE BASIN
(Sheet 1 of 3)

Station	Discharge (ft^3/s)	Water Surface Elevation	Top of Berm Elevation	Channel Invert Elevation	Critical Depth Elevation	Average Velocity (ft/s)
0+00	6030	75.0	77.0	66.0	76.4	17.9
1+00	6030	77.1	80.0	69.0	78.4	17.3
3+00	6030	81.2	85.5	72.2	83.0	19.4
5+00	6030	87.9	90.0	76.0	85.5	7.6
7+00	6030	88.7	91.0	79.0	86.0	6.8
9+00	6030	89.8	92.5	81.0	89.9	13.1
11+00	6030	92.7	95.0	81.5	90.3	8.8
13+00	6030	93.2	95.5	83.0	91.7	10.0
15+00	6030	94.4	96.5	83.5	91.8	8.4
17+00	6030	94.9	97.0	84.0	93.5	10.2
19+00	6030	96.0	98.0	85.0	94.5	10.0
21+00	6030	97.0	99.0	86.5	95.6	10.1
23+00	6030	98.2	100.5	87.5	96.2	9.2
25+00	6030	99.2	102.0	88.0	97.0	8.6
27+00	6030	99.8	103.0	90.0	99.2	13.8
29+00	6030	102.4	104.5	91.0	101.0	10.7
31+00	6030	103.8	106.0	93.0	103.8	13.6
33+00	6030	106.2	108.8	96.1	105.8	12.8

Table 2.4-9
PMF WATER SURFACE PROFILE AT PEAK DISCHARGE - FOOTHILL DRAINAGE BASIN
(Sheet 2 of 3)

Station	Discharge (ft^3/s)	Water Surface Elevation	Top of Berm Elevation	Channel Invert Elevation	Critical Depth Elevation	Average Velocity (ft/s)
35+00	5670	107.1	111.0	98.5	109.1	13.8
37+00	5670	111.4	114.0	101.0	110.0	12.5
39+00	5670	114.0	118.0	106.0	115.4	18.2
41+00	5670	117.8	122.0	109.0	119.4	18.8
43+00	5670	123.4	126.0	113.0	123.4	14.8
45+00	5670	126.1	129.0	115.0	126.1	10.0
47+00	4340	126.6	131.0	118.5	127.4	12.4
49+00	4340	129.7	133.1	118.5	129.7	13.3
51+00	4340	132.6	135.0	121.0	130.2	8.9
53+00	4340	133.1	137.0	123.0	133.0	13.6
55+00	4340	138.9	141.5	125.0	138.5	13.0
57+00	4340	141.2	143.5	126.0	139.5	10.8
59+00	4340	144.8	146.5	126.4	140.9	5.2
61+00	4340	144.9	146.5	128.0	142.5	6.7
63+00	4340	145.3	146.5	130.0	143.9	8.6
65+00	4340	146.6	148.8	132.0	145.0	12.1
67+00	2995	150.2	151.4	134.6	150.0	7.1
69+00	2995	151.8	155.0	140.0	153.1	16.6

Table 2.4-9
PMF WATER SURFACE PROFILE AT PEAK DISCHARGE - FOOTHILL DRAINAGE BASIN
(Sheet 3 of 3)

Station	Discharge (ft^3/s)	Water Surface Elevation	Top of Berm Elevation	Channel Invert Elevation	Critical Depth Elevation	Average Velocity (ft/s)
71+00	2995	158.1	160.0	145.0	158.0	11.2
73+00	2995	160.5	162.0	150.0	161.5	14.9
75+00	2995	167.0	168.5	155.6	167.0	9.5
77+00	2140	166.3	173.6	159.9	169.0	5.2
79+00	2140	172.4	179.0	167.0	173.7	9.3
81+00	2140	184.9	191.0	179.0	185.8	14.4
83+00	2140	196.9	203.0	193.5	198.2	17.6
84+00	2140	204.7	206.5	198.2	204.7	8.7

Note: Refer to figure 2.4-22 for plot of profiles shown in table 2.4-9.

2.4.5 PROBABLE MAXIMUM SURGE AND SEICHE FLOODING

2.4.5.1 Probable Maximum Winds and Associated Meteorological Parameters

The probable maximum winds associated with maximum surge and seiche water levels at the San Onofre Nuclear Generating Station are caused by northeast Pacific tropical cyclones that reach the southern California coast. Although relatively strong winds may result from severe Santa Ana conditions or extratropical storms, these weather systems will not cause water levels along the southern California coast as high as those resulting from a tropical storm entering the coastal areas. The Santa Ana conditions cause winds that blow offshore and tend to cause low water levels rather than high levels. The winter and spring extratropical storms usually enter California from the west. The centers of these storms move into central or northern California. Cold fronts embedded in these storms move rapidly through southern California. Maximum winds speeds of 35 to 45 knots are associated with the frontal systems. The winds shift from southwest to west and then to northwest as the frontal system moves through the area. The rapid variation of the wind speed and directions are not conducive to producing high water levels along the coast. The tropical storm, on the other hand, moves more slowly with very high wind speeds embedded in the storm circulation. This provides the necessary time and fetch to generate maximum surge and seiche water levels as the storm moves across the coast.

Before meteorological satellite observations were available, it was generally believed that on the average about 10 tropical cyclones occurred each year during the tropical storm season, May to November. However, satellite data has shown that the average may be as high as 15 to 20 per year, making the northeast Pacific one of the most active regions of the world.(15)

Most of the northeast Pacific tropical storms form a few hundred miles off the west coast of Mexico between 10° and 20° north latitude over very warm waters. Massive moisture and cloudiness from the Intertropical Convergence Zone (located at around 10°N latitude in the summer) are fed directly into the southern half of the cyclones. During the midsummer months, the storms move northwestward and westward, paralleling the coast of Mexico. During the early and late season months, several storms usually curve to the northeast and cross the Mexican mainland coast or Baja California and eventually die over land.

The forward speed of movement of the tropical cyclones usually ranges between 5 to 15 knots. The average observed duration is from 4 to 5 days, with late-season cyclones on the average being of slightly shorter duration. Several storms have been tracked for more than 10 days.(16)

While tropical storms in the northeast Pacific are rather frequent during the summer months, they are not as intense as their counterparts in the Atlantic and western Pacific. In these latter regions about 65% of the tropical storms reach hurricane or typhoon intensity, while in the northeast Pacific only about 33% reach hurricane force.(17)

The size of the average storm is smaller than that of its Atlantic and western Pacific counterparts. The radius of gale force winds is thought to be relatively small. Now and then, however, a cyclone of huge proportions is encountered. Due to the scarcity of observational data, the intensity of the cyclones is often difficult to determine or estimate. The more severe ones can be assumed to have maximum sustained winds in the 100- to 125-knot range, while most of the storms that reach hurricane force (>64 knots) barely do so. However, a majority of the storms do not attain even this force.

Only one of these tropical cyclones has entered southern California in the last 50 years, with high winds and extensive damage. This occurred on September 25, 1939, when a tropical storm moved inland near Los Angeles.[18] September 1939 was unusual in that five tropical cyclones crossed the coast from Baja California northward. In addition to the southern California case, two other storms passed less than 200 miles south of San Diego (see figure 2.4-23).

The southern California tropical storm of September 25, 1939, was a violent storm in its early offshore history. On the morning of September 22, while west of the southern tip of Baja California, winds of 60 knots and barometer 971 millibars (28.67 inches) were reported near the center. Its offshore track from that point was parallel to the coast (see figure 2.4-23). This track was made possible by a strong ridge over the western United States and another offshore separated by an elongated inverted trough extending along the coast both at the surface and aloft. This trough persisted from the 19th through the 23rd, resulting in extremely high coastal and coastal mountain temperatures in southern and central California.

By the 24th, the ridge over the west had weakened, allowing the storm to veer toward the north-northeast and then toward northeast as it approached the southern California coast. The windspeed at San Pedro reached 41 knots before the 996-millibars (29.47 inches) low center entered the coast in that vicinity about 0800 PST on the morning of the 25th. The severity of this storm along the coast is indicated by a loss of 45 lives at sea, and property damage of approximately $2 million, largely from wave action at the coast.[19]

Although only one tropical storm has caused severe damage in the last 50 years, it is possible for conditions to occur that may produce several storms over a shorter period of time. Conditions conducive to entry of tropical storms into the southern California coast are weak summer north-westerly winds and a sluggish California current which results in high water temperatures along the coast. In some periods in the early 1800's the southern California coast apparently was affected by these tropical storms several times a year. These conditions were probably accompanied by abnormally high temperatures in autumn.[20]

Utilizing the climatology of the northeast Pacific tropical cyclones and the structure of typical hurricanes,[21] the track and the surface wind structure of the hypothetical maximum probable storm for the San Onofre site were constructed and are shown in figures 2.4-24, 2.4-25, and 2.4-26. The track of the storm is shown in figure 2.4-24. The speed of the hurricane is about 10 knots. The wind field for the hurricane for positions 1, 2, and 3 is shown in figure 2.4-25. Wind speeds in excess of 110 knots occur in these positions but diminish to 70-75 knots (figure 2.4-26) as it moves to position 4 on the southern California coast. The center of the hurricane crosses the coast near San Onofre and moves inland and dissipates.

2.4.5.2 Surge and Seiche Water Levels

Water levels antecedent to probable maximum surge and seiche levels are first discussed. Accepted conservative high and low tide levels and sea level anomalies for the San Onofre area are considered in establishing the antecedent water levels.

2.4.5.2.1 Tides

The character of the San Onofre tides is semidiurnal; i.e., two high tides and two low tides of differing amplitudes occur each day, on the average. The average of only the lower of the two low water levels is taken as the local bathymetric chart datum: mean lower low water (mllw). Mean lower low water thereby acts as a reference point in sea level variation discussions. The tide reference station local to San Onofre is at San Diego Bay.[22]

The specific location of San Onofre along the open coast north of San Diego necessitates that an amplitude ratio of 0.92 be applied to the San Diego reference data. This restricts the diurnal tidal range of the reference data to accurately reflect the tidal conditions at San Onofre.

Historically, at San Diego, where accurate absolute tidal levels have been recorded since 1906, the highest tide observed was on December 20, 1968, and the lowest tide observed was on December 17, 1933.[23] The extreme tides of 1968 and 1933 adjusted for the San Onofre location are +7.18 feet mllw and -2.66 feet mllw, respectively.

To establish the accepted conservative high and low tide levels for San Onofre, the 10% exceedance monthly spring tide was calculated. Semi-monthly spring tidal elevations at San Diego were obtained for the years 1968, 1970, 1972, 1974, and 1976 from tables of predicted tides (U.S. Department of Commerce for cited years). These data are shown in table 2.4-10. The cumulative of highest and lowest tides at San Diego is given in tables 2.4-11 and 2.4-12, respectively; these have been constructed from the data presented in table 2.4-10. In these, the 10 and 90% exceedance levels are identified to the nearest 0.1 foot, as being +7.6 feet and -1.9 feet mllw (the tide is above +7.6 feet and below -1.9 feet mllw, only 10% of the time). To these are applied the amplitude ratio correction (0.92) for San Onofre to convert San Diego tides to ones accepted as representative of San Onofre. This yields a spring high tide at San Onofre that has a 10% probability of exceedance of +7.0 feet mllw and a spring low tide that has only a 10% chance of being lower than -1.75 feet below mllw.

2.4.5.2.2 Sea Level Anomalies

Mean sea level variations (sea level anomalies) that are significant in a time frame of 2 weeks or larger and that occur off of coastal southern California are principally attributable to two factors: variations in atmospheric pressure and changes in the specific volume of sea water.[24] Both factors vary seasonally. Specific volume changes can be ascribed mainly to oceanic temperature variations. The sea can be expected to respond to the local atmospheric pressure changes at a rate of 1 cm or -1 cm change in water level for each millibar atmospheric pressure decrease or increase, respectively. The pressure and specific volume contributions summed together form isostatic sea level variations.

Table 2.4-10
SPRING TIDAL ELEVATIONS AT SAN DIEGO, CALIFORNIA (FEET, RELATIVE TO MLLW)

Month	1968			1970			1972			1974			1976		
	Day	High	Low	Day	High	Low	Day	High	Low	Day	High	Low	Day	High	Low
January	1	7.2	-1.6	8	7.7	-2.1	1	7.2	-1.6	8	7.7	-2.1	1	7.1	-1.5
	28	7.2	-1.8	21	6.4	-1.0	16	6.9	-1.3	22	6.3	-0.8	17	6.9	-1.3
							19	7.0	-1.5						
February	13	6.7	-1.3	5	7.5	-2.1	14	6.7	-1.3	6	7.3	-1.8	15	6.8	-1.2
	25	6.6	-1.5	19	6.1	-0.8	26	6.3	-1.1	20	6.0	-0.6	26	6.0	-0.8
March	13	6.3	-1.0	5	6.9	-1.6	13	6.2	-0.9	6	6.6	-1.3	14	6.2	-0.9
	25	5.8	-0.9	20	5.5	-0.3	25	5.4	-0.6	26	5.8	-0.3	26	5.2	-0.2
April	14	6.8	-1.4	6	6.5	-1.1	14	7.0	-1.6	7	6.4	-1.0	14	6.9	-1.5
	28	5.9	-0.5	23	6.2	-0.8	29	5.9	-0.5	23	6.4	-1.0	29	5.9	-0.5
May	12	7.3	-2.0	5	6.7	-1.3	13	7.3	-2.0	5	6.6	-1.2	13	7.2	-1.9
	27	6.2	-0.7	21	6.8	-1.4	27	6.2	-0.7	22	6.9	-1.6	29	6.2	-0.7
June	10	7.6	-2.1	2	6.8	-1.3	11	7.5	-2.0	3	6.6	-1.1	11	7.3	-1.8
	25	6.4	-0.8	19	7.3	-1.7	27	6.5	-0.9	20	7.4	-1.8	27	6.6	-0.9
July	9	7.7	-1.9	2	6.7	-1.1	10	7.5	-1.8	2	6.6	-0.8	10	7.3	-1.4
	25	6.7	-0.8	18	7.6	-1.8	25	6.8	-0.8	19	7.6	-1.7	26	6.8	-0.8
				31	6.6	-0.8									
August	7	7.5	-1.5	16	7.6	-1.5	7	7.2	-1.3	1	6.5	-0.5	7	7.0	-0.9
	22	6.7	-0.7	29	6.3	-0.4	23	6.7	-0.6	16	7.4	-1.3	24	6.8	-0.6
										30	6.1	0.1			
September	3	6.8	-1.1	13	7.1	-1.0	5	6.6	-0.7				5	6.4	-0.4
	24	6.5	-0.3	26	5.7	0.0	25	6.9	-0.5	14	6.8	-0.7	25	6.8	-0.5
October	3	6.1	-0.5	15	7.2	-1.0	2	5.7	-0.2	3	6.2	+0.2	8	6.1	+0.2
	23	7.2	-1.1	31	6.5	-0.4	23	7.4	-1.4	15	7.1	-0.9	24	7.4	-1.3
November	1	6.5	-0.4	13	7.3	-1.4	5	6.4	-0.4	1	6.7	-0.6	7	6.3	-0.3
	21	7.6	-1.8	29	6.9	-1.1	21	7.7	-1.9	13	7.2	-1.2	22	7.6	-1.8
										30	7.1	-1.3			
December	4	6.5	-0.7	12	7.2	-1.5	5	6.5	-0.7	12	7.0	-1.2	6	6.4	-0.6
	20	7.8	-2.1	29	7.2	-1.6	20	7.8	-2.1	29	7.4	-1.7	20	7.6	-1.9

Table 2.4-11

DISTRIBUTION OF SPRING HIGH TIDES AT SAN DIEGO
DURING FIVE YEARS

Elevation Above mllw (ft)	No. Occurrences	Percentage Probability of Occurrence	Cumulative Percentage Probability of Equaling or Exceeding Given Elevation
5.2	1	0.7	100.0
5.3	0	0	99.3
5.4	1	0.8	99.3
5.5	1	0.8	98.5
5.6	0	0	97.7
5.7	2	1.6	97.7
5.8	2	1.6	96.1
5.9	3	2.4	94.5
6.0	2	1.6	92.1
6.1	4	3.3	90.5
6.2	7	5.7	87.2
6.3	5	4.1	81.5
6.4	7	5.7	77.4
6.5	8	6.5	71.7
6.6	8	6.5	65.2
6.7	8	6.5	58.7
6.8	10	8.1	52.2
6.9	7	5.7	44.1
7.0	4	3.3	38.4
7.1	4	3.3	35.1
7.2	10	8.1	31.8
7.3	7	5.7	23.7
7.4	5	4.1	18.0
7.5	4	3.3	13.9
7.6	7	5.7	10.6
7.7	4	3.3	4.9
7.8	2	1.6	1.6
	123	100.0%	

Table 2.4-12

DISTRIBUTION OF SPRING LOW TIDES AT SAN DIEGO
DURING FIVE YEARS

Elevation Below mllw (ft)	No. Occurrences	Percentage Probability of Occurrence	Cumulative Percentage Probability of Equaling or Exceeding Given Elevation
+0.2	2	1.6	1.6
+0.1	0	0	1.6
0.0	1	0.8	2.4
-0.1	1	0.8	3.2
-0.2	2	1.6	4.8
-0.3	4	3.3	8.1
-0.4	5	4.0	12.1
-0.5	7	5.7	17.8
-0.6	6	4.9	22.7
-0.7	8	6.5	29.2
-0.8	10	8.1	37.3
-0.9	7	5.8	43.1
-1.0	6	4.9	48.0
-1.1	7	5.7	53.7
-1.2	4	3.3	57.0
-1.3	11	8.9	65.9
-1.4	5	4.0	69.9
-1.5	7	5.7	75.6
-1.6	6	4.9	80.5
-1.7	3	2.4	82.9
-1.8	8	6.5	89.4
-1.9	4	3.3	92.7
-2.0	3	2.4	95.1
-2.1	6	4.9	100.0
	123	100.0%	

Pattullo[25] has tabulated the monthly sea level anomaly found at the La Jolla pier of the Scripps Institution of Oceanography, 34 nautical miles southeast of San Onofre. Pattullo found monthly deviations from msl to vary anywhere between +8 cm to -9 cm. Therefore, a conservative estimate of the maximum isostatic sea level rise due to effects with monthly time scales would be +10 cm (0.33 foot), and the minimum stand of sea level due to such causes would be -0.33 foot. Referring to graphical presentations of La Jolla sea level data for a longer duration (Roden)[26] it is apparent that the ±0.33 foot figures for the isostatic anomaly, in fact, do represent appropriate extremes at perhaps even less than the 10% probability of exceedence level.

2.4.5.2.3 Maximum Surge Conditions

The maximum surge water level hypothetically possible and applicable to the site would result from the hypothetical maximum probable storm diagrammed in paragraph 2.4.5.1. In developing the hypothetical maximum tropical storm, particular attention was given to the configuration of its radius of maximum winds, the storm's forward speed, and the storm's track. A storm center trajectory lying farther offshore than the 1939 event (paragraph 2.4.5.1) was chosen. This would provide less frictional and thermal energy loss over land. Additionally, as with the 1939 storm this hypothetical storm would pass over an unseasonably warm seawater surface, which reduces the storm's dissipation rate as it moves northward. Just north of San Onofre the storm would curve sharply and slowly toward the coast; its center passing over the coast somewhat north of San Onofre so that its maximal south, south-southwest, and southwest winds would occur over San Onofre. For these wind directions the tropical storm with the above mentioned combination of maximizing conditions would produce the highest sustained winds at San Onofre with virtually no risk of being exceeded.

The high wind speeds from the south and southwest were obtained by: (1) elongating the axis of the storm in a northeast-southwest direction so as to direct the maximum winds to be in the southeast quadrant, and (2) moving the storm at a slow forward speed of 10 knots. The 6-hour durations are: 60 knots from south, 55 knots from south-southwest, and 50 knots from southwest. The 1-hour maximum credible wind speed for this storm is 70 to 75 knots.

2.4.5.2.3.1 Barometric Pressure Contribution to Surge

The lowest pressure with virtually no chance of being lower at San Onofre, associated with the hypothetical tropical storm moving into the area would be 985 millibars (29.10 inches Hg). The hydrostatic change in water level at San Onofre associated with a 985-millibar tropical storm would be +1.00 feet and the barometric increase in water level would be +1.20 feet.

2.4.5.2.3.2 Wind Stress Tide and Coriolis Tide Contributions to Surge

Storm surges are transient in nature and exhibit a main peak that is manifest as a sharp rise in water level and occurs near the vicinity of maximum storm winds, just before the arrival of the tropical storm. A simplified steady-state surge model can be employed to treat the case of the main surge peak. The model which has been employed is due to Bretschneider[27][28] and his co-workers. Bodine[29] has refined this technique and reduced it to a digital computer code. The general methodology[23] assumes that convective momentum terms can be ignored, the response to onshore wind stress is instantaneous, and relies on the existence of parallel depth contours in the longshore direction, a condition rather well met at San Onofre. The computations are carried out along lines transverse and perpendicular to the assumed parallel bottom contours. The transverse bathymetry sections used for this purpose are shown in figure 2.4-27. The fact that the storm center is necessarily over deep water most of the time where the phase velocity of long waves is great makes it indicative that large storm surges would not develop in the vicinity of San Onofre.

Besides the barometric surge of +1.20 feet already discussed, the model employed views a storm surge as composed of two parts: (1) the wind stress tide caused by winds directed normal to shore; and (2) the coriolis tide. The latter is a rise (or drawdown) of water caused by a current flowing parallel to shore, which may result from wind stress in that direction. However, the coriolis tide can persist even in the absence of local winds. Following passage of a storm, for instance, the longshore current it generates may inertially move along for days while slowly decaying through various frictional effects. Both surge components have been combined to give the heights above pre-existing elevations according to wind directions over a 6-hour duration: south, +0.78 foot; south-southwest, +0.40 foot; and southwest, +0.31 foot. Pre-existing water levels already discussed were added to the depths shown in figure 2.4-27 prior to surge computations due to the nonlinear addition of the various effects.

2.4.5.2.3.3 Summation of Maximum Surge Contributions

The maximum likely storm surge height, therefore, has been determined to be +1.98 feet above the antecedent water level. This figure is the sum of the barometric surge of +1.20 feet and the maximum surge components derived from the model of +0.78 foot. Hence, it is concluded that large surges will not develop in the vicinity of San Onofre. This precludes the necessity for a detailed two-dimensional treatment of surge such as a surge hydrograph.

2.4.5.2.4 Seiche Water Levels

Some of the most detailed measurements and analyses of long-period waves (normal shelf seiching background levels) over the continental borderland have been conducted near Oceanside, California, about 17 miles southeast of San Onofre.[(30)(31)] Seiche has been found to affect sea surface elevation by only 0.7 cm, which is considered negligible for water level calculations for southern California.

2.4.5.2.5 Still Water Level Extremes - Summary

Extreme high and low still water levels have been estimated at +9.3 and -2.6 feet mllw, respectively. These figures arise through the following causal factors, as discussed in paragraphs 2.4.5.2 and 2.4.11.2.

Causal Factor	Elevation (ft. mllw) High Water	Elevation (ft. mllw) Low Water
Astronomical tides (paragraph 2.4.5.2.1)	+7.0	-1.75
Isostatic anomaly (paragraph 2.4.5.2.2)	+0.33	-0.33
Maximum surge (paragraph 2.4.5.2.3)	+1.98	-0.55[a]
	+9.31	-2.63

2.4.5.3 Wave Action

Severe deep water storm waves determine the lowest and highest instantaneous water elevations in conjunction with long period phenomena; i.e, tide and storm surge. As severe waves are infrequent, it is usually necessary to determine their characteristics by hindcasting. A careful selection of past storms based on reported wave damage and strong winds is a prerequisite. Then, the deep water significant wave characteristics for each storm are hindcast from weather maps. A wave height distribution function[32] is used to determine the highest individual shallow water wave height, H_{max}, in the storm from the hindcasted significant wave height and period time histories.

Marine Advisors[33] and Intersea Research[34] examined a total of approximately 60 storms that occurred between 1900 and 1967 and that occurred near enough to San Onofre to be applied to this study. Twenty-five of the most severe storms were selected and hindcast (table 2.4-13).

The deep water wave data were corrected for refraction and shoaling at the San Onofre site, and also for island sheltering.

As waves enter into shallow water they are transformed by the bottom topography. This transformation is apparent as a decrease in wave velocity and a change in wave height. These two changes are brought about by separate physical processes. One, shoaling, always applies when waves travel into shallow water. It is the effect of the shoaling bottom on the advance of the wave form. As a result of this restricting effect the wave velocity is reduced; at first the wave height decreases but upon traveling into shallower water it increases until the wave breaks. The other process, refraction, occurs when the wave crest advances toward shore over a shallow, irregular bottom or over a shallow and smooth bottom at an angle to the bottom contours. The portion of wave that is in deeper water has a greater velocity than the portions in shallow water. This causes a bending of the crest and as a result wave heights are increased in some shore areas and reduced in others. Corrections for island sheltering are made by proportionately decreasing

[a] Surge drawdown is discussed in paragraph 2.4.11.2.

Table 2.4-13

HINDCASTED WAVES FOR PAST SEVERE STORMS

Storm Date	Deep Water			Shallow Water
	Direction (°T)	H_{max} (ft)	T_s (s)	H_{max} (ft)
March 9-11, 1904	228	22.6	8.5	18.5
March 8-10, 1912	155	40.9	12.3	27.8
December 16-17, 1914	220	29.3	10.0	25.8
January 28-30, 1915	272	35.1	10.4	20.7
February 1-3, 1915	268	36.3	11.4	22.5
April 29 - May 1, 1915	278	34.2	10.5	19.8
January 26-28, 1916	260	50.0	12.9	33.5
June 28-30, 1922	165	40.0	11.7	29.2
February 1-2, 1926	266	32.3	13.8	20.7
April 6-8, 1926	262	23.5	12.8	15.7
March 14-15, 1930	265	23.3	9.5	14.9
December 6-7, 1937	270	26.5	16.0	16.7
September 15-25, 1939	165	60.0	16.1	43.8
January 20-23, 1943	255	44.2	12.7	30.5
March 13-14, 1952	268	33.3	12.8	21.0
January 6-8, 1953	268	38.0	18.0	25.1
April 2-4, 1958	285	49.2	20.7	30.0
February 14-17, 1959	265	37.0	14.3	24.4
February 8-10, 1960	290	51.6	18.8	29.9
March 13-15, 1961	273	32.8	16.8	21.0
March 4-7, 1962	280	31.6	15.5	18.6
December 11-15, 1962	270	32.7	20.5	21.9
January 20-22, 1964	267	21.2	8.9	13.4
November 15, 1965	280	34.0	14.2	20.1
November 18, 1965	289	29.0	14.8	15.7

wave height and shifting the resulting wave mean direction when the deep water wave data has a path that is interfered with either by coastal islands or by parts of the coast itself for waves approaching the mainland obliquely.[35] The storm waves given in table 2.4-13 represent 64 years of record. The deep water wave data provided in this table include correction for refraction, shoaling, and island sheltering. The shallow water H_{max} values at San Onofre have been treated statistically by the Weibull method of extremes[36] with the assumption that the 1-in-64-year wave equals the highest H_{max} of the period covered. An extrapolation gives the 100-year highest individual shallow water wave of 46 feet. The highest hindcast wave was produced by the tropical storm of September 24-25, 1939 (described in paragraph 2.4.5.1). This is the only tropical storm in the past 75 years which followed such a trajectory as to produce severe waves in southern California waters. As shown in table 2.4-13, the greatest shallow water wave height offshore at San Onofre during the 1939 storm was 43.8 feet.

A hypothetical tropical storm was considered based on the concurrence of individual worst parameters. This storm was designed around the 1939 hurricane which was able to reach latitude 34 N with strong winds. Certain modifications were made, however, so that higher extreme wind conditions would be postulated in the San Onofre area than experienced in the 1939 case. The track, size, and configuration of the storm were all designed with the idea of a realistic storm that could reach San Onofre. Specific storm parameter modifications are discussed in paragraph 2.4.5.2.

Having an optimum final trajectory from the south, then credible conditions for worst storm wave generation includes an effective fetch of 400 miles with wind speed of 50 knots for 24 hours. This would generate a significant deep water wave height of 34 feet. The highest individual wave, corrected for sheltering, shoaling, etc., at the site is calculated at 54 feet (shallow water height). Its associated wave period would be 13 seconds. According to the extrapolation in figure 2.4-28, it would be 200-year return interval wave.

Using techniques and graphs given in U.S. Army CERC,[23] calculations have been made of the extreme instantaneous water levels due to the hypothetical storm waves. The highest crest elevation was determined for a +9.3 feet mllw still water level, and the lowest wave trough elevation was derived for a -2.6 feet mllw still water level as discussed in paragraph 2.4.5.2. Tsunami effects were not included because the simultaneous occurrence of the hypothetical worst storm and hypothetical worst tsunami is infinitesimal. Another objective of this calculation was to determine the lowest wave trough at the cooling system seawater intakes. The results of the calculation are presented in figure 2.4-29. Figure 2.4-29 shows that the lowest wave trough remains above -10 feet mllw until farther than 9000 feet distance from the coast. In preparing figure 2.4-29, the highest crest elevation is based on a 13-second breaking wave height that is limited by the still water depth. The lowest trough elevation is based on breaking waves of shorter period, yet whose height is depth-limited; these have a larger portion of their height lying below the still water level. As seen from the graph, the worst storm-generated wave of 54 feet would begin feeling the bottom at a distance offshore of approximately 11,000 feet and would be completely dissipated by the time it reaches the beach in front of the San Onofre seawall.

Intersea Research Corporation[37] calculated the seasonal frequency of occurrence of breaking wave significant height, period and direction at the San Onofre beach. The significant breaking wave height exceedance values from that report and the annual average values are presented in table 2.4-14.

Table 2.4-14

BREAKING WAVE SIGNIFICANT HEIGHT (H_b)
(feet)

H_b>	1	2	3	4	5	6	8	10	12
Summer (J-S)	100.0	86.4	62.5	18.6	10.4	1.4			%
Transition (A,M,O,N)	100.0	67.2	47.2	16.4	10.2	3.9	0.5		%
Winter (D-M)	91.2	52.6	33.0	16.1	11.9	5.9	1.3	0.5	0.2%
Annual	97.2	68.7	47.6	17.0	10.8	3.7	0.6	0.2	0.1%

Inasmuch as the significant wave height is the average of the highest one-third of the waves present, then two-thirds of the waves would be lower, and on an annual basis, the 1% height exceedance for all waves would be about 6 feet.

Thus, the calculated highest run up at the seawall of +27 feet mllw due to storm waves occurring during an extreme high water level of 15.6 feet mllw (including tsunami) presented by Intersea Research Corporation[34] is far more severe than the 1% exceedance surf height.

Wave action will not generate water levels above the elevation at the top of the seawall (+30.00 feet mllw); therefore, no further design provisions for protection of safety-related structures from waves are necessary.

2.4.5.4 Resonance

The possibility of oscillations of waves at natural periodicity, defined as resonance, is most applicable to a closed body of water such as a lake or embayment. The San Onofre site contrasts these types of physical confinements by being located on a coastal marine terrace adjacent to a long straight stretch of coastal shelf.

Aside from the tsunami consideration (see subsection 2.4.6), resonance of the entire continental borderland is responsible for the bulk of the evident seiche, which has been measured extensively (paragraph 2.4.5.2.4) 17 miles south of San Onofre, at Oceanside. Seiche has been found to affect sea surface elevation by only 0.7 centimeter which is considered negligible for all practical purposes.

2.4.5.5 Protective Structures

The San Onofre Units 2 and 3 plant grade is elevation +30.0 feet mllw. This is well above the maximum seawater elevation predicted for the occurrence of a maximum tsunami coincident with storm surge. Special structures designed to protect the site against wave action include the seawall and screen well perimeter wall. The onshore intake structure is arranged so that all penetrations, except in the screen well, are sealed against leakage of rising or surging seawater.

The screen well is surrounded by perimeter walls with top elevation at +30.0 feet mllw. This is well above the maximum water elevation predicted.

The recirculation gate slot openings of the circulating water system are set at elevation +30.5 feet mllw.

The seawall is a poured in place reinforced concrete cantilevered retaining structure. The top of wall elevation matches the Units 2 and 3 plant grade at elevation +30.0 feet mllw. The seawall is designed to withstand, without loss of functional capability, the design basis earthquake (DBE) followed by the maximum predicted tsunami with coincident storm wave action.[38] Additional design criteria include:

A. Location - generally following the natural bluff line

B. A seismic design factor - Seismic Class II criteria with additional requirement to maintain functional under DBE followed by tsunami

C. Wind load: W = 15 lb/ft^2 (Uniform Building Code)

D. Vehicle surcharge on retained earth = 250 lb/ft^2 (assumed)

E. In-place density = 130 lb/ft^2, with coefficient of friction ϕ = 35 degrees (San Mateo sand)

The offshore intake terminal structures and diffuser ports are designed to withstand maximum uprush and withdrawal velocities of current associated with the postulated tsunami.

The offshore conduits are buried with a minimum cover of 4 feet. This prevents the conduits from being subjected to any forces created by wave actions or currents.

2.4.6 PROBABLE MAXIMUM TSUNAMI FLOODING

2.4.6.1 Probable Maximum Tsunami

The maximum possible tsunami wave heights that could occur at the San Onofre site would be generated by local offshore earthquake activity. Major tsunamis have been occurring in the Pacific Ocean at the rate of about one every 4 years. Over the period of adequate historical records, tsunamis of large energy, generated in all known seismically active regions around the Pacific, have been noticed at southern California, but have generally not produced damage.[39]

The effects of a distantly generated tsunami are minimized at southern California by the presence of a broad continental borderland, which apparently reflects much incident low frequency energy back out to sea. The regions of known high runup from remote tsunamis seem to be confined to those having steep offshore slopes (e.g., Japan, Hawaii, western South America).

Because of the moderating affect of southern California's offshore border-land on distant tsunami waves, local offshore fault zones are considered to be the most probable generators for large waves at San Onofre. The closest such zone to the Unit 2 and 3 site is the hypothesized offshore Zone of Deformation discussed in paragraph 2.5.2.4.5. The closest portion of this zone is approximately 5 miles southwest of the Unit 2 and 3 site.

To study the affect at San Onofre caused by sea floor displacements on the offshore Zone of Deformation, detailed specific analyses were completed by Dr. Basil W. Wilson.[(40)]

Mathematical modelling of the hypothetical tsunami was conducted assuming an earthquake with a 7-foot vertical displacement component of the sea floor 5 miles offshore from San Onofre as the generating mechanism. This vertical displacement is much larger than would be expected to occur on the hypothesized offshore Zone of Deformation which, because of its northwest trend, should be characterized by predominately strike slip displacement. Dr. Wilson's study concluded that the wave induced by 7-foot sea floor displacement and occurring during simultaneous high tide and storm surge would have a maximum runup to elevation +15.6 feet mllw at the Unit 2 and 3 seawall. The event modelled by Dr. Wilson is certainly the maximum probable tsunami that could reach San Onofre.

Normal faulting was postulated for the hypothesized offshore Zone of Deformation because the conversion of large strike-slip movements on the sea floor to a tsunami wave near San Onofre would be inefficient. Further, there are no large topographic features oriented normal to the direction of strike-slip movement on the offshore Zone of Deformation. This is consistent with the lack of a tsunami associated with the 1906 San Francisco earthquake, which caused lateral motions of the floor of San Francisco Bay.

2.4.6.2 <u>Historical Tsunami Record</u>

Table 2.4-15 summarizes the available information concerning tsunami indications associated with offshore California earthquakes at various regional locations.

Table 2.4-15

EXAMINATION OF TIDE GAGE RECORDS ASSOCIATED WITH OFFSHORE CALIFORNIA EARTHQUAKES (Sheet 1)

Date Day/Mo/Yr	Earthquake Magnitude	Location	Tide Record Location	Tsunami Indication
14, 15 July 1918	6.5	Humboldt County	San Diego San Francisco	None None
19, 20 Nov 1918	VII[a]	Santa Monica Bay	San Diego San Francisco	None None
31 Jan 1 Feb 1922	7.6	Cape Mendocino	San Francisco	None
22, 23 June 1923	7.3	Cape Mendocino	San Francisco San Diego	Questionable None
29, 30 June 1925	6.3	Santa Barbara	San Diego Long Beach	None[b] Questionable
22, 23 Oct 1926	6.1	Monterey Bay	San Diego	Questionable
4, 5, Nov 1927	7.5	Point Arguello	La Jolla	Present Very small
			San Diego	Present Very small
			San Francisco	Present Very small
10, 11 Mar 1933	6.3	Long Beach	San Pedro	None[c]
		Newport Beach	San Diego	None
		Offshore	La Jolla	Questionable

(a) Modified Mercalli scale

(b) The time sequence of this record seems confused. There is a long period oscillation in the tide record (about 30 minutes) which seems to have preceded the earthquake by a few hours.

(c) Actual ground motion appears on this tide gage record. A 6-inch seiche of about 1-hour period was in oscillation at the time. The earthquake failed to produce a tsunami or materially disturb the existing seiche.

(d) Conspicuous lack of any long-period waves above background.

Table 2.4-15

EXAMINATION OF TIDE GAGE RECORDS ASSOCIATED WITH OFFSHORE CALIFORNIA EARTHQUAKES (Sheet 2 of 2)

Date Day/Mo/Yr	Earthquake Magnitude	Location	Tide Record Location	Tsunami Indication
30 June - 1 July 1941	5.9	Carpenteria	San Diego La Jolla	None None
9, 10 Feb 1941	6.6	Cape Mendocino	San Diego	Slight increase in seiching 14 hours later
			La Jolla Santa Monica Port Hueneme	None None General increase in seiching 36 hours later
			San Francisco	Increase in what appears to be a harbor resonance 14 hours later.
25, 26 Dec 1951	5.9	San Clemente Islands	San Diego[d] La Jolla[d] Long Beach[d] Santa Monica[d] Port Hueneme[d]	None None None None None

2.4.6.3 Source Tsunami Wave Height

2.4.6.3.1 Distant Tsunami Generating Sources

The cumulative world incidence of severe tsunamis has averaged 10 per century for 2400 years; most of these are generated by earthquakes associated with the great oceanic trench systems and volcanic arcs. Three trench systems (the Atacama, Aleutian, and Japan Trenches) have accounted for 33% of all Pacific Ocean tsunamis in the past 200 years.

The closest active trench system which could cause tsunamis at San Onofre is the Aleutian Trench. Because of its broad shelf topography, the southern California coast is not sensitive to such distantly-generated waves. Remotely-generated tsunamis have amplitudes on the same order as the astronomical tides in southern California as given in Marine Advisors Report A-163.[39]

Besides the great trench systems, there are other large scale tectonic structures which might be considered capable of generating tsunamis. These include the large east-west trending fracture zones (Mendocino, Murray, and Clarion Fracture Zones) and the north-south trending East Pacific Rise, which enters the Gulf of California. These structures have earthquakes associated with them, and thus might generate tsunamis. However, the predominant displacement on these tectonic zones is strike-slip, which does not produce a large amount of tsunami wave energy. In contrast, the ocean floor displacements which occur in the ocean trench systems are predominantly vertical, and are relatively efficient in producing tsunami wave energy. Strike-slip structures in the East Pacific are therefore not regarded as significant sources for distantly generated tsunamis.

2.4.6.3.2 Tsunamis of Local Origin

The hypothesized offshore Zone of Deformation is the controlling generator for protective design at San Onofre as discussed in paragraph 2.4.6.1. Estimates of the maximum tsunami wave height at this hypothesized Zone of Deformation is provided in paragraph 2.4.6.4.

Other faults in the Pacific Ocean are at a greater distance from the San Onofre coast, and could not have a tsunami effect at the Unit 2 and 3 site greater than the Postulated event 5 miles southwest of the site discussed in the Wilson Report.[40]

2.4.6.4 Tsunami Height Offshore

The maximum hypothetical tsunami to approach San Onofre would be generated from an assumed earthquake with a 7.07-foot vertical displacement component of the sea floor 5 miles offshore from San Onofre. To simulate the wave generated from this assumed maximum situation, a Fourier series representation of a sawtooth wave form of maximum height was considered by Dr. Wilson.[40] The main objective of this analysis was to identify the principal wave components that can be considered to be present and to be representative of the approximate stroke of the initial sea disturbance set up by the earthquake. From Wilson's Table III[40] it can be seen that the maximum wave height generated from a maximum vertical (dip slip) bottom offset of 7.07 feet would be 6.32 feet (with a period of 12.7 minutes).

2.4.6.5 Hydrography and Harbor or Breakwater Influences on Tsunami

The analysis used to translate design (controlling) tsunami waves from the 5-mile offshore generator location to the San Onofre site is presented by Wilson.[40] The fault location of the earthquake that would generate the controlling tsunami is approximately along the 60 meter (200 feet) depth contour between Dana Point and Oceanside as shown in Wilson's Figure 1. Wilson's Figure 13, a profile of the continental shelf off San Onofre, shows that the continental shelf off San Onofre cannot satisfactorily be approximated by a single uniform slope. Wave refraction diagrams were calculated numerically and computer plotted.

Wilson's Figure 14 gives part of the grid pattern of depths, in meters, used to define the topography of the area and Figure 15 shows the wave fronts and rays as computer plotted from calculations which determine the refraction of the waves. Furthermore, Wilson's Figure 16 gives the initial profile of the sea disturbance in conformity with the hydrodynamic properties of the continental shelf.

Bore formation and resonance effects would not become an influence in estimating the maximum tsunami run up from the controlling tsunami. No effect to safety-related facilities is expected from the occurrence of this tsunami.

2.4.6.6 Effects on Safety-Related Facilities

The controlling tsunami occurring during simultaneous high tide and storm surge produces a maximum runup to elevation +15.6 feet mllw at the Unit 2 and 3 seawall. When storm waves are superimposed, the predicted maximum runup is to elevation +27 mllw, as discussed in paragraph 2.4.5.3.

Tsunami protection for the Unit 2 and 3 site is provided by a reinforced concrete seawall constructed to elevation +30.0 mllw. Design parameters for the seawall are presented in paragraph 2.4.5.5.

No effect to safety-related facilities is expected from the occurrence of the controlling tsunami.

2.4.7 ICE EFFECTS

As described in section 2.3 the mild climate and general lack of freezing temperatures in this region make ice formation highly unlikely and it is, therefore, not considered credible.

2.4.8 COOLING WATER CANALS AND RESERVOIRS

Cooling water for San Onofre Units 2 and 3 is supplied by the Pacific Ocean and is transported to each unit by one intake conduit. No reservoirs or cooling water canals are needed or used in the system. For further information on San Onofre Units 2 and 3 cooling water system refer to subsections 9.2.5, ultimate heat sink, and 10.4.5, circulating water system.

2.4.9 CHANNEL DIVERSIONS

Upstream diversions associated with rivers, where low flow has an impact on dependable cooling water sources, is not a factor at the San Onofre site. Cooling water is exclusively supplied by the Pacific Ocean through conduits which have been designed to supply the minimum of 4% total intake conduit flow required for emergency cooling during any conceivable accident. For further information refer to subsection 9.2.5, ultimate heat sink.

2.4.10 FLOODING PROTECTION REQUIREMENTS

Flooding of the plant site from runoff initiating from the east side of Interstate 5 is prevented by the flood protection structure shown on Figure 2.4-4 and as discussed in paragraph 2.4.2.2.

Runoff resulting from precipitation along Highway 101 and directly onsite will be diverted by the site drainage facilities to the ocean. In areas where water may pond against openings or structures special provisions have been made. These features are discussed in section 3.4.

2.4.11 LOW WATER CONSIDERATIONS

2.4.11.1 Low Flow in Streams

Local rivers and streams are not involved in plant operations. Plant cooling water is supplied exclusively by the Pacific Ocean; therefore, low flow conditions in streams do not affect the plant.

2.4.11.2 Low Water Resulting from Surges, Seiches, or Tsunami

2.4.11.2.1 Surge and Seiche Low Water

Winds that blow offshore at San Onofre would cause the greatest lowering of water as a result of surge. Surge drawdown would be most pronounced during Santa Ana wind conditions. A maximum credible Santa Ana condition for San Onofre would produce northeast winds of 35 knots sustained for 12 hours. The greatest correspondent drawdown from the antecedent water level associated with maximum Santa Ana wind conditions is- 0.55 foot.

As mentioned in paragraph 2.4.5.2, seiche has been extensively measured near San Onofre and has been found to affect sea surface elevation by only 0.7 centimeter.

It is therefore concluded that neither surge- nor seiche-caused maximum drawdown conditions would affect the ability of safety-related features at San Onofre to function adequately.

2.4.11.2.2 Tsunami Low Water

The most severe low water that could hypothetically be assumed would involve the worst tsunami drawdown combined with the hypothetical extreme low still water level. The extreme low still water level at San Onofre is estimated to be -2.63 feet mllw. This is derived from a situation consisting of a severe Santa Ana wind condition, as discussed above, causing a

0.55-foot sea level depression, promptly following passage of a deep low-pressure center in a winter storm causing an isostatic anomaly of - 0.33 foot (from paragraph 2.4.5.2.2) and occurring simultaneously with the lowest probable astronomical tide of -1.75 feet mllw (from paragraph 2.4.5.2.1). The maximum high water level of +15.6 feet would also cause the worst tsunami drawdown, which would be -11.9 feet mllw at the coast. This incident could persist for only a few minutes and only under the improbable condition that all of the contributing influences occur and reach their limits simultaneously.

The worst low water case described above would not affect the ability of safety-related features to function at San Onofre. The circulating water system receives its cooling water from intakes located 3330 feet from the protective seawall and at maximum inlet depth of -20.75 feet mllw. Worst tsunami drawdown at the offshore intakes is -4.0 feet, as discussed in Dr. Wilson's report[(40)].

2.4.11.3 Historical Low Water

The lowest tide elevation determined for the site is -2.3 feet mllw. This is a historical low tide and was considered in determining the bottom of the circulating water pump suction bell. This was to ensure proper impeller submergence. For information on the calculated low water elevations associated with tsunamis, refer to paragraph 2.4.11.2.

The above information is graphically shown in figure 2.4-30 and was determined using tables from the Army Corp of Engineers Shore Protection Manual, and the U.S. Department of Commerce Tide Table (1976)[(41)].

2.4.11.4 Future Control

Plant cooling water is supplied exclusively by the Pacific Ocean. Anticipated future uses of the Pacific Ocean will not limit the cooling water flowrate. Therefore, there is no impact to safety-related facilities.

2.4.11.5 Plant-Requirements

The minimum required safety-related cooling water flow is discussed in section 9.2. The system is designed to ensure an adequate supply of cooling water during the most severe sequence of events that could reasonably be postulated. The suction bells of the saltwater cooling pumps are located at elevation minus 22 feet 6 inches mllw, as shown in figure 2.4-31, to assure impeller submergence. The minimum design submergence is 2.5 feet, or a low water elevation of minus 20 feet mllw. This is much lower than the lowest calculated water elevation of minus 11.9 feet mllw for the San Onofre site associated with the maximum possible tsunami drawdown. The saltwater cooling pumps were designed for an operating head of 80 feet. No special design is necessary for the saltwater cooling pump system effluent submergence, mixing, and dispersion since the effluent is discharged through the same facilities as the main circulating water pumps effluent. For the mixing and dispersion of the effluent, the system was designed with thermal diffusers that meet the State of California's thermal standard of 4° (Δ)T (change in temperature) over ambient at a 1000-foot radius from the discharge point at the shoreline and ocean bottom. To ensure submergence of the intake and discharge conduits at San Onofre Units 2 and 3 the conduit trenches will be backfilled with a select aggregate backfill material for a distance of 1125 feet seaward of the normal shoreline. This is to protect the conduit from effects of liquefaction during a severe postulated design basis earthquake. The design maximum tsunami drawdown at the coast was determined to be -11.9 feet, which would effectively produce a new shoreline 1000 feet from the normal shoreline. The maximum tsunami drawdown at the offshore water intake structures located 3330 feet from the protective seawall is -4.0 feet, as discussed in Dr. Wilson's report[40].

Low flows resulting for 100-year drought are not applicable to San Onofre since the cooling water supply is the Pacific Ocean. Periods of low water flow that could occur at the site during a tsunami drawdown or conduit blockage were considered in the plant's design. The minimum cooling water flow can always be supplied by the saltwater cooling pumps as mentioned above. For further information refer to paragraph 2.4.11.2, low water resulting from surges, seiches, or tsunami, and subsection 9.2.1, saltwater cooling system. The normal operation cooling water flow is 1849 ft^3/s per unit. For normal operating conditions this flowrate will always be met since the water supply from the Pacific Ocean is virtually unlimited. For further information refer to subsections 9.2.5, ultimate heat sink, and 10.4.5, circulating water system.

2.4.11.6 Heat Sink Dependability Requirements

The Pacific Ocean provides the ultimate heat sink, as described in subsection 9.2.5. The cooling water system (subsection 9.2.1) is designed so that it will perform all safety functions throughout all postulated surges and tides, with a minimum steady water level of -20.00 feet mllw. Dependability of the ultimate heat sink with respect to postulated failures in associated components or structures is described in subsection 9.2.1.

San Onofre is consistent with the applicable recommendations of Regulatory Guide 1.27. Subsection 9.2.5 provides further discussion of this regulatory guide.

2.4.12 DISPERSION, DILUTION, AND TRAVEL TIMES OF ACCIDENTAL RELEASES OF LIQUID EFFLUENTS IN SURFACE WATERS

The only release point from the plant to the surface waters is the discharge of the circulating water system (outfall diffusers) to the Pacific Ocean, which is not considered a source of potable water. The flowrate of this discharge is discussed in section 9.2. The locations and users of the surface waters are discussed in paragraph 2.4.1.2.

Subsection 15.7.3 discusses the design features of the plant which mitigate the effects of a tank leak or failure. In addition, subsection 15.7.3 and section 11.2 discuss the administrative controls and automatic interlocks, together with the fail-safe design of the instrumentation and control devices, which provide assurance against any release of liquid waste to the environs in excess of 10CFR20 limits.

Failure or overflow from an unprotected tank could result in an uncontrolled, unmonitored release of radioactive liquid. The storm drain system would route the release to the circulating water system's discharge to the Pacific Ocean. Administrative controls would limit the total radioactive inventory in unprotected tanks in accordance with NUREG-0472 and NUREG-0133. Control of the radioactive inventory ensures that the concentrations of radioactive material will remain below the limits in 10 CFR 20, Appendix B, Table II, Column 2, in the event of an uncontrolled release. Doses resulting from an uncontrolled release will remain below the limits of 10 CFR 50, Appendix I, and 10 CFR 100.

Routine releases of radioactive liquids are performed in accordance with the requirements of the Effluent Control Program and the Offsite Dose Calculation Manual. All releases of radioactive liquids are routed to the circulating water system for discharge to the unrestricted area. Section 11.2 describes the operation of the liquid radioactive waste systems.

The maximum pumping flowrate available is 140 gal/min by the primary radwaste tank pumps (see table 11.2-1). The flowrate in the circulating water system is approximately 830,000 gal/min. Therefore, an accidental release of radioactive effluent would result in about a 5900:1 dilution within the circulating water system. The initial dilution of circulating water being discharged through the outfall diffusers is about 10 times the total volume rate of flow.[42] Therefore, any accidentally discharged radioactive effluent would be diluted about 59,000:1 in the near field zone.

2.4.13 GROUNDWATER

2.4.13.1 Description and Onsite Use

San Onofre Units 2 and 3 are located at the southern boundary of the San Onofre Valley Groundwater Basin (Basin No. 9-3).[43] The Basin lies within the South Coastal Hydrologic subregion of California as defined by the California Region Framework Committee (1968).[43] The Basin extends inland from the coast about 21 kilometers (13 miles) and dissects the Santa Margarita Mountains which lie inland to the east (figure 2.4-5). The Basin is bounded on the south by the northwest-trending San Onofre Mountains which form a barrier to drainage toward the coast. A southwest-trending ridge separates the San Onofre Valley Basin from the San Mateo Creek Basin which lies immediately north.

San Onofre Valley Groundwater Basin is drained by San Onofre Creek and its tributaries Jardine, San Onofre North Fork, and San Onofre Canyons to the northeast, and San Onofre Canyon South Fork to the east. The drainage area of the San Onofre Valley Basin covers about 112 square kilometers (43 square miles) of which about 85% consists of steep sided mountains, about 10% consists of unconsolidated alluvium in the valleys, and about 5% of elevated terrace deposits. The Santa Margarita Mountains range in elevation from 122 to 152 meters (400 to 500 feet) near the coast to 975 meters (3198 feet) at Margarita Peak near the eastern boundary of the San Onofre Creek drainage divide. The valley floor of the San Onofre Basin ranges in elevation from 3 meters (10 feet) near the ocean to a maximum of 244 meters (800 feet) at the head of Jardine Canyon.

Stream gradients in lower San Onofre Creek range below 1%. Gradients in the tributary canyons range from 1.5 to 2.5% in the lower reaches, increasing up to 7.5% in the upper reaches.

The important water-bearing formations in the San Onofre Valley Basin consist of sedimentary strata of Pliocene, Pleistocene, and Recent age.(44) Older formations are well indurated and are essentially nonwater-bearing.(45) These older rocks consist of the Miocene Monterey, the San Onofre Breccia, the older La Jolla Group and pre-Tertiary sedimentary rocks.

The oldest of the productive water-bearing strata is the Capistrano Formation. The Capistrano consists of poorly to semi-consolidated, thinly-bedded marine siltstone, fine-grained sandstone and shale with local limestone concretions, conglomerate, and breccia. The Capistrano Formation crops out immediately to the northwest of San Mateo Creek in southern Orange County.(45)

The water-bearing San Mateo Formation underlies the portion of the San Onofre Valley Basin west of the Cristianitos fault (see figure 2.4-32). Beneath the San Onofre Generating Station, the San Mateo consists of about 274 meters (900 feet) of light brown to yellow, medium- to coarse-grained sandstone. The formation is massive to thickly bedded, poorly cemented and well consolidated.

Alluvium is the most important of the water-bearing strata of the San Onofre Valley Basin,(43)(45) and occurs as unconsolidated valley fill reaching a maximum depth of about 30 meters (100 feet) and an average depth of about 21 meters (70 feet).(44) Alluvium is composed of boulders, gravel, sand, and silt. Production wells in the San Onofre Basin are located exclusively in the alluvial area which is the primary source of groundwater.

The principal recharge areas are the stream channels and alluvium in the upper parts of valleys.(43) Percolation of precipitation is the principle source of recharge. Minor amounts of water recharge the basin from percolation of recycled sewage effluent.(46)

Figures 2.4-33 and 2.4-34 show locations of wells and groundwater contours for typical basin high and low groundwater conditions. Groundwater occurrence, east of the Cristianitos fault, is restricted almost entirely to the alluvium. This is due to the thick sequence of relatively impermeable formations underlying the alluvium in this location. Groundwater moves downstream through the alluvium and passes over the Cristianitos fault. West of the fault the alluvium is underlain by, and in hydraulic continuity with, the San Mateo Formation. The contours (figures 2.4-33 and 2.4-34) indicate that groundwater flow through the alluvium has a shallower gradient and is less restricted than movement occurring within the San Mateo Formation. Contours indicate that groundwater movement is to the west and southwest toward the ocean. Geologists at Camp Pendleton have indicated that well data suggests that little groundwater movement occurs between the San Onofre Valley and San Mateo Valley groundwater basins. For this reason groundwater conditions in the San Mateo Basin should have no effect on the groundwater conditions beneath or in the vicinity of the site.

Fresh water requirements of the San Onofre plant will be met totally by water obtained from local water agencies and therefore no water will be derived from aquifers beneath or in the vicinity of the site for plant-related use.

2.4.13.2 Sources

The San Onofre Valley groundwater basin lies completely within the boundary of the Camp Pendleton Marine Corps Base. Groundwater use within the basin is under the direction and control of the Marine Corps. Presently, all water derived from the San Onofre Basin is for military use. Military security dictates that detailed information concerning amounts of water withdrawn, water levels, and locations of production wells remains classified. However, general information is available, including a limited amount of well data. San Onofre Valley Basin groundwater supplies only a partial quantity of Camp Pendleton's total consumption and is limited directly by the amount of precipitation and recharge which occurs. Marine Corps policy requires the maintenance of a seaward gradient of the groundwater table at all times to prevent intrusion of saline water into fresh water aquifers.(47) This policy prohibits the withdrawal of considerable amounts of groundwater stored in alluvium below or near sea level. Past groundwater withdrawals have fully utilized the basins potential up to the policy limits. Future groundwater usage from the San Onofre Basin is expected to remain the same as past usage with no projected changes.

Groundwater fluctuations within the San Onofre Basin are controlled primarily by recharge and groundwater pumpage by the Marine Corps. Indications are that the basin rapidly accepts recharge. Well data have shown the basin to be almost completely replenished within 1 year (1952) following a 6-year dry spell.(45)(44) Largest fluctuations of the groundwater table occur in the upper portion of San Onofre Creek and the area immediately west of the Cristianitos fault (see figure 2.4-32).

The average groundwater elevation beneath the site is +5 mllw.(46) Fluctuations within the pumped regions of the San Onofre basin have had little impact on the level of groundwater at the San Onofre site. Monitoring of groundwater levels at the San Onofre site for a ten-year period between 1963 and 1974 has shown the water table to vary from +2.7 feet to +5.7 feet mllw in the vicinity of the containment spheres.

Tidal effects on the groundwater levels in piezometers at the site have been monitored. Wells located closer to the ocean are generally more responsive to tidal fluctuations. Amplitudes of the fluctuations in observation wells are proportional to amplitudes of tidal fluctuations. The ratio of observation well to tidal fluctuations range from 0.1 to 0.3 for wells located between the containment spheres and the shore. Wells located a few hundred meters east of the unit's centerline are less responsive. The time lag between tidal highs and lows and the corresponding change in observations wells is generally about an hour (appendix 2.4A).

Groundwater contours are shown in figures 2.4-33 and 2.4-34 for typical high and low groundwater conditions.(47) Groundwater gradients within the alluvium to the east of the Cristianitos fault range from about 0.83% to 1.00%. Gradients in the alluvial portions of the lower basin to the west of the Cristianitos fault range from about 0.11% to 0.50%. Gradients within the San Mateo Formation are slightly higher ranging from 0.17% to nearly 1.0% with groundwater elevations dropping to sea level at the coast. Groundwater gradients are steepest over the Christianitos fault ranging from 1.25% to 1.67%.

The San Mateo Formation underlies the site to a depth of approximately 274 meters (900 feet). Boring logs indicate that the San Mateo is quite homogenous from the surface to below 91 meters (300 feet).

Pump test data indicate an average horizontal permeability for the San Mateo Formation of 0.0076 m/min (0.025 ft/min). Data were evaluated on the basis of several approaches. These included: (1) equilibrium methods, i.e., methods based on the assumption that a steady-state drawdown condition had been reached, and (2) nonequilibrium methods: methods based on the mathematical relationship between the rate of water lowering to permeability prior to reaching a steady-state (appendix 2.4A, page 3).

Detailed data and the pump test report are included in appendix 2.4A. A minimum value for vertical permeability for the San Mateo Formation of 0.0015 m/min (0.005 ft/min) was determined on the basis of grain size (using Allen Hazen's formula and correction).(48)

Studies have shown that reversal of groundwater flow from the site toward pumping wells in San Onofre Valley Basin cannot reasonably occur. According to SCE San Onofre Unit 1, Final Engineering Report and Safety Analysis, page 8 (1965),(47) "The established minimum pumping level for San Onofre Creek wells is above the elevation of the water table at the site so that even under extreme pumping conditions in San Onofre Creek, a seaward gradient will exist. Hence, a flow of groundwater toward the ocean from both San Onofre Creek and the site will be assured."

The groundwater table beneath the site approaches sea level as movement toward the ocean occurs. The groundwater gradient across the site is therefore influenced by tidal fluctuations. Piezometer measurements at the site indicate the gradient ranges below 0.3%. Available groundwater elevation data at the nearest Marine Corps observation well (9/7 -24H1), which lies about 914 meters (3000 feet) northwest of the site (figure 2.4-5), indicate that the groundwater table normally ranged between +10 feet and +12 feet mllw at that well from 1951 to 1972. The only time during this period when the measured level fell outside of this range was in 1964 when the elevation dropped to 7.8 feet mllw. This was probably caused by the continuous pumping of 15 dewatering wells at the site during construction of San Onofre Unit 1.

Recharge of the San Onofre Valley Basin occurs in the upstream parts of stream channels and alluvium in the upper region of the valley.(43) There are no potential groundwater recharge areas within the influence of the plant.

2.4.13.3 Accident Effects

As discussed in paragraphs 2.4.13.1 and 2.4.13.2 there is a groundwater gradient toward the ocean of approximately 0.4 to 0.6%. The nearest water supply wells serve the Marine Corps and are located in San Onofre Creek over 1 mile inland from the plant site. Marine Corps policy is to maintain the groundwater table throughout Camp Pendleton sufficiently above mean sea level to eliminate the possibility of saline water intrusion from the ocean into the freshwater aquifers. The established minimum pumping level for the San Onofre Creek wells is above the elevation of the water table at the site. Thus a seaward gradient will exist even under extreme pumping conditions in San Onofre Creek and the flow of groundwater toward the ocean from both San Onofre Creek and the site is assured. Based upon this gradient, groundwater movement from the site toward any present or projected users will not occur. There is no present or projected usage of groundwater at the San Onofre site. In addition, subsection 15.7.3.3 discusses the design features of the plant which mitigate the effects of a tank leak or failure. Based upon the above, no analysis of an accidental release of liquid radioactive material is required.

2.4.13.4 Monitoring for Safeguard Requirements

Observation wells around the site were established and monitored during the construction phase of Units 2 and 3. Monitoring of these confirmed a seaward gradient.

2.4.13.5 Design Bases for Subsurface Hydrostatic Loading

The design bases for groundwater induced hydrostatic loading on subsurface portions of safety-related structures, systems and components are as follows:

A. Elevation +5.00 feet above MLLW is the design limit for hydrostatic loading.

B. The soil below elevation +5.00 feet is either compacted to 100% optimum density or is left in an undisturbed condition.

C. Hydrostatic lateral earth pressures are determined by the relative elevation of each structure with the maximum groundwater pressures added directly to the equivalent soil fluid pressures.

Dewatering during construction is carried out so that any portion under construction is completely dewatered. Water is not permitted to rise until the structure is completely stable against hydrostatic forces. When construction and backfill is complete, the dewatering system is removed. No permanent dewatering for San Onofre Units 2 and 3 is required. Dewatering is discussed in detail in paragraph 2.5.4.6.

All safety-related structures are designed to withstand the appropriate design loads at the design groundwater condition.

An evaluation of the groundwater level at San Onofre Unit 1 was conducted as reported to the NRC in reference 49. The results of this evaluation indicate that the median groundwater level would be 5.6 feet MLLW. The new value of 5.6 feet is in close agreement to the design value of 5.0 feet and is considered to impose equivalent structural loads. It is expected that additional data would provide additional small variations in the median groundwater value which would also not affect design.

2.4.14 TECHNICAL SPECIFICATIONS AND EMERGENCY OPERATION REQUIREMENTS

No technical specifications or emergency procedures are required in the event of probable maximum precipitation rainfall to minimize the impact on safety-related facilities.

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