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RELATED CORRESPONDENCE

DOCKETED USNRC NOV 4 1977

November 2, 1977

John F. Stolz, Chief
Light Water Reactors Branch No. 1
Division of Project Management
U.S. Nuclear Regulatory Commission
Washington, D.C. 20555

Re: Docket No. 50-275-OL
Docket No. 50-323-OL
Diablo Canyon Units 1 & 2

Dear Mr. Stolz:

Your letter dated August 24, 1977 requested additional information on design criteria for structures, components, equipment, and systems included in Amendment 50.

Enclosed are 20 copies of answers to the following questions:

3.55 through 3.60	3.67Ad	3.78A
3.62	3.67Ae	3.80 through 3.83
3.63	3.69 through 3.73	3.86
3.66	3.75	3.88 through 3.92
3.67c	3.77	3.97
3.67Ac	3.78	3.98

The remaining questions will be answered before December 1, 1977.

Very truly yours,

PHILIP A. CRANE, JR.

PHILIP A. CRANE, JR.

PAC:vm
Enclosures (20)
cc w/enc.: Service List

RELATED CORRESPONDENCE

PARTIAL RESPONSE TO NRC
LETTER OF AUGUST 24, 1977

3.55 On Page 2-7, Par. 2, the statement is made that "for non-linear analysis that considers gaps between supports, impact damping is also considered." Define the term "impact damping," providing the value assigned thereto, and the method of its use and justification therefor.

Response:

Impact damping is representative of energy losses resulting from impact and is represented in mathematical models by dash-pots acting as viscous dampers during impact. The viscous damper is adjusted in the model to produce an energy loss not to exceed experimentally determined values. These values are expressed as coefficients of restitution and/or impact forces which are then converted to damping percentages.

Impact damping was used only in the analyses of the CRDM's (5 percent) and fuel (25 percent). These values have been justified by tests.

The design of structures was made on an elastic basis and did not require consideration of impact damping.

3.56 On Page 2-10, it is stated that a component or piping was considered to be rigid if the natural period was less than 0.05 seconds (2.2.3.3) and 0.066 seconds (2.2.3.5). Indicate if the same criteria were used in the reevaluation. If so, please provide justifications. It is normally the staff's position that a component or equipment is considered rigid if its natural frequency is equal to or above 33 cps.

Response:

Chapter 2 of Amendment 50 is concerned with the original design of the plant. At that time, components and piping were considered to be rigid if the natural period was 0.05 second or less, as stated in Paragraph 2.2.3.3. Paragraph 2.2.3.5 is a brief description of a particular method of qualifying piping and has been discussed with the Mechanical Equipment Branch. For the Hosgri evaluation, equipment is considered to be rigid if there is no natural frequency below 33 Hz.

3.57 On Pages 2-11, and 2-15, a seismic acceleration of "2g" is quoted as the upper bound used for certain piping systems. Indicate if the same criterion was used in the reevaluation. If so, justification should be provided since some in-structure response spectra may have special acceleration higher than 2g.

Response

The design of the piping system restraints for the Design Earthquake (DE) used an upper bound seismic support load of 2g. In the Hosgri reevaluation, the various loading values were based on the Hosgri spectra accelerations.

3.58 Define the term "State vector" used in Page 2-16 and other places.

Response:

The term "State Vector" is used to refer to the set of forces (F_x, F_y, F_z) and moments (M_x, M_y, M_z) acting at a point in the piping equipment.

3.59 On Page 3-10, last paragraph, the statement is made: "....the probability of exceeding the .75g effective ground acceleration in 50 years is 0.1% or an average return period of 52,600 years...." Although these values are consistent, the statement is not worded clearly and is subject to misinterpretation. Please clarify.

<u>Response</u>:

In response to a recommendation contained in the letter from Raymond F. Fraley, Executive Director, ACRS to Benard C. Rusche, Director, Office of Nuclear Reactor Regulation, dated December 20, 1976, a probabilistic assessment of seismic risk for the plant was undertaken.

This report, which is contained in Appendix D-LL11, presents the results of an intensive study including analysis of the seismic history of a large area representative of the site conditions; development of return period-magnitude relationships under the assumption that earthquakes as large as 7.5M can be generated by the Hosgri and other fault zones near the Diablo Canyon plant as well as a great earthquake on the San Andreas fault; development of a source-to-site model; use of appropriate attenuation relationships; development of peak site accelerations from all possible sources; and development of spectral response accelerations for various damping ratios and probabilities of exceedance. From this work have been determined the acceleration levels as well as probability relationships for the entire chain of phenomena from the faults to the plant. Finally, the data were related to the spectra used in the plant design and in the Hosgri evaluations.

The results of this study indicate that the probabilities of exceedance of the Hosgri ground accelerations and spectral response accelerations during the life of the plant are exceedingly small, for example, the probability of exceeding the .75g effective ground

3.59 (Continued)

acceleration in 50 years is 0.1%, while the probability of exceeding the 7% spectral response accelerations varies from .01% to .08%, depending on period. If one were to consider a very long time period (i.e., several million years) one would expect that, on the average, 0.75g effective acceleration at the site would be exceeded once every 52,600 years under the assumption of 7.5M earthquake opposite the plant in the Hosgri fault zone.

3.60 On Page 4-3, last paragraph, the statement is made that, in the DDE analyses, the vertical response and one horizontal response were combined on an absolute sum basis. This is inconsistent with the statements on Page 2-6 (see Question 1). Please clarify.

<u>Response:</u>

The DDE analysis of the structures used the absolute sum of one horizontal and one vertical response.

3.62 On Page 4-3, Par 5, the statement is made that "this eccentricity was either 5%....or....7%.... with the translational results, whichever was greater."

This statement does not indicate how the response due to 5% additional eccentricity was combined with the translational results. Please clarify.

Response:

The response due to 5% additional eccentricity was combined with the translational results on an absolute sum basis.

3.63 On Page 4-5, Par. 3, the statement that "this is in accordance with.... greater than 3000 fps" is incorrect. The number 3000 fps should be 3500 fps in accordance with Section 3.7 of the SRP.

Response:

The number "3000 fps" in Paragraph 3 of Page 4-5 should read "3500 fps" in accordance with Section 3.7 of the Standard Review Plan.

3.66 On Page 4-7, Par. 2, the statement is made that "in generating the floor response spectra for.... elastic and inelastic using.... of material strength." Explain how floor response spectra were generated by means of an inelastic analysis of the building.

Response:

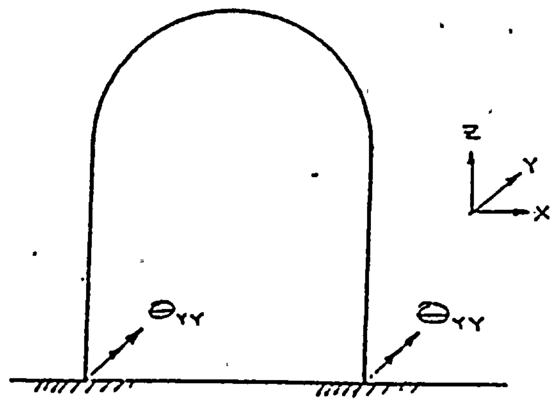
Because proposed modifications to the Turbine Building prevent inelastic responses to the Hosgri Earthquake, it was not necessary to generate floor response spectra using an inelastic analysis.

3.67 On Page 4-10, Paragraph 4.2.1.1, clarify:

(c) How does the fixed model as shown in Figs. 4-24 and 4-26 take account of the pin connection at the containment base?

Response:

The pin connection at the base of the Containment was accounted for by releasing the rotational degree of freedom θyy (see figure) at the base of the axisymmetric model.



3.67A On Page 4-9, Section 4.2.1, please clarify the following:

(c) For responses combined on an absolute sum basis, Equation (1) on Page 4-11 should be revised to read:

$$H_T = |H_{TR}| + |X * H_{TOR}|$$

<u>Response:</u>

For response combined on an absolute sum basis, Equation (1) on Page 4-11 should be revised to read:

$$H_T = |H_{TR}| + |X * H_{T05}|$$

3.67A On Page 4-9, Section 4.2.1, please clarify the following:

(e) On Page 4-11, H_{T05} was defined as the torsional response at the center line of the containment. In Fig. 4-26, the coordinate axes were defined at each mass point. Clarify how the difference between these definitions was accounted for in the analysis.

Response:

In Fig. 4-26, the degrees of freedom of the structure were defined at the mass points. However, torsional responses were monitored at the centerline of the containment and used in the analysis.

3.69 On Page 4-13, Section 4.2.1.2, clarify what were the criteria used in constructing the equivalent axisymmetric model and how was the axisymmetric finite element model modified to analyze the shear walls unsymmetric about the structural axis?

Response:

In accordance with the "Specification for Seismic Review of Major Structures for 7.5M Hosgri Earthquake," February 2, 1977, revised February 8, 1977, the procedures used in the DDE analysis remain valid. For the DDE analysis, an axisymmetric model possessing a shear stiffness equivalent to the actual structure was used. For the Hosgri reanalysis, a check was made to verify that the shear stiffnesses were equivalent and that the difference in bending stiffnesses between the axisymmetric model and the actual structure was negligible.

3.70 Fig. 4-20 shows that the idealized center line of the containment and its interior structures does not coincide with that of the reactor vessel. Clarify how this effect is accounted for in the axisymmetric model.

Response:

The reactor vessel is one of many elements making up the total mass of the structure. While the reactor vessel is eccentric to the centerline of the containment, the other masses with their respective locations combine to yield a negligible total eccentricity.

3.71 Provide justifications for neglecting walls above El. 140' in the mathematical model for the interior structure.

Response:

The walls above El. 140' in the interior structure were included in the mass of the mathematical model, but because they are free-standing, they do not contribute to the stiffness of the structure. Stresses in the walls have been checked using the floor response spectra of the 140' level applied at the base of the walls.

3.72 Fig. 9 cited on Page 4-18, Par. 3 is not included. Please provide.

Response:

Figure 9 cited in Paragraphs 3 and 4, Page 4-13 should read "Figure 4-27."

3.73 On Page 4-13, Par. 4, the statement is made that "Model 1 in---- to the applied loads." Since the model is a coupled model which includes both geometric and 5% equivalent eccentricities, the contribution from the equivalent eccentricity cannot be separated from that of the geometric eccentricity. Clarify how the ABS (or SRSS) of the response due to 5% (7%) eccentricity and the remaining responses were combined.

Response:

The effects of the geometric and accidental eccentricities were not separated since the torsion produced due to both kinds of eccentricity is the relevant response. The analyses of the interior structure yielded a horizontal response at the structure axis from the axisymmetric model shown in Figure 4-24 and a torsional response (including the effect of both types of eccentricities) at the structure axis from the model shown in Figure 4-27. These two responses (horizontal and torsional) were then used to compute the total horizontal response (H_T) according to equations 1 and 2, and the higher value from these equations was used to check the structure and components.

3.75 On Page 4-15, the last paragraph refers to the combination of effects due to horizontal input motions. It is not clear that in all cases the horizontal motion effects are due entirely to one direction of motion. For example, the maximum vertical stress due to motion in one direction may occur at the same time as the maximum shear due to motion at right angles to that direction. Please explain.

<u>Response</u>:

In an axisymmetric shell each component of the ground motion will produce 3 sets of responses at a given point in the structure (radial, tangential, and longitudinal). The SRSS combination of similar responses due to different components of ground motion gives the sum of the three radial responses at a point due to the three components of ground motion. Therefore, maximum vertical and shear stresses are not combined in the results shown in Tables 4-9 and 4-10.

The last paragraph on Page 4-15 explains that the horizontal responses at a given elevation due to the horizontal components of ground motion vary as a cosine function, as shown in Figures 1 and 2 below. As an example, consider the absolute radial accelerations. Let α_{N-S} be the absolute radial acceleration response due to the N-S component of ground motion, and let α_{E-W} be that due to the orthogonal component. The distribution of these accelerations is shown in Figures 1 and 2. The SRSS combination of the radial acceleration (α^{H}_{radial}) at any point A due to these components of ground motion is:

$$(\alpha_{radial})^{H} = \sqrt{\alpha_{EW}^{2}\cos^{2}\beta + \alpha_{NS}^{2}\cos^{2}\alpha} \qquad (1)$$

$$\sin\alpha = \cos\beta$$

$$\therefore (\alpha_{radial})^{H} = \sqrt{\alpha_{EW}^{2}\sin^{2}\alpha + \alpha_{NS}^{2}\cos^{2}\alpha} \qquad (2)$$

$\alpha_{EW} = \alpha_{NS}$ For axisymmetric shell subjected to equal horiz. ground motion components.

$$\therefore (\alpha_{radial})^{H} = \alpha_{NS}\sqrt{\sin^{2}\alpha + \cos^{2}\alpha} = \alpha_{NS}$$

3.75 (Continued)

The total maximum SRSS radial acceleration due to the three ground motion components is:

$$(\alpha_{radial})^{Total} = \sqrt{[(\alpha_{radial})^{H}]^2 + [(\alpha_{radial})^{V}]^2}$$

$$= \sqrt{\alpha^2_{NS} + [(\alpha_{radial})^{V}]^2}$$

where $(\alpha_{radial})^{V}$ = Max. radial acceleration at point A due to vertical ground motion component.

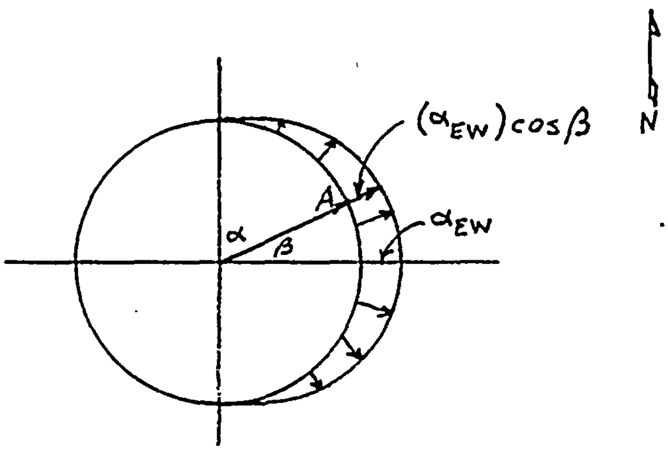


Fig 1 Response Due to E-W Component of Ground Motion

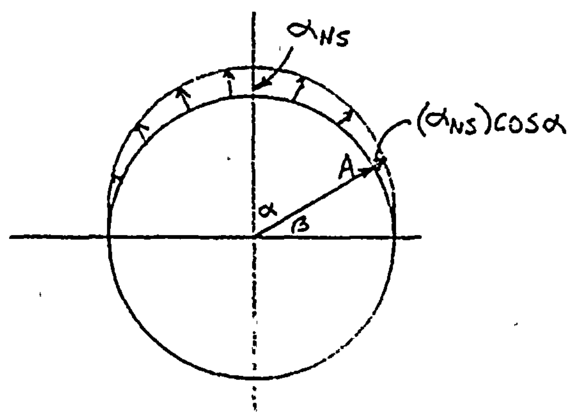


Fig 2 Response due to N-S Component of Ground Motion

3.77 On Page 4-16, Par. 2, the statement is made that "in specific situations, a reduction....cannot be reduced." The staff position has been that in using Newmark Spectra, the reduction due to ductility consideration is not permitted. Delete the words "in general" or clarify your intention.

Response:

On Page 4-16, the second sentence of Paragraph 2 should read, "The Blume input generally produced a higher structure response than the Newmark input, but the criteria outlined in Section 4.1 allow, in specific situations, a reduction of the response due to the Blume input by considerations of ductility, while the responses due to the Newmark inputs cannot be reduced."

3.78 Equation (5) on Page 4-16 should be revised as follows:

$$g * A_H = | g * H_{HCL} | + | X * A_{TCL} |$$

Response:

Equation (5) on Page 4-16 should be revised as follows:

$$g * A_H = |g * H_{HCL}| + |X * A_{TCL}|$$

3.78A Table 11a cited on Page 4-17 is not included and should be provided.

Response:

Table 11a cited on Page 4-17 should read "Table 4-15."

3.80 With reference to the evaluation of the 200 Ton Polar Gantry Crane on Pages 4-18 and 19, clarify the following:

(a) What was the input used in the analysis?

(b) What was the boundary conditions of the mathematical model shown in Fig. 4-29?

(c) What were the natural frequencies of the crane system?

Response:

(a) Newmark Hosgri response spectra (Figures 4-71, 4-81 and 4-86) were used as input.

(b) Hinge supports are assumed at Points 1, 2, 3, and 4.

(c) The following are the natural frequencies of the crane system:

Horizontal-longitudinal	(x)	- 1.05 CPS
Horizontal-transverse	(z)	- 3.57 CPS
Vertical	(y)	- 6.25 CPS

3.81 On Page 4-20, Par. 4, the statement is made that "The combined translational and torsional spectra---- shown in Figs. 4-30 to 4-47." Clarify how this was done with a mathematical model as shown in Fig. 4-24.

Response:

The horizontal spectra at any point in the containment shell have four components. One component each is due to the E-W, N-S and vertical components of the ground motion. The four component is the horizontal response due to torsion about the containment axis. These four horizontal responses are combined on an SRSS basis. The Response to Question 3.7 describes how the floor response spectra due to the two horizontal ground motions are combined. Call this combined floor response spectra S_1. The torsional floor response spectra from analyzing the model shown in Figure 4-26 yields another horizontal floor response spectra, S_2, by multiplying the torsional floor response spectra at the structure axis by the radius of the shell. Finally, at each level there is a horizontal floor response spectra, S_3, due to the vertical component of the ground motion. Hence, the final horizontal floor response spectra, S_H, of Figures 4-30 to 4-47 for each frequency, f, is:

$$(S_H)_f = \sqrt{(S_1)_f^2 + (S_2)_f^2 + (S_3)_f^2}$$

3.82 Clarify if there is any difference between Equations (1) on Page 4-11 and (5) on Page 4-16.

Response:

Equation (1) on Page 4-11 is a general equation relating to any horizontal response; Equation (5) on Page 4-16 is specifically applicable only to horizontal accelerations.

3.83 How was the floor response of the auxiliary building at El. 125'-0" computed? The model, Fig. 4-108, did not show a mass point at this elevation.

Response:

The floor at El. 128'-0" is a partial floor, located midway between floors at El. 140'-0" and El. 115'-0". In the structural dynamic analysis the mass of this partial floor was distributed to the upper and lower floors. The floor acceleration at El. 128'-0" was obtained by interpolation between computed floor accelerations at El. 115'-0" and El. 140'-0".

3.86 On Page 4-26, Par. 3, the statement is made that "locations of the centers of mass....of vibration in the analysis." In order to accomplish this, the location and mass of each and every piece of equipment must already be known. Explain why a 100 psf load had to be used in the computation of lateral masses to account for these equipment.

Response:

In accordance with the "Specification for Seismic Review of Major Structures for 7.5M Hosgri Earthquake," February 2, 1977, revised February 8, 1977, the procedures used in the DDE analysis remain valid. The mass of the accumulated equipment was calculated to be equivalent to a 100 psf load in the DDE computer model and remains valid for the Hosgri analysis.

3.88 It is indcated on Page 4-34, Par. 1 that the detailed review of critical shear walls considered the effects of embedded columns, "flange" effect of perpendicular walls, and variable wall thickness. Elaborate on the method used for the analysis of these effects.

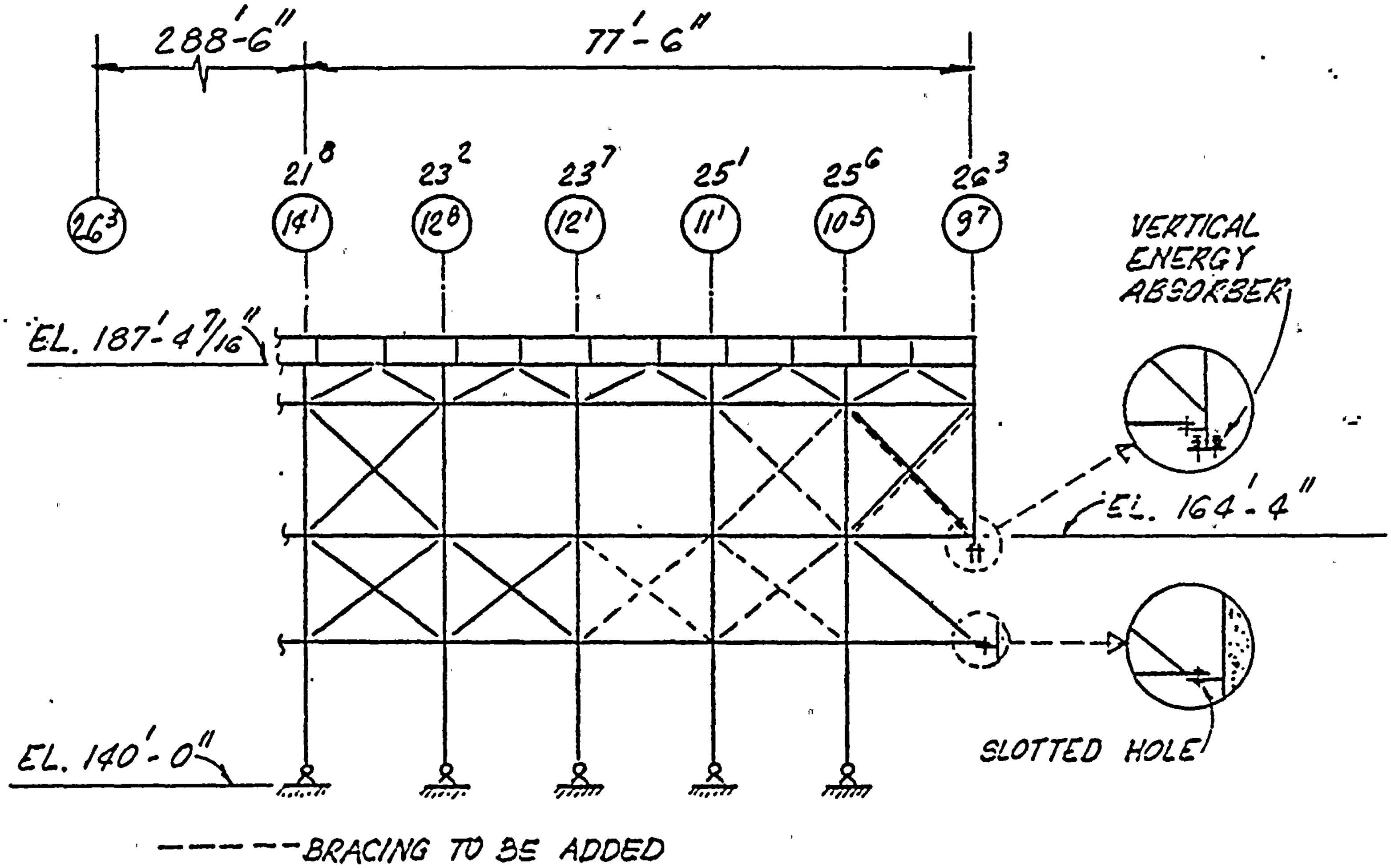
Response:

No credit was taken in the original design for the additional structural capacity which could be obtained by considering the effects of embedded columns, the "flange effect" of perpendicular walls, and variable wall thickness. Nor is there any need for these factors to be included in the Hosgri evaluation. Adequate safety margins have been provided by consideration of the additional strength resulting from reinforcing quantities exceeding design requirements, and from actual values of material properties which are better than design values.

3.89 The details of the fuel-handling crane support structure referred to on Page 4-36 are not included. Please provide.

<u>Response</u>:

Specific modifications for the fuel handling area are presented in Figures 4-164 through 4-166. Partial release of the supports shown in Figure 4-166 enables the structure to deflect uniformly throughout its entire length under earthquake loads in the E-W direction; hence the seismic loads will be evenly distributed to all column anchorages, rather than being taken primarily by the anchor bolts of the end bents. Also, the torsional stresses introduced previously by the rigid condition of the end bents will be reduced considerably. The release of the supports shown in Figures 4-164 and 4-165 provides, for the N-S direction, the same solution as described above. The added bracing eliminates uneven distribution of the pullout forces on the column anchorages. As a result of these modifications, critical stresses in the anchorages will be less than allowable.

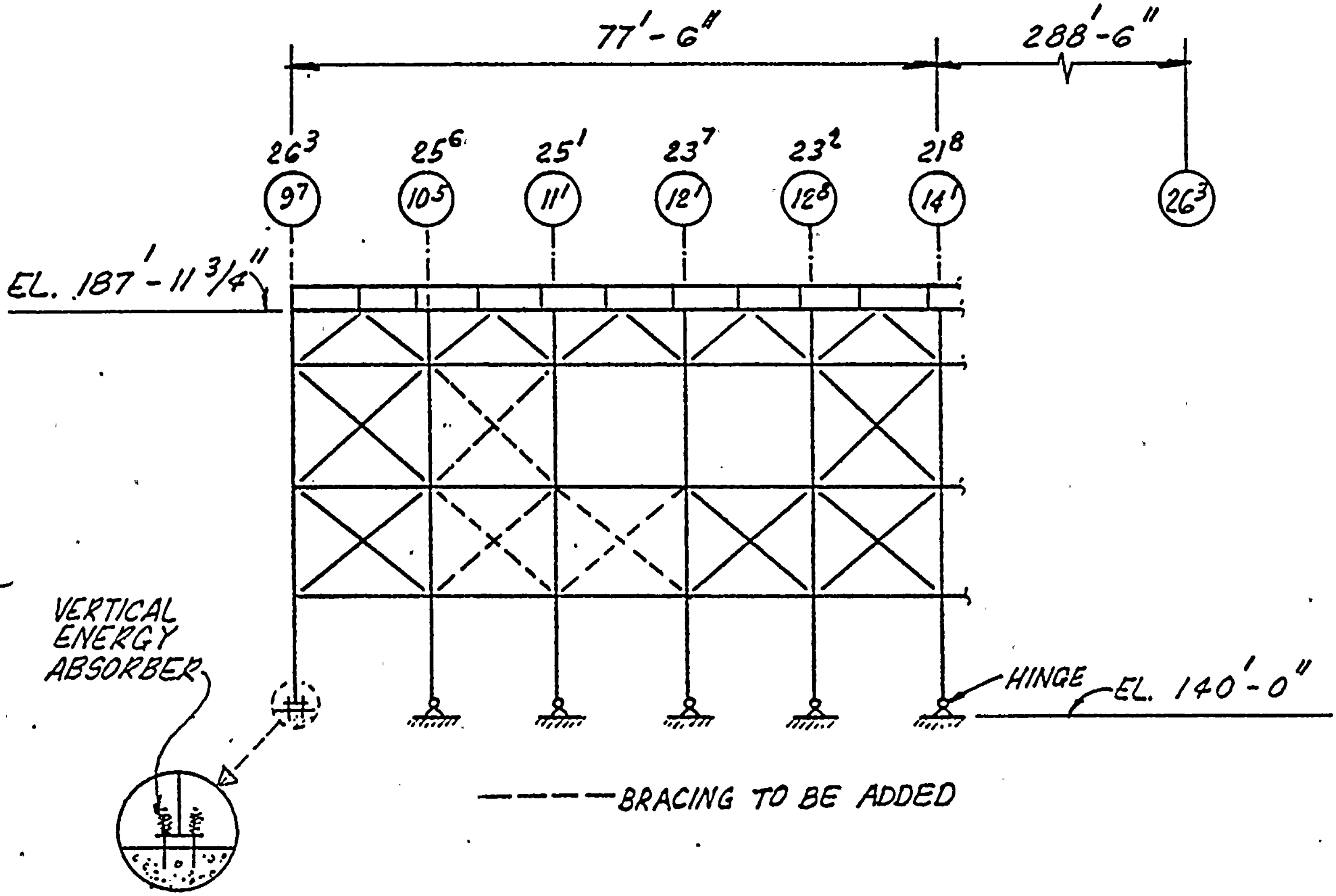


MODIFICATIONS IN FUEL HANDLING CRANE SUPPORT STRUCTURE

EAST ELEVATION

FIG. 4 - 164

DIABLO CANYON
AUXILIARY BUILDING

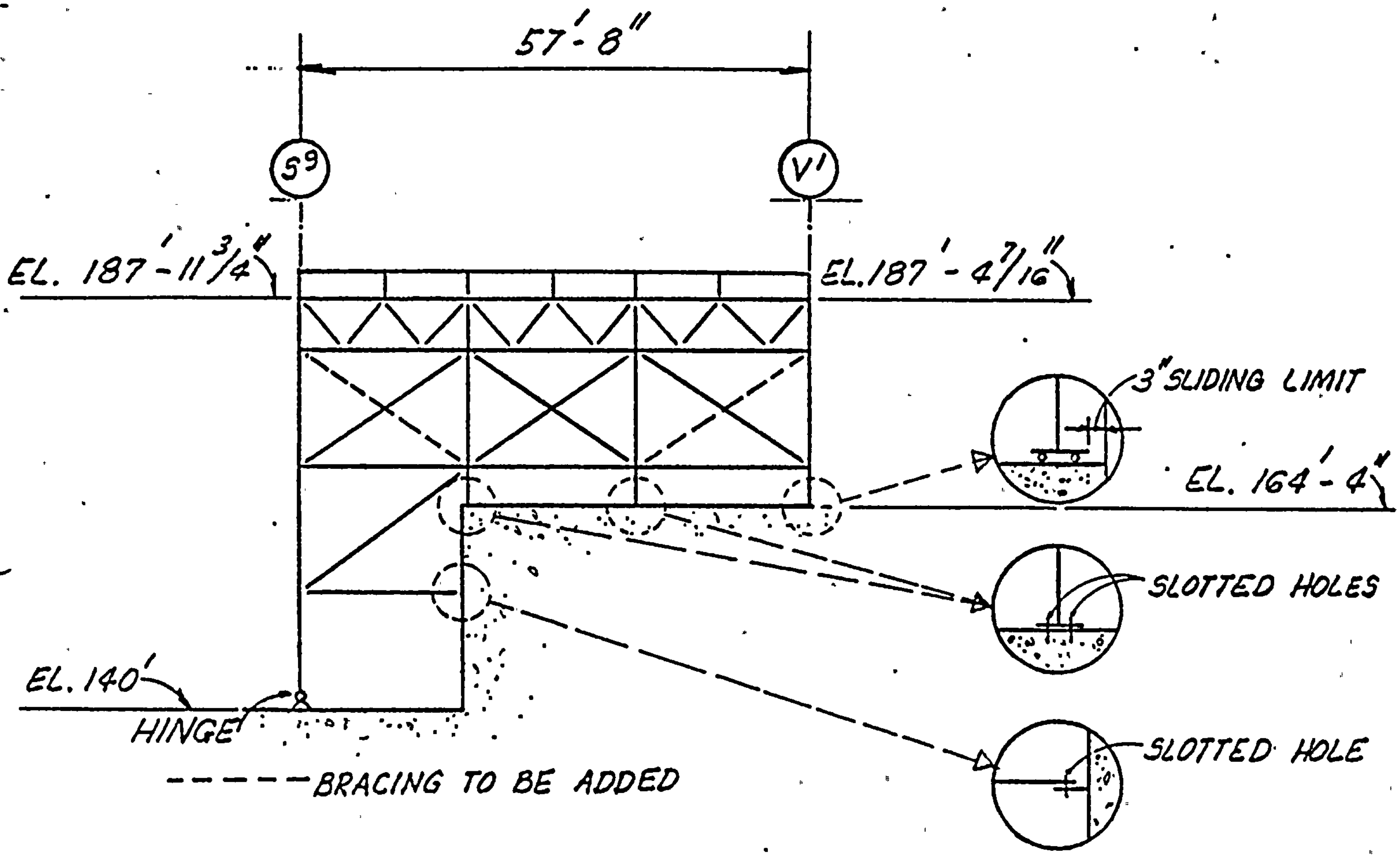


MODIFICATIONS IN FUEL HANDLING CRANE SUPPORT STRUCTURE

WEST ELEVATION

FIG. 4-165

DIABLO CANYON
AUXILIARY BUILDING



MODIFICATIONS IN FUEL HANDLING CRANE SUPPORT STRUCTURE

SOUTH ELEVATION

NORTH ELEVATION OPP. HAND

FIG. 4 - 166

DIABLO CANYON
AUXILIARY BUILDING

3.90 On Page 4-37, Par. 6, the statement is made that "For a north-south input,......according to Equation (4)." It is not clear which Equation (4) was referred to. Please clarify.

Response:

In Paragraph 6 on Page 4-37, Equation (4) refers to the equation

$$g * A_H = g * A_{HCL} + X A_{TCL} \tag{4}$$

found at the bottom of Page 4-32.

Similarly, the equation

$$\%PF_i = \frac{PF_i}{\sum_{i=1}^{n} PF_i} * 100 \tag{3}$$

at the top of Page 4-32 should be denoted as Equation (3).

3.91 With reference to Sections 4.4.5 and 4.4.6 on Pages 4-43 and 4-44, please indicate if there was any analysis performed after the modifications. If so, what were the resulting stresses and/or deformations? Elaborate on whether an elastic and inelastic analysis was employed in the computation.

Response:

Work is currently being performed to determine the stresses and deformations of the modified Turbine Building when subjected to the Hosgri earthquake. It is possible that the two end frames of the structure, Lines 1 and 35, will undergo inelastic behavior; the remainder of the structure will remain elastic.

3.92 Figs. 4-308 and 4-309 used as seismic input for the analysis of the intake structure appear to correspond to the longer dimension of the intake structure. Justify the use of these spectra for the analysis in the shorter dimension.

Response:

The response spectra shown in Figs. 4-308 and 4-309 used as seismic input for the analysis of the intake structure correspond to a tau-filtered value of 0.04.

This value results from the use of the dimension of an equivalent area square foundation. These spectra were applied in both horizontal directions. Consideration of the actual foundation dimensions would require spectra in the north-south and east-west directions corresponding to tau values of 0.05 and 0.03, respectively. Use of the tau = 0.05 spectra would, of course, result in lower forces than those calculated. Use of the 0.03 spectra would result in higher accelerations, but considering the fact that the embedment on three sides of the structure was neglected in the analysis, it is believed that the use of the spectra shown in Figures 4-308 and 4-309 is justified.

3.97 Table 4-8 shows vertical displacements due to both horizontal seismic inputs and vertical inputs. Explain how you obtain vertical displacements due to horizontal inputs in an axisymmetric structure on a fixed base. In addition explain why the displacements are less near the crown of the dome than further down at the cylindrical section of the containment building.

Response:

As a result of the overturning moments and rotations from the horizontal seismic input, there will be local vertical displacements of the axisymmetric structure, one side of the centerline displacing upward, the other side an equal amount downward. Because the total vertical displacement of a point is proportional to the rotation and the distance of the point from the centerline of the structure, the spring line of the dome experiences the maximum vertical displacement from the horizontal input. Points above the spring line displace less due to the reduced radius, while points below the spring line are increasingly restrained by the base as one moves downward on the structure.

3.98 In addition to the shears included in Table 4-11, provide the shear forces in the dome shell due to the vertical input. In what directions are the shears in the shell computed and how are they combined with the net shear forces on the wall.

Response:

Calcuation of shear forces in the dome due to the vertical input is possible using the data provided in the analysis. These forces would then be combined with dead and live load forces to obtain the total shear forces in the dome.