2 SITE CHARACTERISTICS

2.1 Conduct of Review

Chapter 2 of the Diablo Canyon ISFSI Safety Analysis Report (SAR) discusses the geographical location of the Diablo Canyon ISFSI and meteorological, hydrological, seismological, and geological characteristics of the site and the surrounding area. It also describes the population distribution around the Diablo Canyon ISFSI, land and water uses, and associated site activities. Chapter 2 of the SAR also evaluates site characteristics with regard to safety and identifies assumptions that need to be applied when evaluating safety, establishing installation design, and providing design bases in other evaluations in the SAR.

The information and analyses in SAR Chapter 2 were reviewed with respect to the applicable siting evaluation regulations in 10 CFR Part 72, Subpart E, and 10 CFR §72.122(b). Where appropriate, findings of regulatory compliance were made in this chapter of the staff's Safety Evaluation Report (SER) for the 10 CFR Part 72 requirements that are fully addressed in Chapter 2 of the SAR. Findings of technical adequacy and acceptability were made for each section in Chapter 2 of the SAR. However, because compliance with some regulations was determined by an integrated review of several sections in Chapter 2 and other chapters within the SAR, a finding of regulatory compliance was not made in each major section of this chapter unless the specific regulatory requirement was fully addressed.

In addition to the information presented in the ISFSI SAR (Pacific Gas and Electric Company, 2002), much of the information pertaining to the site characteristics was derived from the Diablo Canyon Power Plant (DCPP) Final Safety Analysis Report (FSAR) (Pacific Gas and Electric Company, 2001). The DCPP FSAR is periodically updated in accordance with 10 CFR §50.71(e). Additionally, in response to License Condition Item 2.C.(7) of the Unit 1 Operating License issued in 1980, PG&E was required to reevaluate the seismic design for the DCPP.

The reevaluation included a 10-year study of the geologic, tectonic, and seismological characteristics of the DCPP site. That 10-year study was documented in a reported titled "Diablo Canyon Long-Term Seismic Program" (Pacific Gas and Electric Company, 1988, 1991). In 1991, the NRC staff concluded that the analyses presented in the Diablo Canyon long-term seismic program (LTSP) report were sufficient to satisfy License Condition Item 2.C.(7) (U.S. Nuclear Regulatory Commission,1991).

2.1.1 Geography and Demography

This section contains the review of Section 2.1, "Geography and Demography," of the ISFSI SAR. Subsections discussed include (i) Site Location, (ii) Site Description, (iii) Population Distribution and Trends, and (iv) Land and Water Uses. The staff reviewed the discussion on geography and demography with respect to the following regulatory requirements:

- 10 CFR §72.24(a) requires a description and safety assessment of the site on which the ISFSI is to be located, with appropriate attention to the design bases for external events. Such assessment must contain an analysis and evaluation of the major structures, systems, and components of the ISFSI that bear on the

suitability of the site when the ISFSI is operated at its design capacity. If the proposed ISFSI is to be located on the site of a nuclear power plant or other licensed facility, the potential interactions between the ISFSI and such other facility-including shared common utilities and services-must be evaluated.

- 10 CFR §72.90(a) requires that site characteristics that may directly affect the safety or environmental impact of the ISFSI be investigated and assessed.

- 10 CFR §72.90(b) requires that proposed sites for the ISFSI must be examined with respect to the frequency and the severity of external natural and man-induced events that could affect the safe operation of the ISFSI.

- 10 CFR §72.90(c) requires that design-basis external events must be determined for each combination of proposed site and proposed ISFSI design.

- 10 CFR §72.90(e) requires that pursuant to Subpart A of Part 51 of Title 10 for each proposed site for an ISFSI, the potential for radiological and other environmental impacts on the region must be evaluated with due consideration of the characteristics of the population, including its distribution, and of the regional environs, including its historical and aesthetic values.

- 10 CFR §72.90(f) requires the facility to be sited so as to avoid to the extent possible the long-term and short-term adverse impacts associated with the occupancy and modification of floodplains.

- 10 CFR §72.98(a) requires that the regional extent of external phenomena, man-made or natural, that are used as a basis for the design of the ISFSI must be identified.

- 10 CFR §72.98(b) requires that the potential regional impact caused by the construction, operation, or decommissioning of the ISFSI must be identified. The extent of regional impacts must be determined on the basis of potential measurable effects on the population or the environment from ISFSI activities.

- 10 CFR §72.98(c) requires that those regions identified pursuant to paragraph 10 CFR §72.98(a) and (b) must be investigated as appropriate with respect to (1) the present and future character and the distribution of the population, (2) consideration of present and projected future uses of land and water within the region, and (3) any special characteristics that may influence the potential consequences of a release of radioactive material during the operational lifetime of the ISFSI.

- 10 CFR §72.100(a) requires that the proposed site must be evaluated with respect to the effects on populations in the region resulting from the release of radioactive materials under normal and accident conditions during operation and decommissioning of the ISFSI; in this evaluation, both usual and unusual regional and site characteristics shall be taken into account.

2.1.1.1 Site Location

Section 2.1.1 of the SAR, "Site Location," and relevant literature cited in the SAR describe the site location. The Diablo Canyon ISFSI is to be located within the PG&E-owned area at the DCPP.

The DCPP site consists of 300 hectares [750 acres] of land located along the Pacific Ocean in San Luis Obispo County, California. The site is approximately 19 km [12 mi] west-southwest of the city of San Luis Obispo, California, and approximately 10 km [6 mi] northwest of Avila Beach, California. The SAR reports the latitude and longitude, and the Universal Transverse Mercator coordinates of the Diablo Canyon ISFSI.

The staff reviewed the description of the site location and found it acceptable because it clearly describes the geographic location of the site, including its relationship to political boundaries and natural anthropogenic features. The maps provided in the SAR are acceptable because they provide sufficient detail to review the Diablo Canyon ISFSI. This information is acceptable for use in other sections of the SAR to develop the design bases of the ISFSI, perform additional safety analysis, and demonstrate compliance with the regulatory requirements in 10 CFR §72.24(a), §72.90(a), §72.90(e), and §72.98(a) with respect to this topic.

2.1.1.2 Site Description

Section 2.1.2 of the SAR, "Site Description," describes the site with maps to delineate the site boundary and controlled area. The Diablo Canyon ISFSI site is to be located within the PG&E-owned area at the DCPP.

The location and orientation of the Diablo Canyon ISFSI structures with respect to nearby roads and waterways are shown on various maps and plots, and there is no obvious way in which traffic on adjacent transportation links can interfere with ISFSI operations. The proposed Diablo Canyon ISFSI site is located within the site boundaries of the existing PG&E DCPP site. PG&E owns the land along the coast, north 9.5 km [5.9 mi] to the Montana de Oro State Park and south approximately 13 km [8 mi] to Avila Beach and inland between 0.8 km [0.5 mi] and 2.9 km [1.8 mi]. The area surrounding the DCPP site is encumbered by grazing leases. Access to the ISFSI and the DCPP-controlled areas is restricted by fencing. The PG&E owner-controlled area is not traversed by any public roads or railroads. Ingress and egress of site personnel will be controlled. Access to the site is from the south, at Avila Beach, along a 10.5-km [6.5-mi] private road through the PG&E-owned site.

The DCPP owner-controlled area rests on a coastal terrace and adjacent coastal uplands. Elevations of the DCPP site range from 18 to 425 m [60 to 1,400 ft] above mean sea level (MSL). The ISFSI site is located on bedrock between hillsides at an elevation of 94.5 m [310 ft]. Drainage of the area is through Diablo Creek, located just north of the ISFSI pad site.

The staff reviewed the site description and relevant literature cited in the SAR. The staff finds that the site description is adequate because the descriptive information and maps clearly delineate the site boundary, controlled area, general natural and man-made features, topography, and surface hydrologic features. The maps have a sufficient level of detail and are of appropriate scale and legibility required for the review of the site and the Diablo Canyon

ISFSI. The information is also acceptable to determine distances between the ISFSI and nearby facilities and cities. This information is acceptable for use in other sections of the SAR to develop the design bases of the ISFSI, perform additional safety analysis, and demonstrate compliance with regulatory requirements in 10 CFR §72.90(a), §72.90(e), and §72.98(a) with respect to this topic.

2.1.1.3 Population Distribution and Trends

Section 2.1.3 of the SAR, "Population Distribution and Trends," and relevant literature cited in the SAR describes the population distribution and trends. The population data used in the SAR were derived from the 2000 census and from estimates of future population provided by the California Department of Finance. The population distribution and trends description in the SAR is based on information from the DCPP FSAR update.

The 2000 census data show the population within 80.5 km [50 mi] of the ISFSI site to be 424,000 people. The same census data show that 23,700 people live within a 16-km [10-mi] radius of the ISFSI site. The population within a 9.6-km [6-mi] radius of the site, defined by PG&E as the low population zone, is approximately 100 residents. The nearest residence is approximately 2.4 km [1.5 mi] from the ISFSI site. Information in the SAR indicates that the maximum number of people in the low population zone is 5,000. This population estimate is based on the maximum number of day-use visitors to the Montana de Oro State Park, based on information from the California Department of Parks and Recreation.

PG&E indicates that projected population growth near the ISFSI will be limited based on projected trends in the population data and on the assumption that the land use will not change in character within the next 25 years. This assumption is supported by information on land and water uses, as described in Section 2.1.4 of the SAR. Land use is expected to remain either agricultural or as wilderness within state parks or national forests.

The staff reviewed the information presented in the SAR and has determined that the population distribution and trends in the region have been adequately described and assessed. The source of the population data used in the SAR is appropriate, and the basis for population projections is reasonable. The staff found that 10 CFR §72.98(c)(1) is met because the region has been appropriately investigated with respect to the present and future character and distribution of the population. This information is also acceptable for use in other sections of the SAR to develop the design bases of the ISFSI, perform additional safety analysis, and demonstrate compliance with regulatory requirements in 10 CFR §72.90(e), §72.98(a), and §72.100(a), with respect to this topic.

2.1.1.4 Land and Water Uses

Section 2.1.4 of the SAR, "Uses of Nearby Lands and Waters," describes land and water uses in the region surrounding the DCPP site. The SAR notes that the area between the DCPP site and U.S. Route 101 is dominated by the San Luis Range, which consists of rugged topography with elevation up to 550 m [1,800 ft]. Land use in the San Luis Range is limited to beef cattle grazing, minor dairy cattle grazing, wild and domestic goats, and support of other wildlife. A large portion of this land area is contained within the Los Padres National Forest.

Farming within surrounding regions of San Luis Obispo County is limited to the narrow coastal valleys. Wine grapes are the principal cash crop. The only dairy production is at the California Polytechnic State University, approximately 19 km [12 mi] northeast of the Diablo Canyon ISFSI site. The region also supports sport and commercial deep-sea fishing. Fishing vessels harbor at Morro Bay, located 16 km [10 mi] north of the Diablo Canyon ISFSI site, and at Port San Luis Harbor, located 10 km [6 mi] south of the Diablo Canyon ISFSI site.

PG&E has identified two public groundwater suppliers within 16 km [10 mi] of the site. These suppliers are the Avila Beach County Water and Sewer District and the San Miguelito Mutual Water and Sewer Company. Property owners north and east of the DCPP also capture surface and spring water for limited domestic use. PG&E captures surface water from Crowbar Canyon, located 1.6 km [1 mi] north of the DCPP. DCPP also gets water from an ocean water desalinization plant, which was built at the DCPP in 1985.

The staff reviewed the description of the land and water use in the SAR and found that it has been adequately described and assessed. The region has been investigated as appropriate with respect to consideration of present and projected future uses of land and water within the region. This information is acceptable for use in other sections of the SAR to develop the design bases of the Diablo Canyon ISFSI, perform additional safety analysis, and demonstrate compliance with regulatory requirements in 10 CFR §72.98(a), §72.98(b), and §72.98(c) with respect to this topic.

2.1.2 Nearby Industrial, Transportation, and Military Facilities

Section 2.2 of the SAR, "Nearby Industrial, Transportation, and Military Facilities," and relevant literature cited in the SAR describe each nearby facility and identify potential hazards from all of these facilities. This information is necessary to evaluate credible scenarios involving manmade facilities that may endanger the proposed facility site. The staff reviewed nearby industrial, transportation, and military facilities with respect to the following regulatory requirements:

- 10 CFR §72.24(a) requires a description and safety assessment of the site on which the ISFSI is to be located, with appropriate attention to the design bases for external events. Such assessment must contain an analysis and evaluation of the major structures, systems, and components of the ISFSI that bear on the suitability of the site when the ISFSI is operated at its design capacity. If the proposed ISFSI is to be located on the site of a nuclear power plant or other licensed facility, the potential interactions between the ISFSI and such other facility—including shared common utilities and services—must be evaluated.

- 10 CFR §72.94(a) requires that the region must be examined for both past and present man-made facilities and activities that might endanger the proposed ISFSI. The important potential man-induced events that affect the ISFSI design must be identified.

- 10 CFR §72.94(b) requires that information concerning the potential occurrence and severity of such events must be collected and evaluated for reliability, accuracy, and completeness.

- 10 CFR §72.94(c) requires that appropriate methods must be adopted for evaluating the design-basis external man-induced events, based on the current state of knowledge about such events.

- 10 CFR §72.98(a) requires that the regional extent of external phenomena, man-made or natural, that are used as a basis for the design of the ISFSI must be identified.

- 10 CFR §72.98(b) requires that the potential regional impact caused by the construction, operation, or decommissioning of the ISFSI must be identified. The extent of regional impacts must be determined on the basis of potential measurable effects on the population or the environment from ISFSI activities.

- 10 CFR §72.98(c) requires that those regions identified pursuant to paragraphs 10 CFR §72.98(a) and 10 CFR §72.98(b) must be investigated as appropriate with respect to (1) the present and future character and the distribution of population, (2) consideration of present and projected future uses of land and water within the region, and (3) any special characteristics that may influence the potential consequences of a release of radioactive material during the operational lifetime of the ISFSI.

- 10 CFR §72.100(a) requires that the proposed site must be evaluated with respect to the effects on populations in the region resulting from the release of radioactive materials under normal and accident conditions during operation and decommissioning of the ISFSI; in this evaluation both usual and unusual regional and site characteristics shall be taken into account.

- 10 CFR §72.100(b) requires that each site must be evaluated with respect to the effects on the regional environment resulting from construction, operation, and decommissioning of the ISFSI; in this evaluation, both usual and unusual regional and site characteristics must be taken into account.

The SAR identifies potential hazards, including the identification of facilities and determination of credible scenarios that may endanger the proposed ISFSI. Industry in the vicinity of the Diablo Canyon ISFSI mainly consists of food processing and crude oil refining. Vandenberg Air Force Base is the largest industrial complex near the proposed facility and is located approximately 56 km [35 mi] south-southwest of the proposed ISFSI. This base is the site for launching missiles and polar-orbiting satellites and also the designated alternate landing site for space shuttles. Coastal shipping lanes are approximately 32 km [20 mi] offshore. The Port San Luis tanker loading pier is approximately 9.6 km [6 mi] east-southeast of the proposed site. This pier is no longer active because tanker traffic into Port San Luis has been discontinued. The local tanker terminal at Estero Bay ceased operations in 1994 because of lack of tanker traffic. For the same reason, the Avila Beach pier closed in 1998. Some petroleum products and crude oil are stored at Estero Bay, which is approximately 16 km [10 mi] from the DCPP site. This storage site is sufficiently far from the ISFSI site to have a negligible impact and thus is beyond the distance of regulatory concern.

PG&E identified the potential crash of aircraft onto the proposed site as a potential hazard in the SAR. Civilian aircraft with a potential to crash at the proposed ISFSI include aircraft taking

off and landing at San Luis Obispo Regional Airport and Oceano Airport, and aircraft flying on federal airway V-27. Military aircraft that have a potential to crash at the proposed ISFSI include aircraft flying the military training route VR-249. Additionally, PG&E considered hazards associated with launch and landing of the space shuttle, rockets, and missiles at Vandenberg Air Force Base.

The staff finds that the Diablo Canyon ISFSI SAR adequately identifies all nearby facilities that may present a hazard to the proposed ISFSI and demonstrates compliance with regulatory requirements in 10 CFR §72.24(a). The potential hazards from these facilities are evaluated in Chapter 15 of this SER.

2.1.3 Meteorology

The staff has reviewed the information presented in Section 2.3 of the SAR, "Meteorology." Subsections discussed below include (i) Regional Climatology, (ii) Local Meteorology, and (3) Onsite Meteorological Measurement Program. The staff reviewed the discussion on meteorology with respect to the following regulatory requirements:

- 10 CFR §72.24(a) requires a description and safety assessment of the site on which the ISFSI is to be located, with appropriate attention to the design bases for external events. Such assessment must contain an analysis and evaluation of the major structures, systems, and components of the ISFSI that bear on the suitability of the site when the ISFSI is operated at its design capacity. If the proposed ISFSI is to be located on the site of a nuclear power plant or other licensed facility, the potential interactions between the ISFSI and such other facility—including shared common utilities and services—must be evaluated.

- 10 CFR §72.90(a) requires that site characteristics that may directly affect the safety or environmental impact of the ISFSI be investigated and assessed.

- 10 CFR §72.90(b) requires proposed sites for the ISFSI to be examined with respect to the frequency and severity of external natural and man-induced events that could affect the safe operation of the ISFSI.

- 10 CFR §72.90(c) requires design-basis external events to be determined for each combination of proposed site and proposed ISFSI design.

- 10 CFR §72.90(d) requires that proposed sites with design-basis external events for which adequate protection cannot be provided through ISFSI design shall be deemed unsuitable for the location of the ISFSI.

- 10 CFR §72.90(e) requires that, pursuant to Subpart A of Part 51 of Title 10, for each proposed site for an ISFSI, the potential for radiological and other environmental impacts on the region must be evaluated with due consideration of the characteristics of the population, including its distribution, and of the regional environs, including its historical and aesthetic values.

- 10 CFR §72.90(f) requires the facility to be sited so as to avoid to the extent possible the long-term and short-term adverse impacts associated with the occupancy and modification of floodplains.

- 10 CFR §72.92(a) requires that natural phenomena that may exist or that can occur in the region of a proposed site be identified and assessed according to their potential effects on the safe operation of the ISFSI. The important natural phenomena that affect the ISFSI design must be identified.

- 10 CFR §72.92(b) requires that records of the occurrence and severity of those important natural phenomena must be collected for the region and evaluated for reliability, accuracy, and completeness. The applicant shall retain these records until the license is issued.

- 10 CFR §72.92(c) requires that appropriate methods must be adopted for evaluating the design-basis external natural events based on the characteristics of the region and the current state of knowledge about such events.

- 10 CFR §72.98(a) requires that the regional extent of external phenomena, man-made or natural, that are used as a basis for the design of the ISFSI must be identified.

- 10 CFR §72.98(c) requires that those regions identified pursuant to paragraphs 10 CFR §72.98(a) and (b) be investigated as appropriate with respect to (1) the present and future character and the distribution of population, (2) consideration of present and projected future uses of land and water within the region, and (3) any special characteristics that may influence the potential consequences of a release of radioactive material during the operational lifetime of the ISFSI.

- 10 CFR 72.122(b) requires that (1) Structures, systems, and components important to safety must be designed to accommodate the effects of, and to be compatible with, site characteristics and environmental conditions associated with normal operation, maintenance, and testing of the ISFSI and to withstand postulated accidents. (2)(i) Structures, systems, and components important to safety must be designed to withstand the effects of natural phenomena such as earthquakes, tornadoes, lightning, hurricanes, floods, tsunami, and seiches, without impairing their capability to perform safety functions. The design bases for these structures, systems, and components must reflect (A) appropriate consideration of the most severe of the natural phenomena reported for the site and surrounding area, with appropriate margins to take into account the limitations of the data and the period of time in which the data have accumulated, and (B) appropriate combinations of the effects of normal and accident conditions and the effects of natural phenomena. (2)(ii) The ISFSI should also be designed to prevent massive collapse of building structures or the dropping of heavy objects as a result of building structural failure on the spent nuclear fuel or high-level waste or onto structures, systems, and components important to safety. (3) Capability must be provided for determining the intensity of natural phenomena that may occur for comparison with design bases of structures, systems, and components important to safety. (4) If the ISFSI is

located over an aquifer that is a major water resource, measures must be taken to preclude the transport of radioactive materials to the environment through this potential pathway.

2.1.3.1 Regional Climatology

Section 2.3.1, “Regional Climatology,” in the SAR briefly describes the regional climatology for the region surrounding the DCPP. The description is based on a summary of regional climatology from Section 2.3 of the DCPP FSAR update. Because the Diablo Canyon ISFSI site is within the DCPP site and only 0.35 km [0.22 mi] from the Unit 1 reactor building, the regional climatic conditions at the ISFSI site are the same as those identified at the DCPP.

Climate at the DCPP is characterized by moderate temperatures and precipitation, with small diurnal and seasonal changes. Dry conditions are prevalent from May through September. The rainy season is from October through April. The mean annual temperature at San Luis Obispo is 13 °C [55 °F], and the average yearly precipitation is approximately 56 cm [22 in].

The staff reviewed the description of the regional climate in the SAR and found it to be acceptable, because it is based largely on general information provided for the DCPP that is also applicable to the ISFSI site. The regulations at 10 CFR §72.40(c) indicate that a reevaluation of a site is not required when relevant information is covered under previous licensing actions, except where new information could alter the original site evaluation findings. The staff has previously accepted the regional climatology description for DCPP, and the staff has discovered no new information that might alter the relevance of the regional climatology description for the proposed ISFSI location. Therefore, the staff has determined that this information is acceptable for use in other sections of the SAR to develop the design bases of the Diablo Canyon ISFSI, perform additional safety analysis, and demonstrate compliance with the regulatory requirements of 10 CFR §72.90(a), §72.90(b), and §72.122(b) with respect to this topic.

2.1.3.2 Local Meteorology

Section 2.3.2 of the SAR, “Local Meteorology,” in the SAR describes and characterizes the local meteorology of the DCPP site. The description is based on a summary of local meteorological information from Section 2.3 of the DCPP FSAR update. Because the Diablo Canyon ISFSI site is within the DCPP site and is only 0.35 km [0.22 mi] from the Unit 1 reactor building, the local meteorological conditions at the ISFSI site are considered to be the same as those identified at the DCPP.

The ISFSI SAR indicates that the maximum recorded temperature for the site was 36 °C [97 °F] in October 1997 and that the minimum recorded temperature was just below 0 °C [32 °F] in December 1990. The mean annual temperature is approximately 13 °C [55 °F], based on measurement from the DCPP primary meteorological tower.

Solar insolation data collected by the California Polytechnic State University, Department of Water Resources, and cataloged in the California Irrigation Management Information System, are applicable to the Diablo Canyon ISFSI site. These data are measured at a location approximately 19.3 km [12 mi] northeast of the proposed ISFSI. For data collected between

1986 and 1999, the maximum measured insolation for a 24-hour period was 766 g-cal/cm^2 per day {371 W/m^2 [118 BTU/hr-ft^2]} and, for a 12-hour period, 754 g-cal/cm^2 per day {365 W/m^2 [116 BTU/hr-ft^2]}. The daily (24-hour) average for the period of record was 430 g-cal/cm^2 per day {208 W/m^2 [66 BTU/hr-ft^2]}.

The SAR also notes that the average annual precipitation at the DCPP is 40 cm [16 in]. The maximum daily precipitation is 8.33 cm [3.28 in]. The maximum yearly precipitation, recorded at San Luis Obispo, was 138.5 cm [54.53 in] in 1969. The highest wind gust recorded at the meteorological tower was 135 km/hr [84 mph]. The prevailing wind direction is from the northwest.

The staff reviewed the description of the local meteorology in the SAR and found that it has been adequately described and assessed. The staff found the description of the local meteorology in the SAR to be acceptable because it is based largely on general information provided for DCPP that is also applicable to the ISFSI site. The regulations at 10 CFR §72.40(c) indicate that a reevaluation of a site is not required when relevant information is covered under previous licensing actions, except where new information could alter the original site evaluation findings. The staff has previously accepted the local meteorology description for DCPP, and the staff has discovered no new information that might alter the relevance of the local meteorology description for the proposed ISFSI location. Therefore, the staff has determined that this information is acceptable for use in other sections of the SAR to develop the design bases of the Diablo Canyon ISFSI, perform additional safety analysis, and demonstrate compliance with the regulatory requirements of 10 CFR 72.92(a), 72.98(a), 72.98(c)(3), and 72.122(b) with respect to this topic.

2.1.3.3 Onsite Meteorological Measurement Program

Section 2.3.3 of the SAR, “Onsite Meteorological Measurement Program,” describes the onsite meteorological measurement program at the DCPP. PG&E noted that the onsite meteorological monitoring system supporting the DCPP will also be used to support the proposed ISFSI. The system consists of two independent subsystems and is designed to conform with NRC Regulatory Guide 1.23 (U.S. Nuclear Regulatory Commission, 1976).

The staff reviewed the description of the onsite meteorological measurement program in the SAR and found that it has been adequately described and assessed. The staff found the description of the onsite meteorological measurement program in the SAR to be acceptable because it is based largely on information previously provided for DCPP that is also applicable to the ISFSI site. The regulations at 10 CFR §72.40(c) indicate that a reevaluation of a site is not required when relevant information is covered under previous licensing actions, except where new information could alter the original site evaluation findings. The staff has previously accepted the onsite meteorological measurement program description for DCPP, and the staff has discovered no new information that might alter the relevance of the onsite meteorological measurement program description for the proposed ISFSI location. Therefore, the staff has determined that this information is acceptable for use in other sections of the SAR to develop the design bases of the Diablo Canyon ISFSI, perform additional safety analysis, and demonstrate compliance with the regulatory requirements of 10 CFR §72.24(a), §72.92(a), §72.98(a), §72.98(c)(3), and §72.122(b) with respect to this topic.

2.1.4 Surface Hydrology

The staff has reviewed the information presented in Section 2.4 of the SAR, “Surface Hydrology.” Subsections discussed in the following text include: (1) Hydrologic Description, (2) Floods, (3) Probable Maximum Flood (PMF) on Streams and Rivers, (4) Potential Dam Failures (seismically induced), (5) Probable Maximum Surge and Seiche Flooding, (6) Probable Maximum Tsunami Flooding, (7) Ice Flooding, (8) Flood Protection Requirements, and (9) Environmental Acceptance of Effluents. The staff reviewed the discussion on surface hydrology with respect to the following regulatory requirements:

- 10 CFR §72.24(a) requires a description and safety assessment of the site on which the ISFSI is to be located, with appropriate attention to the design bases for external events. Such assessment must contain an analysis and evaluation of the major structures, systems, and components of the ISFSI that bear on the suitability of the site when the ISFSI is operated at its design capacity. If the proposed ISFSI is to be located on the site of a nuclear power plant or other licensed facility, the potential interactions between the ISFSI and such other facility—including shared common utilities and services—must be evaluated.

- 10 CFR §72.90(a) requires that site characteristics that may directly affect the safety or environmental impact of the ISFSI be investigated and assessed.

- 10 CFR §72.90(b) requires that proposed sites for the ISFSI must be examined with respect to the frequency and severity of external natural and man-induced events that could affect the safe operation of the ISFSI.

- 10 CFR §72.90(c) requires that design-basis external events must be determined for each combination of proposed site and proposed ISFSI design.

- 10 CFR §72.90(d) requires that proposed sites with design-basis external events for which adequate protection cannot be provided through ISFSI design shall be deemed unsuitable for the location of the ISFSI.

- 10 CFR §72.90(e) requires that, pursuant to subpart A of Part 51 of CFR Title 10, for each proposed site for an ISFSI, the potential for radiological and other environmental impacts on the region must be evaluated with due consideration of the characteristics of the population, including its distribution, and of the regional environs, including its historical and aesthetic values.

- 10 CFR §72.90(f) requires the facility must be sited so as to avoid to the extent possible the long-term and short-term adverse impacts associated with the occupancy and modification of flood plains.

- 10 CFR §72.92(a) requires that natural phenomena that may exist or that can occur in the region of a proposed site must be identified and assessed according to their potential effects on the safe operation of the ISFSI. The important natural phenomena that affect the ISFSI design must be identified.

- 10 CFR §72.92(b) requires that records of the occurrence and severity of those important natural phenomena must be collected for the region and evaluated for reliability, accuracy, and completeness. The applicant shall retain these records until the license is issued.

- 10 CFR §72.92(c) requires that appropriate methods must be adopted for evaluating the design-basis external natural events based on the characteristics of the region and the current state of knowledge about such events.

- 10 CFR §72.98(a) requires that the regional extent of external phenomena, man-made or natural, that are used as a basis for the design of the ISFSI must be identified.

- 10 CFR §72.92(b) requires that records of the occurrence and severity of those important natural phenomena must be collected for the region and evaluated for reliability, accuracy, and completeness. The applicant shall retain these records until the license is issued.

- 10 CFR §72.98(c) requires that those regions identified pursuant to 10 CFR §72.98(a) and 10 CFR §72.98(b) must be investigated as appropriate with respect to (1) the present and future character and the distribution of population, (2) consideration of present and projected future uses of land and water within the region, and (3) any special characteristics that may influence the potential consequences of a release of radioactive material during the operational lifetime of the ISFSI.

- 10 CFR 72.122(b) requires that (1) Structures, systems, and components important to safety must be designed to accommodate the effects of, and to be compatible with, site characteristics and environmental conditions associated with normal operation, maintenance, and testing of the ISFSI and to withstand postulated accidents. (2)(i) Structures, systems, and components important to safety must be designed to withstand the effects of natural phenomena such as earthquakes, tornadoes, lightning, hurricanes, floods, tsunami, and seiches, without impairing their capability to perform safety functions. The design bases for these structures, systems, and components must reflect (A) appropriate consideration of the most severe of the natural phenomena reported for the site and surrounding area, with appropriate margins to take into account the limitations of the data and the period of time in which the data have accumulated, and (B) appropriate combinations of the effects of normal and accident conditions and the effects of natural phenomena. (2)(ii) The ISFSI should also be designed to prevent massive collapse of building structures or the dropping of heavy objects as a result of building structural failure on the spent nuclear fuel or high-level waste or onto structures, systems, and components important to safety. (3) Capability must be provided for determining the intensity of natural phenomena that may occur for comparison with design bases of structures, systems, and components important to safety. (4) If the ISFSI is located over an aquifer that is a major water resource, measures must be taken to preclude the transport of radioactive materials to the environment through this potential pathway.

2.1.4.1 Hydrologic Description

Section 2.4.1 of the SAR, "Hydrologic Description," describes and characterizes the surface hydrological conditions and features pertaining to the proposed ISFSI site. The applicant provides a general description and maps of the drainage basin in the region surrounding the proposed ISFSI site, summarizes the seasonal flow characteristics for Diablo Creek, and indicates sources of water supply for the DCPP. The applicant documents similar hydrologic information pertaining to the Diablo Canyon area in the DCPP FSAR update, and the applicant indicates that much of that information pertains also to the ISFSI location, because the hydrologic characteristics in the Diablo Canyon area do not vary significantly in the general vicinity of the ISFSI and power plant facilities.

The surface hydrologic conditions most relevant to the proposed ISFSI include those influencing storm run-offs within the Diablo Creek watershed and those influencing wave run-up caused by disturbances in the water level of the Pacific Ocean. The area encompassing the drainage basin for Diablo Creek is approximately 13 km^2 [5 mi^2]. The drainage basin is elongated roughly east-west, with highest elevation at 555 m [1,819 ft] above MSL. The maximum flow in Diablo Creek during the flood of January 18-25, 1969, has been estimated at 12,176 l/s [430 cfs]. Typical flows are described as being nearer the minimum flow value of 12.5 l/s [0.44 cfs].

The staff reviewed information concerning the surface hydrologic features of the Diablo Canyon vicinity, including information on the location, size, and flow rates of streams and other significant water features; topographic maps and watershed characteristics; relevant oceanographic and tidal data; proposed alterations of natural drainage features; and other applicable information. The staff also reviewed meteorological data significant to the characteristics and hazards associated with surface hydrology.

The staff reviewed the description of the surface hydrologic features in the SAR and found that these features have been adequately described and assessed. The staff found the description of the surface hydrologic features in the SAR to be acceptable because it is based largely on general information provided for DCPP that is also applicable to the ISFSI site. The regulations at 10 CFR §72.40(c) indicate that a reevaluation of a site is not required when relevant information is covered under previous licensing actions, except where new information could alter the original site evaluation findings. The staff has previously accepted the surface hydrologic features description for DCPP, and the staff has discovered no new information that might alter the relevance of the surface hydrologic features description for the proposed ISFSI location. Therefore, the staff has determined that this information is acceptable for use in other sections of the SAR to develop the design bases of the Diablo Canyon ISFSI, perform additional safety analysis, and demonstrate compliance with the regulatory requirements of 10 CFR §72.24(a), §72.90(a), §72.90(b), §72.90(c), §72.90(d), §72.90(f), §72.92(a), §72.92(b), §72.92(c), §72.98(a), §72.98(c)(3), and §72.122(b) with respect to this topic.

2.1.4.2 Floods

Section 2.4.2 of the SAR, "Floods," discusses the potential for and effects of floods at the proposed ISFSI site. The applicant identifies flood considerations from the DCPP FSAR update that are applicable to the proposed ISFSI location. The applicant flood design considerations

pertain primarily to local floods along Diablo Creek. PG&E states that the canyon confining Diablo Creek is more than sufficient to channel any conceivable flood without hazard to the proposed ISFSI. Additionally, any damming of water caused by channel blockage resulting from a landslide downstream of the ISFSI would not be sufficient to flood the ISFSI area, and there are no dams or natural features along Diablo Creek that would hinder run-off for a significant period of time. The applicant also indicates that run-off is efficiently drained at the proposed ISFSI site by the adjacent natural and constructed drainage features. Ponding caused by intense local precipitation can occur at the proposed ISFSI site, but the applicant indicates that such ponding would not be significant, would be temporary, and would not adversely impact ISFSI operation or spent nuclear fuel confinement. Similarly, the applicant notes that overflow of nearby constructed reservoirs would drain toward Diablo Creek and the Pacific Ocean and would not adversely impact the ISFSI.

The staff review of this information focused on the potential for and effects of locally intense precipitation at the proposed ISFSI. Section 2.1.4.3 of this SER discusses the staff review of information concerning the potential and effects of flooding along streams and rivers. Values for the probable maximum precipitation (PMP) for various time periods are provided in Table 2.4-1 of the DCPP FSAR update, with the 1-, 6-, and 24-hour PMPs reported as 10.9 cm [4.3 in], 23.1 cm [9.1 in], and 42.16 cm [16.6 in]. The staff also considered local and regional precipitation data, and historical flood data, as well as local grading and drainage provisions at the proposed ISFSI location.

The staff reviewed the description of floods resulting from locally intense precipitation in the SAR and found that flooding has been adequately described and assessed. The staff found the description in the SAR of floods resulting from locally intense precipitation to be acceptable because the description is based largely on information previously provided for DCPP that is also applicable to the ISFSI site. The regulations at 10 CFR §72.40(c) indicate that a reevaluation of a site is not required when relevant information is covered under previous licensing actions, except where new information could alter the original site evaluation findings. The staff has previously accepted the description of floods caused by locally intense precipitation for the DCPP, and the staff has discovered no new information that might alter the relevance of the description of floods caused by locally intense precipitation for the proposed ISFSI location. Therefore, the staff has determined that this information is acceptable for use in other sections of the SAR to develop the design bases of the Diablo Canyon ISFSI , perform additional safety analysis, and demonstrate compliance with the regulatory requirements of 10 CFR §72.24(a), §72.90(a), §72.90(b), §72.90(c), §72.90(d), §72.90(f), §72.92(a), §72.92(b), §72.92(c), §72.98(a), §72.98(c)(3), and §72.122(b) with respect to this topic.

2.1.4.3 Probable Maximum Flood on Streams and Rivers

Section 2.4.3 of the SAR, “Probable Maximum Flood (PMF) on Streams and Rivers,” discusses probable maximum flooding at the proposed ISFSI site caused by stream flooding. Diablo Creek is identified as the only significant channel within the drainage basin encompassing the proposed ISFSI site. In the DCPP FSAR update, the PMF upstream of DCPP is reported to have a peak discharge of approximately 195,386 l/s [6,900 cfs], with a total volume of approximately 5.3×10^6 m^3 [4,300 acre-ft] for a 24-hour storm. The applicant indicates that the drainage capacity of Diablo Creek, including the portion defined by the 3.1-m [10-ft] diameter culvert passing under the 500-kV switchyard (even if temporarily clogged) is sufficient to shed

the PMF volume directly into the Pacific Ocean, with no water retention. Even if the culvert becomes plugged indefinitely, the applicant indicates that local floodwaters would pass through water-diversion features with adequate freeboard around switchyards and would not cause any flooding of the proposed ISFSI.

The staff's review of this information focused on the potential for and the effects of stream overflows at the proposed ISFSI location. Section 2.1.4.2 of this SER discusses the staff's review of information concerning the potential and effects of flooding caused by locally intense precipitation. The value of PMF chosen for the proposed ISFSI is the same as that determined in the DCPP FSAR update and is based on shedding a 24-hour PMP that occurs over the entire Diablo Creek drainage basin, under the assumptions that all culverts along Diablo Creek are plugged and that antecedent ground wetness is such that flood run-off is high. The staff also considered local and regional precipitation data, historical flood data, characteristics of the Diablo Creek drainage basin, and the specific locations and elevations of interest for the proposed ISFSI.

The staff reviewed the description of the PMFs on streams and rivers in the SAR and found that the PMF has been adequately described and assessed. The staff found the description of the PMFs on streams and rivers caused by locally intense precipitation in the SAR to be acceptable because it is based largely on information previously provided for DCPP that is also applicable to the ISFSI site. The regulations at 10 CFR §72.40(c) indicate that a reevaluation of a site is not required when relevant information is covered under previous licensing actions, except where new information could alter the original site evaluation findings. The staff has previously accepted the description of the PMFs on streams and rivers caused by locally intense precipitation for DCPP, and the staff has discovered no new information that might alter the relevance of the description of the PMFs on streams and rivers caused by locally intense precipitation for the proposed ISFSI location. Therefore, the staff has determined that this information is acceptable for use in other sections of the SAR to develop the design bases of the Diablo Canyon ISFSI, perform additional safety analysis, and demonstrate compliance with the regulatory requirements of 10 CFR §72.24(a), §72.90(a), §72.90(b), §72.90(c), §72.90(d), §72.90(f), §72.92(a), §72.92(b), §72.92(c), §72.98(a), §72.98(c)(3), and §72.122(b) with respect to this topic.

2.1.4.4 Potential Dam Failures (Seismically Induced)

Section 2.4.4 of the SAR, "Potential Dam Failures (Seismically Induced)," discusses the potential for flooding at the proposed ISFSI site as a result of seismically induced dam failures. The applicant states that there are no dams in the watershed area of the proposed ISFSI and that any seismically induced failures outside the watershed area would not affect the ISFSI. Similar information from the applicant concerning potential seismically induced dam failures is provided in the DCPP FSAR update.

The staff reviewed the applicant's information concerning potential seismically induced dam failures and found it to be acceptable because no dams or reservoirs exist whose failure could cause flooding of the ISFSI. Thus, seismically induced dam failure is not considered to be a credible threat to the proposed ISFSI site. The regulations at 10 CFR §72.40(c) indicate that a reevaluation of a site is not required when relevant information is covered under previous licensing actions, except where new information could alter the original site evaluation findings.

The staff has previously accepted the assessment of seismically induced dam failures in the FSAR for the DCPP, and the staff has discovered no new information that would alter the finding that seismically induced dam failures do not pose a credible threat to the proposed ISFSI. Therefore, the staff has determined that this information is acceptable for use in other sections of the SAR to develop the design bases of the Diablo Canyon ISFSI, perform additional safety analysis, and demonstrate compliance with the regulatory requirements of 10 CFR §72.24(a), §72.90(a), §72.90(b), §72.90(c), §72.90(d), §72.90(f), §72.92(a), §72.92(b), §72.92(c), §72.98(a), §72.98(c)(3), and §72.122(b) with respect to this topic.

2.1.4.5 Probable Maximum Surge and Seiche Flooding

Section 2.4.5 of the SAR, “Probable Maximum Surge and Seiche Flooding,” discusses flooding from a maximum surge or seiche at the proposed ISFSI site. The applicant states that, because of the elevation of the ISFSI, there is no credible scenario that would create any flooding from a maximum surge or seiche. Additional information from the applicant concerning surge and seiche flooding is provided in the DCPP FSAR update.

The staff reviewed the applicant’s information concerning probable maximum surge and seiche flooding and found it to be acceptable because the ISFSI pad and transporter route (which bound the vertical extent of the overall ISFSI) are located at elevations that are significantly higher than the DCPP. The regulations at 10 CFR §72.40(c) indicate that a reevaluation of a site is not required when relevant information is covered under previous licensing actions, except where new information could alter the original site evaluation findings. The staff has previously accepted the DCPP analysis of probable maximum surge and seiche flooding and has discovered no new information in this review that alters the previous findings. The staff concludes that the probable maximum surge and seiche flooding do not pose credible threats to the proposed ISFSI. Therefore, the staff has determined that this information is acceptable for use in other sections of the SAR to develop the design bases of the Diablo Canyon ISFSI, perform additional safety analysis, and demonstrate compliance with the regulatory requirements of 10 CFR §72.24(a), §72.90(a), §72.90(b), §72.90(c), §72.90(d), §72.90(f), §72.92(a), §72.92(b), §72.92(c), §72.98(a), §72.98(c)(3), and §72.122(b) with respect to this topic.

2.1.4.6 Probable Maximum Tsunami Flooding

Section 2.4.6 of the SAR, “Probable Maximum Tsunami Flooding,” discusses probable maximum tsunami flooding at the proposed ISFSI site. The proposed ISFSI pad is to be located at an elevation of +94.5 m [+310 ft] relative to MSL, which is +95.3 m [+312.6 ft] relative to mean lower low water level. The lowest portion of the ISFSI cask transporter route is to be at an elevation of approximately 24.4 m [+80 ft] MSL. The applicant states that, because of the elevation of the ISFSI, a maximum tsunami would not cause any flooding to the ISFSI. The applicant cites data and analysis from the DCPP FSAR update as supporting bases for this statement. In the DCPP FSAR update, the maximum combined wave run-up from a distantly generated tsunami is reported as +9.1 m [30 ft] mean lower low water level, and the maximum combined wave run-up for near shore tsunamis is reported as +10.5 m [+34.6 ft] mean lower low water level.

PG&E indicates that little new offshore geologic or geophysical data for central coastal California have been gathered since the earlier studies conducted in the 1970s and 1980s. More information about the occurrence and generation of tsunamis in other parts of California and the world have recently become available. The SAR and responses to the staff's request for additional information (RAI) (Pacific Gas and Electric Company, 2002a) provide an overview of available new information, including recent worldwide tsunami data, recent results from tsunami inundation mapping studies for locations in California, and information concerning sub-aerial and submarine landslide potential along central coastal California. These documents provide the applicant's assessment of implications of that new information relative to the proposed ISFSI and to tsunami scenarios potentially affecting the Diablo Canyon site. Considering the new information, the applicant concludes that the run-up from tsunami scenarios would not exceed the DCPP design-basis tsunami and would be significantly below the Diablo Canyon ISFSI site (pad location) and the transporter route. The applicant also assessed the potential for tsunami scenarios to reactivate movement along the existing Patton Cove landslide, and concluded that tsunamis would not cause a major reactivation of the slide.

The staff reviewed the applicant's information concerning probable maximum tsunami flooding and found it to be acceptable because the ISFSI pad and transporter route (which bound the vertical extent of the overall ISFSI) are located at elevations significantly higher than the DCPP. The regulations at 10 CFR §72.40(c) indicate that a reevaluation of a site is not required when relevant information is covered under previous licensing actions, except where new information could alter the original site evaluation findings. The staff has previously accepted PG&E's analysis of probable maximum tsunami flooding for the DCPP, and has reviewed PG&E's assessment of more recent information on tsunamis. The staff agrees with the applicant's conclusion that this information does not alter the previous findings for the DCPP site. Therefore, the staff found that, relative to probable maximum tsunami flooding, the proposed ISFSI is adequately covered in accordance with the previous licensing actions for the DCPP. Therefore, the staff has determined that this information is acceptable for use in other sections of the SAR to develop the design bases of the Diablo Canyon ISFSI, perform additional safety analysis, and demonstrate compliance with the regulatory requirements of 10 CFR §72.24(a), §72.90(a), §72.90(b), §72.90(c), §72.90(d), §72.90(f), §72.92(a), §72.92(b), §72.92(c), §72.98(a), §72.98(c)(3), and §72.122(b) with respect to this topic.

2.1.4.7 Ice Flooding

Section 2.4.7 of the SAR, "Ice Flooding," discusses the potential for ice flooding at the proposed ISFSI site. The applicant indicates that flooding caused by ice melt events is not credible because of the mild climate and infrequency of freezing temperatures in the region.

In reviewing the SAR in regard to ice flooding, the staff considered regional climatology, local meteorology, historical meteorological data, surface hydrology, and other information relevant to the potential for ice-jam flood formation, wind-driven ice ridges, and ice-producing forces that may affect the proposed ISFSI site. The staff also reviewed information provided in the DCPP FSAR update regarding ice flooding.

The staff found the applicant's information regarding ice flooding to be acceptable because, based on current knowledge and data, the proposed ISFSI site is not subject to ice flooding hazards, and thus, ice flooding is not considered to be a credible threat to the proposed ISFSI site. Therefore, the staff has determined that this information is acceptable for use in other sections of the SAR to develop the design bases of the Diablo Canyon ISFSI, perform

additional safety analysis, and demonstrate compliance with the regulatory requirements of 10 CFR §72.24(a), §72.90(a), §72.90(b), §72.90(c), §72.90(d), §72.90(f), §72.92(a), §72.92(b), §72.92(c), §72.98(a), §72.98(c)(3), and §72.122(b) with respect to this topic.

2.1.4.8 Flood Protection Requirements

Section 2.4.8 of the SAR, "Flood Protection Requirements," discusses flood protection requirements for the proposed ISFSI. The applicant indicates that no cooling water canals, reservoirs, rivers, or streams are used in the operation of the proposed ISFSI. Because the ISFSI does not depend on such water sources, low flow conditions or diversion of water flow are not considered. The applicant states that there are no credible hydrological scenarios that could adversely affect the proposed ISFSI, and thus, other than normal engineering provisions for grading and drainage of storm run-off, no specialized hydrological considerations or flood protection requirements are needed.

The staff reviewed the applicant's information pertaining to flood protection requirements and found it to be acceptable because the analyses pertaining to surface hydrology and flooding indicate that enclosed areas of the proposed ISFSI [such as the Cask Transfer Facility (CTF)] are not subject to significant flooding or uncontrolled moisture intrusion, and exposed areas of the proposed ISFSI (such as the ISFSI pad) are not subject to levels of flooding that would adversely affect the ISFSI. In addition, ISFSI operating procedures will prohibit cask transport during any times of severe weather that could cause flooding along the transporter route.

2.1.4.9 Environmental Acceptance of Effluents

Section 2.4.9 of the SAR, "Environmental Acceptance of Effluents," discusses the potential for release and transport of radionuclides from the proposed ISFSI site via the hydrologic system. PG&E states that no radioactive wastes are created by the HI-STORM 100SA Cask System while in storage, while in transport, while at the CTF, or because of accidents. Because the ISFSI will not produce radioactive waste that can be incorporated into surface run-off, PG&E also asserts that surface run-off from the proposed ISFSI will have no radioactive contamination and will not adversely affect the surrounding ecosystem. Moreover, PG&E asserts that there is no public use of any surface waters or groundwater sources from the DCPP site. Thus, PG&E concludes that because no radioactive waste will be produced by the ISFSI and because surface and groundwater are not used by the public, detailed analysis of the acceptance of effluents via surface waters or groundwater as a result of ISFSI operation is not necessary.

The staff reviewed the applicant's information concerning the potential for release and transport of radionuclides from the proposed ISFSI site via the hydrologic system and found it to be acceptable. The applicant has shown that the multipurpose canister (MPCs) will contain the waste. Consequently, no waste will be incorporated into the surface or groundwater systems. Thus, the staff concludes that there will be no radioactive effluents. Therefore, the staff has determined that this information is acceptable for use in other sections of the SAR to develop the design bases of the Diablo Canyon ISFSI, perform additional safety analysis, and demonstrate compliance with the regulatory requirements of 10 CFR §72.24(a)(m), §72.90(a), §72.92(a)(c), §72.98(b), and §72.122(b) with respect to this topic.

2.1.5 Subsurface Hydrology

The staff has reviewed the information presented in Section 2.5, "Subsurface Hydrology," of the SAR. The staff reviewed the discussion on subsurface hydrology with respect to the following regulatory requirements:

- 10 CFR §72.98(a) requires that the regional extent of external phenomena, man-made or natural, that are used as a basis for the design of the ISFSI be identified.

- 10 CFR §72.98(b) requires that the potential regional impacts caused by the construction, operation, or decommissioning of the ISFSI must be identified. The extent of regional impacts must be determined on the basis of potential measurable effects on the population or the environment from ISFSI activities.

- 10 CFR §72.98(c) requires that those regions identified pursuant to 10 CFR 72.98(a) and (b) must be investigated as appropriate with respect to (1) the present and future character and distribution of population, (2) consideration of present and projected future uses of land and water within the region, and (3) any special characteristics that may influence the potential consequences of a release of radioactive material during the operational lifetime of the ISFSI.

- 10 CFR 72.122(b) requires that (1) Structures, systems, and components important to safety must be designed to accommodate the effects of, and to be compatible with, site characteristics and environmental conditions associated with normal operation, maintenance, and testing of the ISFSI and to withstand postulated accidents. (2)(i) Structures, systems, and components important to safety must be designed to withstand the effects of natural phenomena such as earthquakes, tornadoes, lightning, hurricanes, floods, tsunami, and seiches, without impairing their capability to perform safety functions. The design bases for these structures, systems, and components must reflect (A) appropriate consideration of the most severe of the natural phenomena reported for the site and surrounding area, with appropriate margins to take into account the limitations of the data and the period of time in which the data have accumulated, and (B) appropriate combinations of the effects of normal and accident conditions and the effects of natural phenomena. (2)(ii) The ISFSI should also be designed to prevent massive collapse of building structures or the dropping of heavy objects as a result of building structural failure on the spent nuclear fuel or high-level waste or onto structures, systems, and components important to safety. (3) Capability must be provided for determining the intensity of natural phenomena that may occur for comparison with design bases of structures, systems, and components important to safety. (4) If the ISFSI is located over an aquifer that is a major water resource, measures must be taken to preclude the transport of radioactive materials to the environment through this potential pathway.

Section 2.5 "Subsurface Hydrology," of the SAR discusses the characteristics of groundwater at the DCPP, ISFSI pad, and CTF sites. The descriptions are based on a summary of

groundwater characteristics provided in the DCPP FSAR. The DCPP FSAR is maintained through periodic revisions in accordance with 10 CFR §50.71(e). Because the Diablo Canyon ISFSI site is within the DCPP site and is only 0.35 km [0.22 mi] from the Unit 1 reactor building, the regional characteristics of groundwater conditions at the ISFSI site are the same as those at the DCPP.

The SAR concludes that the groundwater table beneath the entire DCPP site is influenced by sea level at the coastline, drainages such as Diablo Creek, and the topography of the hills east and southeast of the DCPP. Several monitoring wells at the DCPP also note perched groundwater, especially above impermeable clay beds in the Obispo Formation. Groundwater is found in the alluvium of Diablo Creek, within the terrace deposits along the coast, and in the fractured bedrock of the Obispo Formation.

The SAR notes that the groundwater table at the ISFSI and CTF site is largely controlled by Diablo Creek. The elevation of the creek adjacent to the ISFSI site is approximately 30 m [100 ft] above MSL. The elevation of the ISFSI pad site and CTF is approximately 95 m [310 ft] above MSL. Thus, there is 65 m [210 ft] of distance from the groundwater table to the ISFSI site. In addition, PG&E concludes in the SAR that the intermittent clay beds observed within the Obispo Formation would act to temporarily impede groundwater flow from the ISFSI pads to the water table.

PG&E also concludes in the SAR that groundwater quality or quantity will not be affected by the proposed ISFSI in any way. Construction and operation of the ISFSI, including the CTF, will not include use of groundwater. In addition, there is no public use of the local groundwater. The occurrences of perched groundwater were shown not to impact construction or operation of the ISFSI.

The staff found the description of the regional characteristics of groundwater in the SAR to be acceptable because it is based largely on general information provided for the DCPP that is also applicable to the ISFSI site. The regulations at 10 CFR §72.40(c) indicate that a reevaluation of a site is not required when relevant information is covered under previous licensing actions, except where new information could alter the original site evaluation findings. The staff has previously accepted the applicant's description of the subsurface hydrology for the DCPP, and the staff has discovered no new information that might alter the relevance of the description of the subsurface hydrology for the proposed ISFSI location. New information presented in the SAR concerning groundwater characteristics relative to the proposed ISFSI pad and CTF shows that groundwater characteristics at the site will not be adversely affected by the ISFSI, nor will groundwater conditions at the DCPP site impact construction and operation of the ISFSI. Therefore, the staff has determined that this information is acceptable for use in other sections of the SAR to develop the design bases of the ISFSI, perform additional safety analysis, and demonstrate compliance with regulatory requirements in 10 CFR §72.98(c)(2) and §72.122(b) with respect to this topic.

2.1.6 Geology and Seismology

Section 2.6 of the SAR, "Geology and Seismology," describes the geological and seismological setting of the DCPP. The staff reviewed the following sections of the SAR: 2.6.1, "Basic Geologic and Seismic Information;" 2.6.2, "Vibratory Ground Motion;" 2.6.3, "Surface Faulting;"

2.6.4, “Stability of Subsurface Materials;” and 2.6.5, “Slope Stability.” The staff reviewed the geology and seismology of the site with respect to the following regulatory requirements:

- 10 CFR §72.90(a) requires that site characteristics that may directly affect the safety or environmental impact of the ISFSI must be investigated and assessed.

- 10 CFR §72.90(b) requires that proposed sites for the ISFSI must be examined with respect to the frequency and severity of external natural and man-induced events that could affect the safe operation of the ISFSI.

- 10 CFR §72.90(c) requires that design basis external events must be determined for each combination of proposed site and proposed ISFSI design.

- 10 CFR §72.90(d) requires that proposed sites with design-basis external events for which adequate protection cannot be provided through ISFSI design shall be deemed unsuitable for the location of the ISFSI.

- 10 CFR §72.92(a) requires that natural phenomena that may exist or that can occur in the region of a proposed site be identified and assessed according to their potential effects on the safe operation of the ISFSI. The important natural phenomena that affect the ISFSI design must be identified.

- 10 CFR §72.92(b) requires that records of the occurrence and severity of those important natural phenomena be collected for the region and evaluated for reliability, accuracy, and completeness. The applicant shall retain these records until the license is issued.

- 10 CFR 72.92(c) requires that appropriate methods be adopted for evaluating the design-basis external natural events based on the characteristics of the region and the current state of knowledge about such events.

- 10 CFR §72.98(a) requires that the regional extent of external phenomena, man-made or natural, that are used as a basis for the design of the ISFSI be identified.

- 10 CFR §72.98(b) requires that the potential regional impact caused by the construction, operation, or decommissioning of the ISFSI be identified. The extent of regional impacts must be determined on the basis of potential measurable effects on the population or the environment from ISFSI activities.

- 10 CFR §72.98(c) requires that those regions identified pursuant to paragraphs 10 CFR §72.98(a) and (b) be investigated as appropriate with respect to (1) The present and future character and the distribution of population, (2) Consideration of present and projected future uses of land and water within the region, and (3) Any special characteristics that may influence the potential consequences of a release of radioactive material during the operational lifetime of the ISFSI.

- 10 CFR §72.102(b) requires that west of the Rocky Mountain Front (west of approximately 104° west longitude), and in other areas of known potential

seismic activity, seismicity will be evaluated by the techniques of Appendix A of Part 100 of this chapter. Sites that lie within the range of strong near-field ground motion from historical earthquakes on large capable faults should be avoided.

- 10 CFR §72.102(c) requires that sites other than bedrock sites must be evaluated for their liquefaction potential or other soil instability caused by vibratory ground motion.

- 10 CFR §72.102(d) requires that site-specific investigations and laboratory analyses show that soil conditions are adequate for the proposed foundation loading.

- 10 CFR §72.102(e) requires that in an evaluation of alternative sites, those that require a minimum of engineered provisions to correct site deficiencies are preferred. Sites with unstable geologic characteristics should be avoided.

- 10 CFR §72.102(f) requires that the design earthquake (DE) for use in the design of structures must be determined as follows: (1) for sites that have been evaluated in accordance with the criteria of Appendix A of 10 CFR Part 100, the DE must be equivalent to the safe shutdown earthquake (SSE) for a nuclear power plant, (2) regardless of the results of the investigations anywhere in the continental United States the DE must have a value for the horizontal ground motion of no less than 0.10g with the appropriate response spectrum.

- 10 CFR 72.122(b) requires that (1) Structures, systems, and components important to safety must be designed to accommodate the effects of, and to be compatible with, site characteristics and environmental conditions associated with normal operation, maintenance, and testing of the ISFSI and to withstand postulated accidents. (2)(i) Structures, systems, and components important to safety must be designed to withstand the effects of natural phenomena such as earthquakes, tornadoes, lightning, hurricanes, floods, tsunami, and seiches, without impairing their capability to perform safety functions. The design bases for these structures, systems, and components must reflect (A) appropriate consideration of the most severe of the natural phenomena reported for the site and surrounding area, with appropriate margins to take into account the limitations of the data and the period of time in which the data have accumulated, and (B) appropriate combinations of the effects of normal and accident conditions and the effects of natural phenomena. (2)(ii) The ISFSI should also be designed to prevent massive collapse of building structures or the dropping of heavy objects as a result of building structural failure on the spent nuclear fuel or high-level waste or onto structures, systems, and components important to safety. (3) Capability must be provided for determining the intensity of natural phenomena that may occur for comparison with design bases of structures, systems, and components important to safety. (4) If the ISFSI is located over an aquifer that is a major water resource, measures must be taken to preclude the transport of radioactive materials to the environment through this potential pathway.

2.1.6.1 Basic Geologic and Seismic Information

Basic geologic and seismic characteristics of the site and vicinity are presented in Section 2.6.1 of the SAR, “Basic Geological and Seismological Information.” Much of the information presented by PG&E is a summary of information developed for the DCPP FSAR and LTSP.

In the SAR, PG&E summarizes the additional analyses conducted at the site to support the applicability of the DCPP geologic and seismic information to the proposed ISFSI. In particular, PG&E examined the site geology from aerial photography, made direct field observations and updated site geologic maps and cross sections, gathered and interpreted subsurface data from boreholes and trenches beneath the ISFSI pad and CTF sites, and collected and interpreted seismic refraction data at the site to obtain compression and shear wave velocity profiles for that site-response analysis.

<u>Regional Geology and Tectonics</u>

Section 2.6.1.3 of the SAR summarizes the regional geology and tectonic setting of the DCPP site. Detailed discussion is provided in the DCPP FSAR update and in the Diablo Canyon LTSP.

The DCPP lies within the tectonic plate boundary between the Pacific and North American tectonic plates. Along this plate boundary, relative motions between the Pacific and North American plates are largely manifested as dextral strike-slip motion along the San Andreas strike-slip fault system. Although the main trace of the San Andreas fault lies 77 km [48 mi] to the east of the DCPP site, the width of the plate boundary encompasses much of coastal California from the main fault trace westward to the Santa Maria Basin, several kilometers offshore.

Tectonically, the plate boundary in central California consists of a melange of crustal blocks or exotic and suspect terranes. These terranes were accreted to North America during the latter half of the Mesozoic [170–65 Ma] and are currently undergoing complex dextral strike-slip and transpressional deformation. A large area of transpression occurs south of the DCPP where the San Andreas fault makes a left bend and interacts with the left lateral Garlock fault zone. The result is the uplifted Transverse ranges.

The DCPP site lies within what has been defined as the Los Osos domain, a triangular crustal block that is undergoing north-northeast contraction in response to right-lateral motion on the San Andreas fault to the east and clockwise rotation and uplift of the Western Transverse mountains to the south. The western edge of the Los Osos domain is delineated by the Hosgri fault, which is located offshore, approximately 5 km [3 mi] to the west of the DCPP. The Hosgri fault is the largest active fault near the DCPP. As a result of analyses documented in the Diablo Canyon LTSP, an earthquake with a moment magnitude ($\mathbf{M_W}$) of 7.2 on the Hosgri fault was determined to be the controlling earthquake in the development of the design-basis ground motion spectra for the DCPP.

There is some debate in the scientific literature whether the Hosgri fault zone is a nearly vertical strike-slip fault or a thrust or reverse fault that could dip eastward underneath the DCPP site. McIntosh, et al. (1991) use reflection seismic data (profile RU-3, which crosses the Hosgri fault

zone at a near 90° angle) to conclude that the Hosgri fault consists of blind thrusts and high-angle fault strands. McIntosh, et al. (1991) interpret the high-angle strands of the fault to have originally formed as basinward-dipping normal or transform faults in the early Miocene [22.7–16.6 Ma] transtensional stress regime, coinciding with the formation of the Santa Maria Basin. In the middle and late Miocene [16.6–5.3 Ma], the stress regime changed to one of compression, and the normal faults were rotated to their present orientation. According to the McIntosh, et al. (1991) interpretation, low-angle thrust faulting has occurred on the Hosgri fault since the late Miocene. McIntosh et al. (1991), however, also note right-lateral earthquake focal mechanisms along the Hosgri fault from Gawthrop (1975) and concede that strike-slip motion on the fault is permissible within their interpretation.

More recent scientific studies conclude that the Hosgri fault is a strike-slip fault with only minor vertical motion. For example, McLaren and Savage (2001) use earthquake data to show that the Hosgri fault is a strike-slip fault with 1–3 mm/yr [0.04–0.12 in/yr] of horizontal motion. In the McLaren and Savage (2001) interpretation, the transtensional strain is partitioned between strike-slip motion on the Hosgri fault and contraction and uplift of the Los Osos domain along a number of smaller reverse faults. Two of the reverse faults nearest the DCPP are the Los Osos fault and the Southwestern Boundary fault zone. These two faults bound a crustal block in the Los Osos domain called the Irish Hills subblock. Marine terraces near the DCPP document minor uplift of the Irish Hills subblock in the Late Quaternary [last 750,000 years] with vertical displacement rates of 0.02 and 0.4 mm/yr [0.008 and 0.16 in/yr], nearly an order of magnitude smaller than the horizontal slip rates on the Hosgri fault.

This tectonic interpretation summarized by McLaren and Savage (2001)—strike-slip faulting on Hosgri fault with smaller amounts of reverse faulting on other faults near the DCPP—is not substantially different from the tectonic interpretation used by PG&E in the development of the design-basis ground motion spectra for the DCPP and accepted by the NRC (U.S. Nuclear Regulatory Commission, 1991). Therefore, the staff concludes that there is no new information on the tectonic and seismic sources near the DCPP that would require PG&E to update the technical bases for earthquake sources in the existing 10 CFR Part 50 license for the DCPP. The new information on the regional geology and tectonics does not alter the previous site-evaluation findings.

<u>Site Stratigraphy</u>

Section 2.6.1.4 of the SAR summarizes the stratigraphy of the DCPP site, including the detailed stratigraphy beneath the proposed ISFSI pad and CTF. The detailed stratigraphic analyses of strata at the DCPP were conducted by PG&E and reported in the SAR to support two aspects of the Diablo Canyon ISFSI license application. First, PG&E used the information to show that the bedrock stratigraphy beneath the proposed ISFSI site was the same as the bedrock stratigraphy under the DCPP. This condition provides the technical bases for use of the existing design-basis ground motion spectra, originally developed for the DCPP license application. Second, PG&E used the stratigraphic information to develop appropriate rock property data for the slope-stability analyses of rocks in the hillslope above the proposed ISFSI pad site, the pad site itself, and the hillslopes along the transport route.

The SAR shows that both the DCPP and the proposed ISFSI site are underlain by the early-to-middle Miocene [22.7–11.2 Ma] Obispo Formation. This formation consists of fine-grained and massively bedded zeolitic tuff and a thick sequence of interbedded marine

sandstone, siltstone, and dolomite. The Obispo Formation has been subdivided in the SAR into three laterally interfingering units based on lithologic changes. These units are, from east to west across the site: (i) Unit Tof_a—a massively bedded diatomaceous siltstone and tuffaceous sandstone; (ii) Unit Tof_b—a medium to thickly bedded dolomite, dolomitic sandstone, dolomitic siltstone; and sandstone; and (iii) Unit Tof_c—a thin-to-medium bedded shale, claystone, and siltstone. The Obispo Formation is intruded in places by late Miocene diabase and gabbro sills and dikes. None of these intrusive rocks is known to be under the DCPP or the proposed ISFSI pad sites. Overlying the Obispo Formation are Quaternary [1.6 Ma to the present] deposits consisting of coastal marine terrace platforms, debris and colluvial fans, and alluvium. Quaternary deposits do not exist at the proposed ISFSI pad site because the proposed pad site is located in an area of extensive borrowing operations during plant construction in the 1970s. Quaternary deposits removed from this site were used by PG&E as fill at the adjacent switchyard location.

The proposed ISFSI pad and CTF will be founded on dolomitic sandstone and dolomite of Unit Tof_b. The proposed cutslope above the pad site is also underlain by dolomitic sandstone and dolomite. Thin clay beds, 5.0–10 cm [2–4 in] thick, are present within both the dolomitic sandstone and dolomite. The clay beds are composed of kaolinite, ganophyllite, and sepiolite. They are generally parallel to bedding but laterally discontinuous. Based on trenches and boring, some of the clay beds have been correlated across distances up to 60 m [200 ft], while other clay beds appear to pinch out laterally within 15–30 m [50–100 ft]. The clay beds are more common in the dolomite than in the dolomitic sandstone. Presence of the clay beds is important to slope stability analyses and they are discussed in greater detail in Section 2.1.6.4, “Stability of Subsurface Materials,” of this SER.

The staff’s review found the description of the basic geologic and seismic information in the SAR to be acceptable because it is based largely on information previously provided for the DCPP that is also applicable to the ISFSI site. The regulations at 10 CFR §72.40(c) indicate that a reevaluation of a site is not required when relevant information is covered under previous licensing actions, except where new information could alter the original site evaluation findings. The staff has discovered no new information that would alter the original site evaluation findings. In addition, the staff reviewed information specific to the proposed ISFSI site in Section 2.6.1.1 of the SAR and found it acceptable because the basic geologic and seismic characteristics of the site and vicinity have been adequately described in detail to allow investigation and evaluations of the seismic characteristics of the Diablo Canyon ISFSI. The staff has determined that this information is acceptable for use in other sections of the SAR to develop the design bases of the Diablo Canyon ISFSI, perform additional safety analysis, and demonstrate compliance with regulatory requirements in 10 CFR §72.92(a), §72.92(b), §72.98(a), §72.102(e), and §72.122(b) with respect to this topic.

2.1.6.2 Ground Vibration

Section 2.6.2 of the SAR, “Vibratory Ground Motions,” discusses the development of design bases associated with credible levels of vibratory ground motions that may be experienced at the proposed ISFSI site. The applicant provided (i) a general overview of the approach taken in developing design-basis ground motions for the proposed ISFSI; (ii) a summary of the existing licensing-basis ground motions for the DCPP; (iii) a comparison of factors influencing rock site response and ground motions at the DCPP power block site and the proposed ISFSI site; (iv) a

description of the derivation and results of rock design spectra for proposed ISFSI pads, casks, and cask anchorage, and CTF; (v) a description of the derivation and results of ISFSI long-period (ILP) rock design spectra and time histories used as input for analyses of ISFSI pad sliding, seismic stability of slopes, and transporter seismic stability; and (vi) a description of soil site response along the transporter route and ground motion input used for the analysis of transporter stability.

The applicant cites 10 CFR 72.102(f) as the basis for determining ISFSI DE ground motions and indicates that seismic analyses for the proposed ISFSI use ground motions that meet or exceed DCPP ground motions based on SSE criteria of 10 CFR Part 100, Appendix A. The approach followed by the applicant consisted of: (i) using the existing DCPP SSE ground motions; (ii) confirming the applicability of DCPP ground motions to the ISFSI site by showing similarity in site conditions and other factors influencing ground-shaking hazard; (iii) determining ground response spectra at longer vibration periods than those developed for the DCPP, accounting for near-source effects such as directivity; and (iv) developing spectra-compatible time histories for use in analyses and design.

The applicant indicates that DCPP design-basis ground motions are discussed in the DCPP FSAR update and notes the following three design spectra relevant to DCPP, the DE, the double design earthquake (DDE), and the Hosgri earthquake (HE). The existing seismic qualification basis for the DCPP (applicable also to future additions and modifications at the plant) consists of the DE and DDE (used for original design at DCPP) and the HE (used for seismic re-evaluation at DCPP) combined with the respective analytical methods, acceptance criteria, and conditions defining their implementation. In addition to these design-basis ground motions, response spectra have been developed as part of the LTSP for use in verifying the adequacy of seismic margins of specific structures, systems, and components at DCPP (Pacific Gas and Electric Company, 1988). The applicant uses the Diablo Canyon LTSP spectra as an additional basis for seismic analysis and design of the proposed ISFSI.

For the DCPP and proposed ISFSI locations, the applicant compares site conditions and distances to the seismic source having dominant contribution to local ground-shaking hazard. In comparing site conditions, the applicant notes that the DCPP (power block) and the proposed ISFSI are both sited within the same continuous, thick sequence of sandstone and dolomite beds defining a single unit of geologic formation. Additionally, the applicant considers shear-wave velocity profiles for both sites and finds that shear-wave velocities at the ISFSI site are within the range of values obtained for the DCPP site. The applicant states that both sites can be classified as rock having similar ranges of shear-wave velocities. In comparing source-to-site distances, the applicant cites results of the Diablo Canyon LTSP seismic hazard analysis that indicated the Hosgri fault zone, at a distance of 4.5 km [2.8 mi] from DCPP, to be the controlling seismic source. The applicant notes that the ISFSI is just slightly farther, 245 to 366 m [800 to 1,200 ft], from the Hosgri fault zone than DCPP and states that the distances to the controlling seismic source are essentially the same for both locations. Based on similarity in site conditions and distance to the controlling seismic source, the applicant concludes that the DCPP ground motions are applicable to the ISFSI design.

For analysis and design of the ISFSI pads, casks, and cask anchorage, and the CTF, the applicant notes that the DCPP spectra for the DE, DDE, HE, and LTSP are applicable. These spectra are defined, up to vibration periods of 1.0 second (DE), 1.0 second (DDE), 0.8 second (HE), and 2.0 seconds (LTSP). Details regarding design criteria for these ground motions are

evaluated in Chapters 4 and 5 of this SER. For design of the ISFSI cask anchorage, the HE spectrum is used for vibration periods up to 0.8 second (i.e., the longest period for which the HE spectrum is defined), and the LTSP is used for vibration periods up to 2.0 seconds. Using the spectral-matching approach, time histories were developed consistent with the resulting (composite) spectrum. The Standard Review Plan criteria of Section 3.7.1 of NUREG–0800 (U.S. Nuclear Regulatory Commission, 1981) were used in deriving the spectral match, with the exception of the requirement for a minimum power spectral density for an NRC Regulatory Guide 1.60 spectral shape, which the applicant determines not to be appropriate for use with the HE or LTSP spectra. The applicant states that the objective of a minimum power spectral density is met by requiring the spectrum of each time history to be less than 30 percent above and 10 percent below the target spectrum.

For analysis and design pertaining to ISFSI pad sliding, slope stability, and transporter stability, cases of interest include vibration periods beyond 2.0 seconds, and the applicant thus developed the ILP spectra and associated time histories for use in these cases. The applicant indicates that ILP spectra represent 84th-percentile values of horizontal and vertical spectra that extend out to a vibration period of 10 seconds. In developing ILP spectra and associated time histories, the applicant addresses near-source effects associated with fault rupture directivity and tectonic deformation (fault fling). The applicant summarizes various considerations and assumptions incorporated into the development of ILP horizontal and vertical spectra. These assumptions and considerations include (i) assuming strike-slip faulting as the dominant slip mechanism on the Hosgri fault; (ii) identifying and using a rupture scenario with maximum directivity effects; (iii) using near-source models that incorporate directivity effects; (iv) defining the 5-percent-damped ILP horizontal and vertical spectra, for periods less than 2 seconds, as the envelope of the DDE, HE, and LTSP spectra; (v) using the 84th-percentile spectral shape to extrapolate the enveloped horizontal spectrum over the period range of 2 to 10 seconds; (vi) using a conservative vertical-to-horizontal ratio of 2/3 to extrapolate the enveloped vertical spectrum over the period range of 2 to 10 seconds; (vii) increasing the 5-percent damped horizontal and vertical spectra to ensure that they envelope corresponding HE spectra at 4- and 7-percent damping; (viii) increasing the fault-normal spectrum, over the period range of 0.5 to 3.0 seconds, to account for directivity effects near 1-second period for earthquakes having magnitude less than 7.2; and (ix) using an 84th-percentile ground-motion model of the effects of tectonic fling on the fault-parallel horizontal spectrum. The applicant provides plots of ILP ground motions, including results for fault-normal and fault-parallel spectra.

The applicant develops five sets of time histories compatible with the ILP ground-motion spectra. Representative empirical motions were used as starting bases for the spectral matching procedure. Although the Standard Review Plan spectral matching requirements of Section 7.1 of NUREG–0800 recommend the use of at least 75 frequencies for matching, the applicant used 104 frequencies as an enhancement to cover the broader period range. The requirements that no more than five ordinate frequencies fall below the (5-percent-damped) target spectrum, and no ordinates fall below 0.9 times the target spectrum, were applied. The average spectra for 2-, 4-, 5-, and 7-percent damping, produced from the five sets of time histories, enveloped target spectra for corresponding damping levels.

For analysis of transporter stability, the applicant considers the effects of soil site response on ground motions along the transporter route. The applicant notes that approximately one-third of the transporter route is founded on the same bedrock unit as the DCPP and the ISFSI. Hence, the ILP ground motions are considered applicable to that portion of the route. For the

remaining two-thirds of the transporter route that crosses over surficial deposits, the applicant considers the effects of site response. The applicant performed soil response analyses at selected locations, with the finding that surface peak ground acceleration (PGA) values can be amplified by a factor of approximately 1.5 to 2.0 times the PGA value on bedrock. The applicant does not make any direct comparison of spectral results but suggests that the ILP motions for bedrock are also applicable to transporter stability analysis for the portions of the route on surficial deposits. Additionally, the applicant performed analyses that indicate that the transporter would remain stable when experiencing accelerations twice the ILP accelerations. The applicant further evaluated the risk of scenarios where an earthquake produces twice ILP ground motions coincident with a transport operation and concluded that such scenarios are not credible (i.e., they have an annual frequency less than 10^{-7}/yr).

In reviewing the applicant's analysis of vibratory ground motions, the staff considered factors related to the principal elements of seismic hazard analysis: (i) seismotectonic modeling; (ii) ground-motion prediction and near-source wave propagation; and (iii) dynamic site response. Documentation and data from the DCPP FSAR update and the Diablo Canyon LTSP seismic hazard study were reviewed, including information on (i) historical seismicity; (ii) seismogenic faults and area sources; (iii) parameters (e.g., activity rates, slip rates, b-values, maximum magnitudes) derived from seismological, geological, and geophysical investigations; (iv) applicable ground-motion relations, including models that incorporate directivity effects; (v) empirical time histories, and methods for developing time histories that match target spectra; (vi) controlling seismic sources and earthquake scenarios; and (vii) geotechnical parameters related to site response. The staff also considered new information concerning current data and available methods developed since the time of the Diablo Canyon LTSP report.

The staff's review of the applicant's information found it to be acceptable because, where applicable, it makes use of ground motions that meet or exceed the SSE ground motions for DCPP that have already been accepted by the NRC. The regulations at 10 CFR §72.40(c) indicate that a reevaluation of a site is not required when relevant information is covered under previous licensing actions, except where new information could alter the original site evaluation findings. The NRC has previously accepted PG&E's analyses of vibratory ground motions in its previous determination that the Diablo Canyon LTSP met the applicable license condition for the DCPP Unit 1 operating license. The staff has discovered no new information that alters the applicability of the current DCPP licensing basis for vibratory ground motions. Furthermore, the staff found that, in verifying the applicability of DCPP ground motions to the proposed ISFSI, the applicant has used parameters and methods of evaluation consistent with the current state of knowledge. The staff notes that, in extending the existing DCPP motions to longer vibration periods, for the purpose of developing the ILP spectra, the applicant has used an approximate approach. The applicant has nonetheless incorporated assumptions and criteria (e.g., 84th-percentile ground motions, near-source scenario that produces the maximum effects of directivity, enveloping of spectra, and other factors) that are clearly conservative and consistent with the requirements of 10 CFR Part 100, Appendix A. Therefore, the staff concludes that the ILP spectra are acceptable for use in ISFSI design analyses. The staff has determined that this information is acceptable for use in other sections of the SAR to develop the design bases of the Diablo Canyon ISFSI, perform additional safety analysis, and demonstrate compliance with regulatory requirements in 10 CFR §72.92(a), §72.92(b), §72.102(b), §72.102(e), and §72.122(b) with respect to this topic.

2.1.6.3 Surface Faulting

The description and characterization of surface faulting at the proposed Diablo Canyon ISFSI site are provided in Section 2.6.3, "Surface Faulting," of the SAR. Much of the information presented by PG&E is a summary of information developed for the DCPP FSAR and LTSP. In addition, potentially active faults at the DCPP site and the surrounding region were also identified and characterized by PG&E in the LTSP. From the geological and geophysical studies, PG&E showed that there is no evidence for capable faults at the DCPP site. Detailed mapping of marine terraces coupled with paleoseimologic studies (e.g., trenches, test pits, and boreholes) summarized in the SAR show that there was no significant active faulting at the DCPP site in the last 120,000 years.

The principal geologic structure at the DCPP site is the Pismo syncline. This structure folds strata of the Miocene [22.7 to 5.3 Ma] Monterey and Obispo and Pliocene [5.3 to 1.6 Ma] Pismo formations. Folding occurred in the late Pliocene or earliest Quaternary [1.6 Ma to the present]. Maps of Quaternary marine terraces across the fold axis show continuity such that folding and associated faulting must have ended at least 500,000 years ago. Thus, in the present tectonic regime, the Irish Hills subblock appears to be uplifting *en masse* without additional internal deformation.

Excavation of trenches near the proposed ISFSI site revealed minor bedrock faults with displacements up to 3 m [10 ft]. The faults are similar to minor bedrock faults encountered beneath the DCPP foundation and to bedrock faults found throughout the Obispo and Monterey formations in the Irish Hill subblock. The SAR concludes that development of these bedrock faults is either related to the growth of the Pismo syncline or to intrusion of the diabase into the Obispo formation. Both folding and intrusion have been demonstrated to have occurred in the Miocene [before 5.3 Ma].

The staff found the description of surface faulting in the SAR to be acceptable because it is based in part on information previously provided for the DCPP that is also applicable to the ISFSI site. The regulations at 10 CFR §72.40(c) indicate that a reevaluation of a site is not required when relevant information is covered under previous licensing actions, except where new information could alter the original site evaluation findings. Information concerning the potential surface faulting hazard that is specific to the proposed ISFSI pad and CTF shows that surface faulting at the site is not a credible hazard and will not adversely impact the ISFSI nor will surface faulting at the DCPP site impact construction and operation of the ISFSI. Therefore, the staff concludes that this information is also acceptable for use in other sections of the SAR to develop the design bases of the Diablo Canyon, perform additional safety analysis, and demonstrate compliance with the regulatory requirements of 10 CFR §72.90(b), §72.90(c), §72.90(d) §72.92(a), §72.92(b), §72.92(c), 72.98(b), 72.98(c)(3), and 72.122(b) with respect to this topic.

2.1.6.4 Stability of Subsurface Materials

The staff has reviewed the applicant information provided in SAR Sections 2.6.1, "Geologic, Seismologic and Geotechnical Investigations;" 2.6.4, "Stability of Subsurface Materials;" 2.6.5, "Slope Stability;" 4.1, "Location and Layout;" 4.2, "Storage Systems;" and the supporting calculations and data reports (provided as attachments or references in the SAR). The staff

also reviewed documents submitted by the applicant in response to staff RAIs (Pacific Gas and Electric Company, 2003).

Geotechnical Site Characterization

The following geotechnical and geological studies were performed at the site by the applicant to obtain information needed for the design of the ISFSI: (i) Surface geologic mapping; (ii) continuous rock coring, down-hole velocity measurements, and caliper and televiewer borehole logging; (iii) shallow seismic refraction; (iv) trenching, *in-situ* measurement of rock-discontinuity properties, and sampling of clay beds exposed in the trenches; and (v) laboratory testing of intact rock and clay-bed samples. The studies are documented in PG&E Calculation 52.27.100.731, “Analysis of Bedrock Stratigraphy and Geologic Structure at the DCPP ISFSI Site” (also cited as GEO.DCPP.01.21, Revision 2). Based on these studies, the applicant concluded that the ISFSI site and the hill slope to the south and southeast of the site are underlain by a thick sequence of sandstone and dolomite, within which there are several discontinuous intercalations of clay beds and friable rock. The hill slope above the ISFSI site is underlain by dolomite, but the ISFSI site is underlain by sandstone. The bedding planes in the sandstone and dolomite are generally tight and bonded. The other geologic structures in the bedrock include folds, faults, joints, and fractures.

The staff accepts the applicant’s description of the geologic structure and stratigraphy of the site (see Section 2.1.6.1, “Basic Geology and Seismic Information,” of this SER). The geologic features of the site essential for assessing the stability of the subsurface materials are (i) the clay beds, because the clay-bed material is significantly weaker than the surrounding rock; and (ii) the rock discontinuities, specifically joints and fractures.

Characterization of the Clay Beds

The geometry (i.e., thickness, lateral persistence, and structural attitude) and engineering properties (i.e., compressibility and shear strength) of the clay beds are essential to the stability of the subsurface materials. The applicant’s analysis of trench and road-cut exposures and borehole encounters of the clay beds indicates that the clay beds have a maximum thickness of approximately 10 cm [4 in] and lateral persistence of approximately a few tens of meters (tens to a few hundred feet). The effective lateral persistence of the clay beds (i.e., the lateral distance over which a slip on a clay bed would not encounter any rock-to-rock contact) is likely smaller than the estimated persistence because rock-to-rock contacts occur through clay beds that have a thickness smaller than approximately 0.64 cm [0.25 in].

The compressibility of the clay beds is not important because the thinness of the clay beds and the fact that the clay is overconsolidated (through previous erosion and excavation of the overburden material) imply that any subsequent compression of the clay beds would be small and of no significance to the stability of the subsurface materials. The applicant’s evaluation of the shear strength of the clay beds through laboratory testing is documented in SAR Section 2.6.5.1.2.3, “Material Properties,” and in PG&E Data Report G, “Soil Laboratory Test Data.” Three types of laboratory shear-strength tests were performed, namely, triaxial compression, monotonic direct shear, and cyclic direct shear. The applicant lumped the results from the three types of tests (SAR Figure 2.6-50) to obtain Eq. (2-1) for the shear strength of the clay bed:

$$\tau_f = 38.3 + \sigma_{fc} \tan(15°) \tag{2-1}$$

where τ_f is the total strength (kPa) and σ_{fc} is the overburden pressure {kPa [1 kPa ≈ 21 psf]}. Because the clay-bed thickness is small and the clay-bed material is much weaker than the overlying and underlying rocks, the stress conditions during any potential failure of the clay bed are appropriately represented by the direct-shear test loading conditions but not by the triaxial compression test. The monotonic direct-shear test represents the behavior of the clay under long-term static loading conditions, whereas the cyclic direct shear represents the behavior during seismic loading conditions. Lumping the three sets of data, therefore, is inappropriate. It can be shown by separating the applicant's data into three sets (each set consisting of data from one type of test) that the cyclic direct-shear data would be appropriately represented by Eq. (2-1), but the following equation is more appropriate for representing the monotonic direct-shear data:

$$\tau_f = 143.6 + \sigma_{fc} \tan(6°) \tag{2-2}$$

Equation (2-1) may be used for the analysis of stability during seismic loading conditions, but Eq. (2-2) is more appropriate for long-term static stability analyses. An examination of the two equations indicates that using Eq. (2-1) for static stability analysis would result in overestimating the applicable shear strength of the clay bed for values of overburden pressure greater than 0.65 MPa [13.5 ksf] such as for clay beds at depths greater than approximately 29.4 m [96.4 ft].

The staff accepts the applicant's conclusion that the geometry of a clay bed can be approximately represented by a thin horizontal bedding plane that extends laterally approximately 30 to 60 m [100 to 200 ft]. The staff also accepts that the shear strength of a clay bed under seismic-loading conditions is approximately represented by Eq. (2-1). As explained subsequently in "Stability Against Bearing-Capacity Failure Under Dynamic Loading," the clay beds need not be considered in the evaluation of static-bearing capacity of the foundation because the foundation loading under static conditions is vertical and would not induce any shear stress in the horizontal clay beds. The difference between Eqs. (2-1) and (2-2), therefore, has no impact on foundation-stability evaluations. Furthermore, any uncertainty introduced by using Eq. (2-1) instead of Eq. (2-2) to evaluate the static stability of slopes would be adequately accounted for by the safety factor required to demonstrate stability under static conditions. The staff concludes that the applicant characterization of the clay beds is based on accepted techniques. Therefore, the information presented is adequate for use in other sections of the SAR to perform additional safety analyses and demonstrate compliance with regulatory requirements in 10 CFR §72.90(a), §72.90(b), §72.90(c), §72.90(d), §72.92(a), §72.92(b), §72.92(c) §72.102(c), §72.102(d) and 72.122(b).

Characterization of Rock Discontinuities and the Discontinuous Rock Mass

Rock Discontinuities: The *in-situ* friction angle of the discontinuities is estimated using Barton's equation (Barton and Choubey, 1977) in PG&E Calculation 52.27.100.730, "Development of Strength Envelopes for Shallow Discontinuities at DCPP ISFSI Using Barton's Equations" (also cited as GEO.DCPP.01.20). At the slope face, the stress-relieved rock tends to dilate along preexisting discontinuities as asperities on the joint surfaces override one another. The nonlinear shear strength in Barton's joint model is dependent on the joint roughness coefficient

(JRC), joint compressive strength (JCS), and a base friction angle. The JRC was estimated from field and laboratory assessments either by measurement or by comparison with a standard roughness profile (Barton and Choubey, 1977). The JCS was assumed to be 25 percent of the unconfined compressive strength of intact rock. The analysis of laboratory test data of unconfined compressive strength for dolomite and sandstone is given in PG&E Calculations 52.27.100.727, "Determination of Mean and Standard Deviation of Unconfined Compressive Strength for Hard Rock at DCPP ISFSI, Based on Laboratory Tests" (also cited as GEO.DCPP.01.17). The mean and standard deviations of JRC and JCS were evaluated for joint, bedding planes, and faults in dolomite and sandstone. The basic friction angle was estimated from laboratory direct-shear tests as described in PG&E Calculation 52.27.100.728, "Determination of Basic Friction Angle Along Rock Discontinuities at DCPP ISFSI, Based on Laboratory Tests" (also cited as GEO.DCPP.01.18). Shear strength failure envelopes were developed for joints, bedding planes, and faults in dolomite and sandstone for a range of JCS, JRC, and base friction angles varied about their mean values. The *in-situ* friction angle was determined from the inclination of the tangent line on the shear-strength failure envelope at the midpoint of the normal stress range of 0–0.15 MPa [21.75 psi]. The normal stress on the wedge surfaces is not expected to exceed approximately 0.2 MPa [29 psi] considering the height of the slope {52.3 ft [5.9 m]}, density of the rock {140 pcf [0.022 MN/m^3]} and average orientation of the discontinuities. The range of normal stress used for the analysis is, therefore, consistent with the applicable normal stress at the site. The *in-situ* friction angles varied between 17.5–54° for dolomite and 16–46° for sandstone. The average friction angle of 34° for dolomite or 30° for sandstone is higher than the base friction angle of 28°, which is expected considering the effect of surface roughness on the shear strength of discontinuities (e.g., Barton and Choubey, 1977).

The staff concludes that the applicant's evaluation of the *in-situ* strength of rock discontinuities is based on standard methods (Hoek, 2000a; Barton and Choubey, 1977). Therefore, the information presented is adequate for use in other sections of the SAR to perform additional safety analyses and demonstrate compliance with regulatory requirements in 10 CFR §72.90(a), §72.90(b), §72.90(c), §72.90(d) §72.92(a), §72.92(b), §72.92(c) §72.102(c), §72.102(d), and 72.122(b).

<u>*Strength of the Discontinuous Rock Mass*</u>: The strength of the discontinuous rock mass modeled as a continuum is needed to evaluate the stability of the rock mass against potential failure modes for which the rock-mass behavior can be considered as similar to the behavior of a homogeneous continuum. The staff evaluated the applicant's estimation of rock-mass strength at the ISFSI pad site presented in PG&E Calculation 52.27.100.729, "Development of Strength Envelopes for Jointed Rock Mass at DCPP ISFSI Using Hoek-Brown Equation," (also cited as GEO.DCPP.01.19) and supporting documents. The applicant used the Hoek-Brown criterion (Hoek, 2000b) to develop the shear strength envelopes and estimate the shear strength parameters of the jointed rock mass in dolomite and sandstone. The rock-mass strength was utilized in foundation design for the ISFSI pads and the CTF and the analysis of slopes above the ISFSI pads.

The Hoek-Brown criterion defines a nonlinear empirical relationship between principal stresses associated with failure of moderately to heavily jointed rock mass. The failure criterion considers the characteristics of the intact rock and the geologic structures and surface conditions of discontinuities. The rock-mass strength is estimated using two intact-rock

strength parameters, namely, uniaxial compressive strength, $\boldsymbol{\sigma}_{ci}$, and material index, $\boldsymbol{m}_i$, and one rock-mass quality parameter referred to as the Geological Strength Index (GSI).

The unconfined compressive strengths of dolomite and sandstone were evaluated from laboratory tests on intact rock as discussed in PG&E Calculation "Determination of Mean and Standard Deviation of Unconfined Compressive Strengths for Hard Rock at DCPP ISFSI, Based on Laboratory Tests" (also cited as GEO.DCPP.01.17). The average value of $\boldsymbol{\sigma}_{ci}$ was determined to be 31.1 MPa [4,517 psi] with standard deviation of 14.7 MPa [2,131 psi] for dolomite based on test results of 12 samples, and 21.8 MPa [3,165 psi] with standard deviation of 9.3 MPa [1,348.9 psi] for sandstone based on test results of five samples.

The applicant estimated GSI and $\boldsymbol{m}_i$, from visual observation by individual members of a team consisting of two to three field geologists. The value of $\boldsymbol{m}_i$, was assigned by comparing the mineralogical and sedimentological characteristics of the exposed rock in exploratory trenches against the estimated values of $\boldsymbol{m}_i$ given by Hoek (2000b) for similar rock types. The means and standard deviations of $\boldsymbol{m}_i$ values for dolomite were estimated as 15.4 and 2.0, and for sandstone, 17.8 and 1.0. The Hoek-Brown constant, $\boldsymbol{m}_i$, is usually determined by statistical analysis of sets of triaxial tests conducted on intact rock core samples (Hoek, 2000b). The applicant's estimate for $\boldsymbol{m}_i$ for sandstone compares well with the estimated value of 14.3 from laboratory tests (Hoek and Brown, 1980). However, the applicant's estimate for dolomite is significantly larger than the laboratory-determined value of 6.8 (Hoek and Brown, 1980). An $\boldsymbol{m}_i$ value of 6.8 for dolomite, compared with the value of 15.4 used by the applicant, implies that the rock-mass friction angle calculated by the applicant (considering the values of GSI discussed subsequently) could be smaller by approximately 5°. This uncertainty in the rock-mass friction angle is acceptable as will be discussed subsequently in this section.

The applicant estimated the GSI by visual inspection of the rock exposures in the exploratory trenches and road cuts. The GSI was estimated by comparing the rock-mass structure and discontinuity surface conditions against the GSI classification developed by Hoek (2000b). The means and standard deviations of GSI for dolomite were 55.7 and 9.3, and for sandstone, 64.8 and 3.1.

The applicant performed several spreadsheet calculations and developed nonlinear shear strength failure curves with different combinations of mean, lower-bound, and upper-bound values of the three parameters. The lower-bound and upper-bound values are smaller and larger than the mean value by one standard deviation.

The applicant considered a straight-line failure envelope, with a friction angle of 50° and cohesion of zero, which represented a lower bound for all the calculated rock-mass shear strength curves for dolomite and sandstone. The applicant indicated that the rock-mass shear and compressive strengths are significantly smaller than the uniaxial compressive strength of intact rock determined from laboratory test data. The estimated rock-mass shear strength shows a zero uniaxial compressive strength and a steeply rising shear-strength envelope. The applicant did not provide any triaxial test data for the ISFSI-site intact rock. The staff, therefore, used intact-rock shear strength envelopes available in the literature (e.g., Goodman, 1983) in combination with the applicant's unconfined compressive strength data to assess the applicant's rock-mass strength information. Using a friction angle of 37.2° and unconfined

compressive strength of 21.8 MPa [3,165 psi] for sandstone, a cohesion of 5.4 MPa [783 psi] was determined using the relation given in Goodman (1983)

$$q_u = 2c \tan\left(45 + 0.5\phi\right) \tag{2-3}$$

where q_u is the unconfined compressive strength, c is the cohesion, and ϕ is the friction angle. Similarly, using a friction angle of 35.5°(Goodman, 1983) and unconfined compressive strength of 31.1 MPa [4,517 psi] for dolomite, a cohesion of 8.12 MPa [1,177 psi] was calculated for intact dolomite rock. The straight-line failure envelope of the rock mass defined by a friction angle of 50° and zero cohesion is compared with the resulting intact-rock strength envelopes for sandstone and dolomite defined by the c and φ values. The rock-mass strength envelope would intersect the intact-rock envelopes at a normal stress of 12.5 MPa [1,813 psi] for sandstone and 16.7 MPa [2,422 psi] for dolomite, which suggests that the rock-mass strength obtained using the friction angle of 50° would not exceed the strength of intact sandstone for values of normal stress smaller than 12.5 MPa [1,813 psi] or the strength of intact dolomite for values of normal stress smaller than 16.7 MPa [2,422 psi]. These normal stresses are substantially higher than the normal stress of 1.3 MPa [188.5 psi] on a typical failure plane at a maximum depth of 60 m [200 ft]. The applicant's characterization of the rock-mass strength, therefore, satisfies the condition that the rock-mass strength should not exceed the intact rock strength.

The rock-mass strength was evaluated further by considering a disturbance factor, *D*, that was recently introduced by Hoek, et al. (2002) to account for rock-mass weakening from blast damage or stress relaxation. The value of *D* varies between 0 for the undisturbed case to 1.0 for the disturbed case. The rock properties evaluated using the Hoek-Brown criterion in PG&E Calculation 52.27.100.729 represent rock-mass strength for the undisturbed condition. Stress relief from the removal of overburden, such as by excavation, would cause a disturbance of the rock mass. Hoek, et al. (2002) recommended a *D* value in the range of 0.7–1.0 and suggested that assuming an undisturbed *in-situ* rock condition for slopes would result in an overestimation of the applicable rock-mass strength. The rock mass underlying the ISFSI site and the hill-slope area close to the ISFSI site has been subjected to such a stress relief because of the previous excavation of the slope for borrow material. Additional stress relief is expected to result from future excavation needed for the ISFSI construction.

Additional information provided by the applicant (Pacific Gas and Electric Company, 2003) indicated that any effects of stress relief from the 1971 excavation at the site are implicitly included in the measured geologic parameters used as input into the analyses to determine the Hoek-Brown parameters for the rock-mass. The applicant also indicated that the proposed excavation at the ISFSI site would be performed using large earth-moving equipment and explained that a zero value for *D* is appropriate for slopes excavated using such techniques. The applicant obtained an expert opinion from E. Hoek (Pacific Gas and Electric Company, 2003) to support its view regarding the applicable value of *D*. The staff accepts that a zero value for *D* is appropriate for the ISFSI site rock mass, considering that any stress-relief effect from the 1971 excavation has been implicitly accounted for and the excavation technique proposed by the applicant would not cause any appreciable rock-mass disturbance.

The staff concludes that the applicant's determination of an equivalent-continuum strength for the discontinuous rock mass is based on standard methods (Hoek, 2000b; Hoek, et al., 2002).

Therefore, the information presented is adequate for use in other sections of the SAR to perform additional safety analyses and demonstrate compliance with regulatory requirements in 10 CFR §72.90(a), §72.90(b), §72.90(c), §72.90(d), §72.92(a), §72.92(b), §72.92(c), §72.102(c), §72.102(d) and 72.122(b).

Stability of Cask-Storage Pad Foundation

The proposed ISFSI design (SAR Section 4.1, "Location and Layout") consists of seven contiguous reinforced concrete pads covering an area of approximately 152.4 m [500 ft] by 32.0 m [105 ft]. Each storage pad is 2.29 m [7.5 ft] thick and is designed to accommodate up to 20 storage casks in a 4 by 5 array. Each storage cask is approximately 6.10 m [20 ft] high, has a circular cross section of 3.35 m [11 ft] in diameter, and weighs approximately 163,293 kg [360,000 lb] when loaded. The base of the storage pad, therefore, will be subjected to a maximum static pressure of approximately 0.10 MPa [14.4 psi], considering the weight of a fully loaded pad. The potential dynamic loading for the design of the ISFSI foundation is reviewed in Section 2.1.6.2 of this SER.

Stability Against Bearing-Capacity Failure Under Static Loading: The allowable bearing pressure for the storage pad under static loading was developed in PG&E Calculation 52.27.100.713 (also designated as GEO.DCPP.01.03). The calculation was performed using two methods. First, considering the subsurface material as a homogeneous rock characterized by an estimated rock quality designation of 18 resulted in an allowable bearing pressure equal to 1.92 MPa [276.5 psi]. Second, the allowable bearing pressure was calculated by considering the subsurface material as a homogeneous soil characterized by zero cohesion and a friction angle of 50°, and using a bearing-capacity equation applicable to a shallow foundation subjected to a centered vertical static load. This approach resulted in an allowable bearing pressure equal to 3.83 MPa [551.6 psi]. Each fully loaded storage pad would be subjected to a centered vertical load resulting in a maximum static-bearing pressure of 0.10 MPa [14.4 psi], considering the dead weight of one concrete pad loaded with 20 casks.

The staff reviewed the applicant's evaluation of the stability of the cask storage pads with respect to the potential for bearing-capacity failure under static loading. The applicant's evaluation was performed using standard methods. The staff noted in the previous subsection that the rock-mass friction angle can be smaller by approximately 5° than the value used by the applicant. This potentially smaller value of rock-mass friction angle, however, does not affect the staff's assessment of the applicant's evaluation of the static-bearing capacity of the storage pads. The allowable bearing pressure remains significantly larger than the maximum static-bearing pressure of 0.10 MPa [14.4 psi] even considering a substantially smaller rock-mass friction angle. Therefore, the staff concludes that the proposed cask-pad design is adequate considering the potential for bearing-capacity failure under static loading. The staff also concludes that the information provided in the SAR regarding the static-bearing capacity of the storage pads is adequate for use in other sections of the SAR to perform additional safety analysis and demonstrate compliance with regulatory requirements in 10 CFR §72.102(c), §72.102(d) and §72.122(b).

Stability Against Bearing-Capacity Failure Under Dynamic Loading: The applicant's evaluation of the bearing capacity of the storage-pad foundation subjected to dynamic loading from the design-basis earthquake is documented as part of the applicant's response to the staff's RAI (Pacific Gas and Electric Company, 2003). The evaluation considered two failure modes of the

subsurface materials beneath the ISFSI: 1) potential failure dominated by movement on a horizontal bedding surface; or 2) potential rotational failure along a combined surface consisting of inclined joints and horizontal bedding. These potential failure modes needed to be considered because of the occurrence of near-horizontal clay beds that are significantly weaker than the surrounding rocks. The occurrence of the clay beds does not constitute a problem under static-loading states, because the static load is applied vertically and, as a result, would not cause any significant shear stress in any of the clay beds. Dynamic loading from the design-basis seismic ground motion, however, has horizontal and vertical components. The clay beds within the subsurface materials under the storage pad could, therefore, be subjected to shear stresses during an earthquake. The shear stress in the clay beds could potentially result in a bearing-capacity failure of the storage pad following one of the two failure modes.

The applicant's analysis for the horizontal-sliding failure mode is documented in PG&E Calculation GEO.DCPP.03.01, Revision 0, "Determination of Earthquake-Induced Displacements of Potential Sliding Mass Beneath DCPP ISFSI Site." The analysis considered the stability of an approximately rectangular block of rock 155.4 m [510 ft] long, 149.4 m [490 ft] wide, and 15.2 m [50 ft] thick. The top of the block includes seven ISFSI pads carrying 20 fully loaded storage casks each, and the base surface of the block is assumed to pass through a thin but continuous clay bed. The dimensions of the block were determined based on the geometry of the ISFSI pad, the canyon wall to the north of the pad, and the clay beds. The driving force considered in the analysis consists of a static representation of the lateral earthquake force calculated as αW, where W is the total weight of the block and, $\alpha = 0.43$, which is about half of the PGA for the design-basis ground motion. This evaluation of the driving force is consistent with standard practice (e.g., Kramer, 1996, p. 436). Resistance to sliding was considered as arising from the shear resistance of the clay bed at the base of the block and the shear resistance of the rock-mass on the vertical boundary surface of the block, except the canyon face. The shear resistance parameters used in the analysis are consistent with the information described previously in this subsection of the SER, under "Geotechnical Site Characterization."

A factor of safety of 1.02 against horizontal sliding was obtained through the analysis. This factor of safety indicates that a potential horizontal sliding of the ISFSI foundation during a design-basis earthquake cannot be ruled out based on the analysis provided by the applicant. The applicant performed an evaluation of the potential sliding displacement using the Newmark (1965) sliding-block approach. The analysis indicates that the foundation may experience a sliding displacement on the order of a few inches {approximately 6.35 cm [2.5 in]} during a design-basis earthquake. The staff concludes that such horizontal displacement would not have any significant impact on any safety function of either the pads or the storage casks.

The applicant's analysis for the rotational failure mode is documented in a PG&E Calculation, "Evaluation of Potential Rotational Failure of Foundation Beneath DCPP ISFSI Pads" ((Pacific Gas and Electric Company, 2003, Attachment 2-2). The analysis consists of evaluating the allowable bearing pressure for the pad and comparing it against the bearing pressure that may be imposed on the pad during a design-basis earthquake. The allowable bearing pressure for a shallow foundation such as the storage pad is equal to the ultimate bearing capacity, such as the value of foundation pressure that would cause a rotational failure, divided by a safety factor.

The applicant evaluated the ultimate bearing capacity of the pad using a standard bearing-capacity formula (e.g., Terzaghi, et al., 1996, p. 260) for a homogeneous soil deposit or a deposit for which strength increases continuously with depth. To apply the formula, the applicant assumed that the subsurface material at the pad site is homogeneous with a strength equal to the strength of the clay beds. The applicant's approach is accepted because it would result in a lower-bound ultimate bearing capacity for the pad foundation. The clay-bed strength used in the analysis is consistent with the information reviewed previously in this section under "Geotechnical Site Characterization." The analysis resulted in an ultimate bearing capacity of approximately 0.44 MPa [9,238 psf] for the pad.

The applicant estimated the bearing pressure imposed by the design-basis earthquake from the results of a finite element model analysis of the pad foundation (PG&E Calculation PGE–009–CALC–003, Revision 2, "ISFSI Cask Storage Pad Seismic Analysis"). The finite element analysis has been reviewed and accepted as documented in Section 5.1.3.4, "Structural Analysis for Reinforced Concrete Structures," of this SER. The analysis indicates that the storage pad would tend to rotate about a horizontal axis during a design-basis earthquake, resulting in a bearing pressure that attains a maximum value along an edge of the pad and decreases to zero near the middle of the pad. Such a rotation would cause a rotational failure of the pad foundation if the maximum pressure equals the ultimate bearing capacity of the foundation. A maximum bearing pressure of approximately 0.33 MPa [6,950 psf] was determined based on the analysis.

The applicant estimated the pressure on a clay layer at a depth of 7.92 m [26 ft] below the pad by applying a geometric attenuation factor to the maximum foundation bearing pressure. The estimated clay-layer pressure was compared with an allowable bearing pressure calculated as $(1.3/3)P_u$ where P_u is the ultimate bearing capacity of 0.44 MPa [9,238 psf]. The applicant explained that the factor of 1.3 accounts for an increase in dynamic bearing capacity relative to the static value and the factor of one-third is the inverse of the applicable safety factor. The staff did not agree with this aspect of the applicant's approach because a comparison of the pressure at the base of the foundation (the allowable bearing pressure) with the pressure on a clay layer at a depth of 7.92 m [26 ft] below the foundation is inconsistent with its interpretation of this failure mechanism. However, the staff determined that the information provided by the applicant is sufficient to complete an evaluation of the stability of the pad foundation during a design-basis earthquake. In the staff's view, the allowable bearing pressure can be obtained by applying a safety factor of 1.1, based on U.S. Nuclear Regulatory Commission staff guidance (U.S. Nuclear Regulatory Commission, 1981, pp. 3.8.5-7), to the ultimate bearing capacity, which gives 0.40 MPa [8,398 psf] as the allowable bearing pressure for the pad subjected to dynamic loading. Because this allowable bearing pressure is larger than the maximum bearing pressure of 0.33 MPa [6,950 psf], the staff concludes that the information provided by the applicant is sufficient to show that the storage pad foundation would not experience a rotational failure during a design-basis earthquake.

Therefore, the staff concludes that the proposed ISFSI storage pad design is adequate considering the potential for bearing-capacity failure under dynamic loading from the design-basis earthquake. The information provided in the SAR regarding the dynamic bearing capacity of the storage pads is adequate for use in other sections of the SAR to perform additional safety analysis and demonstrate compliance with regulatory requirements in 10 CFR §72.102(c), §72.102(d) and §72.122(b).

Staff Evaluation

The staff has reviewed Section 2.6.4, “Stability of Subsurface Materials,” of the SAR and concludes that the information presented in this section is adequate for use in other sections of the SAR to develop the design bases of the ISFSI, perform additional safety analysis, and demonstrate compliance with regulatory requirements in 10 CFR §72.102(c), §72.102(d), and §72.122(b).

2.1.6.5 Slope Stability

The staff has reviewed information presented in SAR Section 2.6.5, “Slope Stability,” which provides an evaluation of the stability of natural and cut slopes as follows: 2.6.5.1, “Stability of the Hill Slope Above the ISFSI;” 2.6.5.2, “Stability of Cut Slopes;” 2.6.5.3, “Slope Stability at CTF Site;” and 2.6.5.4, “Slope Stability Along the Transport Route.”

Stability of the Hill Slope Above the Storage Pad

The area to the south and southeast of the proposed ISFSI storage pad site is occupied by a hill that rises at an average slope of approximately one vertical to three horizontal units to a maximum elevation of approximately 182.9 m [600 ft] above the pad site. The hill is formed of jointed dolomite and sandstone with discontinuous intercalated clay beds. The applicant’s interpretation of geologic data from boreholes, trenches, and outcrops indicates that the stability of the slope would be controlled by potential sliding on compound surfaces formed by a combination of joints, clay beds, and new tension cracks. The clay beds are the weakest materials within the hill rock mass, such that any rock failure would likely initiate through slip on a clay bed. Because the clay beds are discontinuous, any such slip surfaces would have to connect through slip on joints and, if necessary, failure of rock bridges between joints or clay beds.

The stability of the hill slope above the storage pad is evaluated for the following reasons. First, a potential failure along a surface that daylights above the pad may result in the casks being impacted by rock blocks and debris, and the accumulation of these materials on the storage pads. Second, a potential failure along a surface that daylights to the north of the pad could cause a rotational failure of the pad foundation, similar to the second potential failure mode discussed in the previous subsection of this SER. The applicant has chosen not to assess the consequences of such events but instead to establish, through a slope-stability evaluation, that such events are not credible.

The applicant’s approach to evaluating the stability of the hill slope consists of making a case that (i) a failure of the hill slope during nonseismic (i.e., long-term static) conditions is unlikely; (ii) the displacement of any potentially unstable mass during a potential earthquake would be small and, therefore, of no concern to the safety of the facility; and (iii) adequate rockfall mitigation measures would be provided such that any rockfall generated from any seismically induced slope instability would not impact the storage casks.

Static Stability of the Hill Slope Above the Storage Pad: The applicant’s evaluation of the long-term stability of the hill slope subjected to static loading is documented in Section 2.6.5.1.2 of the SAR, in PG&E Calculation 52.27.100.734 (also cited as GEO.DCPP.01.24, Revision 1),

and in its response to staff RAIs (Pacific Gas and Electric Company, 2003, Attachment 3-1). The evaluation was based on a series of two-dimensional limit-equilibrium analyses. The analysis procedure, referred to as the method of slices, is standard and documented satisfactorily. Three sets of analyses were performed. The first set considered the stability of potentially unstable masses that were predefined based on the interpreted geometry of clay beds. The second set considered the stability along circular slip surfaces located using a random search routine. The third set of analyses also considered the stability of randomly selected circular slip surfaces, but differs from the second analysis set because the geometry of the proposed ISFSI excavation was included in the analyses.

The applicant did not include water pressure in the analyses, because the groundwater table is at a depth of approximately 65 m [210 ft] below the storage pad. Furthermore, rainwater is expected to drain freely through the fractured rock, and any perched water that may accumulate on a clay bed is likely to be small and short-lived because the clay beds are discontinuous and surrounded by fractured rock. The staff accepts the applicant's justification for not including water pressure in the stability analysis of the hill slope.

The first set of analyses consists of ten cases, each considering the equilibrium of a different potentially unstable mass (PG&E Calculation 52.27.100.734, Figures 1–10). The failure surface in each case is a composite surface that involves slip on one or more clay beds and failure of the fractured rock mass. The strength of the fractured rock was represented using a friction angle of 50° and zero cohesion. The shear strength of the clay beds was assigned using Eq. (2-1).

The results of the analyses are summarized in SAR Table 2.6-3. Nine analysis cases gave values of safety factor greater than 2.1, and the tenth gave a safety factor of 1.62. The use of a friction angle of 50° for the rock mass is not likely to have a significant effect on the calculated safety factor because the rock-mass sections of the failure surfaces are steeply inclined. Because the normal stress on such steeply inclined surfaces is close to zero, any frictional resistance contributed by such sections of the failure surface would be small, such that changing the friction angle for the rock-mass sections of the failure surfaces would not have a significant effect on the calculated safety factor. Any uncertainty in the rock-mass friction angle, therefore, would not have a significant effect on the calculated safety factor. The staff also determined that the effect of using Eq. (2-1) instead of Eq. (2-2) for the clay-bed strength would be to overestimate the safety factor by a factor of approximately 1.4 considering the strength contribution of sections of the clay beds subjected to a vertical stress greater than 0.65 MPa [94.3 psi], that is, clay-bed sections under a rock column of height greater than 29.3 m [96 ft], which excludes the analyses for slide mass 1a and 1b (SAR Figure 2.6-47). A minimum safety factor of approximately 1.51 would be obtained by dividing the applicable safety factors in SAR Table 2.6-3 by 1.4, except for the safety factor of 1.62 from slide mass 1b.

In the second set of analyses, the rock mass was considered as a homogeneous continuum with a shear strength defined by a friction angle of 50°. The analyses consist of an evaluation of the conditions for limit equilibrium along several randomly selected circular slip surfaces. The analyses, documented in the applicant's response to staff RAIs (Pacific Gas and Electric Company, 2003, Attachment 3-1), produced a minimum safety factor of 3.26.

As explained previously, the third set of analyses was similar to the second set, but also included the geometry of the proposed ISFSI excavation, which was not included in the second

set. The analyses also included rock anchors assumed to be installed on the cut slope face following a regular pattern and a center-to-center separation of 1.52 m [5 ft]. Each anchor, as represented in the analyses, was 9.14 m [30 ft] long, includes a bond section 3.35 m [11 ft] long, and was prestressed to approximately 340 kN [76,500 lb]. The analyses produced a minimum safety factor of 3.27.

A safety factor in the range of 1.25–1.5 is an acceptable safety margin against failure of a natural slope subjected to static loading (e.g., National Research Council Transportation Research Board, 1978, p. 172). The higher value of 1.5 is preferred for critical slopes (e.g., Hoek and Bray, 1977; U.S. Nuclear Regulatory Commission, 1977). Based on these considerations, the staff concludes that the long-term static stability of the hill slope, including the change in slope geometry that would result from the proposed ISFSI excavation, would be adequate to maintain safety. The information provided in the SAR regarding the long-term static stability of the hill slope is adequate for use in other sections of the SAR to perform additional safety analysis and demonstrate compliance with regulatory requirements in 10 CFR §72.102(d) and §72.122(b).

Seismic Stability of the Hill Slope Above the Storage Pad: Slope stability analyses provided by the applicant (SAR Section 2.6.5.1.3, "Seismically Induced Displacements") indicate that rock and debris may be dislodged from the hill slope above the ISFSI during a design-basis earthquake. The applicant provided additional information in SAR Section 2.6.5.1.3 (Pacific Gas and Electric Company, 2003a) to make a case that the displacement of any rock volume dislodged from the hill slope during a design-basis earthquake would be small. The applicant also committed to provide a sufficient rockfall mitigation to protect the ISFSI storage casks from rockfall impact that may arise from such dislodged rock. The applicant calculated seismically induced displacements through a series of analyses based on an approach proposed by Newmark (1965). The calculation consists of three steps for a selected potentially unstable mass.

First, the value of yield acceleration, k_y, was calculated through a limit-equilibrium analysis similar to the analysis described previously in "Static Stability of the Hill Slope Above the Storage Pad." The parameter k_y is the horizontal acceleration that would cause the value of the safety factor against sliding of the potentially unstable mass to decrease to 1.0 from the value calculated for the long-term static condition. The acceleration is represented in the analysis as a static horizontal force k_yM, where M is the mass of the potentially unstable mass. The calculation of k_y is documented in PG&E Calculation 52.27.100.734 (also cited as GEO.DCPP.01.24, Revision 1). The calculated values of k_y are given in SAR Table 2.6-3 for ten different potentially unstable masses and are in the range of 0.19–0.44g.

Second, an average horizontal acceleration time history, a_{ht}, for the potentially unstable mass was calculated [PG&E Calculation 52.27.100.735 (also cited as GEO.DCPP.01.25, Revision 1)]. The average acceleration time history is typically based on the ratio F_t/M where F_t is the resultant down-slope force along the potential failure surface (e.g., Kramer, 1996, p. 446). The applicant, instead, calculated the acceleration by averaging the nodal horizontal accelerations in a finite element model of the potentially unstable mass. The applicant asserted that this nodal average is a satisfactory representation of the average acceleration because the rock material is stiff enough that the acceleration of the rock at every point within the potentially unstable mass is similar to the input acceleration time history. Additional information was provided by the applicant in response to a staff request for a verification of the applicant's approach of

calculating a_{ht} by averaging nodal accelerations (Pacific Gas and Electric Company, 2003, Attachment 4-1). The additional information indicates that the values of a_{ht} calculated using F_t /M are essentially the same as the values calculated by averaging nodal accelerations.

Third, the difference $a_{ht} - k_y$ for $a_{ht} > k_y$ was integrated twice with respect to time to obtain a displacement, the so-called Newmark displacement. This calculation is documented in SAR Section 2.6.5.1.3 and in PG&E Calculation 52.27.100.736 (also cited as GEO.DCPP.01.26, Revision 1). The calculated displacements are given in SAR Table 2.6-5 and are in the range of 12–94 cm [0.4–3.1 ft]. The applicant interprets the calculated Newmark displacements as representing the magnitude of seismically induced displacements of the potentially unstable masses. Information obtained from other literature (as described subsequently), however, suggests that the Newmark displacements should be interpreted qualitatively when used for the assessment of the stability of a rock slope.

The Newmark displacement calculation is based on an approach proposed in Newmark (1965) for evaluating the potential seismically induced deformations of an embankment dam. The approach is based on the principle that significant displacements of the dam material would begin when the inertial forces on a potentially unstable mass are large enough to overcome the yield resistance and would stop when the inertial forces decrease to values smaller than the yield resistance. The method is implemented by performing a double integration of the acceleration difference $a_{ht} - k_y$ for $a_{ht} > k_y$. The method has been used extensively for earth dams and embankments, but the staff has not found any documentation of a successful use of the method for estimating seismically induced displacements in a rock slope. The method has been discussed in several review articles (e.g., Seed, 1979; Ambraseys and Menu, 1988) and textbooks (e.g., Kramer, 1996; Abramson, et al., 2002). All the applications of the Newmark calculation cited in Seed (1979), Ambraseys and Menu (1988), and Abramson, et al. (2002) are for earth dams and embankments. Similarly, seven of the nine publications on the Newmark method cited in Kramer (1996) are for earth dams and embankments and the other two are for landslides. One of the two landslide examples, [Jibson (1994)], is a review article. Only one, [Wilson and Keefer (1983)], reports an actual application of the method for estimating seismically induced landslide displacements.

This lack of published information on any application of the method to rock slopes accentuates the uncertainty regarding potential interpretations of seismically induced down-slope rock displacements calculated using the method. Materials used for earth dams and embankments typically have high damping, such that any seismically induced motion of such materials is likely to stop as soon as the source of excitation is removed. This behavior may explain why the double integration of the excess acceleration ($a_{ht} - k_y$ for $a_{ht} > k_y$) appears to give a reliable order-of-magnitude estimate of the seismically induced embankment deformations. Rock slopes, in comparison, are typically composed of low-damping materials. It is, therefore, conceivable that a rock body dislodged from a slope may continue its down-slope motion even if the condition $a_{ht} > k_y$ is no longer satisfied. In that case, any displacement calculated for such a rock body using the Newmark approach would only be a lower-bound estimate of the seismically induced displacement.

As suggested in Abramson, et al. (2002, p. 408), the Newmark displacements should be interpreted as a qualitative indication of the stability of a slope when subjected to the ground motion specified in the analysis. For example, the State of California guideline (State of California Division of Mines and Geology, 1997, Chapter 5) suggests a three-category

qualitative classification of the seismic stability of a slope as (i) likely stable, (ii) likely unstable, and (iii) definitely unstable. The Newmark displacements calculated by the applicant suggest that the hill slope could fall into the second or third category, depending on which potentially unstable mass is considered, if subjected to ground motion from the design-basis earthquake. An application of the State of California guideline to the displacements calculated by the applicant, therefore, suggests a conclusion that the hill slope above the ISFSI could be unstable during the design-basis earthquake.

In its responses to staff RAIs (Pacific Gas and Electric Company, 2003, 2003a), the applicant acknowledges that the toe region of a dislodged rock volume, such as the hypothetical slide masses used in the applicant's analyses, would be susceptible to calving and progressive raveling and would, as a result, be a potential source of seismically generated rockfall hazard. Therefore, the proposed ISFSI design includes the following rockfall mitigation features to minimize the potential impact of rockfall on the storage casks.

(1) The tower access road (SAR Figure 2.6-1) provides a horizontal bench 3.0–3.7 m [10–12 ft] wide that is expected to reduce the velocity of any falling rock.

(2) A drainage ditch 0.61–1.83 m [2–6 ft] wide and 0.30 m [1 ft] deep will be constructed at the top of the ISFSI excavation (on the hill slope side). The ditch is expected to stop small rock blocks.

(3) A rockfall barrier fence 2.4–3.0 m [8–10 ft] high will be constructed at the top of the ISFSI cut slope. The fence has an impact capacity of 8.1×10^4 m-kg [295 ft-ton]. The applicant's analysis indicates that the fence alone would be adequate to protect the ISFSI from rockfall impact.

(4) The ISFSI cut slope includes a mid-slope horizontal bench 7.6 m [25 ft] wide that is an additional energy dissipation barrier against any rockfall that may break through the barrier fence.

(5) The ISFSI pad is set back from the toe of the cut slope by another horizontal bench 12.5 m [41 ft] wide. This bench is a significant energy-absorption buffer against any rockfall that may break through the barriers enumerated previously.

The staff accepts that the proposed rockfall mitigation features will be adequate because of the use of several layers of barriers and energy-dissipation features. The staff, therefore, concludes that the information provided in the SAR regarding the safety of the hill slope with respect to seismically induced instability and rockfall is adequate for use in other sections of the SAR to perform additional safety analysis and demonstrate compliance with regulatory requirements in 10 CFR §72.102(d) and §72.122(b).

<u>Stability of Cut Slopes</u>

The three cut slopes designated as Eastcut, Backcut, and Westcut are situated along northeast, southeast, and southwest sides of the ISFSI storage-pad site. These slopes will be excavated in dolomite (unit Tof_b-1), sandstone (unit Tof_b-2), and friable dolomite and sandstone (Tof_b-1a and Tof_b-2a). The dolomite and sandstone are jointed and faulted. The stability of the

cut slopes in these rock units was evaluated using kinematic and pseudostatic analyses. The kinematic stability analyses on all three slopes provided potential modes of failure controlled by the discontinuities. The slopes with the potential for wedge failure were further analyzed in detail using a pseudostatic method for stability and design of mitigative measures. The friable or altered rocks are essentially not jointed, hence the kinematic and pseudostatic methodologies were not used for analysis of slopes in this medium. The stability of the slope in the altered rock region, which constitutes a limited area in the Backcut, was not analyzed. The applicant, however, discussed the support measures. The cutslope analysis is supported by the following PG&E calculation packages: GEO.DCPP.01.05, GEO.DCPP.01.08, GEO.DCPP.01.16, GEO.DCPP.01.17, GEO.DCPP.01.18, GEO.DCPP.01.20, GEO.DCPP.01.21, GEO.DCPP.01.022, and GEO.DCPP.01.23.

Kinematic Analysis: The kinematic analysis of cut slopes inclined at 70° is presented in PG&E Calculation 52.27.100.732, "Kinematic Stability for Cut Slopes at the DCPP ISFSI Site" (also cited as GEO.DCPP.01.22). The analysis identified three modes of failure typically associated with hard rock slopes, namely, plane, wedge, and toppling failures. The fracture data (joints, faults, and bedding planes) used in the analysis were collected from rock outcrop, exploratory trenches, and boreholes from each cutslope area. The fracture data were analyzed graphically by plotting, clustering, and contouring of poles on stereonets using DIPS Version 5.0 software. Four discontinuity sets were identified on each cutslope face, and average dip and dip direction for each discontinuity set were determined from the stereographic plots of discontinuity data. The discontinuity sets, local faults, slope geometries, and discontinuity friction angles were used for each mode of failure in kinematic analysis using the DIPS Version 5.0 software. The joint friction angle used in the analysis is based on direct shear tests of clean (rock-to-rock contact) and clay-coated rock joints in dolomite and sandstone as described in PG&E Calculation 52.27.100.728, "Determination of Basic Friction Angle Along Rock Discontinuities of DCPP ISFSI, Based on Laboratory Tests" (also cited as GEO.DCPP.01.18, Revision 2). The post-peak friction angles were evaluated to be 35° for clean and 14° for clay-coated joint samples. The applicant used in the kinematic analyses an average friction angle of 28°, evaluated by aggregating the direct-shear strength data from clean and clay-coated joint samples. The kinematic analyses ascertained moderate to high potential for plane failure and moderate potential for wedge failure for Eastcut. The Backcut has low to moderate potential for plane failure and high potential for wedge failure. The Westcut has high potential for toppling failure and low potential for planar and wedge failure. Eastcut and Backcut slopes were further analyzed for wedge stability and support design. Additional rockfall mitigation measures will be provided. These additional mitigation measures are reviewed in this subsection under "Seismic Stability of Hill Slope Above the Storage Pad."

The staff reviewed the kinematic stability analysis of cut slopes at the ISFSI pad site. The applicant used a standard approach to identify the major joint sets on each cut slope by stereo plots (Hoek and Bray, 1977). The kinematic analysis is also based on standard methodology to identify plane, wedge, and toppling failures using a stereographic projection approach (Hoek and Bray, 1977; Goodman, 1983). The average joint friction angle of 28° used in the analysis is within the range of friction angles for dolomite (27–31°) and sandstone (25–31°) given in Hoek and Brown (1980). The applicant has also provided a detailed description of the DIPS Version 5.0 software used in the analysis, including verification analysis.

Pseudostatic Analysis: The pseudostatic analysis of potential unstable wedges, identified by kinematic analysis, in Eastcut and Backcut slopes are presented in PG&E Calculation

52.27.100.733, “Pseudostatic Wedge Analysis of DCPP ISFSI Cutslopes, CSWEDGE Analysis” (also cited as GEO.DCPP.01.23). The stability analysis was performed using SWEDGE software, which is based on the limit equilibrium methodology described in Hoek and Bray (1977). Probabilistic and deterministic analyses were conducted on Eastcut and Backcut, considering pore water conditions, seismic forces, tension cracks, and rock anchors. Probabilistic analyses determined the potential unstable wedges with high probability of failure, taking into account uncertainty and variability in discontinuity orientation and strength. The deterministic analyses were conducted on these critical wedges to determine the anchor forces required to achieve stability.

Kinematic analysis identified four potential wedges for Backcut and three potential wedges for Eastcut. The proposed geometry for Backcut consists of two 70° cut slopes with a 7.62 m-[25-ft]-wide bench. The lower cut slope of height 6.25 m [20.5 ft] and upper cut slope of height 9.69 m [31.8 ft] were modeled separately. Because SWEDGE software cannot model a bench profile, a composite slope height of 15.9 m [52.3 ft] inclined at 47° was also analyzed. The Eastcut was modeled as a single slope of height 7.17 m [23.5 ft] sloping at 70°. Stability of the slope models was analyzed probabilistically by performing 1,000 Monte Carlo realizations on each model considering normal distribution in discontinuity parameters such as dip, dip direction and friction angles, variation of tension crack distances from the slope face, and inclusion or exclusion of seismic forces and joint saturation.

The dimensions of the wedges are controlled by the length of joints. The block dimensions typically range between 0.6 and 0.9 m [2 and 3 ft] locally up to a maximum of 4.2 m [14 ft] based on the field measurements of the joint continuity. The maximum depth of wedges in the model was 6.1 m [20 ft]. In probabilistic analysis, the dip and dip direction of the joints, obtained from kinematic analysis, was varied between 5 and 10°. The *in-situ* friction angle of the discontinuities estimated using Barton's equation (Barton and Choubey, 1977) is discussed in Section 2.1.6.5 of this SER. In probabilistic analysis, the friction angle was varied between 16° and 46°. For deterministic analysis, the friction angle was varied between 26 and 31°. The range of friction angles was obtained from the probabilistic analysis.

The horizontal force for pseudostatic stability analysis was determined from a seismic coefficient of 0.5. The approach used to evaluate the seismic coefficient is described in PG&E calculation package GEO.DCPP.01.05. The applicant used a methodology described in Ashford and Sitar (1994) to estimate the design seismic coefficient. The staff finds that the design seismic coefficient of 0.5g used in the pseudostatic analysis is between 50 and 67 percent of the PGA [0.83g] and is consistent with the recommendations in Kramer (1996, p. 436) and NUREG–1620 (NRC, 2002).

A water pressure head equivalent to one-half of the slope height was included in the wedge analysis to account for potential temporal accumulation of infiltrated rain water. In general, the cutslope area is dry, and the measured water table is 65 m [210 ft] below the ISFSI pad site. In addition, installation of a drainage system in the cut slopes has been proposed to prevent accumulation of perched water. The staff agrees that consideration of a water table at half the slope height for cutslope stability analysis is a reasonable assumption. Specific information on the drainage system was not provided in the Diablo Canyon ISFSI SAR; it is, however, expected that the design would include adequate spacing and depth of weeping holes to drain potential accumulation behind the slope face, which will be lined with shotcrete and wire mesh.

The probability of failure of wedges in Backcut varied between 0 and 1. Probability analysis, conducted on 17 models, showed most wedges were stable under dry joint and nonseismic conditions, but the probabilities were high for the cases analyzed with seismic forces or joint saturation or both. The maximum weight of unstable wedges varied between 18.1 MT [40 kips] and 2,030 MT [4,474.6 kips]. A deterministic analysis was conducted on the unstable slopes to design the rock anchor supports with a target factor of safety of 1.3. The rock anchor forces per anchor required to support the wedges ranged from 40 to 151.2 KN [9 to 34 kips] for a support pattern of 1.5 by 1.5 m [5 by 5 ft] placed in a staggered manner and inclined at 15° from the horizontal. Minimum anchor length, not including the bond length, required to penetrate the wedges varied between 1.2 and 7.0 m [4 and 23 ft]. The Eastcut slope was analyzed on 7 models, and the calculated probability of failure varied between 0.12 and 1.0. The maximum weight of the wedges ranged between 10.8 and 15.4 MT [23.8 and 34.0 kips] and the anchor force per anchor required to achieve a factor of safety of 1.3 was between 37.6 and 40.0 KN [8.4 and 9.0 kips]. The minimum anchor length in Eastcut was 0.9 m [3 ft]. The staff reviewed the methodology for anchor design of unstable wedges and finds it consistent with the standard practice. The staff also concurred with the factor of safety of 1.3, considering the recommendation by the U.S. Nuclear Regulatory Commission (1977) used in the analysis, and concluded that the proposed methodology is acceptable.

The methodology used for anchor design is described in PG&E Calculation 52.27.100.718, “Determination of Rock Anchor Design Parameters” (also cited as GEO.DCPP.01.08). The anchor design is based on Post-Tensioning Institute (1996) and the American Society of Civil Engineers (1996). The anchor system consists of 2.54-cm [1-in] diameter Cone-Tech Systems Grade 95H bars inserted in 5.08- to 7.6-cm [2- to 3-in] diameter holes of approximately 9 m [30 ft] deep. The anchors would be installed at 15° inclined to the horizontal with 0.61-m [2-ft] square reinforced concrete bearing pads and spaced at 1.52 m [5 ft] in a staggered pattern. The bond length of the anchor system is 3.1 m [10 ft]. The bond length is greater than computed bond length of 2.04 m [6.7 ft], which is based on a maximum design load of 169.0 KN [38 kips] per anchor from the wedge analysis for the seismic case, a factor of safety of two on the ultimate bond stress between rock and grout, and ultimate bond stress of 1.03 MPa [150 psi] for sandstone. The anchors would be stressed at 191.2 kN [43 kips], which is approximately 0.6 times the ultimate load. Staff evaluated the methodology used in the anchor design and concluded that the proposed methodology is acceptable.

The applicant proposed that the anchor design would be modified as follows for any altered or friable rock zones. The anchor system in altered rock would be improved, based on site conditions, with pressure grouting, increased bond length, reduced anchor spacing, extended anchor length beyond the altered zone, and post grouting in the bonded zones. Staff concluded that reinforcing the slope with anchors and protecting the slope face with wire mesh and shotcrete as proposed by the applicant would be adequate to stabilize the slope in the altered rock region.

The effect of the proposed ISFSI excavation on the stability of the hill slope was included in the applicant’s evaluation of the hill-slope stability. The staff’s review of this aspect of the slope stability evaluation is documented in earlier in this SER subsection under “Static Stability of the Hill Slope Above the Storage Pad.”

The staff, therefore, concludes that the information provided in the SAR regarding the stability of the cut slopes is adequate for use in other sections of the SAR to perform additional safety

analysis and demonstrate compliance with regulatory requirements in 10 CFR §72.102(d) and §72.122(b).

Slope Stability at the Cask Transfer Facility Site

There is no slope stability concern for the CTF site. Any potential instability of the subsurface materials under seismic loading conditions is bounded by the analysis reviewed in SER Section 2.1.6.4, “Stability of Cask-Storage Pad Foundation,” which indicates that the subsurface materials are sufficiently stable to withstand the foundation loading.

Slope Stability Along the Transport Route

The slope-stability concerns along the proposed transport route are (1) the potential encroachment of the Patton Cove landslide toward the transport route, (2) potential rockfall hazard from debris flows and toppling failure of rock blocks from the natural slope above the transport route, and (3) sliding failure of the subsurface material below the transport route. The applicant provided analyses in SAR Section 2.6.5.4 that address these concerns.

Potential Encroachment of the Patton Cove Landslide: Information provided in the SAR (Section 2.6.1.12, “Landslides;” and Section 2.6.5.4, “Slope Stability Along the Transport Route”) and in a subsequent clarification of the information by the applicant (Pacific Gas and Electric Company, 2002, Response to Question 2-17) indicates the following: (1) the Patton Cove slide surface intersects the ground surface at least 30.5 m [100 ft] from the transport route, (2) the slide consists of two slide surfaces constrained by a stable bedrock topography (as illustrated in Figures RAI 2-17-2 and 2-17-3), (3) any continued movement on the lower slide surface will not impact the proposed transport route because the headward growth of the landslide is constrained by a buried sea cliff 67.0 m [220 ft] south of the edge of the proposed transport route, and (4) a headward migration of the upper slide surface is not likely because the geometry of the slide is constrained by a horizontal wave-cut bedrock platform. The applicant also indicated that the landslide will be monitored through inclinometer measurements and visual inspections for cracks and that such monitoring will permit a sufficient warning of any impending landslide hazard prior to its occurrence.

The staff accepts the applicant’s conclusion that an encroachment of the Patton Cove landslide onto the transport route is unlikely because of the buried bedrock topography. Moreover, the applicant’s monitoring of the landslide, which includes a walk-down inspection of the transport route prior to any movement of the transporter, will provide a timely warning of any impending landslide hazard.

Potential Rockfall Hazard from Debris Flows and Toppling Failure of Rock Blocks: The applicant identified two potential sources of rockfall hazard along the transport route (SAR Section 2.6.1.12, “Landslides;” and Section 2.6.5.4, “Slope Stability Along the Transport Route”). The applicant’s analysis indicates a potential for toppling failure of isolated rock blocks from the hill slope above the transport route. Also, the applicant identified several colluvial or debris-flow swales above the transport route and concluded that debris flows may develop within such swales during severe weather events. Localized debris accumulations up to 1-m [3-ft] thick may occur on the transport route after such a debris flow.

The applicant identified several existing remedial measures to protect the roadway and the transporter from potential rockfall hazards. The remedial measures are (1) drainage ditches on the inboard edge of the road, (2) graded benches for an abandoned leach field system above a portion of the road, and (3) concrete ditches and culverts. The applicant also indicated that any debris accumulation on the road would be removed prior to any movement of the transporter.

The staff accepts the applicant's argument that potential rockfall hazards would be mitigated by the existing remedial measures. Such remedial measures are known to be effective for mitigating rockfall hazards along roadways (e.g., Hoek, 2002). Moreover, the applicant will conduct a walk-down inspection and implement any necessary cleanup of the transport route prior to any movement of the transporter, as part of the Cask Transportation Evaluation Program required by Diablo Canyon ISFSI Technical Specification 5.1.5.

Stability of the Subsurface Material Below the Transport Route Under Static Loading Conditions: The subsurface materials that form the foundation of the transport route may fail by sliding along vectors pointing approximately west to southwest. Information provided by the applicant (SAR Figures 2.6-7, 2.6-11, 2.6-16, 2.6-17, and 2.6-19) indicates that the subsurface material consists of Quaternary deposits up to approximately 15–27 m [50–90 ft] thick that overlie bedrock along a major portion of the transport route, except at the north end where the transport route lies directly on bedrock (sandstone and dolomite with intercalated clay beds). A part of the transport route near the south end will be constructed on engineered fill placed on the Quaternary deposits.

The stability of the transport route will be affected by the potential for a sliding failure of the subsurface material below the roadway. The occurrence of any such sliding failure would be controlled by slip on clay beds in the northern part of the road or by deformation of the Quaternary deposits along the remaining parts of the road.

Transport Route on Bedrock Foundation: Information provided by the applicant indicates that the transport route lies directly on bedrock from approximately Station 34+50 {i.e., 1,052 m [3,450 ft] from the south end of the route} to approximately Station 53+50 {i.e., 1,631 m [5,350 ft] from the south end}. The applicant further divided this section of the route into two parts: a southern part, from approximately Station 34+50 to 46+10, in which bedding surfaces dip into the slope; and a northern part, from approximately Stations 46+10 to 53+50, in which the bedding surfaces dip out of the slope. Failure of the slope below the transport route is unlikely in the southern part (Stations 34+50 to 46+10) because sliding on the up-slope dipping clay beds is kinematically impossible. Failure of the slope in the northern part (Stations 46+10 to 53+50) could not be ruled out based on kinematic considerations alone.

The applicant presented stability analysis to make a case that the northern part of the roadway from Stations 46+10 to 53+50 would be stable. The geologic cross section and the clay beds and potential sliding masses considered in the analysis are described in the applicant's response to the staff's RAI (Pacific Gas and Electric Company, 2003, Attachment 5-1, Figures TR–1 through TR–4). The location and attitude of the clay beds were inferred based on information from a borehole and bedding-plane measurements at locations close to the cross section. The slope stability analysis, to assess the stability of the transport route during static loading conditions (including the weight of the transporter), is documented in PG&E calculation GEO.DCPP.01.28, Revision 3, "Stability and Yield Acceleration Analyses of Potential Sliding Masses Along DCPP ISFSI Transport Route" in the applicant's RAI response (Pacific Gas and

Electric Company, 2003, Attachment 6-1). The clay-bed and rock-mass strength and unit weight used for the analysis are consistent with information reviewed in SER subsection 2.1.6.4, "Geotechnical Site Characterization." The transporter loading used in the analysis is consistent with information (weight of empty transporter and a loaded HI-TRAC 125 Transfer Cask) given in a supporting calculation: ENOVA Engineering Calculation 0104-021-C01, "Seismic Stability Analysis of Transporter on Soil" (Pacific Gas and Electric Company, 2002). The minimum safety factor of 2.07 obtained from the analysis in the applicant's RAI response (Pacific Gas and Electric Company, 2003, Attachment 5-1, Table 5-1) indicates the existence of an adequate stability margin for the slope.

A safety factor in the range of 1.25–1.5 is an acceptable safety margin against failure of a natural slope under static loading conditions (e.g., National Research Council Transportation Research Board, 1978, p. 172). The minimum value of 2.07 obtained from the analysis is larger than the range of acceptable values. Based on these considerations, the staff concludes that the long-term static stability of the subsurface material under the transport route, for sections of the transport route that directly overlie bedrock, is acceptable. The information provided in the SAR regarding the long-term static stability of the transport route foundation where the transport route lies directly on bedrock is adequate for use in other sections of the SAR to perform additional safety analysis and demonstrate compliance with regulatory requirements in 10 CFR §72.102(d) and §72.122(b).

Transport Route Underlain By Quaternary Deposits: In its RAI response, the applicant provided an evaluation of the long-term stability of the subsurface material below the transport route through static slope-stability analyses of three typical cross sections representing the areas of the transport route underlain by Quaternary deposits (Pacific Gas and Electric Company, 2003, Attachment 6-1, Table 2, and Figure TR–1). The analyses are documented in PG&E Calculation GEO.DCPP.01.28, Revision 3, "Stability and Yield Acceleration Analysis of Potential Sliding Masses Along DCPP ISFSI Transport Route."

The material properties used for the analyses (shear strength and density) were taken from Appendix 2.5C of Volume III of the DCPP FSAR (Pacific Gas and Electric Company, 2001) and the applicant's RAI response (Pacific Gas and Electric Company, 2003, Attachment 6-1, Table 1 and Attachment B).

The transporter loading was represented as an equivalent line load of approximately 25,538 N/m [1,750 lb/ft] applied uniformly over the transporter footprint. This loading is consistent with information (weight of empty transporter and a loaded HI-TRAC 125 Transfer Cask) given in the previously identified supporting calculation, ENOVA Engineering Calculation 0104–021–C01.

The calculated factors of safety indicate that there would be a sufficient safety margin against sliding failure of the subsurface material below the transport route during static loading conditions (including the transporter loading), for the sections of the transport route that are underlain by Quaternary deposits. The staff concludes that the long-term static stability of the subsurface material under the transport route, for sections of the transport route underlain by Quaternary deposits, is acceptable. The information provided in the SAR regarding the long-term static stability of the transport route foundation where the transport route lies on Quaternary deposits is acceptable for use in other sections of the SAR to perform additional

safety analysis and demonstrate compliance with regulatory requirements in 10 CFR §72.102(d) and §72.122(b).

Seismic Stability of the Subsurface Material Below the Transport Route: The applicant provided an analysis of the effect of a potential earthquake on the stability of the subsurface materials below the transport route, considering an occurrence of such an earthquake while the transporter is on the transport route. The analysis was performed using four geologic cross sections (Pacific Gas and Electric Company, 2003, Attachment 6-1, Figure TR–1). One cross section represents areas of the transport route supported directly on bedrock, and the other three represent areas of the transport route underlain by Quaternary deposits. The analysis is based on calculating the seismically induced displacements following an approach proposed by Newmark (1965).

As described previously in "Seismic Stability of the Hill Slope Above the Storage Pad," the Newmark approach consists of three key steps: (1) the calculation of the yield acceleration, k_y; (2) the calculation of the average acceleration time history, a_{ht}; and (3) the double integration of $a_{ht} - k_y$, for $a_{ht} > k_y$, to obtain the Newmark displacement.

The calculation of k_y is documented in PG&E Calculation GEO.DCPP.01.28, Revision 3, "Stability and Yield Acceleration Analysis of Potential Sliding Masses Along DCPP ISFSI Transport Route" (Pacific Gas and Electric Company, 2003, Attachment 6-1). This calculation also documents the analysis of the long-term stability of the transport route foundation as described previously in "Stability of the Subsurface Material Below the Transport Route Under Static Loading Conditions." As discussed in that section, the analysis was performed using standard techniques and appropriate values for the transporter loading and for the strength and unit weight of the bedrock and Quaternary deposit materials. The staff, therefore, concludes that the values of k_y calculated from the applicant's analysis are appropriate for conducting an assessment of the seismic stability of the transport route.

The calculation of a_{ht} is documented in PG&E Calculation GEO.DCPP.01.29, Revision 3, "Determination of Seismic Coefficient Time Histories for Potential Sliding Masses Along DCPP ISFSI Transport Route" (Pacific Gas and Electric Company, 2003, Attachment 7-2). The calculation was performed through a dynamic finite element analysis using a two-dimensional finite element model of each slope cross section. The base of each model (at a depth of approximately 91.4 m [300 ft] below the toe of the slope) was subjected to an input acceleration time history. The base input time history was calculated from a deconvolution of the design ground motion using a one-dimensional site response analysis code. The applicant verified in PG&E Calculation 52.27.100.744, "Verification of Computer Code QUAD4M" (also cited as GEO.DCPP.01.34) that the free-field ground motion calculated from the finite element model matches the design ground motion satisfactorily. The finite element analysis and the approach used to calculate the input acceleration time history for the finite element model are consistent with standard practice (cf. Ofoegbu and Gute, 2002). The applicant calculated a_{ht} using two methods: first, by averaging the acceleration time history at selected nodal points in each finite element model; and second, by using the ratio F_t / M where F_t is the resultant down-slope force along the potential failure surface and M is the potentially unstable mass. The second approach used by the applicant is the standard approach for calculating the seismic coefficient time history for a deformable medium (e.g., Kramer, 1996, p. 446). The values of a_{ht} calculated using the second approach were used by the applicant to calculate potential seismically induced displacements as explained presently. The staff, therefore, concludes that the values

of a_{ht} calculated from the analysis (Pacific Gas and Electric Company, 2003, Attachment 7-2, Figure 18) are appropriate for conducting an assessment of the seismic stability of the transport route.

The calculation of Newmark displacements through a double integration of $a_{ht} - k_y$ for $a_{ht} > k_y$ is documented in PG&E Calculation GEO.DCPP.01.30, Revision 3, "Determination of Potential Earthquake-Induced Displacements of Potential Sliding Masses Along DCPP ISFSI Transport Route (Newmark Analysis)" (Pacific Gas and Electric Company, 2003, Attachment 7-1). The calculated Newmark displacements generally lie in the range of 15.2–45.7 cm [0.5–1.5 ft. The calculated displacements should be interpreted as order-of-magnitude estimates of the potential deformation of the transport-route foundation when subjected to the design-basis earthquake and the transporter loading simultaneously. The results indicate that a deformation on the order of tens of centimeters would occur under such conditions. The direction of such deformation would be downward with a lateral component normal to the roadway. The potential implications of such deformation for the stability of the transporter during a potential seismic event are reviewed in Section 15.1.2.6, "Earthquake," of this SER.

The staff concludes that the applicant's evaluation of the seismic stability of the transport route foundation is based on standard methods and is acceptable. Therefore, the information presented is adequate for use in other sections of the SAR to perform additional safety analyses and demonstrate compliance with regulatory requirements in 10 CFR §72.90(a), §72.90(b), §72.90(c), §72.90(d), §72.92(a), §72.92(b), §72.92(c), §72.102(c), §72.102(d), and §72.122(b).

Staff Evaluation

The staff has reviewed Section 2.6.5, "Slope Stability," of the SAR and concludes that the information presented in this section is adequate for use in other sections of the SAR to develop the design bases of the ISFSI, perform additional safety analysis, and demonstrate compliance with regulatory requirements in 10 CFR §72.102(c) and §72.102(d).

2.2 Evaluation Findings

The staff has reviewed the site characteristics presented in the SAR and finds that the SAR provides an acceptable description and safety assessment of the site on which the Diablo Canyon ISFSI is to be located, in accordance with 10 CFR §72.24(a). The staff also finds that the proposed site complies with the criteria of 10 CFR Part 72 Subpart E, as required by 10 CFR §72.40(a)(2).

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