AUG 25 1976

DOCKET NOS. 50-275 and 50-323

APPLICANT: Pacific Gas and Electric Company (PG&E)

FACILITY: Diablo Canyon Nuclear Power Station, Units 1 and 2 (Diablo Canyon)

SUMMARY OF MEETING HELD ON AUGUST 11, 1976 TO DISCUSS SEISMIC REEVALUATION OF DIABLO CANYON

We met with the applicant and the applicant's consultant on August 11, 1976 in Bethesda, Maryland to discuss the seismic reevaluation of Diablo Canyon.

A list of attendees is provided in Enclosure 1.

BACKGROUND

The seismic design of Diablo Canyon was being reevaluated in terms of a magnitude 7.5 earthquake on the Hosgri fault with an effective site acceleration of 0.75g. This earthquake, which involved more severe ground shaking than the original design earthquake was described in Supplement No. 4 to the Safety Evaluation Report (SER) on May 11, 1976. Following Supplement No. 4 to the SER it was still necessary to develop design response spectra to be used as inputs for engineering calculations in reevaluating the plant.

Since May 11, tentative or preliminary response spectra had been developed independently by the applicant's consultant, Dr. John Blume, and the NRC's consultant, Dr. Nathan Newmark. These tentative spectra and their bases had been discussed with the Advisory Committee on Reactor Safeguards (ACRS) in June and July.

DESIGN RESPONSE SPECTRA

In Amendment 44 to the Final Safety Analysis Report (FSAR), the applicant had submitted the design response spectra he proposed to use in the reevaluation and had described the bases for them. These proposed spectra were similar to the tentative spectra developed by our consultant, but some difference remained. We and our consultant had reviewed the amendment and had provided a set of draft questions to the applicant by telecopy. A copy of these questions is attached in Enclosure No. 2

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At this meeting (August 11, 1976) the applicant provided responses to the draft questions. A copy of the applicant's responses is attached as Enclosure No. 3. We discussed these responses with the applicant.

We agreed to consider the applicant's responses and to let the applicant know which questions would be formally transmitted. The applicant was prepared to revise the FSAR to include responses to any or all of the draft questions as needed.

FUEL STORAGE FACILITIES

On July 9, 1976 the applicant had applied under 10 CFR Part 70 for a license to store fuel in Unit 2. On this application it had been stated that the fuel storage facilities design was considered adequate to maintain spacing between fuel elements for the seismic loading conditions resulting from the magnitude 7.5 earthquake described in Supplement No. 4 to the SER. At this meeting (August 11, 1976) the applicant discussed the bases for that statement.

The applicant stated that calculations had been performed, using the applicant's proposed design response spectra, which indicated smaller floor responses at the location of the new and spent fuel storage racks than were considered in the original design. The reasons for the smaller responses were: (1) taking credit for additional embedment, (2) increased damping values and, (3) reductions due to wave/base slab interactions. In addition, the applicant stated that calculations had been performed which indicated that the capabilities of the fuel storage facilities were greater than necessary to meet the floor responses considered in the original design. A brief summary of this information is attached as Enclosure No. 4.

Accordingly, the applicant felt that the seismic design of the fuel storage facilities (new fuel racks, new fuel storage vault, spent fuel racks and spent fuel pool) would be adequate, even in the event that the staff required some changes in the proposed design response spectra. We agreed that this conclusion was reasonable and any changes that might be made to the proposed design response spectra were unlikely to affect the acceptability of the design in this area.

/S/
D. P. Allison, Project Manager
Light Water Reactors Branch No. 1
Division of Project Management

Enclosures:
As stated

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|---|---|---|---|---|---|---|
| cc: | See page 3 | | | | | |
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| SURNAME➤ | | DAllison/red | JStolz | | | |
| DATE➤ | | 8/ /76 | 8/ /76 | | | |

AUG 25 1976

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ENCLOSURE NO. 1

ATTENDANCE LIST

DIABLO CANYON NUCLEAR POWER STATION

MEETING HELD ON AUGUST 11, 1976

PG&E

W. Lindblad
J. Blume
G. Linfesty
R. Bettinger
J. Hoch
V. Ghio

PG&E Consultants

J. Blume
R. Gallagher

Westinghouse

T. Esselman

NRC

I. Sihweil
K. Kapur
D. Allison
L. D. Davis
E. Ketchen
J. O'Brien
M. Lehman
W. Gammill

ENCLOSURE NO. 2

DIABLO CANYON NUCLEAR POWER PLANT
DOCKET NOS. 50-275, 50-323
STRUCTURAL ENGINEERING BRANCH
DRAFT REQUEST FOR ADDITIONAL INFORMATION

1. Provide justification for not using the original set of 10 earthquakes utilized in developing the Hosgri spectra. What would be the impact of the two omitted earthquakes on proposed response spectra?

2. Provide justification for the magnitude of the adjustment made to the response spectrum above 0.4 sec. for the higher magnitude earthquake. Provide graph showing the spectra shape as derived from the strong motion records cited on page 2 and the adjustment for long periods for the higher magnitude earthquake.

3. (a) Provide the mean, median and the standard deviations of the spectral response. It is not clear whether mean + standard deviation ($1\sigma$) or mean + $1/2\sigma$ was used in developing the final spectra. Provide justification if mean + $1/2\sigma$ was used.

   (b) Provide figures illustrating the spectra for the earthquakes on page 2 all on the same graph scaled to the .75g high frequency asymptote.

4. What was the shear wave velocity $V_s$ used in calculating Tau? Provide the basis for the use of this shear wave velocity. What is the difference between t and Tau?

5. Provide the basis for reducing the response spectra because of embedment and wave scattering phenomenon (Ref. 4). Results obtained are different whether the larger or smaller foundation dimension is used in calculations using Ref. 4 (Wang and Trifunac). State what dimension was used in calculations and provide basis. Also provide a typical set of calculations. The statistical treatments, smoothing and adjustments mentioned in the first

paragraph of page 3 should be discussed in some detail with a clear description, justification and evidence supporting each procedure.

6. Provide justification for the statement that "torsion represents a relatively small contribution to the total seismic load".

7. Provide the basis for the use of nTau for torsionally induced tangential acceleration at the extremities of the foundation.

8. Provide the basis for the formula for $T_0$ on page 5. Provide the basis for various values of n used in calculating $T_0$.

9. Provide justification for the statement that "tilting from some waves is only possible where the foundations are relatively deep".

*10. Provide the material properties and allowable stresses to be used in analysis or design for Diablo Canyon plant.

11. Provide justification for the use of ductility factor of 1.3 for Category I structures other than containment and for Category I equipment or its supports.

*12. Provide justification for any Category I structures, systems and components not to be included in the analysis to verify their adequacy and to identify any necessary modifications to structures, systems and components.

13. The table on page 3 should be discussed in detail. How was the "Effective length of foundation" or perhaps more appropriately the "Effective Shear Velocity" determined. Support this with shear velocity field data. The discussion should be complete enough that the reviewer can proceed from field velocity data, dimension of structures, and a rationale for the "Tau Effect" to reproduce the table and defend its adequacy.

14. Provide justification for the differences in response spectra obtained as compared to the response spectra recommended by Dr. Newmark in his report to NRC. The differences appear to be

large around 2 cps and between 10 to 30 cps (especially in Fig. 13)

15. Provide detailed explanation of Fig. 14 regarding containment structure torsional motion and its use in analysis.

16. Provide justification for the following assumptions made in Ref. 4 (by Wang and Trifunac) and the paper's applicability to Diablo Canyon plant structures.

    (i) Only SH waves are treated
    (ii) Wave front is planar
    (iii) Earthquake motions are harmonic
    (iv) Structure (shear wall) is infinitely long
    (v) Foundation is infinitely long
    (vi) Foundation is infinitely rigid

17. Figures 1 through 13 should be discussed in detail. Which of the references cited were used in calculating the response spectra? Provide typical calculations.

ENCLOSURE NO. 3

1. Question) Provide justification for not using the original set of 10 earthquakes utilized in developing the Hosgri spectra. What would be the impact of the two omitted earthquakes on proposed response spectra?

Reply) The original set of 10 earthquake records included the 2 components of horizontal motion from the 1967 Koyna Dam Record. Subsequent to the original work it was learned that the Koyna Dam records were not reliable, either due to the effects of the dam or instrumental problems. It was thus decided to eliminate the Koyna Dam records. The impact of omitting these records must be considered academic due to the spurious nature of the Koyna records, however, it will be shown in a later figure ~~if needed~~.

DRAFT

JB 8/3/76

Supplementary Material for Question No. 1

Figure K shows 7% damped spectra for elastic conditions, free field (Tau = 0). The Hosgri curve is shown together with the two Koyna curves. The Hosgri curve is above the Koyna longitudinal curve for almost the entire spectral range and it is above both Koyna curves for most of the spectral range. The greatest divergence is at 0.11 sec period. However, the Hosgri curve has a weight of 8 (8 records) while the Koyna has 2 (2 records). Taking a weighted average at the worst period, 0.11 sec, the result is about 1.6 g which is shown by the cross in the figure. The Koyna records would have little effect on the average curve if they were used. However, the transverse Koyna peak is no doubt a result of dam response and should not be used.

DRAFT

JAB 8/5/76

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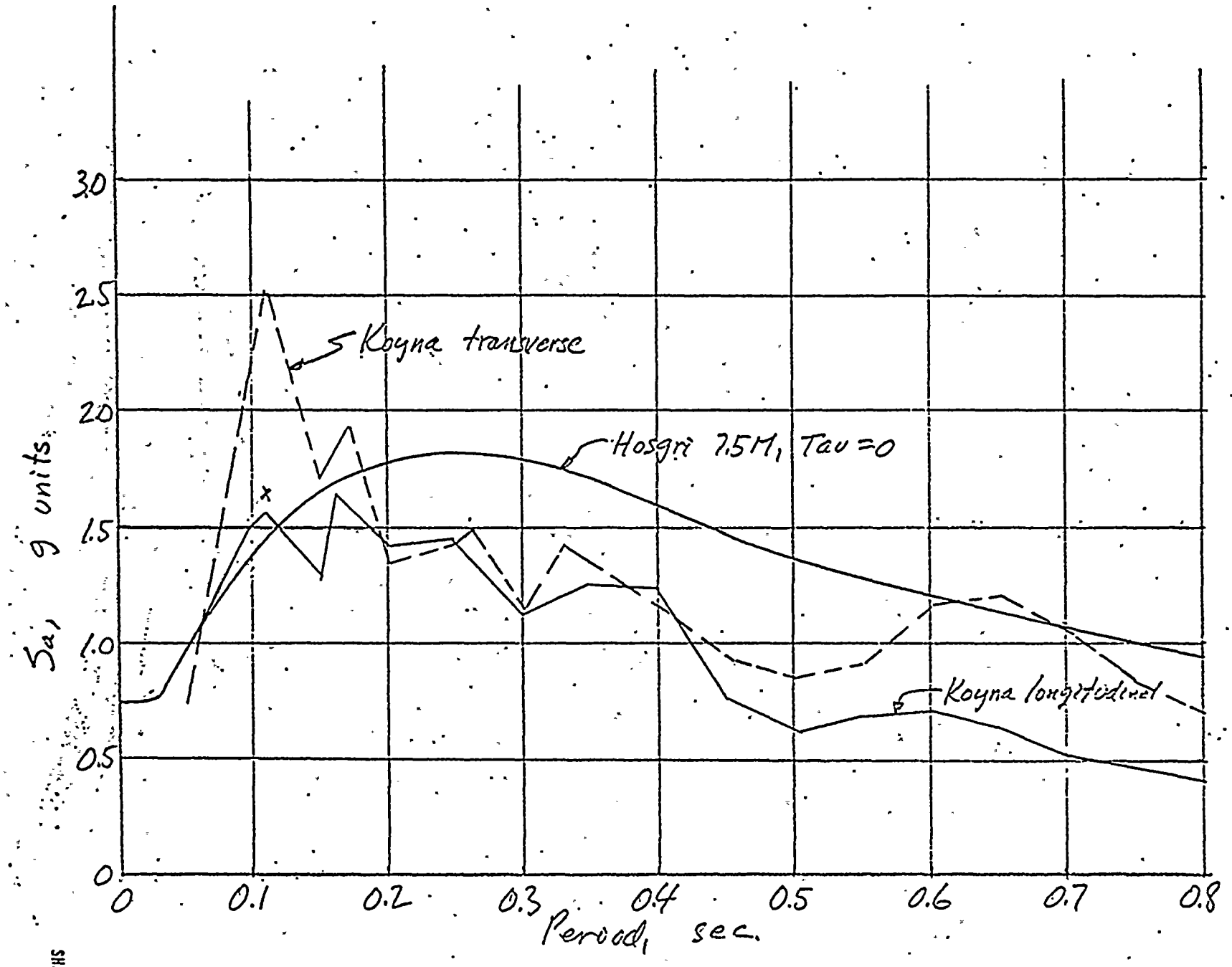
JOB NO. | JOB | BY | DATE 8/4/76
CLIENT | SUBJECT Diablo Canyon | CHK'D | DATE




SHEET NO.

FIG. K — 7% DAMPED SPECTRA, ELASTIC, TAU=0

2. Question) Provide justification for the magnitude of the adjustment made to the response spectrum above 0.4 sec. for the higher magnitude earthquake. Provide graph showing the spectra shape as derived from the strong motion records cited on page 2 and the adjustment for long periods for the higher magnitude earthquake.

Reply) Figure N shows the spectral diagrams for Tau = 0.08, 7% damping, elastic conditions. Curves 2 and 3 provide the spectral response data from the 8 close-in rocky site records for the mean plus one-half standard deviation and the mean plus one standard deviation, respectively. Curve 1 in Figure N is the proposed Hosgri curve and it is considerably greater than curves 1 and 2 beyond 0.5 sec period. This increase is to provide for magnitude scaling effects.

The 8 close-in records were treated as follows for a study of magnitude scaling. Each record was normalized to 0.1g and then each pair of horizontal components for each station was averaged and the pseudo spectral response velocity was computed for 7% damping and the various periods of the spectrum. Figure V, attached, shows the spectral velocity plotted versus magnitude for several periods ranging from 0.6 secs to 2 secs. Also shown on the plot are the points at 7.5M taken from the Hosgri curve, converted to spectral velocity and normalized to 0.1g. It can be seen that the points taken from the curve represent a reasonable extrapolation of the results from the four earthquakes having magnitudes of 5.3, 5.6, 6.0, and 6.5.

DRAFT
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JOB NO. | CLIENT | JOB DIABLO CANYON | SUBJECT HOSGRI 7.5M | BY | CHK'D | DATE | DATE 5/4/72 | SHEET NO.

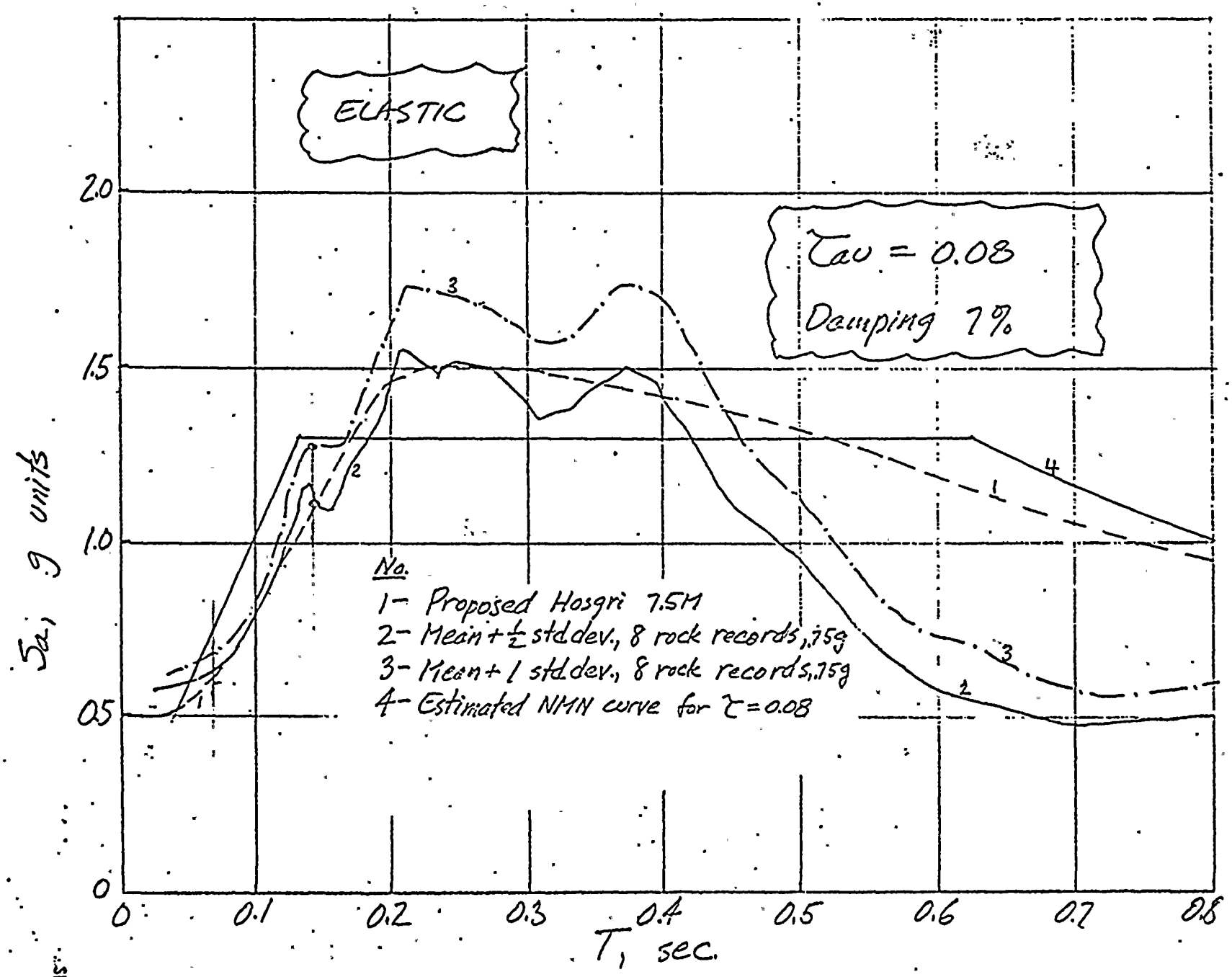


FIG. N — COMPARISON OF SPECTRAL CURVES, τ=0.08, Dpg 7%

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CLIENT | SUBJECT DIABLO CANYON | CHK'D | DATE

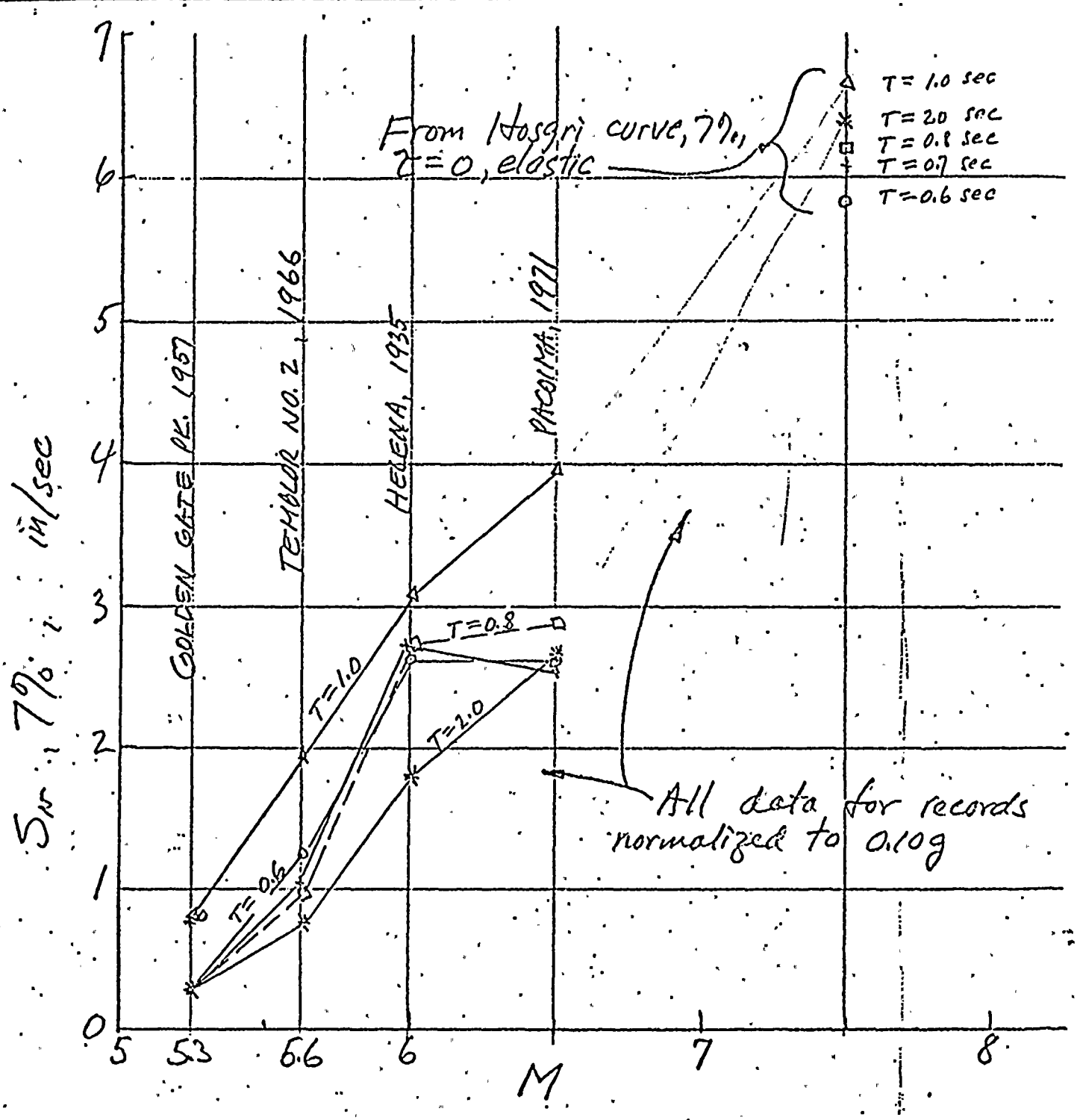


FIG. V — SPECTRAL VELOCITY VS. MAGNITUDE

SHEET NO.

3. Question) (a) Provide the mean, median and the standard deviations of the spectral response. It is not clear whether mean + standard deviation ($\sigma$) or mean + $1/2\sigma$ was used in developing the final spectra. Provide justification if mean + $1/2\sigma$ was used.

(b) Provide figures illustrating the spectra for the earthquakes on page 2 all on the same graph scaled to the .75g high frequency asymptote.

Reply) (a) Figure N shows curve 2 for the mean plus one-half standard deviation for the 8 close-in rock records normalized to 0.75g and adjusted for Tau equal to 0.08. All curves on the sheet are elastic, and for 7% of critical damping. Curve 3 is the same except it is for the mean plus one standard deviation. The value of one standard deviation at any period is twice the difference between curves 2 and 3.

Curve 1 is the proposed spectral curve for the analysis to resist a 7.5M Hosgri zone earthquake. It can be seen that this smooth curve averages slightly greater than curve 2 except in the extreme high frequency range where it is less (with some allowance for embedment values not otherwise considered) and beyond 0.4 sec period where allowances have been made for magnitude effects.

Curve 2 was used as the primary basis because (1) the carefully selected 8 close-in rocky site records are considered to closely model the site conditions and thus fall above the values that would be obtained by more conventional methods using a mixture of alluvium records at greater distances. In other words, similar or lesser results would have been obtained using mean plus one standard deviation with a hybrid mix of records less appropriate to the site; and (2) the joint probability of peak ground acceleration and the spectral dynamic amplification was estimated for the site conditions, assuming up to a 7.5M earthquake could occur on the Hosgri. It was found that with the mean plus 0.4 standard deviation on shape (or DAF) the joint probabilities of exceedance of spectral acceleration were the same as if one standard

deviation above the mean for shape had been combined with a more generally used mean peak ground acceleration for site exposure. A value of 0.5 was used in lieu of 0.4, thus providing even less chance of exceedance than is generally considered a satisfactory level.

Reply)(b) Figure M, attached, provides the requested plot of the 8 spectral diagrams, each normalized to 0.75g and each for 7% of critical damping. These are elastic curves with no adjustment for Tau effects.

DRAFT

JCB 8/5/76

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JOB NO. | CLIENT | JOB DIABLO CANYON | SUBJECT HOSGRI 7.5M | BY | CHK'D | DATE 6/4/73 | DATE | SHEET NO.

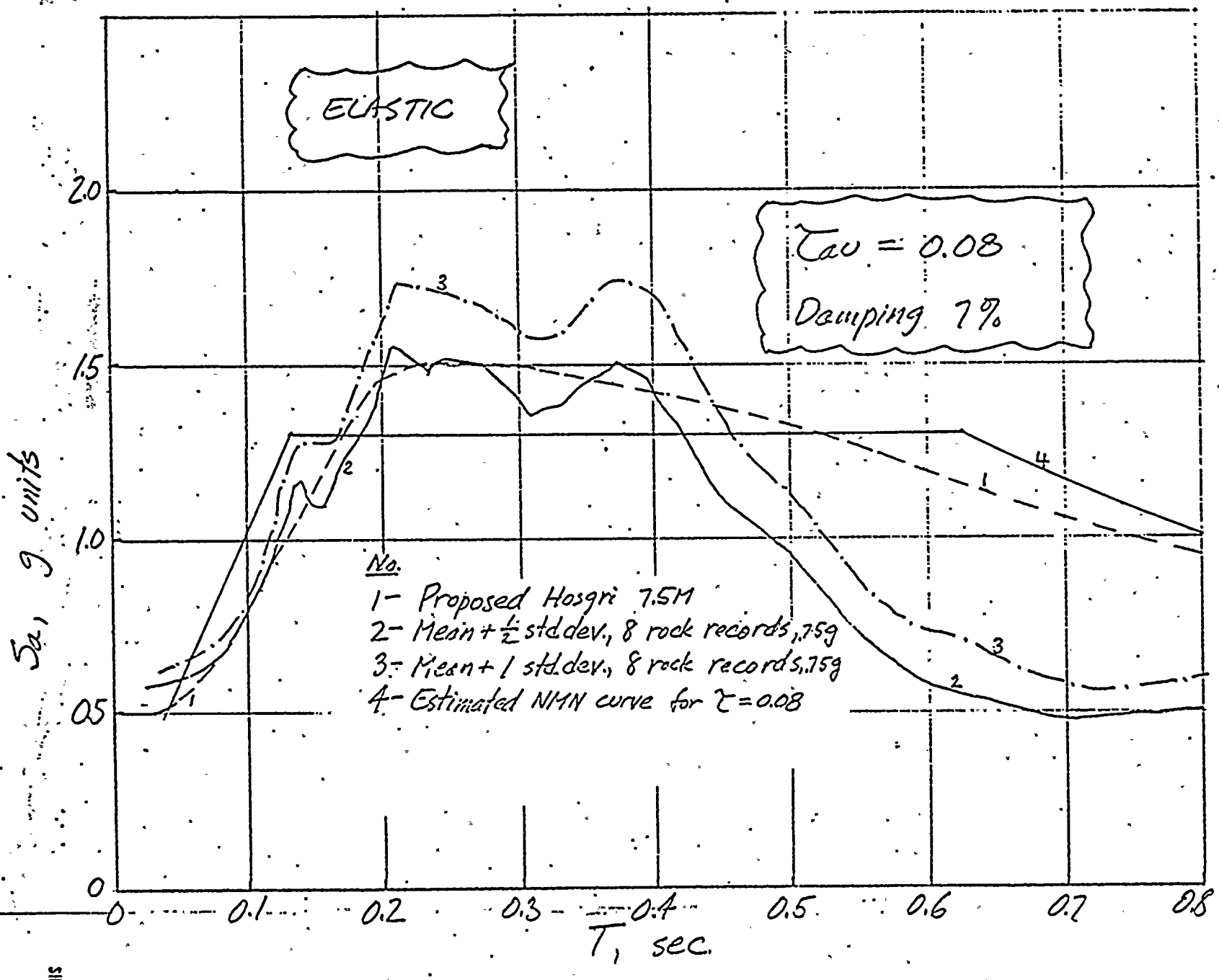


FIG. N — COMPARISON OF SPECTRAL CURVES, $\tau$=0.08, Dpg 7%

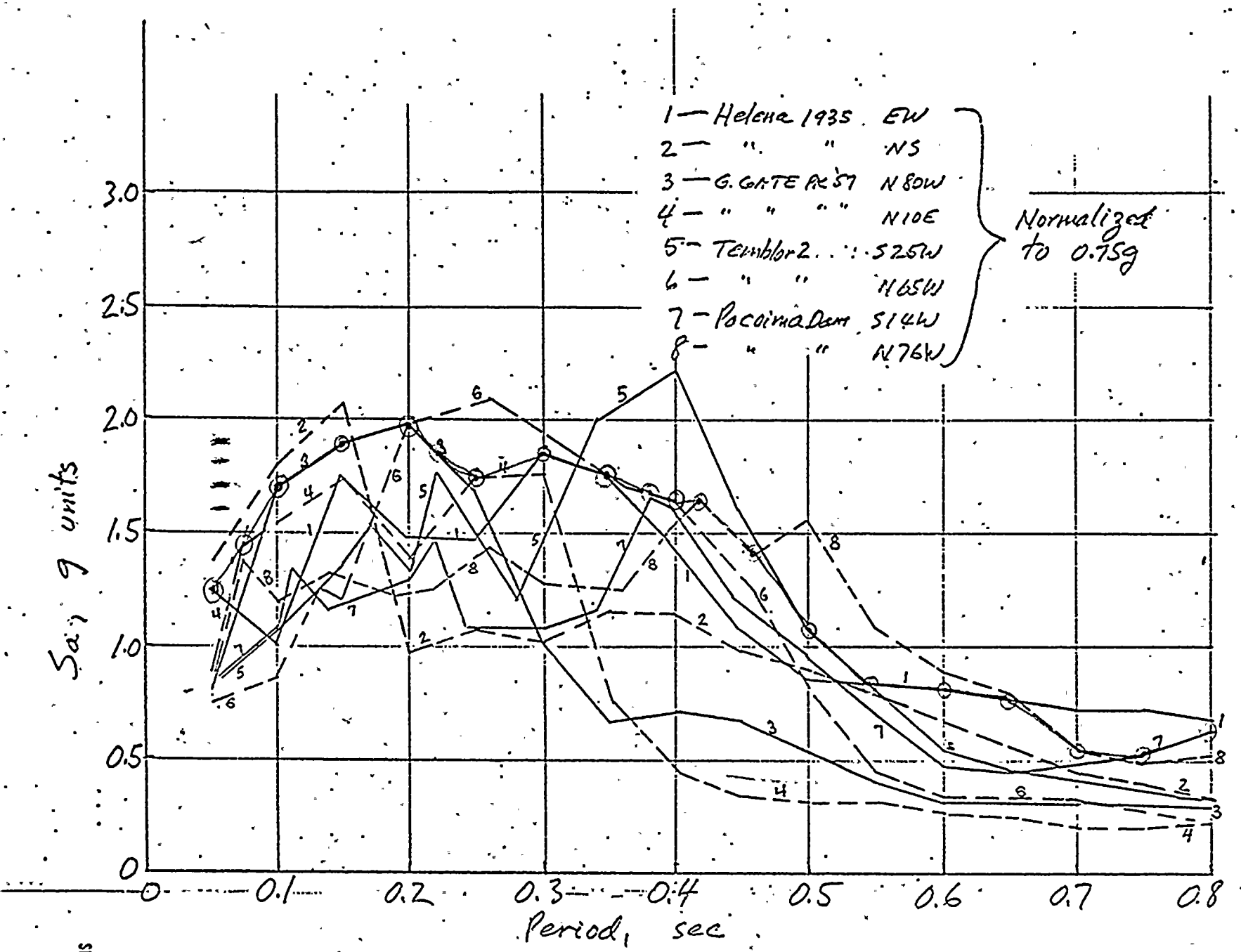


FIG. M — 7% DAMPED SPECTRA, ELASTIC, TAU = 0

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JOB NO. | CLIENT | JOB | SUBJECT | BY | CHK'D | DATE | DATE | SHEET NO.

4. Question) What was the shear wave velocity $V_s$ used in calculating Tau? Provide the basis for the use of this shear wave velocity. What is the difference between t and Tau?

Reply) The determination of the Tau values was based upon a study of the dimensions of the different structures of the plant and assumption for the shear wave velocity. The shear wave velocity determined for low strain values averaged about 5,000 feet per second for the materials underlying the foundations. However, it is expected that under much greater strains from a severe earthquake the shear wave velocities would tend to decrease. Thus the 5,000 fps value is considered an upper bound.

The following tabulation illustrates the range of parameters considered and the final selection of Tau values by weighting and judgement.

| Structure | $\sqrt{ab}$ (ft) | $\sqrt{ab}/4000$ | $\sqrt{ab}/4500$ | Use for Tau |
|---|---|---|---|---|
| Containment | 148 (dia) | .037 | .033 | 0.04 |
| Auxiliary | 208 | .052 | .046 | 0.06 |
| Combined Containment & Auxiliary | 260 | .065 | .058 | |
| Turbine | 321 | .080 | .071 | 0.08 |
| Intake | 158 | .040 | .035 | 0.04 |

The last part of the question refers to the differences between t and Tau. I believe this was prompted by the t in Table 1, page 3.7A-7, dated July 1976. The t in that table is a typographical error and should be Tau.

DRAFT

JRS 8/3/76

5. Question) Provide the basis for reducing the response spectra because of embedment and wave scattering phenomenon (Ref. 4). Results obtained are different whether the larger or smaller foundation dimension is used in calculations using Ref. 4 (Wong and Trifunac). State what dimension was used in calculations and provide basis. Also provide a typical set of calculations. The statistical treatments, smoothing and adjustments mentioned in the first paragraph of page 3 should be discussed in some detail with a clear description, justification and evidence supporting each procedure.

Reply) Reference to the Wong and Trifunac paper was made in support of the general procedures used in reducing response spectra because of the so-called Tau effect. The Wong and Trifunac data indicated for their basic assumptions that the Tau effects as determined by Yamahara and Scanlan would be conservative for a structure with considerable embedment. However, in the final determination of the Hosgri response spectra very little credit was given to the Wong and Trifunac type phenomena and then only in a judgemental sense in rounding the very high frequency accelerations to the values of zero period acceleration shown in the response spectra, Figures 1 to 13 inclusive. In other words, the reductions taken were largely those due to the Tau effect. Thus it is not feasible to provide determinations and calculations for the minor adjustments made solely at the extreme high frequency end of the spectra.

Further discussion of the procedures followed will appear in replies to subsequent questions.

DRAFT
RB 8/3/76

6. Question) Provide justification for the statement that "torsion represents a relatively small contribution to the total seismic load".

Reply) Figures T1, T2, and T3, attached, show absolute tangential acceleration for various heights and radial distances in the Exterior Containment Structure, the Interior Containment Structure, and the Auxiliary Structure, respectively. These plots are based on the submitted Appendix 3.7A and the supplementary data provided under Question No. 15. The circles in these plots represent cases of q and z (height, q, above base and radius, z) that will be considered herein as extreme situations. Most qz pairs will be much less critical.

Table 6.1 shows the development of translational acceleration, $S_a$, from the spectra, and the tangential acceleration from the curves of T1, T2, and T3.

TABLE 6.1 - TANGENTIAL ACCELERATIONS

| Structure | $T_1$ (sec) | From Hosgri curves 7% $S_a$ (g) | $T_{T_1}$ (sec) | Assumed z (ft) | From Figs. T1, T2, T3 $\alpha_o$ (g) | q (ft) | $\alpha_{qz}$ (g) |
|---|---|---|---|---|---|---|---|
| Containment Exterior | 0.20 | 1.55 | 0.10 | 70 | 0.23 | 150 | 0.537 |
| Containment Interior | 0.067 | 0.90 | 0.035 | 70± | 0.23 | 50 | 0.447 |
| Auxiliary, concrete | 0.096 | 1.00 | 0.079 | 110 | 0.30 | 80 | 0.950 |

Table 6.2 shows the relative impact (last column) of the wave induced torsion when the two accelerations are compounded by SRSS. The tangential acceleration will be resolved into vectors parallel with the translational motion under consideration. For this purpose (only) an average (45 degree) alignment is considered; hence 0.707 times the tangential acceleration is used in the table. The last column in Table 6.2 shows the relative impact, or the ratio of the SRSS combination to the translational acceleration. These ratios are not much greater than

unity except for the Auxiliary structure and that is for an extreme qz situation and subject to reduction as noted.

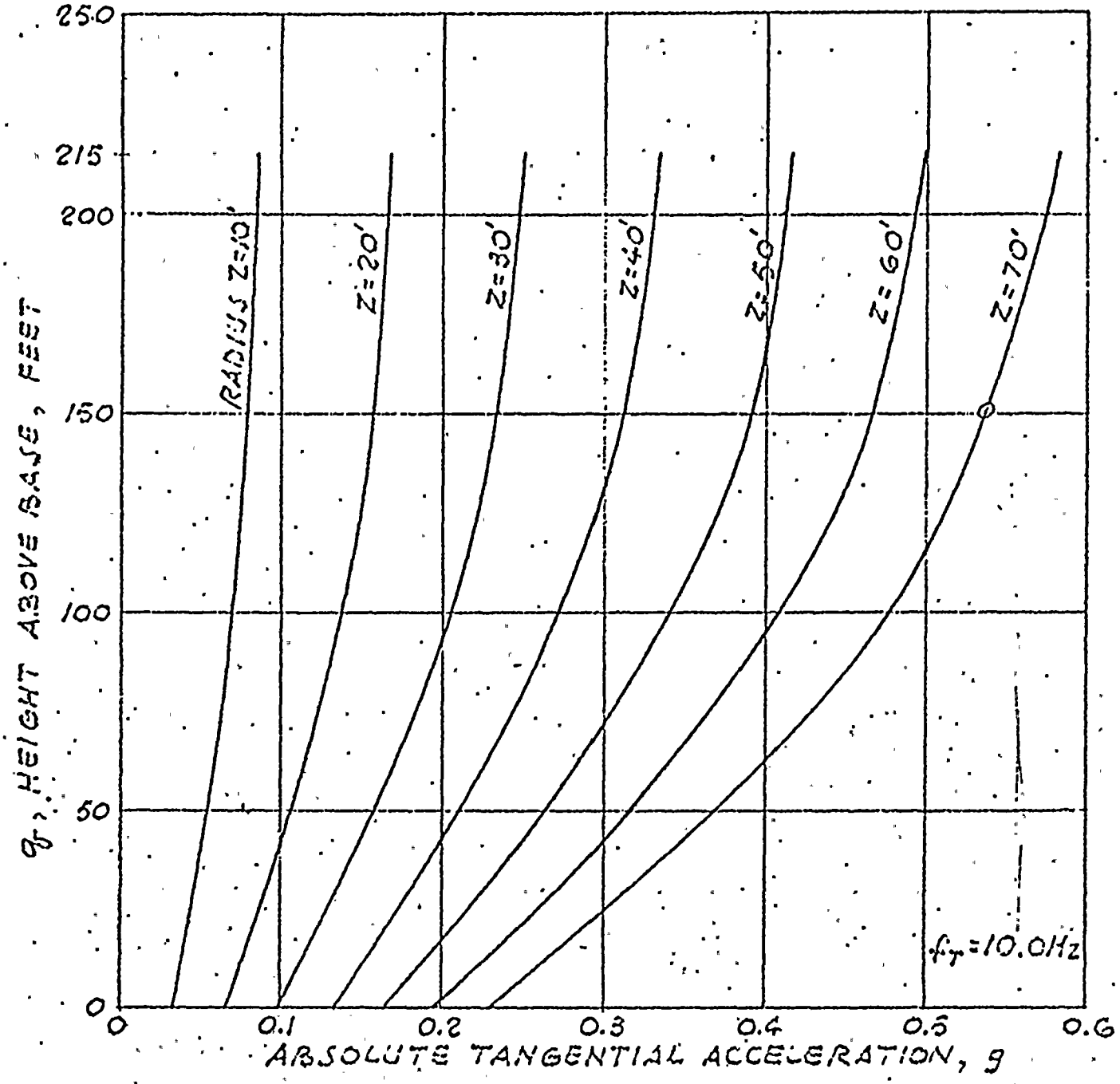
In general, wave induced torsion will have a small effect on the total seismic load.

TABLE 6.2 - RELATIVE IMPACT OF WAVE TORSION

| Structure | z (ft) | q (ft) | 7% $S_a$ (g) | $\lambda = 0.707\alpha_{qz}$ (g) | $\sqrt{S_a^2 + \lambda^2}$ (g) | $\frac{\sqrt{S_a^2 + \lambda^2}}{S_a}$ |
|---|---|---|---|---|---|---|
| Containment Exterior | 70 | 150 | 1.55 | 0.376 | 1.59 | 1.03 |
| Containment Interior | 70 | 50 | 0.90 | 0.313 | 0.953 | 1.06 |
| Auxiliary, concrete* | 110 | 80 | 1.00 | 0.665 | 1.20 | 1.20 |

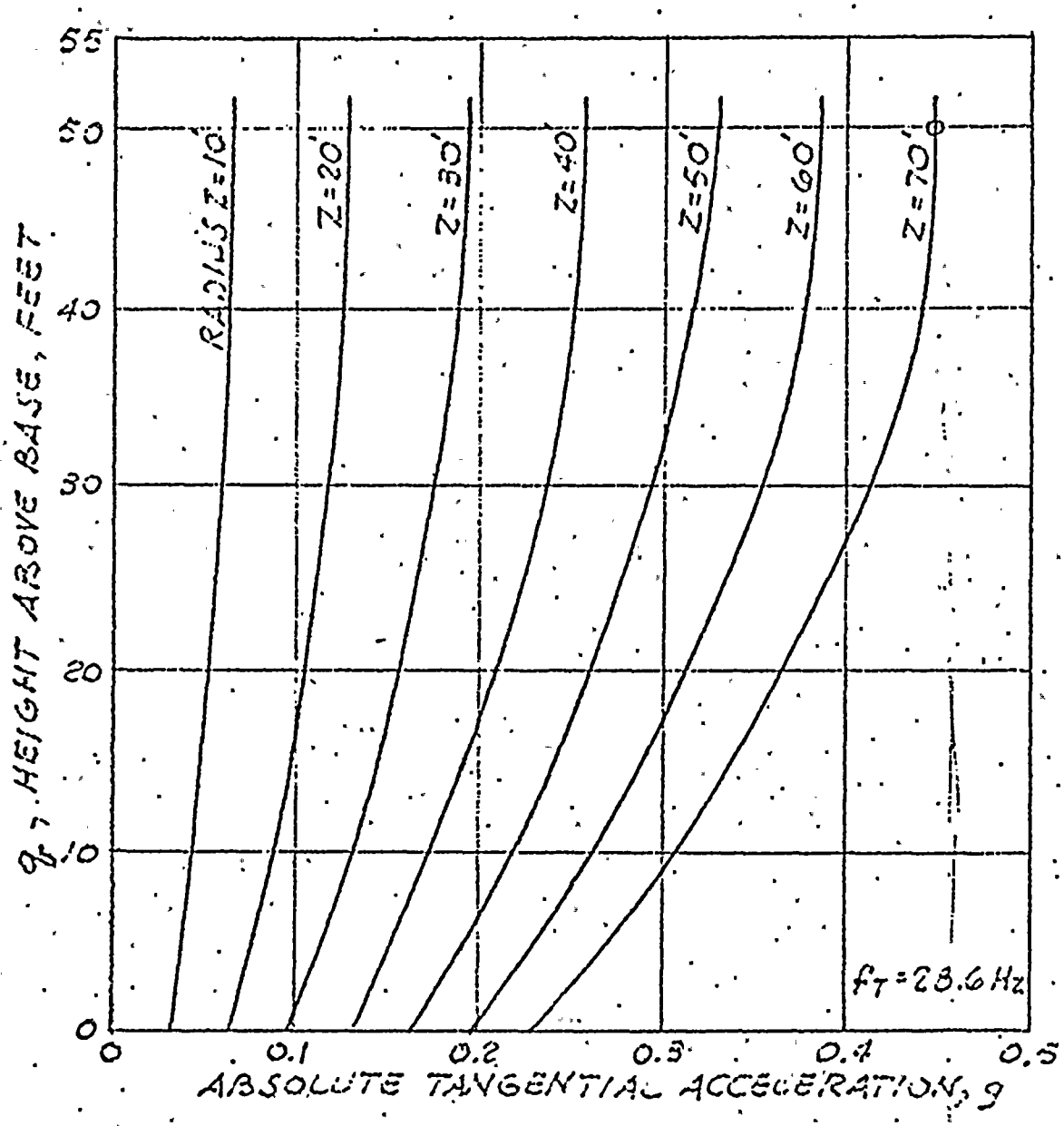
*Note: There is "geometric" torsion in this structure, not included here, but which would decrease the relative impact of wave torsion. It will be included in the actual analysis.

DRAFT
JAB 8/5/76



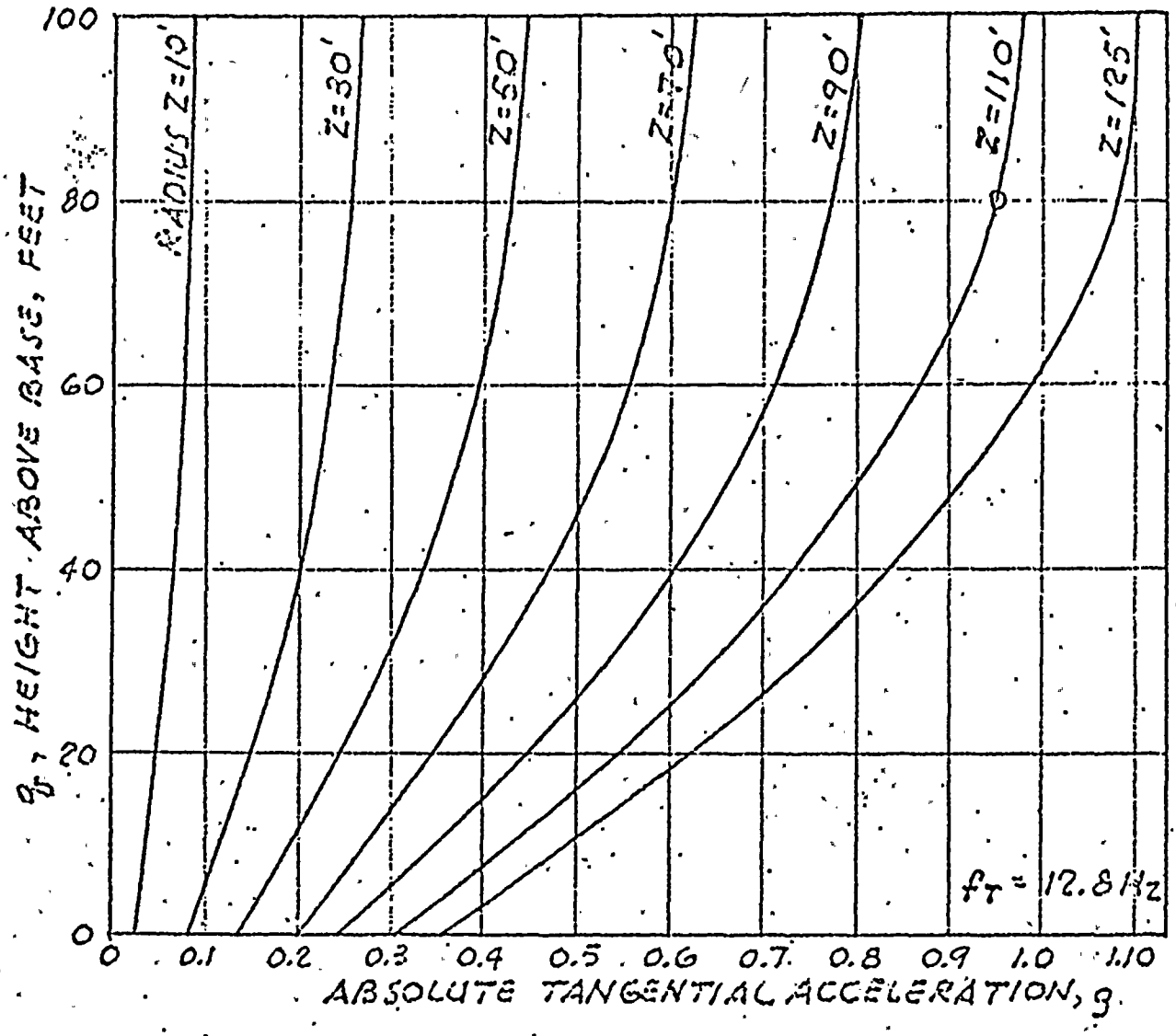

EXTERIOR CONTAINMENT STRUCTURE TORSIONAL MOTION

FIG. T1




INTERIOR CONTAINMENT STRUCTURE TORSIONAL MOTION

FIG. T2




AUXILIARY STRUCTURE TORSIONAL MOTION

FIG. T3

7. Question) Provide the basis for the use of nTau for torsionally induced tangential acceleration at the extremities of the foundation.

Reply) Two of the earthquake records from the 8 listed on page 3.7A-4, namely Temblor N65W and Pacoima S16E, were used to compute the tangential accelerations at the extremities of the foundation of the Containment Structure. These computations were made with the actual time histories of motion normalized to 0.75g, and the dimensions of the structure. Allowances were then made for a possible increase from the values calculated because of statistical variations. The tangential acceleration and Tau were known for the Containment Structure, so its n-value was computed and rounded to 6. The values of n for the other structures were determined in proportion (to n = 6 for the Containment Structure) in general accordance with the aspect ratios provided by Newmark* and the dimensions of the other foundations.

DRAFT
DAS 8/3/76

*N. M. Newmark, "Torsion in Symmetrical Buildings," 4WCEE

8. Question) Provide the basis for the formula for $T_o$ on page 5. Provide the basis for various values of n used in calculating $T_o$.

Reply) The formula for $T_o$ was determined from the mass and the tangential acceleration based upon nTau as described under 7 above and in the Appendix 3.7A, in accordance with Newton's second law, together with the assumption that the diameter of each foundation would be taken as $\sqrt{ab}$.

A more elegant formula for $T_o$ was proposed subsequently as a substitute for the one shown on page 3.7A-7. (However there would be little if any difference in the numerical results.) The formula now proposed is

$$T_o = 2\, I_W\, \alpha_o / \sqrt{ab}$$

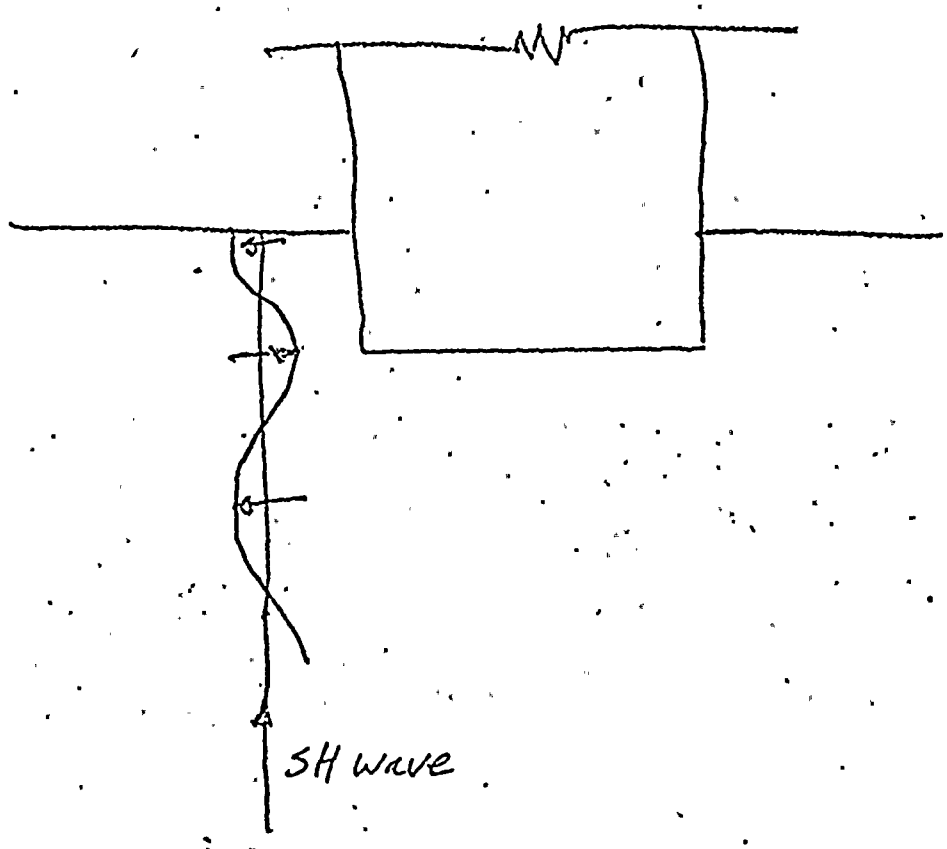
which is derived directly from the classical formula for torque, the mass polar moment of inertia times the angular acceleration. The symbol $I_W$ is the polar moment of inertia of all the weight above the foundation, $\frac{\text{KIP-ft}^2}{g}$.

DRAFT
JCB 8/3/76

9. Question) Provide justification for the statement that "tilting from some waves is only possible where the foundations are relatively deep."

Reply) This statement applies to SH waves. In the absence of embedment, there is no tilting due to SH excitation. In the case of an embedded foundation, SH waves can cause tilting. This effect is greatest at vertical incidence, and for frequencies corresponding to half-wavelengths which are comparable with the depth of embedment.

For the Containment Structure, with maximum embedment of about 40 ft., this characteristic frequency is about 50 hz., taking the shear velocity to be 4,000 ft./sec. The shallower the embedment, the higher this frequency will be.




DRAFT
8/3/76

10. Question) Provide the material properties and allowable stresses to be used in analysis or design for Diablo Canyon plant.

Reply) The material properties and allowable stresses to be used are shown in the attached.

[Note: The material provided by URS/Blume but deleted from the 3.7A draft is appropriate here.]

DRAFT
JAB 8/3/76

DRAFT 7/19/76
JAB

## Material Properties and Allowable Stresses

Allowable stress under severe earthquake conditions depends upon the actual properties of the materials in a structure. Although it is well known that such properties vary over a controlled range of values, and an individual member or connection must be designed for stresses below the average, in a whole structure there is available the spectrum of values, most or all of which are available in emergencies. It is therefore appropriate to utilize average actual material properties in analysis for severe seismic events rather than highly conservative and inappropriate design values intended for individual members; alternatively the problem can be treated probabilistically.[1]

Actual average strengths, as determined by tests, will be used in this analysis. In addition, the fact that concrete strength increases with time will also be considered up to an age of 2 years.* A third factor, the fact that allowable stresses for design purposes are generally determined by conservative evaluation of laboratory test results,[1] will not be used herein, or at all, unless found necessary to demonstrate the adequacy of the plant structures.

### Concrete

A variety of concrete mixes was used; all mixes utilized Type II, low alkali, moderate heat, Portland cement. Three basic mixes constitute most of the concrete. 3038 cylinder tests were performed on concrete, as summarized in Table 1 for all major structures for Units 1 and 2. Table 1 further summarizes the average strength and related statistical data.

Table 1

| Concrete mix | Specified $f'_c$ (psi) at days | Average test strength $f'_c$ (psi) | Average strength increase above design (%) | Standard deviation (psi) | Coefficient of variation (%) | No. of tests |
|---|---|---|---|---|---|---|
| A | 5000 @ 60 | 6340 | 27 | 412 | 6.5 | 511 |
| B | 3000 @ 28 | 3819 | 27 | 369 | 9.5 | 1698 |
| C | 5000 @ 28 | 5640 | 13 | 392 | 7.0 | 829 |
| | | | | | | 3038 |

*There is further increase beyond 2 years, at a declining rate.

In general, the strength of the concrete in the existing structures exceeds the specified design strength between 13% and 27%. Note also the low coefficients of variation, indicating excellent mix control and also that mean values can be used with confidence.

Concrete utilizing Type II Portland cement experiences a significant increase in strength with age (References 2, 3, 4, 5). Most of the strength increase above the specified strength is achieved at the end of two years. The following strength increases are assumed for the various concrete mixes at the end of two years after placement.

Table 2

| Concrete mix | Specified $f'_c$ (psi) @ days | Strength increase at 2 years, % |
|---|---|---|
| A | 5000 @ 60 | 35 |
| B | 3000 @ 28 | 50 |
| C | 5000 @ 28 | 50 |

The concrete strengths and their related moduli of elasticity, as summarized in Table 3 will be used in analysis. The design strength is increased above the original specified strength to account for (1) the higher actual average strength of the in-place concrete, and (2) the increase of concrete strength due to age since testing. The results are summarized for the different major structures.

Table 3

| Structure | Concrete mix | Average $f'_c$ test value (psi) | Age factor increase | Analysis $f''_c$ (psi) | $E_c$* (psi x $10^{-6}$) |
|---|---|---|---|---|---|
| Containment | | | | | |
| Exterior | B | 3850 | 1.50 | 5760 | 4.32 |
| Interior | A | 6330 | 1.35 | 8550 | 5.27 |
| Auxiliary Building | | | | | |
| Above El. 85' | B | 3920 | 1.50 | 5880 | 4.37 |
| Below El. 85' | C | 5650 | 1.50 | 8480 | 5.25 |
| Turbine Building | | | | | |
| Shear Walls @ Col. lines 5 & 17 | C | 5540 | 1.50 | 8310 | 5.20 |
| Intake Structure | B | 3630 | 1.50 | 5440 | 4.21 |
| " " | C | 5490 | 1.50 | 8230 | 5.17 |

*$E_c = 57{,}000 \sqrt{f''_c}$ per ACI 318-71, in psi.

## Steel

The average reinforcing and structural steel yield strengths as summarized in Table 4 will be used in the analysis. The results are summarized for the major structures and are based on 100% sampling of the containment reinforcing steel, 80% sampling of the other reinforcing steel, and approximately 90% sampling of the structural steel.

Table 4

| Structure | Steel grade | Average yield strength by test (psi) | Average ultimate strength by test (psi) |
|---|---|---|---|
| | | Reinforcing Steel | |
| Containment | | | |
| Exterior # 18's | 60 | 66,780 | 105,980 |
| Interior # 11's | 60 | 68,720 | 105,980 |
| Auxiliary Building | 60 | 66,170 | 102,840 |
| | 40 | 51,800 | 80,000 |
| Turbine Building | 60 | 65,940 | 101,350 |
| | 40 | 51,390 | 80,550 |
| Intake Structure | 40 | 51,430 | 78,940 |
| | | Structural Steel | |
| All Structures | A36 | 43,950 | 68,040 |
| | A441 | 51,620 | 75,910 |
| | A516 | 51,040 | 79,170 |
| | A572 | 54,160 | 78,650 |

It is to be noted that in flexure of a reinforced concrete section, an increase in concrete strength and modulus of elasticity with no increase in reinforcing steel strength causes some reduction in reinforcing bar stress for the same loading conditions. Thus there is some increase in the value of the overall section even without an increase in the strength of the reinforcing steel.

## References


1. Blume, John A., "Allowable Stresses and Earthquake Performance," Sixth World Conference on Earthquake Engineering, New Delhi, India, 1977.

2. Troxell, G. A., and H. E. Davis, "Composition and Properties of Concrete," McGraw-Hill, New York, 1956.

3. U. S. Bureau of Reclamation, Concrete Manual, 1963.

4. Kee, C. F., "Relation Between Strength and Maturity of Concrete," Title No. 68-21, ACI Journal, March 1971.

5. Washa, G. W., and K. F. Wendt, "Fifty Year Properties of Concrete," Title No. 71-4, ACI Journal, January 1975.

11. Question) Provide justification for the use of ductility factor of 1.3 for Category I structures other than containment and for Category I equipment or its supports.

Reply) It is felt that the ductility factor of 1.3 is a conservative value for such an extreme earthquake as the 7.5 magnitude on the Hosgri fault. An indication of the minor distress at low ductilities may be seen in pages 126 and 127 of the book "Design of Multi-Story Reinforced Concrete Buildings" by Blume, Newmark and Corning. The minor cracks shown are for the condition with load fully applied. Most of these cracks would almost disappear from view upon release of the load at such low ductilities.

For additional conservatism with structures functioning in containment, a ductility factor of 1.2 is proposed.

DRAFT
JB 8/3/76

13. Question) The table on page 3 should be discussed in detail. How was the "Effective length of foundation" or perhaps more appropriately the "Effective Shear Velocity" determined. Support this with shear velocity field data. The discussion should be complete enough that the reviewer can proceed from field velocity data, dimension of structures, and a rationale for the "Tau Effect" to reproduce the table and defend its adequacy.

Reply) Shear wave measurements are discussed in Appendix 2.5A and shown on pages 2.5A-14, 2.5A-15, 2.5A-16, 2.5A-50, 2.5A-51, 2.5A-52, 2.5A-53, 2.5A-54, 2.5A-55, 2.5A-56, 2.5A-57 and 2.5A-58.

As indicated in the table on page 3, the characteristic dimension of a foundation is taken to be the square root of its area, except for the containment structures, in which case the diameter is used. The rationale for computing the characteristic dimension in this way is that elastic waves from the postulated earthquake on the Hosgri fault would impinge upon the site over a wide range of azimuths: almost 180 degrees.

The parameter Tau is calculated as the transit time across the foundation of a horizontally propagating shear wave: i.e., Tau is the characteristic dimension divided by the shear wave velocity. Values of the shear wave velocity are expected to vary with strain. In general they were taken in the 4000 to 4500 fps range as shown for Question 4.

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JaB 8/3/76

14. Question) Provide justification for the differences in response spectra obtained as compared to the response spectra recommended by Dr. Newmark in his report to NRC. The differences appear to be large around 2 cps and between 10 to 30 cps (especially in Fig. 13).

Reply) The differences between Dr. Newmark's curves and the curves submitted are not great considering the fact that there were completely different sets of data and different approaches used by Dr. Newmark and Dr. Blume who used 8 close-in rocky-site records, with adjustment for magnitude scaling which are considered to be the best available models for the conditions at Diablo Canyon. Dr. Newmark's data, although including more records, generally includes alluvial sites and greater epicentral distances.

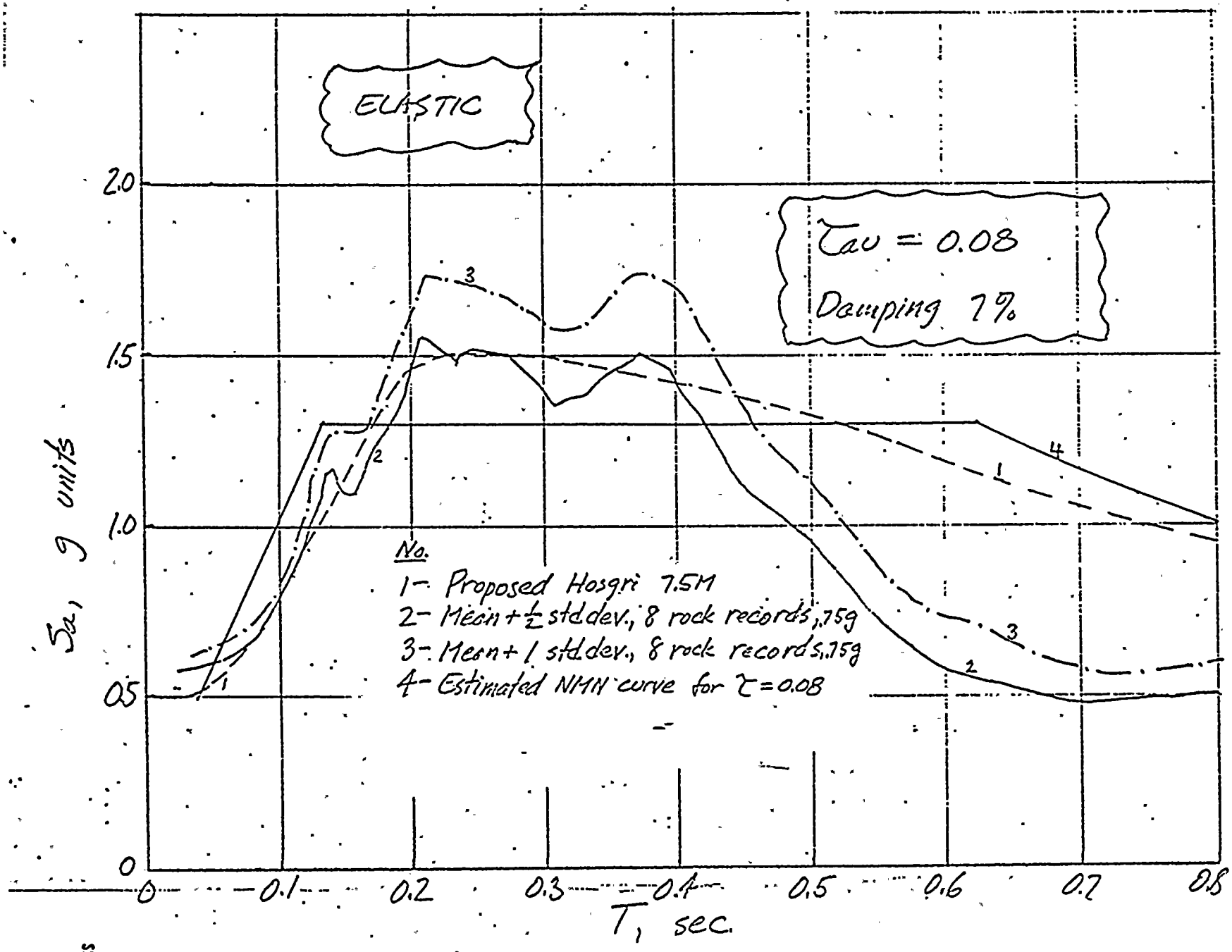
There are scaling differences between Dr. Newmark's Table 1 and his curves. For that reason we have used his Table 1 numerical data for comparison. In Figure N we have plotted an interpolated "NMN" curve for Tau = 0.08 taken at 1.3g which is between his values for Tau of 0.04 and 0.12. There seems to be no difference at 2 cps (0.5 sec period) and rather minor differences at longer periods. We don't think Dr. Newmark would insist upon his "corner" points which develop from straight line plotting convenience.

We exceed Dr. Newmark between 0.175 and 0.50 sec period, because that is the way our data fell.

At the high frequency end there are some differences, some of which may be due to our interpretation of his plots. In any event, we have generally followed our data and feel the results are adequate for the problem at hand.

Results would be similar for other Tau values.

DRAFT
RB 8/5/76

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JOB NO. | CLIENT | JOB DILBLO CANYON | SUBJECT HOSGRI 7.5M | BY | CHK'D | DATE | DATE: 6/4/75

FIG. N — COMPARISON OF SPECTRAL CURVES, τ = 0.08, Dpg 7%

SHEET NO.

17. Question) Figures 1 through 13 should be discussed in detail. Which of the references cited were used in calculating the response spectra? Provide typical calculations.

Reply) In developing the response spectra, the procedure was as follows:

1. Select reliable, strong, close-in, rocky site records, corrected.
2. Normalize to 0.75g.
3. Filter for Tau effects and make response spectra for 7% damping.
4. Conduct statistical analysis over the spectrum range.
5. ~~Select a suitable probability of being exceeded.~~
6. Make smoothed, 7%-damped, spectral curves for each Tau value of interest.
7. Adjust slightly at the very high frequency end for embedment effects not otherwise considered.
8. Adjust beyond 0.4 sec period for magnitude scaling.
9. Compare with other data and make minor local adjustments.
10. Develop curves for other than 7% damping by reference to statistical studies made for NRC (AEC). This produced the following ratios of spectral values.

Ratio of $S_{a_i}$ to 7%-damped $S_a$

| Damping, i (%) | Period (sec) | | | | | | |
|---|---|---|---|---|---|---|---|
| | .033 | .115 | .175 | .25 | .35 | .60 | .80 |
| 2 | 1 | 1.481 | 1.484 | 1.526 | 1.543 | 1.513 | 1.492 |
| 3 | 1 | 1.328 | 1.338 | 1.366 | 1.391 | 1.373 | 1.363 |
| 4 | 1 | 1.205 | 1.210 | 1.262 | 1.270 | 1.258 | 1.268 |
| 5 | 1 | 1.136 | 1.146 | 1.169 | 1.174 | 1.178 | 1.175 |

The significant figures shown are for relative consistency and do not reflect that level of accuracy. They allow not only for the damping but for the probability level.

The references cited were reviewed and, in some cases, checked. In the final analysis, we used our own procedures but we first found concurrences with the references, or the reasons for nonconcurrence. We did not use assumptions of harmonic motion but actual time histories, and the actual ground motion frequencies as related to the foundation dimensions. In short, none of the references was followed explicitly. Typical calculations will be provided.

DRAFT

DCB 8/5/76

15. Question) Provide detailed explanation of Fig. 14 regarding containment structure torsional motion and its use in analysis.

Reply) The tangential acceleration applied to the foundation causes torsional twist of the superstructure and amplification due to inertial effects. The torsional acceleration increases with height above the base and with the radius from the center of rotation. It is zero at the center of rotation at any height in the structure. The following formula was derived under the assumption that the torque varies from $T_o$ at the base to zero at the top.

$$\phi_q = \frac{T_o q}{GJ}\left[1 - \frac{q}{2h}\right], \text{ radians}$$

wherein $\phi_q$ = the twist at any level q above the base, radians

$G$ = the shear modulus, kip $ft^{-2}$

$J$ = the polar moment of inertia, $ft^4$

$h$ = the total height, feet

The angular acceleration will be obtained from $\phi_q$ assuming harmonic motion in the fundamental torsional mode with frequency $f_T$, hertz

$$\ddot{\phi}_q = 4\pi^2 f_T^2 \phi_q, \text{ rad sec}^{-2}$$

The absolute tangential acceleration due to torsion at any level q and any radius z from the center of rotation will be $\alpha_{qz}$, as follows

$$\alpha_{qz} = \frac{z}{g}(\ddot{\phi}_q + \ddot{\phi}_o), \text{ g units}$$

$$\ddot{\phi}_o = 64.4\, \alpha_o/\sqrt{ab}, \text{ rad sec}^{-2}$$

Plots are being developed for all the structures subjected to torsion.

DRAFT
DZS 8/3/76

16. Question) Provide justification for the following assumptions made in Ref. 4 (by Wong and Trifunac) and the paper's applicability to Diablo Canyon plant structures.

(i) Only SH waves are treated
(ii) Wave front is planar
(iii) Earthquake motions are harmonic
(iv) Structure (shear wall) is infinitely long
(v) Foundation is infinitely long
(vi) Foundation is infinitely rigid

Reply) In view of the fact that the Wong and Trifunac material was referenced as complementary information and was not used formally or to any significant degree in the development of curves the detailed questions are really not applicable.

DRAFT
JLS 8/3/76

ENCLOSURE NO. 4

SEISMIC CAPABILITY OF FUEL STORAGE FACILITIES
Additional Information for Special Nuclear Material
License Application - Docket No. 50-323-OL
Diablo Canyon Site, Unit 2

I. Spent Fuel Storage Pool and New Fuel Storage Area

The seismic response of the spent fuel storage pool and new fuel storage area to the postulated 7.5 M event on the Hosgri fault with 7% critical damping, including allowance for the appropriate torsional effect, is less than that produced by the DDE with 5% critical damping. It is therefore concluded that the Hosgri earthquake is not critical for the Spent Fuel Storage Pool and the New Fuel Storage area.

2. New and Spent Fuel Racks

The new and spent fuel racks have a $T < 0.03$ seconds and therefore are rigid with respect to the floor on which they are supported. Accordingly, they will be subjected to the peak accelerations of the floor. The maximum floor accelerations, including torsional effect, for the postulated Hosgri 7.5 M event with 7% of critical damping are less than for the DDE with 5% critical damping. It is therefore concluded that the DDE, which was used in the original design, governs.

Additional Information:

We have reviewed the capability of the spent fuel storage pool and the storage racks to withstand seismic events. Results are shown below:

| | Element | Capability | |
|---|---|---|---|
| 1. | Spent Fuel Pool (concrete) | 2.00g | * |
| 2. | Spent Fuel Racks | 1.00g | * |
| 3. | Spent Fuel Racks Anchor Bolts | 0.84g + | * |
| 4. | New Fuel Racks | 1.20g + | ** |
| 5. | New Fuel Racks Anchorage | 1.50g | ** |

* Floor acceleration at El. 100'

** Floor acceleration at El. 125' - 8"

MEETING SUMMARY

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Local PDR
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IE (3)
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F. Williams
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