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March 11, 2015

PG&E Letter DCL-15-034

U.S. Nuclear Regulatory Commission
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10 CFR 50.54(f)

Docket No. 50-275, OL-DPR-80
Docket No. 50-323, OL-DPR-82
Diablo Canyon Power Plant Units 1 and 2
Final Response to Request for Information Pursuant to 10 CFR 50.54(f) Regarding Recommendation 2.1 Flooding

References:

1. NRC Letter, "Request for Information Pursuant to Title 10 of the Code of Federal Regulations 50.54(f) Regarding Recommendations 2.1, 2.3, and 9.3 of the Near-Term Task Force Review of Insights from the Fukushima Dai-ichi Accident," dated March 12, 2012

2. NRC Letter, "Prioritization of Response Due Dates for Request for Information Pursuant to Title 10 of the Code of Federal Regulations 50.54(f) Regarding Flooding Hazard Reevaluations for Recommendation 2.1 of the Near Term Task Force Review of Insights from the Fukushima Dai-Ichi Accident," dated May 11, 2012

Dear Commissioners and Staff:

On March 12, 2012, the Nuclear Regulatory Commission (NRC) issued Reference 1 to Pacific Gas and Electric Company (PG&E). Enclosure 2 of Reference 1 contains specific Requested Actions, Requested Information and Required Responses associated with Recommendation 2.1 for Flooding Hazard Reevaluation. Item 1 of the Requested Information asks for a final Hazard Reevaluation Report.

Per Reference 2, the Flood Hazard Reevaluation Report for Diablo Canyon Power Plant (DCPP) is to be submitted by March 12, 2015, which corresponds to three years from the date of Reference 1.

Enclosure 1 contains PG&E's DCPP Unit 1 and Unit 2 Flood Hazard Reevaluation Report. The report includes an Interim Action Plan that documents actions planned or taken to address the reevaluated hazard. These interim actions are being taken to address the results of the beyond design basis localized intense precipitation reevaluation which shows flooding levels in excess of those assumed to establish the current licensing basis requirements for flooding.

In accordance with the required responses as identified in Enclosure 2 of Reference 1, PG&E will complete and submit to NRC the DCPP Unit 1 and Unit 2 Integrated Assessment Report prior to March 13, 2017. PG&E has implemented, and will maintain, interim actions as set forth in Enclosure 1 until PG&E has completed the Integrated Assessment Report. PG&E is making a new regulatory commitment (as defined by NEI 99-04), as shown in Enclosure 2.

If you have any questions, or require additional information, please contact Mr. Patrick Nugent at (805) 781-9786.

I declare under penalty of perjury that the foregoing is true and correct.

Executed on March 11, 2015.

Sincerely,

Barry Allen

Barry Allen
Vice President, Nuclear Services

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Diablo Canyon Power Plant Units 1 and 2
Flood Hazard Reevaluation Report

In Response to 50.54(f) Information Request
Regarding Near-Term Task Force Recommendation 2.1

Revision 0
March 2015

Revision	Prepared by:	Reviewed by:	Approved by:
0	M. Olsofsky	J. Neale	S. Maze
Date	2/27/2015	3/6/2015	3/6/2015

Revision Summary

Revision	Required Changes to Achieve Revision	Date
0	Initial Issuance	3/6/2015

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1.0 PURPOSE

This report provides the Pacific Gas and Electric Company (PG&E) response to the U.S. Nuclear Regulatory Commission's (NRC) March 12, 2012 request for information pursuant to the post-Fukushima Near-Term Task Force (NTTF) Recommendation 2.1 flooding hazards reevaluation of Diablo Canyon Power Plant (DCPP).

1.1 Background

In response to the Fukushima Dai-ichi nuclear facility accident resulting from the March 11, 2011 earthquake and subsequent tsunami, the NRC established the NTTF to conduct a systematic review of NRC processes and regulations, and to make recommendations to the NRC for its policy direction. The NTTF reported a set of recommendations that were intended to clarify and strengthen the regulatory framework for protection against natural phenomena.

On March 12, 2012, the NRC issued an information request pursuant to Title 10 Code of Federal Regulations (CFR) 50.54(f) (NRC, 2012a) which included six enclosures:

1. NTTF Recommendation 2.1: Seismic
2. NTTF Recommendation 2.1: Flooding
3. NTTF Recommendation 2.3: Seismic
4. NTTF Recommendation 2.3: Flooding
5. NTTF Recommendation 9.3: Emergency Preparedness
6. Licensees and Holders of Construction Permits

In accordance with Enclosure 2 of the NRC 10 CFR 50.54(f) letter request (NRC, 2012a), licensees are required to reevaluate the flooding hazards at their sites using the present-day regulatory guidance and methodologies that are being used for early site permits (ESP) and combined license applications (COLA).

1.2 Requested Actions

Per Enclosure 2 of the NRC 10 CFR 50.54(f) letter request (NRC, 2012a),

> *Addressees are requested to perform a reevaluation of all appropriate external flooding sources, including the effects from local intense precipitation on the site, probable maximum flood (PMF) on stream and rivers, storm surges, seiches, tsunami, and dam failures. It is requested that the reevaluation apply present-day regulatory guidance and methodologies being used for ESP and COL reviews including current techniques, software, and methods used in present-day standard engineering practice to develop the flood hazard. The requested information will be gathered in Phase 1 of the NRC*

staff's two phase process to implement Recommendation 2.1, and will be used to identify potential vulnerabilities.

For the sites where the reevaluated flood exceeds the design basis, addressees are requested to submit an interim action plan that documents actions planned or taken to address the reevaluated hazard with the hazard evaluation.

Subsequently, addressees should perform an integrated assessment of the plant to identify vulnerabilities and actions to address them. The scope of the integrated assessment report will include full power operations and other plant configurations that could be susceptible due to the status of the flood protection features. The scope also includes those features of the ultimate heat sinks (UHS) that could be adversely affected by the flood conditions and lead to degradation of the flood protection (the loss of UHS from non-flood associated causes are not included). It is also requested that the integrated assessment address the entire duration of the flood conditions.

PG&E submitted a 90-day response letter (PG&E Letter DCL-12-059) to the NRC, titled "Pacific Gas and Electric Company's Response to NRC Request for Information Pursuant to 10 CFR 50.54(f) Regarding the Flooding Aspects of Recommendations 2.1 and 2.3 of the Near-Term Task Force Review of Insights from the Fukushima Dai-ichi Accident," dated June 7, 2012 (PGE, 2012a). In the letter, PG&E committed to responding to the request for information (RFI) in the NRC's Section 50.54(f) letter.

1.3 Requested Information

This report provides the following requested information for DCPP, in accordance with Enclosure 2 of the NRC 10 CFR 50.54(f) letter request (NRC, 2012a):

a. Site information related to the flood hazard. Relevant structure, systems and components (SSCs) important to safety and the ultimate heat sinks (UHS) are included in the scope of this reevaluation, and pertinent data concerning these SSCs should be included. Other relevant site data includes the following:
    i. Detailed site information (both designed and as-built), including present-day site layout, elevation of pertinent SSCs important to safety, site topography, as well as pertinent spatial and temporal data sets (Section 2.0);
    ii. Current design basis flood elevations for all flood causing mechanisms (Section 2.3);
    iii. Flood-related changes to the licensing basis and any flood protection changes (including mitigation) since license issuance (Section 2.3.3);
    iv. Changes to the watershed and local area since license issuance (Section 2.4);
    v. Current licensing basis flood elevations for all flood causing mechanisms (Section 2.3.2);
    vi. Additional site details, as necessary, to assess the flood hazard (i.e., bathymetry, walkdown results, etc.)

b. Provide evaluations of the flood hazard for each flood causing mechanism, based on present-day methodologies and regulatory guidance. Analyses are provided for each flood causing mechanism that may impact the site including local intense precipitation and site drainage, flooding in streams and rivers, dam breaches and failures, storm surge and seiche, tsunami, channel migration or diversion, and combined effects. Mechanisms that are not applicable at the site may be screened-out; however, a justification should be provided. Provide a basis for inputs and assumptions, methodologies and models used including input and output files, and other pertinent data (Section 3.0).

c. Comparison of current and reevaluated flood causing mechanisms at the site. Provide an assessment of the current design basis flood elevation to the reevaluated flood elevation for each flood causing mechanism. Include how the findings from Enclosure 2 of the 10 CFR 50.54(f) letter (i.e., Recommendation 2.1 flood hazards reevaluation) support this determination. If the current design basis flood bounds the reevaluated hazard for all flood causing mechanisms, include how this finding was determined (Section 4.0).

d. Interim evaluation and actions taken or planned to address any higher flooding hazards relative to the design basis, prior to completion of the integrated assessment described below, if necessary (Section 5.0).

e. Additional actions beyond Requested Information Item 1.d taken or planned to address flooding hazards, if any (Section 6.0).

1.4 Applicable Guidance Documents

The following documents were used as guidance in performing the flooding hazards reevaluation analyses:

ANSI/ANS, 1992, American Nuclear Society (ANSI/ANS), "Determining Design Basis Flooding at Power Reactor Sites ANS 2.8-1992," La Grange Park, Illinois, 1992.

NRC, 1977, NRC, "Design Basis Floods for Nuclear Power Plants," Regulatory Guide 1.59, Revision 2 Washington, D.C., 1977.

NRC, 1978a, NRC) "Standard Format and Content of Safety Analysis Reports for Nuclear Power Plants," Regulatory Guide 1.70, Revision 3, Washington, D.C., 1978.

NRC, 2007, NRC, "Standard Review Plan for the Review of Safety Analysis Reports for Nuclear Power Plants: LWR Edition," NUREG-0800, Washington, D.C., March, 2007.

NRC, 2009, NRC, "Tsunami Hazard Assessment at Nuclear Power Plant Sites in the United States of America - Final Report," NUREG/CR-6966, PNNL-17397, Richland, WA, March 2009.

NRC, 2011, NRC, "Design-Basis Flood Estimation for Site Characterization at Nuclear Power Plants in the United States of America," NUREG/CR-7046, Washington, D.C., November, 2011.

NRC, 2013, NRC, "Guidance for Performing a Tsunami, Surge and Seiche Flooding Safety Analysis Revision 0," Japan Lessons-Learned Project Directorate Interim Staff Guidance, JLD-ISG-2012-06, January 4, 2013.

1.5 Notes on Terminology

JLD-ISG-2012-06 suggests that the term "probable maximum" should be replaced with "design basis" for flood-causing mechanisms (e.g., "probable maximum storm surge" would be replaced with "design basis storm surge"). However, to avoid confusion with the current design basis, "probable maximum" terminology will be used to describe the reevaluated flood-causing mechanisms, as the new analyses are not being adopted at this time as the plant's "design basis."

2.0 SITE INFORMATION

The DCPP site consists of approximately 750 acres located in San Luis Obispo County, California, adjacent to the Pacific Ocean and roughly equidistant from San Francisco and Los Angeles. The DCPP site is approximately 12 miles west-southwest of the city of San Luis Obispo. San Luis Obispo County is bounded on the north by Monterey and Kings Counties, on the west by the Pacific Ocean, on the east by Kern County, and on the south by Santa Barbara County. San Luis Obispo County covers approximately 3,300 square miles of land area. The site location map is presented as Figure 2-1.

2.1 Datums and Projections

Various horizontal and vertical datums and mapping projections are referenced throughout this Report. This section describes the horizontal and vertical datums and mapping projections used, their definitions and relationships, and the methods used to convert from one datum or projection to another.

2.1.1 Horizontal Datums and Projections

A horizontal datum is a system which defines an idealized surface of the earth for positional referencing. The North American Datum of 1983 (NAD83) is the official horizontal datum for United States surveying and mapping activities (replacing NAD27, which was used during DCPP design, construction and licensing). The NAD83 datum was computed as a geocentric reference system and is based on the adjustment of 250,000 points, including 600 satellite Doppler stations, which constrain the system to a geocentric origin. The computation of the NAD83 removed significant local distortions from the network which had accumulated over the years, using the original observations, and made the NAD83 much more compatible with modern survey techniques. Therefore, NAD83 is the preferred horizontal datum referenced in new site survey studies performed for the DCPP site. Measurements are typically recorded using a complex spherical coordinate system (i.e., the geographic coordinate system of latitude and longitude).

A map projection is a mathematical transformation that converts a three-dimensional (spherical) surface onto a flat, planar surface. Different projections cause different types of distortions, and depending on their intended use, projections are chosen to preserve different relationships of characteristics between features. Projections in the United States are typically defined as State Plane coordinate systems with units of Northing and Easting. The extent of State Plane coordinate systems are limited by the acceptable distortion. The United States is divided into many State Plane maps; large states, such as California, are defined by several maps, i.e. Zones. The most recently recorded DCPP site survey data, provided as input for the data contained in this Report, uses the NAD83 horizontal datum projected onto the California State Plane Coordinate System, Zone 5 (CCS Z5), referencing the NAD83 (2007 Epoch) adjustment, and is based on Global Positioning System (GPS) observations.

2.1.2 Vertical Datums

A vertical datum is a surface of zero elevation to which heights of various points are referred in order that those heights be in a consistent system. The most predominant types of vertical datums that are used today are tidal datums and fixed geodetic datums. Tidal datums are determined by averaging the level of water at a tide gage over time. Fixed geodetic datums are predominantly determined through a process of surveying known as geodetic leveling, determining the height differences between points in the ground known as bench marks. These height differences can only yield actual heights at the benchmarks if at least one datum origin point is chosen to serve as the absolute level of the vertical datum. It is frequently the practice of those responsible for defining a geodetic datum, to choose a datum origin point that is also at a tide gage so a relationship between the tidal and geodetic datums exists. The following is a list of commonly referenced tidal and fixed datums, as defined by the National Oceanic and Atmospheric Administration (NOAA):

- Mean Higher High Water (MHHW) - the average of the higher high water height.
- Mean High Water (MHW) - the average all high water heights.
- Mean Sea Level (MSL) - the arithmetic mean of hourly heights.
- Mean Low Water (MLW) - the average of all the low water heights.
- Mean Lower Low Water (MLLW) - the average of the lower low water height.
- North American Vertical Datum of 1988 (NAVD88) - fixed vertical control datum, referenced to the tide station and benchmark at Pointe-au-Pere, Rimouski, Quebec, Canada.
- National Geodetic Vertical Datum of 1929 (NGVD29) - fixed vertical control datum, affixed to 21 tide stations in the United States and 5 in Canada.

During DCPP construction and licensing (1960s through mid 1980s), the "accepted equation" between 'Sea Level Datum (SLD) of 1929 General Adjustment' (also commonly referred to as MSL '29 and National Geodetic Vertical Datum of 1929 [NGVD29]) was a difference of 2.6 feet from MLLW, which is the relationship published by the U.S. Coast and Geodetic Survey (USC&GS) for May of 1953. This "accepted equation" of 2.6 feet was adopted by DCPP for its construction and licensing. Therefore, for historical design and licensing documents at DCPP, Elevation 0 MLLW is equal to Elevation -2.6 feet (ft.) MSL. Also noteworthy for DCPP, is that the historical use and reference to the datum "MSL" during design, licensing, and construction at DCPP is not necessarily the true "mean sea level" tidal datum of record for when the survey data was recorded. The historical use of the term "MSL" during DCPP design and licensing has been commonly interchanged with, and is equal to, the NGVD29 (SLD29) datum (which is the true original reference datum for the plant).

Considering that NAVD88 is the current official vertical datum for surveying and mapping activities in the United States by federal agencies using vertical height information (replacing NGVD29), the NRC has expressed a preference for flood level reporting in NAVD88. Therefore, NGVD29), the NRC has expressed a preference for flood level reporting in NAVD88. Therefore, the most recent DCPP site surveys referencing a local PG&E specific vertical datum have been converted to NAVD88. For convenience of applicability, when other datums are referenced in this Report, the equivalent elevation in NAVD88 will also be provided.

For reference in this report, MLLW is 0.32 ft. above NAVD88 and MSL is 2.92 ft. above NAVD88. The mathematical equation for conversion is provided below:

NAVD88 = MLLW + 0.32 ft. = MSL + 2.92 ft.

2.2 DCPP Description

The DCPP site occupies a coastal terrace that ranges in elevation from 62.9 to 152.9 ft. NAVD88 (60 to 150 ft. MSL) above sea level and is approximately 1000-ft. wide. The seaward edge of the terrace is a near vertical cliff. With the exception of the intake and discharge facilities, plant grade is at elevation 87.9 ft. NAVD88 (85 ft. MSL) and entrance to major plant buildings is at or above this elevation. In addition, the plant site is generally sloped away from the major plant buildings and toward the ocean or Diablo Creek. (PGE, 2012b) The DCPP site location map is presented as Figure 2-1.

Topography and plant site arrangement limit flood design considerations to local floods from Diablo Creek and sea wave action from the Pacific Ocean (PGE, 2013).

2.3 Current Licensing Basis

2.3.1 Flood-Related and Flood Protection Changes to the Licensing Basis Since License Issuance

Since the issuance of the original licenses for Units 1 and 2, a modification of the auxiliary saltwater (ASW) system was completed to install bypass piping (late 1990s). The bypass piping installation was done due to a concern that localized corrosion was occurring in the portion of the ASW piping buried in the tidal zone outside the intake structure. The modifications involved bypassing approximately 800 ft. of Unit 1 and 200 ft. of Unit 2 safety-related piping. Storm and tsunami protective measures for the bypass piping consisted of gabion mattresses, reinforced concrete pavement above buried ASW system piping, and an armored embankment southeast of the intake structure, which were designed and installed to resist the effects of tsunami and storm waves. NRC approved these flooding modifications in Amendments 131 and 129 to the Units 1 and 2 licenses, respectively (NRC, 1999).

2.3.2 Current Licensing Basis for Flooding Hazards

The following describes the flood causing mechanisms and their associated water surface elevations and effects that were considered for the DCPP current licensing basis (CLB).

2.3.2.1 CLB - Local Intense Precipitation (LIP)

As discussed in Updated Final Safety Analysis Report (UFSAR) Section 2.4.10 (PGE, 2013), roofs of safety-related buildings have a drainage system designed in accordance with the Uniform Plumbing Code for an adjusted regional probable maximum precipitation (PMP) of 4 inches per hour. In addition, overflow scuppers are provided in parapet walls at roof level to prevent ponding of accumulated rainwater in excess of drain capacity. Yard areas around safety-related buildings are graded to provide positive slope away from buildings. Storm runoff is overland and unobstructed. Therefore, based on the design in accordance with the CLB, it is not possible for ponding from local PMP to flood safety-related buildings.

2.3.2.2 CLB - Riverine (Rivers and Streams) Flooding

As discussed in UFSAR Section 2.4.3 (PGE, 2013), the only stream on the site subject to a probable maximum flood (PMF) is Diablo Creek. Diablo Creek collects runoff from a drainage area of 5.19 square miles (sq-mi). The PMF was obtained by deriving an estimated PMP with a duration of 24 hours over the subject drainage area. The DCPP PMP for a 24-hour duration was determined to be 16.6 inches.

The PMF study assumed the most severe antecedent condition of ground wetness favorable to high flood runoff and that during a PMF, all culverts are plugged, and water is impounded to the crest of the lowest depression of the switchyard's fill, which is along the border of Diablo Creek. The study determined that the PMF could not affect the plant. For a drainage area of 5.19 square miles, the PMF was found to have a peak discharge of 6,878 cubic feet per second (cfs) (1,325 cfs/sq-mi) or a total volume of about 4,306 acre-feet for the 24-hour storm.

As discussed in UFSAR Section 2.4.2.2.1 (PGE, 2013), the canyon confining Diablo Creek remains intact and will pass floods without hazard to safety-related equipment. In addition, channel blockage from landslides downstream of the plant, sufficient to flood the plant yard, is not possible because of the topographic arrangement of the site.

As discussed in UFSAR Section 3.4.1 (PGE, 2013) and Supplemental Safety Evaluation Report (SSER) 1 (NRC, 1975), Diablo Creek is adequate to handle the PMF. Estimated maximum water surface elevation during a PMF at a point nearest the plant was approximately 6 ft. below plant grade for the worst case. Thus, the depth of water at the plant location for the PMF is zero.

2.3.2.3 CLB – Dam Breaches and Failure Flooding

There are no dams in the watershed and failure of dams outside the watershed could not generate waves higher than those discussed in Sections 2.3.2.4, 2.3.2.5, and 2.3.2.6. As described in UFSAR Section 9.2.3.3, there are two raw water storage reservoirs (RWSRs) located onsite, each of 2.5 million gallons storage capacity, that were excavated in a stable rock terrace east of and above the power block at approximately 312.9 ft. NAVD88 (310 ft. MSL). A portion of the reservoir bench drains toward the power plant. As such, the reservoir was lowered by excavating the basin entirely in rock, eliminating the risk of flooding due to dike failure. The drainage capacity of Diablo Canyon is sufficient to pass the entire reservoir volume safely by the plant in less than 1 minute (PGE, 2013). As described in SSER 8 (NRC, 1978b), the reservoirs were qualified to demonstrate acceptable foundation materials during a seismic event.

2.3.2.4 CLB - Storm Surge

UFSAR Section 2.4.5.4 (PGE, 2013) indicates that wave action behavior at DCPP was originally developed based on a statistical evaluation of historical data. PG&E conducted an extensive review of the historical data that led to the estimation of the return periods of the critical storms. A major Pacific storm in January 1981 resulted in extensive damage to the west breakwater protecting the intake basin, and led to a review of all the design waves and water levels. As a result of the damage, PG&E undertook a test program to determine critical wave behavior at the intake basin, including wave height, wave direction, wave runup, resulting forces, and the effects of wave splash on the intake structure. A three-dimensional physical model of the basin and its surroundings was constructed representing the sea floor, the intake structure, and the breakwaters in storm-induced damage conditions.

The design basis storm flood consisted of the maximum credible wave event combined with high tide and sea level anomaly with postulated degradation of both breakwaters to 0.3 ft. NAVD88 (-2.6 ft. MSL). Waves for the scale model tests were mechanically generated. Wave heights, outside the breakwater, of up to 45 ft., with periods of 12, 16, and 20 seconds were generated. The results for the model testing indicated that the response waves within the intake basin reached a maximum height that did not increase further in response to increases in the offshore wave height. This phenomenon is due to the effects of the natural terrain and the presence of the degraded breakwater. Therefore, the maximum credible wave event is based on the maximum response of the wave height within the basin, in combination with the still water level in the basin, and was used for assessing the maximum inundating effects and wave forces at the intake structure.

Storm surge is also considered in the CLB in conjunction with tsunami, wind-generated waves, and tides, as discussed in Section 2.3.2.13.

2.3.2.5 CLB – Seiche

As discussed in UFSAR Section 2.4.5.3 (PGE, 2013), seiche is considered in the CLB as part of storm surge effects. Seiche is also considered in the qualification of the RWSRs. SSER 8 (NRC, 1978b) concludes that there would be no significant loss of water from the RWSRs due to seiche.

2.3.2.6 CLB - Tsunami Flooding

The DCPP tsunami evaluation and design have evolved as a result of a number of studies and analyses during the original plant design and licensing period, the operating license review period, and following the breakwater damage caused by a major Pacific storm in January 1981.

Wave heights for the two classes of tsunamis considered in the design of DCPP are described in the following sections. For a discussion of the combined effects of tsunami, wind-generated storm waves, storm surge ("piling up" of water near the shore due to a storm), and tides, refer to Section 2.3.2.13.

2.3.2.6.1 Distantly-Generated Tsunamis

As discussed in UFSAR Section 2.4.6.1.1 (PGE, 2013), the predominant sources of distantly-generated tsunamis are limited to areas of earthquake and volcanic activity on the circum-Pacific belt. Distant sources relative to DCPP include the Aleutian area, the Kuril-Kamchatka region, and the South American coast.

Because of the lack of historical data for the site during the construction permit review, in 1967, the Atomic Energy Commission (AEC) staff and its consultants, the United States Coast and Geodetic Survey (USCGS), agreed that the probable maximum tsunami at the site, that had virtually no risk of being exceeded, would be less than the 17 to 20 foot waves experienced at Crescent City, California, as a result of the 1964 Anchorage, Alaska earthquake. PG&E conservatively decided to use 20 ft. as the maximum distantly-generated tsunami wave height.

2.3.2.6.2 Near-Shore Tsunamis

As discussed in UFSAR Sections 2.4.6.1.2 and 2.4.6.4 (PGE, 2013), a number of investigations and analyses to determine the tsunami-generation potential of near-shore earthquake faults were performed during the period from 1966 to 1975. The design basis tsunami wave heights are based on the analysis performed in 1975 (Hwang, 1975), which considered seismic activity and submarine landslides. The following earthquake sources and characteristics were considered in the analysis:

- Santa Lucia Bank fault, located approximately 29 miles from the site, considering a resultant displacement of 9.8 ft. and a vertical displacement (6.6 ft.) equal to two-thirds of the resultant displacement
- Santa Maria Basin fault (later identified as the Hosgri fault), located approximately 3.5 miles from the site, considering a resultant displacement of 11 ft. and a vertical displacement (7.3 ft.) equal to two-thirds of the resultant displacement

The analysis considered the cases of the breakwaters (a) present as originally constructed, (b) completely absent, and (c) in damaged conditions, in which the sides of the breakwaters slump to a 1-on-4, 1-on-5, or 1-on-6 vertical-to-horizontal slope.

The Santa Maria Basin fault source controls, producing a maximum runup of 9.2 ft. and a maximum drawdown of 0.0 ft.

2.3.2.7 CLB - Ice Induced Flooding

The mild climate and general lack of freezing temperatures in this region make regional ice formation highly unlikely. Therefore, it was not considered. (PGE, 2013)

2.3.2.8 CLB - Channel Migration or Diversion

Upstream diversions associated with rivers, where low flow has an impact on dependable cooling water sources, is not a factor for the DCPP site. (PGE, 2013)

2.3.2.9 CLB - Wind-Generated Waves

Wind-generated waves are considered in the CLB in conjunction with tsunami, storm surge, and tides, as discussed in Section 2.3.2.13.

2.3.2.10 CLB - Hydrodynamic Loads

The structural design of the intake structure, including the ASW compartment ventilation shafts and their extensions, required the determination of the forces acting on the submerged portions of these structures due to the design water level. These forces include both hydrostatic loading and hydrodynamic loading due to the wave action.

To provide input for the determination of the loading on structural elements, a 3-D physical model study (testing program) was completed, which included monitoring of loading on the model of the intake structure and ventilation shafts including extensions by the use of load cells, bending moment gages, and pressure transducers attached to the model. The test results yielded the combined wave runup (34.9 ft. NAVD88; 32.0 ft. MSL) and other design loads for the following components:

- Vent shafts (concrete huts)
- Vent shaft extensions (coaxial steel pipes)
- Top deck of intake structure
- Curtain wall of intake structure (concrete wall forming ocean side face of structure)
- ASW pump forebay ceiling (underside of slab supporting the ASW pump)

Numerical values of design basis wave pressures reflecting the model studies are as follows:

- Vent Shafts (Huts): SSER 17 (NRC, 1984) evaluated wave-induced forces on the ASW ventilation huts. The maximum force that could be generated by wave action on the concrete ventilating structure was determined by hydrodynamic testing to be 300 kips. The ASW ventilation structure was evaluated for a maximum force of 780 kips applied at the top of the concrete ventilation structure. This yielded the following pressure profiles:
    - On the seaward (West) face, pressure varies linearly from 2.5 ksf on the top (Elevation 34.9 ft. NAVD88 [32 ft. MSL]) to 1.6 ksf at the bottom (Elevation 20.4 ft. NAVD88 [17.5 ft. MSL]).
    - On the side (North) face, pressure varies linearly from 0.5 kips per square-foot (ksf) on the top to 3.0 ksf at the bottom.
    - Seaward and side pressures are not concurrent.

- Vent Extension (Snorkels): While splashing of water may occur, the wave forces on the vent extension are negligible.

- Top Deck - Elevation 20.4 ft. NAVD88 (17.5 ft. MSL): Uniform water pressure of 0.93 ksf is applied over the top of the entire deck.

- Curtain Wall: On the seaward face of the seaward curtain wall, the center panel is rated for a uniform dynamic pressure of 2.4 ksf. The side panels of the curtain wall have an applied pressure of 1.54 ksf based on observations during the model testing.

- ASW Pump Forebay Ceilings: The uniform upward pressure on the bottom of the slab over the ASW pump forebay is 6.2 ksf.

2.3.2.11 CLB - Waterborne Projectiles and Debris Loads

SSER 17 (NRC, 1984) evaluated degradation of the breakwater due to a severe storm combined with the probability of a large vessel (greater than 250 tons) crossing the degraded breakwater and impacting the intake structure. SSER 17 concluded that the probability of this occurrence was acceptably low (storm-independent case was 6.7 E-6 events per year). SSER 17 also concluded that, with respect to the safety-related function of the ASW pumps, the impact of vessels displacing less than 250 tons on the intake structure would be inconsequential.

Based on hydrodynamic testing, the steel outer coaxial 40-inch ASW ventilation snorkel pipe was not loaded by the wave forces. Thus, only wind forces and tornado missiles needed to be considered in the evaluation of the snorkel pipe. The controlling 4000-lb automobile tornado missile load is shown to be bounding in the evaluation of the snorkel pipe.

2.3.2.12 CLB - Debris and Sedimentation

As discussed in UFSAR Sections 9.2.7.3 and 10.4.5.2 (PGE, 2013), a curtain wall at the front of the intake structure limits the amount of floating debris entering the intake structure. Bar racks near the front of the intake structure intercept large submerged debris. The bar racks have 3/8-inch thick bars at 3-3/8 inch centers. Traveling screens intercept all material larger than the screen mesh opening (3/8 inch clear square openings). If the traveling screen for a unit becomes clogged with debris, seawater may be supplied to the ASW pump bays from the unit's circulating water pump bays. There are no specific debris considerations in the tsunami licensing basis.

There is no mention of sedimentation control in the CLB documents. The tsunami licensing basis does not address sedimentation.

2.3.2.13 CLB - Combined Events

As discussed in UFSAR Section 2.4.2.2.2 (PGE, 2013), the licensing basis includes the combined effects of a tsunami, wind-generated storm waves, storm surge ("piling up" of water

As discussed in UFSAR Section 2.4.5.1 (PGE, 2013), hurricanes or line squalls of sufficient magnitude to generate surge flooding (storm-generated long-period sea waves) have not been recorded on the Pacific coastline. This lack of observed events in 200 years of record provides reasonable assurance that such an event will not occur during the lifetime of DCPP. However, the effects of wind-generated storm waves, storm surge, and tides are conservatively considered in the evaluation of water level and its effects on safety-related equipment and structures.

As described in Section 2.3.2.4, UFSAR Section 2.4.5.4 (PGE, 2013) indicates that wave action behavior at DCPP was originally developed based on a statistical evaluation of historical data. PG&E conducted an extensive review of the historical data that led to the estimation of the return periods of the critical storms. A major Pacific storm in January 1981 resulted in extensive damage to the west breakwater protecting the intake basin, and led to a review of all the design waves and water levels. As a result of the damage, PG&E undertook a test program to determine critical wave behavior at the intake basin, including wave height, wave direction, wave runup, resulting forces, and the effects of wave splash on the intake structure. A three-dimensional physical model of the basin and its surroundings was constructed representing the sea floor, the intake structure, and the breakwaters in storm-induced damage conditions. The tests included the effects of degraded breakwaters concurrent with: (a) wind-generated storm waves, including storm surge and tides, and (b) the effects of tsunami plus storm waves.

As discussed in UFSAR Section 2.4.6.1.2 (PGE, 2013), the combined wave runup for distantly-generated tsunamis is 30.3 ft. NAVD88 (27.4 ft. MSL) and the combined wave runup for near-shore tsunamis is 34.9 ft. NAVD88 (32.0 ft. MSL). As discussed in UFSAR Section 2.4.6.1.5 (PGE, 2013), the combined wave drawdown for distantly-generated tsunamis is -8.7 ft. NAVD88 (-11.6 ft. MSL) and the combined wave drawdown for near-shore tsunamis is -3.8 ft. NAVD88 (-6.7 ft. MSL).

The ASW pumps are designed to operate with a water level of -17.1 ft. NAVD88 (-20 ft. MSL). This is well below the minimum water level of -8.7 ft. NAVD88 (-11.6 ft. MSL) that results from the combined wave drawdown for distantly-generated tsunamis. In the event of a tsunami drawdown, the ASW pumps are capable of performing their safety function for this temporary condition (NRC, 1976b).

As discussed in UFSAR Section 2.4.6.6 (PGE, 2013), the potential effects of splash and spray of the sea waves on safety-related equipment were also evaluated. Splashing of water up to and above the top of the ventilation shaft (52.3 ft. NAVD88 [49.4 ft. MSL]) for the ASW pump rooms was observed during the performance of the scale model testing. The testing demonstrated that the ventilation shaft extensions remained free of the upward splashed water as they are set back from the seaward edge of the concrete vent huts at a considerable distance from the seaward edge of the intake structure, and the openings face away from the sea. Although the air intake would not be inundated by splashing of water, it could be subject to windborne spray. This spray could potentially wet the vent openings and water could enter the ASW pump rooms.

sea. Although the air intake would not be inundated by splashing of water, it could be subject to windborne spray. This spray could potentially wet the vent openings and water could enter the ASW pump rooms.

Using the model of the intake structure and intake basin, testing was performed to determine the potential for ingestion of spray water by the ASW pump room ventilation shafts. The conclusion of the study, and that of the NRC's evaluation (NRC, 1984), was that the combination of degraded breakwater, tsunami, high tide, severe storm, and extreme winds in the offshore direction necessary to result in enough water to render the ASW pumps inoperable was inconceivable.

2.3.2.14 CLB - Low-Water Considerations

As discussed in UFSAR Section 2.4.11 (PGE, 2013), there are no rivers or streams involved in plant operations; therefore, low flow conditions were not evaluated. Low water, as a result of tsunami drawdown occurring coincident with low tide and short-period storm waves, was projected to result in a possible low water elevation of -8.7 ft. NAVD88 (-11.6 ft. MSL) (MAI, 1966).

2.3.3 Current Flood Protection Features and Protected Equipment

The following flood protection features are included in the DCPP CLB as summarized in the UFSAR.

<u>ASW Watertight Pump Rooms</u>

As discussed in UFSAR Section 2.4.5.7 (PGE, 2013), the only safety-related system that has components within the projected tsunami and storm wave zone is the ASW system. Each ASW pump motor is housed in its own watertight room within the intake structure. These rooms are designed for a combination tsunami-storm wave activity to elevation 48.3 ft. NAVD88 (45.4 ft. MSL). As discussed in UFSAR Section 9.3.3.1 (PGE, 2013), the floor drainage system in the ASW vaults is designed with consideration of the potential for back flow. As a result, a design feature of the floor drain system for each of the ASW pump rooms includes a backflow check valve to maintain the pump rooms dry.

<u>ASW Buried Piping</u>

As discussed in Section 2.3.1, the ASW pumps, the buried ASW piping outside of the intake structure is vulnerable to the effects of tsunami and storm waves. Erosion protection consisting of gabion mattresses, reinforced concrete slabs, and pavement above this buried piping, and an armored embankment southeast of the intake structure are installed to resist the effects of tsunami and storm waves.

Tsunami Warning Response Procedure

As discussed in UFSAR Section 9.2.7.5 (PGE, 2013), the watertight doors of the ASW pump rooms are alarmed and indicated in the control room. Procedurally, activities at the intake which involve opening an ASW pump room door require posting a person to close the door. In addition, there is a tsunami warning procedure which requires closure of the ASW pump room doors if they are open, and the removal of all personnel from the intake structure area.

Breakwater System

DCPP has two breakwaters at the intake cove that provide protection to the intake structure from waves. They are constructed of precast concrete interlocking tri-bars with a reinforced concrete cap slab.

Diesel Fuel Oil (DFO) System

The DFO system contains two buried DFO storage tanks and a DFO transfer system, which consists of pumps and piping in underground vaults and trenches. The design considerations to prevent water from flooding or groundwater from entering the DFO storage tanks, concrete rooms, and pipe trenches are discussed below.

Based on a discussion in UFSAR Section 2.4 (PGE, 2013), the risk of surface water flooding at this site is essentially zero. No groundwater has been encountered at or below the buried tanks, pump rooms, or pipe trenches. Therefore, the source potential for groundwater flooding the fuel oil system that could affect the functionality is negligible.

DFO Storage Tanks

The below-ground storage tanks are completely sealed with the vent line extending approximately 2 ft. above ground. The tanks' access hatch covers are elevated approximately 6 inches above local grade to prevent water intrusion.

DFO Transfer System

The two DFO transfer pumps that transfer diesel fuel from the main storage tanks to the individual diesel engine day tanks are in separate, underground, reinforced concrete vaults with solid covers protected from surface runoff due to their location inside the west buttress and condensate polishing system structure. The vault's manway hatch covers are provided with approximately 6 inches of concrete curbing to prevent water intrusion into the vaults. These vaults are drained to the turbine building sump and are protected with backwater check valves.

The two DFO supply headers are in separate, below-ground reinforced concrete pipe trenches with covers that are generally flush with the adjacent ground level. Because the trenches collect water from surface runoff, drainage is provided through floor drains to manholes, which are pumped to the turbine building sumps or standpipes that can be connected to portable pumps. Thus, the piping within the trenches is unaffected by a flood or filling with water. Flood barriers exist between the trenches, vaults, and emergency diesel generators.

Roof Drains and Yard Area Slope

The DCPP roof drain systems are designed to handle a maximum rate of 4 inches of rain per hour, which exceeds the current licensing basis PMP rate for the site. Yard areas around safety-related buildings are graded to provide positive slope away from buildings. Storm runoff is overland and unobstructed. The current licensing basis indicated that it is, therefore, not possible for ponding to flood safety-related buildings.

2.3.4 Flooding Walkdown Summary

PG&E has submitted a Flooding Walkdown Report in response to the Section 50.54(f) information request regarding NTTF Recommendation 2.3: Flooding for DCPP (PGE, 2012b). The walkdowns were performed in accordance with NEI 12-07, "Guidelines for Performing Verification of Plant Flood Protection Features," dated May 2012 (NEI, 2012), which was endorsed by the NRC on May 31, 2012 (NRC, 2012b). No operability issues were identified. There are no planned flood protection enhancements or flood mitigation measures at DCPP resulting from the flood protection walkdowns. The NRC Staff evaluated the DCPP Flooding Walkdown Report in its Staff Assessment (NRC, 2014).

2.4 Hydrosphere

The hydrologic characteristics of the site are influenced by the Pacific Ocean on the west and by local storm runoff collected from the approximately 5 square mile oval area drained by Diablo Creek (see Figure 2-2). The maximum and minimum flows in Diablo Creek are highly variable. Average flows tend to be nearer the minimum flow value of 0.44 cfs. Maximum flows reflect short-term conditions associated with storm events. Usually within 1 or 2 days following a storm, flows return to normal. Flows during the wet season (October-April) vary daily and monthly. Dry season flows are sustained by groundwater seepage and are more consistent from day to day, tapering off over time. There is no other creek or river within the site area.

3.0 FLOODING HAZARDS REEVALUATION

The following sections discuss the flood causing mechanisms and the associated water surface elevations that were considered in the DCPP flooding hazards reevaluation.

3.1 Local Intense Precipitation

As discussed in Section 3.2.1, PMP was calculated in accordance with the NUREG/CR-7046 (NRC, 2011). The PMP was then used as input into a LIP and PMF reevaluation for the entire DCPP site. To improve the accuracy of the LIP at the power block and surrounding structures, a site-specific PMP (SPMP) was also calculated. The methodology for determining the site-specific PMP, calculating LIP with the site-specific PMP, and the associated results, are presented in this section.

3.1.1 Site-Specific PMP Determination

SPMP has been computed at the location of the DCPP site for use only as input to the LIP calculation as it affects safety-related SSCs. The analysis followed the storm-based approach used in the most recent Hydrometerological Reports (HMRs) (e.g., HMR 59) and the World Meteorological Organization (WMO) Manual for PMP determination (WMO, 1986; WMO, 2009).

The storm-based approach utilizes actual data from rainfall events which have occurred over the site and in regions where historic storms are transpositionable to the DCPP location. These rainfall data are maximized in-place following standard maximization procedures, then transpositioned to the DCPP location. The transpositioning process accounts for differences in moisture and elevation between the original storm location and the DCPP site. The process produces a total adjustment factor that is applied to the original rainfall data for each storm. The result represents the maximum rainfall each storm could have produced at the DCPP site had all meteorological factors leading to the rainfall occurred in an ideal and physically possible combination.

Several improvements and advancements compared to HMRs 58 (NOAA, 1998) and 59 (NOAA, 1999) were used in this calculation. The process of explicitly considering only storms that are considered transpositionable to the DCPP site is a significant improvement from methodology employed in HMR 59. HMR 59 covered the entire state of California, with no specific consideration applied to any particular basin. Therefore, regional smoothing of data was necessarily employed and storms were implicitly transpositioned well beyond appropriate limits. This produced PMP values which did not accurately represent the explicit characteristics of a given location. In addition, the storm database used in this study was updated with nearly 20 years of data not available during the development of HMR 59. This resulted in a more comprehensive data set from which LIP values were developed. Finally, more accurate storm maximizations were completed using updated climatologies and moisture inflow identification processes than were available in HMR 59.

For storm maximization calculations, a specific latitude and longitude with elevation is required. Table 3-1 displays the data used in these calculations.

A storm search was conducted which included all rainfall reports of 1-hour, 6-hours, and 24-hours which exceeded the 100-year recurrence interval for each station location. The search encompassed a region where storms that occurred were considered transpositionable to the DCPP site. Several criteria were used to further refine the storm list, including quality controlling the rainfall observation, comparing the magnitudes of individual reports to other reports in the same area on the same date, and comparing the magnitude of a validated report for a given duration to the largest amounts at that location for the same duration. Results of this initial analysis produced a list of storms which all accumulated more than 1.95 inches of rainfall in a 1-hour period. This value was used as the cutoff because all storms needed to be within at least 40 percent of the largest value. Storms smaller than this level would not be able to produce the LIP values after all maximizations and adjustments were applied, and therefore, were not included for further evaluation. This resulted in 14 storm events being evaluated and used for LIP development (Table 3-2, Figure 3-1).

The 14 storms on the final storm list used to calculate the LIP values consisted of the largest hourly rainfall report for each storm analyzed. Each storm event was then maximized in-place to produce a scenario representing how much larger the rainfall could have been had all atmospheric processes been combined in ideal conditions. Finally, the resulting largest 1-hour accumulation was distributed temporally using guidance provided in HMR 58 (NOAA, 1998).

Storm calculations yielded the LIP values for sub-hourly and hours one to six, as shown in Table 3-3.

3.1.2 LIP Methodology

The LIP reevaluation was conducted to determine the water surface elevation (WSE) associated with the effects of site-specific LIP inside the protected area at the two main elevations where the power block and other safety-related structures and commodities are located: 117.9 ft. NAVD88 (115 ft. MSL) and 87.9 ft. NAVD88 (85 ft. MSL). Additionally, the calculation verified that the site-specific LIP, determined using a hydrometeorological procedure, is within the range of National Research Council suggested return period of 10^5 and 10^9 years (NRC, 1994). Then the calculation determined the maximum WSE, the duration of flooding, and associated hydrodynamic loading near the potential water entry points to the turbine building, auxiliary building and fuel handling building. The calculation evaluated maximum WSE and duration for the auxiliary building, the northwest and southwest corners of the turbine building (where the safety-related diesel generator room air inlet louvers are located), and areas proximate to the north and south buttress buildings where important safety-related components for the diesel generator fuel oil transfer system are located.

The methodology used in the reevaluation is consistent with the following standards and guidance documents:

- NRC, "Standard Review Plan for the Review of Safety Analysis Reports for Nuclear Power Plants", NUREG-0800, March 2007 (NRC, 2007);
- NRC, "Design-Basis Flood Estimation for Site Characterization at Nuclear Power Plants in the United States of America", NUREG/CR-7046, November 2011 (NRC, 2011); and
- ANS, "American National Standard for Determining Design Basis Flooding at Power Reactor Sites:, (ANSI/ANS 2.8) 1992 (ANSI/ANS, 1992)

NUREG/CR-7046 introduces use of the Hierarchical Hazard Assessment (HHA) Approach. The HHA is a series of progressively refined methods that increasingly uses site-specific data to demonstrate whether safety-related SSCs are adequately protected from adverse flooding effects.

The hydraulic analysis of the runoff was completed using FLO-2D PRO computer software, two-dimensional (2D) hydrodynamic modeling software (FLO-2D, 2014). The reevaluation assumed that all the drainage system components (e.g., gravity storm drain systems, culverts, inlets) are non-functional or completely blocked during the LIP event. This approach is consistent with the HHA approach described in NUREG/CR-7046. All the precipitation falling on the buildings was assumed to discharge onto the ground and contribute to ground surface runoff.

As discussed in Section 3.1.1, the LIP values at the DCPP power block calculated for various durations are shown in Table 3-3. The cumulative depth of the 1-hour and 6-hour LIP was 4.5 and 5.9 inches, respectively. The highest 15-minute incremental site-specific LIP was 2.5 inches. This data was utilized in the LIP water surface calculation. The study developed five different temporal distributions of 15-minute incremental local storm PMP values to determine the critical temporal distribution that produces the maximum water surface elevation (Table 3-4).

3.1.3 LIP Results

Based on frequency analysis, the 90 percent confidence interval for 10^6 years return period varied between 3.62 and 2.86 inches (Table 3-7). The calculated 1-hour maximum LIP is 4.5 inches and has a return period of approximately 1.29×10^8 years. The LIP is within the recommended range of the National Research Council suggested return period of 10^5 and 10^9 years (NRC, 1994). Therefore, the LIP storm event, determined using the hydrometeorological procedure, is conservative because it falls within the National Research Council range and can be used to evaluate flooding at the DCPP site.

The WSE and water depths above thresholds resulting from the LIP flooding event at the power block area, safety and non safety-related structures, doors, and areas to the west of the turbine and buttress buildings are presented in Table 3-5 (locations are shown in Figures 3-2 and water depths in Figure 3-3). Based on the results, the water depth above the door thresholds and areas to the west of the turbine and buttress buildings varied between 0.09 ft. and 1.4 ft., with five of the doors/areas showing no inundation. The duration of time dependent water depths varied between 0.00 hours and 4.41 hours. For the doors and areas that experienced inundation, mitigation is required to prevent adverse impact from LIP at DCPP. See Section 5.1 for mitigation actions.

Area A3 (Figures 3-2 and 3-3) includes commodities related to the safety-related DFO transfer system. Area A3 shows a water depth value of 0.13 ft. (Table 3-5, Figure 3-3). The safety-related fuel oil transfer commodities are elevated 6 inches above grade, and therefore, do not experience any flooding.

Debris flow is not a concern for water flow depth less than 1 foot and velocity less than 1 ft/sec (Prochaska et al, 2008). Maximum velocity values are less than 1 ft/sec for all but two commodities listed on Table 3-5. Area B1 shows a maximum velocity of 1.16 ft/sec, however, this area is proximate to the Unit 2 diesel generator air intake louvers, which are recessed and located behind a security fence. Debris ingress to the louvers is not credible. Door 191-2 shows a maximum velocity of 1.02 ft/sec, sufficiently close to the 1 ft/sec threshold so as to safely assume that debris loading will not be an issue for this door.

The total associated effect is the force (per linear foot of surface) due to hydrostatic and hydrodynamic loading of the LIP and was generally small for all the doors and safety-related structures. It varies from 0.03 lb/ft. to 21.51 lb/ft. (Table 3-6). Forces due to associated LIP flood event effects will not adversely impact the doors or power block and surrounding structures.

3.2 Flooding in Streams and Rivers

3.2.1 Probable Maximum Precipitation

The PMP reevaluation was conducted using the current applicable guidance contained in HMRs 58 and 59 (NOAA, 1998; NOAA, 1999). PMP values calculated for input to the PMF and LIP are computed in accordance with the NUREG/CR-7046 (NRC, 2011) hierarchical hazard assessment approach. PMP calculations required as input to the PMF analysis include the 72-hour general storm PMP, 6-hour local storm PMP, 72-hour cool/snow season PMP, snowmelt parameters, and snowmelt rates.

The PMP reevaluation also requires calculation of the 100-year and probable maximum snowpack. Snowpack calculations were conducted using snow depth data at several NOAA Climate Stations near the DCPP (NOAA, 2013b; USACE, 1998).

The methodology used in the reevaluation is consistent with the following standards and guidance documents:

- NRC, "Standard Review Plan for the Review of Safety Analysis Reports for Nuclear Power Plants," NUREG-0800, March 2007 (NRC, 2007);
- NRC, "Design-Basis Flood Estimation for Site Characterization at Nuclear Power Plants in the United States of America," NUREG/CR-7046, November 2011 (NRC, 2011); and
- ANS, "American National Standard for Determining Design Basis Flooding at Power Reactor Sites," (ANSI/ANS 2.8) 1992 (ANSI/ANS, 1992)

PMF calculations require analysis of various flood causing precipitation events. The ANS guidance provided in the NUREG/CR-7046 (NRC, 2011) states that determination of design bases from flood hazards should include concurrent flood-causing mechanisms. To evaluate the highest flood water elevation associated with the PMF due to precipitation events, analysis of the following three alternatives combining various flood conditions were considered, in accordance with ANS recommendations:

Alternative 1 – Combination of

- Mean monthly base flow;
- Median soil moisture;
- Antecedent of subsequent rain: the lesser of (1) rainfall equal to 40 percent of the PMP and (2) a 500-year rainfall;
- PMP; and
- Waves induced by 2-year wind speed applied along the critical direction.

Alternative 2 – Combination of

- Mean monthly base flow;
- Probable maximum snowpack;
- 100-year snow season rainfall; and
- Waves induced by 2-year wind speed applied along the critical direction.

Alternative 3 – Combination of

- Mean monthly base flow;
- 100-year snowpack;
- Cool/Snow season PMP; and
- Waves induced by 2-year wind speed applied along the critical direction.

Procedures for calculating PMP values are provided in HMRs 58 and 59 (NOAA, 1998; NOAA, 1999). HMRs 58 and 59 apply to the state of California and provide procedures to compute all season general storm PMP, seasonal general storm PMP, and local storm PMP values.

For drainage areas less than 500 square miles, HMR 58 (NOAA, 1998) recommends that both the general and local storm PMP be calculated to determine the governing PMP. The all season general and local storm PMP were computed for the Diablo Creek watershed to develop Alternative 1 PMP for input to the PMF calculation. PMP Alternative 2 includes a combination of 100-year rainfall and snowmelt with probable maximum snowpack. Seasonal general storm PMP calculations were conducted to determine the cool/snow season PMP needed to develop Alternative 3 PMP also for input to the PMF calculation.

3.2.1.1 100-Year and 500-Year Rainfall

100-year and 500-year point precipitation depth-duration values are determined using the NOAA Atlas 14 (NOAA, 2012) website. Output obtained from the NOAA Atlas 14 website included annual exceedance precipitation frequency estimates for the 100- and 500-year storm for durations ranging from 5-minutes up to 60-days. Point precipitation frequency estimates were determined at the centroid of the Diablo Creek basin. The NOAA Atlas 14 data also provides 90 percent confidence bounds around the precipitation estimates. The upper 90 percent confidence bound rainfall for the 100- and 500-year events were applied to Alternative 1 and 2 computations.

3.2.1.2 100-Year and Probable Maximum Snowpack

The 100-year snowpack estimates for the Diablo Creek watershed were determined by conducting an assessment of historical observed snow depth data available from the NOAA interactive snow information website (NOAA, 2013a). Daily snow depth data at NOAA Climate Stations similar in elevation and near the Diablo Creek basin were analyzed in the absence of a station located within the Diablo Creek watershed. Historic snow depth data at four NOAA Climate Stations were evaluated. Typically, a statistical analysis of historic data would be used to estimate the 100-year snowpack. Due to an extremely low number of days (one day) in the observed data set, a statistical analysis to determine 100-year snowpack was not possible. A conservative estimate of the 100-year snowpack was made based upon available data.

3.2.1.3 PMP Results

The Diablo Creek watershed delineation, computed drainage basin area, and the location of the centroid are shown graphically in Figure 2-2. The computed drainage basin area is 5.197 square miles.

The 1-, 6-, 12-, 18-, 24-, 48-, and 72-hour general storm all-season cumulative PMP values are shown in Figure 3-4, and are fit with a smooth curve to interpolate 15-minute incremental values.

All-season general storm PMP values for a 72-hour storm in 15-minute increments and five temporal distributions are provided in Table 3-8. The maximum computed PMP value is 30.90 inches.

Local storm PMP values were calculated for a 6-hour storm computed in 15-minute increments in five temporal distributions for the Diablo Creek watershed. The Diablo Creek watershed local storm PMP was developed as part of PMP Alternative 1. The all-season general storm and local storm PMP values computed for the Diablo Creek watershed were used as input to Diablo Creek PMF Analysis.

Precipitation computations developed for PMP Alternative 2, combining 100-year rainfall and snowmelt with probable maximum snowpack for a 72-hour storm in 15-minute increments and five temporal distributions, were applied as input to the Diablo Creek PMF Analysis. The maximum computed PMP value is 20.55 inches.

Computed precipitation values including a combination cool/snow season PMP and snowmelt with 100-year snowpack for a 72-hour storm in 15-minute increments and five temporal distributions representing PMP Alternative 3 were applied as input to the Diablo Creek PMF Analysis. The maximum computed PMP value is 32.09 inches.

3.2.2 Probable Maximum Flooding

Hydrologic and hydraulic models were developed to calculate the PMF for Diablo Creek. Hydrologic modeling of the Diablo Creek watershed was conducted using the USACE Hydrologic Engineering Center – Hydrologic Modeling System (HEC-HMS) Version 3.5 computer software (USACE, 2010a). Hydrologic modeling was developed to estimate a PMF runoff hydrograph at the DCPP site resulting from the PMP. The critical PMF runoff hydrograph was then used as input to hydraulic modeling. Hydraulic modeling, using the USACE Hydrologic Engineering Center – River Analysis System (HECRAS) Version 4.1.0 computer software (USACE, 2010b), was used to compute PMF water surface elevations along Diablo Creek through the DCPP site.

The methodology used in the calculations is consistent with the following standards and guidance documents:

- NRC, "Standard Review Plan for the Review of Safety Analysis Reports for Nuclear Power Plants," NUREG-0800, March 2007 (NRC, 2007);
- NRC, "Design-Basis Flood Estimation for Site Characterization at Nuclear Power Plants in the United States of America," NUREG/CR-7046, November 2011 (NRC, 2011); and
- ANS, "American National Standard for Determining Design Basis Flooding at Power Reactor Sites," (ANSI/ANS 2.8) 1992 (ANSI/ANS, 1992)

3.2.2.1 Hydrologic Modeling

The HEC-HMS hydrologic model of the Diablo Creek watershed incorporates mean monthly base flow, PMP, and a synthetic unit hydrograph adjusted to account for nonlinear basin response to large flood events. The HEC-HMS modeling conservatively assumes no precipitation losses and instantaneous routing in channels. It should also be noted that the Diablo Creek watershed does not have any storage reservoirs or dams.

Mean monthly base flow for Diablo Creek was estimated using available United States Geological Survey (USGS) flow gage data (USGS, 2013a) at gages located in similar sized basins near the DCPP site.

Probable maximum precipitation input was obtained from results discussed in Section 3.2.1.3. A total of twenty probable maximum precipitation events representing three different precipitation alternatives were run separately in the HEC-HMS model to determine which alternative combination produces the largest runoff hydrograph. PMP data was calculated and input into HEC-HMS in 15-minute increments.

A region specific synthetic unit hydrograph adjusted to simulate a non-linear basin response was developed as input to the HEC-HMS model to estimate the rainfall to runoff transformation occurring in the Diablo Creek basin. The U.S. Bureau of Reclamation (USBR) synthetic unit hydrograph method for coast and cascade ranges of California, Oregon, and Washington provided in the USBR Flood Hydrology Manual (USBR, 1992) was used.

The derived synthetic unit hydrograph was then modified to account for a non-linear basin response to large flood events, per guidance provided in NUREG/CR-7046 (NRC, 2011). Modification to the derived unit hydrograph included increasing the peak discharge by one-fifth (20 percent) and decreasing the time to peak by one-third (33 percent). The rising and falling limbs of the modified synthetic unit hydrograph were then manually adjusted to maintain the unit ratio of runoff volume to unit depth over the drainage area.

3.2.2.2 Hydraulic Modeling

Probable maximum flooding in Diablo Creek at the DCPP site is due to runoff from the Diablo Creek watershed resulting from the PMP. There are no dams or storage reservoirs located within the watershed, and therefore dam breach analyses were not required for this PMF. Hydraulic analyses were conducted assuming that all pipes and culverts along Diablo Creek are 100 percent plugged. Attenuation of PMF flows in Diablo Creek was not considered. Flooding due to failure of embankment, pipes, or culverts on the DCPP site was not found to be a factor.

3.2.2.3 PMF Results

The critical PMF peak discharge resulting from PMP was determined to be 6,541 cfs. Results of the hydrologic and hydraulic analysis of Diablo Creek indicate that no safety-related SSCs are inundated by PMF. The 230kV switchyard (non safety-related) would be inundated during the PMF event, as shown in Figure 3-5. All other DCPP facilities and site features remain above calculated PMF water surface levels, including the intake structure and the entire DCPP power block, which consists of the fuel handling building, the auxiliary building, the turbine building, and the two unit containment buildings.

The results of the analysis summarized in the above paragraph, and the methodology of the analysis, satisfy the first two steps in the HHA Approach described in Chapter 2 of NUREG/CR-7046 (NRC, 2011), which states:

> *1. Identify flood-causing phenomena or mechanisms by reviewing historical data and assessing the geohydrological, geoseismic, and structural failure phenomenon in the vicinity of the site and region.*
>
> *2. For each flood-causing phenomenon, develop a conservative estimate of the flood from the corresponding probable maximum event using conservative simplifying assumptions.*

No safety-related SSCs are adversely affected by flood hazards. Therefore, Step 3 of the HHA Approach described in Chapter 2 of NUREG 7046 is not applicable to evaluation of impact to DCPP safety-related SSCs. Step 3 states:

> *3. If any Safety-Related SSC is adversely affected by flood hazards, use site-specific data to provide more realistic conditions in the flood analyses while ensuring that these*

conditions are consistent with those used by Federal agencies in similar design considerations. Repeat Step 2; if all Safety-Related SSCs are unaffected by the estimated flood, or if all site-specific data have been used, specify design bases for each using the most severe hazards from the set of floods corresponding to the flood-causing phenomena.

Although the 230kV switchyard is not a safety-related SSC, a sensitivity analysis that included the methodology outlined in Step 3 above was employed to determine if this SSC would be unaffected under more realistic conditions. The sensitivity analysis involved changing two of the calculation inputs to more realistic conditions. The first change was an adjustment of the Average Weighted Manning's n value (Kn) from 0.120 to 0.150, which by itself resulted in a 15 percent reduction in the PMF. The second change was an examination of reduced capacity, as opposed to total blockage, of the 10-ft. diameter culvert under the switchyards. Iterative evaluation of the HEC-RAS model determined that a 27 percent blockage of the culvert could occur prior to PMF resulting in inundation of the 230kV switchyard.

The sensitivity analysis demonstrates that utilizing a higher Kn value and crediting a more realistic partial functional capability of the culvert does not result in inundation of the non safety-related 230kV switchyard from the PMF. The sensitivity analysis represents an effort that comports with Case 2 of Appendix B of NUREG/CR-7046 – "Fully Functional Site Grading and Partially Blocked Drainage Channels."

3.2.2.4 PMF Coincident with Wind Wave Analysis

Calculations were completed to determine the coincident wind wave activity to be added to the WSE of the PMF at Diablo Creek. Topographic information was obtained for development of a fetch line profile. The wind speed and maximum wind wave height was determined using the recommendations outlined in ANSI/ANS-2.8 (ANSI/ANS, 1992). The guidelines provided in USACE Coastal Engineering Manual (CEM) (USACE, 2008) were used to perform wind speed adjustments, calculate the wind wave generation and run-up, and determine wind setup.

The PMF limit, determined in the PMF analysis (see Section 3.2.2.3), was used as a boundary for the fetch line, as shown in Figure 3-6. The critical location (worst case scenario) was determined to be where the wind wave generated along the fetch line would hit the retaining wall located along the southern PMF boundary, which creates the possibility of a wave run-up overtopping the retaining wall and progressing toward the turbine building. This is the critical location for the wind-related analysis at DCPP because the input parameters contributing to the wind wave height are the most conservative at this location. For any location along Diablo Creek, the input parameters to calculate the resulting wind wave are less conservative (i.e., the fetch line is shorter). Because the worst case scenario bounds any other possible scenarios, no additional investigation was required for other locations along Diablo Creek.

The maximum WSE of the PMF coincident with wind wave activity was calculated at 77.9 ft. NAVD88 (75.0 ft. MSL). The top of the retaining wall at the same location is approximately 86.3 ft. NAVD88 (83.4 ft. MSL), which is about 8.4 ft. above the maximum WSE of the PMF coincident with wind wave activity. Therefore, there is no hazard posed to the power block from the PMF coincident with wind wave activity at Diablo Creek.

3.3 Dam Breaches and Failures

As described in Section 2.3.2.3, DCPP is not affected by flooding from dam breaches or failures. Two raw water storage reservoirs located onsite, each of 2.5 million gallons storage capacity, were excavated in a stable rock terrace east of and above the power block at approximately 312.9 ft. NAVD88 (310 ft. MSL). A portion of the reservoir bench drains toward the power plant. As such, the reservoir was lowered by excavating the basin entirely in rock, eliminating the risk of flooding due to dike failure. The drainage capacity of Diablo Canyon is sufficient to pass the entire reservoir volume safely by the plant in less than 1 minute (PGE, 2013). As described in SSER 8 (NRC, 1978b), the reservoirs were qualified to demonstrate acceptable foundation materials during a seismic event.

3.4 Storm Surge

For the storm surge at DCPP, a computer-based numerical model was used to estimate the surge and wave effects to determine the probable maximum storm surge (PMSS) and seiche with wind-wave activity combined with the antecedent 10 percent exceedance high tide.

The methodology used in the calculations is consistent with the following standards and guidance documents:

- NRC, "Standard Review Plan for the Review of Safety Analysis Reports for Nuclear Power Plants," NUREG-0800, March 2007 (NRC, 2007);
- NRC, "Regulatory Guide 1.59 - Design Basis Floods for Nuclear Power Plants," Revision 2, August 1977 (NRC, 1977);
- NRC, "Design-Basis Flood Estimation for Site Characterization at Nuclear Power Plants in the United States of America," NUREG/CR-7046, November 2011 (NRC, 2011); and
- NRC "Guidance for Performing a Tsunami, Surge, and Seiche Flooding Safety Analysis Revision 0," Japan Lessons-Learned Project Directorate Interim Staff Guidance, JLD-ISG-2012-06, January 4, 2013 (NRC, 2013).

3.4.1 California Hurricanes and Storm Surges

Hurricanes are rare events in California. Historically, even if a hurricane travels towards California, by the time it reaches the California coast, the hurricane has spent considerable time in the colder Pacific waters, becoming a Tropical Depression or Storm. One such event occurred in 1997, when Tropical Storm Ignacio made its way towards California, making landfall on August 20, 1997. Maximum wave heights observed at the DCPP Waverider Buoy showed a significant wave height of 6.6 ft., with a period of approximately 4 seconds (Figure 3-7). This is not atypical of such events along the California Coast, and the significant wave height of 6.6 ft. is small compared to historical wind waves produced by the Pacific Ocean fetch. Larger wave events tend to not come from a southerly direction (based on observations at the DCPP buoy), but from the westerly direction.

Wave dynamics on the US West Coast are significantly different than those associated with those on the Gulf and East Coasts. The primary factors that result in the different (and significantly larger) West Coast wind wave heights are long prevailing winds that can occur across the Pacific Ocean fetch. Waves do not significantly lose energy once wind action has imparted energy to them (unless they run into winds blowing in the opposite direction), so waves produced by winds and strong storms across the very long Pacific Ocean fetch join together in wave trains that then impact the US West Coast, with large wave heights and long periods. The major wave heights associated with the California Central Coast are generally prevailing from the west-south-west to westerly directions.

For approximately 30 years, an increasing number of wave buoys have been collecting time-averaged wave height, period, and directional data (and in some cases much more detailed wave data) along the West Coast. Each of these data sets acts as an "integrator" of actual storm and wind wave events that cross the Pacific Ocean and land on coastal environs, and can be used to statistically predict the joint significant wave height and period for return periods of up to 10,000 years (Galiatsatou and Prinos, 2012). The usage of these data sets allow for a much higher confidence in the base wave height and periods that can be used as a source to predict impacts at the DCPP breakwater and intake area.

3.4.2 Return Period Estimates Based on Historical Records

For the PMSS at DCPP, a computer-based numerical model was used to estimate the surge and wave effects based on a statistical assessment of measured storm waves at the DCPP Waverider Buoy (National Data Buoy Center [NDBC] 46215) to determine a 200 year return period storm surge at the breakwater. Additionally, 200 year return periods were determined based on the historical data collected from four other NOAA Buoys found along the California Central Coast (Figure 3-8, NDBC 46028, NDBC 46011, NDBC 46023, and NDBC 46218).

Significant wave height, peak period, wave direction, as well as wind speed and wind direction were examined in detail using statistics in order to get a better understanding of the wave and wind dynamics found along the Central California Coast. All wave heights show a pronounced bimodal directionality based distribution, with the greatest wave heights coming from a westerly to northwesterly direction. This is especially true at the DCPP buoy (NDBC 46215).

Table 3-9 and Table 3-10 shows the largest maximum daily significant wave heights, wind speeds, and directions by date.

Figure 3-9 shows the 200 year return period for the five buoys. The estimated return period heights can be seen in Table 3-11, and these are used to calibrate the model as described in Section 3.4.4 below.

3.4.3 Numerical Storm Surge Model

Wave transformation was performed using Simulating WAves Nearshore (SWAN), part of DELFT3D WAVE/SWAN. DELFT3D WAVE/SWAN (Deltares, 2014) allows for accurate representation of the coastline near and surrounding the DCPP facility, and includes the ability to model wave processes, such as refraction, diffraction, generation, and dissipation. The maximum wave setup calculated with a detailed DELFT3D WAVE/SWAN model system was determined by comparing model storm surge results with wave heights statistically found from the NOAA NDBC Central Coast Buoy Network.

The physical features of the numerical model were created from regional and local bathymetry and topography. The model was calibrated to the calculated 200 year return period significant wave heights based on observations from the Central Coast NOAA NDBC Buoy network. The following antecedent water level conditions were included in the numerical model (Section 3.6.2):

High antecedent water level (HAWL): 7.0 ft. NAVD88

Alternate high antecedent water level (HHWL): 8.7 ft. NAVD88

In order to address storm surge (associated with low atmospheric pressures) the historically measured average and minimum pressures at NDBC Buoy 46028 were used to determine a surge antecedent water level (SAWL) of 9.9 ft. NAVD88.

The numerical model extended to the edge of the continental shelf, to the south of Point Arguello, and to the north of Monterey. The numerical model domain used a nested grid approach to account for the regional (deep ocean) and local characteristics (i.e., shallow coastal waters and the DCPP breakwater and inlet area).

Many of the default parameters for the DELFT3D WAVE/SWAN model were used without change. Table 3-12 shows boundary and physical inputs to the DELFT3D numerical model that are not default parameters.

3.4.4 Storm Surge Results

The maximum estimated wave height outside the breakwaters was 44.6 ft. (10.3 m). The maximum crest wave level inside the breakwaters was 12.8 ft. NAVD88 (9.9 ft. MSL). The maximum crest wave level at various locations in and around the DCPP intake cove are provided in Figure 3-10[1]. While seiche effects were noted in the intake cove, the wave heights were found to be less than 3.2 ft. of the maximum estimated wave height, and are therefore, not a concern.

3.5 Seiche

The onsite RWSRs were reevaluated for seiche during seismic loading. Using the Arbitrary Lagrangian-Eulerian methodology, a RWSR finite element model was developed and used in the analysis. The target spectra used to develop the time histories envelope the ground motion response spectra for the Hosgri Earthquake and Long Term Seismic Program earthquake. The

[1] In Figure 3-10, maximum crest wave levels are presented in meters.

reevaluation determined that the water sloshing height is approximately 2 ft. The maximum expected water volume loss from each of the RWSRs is 14,684 gallons.

3.6 Tsunami

In accordance with the guidelines presented in NUREG/CR-6966 (NRC, 2009), a HHA approach was used to evaluate the tsunami hazard. This approach uses a series of stepwise, progressively more refined analyses to evaluate the hazard. If the safety of the plant can be demonstrated by a simple and bounding analysis, more refined analyses do not need to be performed. Relative to tsunami hazards, the HHA approach consists of the following steps:

1. A Regional Screening Test involving an evaluation of the regional hazard based on a review of the historical record and the best available scientific data.
2. A Site Screening Test to compare the location and elevation of the plant site with the areas affected by tsunamis in the region. This screening test considers the local site characteristics of ground elevation (the plant grade relative to the water surface elevation) and the distance of the plant from the shoreline.

A detailed tsunami hazard assessment is performed if the screening tests do not conservatively establish the safety of the plant. A detailed, site-specific tsunami hazard assessment typically involves identification and modeling of applicable (near-field and far-field) tsunamigenic sources, numerical modeling of wave propagation from the tsunamigenic source to the near shore, and numerical inundation modeling of the plant site and vicinity.

The methodology used in the calculations is consistent with the following standards and guidance documents:

- NRC, "Standard Review Plan for the Review of Safety Analysis Reports for Nuclear Power Plants," NUREG-0800, March 2007 (NRC, 2007);
- NRC, "Regulatory Guide 1.102 - Flood Protection for Nuclear Power Plants," Revision 1, September 1976 (NRC, 1976a);
- NRC, "Regulatory Guide 1.59 - Design Basis Floods for Nuclear Power Plants," Revision 2, August 1977 (NRC, 1977);
- NRC, "Design-Basis Flood Estimation for Site Characterization at Nuclear Power Plants in the United States of America," NUREG/CR-7046, November 2011 (NRC, 2011); and
- NRC, "Tsunami Hazard Assessment at Nuclear Power Plant Sites in the Unites States of America," NUREG/CR-6966 (NRC, 2009).

3.6.1 Regional and Site Screening Evaluation

A regional screening and site screening evaluation was performed to identify the tsunamigenic sources with the potential to result in the Reevaluated Probable Maximum Tsunami (RPMT) at DCPP. Based on a review of (1) the historical records from Port San Luis (PSL) and Avila Beach (AB) tide gauges from all historical far-field coseismic tsunamis since 1946, (2) previous tsunami modeling studies, and (3) consideration of potential submarine mass failures (SMF), four far-field sources and five near field sources were selected as RPMT sources.

3.6.1.1 Far-Field Seismic Sources

Historical Records

Review of the historical records from the PSL/AB tide gauges indicates that, since 1946, 32 tsunamis were recorded near DCPP as a result of far-field earthquakes around the Pacific Ocean basin (Table 3-13). The maximum recorded wave amplitude (i.e., maximum surface elevation above mean sea level at the time of the event) was 6.6 ft. from the Tohoku 2011 Japan earthquake and tsunami. In addition to those far-field events, there were also large earthquakes events that occurred in the Pacific Ocean, but did not cause a measurable signal at the PSL/AB tide gauges.

To provide a fully comprehensive historical view of the impact of all significant far-field earthquakes on the DCPP site, data was extracted from the USGS data base (all seismic events larger than Mw 8 originating in the Pacific Rim (including interior seas such as Sea of Okhotsk (West Kamchatka), Celebes Sea (West Philippine), up to 140 degree in Longitude E, as a Western limit.) and compared them to those found to have caused a measurable surface elevation at the PSL/AB tide gauge, which are listed in Table 3-13.

Modeling Studies

Uslu (2008) conducted a comprehensive modeling study of far-field tsunamis in the Pacific Ocean Basin that could have a potential impact on the California Coast. This work showed that near the DCPP site, the potential far-field tsunami hazard is expected mostly from large sources occurring in the Aleutian Alaska Subduction Zone (AASZ), the Kamchatka Subduction Zone (KSZ), and Japan Subduction Zone (JSZ). Sources from the South American Subduction Zone (SASZ) and the Cascadia Subduction Zone (CSZ) will cause a smaller or negligible impact.

In 2013, in the wake of the catastrophic Tohoku 2011 tsunami, a far-field source was designed by a group of experts under the auspice of USGS in California, to represent the largest potential far-field tsunami hazard to impact central and southern California (Ross et al., 2013). This source, referred to as Semidi Subduction Zone (SSZ), represented a Mw 9.1 magnitude event (similar in magnitude and size to Tohoku 2011) in the Aleutian trench more precisely the Semidi Subduction Sector (SSS), which is part of the AASZ.

Submarine Mass Failure

PG&E evaluated the effect at DCPP from scenarios of dramatic SMFs and volcano collapses in the Hawaiian Islands. Despite the large size of the SMF slump, and significant magnitude of the earthquake, numerical runup models at the PSL/AB gauge indicate a relatively low potential impact at DCPP. There are larger impacts expected from other far and near-field events that bound the Hawaiian Island source. Thus, the effect at DCPP from scenarios of dramatic SMFs and volcano collapses in the Hawaiian Islands is not considered a RPMT source for DCPP.

Far-Field Selections

Based on the above research, the four far-field seismic sources (see Figure 3-11) that were modeled for DCPP were:

1. Aleutian Alaska Subduction Zone (AASZ)
2. Semidi Subduction Zone (SSZ)
3. Kamchatka Subduction Zone (KSZ)
4. Japan Subduction Zone (JSZ)

3.6.1.2 Near-Field Seismic Sources

Historical Records

Review indicates that, since 1946, only one near-field event resulted in a small observation at the PSL tide gauge – the April 1992 California/Humboldt earthquake. A survey of records from the NOAA NGDC database of events identified two additional events – an 1878 and a 1927 event (see Table 3-14).

Literature for the 1878 event based on eyewitness accounts found that a drawdown and tsunami of approximately one meter occurred, in the absence of a storm or earthquake. It is reasonable to attribute this event to submarine mass failure.

Contemporary literature for the 1927 event finds that this tsunami was likely a result of the Lompoc earthquake, however the contribution from a concomitant submarine mass failure has not been determined and remains unsettled.

Modeling Studies

Physiographic features in the DCPP area are shown in Figure 3-12. The figure shows the location of the continental shelf, including the upper-continental shelf, which consists of the Offshore Santa Maria Basin, Sur Basin, and the Santa Lucia Bank, and the lower continental slope, which includes the Santa Lucia basin and Santa Lucia Escarpment. The Santa Lucia Escarpment marks the boundary between the lower continental slope and the abyssal plain. The Santa Maria Basin is bounded to the east by the Hosgri fault zone and to the west by the Santa Lucia Bank fault. These fault systems are shown in Figure 3-13. The Hosgri fault zone is the southernmost component of a complex system of right-slip faults that run parallel to the Central California coastline, with a slip rate of 2 to 4 millimeters/year (mm/yr) (Johnson et al., 2012; PGE 2014). Hanson et al (2004) assumed rupture scenarios for the Hosgri fault zone as a convergent right-slip fault, with vertical displacements between 6.6 to 16.4 ft. Recent mapping of the Hosgri fault (Johnson et al., 2012) indicated a continuous fault zone, from Point Sal to Piedras Blancas, while previously the Hosgri fault was assumed to terminate at Point Estero. Consequently, Johnson et al (2012) recommend a minimum rupture length of 110 kilometers (km) for any seismic or tsunami assessment. The Santa Lucia Bank fault zone separates the offshore Santa Maria basin from the Santa Lucia high, a structurally uplifted block of Cretaceous rock.

Owing to their proximity to the DCPP site and the largest observed near-field event, the 1927 Lompoc earthquake (Mw 7-7.3) that occurred in between the two faults systems and caused a 3.9 ft. surface elevation at the PSL/AB gauge location, near-field seismic tsunami hazard at DCPP is clearly dominated by the Hosgri and Santa Lucia Banks faults.

Rupture of the Hosgri Fault and Santa Lucia Bank Fault are thus considered significant seismic sources for DCPP and were selected as near-field RPMT sources for DCPP. As recommended by Ellsworth (2003), a preliminary maximum magnitude of Mw 7.3 is selected for the Hosgri Fault event, and a Mw 7.0 magnitude for the Santa Lucia Fault event (the initial magnitude attributed to the 1927 event).

<u>Submarine Mass Failure</u>

SMF analogs that describe characteristics of historic SMFs in the DCPP area for use as potential tsunami sources were reviewed for their impact of tsunami. These analogs include the Goleta slide, a smaller Gaviota slide, and the Sur Slide:

- The largest SMF analog is the Goleta slide, approximately 14.6 km long, 10.5 km wide with a displacement volume of 1.51 km^3 (Greene et al. 2006). The slide is located in the Santa Barbara Basin, approximately 160 km south of DCPP. The location of the slides with respect to the California coastline is shown in Figure 3-14. The Goleta slide consists of three major lobes and may represent as many as 24 different independent slides (Fisher et al 2005; Greene et al 2006). Seismic data of the Goleta slide indicates the three lobes of the slide. Failure of the entire slide as one unit is not considered a likely scenario.

- A smaller Gaviota slide is located approximately 10 km to the west of the Goleta slide also within the Santa Barbara Basin. The Gaviota slide is 1.65 km wide, 2.6 km wide (Greene et al. 2006).

- The second largest SMF analog is the Sur Slide, which is located on the lower continental slope in the northern part of the Santa Lucia Escarpment Zone. This slide is considered a possible analog for the maximum size landslide that may occur elsewhere along the lower continental slope of the Santa Lucia Escarpment. The location of the Sur Slide is shown in Figure 3-15, and is located approximately 190 km north west of DCPP. The size of the slide is approximately 10 km wide by 10 km long.

To determine which SMFs should be selected as near-field sources, a preliminary analysis was completed for impact at DCPP. Based on the historical SMFs of Goleta to the east (Figure 3-14) and Sur to the North (Figure 3-15), one deep water SMF proxy (Big Sur proxy) and a shallow water proxy (Goleta proxy) were parameterized, sited, and modeled. The Goleta proxy was modeled on the Santa Maria Slope Break Zone (SMSB), and the Big Sur proxy was modeled on the Central Santa Lucia Escarpment Zone (ECZ), respectively (Figure 3-16). While SMSB is in fairly shallow water (200-400 m), the ECZ is located in deeper water (1,500-3,500 m). Tsunami generation and propagation from these slides were simulated using state-of-the-art models, and tsunami impact, in terms of runup and inundation, was computed at the DCPP site, in a series of gradually finer resolution nested grids.

The two slide proxies were sited in their respective areas (SMSB, ECZ), based on the estimated characteristics of the historical Goleta and Sur slides, and site-specific considerations within each area, such as water depth, seafloor morphology, maximum sediment thickness, and volume of sediment. Conservative choices and assumptions were made to maximize tsunami generation and impact at DCPP.

The Sur Slide was divided into two proxies based on water depth and other considerations. The worst case of the two (North Big Sur Proxy) was selected.

Near-Field Selections

Based on the above research, five near-field sources were selected.

The two near-field seismic sources that were selected were:

1. Hosgri fault (HFS)
2. San Lucia fault (SLFS)

Two SMFs were selected as near-field sources:

3. Goleta proxy
4. Big Sur proxy

In addition, because the Hosgri fault poses the largest near-field seismic hazard[2], the Goleta proxy combined with the Hosgri fault event was also selected as a near-field RPMT.

5. Goleta proxy combined with the Hosgri fault

3.6.2 Antecedent Water Level and Sea Level Rise Estimates

In accordance with NUREG/CR-6966 (NRC, 2009), the runup from a tsunami should be evaluated coincidentally with an antecedent water level equal to the 10 percent exceedance high tide. The 10 percent exceedance high tide is the high water level that is equaled or exceeded by 10 percent of the maximum monthly tides over a continuous 21-year period (ANSI/ANS, 1992). In accordance with JLD-ISG-2012-06 (NRC, 2013) and NUREG/CR-7046 (NRC, 2011), consideration should also be given to the long-term effect of sea level rise (for the lifetime of the plant). Therefore, the high tide antecedent water level should be taken as the 10

[2] Using the latest Shoreline fault data (PGE, 2014), an evaluation of the Shoreline fault was completed to determine if it should be combined with the Goleta proxy and Hosgri fault. The resulting increase to the RPMT wave height was 10 mm. Since the contribution of the Shoreline fault is minimal, it was not selected for inclusion as a near-field RPMT.

percent exceedance high tide plus long-term sea level rise calculated for the lifetime of the plant. This is referred to as the high antecedent water level.

In accordance with NUREG/CR-6966 (NRC, 2009), the drawdown from a tsunami should be evaluated coincidentally with an antecedent water level equal to the 90 percent low tide. The 90 percent low tide, defined as the low tide level that is equal to or less than 90 percent of the minimum monthly tides over a continuous 21-year period (e.g., 10 percent of low tides are below this value). This is referred to as the low antecedent water level.

The antecedent water levels for DCPP were calculated using verified tide data from the Port San Luis NOAA COOP station (Port San Luis, CA - Station 9412110), which is the closest station to DCPP (approximately 6 miles southeast). The results for the antecedent water levels at DCPP were:

High Antecedent Water Level (HAWL): 7.0 ft. NAVD88

Low Antecedent Water Level (LAWL): -1.9 ft. NAVD88

To determine sea level rise, the observed average (linear) rate at the Port San Luis gauge from 1946 to 2006 was used (0.0311 inches/year [0.79 mm/yr]). Assuming the remaining plant life is 40 years, sea level rise through 2054 was estimated at 0.104 ft. JLD-ISG-20120-06 also allows regional or global sea level rise trends to be added to tsunami simulations for additional margin. California's "California Coastal Commission Draft Sea-Level Rise Policy Guidance" (CCC, 2013) is considered the best available science on sea level rise for California. This draft guidance document adopts the National Research Council's 2012 Report as the best available science on sea level rise in the state of California. Using Appendix B of California's draft guidance, the regional projected sea level rise was determined to be 1.7 ft. for the expected plant life of 40 years.

Using the alternative sea level rise method, the alternative high antecedent water level at Diablo Canyon was:

Alternative High Antecedent Water Level (HHWL): 8.7 ft. NAVD88

3.6.3 Tsunami Reevaluation

Simulations of wave propagation and inundation for each tsunamigenic source were performed on a series of nested grids using the FUNWAVE-TVD 2.0 model (2012). Inputs for modeling each of the simulations were based on either existing published (and peer-reviewed) research or new calculations. For the near-field (SMF) sources, the NHWAVE 1.1 model (2013) was used to first compute the initial sea surface and velocities based on slide motion. The generated waves were then propagated toward the site using FUNWAVE-TVD.

3.6.3.1 Parameterization of Seismic RPMT Sources

Tsunami propagation and coastal impact resulting from each of the selected seismic RPMT sources was simulated using the long wave model FUNWAVE-TVD (Shi et al., 2012), in a series of nested grids of increasingly fine resolution towards DCPP. To do so, as is standard for coseismic tsunamis, the initial tsunami elevation in FUNWAVE-TVD was initialized using the seafloor elevation computed using the Okada (Okada, 1985) model, without initial velocity.

For the seismic RPMT sources, the maximum expected earthquake magnitude Mw (Table 3-15) was defined either based on the literature or, in some cases, based on new calculations. For some RPMTs, other related fault parameters were defined as required, and the initial seafloor elevation of each seismic RPMT source was computed using Okada's (1985) method (for KSZ, HFS, and SLFS), or it was obtained from reliable references (for the ASZ, SSZ). For the HFS, the fault parameters are consistent with those provided in the 2014 Central Coastal California Seismic Imaging Project Report (PGE, 2014). For the JSZ, the initial tsunami surface elevation and horizontal Tsunami Analyses Results velocity were obtained from earlier modeling of both the Japan Trench deformation (induced by the Tohoku 2011 event) and resulting tsunami generation (Grilli et al., 2013a, b; Tappin et al., 2014).

3.6.3.2 Parameterization of Submarine Mass Failure Sources

The methodology used to model SMF geometry and kinematics, for the purpose of tsunami generation simulations using the model NHWAVE (Ma et al., 2012), was approved for tsunami hazard assessment and inundation mapping by the U.S. National Tsunami Hazard Mitigation Program's (NTHMP) Mapping and Modeling Subcommittee (MMS). Details of the methodology and its application for calculating SMF tsunami hazard along the US East coast can be found in Grilli et al. (2014). For SMF locations and bathymetry, see Figures 3-17 and 3-18. Parameters used to define boundary conditions for use in the NHWAVE simulations are provided in Table 3-16.

3.6.4 Tsunami Reevaluation Results

Coarse grid simulations indicated that the dominant far-field seismic RPMT is the Semidi (SSZ) event, and the dominant near-field SMF RPMT is the Goleta proxy. HFS is the dominant near-field seismic RPMT; however, the impact at DCPP of this near-field seismic event is so small (3.3 ft. NAVD88, 0.4 ft. MSL) as compared to that of the two other RPMTs (several meters) that it did not warrant continued computations in the finer grids.

Fine grid simulations were then performed on the SSZ event and Goleta proxy. Fine grid simulations were also run for the worse case RPMT, Goleta proxy, assuming a modified breakwater (3 ft. of the crest reduced) to account for an event where the DCPP breakwater was partially damaged by the Hosgri or San Lucia fault earthquake that would trigger the SMF. As discussed in Section 3.6.2, fine grid simulations were run for three water levels:

- HAWL at 7.0 ft. NAVD88
- LAWL at -1.9 ft. NAVD88

- HHWL at 8.7 ft. NAVD88

In all cases, maximum runup in the intake area occurred in back of (north of) the intake structure for the HHWL. The Goleta proxy was the controlling RPMT for maximum water level.

The ASW system is the only safety-related system that could be affected by a tsunami. The intake structure (as shown in Figure 3-19), which houses this system, has a top deck elevation of 20.4 ft. NAVD88 (17.5 ft. MSL). The ASW ventilation snorkels, which were designed to prevent water intrusion into the ASW pump rooms, have openings at 48.5 ft. NAVD88 elevation (45.6 ft. MSL). The top of ASW ventilation snorkels extend to 52.3 ft. NAVD88 elevation (49.4 ft. MSL).

The maximum RPMT water level in the area of the intake structure was 32.8 ft. NAVD88 (29.9 ft. MSL).

For the case of the Goleta proxy and the modified breakwater bathymetry, water was predicted to runup the steep slope behind the intake structure to an elevation of 62.3 ft. NAVD88 (59.4 ft. MSL).

For all RPMT cases, the maximum water drawdown in front of the intake structure was -15.7 ft. NAVD88 (-18.6 ft. MSL).

All results discussed above are provided in Table 3-17. Section 4.6 evaluates the impact of RPMT maximum water levels, runup, and drawdown.

In addition to wave runup and drawdown, maximum current velocities and impulse force were calculated for use in evaluating hydrostatic, hydrodynamic and impact on civil structures as well as scour and sediment transport in the intake cove. The results of these calculations are provided in Table 3-18.

3.6.4.1 Hydrostatic and Hydrodynamic Forces

Using the results of the RPMT, potential load forces were calculated for the five civil commodities that house safety-related SSCs at the intake structure. These commodities include the west-facing intake structure curtain wall, the intake structure top deck, the ASW forebay ceilings, the ASW ventilation huts, and the ASW ventilation snorkels. An elevation profile of the commodities is shown in Figure 3-19.

Using state of the art software, assumptions, and methodologies and employing contemporary guidance provided by NRC (NRC, 2009) and FEMA publications (FEMA, 2011; FEMA, 2012), combined hydrodynamic and hydrostatic forces were determined for each of the five commodities. Results are provided in Figure 3-20. As discussed in Section 4.6, the RPMT loads are bounded by the CLB loads.

3.6.4.2 Waterborne Projectiles and Debris Loads

A probabilistic analysis was performed to estimate the probability of a large marine vessel arriving at the DCPP breakwater and impacting the intake structure. In 2002, the US Coast Guard established a security zone with a radius centered at DCPP. No vessel may enter this security zone without authorization. Thus, only vessels grounded at the breakwater were considered because a traveling vessel is required to remain outside the security perimeter. A tsunami is expected to break well within the security perimeter; therefore, a passing vessel will not contribute to debris during a tsunami event. Based on the above assumptions, the probability of a large marine vessel arriving at the DCPP breakwater and impacting the intake structure was determined to be 3.1×10^{-5} events per year.

To determine the type of objects that may be susceptible as debris during a tsunami event, a walkdown was completed for the intake cove area. Table 3-19 provides a listing of susceptible commodities, their weight class, and material type. The maximum weight from each class except the heaviest class was evaluated for impact to the five civil commodities that house safety-related SSCs at the intake structure. For the heaviest weight class (greater than 10,000 pounds [lbs.]), a projectile weighing 20,000 lbs. (representing an Intake Cove kelp harvesting vessel) was evaluated. Additionally, in order to account for future potential commodities and temporary maintenance activities in the intake cove area, PG&E conservatively evaluated hypothetical submerged projectiles of up to 100,000 lbs. impacting the ASW curtain wall.

Projectiles less than 10,000 lbs. were assumed to be floating or at any elevation in the flow stream and to be traveling at maximum fluid velocity. Other projectiles in the heaviest weight class were assumed capable of entrainment in the tsunami flowstream, but incapable of floatation. A projectile greater than 10,000 lbs. located on the intake structure top deck was also evaluated for impact to the top deck and the ASW ventilation huts.

The maximum loads resulting from projectile impact forces, debris damming, and combined forces for each of the civil commodities is provided in Table 3-20. As discussed in Section 4.6, the RPMT projectile loads are bounded by the CLB tornado-generated missile loads.

3.6.4.3 Debris and Sedimentation

An evaluation was performed to identify the sediment erosion, suspension and deposition resulting from the RPMT. Detailed tsunami numerical modeling was performed to estimate the tsunami effects resulting from several tsunami sources. The Goleta proxy was determined to be the controlling RPMT relative to maximum run-up and current velocity.

The ASW bypass piping was determined to be the only safety-related component vulnerable to negative impacts from scour. During the ASW bypass project performed during the late 1990's, a scour and erosion assessment, due to a tsunami combined with storm waves, was performed and scour and erosion mitigation measures were designed and constructed (see Section 2.3.1). These analyses evaluated several different wave and stillwater elevation conditions and developed tsunami flow velocities (37 ft/sec).

The maximum RPMT water velocities over the ASW bypass piping are 82 ft/sec for intermittent short durations (few seconds). Maximum water velocities over the top deck of the intake structure were also 82 ft/sec. Section 4.6 evaluates the impact of RPMT maximum water velocities on safety-related SSCs.

3.7 Ice Induced Flooding

As described in Section 2.3.2.7, DCPP is not affected by ice-induced flooding.

3.8 Channel Diversion and Migration

As described in Section 2.3.2.8, DCPP is not affected by channel diversion and migration.

3.9 Combined Events

Consistent with NUREG/CR-7046, Appendix H (NRC, 2011), PG&E considered combined events, as described below.

3.9.1 Floods Caused by Precipitation Events

As discussed in Section 3.2.1, the ANS guidance provided in NUREG/CR-7046 was used to evaluate three combination alternatives for reevaluated PMP (RPMP). The RPMP determined the following maximum values, all of which were used as inputs into the reevaluated PMF:

Alternative 1 – all-season general storm PMP for a 72-hour storm duration is 30.90 inches (see Table 3-8)

Alternative 2 – combining 100-year rainfall and snowmelt with probable maximum snowpack for a 72-hour storm duration is 20.55 inches

Alternative 3 – combination of cool/snow season PMP and snowmelt with 100-year snowpack for a 72-hour storm duration is 32.09 inches

3.9.2 Floods Along the Shore of Open Bodies of Water

As discussed in Section 3.4, for a shore location, the ANS guidance provided in NUREG/CR-7046 was used to evaluate a combination of probable maximum surge and seiche with wind-wave activity combined with the antecedent 10 percent exceedance high tide. The maximum estimated wave height outside the breakwaters combined with SAWL was 44.6 ft. The maximum crest wave level inside the breakwaters was 12.8 ft. NAVD88 (9.9 ft. MSL).

3.9.3 Floods Caused by Tsunamis

As discussed in Section 3.4, for a shore location, the guidance provided in NUREG/CR-7046 and NUREG/CR-6966 was used to evaluate a combination of (1) probable maximum tsunami runup combined with the antecedent 10 percent exceedance high tide (HAWL); and (2) probable maximum tsunami drawdown combined with the antecedent water level equal to the

90 percent low tide (LAWL). In addition, PG&E completed a third combination of the probable maximum tsunami runup combined with the antecedent 10 percent exceedance high tide using California-specific sea level rise information (HHWL). The RPMT reanalysis determined the following:

(1) The Goleta proxy was the controlling RPMT for runup. The maximum Goleta proxy runup combined with the HAWL was 32.8 ft. NAVD88 (29.9 ft. MSL) (along the steep slope in back of the intake structure).

(2) The Goleta proxy was the controlling RPMT for drawdown. The maximum Goleta proxy drawdown combined with the LAWL was -15.7 ft. NAVD88 (-18.6 ft. MSL) (in front of the intake structure).

(3) The maximum Goleta proxy runup combined with the HHWL was 62.3 ft. NAVD88 (59.4 ft. MSL) (along the steep slope in back of the intake structure).

4.0 COMPARISON OF CURRENT AND REEVALUATED FLOOD-CAUSING MECHANISMS

4.1 Local Intense Precipitation

For LIP (see Section 2.3.2.1), the current analysis concludes that it is not possible for ponding from local PMP to flood safety-related buildings. The reevaluated LIP (see Section 3.1) determined that the water depth above the door thresholds and areas to the west of the turbine and buttress buildings varied between 0.09 ft. and 1.4 ft., with five of the 29 doors/areas showing no inundation. The duration of time dependent water depths varied between 0.00 hours and 4.41 hours. Several doors and areas show the potential to experience inundation. The reevaluated LIP is not bounded by the current analysis. Therefore, PG&E implemented interim actions, as described in Section 5.1.

4.2 Riverine (Rivers and Streams) Flooding

4.2.1 Probable Maximum Precipitation

The current PMP analysis (see Section 2.3.2.2) concludes that the DCPP PMP for a 24-hour duration is 16.6 inches. The RPMP (see Section 3.2.1) determined the DCPP all-season general storm PMP for a 24-hour duration is 18.2 inches.

Time (hrs)	PMP (inches)	Reevaluated PMP (inches)	Bounded/Not Bounded
1	4.3	2.4	Bounded
3	7.1	4.9	Bounded
6	9.1	8.2	Bounded
12	12.0	13.5	Not Bounded
18	14.8	15.9	Not Bounded
24	16.6	18.2	Not Bounded

Although the RPMP is not bounded by the current PMP, it is used as input into the reevaluated PMF, which is bounded by the current PMF analysis (see Section 4.2.2).

The RPMP also determined the following maximum values, all of which were used as inputs into the reevaluated PMF:

Alternative 1 – all-season general storm PMP for a 72-hour storm duration is 30.90 inches (see Table 3-8)

Alternative 2 – combining 100-year rainfall and snowmelt with probable maximum snowpack for a 72-hour storm duration is 20.55 inches

Alternative 3 – combination of cool/snow season PMP and snowmelt with 100-year snowpack for a 72-hour storm duration is 32.09 inches

4.2.2 Probable Maximum Flooding

The current PMF analysis (see Section 2.3.2.2) determined that Diablo Creek is adequate to handle the PMF. Estimated maximum water surface elevation during a PMF at a point nearest the plant was approximately 6 ft. below plant grade for the worst case. Thus, the depth of water at the plant location for the current PMF is zero. For a drainage area of 5.19 sq-mi, the current PMF was found to have a peak discharge of 6,878 cfs for the 24-hour storm. The reevaluated PMF (RPMF) (see Section 3.2.2) determined a maximum water level of approximately 8 ft. below plant grade (including wind-wave activity). No safety-related SSCs were inundated by RPMF. The critical RPMF peak discharge resulting from RPMP was determined to be 6,541 cfs. As the reevaluated peak discharge for the drainage area is less than the current PMF and flooding levels would not adversely affect safety-related SSCs, the RPMF is bounded.

4.3 Dam Breaches and Failures

As described in Section 2.3.2.3, DCPP is not affected by flooding from dam breaches or failures.

4.4 Storm Surge

The current storm surge analysis (see Section 2.3.2.4) considered waves up to 45 ft. outside the breakwater. The results indicated that the response waves within the intake basin reached a maximum height and did not increase further in response to increases in the offshore wave height. For the reevaluated PMSS (see Section 3.4), the maximum estimated wave height outside the breakwaters was 44.6 ft. The maximum crest wave level inside the breakwaters was 12.8 ft. NAVD88 (9.9 ft. MSL). The reevaluated maximum estimated wave height outside the breakwaters is less than what was analyzed in the current storm surge analysis. Further, the resulting crest wave level inside the breakwaters is much less than the current PMT wave height of 34.9 ft. NAVD88 (32.0 ft. MSL). Thus, there is no impact to safety-related SSCs from reevaluated PMSS maximum estimated wave heights and the reevaluated PMSS is bounded.

4.5 Seiche

for the current seiche analysis (see Section 2.3.2.5) determined that there would be no significant loss of water in the RWSRs due to seiche The seiche reevaluation (see Section 3.2.5) determined that, during a seismic event, the maximum expected water volume loss from the 2.5 million gallon RWSRs is 14,684 gallons per RWSR. The current design basis for the raw water reservoirs to perform their design function is a minimum of 2 million gallons (1 million gallons per reservoir). The raw water storage reservoirs are able to perform their design function with only 1 million gallons per reservoir. As such, loss of 14,684 gallons is not significant. Thus, the reevaluated seiche continues to maintain the RWSR safety function and is bounded by the current design basis.

4.6 Tsunami Flooding

Current PMT (see Sections 2.3.2.6 and 2.3.2.13) wave runup and drawdown values are provided in Table 3-17. For the RPMT (see Section 3.2.6), runup and drawdown were determined and compared to the current PMT (Table 3-17):

- The maximum RPMT water level the area of the intake structure was 32.8 ft. NAVD88 (29.9 ft. MSL). This water level is below the current PMT water level (34.9 ft. NAVD88, 32.1 ft. MSL) and the ASW ventilation snorkel openings (48.5 ft. NAVD88, 45.6 ft. MSL) which protect the safety-related ASW system SSCs. Thus, there is no impact to safety-related SSCs from RPMT water levels at the intake structure and the RPMT water level is bounded by the current PMT water level.

- The RPMT shows water runup on the steep slope behind the intake structure to an elevation of 62.3 ft. NAVD88 (59.4 ft. MSL). Safety-related SSCs in this area include the buried Unit 1 and Unit 2 ASW bypass piping (discussed in Section 2.3.1), which follow separate paths from the intake structure to the plant.

    - The RPMT runup over the Unit 1 ASW bypass piping is below the elevation used in the design and construction of erosion protection measures (i.e., below the top of the gabion mattress). Thus, there would be no impact to the Unit 1 ASW bypass piping.

    - The portion of Unit 2 ASW bypass piping that experiences the RPMT runup is either encased in concrete (thrust blocks) or provided with a minimum of 6 ft. of soil cover. A review of the RPMT velocities in this area shows that higher velocities are only seen in the area where these pipes are encased in a concrete thrust block, which is considered extremely robust and capable of protecting the pipes. Lower velocities are seen at the uphill portions of the piping where it is provided with adequate soil cover to preclude exposure. Thus, there would be no impact to the Unit 2 ASW bypass piping.

  Therefore, the Unit 1 and Unit 2 ASW bypass piping is adequately protected from the predicted runup levels. For the short duration RPMT velocities, see discussion further below.

- The RPMT drawdown (-15.7 ft. NAVD88, -18.6 ft. MSL) is not bounded by the current limiting PMT drawdown (-8.7 ft. NAVD88, -11.6 ft. MSL). However, as described in Section 2.3.2.13, the current design basis for ASW pump operation is a minimum water elevation of -17.1 ft. NAVD88 (-20 ft. MSL) for ASW pump operation (NRC, 1976b). This is below the RPMT drawdown. Thus, in the event of a tsunami drawdown, the ASW pumps are capable of performing their safety function for this temporary condition and are bounded by the current design basis.

Current PMT hydrostatic and hydrodynamic loads (see Section 2.3.2.10) are provided below. For the RPMT (see Section 3.2.6), loads were calculated and compared to the current PMT (Figure 3-20). The RPMT loads are less than (i.e., bounded by) the current PMT loads, as shown below.

Safety-Related SSC	Current PMT Loads (ksf)	RPMT Loads (ksf)	Bounded/Not Bounded
ASW Ventilation Huts (seaward facing side)	2.5 to 1.6 (top to bottom)	1.1 to 1.3 (Note 1)	Bounded
ASW Ventilation Huts (North facing side)	0.5 to 3.0 (top to bottom)	1.1 to 1.3 (Note 1)	Bounded
ASW Ventilation Snorkels	0.0	0.0	Bounded
Intake Structure Curtain Wall	1.54 (side panels) 2.4 (center panel)	0.7 to 0.8 (entire surface)	Bounded
ASW Forebay Ceiling	6.2	2.5	Bounded
Intake Structure Top Deck	0.93	0.3	Bounded

1. Loads do not extend higher that the RPMT inundation elevation

As discussed in Section 2.3.2.11, the bounding projectile loads at DCPP are not generated from tsunami projectiles, but from tornado-generated missiles. Thus, the projectile loads for the RPMTs were conservatively compared to the loads that would occur from a bounding tornado-generated missile (4,000 lb. automobile). As shown in Table 3-20, the RPMT projectile loads are bounded by the current tornado-generated missile loads.

The ASW bypass piping was designed and constructed with scour and erosion mitigation measures (see Section 3.6.4.3). Erosion protection measures were designed using tsunami flow velocities of 37 ft/sec and consist of thickened asphalt concrete, 12-inch thick concrete cover slabs, and concrete slope protection. The RPMT analysis shows maximum velocities of approximately 82 ft/sec for intermittent short durations (few seconds), which exceed the maximum velocities of 37 ft/sec used in the design of scour and erosion mitigation measures. There are two areas of the ASW bypass piping that have the potential to be impacted:

- Area 1 – this area consists of both Unit 1 and Unit 2 piping parallel to the back of the intake structure (East wall). Concrete cap slabs are installed directly over the piping in shallow locations, and concrete slope protection is installed in areas along the main roadway. As discussed in the design basis analysis, concrete provides protection for velocities up 120 ft/sec; therefore, these components will provide adequate protection. Asphalt concrete in the areas around the concrete cap slabs is not qualified for velocities higher than 37 ft/sec. The higher velocities only occur for brief periods of time (few seconds) and are not postulated to cause complete failure of the thickened asphalt roadway. If these features were to fail; however, erosion next to the 12-inch thick concrete cap slabs over the piping is not expected to cause failure of these cap slabs.

Some localized erosion could occur; however, the cap slab would remain and provide the required protection.

- Area 2 – this area consists of both Unit 1 and Unit 2 piping perpendicular to the back of the intake structure (East wall). In this location, the piping is a minimum of 6-ft. deep and overtopped with 5-inch thick asphalt concrete. As discussed above, the asphalt concrete is not rated for velocities greater than 37 ft/sec; however, these velocities are for very short durations and are not expected to cause failure of the asphalt concrete paving. Additionally, the existing backfill provides more than adequate protection over the pipes in the unlikely event of pavement failure.

Therefore, the Unit 1 and Unit 2 ASW bypass piping is adequately protected for the short duration RPMT velocities.

4.7 Ice Induced Flooding

As described in Section 2.3.2.7, DCPP is not affected by ice-induced flooding.

4.8 Channel Migration and Diversion

As described in Section 2.3.2.8, DCPP is not affected by channel migration or diversion.

4.9 Combined Events

Refer to Sections 4.2.1, 4.4., and 4.6.

5.0 INTERIM EVALUATION AND ACTIONS

5.1 Local Intense Precipitation

The LIP reevaluation indicates that LIP will result in positive water depth at doors and areas adjacent to safety and non safety-related structures. The flood hazard reevaluations are distinct from the current design and licensing bases of DCPP and do not alter the terms of the license. NRC Staff considers the flood hazard reevaluations being performed to be beyond the current design/licensing basis of operating plants (NRC, 2012c). The 10 CFR 50.54(f) (CFR, 2014) guidance requests interim actions (taken or planned) to address the reevaluated flooding hazard be included in the licensee response when results are not bounded by the current design basis.

In response to the LIP exceedances, a LIP Mitigation Evaluation was developed to evaluate options for interim actions. The FLO-2D computer model used for the LIP reevaluation was used in the LIP Mitigation Evaluation in order to evaluate the feasibility of reducing the WSE and water depth at the various doors/entryways. The Evaluation found that modifications to the site topography or addition of surface drainage features would not completely mitigate the WSE and water depth conditions. The LIP Mitigation Evaluation did not reduce conservatisms used in the LIP modeling (roof drains blocked, storm drains blocked, etc.) which are potentially acceptable in a HHA approach.

The LIP Mitigation Evaluation considered options for mitigation in accordance with guidance provided in FAQ-033, "Hazard Reevaluation Report (HRR) - Options for Interim Actions for Challenging HRRs," (NRC, 2014) and FAQ-031, "Hazard Reevaluation Report (HRR) - Interim Action Responses," (NRC, 2013).

The beyond design basis LIP Mitigation Actions consist of an approach utilizing existing weather forecasting technologies combined with deployment of temporary barriers (sandbags or equivalent) at affected doors and safety and non-safety related structures. This approach was entered into the corrective action program.

PG&E will perform an integrated assessment for the beyond design basis LIP event in accordance with NRC-approved guidance. Long-term mitigation actions will be addressed in the integrated assessment.

5.2 Riverine (Rivers and Streams) Flooding

No interim actions are required because the flooding levels would not adversely affect safety-related SSCs. Therefore, this hazard will not be addressed in the integrated assessment.

5.3 Dam Breaches and Failures Flooding

No interim actions are required because this hazard does not apply to DCPP. Therefore, this hazard will not be addressed in the integrated assessment.

5.4 Storm Surge

No interim actions are required because the reevaluated wave heights would not adversely affect safety-related SSCs and is bounded by the current analysis wave heights. Therefore, this hazard will not be addressed in the integrated assessment.

5.5 Seiche

No interim actions are required because the loss of water due to seiche does not impede the RWSRs from performing their design function. Therefore, this hazard will not be addressed in the integrated assessment.

5.6 Tsunami

The RPMT reevaluation indicated that the RPMT drawdown (-15.7 ft. NAVD88, -18.6 ft. MSL) is not bounded by the current PMT drawdown (-8.7 ft. NAVD88, -11.6 ft. MSL). While the RPMT drawdown is not bounded by the current PMT drawdown, the ASW pumps' current design basis is to operate with a water level of -17.1 ft. NAVD88 (-20 ft. MSL), which is below the RPMT drawdown. Thus, in the event of a reevaluated tsunami drawdown, the ASW pumps are capable of performing their safety function for this temporary condition. The 10 CFR 50.54(f) (CFR, 2014) guidance requests that interim actions (taken or planned) to address the reevaluated flooding hazard be included in the licensee response when results are not bounded by the current design basis. The ASW pumps' current design basis analysis is bounding for the RPMT drawdown. Therefore, interim actions are not necessary, and this will not be addressed in the integrated assessment.

The RPMT reevaluation indicated that the RPMT runup behind the intake structure (62.3 ft. NAVD88, 59.4 ft. MSL) is not bounded by the current PMT runup. As discussed in Section 4.6, while the RPMT water runup behind the intake structure is not bounded by the current PMT runup, the existing erosion protection measures analyzed in the current design basis are sufficient to handle the RPMT velocities. The existing erosion protection measures analyzed in the current design basis analysis are adequate to address the RPMT runup. Therefore, interim actions are not necessary, and this will not be addressed in the integrated assessment.

No interim actions are required for the RPMT maximum water level because it is bounded by the current PMT maximum water level. Therefore, the impact of the RPMT runup will not be addressed in the integrated assessment.

No interim actions are required for the RPMT hydrostatic and hydrodynamic loads because they are bounded by the current PMT loads. Therefore, the impact of the RPMT loads will not be addressed in the integrated assessment.

No interim actions are required for the RPMT projectile loads because it is bounded by the current tornado-generated missile loads. Therefore, the impact of the RPMT projectile loads will not be addressed in the integrated assessment.

While no interim actions are required, it should be noted that, in response to NRC Security Order EA-02-026, Section B.5.b, PG&E maintains on-site portable diesel-driven emergency ASW (EASW) pumps. In the event the ASW system can no longer perform its design function, the EASW pumps and rigid piping segments would be used to restore the ultimate heat sink function for safe shutdown cooling.

5.7 Ice Induced Flooding

No interim actions are required because this hazard does not apply to DCPP. Therefore, this hazard will not be addressed in the integrated assessment.

5.8 Channel Diversion and Migration

No interim actions are required because this hazard does not apply to DCPP. Therefore, this hazard will not be addressed in the integrated assessment.

5.9 Combined Events

Refer to Sections 5.2.1, 5.4., and 5.6.