PORTLAND GENERAL ELECTRIC COMPANY
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PORTLAND, OREGON 97204

D. J. BROEHL
ASSISTANT VICE PRESIDENT

October 10, 1978

Trojan Nuclear Plant
Docket 50-344
License NPF-1

Director of Nuclear Reactor Regulations
ATTN: Mr. A. Schwencer, Chief
Operating Reactors Branch #1
Division of Operating Reactors
U. S. Nuclear Regulatory Commission
Washington, D. C. 20555

Dear Sir:

Attached are responses to the NRC Staff questions of October 2, 1978 based on information provided by Bechtel in confirmation of telephone conversations between Portland General Electric Company (PGE), Bechtel and the NRC Staff.

This letter and attachments are being served on the Atomic Safety and Licensing Board (ASLB) and all parties to the Control Building Hearings.

Sincerely,

DJBroehl

CLARIFYING INFORMATION TO SUBMITTALS ENTITLED

TROJAN CONTROL BUILDING
SUPPLEMENTAL STRUCTURAL EVALUATION
SEPTEMBER 19, 1978

and

RESPONSE TO QUESTIONS FROM THE
NUCLEAR REGULATORY COMMISSION
DATED AUGUST 30, 1978
September 20, 1978

QUESTION # 1

"On Page B-3 of your submittal, Equations 3 and 4, there is a statement about σ_n being assumed to be equal to v_c'. Justify that statement."

CLARIFICATION # 1

To develop the basic criteria (Figure 4-1 and Appendix B), the empirical relationship obtained by Schneider was used as a basis. Schneider's test specimens (those used for establishing the basic criteria) had overall height equal to overall length The load was applied diagonally to the square specimens which had struts on each side of the pier. The testing mechanism, when studied in detail, showed that each strut received some fraction of the vertical component of the load. The values of this component in these struts are not documented in the testing report. It was assumed that the total vertical component of the diagonally applied load is resisted by the pier tested. Therefore, $\sigma_n = v_u$ (test). It is noted that if the amount of compressive force in each strut were known, then the compression force in the pier would be less than v_u and for the correlation in Figure B-3, the calculated values for Schneider's test specimens would have been lower. Therefore, the assumption $\sigma_n = v_u$ (test) is conservative. For the Berkeley test data, the actual compressive stress is documented and was used for the correlation.

A clarification of the designations used on Page B-3 of Appendix B of the Trojan Control Building Supplemental Structural Evaluation is appropriate. Equation 2 refers to ACI 318-71; v_c' in this equation represents the ultimate shear stress of the concrete without contribution of the reinforcement. Generally, v_u represents the ultimate shear stress capacity including the contribution of the reinforcement. In the basic criteria, the contribution of the reinforcement was not explicitly included as a parameter. Therefore, v_u in equation 5 corresponds to v_c' in equation 2.

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QUESTION # 2

"Provide a basis for breaking up the walls between column lines with the new criteria as opposed to not doing it under the older criteria. Justify why it is different and yet does not violate any conclusions from the old analysis."

CLARIFICATION # 2

In the supplementary evaluation, the determination of shear stress capacity is dependent on height-to-length ratio (H/W) of walls. Therefore, dividing the walls into appropriate segments was required. This division was based on continuity or discontinuity of the horizontal reinforcement in the core. If the steel column was encased in the core with continuous horizontal core reinforcement, then the wall was considered as continuous and the overall length was taken. If the core horizontal reinforcement was interrupted by the encased steel columns, then the wall was divided accordingly. If the walls were not divided into segments, capacity would be higher and thus less conservative.

In the criteria used in the re-evaluation study, the shear capacity is not dependant on H/W ratio as a parameter. Rather, the shear capacity is a function of the concrete strength and the percentage of reinforcement. Therefore, in the re-evaluation study dividing the walls into segments was not necessary.

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QUESTION # 3

"Justify the use of 150 psi on block walls without horizontal reinforcement as an upper limit."

CLARIFICATION # 3

For walls without core steel, the ultimate shear stress capacity is taken as 150 psi. The explanation of this limiting value is as follows:

1. The allowable shear stress, according to the UBC 1967, is 50 psi for members with no shear reinforcement (without 1/3 increase for earthquake loading allowed in the UBC). For masonry-type structures, the minimum factor of safety used in arriving at elastic allowable shear stresses is taken as three*. Therefore, it is reasonable to take the ultimate shear stress as 150 psi.

2. The value 150 psi is approximately $2\sqrt{f'_c}$ (for $f'_c = 5000$ psi) which is only the contribution of the concrete in the criteria presented in the re-evaluation study, where $v_u = v_c + v_s$.

*See "Building Code Requirements for Concrete Masonry Structures," proposed ACI Standard, ACI Journal (August 1978); and also commentary, ACI Journal (September 1978) p. 485.

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QUESTION # 4

"Describe the manner of installation of the drypack and discuss in detail the effect of the drypack on the dowel capacity and overall wall capacity."

CLARIFICATION # 4

Drypack was used in the construction of some of the interior walls. The drypack used was a stiff mortar. Since strength increases with a reduction in the water-cement ratio, the stiff mortar has a strength equal to or greater than the mortar used in the joints of the block masonry.

Different considerations govern the installation techniques for walls with a concrete core and walls composed of block only.

In the construction of walls with no concrete core, the concrete blocks were placed up to the underside of the floor slab. The top course of block was a bond-beam type block. Where the ribs in the metal decking ran perpendicular to the wall, grout was pumped into the cells and brought up to the top of the block. The remaining volume between the ribs and the top of the blocks was filled with a stiff mortar. Where the ribs ran parallel to the wall and the reinforcing steel came through the top of a rib, the following sequence of construction was used:

1. A horizontal mortar joint was placed between the edges of the block and the bottoms of the two adjacent ribs.

2. A 2-in. to 3-in. hole was cut in the side of the top row blocks every 4 ft to 6 ft and grout was pumped in through the holes. This was done under enough pressure so that the grout was forced up to the top of the rib. Where the

CLARIFICATION # 4, continued

ribs ran parallel to the wall, and the vertical reinforcing steel penetrated the bottom of the rib, the grout is brought up to the top of the block, which is nearly the bottom of the rib and the remaining volume is filled with stiff mortar.

For block wythes which were topped out in this manner, the ultimate capacity of the dowels will be able to develop.

The walls with concrete cores were constructed by one of two methods. The primary method was to build both block wythes up to the bottom of the slab by one of the methods described above. After the block wythes were constructed, the concrete core was placed from above through holes in the slab. In some situations, access from above was not available, in which case the following sequence was used.

1. One block wythe was built to the top as described above.

2. The second wythe was built one block short of the top and concrete core was placed up to the top of this wythe.

3. Solid concrete brick was used to fill the remainder of the concrete core volume and packed solid with mortar.

4. The second block wythe was finished with solid concrete brick with the vertical reinforcing steel in the mortar joint. The volumes at the top of the concrete brick were filled with stiff mortar.

This type of construction will allow the ultimate capacity of the dowels to develop.

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QUESTION # 5

"Justify the use of a factor of 2 on top of the AISC criteria for the joint capacity."

CLARIFICATION # 5

As it is stated in Section 6 of the Trojan Control Building Supplemental Structural Evaluation, the steel beam to column connection capacity is based on twice the AISC Part I allowable capacity. This factor of 2 is based on experiments conducted by Fisher and Beedle* and summarized in ASCE Manual No. 41.** These tests show a factor of safety of 2 to 3.3 for bearing and larger than 3 for shear of bolts.

* Fisher, J. W., and Beedle, L. S., "Criteria for Designing Bearing Type Bolted Joints," ASCE (ST 5), Paper 4511 (October 1965).

** Plastic Design in Steel, A Guide and Commentary, ASCE Manual No. 41, (1971) p. 211.

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QUESTION # 6

"State the number of modes considered in the STARDYNE analysis, the upper frequency cutoff of the significant modes, and why the other higher modes do not have a significant effect on the response. Justify that."

CLARIFICATION # 6

In the fixed-base STARDYNE analysis, the first 30 modes were included in determining the SRSS responses. In combining the modes, closely spaced modes were considered and combined by the "10% grouping method" described in BC-TOP-4A. The highest frequency was 18.7 cps. Since the sum of the effective modal weights of these modes in the N-S and E-W directions are 94% and 91% respectively of the total weight, the higher modes which have not been included cannot contribute significantly to the global response, and the global response governs the shear force in the walls.

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QUESTION # 7

"Supply the sum of the effective weights in both the north-south and east-west directions for all the modes considered in 6 above."

CLARIFICATION # 7

Please refer to clarification offered in response to Question # 6.

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QUESTION # 8

"On page 1-2 of the response to staff questions, did supports and the cable trays meet all the appropriate FSAR criteria?"

CLARIFICATION # 8

As discussed in FSAR section 3.10.2.4, the safety-related cable tray supports and their associated loads (cable trays and cable) were designed and built in accordance with the requirements stated in section 3.10.1, with allowable stresses as stated in section 3.8.1.3.3. These sections of the FSAR constitute the appropriate criteria. The supplemental analyses did not affect the qualification of the original installation.

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QUESTION # 9

"Provide the significant natural frequencies of the Turbine Building (for both directions)."

CLARIFICATION TO # 9

The significant natural frequencies of the Turbine Building are as follows:

	1st mode	2nd mode
N - S Direction	1.93 cps	3.96 cps
E - W Direction	0.89 cps	3.63 cps

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QUESTION # 10

"Confirm that the September 1 submittal just documented the results of the meeting that we had to discuss this new information, that the results were preliminary in nature and the September 1 submittal has been superseded by the "Supplemental Evaluation" submittal.

CLARIFICATION # 10

On August 28, 1978, a meeting was held with the NRC to discuss new information regarding Control Building design. At this meeting, presentations were given by Bechtel engineers on various technical details. Copies of the overhead slides used for these presentations were left with the NRC, and they were attached to the NRC meeting notes. The engineers who gave these presentations emphasized that the technical results presented were preliminary, since Bechtel was still checking data. Subsequent to this meeting, the following two documents were submitted to the NRC attached to PGE's letter of September 1, 1978.

a. Attachment 1, "Preliminary Results of STARDYNE Finite Element Analyses of Trojan Control-Auxiliary-Fuel Building Complex," August 28, 1978.

b. Attachment 2, "Supplementary Information On:

    1. Preliminary Assessment of Fuel Building to Resist Seismic Loads Based on Results of the STARDYNE Finite Element Analysis.

    2. Transferring Lateral Earthquake Force From the Structures to the Rock Subsoil.

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CLARIFICATION # 10, continued

3. Evaluation of Deflections and Displacements" dated August 28, 1978.

These two documents summarized the oral presentation given to the NRC on August 28.

The information given at the NRC meeting and in the foregoing two documents was preliminary in nature; it should not be used as a basis for evaluation of the structures, and has now been superseded by the information given in the documents titled "Trojan Control Building, Supplementary Structural Evaluation," September 19, 1978; and "Response to Questions from the Nuclear Regulatory Commission Dated August 30, 1978," September 20, 1978.

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QUESTION # 11

"For Table 2.1 on Page 2-4 of the response to NRC questions specify the moment and the shear capacities of Fuel Building walls according to ACI 318-63. Include a statement that the 1963 code is met in total for those walls."

CLARIFICATION # 11

The additional data requested are incorporated into the expanded Table 2.1 as revised October 1978 and attached to the response to questions 13 and 14. Also, the required statement regarding the ACI 318-63 code is given in the response to questions no. 13 and 14 of this supplemental information.

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QUESTION # 12

"Provide justification for using the ductility ratio of 1.5 in the discussion on Page 3b-2 of the response to NRC questions. Show why it is conservative and why it is consistent considering the information in Appendix D for both frequency shifts and reductions in amplitude."

CLARIFICATION # 12

The information presented in Appendix D was intended only to give an upper-bound estimate of the Control Building displacement under the SSE load. The upper-bound displacement was determined based on the displacement of the most highly loaded wall relative to its capacity (wall 1) of the Control Building, using the lowest bound stiffness. The total system response was not addressed.

To assess the ductility ratio consistent with the criteria given in Appendix E, and the possible frequency shifts due to the Control Building inelastic response based on the frequencies from the STARDYNE finite element analysis, the total system behavior of the Control-Auxiliary-Fuel Building Complex has to be considered because the three buildings are tied together, and the STARDYNE frequencies are the system frequencies rather than the frequencies of the Control Building alone. To apply the criteria given in Appendix E, an equivalent elasto-perfectly plastic system representing the total system must be used. The stiffness of the total system is the combined stiffnesses of the Control, Auxiliary, and Fuel Buildings; and the "yield" capacity of the total system is the combined ultimate capacities of the Control, Auxiliary, and Fuel Buildings. Since the total SSE load on the entire building complex has not reached the "yield" capacity of the total system, a rational determination of the system ductility ratio cannot be made (because the system ductility ratio based on

CLARIFICATION # 12, continued

the criteria given in Appendix E in this case would be less than one; see attached Figure 12-1). The ductility ratio of 1.5 used in the response to the NRC question 3(b) was selected as an upper bound to illustrate the possible effect of the Control Building inelastic response on the response of the total system.

Due to the Control Building inelastic behavior, the total stiffness of the entire building complex will be somewhat lower than the initial elastic stiffness. However, the reduction of the total stiffness is expected to be not more than one-half of the total initial stiffness because under the SSE load the Fuel Building end of the structural complex still remains in the elastic range. Thus, the lowering of the system frequency due to the Control Building inelastic behavior is expected to be not lower than f/1.414, where f is the STARDYNE system frequency. This shifts the frequency into the frequency range of the original floor spectral peak, as shown in the response to question 3(b).

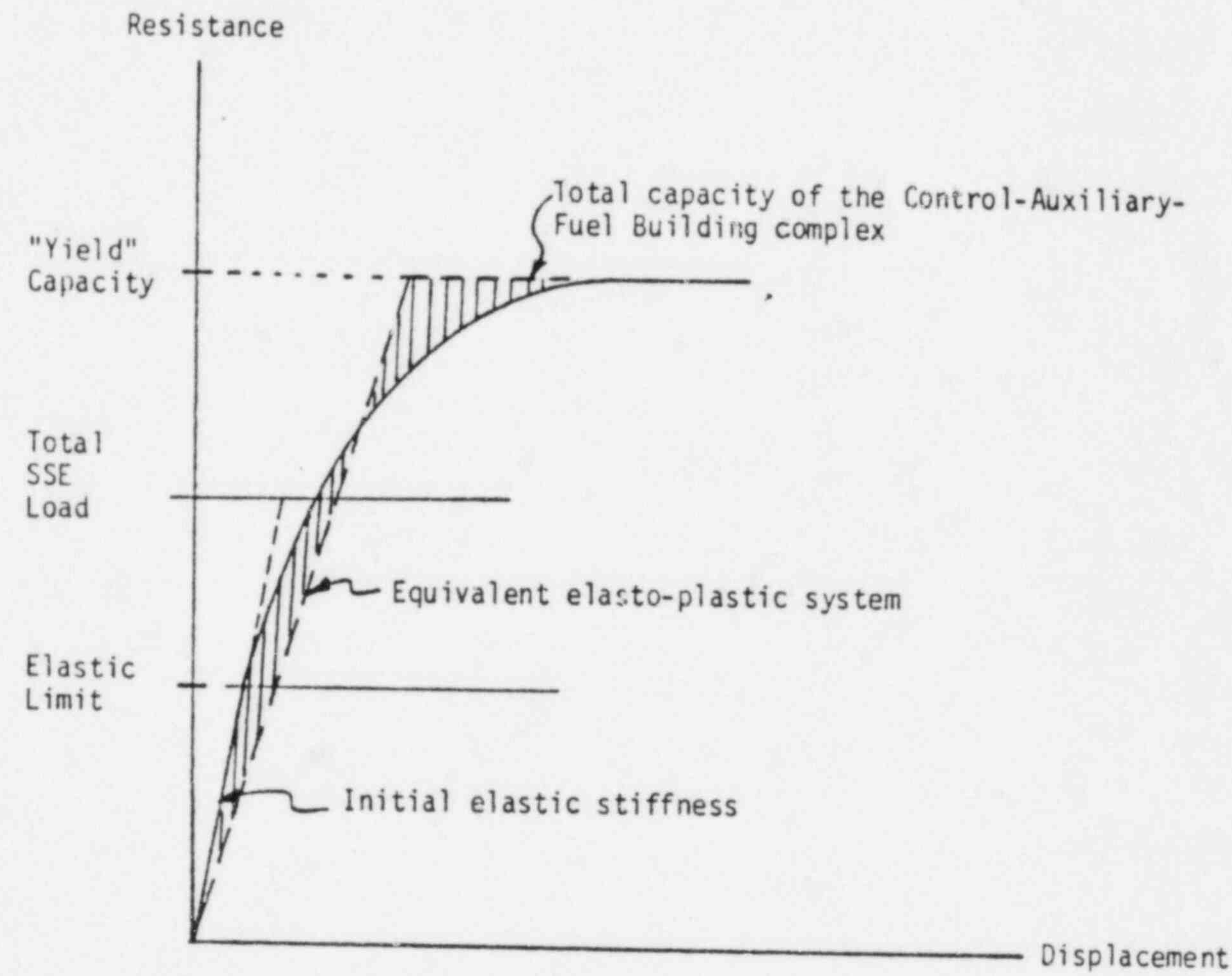

Figure 12-1 Equivalent Elasto-Plastic System Based On The Newmark Criteria

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QUESTION # 13

"Relative to the Fuel Building walls where you used ACI 318-77, are all provisions of that code met?"

CLARIFICATION # 13

Response to Question 2, Item 1 (in the document titled "Responses to Questions from Nuclear Regulatory Commission, August 30, 1978, submitted on September 20, 1978) was addressed to the integrity of the Fuel Building. The response included a table (Table 2.1) with information on loads vs capacities. A revised table (Table 2.1, revised October 1, 1978) giving additional requested data is attached to this supplementary response. We included herein an explanation of the development of the additional information and our conclusion regarding the seismic capability of the Fuel Building.

Information on factored OBE loads (0.15g with 2% damping), derived from the original stick model and the supplementary STARDYNE finite element analyses, as well as the values of "Shear Capacity" given in the original Table 2.1 are unchanged. However, the heading of "Shear Capacity" was revised to "Design Shear Strength" in accordance with the definition given in the ACI 318-77 code. As it is explained in the original response, the values in this column are based, conservatively, on $2(f'_c)^{\frac{1}{2}}$ shear stress. Formulae 11-33 or 11-34 of the ACI 318-77 code were not used. Calculations based on these two formulae would have resulted in considerably higher Design Shear Strengths.

The "Ultimate Shear Strength" of each wall as defined by the ACI 318-63 code have been calculated and added to the table. (It should be noted that the meaning of "Design Shear Strength" defined by the ACI 318-77 code and "Ultimate Shear

CLARIFICATION # 13, continued

Strength" defined by the ACI 318-63 code are identical.) The ACI 318-63 code did not include specific provisions for calculating shear strength of shear walls. The values in the table under "Ultimate Shear Strength" are based on $2\sqrt{f'_c}$ shear stress, 0.85 capacity reduction factor and specified design strength (not "as-built" strength) of materials. In computing the "d" value (distance from extreme compression fiber to the centroid of tension reinforcement), a flange width extended to the centerline between two walls, the reinforcement in the tensile flange and two thirds of the reinforcement in the web were considered. Since formulae (17-2) and (17-3) of the code have not been used, additional conservatism was introduced. Based on these parameters, most of the "Ultimate Design Strengths" as defined by the ACI 318-63 code are slightly lower than the "Design Shear Strength" based on the ACI 318-77 code. However, even these lower strengths are higher than the loads derived from both the original stick model analysis and the STARDYNE analysis based on factored load condition.

The revised table also provides information on shear strengths governed by ultimate resisting moments. The calculation of these values has been based on the ACI 318-63 code as described above. Since both the hold up tank enclosure structure and the fuel pool are "box" type reinforced concrete structures, with 0.6-0.8 percent reinforcing steel, the moment capacity of these walls are high. There is only one wall for which the shear strength is governed by ultimate resisting moment.

The results of our investigation clearly demonstrate that the Fuel Building resists the SSE and factored OBE loads well within the FSAR criteria, based on both ACI 318-63 and ACI 318-77 codes.

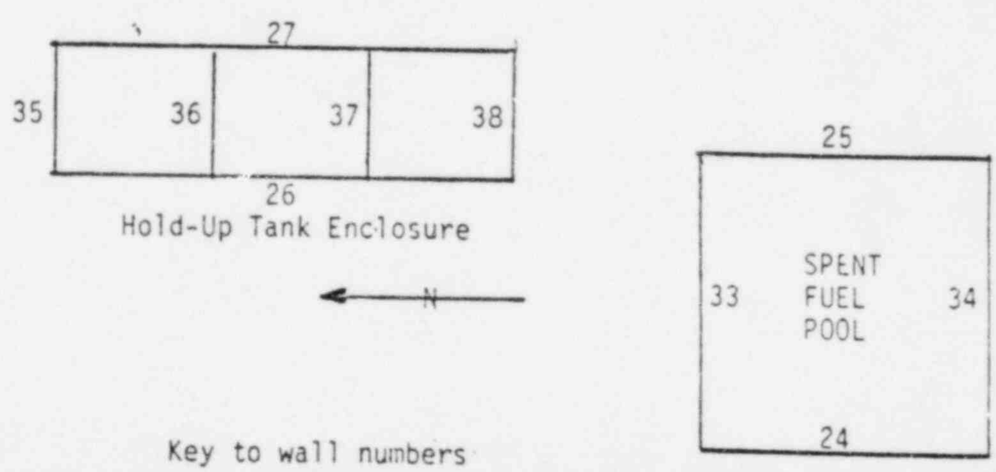
TABLE 2.1 (REVISED 10/1/78)

LOAD vs STRENGTH OF EACH WALL OF THE HOLD-UP TANK ENCLOSURE AND THE SPENT FUEL POOL

(KIPS)

	WALL ELEMENT	Shear Strength Governed by Ultimate Resisting Moment	"Ultimate Shear Strength" ACI STD. 318-63	"Design Shear Strength" ACI STD. 318-77	1.4 OBE (.15 g)			
					Orig. Stick Model		Supplemental*	
					N-S	E-W	N-S	E-W
HOLD-UP TA ENCLOSURE	26	8900	6600	7363	1320		2130	349
	27	8600	6600	7363	1320		1764	878
	35	2710	2630	2718		1360	956	1890
	36	2190	1730	1616		1360	193	729
	37	2370	1730	1616		1360	60	671
	38	1830	2630	2718		1360	140	596
SPENT FUEL POOL	24	17200	10400	11124	4500		1703	823
	25	14700	12900	13792	4500		1465	515
	33	13700	13400	14352		4340	1304	2834
	34	15700	15200	16561		2980	555	1346

*STARDYNE finite element/analysis



Key to wall numbers

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QUESTION # 14

"With regard to the Fuel Building walls, provide the details of not only how the shear capacities for the walls were calculated but also how the moment capacities for the Fuel Bulding walls were calculated."

CLARIFICATION # 14

Please refer to clarification offered in response to question # 13.

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QUESTION # 15

"Relative to Table 4-2, for the Auxiliary Building walls, compare their dowel capacities to their moment and shear capacities and justify how their capacities can be developed. Were they governed by moment or shear? Also, provide details for how the moment capacities were calculated.

CLARIFICATION # 15

Referring to Table 4-2 for the Auxiliary Building, the walls which have "dry pack" in the space on top of masonry and below the metal deck are walls 4, 5, and 6 in the N-S direction, and walls 10, 11, and 12 in the E-W direction (refer to Figure 3-3). There are two walls in the Auxiliary Building at this elevation (61'-77') which have dowel action shear capacity smaller than shear capacity due to bending. In wall 4 the ratio of the shear capacity governed by bending to the shear capacity by dowel action is 1.11. This ratio in wall 12 is 1.15. All the others have dowel capacities larger than the shear capacity due to bending moments. The bending capacity of shear walls was evaluated according to Appendix B of the May 5, 1978 submittal.

The capacities for walls 4, 5, and 6 in the N-S direction and 10, 11, and 12 in the E-W direction are controlled by moment. This capacity depends on forces in the vertical reinforcing steel. Since this reinforcing steel is adequately anchored in the slabs, the capacity can develop. In addition, there is some dead load on these walls. The capacities of the other two walls in the Auxiliary Building shown in Table 4-2 (wall 9 and part of wall 8) are controlled by the basic criteria. Since these walls satisfy the conditions of the basic criteria, namely, reinforcing ratios and dead load, their capacities can develop.

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CLARIFICATION # 15, continued

Capacity based on dowel resistance relates to the behavior of walls in the final stages of resistance. Before this final resistance is reached, other actions such as development of compression strut will develop capacity higher than dowel capacity.

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QUESTION # 16

"Relating to (12 above), describe in detail how the spectra were developed. How were Newmark's criteria related to the Trojan criteria, and also how were the peaks smoothed."

CLARIFICATION # 16

A description of how the pseudo-elastic floor response spectra were developed is provided on page 3b-2. Additional details follow.

The pseudo-elastic floor response spectra amplitudes were obtained by linear scaling of the STARDYNE elastic floor response spectra amplitudes by the ratio of pseudo-elastic to elastic maximum (zero period) floor accelerations, and the spectra peak band widths were located at $\pm$ 10 percent about the fundamental frequencies represented by the pseudo-elastic stiffness (i.e., STARDYNE elastic stiffness divided by the ductility ratio, μ).

The pseudo-elastic Control Building floor accelerations were determined by the methods described in Reference 1 of Appendix E using as a basis the Trojan elastic 0.25g SSE ground response spectrum at 5-percent damping (see FSAR Figure 3.7-2). This ground response spectrum was first "linearized" on the tripartite log plot by constructing straight lines identifying the frequency regions of constant displacement, velocity and acceleration, and the transition between constant acceleration and ground motion. Values determined were 0.22 Hz and lower for displacement, 0.22 Hz to 1.25 Hz for velocity, 1.25 Hz to 6.67 Hz for acceleration, and 6.67 Hz to 20 Hz for the acceleration transition zone. Pseudo-elastic ground response acceleration spectra were then constructed for selected ductility ratios (μ's) by multiplying the "linearized" elastic ground spectrum values by $1/\mu$ in the constant displacement frequency region, by $1/(2\mu - 1)^{\frac{1}{2}}$ in the velocity response

CLARIFICATION # 16, continued

preserving frequency region, and by maintaining the peak ground response acceleration at 20 Hz and above.

Based upon the inelastic ground response spectra developed as described above, Control Building pseudo-elastic flor accelerations were determined using STARDYNE elastic fundamental frequencies (per Reference 1 of Appendix E). Amplitudes of the pseudo-elastic floor response spectra were then calculated by multiplying the STARDYNE elastic floor response spectra amplitudes by the ratio of pseudo-elastic to STARDYNE elastic floor accelerations. Since the response of the Control Building is dominated by the fundamental mode, and the fundamental mode frequency is a function of the stiffness, the pseudo-elastic fundamental frequency was determined by multiplying the elastic fundamental frequency by $(K_{pseudo\text{-}elastic}/K_{elastic})^{\frac{1}{2}}$ which is equivalent to $(\frac{1}{\mu})^{\frac{1}{2}}$.

For purposes of review of safety-related equipment in the Control Building, envelopes of the original elastic, STARDYNE elastic, and pseudo-elastic (for $\mu = 1.5$) floor response spectra were used. For conservatism, the envelopes of the original and the STARDYNE spectral peaks were used, and the "valleys" between the spectral peaks were not considered as illustrated in Figures 3b-9 through 3b-12.

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QUESTION # 17

"With regard to these spectra, compare the STARDYNE accelerations to the accelerations obtained in the original modal analysis so as to justify taking the unnecessary conservatism from the original synthetic time-history which was used to develop the original spectra."

CLARIFICATION # 17

For illustration, the respective values of Control Building floor accelerations at elevation 93 ft. for the 0.25 g SSE (5% damping ratio) determined from the original (1971) response spectrum analysis based on the SRSS modal response combination method, the STARDYNE response spectrum analysis, and the original (1971) time-history analysis, are compared as follows:

	Maximum Acceleration N-S	Maximum Acceleration E-W
Original Response Spectra	0.32g	0.45g
STARDYNE Response Spectra	0.63g	0.47g
Original Time-History	0.94g	0.89g

The data tabulated above show that the original time-history analysis gives values of floor accelerations much larger than those determined from the original response spectrum analysis. As described in 3(b) the larger floor accelerations produced by the original time-history analysis are primarily due to the excessively conservative synthetic time-history used in the original analysis. As shown in FSAR Figures 3.7-6a, 6b, and 6c, the response spectrum produced from the synthetic time-history exceeds the normalized design ground response

CLARIFICATION # 17, continued

spectrum by large amounts in most frequency ranges , and particularly in the frequency range of the Control Building fundamental horizontal frequencies. With use of a synthetic time history which has a response spectrum closely matching the design ground response spectrum, time-history analysis of a structure would produce floor accelerations very close to those that would result from the response spectrum analysis of the structure. For the Control Building, which has its response dominated by the fundamental mode, floor accelerations resulting from a time-history analysis and a response spectrum analysis would be essentially equal.

Floor response spectra representing the expected STARDYNE results were calculated by linearly scaling the original floor response spectra from the time-history analysis by the ratios of peak horizontal floor accelerations, and centering the broadened spectra on the STARDYNE calculated fundamental frequencies. These calculated spectra were broadened by maintaining the peak spectral response through the frequency range of ± 10 percent about the fundamental frequency. These predicted spectra remain conservative because the STARDYNE response spectrum analysis floor acceleration values represent upper bound elastic responses, and the original ratios of spectra peak accelerations to floor accelerations are maintained in constructing STARDYNE floor response spectra.

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QUESTION # 18

"Justify the apparent discrepancies between the shear modulus in the Appendix D analysis and the shear modulus used in your reevaluation, and why these discrepancies would have no effects on the results of the STARDYNE analysis, such as forces, displacements, accelerations, response spectra (considering the effects of ductility), etc. Also, in your response to NRC questions on pages 3b-4 and 3b-5 you discussed the effects of increased displacements for the Auxiliary and Fuel Buildings, yet the discussion prior to this regarding effects on response spectra neglects the consideration of the effects of nonlinear behavior on these buildings. Verify that the effects on the spectra were considered for these buildings and were found to have insignificant effects on the safety-related (particularly those required for ECCS and safe shutdown) components, equipment and piping (including their supports) in these buildings.

CLARIFICATION # 18

The uncracked elastic shear modulus used in the STARDYNE analysis was 1.59×10^6 psi. The shear modulus used in Appendix D was 0.45×10^6 psi. This was obtained from averaging reported test data; it represents the cracked elastic modulus. For the purpose of estimating an upper bound value of the Control Building deflection, the cracked modulus was used. Furthermore, the deflection estimation in Appendix D was based on only the most highly stressed wall (i.e. wall 1) of the Control Building and a low estimate of the stiffness.

The seismic forces and the structural accelerations will show insignificant change due to any anticipated reduction in shear

CLARIFICATION # 18, continued

modulus. This is for two reasons. First, the spectral acceleration associated with the reduced frequency will still be obtained from the flat portion of the design response spectra and therefore remain unchanged. Second, the response in each direction is dominated by the fundamental mode in the respective direction. Therefore higher modes will not offer a significantly greater contribution even with the increase in their spectral accelerations. The displacements will increase, but this will not affect the upper limit prediction given in Appendix D.

The frequency change due to reduction in shear modulus will result in a shift of the floor response spectra peaks. The effect of frequency shift and that of ductility is discussed in the response to NRC Question 3(b) and Clarifications 12 and 16.

On pages 3b-4 and 3b-5 of the response to Question 3(b), the discussion on the effect of increased displacements on the Auxiliary and Fuel Buildings was based upon the upper bound displacement estimate for the Control Building as determined from Appendix D. The upper bound displacements used in the discussion was for conservative purposes. The possible effect of the Control Building inelastic response on the Control Building floor response spectra was discussed in the response to 3(b). It was concluded that the effect of inelastic behavior is insignificant. Since the Fuel Building still remains in the elastic range under the SSE loads, the effect of the Control Building inelastic behavior on the Fuel Building floor response spectra would be even less.

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In the response to question 3(b), particular emphasis was placed on the verification of the Control Building safety-related equipment which is generally located at higher elevations where the effects on response spectra are more pronounced. Most of the equipment required to achieve safe shutdown located in the Fuel and Auxiliary Buildings is at elevation 45 ft or below where the changes in response spectra are felt the least. Additionally, this equipment is located at the east end of the Auxiliary Building and at the north end of the Fuel Building where the effects on the response spectra are relatively unchanged. (See Clarification 12.) The fact that the Control Building equipment continues to meet the original criteria provides substantial assurance that the safety-related equipment, components, and piping (including their supports) located in the Fuel and Auxiliary Buildings, which is required for safe shutdown, will continue to meet their original seismic qualifications as well.

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QUESTION # 19

With regard to the foundation:

a. "Describe the foundation and any peculiarities, such as mud sills, waterproofing, etc., that would influence the values of C and μ at the interface of the foundation with the bedrock. Discuss the construction technique, etc., that was used to assure that there are no peculiarities, such as some type of weakening of that interface, through the construction technique used and weakening of the bond value. Justify the bond value between the foundation and that interface is such that the values used for the tuff material are indeed the weakest link."

b. "Describe in detail how the dead loads and earthquake loads were considered in arriving at a pressure distribution for the foundation."

c. "Describe your conformance to Standard Review Plan Sections 2.5.4 and 3.8.5, and justify any deviations from these two sections."

d. "Explain how with a wide range of velocities, it is still conservative to use an average value. Also explain the obvious discrepancy between the shear wave velocities used in your analysis and the shear wave velocities quoted in the FSAR. In detail, justify the values for the tuff considering that we heard that some tests at that time were not performed and that a lot of this was extrapolation through current knowledge."

e. "Also expand on the discussions such that you are assured that there are no seams, fissures, etc., in the rock that would affect the values you are using."

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1. The footings and grade beams for the Control Building were constructed by placing concrete directly on the rock foundation. There are no mudsills or waterproofing materials under the footings or grade beams of the Control Building.

2. Prior to placing concrete on the foundation, debris was removed and the rock was thoroughly wetted. The foundation was inspected by the contractor's QC personnel.

3. Foundation materials underlying the structure are volcanic rocks, primarily basalts and tuffs. The geologic mapping during the excavation phase of construction revealed the rocks are gently folded (dipping 15° to 20°), discontinuous masses of basalts and tuffs with some volcanic agglomerates. These rocks have been intruded by igneous dikes which interrupt the stratification of the other rock units. Measured angles of dip of joint planes ranged from 30° to 80°. Seismic P (compressional) and S (shear) wave velocity measurements were made as the excavation neared final grade. Four geophysical survey lines were made in the power block area; these measurements indicated that P wave velocities ranged from 8,200 to 10,600 fps, and the shear wave (S) from 4,500 to 5,000 fps. (See attached table A.) These values are actual in-place measurements of the foundation rocks, near final grade, and include the effects of any fractures, weathered zones, and any other weakness the rock may have. The attached table B, showing static properties, summarizes the laboratory test results of core samples obtained during the exploration program. The compressive strengths ranged from 360 to 13,150 psi, and averaged 2,497 psi.

   As it is shown on the attached photographs, the foundation conditions as exposed during final excavations are competent rock masses as described in the FSAR. Although

CLARIFICATION # 19, continued

there are fractures, shears, and joints in the foundation, there are no seams or fissures in the rock which would affect the results of the analysis.

In order to provide a very conservative estimate of the shear strength of the foundation rocks and their resistance to sliding, the following assumptions were made:

a) The foundation for the structure is composed entirely of tuff, the weakest rock unit present at the site. See FSAR, section 2.5.1.5, and Table 2.5-1. As described above, this is a highly conservative assumption consistent with known foundation conditions.

b) This hypothetical mass is assumed to have a compressive strength of 1,225 psi, the average of the unconfined compressive laboratory test results on the tuff. This value is also highly conservative as the P and S wave velocities measured near foundation grade indicate a much stronger rock than one having 1,225 psi compressive strength; a value of 3,000 to 5,000 psi is more realistic.

The coefficient of friction, μ, is estimated to be at least 0.7. This is a reasonable value frequently used in analysis of sliding for concrete dams on similar rocks. The US Bureau of Reclamation in their Design Standards No. 2, Treatise on Dams, chapter 9, suggests a value 0.8 should be used. The Handbook of Applied Hydraulics, C V Davis, editor (McGraw-Hill), quotes values ranging from 0.65 to 0.75.

Rock test results of tuffs from the Howard Prairie Dam Site east of Medford, Oregon, showed a coefficient of friction of 0.9, based on the tests of 15 specimens.

CLARIFICATION # 19, continued

These are believed to be weaker rocks than the Trojan tuffs, as the compressive strengths averaged only 530 psi, compared with the 1,225 psi for 41 samples at Trojan.

The shear strength, C ,of this weaker hypothetical rock mass is estimated to be in excess of 130 psi. One method of estimating the shear strength of rock is to divide the unconfined compressive strength by three. For this material, this would be 1,225 divided by 3, or 408 psi. If we use a safety factor of three, the result would be about 130 psi.

Another "check method" frequently used by rock mechanics and foundation specialists is to take 10% of the average unconfined compressive test results, or 10% of 1,225, which is 122.5; or about 130 psi.

Additional conservatism is included in this estimated 130 psi shear strength as these values are assuming zero confining load--clearly not the case at Trojan--and secondly, all of this evaluation of rock properties is based on normal stress conditions. These values are those used for normal design conditions. Much higher strengths of the rocks are normally used for SSE conditions. For example, normal bearing capacity values are frequently increased by at least a factor of two, often three or more, for SSE conditions.

4. The stress distribution on foundation level caused by gross moments due to lateral seismic loads is determined from the STARDYNE analysis output. The stresses due to the dead load and the vertical seismic acceleration are then superimposed and the total stress distribution pattern under the foundation is established. This stress distribution indicates the areas of the foundation that

CLARIFICATION # 19, continued

may be subject to uplift. These areas are excluded in evaluating the contribution from the cohesion, C.

The stress distribution has no influence on the frictional contribution, μ, because the net vertical load, D is unchanged by the non-uniform distribution of load on the foundation level.

5. It should be emphasized that the Standard Review Plans (SRP) were not issued at the time the Trojan Nuclear Plant was being designed and constructed.

   Our evaluation of section 2.5.4, Stability of Subsurface Materials and Foundations (May 1975), has shown general compliance with it.

   Our evaluation of section 3.8.5, Foundations (November 24, 1975), shows that the design and construction of the Control Building generally complies with it, with the following two clafifications:

   a) Some of the load factors and load combinations are as defined in the Trojan FSAR and differ from the SRP.

   b) The SRP lists more updated versions of some codes, and lists some standards that were not in existence when the Trojan Nuclear Plant was being designed and constructed.

6. Using the average compressive strength of 1,225 psi for the tuff is reasonable because the confining load is assumed to be zero. Additional conservatism is obtained by using a factor of safety equal to 3, and 10% of the unconfined compressive strengths as discussed in 3, above.

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CLARIFICATION # 19, continued

7. The shear wave velocity used in the STARDYNE flexible base analysis was computed by using the rock properties as determined by laboratory testing and the following formula:

$$V = \left(\frac{E\, g}{2\, \gamma (1 + \nu)} \right)^{1/2}$$

where, V = shear wave velocity, ft/s
E = dynamic modulus of elasticity, lb/sq in
g = acceleration due to gravity, ft/s sq
γ = unit weight, lb/cu ft
ν = Poisson's ratio

Using the numerical values given in the FSAR, the resulting shear wave velocity is 5,473 ft/s.

8. The strength values of tuffs are discussed in section 2.5.1.5 of the FSAR.

CLARIFICATION # 19, continued

TABLE B

TESTS ON ALL ROCK SAMPLES TESTED

Test	No. of Tests	High	Low	Average
Specific Gravity	34	2.51	1.84	2.13
Porosity, (%)	34	32.40	9.30	20.15
Absorption, (%)	34	17.30	3.70	9.71
Unconfined Compressive Strength, (psi)	55	13,510.00	360.00	2,497.00
Modulus of Elasticity at 150 psi, (psi)	21	7.5×10^6	0.09×10^6	1.7×10^6

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CLARIFICATION # 19, continued

TABLE A
(Revised 10-6-70)

Line	V	V	ν	E	μ	K
A	8,500	4,500	0.31	1.75	0.62	1.52
B	8,500	4,500	0.31	1.75	0.62	1.52
C	8,200	4,500	0.32	1.6	0.62	1.45
D	10,600	5,000	0.35	2.3	0.75	2.55

where, V = compressional wave velocity, ft/sec.
V = shear wave velocity, ft/sec.
ν = Poisson's ratio
E = Young's modulus (compression) x 10^6 psi
μ = shear modulus x 10^6 psi
K = bulk modulus x 10^6 psi

Note: Revised specific gravity based on density of 153 lb/cu ft (saturated) = 2.45 specific gravity. Rock material specific gravity 2.25 (dry) and 20% porosity.

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QUESTION #20

"Provide the minimum clearance between the Control and Turbine Buildings, in light of the discrepancy between your previous response to our question and the findings of our site visit regarding clearances of lateral bracing. Explain why the lateral bracing was not discovered in the previous walkdown and why the information presented based upon that previous walkdown is still accurate. What are the effects of the lateral bracing and the three I-beams connected to the 4160 volt switchgear room wall on your previous conclusions with respect to the adequacy of the minimum clearance between the Control and Turbine Buildings? Will any repairs be performed to improve this situation?"

CLARIFICATION #20

Clearances between the Turbine Building and the Control Building as well as between the Category I structures within the Turbine Building and the Turbine Building structure itself were discussed in the Response to Question 1. Based on a survey of the main structural columns of the Turbine Building/Control Building interface, it was reported that the minimum clearance is 3 in. (except at El 99 ft where a steel plate is attached to Turbine Building columns). Similarly, based on a review of design drawings it was reported that a 3-in. gap separates the Category I structures within the Turbine Building from Turbine Building structural members.

In light of the discrepancies identified during the NRC site visit, related to secondary structural members of the Turbine Building and I-beams connected to the 4160-volt switchgear room wall, additional evaluations through detailed field surveys have been conducted on these clearances.

We have compared the results of this survey with the ABS differential displacements under 0.25g SSE conditions and find that a minimum ratio of static clearance to differential displacement in the E-W direction is at the 2-in. column clearance at El 99 ft and at the Control Building roof at El 117 ft in the N-S direction. Thus, our original assessment and conclusions remain unchanged.

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CLARIFICATION #20, continued

As to the separation between Category I structures located within the Turbine Building and Turbine Building structural members, the survey identified instances where structures housing safety-related equipment in the Turbine Building are separated from the Turbine Building by less than 3 in., several instances where miscellaneous steel, not part of the Turbine Building structural system, were connected to the Auxiliary Feed Pump structure, a column-slab interference between a Turbine Building column and the roof of the Emergency Diesel Generator Room, and one instance where three small I-beams spanning between the north wall of the 4160-volt switchgear enclosure and an interior concrete block wall of the Turbine Building are installed. In each instance these conditions will be corrected, and the corrections are not difficult. For example, with respect to the I-beams spanning between the switchgear enclosure and the Turbine Building wall, the design clearances between the ends of the I-beams and the walls are sufficient to accommodate the 0.25 SSE relative displacements in the N-S direction between the switchgear enclosure and the Turbine Building at that location. However, it has been identified that the actual clearance at the beam ends and movement at the I-beam bolted end connections is not sufficient, and this condition can be remedied simply by modifying the end connections.

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QUESTION # 21

"Discuss and compare the reinforcement anchorages in the tests that you are using to assure they are continuous. The comparison should be made with the steel in the Trojan walls."

CLARIFICATION # 21

In the Trojan Control Building walls, all the reinforcement in the blocks is continuous or anchored. Some of the core horizontal reinforcement is continuous also. In consideration of the reinforcement percentages for applicability of the basic criteria, only the continuous or anchored reinforcement in the block walls over the gross section was considered. For Schneider's test specimens, the report does not give a detailed description of the anchorages. The figures in the report show 180' bends, but the extension of the bend is not shown. The Berkeley test specimens had 90' bends with 9 inches of bars extending vertically upward or downward. The continuous reinforcing bars of the Trojan walls have at least the same effectiveness in anchorage as that of the reinforcing bars of the test specimens.

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QUESTION # 22

"On Page 3d-1 of the response to our questions, your equation for shear friction comes from the ACI 318-71 code. Verify that all the provisions of that particular section, namely 11.15, with regard to shear friction, steel anchorages, etc., are met. Also, on Page 3d-2, clarify that Items 3 and 4 on that list are not considered. Document where the criteria for the dowel capacity of encased columns have been previously set forth. Also for Criterion 1, justify the $0.9A_s$ times the ultimate strength of the steel in more detail than merely a reference to Appendix B".

CLARIFICATION # 22

a. The shear capacity of slabs based on shear friction is evaluated from equation 11-30 of ACI 318-71. The shear friction provisions of section 11.15 of ACI 318-71 are met. In evaluating the compliance with section 11.15, the anchorages of top and bottom steel in the slabs have been considered, and wherever anchorage of bars is inadequate due to being interrupted by steel girder webs or other effects, contribution to the shear capacity of these bars is conservatively neglected.

b. In the September 19, 1978 supplement, the load transfer from the upper wall-slab connection to the lower wall represented by items 1 and 2 on page 3d-2 is adequate for all elevations except the lower two elevations of the West Wall (Wall 1 on elevations 45'-61' and 61'-77', refer to Figures 3-2 and 3-3). For this wall only, the load transfer represented by item 4 (page 3d-2) was used additionally. The contribution of the dead load was added to that of items 1 and 2. If a coefficient of friction of 0.6 is used at elevation 59 ft, then the load transfer

CLARIFICATION # 22, continued

capacity from items 1, 2, and 4 equals the applied load. The weakest horizontal plane at this elevation is at the lower end of the 8-in. Nelson studs welded to the bottom flange of the encased beam. The coefficient of friction along this plane would correspond to concrete cast monolithically. For this condition, 0.6 is a conservative coefficient of friction. At elevation 75 ft., a coefficient of 0.17 is needed for the combined load transfer from items 1, 2, and 4 to equal the applied loads. The weakest horizontal plane at this elevation is along the bottom flange of the encased beam which does not have Nelson studs attached. The coefficient of friction of 0.17 is conservative for concrete placed against rolled structural steel. The additional shear transfer mechanisms present, but not used, will provide added capacity.

c. The criteria for the dowel capacity of encased steel columns is documented on page 4-4 of the May 24, 1978 supplement.

d. The dowel capacity of the vertical reinforcing bars is based on 0.9 $A_s f_u$. The factor of 0.9 is taken due to the fact that the embedment condition in the bars allows them to be considered to have at least the capacity of the Nelson studs. Further discussion of this is provided in Section 5 of Appendix B.

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QUESTION # 23

"On Page 3f-2, Table 3f-1, in the responses to our questions you talk about the horizontal resistance of the Fuel Building being considered using 0.7W. Define "W". Expand the discussion of the Control Building foundation to include a discussion of the effect of all parameters considered for the Control Building on the Fuel Building foundation."

CLARIFICATION # 23

In Table 3(f)-1, (as revised on October 5, 1978) "W" is defined as the dead load, reduced by the vertical SSE acceleration of 0.167g. In the original Table 3(f)-1, the effect of vertical SSE acceleration was not incorporated.

In computing the horizontal sliding resistance of the holdup tank enclosure and spent fuel pool, only the resistance from μW was used. While some limited tension areas are expected, the stress distribution has no influence on the frictional resistance since the net vertical load "W" is unchanged by the nonuniform distribution of load on the foundation level.

The rock foundation under the spent fuel pool and holdup tank enclosures in the Fuel Building is similar to the Control Building. Therefore, for a discussion regarding the adequacy of the foundation material, justification for the value of μ, and preparation of the foundation prior to concrete placement, please refer to Clarification # 19.

Prior to placing the concrete for the spent fuel pool and holdup tank base slabs, a 2-in. thick concrete "work slab" was placed on top of the rock foundation. This, however, does not affect the resistance to sliding because the coefficient of friction, μ, between the concrete surfaces would be at least 0.7. No waterproofing materials were used under the base slabs. Conformance to Standard Review Plan sections 2.5.4 and 3.8.5 for the Fuel Building are as discussed in Clarification # 19.

TABLE NO. 3 (f) - 1

(As revised on Oct. 5, 1978)

FUEL BUILDING

HOLD-UP TANK ENCLOSURE AND SPENT FUEL POOL

SAFE SHUTDOWN EARTHQUAKE

(.25g @ 5% damping)

STRUCTURE	HORIZONTAL RESISTANCE .7W* (Kip)	DIRECTION OF EARTHQUAKE N-S BASE SHEAR (Kip)	N-S FACTOR OF SAFETY	E-W BASE SHEAR (Kip)	E-W FACTOR OF SAFETY
HOLD-UP TANK ENCLOSURE	5230	2780	1.88	2780	1.89
SPENT FUEL POOL (WITH WATER)	8400	2260	3.72	2990	2.81
SPENT FUEL POOL (WITHOUT WATER)**	6820	2260	3.02	2990	2.28

*W, dead load reduced by vertical seismic acceleration (.167g).

**Seismic analysis was not performed for empty pools; conservatively, the base shear forces corresponding to pool filled with water were used.

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QUESTION # 24

"Where in the FSAR are the minimum factors of safety against overturning and sliding for the SSE and OBE documented?"

CLARIFICATION # 24

The FSAR does not specify minimum factors of safety against overturning and sliding for the SSE and OBE. However, the Control-Auxiliary-Fuel Building complex meets the minimum factors of safety for sliding and overturning as specified in Standard Review Plan, Section 3.8.5.

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QUESTION # 25

Supply the new information from the Berkeley tests on more squat walls and compare your criteria to the results of these tests to substantiate the stress levels indicated by the new analyses, and also the trend which seems to be indicated by the test data that if you have horizontal reinforcing greater than vertical steel you can go to higher stress levels indicated by the specimen tests with higher reinforcement ratios than are present in the Trojan walls.

CLARIFICATION # 25

Table B-4 lists results of tests performed by the Earthquake Engineering Research Center, University of California, Berkeley, which were correlated with the basic criteria. The test results show excellent correlation with the basic criteria. The reinforcement percentages of the Berkeley test specimens varied, as shown in Table B-4. Most of Schneider's test specimens used for developing the basic criteria did not have any horizontal reinforcement (see Table B-1). Thus, the tests and the correlation indicate that the variation in reinforcement percentages is not a significant parameter. An increase in the horizontal reinforcement improves inelastic deformability but does not increase shear strength significantly. Rather, it is the load-resisting mechanism that takes place during the lateral deformation (such as the formation of a compression strut) that contributes to the strength. This is shown by the fact that considerable strength develops even for the specimens without any reinforcement (specimen HCBL-11-1).

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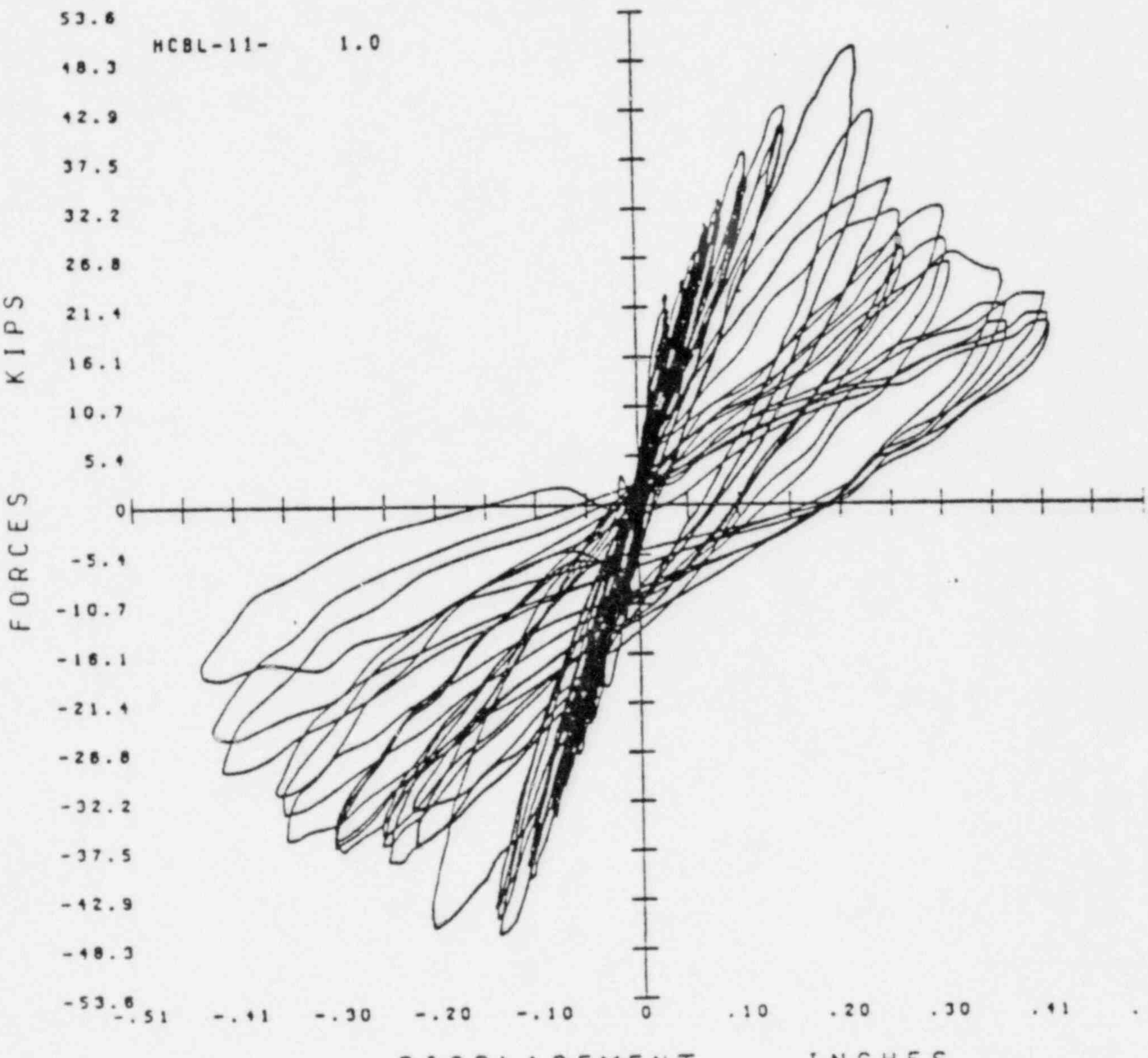
CLARIFICATION # 25 continued

The hysteresis curves of the Berkeley HCBL test specimens used in the correlation of data in Appendix B are presented in Figures 25-1 to 25-7.

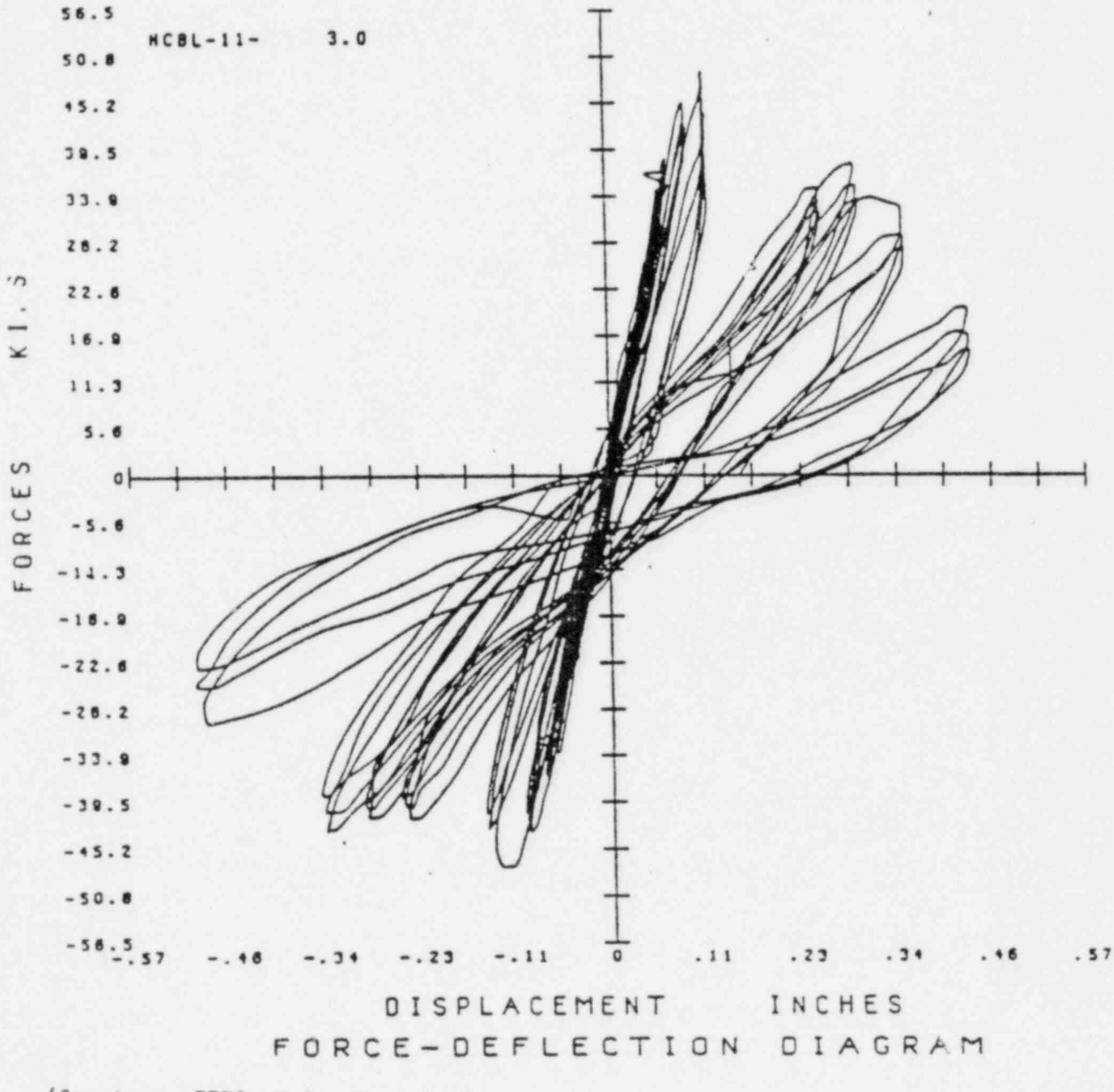
Figure 25-8 shows the only hysteresis curve available from more recent tests for a squat wall with H/W = 0.5. (Five other specimens with H/W = 0.5 have been recently tested. The hysteresis curves are being developed, but are not available at this time). The reinforcement of these six squat specimens varies. Through verbal communication with the Berkeley test group, the reported axial stresses at ultimate vary between 120 and 160 psi. Assuming an upper value of σ_n = 160 psi on these specimens, according to the basic criteria, the calculated v_u is 260 psi. The reported ultimate shear stresses from tests vary from 320 to 427 psi which is higher than the calculated v_u.



FORCE-DEFLECTION DIAGRAM

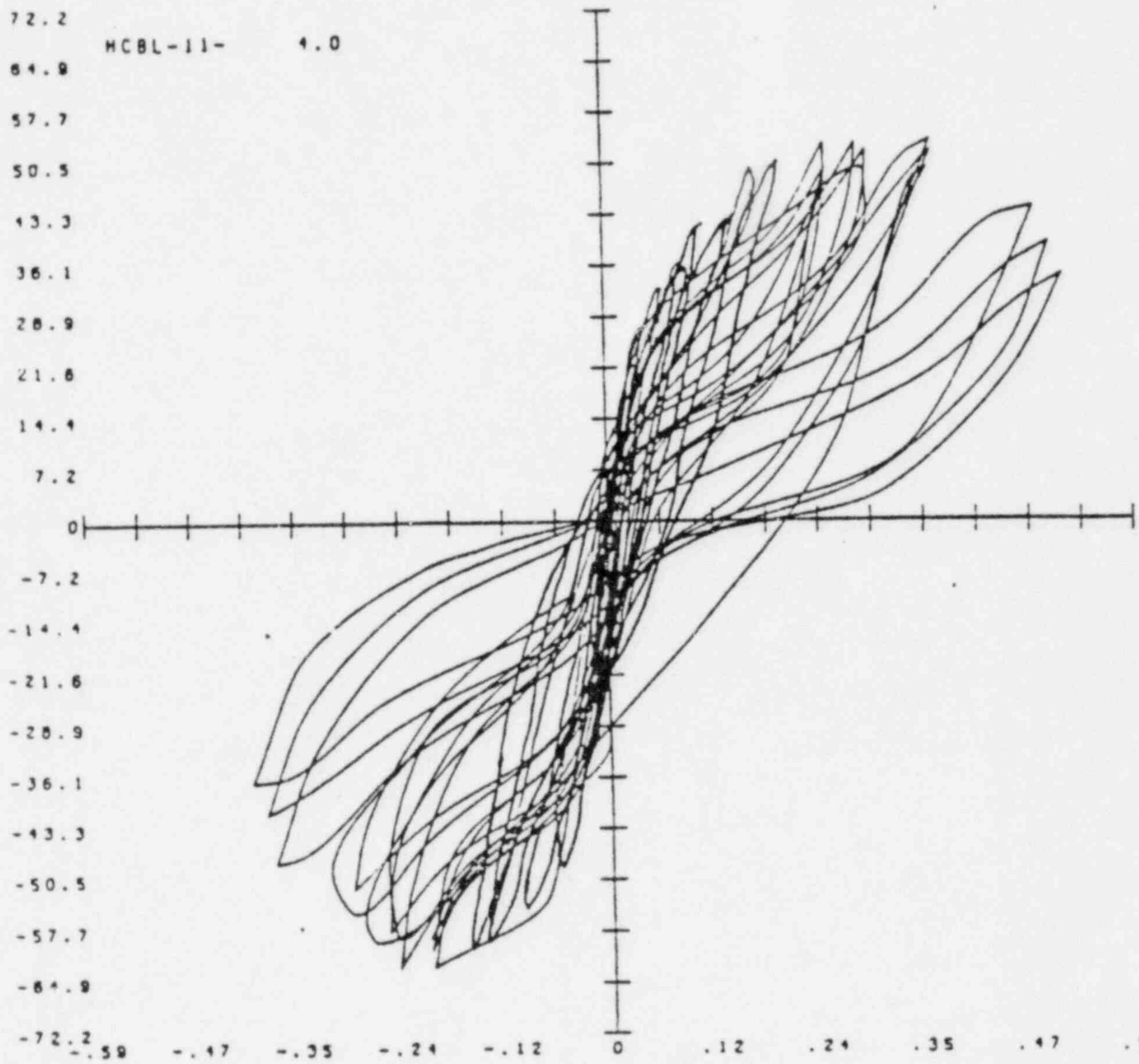
(Courtesy, EERC, U.C., Berkeley)

Figure 25-1



(Courtesy, EERC, U.C., Berkeley)

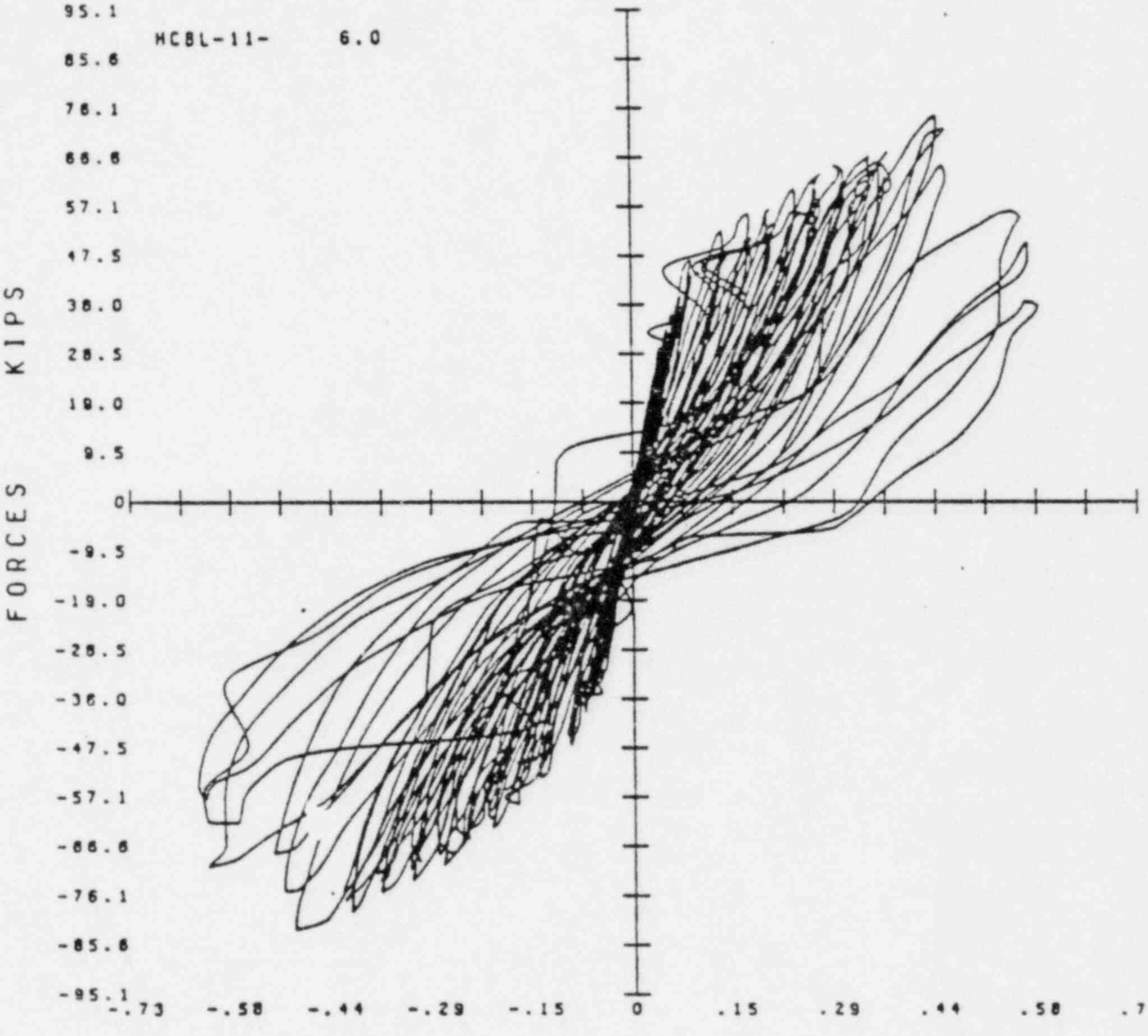
Figure 25-2



FORCE-DEFLECTION DIAGRAM

(Courtesy, EERC, U.C., Berkeley)

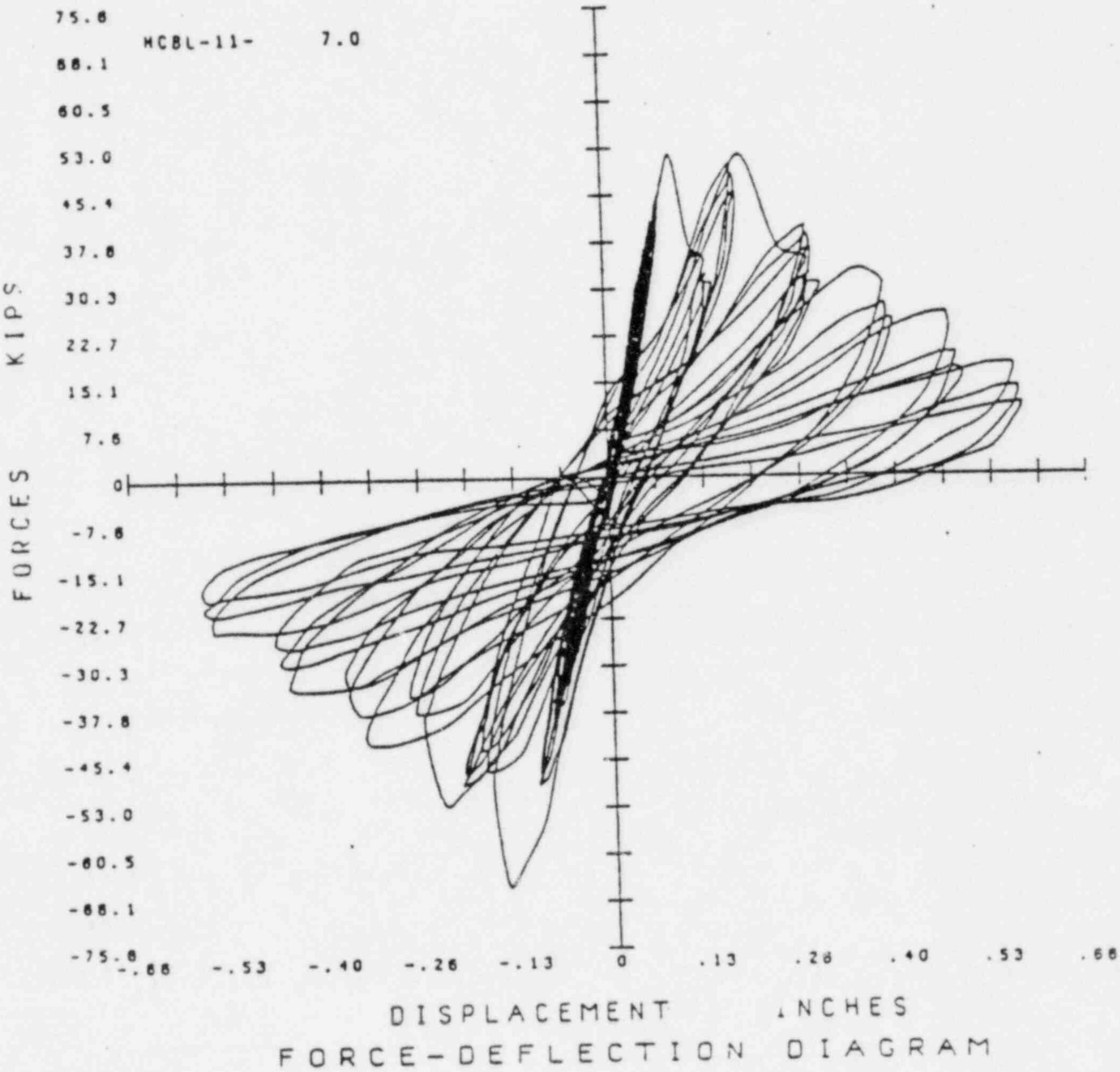
Figure 25-3



FORCE-DEFLECTION DIAGRAM

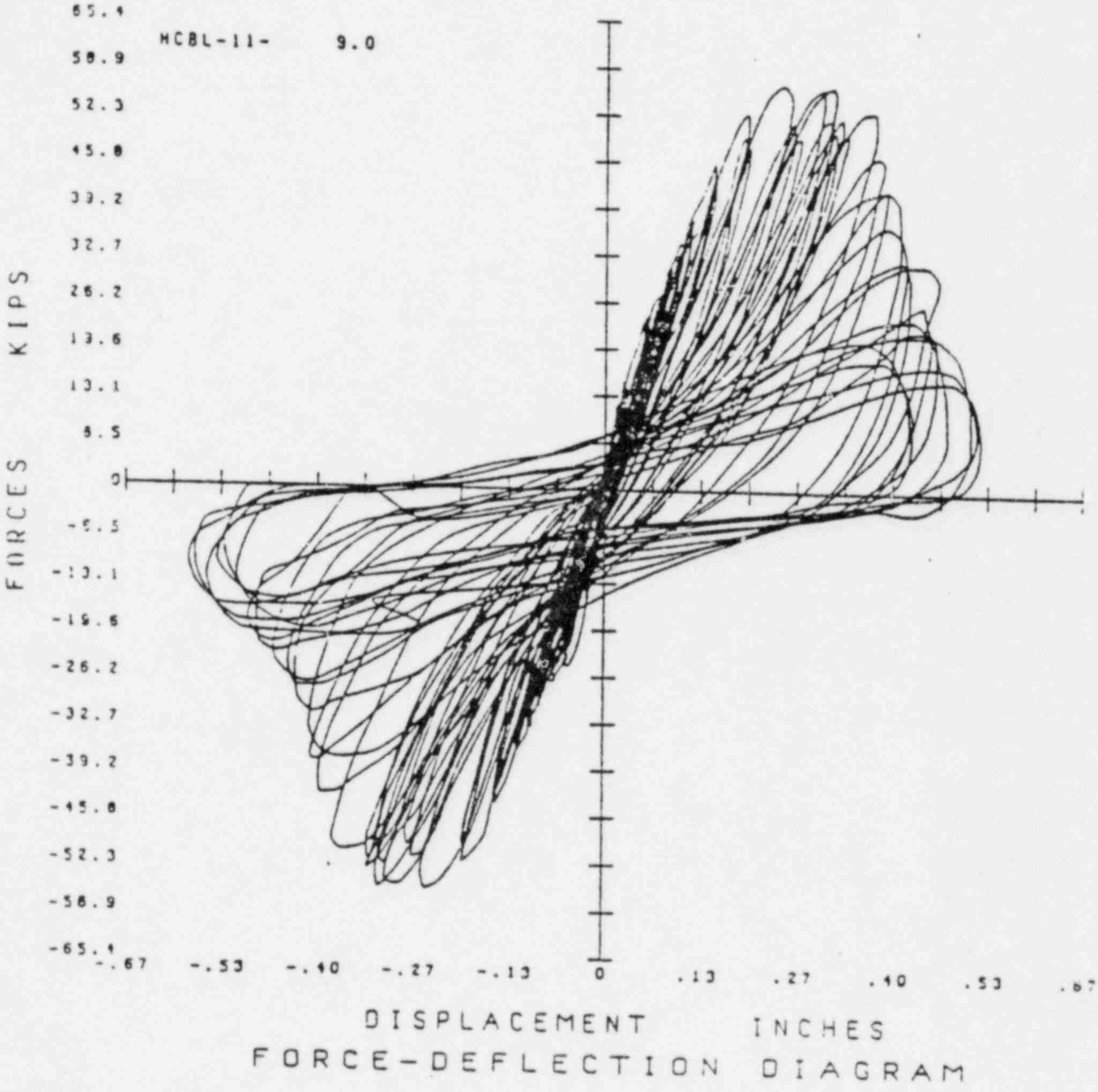
(Courtesy, EERC, U.C., Berkeley)

Figure 25-4



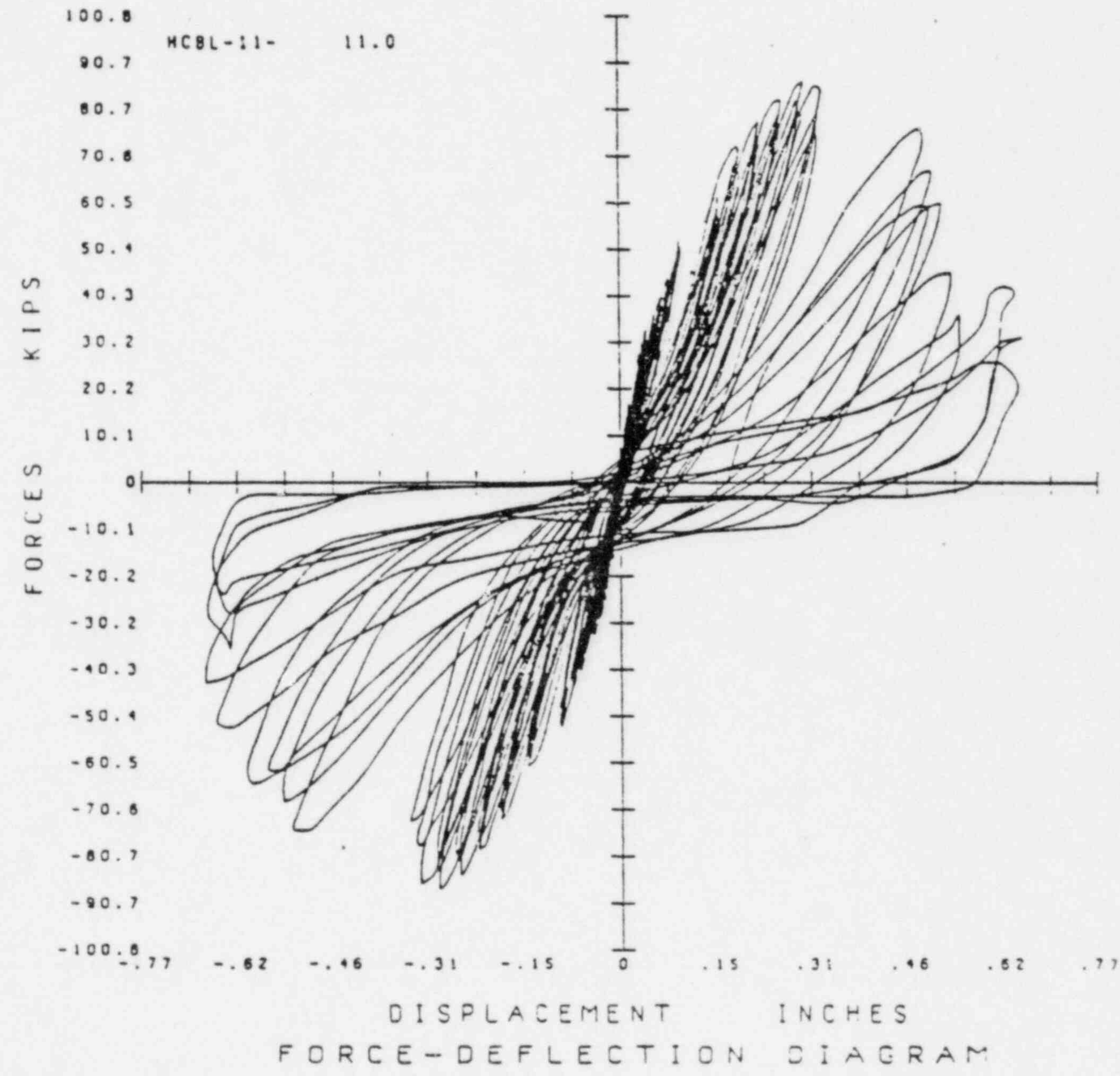
(Courtesy, EERC, U.C., Berkeley)

Figure 25-5



(Courtesy, EERC, U.C., Berkeley)

Figure 25-6



(Courtesy, EERC, U.C., Berkeley)

Figure 25-7

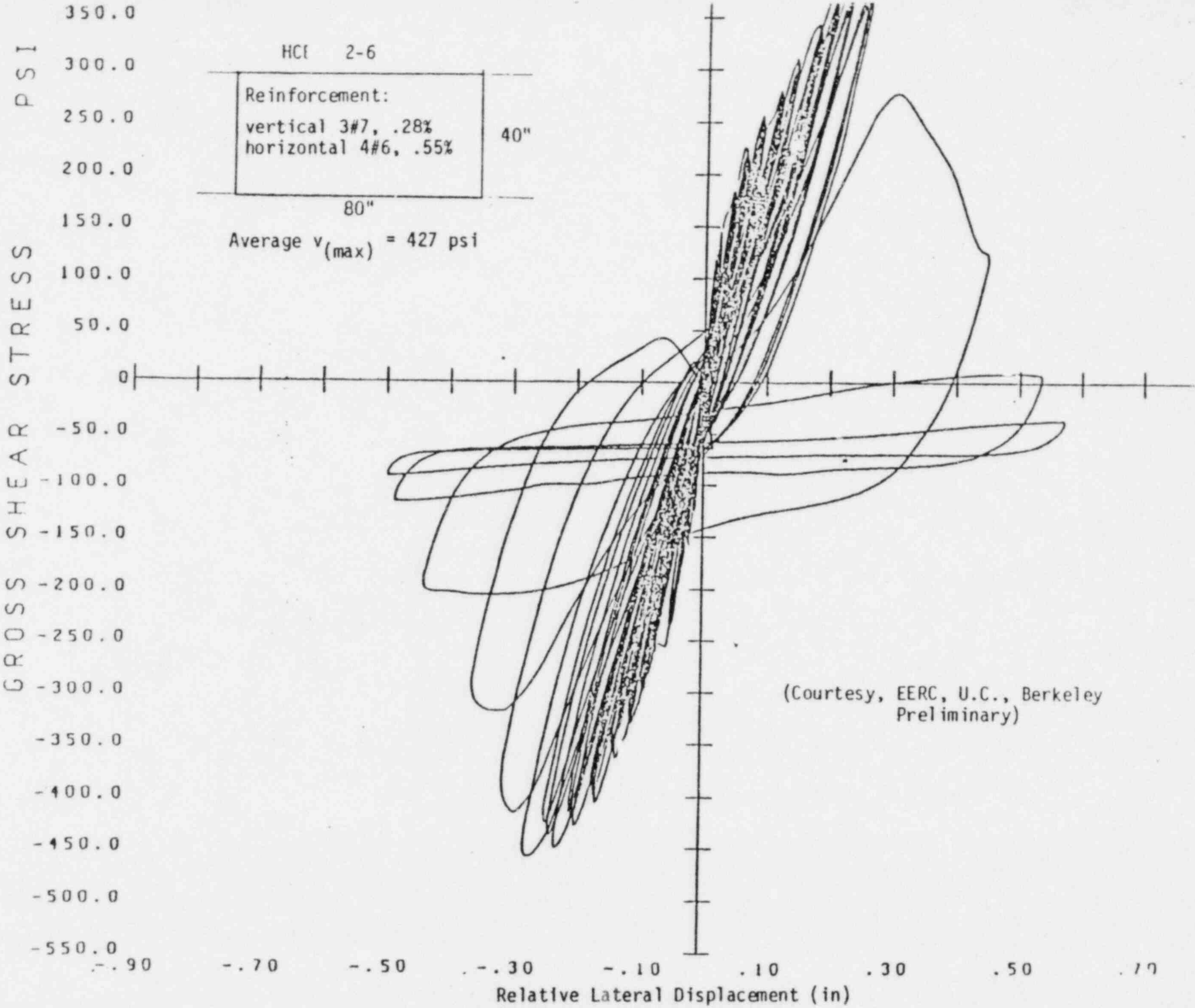

HCI 2-6
Reinforcement:
vertical 3#7, .28%
horizontal 4#6, .55%
40"
80"
Average $v_{(max)}$ = 427 psi
(Courtesy, EERC, U.C., Berkeley
Preliminary)
GROSS SHEAR STRESS PSI
350.0
300.0
250.0
200.0
150.0
100.0
50.0
0
-50.0
-100.0
-150.0
-200.0
-250.0
-300.0
-350.0
-400.0
-450.0
-500.0
-550.0
-.90
-.70
-.50
-.30
-.10
.10
.30
.50
.70
Relative Lateral Displacement (in)

Figure 25-8

CLARIFYING INFORMATION TO SUBMITTALS ENTITLED

TROJAN CONTROL BUILDING
SUPPLEMENTAL STRUCTURAL EVALUATION
SEPTEMBER 19, 1978

and

RESPONSE TO QUESTIONS FROM THE
NUCLEAR REGULATORY COMMISSION
DATED AUGUST 30, 1978
September 20, 1978

QUESTION # 26

"Indicate the differences in modeling between previous analyses and the STARDYNE analyses with regard to which walls were considered. What were the additional walls considered and verify that those additional walls (just as all the others) do not have any problem with any excessive cracking, etc. Also, relate Figure 3-2 in the September 19 report to Figure 2-1 in the August 19 report. Each figure shows a wall layout and these are not obviously similar. Which are which?"

CLARIFICATION # 26

The walls that were considered as part of the Control Building in the original seismic analysis performed in 1971 are shown in Figure 2-1 of the August 19, 1978 submittal. In the original analysis, the only walls in the Auxiliary Building that were considered as part of the Control Building were walls 6 and 7 in the N-S direction and walls 8A and 9A, Figure 2-1, in the E-W direction. The re-evaluation study considered all the walls in the original analysis as well as some additional walls in the Auxiliary Building (walls 6, 7, 8, 14, 15, and part of 10 and 13 shown on Figure 3-2, and walls 4, 5, 6, 9, 10, 11, 12, and part of wall 8 shown on Figure 3-3). In the finite element model, all walls considered in the re-evaluation study plus the small walls 11 and 12 at elevations 45'-61' (Figure 3-2) in the Control Building were considered.

In the finite element model, some of the walls in the Auxiliary Buiding have been shifted slightly for modeling convenience. For example, walls 6 and 7 shown in Figure 2-1 of the August 19, 1978 submittal, which straddle Column Line L, have been moved to Column Line L in the finite element model and constitute wall 5 shown in Figure 3-2.

A field survey has verified that none of the walls being relied upon for lateral resistance in any of the analyses have excessive cracking.

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QUESTION # 27

"Verify that in the supplemental analysis, just as in the previous analyses, the critical stories are 45 to 77 and that the other members (walls and floors) are well within criteria such that we do not have to look at them."

CLARIFICATION # 27

In the supplemental analysis, just as in the previous analyses, the critical stories are those at el 45 ft to 77 ft. Walls above el 77 ft have capacities at least equal to the capacities of the walls at el 45 ft to 77 ft, and the loads on these walls above el 77 ft are lower. The slabs that need to act as shear diaphragms are discussed in the response to NRC question 3(d).

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QUESTION # 28

With regard to the Service Water lines that pass from the Control Building across the hallway at Turbine Building Elevation 69 ft: show that with displacements (ABS) considered, there is no failure of these lines; or if there is a failure, that the effects of such a failure are acceptable. If the line must be isolated, what other equipment is affected?

CLARIFICATION # 28

In the event of an SSE, the failure of service water lines (2"-HKD-1-71 and 2"-HKD-2-68) at Turbine Building elevation 69 ft has been considered. The subject lines supply cooling water to the room coolers (V-163A, B, C, and V-164) located in the Turbine Building switchgear room. The switchgear room houses only "A" train components. With complete loss of cooling water, the switchgear room temperature would require more than three hours to rise to the maximum postulated temperature limit of 107 F. This permits time not only to achieve a safe shutdown condition, but also time for operator action to achieve alternative cooling means.

Although the lines in question would be overstressed by the displacements resulting from an SSE, it is highly unlikely that an end-to-end break would occur due to the very ductile materials (CuNi) employed. Even assuming that such a break were to occur, the inventory carried in these lines would empty into the Turbine Building. The Control Building areas would not be flooded. The Turbine Building has sufficient drain flow capacity in terms of floor drains and stairwells to accommodate the flow (approximately 300 gpm) due to an end-to-end line break. Therefore, flooding of any safety-related equipment would not occur. Further, the loss of

CLARIFICATION # 28, continued

inventory expected from such a break is insufficient to have any significant effect on the complete Service Water System. These lines can not be isolated; however, their rupture will not affect other safety-related equipment. For these reasons, the postulated failures of these service water lines can be accommodated without affecting the ability to achieve safe shutdown.

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QUESTION # 29

What are the displacements between the Control Building and the Containment at Elevation 77 ft? State the loadings considered for containment deflection.

CLARIFICATION # 29

Relative displacements between the Control Building and Containment at Elevation 77' are given below. The displacements for the Containment are for the SSE of 0.25g.

	Maximum Displacement(inch) SSE 0.25g	
	N-S Direction	E-W Direction
Control Building @ 5% damping	0.72	0.06
Containment @ 5% damping	0.04	0.04
ABS Combination	0.76	0.1
SRSS Combination	0.72	0.072

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September 20, 1978

QUESTION # 30

"Relative to parts (a) and (d) of Question 19: (a) state whether or not blasting was performed and discuss its effect on the foundation rock, and (b) state whether shear values on cores have also been obtained as a control."

CLARIFICATION # 30

a. Although blasting was employed in excavation, it had no significant effect on the foundation rock. The excavated rock was cleaned by barring and prying of loose and detached rock fragments disturbed by blasting.

b. Shear tests on cores were not performed because information sufficient to adequately evaluate the shear strength of the foundation work was obtained from inspecting core samples, unconfined compression tests, compressional and shear wave velocity tests of the actual in situ foundation rock, and inspection of the exposed foundation conditions.