72-26

Pacific Gas and Electric Company

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PG&E Letter DIL-03-003

U.S. Nuclear Regulatory Commission
ATTN: Document Control Desk
Washington, DC 20555-0001

Docket No. 72-26
Diablo Canyon Independent Spent Fuel Storage Installation
Revised Response to NRC Request for Additional Information 5-1 for the Diablo Canyon Independent Spent Fuel Storage Installation Application (TAC No. L23399)

Dear Commissioners and Staff:

By letter dated December 21, 2001, the Pacific Gas and Electric Company (PG&E) submitted an application to the U. S. Nuclear Regulatory Commission (NRC) for a 10 CFR 72 site-specific license to build and operate an independent spent fuel storage installation (ISFSI) at the Diablo Canyon Power Plant site. The application included a Safety Analysis Report (SAR), Environmental Report, and other required documents in accordance with 10 CFR 72.

By letter dated August 29, 2002, the NRC staff requested additional information (RAI) needed to continue their review of the Diablo Canyon ISFSI License Application. PG&E submitted its response to the NRC staff by letter dated October 15, 2002 (PG&E Letter DIL-02-009).

Enclosure 1 contains PG&E's revised response to its October 15, 2002 RAI 5-1.

Enclosure 2 contains a draft of SAR Figure 4.2-2.

Enclosure 3 contains a list of revised and new calculations followed by the calculations.

The draft SAR Figure 4.2-2 will be incorporated into the Diablo Canyon ISFSI SAR, Amendment 2, and submitted to the NRC at a later date coincident with final responses to all outstanding NRC information requests and open items.

If you have any questions regarding this response, please contact Mr. Terence Grebel at (805) 545-4160.

NMSS01

Sincerely,

Lawrence F. Womack
Vice President Nuclear Services

pns3/4998
Enclosures

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UNITED STATES OF AMERICA
NUCLEAR REGULATORY COMMISSION

In the Matter of PACIFIC GAS AND ELECTRIC COMPANY Diablo Canyon Independent Spent Fuel Storage Installation	))))))))	Docket No. 72-26

AFFIDAVIT

Lawrence F. Womack, of lawful age, first being duly sworn upon oath states that he is Vice President, Nuclear Services of Pacific Gas and Electric Company; that he is familiar with the content thereof; that he has executed this revised and supplemental information to an NRC request for additional information regarding the Diablo Canyon Independent Spent Fuel Storage Installation license application on behalf of said company with full power and authority to do so; and that the facts stated therein are true and correct to the best of his knowledge, information, and belief.

Lawrence F. Womack
Vice President Nuclear Services

Subscribed and sworn to before me this 27th day of March 2003.

Notary Public
State of California
County of San Luis Obispo

CHUCK MACKEY
Commission # 1397547
Notary Public - California
San Luis Obispo County
My Comm. Expires Feb 1, 2007

Revised Response to Request for Additional Information for Diablo Canyon Independent Spent Fuel Storage Installation (ISFSI) License Application

Chapter 5 - Installation and Structural Evaluation

<u>*Question 5-1*</u>

Provide an evaluation to support the conclusion that the concrete will not break out of the storage pads prior to failure of the ductile metal members.

The Diablo Canyon ISFSI Safety Analysis Report (Section 3.3.2.3, ISFSI Concrete Storage Pad: Design Criteria) states that the design strength capacity of the embedded base plate, concrete bearing, and diagonal tension shear capacity are in accordance with the design provisions of American Concrete Institute 349-97 and the embedded anchorage will meet the ductile anchorage provisions of the Proposed Draft New Appendix B to American Concrete Institute 349-97 (dated October 1, 2000). Supporting evidence that the concrete will not break out prior to failure of the ductile metal members is not provided in the Diablo Canyon ISFSI Safety Analysis Report. An evaluation of concrete breakout strength in tension of a group of anchor rods considering spacing and edge distance should be provided.

This information is necessary to determine compliance with 10 CFR §72.24(a), §72.24(b), §72.24(c), §72.24(d), §72.24(i), and §72.122(b).

<u>Revised Response to Question 5-1:</u>

<u>BACKGROUND</u>

Each loaded cask delivers a total shear load of 515 kips (for all 16 anchors) and a maximum pull-out tension load of 62.13 kips in the highest loaded anchor under a seismic event. These loads must be carried into the ISFSI concrete by the anchorage design as outlined in Calculation PGE-009-CALC-001, Revision 5 (Attachment 1 to this submittal). These loads, along with other design requirements such as satisfying stiffness requirements and minimizing pad uplift in a seismic event resulted in an anchorage system, consisting of 16 anchors per cask perimeter each 2.5 inches in diameter and 71.13 inches in length (See Attachment 1, Page 20).

At the time of design of this anchorage system, the design team was aware of the pending new Appendix B to ACI 349-97 (draft dated October 2000). The design team was also aware that the NRC had never officially endorsed the old Appendix B to ACI 349-97 code (Reference 1). Therefore it was decided that the design would meet the requirements of the new proposed Appendix B. Since then, the new Appendix B has been officially issued as part of release of ACI 349-01 Code (Reference 2).

Section B.4.2.2 of Reference 2 states that for anchorage systems having diameter larger than 2 inches and anchorage length greater than 25 inches, the anchorage capacity shall be determined based on test data, and that the general equations of anchor pull-out and shear capacities are not applicable to anchors exceeding these size limitations because of lack of available test data.

Since it was obvious that the Diablo Canyon ISFSI cask anchorage system would exceed these size limitations, the design team decided to adopt a new design philosophy, which would meet the intent of Appendix B as described below.

DESIGN PHILOSOPHY

The design philosophy adopted was to deliver the design pull-out and shear loads into concrete as follows:

(a) The tension pull-out load was to be carried from the cask using 16 tension tie-rods (also referred to as round bars in Attachment 1) which would not only satisfy the stiffness requirements of design, but were designed to deliver the load to the concrete through upward compression imposed by a large 12-inch square plate (called the anchor plate in Attachment 1) attached at the lower end of these tension tie-rods (see draft Diablo Canyon ISFSI SAR, Figure 4.2-2 (Enclosure 2 to this submittal)). This upward compression load into concrete would in turn, be resisted by concrete bearing and diagonal shear of the appropriate concrete section.

(b) The imposed horizontal shear load was to be delivered into concrete by bearing action of the "coupler" mechanism, which is located at the top of the tension tie-rods, against the surrounding concrete. The shear load is then resisted by concrete through side bearing action. The tension tie-rods are not relied on to deliver any shear into the concrete as a typical anchor would.

By adopting this design philosophy, the tension tie-rods supported by large base-plates (referred to as anchor plates in Attachment 1) were treated as inverted columns on base plate, and were no longer treated as anchorage under jurisdiction of Appendix B. Therefore the design was carried out in accordance with shear provisions of the main body of the code (Sections 11.1 through 11.5 of Reference 1). However, to ensure that the intent of Appendix B is met, the design was carried out to achieve a ductile design by ensuring the following:

(a) That the concrete bearing and diagonal shear pull-out capacities as adjusted by appropriate code required strength reduction ϕ factors are larger than the ultimate capacity of tension tie-rods

(b) That the ultimate capacity of a tension tie-rod is larger than it's corresponding yield capacity

(c) That the yield capacity of a tension tie-rod was significantly larger than the demand tension pull-out load

This design philosophy ensured that not only there would be significant margin between the demand load versus the yield capacity of the tension tie-rods, but also the tension tie-rod yield capacity would be lower than it's ultimate capacity and the code capacity of all other supporting members in the load path such as concrete and anchor plate, thus ensuring ductile behavior of tension tie-rods in the unlikely event that the imposed demand would equal the yield capacity of the tension tie-rods.

The detailed load paths for tension pull-out and shear loads are further described below.

TENSION PULL-OUT LOAD PATH

The load path for the tension pull-out load from the cask into concrete is through 16 pre-loaded bolts, which are each, tied to a coupler at top of the concrete. This load is then transmitted through the coupler into what is called a tension tie-rod (also referred to as round bar in Attachment 1). These tension tie-rods are 2.5 inches in diameter and 71.13 inches in length (Attachment 1, Page 20). At the lower end of each tension tie-rod is a 12-inch square anchor plate (Attachment 1, Page 22, Section 6.4). The tension pull-out load is transmitted into concrete through the bearing action of the anchor plate as an upward compressive force against the concrete. This vertical compressive load can also be resisted by concrete as a diagonal shear through the ISFSI cross section and as a plug of concrete and reinforcing steel above each anchor plate evaluated using the shear-friction provisions of the Code.

To ensure that the intent of Appendix B is met, and a ductile design is achieved, first it was ensured that the ultimate capacity of each tension tie-rod exceeds its yield capacity. This check is done in Attachment 1, Page 21, Section 6.3, where the ultimate capacity is calculated as 220.34 kips versus yield capacity of 176.72 kips. This section also calculates the required ductile design strength for each tension tie-rod as 235.63 kips. The 12-inch square anchor plate is then sized (Attachment 1, Page 22, Section 6.4) so that its design strength and corresponding concrete strength in bearing exceed the required ductile design strength of the tension tie-rod. Furthermore, the diagonal shear strength of the concrete is checked (Attachment 1, Section 6.6) to ensure that it also exceeds this required ductile design capacity of 235.63 kips. In addition to the diagonal shear check, the portion of the pad above the 12-inch anchor plate is examined for capacity as a plug of reinforced concrete that the anchor plate is attempting to expel from the pad (Attachment 1, Page 26, Section 6.6). The shear-friction provisions of the Code are applied on assumed crack planes and the amount of steel necessary to comply with the Code shear-friction provisions is computed. One of the two layers of bottom steel, #10 bars at 9 inches each way, is located 2 inches above the anchor plates. The geometry of the 12-inch square anchor plates and the bar spacing of 9 inches ensure that at least 2 bars in each direction cross the assumed crack planes. The minimum steel area to resist the diagonal shear demand in the concrete is 0.444 sq. in. (Attachment 1, Page 27, Section 6.6).

The pad will be constructed using 5,000 psi concrete fully cured at 90 days. The temperature stresses in the reinforcement are expected to reduce to nominally zero psi due to heat dissipation and creep well before the pad is loaded with the casks. Thus, the reinforcement calculation for the shear friction provisions of ACI indicates no initial stress in the bars.

Therefore this design would ensure that the tension tie-rods are the ductile element in the entire tension load path since its ultimate capacity of 220.34 kips is lower than design strength of the supporting anchor plate, and the various concrete strengths based on code equations reduced by appropriate ϕ factors. Furthermore, this ultimate tensile capacity of 220.34 kips was calculated based on the reduced section at the threaded root of the tension tie-rods (Attachment 1, Page 21) which is approximately 125 percent of the yield capacity of the unreduced gross section of the tension tie-rods computed as 176.72 kips (See Attachment 1, Page 21). These tension tie-rods are made of A36 steel, which has a well-defined yield plateau. Thus, if any overload occurs, the tension tie-rods will yield before any less ductile failure could occur, therefore ensuring a ductile behavior as intended by Appendix B of References 1 and 2.

Lastly, the yield strength of the tension tie-rods (176.72 kips) is more than 280 percent of the computed demand load of 62.13 kips per tension tie-rod thus ensuring substantial margin against yielding.

COMBINED TENSION PULL-OUT AND PAD FLEXURE LOADING (SEISMIC)

Additional steel is required to resist the applied tension due to the seismic forces from pad flexure. The bounding value for this force is 21.536 kips. Refer to Calculations PGE-009-CALC-006, Revision 1, and PGE-009-CALC-007, Revision 0 (Attachments 2 and 3 respectively to this submittal). The required area, for 21.353 kips is 21.536/Fy = 21.536/60 = 0.359 sq. in. Thus, the total area of steel required is A = 0.444 + 0.359 = 0.803 sq. in. and each #10 bar provides 1.27 sq. in. which is 1.5 times the Code requirement. The anchor plates are 12 inches square, thus with a bar spacing of 9 inches, two bars must cross each of the shear planes. In addition, the mat of steel above the plates is throughout the pad, thus an assumed crack that might begin at a plate and migrate horizontally cannot find a location where it can progress vertically through the pad.

SHEAR LOAD PATH

Each cask has a total demand horizontal shear load of 515 kips, which needs to be delivered into the pad concrete. Even though the cask bolts have high pre-load in each of the cask bolts thus ensuring adequate friction to resist the applied shear load, the design conservatively did not take credit for this frictional resistance. The load path of the applied shear load is through the couplers at the top end of tension tie-rods into bearing resistance of the concrete. This check is done in Attachment 1, Pages 16 through 19, Section 6.2. As stated in Attachment 1, Page 19, the allowable shear load on the coupler as limited by concrete bearing is calculated as 1,350.4 kips, which is 260 percent of the demand shear load of 515 kips.

The tension tie-rods are not counted on to deliver any shear into concrete as a normal anchor system would; therefore provisions of B.4.1.1 and B.4.1.2 of Reference 2 are not applicable.

SUMMARY

The RAI requests supporting evidence that the concrete will not break out prior to failure of ductile metal members allowing for grouping of anchors and edge distance. The tension tie-rods are not treated like anchors for the reasons stated above in the design philosophy. They are treated as inverted columns on base plates and are sized to have lower ultimate strength than the surrounding concrete strength in bearing and diagonal shear as provided by the provisions of the main body of the code. As such, the design ensures ductile behavior, which meets the intent of Appendix B to the ACI 349 code. Attachment 1 provides various capacity calculations for different elements in the load path, thus providing the required evidence as stated above. Furthermore the design has substantial margin between the yield capacity of the weakest element (tension tie-rods) and the imposed tension pull-out demand load.

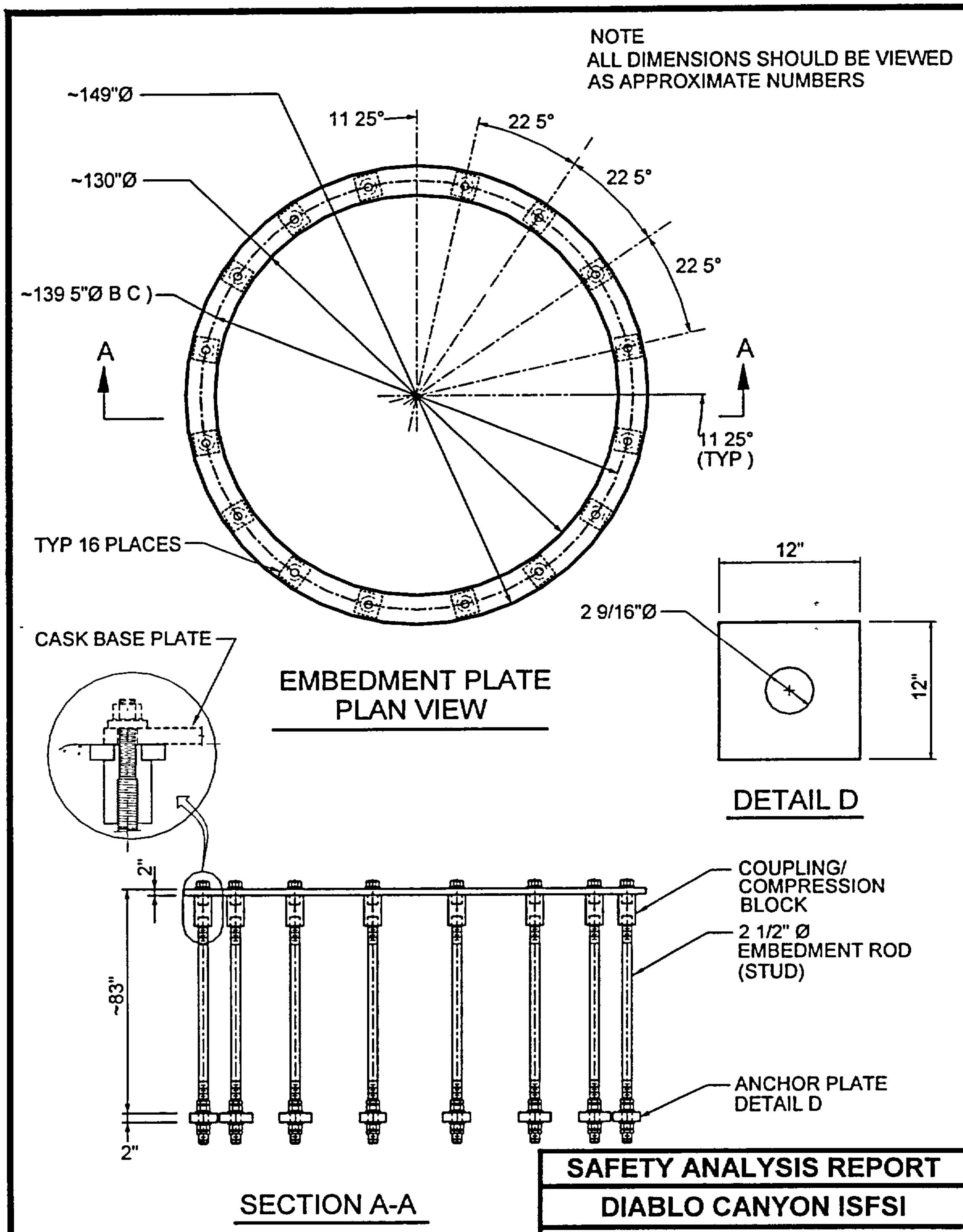
The load path for delivery of shear load into concrete is through the coupler at the top of the tension tie-rods. As such, the tension tie-rods are not relied on to deliver any shear load into concrete.

Lastly, the combination of the tension (pad flexure) and shear (pull-out) loading in the reinforcing steel and the minimum required steel area has been demonstrated and shown to have considerable margin.

REFERENCES

1. Code requirements for Nuclear Safety Related Concrete Structures, ACI-349-97 code.

2. Code requirements for Nuclear Safety Related Concrete Structures, ACI 349-01 code.



SAFETY ANALYSIS REPORT
DIABLO CANYON ISFSI
FIGURE 4.2-2 EMBEDMENT ANCHORAGE STRUCTURE

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List of Calculations

Attachment No.	TITLE
1	Embedment Support Structure Calculation PGE-009-CALC-001, Revision 5
2	ISFSI Cask Storage Pad Concrete Shrinkage and Thermal Stresses Calculation PGE-009-CALC-006, Revision 1
3	ISFSI Cask Storage Pad Steel Reinforcement. Calculation PGE-009-CALC-007, Revision 0

ATTACHMENT 1