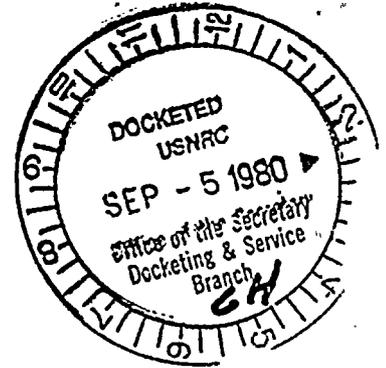UNITED STATES OF AMERICA
NUCLEAR REGULATORY COMMISSION

ATOMIC SAFETY AND LICENSING APPEAL BOARD

Richard S. Salzman, Chairman
Dr. John H. Buck
Dr. W. Reed Johnson

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                                )
In the Matter of                )
                                )
PACIFIC GAS AND ELECTRIC COMPANY )     Docket Nos. 50-275 OL
                                )                   50-323 OL
(Diablo Canyon Nuclear Power    )
 Plant, Units 1 and 2)          )
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4 Sept 80

TESTIMONY OF DR. ENRIQUE LUCO

The attached testimony of Dr. Luco is provided pursuant to
the Board's order of August 7, 1980 (ALAB-604), with the Board's
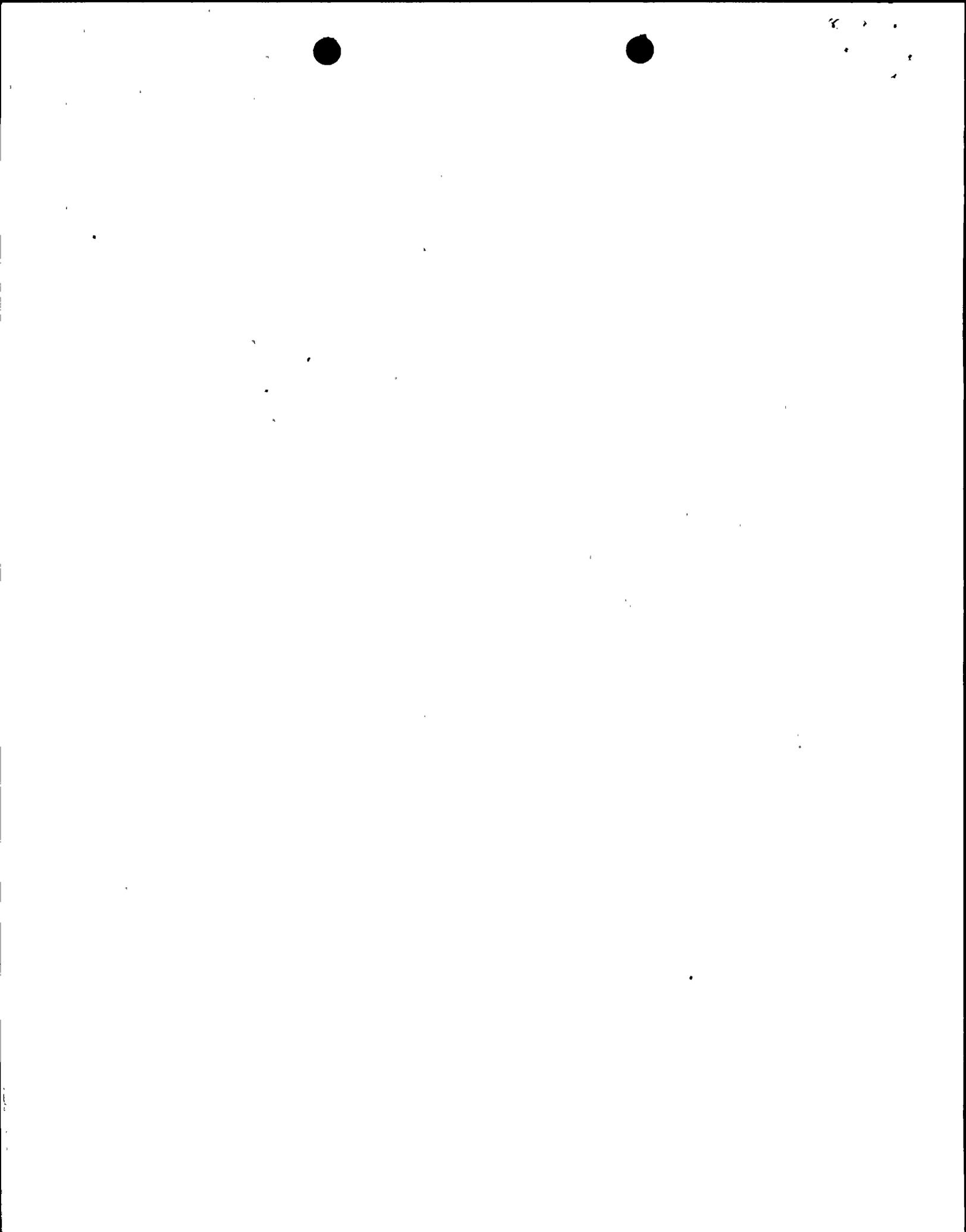permission for later filing.

Respectfully submitted,

Marjorie S. Nordlinger, Attorney
Counsel for Drs. Luco and Trifunac

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UNITED STATES OF AMERICA
NUCLEAR REGULATORY COMMISSION

ATOMIC SAFETY AND LICENSING APPEAL BOARD

In the Matter of

PACIFIC GAS AND ELECTRIC COMPANY
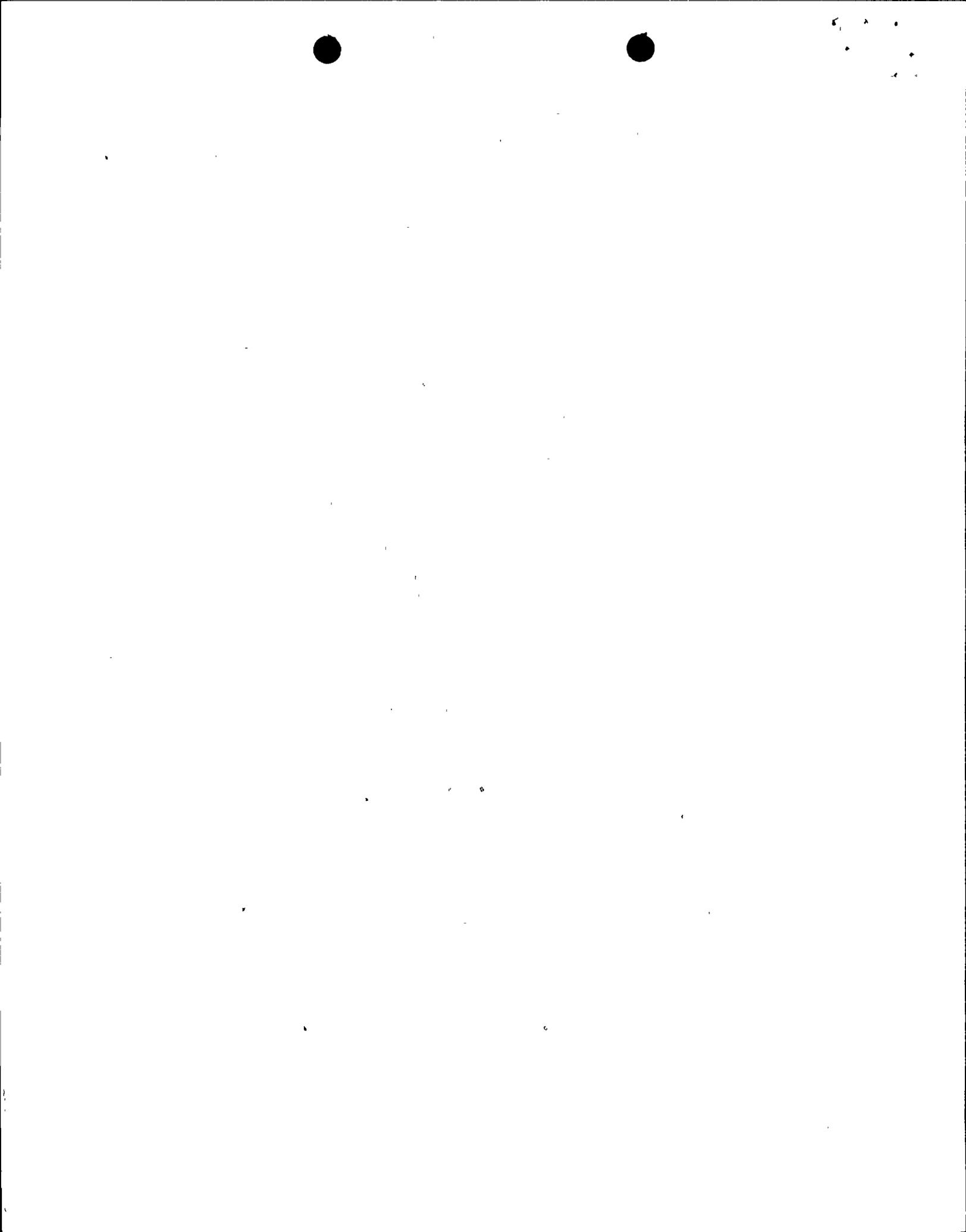
(Diablo Canyon Nuclear Power Plant, Units 1 and 2)

Docket Nos.   50-275
               50-323

Comments by

J. Enrique Luco

September 1980

In compliance with the order of the Atomic Safety and
Licensing Appeal Board (August 7, 1980, ALAB-640), I have
reviewed the testimony offered by the parties and have con-
sidered the questions raised by the Board. My comments to
the first six questions are included herewith.

J. Enrique Luco   9/4/80
J. Enrique Luco   (Date)

I.   The October 15, 1979, Imperial Valley Earth-
quake (IV-79, M_L=6.4-6.9) provided an extensive set of
strong motion records in the near field of a rather severe
earthquake. 33/ The parties should compare the horizontal
peak acceleration values recorded for various instrument
positions with earlier predictions and compilations of such
motion, e.g., those contained in the Final Safety Analysis
Report (FSAR) on the Diablo Canyon Nuclear Power Plant,
Amendment 50, Appendix D LL 11B, Figures 2, 3 and 4; and the
United States Geological Survey (USGS) Circular 795, Figures
4, 24, 47, and 48. Those comparisons should (if possible)
address whether there is magnitude independence or a
saturation effect for ground motion intensity in the near
field of earthquakes. 34/

33/ Preliminary Summary of the U.S. Geological Survey
     Strong-Motion Records from the October 15, 1979
     Imperial Valley Earthquake by R.L. Porcella and R.B.
     Mathiesen (October 1979), included in Board Notifi-
     cation, December 17, 1979.

34/ See, for instance, TR. 8597; 10,105; 5889-90.

COMMENTS IN RESPONSE TO QUESTION 1.

1. To consider the 1979 Imperial Valley earthquake data
in proper context it is necessary to establish first the mag-
nitude of the earthquake. The USGS has assigned a value of
6.9 to the surface wave magnitude M_S for this event (M_S is a
measure of the strength of the earthquake in the long period
band (20 sec) as recorded by stations at large epicentral dis-
tances (> 2000 km)). In a recent study, Kanamori and Regan
have determined the seismic moment for the IV79 event. Employ-
ing correlations between seismic moment and magnitude they have
found a value of $M_W = 6.5$ (M_W is a magnitude determined by ultra-
long period seismographs, which, in this range of values, is
equivalent to M_S) (Kanamori and Regan, Long-Period Surface Waves
Generated by the Imperial Valley Earthquake of 1979, preprint).
It would seem then that the surface wave magnitude for this
event is in the range $M_S = 6.5$ to $M_S = 6.9$.

The local magnitude reported by Pasadena is $M_L = 6.6$ (local
magnitude is based on the amplitude recorded by the Wood-Anderson
seismograph with a natural period of 0.8 sec and a damping of 80
percent of critical). Calculations of synthetic Wood-Anderson seismo-
graph responses based on the strong-motion accelerographs located
in Mexico lead to a value of $M_L = 6.3$ (Brune, Prince, Vernon,

Mena and Simons, Strong Ground Motion Recorded in Mexico During the Oct. 15, 1979 Main Shock, preprint). Finally, preliminary calculations of M_L on the basis of the strong-motion accelerograms recorded in U.S. stations lead to a value of $M_L \sim 6.2$ (Kanamori, personal communication). Thus, the local magnitude for the IV79 event appears to be in the range from $M_L = 6.2$ to $M_L = 6.6$. It should be said that local magnitude M_L is more relevant for engineering applications since it is determined closer to the period range of interest.

The relation of the IV79 event to other California earthquakes (and to the Guatemala earthquake) is shown in Fig. 1-1. This figure taken from the work of Kanamori (Bull. Seism. Soc. Am. 69, pp. 1645-1670, 1979) has been modified by superimposing the possible region of (M_L, M_S) values for the IV79 event. This figure indicates that given M_S there is a considerable range in the possible values of M_L.

For the Hosgri $M_S = 7.5$ event, the local magnitude M_L could vary from a value of about 6.6 to a value of 7.2. The implication of this is that given a magnitude $M_S = 7.5$ the peak response of a structure with a period of 0.8 sec (1.25 Hz) and 80 percent damping could vary by a factor of 4. For a structure with lower damping and shorter period (higher frequency) the margin of possibilities is larger still. This example underscores the uncertainties associated with prediction of high frequency components of motion. In the range of local magnitude from
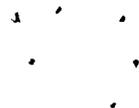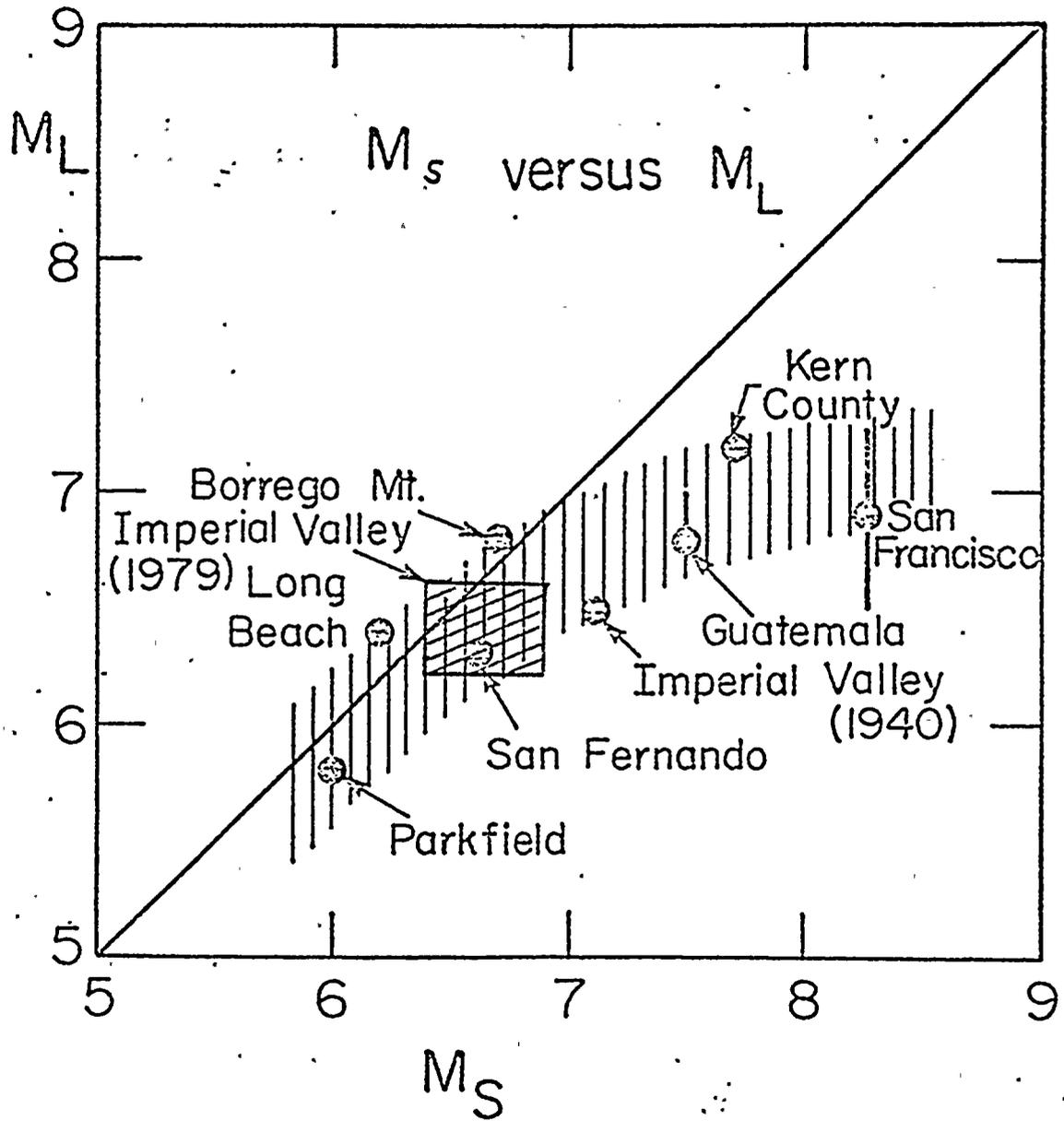
Fig. 1-1

(Luco)

$M_L = 6.6$ to $M_L = 7.2$ and for distances shorter than 10 km to the fault the situation is aggravated by the absolute lack of recorded data. The only records available which may fall within this group are those obtained during the Gazli, Russia $M_S \sim 7.0$ earthquake (peak horizontal accelerations 0.85g and 0.65g, peak vertical acceleration 1.3g, soft rock) and during the Tabas, Iran $M_S \sim 7.7$ earthquake (peak horizontal acceleration 0.8g, alluvium).. Unfortunately, the local magnitude for these events is not known.

2. Before proceeding with the comparison of the Imperial Valley 1979 data with the data in USGS Circular 795 and with various correlations, it is necessary to examine some of the characteristics of the distribution of peak accelerations recorded during the IV79 event. In general, the distribution of peak accelerations on the ground surface will exhibit a two-dimensional pattern that cannot be described by just one parameter such as distance to the fault, epicentral distance, hypocentral distance or distance to the center of energy release. To avoid, in part, this difficulty I have plotted in Fig. 1-2 the corrected peak accelerations versus distance to the fault in three separate regions. The first region corresponds to stations located in the vicinity of the southern half of the fault (Fig. 1-2a). The second region corresponds to stations located to the east of the northern end of the fault (Fig. 1-2b), and, the third region

Figure 1-2
(Luco)

1-6
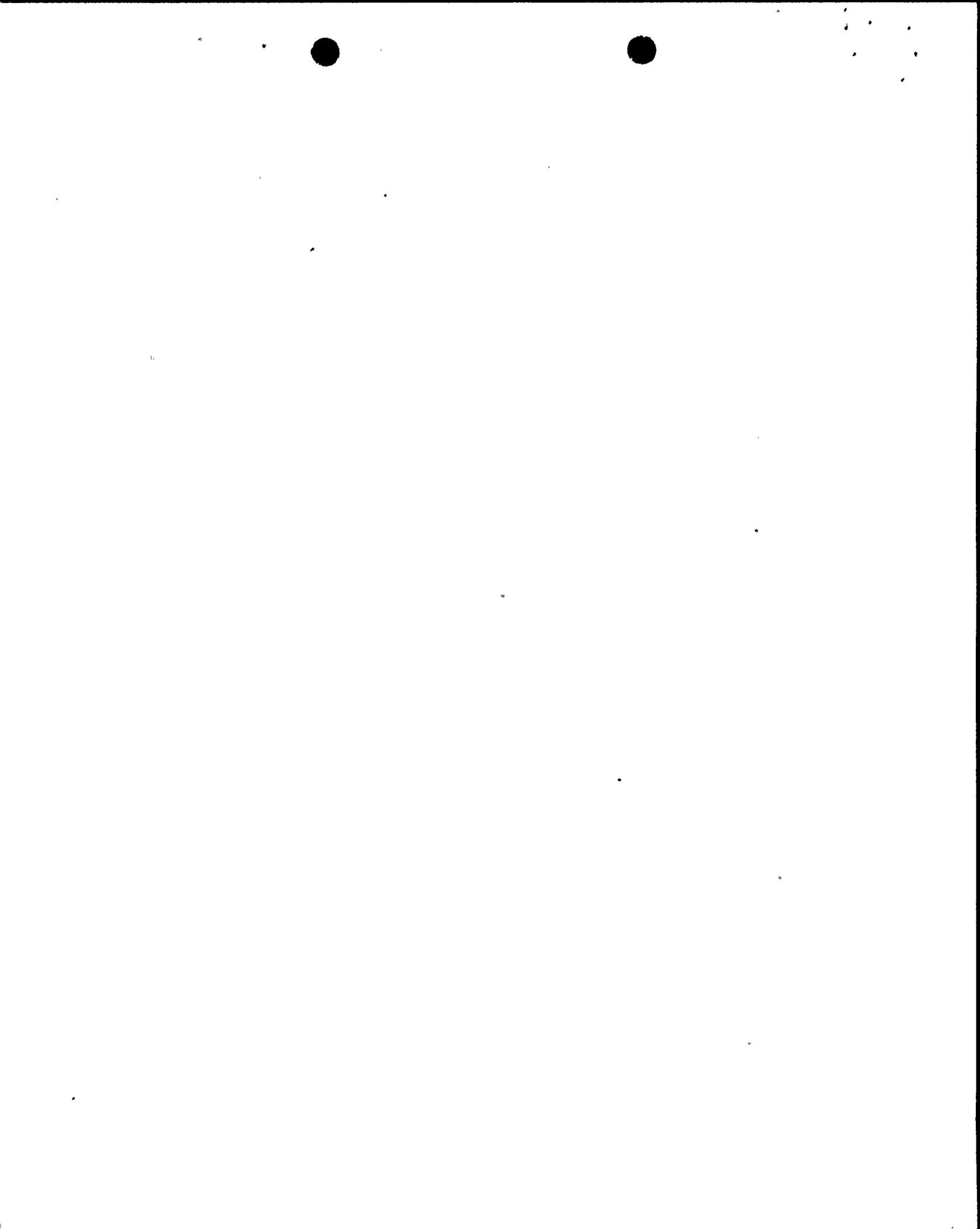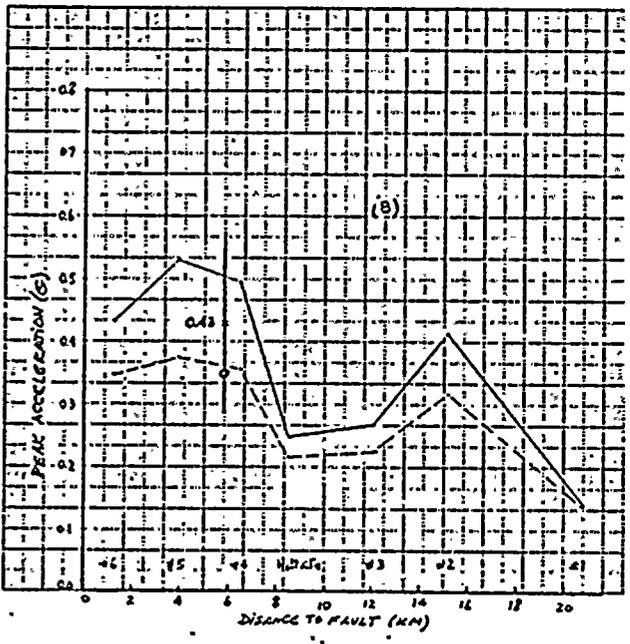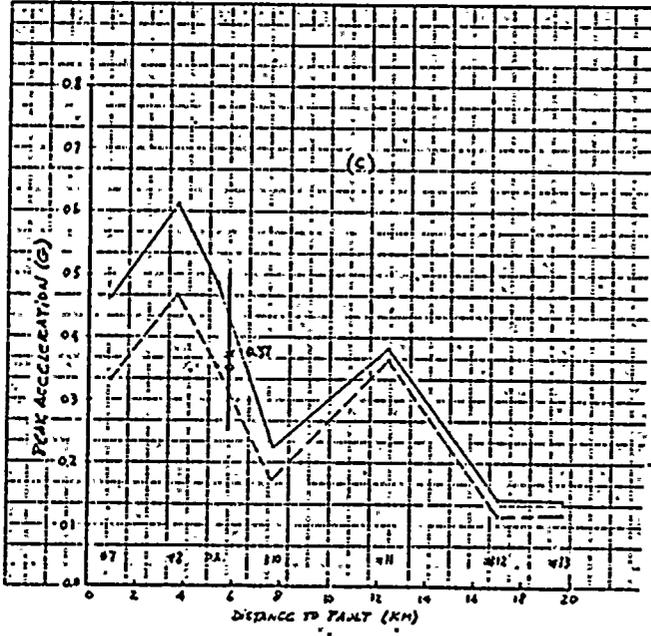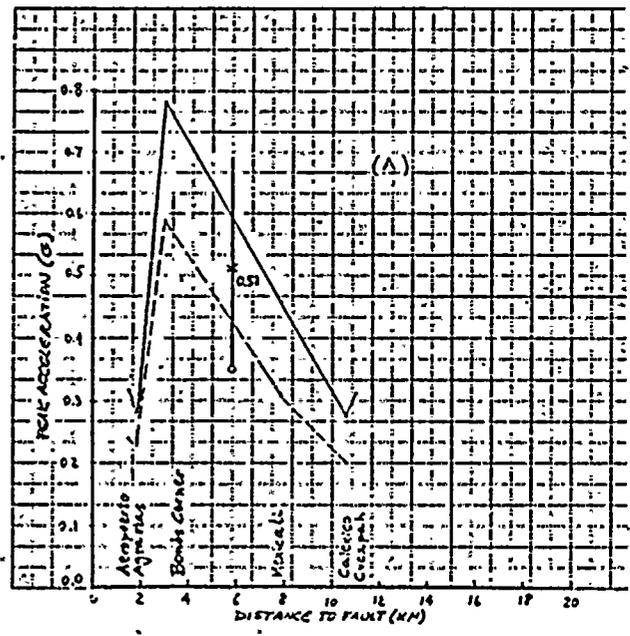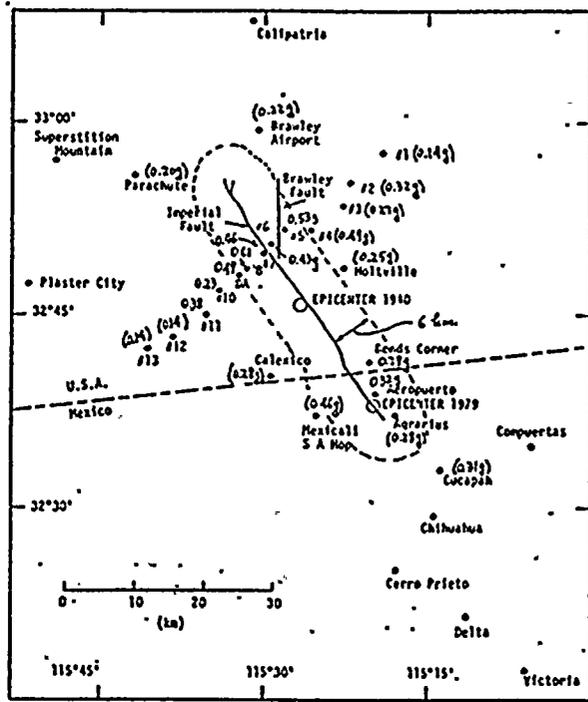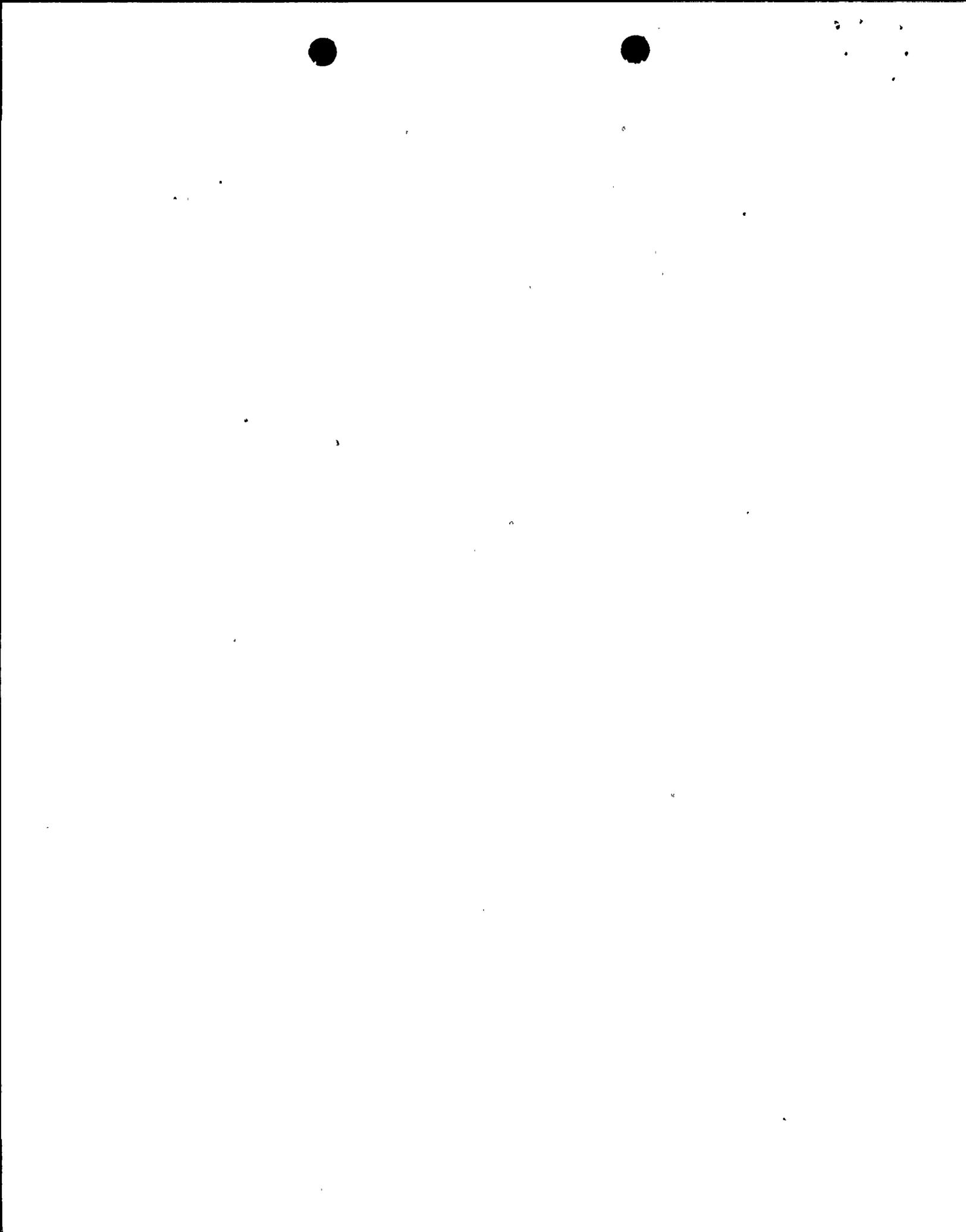
corresponds to stations to the west of the northern end of the fault. The maximum and minimum values of the peak accelerations recorded at different stations have been connected by lines to estimate trends of variation. In each graph a vertical line has been drawn at a distance of 5.8 km from the fault.

Inspection of Fig. 1-2 reveals some trends common to all three regions. In the first place, stations within 1-2 km from the fault recorded significantly lower peak accelerations than those 3-4 km away from the fault. Second, the largest peak accelerations developed at distances of 3-6 km from the fault. Third, at distances of the order of 8-11 km the distribution of peak accelerations shows a second zone of low values. Fourth, in the region from 12-16 km a second region of higher peak accelerations is observed. These trends must be associated with the radiation pattern from the fault, with the depth of the fault and with the characteristics of the local geology.

The mean accelerations at 5.8 km, interpolated from the closest stations, are 0.51g, 0.43g and 0.37g for the three zones considered (represented by crosses in the graphs). The global mean value at this distance is 0.44g. These values can be compared with the estimates of 0.36g and 0.34g for the mean peak IV79 acceleration at 5.8 km obtained by Blume (Affidavit) and Frazier (Affidavit). An average value of 0.35g is shown as an open circle in the graphs. Given the fluctuating trend of the

data, correlations which assume a smooth variation of peak acceleration with distance may not be adequate. In particular, the low values of peak accelerations in the distance ranges of 0-2 km and 8-11 km introduce a bias in the estimates.

The low peak acceleration values in the range from 0-2 km may be attributed to the following causes:

(i) The soil in the immediate vicinity of the fault has been altered by previous faulting. It is possible that material attenuation within this severely disturbed region is significantly higher than further away from the fault. This increased material attenuation may have locally filtered the high frequency components of horizontal motion.

(ii) Theory indicates that the horizontal motion at a point located on the soil surface exactly above a small buried strike-slip fault is exactly zero. The maximum response occurs at a horizontal distance of the order of the depth of the fault. These effects are illustrated in Fig. 1-3 where calculated quantities proportional to the amplitudes of the tangential ($\Sigma_{\theta x}$) and vertical (Σ_{zx}) components of motion on the ground surface are presented versus horizontal distance (r) to a small (point) strike-slip fault buried 5.5 km in a typical geologic structure (Luco and Sotiropoulos, 1980). In the case of an extended fault such as the Imperial Fault the situation is more complex. The previous discussion suggests, however, that the peak response may not necessarily occur immediately above the fault.
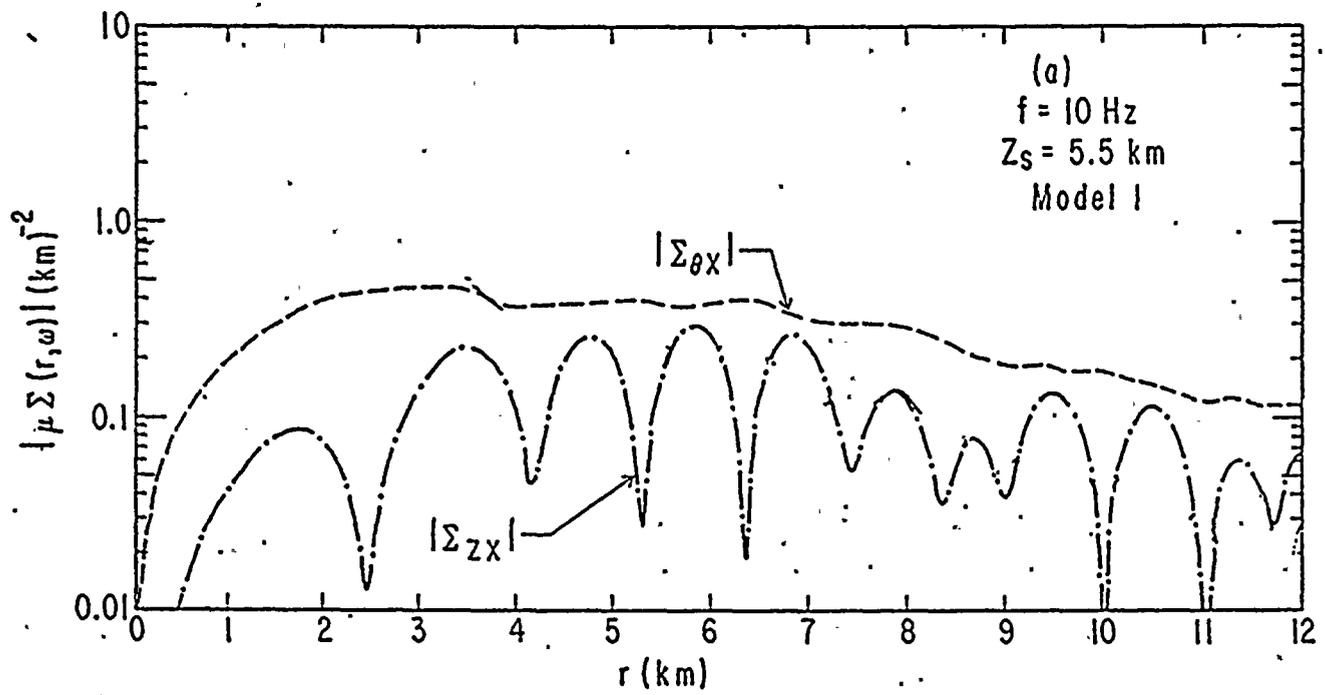
(a)
f = 10 Hz
Z_S = 5.5 km
Model 1

$|\Sigma_{\theta X}|$

$|\Sigma_{ZX}|$

Figure 1-3
(Luco)

3.   The peak horizontal accelerations recorded during the
1979 Imperial Valley earthquake superimposed on Fig. 24 of USGS
Circular 795 are shown in Fig.1-4 .   In this figure peak hori-
zontal accelerations plotted versus distance to the fault are
compared with the corresponding peak accelerations recorded
during the Point Mugu (M_L = 6.0), Managua (M_L = 6.2), Imperial
Valley 1940 (M_L = 6.4), Borrego Mountain (M_L = 6.4) and San
Fernando (M_L = 6.4) earthquakes.   Fig. 1-4 corresponds to Fig.
I-7 of Blume's testimony (P.G. and E Written Testimony), except
that to be consistent with the USGS Circular 795 criterium only
the largest peak acceleration at each station has been considered.

Inspection of Fig. 1-4 reveals that for distances to the
fault larger than 10 km, the IV79 horizontal accelerations are
not different from accelerations recorded during previous earth-
quakes in the magnitude range M_L 6.0 - 6.4.   In this distance
range, only two IV79 data points fall outside the 70 percent con-
fidence limits of Circular 795 and all of the IV79 data points
fall within the 95 percent confidence limits.

In the distance range from 3 to 10 km, the trend of the IV79
data points is somewhat higher than that defined by the records
824 Orion (0.258g, 7.7 km, San Fernando) and Esso refinery (0.39g,
5 km, Managua) and somewhat lower than the peak acceleration at
Pacoima Dam (1.25g, 3.2 km, San Fernando).   The attenuation

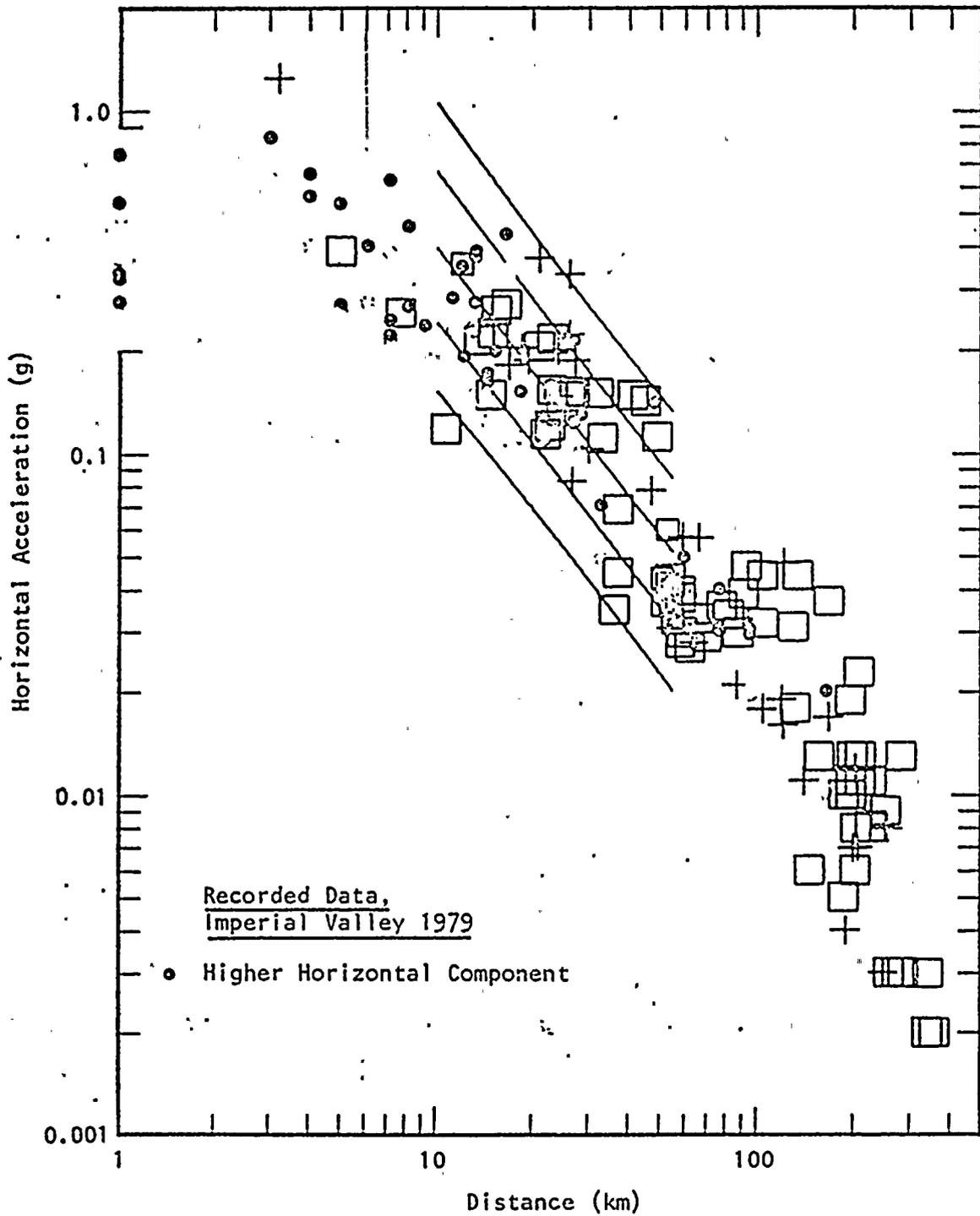FIGURE I-7.   FIGURE 24 OF USGS CIRCULAR 795 WITH
             IMPERIAL VALLEY OCTOBER 15, 1979
             EARTHQUAKE DATA SUPERIMPOSED

Figure 1-4

(Luco)

of peak acceleration with distance in the range from 3-10 km seems to be lower than that from 10 to 100 km.

Fig. 1-4 indicates that the average peak acceleration for stations within 1 km from the fault is somewhat lower than the accelerations recorded 3-4 km from the fault as previously discussed.

Figure 1-5 shows the comparison of the recorded IV79 peak horizontal accelerations with the 70 percent prediction intervals of USGS 795 for the magnitude ranges 5.0-5.7, 6.0-6.4 and 7.1-7.6. The data and the confidence intervals are plotted versus distance to the fault. Fig. 1-5 corresponds to Fig. I-6 of Blume's testimony except that for consistency with the criterium of USGS 795, only the largest peak acceleration at each station has been considered.

In the distance range from 40 to 150 km the IV79 data points fall close to the lower limit of the 70 percent interval for magnitudes 7.1-7.6 indicating that in this distance range peak acceleration increases with magnitude. In the distance range from 15 to 40 km, the IV79 data points fall within or close to the 70 percent interval for magnitudes 6.0-6.4 indicating that the IV79 earthquake is not different from other earthquakes in the magnitude range 6.0-6.4. Finally, in the distance range from 5 to 30 km the IV79 peak accelerations fall in the upper half of the 70 percent confident limit for the magnitude range 5.0-5.7 indicating that peak acceleration increases with magnitude in this distance range.
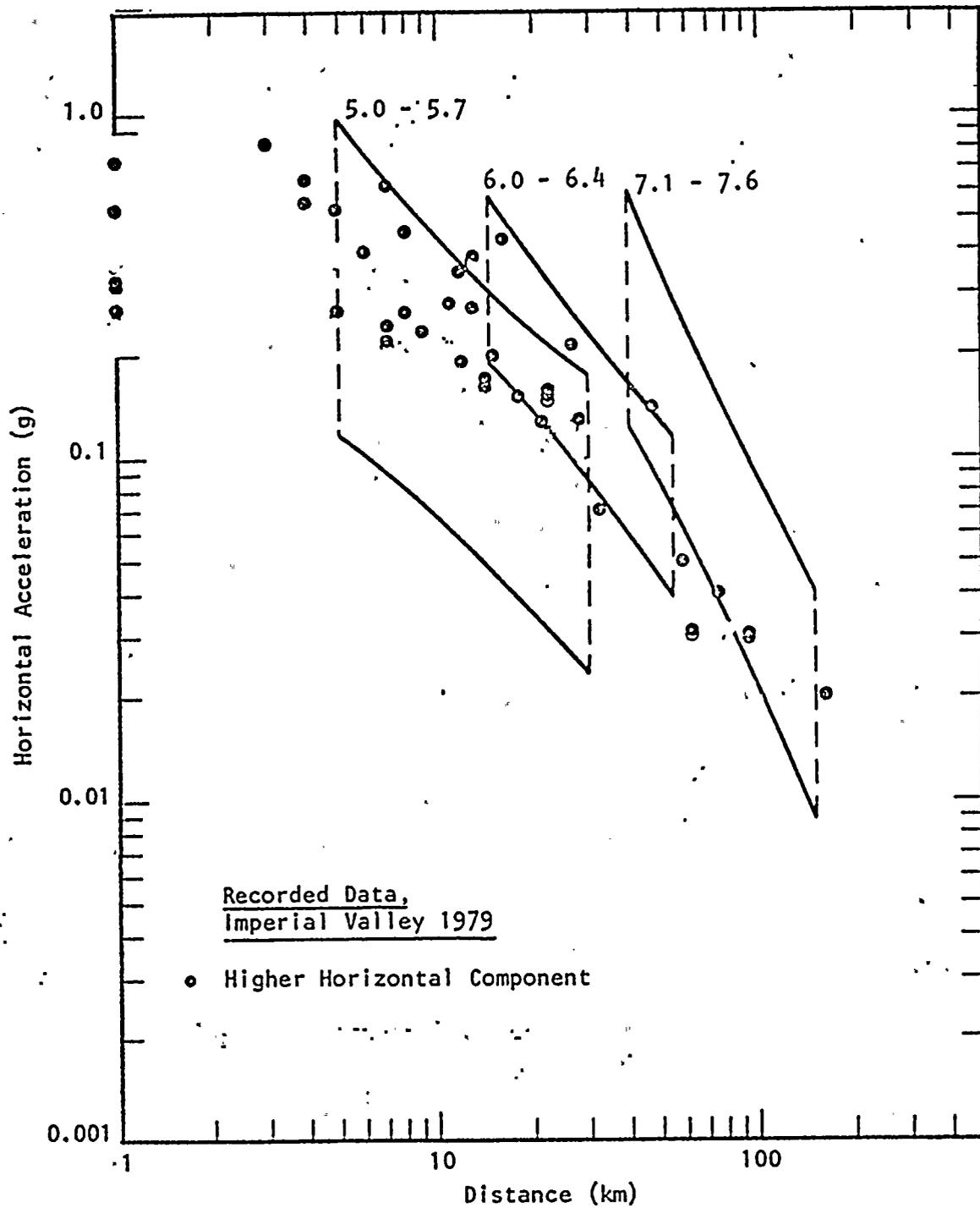
FIGURE I-6.   FIGURE 4 OF USGS CIRCULAR 795 WITH
             IMPERIAL VALLEY OCTOBER 15, 1979
             EARTHQUAKE DATA SUPERIMPOSED

Figure 1-5..
(Luco)

The comparison shown in Fig. 1-5 indicates that peak acceleration increases with magnitude in both the distance ranges of 5 to 30 km and 40 to 150 km.

4. The distribution of peak accelerations on the ground surface and in the vicinity of an extended fault, such as the Imperial fault, follows a two-dimensional pattern which depends on geometry of the fault, on the characteristics of the rupture process and on the local geology. In a typical licensing situation all that is known, in geometric terms, is the distance from the site to the fault. To estimate the ground motion two alternatives are possible. One is to attempt to predict the maximum peak acceleration that could develop at that distance. The second is to estimate the average peak acceleration over the length of the fault at the assumed distance. These two approaches are different as indicated by the following example. Consider a 50 km vertical fault and a line parallel to the trace of the fault at a distance of 5 km. Assume that over ten kilometers of this line peak accelerations of 1g are recorded while over the remaining 40 km the recorded peak accelerations are 0.5g. The average peak acceleration over the line is 0.6g while the largest peak acceleration is 1.g. If we consider a number of different earthquakes, each estimate will have a standard deviation associated with it. The decision of which type of estimate to use goes beyond the technical aspects of the problem.

Correlations of peak accelerations based on shortest dis-
tance to the fault by construction give estimates of the second
type, i.e. estimates of the average peak acceleration over the
length of the fault at a certain distance.

The predictions of various correlations for an earthquake
with a magnitude M ~ 6.5 for a soil site at a "distance" of
5.8 km are compared in Table 1-1. The estimates range from 0.20g
to 0.97g.

Table 1-1. Comparison of various correlations for M ~ 6.5, soil
site and "distance" of 5.8 km.

Correlation	Predicted Peak Acc.(g)	Observation
SAM IV (Blume, 1977)	0.20	$\bar{b}$=1.51, depth 8 km
Donovan (1973)	0.30	M ~ 6.6
Seed et al. (1977)	0.36	soil, distance to fault
Trifunac (1976)	0.97	soft soil, s=0, epi-central distance

Although the magnitude for the IV79 earthquake may not corres-
pond to 6.5, the Table above can be used for a rough comparison.
The estimates of the average of the peak acceleration along a line
at a distance of 5.8 km of the IV79 fault as calculated by Blume
(affidavit) and Frazier (Affidavit) are 0.36g and 0.34g, respec-
tively. My own estimate as discussed above is 0.44g. The pre-
diction of Seed et al. (1976) seems to be equal or slightly lower
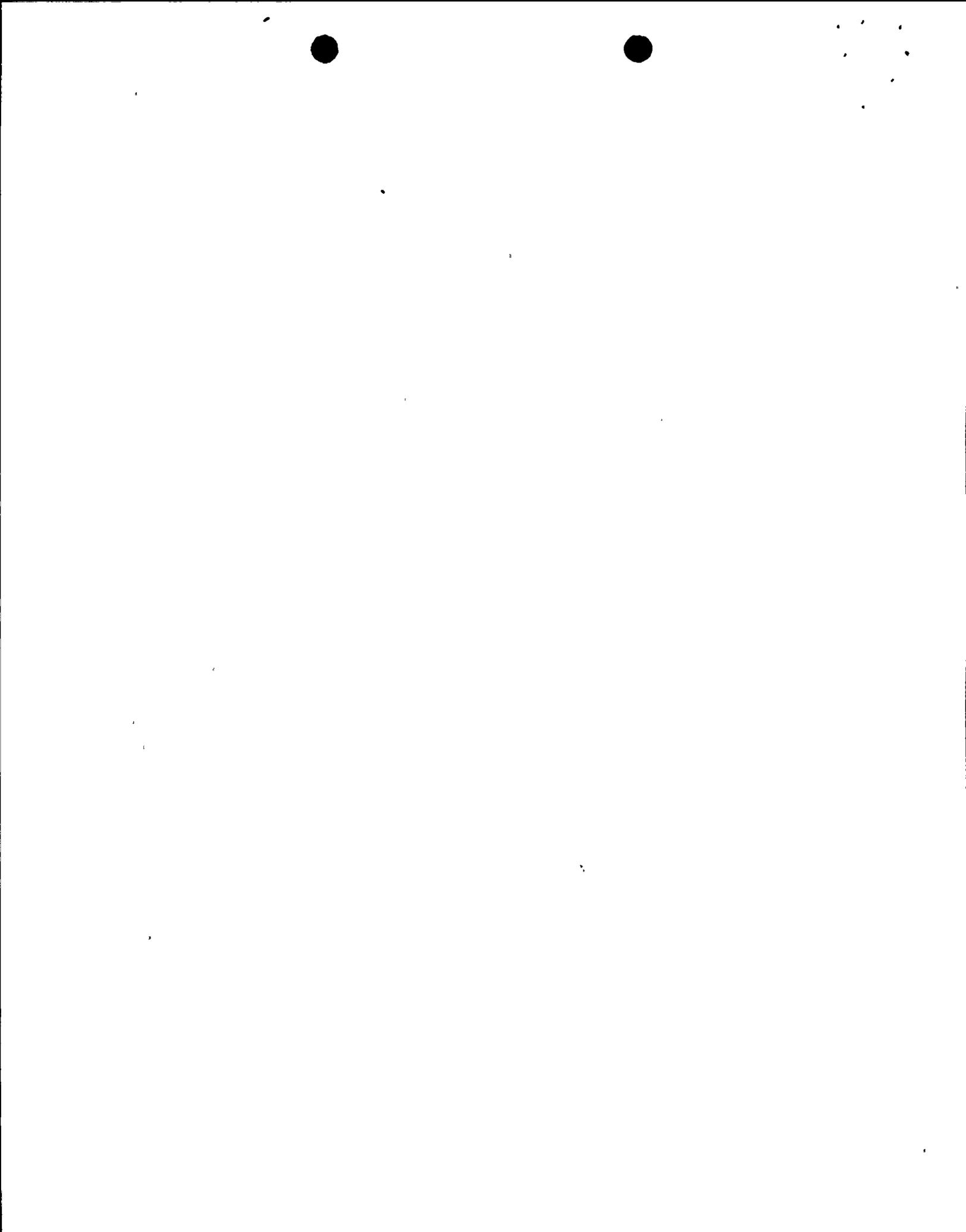
than the 5.8 km-average of the IV79 data. It appears then
that the correlation of Seed et al. provides an estimate of the
second type discussed above. The correlation of Trifunac, on
the other hand, is based on epicentral distance and should not
be compared with the average of the data along a line 5.8 km from
the fault. Since the local magnitude for the IV79 event is
probably somewhat lower than 6.5, I find that the correlation
of Trifunac gives a fair estimate of the peak acceleration re-
corded at Bonds Corner (0.81g) at an epicentral distance of 6 km.
At least on the basis of the IV79 event, it appears that the esti-
mate of Trifunac, at short epicentral distances, provides an
estimate of the first type discussed, i.e. maximum acceleration
at a given distance.

The predictions of the various correlations for a M=7.5
event at a rock site at a distance of 5.8 km are listed in Table
1-2.

Table 1-2. Predictions of Various Correlations for $M_s \sim 7.5$,
rock site and distance of 5.8 km.

| Correlation	Mean Peak Acc.(g)	Observations
SAM V (Blume, 1977)	0.43	$\bar{b}$ = 2.04, rock
Donovan (1973)	0.50	M ~ 7.6
Schnabel and Seed (1973)	0.64	M ~ 7.6 rock
Trifunac (1976)	1.14	rock, s = 2

These predictions should be viewed in the light of the only near-field record in this magnitude range. In the Tabas, Iran earthquake of magnitude M_s = 7.7 a peak acceleration of 0.8g was recorded on recent alluvium at an estimated distance to the fault of 3.0 km (Table B-1, Exhibit 1, P.G. and E Written Testomony). Since the site is on alluvium, the corresponding peak acceleration on rock would be higher. On the other hand, the estimated distance may be shorter than that to the Hosgri fault.

Finally, the predictions listed above are associated with a variance. Since peak accelerations appear to be log-normally distributed, the inclusion of one standard deviation corresponds to a multiplication by a factor which we can denote as $(M+\sigma)/M$. This factor varies from correlation to correlation. Some typical values are: SAM IV (Blume, 1977) 2.53; SAM V (Blume, 1977) 1.81; Seed et al. (1976) 1.67 for soil, 1.45 for rock; Trifunac (1969) 2.02. The correlations in USGS Circular 795 have variance factors ranging from 1.58 to 2.34.

II. Response spectra have been developed from the near-field (1 to 11 km) ground motion records produced by IV-79. The records contain horizontal peak acceleration values in the range of 0.81g to about 0.2g. The applicant calculated a mean peak acceleration of 0.36g for IV-79 at the 5.8 km site-to-fault distance that characterizes the Diablo Canyon site (Applicant's Brief). Despite the fact that the IV-79 peak acceleration values are generally lower than the 1.15g peak acceleration or 0.75g zero-period acceleration used as the design basis for the Diablo Canyon plant (resulting from a postulated 7.5M event on the Hosgri fault), there are instances (although only those from the El Centro Arrays are significant) for which the IV-79 horizontal responses exceed the Newmark Design Response Spectrum for Diablo Canyon. (See staff brief at page 9; Brune Affidavit, Attachments A and B.) In view of this, the parties should discuss whether the Newmark Spectrum is an appropriate and sufficiently conservative representaton of the 7.5M event at Hosgri. 35/

35/ In other words, if the various IV-79 near-field response spectra were used to generate a smoothed, average response spectrum for a zero-period acceleration appropriate to that event (in accordance with techniques explained in Blume's testimony fol. Tr. 6099 at page 6 and pages 39 and 40), and if this spectrum were scaled to a 0.75g zero-period acceleration, would the resulting response spectrum be bounded by the Newmark Spectrum for Diablo Canyon?
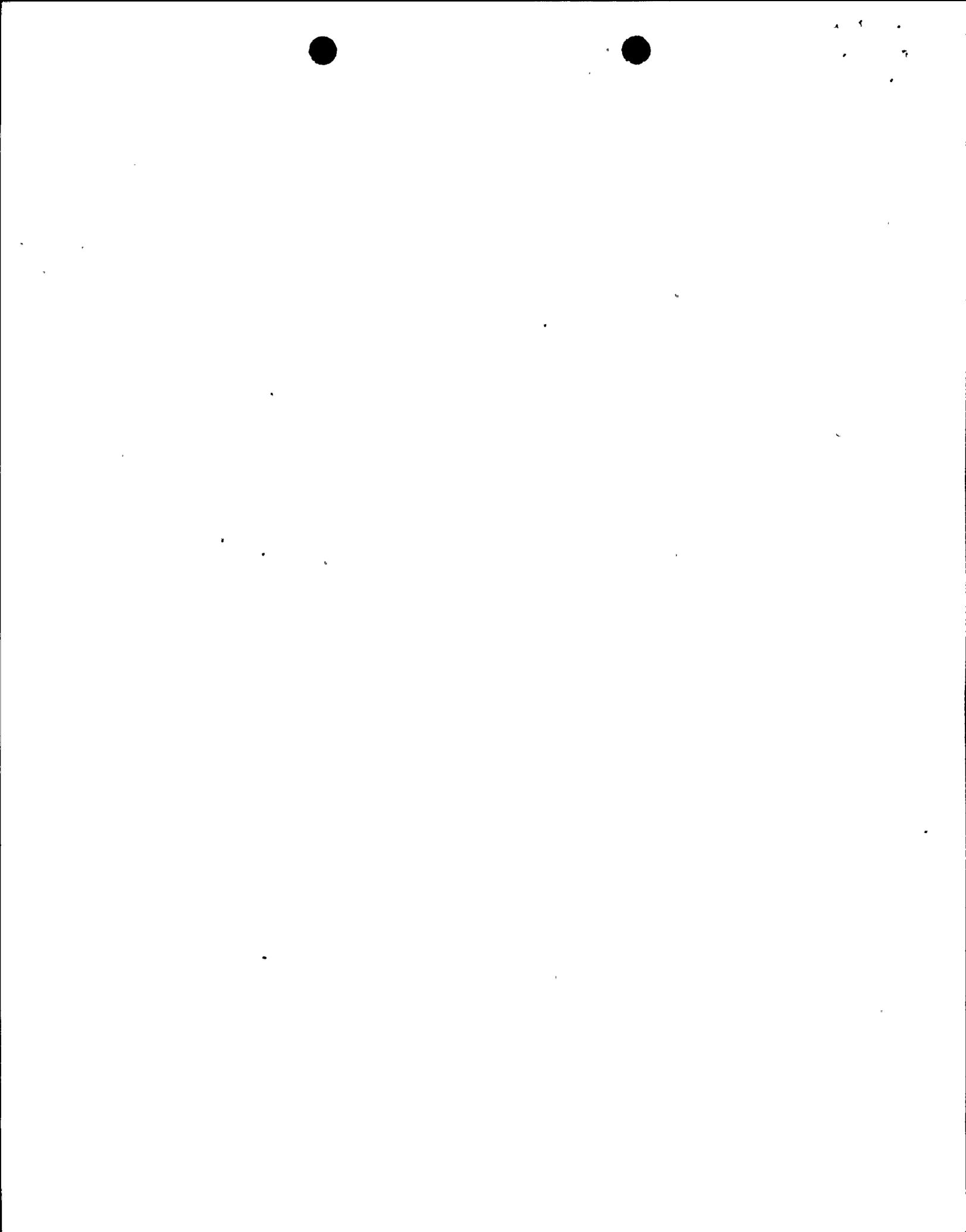
COMMENTS IN RESPONSE TO QUESTION 2.

1. The Newmark Hosgri design spectrum is based on values of 0.75g, 61 cm/sec and 20 cm for the peak acceleration, velocity and displacement, respectively. The corresponding mean values for the IV79 earthquake at 5.8 km, as calculated by Blume (Affidavit, Fig. 1) and Frazier (Affidavit, Table 1), are: `0.35g, 45 cm/sec and 20 cm, respectively. The corresponding 84th percentile values are 0.52g, 75 cm/sec and 32 cm, respectively. Under these conditions it is not surprising to find that the Newmark Hosgri design spectrum will be exceeded by some of the IV79 spectra, particularly in the regions controlled by peak velocity (0.5 - 2 Hz) and peak displacement (<0.5 Hz).

2. Comparison of IV79 and Newmark design spectra for 2 percent damping (Brune, Affidavit p.1, Attachment 1) indicates that the Newmark design spectrum is exceeded in different frequency ranges by the spectra at ten IV recording stations.

Comparison at 5 percent damping level (Rothman and Kuo, Affidavit) indicates that for frequencies in the range of 2 - 33 Hz the Newmark spectrum is exceeded only by the spectra at stations 5 (4 km) and Bonds Corner (3 km). For frequencies in the range of (0.5 - 2 Hz) the Newmark spectrum is exceeded at stations 7 (1 km) and Bonds Corner (3 km). For frequencies lower than 0.5 Hz the spectra at several stations exceed the Newmark spectrum.

At 7 percent damping level and for frequencies higher than 2 Hz, the Newmark spectrum is exceeded only by that at the Bonds Corner station (Blume, Testimony, Figs. II-1, II-2).

The effect of damping on the comparisons is explained by the fact that damping has a stronger effect in reducing the peak spectral amplitudes than in reducing the mean spectral ordinates.

In general, it seems that for frequencies above 2Hz the Newmark free-field horizontal spectrum would provide an adequate design spectrum for a site at a distance of 5 km in the event of the Imperial Valley earthquake. For frequencies lower than 0.5 Hz, the Newmark spectrum is not adequate.

3. To study whether the Newmark spectrum is an appropriate and sufficiently conservative representation for the 7.5M event at Hosgri I have used the following procedure:

(i)   I have assumed that 0.75g is an appropriate value for the mean expected acceleration (not mean plus one standard deviation) at a distance of 5.8 km of the Hosgri fault.

(ii)   I have scaled the average spectrum obtained by Blume (Blume, Testimony, Fig. II-2) from 24 Imperial Valley records within 11 km from the fault to a zero period peak acceleration of 0.75g.

(iii)   I have assumed that the standard deviation for spec-
        tral amplitudes corresponds to the same factor as
        for peak acceleration. The value of M+σ/M calculated
        by Blume (Affidavit, Fig. 1) for peak acceleration is
        1.47. The corresponding value as calculated by Frazier
        (Affidavit, Table 1) is 1.56. As an average, I have
        used a value of 1.5.

(iv.)   To obtain an estimate of the mean plus one standard
        deviation for the Imperial Valley spectra normalized
        to an expected peak acceleration of 0.75g, I have
        multiplied the results of (ii) by 1.5.

(v)   The results of (ii) and (iv) are compared in Fig. 2-1
        with the Newmark free-field horizontal design spectrum
        for 7 percent damping.

Inspection of Fig.2-1 reveals that the Newmark spectrum exceeds
the average IV spectrum normalized to an expected peak acceleration
of 0.75g. The Newmark spectrum falls below the corresponding mean-
plus-one-standard-deviation spectrum. For periods shorter than
0.05 sec (frequencies higher than 20 Hz), the 0.75g IV mean-plus-
one-standard-deviation spectrum exceeds the Newmark spectrum by 50
percent. For a period of 0.2 sec (5 Hz), the difference is of the
order of 28 percent. In conclusion, the Newmark spectrum is low-
er than a mean-plus-one-standard-deviation spectrum based on the
Imperial Valley data and a mean expected peak acceleration of
0.75g.
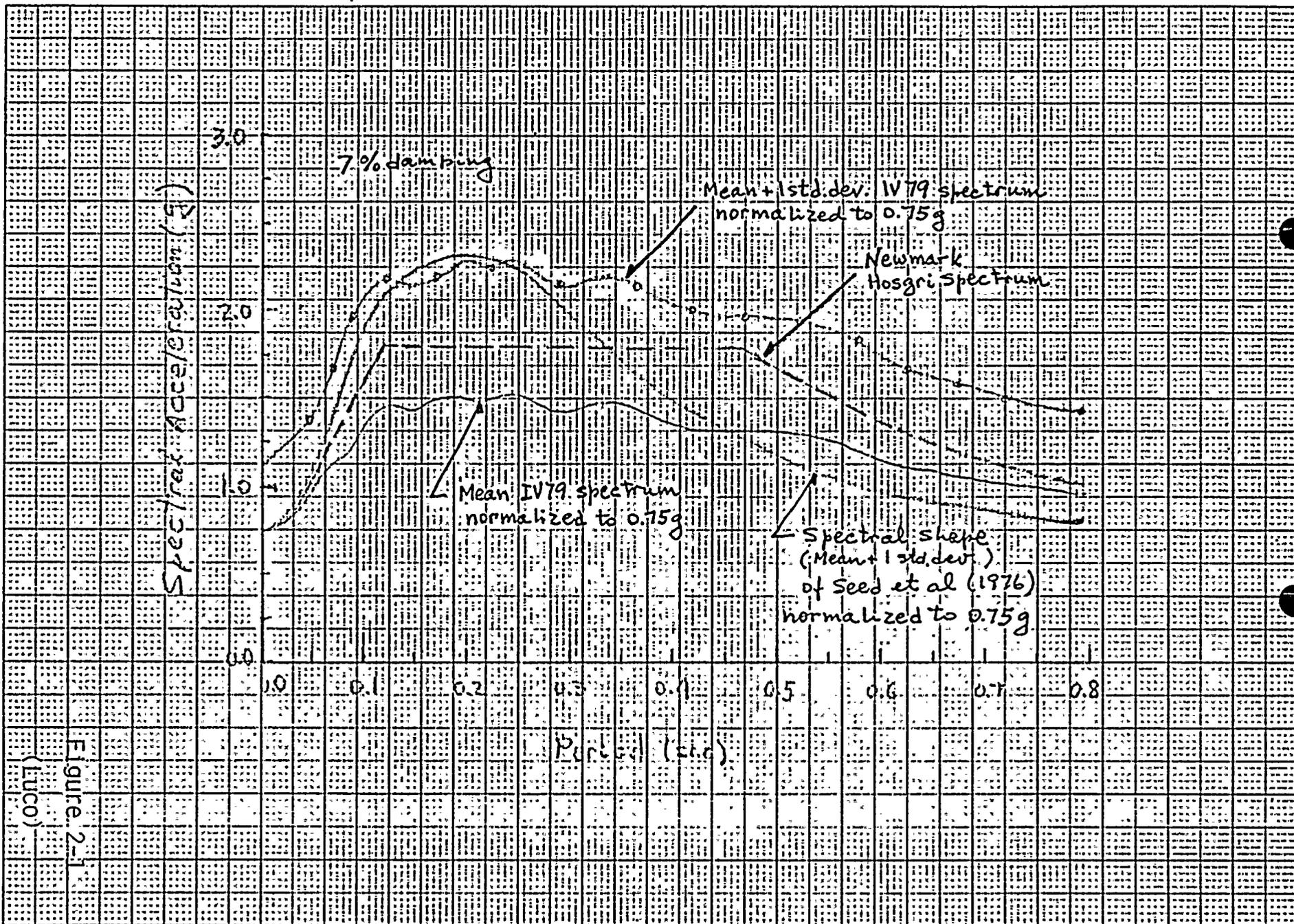
Figure 2-1
(LUCO)

4. To further study the adequacy of the Newmark design spectrum, I have used the normalized spectral shape (mean plus one standard deviation level) for rock obtained by Seed et al. (Bull. Seism. Soc. Am. 66, 1976, pp. 221-243). Since the spectral shape is given for a damping ratio of 5 percent, I have reduced the amplitudes by the factor $(0.5/0.7)^{0.4} = 0.874$ to compare with the 7 percent damped spectra. The resulting spectral shape was scaled to a 0.75g mean expected acceleration. The result is compared with the Newmark spectrum in Fig. 2-1. For periods shorter than 0.35 sec (frequencies higher than 2.9 Hz), the Newmark spectrum falls below the mean-plus-one-standard-deviation rock spectrum of Seed et al.

It should be noted that the use of normalized mean-plus-standard-deviation spectral shapes is inadequate at periods shorter than 0.1 sec. Due to the normalization at zero period, the standard deviation in that region is artificially reduced to zero. This aspect, which is at the root of the problem with 'effective' accelerations, is discussed in the testimony of G. A. Young (pp. 22-23). Also, for periods longer than 0.4 sec, the spectral shape of Seed et al. may be too low due to the lack of near-field large magnitude data in the sample considered.

5. It has been mentioned (Blume, Affidavit, Para. 9) that the period range of interest for Diablo Canyon structures is less than 0.5 sec and that peak velocities and displacements have no effect in that range. The turbine building, however, has a period

(after modifications) of 0.71 sec which is in the region controlled
by peak velocity (FSAR, Amendment 50, D-LL 42). The design peak
velocity is 61 cm/sec while the 84th percentile for the IV79 earth-
quake at 5.8 km is 75 cm/sec (Frazier, Affidavit, Table 1). Clearly,
a design peak velocity of 61 cm/sec at 5.8 km is not conserva-
tive for a 7.5M earthquake. Peak velocities of 113 (Pacoima Dam,
San Fernando) and 108.6 cm/sec (IV79, Sta. 6, 1 km) have been re-
corded. The turbine building houses Category I equipment (com-
ponent cooling water heat exchangers, 4160 volt vital switchgear,
emergency diesel generators and cardox system).

III.   We are told that IV-79 data are not relevant to the Diablo Canyon seismic analysis because that plant is a "rock" site, whereas the Imperial Valley data were obtained on soil sites. (Rothman - Kuo Affidavit at page 3; Blume Affidavit, Paragraph 8.) What is the significance of this difference in view of the conclusion of the authors of USGS Circular 795 (based on an analysis of data provided in that document) that, for comparable earthquake magnitude and distance, there are no significant differences between peak horizontal accelerations measured on soil or rock? (USGS Circular 795 at pages 1, 17, and 26.) This question should be considered in light of statements by applicant's witness Blume to the effect that acceleration, rather than velocity or displacement, is the critical parameter in the design of Diablo Canyon (Blume Affidavit, Paragraph 9; Testimony fol. Tr. 6099, p. 33).

COMMENTS IN RESPONSE TO QUESTION 3.

1. Several studies indicate that the high frequency components of motion in the near field of an earthquake should be higher on rock than on soils [Duke et al. (1972), Donovan (1973), Seed et al. (1975), Seed et al. (1976), Trifunac (1976), McGuire (1978)]. In particular, peak accelerations on rock at short epicentral distances can be 50 percent higher (0.5g level on rock) than those on deep cohensionless soils as shown in Fig. 3-1 reproduced from Seed et al. (EERC Report 75-17, UCB, July 1975). For large epicentral distances, the peak accelerations on alluvium exceed those on rock. To understand this change in behavior it is convenient to refer to the work of McGuire (Bull. Seism. Soc. Am., 68, pp. 803-822, 1978) on Fourier amplitude spectra. By correlation of the Fourier spectra of recorded ground motion, McGuire found that components with frequencies higher than 3 Hz were larger on rock than on soil. At frequencies of 5, 10 and 20 Hz the ratios rock/soil were 1.6, 2. and 1.4, respectively. For frequencies lower than 3 Hz, the components on soil exceeded those on rock. For a frequency of 1 Hz, the ratio soil/rock was 2.0. Similar results had been obtained in a previous study by Trifunac (Bull. Seism. Soc. Am. 66, pp. 1343-1373, 1976). At short epicentral distances, the pulse with the highest acceleration has a high frequency content, and, consequently, we should expect higher peak accelerations on rock than on soil. At large

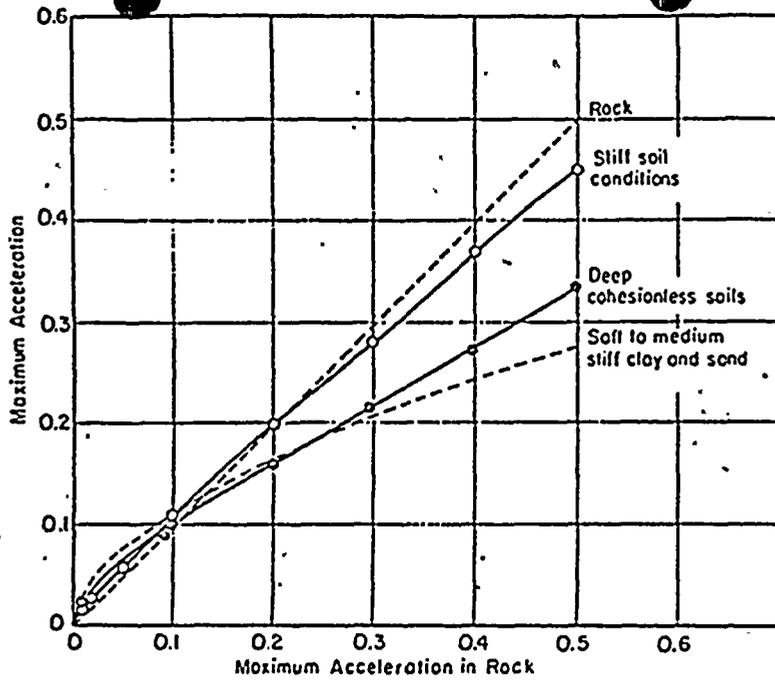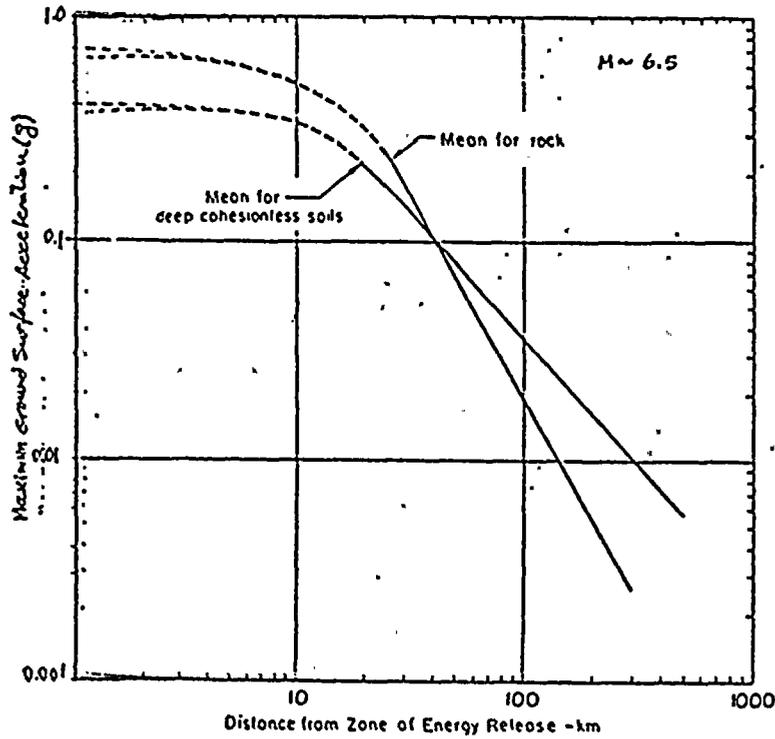Fig. 10 APPROXIMATE RELATIONSHIPS BETWEEN MAXIMUM ACCELERATIONS ON ROCK AND OTHER LOCAL SITE CONDITIONS

Fig. 9 COMPARISON OF MAXIMUM GROUND SURFACE ACCELERATIONS FOR ROCK AND DEEP COHESIONLESS SOIL CONDITIONS

(Seed et al., Relationship between maximum acceleration, maximum velocity, distance from source and local site conditions for moderately strong earthquakes, EERC Report 75-17, UCB, July 1975).

Figure 3-1
(Luco)

epicentral distances, the peak acceleration pulse, due to selective attenuation of high frequencies with distance, has a low frequency content and then we should expect higher accelerations on soil than on rock. The transition takes place at a distance to the fault of the order of 30 km (Fig. 3-1) as shown by the studies of Duke et al. (Report Eng.-7247, UCLA, June 1972), Donovan (Proc. V WCEE, Rome, 1973) and Seed et al. (cited above). The correlations of Boore et al. (USGS Circular 795, 1978) involve data in the range from 10 to 100 km which brackets the transition zone, and, consequently, are not able to detect the difference in behavior. A similar situation occurs with the correlations of Trifunac for peak acceleration, which indicate slightly higher acceleration on soil than on rock independent of epicentral distance (Bull. Seism. Soc. Am., 66, pp. 189-219, Feb. 1976).

The pulses associated with peak velocities and peak displacements involve lower frequency components and will be higher on soils than on rock.

The effects described above must be considered when interpreting the peak motions recorded on soil during the IV79 earthquake.

2. The effects described above also affect the high frequency portion of response spectra. Fig. 3-2, taken from a study by Seed, Ugas and Lysmer (Bull. Seism. Soc. Am., 66, pp. 221-243, 1976) illustrates this point. For frequencies higher than 2 Hz (periods shorter than 0.5 sec) the anticipated mean spectrum for rock exceeds that on deep cohensionless soils by approximately 50 percent
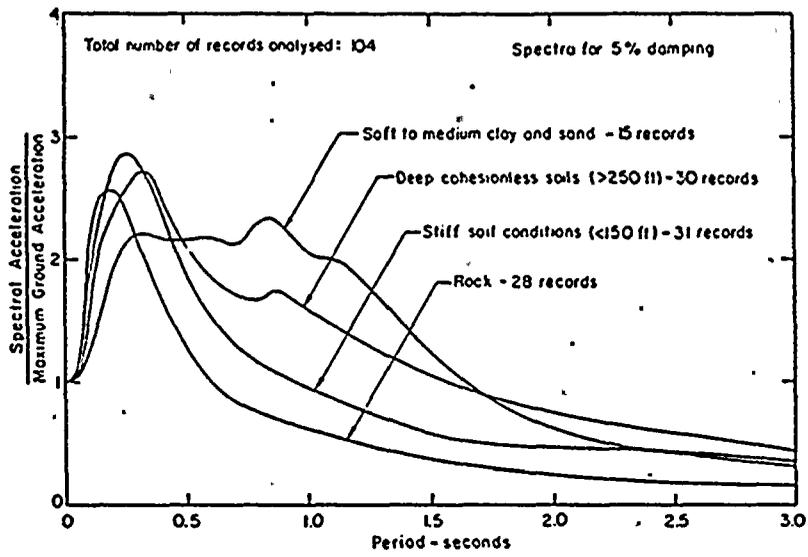
Fig. 9. Average acceleration spectra for different site conditions.

Figure 3-2
(Luco)

(M~6.5, distance of 5 miles). The results of Trifunac also in-
dicate higher spectral amplitudes on rock than on soils for fre-
quencies higher than 5 Hz (Jour. Engrg. Mech. Div. ASCE, Vol.
104, EM5, pp. 1081-1097, Oct. 1978).

3.   The data obtained during the IV79 earthquake is indeed
relevant to the Diablo Canyon analysis but it must be properly
scaled to account for the difference of soil conditions and mag-
nitude.

IV. The magnitudes of vertical and horizontal acceleration values measured at IV-79 are generally comparable. (Mean values calculated at a distance of 5.8 km from the fault are virtually identical.) 36/ The response spectra developed for vertical motion within 11 km of the Imperial Fault during IV-79 appear to show generally equivalent values of vertical and horizontal response for periods less than about 0.2 seconds (i.e., frequencies in excess of 5 cps). 37/ Finally, in some instances the higher frequency portions of the IV-79 response spectra for vertical motion exceed comparable portions of the Diablo Canyon Design Response Spectrum. 38/

Observations made of the IV-79 data and response spectra appear to be consistent with the criteria set forth in NRC Regulatory Guide 1.60. These require that vertical accelerations in the higher frequency range be equal to horizontal accelerations. As the guide states:

> It should be noted that the vertical Design Response Spectra are 2/3 those of the horizontal Design Response Spectra for Frequencies less than 0.25; for frequencies higher than 3.5 they are the same, while the ratio varies between 2/3 and 1 for frequencies between 0.25 and 3.5. 39/

The references to vertical motion made in the Diablo Canyon record, however, indicate that a 2/3 ratio between vertical and horizontal motion was apparently utilized at all frequencies. 40/ The parties should address this apparent

inconsistency and explain it, if possible. Should there be substantive and relevant analyses suggesting that vertical motion records do not reflect the true vertical motion, these should be provided. 41/

36/ Blume Affidavit, Table 1, Figures 1 and 2.

37/ Rothman - Kuo Affidavit, Figures.

38/ Ibid.

39/ We note that elsewhere in the Regulatory Guide frequencies are presented with accompanying units of cycles per second (cps), and assume that these units are inadvertently omitted in the portion we have quoted.

40/ SER Supplement 7, pages 3-18; Knight Testimony, page 13, fol. Tr. 8697, Ghio Test., page 1, fol. Tr. 6993. Blume Testimony, page 41, fol. Tr. 6099.

41/ See, for example, Newmark Testimony, fol. Tr. 8552, Reference B at pages 4, 5; Tr. 9349.

COMMENTS IN RESPONSE TO QUESTION 4.

1. In 1976 during the Gazli, Russia earthquake ($M_s \sim 7.0$),
a near-field vertical peak acceleration of 1.3g was recorded.
This vertical acceleration exceeded the peak horizontal accelera-
tions at the site (0.81 and 0.65g). This high value has been
variously attributed to instrument malfunction or to the collapse
of the structure in which the instrument was located. In 1979, dur-
ing the Imperial Valley earthquake a significant number of near-
field stations recorded vertical peak accelerations larger than the
horizontal peak accelerations. The ratios of the uncorrected ver-
tical peak accelerations to the maximum and minimum horizontal peak
accelerations recorded at each station are shown in Fig. 4-1 versus
distance to the Imperial Fault. The ratios for stations to the
N-E and S-W of the fault have been connected by lines to illustrate
the variation with distance to the fault. A clear trend may be
observed in Fig. 4-1. At short distances the vertical to hori-
zontal ratio is larger than 1., it becomes 1. at about 6 km
from the fault and then decreases to a value ranging from 0.5 to
0.75 at about 10 km from the fault. The vertical to horizontal
peak acceleration ratios for the Gazli earthquake are also shown
in Fig. 4-1 and seem to fit with the general pattern for the IV79
earthquake. Except for the records at station 4, 5 and Bonds
Corner, which were housed on instrument shelters, all other (US)
records were obtained at ground level of one-story buildings.

RATIO VERTICAL/HORIZONTAL PEAK ACCELERATION

Diff Array

Gazli

Bear Canyon

DISTANCE TO FAULT (KM)

Figure 4-1 (LUCO.)

4-4

The ratios shown in Fig. 4-1 indicate that the records at stations 4 and 5 do not deviate from the general trend. The ratio at E.C. Differential Array deviates from the trend but is not different from the ratios at stations 1 and 2 of the Differential Array (Table V-1, Smith Testimony).

2. For a possible interpretation of the behavior shown in Fig. 4-1, it is convenient to refer to the correlations developed by Trifunac (Bull. Seism. Soc. Am., 66, 4, pp. 1343-1373, August 1976) for Fourier amplitude spectra. In that study Trifunac found that the Fourier components of vertical acceleration are higher than the corresponding horizontal components for frequencies higher than 10 Hz. In particular, at 25 Hz the vertical components are 25 percent higher than the horizontal components. For frequencies lower than 10 Hz, the horizontal components are higher. At about 1 Hz, the horizontal components are almost twice the vertical components. In the near-field, the peak acceleration pulses have a high frequency content and, then, it is possible to find larger peak accelerations in the vertical components. At larger distances from the fault the peak acceleration pulses have a lower frequency content and the peak horizontal accelerations exceed the peak in the vertical component. The transition for the IV79 data occurs at a distance of about 6 km from the fault. Since most correlations for peak accelerations are based on data heavily biased to distances in the range from 20 to 100 km it is not surprising to find that these correlations predict vertical peak accelerations of the order of 2/3 of the peak horizontal values.

3. The extremely high ratios (>1) of vertical to horizontal peak accelerations observed at stations 5, 6 and 7 may be associated with the fact that the surface expression of the fault in the vicinity of station 6 indicates a vertical offset (~30 cm) larger than the horizontal relative displacement (10-20 cm). This vertical slip may have contributed to the vertical motion recorded at stations 5, 6 and 7.

4. In view of the Imperial Valley data, the Regulatory Guide 1.60 requirement of equal vertical and horizontal spectral amplitudes for frequencies higher than 3.5 Hz seems proper provided that this requirement is applied to sites at short distances to faults.

5. It has been argued that the particular Imperial Valley geologic structure enhanced the vertical components of motion relative to the horizontal components (Frazier, P.G. and E. Written Testimony). In support of this argument, calculations have been presented indicating that for small strike-slip sources buried at various depth in the Imperial Valley geologic structure, the ratios of vertical to horizontal peak accelerations would vary from 2.0 to 3.1 as an average over epicentral distances less than 10 km (Table IV-2, Frazier IV-15). These values are significantly different from the ratios appearing in a recent report by Del Mar Technical Associates (Tera) (Simulation of Earthquake Ground Motions for San Onofre Nuclear Generating Station Unit 1, Supplement III, August 1980). In this report the ratios calculated by the same

procedure for the Imperial Valley geologic structure range from
0.6 to 1.0. It seems that the differences are due to the use of
different models for the geologic structure of the Imperial Valley.
If such large differences (from ratios of 2.0 - 3.1 to ratios of
0.6 - 1.0) can be obtained by modification of the geologic model,
it is apparent that these comparisons are not reliable. In par-
ticular, the calculated ratios are highly dependent on the assumed
values for the attenuation of shear and compressional waves. These
attenuation values are highly uncertain and it appears that
by selective modeling various ratios can be obtained.

It would be interesting to determine whether the increase
in vertical/horizontal ratio for the two different geologic
structures is due to enhancement of the vertical component or
to a reduction of the horizontal component.

V.   Peak horizontal acceleration values measured at the base of the Imperial Valley Services Building during IV-79 exceed those measured in the free field 103 meters away from the building. The motion records are described as showing similar amplitudes but greater low frequency motion in the building than in the free field. 42/ No response spectra for the two recording locations have been provided. The acceleration data, however, may be taken to indicate that no reduction in building motion due to the tau effect was realized in this instance.

Based on these observations, intervenors question the validity of the tau concept as well as its use to reduce the higher frequency portions of the Diablo Canyon Design Spectrum. The staff and the applicant answer that, because the Imperial County Services Building was supported on piles in a deep soil structure, these observations are irrelevant to the use of a tau effect in the seismic reanalysis of Diablo Canyon, which is built on a rock site. 43/ Staff witness Newmark, however, used recorded earthquake motions at the Hollywood Storage Building to demonstrate the use of a tau effect analysis. 44/ The Hollywood Storage Building itself is built on piles in soil. Thus, the "built-on-piles" rationale appears insufficient to explain why no tau effect was evident at the Imperial Valley Services Building.

One feature distinguishing the two buildings that no party commented upon is that the Hollywood Storage

Building has a basement and the Services Building does not. Intervenors' witness, Dr. Luco, used this fact to explain in part why he believes the Hollywood building should have a large tau value. 45/ Rojahn and Ragsdale's discussion implies that to some extent ground level instrumental responses within the Imperial Valley Services Building may have been influenced by the response (and failure) of the building itself. 46/

In any event, given the apparent similarities between the structural foundations of the two buildings, the explanations provided thus far for a seeming lack of a tau effect at the Imperial Valley Services Building are inadequate. The parties should provide additional information on this point and relate their analyses to both geologic and structural conditions prevailing at the Diablo Canyon site.

42/ See "A Preliminary Report on Strong-Motion Records from the Imperial county Services Building by Christopher Rojahn, U. S. Geological Survey and J.D. Ragsdale, California Division of Mines and Geology (undated but issued early January 1980), pages 7 and 8.

43/ Blume Affidavit, Paragraph 10; Rothman - Kuo Affidavit, page 7.

44/ SER Supplement 5, Appendix C.

45/ Tr. 8949.

46/ Rojahn and Ragsdale, pages 7 and 8. That report also reflects information regarding the Services Building asymetric structure (at pages 2 and 3) which may explain why .is was susceptible to damage (see Newmark Testimony fol. Tr. 8552, Attachment B, pages 14 and 15).

COMMENTS IN RESPONSE TO QUESTION 5.

1. The Newmark free-field horizontal design spectrum has been considerably reduced for periods shorter than 0.5 sec (frequencies higher than 2 Hz) by use of the so-called tau effect. The nature of the reductions is shown in Fig. 5-1 (Figs. 3-5, 4-5, 4-6 and 4-7, Seismic Evaluation for Postulated 7.5M Hosgri Earthquake, P.G. and E., Vol. I). Reductions of this magnitude require detailed justification.

2. Before proceeding with the discussion of this issue, it is necessary to refer to a more detailed description of the complete soil-structure interaction effects. Consider first the case of a structure supported on a flat mat foundation resting on the soil surface. In the linear range, the calculation of the response of the structure can be divided in two parts. The first part entails the calculation of the foundation response to the seismic excitation in absence of the structure. The second part involves the evaluation of the motion of the foundation and distortion of the soil due to the forces and moments that the structure exerts on the foundation and soil. This second part, which we may call soil-structure interaction proper, also involves the calculation of the structural response incorporating the distortion induced in the soil by the structure. Depending on the characteristics of the structure, foundation, soil and seismic excitation, soil-structure interaction proper may reduce or increase the response at foundation level resulting from the first part.
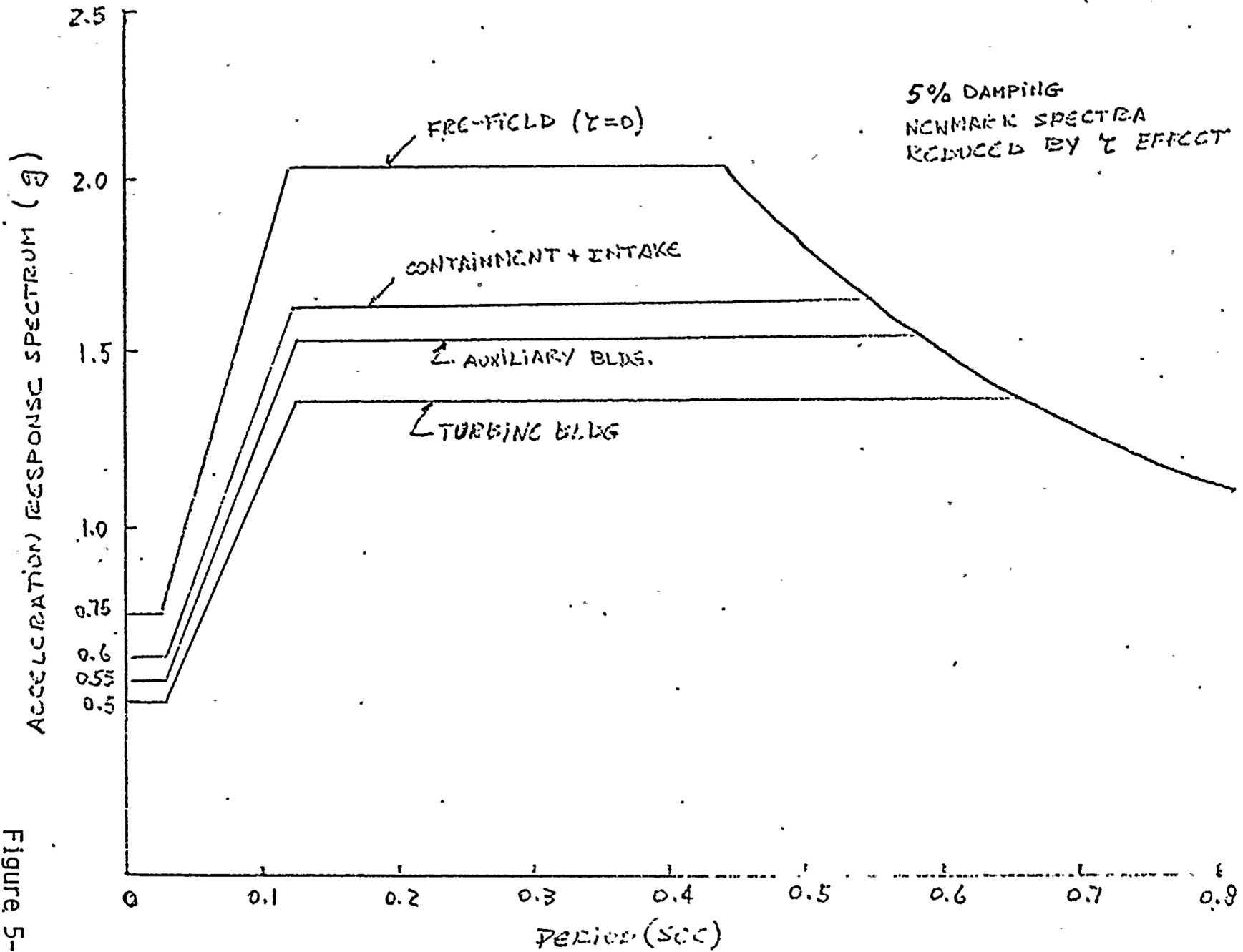
Figure 5-1
(Luco)

5-4

For very stiff soils, the effects of interaction proper may be small and are often neglected. In the case of Diablo Canyon this was the option taken by the applicant. The discussion, then, reduces to the first part of the complete soil-structure problem, i.e., the interaction between the foundation and the seismic excitation.

The response of a flat massless foundation placed on the ground surface depends on the type of seismic excitation. For vertically incident plane waves, the response of the foundation would have exactly the same amplitude as in the free-field. If the waves are inclined, then, the translational response of the foundation at high frequencies is reduced and torsional and rocking effects are introduced. The extent of the reduction of the translational response and the creation of torsional and rocking effects depend on the dimensions of the foundation, on the apparent horizontal velocity of the inclined wave, on frequency and on the relative stiffness of the foundation to that of the soil. For very stiff foundations, the critical parameter is $f\tau$, where f is the frequency and $\tau = D/C$ is the time it takes for the wave with apparent horizontal velocity C to travel the width D of the foundation. If tau is zero ($\tau=0$), then there is no reduction in translational response and no rocking or torsional response induced in this first part of the interaction problem.

3. For the containment building at Diablo Canyon a value of $\tau = 0.04$ sec has been used. Since the diameter of the foundation

is 153 ft, the corresponding value for the apparent velocity is
C = 3825 ft/sec. Since the shear wave velocity at the depth of
the foundation is of the order of 3600 ft/sec, this value of C
implies that it is assumed that waves propagate almost horizon-
tally; clearly, an untenable assumption.

A recent study (Luco and Sotiropoulos, Bull. Seism. Soc. Am.,
1980, in press) indicates that for epicentral distances in the
range from 5 - 12 km the apparent wave velocities are of the
order of the velocities in rock at the depth of source (~3.5 km/
sec). Observations by Tamura et al. (Proceedings, 6th WCEE, India,
2;2-63/2-68) employing arrays of seismometers and accelerometers
separated by distances of 100 m lead to values of C in the range
from 2.6 to 2.9 km/sec at epicentral distances in the range 30 -
50 km. In a report by Del Mar Technical Associates (Delta-R-79-
034, Studies on Phase Velocity versus Frequency) values in excess
of 3 km/sec for the phase velocity at an epicentral distance of
5 km are calculated for the geologic structure at Diablo Canyon.

4. When it was clear that the assumption of almost horizon-
tally incident waves could not be supported, the interpretation of
the tau effect was modified. It was assumed that the reduction of
translational response of the foundation was associated with in-
coherence of the seismic excitation. Incoherence of the seismic
motion would produce effects similar to those for inclined waves.
The critical parameter in this case would be D/λ (=$f\tau$) where λ is
the characteristic wave-length of the incoherence at a frequency
f. To produce the same effects on the containment building as
$\tau = 0.04$ for f = 10 Hz, the characteristic length of the incoher-

ence would have to be $\lambda = 383$ ft; or, in other words, at 10 Hz, points separated by 190 ft would have to be moving in opposite directions. The fact that Tamura et al. (1980, cited above) were able to calculate consistent apparent phase velocities using arrays of five to six instruments 100 m apart argues against the existance of such extreme incoherence over short distances.

During the Imperial Valley earthquake strong motion records were obtained in a linear array of five instruments (Bycroft, El Centro California Differential Ground Motion Array, USGS Open File Report 80-919, 1980). The array is approximately 3 km from the Imperial Fault. Stations 2, 3, 4 and 5 are at distances of 60, 180, 420 and 700 ft from station 1. The acceleration time histories recorded in the E-W direction are shown in Fig. 5-2. Since the timing system failed, the time axis shown in the figure does not correspond to absolute or relative time. The similarity of the records (at least to the naked eye) at stations 1, 2 and 3 is striking. Some slight differences between the records at stations 4 and 1 (separated by 420 ft) can be observed. Although more detailed analysis of these records is necessary, it is possible to say that the motion of points separated by distances of 180 ft is coherent at least up to frequencies of 5 Hz.

In the testimony of S. Smith (Applicant Written Testimony, V-2, V-6), uncorrected peak accelerations recorded at the array are presented. The quoted peak accelerations are given in the following Table.
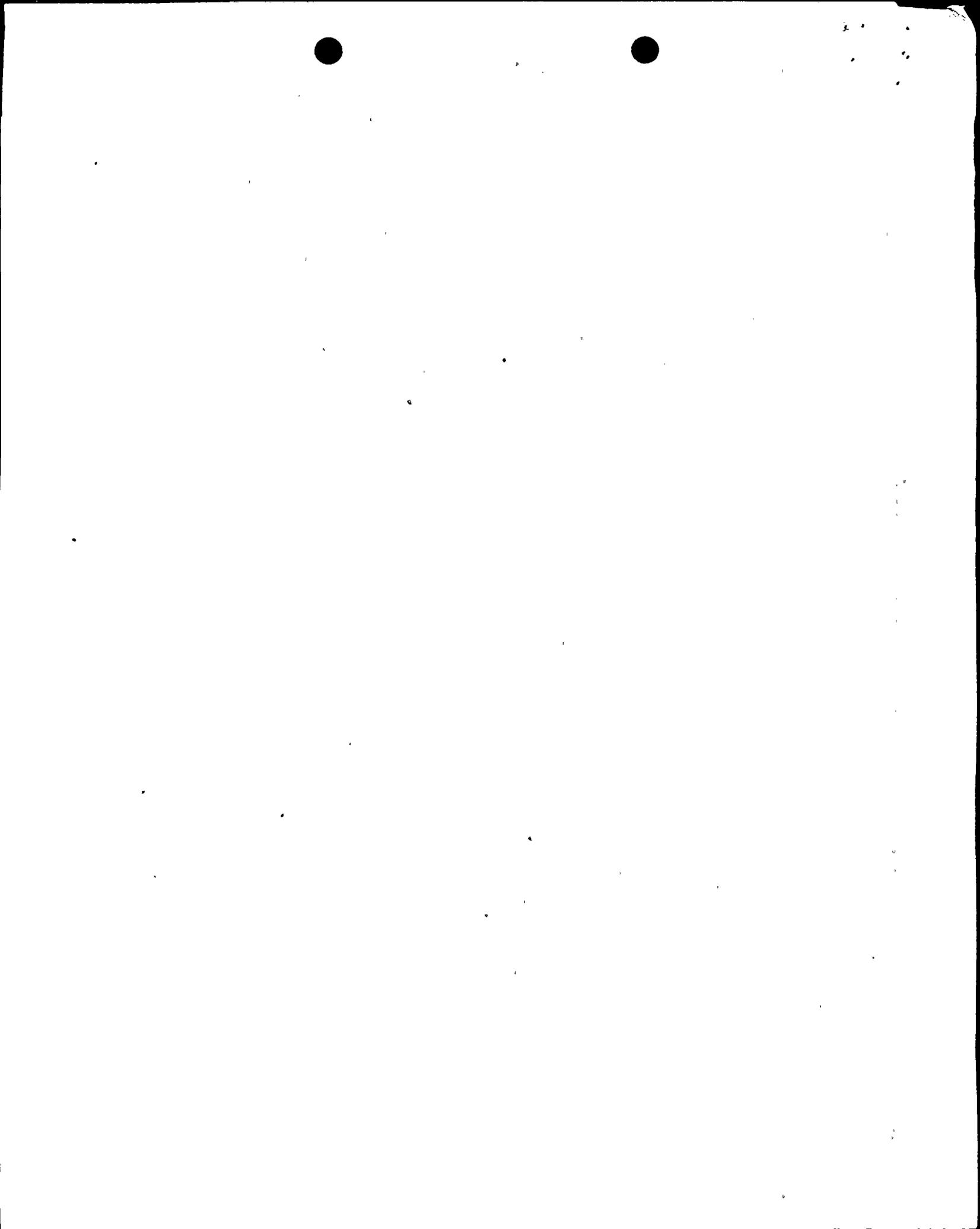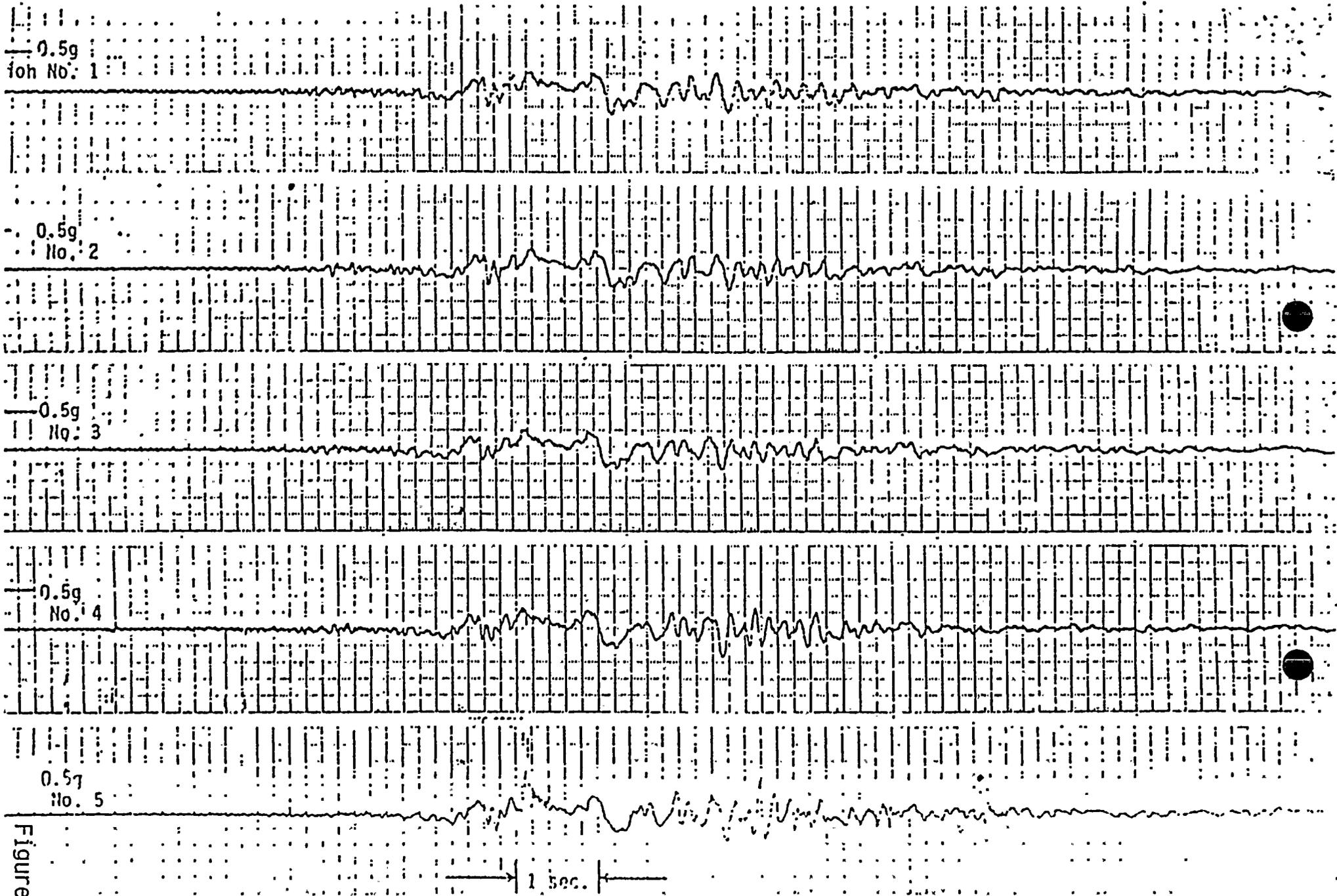
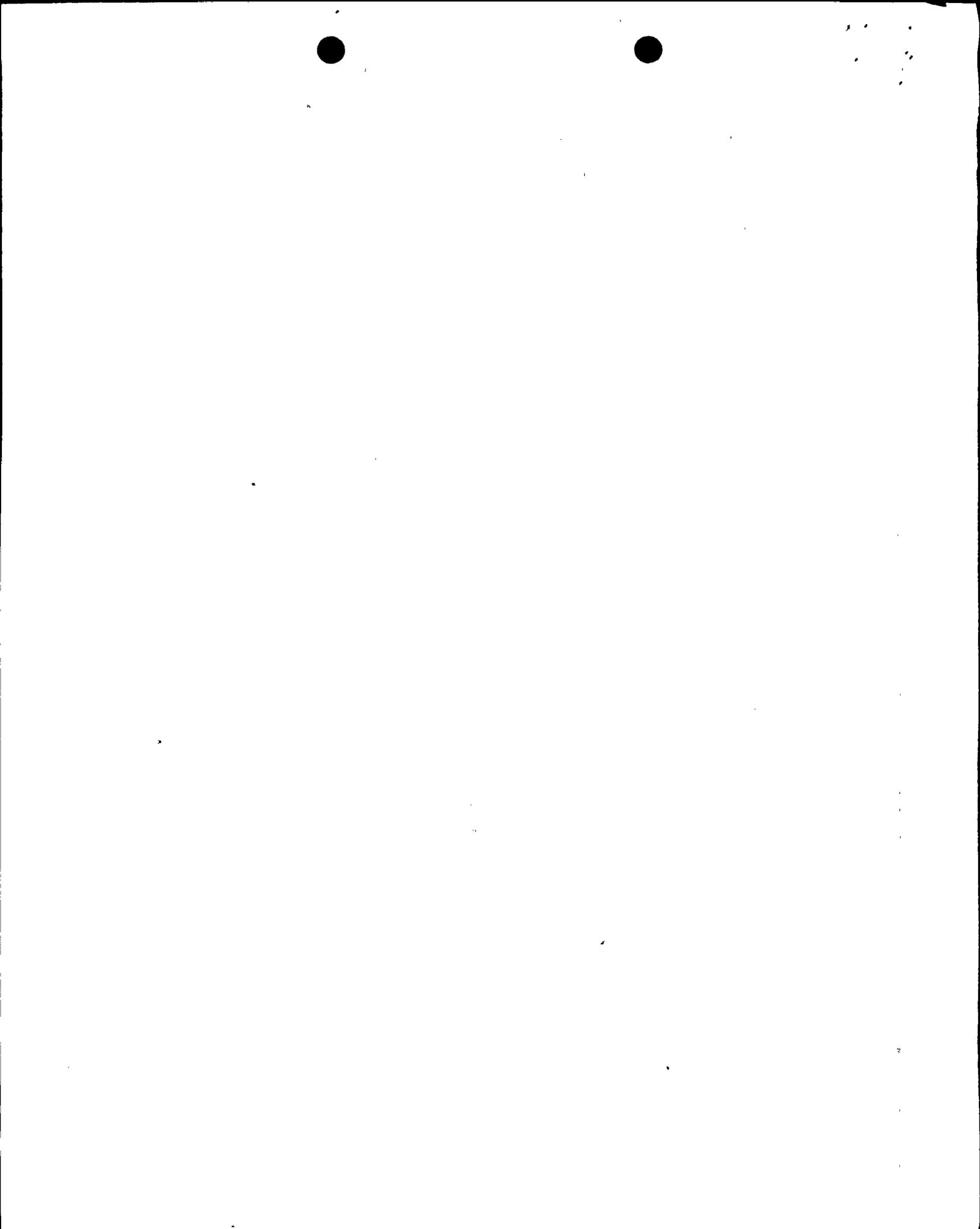Figure 5. East-west accelerations at the El Centro Differential Ground Motion Array.

Figure 5-2 (Luc

Table 5-1.   Uncorrected Peak Acceleration (g) El Centro
             Differential Array.

Station	#1	#2	#3	#4	#5
Distance	|—60 ft—|—120 ft—|— 240 ft-|—280 ft—|				
N-S·	0.29	0.28	0.24	0.42	0.45
E-W	0.38	0.37	0.38	0.43·	0.39
Vert	0.63	0.66	0.50	0.51	0.28

The peak accelerations listed in Table 5-1 indicate that the horizontal components of motion are coherent within distances less than 180 ft. Some differences can be observed in the vertical components. The significance of these differences will have to be established after the records are corrected.

Dr. Smith has calculated an estimate of the effect of incoherence on the vertical response spectra by averaging the vertical time histories and dividing the resulting response spectrum by the vertical spectrum at station 2. Based on this study he concludes that 'The spectrum of the array average is essentially identical to an individual member of the array up to frequencies of about 5 Hz. Above this frequency, the array sum has its spectral components reduced, the reduction becoming as large as 40% at a frequency of about 30 Hz.' That this is an extreme upper bound for the effects of incoherency can be surmised from the following facts:

(i) The calculation is referred to the vertical components where maximum deviations in peak accelerations are observed. Lower deviations would be obtained for the horizontal components.

(ii) The 40% reduction of the spectrum for the average
vertical motion is with respect to station #2 which has the largest
vertical peak acceleration. If he had used station #5 as reference,
he would have found a 41% increase in spectral amplitude at 30 Hz.
A more proper comparison is with the average vertical spectrum which
leads to a reduction of less than 23% at 30 Hz.

(iii) This reduction of 23% at 30Hz would take place for a
rigid foundation 700 ft wide. The diameter of the containment build-
ing is only 153 ft and the equivalent diameter of the turbine build-
ing is 267 ft. Assuming that the filtering effect can be represented
by a function of the type $\sin(\omega L/\alpha)/(\omega L/\alpha)$, a reduction of 23 per-
cent at 30 Hz over a distance of 700 ft would indicate reductions at
30 Hz of the order of 1. and 3.6 percent over lengths of 153 and
267 ft, respectively.

(iv) The variation of peak vertical accelerations along the
array follows a clear trend. This trend would produce rocking re-
sponse of the structures. This rocking could induce an additional
vertical acceleration along the perimeter of the turbine building
of the order of 0.07g.

The upper bound calculations of Dr. Smith indicate that the
incoherence effects over the distance range of 150 to 270 ft and,
in the frequency band of interest, are extremely small and partic-
ularly so for the horizontal components which are more important
than vertical accelerations in typical seismic analyses. It is
unfortunate that the applicant did not calculate or present the
reductions for the vertical and horizontal components over the
lengths of interest.

Table 5-2. Examples of Soil-Structure Interaction Effects on High-Frequency Spectral Amplitudes.

Structure	1	2	3
Foundation width (m)	20	20	40
Height (m)	40	40	60
Mass of structure (kg)	10^7	10^7	3×10^7
Fund. frequency (Hz)	2	1	5
Shear wave velocity (m/sec)	400	400	600
Ratio of 25 Hz spectral amplitudes of base to free-field	1.20	1.08	0.90

In these calculations there was no wave scattering involved (no tau effect). In two cases the high frequency (response) spectral amplitudes at the base of the structures were higher than those on the free-field. In the third case, the opposite occurred.

In a second set of soil-structure interaction calculations the response of slightly non-symmetric containment building (Foundation diameter 146 ft, height 238 ft, total weight 88×10^6 lb, fundamental fixed-base frequencies in x- and y- direction 4.2 Hz) supported on a flat foundation resting on a soil with a shear wave velocity of 1400 ft/sec was obtained (Wong and Luco, Identification of Sensitive Parameters for Soil-Structure Interaction,

It seems, then, that the two rational bases for the tau effect (nonvertically incident waves and incoherence) are not of the magnitude required to justify the reductions of the design spectrum for Diablo Canyon shown in Fig. 5-1.

5. The inclusion of a reduction by tau effect has been justified on the basis of the response of the Hollywood Storage Building. The high frequency components of the response recorded in the basement of the Hollywood Storage Building during several earthquakes are significantly lower than those recorded on the free-field. On the other hand, the horizontal peak accelerations recorded on the first floor of the Imperial County Service Building were higher than those recorded in the free-field.

In the first place, it must be understood that since both structures are supported on soft soils, the response recorded at the base is a result of the complete soil-structure interaction phenomenum, and, consequently, it cannot be used as direct evidence for or against the existence of averaging of nonvertically incident waves or of incoherent motion (tau effect). To illustrate this point, consider the theoretical results listed in Table 5-2 and obtained by soil-structure interaction analyses of three structures supported on flat rigid foundations subjected to coherent vertically incident waves (Luco and Wong, Response of Structures to Nonvertically Incident Waves, Report, UCSD, October 1979).
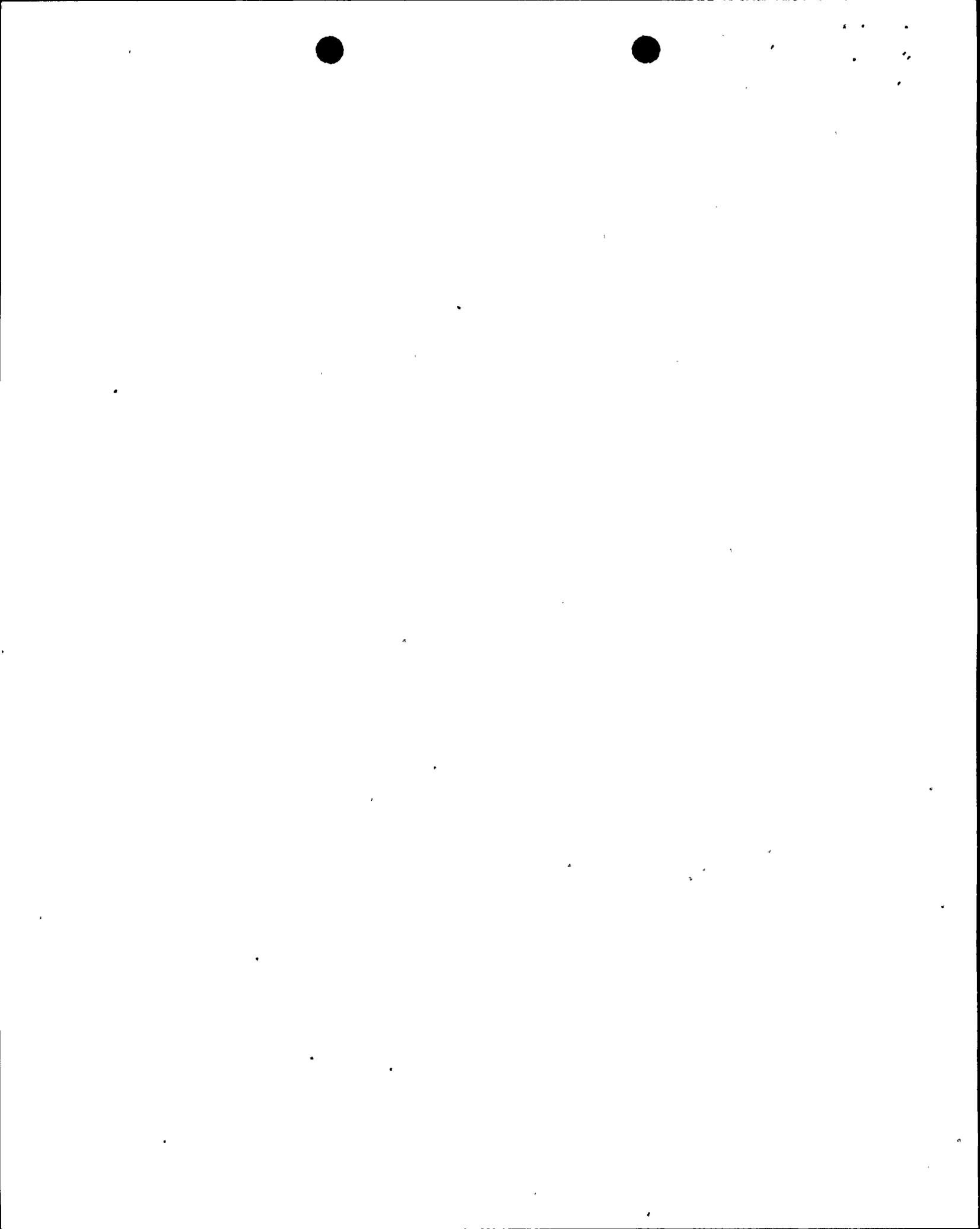
Report, USC, August 1980). In this case, nonvertically incident SH waves with apparent phase velocities of 4 km/sec and P-SV waves with apparent phase velocity of 6 km/sec were used as excitation. The horizontal peak accelerations in the x- and y-directions in the free-field were equal to 0.20g. Peak accelerations at the base of the structure of 0.23g and 0.22g were calculated in the x- and y-directions, respectively. In this case, although scattering was present (tau effect) it was not sufficient to balance other interaction effects and the peak accelerations at the base of the structure were larger than in the free-field.

These examples illustrate the futility of arguing about the so-called tau effect on the basis of simplistic comparisons. In here, I am assuming that the interpretation of this effect has not been modified to encompass the complete soil-structure interaction phenomenum.

6. To analyze the effects observed in the H. S. building, it is necessary to describe the complete interaction effects for structures supported on embedded foundations. In this case, the resultant effect can be separated into three separate components. The first effect occurs in the free-field and is associated with the interference between the upwardly propagating seismic excitation with the downwardly moving wave reflected from the ground surface. At a given depth, certain frequency components are drastically reduced as a result of this interference. The result
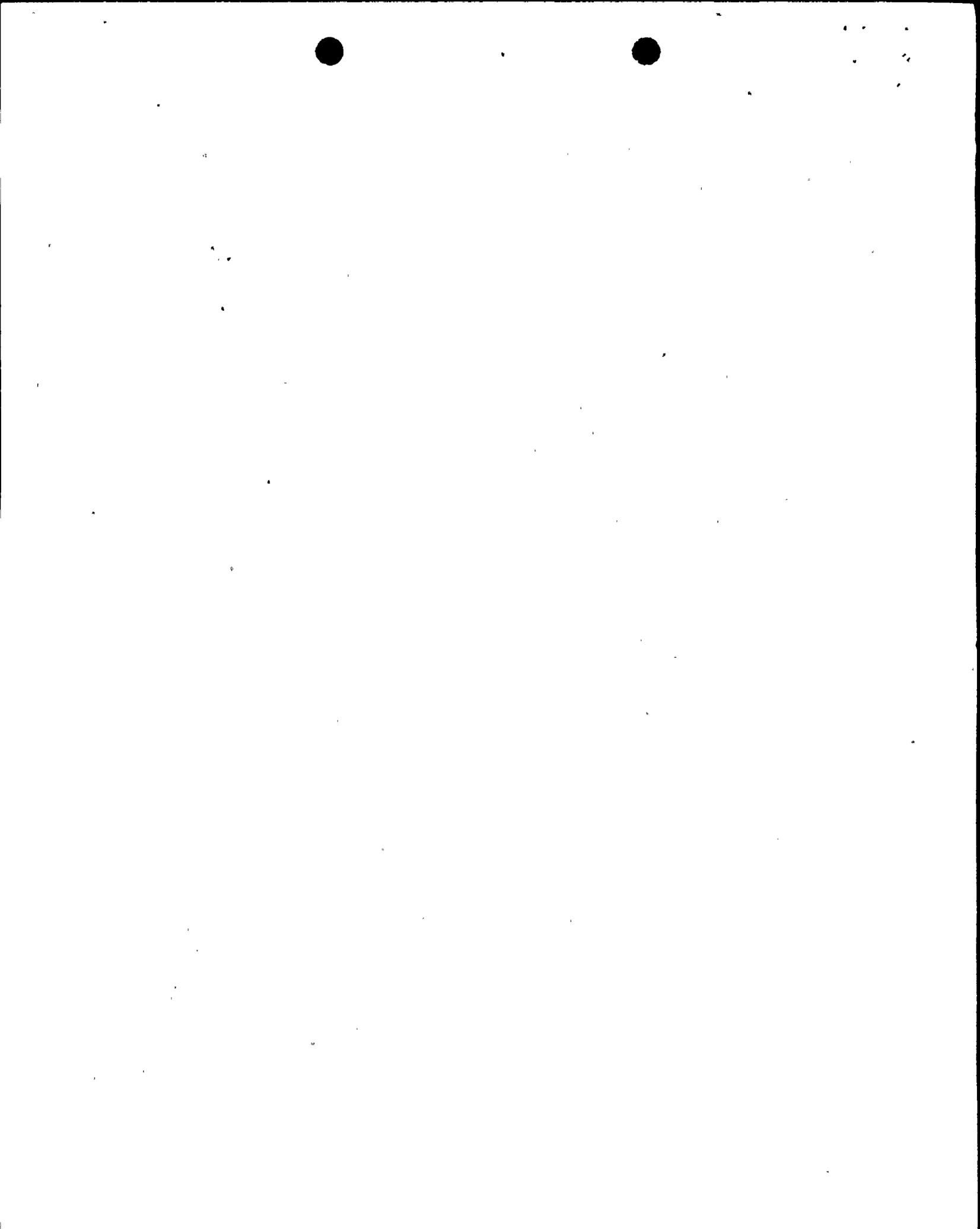
over the top few tens of feet is that the motion has lower amplitudes than those on the ground surface. The extent of the reduction depends on depth, stiffness of the soil and frequency content of the excitation. For shallow depths and hard soils the reduction is not significant except at extremely high frequencies.

The second effect is associated with the presence of the foundation (in absence of the superstructure). Since the motion of the ground varies with depth, and since the embedded foundation has a rigidity different from that of the soil, the motion of the soil in the vicinity of the foundation is altered. The resulting motion at the bottom of the embedded foundation is then different from the motion of a point in the soil at the same depth but away from the foundation (free-field motion at the depth of the foundation). This scattering effect may lead to translational motion of the foundation which may be slightly higher or lower than that of the free-field at the same depth. The scattering effect also induces a rocking response of the foundation.

Finally, the third effect corresponds to the effect of the superstructure on the motion of the foundation (soil-structure interaction proper). In terms of the translational response at the bottom of the foundation, the interaction proper may increase or reduce the motion resulting from the previous two effects. For example, consider the calculated response of the containment building of a typical nuclear power plant. The foundation in this
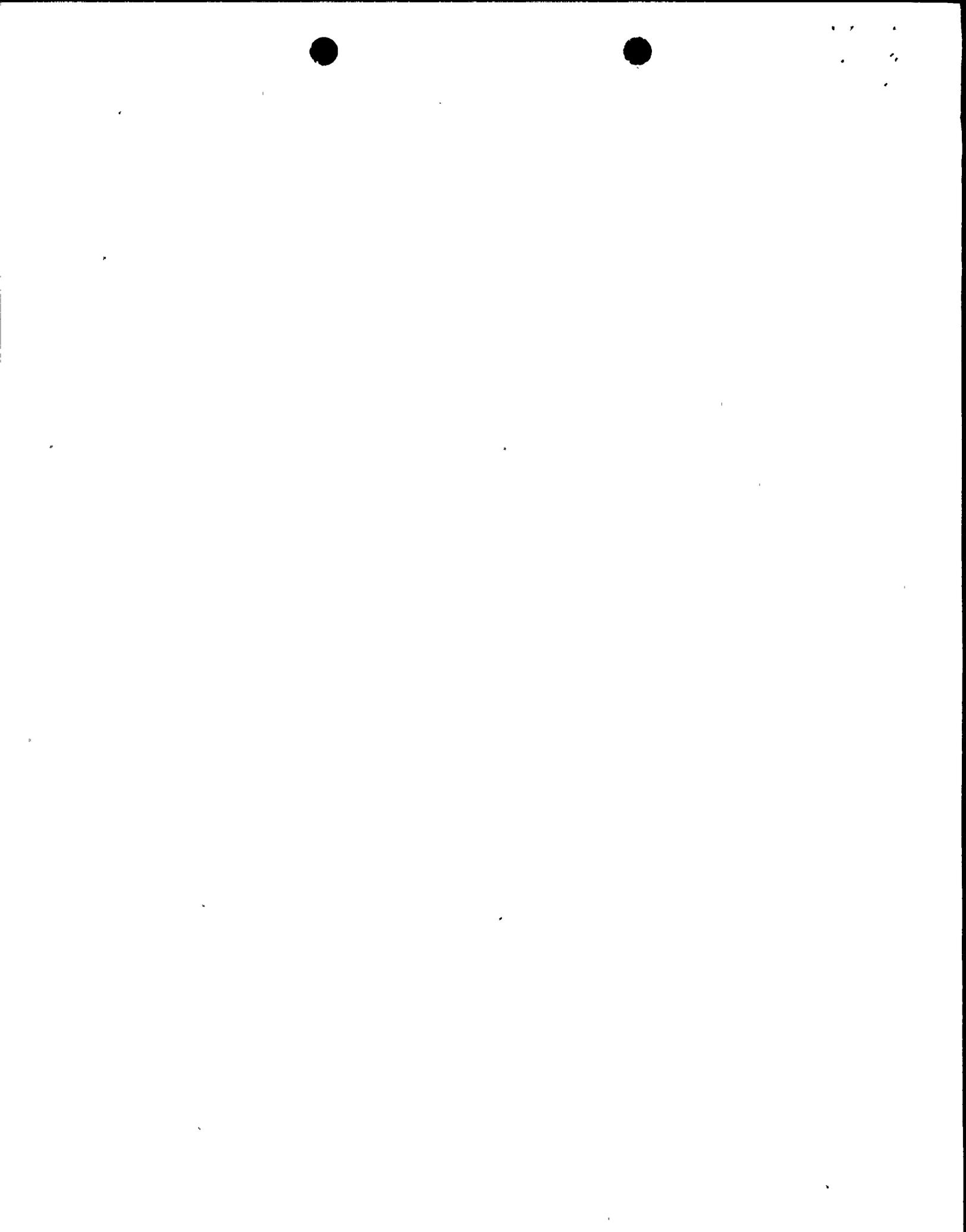
case is embedded 36 ft in a soft soil with shear wave velocities ranging from 600 ft/sec at the ground surface to 1400 ft/sec at the depth of the foundation. For a particular excitation with a peak acceleration of 0.20g on the ground surface, the free-field motion at a depth of 36 ft had a peak acceleration of 0.08g (first effect). The scattering effect by the foundation (second effect) slightly increased this value leading to a response at the bottom of the foundation of 0.083g. Finally, the effect of inter-action proper reduced the previous value to 0.078g. For the same structure and soil but for a different seismic excitation having a peak acceleration of 0.35g on the ground surface, the corres-ponding quantities are 0.319g, 0.288g and 0.295g (Wong and Luco, Identification of Sensitive Parameters for Soil-Structure Inter-action, Research Report, USC, August 1980). It should be noted that peak acceleration at foundation level is not a good measure of the effects of soil-structure interaction which include rock-ing and modification of the response of the superstructure.

The Hollywood Storage Building has a basement embedded approximately 9 ft into a soil with a shear wave velocity of the order of 600 ft/sec (at least over the first 50 ft). In addition, the structure is supported on a large number of piles (I do not have details available). In 1959, I participated in a study of the response of the HSB to the Kern County earthquake of 1952 (Duke et al., Strong Earthquake Motion and Site Conditions:
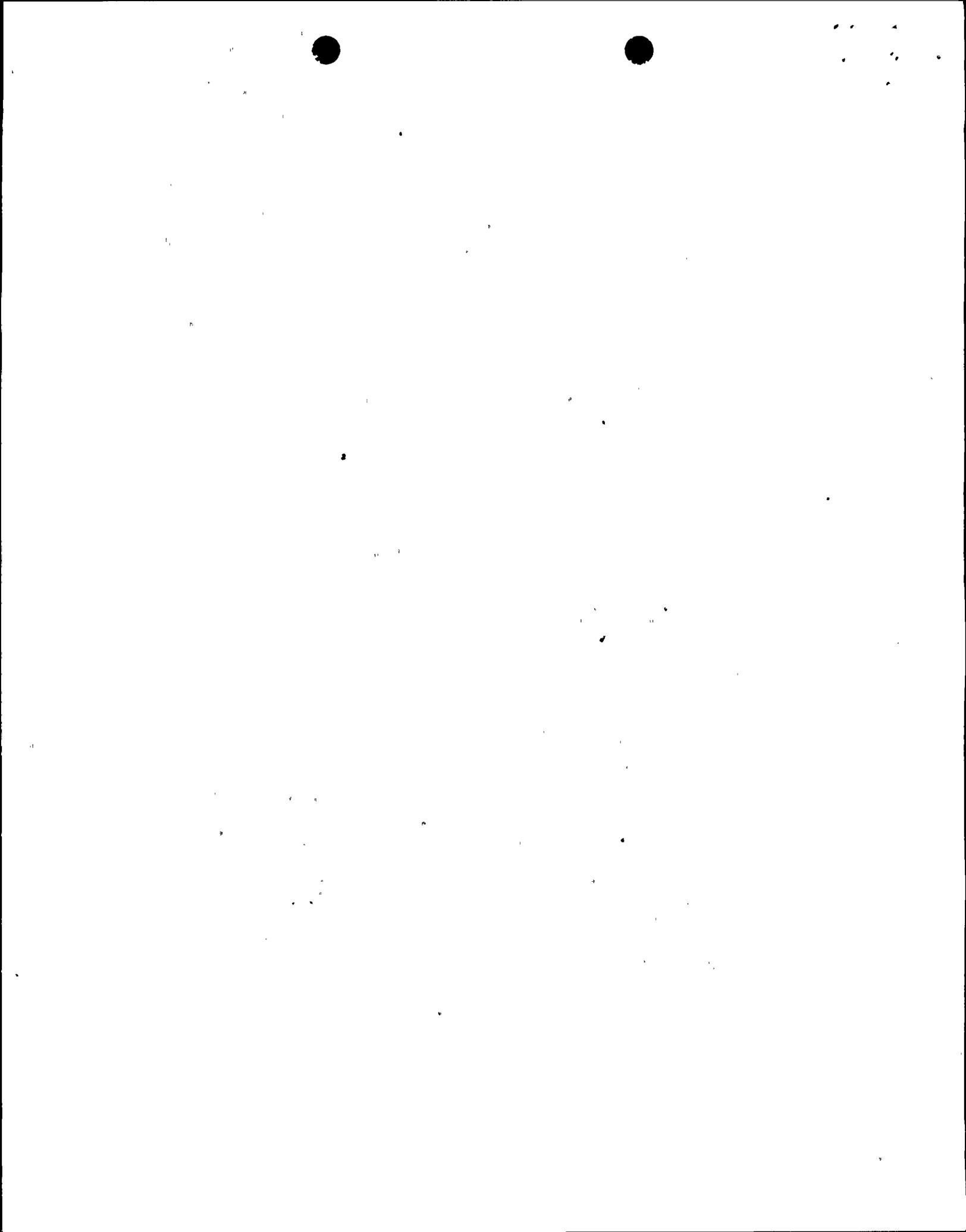
Hollywood, Bull. Seism. Soc. Am. 60, pp.1271-1289, 1970). In
that study, we compared the Fourier transforms of the records
obtained in the basement of the building with those in the free-
field. We found a marked reduction of the components of motion
with frequencies higher than 5 Hz. At 10 Hz, the amplitude of
motion of the basement was approximately half of that on the
free-field. We constructed a mathematical model of the build-
ing, foundation and soil, and found that the reduction in res-
ponse could be explained if we modelled the foundation as a
rigid semicylinder (axis along the length of the building) with
a radius of 26 ft. At the deepest point (26 ft) this rigid
foundation model was more embedded than the basement of the
structure (9 ft). However we felt that, particularly, at high
frequencies the presence of the piles would stiffen the soil
and that this effect could be represented by an equivalent deeper
foundation. The results obtained matched the observations and
suggested that the reduction of response was associated with
scattering by the embedded foundation and by the piles.

To confirm that the reduction was not due to the effects
of interaction proper, later, (Hradilek and Luco, Dynamic Soil-
Structure Interaction, Report, IDIEM, Univ. of Chile, 1970) we
conpared the response calculated assuming a flat foundation with
that for the embedded foundation. The comparison is shown in
Fig. 5-2 where the ratio of the Fourier transform of the motion
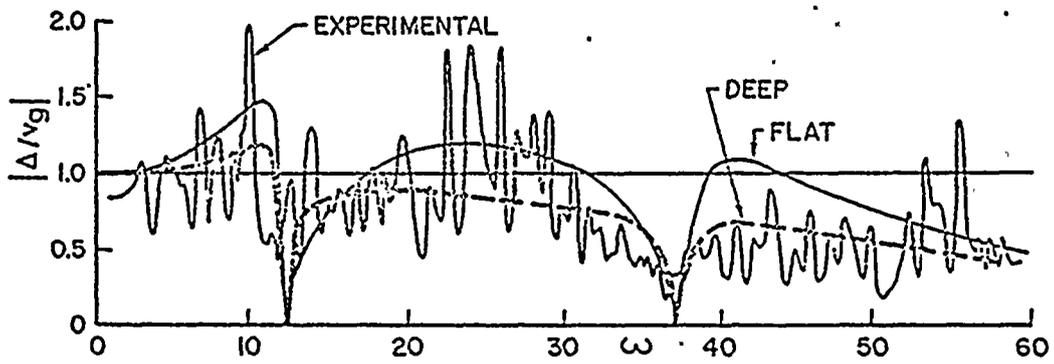of the base of the structure to the corresponding quantity in the

Figure 6.

Figure 5-3
(Luco)

free-field is shown versus frequency in radians per second
(1 rad/sec = 6.28 Hz). Three curves are shown in Fig. 5-3.
One curve corresponds to the recorded data, a second curve corres-
ponds to the flat foundation model, and a third to the model with
an embedded foundation. The results shown in Fig. 5-3 indicate
that, for low frequencies, the flat foundation model fits the
data, while, at high frequencies the embedded foundation model
is more adequate. This could be expected since the piles will
deform without much resistance to low frequency motion (long wave-
length) but will not be able to deform at high frequencies (short
wavelengths) and will act as almost rigid bodies. The conclusion
of this study was that soil-structure interaction proper was not
sufficient to produce the observed reduction and that the reduc-
tion was associated with scattering by the basement and piles.
The particular foundation model employed in these studies (rigid
semi-cylinder) is such that vertically or nonvertically incident
waves lead to exactly the same result. As a result, it was not
possible to discriminate the type of seismic excitation.

7. In the case of the Imperial County Service Building the
foundation is not embedded, is quite flexible and, again, it is
supported on piles. The density of piles, however, seems lower
that that at the HS building.

The behavior of a flexible pile embedded in the soil and
subjected to a vertically incident wave is such that for fre-
quencies below a certain critical value the response at the top

of the pile can be higher than that on the ground surface away from the pile. This increase in response is typically less than 20 percent. For frequencies higher than the critical frequency, the response at the top of the pile is lower than that on the ground surface away from the pile. The critical frequency depends on the stiffness of the pile relative to that of the soil and on the slenderness of the pile. For very flexible piles the critical frequency is very high. It is possible to speculate that the piles at ICSB were so flexible that the critical frequency was sufficiently high to have had amplification over a wide frequency range. I feel, however, that this amplification is not sufficient to explain the observations. Perhaps a combination of this effect with the effects of interaction proper could explain the marked increase in response. It will take considerable research work before the matter is settled.

VI.   Throughout the Licensing Board hearings, parties stressed the role of soil-structure interactions as a mechanism that would reduce the magnitude of structure motion relative to ground motion (e.g., Tr. 8878; 8947-46). Staff and applicant's arguments (in response to intervenors' suggestion of the apparent lack of tau effect during IV-79) point to soil structure interactions as the reason for building motion exceeding that of the ground (Blume Affidavit, Paragraph 10; Rothman - Kuo Affidavit, page 7). (a) Describe and explain the circumstances in which soil-structure interactions produce enhanced or reduced structural response.   (b) Discuss the relevance and applicability for such interactions to the seismic response assumed for Diablo Canyon.

COMMENTS IN RESPONSE TO QUESTION 6.

1. To discuss the effects of soil-structure interaction
on structural response it must be understood that the concept
"structural response" involves several aspects. In the case of
nuclear power plants, the elements of the structural response
of immediate concern are:

(i)   Absolute motion at levels where equipment will be
        mounted. Typically, the absolute accelerations
        developed at different points in the structure need
        to be determined and floor response spectra calculated.

(ii)   Relative displacements between structural elements or
        between adjacent structures. This aspect is required
        for the design of pipes and to avoid pounding.

(iii)   Internal forces or stresses in the different struc-
        tural elements.

(iv)   Base forces and overturning moments at foundation
        level. These elements are needed to study the overall
        stability of the structure resting on soil or rock.

Statements about the effects of soil-structure interaction on
"structural response" are useless or misleading unless they are
made specific. The following comments are addressed to aspects
(i) and (iii). I do not have enough information to comment on
aspects (ii) and (iv)..

It should be mentioned, that, for ordinary structures, the main concern has been with aspects (iii) and (iv) listed above. Typical design codes are based on unrealistically low inputs and unrealistically low strengths. They provide safety for ordinary structures because through experience a balance may be approached. Unfortunately, in the case of nuclear power plants this is not sufficient. At difference with the case of ordinary structures, realistic estimates of motion within a nuclear power plant must be obtained.

2. Effects of soil structure-interaction on motion of the structure. A first effect of soil-structure interaction is a reduction in the fundamental frequency of the system. The reduction in frequency depends, as a first apprximation, on the relative stiffness of the structure defined by the parameter $\omega_1 r/\beta$ (ω_1 = fixed-base natural frequency of the structure in radians per second, r = equivalent radius of the foundation, β = shear wave velocity at foundation level), on the relative mass of the structure defined by the parameter $M_1/\pi\rho_s r^2 H_1$ (M_1 = modal mass of structure which is approximately equal to 0.7 of the total mass, ρ_s = density of the soil, H_1 modal height approximately equal to 0.7 of the total height), and on slenderness of the super-structure defined by H_1/r. The reduction in frequency and, in general, the effects of interaction increase as the values of these parameters increase. The ratio of the frequency with inter-action $\tilde{\omega}_1$ to the fixed-base frequency (or frequency without inter-action) ω_1 is shown in Fig. 6-1. For the containment at Diablo Canyon the parameters indicated above have values of ~0.67, ~0.2,
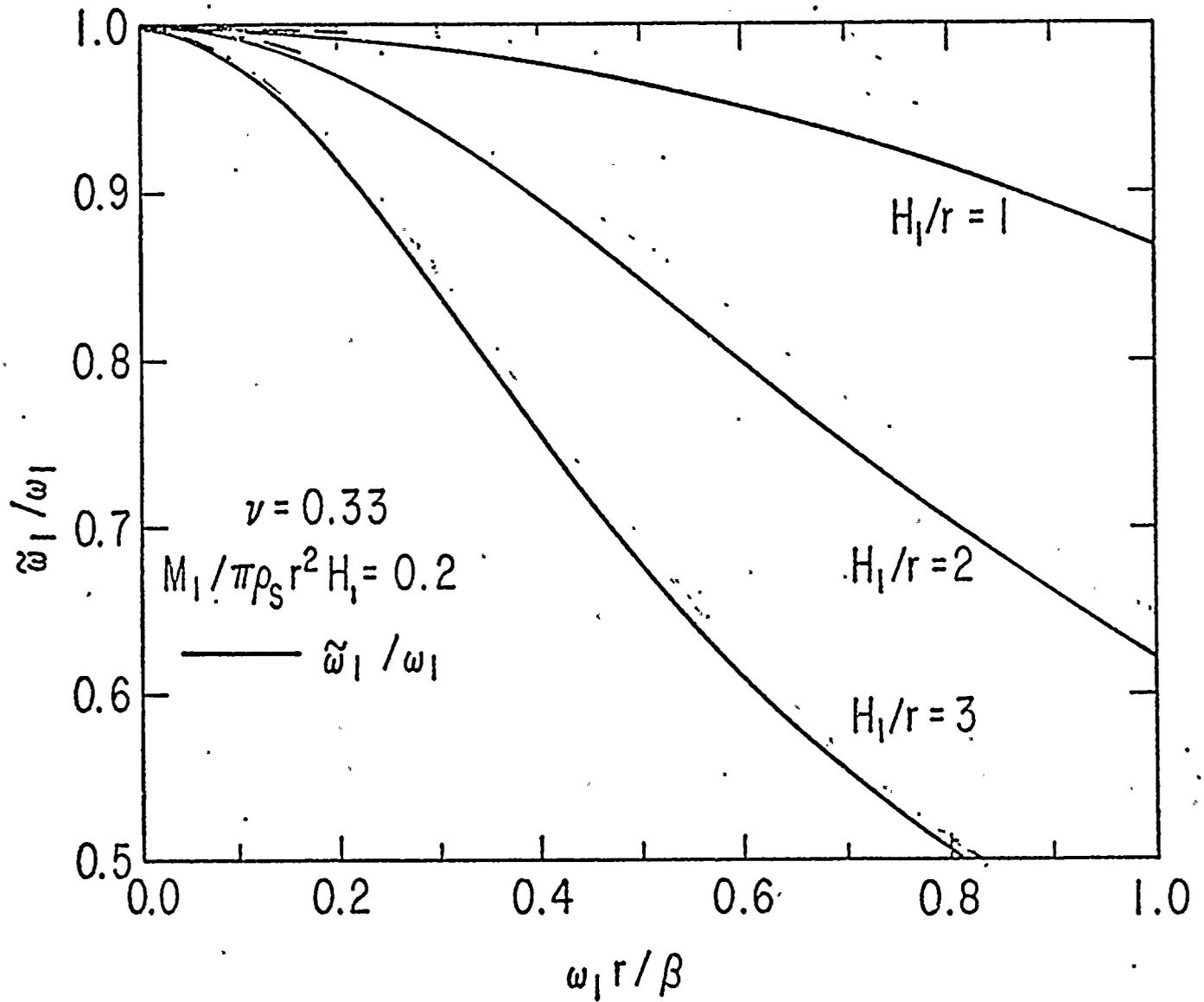
Axis labels: $\tilde{\omega}_1/\omega_1$ (vertical), $\omega_1 r/\beta$ (horizontal)

Curve labels: $H_1/r = 1$, $H_1/r = 2$, $H_1/r = 3$

$\nu = 0.33$

$M_1/\pi \rho_S r^2 H_1 = 0.2$

$\tilde{\omega}_1/\omega_1$

FIG.   System frequency   versus relative
        stiffness and slenderness ratio.

Figure 6-1
(Luco)

and ~2.0, respectively. Fig. 6-1 indicates that as a result of interaction, the frequency of the system will be approximately 18 percent lower than the fixed-base frequency.

A second important effect of interaction is the modification of the damping. This modification depends on the three parameters indicated above, on the damping of the superstructure (ξ_1) and on the material damping in the soil or rock (ξ_s). Fig. 6-2 illustrates the variation of the system damping $\tilde{\xi}_1$ (with interaction) with the relative stiffness $\omega_1 r/\beta$ and with the damping in the soil ξ_s. Fig. 6-2 has been drawn for a damping of five percent in the structure ($\xi_1 = 0.05$). Since Diablo Canyon is supported on rock, ξ_s will have a value of the order of 3% and Fig. 6-2 indicates that the system damping would be similar to the damping in the structure. For Diablo Canyon a damping in the structure of 7% has been used. In this case, the system damping will be lower (~6%).

Finally, a third effect of soil-structure interaction is the introduction of rocking (rotation about an horizontal axis) of the foundation. For frequencies lower than the system frequency, the rocking of the foundation is in phase with the excitation, and, consequently, the low frequency components of motion at elevated points on the structure are amplified. For higher frequencies, rocking is out of phase with the excitation, and, the high frequency components of motion at elevated levels are reduced. This effect as well as the reduction in resonant frequency, are illustrated in Fig. 6-3 taken from the work of Seed and Lysmer for
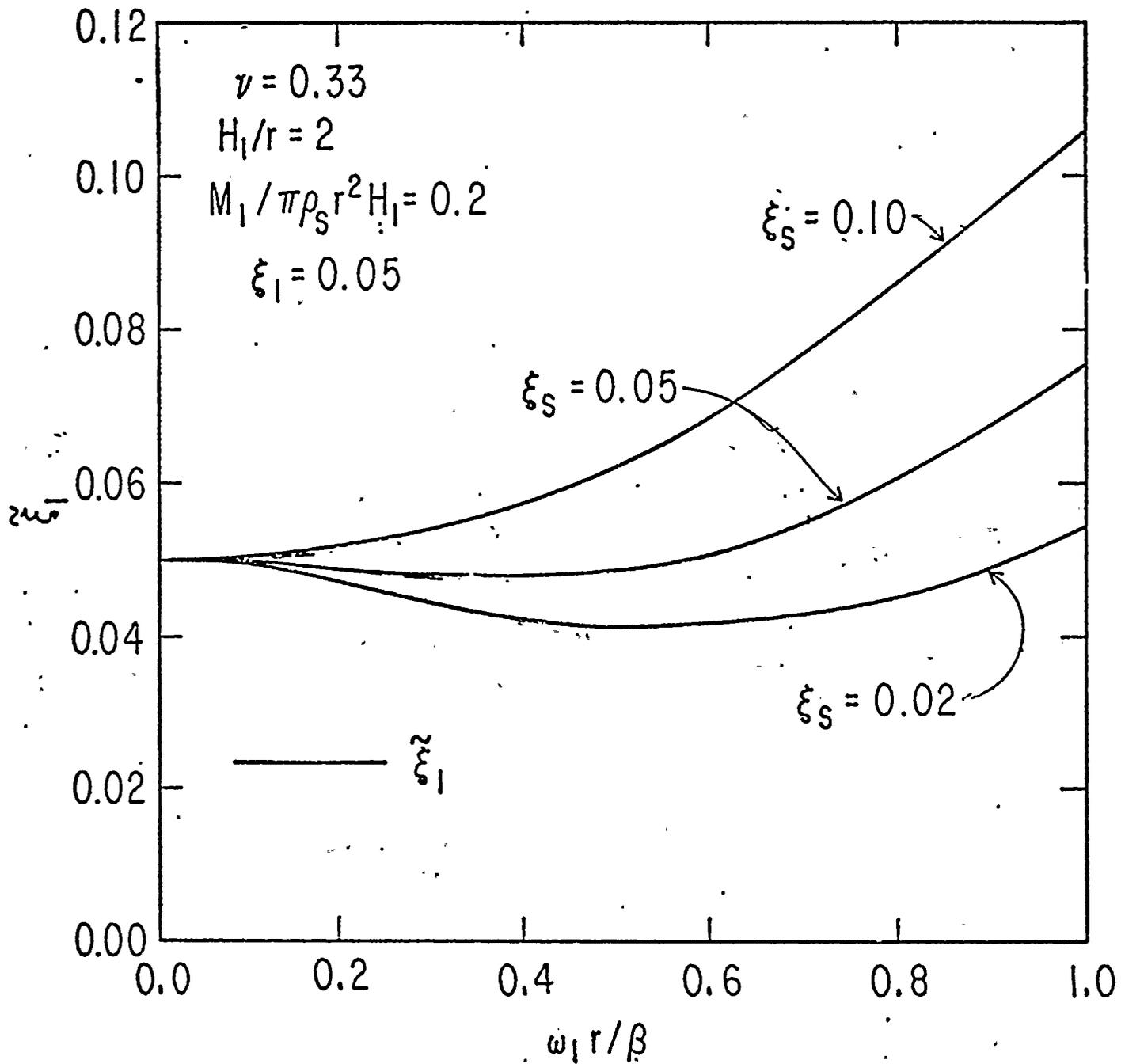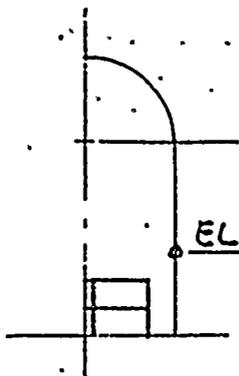
FIG.   Effect of the relative stiffness and of the material soil damping on the overall system damping

Figure 6-2
(Luco)

CONTAINMENT EXTERIOR
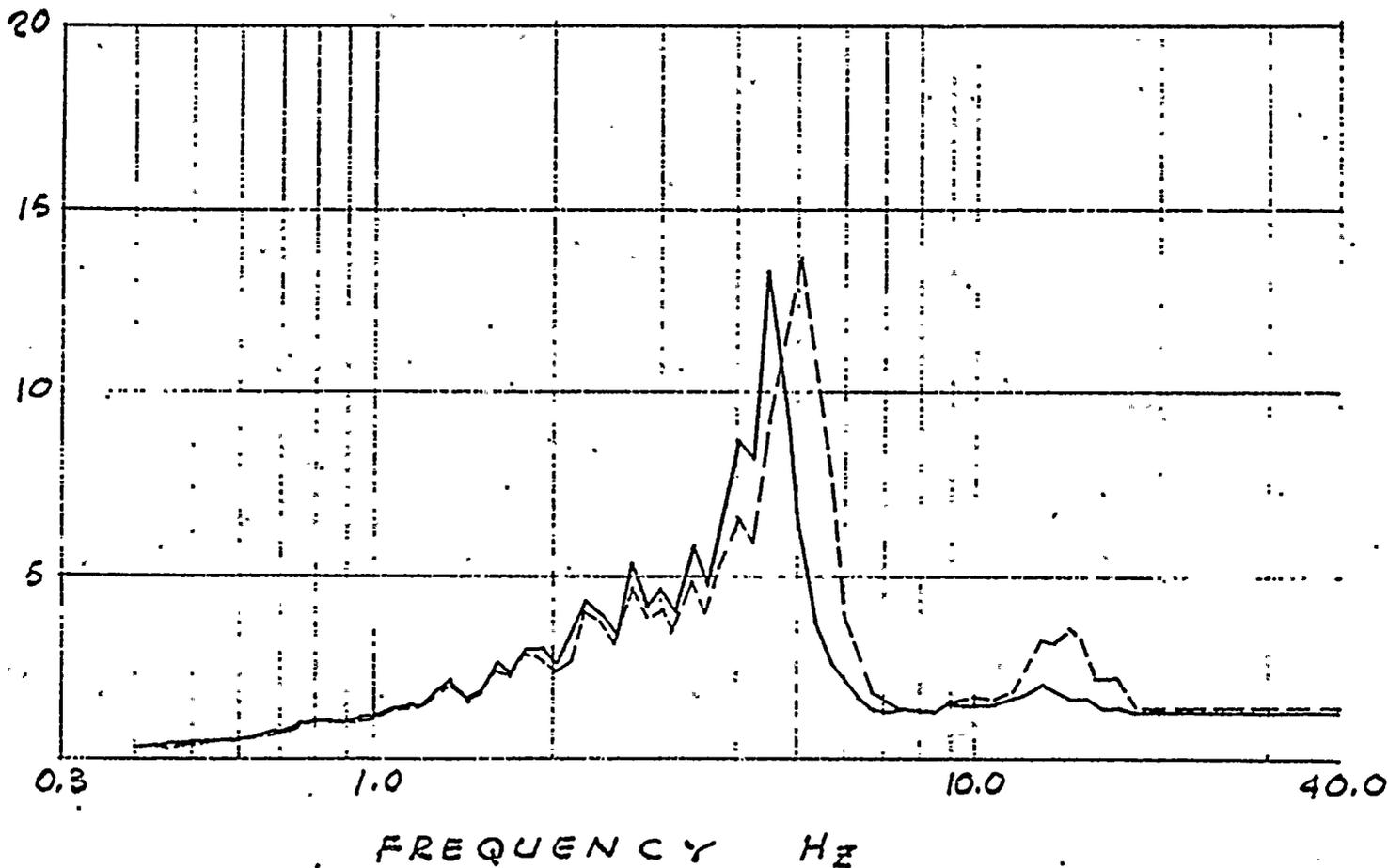FLOOR RESPONSE SPECTRA
ELEVATION 155.25'
DAMPING = 2 %
———————— SOIL STRUCTURE INTERACTION
-------- RIGID BASE

EL. 155.25'

DIABLO CANYON
NUCLEAR POWER PLANT

(September 1977)

Figure 6-3 (Luco)
FIGURE NO. 3B-6

6-7

ABSOLUTE ACCELERATION (g)

FREQUENCY Hz

Diablo Canyon (Appendix D-LL 3B, FSAR, Amendment 53).

The effect of soil-structure interaction on the translational response at foundation level is similar to the effects already described for points at higher elevations, i.e. amplification of low frequency components and reduction of high frequency components. One difference, at foundation level, is that at high frequencies, i.e. where the spectrum tends to the peak acceleration, the motion may be slightly higher or lower than in the free-field. This occurs for surface or shallowly embedded foundations. For deeply embedded foundations, the peak acceleration is typically considerably lower than on the ground surface. These effects on the translational response at the base of the structure are illustrated in Fig. 6-4 taken from Seed's testimony (Fig. VI-1). For frequencies lower than the fundamental system frequency (~4 Hz), the response with interaction is equal or slightly above the results for a rigid soil. For frequencies in the range from 4 to 20 Hz, the response with interaction is lower than in the free-field. Finally, for frequencies above 20 Hz the response with interaction is essentially equal to the free-field motion.

When soil-structure interaction is considered rocking of the foundation occurs. One of the effects of rocking is to induce vertical components of motion along the perimeter of the foundation. These vertical components of motion are not included in a rigid soil analysis. Also, if the structure is not symmetric, or, if the seismic excitation corresponds to nonvertically incident waves, torsional response is induced.
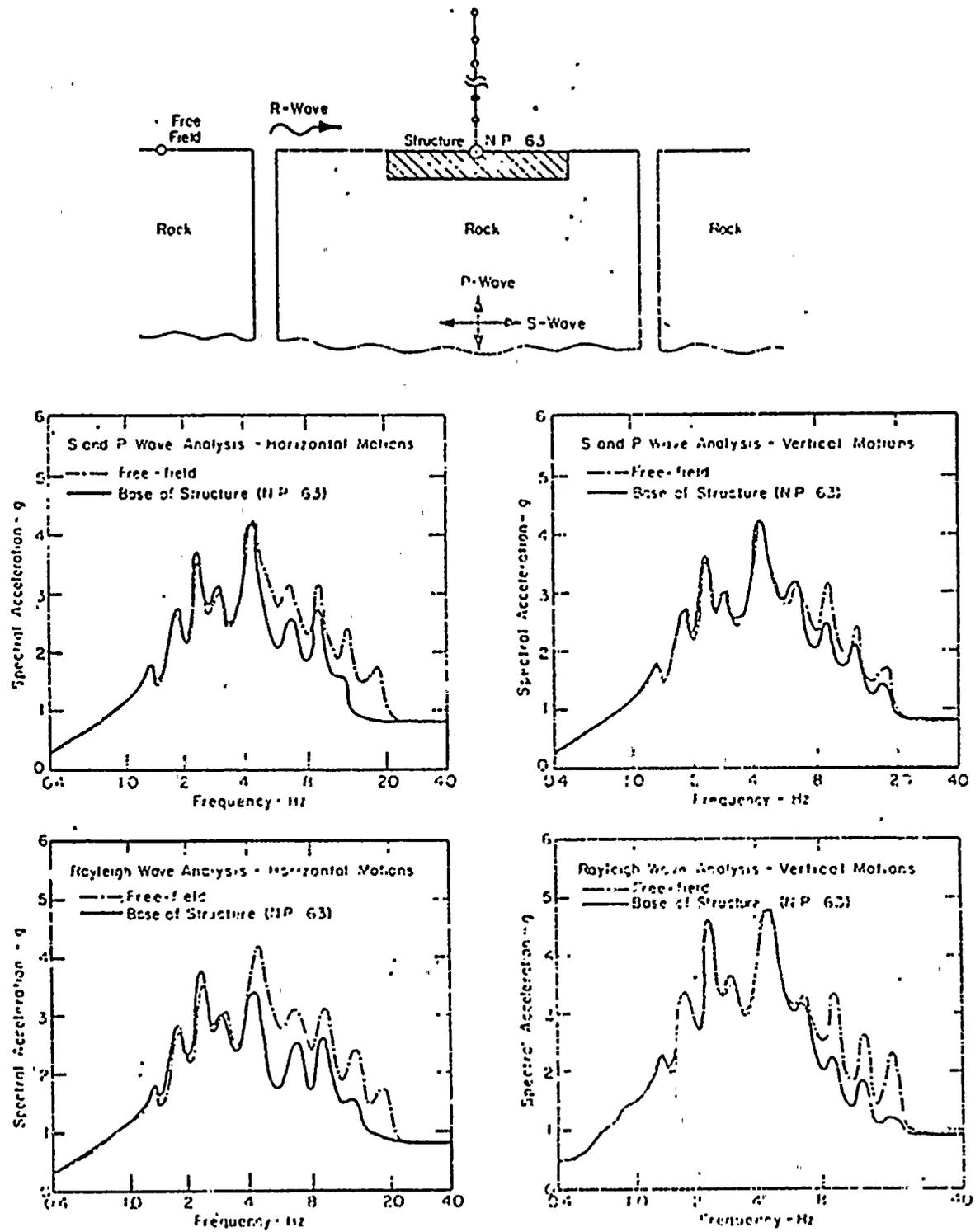
Fig. 13 SOIL-STRUCTURE INTERACTION EFFECTS AT BASE
OF STRUCTURE
(Taken from Seed and Lysmer report on "Analyses of Soil-
Structure Interaction Effects During Earthquakes for
the Diablo Canyon Nuclear Power Station")

Seed FIG. VI-1   Figure 6-4
                                                (Luco)

Finally, in a rigid soil analysis the response of the containment shell and of the internal structure are conducted separately and no coupling exists. If soil-structure interaction is considered, the response of both structures is coupled through the foundation.

The discussion above indicates that soil-structure interaction increases certain components of motion while reducing others.

3. Effects of soil-structure interaction on internal forces. Analysis of the interaction problem reveals that and approximation to the ratio of the internal forces in the structure with interaction to the corresponding forces without interaction is given by $S_a(\tilde{\omega}_1, \tilde{\xi}_1)/S_a(\omega_1, \xi_1)$. In this ratio, $S_a(\tilde{\omega}_1, \tilde{\xi}_1)$ corresponds to the value of the absolute acceleration response spectrum at the system frequency $\tilde{\omega}_1$ and for the damping ratio $\tilde{\xi}_1$ (both terms including interaction). The term $S_a(\omega_1, \xi_1)$ represents the corresponding spectral ordinate at the fixed-base fundamental frequency ω_1 and for the structural damping ratio ξ_1. For the containment building at Diablo Canyon, $\omega_1/2\pi \sim 5$ Hz, $\tilde{\omega}_1/2\pi \sim 4$ Hz, $\xi_1 = 0.07$ and $\tilde{\xi}_1 \sim 0.06$. Since the Newmark response acceleration response spectrum is flat for frequencies in the range from 2 to 10 Hz, the ratio $S_a(\tilde{\omega}_1, \tilde{\xi}_1)/S_a(\omega_1, \xi_1)$ would be equal to one except for the fact that the system damping is slightly lower than that for the structure. From this simplified calculation, we would expect the internal forces in the containment
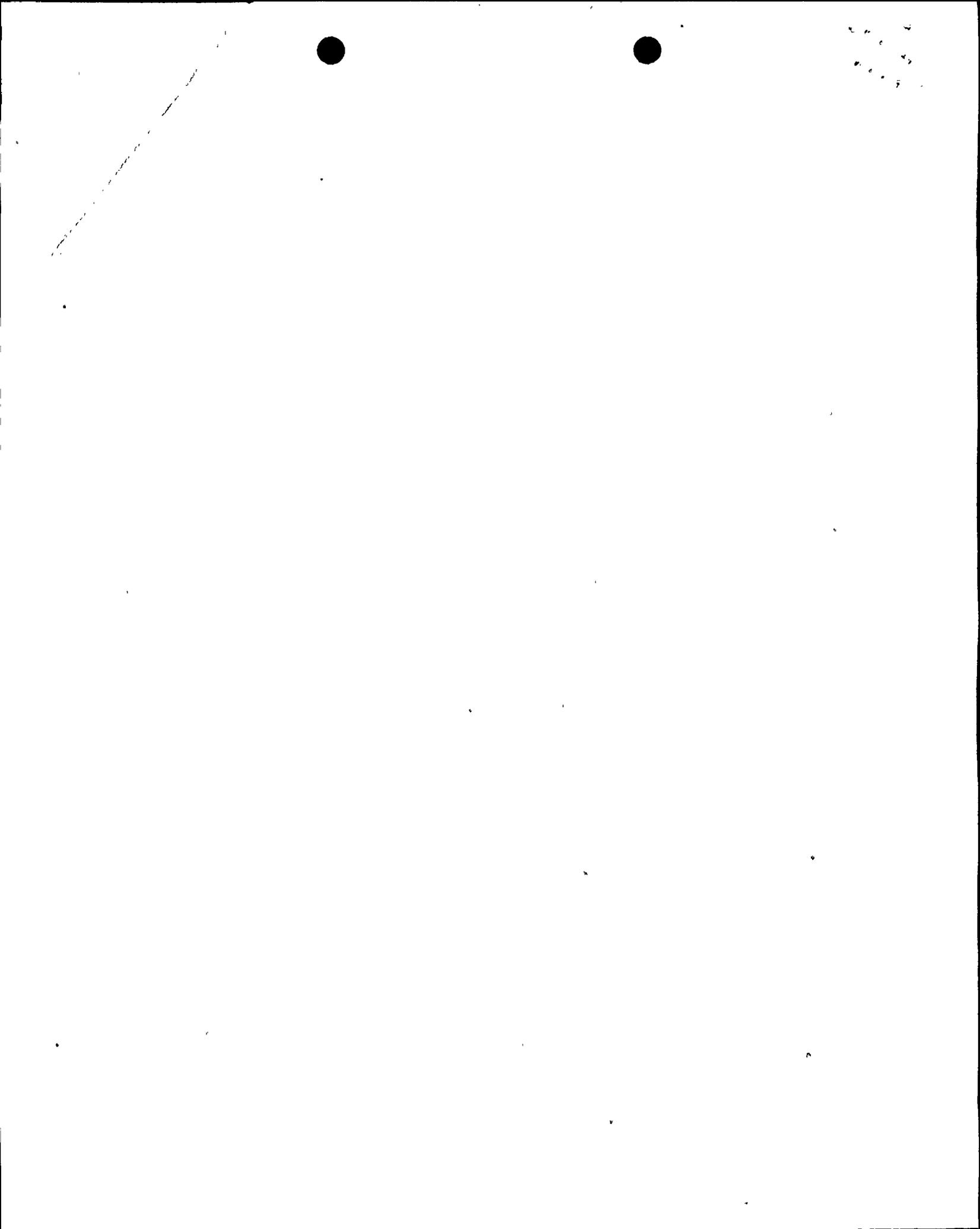
building to be slightly higher when soil-structure interaction
is considered. This conclusion is confirmed by the results of
Seed and Lysmer (Appendix DLL-3B) reproduced in Fig. 6-5. The
results in Fig. 6-5 show that the shear forces on the contain-
ment exterior shell are slightly higher when soil-structure
interaction is included. Similar results are obtained for the
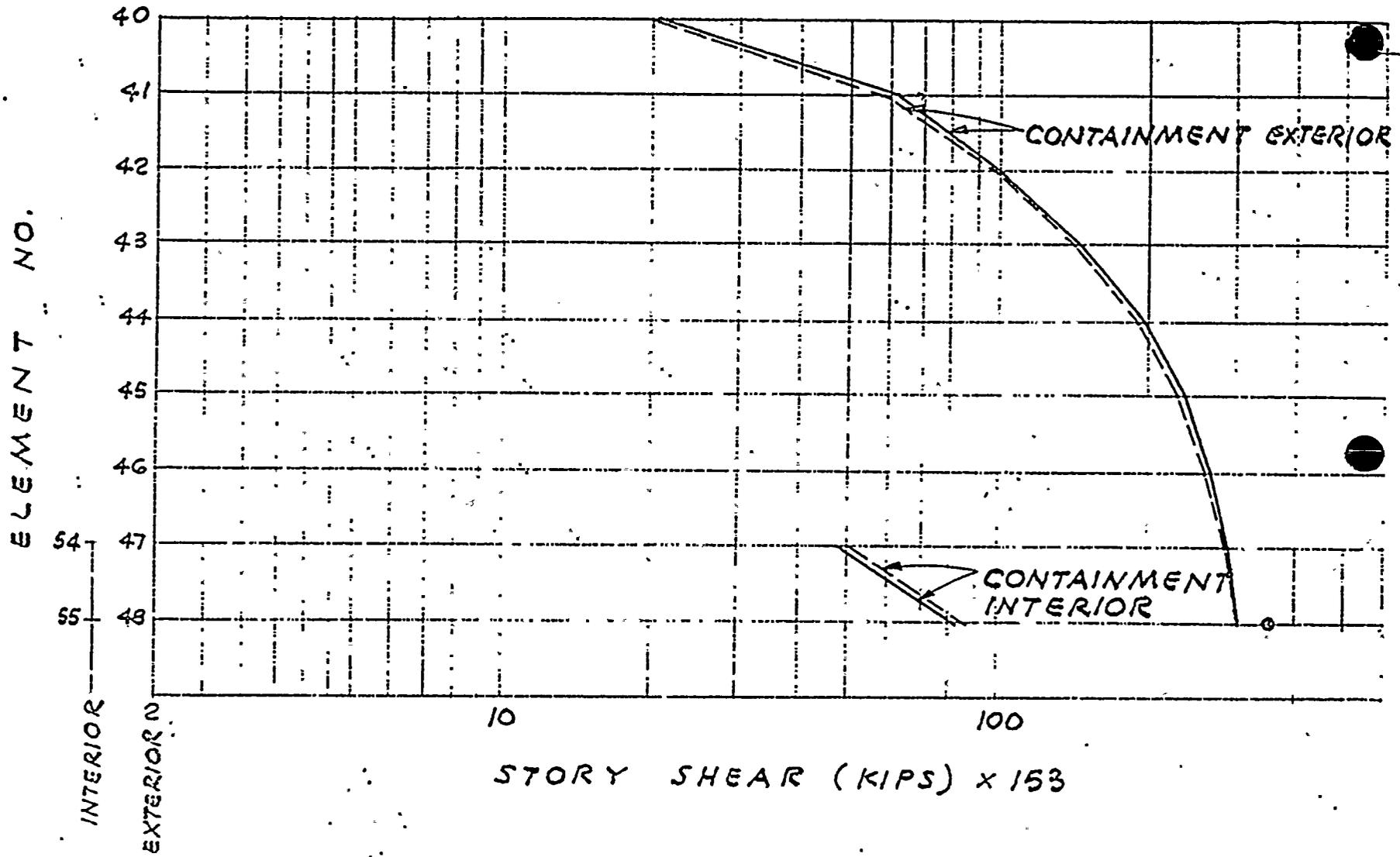flexural moments.

Clearly, general claims of the conservatism of not includ-
ing soil-structure interaction are not warranted.

For cases in which the relative stiffness of the structure
is significantly higher than that of the soil, i.e. $\omega_1 r/\beta > 1$, or,
when material damping in the soil is higher than that on the
structure, the ratio $S_a(\tilde{\omega}_1, \tilde{\xi}_1)/S_a(\omega_1, \xi_1)$ may be less than one.
In these cases the effects of soil-structure interaction may lead to
internal forces lower than those calculated on the basis of a
rigid soil assumption.

CONTAINMENT EXTERIOR & INTERIOR
MAXIMUM STORY SHEAR
———— SOIL STRUCTURE INTERACTION
— — — RIGID BASE

STORY SHEAR (KIPS) × 153

ELEMENT NO.

DIABLO CANYON
NUCLEAR POWER PLANT

(September 1977)

Figure 6-5 (Luco)

FIGURE NO. 3B-3

6-12

Amendment 53