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PG&E Letter DCL-03-077

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

Docket No. 50-275, OL-DPR-80
Docket No. 50-323, OL-DPR-82
Diablo Canyon Units 1 and 2
Response to NRC Questions Regarding License Amendment Request 02-03, "Spent Fuel Cask Handling"

Dear Commissioners and Staff:

By letter dated April 25, 2002, the Pacific Gas and Electric Company (PG&E) submitted an application for amendment to Facility Operating License Nos. DPR-80 and DPR-82, pursuant to 10 CFR 50.90. The License Amendment Request (LAR) submitted, for Nuclear Regulatory Commission (NRC) review and approval, proposed changes in the implementation of the Diablo Canyon Power Plant (DCPP) NUREG-0612 Control of Heavy Loads Program together with other analyses, design, and procedure changes required to implement a dry cask Independent Spent Fuel Storage Installation (ISFSI).

The NRC staff has recently asked additional questions related to their evaluation of LAR 02-03, which will allow handling and loading of Holtec International's (Holtec's) multi-purpose canisters and transfer cask in the DCPP 10 CFR 50 facilities. PG&E's response to these additional questions are included in Enclosures 1, 2, and 3. Enclosure 1 contains PG&E's responses to specific questions. Enclosures 2 and 3 contain the tables and figures, respectively, referenced in Enclosure 1.

This additional information does not affect the results of the safety evaluation and no significant hazards determination previously transmitted in PG&E Letter DCL-02-44, "License Amendment Request 02-03, Spent Fuel Cask Handling," dated April 15, 2002.

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

A001

Sincerely,

Lawrence F. Womack
Vice President - Nuclear Services

Enclosures

cc:	Diablo Distribution
cc/enc.	Edgar Bailey, DHS
	James R. Hall
	Thomas P. Gwynn
	David L. Proulx
	David A. Repka
	David Jaffe
	Jearl Strickland

UNITED STATES OF AMERICA
NUCLEAR REGULATORY COMMISSION

In the Matter of PACIFIC GAS AND ELECTRIC COMPANY	)	Docket No. 50-275 Facility Operating License No. DPR-80
Diablo Canyon Power Plant Units 1 and 2	)	Docket No. 50-323 Facility Operating License No. DPR-82

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 supplemental response to additional NRC questions regarding License Amendment Request 02-03, "Spent Fuel Cask Handling" 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 24th day of June 2003.

Amy Emiko Dong
Notary Public
State of California
County of San Francisco

AMY EMIKO DONG
Commission # 1393845
Notary Public - California
San Francisco County
My Comm. Expires Jan 14, 2007

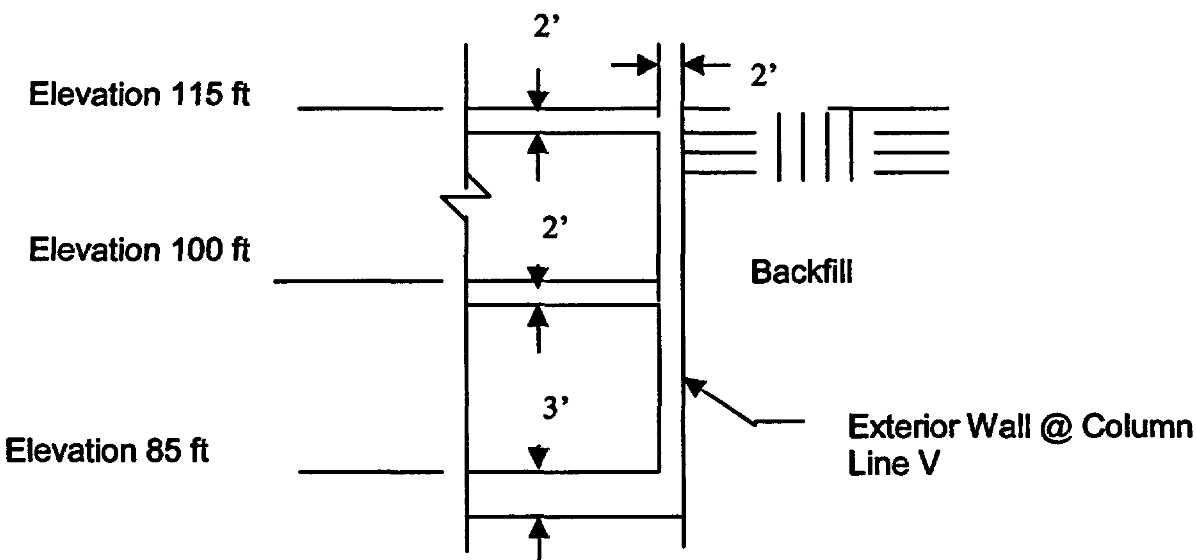
<u>NRC Question 1</u>

You stated in page 16 of the Reference that "...The wall has been judged adequate with respect to lateral pressure related to the surcharge loads..." Provide the following:

a) General description of the wall system (e.g., physical dimension, material type and current lateral pressure).
b) The increased lateral pressure along the wall due to the surcharge.
c) Technical justification on how the wall is adequate with respect to the lateral pressure related to the surcharge loads.

<u>PG&E Response</u>:

a) The 2 ft thick wall is located on the east side of the auxiliary building and extends from the south end at Unit 2 to the north end at Unit 1. The wall extends vertically from elevation 82 ft to elevation 140 ft. The back filled portion of the wall extends from the bottom to elevation 115 ft. There are 2 ft thick slabs at elevation 115 ft and elevation 100 ft, and a 3 ft thick slab at elevation 85 ft, which provide horizontal support to the wall from the inside of the building.



Materials:

Concrete:

$f_{c'}$ (28 day) = 5000 psi

Reinforcing Steel:

f_y = 40 ksi

Size and spacing:

#11 @ 7 inches on center outside face
#11 @ 14 inches on center inside face

Backfill: Crushed rock and concrete. An angle of internal friction, φ, for sand, equal to 55° was conservatively assumed.

The current maximum design lateral earth pressure is 0.75 ksf at elevation 85 ft. This equates to a maximum moment (8.41 ft-k) vs allowable moment (82 ft-k) ratio of 0.103 for the wall.

b) The addition of the surcharge of the loaded cask transporter increases the lateral earth pressure by 0.355 ksf max.

c) Calculations have been performed to qualify the wall for the increased pressure. The transporter's track pressure of 3.57 ksf was conservatively assumed to occur as a uniform surcharge over the entire ground behind the wall in the calculations. The maximum clear span for calculating moments is 13 ft. A fixed-fixed wall end condition was used for the critical span between elevation 85 ft and elevation 97 ft. The maximum wall moment to allowable moment ratio increased from 0.103 to 0.133, which is still within allowable limits.

<u>*NRC Question 2*</u>

You stated in page 16 of the Reference that "...Vital water tank piping vault ... Initial assessment indicates a need for more detailed evaluation and/or analyses ..." Provide the following:

a) What evaluation and/or analyses do you plan to perform?
b) Submit the results of the evaluation and/or analyses to the NRC for review.

<u>PG&E Response</u>:

a) A detailed structural evaluation was performed to verify that the underground non-safety-related walls of the piping vaults for the Design Class I outdoor water storage tanks and the non-safety-related primary water storage tanks can accommodate normal and abnormal loading conditions per the acceptance criteria in FSAR Update Section 3.8.2.

b) The results show that the vault walls are overstressed for the normal condition and for the abnormal condition that combines seismic and cask transporter surcharge loading. Capacity to demand ratios for the various loading conditions are tabulated in Table 2-1. The vault walls will be remediated prior to cask transport operations so that the walls meet the appropriate acceptance criteria. A conceptual remediation is shown in Figure 2-1. The remediation deploys shoring struts at selected elevations in the vault to support the vault walls. The wall flexure load is reduced by the distributed action of the axial forces in the struts to the opposing wall surface.

NRC Question 3

You stated in page 16 of the Reference that "... PG&E analyses provide input to the Holtec analyses and demonstrate the adequacy of the affected structures during the analyzed events ..." Identify the affected structures.

PG&E Response:

The structures affected are in the fuel handling building/auxiliary building (FHB/AB), specifically the spent fuel pool floor and walls of the cask recess area (CRA), the cask washdown area (CWA) floor and walls, the receiving/shipping Area (RSA) floor, the FHB crane, and FHB steel superstructure. Refer to FSAR Update Figures 3.8-60 to 3.8-62, to locate the grid and column line designations used below.

Structure	Analysis	Details in:
FHB Crane & building superstructure	Seismic[1]	DCL-03-047[2] Responses to Question 1-3
FHB Crane Auxiliary Lift	Crane structural code AISE Standard No. 6, (tentative) May 1,1969	DCL-03-047[2] Response to Question 1-4
Spent Fuel Pool CRA floor	Cask Drop analysis	DCL-03-047[2] Response to Question 3-1
Spent Fuel Pool CRA floor	Seismic[1]	See summary of PG&E Calculation 52.15.130 below.
Walls on plant column lines 12.9 and T, loads from SFP frame[3]	Seismic[1]	This letter, Response to Question 12
Spent Fuel Pool wall on plant column line 12.9, loads from CWA Restraint	Seismic[1]	See summary of PG&E Calculation 52.15.129 below.
Cask Washdown Area floor slab and supporting walls	Cask Drop analysis	DCL-03-047[2] Response to Question 3-1
Cask Washdown Area floor slab and supporting walls	Seismic[1]	See summary of PG&E Calculation 52.15.131 below.

Structure	Analysis	Details in:
Receiving/Shipping Area slab and supporting walls	Cask Drop analysis	DCL-03-047[2] Response to Question 3-1
Receiving/Shipping Area slab and supporting walls	Seismic[1]	See summary of PG&E Calculation 52.15.131 below.

[1] Includes loads from DE, DDE, HE and plant seismic margin assessment spectra (LTSP) per FSAR Update Section 3.7.

[2] PG&E Letter DCL-03-047, Response to NRC Request for Additional Information Regarding License Amendment Request 02-03, "Spent Fuel Cask Handling," dated April 25, 2003.

[3] SFP frame analysis not performed by Holtec.

<u>Calculation</u>	**PG&E Calculation 52.15.129, Spent Fuel Pool Wall Evaluation for CWA Seismic Restraint Reaction Forces**
Scope	The purpose of this calculation is to demonstrate that with the addition of the CWA, restraint reaction forces, in conjunction with previously defined load conditions, the walls are capable of resisting the higher demand loads. Note: These same walls are evaluated for loading on the opposite face due to the attachment of the spent fuel pool frame (this letter, Response to Question 12). Since only a single transfer cask will be in use, both the restraint and the frame will not be loaded concurrently.
Key Inputs	1. Frame support reaction loads are obtained from Holtec Calculation HI-2002507. 2. Anchor baseplates are assumed to be 2 ft square. 3. The cumulative effect from high-density fuel storage rack impact loads, dead, seismic inertia, hydrostatic, hydrodynamic and thermal loads are from PG&E Calculation 52.15.13.15. 4. Wall capacities are from PG&E Calculation 52.15.26.
Methodology	The walls are analyzed for in-plane and out-of-plane demand loads and demand ratios are calculated.

Calculation	PG&E Calculation 52.15.129, Spent Fuel Pool Wall Evaluation for CWA Seismic Restraint Reaction Forces
Results	See tables below

In-Plane

Seismic Event	Load Condition	Capacity/Demand
DE	Shear	3.05
	Flexure	2.68
DDE	Shear	2.56
	Flexure	2.47
Hosgri	Shear	1.67
	Flexure	1.61
LTSP	Shear	1.41
	Flexure	1.35

Out-of-Plane – Shear

Seismic Event	Load Condition	Capacity/Demand
DE	Shear	1.82
DDE	Shear	1.68
Hosgri	Shear	1.57
LTSP	Shear	1.53

Out-of-Plane – Flexure

Seismic Event	Load Condition	Capacity/Demand
DE	Outward	2.62
	Inward	11.6
DDE	Outward	1.96
	Inward	2.57
Hosgri	Outward	1.88
	Inward	2.62
LTSP	Outward	1.41
	Inward	1.97

Calculation	**PG&E 52.15.130, Structural Evaluation of Slab in the Cask Loading Pit Area of Aux. Building Subject to Cask Loading**
Scope	The purpose of this calculation is to evaluate and qualify the cask loading pit area in the auxiliary building at elevation 94 ft 6 inches for the design basis loadings, considering the presence of the cask and associated SFP frame. (Note: Cask drops are evaluated in PG&E Calculation 52.15.132, see DCL-03-047 RAI 3-2)
Assumptions, Inputs	Assumptions: 1. The limiting friction coefficient between the cask and the stainless steel liner plate is 0.80. 2. The cask and contents are a rigid body and its response is determined by the ZPA for vertical motions. For horizontal motions the smaller of the peak value of the spectra and 0.80 g (corresponding to the upper bound friction coefficient) is used. 3. Due to the small gap between the cask and the frame, the rocking effect is assumed to be limited. To account for rocking, 10% of the base area is used for bearing and the vertical earthquake load is doubled. 4. The pool water temperature for normal operating and accident conditions are used and material properties are reduced as appropriate. Inputs: 1. The foundation rock strength is from the FSAR Update Section 2.5.1.2.6.5 (800-1300 ksf) with a safety factor of 5 for the DDE analysis and 60 ksf for the DE analysis. 2. The frame loads, bearing and horizontal reactions at wedges and screw jacks are from PG&E Calculation 52.15.128 and are the enveloping values.
Methodology	The demand and allowable loads were calculated for the DE condition. Enveloping conditions were used for DDE, Hosgri and LTSP for demand, whereas DDE capacities were used conservatively for the remaining three seismic events.

Calculation	PG&E 52.15.130, Structural Evaluation of Slab in the Cask Loading Pit Area of Aux. Building Subject to Cask Loading				
Results		DE Condition		DDE/Hosgri/LTSP Condition	
		Demand	Allowable	Demand	Allowable
	Liner Sliding	150 k	300 k	150 k	182 k
	Bearing Cask on liner	0.60 ksi	21.6 ksi	1.0 ksi	32.4 ksi
	Bearing frame leg on liner	1.0 ksi	21.6 ksi	1.0 ksi	32.4 ksi
	Horizontal bearing frame jacks/wedges on liner side wall	7.69 ksi	21.6 ksi	7.69 ksi	32.4 ksi
	Concrete slab bearing	0.6 ksi	1.78 ksi	1 ksi	5.53 ksi
	Rock foundation bearing	27.0 ksf	60.0 ksf	37.6 ksf	160 ksf

Calculation	PG&E 52.15.131, Structural Evaluation of Slabs at Cask Washdown & Receiving/Shipping Areas in the Aux. Building Subject to Cask Loading
Scope	The purpose of the this calculation is to: 1. Calculate the capacities of the CWA slab and the new capacity/demand ratios from cask and frame seismic loads for DE, DDE, Hosgri and LTSP per FSAR Update Section 3.7. 2. Calculate the capacities of the receiving and shipping area (RSA) slab and walls on I the capacity/demand ratio from cask frame loads for DE, DDE, Hosgri and LTSP. 3. Specify minimum shoring requirements for the U.3 wall to support the cask seismic demand due to a 6 ft – 8 inch opening (door) in the wall.

Calculation	**PG&E 52.15.131, Structural Evaluation of Slabs at Cask Washdown & Receiving/Shipping Areas in the Aux. Building Subject to Cask Loading**
Key Inputs	1. Seismic loadings for the four seismic events are determined from Holtec Calculation HI-2002507. 2. The floors are qualified in accordance with the acceptance criteria for the plant as defined in the FSAR Update and appropriate PG&E Design Criteria Memoranda. Capacities are calculated in accordance with the applicable code criteria.
Methodology	PG&E Computer Program SAP2000N is used to determine the state of stresses in the concrete slabs as a result of dead and live loads and the seismic loads imposed by the cask, the CWA Restraint, and the cask transport frame (upending/downending) in the RSA.
Results	1. The capacity/demand ratio for the floor and walls under the CWA cask location all exceeded 1.0 for all loading conditions. 2. The capacity/demand ratio for the slab and the T.6 wall under the RSA slab exceeded 1.0 for all loading conditions. 3. The capacity/demand ratio for the U.3 wall in the location of the door blockout was less than 1.0. 4. A temporary support column (W14x61) in the door opening or equivalent structure increases the capacity/demand to greater than 1.0.

<u>*NRC Question 4*</u>

You stated in page 16 of the Reference that "... In addition, potential seismically-induced interaction between non-seismically designed SSCs (sources) and safe shutdown SSCs (targets) will be identified. Those not already analyzed to demonstrated stability during a seismic event will be evaluated ..." Identify the non-seismically designed SSCs (sources) and safe shutdown SSCs (targets), and those not already analyzed to demonstrate structural adequacy during a seismic event.

<u>PG&E Response</u>:

PG&E has performed the seismic interaction studies for the major dry cask equipment, including the transfer cask and all its movements and locations inside and outside the auxiliary building and on the cask transporter when on the elevation 115 ft bench adjacent to the outside of the FHB/AB.

The following SSC targets are located in the area and were evaluated:

1. Outdoor tank area east of the auxiliary building, elevation 115 ft, the refueling water, condensate and firewater storage tanks and associated piping and valves.
2. Cask washdown area and receiving/shipping area, an HVAC duct penetrating the floor on column line 15 near column line U and running up the wall and along the ceiling, and a conduit running along column 15.
3. North-South corridor between column lines U and V, the same HVAC duct along the ceiling and conduit.

The following SSC sources were considered and were seismically qualified to prevent seismic interactions:

1. The transfer cask in the cask transport frame while on the rails inside and leading outside the FHB/AB. See discussion of seismic qualification in the response to Question 10.
2. The cask transporter, both empty and carrying the transfer cask on the cask transport frame while moving along the elevation 115 ft bench. See LAR 02-03, Spent Fuel Cask Handling, page 45.

The LAR reference to "those not already analyzed" refers to the ancillary equipment (sources), which will be used to perform operations such as welding, drying, testing and backfilling the MPC. This equipment will be evaluated for seismically induced system interactions (SISI) when its design is complete and is being integrated into the plant. The DCPP Design Change process requires evaluations for SISI in accordance with the Seismic Configuration Control (SSC) Program. New sources are either located where there are no SSC targets, or seismically qualified to prevent interactions.

NRC Question 5

With respect to the accident evaluation, Drop into the Cask Washdown Area (CWA), provide the following:

a) A summary of the analysis results (i.e., stress, deformation, factor of safety, etc.)
b) Description of the impact limiter (i.e., physical dimension, material type and properties, etc.)
c) A summary of the analysis methods for the impact limiter, and the analysis results.
d) The maximum downward movement of the annular lead column relative to the transfer cask shells.
e) The criteria used to determine that the drop will not cause failure of building structural elements or unacceptable damage to other systems or equipment.
f) Justification as to how CWA seismic restraint would ensure that the cask will remain upright as a result of an impact during an event sequence so that the orientation that causes the most severe consequences would be avoided.

PG&E Response:

a) A summary of the analyses was presented in response to Question 3-1 of an earlier NRC request regarding this LAR (PG&E letter DCL-03-047, dated April 25, 2003).

b) The impact limiter for this accident evaluation is an all-metal, stainless steel exterior, crushable aluminum honeycomb material with deformable carbon steel internal structure that is a cylindrical, flanged, removable subcomponent of the transfer cask assembly. The limiter measures approximately 32 inches in height and 94 inches in diameter and is bolted beneath the bottom lid to the bottom flange of the transfer cask body with 12 bolts. The limiter weighs approximately 5000 lb. The published static crush strength of the honeycomb material is approximately 500 psi +/- 15 percent with maximum permissible deformation up to 60 percent. The impact limiter is an integral part of the NRC-certified structural performance of the cask system and as such, its performance and acceptance criteria has been licensed under 10 CFR 71 and 72 for transportation and storage operations, respectively.

c) A summary of the analysis methods for the impact limiter was presented in response to Question 1-8 of an earlier NRC request (PG&E letter DCL-03-047, dated April 25, 2003). The impact limiter is an integral part of the NRC-certified structural performance of the cask system and as such, its performance and acceptance criteria have been licensed under 10 CFR 71 and 72 for transportation and storage operations, respectively.

d) The maximum downward movement of the annular lead column relative to the transfer cask shells (transfer cask body) is 0.378 inches. This was demonstrated to be acceptable in a separate dose evaluation (Holtec Report HI-2002563) for the 10 CFR 72 Diablo Canyon ISFSI License Application.

e) This criteria was presented in response to Question 3-1 of an earlier NRC request regarding this LAR (PG&E letter DCL-03-047, dated April 25, 2003).

f) As discussed in response to Question 3-2 of an earlier NRC request regarding this LAR(PG&E letter DCL-03-047, dated April 25, 2003), Holtec International Calculation HI-2002506 demonstrated that in a drop into the SFP (without the frame in place) with an instantaneous off-center load applied at the top of the transfer cask representing the lifting device, crane block and rigging would result in a maximum rotation of 2.57 degrees for the 47 ft fall in free air and water. The drop into the CWA is 27 ft and would result in less rotation. The transfer cask has a 2 ft clearance to the restraint upper horizontal beam, and the maximum offset from 2.57 degrees of rotation is less than 8 inches. Therefore the transfer cask will not impact the restraint structure.

<u>NRC Question 6</u>

With respect to the accident evaluation, Drop into the Cask Recess Area, provide the following:

a) A summary of the analyses results (i.e., stress, deformation, factor of safety, etc.) for cask, impact limiter, liner plate and spent fuel pool (SFP) concrete floor.

b) A technical discussion of the fluid-structure dynamic interaction and computer modeling for the vertical drop over the SFP in your accident evaluation.

c) Description of the formulation used to simulate fluid coupling with the structure in the SFP. Discuss the basis for the formulation, key assumptions, limitations, and verification of the methodology by experiment.

(Note: Question revised based on April 24 telecon with NRC).

<u>PG&E Response</u>:

a) A summary of the analyses was presented in response to Question 3-1 of an earlier NRC request regarding this LAR (PG&E letter DCL-03-047, dated April 25, 2003).

b) The accident evaluation for the drop event over the SFP cask recess area was performed in two steps because two fluid media are involved. First, the drop of the cask assembly in air was evaluated to calculate the initial energy of the system at the moment the cask breaks the water surface. Next, using fluid dynamic expressions derived as explained in c) below, accounting for fluid-structure dynamic interaction, with the initial conditions resulting from the drop in air, the forces acting on the cask rigid body are calculated as the wetted cask falls to the target surface (cask recess area floor), deforms its impact limiter, and comes to rest.

Using derived expressions with appropriate limitations as input, a dynamic simulation code, VisualNastran, is used to simulate the drop of the cask with attached impact limiter onto the target surface as a one-dimensional model. VisualNastran Version 6.2 is a commercially available code that has been independently validated by Holtec in accordance with Holtec's Quality Assurance Program.

c) The fluid-structure dynamic interaction is simulated by deriving terms for the buoyancy effect, fluid drag effects, displaced fluid effects of the cask assembly, and fluid effects resulting from the squeezing of the fluid between two closely spaced rigid bodies as the cask assembly approaches the target surface. Energy lost by splashing and wave action as the cask assembly enters the water

is conservatively neglected. A constant, lower bound, conservative value was selected from published work by Roberson and Crowe[1] for the fluid drag coefficient for a cylinder whose axis is parallel to the direction of flow to simplify the correlation of cask velocity to fluid drag.

The formulations for hydrodynamic coupling are based on the published work of Fritz[2]. The hydrodynamic effects are separated into two terms: one representing the displaced fluid mass of the cask assembly and the other a "squeezing effect" of the fluid gap between two closely spaced rigid bodies. The fluid gap theory of Fritz is limited to larger gaps where the film effects of fluid compressibility and viscosity have little contribution. Therefore, it was necessary to ignore the resisting force due to the fluid gap effect as the cask assembly closely approached the target surface. Neglecting this force is conservative in that higher deceleration is demanded of the cask and its impact limiter, and a higher resisting force is applied to the target surface.

The formulations for the model developed herein have not been verified by experiment, but have been simulated using VisualNastran as described above. The VisualNastran results were independently verified using appropriate hand calculations for drop of an object through fluid media as defined in Section 5.2 of BC-TOP-9A "Bechtel Topical Report for Design of Structures for Missile Impact" which is the basis for applicable DCPP Design Criteria Memorandum DCM C-58 for structural analysis of accidental drops of heavy loads. The hand calculation resulted in an impact velocity of 520 inches per second versus 552 inches per second as simulated by VisualNastran. The VisualNastran prediction is slightly conservative due to the factors discussed above.

References

1. J.A. Roberson and C.T. Crowe, "Engineering Fluid Mechanics," Houghton Mifflin, 1975, p. 343, Table 11-1.

2. R.J. Fritz, "The Effect of Liquids on the Dynamic Motion of Immersed Solids," ASME Journal of Engineering for Industry, February 1972.

NRC Question 7

With respect to the accident evaluation, Drop or Tipover onto the AB Floor During Downending, provide the following:

a) A summary of the analyses results (i.e., stress, deformation, factor of safety, etc.) for cask, impact limiter, and FHB/AB Receiving/Shipping Area (RSA) concrete floor.

(Note: Question revised based on April 24 telecon with NRC).

PG&E Response:

a) A summary of the analyses was presented in response to Question 3-1 of an earlier NRC request regarding this LAR (PG&E letter DCL-03-047, dated April 25, 2003).

NRC Question 8

With respect to the accident evaluation, Drop before Loading MPC, provide a summary of the analysis results (i.e., stress, deformation, factor of safety, etc.).

PG&E Response:

The drop before loading MPC evaluation presented in LAR 02-03 Section 4.3.1(a)4 concluded that with respect to the cask handling hardware (i.e., the empty transfer cask assembly), the analytical results are enveloped by those accidents presented in Sections 4.3.1(a)1 to 4.3.1(a)3 of the LAR in which the transfer cask contains a fully loaded MPC.

The intent of the Section 4.3.1(a)4 evaluation was to address the incidental overhead load handling of the storage system and subcomponents into and within in the FHB/AB and indicate that these operations will be in accordance with DCPP Control of Heavy Loads Program commitments.

NRC Question 9

With respect to the SFP frame structural design calculations, provide the following:

a) *A summary of the analyses results that include the impact forces between the cask and frame and/or walls, governing stresses and deformations, and overall factor of safety.*

b) *Explanation and justification for consistency between the stiffness calculation of the structure in 2-D nonlinear dynamic analyses from the stiffness developed from the 3-D linear static analyses.*

c) *In general, a 3-D analysis provides more critical information for evaluation structural stability than a 2-D analysis does. However, you did not perform a 3-D analysis. Provide justification for not performing a 3-D analysis.*

d) *The results of the 3-D static analyses for structural member sizing and support reactions with the structural acceptance criteria used.*

e) *Description of the restraint and the results of the analyses for the restraints on the SFP frame at Elevation 111 ft. and 140 ft.*

f) *The analytical approach used for the analysis of the superstructure, SFP wall and floor and AB wall and floor. Also, provide a summary of the analysis results (i.e., stress, deformation, moment, factor of safety, etc.) in a tabular form.*

PG&E Response:

a) Impact forces between the cask and frame:

The SFP Frame 2-D dynamic ANSYS model (Calculation 52.15.137) was analyzed for the design earthquake (DE), double design earthquake (DDE), Hosgri earthquake (HE), and the LTSP per FSAR Update Section 3.7. The governing scenarios were taken as those which bound the rest of the scenarios for each load case/earthquake/direction. Table 9-1 shows the load scenario selection criteria.

All combinations of the governing loads were applied to the SAP 3-D model (PG&E Calculation 52.15.128) using 100 percent of a load in one direction (NS or EW) and 40 percent of the load in the other direction and vice versa.

These loads were applied to the 3-D SFP Frame SAP 2000 model at the two points (at top and bottom of transfer cask centerline). The largest bumper load was found to be 838 kips.

Impact forces between frame and wall:

Maximum frame impact forces on the SFP wall are included in Table 9-2. All table values in this section have units of kips. All information is taken from PG&E Calculation 52.15.128.

Frame governing stresses:

Values for frame member stress ratios in Table 9-3 are taken from PG&E Calculation 52.15.128. See Response 9e for typical frame member section locations.

b) <u>2-D Model Justification</u>

The dynamic properties of the SFP Frame are dependant on the area moment of inertia, the distance between supports and support conditions, the modulus of elasticity and the mass density of the structure. If all these properties do not vary in one of the three spatial dimensions then the SFP Frame can be modeled by a two dimensional model. The modulus of elasticity and the mass density of the structure are uniform throughout.

The four columns of the structure are the significant contributors to the area moment of inertia (see Figure 9-1). As shown in Figure 9-1, the area moment of inertia of a regular frame structure such as the SFP Frame is not a function of the angle θ. There is an opening on the east side of the frame that allows fuel to be loaded into the cask. The effect of this opening on the overall stiffness of the frame is minimal. Due to the SFP Frame's column connectivity, only small variations in the stiffness in the NS and EW directions at various elevations were found (PG&E Calculation 52.15.136).

The SFP Frame is supported by compression only wedges and jackscrews at the 94 ft 6 inch and 111 ft elevations. The arrangement of these supports around the SFP Frame is such that they provide the same support for any angle θ. The SFP Frame is supported at the 140 ft elevation by lateral supports on the south and west sides of the frame. These supports provide equal support for any angle θ.

Since all the parameters characterizing the dynamic properties of the system are independent of the angle θ, the dynamic response of the SFP Frame is therefore independent of the angle θ. The only major non-symmetry present in the dynamic response of the frame is due to the non-symmetric seismic loading as was shown in PG&E Calculation 52.15.136. The non-symmetric seismic loading is accounted for by analyzing the two dimensional model independently in the North-South and East-West directions. The two-dimensional model used in the SFP Frame Dynamic Analysis (PG&E Calculation 52.15.137) is therefore appropriate.

North-South and East-West Earthquake Decoupling

To show that the analysis earthquake decoupling assumption is appropriate, a representative ANSYS model shown below was analyzed in PG&E Calculation 52.15.137 using a portion of the Hosgri Earthquake time-history acceleration loading. Three sets of analyses were performed as follows:

1. Hosgri NS Earthquake
2. Hosgri EW Earthquake
3. Hosgri NS and EW Earthquake

It was shown in PG&E Calculation 52.15.137 that the resulting time-history reactions for bumpers 1, 2, 4, and 5 (based on the node numbers in Figure 9-2) are identical for all three runs. This proves that the two components of the Hosgri earthquake can be decoupled and analyzed separately as was done in Calculation 52.15.137. Confirmatory analyses were performed for the DE, DDE, and LTSP earthquakes and the same results were found.

The above analysis proves that the orthogonal earthquake load decoupling methodology used in this SFP Frame Dynamic Analysis Calculation (PG&E Calculation 52.15.137) is appropriate.

c) A 2-D dynamic analysis was performed to determine the loads that the cask would impose onto the SFP frame (PG&E Calculation 52.15.137). The justification for the 2-D analysis is included in the response to 9b above. Using the loads determined in the 2-D dynamic analysis, a 3-D static analysis of the SFP frame was performed (PG&E Calculation 52.15.128). North-South and East-West loads were applied simultaneously to the frame to determine the frame response. The SFP frame was shown to meet the design criteria. In summation, allowable member stresses are 1.7 x AISC allowable stresses for the HE and LTSP earthquakes. Allowable member stresses are 1.6 x AISC allowable stresses for the DDE earthquake. AISC allowable stresses are used as the basis for the DE earthquake. Member stresses do not exceed the yield stress for all load cases.

d) Structural member sizing and support reactions are addressed in the Response to 9a above. Allowable member stresses are 1.7 x AISC allowable stresses for the HE and LTSP earthquakes. Allowable member stresses are 1.6 x AISC allowable stresses for the DDE earthquake. AISC allowable stresses are used as the basis for the DE earthquake. Member stresses do not exceed the yield stress for all load cases. The frame is checked for seismic and dead load only. Minimal gaps are left between the frame and the SFP wall to allow for thermal growth. As such, thermal stresses are not an issue.

e) SFP Elevation 111 ft restraint:

The elevation 111 ft restraint frames around the cask recess area in the SFP. The restraining pipes, horizontal and vertical, are 12 inchϕ Schedule XS Type 304 stainless steel. The support was analyzed and found to meet the acceptance criteria as described in Response 9d. A plan view of the support at elevation 111 ft is shown in Figure 9-3. The SFP frame column placement is shown also. The maximum reactions are shown in Table 9-2. It should be noted that screw jacks are located at the NW, SW, and SE frame columns at elevations 111 ft 0 inches and 94 ft 6 inches.

SFP Elevation 140 ft:

The locations of the three lateral supports for the frame are shown in Figure 9-4. The elevation 140 ft restraints were designed to meet the acceptance criteria as described in Response 9d. The design was performed in PG&E Calculation 52.15.134. Elevation 140 ft reactions are included in Response 9a in Table 9-2.

f) A static analysis (PG&E Calculation 52.15.128) was performed for various load cases (see Response 9a). A 3-D computer model of the SFP Frame system was generated using the computer program SAP 2000 for the analysis (PG&E Calculation 52.15.128). Cask bumper loads were taken from PG&E Calculation 52.15.137 (see Response 9a for loads). See Response 12 for discussion of SFP concrete structure.

The restraints at elevation 111 ft 0 inches and elevation 94 ft 6 inches are needed for the various load cases to model the true behavior of the frame. The restraints at elevations 111 ft 0 inches and 94 ft 6 inches can only take compression. Therefore, to replicate the actual behavior of the frame, design loads were applied and supports were added where the deflection of the frame would cause the supports to activate. See Response 9a, Table 9-3, for frame analysis results.

NRC Question 10

With respect to the analyses performed by your contractor, Holtec, to demonstrate the adequacy of the cask system, inside the SFP frame, inside the CWA Seismic restraint structure and on the cask transport frame while the cask system enters or exits the building, provide the following:

a) The analytical approach and methodology used for the analysis.
b) A summary of the analyses results including those for the anchorages.
c) Indicate whether the computer code, DR Frame 2.0, used for the analyses has been reviewed and accepted by the NRC.
d) Indicate whether artificial time histories are generated and used for seismic analyses.

PG&E Response:

a) To demonstrate the adequacy of the cask system in the Part 50 facility, Holtec was required to:

1. Demonstrate that the postulated events do not induce acceleration levels exceeding the Holtec Part 72 license basis for the cask design.

2. Demonstrate the CWA seismic restraint system and cask transport frame/rail system meets the structural integrity limits established for DCPP. (Note: The SFP frame analysis was not performed by Holtec, but is described in detail in the response to Question 9 in this letter)

3. Develop the interface loads on the building floor, slab and walls from the cask restraints at each location (See the response to Question 3 for discussion of the evaluation of the restraint loads on the structure and the response to Question 9 for the frame loads to the SFP structure, both in this letter).

b) Cask System in SFP Frame

Responses to Questions 9 and 12 discuss the details of the SFP frame design, Items 2 and 3 above. To demonstrate the impacts on the transfer cask while in the SFP frame (Item 1 above), PG&E Calculation 51.15.137, "Spent Fuel Pool Frame Dynamic Analysis" developed the seismic loads the cask bumpers apply to the SFP frame. The same loads are applied as impact loads to the cask. The maximum bumper load was 838 kips. Assuming the load is applied to the both top and bottom bumpers simultaneously, the maximum load on the transfer cask is 838/250 = 3.36 g, less than 10 percent of the licensed drop deceleration limit of 45 g for the loaded transfer cask.

Cask System in the CWA Restraint and Cask Transport Frame

Calculation	**Holtec International Calculation HI-2002507 "Seismic Analyses of Loaded HI-TRAC in Diablo Canyon Fuel Building"**
Purpose	The purpose of this calculation is to: 1. Perform a dynamic analysis of the loaded HI-TRAC in the CWA restraint to develop the loads applied to the restraint structure, the supporting walls, and floor, and the accelerations applied to the transfer cask. Evaluate the structural integrity of the restraint structure and develop the loads applied to the FHB/AB structure for analysis by others (see the response to Question 3 in this letter). 2. Perform an analysis of the loaded HI-TRAC secured in the upending/downending frame (cask transport frame) in a horizontal orientation in the receiving/shipping area (RSA). Determine the stability of the system and develop the loads imposed on the FHB/AB structure.
Assumptions, Inputs	For both analyses, acceleration-time histories are used in each of three orthogonal directions with duration of 24 seconds for: 1. DE 2. DDE 3. Hosgri Earthquake 4. Long Term Seismic Program Spectra per FSAR Update Section 3.7. For the CWA analysis: 1. The elevation 140 ft time histories are used in the analysis. 2. The MPC is free to move independently within the HI-TRAC. The restraint lid is not included which allows unrestrained upward movement of the MPC. The horizontal movement is limited by the gap between the MPC and transfer cask. 3. The Kevlar slings used to restrain the transfer cask are characterized by a linear elastic stress strain relationship (tension only) based on manufacturer's data.

Calculation	**Holtec International Calculation HI-2002507 "Seismic Analyses of Loaded HI-TRAC in Diablo Canyon Fuel Building"**
	4. The transfer cask to floor connection is a specially engineered calibrated low friction base plate with a coefficient of friction of 0.18. For the RSA analysis: 1. The elevation 115 ft time histories are used in the analysis. 2. The transfer cask is considered as a rigid cylinder rigidly connected to a rectangular block in contact with the ground. The center of gravity (CG) is conservatively assumed to be the CG of the cylinder. The block is restrained in one direction by side blocks (the sides of the rails) and free to move in the direction of the rails, with a friction factor. 3. The coefficient of friction between the rail and cask transport frame (Hillman rollers) is assumed to be 0.1. 4. The contact spring stiffness between the cask transport frame and the ground is set to 1,000,000 lb/in.
Methodology	For the CWA restraint: 1. VisualNastran (VN) was used to perform the dynamic analysis. 2. Sling loads, vertical impact loads between the cask and floor, and the peak acceleration at the top of the MPC were determined. 3. The bounding sling loads were used to analyze the restraint frame and develop the loads imparted to the FHB/AB structure. For the RSA analysis: 1. A quasi-static analysis was initially employed using the 100 percent-40 percent-40 percent combination rule for the HE and LTSP events and a 2-D absolute sum rule for combining the DE and DDE events. The input acceleration was the ZPA for each direction. 2. As a result of the above analysis, VN was used to dynamically model the system for all four seismic events. Maximum floor loadings were determined.

Calculation	**Holtec International Calculation HI-2002507 "Seismic Analyses of Loaded HI-TRAC in Diablo Canyon Fuel Building"**
Results	The CWA analysis: 1. The bounding acceleration at the top of the MPC was 3.58 g, significantly less then the allowable 45 g. Bounding sling loads were 150 kips, capacity is 375 kips. 2. The restraint frame steelwork was analyzed using the sling loads and the bounding horizontal and vertical accelerations from the dynamic analysis. For analysis of the portion of the restraint structure that directly restrains the slings, the sling capacity of 375 kips was used as the design loading. 3. The restraint frame loads to the FHB/AB structure were analyzed by PG&E (see the response to Question 3) The RSA analysis: 1. The quasi-static analysis of the frame indicated that, except for the DE earthquake, incipient tipping (rocking) would occur. 2. The seismic instability required that a dynamic analysis be performed to obtain the floor loadings. The analysis resulted in floor loadings that were used by PG&E to analyze the floor (see the response to Question 3).

c) DR FRAME 2.0 Computer Program

Holtec International used the computer program DR FRAME 2.0 in the analysis of the stiffness of the CWA Restraint frame and to analyze the restraint frame. The program was qualified for use by Holtec in accordance with their NRC-approved Quality Assurance Program in Holtec Report HI-2012811 (for generic use) "Dr Frame QA Documentation and Validation."

Per Holtec International, the program has not been reviewed by the NRC.

d) Artificial Time Histories

Artificial time histories, enveloping the floor spectra at appropriate elevations of 115 ft or 140 ft were provided to Holtec by PG&E, for use in various analyses as described above.

<u>*NRC Question 11*</u>

Provide a description of the analysis method used to demonstrate that the pool liner will not tear or rupture under the loading conditions.

<u>PG&E Response</u>:

The structural integrity evaluation of the liner is addressed in PG&E Calculations 52.15.122 and 52.15.132. The details of these calculations are provided in response to Question 3-1 of an earlier NRC request regarding this license amendment (ref. PG&E letter DCL-03-047, dated April 25, 2003). However, it is also noted that a pool liner breach attributed to loading conditions from a cask drop is discussed in LAR 02-03 Section 4.3.2(a) and found to be acceptable based on capture of pool inventory by the surrounding AB concrete foundation structure behind the liner. The pool liner does not perform any nuclear safety function to ensure pool inventory is contained. A pool liner breach due to lateral load transfer from the spent fuel pool frame to the AB concrete foundation structure, though not desirable, does not negatively affect the integrity of the SFP to contain the required inventory to cool and shield the nuclear fuel present in the pool. The response to Question 13 in this submittal describes the SFP leakage detection system and its operations.

NRC Question 12

With respect to the SFP frame structural analysis:

a) Provide the physical dimensions of the SFP including the thickness of the reinforced concrete (RC) slab and walls and liner plate.
b) Provide a copy of the mesh used in the analysis.
c) Describe the boundary conditions used.
d) Describe the applied loading conditions including their magnitudes.
e) Explain how the SSE loading was applied to the SFP. If you used equivalent static analysis, provide justification for this approximation. What were the magnitudes of the equivalent static accelerations applied at mass points you used in the analysis?

PG&E Response:

a) The SFP is a Design Class I, safety-related reinforced concrete structure. The pool walls and base-mat provide containment for the borated water within the SFP. A plan view of the pool at elevation 140 ft is shown in Figure 12-1. The walls of interest are labeled Wall AB & AD (on grid lines 12º and T, respectively).

b) The SFP Frame was not a mesh analysis. It was a SAP2000 frame model utilizing beam elements. The SFP wall was constructed using shell elements in SAP2000. The SFP wall model was constructed using a 1 ft x 1 ft mesh.

c) The selected frame configuration is a 9 ft 7 inch x 9 ft 7 inch out-to-out square tower structure. The frame is supported vertically in the cask recess area (elevation 94 ft 6 inches) and supported laterally just above the cask recess area at the existing SFP lateral restraint (elevation 111 ft 0 inches), and at the top of the SFP wall (elevation 140 ft 0 inches). The frame is open at the top (elevation 140 ft 0 inches) to allow the cask to be lowered through it without any vertical resistance to its motion. This is accomplished by providing gaps between lateral bumpers on the cask and the frame members. The bumpers are located at the corners of the frame.

This model is constructed from beam and nonlinear link elements. The support restraints are modeled as nonlinear links. The nonlinear links only behave nonlinearly during a nonlinear time-history analysis. For static analysis the nonlinear links behave linearly. The frame is supported at the 140 ft 0 inches elevation with pinned beams. Restraints are used to model jackscrew and steel plate supports at the 111 ft 0 inch and 94 ft 0 inch elevations. These elements are also used to model the SFP cask recess area floor which supports the four columns of the frame

The restraints at elevation 111 ft 0 inches and elevation 94 ft 6 inches are needed for the various load cases to model the true behavior of the frame. The restraints at elevation 111 ft 0 inches and 94 ft 6 inches can only take compression. Therefore, to replicate the actual behavior of the frame, design loads are applied and supports are added where the deflection of the frame would cause the supports to activate. Figures 9-3 and 9-4 show a picture of the frame at elevation 111 ft 0 inches and elevation 94 ft 6 inches in the SFP.

d) The largest cask to frame load is 838 kips. The applied loading conditions are discussed in the response to Question 9. Loads from the frame are transferred to the SFP walls at the supports at elevations 140 ft, 111 ft, and 94 ft 6 inches.

The maximum frame to wall reactions are tabulated in the response to Question 9 and in Table 9-2.

e) Equivalent static accelerations were not applied. Instead, maximum reactions from the SFP frame 3-D analysis (PG&E Calculation 52.15.128) were applied to reactions locations at the SFP wall simultaneously. SFP frame reaction loads were used in conjunction with hydrostatic, inertia, and rack impact loads to analyze the SFP walls (PG&E Calculation 52.15.126). The SFP walls were found to meet DCPP acceptance criteria. Frame forces applied to the wall are tabulated in Table 9-2. A summary of the results from the SFP wall analysis (PG&E Calculation 52.15.126) is provided in Tables 12-1 and 12-2. Justification for the loading of the SFP frame has been addressed in detail in the response to Question 9.

NRC Question 13

Describe the method of leak detection in the SFP structure. How are leaks monitored? Is there any existing leakage?

PG&E Response:

The SFP leakage detection system consists of embedded, approximately 1 inch square gap channels adjacent to each side of the seams of the seam-welded stainless steel liner on all perimeter walls and the main pool floor including the fuel transfer canal used during refueling. The cask recess area floor in the pool has a perimeter channel at the base of its vertical wall surfaces only. The gap channels lead to one of six embedded, sloped collection pipes that terminate at the isolation valves in the SFP pump room sump.

The monitoring of leakage is a manual activity whereby individual leak chase isolation valves are opened and sampled for water if present. The leak detection system is entirely manual with no remote monitoring functions required. The location of the isolation valves is such that gravity collects the water in each chase and directs it towards the valves. The leak chase isolation valves are located in the SFP pump room sump and allow any leakage to be routed to the AB sump 0-1 for further processing. Surveillance Procedure STP I-1C requires that the leak chase isolation valves, for both Unit 1 and Unit 2, be checked weekly for presence of water. If water is present, it is analyzed and a determination is made if any additional action is required.

The Unit 2 SFP leakage detection system had been intermittently indicating the presence of water behind the SFP liner. The quantity of water was originally observed to be 16.2 liters, but since that time the presence of water has been checked more frequently and has been observed to average much lower volumes. A single valve, out of the six chase isolation valves for Unit 2, consistently contains water and fluctuates between approximately 0.2 and 0.45 liters of water. The other valves fluctuate between approximately 0 and 0.25 liters of water. These fluctuations have been attributed mainly to water overflow from fuel movement and washdowns during plant outages. Evaluations of the causes and inability to locate the leak source location have been performed and concluded that long-term leakage is acceptable if additional surveillance measures are taken. These additional surveillance measures have been incorporated into STP I-1C as the weekly checks of the leak chase isolation valves for presence of water. These surveillance measures are based upon the maximum volume of one of the SFP leakage detection channels to preclude the rise of hydrostatic pressure in the system.

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Table 2-1
Capacity to Demand Ratios for Various Vault Loading Conditions

	Normal (w/cask)	Abnormal (Seismic w/o cask)	Abnormal (Seismic w/cask)	Abnormal (HELB)
Shear	2.23	2.08	1.79	2.70
Flexure	0.43	1.09	0.83	1.06

Table 9-1
Load Scenario Selection Criteria

Criteria	Time of Load Scenario During the 24 Second Event
1	Largest load at either of the two top bumpers
2	Largest load at either of the two bottom bumpers
3	Largest load sum of: the top left and bottom left the top right and bottom right the top left and bottom right or the top right and bottom left bumpers
4	Largest reaction at either of the two elevation 94 ft 6 inch supports
5	Largest reaction at either of the two elevation 111 ft supports
6	Largest reaction at either of the two elevation 140 ft supports

Table 9-2
Maximum Frame Forces on Wall

Seismic Event

	DE		DDE		HE		LTSP	
	Fx	Fy	Fx	Fy	Fx	Fy	Fx	Fy
El. 140'-0"	128	122	353	320	320	375	501	406
El. 111'-0"	106	101	334	342	286	277	407	284
El. 94'-6"	109	106	320	363	282	286	458	303

Note: Fz forces are minimal. See Response 9e for coordinate system.

Table 9-3
Maximum Frame Member Stress Ratios

Section	Seismic Event	Maximum Stress Ratio	Allowable Stress Ratio
TS 12x12x1/2	LTSP	1.434	1.7
TS 8x8x1/2	LTSP	1.49	1.7
TS 8x6x1/2	LTSP	1.296	1.7
TS 8x4x1/2	LTSP	1.571	1.7
TS 6x4x3/8	HE	1.584	1.7

Table 12-1
SFP Wall Out-of-Plane Capacity vs. Demand Comparison

Seismic Event	Load Condition	Capacity/Demand
DE	Shear	2.58
	Flexure	1.86
DDE	Shear	2.05
	Flexure	1.71
Hosgri	Shear	1.88
	Flexure	1.54
LTSP	Shear	1.57
	Flexure	1.02

Table 12-2
SFP Wall In-Plane Capacity vs. Demand Comparison

Seismic Event	Load Condition	Capacity/Demand
DE	Shear	2.91
	Flexure	2.45
DDE	Shear	2.43
	Flexure	2.23
Hosgri	Shear	1.62
	Flexure	1.53
LTSP	Shear	1.59
	Flexure	1.48

LIST OF FIGURES

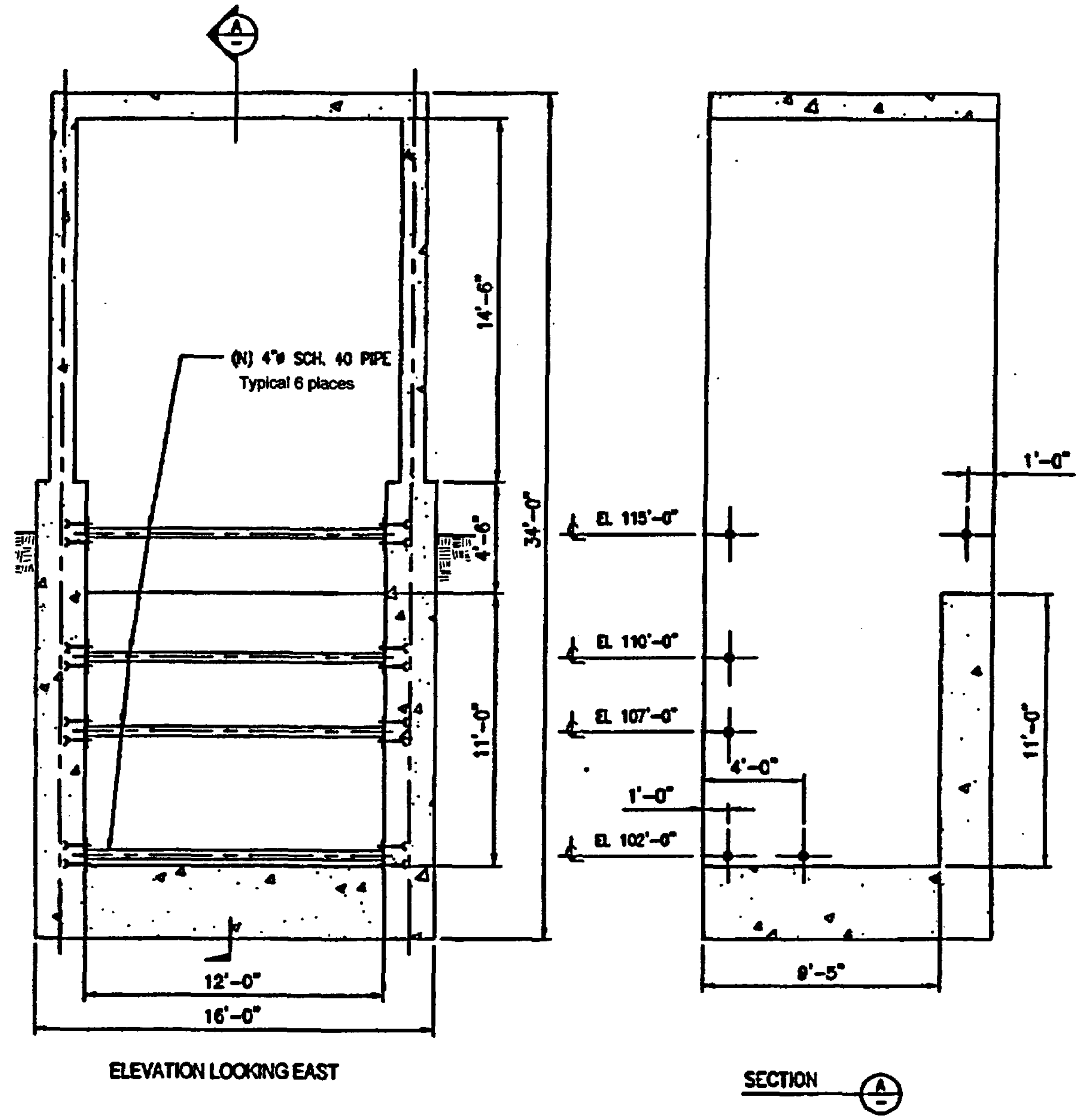

Figure 2-1 – Conceptual Vault Remediation

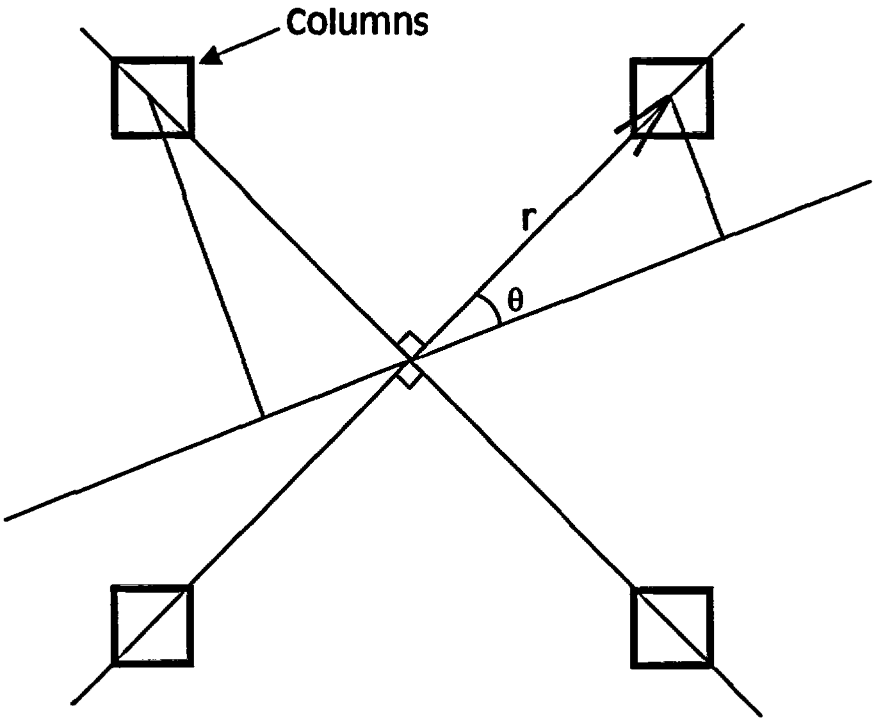

Figure 9-1

$$\text{Area Moment of Inertia} = 2A(r\sin(\theta))^2 + 2A(r\sin(\theta + 90°))^2$$
$$= 2Ar^2(\sin^2(\theta) + \cos^2(\theta)) = 2Ar^2$$

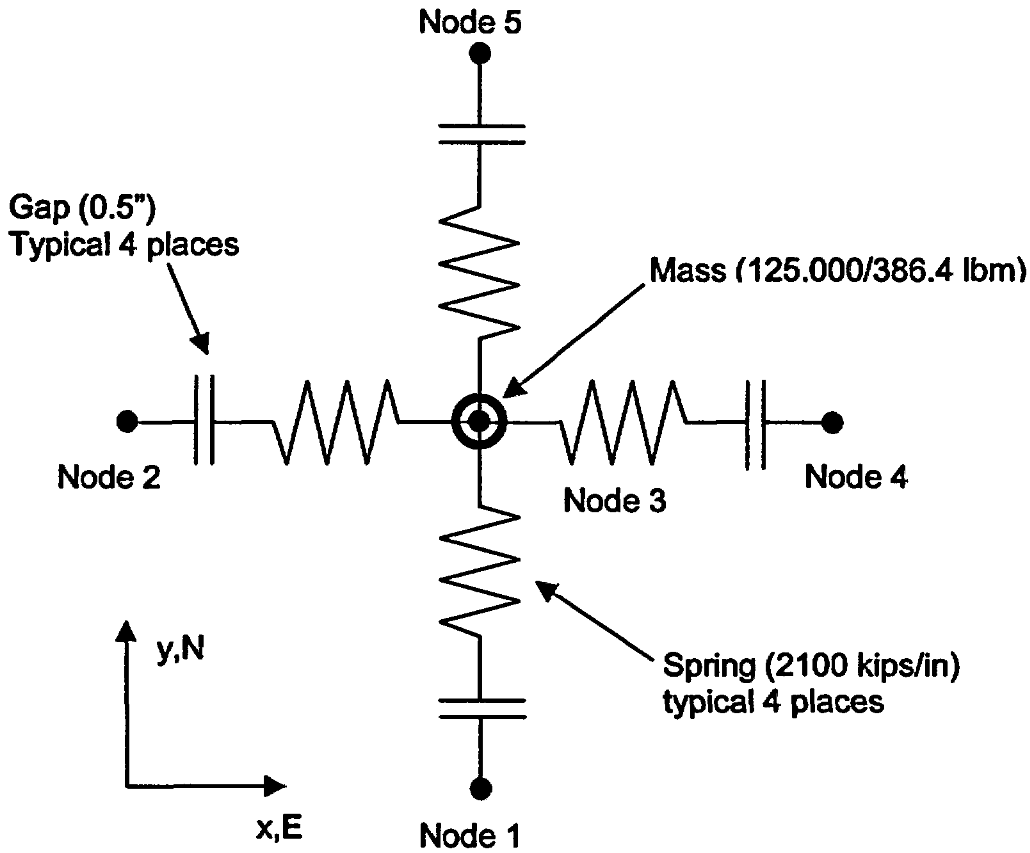

Figure 9-2 - Two Dimensional Plan Model

Note: Nodes 1, 2, 4 and 5 fixed in the x and y directions. All nodes fixed out of plane.

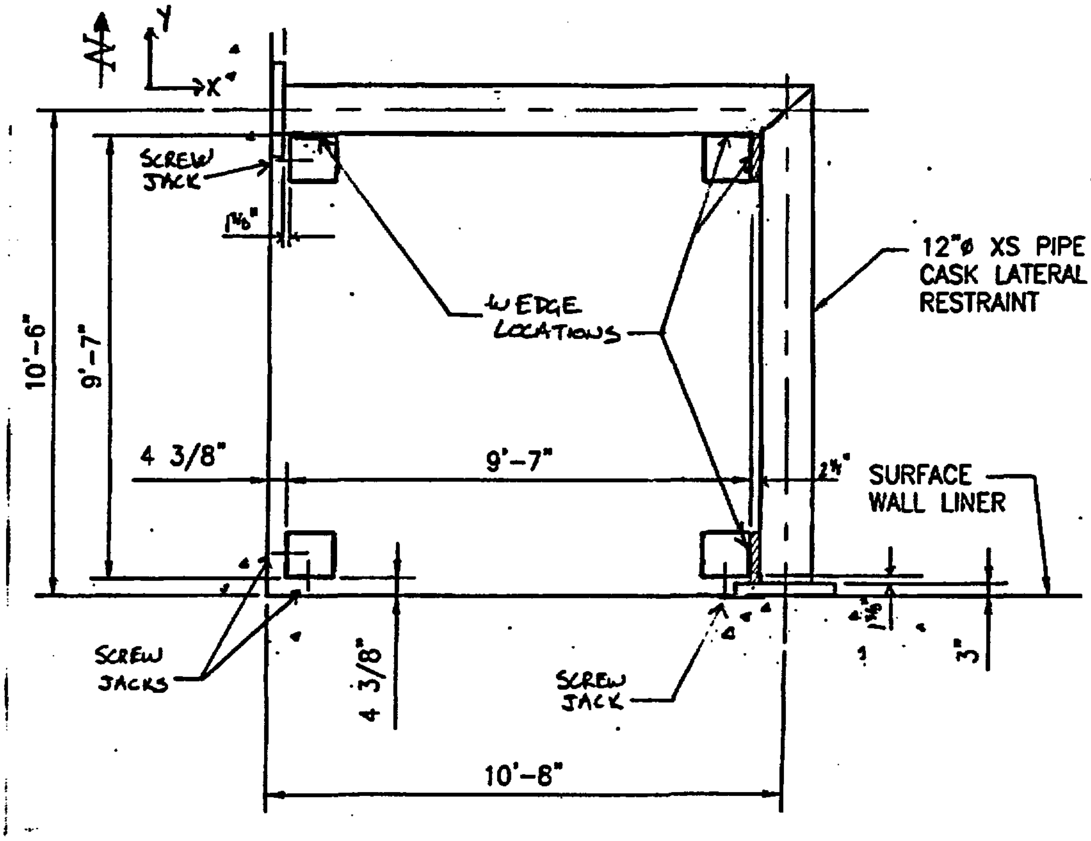

Figure 9-3 – Cask Restraint at Elevation 111 ft 0 inches

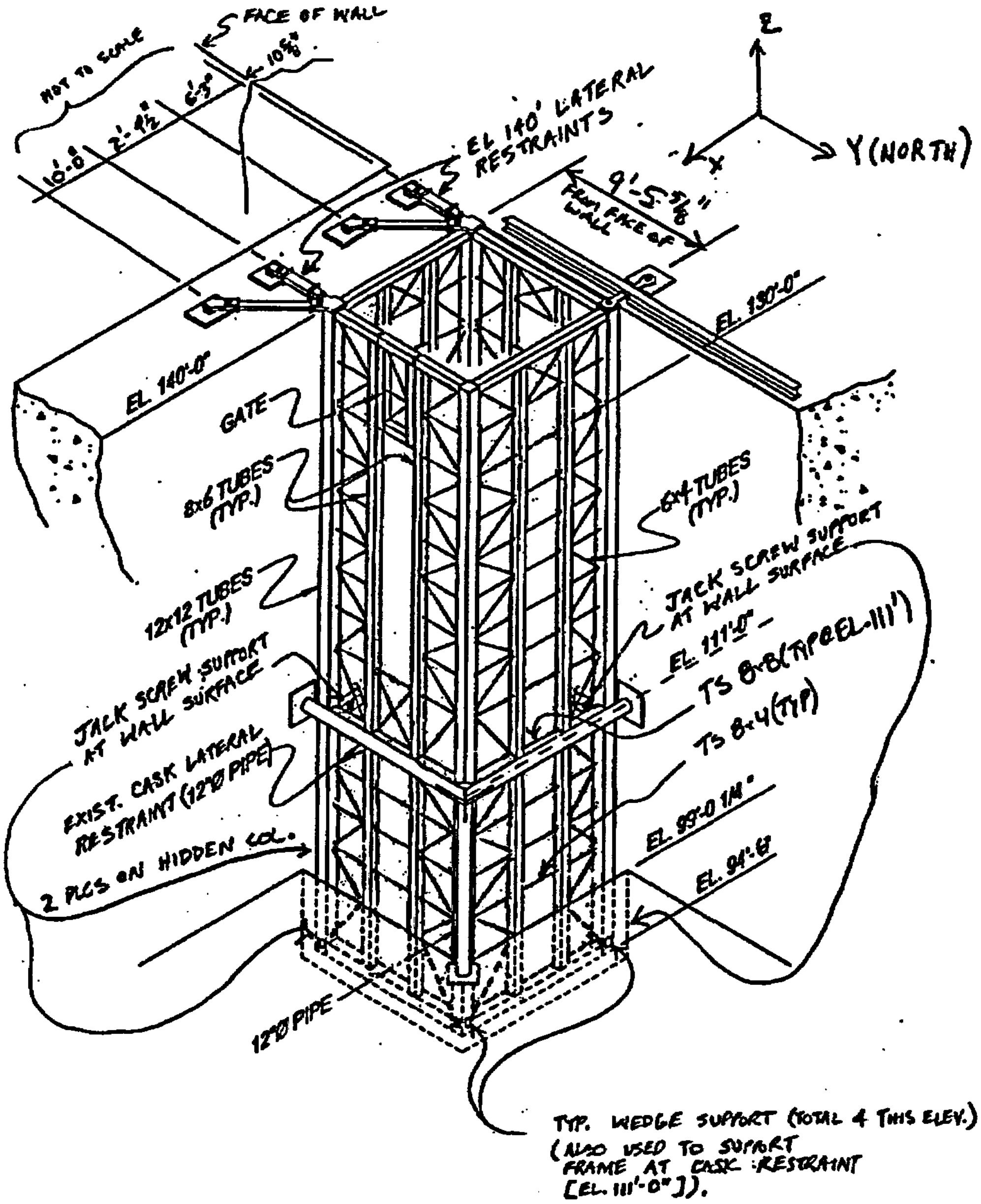

Figure 9-4 – Spent Fuel Pool Frame System Lateral Support

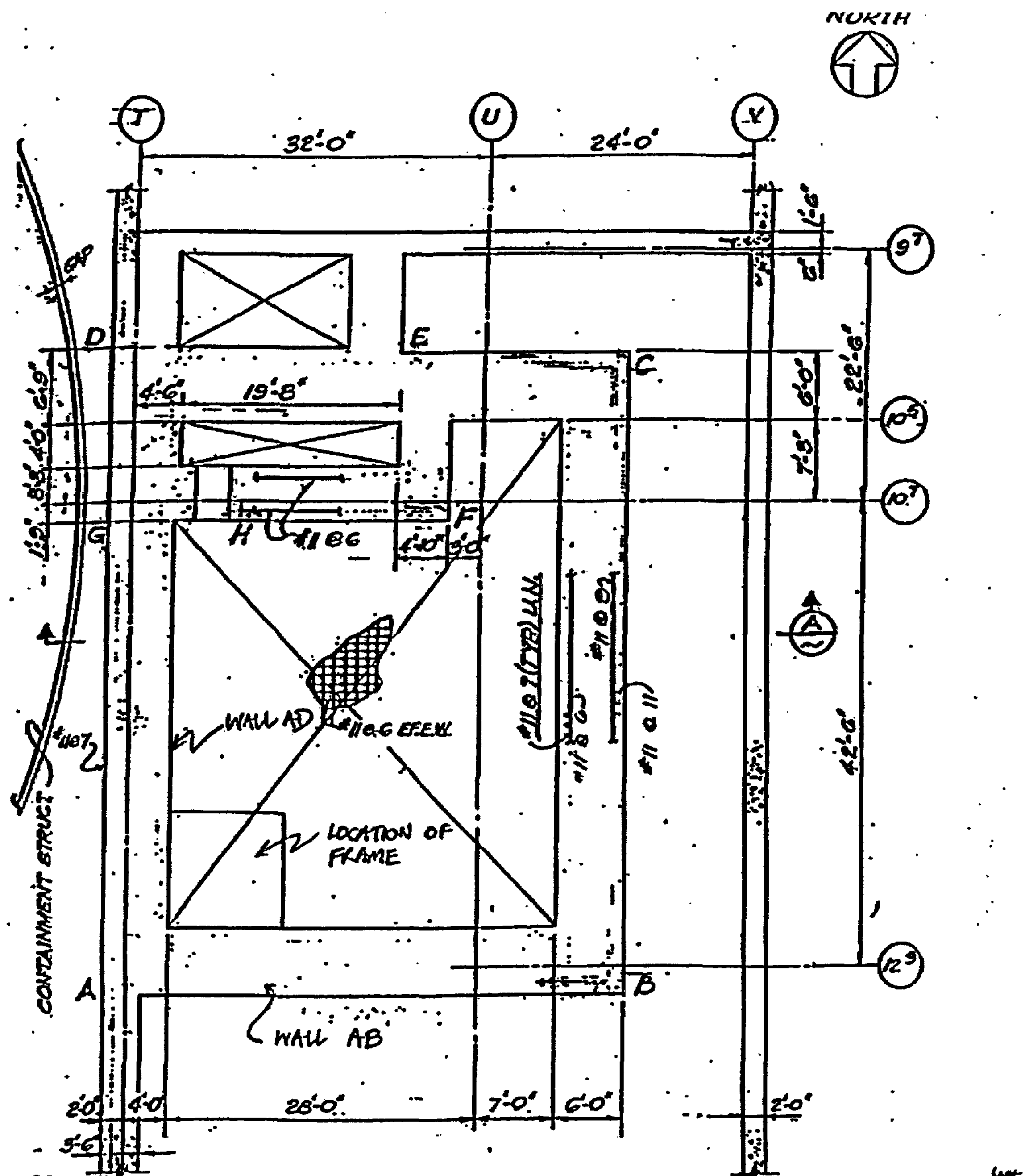

Figure 12-1 – Plan View of SFP Walls (Unit 1 shown)