TENNESSEE VALLEY AUTHORITY

DIVISION OF ENGINEERING DESIGN
CIVIL ENGINEERING BRANCH

JUSTIFICATION OF THE SEISMIC DESIGN CRITERIA USED FOR THE SEQUOYAH, WATTS BAR, AND BELLEFONTE NUCLEAR POWER PLANTS

PHASE II

RESPONSES TO NRC QUESTIONS 1 THROUGH 6

JUNE 1979

CONTENTS

1.0 INTRODUCTION

The Tennessee Valley Authority (TVA) has applied to the Nuclear Regulatory Commission (NRC) for licenses to operate Sequoyah, Watts Bar, and Bellefonte nuclear power plants. In the course of their review for the operating licenses for these facilities, NRC has requested additional information concerning the seismic design basis used for these plants. This report, along with other TVA reports, provides additional information to support the safe shutdown earthquake (SSE) ground motions used in the design of the Sequoyah, Watts Bar, and Bellefonte nuclear plants as discussed in the respective plant's Final Safety Analysis Reports (FSAR). In particular, this report (1) responds to questions the NRC staff ask of our Phase II Report, (2) includes supplements issued to but not included in our Phase II Report, (3) provides additional information to support the present design SSE, (4) summarizes the studies undertaken to resolve this issue, and (5) lists TVA's conclusions. We strongly believe this information along with our previous reports clearly demonstrates that the seismic design bases used at these facilities are conservative.

2.0 RESPONSES TO NRC QUESTIONS 1 THROUGH 6

Our Phase I and Phase II Reports were submitted to the NRC in May and August 1978. The NRC Staff ask no questions of the Phase I Report and ask six questions of the Phase II Report. These questions were issued October 4, 1978, and are contained in Appendix A - Response to NRC Phase II Questions. On November 9, 1978, we met with the staff to discuss their questions and our approach to answering them. As a result of this meeting the staff subsequently issued nine clarifications to the original six questions on November 24, 1978. These nine clarifications are also contained in appendix A. TVA responses to the six questions were submitted informally on December 15, 1979. The responses are now formally submitted and are in appendix A.

3.0 SUPPLEMENTS ISSUED TO PHASE II REPORT

Our Phase II Report was submitted to the NRC in August 1978. TVA and NRC staff also met in August to discuss the report. As a result of these discussions TVA issued a "loose" supplement to the Phase II Report concerning the 5.8 m_{bLg} magnitude assigned to the Giles County earthquake of May 31, 1897. This supplement is contained in Appendix B - Supplements to Phase II Report. Also in appendix B is an errata sheet for the Phase II Report.

4.0 ADDITIONAL INFORMATION TO SUPPORT THE PRESENT DESIGN SSE

The results of four additional investigations are submitted to support the present design SSE used at the three plants. These investigations are summarized below.

4.1 Justification of the Suite of Earthquakes Used in the TVA Strong Motion Analyses.

This work is contained in appendix C. This investigation is concerned primarily with documenting the use of M_L instead of m_{bLg} as the magnitude measure used to select the suite of earthquakes between the specified magnitude range of 5.3 to 6.3.

4.2 The Actual M_L and m_b Magnitudes and the Effect of Variations in the Suite of Earthquakes.

This work is contained in appendix D. This investigation is concerned primarily with delineating the actual M_L and m_b values of the suite of earthquakes and the impact on the statistical measures of magnitude and peak ground acceleration if events are deleted from, replaced in, and added to the suite of earthquakes used.

4.3 Determination of Site Specific Response Characteristics.

This work is reported in Reference 1- Earthquake Ground Motion Study in the Vicinity of the Sequoyah Nuclear Power Plant. The report describes an earthquake ground motion study conducted using portable seismographic instruments at six competent rock sites located at and in the vicinity of the Sequoyah facility.

The report illustrates a large range of variations in the site response of six competent rock sites located in a small area of southeastern Tennessee to regional and distant seismic inputs. This variation in crustal response of rock sites, coupled with the additional ground motion amplification of an overlying soil layer, can account for the large scatter of reported intensities within a restricted epicentral region. Typically, in eastern United States, earthquakes are often characterized by the highest intensity reported, regardless of the fact that this intensity may be reported in only one or few instances and that a lower intensity level clearly prevailed in the epicentral region.

When such a characterization is applied to the design earthquake used in safety analysis, a large amount of conservatism is imposed. All sites

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When such a characterization is applied to the design earthquake used in safety analysis, a large amount of conservatism is imposed. All sites serve to locate and encompass present-day sources of seismicity within the study area. We further believe that the conclusions reached in this report do not indicate any structural continuity in the southern Appalachian region that would warrant migration of the 1897 Giles County, Virginia earthquake to the Sequoyah, Watts Bar, or Bellefonte nuclear plants.

The results of this study strongly suggest the existence of an east-west trending tectonic structural zone (tectonic structure as defined by appendix A) with which the 1897 Giles County, Virginia earthquake was associated and to which a recurrence of an event of this magnitude would be restricted.

It is furthermore felt that the existence of a long northeast-trending lineament transected by three northwest-trending lineaments, as defined by multiple sources of data, serves to develop eight tectonic subdivisions (tectonic provinces as defined by appendix A) having different lithologic, structural, or seismic characteristics. As such, the previously imposed "classical" interpretation that Giles County, Virginia, and the Sequoyah, Watts Bar, and Bellefonte nuclear plants all lie within the same Southern Valley and Ridge Tectonic Province is not warranted.

5.0 Summary of Studies Performed by TVA.

TVA investigated or performed thirteen major studies to address and attempt to resolve the NRC concerns. These thirteen studies are listed in table 1. These studies combine topics from TVA's initial outline, suggestions by the NRC Working Group, and discussions with the NRC staff. In table 1, the numbers in parentheses refer to corresponding sections of the Working Group Report (WGR) where these items are discussed. Our Phase I Report (reference 3) discusses items 1 through 5. Our Phase II Report (reference 4) discusses items 6, 7, and 8. Item 7 is also discussed in our response to question 3. Item 8 is developed in detail in reference 5. Items 9, 10, and 11 are discussed in responses to questions 4, 5, and 6, respectively. Item 12 is summarized in section 4.3 and discussed in reference 1. Item 13 is summarized in section 4.4 and described in detail in reference 2.

The major conclusions for each of these items are summarized below.

1. Evaluation of Giles County Earthquake. After considerable study of this event, it is TVA's opinion the May 31, 1897, Giles County earthquake is best characterized as a MM VII-VIII.
2. Evaluation of Site Conditions on Earthquake Intensity. During a given earthquake, intensities on rock are less than on soil by two to three intensity units. The intensity rating for the 1897 Giles County earthquake is soil biased, inferring that for the same earthquake the intensity on rock would be less.
3. Evaluation of Acceleration Variation with Depth. Earthquake accelerations reduce with depth. The subject plants are all founded on rock at depth.
4. Comparison of Acceleration Recorded on Rock and Soil During a Given Earthquake at a Given Site. Accelerations on rock are less than on soil at a given site during a given earthquake. The subject plants are all founded on rock.

5. Evaluation of Intensity Acceleration Relationship. TVA considers the Murphy-O'Brien intensity-acceleration relationship as the most appropriate.

6. Evaluation of Response Spectra Based on Intensity. This approach is not possible in this case due to lack of data for the intensity level and site conditions of interest.

7. Development of Response Spectra Based on Site Specific Records. Response spectra were developed from twenty-six magnitude, distance, and site specific records. Both actual and normalized response spectra were determined. From these data the following site specific design spectra may be determined:

   a. A Regulatory Guide 1.60 spectrum anchored to the site specific 50th percentile peak ground acceleration (0.10g) determined from the twenty-six records.

   b. The actual 50th and 84th percentile site specific spectra (based on the actual or unnormalized spectra).

   c. The normalized 50th and 84th percentile site specific spectra (based on the normalized spectra).

   The various site specific spectra are compared with the Sequoyah and Phipps Bend design spectra for steel and reinforced concrete structures in figures 1 and 2. These figures show the Sequoyah design spectra are exceeded by only the actual 84th percentile site specific spectra and only over a limited frequency range.

8. Development of Response Spectra Based on Magnitude. On the basis of this method, a top of rock acceleration of 0.08 g is predicted for an earthquake similar to the Giles county event. Anchoring a Regulatory Guide 1.60 spectra to 0.08 g show that the design spectra at the three plants are not exceeded.

9. Calculation of the Probability of Exceedance for Various Response Spectra. The probabilities of exceedance were calculated for various models. The relative differences of probabilities between the various response spectra are less than one order of magnitude for the same models. The probabilities of exceedance vary with the different models and ranges from 10^{-3} to 10^{-5}.

10. Evaluation of the OBE. The return periods for the OBE acceleration levels at the three plants vary from 300 to 1500 years.

11. Additional Probability Studies. These studies are described in our response to question 6. In this response TVA elected to perform various probability studies in addition to the method outlined by the NRC staff. These studies are incorporated in item 9 above.

12. Determination of Site Specific Response Characteristics. These characteristics were determined from field instrumentation. These data show that Sequoyah is a relatively quiet site (as compared to the others surveyed) and has low response characteristics.

13. Southern Appalachian Tectonic Study. This is a regional geophysical-geological study of the southern Appalachian region. The conclusions reached in the report do not indicate any structural continuity in the southern Appalachian region that would warrant migration of the 1897 Giles County, Virginia earthquake to the Sequoyah, Watts Bar, or Bellefonte Nuclear Plants.

6.0 CONCLUSIONS

The seismic design criteria of the Sequoyah, Watts Bar, and Bellefonte nuclear power plants are based on the "tectonic province" approach in

REFERENCES

1. Earthquake Ground Motion Study in the Vicinity of the Sequoyah Nuclear Power Plant, Weston Geophysical Corporation, Boston, Massachusetts, February 1979.

2. Southern Appalachian Tectonic Study, TVA, January 1979.

3. Justification of the Seismic Design Criteria Used for the Sequoyah, Watts Bar, and Bellefonte Nuclear Power Plants. Phase I, TVA, April 1978.

4. Justification of the Seismic Design Criteria Used for the Sequoyah, Watts Bar, and Bellefonte Nuclear Power Plants. Phase II, TVA, August 1978.

5. Prediction of Strong Motions for Eastern North American on the Basis of Magnitude, Weston Geophysical Corporation, Boston, Massachusetts, August 1978.

TABLES

TABLE 1
STUDIES PERFORMED BY TVA

1. EVALUATION OF GILES COUNTY EARTHQUAKE. (WGR-III.A.3)
2. EVALUATION OF SITE CONDITIONS ON EARTHQUAKE INTENSITY. (WGR-III.A.4)
3. EVALUATION OF ACCELERATION VARIATION WITH DEPTH.
4. COMPARISON OF ACCELERATIONS RECORDED ON ROCK AND SOIL DURING A GIVEN EARTHQUAKE AT A GIVEN SITE. (WGR-III.A.4)
5. EVALUATION OF INTENSITY - ACCELERATION RELATIONSHIPS. (WGR-III.B.3)
6. EVALUATION OF RESPONSE SPECTRA BASED ON INTENSITY (WGR-III.B.2)
7. DEVELOPMENT OF RESPONSE SPECTRA BASED ON SITE SPECIFIC RECORDS. (WGR-III.B.1 & III C.1.a)
8. DEVELOPMENT OF RESPONSE SPECTRA BASED ON MAGNITUDE. (WGR-III.B.6)
9. CALCULATION OF THE PROBABILITY OF EXCEEDANCE FOR VARIOUS RESPONSE SPECTRA. (WGR-III.E.1, III.E.2, & III.E.3)
10. EVALUATION OF THE OBE. (WGR-III.D)

ADDITIONAL STUDIES BY TVA

11. ADDITIONAL PROBABILITY STUDIES
12. DETERMINATION OF SITE SPECIFIC RESPONSE CHARACTERISTICS. (WGR-III.A.4)
13. SOUTHERN APPALACHIAN TECTONIC STUDY. (WGR-II.A, III.A.1, & III.A.2)

FIGURES

COMPARISON OF SEQUOYAH AND PHIPPS BEND DESIGN SPECTRA FOR STEEL STRUCTURES WITH VARIOUS SITE SPECIFIC SPECTRA

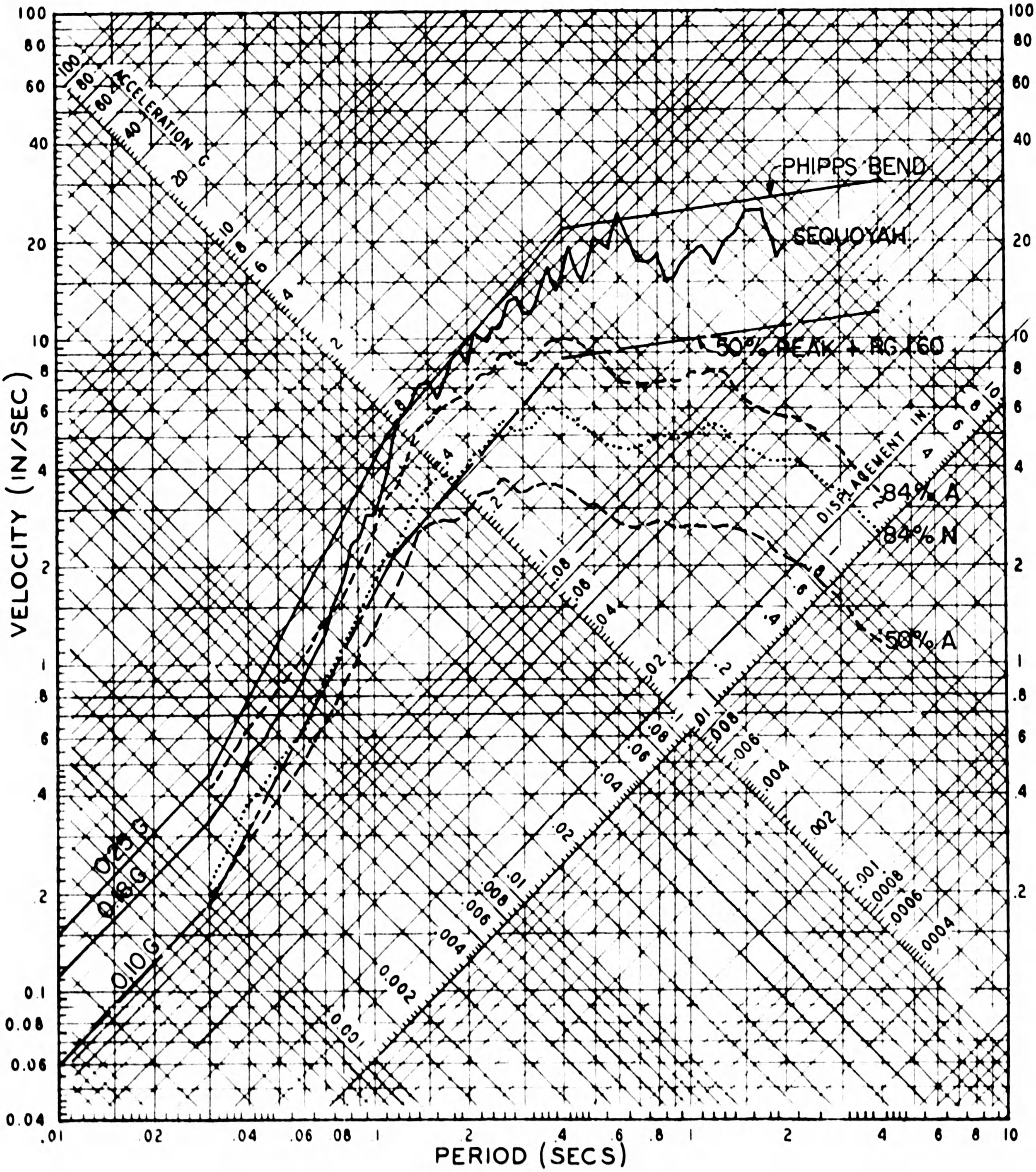


FIGURE 1

COMPARISON OF SEQUOYAH AND PHIPPS BEND DESIGN SPECTRA FOR REINFORCED CONCRETE STRUCTURES WITH VARIOUS SITE SPECIFIC SPECTRA

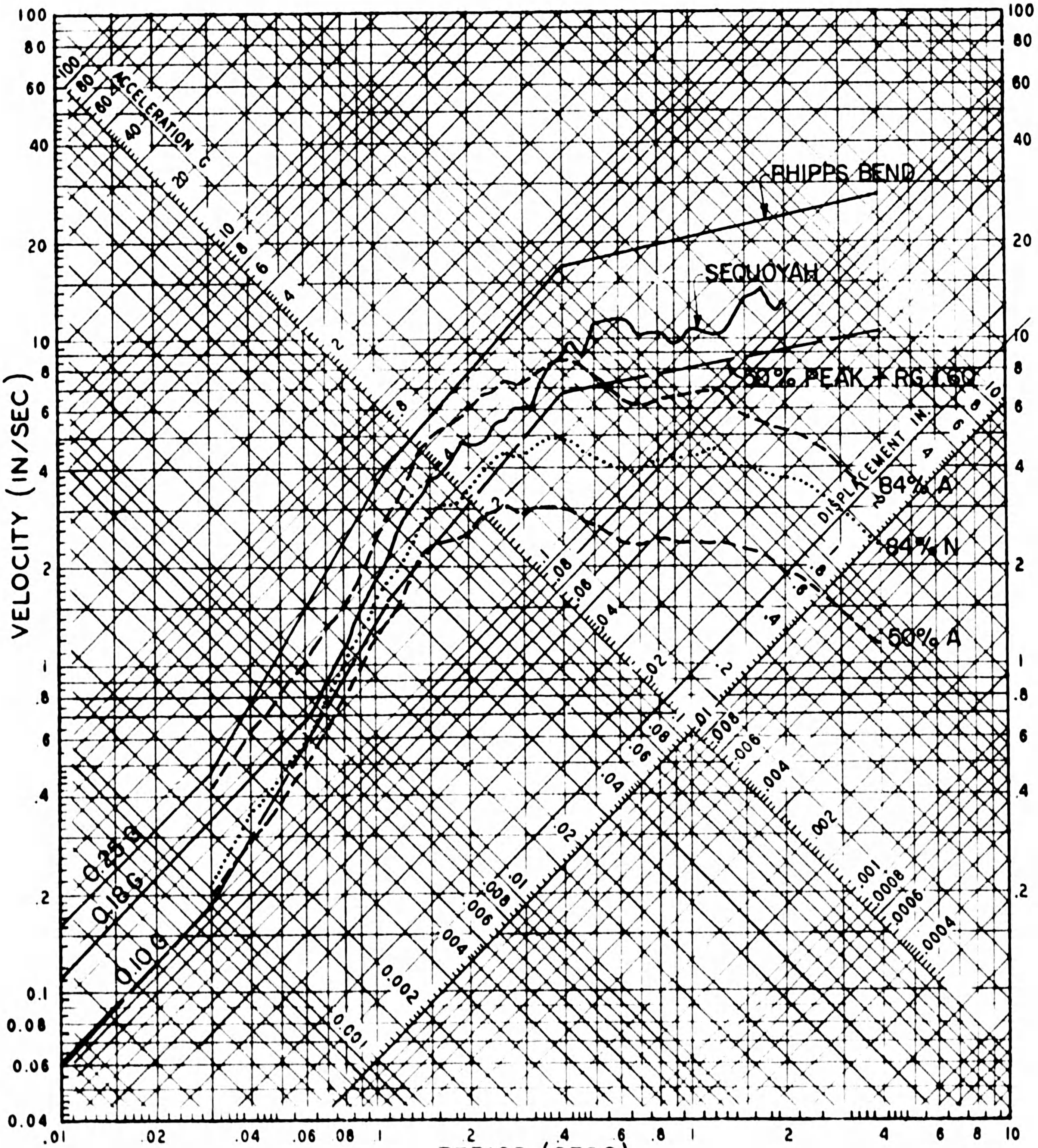


FIGURE 2

APPENDIX A

RESPONSE TO NRC PHASE II QUESTIONS

NRC'S SIX PHASE II QUESTIONS

SEP 29 1978

Seismic Design of
Sequoyah, Watts Bar, and Bellefonte

1. The reference (Muzzi and Vallini) used as your source for epicentral data and instrumental locations for the Friuli earthquakes indicates that there were 3 other sets of strong motion recordings on rock (at Somplago and Tolmezzo) at close distances for the magnitude 6.1 and 6.0 events of September 15, 1976. Figures 37 and 40 of that reference show that while these recordings had "disappeared traces" they also had peak accelerations greater than those recorded at S. Rocco for the same events. Discuss the validity of these measurements and their bearing upon the present study; include any discussions you have had with Muzzi and Vallini regarding this point.

2. The Earthquake Evaluation Study of the Auburn Dam area (Woodward-Clyde, 1977) utilizes the Koyna Dam records of the magnitude 6.0 (mb) Koyna event of December 11, 1967 in estimating nearby motion at rock sites. Although the recordings were made at about mid-height on the dam, investigators (Chopra and Chakrabarti, 1973; Guha and others 1971) have apparently judged that they represent the free field motion at the site. The U. S. Geological Survey in their review of Auburn Dam study has also taken this assumption to be valid. Discuss these records and those of associated events at Koyna and their bearing on the present study.

3. Determine whether the distribution of response spectral values at each frequency for the available data is better fit by a normal or log-normal distribution. Calculate 50th and 84th percentile spectra for the appropriate distribution.

4. Determine to what percentile different frequency bands of the design spectra correspond to with respect to the distribution calculated in 3. Discuss the significance of each such frequency band to the engineering analysis of structural components.
5. Perform a probabilistic analysis to determine whether the OBE meets the criteria of being an earthquake which could reasonably be expected to affect the plant site during the operating life of the plant.
6. Compare the probabilities of exceedence of the SSE design response spectra, the 50th percentile site specific response spectra, and the 84th percentile site specific response spectra at the subject plant with the SSE response spectra at other TVA plants which meet the Standard Review Plan criteria (Phipps Bend, Yellow Creek and Hartsville). If this analysis is done characterizing an earthquake of a given size using a single parameter (such as peak acceleration) it should be verified that the parameter compared accurately represents the spectrum in the frequency band of interest to plant structures. For example, the ratio of the SSE spectra at Phipps Bend to that at Sequoyah is greater than that implied by the ratio of peak accelerations.

SEP 29 1978

References

Chopra, A. K., and Chakrabarti, P., 1973, The Koyna Earthquake and the damage to Koyna dam: Bulletin of the Seismological Society of America, v. 63, p. 381-397.

Guha, S. K., Agarwal, S.P., Agarwal, V. P., Gosavi, P. D., and Marwadi, S. C. 1971, Analyses of accelerograms and structural response; Journal of Engineering Mechanics Division, Proceedings of American Society of Civil Engineers, v. 97, no. 2, p. 589-597.

Muzzi, F. and Vallini, S., The Friuli 1976 earthquake considered as a "Near Source Earthquake", presentation and discussion of the surface recordings.

Woodward-Clyde Consultants, 1977, Earthquake Ground Motions, Vol. 8 of Earthquake Evaluation Studies of the Auburn Dam Area for the U. S. Bureau of Reclamation.

NRC'S NINE CLARIFICATION

1. We do not feel it necessary for TVA to account for the three sets of strong motion recordings from the Friuli earthquakes characterized by "disappeared traces." However, because of the small size of the present data set, we do feel it necessary for TVA to conduct a sensitivity study showing the general effect of the incorporation of additional records of high levels of ground motion upon the 50th and 84th percentile site specific response spectra and the subsequent calculation of relative probabilities.

2. The staff has requested a study of the relative probabilities of exceedence for the various sets of spectra. Although we do not place great reliance upon the accuracy of the absolute probabilities, we feel that estimates of these probabilities are necessary for the full evaluation of the significance and correctness of the relative probabilities.

3. The staff believes that the probabilities should be calculated assuming (1) the maximum possible intensity for each seismic source is the maximum historical intensity and (2) the maximum possible intensity for each seismic source is the maximum historical intensity plus one.

4. The staff believes that the attenuation function used in the probability calculations should have the following characteristics:

a. The epicentral intensity extend out to 10 kilometers.

b. Beyond 10 kilometers, Bollinger's attenuation function and dispersion calculated for the Charleston Earthquake should be used.

c. The dispersion for the attenuation function should be truncated so as not to exceed the epicentral intensity at any distance.

5. The staff believes that the intensity acceleration-relationship and standard error shown in Equations 2-2 and 2-3 of the CSC report (NUREG-0402) is acceptable for use in the probability study.

6. The staff believes the normalized spectral shape calculated from the TVA site specific study would be acceptable for use in the probability study. This spectral shape would be normalized to a peak acceleration at 33 Hz and used with appropriately calculated dispersion factors for amplifications at other frequencies.

7. Historical intensities used in determining activity rates should be those published in Earthquake History of the U. S. unless other values have been specifically changed and accepted by the NRC staff.

8. In the probability calculations all assumptions and procedures should be clearly described so as to permit efficient evaluation by the NRC staff.

9. The reference describing the scatter associated with the Trifunac-Brady acceleration-intensity relationship is

   Trifunac, M.D. A Note on the Range of Peak Amplitudes of Recorded Accelerations, Velocities and Displacements with Respect to the Modified Mercalli Intensity Scale, Earthquake Notes, Vol. 47, No. 1, January-March 1976.

1. The reference (Muzzi and Vallini) used as your source for epicentral data and instrumental locations for the Friuli earthquakes indicates that there were three other sets of strong motion recordings on rock (at Somplago and Tolmezzo) at close distances for the magnitude 6.1 and 6.0 events of September 15, 1976. Figures 37 and 40 of that reference show that while these recordings had "disappeared traces" they also had peak accelerations greater than those recorded at S. Rocco for the same events. Discuss the validity of these measurements and their bearing upon the present study; include any discussions you have had with Muzzi and Vallini regarding this point.

Reference Q1-1 lists five records which had "disappeared traces." Of these five, one occurred at Tarcento, one occurred at Tolmezzo, one occurred at the Somplago turbine level instrument, and two occurred at the Somplago instrument outside of the mountain. Pertinent information on these five recordings and any companion recordings are given in table Q1-1. Only the Tolmezzo and outside Somplago recordings are of interest here.

We have discussed these "disappeared traces" with Francesco Muzzi. He indicated (ref. Q1-2) the following. Apparently the light traces at the high accelerations became so thin they could not be read. He also indicated Tolmezzo is a peculiar site with a complicated geologic structure and considerable fissured rock. It should not be interpreted as a rock site because more recent measurements show very low shear wave velocities.

Muzzi also sent copies of the five "disappeared traces." These traces, although of poor quality, are shown in figures Q1-1 through Q1-5. He has learned recently of a successful attempt by colleagues [Franco Capozza, CNEV (ref. Q1-4)] of the JCS [Joint Study Commission] to reconstruct the traces (ref. Q1-3). We understand this should be completed in 1979.

As shown in table Q1-1 these "disappeared traces" often had higher acceleration prior to "disappearing" than their companion recordings. According to figure 8 of reference Q1-1, all instruments are 0.25 g full scale except for the second instrument at Tolmezzo which is 1.0 g full scale. In figure 40 of reference Q1-1 only one instrument recording at Tolmezzo is reported as a "disappeared trace." Presumably this is the 0.25 g instrument. The recording on the second instrument is not mentioned.

The impact of adding additional records to our data base to approximate these "disappeared traces" is discussed in our response to Question 3 where we report the results of a sensitivity study.

REFERENCES

Q1-1 Muzzi, F. and Vallini, S., The Friuli 1976 earthquake considered as a "Near Source Earthquake," presentation and discussion of the surface recordings.

Q1-2 Muzzi, F., Personal Communication, September 1, 1978.

Q1-3 Muzzi, F., Personal Communication, September 18, 1978.

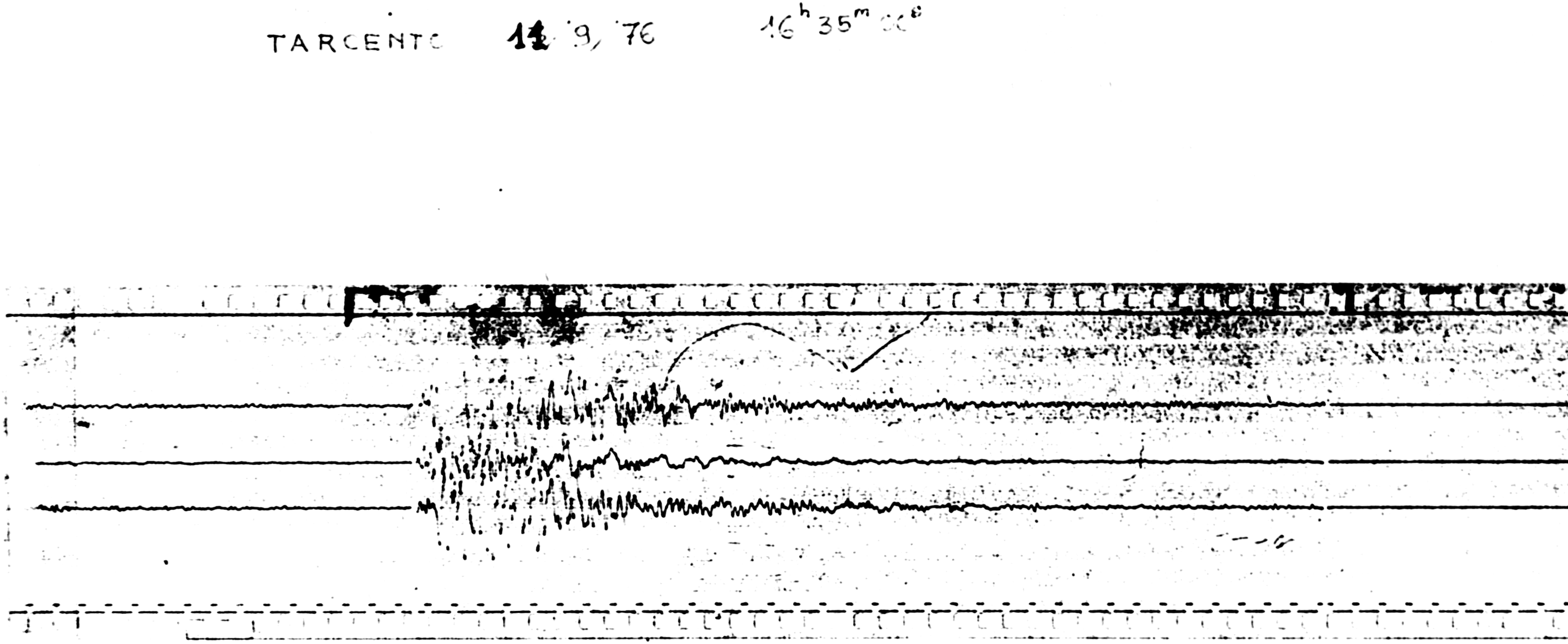
Q1-4 Muzzi, F., Personal Communication, December 14, 1978.

TABLE Q1-1

LIST OF EVENTS WITH "DISAPPEARED TRACES" AND COMPANION RECORDINGS

Date	Time	Magnitude	Station	Epicentral Distance (Km)	Peak Accelerations N-S (g)	Peak Accelerations E-W (g)	Peak Accelerations Vert (g)	Comments
9-11-76	16:35	5.9	Tarcento	15	>.239	>.148	-	Disappeared traces, soil site
9-11-76	16:35	5.9	S. Rocco	14	.092	.095	.048	
9-11-76	16:35	5.9	Somplago (TL)*	6	.063	.062	.034	
9-11-76	16:35	5.9	Buia	14	.233	.108	.093	Soil site
9-11-76	16:35	5.9	Forgaria	14	.133	.235	.119	Soil site
9-15-76	3:15	6.1	Somplago	11	>.137	>.127	-	Disappeared traces
9-15-76	3:15	6.1	S. Rocco	9	.069	.123	.059	
9-15-76	9:21	6.0	Tolmezzo	12	>.283	>.205	-	Disappeared traces
9-15-76	9:21	6.0	Somplago	7.5	>.30	>.196	-	Disappeared traces
9-15-76	9:21	6.0	Somplago (TL)	7.5	>.241	>.236	-	Disappeared traces
9-15-76	9:21	6.0	S. Rocco	20	.145	.238	.083	

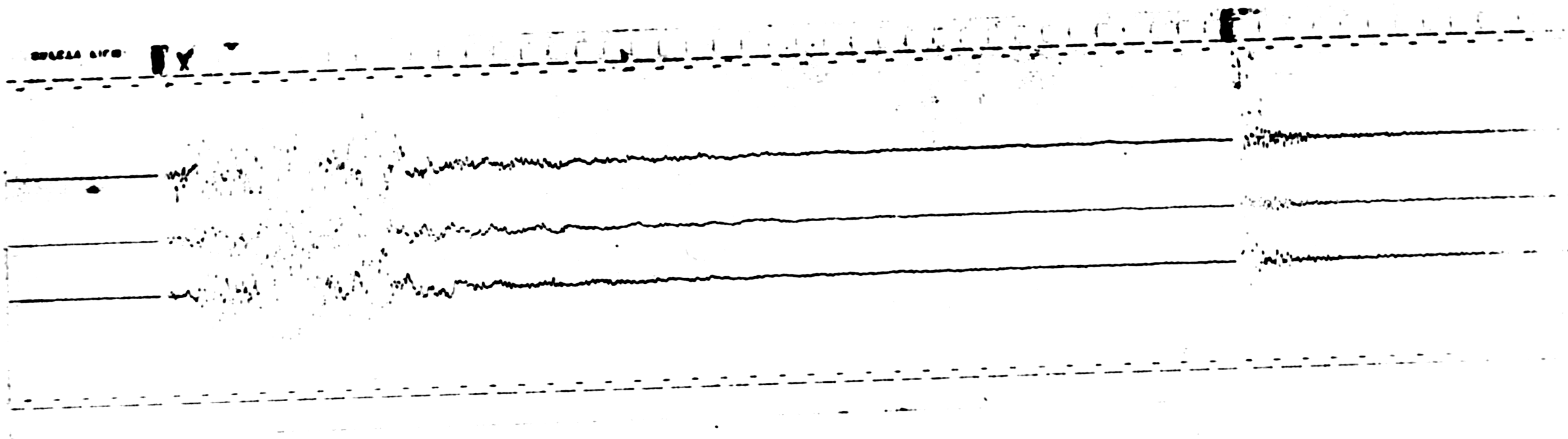
*TL = Turbine Level



"Disappeared Traces" for Tarcento
Date: September 11, 1976 16:35
Record 140
Magnitude 5.9 Epicentral Distance
15 km
$A_{N-S} > 0.239g$, $A_{E-W} > 0.148g$

Figure Q1-1

SOMPLAGO Q.T. 15/9/76 03h 15m 19s

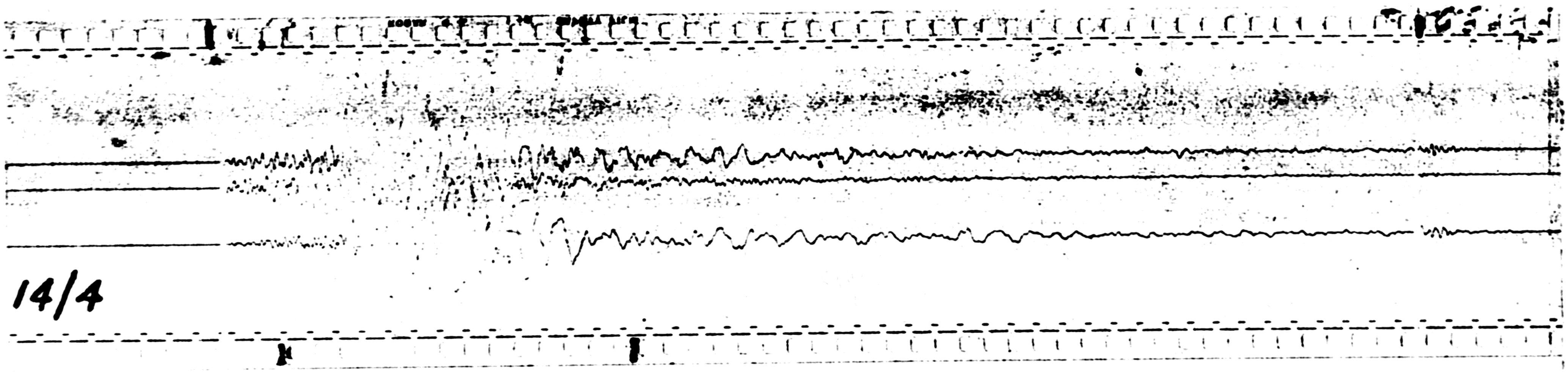


"Disappeared Traces" for Samplago
Date: September 15, 1976 3:15
Record 155
Magnitude 6.1 Epicentral Distance
11 km
$A_{N-S} > 0.137g$, $A_{E-W} > 0.127g$

Figure Q1-2

6.4

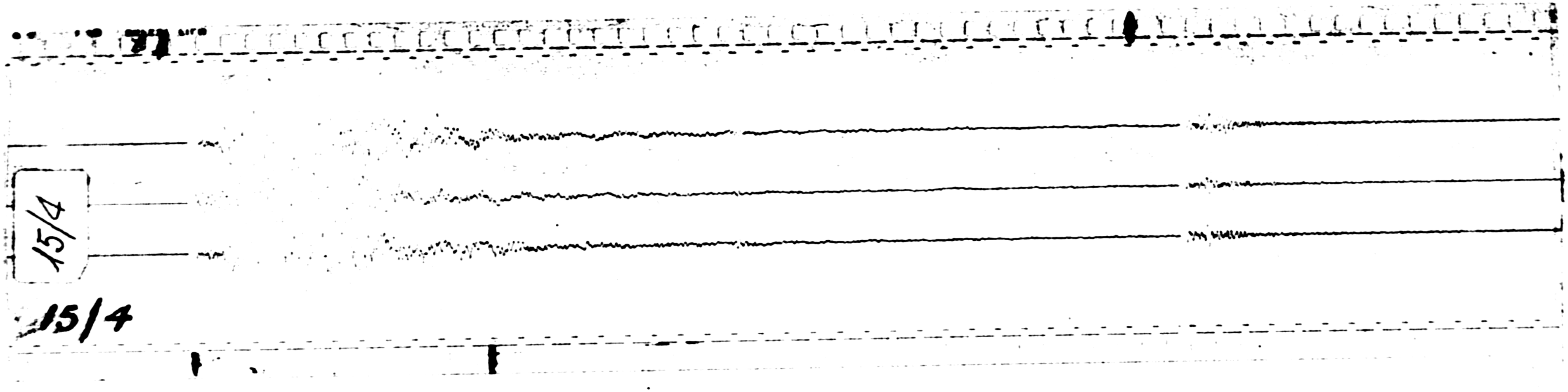
TOLMEZZO 15 9, 76 $09^h\ 21^m\ 18^s$



"Disappeared Traces" for Tolmezzo
Date: September 15, 1976 9:21
Record 173
Magnitude 6.1 Epicentral Distance 12 km
$A_{N-S} > 0.283g$, $A_{E-W} > 0.205g$

Figure Q1-3

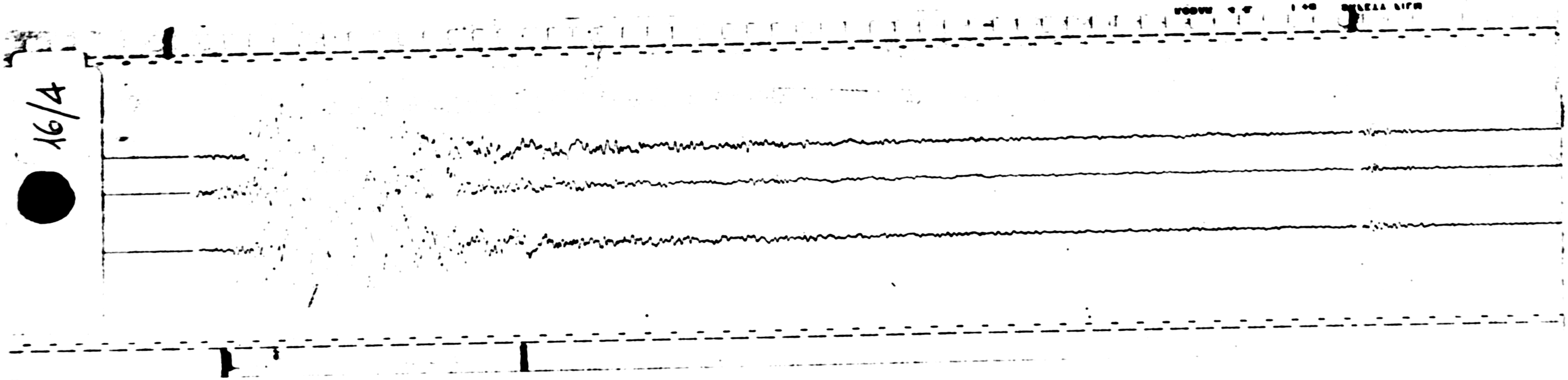
SOMPLAGO Q.T. 15/9/76 09^h 21^m 18^s



"Disappeared Traces" for Samplago
Date: September 15, 1976 9:21
Record 175
Magnitude 6.1 Epicentral Distance
7.5 km
$A_{N-S} > 0.30g$, $A_{E-W} > 0.196g$

Figure Q1-4

SOMPLAGO U.G. 15/9/76 $09^h 21^m 18^s$



"Disappeared Traces" for Samplago - Turbine Level
Date: September 15, 1976 9:21
Record 176
Magnitude 6.1 Epicentral Distance 7.5 km
$A_{N-S} > 0.241g$, $A_{E-W} > 0.236g$

Figure Q1-5

2. The Earthquake Evaluation Study of the Auburn Dam Area (Woodward-Clyde, 1977) utilizes the Koyna Dam records of the magnitude 6.0 (mb) Koyna event of December 11, 1967, in estimating nearby motion at rock sites. Although the recordings were made at about mid-height on the dam, investigators (Chopra and Chakrabarti, 1973; Guha and others 1971) have apparently judged that they represent the free-field motion at the site. The U.S. Geological Survey in their review of Auburn Dam study has also taken this assumption to be valid. Discuss these records and those of associated events at Koyna and their bearing on the present study.

Based on the available published information and our own studies, we do not feel the Koyna Dam earthquake records of December 11, 1967, are representative of free-field motion at a rock site. We do not believe these records should be included in our data set for the reasons discussed below.

The magnitude of the December 11, 1967, event has been variously reported as a Richter magnitude (M_R) 7.0 (ref. Q2-1), a Richter magnitude 6.5 (ref. Q2-2), a surface wave magnitude (M_s) 6.5 and a body wave magnitude (m_b) 6.0 (ref. Q2-3). Assigning the body wave magnitude to the event would allow it to fall within the limitations of our data set (magnitude from 5.3 to 6.3). Since all but one of the assigned magnitudes are beyond our upper limit and the body wave magnitude is in the upper portion of our range, especially since the estimates for the Giles County event range from 5.3 to 5.8 magnitude, we feel it is inappropriate to include this event in our data set. However, should the events at Koyna be included, we feel the September 13, 1967 (M_R = 5.7, ref. Q2-1) and possibly the October 29, 1968 (M_R = 5.25, ref. Q2-1) events should also be included. The impact of adding such additional earthquake records to our data base is discussed in our response to Question 3 where we report the results of a sensitivity study.

The character of the December 11, 1967, event response spectra does not appear to represent typical ground motion spectra. These spectra seem more indicative of structural response spectra as discussed subsequently. In reference Q2-1, Guha states ". . . Due to the comparatively high frequency content in the accelerograms [the September and December events], the instruments might not have acted as a pure accelerometer. However, no attempt has been made to correct the accelerograms on this account." Guha then proceeds to examine the October 29, 1968, event which was recorded on foundation rock and one-third the height of the monolith locations. This October event is a M_R 5.25. Guha then concludes for the October event ". . . the accelerogram recorded on the dam broadly represents the structural response due to ground acceleration . . ." We have attempted to obtain these October records to make a direct comparison of the response spectra at the foundation rock and monolith locations. To date we have not been successful in obtaining the records. Such a comparison could confirm or reject Guha's conclusion about the October event. Nevertheless, should the October records show little difference between the two locations, extrapolating this to a M_R 7.0 event which actually damaged the dam is, at best, risky.

Reference Q2-4 discusses the analysis of the Koyna accelerogram of December 11, 1967. The authors state "There is a possibility that the accelerogram might have been somewhat influenced by the response of the monolith. However, due to various reasons it is not feasible to predict the true ground accelerogram from the one recorded at the gallery level. The monolith in which the instrument was located in an abutment block and had a lot of constraint particularly in the longitudinal direction, that is, parallel to the axis of the dam. It also had a fill on both sides in a direction transverse to the axis of the dam."

In 1973, TVA performed studies on Koyna dam. Part of these studies involved examining the recorded ground motions. According to Krishna, Chandrasekaran, and Saini (ref. Q2-4) the strong motion accelerograph was located in the gallery at about midheight in block 1A. We made an approximate analysis of the block and determined the natural period of the block was about 0.15 second in the transverse direction. Reference Q2-1 determined the natural period to be about 0.12 second in the transverse direction.

Figures Q2-1 and Q2-2 show the response spectra for 2 percent damping in the transverse and longitudinal directions of the dam. Figure Q2-1, for the transverse direction, shows a very predominant peak at a natural period of about 0.12 second which suggests strong amplification of ground motion. Another interesting observation about the response spectra in the transverse direction is the peaks at about 0.09 and 0.06 second. Chopra and Chakrabarti (ref. Q2-2) performed a finite element analysis of the nonoverflow monolith and determined natural periods of 0.326, 0.122, 0.093, and 0.063 second in the transverse direction. The analysis showed the higher modes (particularly the second mode) providing significant response as evidenced by figure Q2-3 which shows the calculated acceleration (using the recorded motion as input). Figure Q2-3 shows significant amplication at midheight caused by the higher modes. The higher modes (0.12, 0.09, and 0.06 seconds) are shown in the response spectra which indicate amplication due to the structure. Since the recording was made at midheight and the second mode was predominant for this location, the first mode (0.33 second) would not be amplified significantly. This is also reflected in the response spectra for the transverse direction. No analyses have been made in the longitudinal direction of the dam. The response spectra in the longitudinal direction are not typical of other observed rock spectra or of any observed ground motions. Since the instrument was located in a monolith near the right bank (ref. Q2-2), it is possible that the abutments and topography affected the longitudinal motion.

Based on the above, it is concluded that the Koyna recordings have been influenced by the dam and probably by the abutments and topography around the abutments. Therefore, it is not representative of a free-field top of rock response spectra and should not be included in the data set.

REFERENCES

Q2-1 Guha, S. K., Agarwal, S. P., Agarwal, V. P., Gosavi, P. D., and Marwadi, S. C., Analyses of Accelerograms and Structural Response; Journal of Engineering Mechanics Division, Proceedings of American Society of Civil Engineers, Vol. 97, No. EM2, April 1971.

Q2-2 Chopra, A. K. and P. Chakrabarti, "The Earthquake Experience at Koyna Dam and Stresses in Concrete Gravity Dams," Earthquake Engineering and Structural Dynamics, Vol. 1, 1972.

Q2-3 Woodward-Clyde Consultants, Earthquake Ground Motions, Vol. 8 of Earthquake Evaluation Studies of the Auburn Dam Area for the U.S. Bureau of Reclamation, 1977.

Q2-4 Krishwa, J., A. R. Chandrasekaran, and S. S. Saini, "Analysis of Koyna Accelerogram of December 11, 1967," Bulletin of Seismological Society of America, Vol. 59, August 1969.

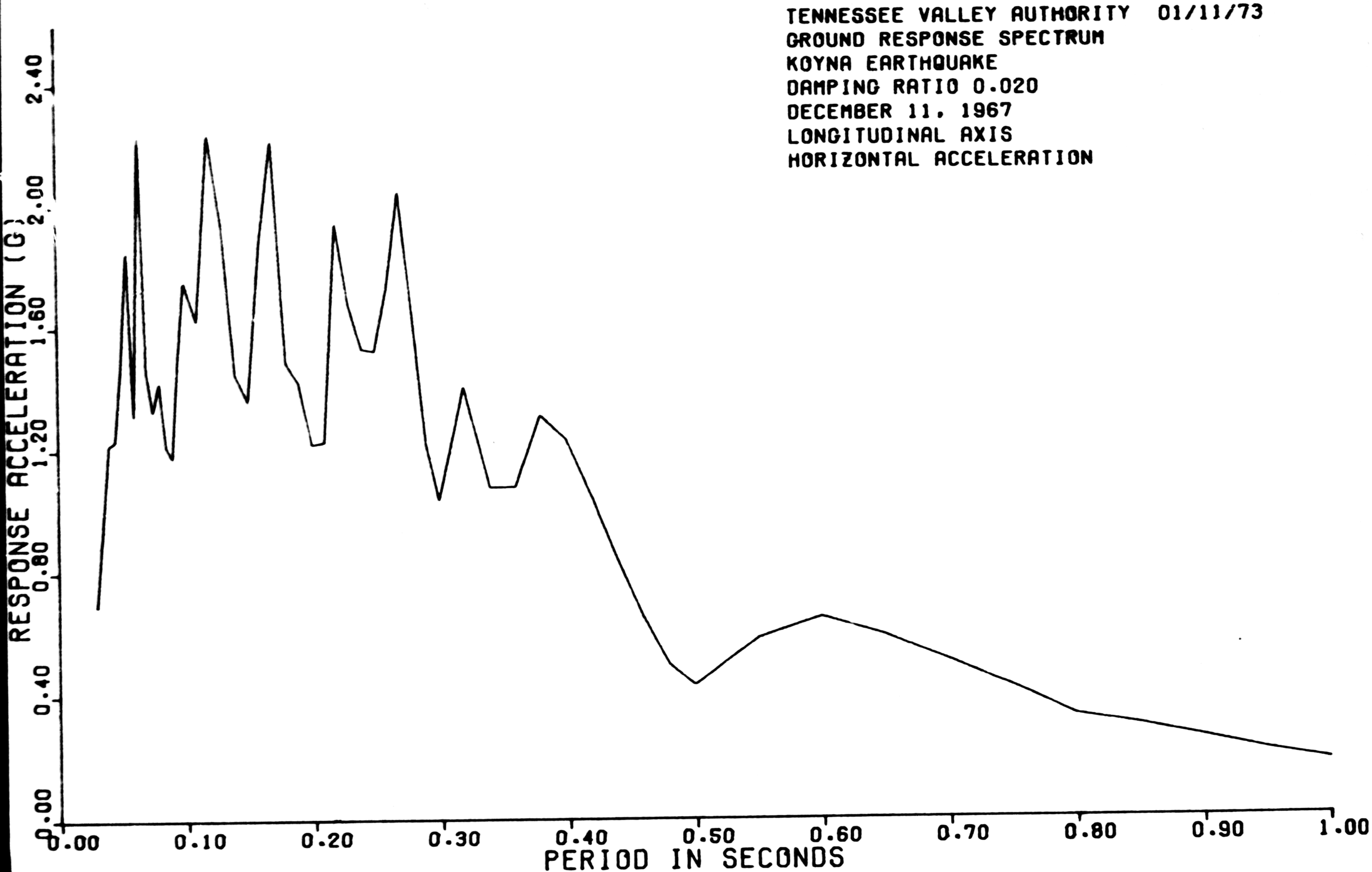

TENNESSEE VALLEY AUTHORITY 01/11/73
GROUND RESPONSE SPECTRUM
KOYNA EARTHQUAKE
DAMPING RATIO 0.020
DECEMBER 11, 1967
LONGITUDINAL AXIS
HORIZONTAL ACCELERATION
RESPONSE ACCELERATION (G)
0.00
0.40
0.80
1.20
1.60
2.00
2.40
0.00
0.10
0.20
0.30
0.40
0.50
0.60
0.70
0.80
0.90
1.00
PERIOD IN SECONDS

FIGURE Q2-2

TENNESSEE VALLEY AUTHORITY 01/12/73
KOYNA DAM
EARTHQUAKE 12/11/67
TRANSVERSE AXIS
GROUND ACCELERATION = 0.49G
SPECTRAL EARTHQUAKE RESPONSE
5.0 PERCENT DAMPING
HORIZONTAL ACCELERATION

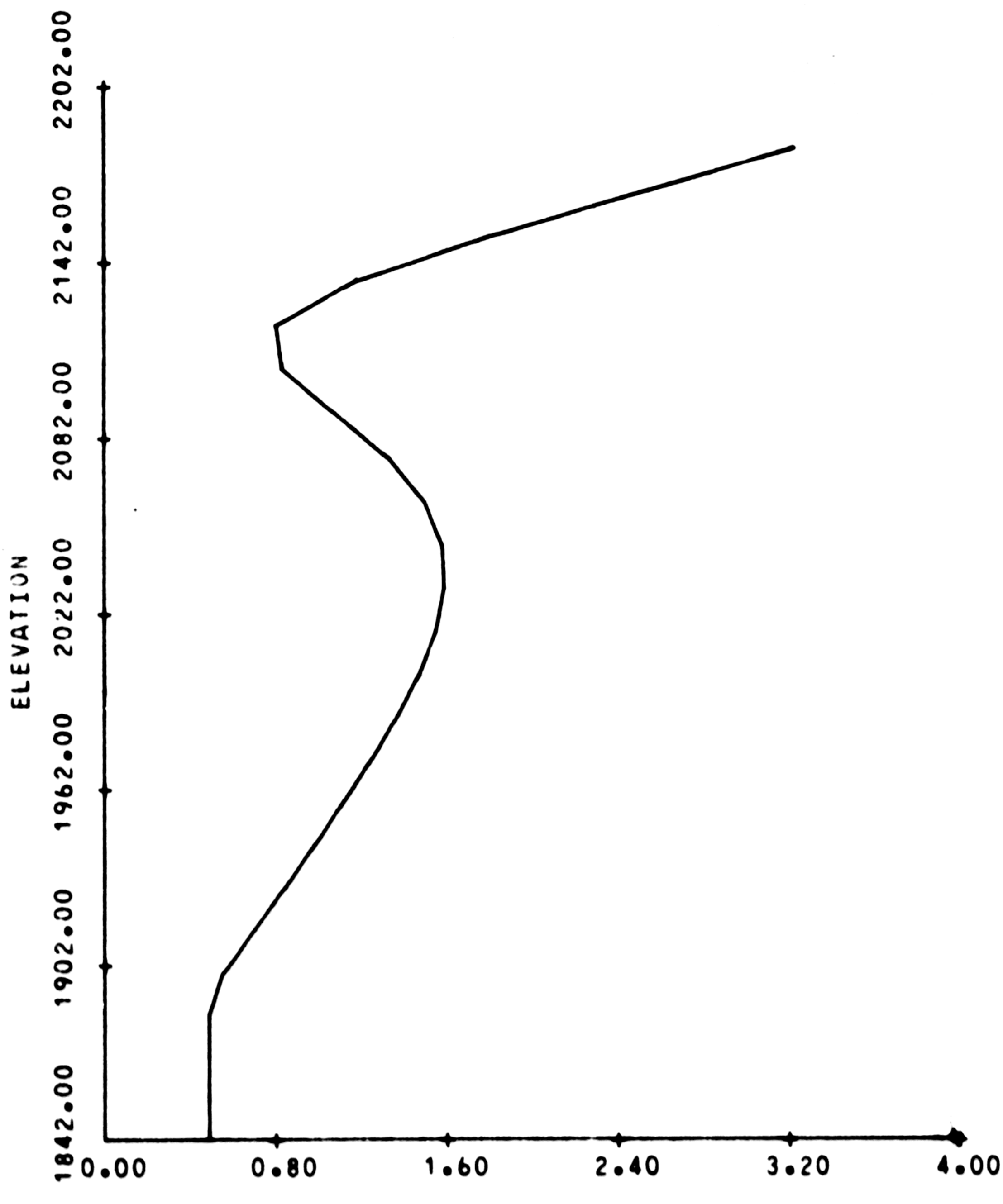


FIGURE Q2-3

3. **Determine whether the distribution of response spectral values at each frequency for the available data is better fit by a normal or lognormal distribution. Calculate 50th and 84th percentile spectra for the appropriate distribution.**

4. **Determine to what percentile different frequency bands of the design spectra correspond to with respect to the distribution calculated in 3. Discuss the significance of each such frequency band to the engineering analysis of structural components.**

The response to Questions 3 and 4 are combined due to their overlapping natures. These questions basically ask for additional statistical treatment of the data. In addition we are including a discussion of normalized response spectra and the development of site specific normalized response spectra. This discussion parallels our previous discussion for actual (unnormalized) spectra.

A sensitivity study was requested by the NRC in our November 9, 1978, meeting and their additional clarification of the original six questions. Their clarification is:

> We do not feel it necessary for TVA to account for the three sets of strong motion recordings from the Friuli earthquakes characterized by "disappeared traces." However, because of the small size of the present data set, we do feel it necessary for TVA to conduct a sensitivity study showing the general effect of the incorporation of additional records of high levels of ground motion upon the 50th and 84th percentile site specific response spectra and the subsequent calculation of relative probabilities.

The results of the sensitivity study are also included in this response. This sensitivity study also covers parts of our response to Questions 1 and 2. The results were also used in our response to question 6.

Statistical Distribution of the Actual Response Spectral Values

Response spectra for both horizontal components of these 13 earthquake records for 4 and 7 percent of critical damping are presented in figures A-1 to A-26 in appendix A of our Phase II report. The use of these damping ratios is discussed in section 3 of that report.

The actual distribution of the response spectra is also shown in the Phase II report. Figures A-27 and A-28 show overplots of the spectra for the six United States records for 4 and 7 percent damping, respectively. Similarly, figures A-29 and A-30 show overplots for the seven Italy records and figures A-31 and A-32 show overplots for all 13 records.

In the Phase II report, figures A-37 and A-38 represent the statistical distribution of the data when they are assumed to be normally (Gaussian) distributed. These figures show the mean (50th percentile), mean plus one standard deviation (84th percentile), maximum and minimum response spectra for all 13 records for 4 and 7 percent damping, respectively.

Figures Q3-1 and Q3-2 represent the statistical distribution of the data when they are assumed to be lognormally distributed. These figures show the maximum, minimum, 16th, 50th, and 84th percentile response spectra for all 13 records for 4 and 7 percent damping, respectively.

Figures Q3-3 through Q3-10 compare the actual data to each assumed distribution. For this comparison only the response spectra for 4 percent damping are used. Figures Q3-3 to Q3-6 compare the actual data to an assumed normal distribution for peak acceleration values and spectral values at periods of 0.15, 0.40, and 4.0 seconds, respectively. Figures

Q3-7 to Q3-10 compare the same data to an assumed lognormal distribution. A comparison of these eight figures indicate these data are more nearly lognormally distributed than normally distributed.

The 16th, 50th, and 84th percentile values for both assumed distributions of the data are given in table Q3-1. The negative entry for the 16th percentile of the normal distribution indicates an acceleration value less than zero. Such a value has no physical meaning in this context. The occurrence of the negative value has been reported also in reference Q3-1. The occurrence of these possible negative values for lower percentile values is one shortcoming of the normal distribution.

This tendency for the data to be lognormally distributed is also supported by a likelihood ratio test of the various spectral values. In this test, between normal and lognormal distributions, a ratio of greater than 1 indicates a preference for normal distribution while a value less than 1 indicates a preference for lognormal distribution. Mathematically, the test is expressed as:

$$\text{LRT (N:LN)} = \frac{\prod_{i=1}^{n} p(x/\mu,\sigma)_{Normal}}{\prod_{i=1}^{n} p(x/\mu,\sigma)_{Lognormal}}$$

$$\frac{\prod_{i=1}^{n}\left\{\frac{1}{\sqrt{2\pi}}\, e^{-\frac{1}{2}\left(\frac{x-\mu_N}{\sigma_N}\right)^2}\right\}}{\prod_{i=1}^{n}\left\{\frac{1}{\sqrt{2\pi}\,x}\, e^{-\frac{1}{2}\left(\frac{x-\mu_L}{\sigma_L}\right)^2}\right\}}$$

where x = the individual spectral values at a given period and damping ratio,

μ = mean of the data, either normal (N) or lognormal (LN),

σ = standard deviation of the data, either N or LN, p = probability density function for either a normal or lognormal distribution, and

n = number of data points considered.

when LRT $> 1 \Rightarrow$ normal distribution

$< 1 \Rightarrow$ lognormal distribution

The results of the likelihood ratio test for each of the 80 frequencies are listed in table Q3-2. Also included in table Q3-2 are the results for frequency clusters with 5 or 20 frequencies in each cluster as well as for all 80 frequencies considered together. These results indicate a strong preference for the lognormal distribution. This is graphically displayed in figures Q3-3 through Q3-10 previously discussed.

Thus, between the assumed normal and lognormal distribution, the actual data are better fit by the lognormal distribution.

Comparison of Site Specific Response Spectra with Plant Design Spectra

To examine the relationship between the seismic design of the Sequoyah, Watts Bar, and Bellefonte Nuclear Power Plants and the actual ground motion induced by the earthquakes, comparison of these 13 records with the seismic design response spectra used at each plant is given in appendix B, figures B-1 to B-52, in our Phase II report.

Figure Q3-11 compares the 50th and 84th percentile site specific response spectra at 4 percent damping to the design spectra used for steel structures at the three plants.

Figure Q3-12 compares the 50th and 84th percentile site specific response spectra at 7 percent damping to the design spectra used for reinforced concrete structures at the three plants.

Figures Q3-13 and Q3-14 show the percent fractile of the site specific response spectra at each frequency for the three plants' design spectra. Figure Q3-13 is for 4 percent damping (steel structures) and figure Q3-14 is for 7 percent damping (reincorced concrete structures). Table Q3-3 gives a listing of these percent fractile for the three plants for all 80 frequencies and both damping ratios.

Development of Normalized Response Spectra from Strong Motion Records of Approximate Magnitude and Distance and Comparison with Design Spectra

The suite of 13 strong motion records for earthquakes of appropriate magnitude and distance for the existing site conditions were also used to develop site specific normalized response spectra for the three plants. This is accomplished by normalizing each of the 26 horizontal components to a peak acceleration of 1.0 g.

Statistical Distribution of Normalized Response Spectral Values

Figures Q3-15 and Q3-16 represent the statistical distribution of the normalized response spectra when they are assumed to be normally distributed. These figures show the maximum, minimum, 50th, and 84th percentile normalized response spectra for all 13 records for 4 and 7 percent damping respectively.

Figures Q3-17 and Q3-18 represent the statistical distribution of these same spectra when they are assumed to be lognormally distributed. These figures show the maximum, minimum, 16th, 50th, and 84th percentile normalized response spectra for all 13 records for 4 and 7 percent damping, respectively.

Figures Q3-19 through Q3-24 compare the normalized data to each assumed distribution. Again, for this comparison only the response spectra for 4 percent damping are used. Figures Q3-19 to Q3-21 compare the actual data to an assumed normal distribution for spectral values at periods of 0.15, 0.40, and 4.0 seconds, respectively. Figures Q3-22 to Q3-24 compare the same data to an assumed lognormal distribution. A comparison of these six figures indicates these data are more nearly lognormally distributed than normally distributed.

The 16th, 50th, and 84th percentile values for both assumed distributions of the data are given in table Q3-4.

This tendency for the data to be lognormally distributed is also supported by the likelihood ratio test of the various normalized spectral values. The results of the likelihood ratio test for each 80 frequencies are listed in table Q3-5. Also included in table Q3-5 are the results for frequency clusters with 5 or 20 frequencies in each cluster as well as for all 80 frequencies considered together. These results indicate a strong preference for the lognormal distribution.

Thus, between the assumed normal and lognormal distribution, the actual data are better fit by the lognormal distribution.

<u>Comparison of Site Specific Normalized Response Spectra with Plant Design Spectra</u>

Figure Q3-25 compares the 50th and 84th percentile site specific normalized response spectra at 4 percent damping to the design spectra used for steel structures at the three plants. These 50th and 84th percentile normalized spectra are anchored to the 50th percentile peak acceleration based on a lognormal distribution. This anchor acceleration is 0.101 g.

Figure Q3-26 compares the 50th and 84th percentile site specific normalized response spectra at 7 percent damping to the design spectra used for reinforced concrete structures at the three plants. Again, these normalized spectra are anchored to 0.101 g.

The procedure used here for determining a normalized site spectra is basically identical with the work of Blume (ref. Q3-2). In reference Q3-2 33 accelerograms were used to represent (1) rock, alluvium, deep, and soft sites, (2) a wide range of magnitudes, and (3) small, intermediate, and large epicentral distances. In the work reported here, we used 26 accelerograms to represent only (1) rock sites, (2) a magnitude range of 5.3 to 6.3, and (3) epicentral distances of less than about 25 kilometers. The work of Blume was combined with additional studies by Newmark (ref. Q3-3 and Q3-4) and form the basis of NRC Regulatory Guide 1.60 response spectra. These spectra are basically the 84th percentile Blume spectra.

NRC Standard Review Plan 2.5.2 sets a general procedure in determining a site response spectrum as:

(1) Determine the mean acceleration for the applicable site intensity, and

(2) Anchor the Regulatory Guide 1.60 spectrum to this mean acceleration.

This procedure was basically followed in the creation of the spectra shown in figures Q3-25 and Q3-26. Here the 50th percentile peak acceleration of our 26 accelerograms (using lognormal distribution) is used as the anchor acceleration. These 84th percentile normalized response spectra are then anchored to this acceleration. As shown in figures Q3-25 and Q3-26, the three plants' design spectra envelop the resulting site specific response spectra.

Sensitivity Study

A sensitivity study was performed on the actual and normalized response spectra to determine the sensitivity of these spectra to including additional records in the data base. For both actual and normalized response spectra, six variations of additional records were considered. These six variations are:

(1) The addition of two high pairs of records (two records of two components each),

(2) The addition of four high pairs of records,

(3) The addition of two low pairs of records,

(4) The addition of four low pairs of records,

(5) The addition of one high and one low pair of records, and

(6) The addition of two high and two low pairs of records.

In all cases the two horizontal components of the Tolmezzo, Italy, May 6, 1976, event (record number 038, see figures A-13 and A-14 in the Phase II report) are used as the high pair of records. Similarly the Tolmezzo, Italy, May 11, 1976, event (record number 063, see figures A-17 and A-18 in the Phase II report) is used as the low pair of records. These records were selected for the following reasons:

(1) The peak accelerations for the Tolmezzo 038 record are 0.346 g and 0.311 g. As a pair these are the highest accelerations of

all 13 records. (Temblor has a higher acceleration, 0.348 g, in one direction but a lower acceleration, 0.270 g, in the other direction. See table 4.2 in the Phase II report for the peak accelerations of all the records.)

(2) The Tolmezzo 063 record represents the pair of records with the lowest acceleration, 0.027 g and 0.027 g, of all 13 records.

(3) Question 1 addresses the Friuli recordings with "disappeared traces." The Tolmezzo station had one of the three "disappeared traces" for free field motions at a rock site. (Somplago had the other two free field "disappeared traces" plus another recorded at the turbine level inside the mountain. See response to Question 1.)

(4) The Tolmezzo 038 records have peak accelerations in excess of the instrument limits (0.25 g full scale) where the "disappeared traces" occurred.

(5) A comparison of the available Tolmezzo and S. Rocco records indicates the normalized response spectrum shape of these records is more affected by differences in the magnitude of the event than in the epicentral distance.

Both high, low, and combined variations are considered to show the full impact of a parametric variation in the data base. It is reasonable to expect high and low values of acceleration for future earthquakes in the parameter range which we are considering. Examination of the recorded data during the Friuli earthquakes support this. For example, the Tolmezzo trace which disappeared was for a magnitude of 6.0 at 12 kilometers distance. The traces disappeared at about 0.28 and 0.20 g.

San Rocco recorded this same event at a distance of 20 kilometers with maximum values of 0.15 and 0.24 g. A magnitude of 6.1 was also recorded at San Rocco at a distance of 9 kilometers which had maximum values of 0.07 and 0.12 g. This illustrates that both high and low values should be considered in a sensitivity study.

The 16th, 50th, and 84th percentile peak acclerations for the original data set and all six parametric variations of this data set are given in table Q3-6. Figures Q3-27 and Q3-29 compare the 16th, 50th, and 84th percentile results for the actual response spectra for 4 and 7 percent damping for the original 13 records, the original records plus four high pairs of records, and the original records plus four low pairs of records. The other four parametric variations are not shown since they fall between the extreme limits for four additional high or low pairs. Some noticeable effect is observed. Figures Q3-28 and Q3-30 give the same comparison for the normalized response spectra for 4 and 7 percent damping. In the higher frequency range (frequencies above 5 Hertz) very little difference is noted between the parametric variations. In fact, these differences are so small that they cannot be plotted and shown on the same figure. Thus, for clarity of the plot, only the curves for the original 13 records are shown in the higher frequency range. In the lower frequency range some deviation is observed and is shown in figures Q3-28 and Q3-30.

Based on the results of the complete sensitivity study and the results presented in figures Q3-27 to Q3-30, it is determined that (1) the actual response spectra are not overly sensitive to reasonable variations in the data base, and (2) the normalized response spectra are very insensitive in the high frequency range and only moderately sensitive in the low frequency range to variations in the data base.

The interpretation of the sensitivity study results should be coupled with some practical judgment. The original data base consists of 13 records recorded over a 43-year period, from 1935 to the present. These records represent the only available data meeting our specific site, magnitude, and distance limitations. The hypothetical inclusion of additional high records should be tempered with a realization of the actual historical distribution of the 13 records. Therefore, it appears unreasonable to assume a large number of high records may be recorded without additional intermediate or low records also being recorded. All of these additional records would then be included in the updated data base. Although no quantitative measures are available to indicate how many additional high, intermediate, and low recordings will be available within a given number of years in the future, the results of the sensitivity study should give a measure to assess the impacts of such hypothetical occurrences.

Effect on Structural Components

The natural periods of the structures at the subject plants are in the range of 0.3 second to 0.03 second or less. Examination of the above various site specific spectra show that the actual 50th percentile spectra is enveloped by the three plants' design spectra and the actual 84th percentile spectra exceeds the Sequoyah, Watts Bar, and Bellefonte design spectra by about 51, 25, and 25 percent, respectively, in this period range. The normalized 84th percentile site specific spectra are also enveloped by the subject plants' design spectra. The percentiles, the Sequoyah, Watts Bar, and Bellefonte design spectra are of the actual site specific spectra, are shown in table Q3-3 and figures Q3-13 and Q3-14.

The smallest value is the 67th percentile. The minimum percentile of the normalized site specific spectra is the 93d percentile. Combining these results with the relative comparisons of the probabilities of exceeding the design response spectra (presented in the response to Question 6), TVA concludes the seismic design bases used at the subject plants are conservative and adequately protect the health and safety of the public.

Soil-Supported Structures

Category I soil-supported structures were analyzed with a different input motion than the rock-supported structures. The input motion for rock-supported structures is defined by the top of rock design response spectra shown in figures 3-1 through 3-6 in our Phase II report. For soil-supported structures this rock motion was amplified to obtain the ground surface motion. The following steps were followed in earthquake analyses of soil-supported structures:

1. The top of rock motion was used as input into a shear beam analysis of the soil deposit using 10 percent soil damping. The shear wave velocity of the soil was varied a minimum of ± 30 percent to account for uncertainties in the properties of the soil. The free field soil response spectra is 2 to 3 times the top of rock spectra in the frequency range of interest to the structures.

2. The soil-supported structures were analyzed using soil-springs calculated as indicated in the FSAR. The predominant motion of the structures was in the translational soil spring. The maximum damping allowed by NRC for this translational motion was 10 percent which is very conservative. The free-field soil motion developed in step 1 was used as input into the soil springs.

The range of design response spectra used as input at the base of soil-supported structures at Sequoyah is shown in figure Q3-31 and at Watts Bar in figure Q3-32. There are no major soil-supported structures at Bellefonte. Examination of these soil spectra show that they envelop the original rock design spectra, the actual 50th and 84th percentile site spectra, and the normalized 84th percentile site specific spectra.

In addition, this method of analyzing soil supported structures is very conservative for the following reasons:

1. The defined top of rock motion used at the plants is based on emperical relationships developed from recorded data at the ground surface (mainly on soil). Therefore amplifying this motion through the soil deposit is very conservative. The maximum soil acceleration obtained from the amplification is 0.42 g, whereas using Trifunac-Brady intensity-acceleration relationship results in 0.25 g.

2. The motion is amplified again through the soil springs using a very conservative damping value. More refined analysis methods have shown that the predominant translational motion does not occur. This is illustrated in reference Q3-5.

Based on the conservative analysis approach and the enveloping nature of the soil spectra, TVA concludes the seismic design bases for soil-supported structures at the subject plants are conservative and adequately protect the health and safety of the public.

REFERENCES

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Q3-2 John A. Blume & Associates, "Recommendations for Shape of Earthquake Response Spectra," San Francisco, California, USAEC Contract No. AT (49-5)-3011, WASH-1254, February 1973.

Q3-3 N. M. Newmark Consulting Engineering Services, "A Study of Vertical and Horizontal Earthquake Spectra," Urbana, Illinois, USAEC Contract No. AT (49-5)-2667, WASH-1255, April 1973.

Q3-4 Newmark, N. M., John A. Blume, and Kanwar K. Kapur, "Design Response Spectra for Nuclear Power Plants," ASCE Structural Engineering Meeting, San Francisco, April 1973.

Q3-5 Hunt, R. Joe, "Structure-Foundation Interaction of Nuclear Power Plant Structure During Earthquakes," Specialty Conference on Structural Design of Nuclear Plant Facilities, Chicago, Illinois, December 17 and 18, 1973.

TABLE Q3-1

NORMAL AND LOGNORMAL DISTRIBUTION 16TH, 50TH, AND 84TH PERCENTILE SITE SPECIFIC RESPONSE SPECTRUM VALUES

Spectral Period	Percent Fractile					
	Normal Distribution			Lognormal Distribution		
(Sec)	16th (g)	50th (g)	84th (g)	16th (g)	50th (g)	84th (g)
Peak acceleration	0.034	0.13	0.23	0.047	0.10	0.22
	For 4% Damping					
0.06	0.059	0.18	0.29	0.066	0.14	0.29
0.15	0.15	0.36	0.57	0.14	0.29	0.60
0.40	-0.053	0.25	0.55	0.048	0.14	0.41
1.50	0.0027	0.041	0.080	0.011	0.027	0.069
4.0	-0.00024	0.0078	0.016	0.0016	0.0047	0.013
	For 7% Damping					
0.06	0.052	0.17	0.28	0.063	0.13	0.27
0.15	0.12	0.31	0.50	0.12	0.25	0.52
0.40	-0.046	0.22	0.48	0.041	0.12	0.35
1.50	0.0016	0.037	0.072	0.0092	0.024	0.062
4.0	-0.00003	0.0074	0.015	0.0016	0.0045	0.013

TABLE Q3-2

LIKELIHOOD RATIO TEST - NORMAL VS LOGNORMAL DISTRIBUTION - ACTUAL SPECTRA

			FOR DAMPING RATIO: 0.04			FOR DAMPING RATIO: 0.07		
NO.	PERIOD SEC	FREQUENCY HZ	STD DEV NORMAL	STD DEV LOGNORMAL	LIKELIHOOD RATIO N/LN	STD DEV NORMAL	STD DEV LOGNORMAL	LIKELIHOOD RATIO N/LN
1	0.030	33.333	0.097810	0.331970	0.2598E-11	0.097360	0.331620	0.2315E-11
2	0.035	28.571	0.100930	0.338140	0.8460E-11	0.099280	0.334530	0.5868E-11
3	0.040	25.000	0.107450	0.345160	0.1325E-10	0.102540	0.338710	0.1035E-10
4	0.045	22.222	0.109600	0.344120	0.1268E-10	0.104130	0.338140	0.1058E-10
5	0.050	20.000	0.112660	0.342590	0.1795E-10	0.107360	0.336600	0.1159E-10
6	0.055	18.182	0.113940	0.327690	0.1931E-10	0.109970	0.325200	0.9479E-11
7	0.060	16.667	0.117360	0.319530	0.9758E-11	0.113480	0.317230	0.4103E-11
8	0.065	15.385	0.118430	0.312580	0.2642E-10	0.115190	0.316440	0.1166E-10
9	0.070	14.286	0.120130	0.308410	0.7659E-10	0.115060	0.304370	0.1941E-10
10	0.075	13.333	0.118760	0.296450	0.7102E-10	0.115980	0.293490	0.1475E-10
11	0.080	12.500	0.120780	0.276490	0.4534E-10	0.118260	0.283120	0.1221E-10
12	0.085	11.765	0.124360	0.273420	0.5292E-10	0.122050	0.280690	0.1408E-10
13	0.090	11.111	0.126010	0.270450	0.1544E-09	0.124970	0.281140	0.2653E-10
14	0.095	10.526	0.139460	0.282850	0.8230E-10	0.131150	0.285880	0.3856E-10
15	0.100	10.000	0.143360	0.287460	0.2445E-09	0.138640	0.300050	0.1020E-09
16	0.110	9.091	0.164770	0.323130	0.7718E-09	0.150060	0.324170	0.2944E-09
17	0.120	8.333	0.169940	0.309830	0.3922E-09	0.162140	0.323290	0.1057E-09
18	0.130	7.692	0.199990	0.326560	0.4635E-09	0.184780	0.330050	0.6821E-10
19	0.140	7.143	0.225210	0.321920	0.2390E-09	0.197470	0.327680	0.8923E-10
20	0.150	6.667	0.209040	0.314920	0.4953E-09	0.191430	0.327130	0.1491E-09
21	0.160	6.250	0.238700	0.335690	0.3428E-10	0.206080	0.338070	0.2463E-10
22	0.170	5.882	0.235340	0.348230	0.3745E-10	0.202570	0.348920	0.2557E-10
23	0.180	5.556	0.226610	0.372260	0.8914E-10	0.195380	0.359960	0.3731E-10
24	0.190	5.263	0.224220	0.371990	0.5531E-10	0.196410	0.367080	0.2550E-10
25	0.200	5.000	0.225080	0.373440	0.4154E-10	0.200390	0.378080	0.1789E-10
26	0.210	4.762	0.240830	0.372640	0.1219E-10	0.207050	0.371950	0.6795E-11
27	0.220	4.545	0.251780	0.368790	0.3497E-11	0.216390	0.372280	0.2541E-11
28	0.230	4.348	0.242800	0.360450	0.1988E-11	0.216620	0.368520	0.1092E-11
29	0.240	4.167	0.252310	0.368470	0.5995E-12	0.220630	0.372680	0.3562E-12
30	0.250	4.000	0.300350	0.382710	0.3115E-13	0.236820	0.377620	0.6403E-13
31	0.260	3.846	0.327190	0.387310	0.3291E-14	0.247260	0.382420	0.1700E-13
32	0.270	3.704	0.305960	0.388790	0.3373E-14	0.235420	0.386200	0.1935E-13
33	0.280	3.571	0.265670	0.390360	0.1750E-13	0.213830	0.388860	0.5766E-13
34	0.290	3.448	0.250050	0.395780	0.2347E-13	0.207940	0.393720	0.4567E-13
35	0.300	3.333	0.238480	0.402380	0.4400E-13	0.206150	0.400560	0.3513E-13
36	0.320	3.125	0.246500	0.393620	0.5723E-14	0.221780	0.404330	0.3095E-14
37	0.340	2.941	0.291470	0.412850	0.9186E-16	0.243680	0.416030	0.1564E-15
38	0.360	2.778	0.326950	0.437830	0.5554E-17	0.261940	0.434080	0.1599E-16
39	0.380	2.632	0.318080	0.452040	0.3950E-17	0.267580	0.448290	0.4412E-17
40	0.400	2.500	0.303530	0.463420	0.3691E-17	0.261240	0.462430	0.3422E-17

TABLE Q3-2 (Continued)

LIKELIHOOD RATIO TEST - NORMAL VS LOGNORMAL DISTRIBUTION - ACTUAL SPECTRA

NO.	PERIOD SEC	FREQUENCY HZ	FOR DAMPING RATIO: 0.04 STD DEV NORMAL	STD DEV LOGNORMAL	LIKELIHOOD RATIO N/LN	FOR DAMPING RATIO: 0.07 STD DEV NORMAL	STD DEV LOGNORMAL	LIKELIHOOD RATIO N/LN
41	0.420	2.381	0.281710	0.472480	0.5816E-17	0.245600	0.467760	0.4393E-17
42	0.440	2.273	0.265820	0.477280	0.5534E-17	0.228170	0.468080	0.4960E-17
43	0.460	2.174	0.255950	0.484590	0.2051E-17	0.218480	0.473370	0.2441E-17
44	0.480	2.083	0.251780	0.479130	0.3901E-18	0.205350	0.466310	0.1293E-17
45	0.500	2.000	0.268680	0.471010	0.2415E-19	0.202350	0.461980	0.4071E-18
46	0.550	1.818	0.221990	0.460240	0.2030E-19	0.183400	0.450730	0.5648E-19
47	0.600	1.667	0.206490	0.442960	0.6884E-21	0.170530	0.442960	0.5170E-20
48	0.650	1.538	0.224460	0.446100	0.7101E-23	0.160470	0.438230	0.1070E-20
49	0.700	1.429	0.194910	0.439560	0.3386E-22	0.144970	0.432080	0.1437E-20
50	0.750	1.333	0.148800	0.433850	0.8424E-20	0.119430	0.432800	0.5434E-19
51	0.800	1.250	0.125040	0.426850	0.2003E-16	0.106180	0.435450	0.4793E-18
52	0.850	1.176	0.107490	0.440750	0.2673E-17	0.093500	0.441580	0.3469E-17
53	0.900	1.111	0.095280	0.452120	0.1184E-16	0.082960	0.449880	0.1897E-16
54	0.950	1.053	0.084970	0.452470	0.7184E-16	0.076610	0.452090	0.4101E-16
55	1.000	1.000	0.082570	0.472730	0.1555E-15	0.071790	0.458390	0.8928E-16
56	1.100	0.909	0.077750	0.474710	0.4474E-16	0.067120	0.469760	0.7233E-16
57	1.200	0.833	0.069310	0.486370	0.1911E-15	0.057950	0.474080	0.4380E-15
58	1.300	0.769	0.057930	0.468520	0.1342E-14	0.049510	0.461760	0.1286E-14
59	1.400	0.714	0.045690	0.432800	0.4493E-14	0.040900	0.434050	0.2854E-14
60	1.500	0.667	0.038480	0.408760	0.7784E-14	0.035310	0.413830	0.4202E-14
61	1.600	0.625	0.034670	0.398500	0.7517E-14	0.032230	0.408140	0.5048E-14
62	1.700	0.588	0.031590	0.397100	0.7778E-14	0.029970	0.410020	0.3692E-14
63	1.800	0.556	0.029680	0.407160	0.4940E-14	0.027780	0.417080	0.2860E-14
64	1.900	0.526	0.028170	0.426920	0.4172E-14	0.025820	0.428530	0.3194E-14
65	2.000	0.500	0.025900	0.437130	0.8578E-14	0.023780	0.436560	0.5236E-14
66	2.100	0.476	0.023580	0.437490	0.1851E-13	0.021790	0.438200	0.9989E-14
67	2.200	0.455	0.022420	0.444360	0.1639E-13	0.020360	0.440580	0.1177E-13
68	2.300	0.435	0.021070	0.445160	0.1077E-13	0.019140	0.447750	0.1103E-13
69	2.400	0.417	0.019690	0.451200	0.1450E-13	0.017780	0.450050	0.1418E-13
70	2.500	0.400	0.018470	0.456480	0.1498E-13	0.016530	0.451740	0.1504E-13
71	2.600	0.385	0.017290	0.458820	0.1253E-13	0.015420	0.450230	0.1302E-13
72	2.700	0.370	0.016210	0.458880	0.9327E-14	0.014680	0.454510	0.1028E-13
73	2.800	0.357	0.015230	0.460540	0.9822E-14	0.013880	0.455560	0.8866E-14
74	2.900	0.345	0.014190	0.459610	0.1000E-13	0.012970	0.452330	0.8551E-14
75	3.000	0.333	0.013100	0.455100	0.1297E-13	0.012040	0.448970	0.1033E-13
76	3.200	0.313	0.011390	0.458360	0.1348E-13	0.010520	0.449120	0.1167E-13
77	3.400	0.294	0.010510	0.472280	0.5130E-14	0.009730	0.454310	0.4936E-14
78	3.600	0.278	0.009560	0.467680	0.2360E-14	0.009000	0.457550	0.2635E-14
79	3.800	0.263	0.008730	0.465150	0.1909E-14	0.008200	0.454220	0.2479E-14
80	4.000	0.250	0.008000	0.460500	0.1315E-14	0.007410	0.448500	0.2456E-14

TABLE Q3-2 (Continued)

LIKELIHOOD RATIO TEST FOR FREQUENCY CLUSTERS

NUMBER		PERIOD (SEC)		FREQUENCY (HZ)		LIKELIHOOD RATIO N/LN	
FROM	TO	FROM	TO	FROM	TO	DAMP 0.04	DAMP 0.07
1	5	0.030	0.050	33.333	20.000	0.6628E-55	0.1725E-55
6	10	0.055	0.075	18.182	13.333	0.2711E-52	0.1298E-54
11	15	0.080	0.100	12.500	10.000	0.7456E-50	0.1794E-52
16	20	0.110	0.150	9.091	6.667	0.1661E-46	0.2825E-49
21	25	0.160	0.200	6.250	5.000	0.2629E-51	0.1072E-52
26	30	0.210	0.250	4.762	4.000	0.1582E-59	0.4302E-60
31	35	0.260	0.300	3.846	3.333	0.2006E-69	0.3044E-67
36	40	0.320	0.400	3.125	2.500	0.43 E-84	0.12 E-83
41	45	0.420	0.500	2.381	2.000	0.62 E-91	0.28 E-90
46	50	0.550	0.750	1.818	1.333	0.28 E-106	0.24 E-100
51	55	0.800	1.000	1.250	1.000	0.71 E-87	0.12 E-86
56	60	1.100	1.500	0.909	0.667	0.4012E-75	0.4887E-75
61	65	1.600	2.000	0.625	0.500	0.1034E-70	0.8916E-72
66	70	2.100	2.500	0.476	0.400	0.7097E-69	0.2763E-69
71	75	2.600	3.000	0.385	0.333	0.1489E-69	0.1048E-69
76	80	3.200	4.000	0.313	0.250	0.4097E-72	0.9243E-72
1	20	0.030	0.150	33.333	6.667	0.22 E-204	0.11 E-212
21	40	0.160	0.400	6.250	2.500	0.36 E-240	0.16 E-264
41	60	0.420	1.500	2.381	0.667	0.50 E-360	0.39 E-363
61	80	1.600	4.000	0.625	0.250	0.45 E-282	0.24 E-283
1	80	0.030	4.000	33.333	0.250	0.18 E-1087	0.17 E-1114

TABLE Q3-3

RESPONSE SPECTRUM PERCENT FRACTILE FOR SEQUOYAH, WATTS BAR, BELLEFONTE, AND PHIPPS BEND NUCLEAR PLANTS

NO.	PERIOD SEC	FREQUENCY HZ	PLANT: DAMPING:	SQN 1%	WBN 1%	BLN 4%	PBN 4%	SQN 5%	WBN 5%	BLN 7%	PBN 7%
1	0.030	33.333		75.90	75.73	75.73	87.02	76.08	76.08	76.08	87.26
2	0.035	28.571		75.56	78.46	77.99	88.37	73.96	77.52	77.86	88.37
3	0.040	25.000		77.52	80.19	79.36	89.11	71.93	78.54	79.15	89.12
4	0.045	22.222		76.72	83.09	82.03	90.94	71.18	80.38	81.19	90.43
5	0.050	20.000		77.83	84.72	83.46	91.76	69.55	81.24	82.23	91.12
6	0.055	18.182		77.00	86.22	85.38	93.16	67.78	82.19	83.36	92.03
7	0.060	16.667		79.42	88.55	87.01	94.22	67.01	83.66	84.95	93.11
8	0.065	15.385		79.47	89.13	87.4[illegible]	94.55	66.83	83.31	84.77	93.02
9	0.070	14.286		79.69	89.38	87.51	94.67	67.33	83.72	85.36	93.58
10	0.075	13.333		85.60	91.17	89.31	95.77	70.04	84.80	86.54	94.42
11	0.080	12.500		87.56	92.38	90.44	96.59	69.20	85.26	87.15	94.92
12	0.085	11.765		92.08	92.79	90.78	96.78	72.22	85.08	87.10	94.94
13	0.090	11.111		90.56	92.70	90.52	96.71	73.00	84.53	86.71	94.74
14	0.095	10.526		91.65	92.15	89.84	96.22	73.41	83.57	85.91	94.24
15	0.100	10.000		88.49	91.65	89.16	95.83	72.06	81.81	84.31	93.09
16	0.110	9.091		88.84	89.53	86.74	94.01	71.15	80.41	83.03	91.85
17	0.120	8.333		94.78	91.19	86.93	94.34	75.97	80.80	82.28	91.43
18	0.130	7.692		92.38	88.95	82.27	91.35	73.73	78.53	78.80	89.10
19	0.140	7.143		91.90	88.23	79.04	89.45	70.65	77.05	76.05	87.35
20	0.150	6.667		91.99	90.86	80.87	90.76	73.86	79.01	76.88	87.92
21	0.160	6.250		86.99	90.21	80.91	90.32	72.34	79.30	77.61	88.12
22	0.170	5.882		88.64	90.43	81.94	90.70	73.84	80.25	78.96	88.76
23	0.180	5.556		91.46	90.40	82.81	90.82	76.38	81.28	80.34	89.44
24	0.190	5.263		91.78	90.97	83.87	91.51	78.78	81.73	81.07	89.78
25	0.200	5.000		88.77	91.09	84.30	91.75	76.40	81.74	81.32	89.74
26	0.210	4.762		92.15	90.71	83.90	91.52	74.11	81.76	81.56	90.01
27	0.220	4.545		90.27	90.80	84.15	91.73	72.83	81.58	81.57	90.02
28	0.230	4.348		89.86	91.95	85.85	92.91	73.60	82.47	82.66	90.79
29	0.240	4.167		91.36	92.02	86.27	93.05	76.30	83.27	83.63	91.34
30	0.250	4.000		88.97	90.80	84.91	92.00	75.90	83.20	83.71	91.32
31	0.260	3.846		89.53	90.77	85.10	92.06	75.37	83.48	84.13	91.51
32	0.270	3.704		92.60	91.92	86.89	93.16	77.13	84.72	85.47	92.31
33	0.280	3.571		93.42	93.13	88.75	94.28	79.26	86.12	86.94	93.19
34	0.290	3.448		93.39	93.75	89.82	94.87	79.43	87.00	87.89	93.72
35	0.300	3.333		91.57	93.97	90.30	95.09	79.35	87.42	88.37	93.94
36	0.320	3.125		91.91	94.66	91.36	95.78	78.35	87.65	88.96	94.26
37	0.340	2.941		92.64	94.28	91.15	95.50	81.82	88.22	89.44	94.44
38	0.360	2.778		93.69	93.82	90.86	95.14	83.73	88.39	89.69	94.44
39	0.380	2.632		90.73	94.07	91.43	95.38	83.95	88.79	90.15	94.61
40	0.400	2.500		93.00	94.43	92.08	95.71	84.80	89.23	90.63	94.81
41	0.420	2.381		94.75	94.87	92.16	95.71	86.89	90.04	90.76	94.86
42	0.440	2.273		92.91	95.35	92.36	95.81	86.59	91.11	91.23	95.16
43	0.460	2.174		91.98	95.87	92.74	96.00	85.58	92.04	91.66	95.39
44	0.480	2.083		93.80	96.55	93.36	96.42	88.29	93.26	92.49	95.95
45	0.500	2.000		96.04	96.94	93.58	96.59	91.07	94.02	92.90	96.22
46	0.550	1.818		96.32	97.68	95.00	97.47	92.65	95.03	94.24	97.08
47	0.600	1.667		98.53	98.36	96.32	98.26	93.93	95.87	95.35	97.73
48	0.650	1.538		97.63	98.42	96.54	98.37	94.22	96.28	95.94	98.07
49	0.700	1.429		96.97	98.51	96.77	98.51	93.26	96.50	96.29	98.28
50	0.750	1.333		96.81	98.53	96.85	98.57	93.38	96.37	96.27	98.27
51	0.800	1.250		97.35	98.55	96.94	98.63	93.01	96.08	96.09	98.16
52	0.850	1.176		95.67	98.37	96.75	98.49	92.32	95.91	96.02	98.11
53	0.900	1.111		95.59	98.27	96.69	98.43	91.27	95.76	95.97	98.05
54	0.950	1.053		96.46	98.23	96.69	98.43	91.50	95.69	95.99	98.05
55	1.000	1.000		96.13	97.76	96.06	98.03	92.48	95.45	95.84	97.95
56	1.100	0.909		96.73	97.80	96.24	98.12	91.88	95.09	95.66	97.81
57	1.200	0.833		95.18	97.51	95.96	97.93	91.18	94.93	95.67	97.80
58	1.300	0.769		96.95	97.85	96.50	98.29	92.23	95.46	96.27	98.18
59	1.400	0.714		98.33	98.71	97.79	99.04	94.94	96.58	97.36	98.82
60	1.500	0.667		99.27	99.15	98.49	99.41	96.97	97.31	98.04	99.19
61	1.600	0.625		99.42	99.31	98.78	99.55	97.39	97.51	98.25	99.30
62	1.700	0.588		99.50	99.39	98.93	99.61	97.80	97.63	98.40	99.36
63	1.800	0.556		99.19	99.37	98.94	99.60	97.20	97.71	98.49	99.39
64	1.900	0.526		98.39	99.25	98.81	99.52	96.74	97.65	98.47	99.37
65	2.000	0.500		98.68	99.18	98.75	99.49	97.27	97.61	98.48	99.36
66	2.100	0.476		0.0	99.22	98.82	99.52	0.0	97.67	98.55	99.39
67	2.200	0.455		0.0	99.19	98.81	99.51	0.0	97.72	98.61	99.42
68	2.300	0.435		0.0	99.25	98.92	99.56	0.0	97.74	98.64	99.42
69	2.400	0.417		0.0	99.24	98.93	99.56	0.0	97.82	98.72	99.46
70	2.500	0.400		0.0	99.25	98.96	99.57	0.0	97.94	98.82	99.50
71	2.600	0.385		0.0	99.29	99.04	99.60	0.0	98.13	98.95	99.57
72	2.700	0.370		0.0	99.36	99.14	99.65	0.0	98.17	98.99	99.58
73	2.800	0.357		0.0	99.39	99.20	99.67	0.0	98.26	99.07	99.61
74	2.900	0.345		0.0	99.45	99.28	99.71	0.0	98.43	99.18	99.67
75	3.000	0.333		0.0	99.53	99.38	99.76	0.0	98.58	99.29	99.72
76	3.200	0.313		0.0	99.60	99.49	99.80	0.0	98.60	99.42	99.78
77	3.400	0.294		0.0	99.60	99.51	99.81	0.0	98.90	99.49	99.80
78	3.600	0.278		0.0	99.69	99.62	99.85	0.0	98.99	99.55	99.83
79	3.800	0.263		0.0	99.73	99.68	99.88	0.0	99.12	99.62	99.86
80	4.000	0.250		0.0	99.78	99.75	99.91	0.0	99.26	99.70	99.89

TABLE Q3-4

NORMAL AND LOGNORMAL DISTRIBUTION 16TH, 50TH, AND 84TH PERCENTILE SITE SPECIFIC NORMALIZED RESPONSE SPECTRUM VALUES

Spectral Period	Percent Fractile					
	Normal Distribution			Lognormal Distribution		
(Sec)	16th	50th	84th	16th	50th	84th
	For 4% Damping					
0.06	1.13	1.40	1.67	1.14	1.38	1.66
0.15	2.25	2.99	3.72	2.23	2.89	3.76
0.40	0.82	1.58	2.33	0.82	1.39	2.37
1.50	0.12	0.32	0.51	0.15	0.27	0.49
4.0	0.016	0.060	0.10	0.021	0.046	0.10
	For 7% Damping					
0.06	1.11	1.31	1.51	1.12	1.30	1.50
0.15	1.94	2.49	3.05	1.95	2.44	3.04
0.40	0.71	1.35	1.98	0.72	1.20	1.99
1.50	0.11	0.28	0.45	0.13	0.24	0.43
4.0	0.018	0.057	0.095	0.022	0.045	0.093

TABLE Q3-5

LIKELIHOOD RATIO TEST - NORMAL VS LOGNORMAL DISTRIBUTION - NORMALIZED SPECTRA

NO.	PERIOD SEC	FREQUENCY HZ	FOR DAMPING RATIO: 0.04 STD DEV NORMAL	STD DEV LOGNORMAL	LIKELIHOOD RATIO N/LN	FOR DAMPING RATIO: 0.07 STD DEV NORMAL	STD DEV LOGNORMAL	LIKELIHOOD RATIO N/LN
1	0.030	33.333	0.079070	0.030410	0.6701E-36	0.073420	0.028520	0.7056E-36
2	0.035	28.571	0.241530	0.074400	0.9529E-38	0.183470	0.060280	0.3044E-37
3	0.040	25.000	0.427470	0.110890	0.5113E-39	0.294160	0.085710	0.3967E-38
4	0.045	22.222	0.414550	0.106670	0.7176E-39	0.274840	0.081070	0.8692E-38
5	0.050	20.000	0.383980	0.102890	0.6694E-38	0.251050	0.075840	0.4413E-37
6	0.055	18.182	0.330010	0.095300	0.1721E-36	0.231700	0.072620	0.4299E-36
7	0.060	16.667	0.270530	0.080540	0.9971E-36	0.199570	0.064060	0.1493E-35
8	0.065	15.385	0.339110	0.091250	0.4367E-36	0.231750	0.070110	0.1431E-35
9	0.070	14.286	0.430100	0.109370	0.5975E-36	0.306060	0.087180	0.1329E-35
10	0.075	13.333	0.498410	0.118890	0.2181E-36	0.323310	0.089010	0.1320E-35
11	0.080	12.500	0.518280	0.115420	0.2215E-36	0.374350	0.096140	0.7577E-36
12	0.085	11.765	0.514080	0.115610	0.9524E-36	0.419560	0.103820	0.9445E-36
13	0.090	11.111	0.588610	0.123000	0.7708E-36	0.436210	0.102840	0.8972E-36
14	0.095	10.526	0.618860	0.119850	0.2477E-36	0.455360	0.101710	0.7270E-36
15	0.100	10.000	0.677220	0.125440	0.3111E-36	0.485620	0.107360	0.1774E-35
16	0.110	9.091	0.703690	0.136500	0.5806E-35	0.518700	0.116320	0.7779E-35
17	0.120	8.333	0.606370	0.113880	0.8481E-35	0.474730	0.101740	0.6903E-35
18	0.130	7.692	0.661520	0.108710	0.5472E-35	0.450020	0.087810	0.6791E-35
19	0.140	7.143	0.657740	0.099680	0.1093E-34	0.496560	0.086750	0.3401E-35
20	0.150	6.667	0.737150	0.113160	0.8076E-35	0.555090	0.096170	0.2132E-35
21	0.160	6.250	0.828650	0.125880	0.2547E-35	0.604800	0.107300	0.1888E-35
22	0.170	5.882	0.865930	0.135850	0.1714E-35	0.592340	0.114810	0.5501E-35
23	0.180	5.556	0.696100	0.126550	0.1244E-34	0.523710	0.112030	0.1813E-34
24	0.190	5.263	0.696410	0.132720	0.2034E-34	0.532720	0.119590	0.2993E-34
25	0.200	5.000	0.719730	0.138680	0.2030E-34	0.570380	0.131550	0.3314E-34
26	0.210	4.762	0.721960	0.134870	0.1634E-34	0.557220	0.126010	0.2700E-34
27	0.220	4.545	0.695190	0.129730	0.1898E-34	0.568360	0.128970	0.3396E-34
28	0.230	4.348	0.748100	0.142740	0.1356E-34	0.615340	0.138030	0.1448E-34
29	0.240	4.167	0.871970	0.168420	0.8606E-35	0.671010	0.152130	0.7496E-35
30	0.250	4.000	0.924520	0.176190	0.1094E-34	0.702960	0.160680	0.7459E-35
31	0.260	3.846	0.927520	0.177360	0.8559E-35	0.706140	0.165510	0.8408E-35
32	0.270	3.704	0.872850	0.180950	0.1135E-34	0.689280	0.170840	0.8843E-35
33	0.280	3.571	0.800200	0.178220	0.8701E-35	0.657400	0.169890	0.5356E-35
34	0.290	3.448	0.786730	0.185370	0.7295E-35	0.641820	0.175240	0.6207E-35
35	0.300	3.333	0.789390	0.192790	0.6596E-35	0.633530	0.182570	0.9915E-35
36	0.320	3.125	0.768000	0.183220	0.1953E-35	0.620720	0.180390	0.5698E-35
37	0.340	2.941	0.807580	0.202850	0.2702E-35	0.626770	0.193690	0.7838E-35
38	0.360	2.778	0.892190	0.229460	0.1299E-35	0.670370	0.211950	0.3149E-35
39	0.380	2.632	0.848460	0.241750	0.2825E-35	0.665110	0.219890	0.2090E-35
40	0.400	2.500	0.758430	0.231660	0.2497E-35	0.639550	0.221550	0.1305E-35

TABLE Q3-5 (Continued)

LIKELIHOOD RATIO TEST - NORMAL VS LOGNORMAL DISTRIBUTION - NORMALIZED SPECTRA

			FOR DAMPING RATIO: 0.04			FOR DAMPING RATIO: 0.07		
NO.	PERIOD SEC	FREQUENCY HZ	STD DEV NORMAL	STD DEV LOGNORMAL	LIKELIHOOD RATIO N/LN	STD DEV NORMAL	STD DEV LOGNORMAL	LIKELIHOOD RATIO N/LN
41	0.420	2.381	0.730480	0.237100	0.1659E-35	0.611260	0.222650	0.9213E-36
42	0.440	2.273	0.699300	0.236670	0.7956E-36	0.576600	0.221670	0.6131E-36
43	0.460	2.174	0.673430	0.239800	0.2785E-36	0.555060	0.224870	0.2848E-36
44	0.480	2.083	0.656710	0.232980	0.4213E-37	0.522860	0.218400	0.9848E-37
45	0.500	2.000	0.702900	0.230980	0.3005E-38	0.513770	0.214630	0.2705E-37
46	0.550	1.818	0.601960	0.224490	0.8649E-39	0.489890	0.217740	0.2769E-38
47	0.600	1.667	0.604350	0.234890	0.4545E-40	0.489240	0.226230	0.1681E-39
48	0.650	1.538	0.663250	0.242240	0.5278E-42	0.468500	0.227070	0.3215E-40
49	0.700	1.429	0.573300	0.246200	0.6346E-41	0.422950	0.232320	0.1150E-39
50	0.750	1.333	0.444420	0.249610	0.2131E-38	0.356820	0.238080	0.4244E-38
51	0.800	1.250	0.393860	0.263190	0.7690E-37	0.332190	0.247950	0.2772E-37
52	0.850	1.176	0.368180	0.265330	0.6229E-37	0.313560	0.250660	0.3040E-37
53	0.900	1.111	0.357500	0.269610	0.2017E-37	0.298670	0.254930	0.2513E-37
54	0.950	1.053	0.324430	0.272370	0.9930E-37	0.277290	0.259280	0.6459E-37
55	1.000	1.000	0.300530	0.283740	0.6920E-36	0.255940	0.263850	0.2291E-36
56	1.100	0.909	0.281150	0.290470	0.3891E-36	0.242530	0.276110	0.2254E-36
57	1.200	0.833	0.305240	0.316530	0.4908E-37	0.245550	0.298350	0.1282E-36
58	1.300	0.769	0.293330	0.302040	0.7156E-38	0.225590	0.286660	0.3996E-37
59	1.400	0.714	0.231780	0.268530	0.8438E-38	0.189370	0.260740	0.2486E-37
60	1.500	0.667	0.194210	0.265170	0.5318E-37	0.167000	0.256940	0.4970E-37
61	1.600	0.625	0.174520	0.264980	0.1045E-36	0.154590	0.258770	0.7143E-37
62	1.700	0.588	0.164090	0.256620	0.2276E-37	0.149260	0.258140	0.1637E-37
63	1.800	0.556	0.158130	0.265250	0.9217E-38	0.143800	0.266550	0.6859E-38
64	1.900	0.526	0.160320	0.283970	0.2406E-38	0.136070	0.276310	0.6056E-38
65	2.000	0.500	0.151470	0.302020	0.6542E-38	0.127490	0.289300	0.1294E-37
66	2.100	0.476	0.136340	0.308640	0.3265E-37	0.116990	0.297320	0.4392E-37
67	2.200	0.455	0.128370	0.315080	0.4255E-37	0.108720	0.303110	0.8548E-37
68	2.300	0.435	0.122160	0.313730	0.1724E-37	0.102280	0.307800	0.7698E-37
69	2.400	0.417	0.115630	0.320980	0.2121E-37	0.096350	0.313230	0.9493E-37
70	2.500	0.400	0.105760	0.321110	0.3158E-37	0.088290	0.314430	0.1469E-36
71	2.600	0.385	0.094330	0.323110	0.9553E-37	0.080560	0.315180	0.2624E-36
72	2.700	0.370	0.083420	0.324360	0.3578E-36	0.073810	0.316320	0.4818E-36
73	2.800	0.357	0.077370	0.328160	0.6498E-36	0.067620	0.317060	0.9449E-36
74	2.900	0.345	0.071530	0.333990	0.1349E-35	0.062160	0.314890	0.1404E-35
75	3.000	0.333	0.068070	0.340470	0.1692E-35	0.058890	0.315870	0.1314E-35
76	3.200	0.313	0.063280	0.347980	0.4524E-36	0.054770	0.321170	0.4479E-36
77	3.400	0.294	0.061370	0.357920	0.4510E-37	0.052230	0.324590	0.8354E-37
78	3.600	0.278	0.056840	0.351800	0.1069E-37	0.048300	0.324170	0.3607E-37
79	3.800	0.263	0.052410	0.348580	0.6094E-38	0.043550	0.321170	0.4225E-37
80	4.000	0.250	0.044030	0.340500	0.2836E-37	0.038180	0.315440	0.8014E-37

TABLE Q3-5 (Continued)

LIKELIHOOD RATIO TEST FOR FREQUENCY CLUSTERS

NUMBER FROM	NUMBER TO	PERIOD (SEC) FROM	PERIOD (SEC) TO	FREQUENCY (HZ) FROM	FREQUENCY (HZ) TO	LIKELIHOOD RATIO N/LN DAMP 0.04	LIKELIHOOD RATIO N/LN DAMP 0.07
1	5	0.030	0.050	33.333	20.000	0.16 E-190	0.33 E-187
6	10	0.055	0.075	18.182	13.333	0.98 E-182	0.16 E-179
11	15	0.080	0.100	12.500	10.000	0.13 E-181	0.83 E-180
16	20	0.110	0.150	9.091	6.667	0.24 E-175	0.26 E-176
21	25	0.160	0.200	6.250	5.000	0.22 E-176	0.19 E-174
26	30	0.210	0.250	4.762	4.000	0.40 E-174	0.74 E-174
31	35	0.260	0.300	3.846	3.333	0.41 E-175	0.25 E-175
36	40	0.320	0.400	3.125	2.500	0.48 E-178	0.38 E-177
41	45	0.420	0.500	2.381	2.000	0.47 E-184	0.43 E-183
46	50	0.550	0.750	1.818	1.333	0.28 E-201	0.73 E-197
51	55	0.800	1.000	1.250	1.000	0.66 E-185	0.31 E-186
56	60	1.100	1.500	0.909	0.667	0.61 E-187	0.14 E-185
61	65	1.600	2.000	0.625	0.500	0.34 E-189	0.63 E-189
66	70	2.100	2.500	0.476	0.400	0.16 E-187	0.40 E-185
71	75	2.600	3.000	0.385	0.333	0.51 E-181	0.28 E-180
76	80	3.200	4.000	0.313	0.250	0.38 E-187	0.46 E-185
1	20	0.030	0.150	33.333	6.667	0.46 E-730	0.12 E-723
21	40	0.160	0.400	6.250	2.500	0.17 E-703	0.13 E-701
41	60	0.420	1.500	2.381	0.667	0.53 E-758	0.14 E-744
61	80	1.600	4.000	0.625	0.250	0.11 E-765	0.26 E-740
1	80	0.030	4.000	33.333	0.250	0.45 E-2938	0.54 E-2911

TABLE Q3-6

VARIOUS PEAK ACCELERATION RESULTS FROM THE SENSITIVITY STUDY

Data Base Used	Peak Acceleration (g)		
	16th Percentile	50th Percentile	84th Percentile
Original 13 records	.047	.101	.215
Original + 4 high pairs	.058	.133	.306
Original + 2 high pairs	.052	.118	.266
Original + 2 low pairs	.037	.084	.195
Original + 4 low pairs	.031	.074	.176
Original + 1 high & 1 low pair	.043	.100	.232
Original + 2 high & 2 low pairs	.040	.099	.244

MAXIMUM, MINIMUM, 16TH, 50TH, AND 84TH PERCENTILE RESPONSE SPECTRA FOR THIRTEEN UNITED STATES AND ITALY EARTHQUAKES
LOGNORMAL DISTRIBUTION - 4% DAMPING

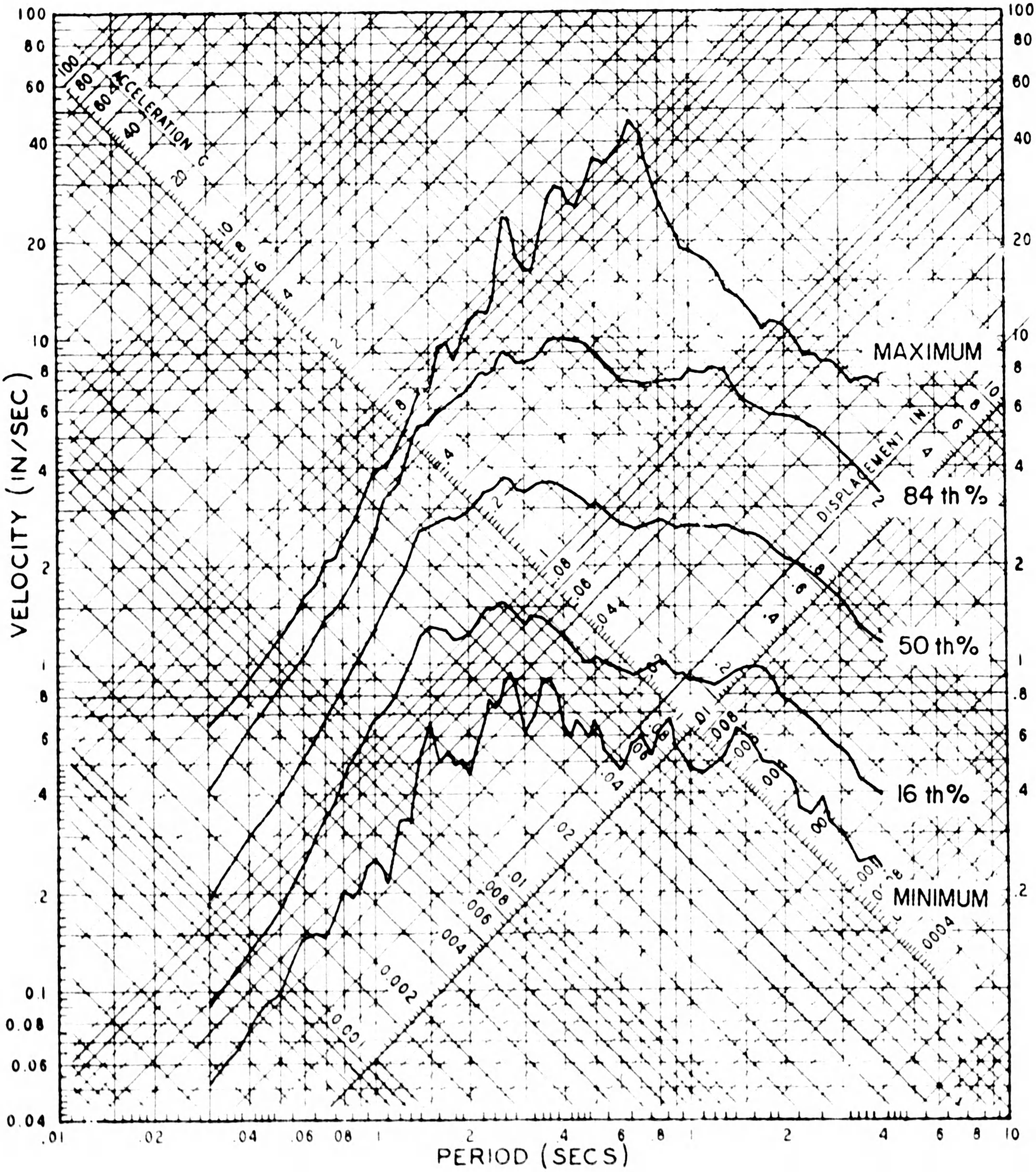


FIG. Q3-1

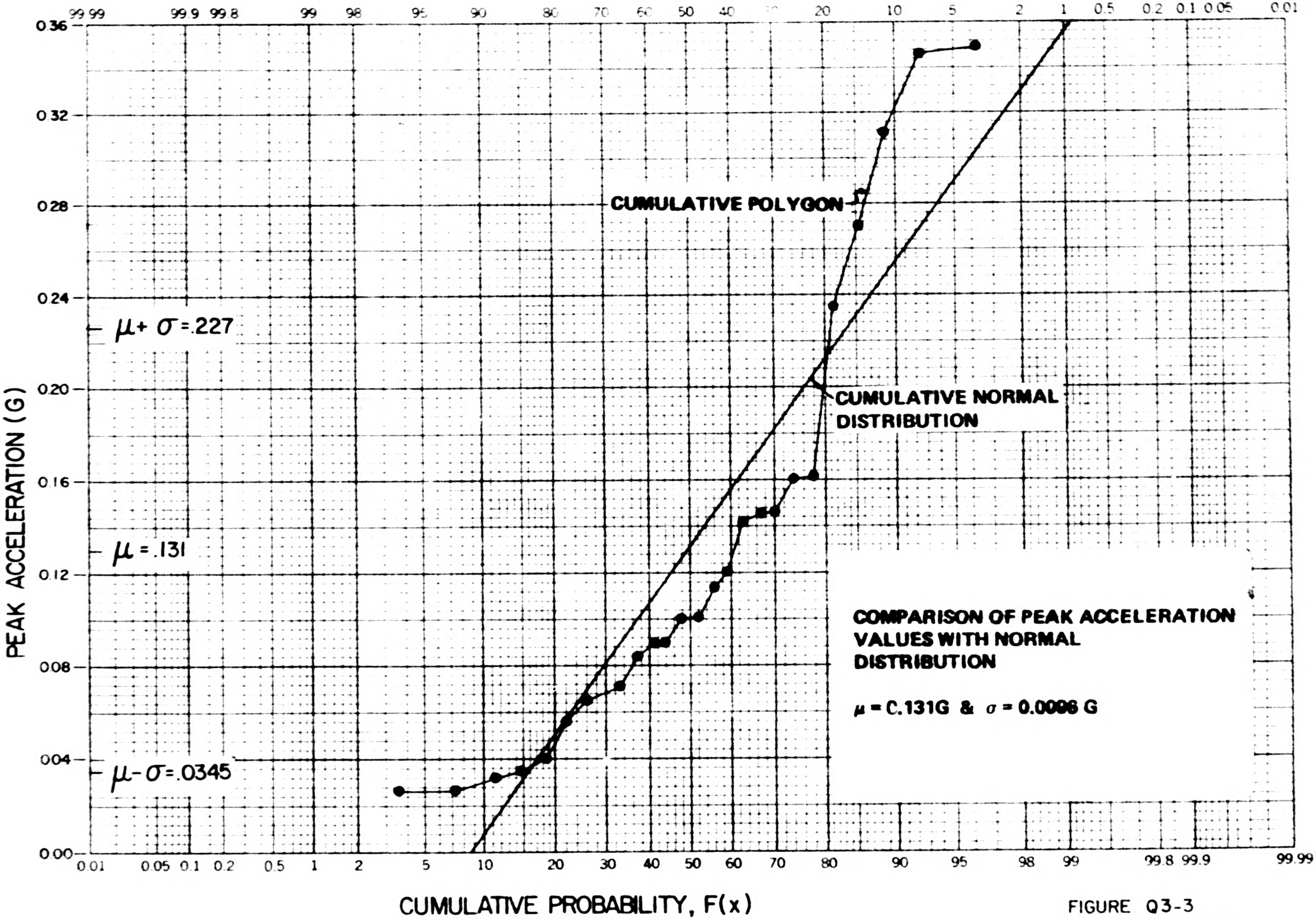

CUMULATIVE POLYGON
CUMULATIVE NORMAL DISTRIBUTION
COMPARISON OF PEAK ACCELERATION VALUES WITH NORMAL DISTRIBUTION
μ = C.131G & σ = 0.0096 G
μ+ σ =.227
μ =.131
μ- σ=.0345
PEAK ACCELERATION (G)
CUMULATIVE PROBABILITY, F(x)

FIGURE Q3-3

K•E PROBABILITY X 90 DIVISIONS KEUFFEL & ESSER CO.

46 8000

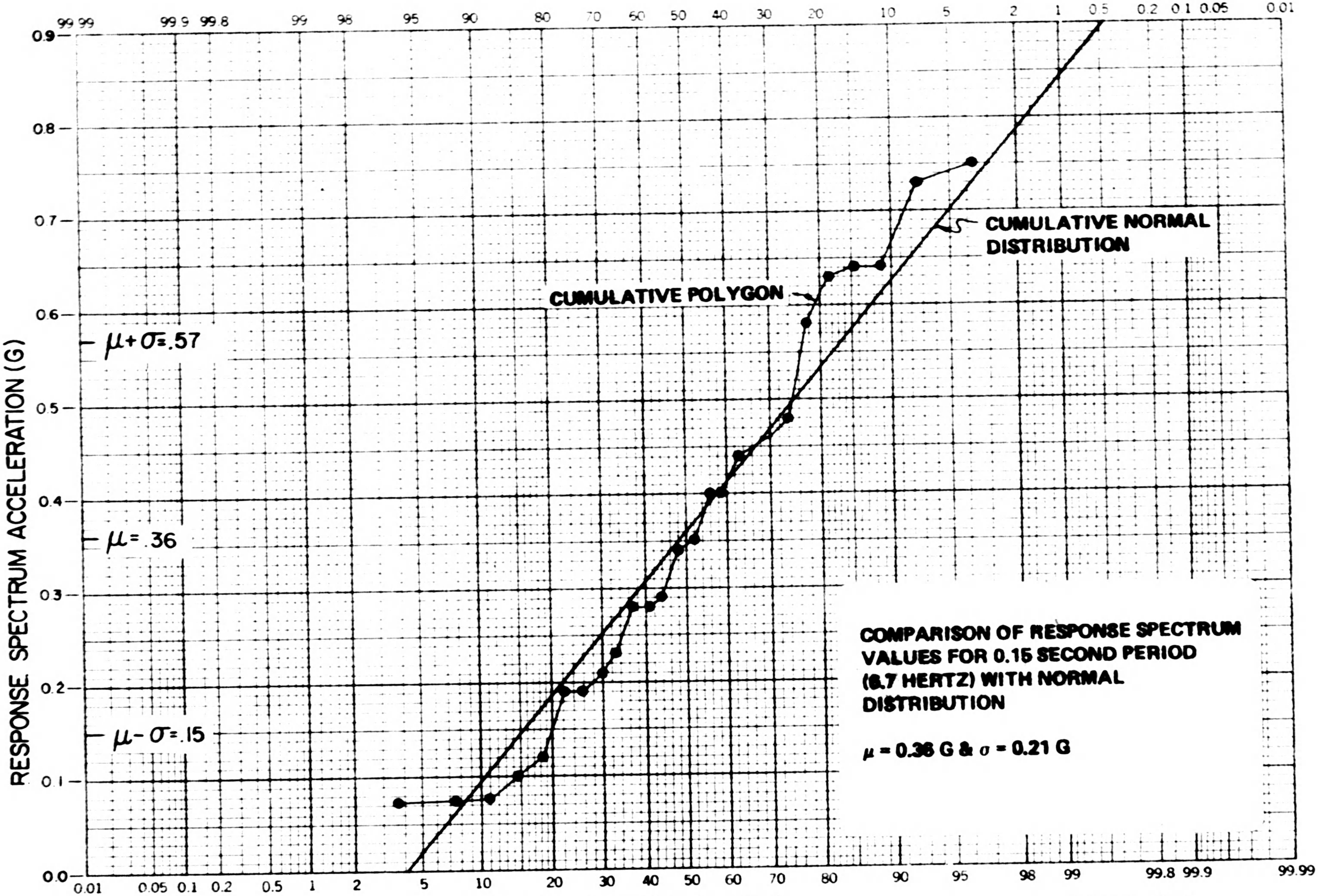


FIGURE Q3-4

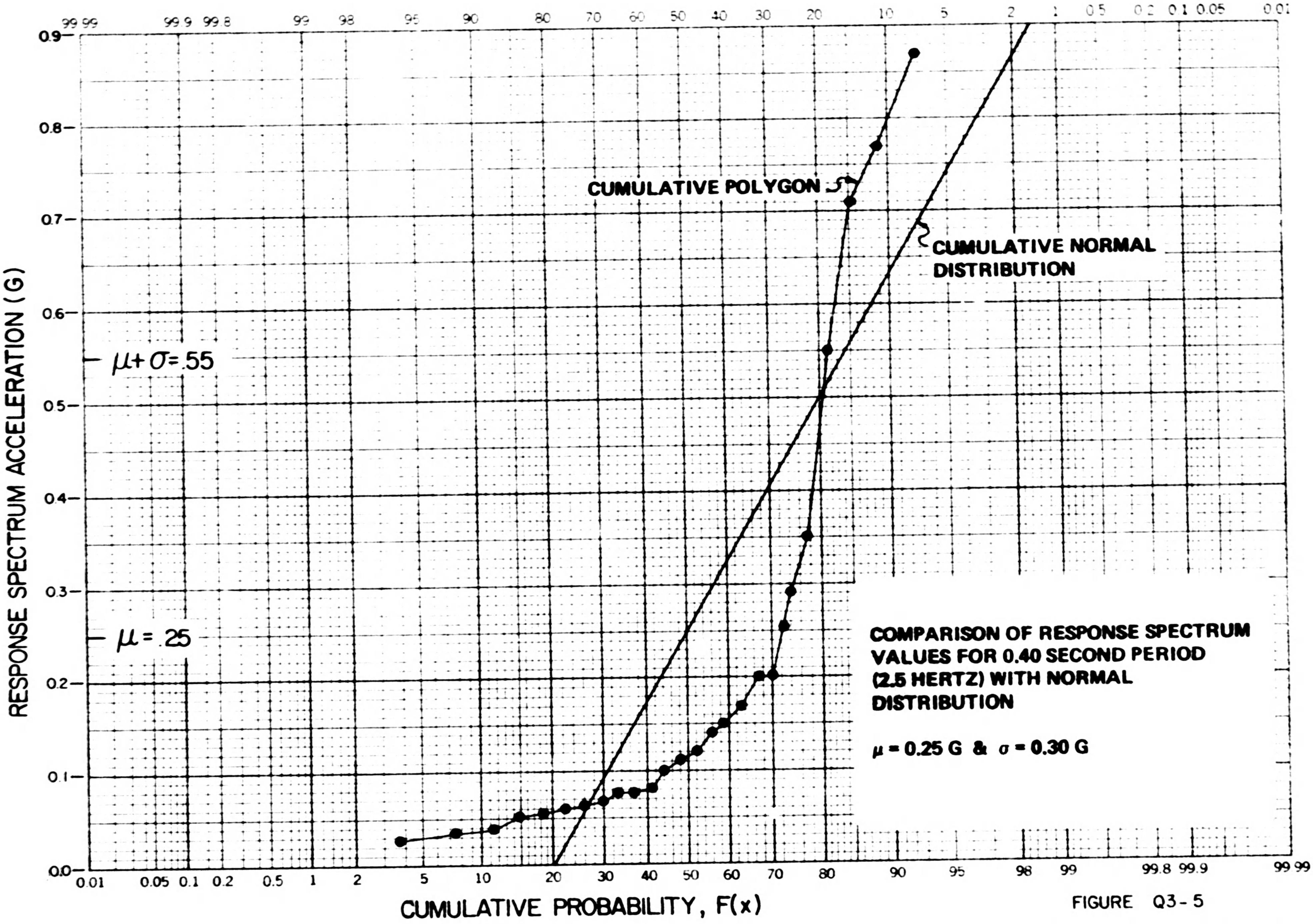

RESPONSE SPECTRUM ACCELERATION (G)
CUMULATIVE PROBABILITY, F(x)
CUMULATIVE POLYGON
CUMULATIVE NORMAL DISTRIBUTION
μ+σ=.55
μ=.25
COMPARISON OF RESPONSE SPECTRUM VALUES FOR 0.40 SECOND PERIOD (2.5 HERTZ) WITH NORMAL DISTRIBUTION
μ = 0.25 G & σ = 0.30 G

FIGURE Q3-5

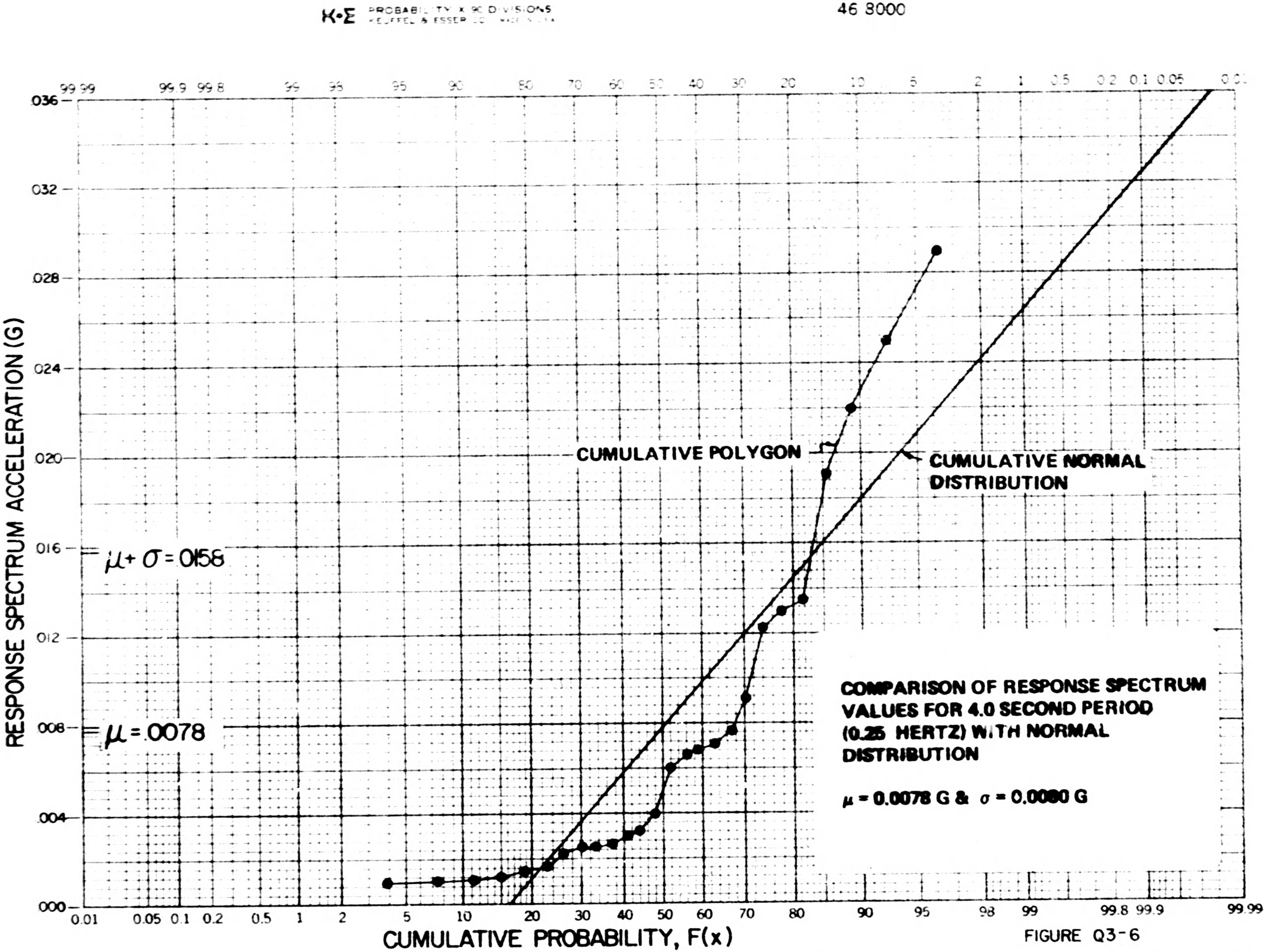
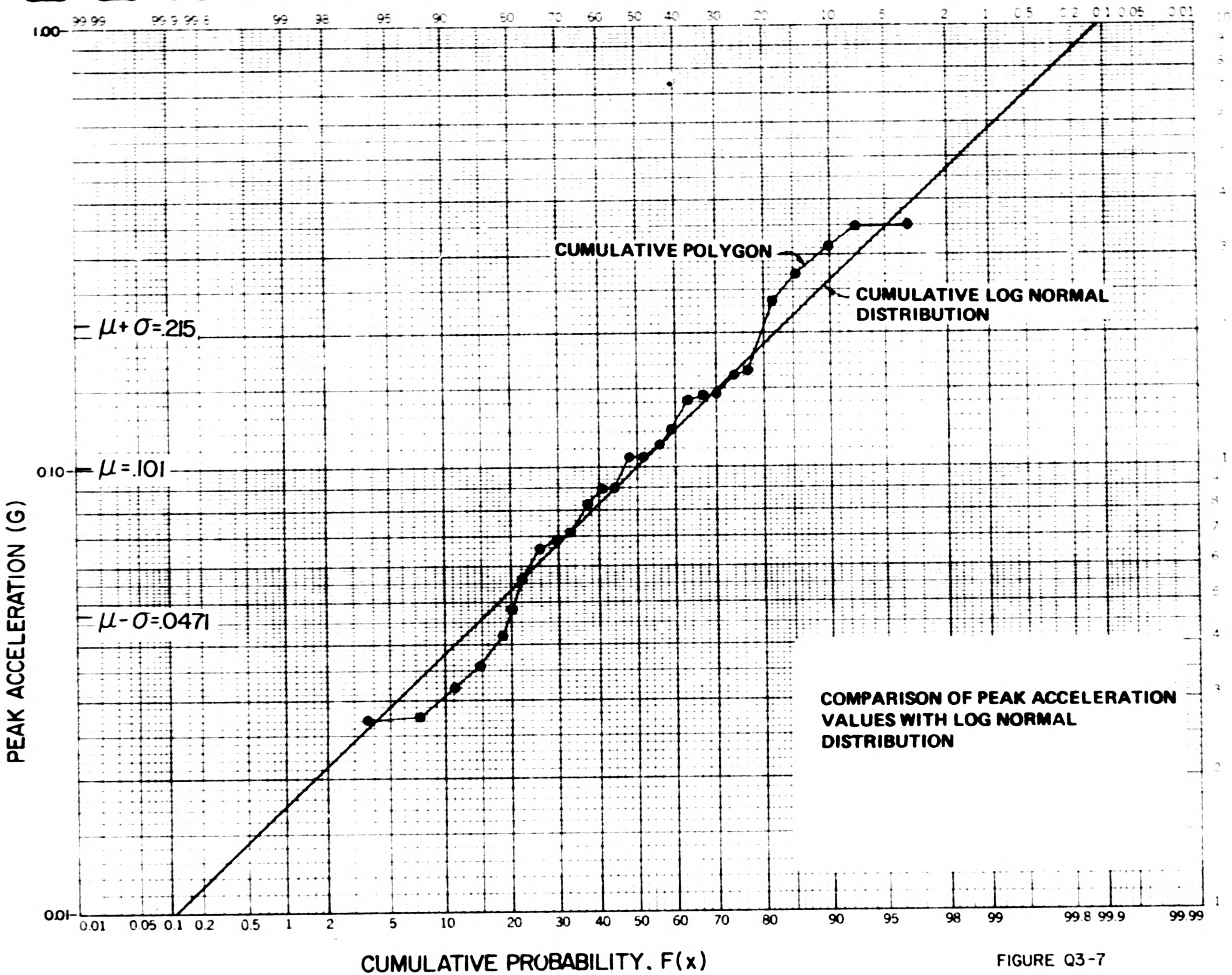

46 8000
RESPONSE SPECTRUM ACCELERATION (G)
CUMULATIVE PROBABILITY, F(x)
CUMULATIVE POLYGON
CUMULATIVE NORMAL DISTRIBUTION
μ+ σ = 0158
μ = .0078
COMPARISON OF RESPONSE SPECTRUM VALUES FOR 4.0 SECOND PERIOD (0.25 HERTZ) WITH NORMAL DISTRIBUTION
μ = 0.0078 G & σ = 0.0080 G

FIGURE Q3-6



CUMULATIVE POLYGON
CUMULATIVE LOG NORMAL DISTRIBUTION
$\mu + \sigma = .215$
$\mu = .101$
$\mu - \sigma = .0471$
COMPARISON OF PEAK ACCELERATION VALUES WITH LOG NORMAL DISTRIBUTION
PEAK ACCELERATION (G)
CUMULATIVE PROBABILITY, F(x)
1.00
0.10
0.01
0.01 0.05 0.1 0.2 0.5 1 2 5 10 20 30 40 50 60 70 80 90 95 98 99 99.8 99.9 99.99

FIGURE Q3-7

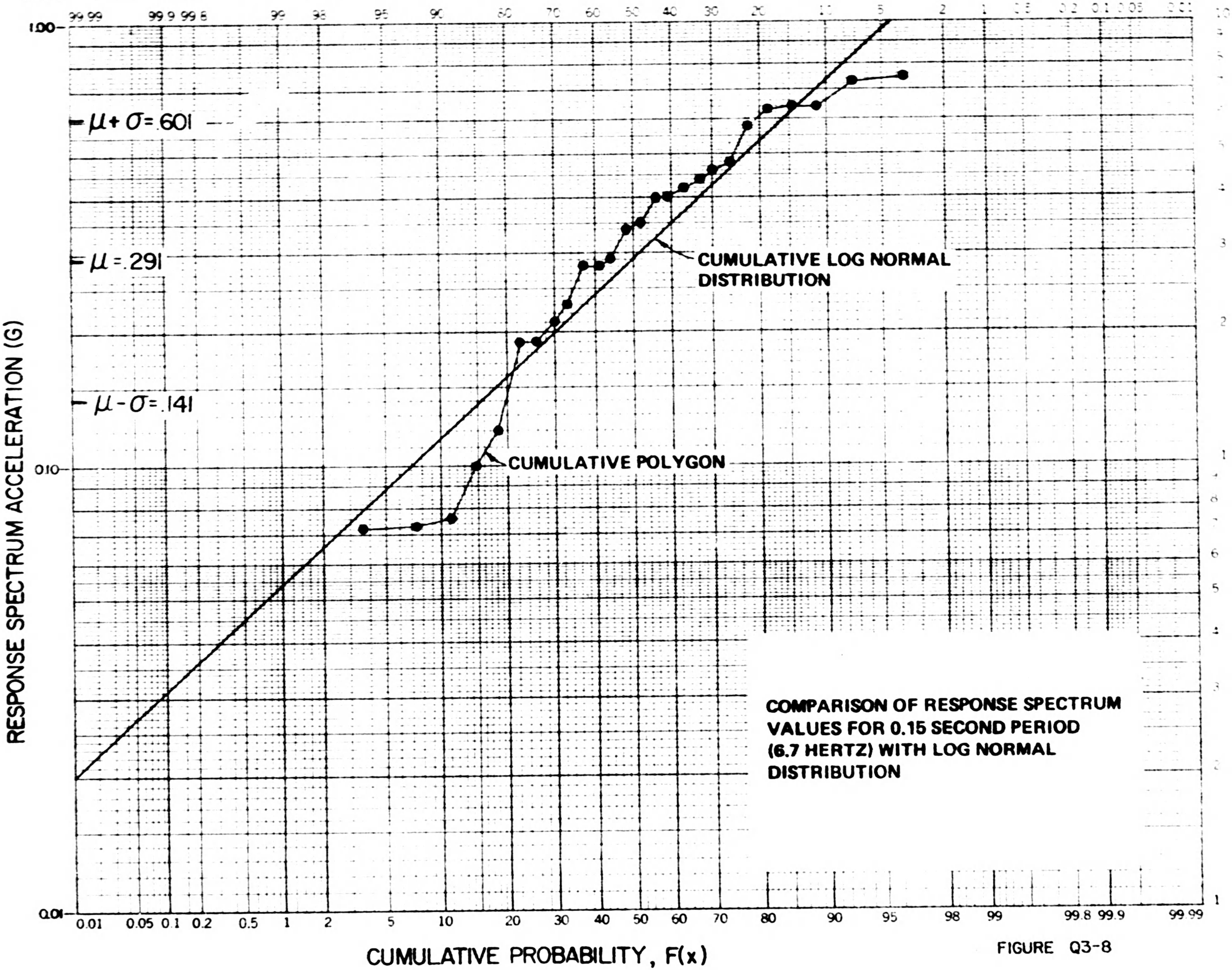
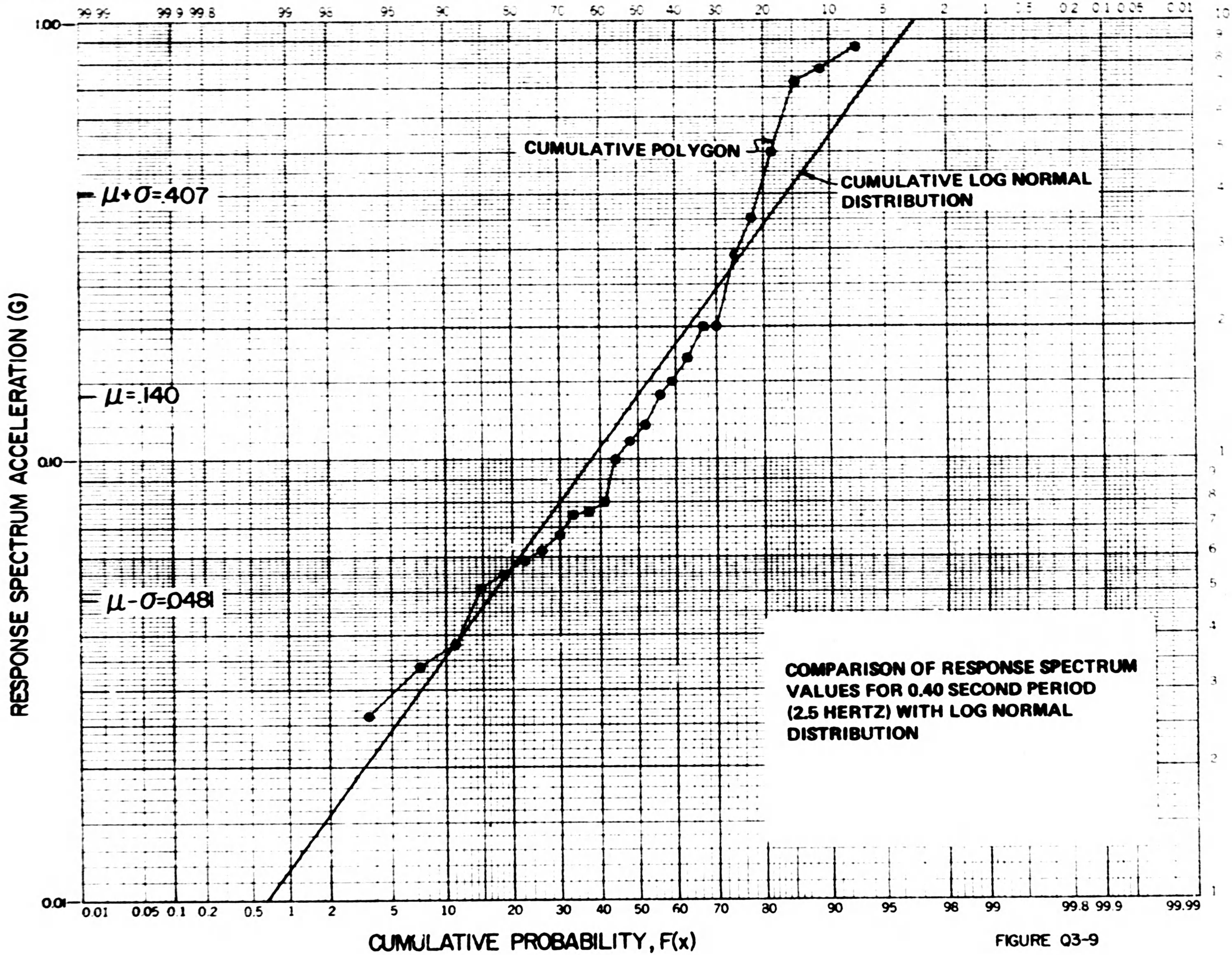

COMPARISON OF RESPONSE SPECTRUM VALUES FOR 0.15 SECOND PERIOD (6.7 HERTZ) WITH LOG NORMAL DISTRIBUTION
$\mu + \sigma = .601$
$\mu = .291$
$\mu - \sigma = .141$
CUMULATIVE LOG NORMAL DISTRIBUTION
CUMULATIVE POLYGON
RESPONSE SPECTRUM ACCELERATION (G)
CUMULATIVE PROBABILITY, F(x)
1.00
0.10
0.01

FIGURE Q3-8



COMPARISON OF RESPONSE SPECTRUM VALUES FOR 0.40 SECOND PERIOD (2.5 HERTZ) WITH LOG NORMAL DISTRIBUTION

FIGURE Q3-9

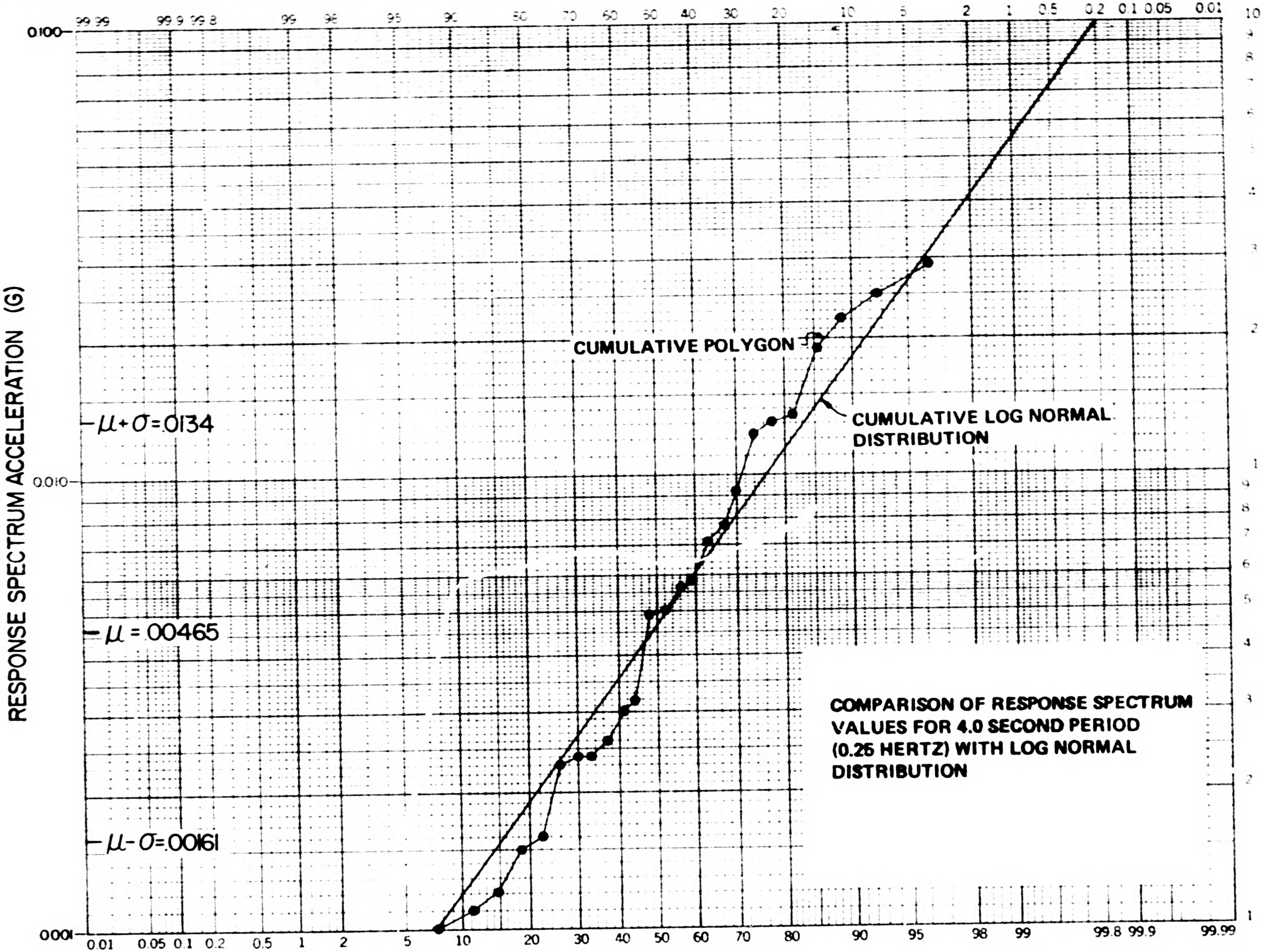

RESPONSE SPECTRUM ACCELERATION (G)
CUMULATIVE PROBABILITY, F(x)
0.100
0.010
0.001
$\mu + \sigma = .0134$
$\mu = .00465$
$\mu - \sigma = .00161$
CUMULATIVE POLYGON
CUMULATIVE LOG NORMAL DISTRIBUTION
COMPARISON OF RESPONSE SPECTRUM VALUES FOR 4.0 SECOND PERIOD (0.25 HERTZ) WITH LOG NORMAL DISTRIBUTION

FIGURE Q3-10