SEISMIC DESIGN DECISION ANALYSIS

NSF GRANTS GK-27955 and GI-29936

GROUND MOTION AMPLIFICATION STUDIES
BURSA, TURKEY

by

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September 1974

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PREFACE

This report is No. 16 in a series of reports prepared under Grants GK-27955 and GI-29936 from the National Science Foundation. A list of previous reports follows this preface.

The studies described in this report were conceived in the winter months of 1971-72. An earlier report by Dr. Tezcan had documented the evidence of apparent soil amplification at Bursa during the 1970 Gediz earthquake, and several engineers from the United States--including Dr. George Housner of the California Institute of Technology and the senior author--had remarked that the soil and geological conditions at the site should be better documented. Dr. Tezcan responded with a proposed program involving field work by agencies of the Turkish government plus data gathering efforts by himself and colleagues in Istanbul, with the effort to be co-ordinated at the Bosphorus University under his direction. Modest outside financial support was required for this proposal. Several means of arranging this support were considered. One possibility was to use funds available to the NEEA Committee on Natural Hazards; this committee had sponsored preliminary site studies of earthquake damage immediately following the Gediz earthquake and could logically support follow-up studies. Prof. Roy W. Clough (University of California - Berkeley), chairman of this committee, contacted the members who agreed to this arrangement. However, after further discussion it appeared best and simplest to include the field studies and associated theoretical interpretations as a task under an extension to Grant GI-29936 from NSF-RANN. Thus appreciation for the financial arrangements must be expressed both to Prof. Clough and his NEEA committee and to Dr. Charles Thiel of NSF-RANN.

The work at MIT under Grant GI-29936 has been under the general direction of Dr. Robert V. Whitman, Professor of Civil Engineering and Head of the Structural Engineering Division. During the period June, 1972 through June, 1973, the Bursa studies were supervised by Dr. John T. Christian, Associate Professor of Civil Engineering. Since June, 1973, Dr. Whitman has provided direct supervision while Dr. Christian has no longer been formally connected with MIT but has continued to provide input to the study. The theoretical studies at MIT were carried out by Messrs. Luis Garcia and Mishac Yegian, both graduate students and Research Assistants.

The work in Turkey was carried out partly under a subcontract from MIT to Dr. Tezcan at Bosphorus University; financial assistance was also provided by Bosphorus University, the Turkish State Waterworks Department (DSI), the State Institute of Mineral Search and Study (MTA) and Kandilli Observatory. Dr. Christian visited Istanbul and Bursa in June, 1972 to participate in the planning of the field work. Dr. Christian and Dr. Whitman met with Dr. Tezcan in Rome during the 5th World Conference on Earthquake Engineering in June, 1973 to review both the work in Turkey and the associated theoretical studies at MIT.

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2. Leslie, S. K. and J. M. Biggs, "Earthquake Code Evolution and the Effect of Seismic Design on the Cost of Buildings", Department of Civil Engineering Research Report R72-20, M.I.T., May 1972.

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15. Panoussis, G., Seismic Reliability of Lifeline Networks, Department of Civil Engineering Research Report R74-57, M.I.T., September 1974 (in preparation).

Chapter 1

BACKGROUND

1.1 DAMAGE AT TOFAS FACTORY FROM GEDIZ STUDY

Tezcan and Ipek (1973) have described the events that occurred near Bursa, Turkey, during and after the Gediz earthquake of 28 March 1970. (See Fig. 1.1 for general locations.) Briefly, the epicenter of the magnitude 7.1 earthquake was about 135 km from an automobile factory located on the Bursa Plain. Severe damage, typical of epicentral regions, was experienced in about Gediz. There was very little damage of any sort in the city of Bursa itself or in the surrounding area, the modified Marcalli intensity was V in this general region. However structures of the TOFAS automobile factory 8 km. north of Bursa experienced severe damage and in some cases collapse. Table 1.1 summarizes the damage at the TOFAS factory.

The most severe damage and collapse occured in the garage and paint workshop, which was under construction at the time of the earthquake. The general arrangement of this reinforced concrete building is shown in Fig. 1.2. There were three separate portions of the workshop. The frames of block A had been completed, and the brick exterior walls were in place. The end frame of this part collapsed while other frames were mildly damaged, and all walls toppled. The frames of block B had been poured from 3 to 10 days before the earthquake, and the erection scaffolding was still in place; this part experienced only minor cracking. The frames of block C had been completed and the erection scaffolding removed, but walls had not yet been constructed. This part collapsed completely.

1.2 GROUND MOTION OBSERVATIONS DURING AFTERSHOCKS

Inasmuch as there were no recordings instruments at the site, there are no recordings of the motions caused by the Gediz earthquake. A SMAC strong motion accelerograph and a Willmore velocity seismograph were installed in the factory area following the earthquake. The Willmore seismograph recorded the vibrations from three small aftershocks which occurred on 25 April 1970. Figure 1.3 shows the spectral acceleration computed from one of these recordings; the plot indicates an unusually strong peak near T = 1.2 seconds. The SMAC strong motion instrument triggered during these aftershocks, but the recorded motions were too small to permit analysis.

A team of engineers (including Dr. J. Penzien of the University of California, Berkeley) was inspecting the site when an aftershock occurred on 19 April 1970. Those sitting in a car at the moment of the aftershock felt the sloshing of gasoline within the tank of the car. All of the engineers in the inspection group were unanimous in feeling that the period of the motion was unusually long.

1.3 ANALYSIS OF DAMAGE

A detailed study of the TOFAS factory buildings was undertaken by Tezcan and Ipek, (1973).

The transverse natural period of the paint and garage workshop was computed to be 1.25 seconds. The yield limit lateral load capacity of the frames was 6.3% of the weight of the structure, while the collapse lateral load capacity was 9.4% of the weight.

Periods were also estimated for other structures in the factory. For the office buildings and service blocks, which experienced no damage, the periods were estimated to be in the range of 0.1 to 0.2 seconds. For the powerhouse and kitchen, which experienced some cracking of concrete, the periods were in the range of 0.6 to 0.8 seconds. Steel workshop buildings, where bolts and bracing and anchor plates failed, had periods in the range of 2.5 to 4 seconds.

All in all, the study indicated unusual damage to longer period buildings, especially a building with a period of about 1.2 seconds. Using several approaches of backcalculating ground motions from damage, the peakground acceleration was estimated to have been about 0.04g.

1.4 SCOPE OF THIS REPORT

The coincidence between the predominant period of motion observed in the aftershock and the computed natural period for the collapsed building strongly suggested that there might be some site effect causing amplification of earthquake ground motions at a period of about 1.2 seconds. The study described in this report was aimed at the question: can this hypothesis be given further verification by theoretical analysis of site conditions? The study consisted of field studies to determine the soil and geological conditions at the site, plus theoretical amplification studies. The field studies, carried out in Turkey under the direction of Dr. Tezcan, are summarized in Chapter 2. The theoretical studies carried out at MIT are discussed in Chapter 3 and Appendices A and B. Concurrent theoretical studies have also been completed at the University of California in Berkeley. Final conclusions will be presented in a joint report.

Chapter 2

GEOPHYSICAL AND GEOTECHNICAL STUDIES

2.1 INTRODUCTION

This chapter summarizes the content of two reports prepared by Dr. Tezcan. The first report, completed in March 1973, is a compilation of geological and geotechnical information concerning the vicinity of Bursa and the site of the TOFAS factory. This information was assembled from various sources, and no new laboratory or field investigations were involved. The second report, completed in 1973, presents results from special field and laboratory investigations made specially for this study: seismic refraction and reflection studies and gravity measurements, resonant column tests to measure wave velocity, and microtremor amplitude spectrums.

Because of the many maps and figures contained in these reports, copying and distribution of the complete reports is not feasible. Both reports are on file at MIT for inspection by interested researchers.

2.2 TOPOGRAPHIC AND GEOLOGIC SETTING

Figure 2.1 shows the topographic features in the vicinity of the TOFAS factory. The site itself is near the north edge of a broad flat plain with an elevation of about 100 meters above sea level. This plain is part of a long valley containing a stream flowing from east to west, with steeply rising hills flanking the valley to the north and south. The city of Bursa is built against the hills on the south side of the valley, on a shelf of land standing somewhat higher than the plain.

Geologically, the valley has been described as a graben. Figure 2.2 shows a geological cross-section through the plain along a more-or-less north-south line, based upon seismic reflection studies. Under the center of the valley, bed rock is at considereable depth and there is a suggestion of nearly vertical, successive faults. The presence of such faults is also suggested by gravity measurements.

The major earth material filling the graben is a soft rock of the tertiary period. (The word "Neocene" is used in the report to denote this earth material.) This material outcrops in the hills immediately north of the TOFAS factory, and in a low flat across the plain at the western edge of Fig. 2.1. In boring logs from the Industrial Zone, this tertiary rock is identified as a marl of limestone. Where deep borings have been made through the alluvium, this tertiary material has been logged as "yellow clay".

The alluvium is heterogeneous, involving lens and strata of gravel, sand and clay. In a boring midway across the valley from the TOFAS factory to the city of Bursa, the alluvium was about 200 meters deep and was predominantly clay. Another boring, approximately 2.5 km west-southwest of the TOFAS factory revealed about 125 meters of alluvium, mostly gravel. Approximately 2.5 km east of the factory was another boring indicating about 100 meters of alluvium, mostly sand. The higher shelf of land on which Bursa is built has been interpreted as an older alluvium.

The water table across the plain was typically found at a depth of 5 to 7 meters, or less in some seasons.

2.3 GEOPHYSICAL STUDIES

The cross-section shown in Fig. 2.2, and the compressive wave velocities shown on that figure, are based upon seismic reflection studies. The reflection measurements were made by the Mineral Research Department of Turkey (MTA) in early 1973, along the approximate line shown in Fig. 2.1. This line passes about 2 km east of the TOFAS factory. At the nearest point to the factory, the profile indicates about 100 meters of alluvium, and then about 170 meters of tertiary rock overlying apparent bedrock.

As mentioned earlier, gravity measurements were also made by MTA at the same time as the reflection studies. The gravity measurements were also strongly suggestive of downfaulting similar to that shown in Fig. 2.2.

Seismic refraction studies were made in September 1972 along approximately the same seismic line, by the State Hydraulics Works of Turkey

(DSI). These studies gave compressive velocities to a depth of about 400 meters. The indicated depths of alluvium were shallower, by about 50% than those given by the reflection study. However, the compressive wave velocities from the two studies were very similar at similar depths. The refraction study identified a seismic boundary in a location similar to the first transition boundary within the tertiary rock in Fig. 2.2.

Thus the geophysical studies give a satisfactory general picture of the pattern of wave velocities under the Bursa plain. Exact details, such as the depth of transition boundaries and the locations of the buried faults, are necessarily somewhat uncertain.

2.4 GEOTECHNICAL STUDIES

2.4.1 Borings at TOFAS Factory

Several deep drill holes were made at the site by DSI in 1960, apparently for the purpose of developing water supply. They revealed alternate lens or strata of gravel, sand and clay. "Yellow clay," identified as being the tertiary rock, was found at a depth of approximately 120 meters, although the available logs do not give any quantitative evidence of a transition at this depth. ("Yellow clay" was also recorded at higher elevations.)

The site explorations for the foundations of the TOFAS factory also included nine bore holes made in 1968, the depth of which varied from about 20 to 30 meters. Standard penetration test results from the bore holes gave an average value of N = 15 with some local values as low as N = 5 and as high as N = 25. The boring logs indicate that the soil profile consists of a top clay layer, 6 meters thick on the average. This clay is brown in color and has a high degree of plasticity. Underlying the clay layer is a sand layer 1.5m thick, below which are clays containing grey sand seams and sand pockets. The color of this second clay is grey and becomes darker with increasing organic matter.

Laboratory tests conducted on the brown and grey clays show that the natural water contents of the clays are close to their plastic limits. The Atterberg limits, natural water contents, unconfined strengths and CUC (consolidated undrained compression) results for the brown and grey clays are summarized in Table 2.1.

2.4.2. Special Geotechnical and Geodynamic Tests

As part of the current study effort, three bore holes were made along the seismic profile at the locations indicated in Fig. 2.1. Eight "undisturbed" and ten disturbed samples were taken for laboratory testing. The depths of the borings varied from 7.00m to 11.50m. In general the boring logs from these three bore holes are similar to those at the TOFAS factory (subsection 2. .1); namely, the presence of the intermittent sand and clay layers. The water table was observed to maintain at 3.0m - 3.5m.

Static tests performed on the "undisturbed" smaples included, consolidation tests, unconfined compression and unconsolidated undrained shear tests. The results show that the unconfined compression strength increases from 0.45 kg/cm^2 at 3.50m to 0.75 kg/cm^2 at 7.00m. Also, the undrained cohesion Cu was observed to increase with depth. A summary of the strength data is presented in Table 2.2.

Laboratory dynamic tests were also performed on undisturbed silty clay samples using the "resonant - column" technique. For various samples taken from various depths, the following dynamic soil parameters were determined under $\varepsilon = 5 \times 10^{-3}$, $\gamma = 5 \times 10^{-3}$ conditions:

1. Longitudinal wave velocity, ranged from 120m/sec to 170m/sec.
2. Shear wave velocity, ranged from 140m/sec to 196m/sec.
3. Dynamic elastic modulus ranged from 1120 kg/cm^2 at 3.50m to 2000 kg/cm^2 at 7.00m.
4. Dynamic shear modulus ranged from 390 kg/cm^2 at 3.50m to 780 kg/cm^2 at 7.00 m.
5. Ratio of shear modulus to undrained shear, ranged from 840 to 1040
6. Poisson's ratio ranged from 0.41 to 0.45
7. Damping ratio ranged from 0.065 to 0.075

Some attempts were made to assess the quality of the laboratory data by comparing them with typical values reported in the literature for similar deposits. The results of these comparisons are presented in Appendix B.

Chapter 3

ONE-DIMENSIONAL AMPLIFICATION STUDIES

3.1 INTRODUCTION

The one-dimensional amplification studies performed at MIT, to evaluate the influence of the local soil conditions upon the earthquake induced ground motions at the site of the TOFAS factory, were carried out in three phases.

The first phase of this work commenced when the site investigation report for the foundations of the TOFAS factory was received in the spring of 1973. Relevant information extracted from this report was utilized to conduct some preliminary amplification studies for the site of the factory. A summary of the conclusions arrived at in this first phase is presented in Appendix A.

In phase 2 of the studies, a number of profile geometries with various shear velocity profiles, soil parameters and earthquake records were utilized to assess the general influences of the various factors upon the calculated ground response at the site of the factory. The results and conclusions arrived at from these analyses are provided herein in Appendix B.

The third phase of this work involved final detailed one-dimensional amplification analyses employing a few basic profiles which were believed to best represent the site conditions at the location of the TOFAS factory. The description of the profiles used and the results and conclusions drawn during this final phase will be presented in this chapter.

3.2 SOIL PROFILES FOR AMPLIFICATION STUDIES

The results of the analyses performed during the first two phases, together with further interpretation of the available data concerning the soil and geological conditions at and near the factory site, were utilized to generate the final profile which was used during the final phase of this study. Simply, the geometery of this profile may be described as consisting of alluvium of 125 meter depth underlain by soft rock of the tertiary period refered to as Neocene rock. Depth to bedrock was established at 320 meter. In the computer analyses, the alluvium was described as consisting of 15m of clay on top of 110 meter of sand and the Neocene rock was specified as clay whenever the shear velocity of the layer was less than 4000 fps. Figure 3.1 shows the geometry of this profile.

A basic shear velocity profile was adopted to describe the dynamic characteristics of each layer in the profile. This basic shear velocity profile was arrived at by employing to some extent the field and laboratory data which are summarized in Chapter 2 and by Tezcan et.al., (1974), together with a knowledge of typical shear wave velocities encountered for similar deposits. Figure 3.2 presents this profile which is hereafter refered to as BP-1 (Bursa Profile - 1.)

After some preliminary studies were completed using BP-1, it was decided to introduce some minor changes in the basic profile to determine the influences of the stiffness of the alluivium, Neocene rock and bedrock upon the computed amplifications. These alternate profiles are presented in Figs 3.2, 3.3, 3.4 and 3.5 and are described as follows:

Profile	Description
BP-1	Basic profile, Fig. 3.2
BP-2	Same as BP-1, Top 7^m of clay softer, C_s = 700 fps
BP-3	Same as BP-1, Top 85^m of alluvium softer, Fig. 3.2
BP-4	Same as BP-1, entire alluvium softer, Fig. 3.3
BP-5	Same as BP-3, damping in Neocene rock doubled
BP-6	Same as BP-1, Neocene rock stiffer and specified as rock in the analyses Fig. 3.4

BP-7	Same as BP-6 shear velocity of Bedrock = 12000 fps, Fig. 3.4
BP-8	Same as BP-7, Neocene rock stiffer. C_s = 5300 fps.
BP-9(a)	Same as BP-6 depth to bedrock = 125^m, shear velocity of bedrock = 8000 fps, Fig. 3.5
BP-9(b)	Same as BP-9(a) shear velocity of bedrock = 12000fps, Fig. 3.5
BP-9(c)	Same as BP-9(a), shear velocity of bedrock = 10^6 fps, Fig. 3.5
BP-9(d)	Same as BP-9(c), 45^m -85^m of alluvium stiffer, Fig. 3.5.

3.3 INPUT EARTHQUAKES

The earthquake record used in the computer analyses of ground response during the main shock was the Hollywood Storage Building record of the Kern county earthquake of 1952. The record was normalized to a peak acceleration of 0.025g. Figure 3.6 shows the response spectra of this record.

Ground response analyses during the aftershock were also carried out using profile BP-6. In view of the fact that most aftershock records are essentially sinusoidal motions*, two sinusoidal records, were generated for use in the aftershock analyses. The two records, refered to as SW -1 and SW-2, were sine waves of period 1.2 seconds and durations 12 seconds and 4 seconds respectively. Both input records were normalized to a peak acceleration of 0.002g. All input motions were specified at an outcrop.

3.4 RESULTS AND DISCUSSION

The computer program SHAKE 3 was used to evaluate the effect of local

* This idea came from the concurrent studies at the University of California.

soil conditions upon the ground response at the site of the TOFAS factory. The normalized nonlinear shear modulus vs strain and damping vs strain curves suggested by Schnabel et.al. (1972) were employed to obtain strain compatible dynamic properties of the soil.

The acceleration response spectrum obtained by using BP-1 is shown in Fig. 3.6.

Also in this figure the response spectrum of the input motion is shown. The observed similarities of the two spectra suggest that the shape of the response spectrum at the ground surface is very much influenced by the shape of the corresponding spectrum of the input motion. Therefore, estimation of the fundamental period of a site which should be independent of the type of the base rock motion, should not be based upon the ground response spectra. Results obtained from Phase 2 studies confirm previous observations from other amplification studies that the shape of the curve obtained by taking the ratio of the response spectrum at the ground surface to the response spectrum of the input motion is in most cases independent of the type of input motion used. Thus, using this curve, which is commonly refered to as Ratio of Response spectra (RRS) to estimate the fundamental period of a site, is more reasonable. Figure 3.7 shows RRS for BP-1 which suggests that the fundamental period of the site is probably close to 2.0 second.

Further computer runs were made by using BP-2, BP-3 and BP-4 which, as a reminder, were obtained from BP-1 by gradually decreasing the stiffness of the alluvium with depth. Comparison between the RRS of BP-1 and BP-2 in Fig. 3.7 indicates that softening the top 7 m of the clay does not significantly change the computed amplifications. On the other hand, softening top 45^{m} (BP-3) or the entire 125 m of the alluvium (BP-4) results in significant changes in the ground response. In general, the softer the alluvium the higher is the response at large periods (2.5 seconds) although its effect can also be observed at smaller periods.

A comparison between the results of the analyses performed by using BP-3 and BP-5 is made in Fig. 3.8. The difference between the two runs was that the damping of the Neocene rock in BP-5 was double the damping in BP-3. It is observed that the greater the internal damping assigned to the Neocene layer, the smaller is the calculated ground response.

The RRS of BP-6, which is similar to the basic profile BP-1 except that the Neocene rock is stiffer, is shown in Fig. 3.9; together with the RRS of BP-1. From Fig. 3.9 it is observed that increasing the stiffness of the deeper earth material increases the amplification at period less than 0.8 seconds and shifts the peak of the RRS curve form 2.5 seconds to 1.5 seconds.

To determine the influence of the assumption regarding the shear velocity of the bedrock upon the ground response, computer runs were made using BP-6, BP-7 and BP-8. The results are presented in Fig. 3.10. It is observed that the stiffer the bedrock the higher is the amplification. This observation is also made from the results of the analyses carried out on BP-9(a), (b), (c) and (d) which will be presented and discussed subsequently.

Since there was great ambiguity in the reported geophysical data regarding the depth to bedrock, which neccesitated use of considerable judgement in the choice of depth to bedrock in the choice of depth to bedrock in the analyses, it was decided to make a few additional runs reducing depth to bedrock from 320 meter to 125 meter. Figure 3.11 presents the results of the analysis for BP-6, BP-9(a),(b),(c) and (d). Two observations are made from this figure. First, the presence of the Neocene rock with shear velocity of 4000 fps, only slightly influences the results. Second, the larger the bedrock velocity the larger is the amplification specially at the fundamental period of the site which is close to 1.5 sec.

Based upon the results and conclusions drawn from the main shock analyses it was decided that profile BP-6 best represented the actual conditions at the site of the factory. Thus, this profile was used in the aftershock analysis. The computed and recorded acceleration response spectra normalized to the maximum ground acceleration are shown in Fig. 3.12. The following observations and comments regarding the results of the aftershock analyses are appropriate here:

1) The shapes of the recorded and computed spectra using SW-1 and SW-2 records suggest that the aftershock ground motions at the site of the factory are probably sinusoidal motions of period 1.2 second.

2) The presence of a peak at 1.2 seconds in the computed spectra may be fortintous since the input sinusoidal motions had a period of 1.2 seconds which is probably very close to the fundamental period of the site. It would be worthwhile to determine the influence of the period of the input sinusoidal motion upon the computed ground response.

3) Increasing the duration of the bedrock motion from 4 second to 12 seconds increases the peak ground response by about 60%.

SUMMARY

The observations made and the conclusions arrived at during the phase 3 amplification studies may be summarized as follows:

1) Decreasing the stiffness of the alluvium deposit significantly increases amplification specially at large periods. (2.0-3.0 sec)

2) Increasing the stiffness of the Neocene rock increases amplification at periods less than 0.8 seconds and shifts the peak in the RRS curve from 2.5 seconds to 1.5 seconds.

3) Profile BP-6 in which the shear velocity of the Neocene rock is 4000 fps probably best represts the site conditions.

4) Establishing bedrock of 125 m below ground surface only slightly changes the ground response computed from BP-6 in which bedrock is at 320 m.

5) The fundamental period of the site is probably between 1.2 to 1.7 seconds.

6) The results of the analyses suggest that amplification of the earthquake ground motions did occur during the 1971 Gediz earthquake at the site of the factory. The magnitude of this amplification ranged, in the analyses, from 2 to 5 within the period range of 1.2 - 1.7 seconds.

7) The aftershock analyses carried out using BP-6 and two sinusoidal input motions of period 1.2 second indicate a peak in the response spectra at 1.2 seconds.

Chapter 4

CONCLUSIONS

Final conclusions will be presented in a subsequent joint report by M.I.T., the University of California (Berkeley) and Bosphorus University. Here only a few observations will be offered.

First, there seems little doubt that soil amplification played an important role in the occurrence of damage at the TOFAS factory. It is difficult to otherwise explain the lack of damage to nearby poorly constructed buildings founded over shallower and/or denser soil. It seems clear also that larger more flexible buildings at the factory suffered more than smaller, stiffer buildings.

However, the recorded aftershock motion does not prove that the soil amplified the motions only over a narrow range of periods. It seems more likely that the aftershock motions thoughout the vicinity had a very strong component of motion with a period of 1.2 seconds.

Also, the one-dimensional amplification studies reported herein do not support a hypothesis that the response spectrum of the main shock had a distinct peak near a period of 1.2 seconds. The theoretical studies do indicate a considerable amplification of motions in the range from 1 to 2 seconds.

On the other hand, to explain the collapse of the garage and paint workshop it is not essential to have had a predominant soil period equal to the elastic fundamental period of the workshop. It is enough that the spectral response at the fundamental period have been large enough to cause yielding, and that the energy at longer periods have been large enough to carry the yielding on to failure. While the workshop was provided with the lateral strength to resist small ground motions without yielding, it seems likely that it was not provided with the ductility to prevent collapse once yielding started.

Considering all these observations, it can be said that the predictions of one-dimensional amplification theory are in accord with the observed damage.

APPENDIX A

Phase 1 Studies

The initial phase of the studies was carried out prior to the completion of the geophysical and geotechnical site and laboratory investigations. Therefore, the soil properties and parameters required for the analysis had to be estimated based on the information available in the report for the TOFAS factory and upon conversations during Dr. Christian's site visit.

Four assumed profiles were used in these preliminary studies. Profiles 1 and 2 are shown in Fig. A.1; profiles 3 and 4 are identical to 1 and 2, respectively, except that bedrock is placed at a depth of 575 meters. In each case, the shear modulus for the upper 175 meters was estimated from Hardin's (1972) formula:

$$G = 1230 \frac{(3-e)^2}{1+e} \sqrt{\bar{\sigma}_o} \qquad \text{(A.1)}$$

where G is shear modulus in psi, $\bar{\sigma}_o$ is average effective stress in psi and e is void ratio. A value of $e = 0.6$ was assumed throughout. For profiles 2 and 4, Eq. A.1 was also used to determine G for the deeper "clay." For profiles 1 and 3, shear wave velocities were assumed in the clay, using 1000, 1200 and 1300 ft/sec respectively for each 200 meter interval.

Two earthquakes were used as input: An artificially generated earthquake time-history with a smooth response spectrum and strong motion record made at Taft during the 1952 Kern County earthquake. Each record was modified to a peak acceleration of 0.05g and 0.1g for different runs. The analyses were performed by a computer program "SHAKE 3" originally developed by Lysmer and Schnabel at the University of California at Berkeley. This program automatically adjusts modulus and damping values in accordance with the computed strain.

Figure A.2 shows a typical result, in the form of a plot of ratio of response spectra. This curve shows the ratio, at each structural period, of the spectral response for computed surface motion to the spectral response for the input motion. There is no suggestion of a sharp amplification effect near a period of 1.2 seconds. Rather, there is amplification over a broad range of periods greater than about 0.5 seconds, and de-amplification at smaller periods. Results for other profiles and inputs were generally similar.

APPENDIX B

Phase 2 Studies

PROFILES FOR AMPLIFICATION STUDIES

The geotechnical studies described in Chapter 2 did not provide completely clear data concerning the sub-surface profile at the site of the TOFAS factory. It does seem clear that there is a surface layer, about 15 meters thick, consisting of clays and sand. The reflection study indicates bedrock at about 320 meters. However, the reflection (100 meters) and refraction (45 meters) studies differed concerning the depth of the alluvium: While the deep borings seemed to indicate a depth of alluvium of about 120-130 meters, they were not without ambiguity. Figure B.1 shows the basic profile adopted for the studies. This profile is believed to be basically correct, although different depths could equally well have been used for all of the transitions.

The dynamic soil properties which were required for the analyses included, shear wave velocity, C_s, and damping values , D, for each layer. In all the analyses, two typical shear wave velocity profiles were employed.

The first profile was estimated based upon the measured compressive wave velocities together with a knowledge of typical shear wave velocities encountered for similar alluvial deposits. Figure B.2 presents this shear wave velocity profile, designated hereafter as C_s - Profile 1. The value of C_s for the bedrock was assumed to be 6000 fps.

The determination of the second shear wave velocity profile (C_s -profile 2) was based on laboratory data combined with the measured values of the compressive wave velocities for the site.

The first step was to investigate the degree of agreement present between the reported laboratory data and typical data for similar deposits reported by Seed & Idriss (1971). Figure B.3 shows a plot of, shear modulus/undrained strength, (G/Su) against shear strain, (γ). The solid line is an average curve suggested by Seed & Idriss; the dashed lines suggest the spread of the data although many data points fall outside these lines. On this figure, the laboratory G/Su values from chapter 2 are plotted.

The agreement between the reported laboratory data with other field and laboratory data is close. Similar investigations were performed to compare the reported laboratory damping values with the values suggested by Seed & Idriss. Figure B.4 shows, in the case of damping, the agreement is quite good.

After assessing the quality of the available laboratory data, the second basic shear wave velocity profile (C_s - profile 2) was generated in the following manner. For the first 15m, using an average value G/Su equal to 1000 at a shear strain of 5×10^{-3} and for Su = 0.5 kg/cm^2 the shear wave velocity at low strains was estimated to be equal to 1460 fPs. Alternatively, using G/Su equal to 2300 at low strains, obtained from Seed & Idriss, the shear wave velocity was estimated to be equal to 780 fPs. Therefore, for the first 15m, an intermediate value of 1000 fPs for the shear wave velocity was adopted. For the rest of the profile, the shear wave velocities were estimated by assuming that the ratio of the shear wave velocities between two successive layers is the same as the ratio of the compressive wave velocities for the same two layers as obtained from the seismic refraction studies:

$$\frac{\text{Shear wave velocity of layer n}}{\text{Shear wave velocity of layer n + 1}} = \frac{\text{compressive wave velocity of layer n}}{\text{compressive wave velocity of layer n}}$$

Inherent in this procedure is the assumption that the Posisson's ratio is constant with depth. With this approach, it makes little difference whether the earth material between 45 and 145 meterd depth is alluvium or soft tertiary rock, since the approach relies on measured compressive velocities.

Figure B.5 presents this second velocity profile which will be designated hereafter as C_s - profile 2. It is important to note that C_s- profile 2 as compared with C_s - profile 1, assumes a stiffer material at shallow depth and less stiff material at greater depths. Both of these final profiles are considerably stiffer than the profiles used for the preliminary studies discussed in Appendix A.

Finnally, several different assumptions concerning the depth to bedrock were used in connection with C_s - profile 1. Thus, a total of five different profiles were used. The descriptions of these five profiles are summarized below. The word in parenthesis identifies the type of input motion used.

PROFILE NAME	DESCRIPTION
BURSA 1 (TAFT) or (KERN):	Depth of bedrock = 320m; C_s- profile 1, Fig. B.2.
BURSA 2 (TAFT):	Depth of bedrock = 320m; C_s- profile 2, Fig. B.5.
BURSA 3 (TAFT):	Depth of bedrock = 145m; C_s- profile 1, Fig. B.6.
BURSA 4 (TAFT):	Depth of bedrock = 45m; C_s- profile 1, Fig. B.7.
BURSA 5 (TAFT):	Depth of bedrock = 320m. From 0-45m, C_s- profile 2, from 45-320m, C_s - profile 1 Fig. B.8.

INPUT EARTHQUAKES

Two earthquake records were used in the computer analyses. The first was the N79W record of the TAFT, 1952 earthquake whose 5% damped response spectrum in Figure B.10 is rich in high frequencies, but less rich in intermediate and low frequencies. The second choice of earthquake record resulted from a desire to have a record with characteristics which might be more similar to the Gediz earthquake. The earthquake record used was the Hollywood Storage Building record of the Kern County earthquake of 1952. This motion was recorded approximately 135 km from the epicenter during an earthquake of magnitude 7. As observed in Fig. B.9 this record is rich in intermediate frequencies in the neighborhood of 1.2 sec, which is the predominant period in the after-shock record at Bursa.

Both records were normalized to a peak acceleration of 0.025g as suggested by Seed, et al. (1969) for an earthquake of magnitude 7 and 135 km away from the epicenter.

RESULTS AND DISCUSSION

For the amplification studies performed and reported herein the computer program "SHAKE 3" was again used. For the analyses performed, to obtain strain compatible soil properties, the relations between dynamic soil properties and strain suggested by Schnabel, et al. (1972) were adopted. In all the analyses the structural damping was assumed to be 5%.

The ratio of response spectra curve obtained using the BURSA 1 (TAFT) profile is presented in Fig. B.10. The results of the analyses for this profile, which may be characterized as a deep layer of firm deposit underlying a relatively softer layer at shallow depths, indicate that amplification will probably occur at all periods. The fundamental period of the deep firm soil has caused large amplifications at long periods. The relatively small peak at 0.6 seconds may be attributed to the shape of the response spectrum of the input which has a peak in that period range, as observed in Fig. B.9, or to the influence of the softer soil deposit at shallow depths.

The several curves in Fig. B.10 indicate the effect of varying the stiffness of the soil and rock, while keeping the overall depth of the profile constant. Recall that BURSA 5 is the same as BURSA 2 to a depth of 45 meters, below which it is the same as BURSA 1. Thus:

1. Comparison of BURSA 1 with BURSA 5 indicates the effect of decreasing the stiffness of the shallow soil. Decreasing the stiffness greatly increases the amplification, especially for periods greater than about 0.8 seconds.

2. Comparison of BURSA 2 with BURSA 5 indicates the effect of decreasing the stiffness of the deeper earth material. Decreasing the stiffness decreases the amplification at periods less than about 0.8 seconds, and shifts the peak of the amplification curve from about 1.5 seconds to about 2.1 seconds.

The effect of the surface layer appears to be predominant in the range from 1 to 2 seconds, while the deeper material has a great effect at the shorter periods.

To investigate the influence of variations in the depth of the profile, analyses were run using BURSA 3 and BURSA 4 profiles which were constructed from the BURSA 1 profile by altering the bedrock elevation. The amplification results presented in Fig. B.11 and the acceleration spectra shown in Fig. B.12 show that the depth of the profile has significant effect upon the ground response at large periods. In general, increasing the depth of the profile significantly increases the response at periods greater than 0.5 seconds.

Finally, a number of the analyses were reworked, with the TAFT earthquake record replaced by the KERN county earthquake record. In Fig. B.15 a comparison is made between the results of BURSA 1 (TAFT) and BURSA 1(KERN) profiles. Generally speaking, for periods less than 0.5 seconds, the response at the ground surface as obtained by the TAFT record is greater than the response obtained by the KERN record. For periods greater than 0.5 seconds the reverse behavior is observed. This may be attributed to the differences in the high frequency contents of the two records. Similar trends in the variation of the response at the ground surface with changes in the input were observed for BURSA 2 and BURSA 5 profiles as shown in Fig. B.14.

SUMMARY

The one dimensional amplification studies performed on typical profiles that are believed to best represent the soil conditions at the site of the TOFAS factory, indicate that, insofar as only two earthquake records of magnitude 7 were employed, a sharp peak of amplification in the neighborhood of 1.2 seconds cannot be observed. However, for the profiles investigated, amplifications of the order of 2 to 7 can be observed for periods greater than 0.6 seconds and less than 3.0 seconds. Therefore, during the 1970 Gediz earthquake, structures at the site of the TOFAS factory with natural periods greater than 0.6 seconds probably did experience response accelerations much larger in magnitude than would structures resting on the bedrock.

Also, it is established that the dynamic behavior of the ground surface, as determined by the one-dimensional amplification studies, is very much influenced by certain factors among which are: the shear wave velocities, the depth of the profile and the input motion. The results show that softening the top 45m thick layer of the profile accompanies a marked increase in the response at periods greater than 0.6 seconds, whereas softening the deep layer of alluvium decreases the response at short periods (less than 0.8 seconds), although its effect is also observed at large periods.

The depth of the profile has significant influence upon the ground response. The results of the analyses, in which the bedrock elevation was altered, show that increasing the depth to bedrock increases the ground response at periods greater than 0.6 seconds. The depth to bedrock has relatively little effect upon the general position of the spectra at periods less than 0.5 seconds, provided that this depth exceeds 45 meters.

Finally, the results of the analyses are also influenced by the form of the input record although to a much lesser extent than by the other factors described above. Of course, using an input with a strong content of periods near 1.2 seconds will be more likely to lead to distinct peaks in surface response spectra at a period of 1.2 seconds.

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5. Tezcan, S.S. and M. Ipek, 1973: "Long Distance Effects of the 28 March 1970 Gediz Turkey Earthquake", Earthquake Engineering and Structural Dynamics, Vol. 1, No. 3/Jan.-March.

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BUILDING		PLAN DIMENS (m)	TYPE	CONSTRUCTION STAGE	DAMAGE
No. 1 Main Workshop		70/180	18 m high steel truss and columns	90% completed	Bolts failed, wind bracings collapsed
No. 3 Press Workshops		60/80	30 m high steel truss and crane beams	90% completed	Bolts of roof trusses failed. Base plates yielded
No. 2 and 4 Office Buildings		24/180	2 storey reinforced concrete	Completed	Absolutely no damage
Service Blocks		24/60	do	Completed	Absolutely no damage
No. 6 Garage and Paint Workshop	A	12/48	7.35 m high from footings one storey high reinforced concrete structure Tile filled joists	Frames completed. Walls finished. No plaster, windows, etc.	Most longitudinal walls toppled. Frame No.1 total collapse, other frames mild damage gradually decreasing from No.1 to 5.
	B	12/36	do	Columns ten days, beams three days old concrete. Slab joists not in place. Under rigid scaffolding	Scattered cracks at columns and beams
	C	12/36	do	All frames completed. No scaffolding, no walls.	Total collapse
	D	12/36	do	Completed up to ground level. Columns not erected	No damage
No. 7 Kitchen		12/24	do	Frames and walls finished, plaster being done	Shear cracks at the support of roof beams

Table 1.1

DESCRIPTIONS OF BUILDINGS AND DAMAGES

(from "March 28, 1970 Gediz Turkey Earthquake and its Long Distance Effects", by S.S. Tezcan and M. Ipek, Research Center, Robert College, Istanbul, Turkey, March 28, 1971)

Table 2.1

AVERAGE GEOTECHNICAL PROPERTIES OF CLAYS AT

SITE OF TOFAS FACTORY

Laboratory Test Description	Brown Clay	Grey Clay
W_L%	55	58
W_P%	29	31
W_N%	29	31
PI%	26	27
q_u(kg/cm^2)	1.4	> 2.0
C_u(kg/cm^2)	0.33 - 0.40	0.20 - 0.64
ϕ_u (Deg.)	0 - 3	0 - 9

Table 2.2

UNDRAINED STRENGTHS OF CLAYS IN SPECIAL

BORE HOLES

Bore Hole No.	Depth (m)	qu (kg/Cm^2)	Cu (kg/Cm^2)	Øu Deg.
1	6.50	---	0.48	19
1	7.00	0.75	--	--
2	3.50	0.45	0.40	17
3	6.00	0.75	--	--

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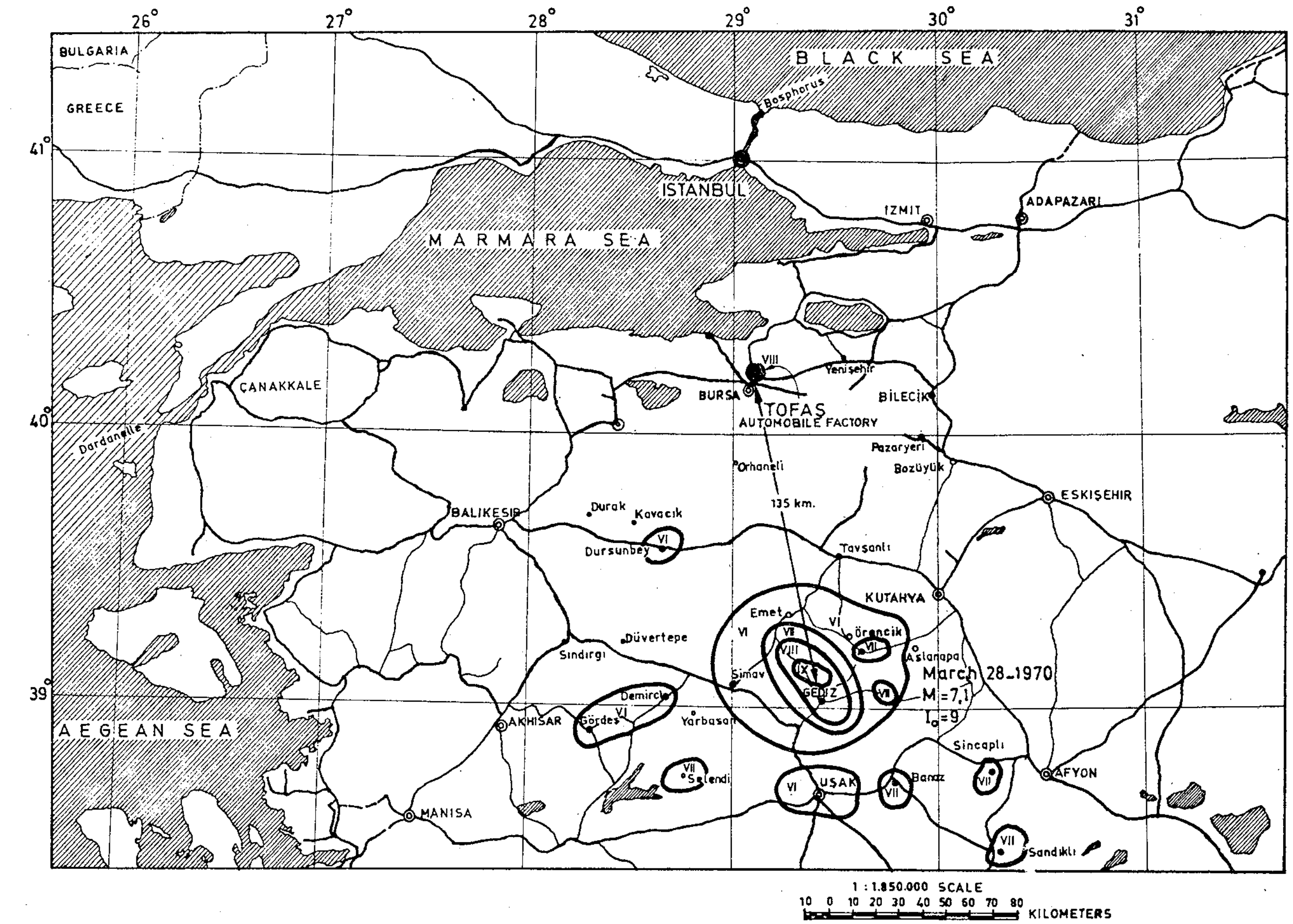

FIGURE 1.1

LOCATION OF TOFAS AUTOMOBILE FACTORY

(from "Long Distance Effects of the 28 March 1970 Gediz Turkey Earthquake", by S.S. Tezcan and M. Ipek, EARTHQUAKE ENGINEERING AND STRUCTURAL DYNAMICS, Vol. 1, No. 3, Jan.-March 1973)

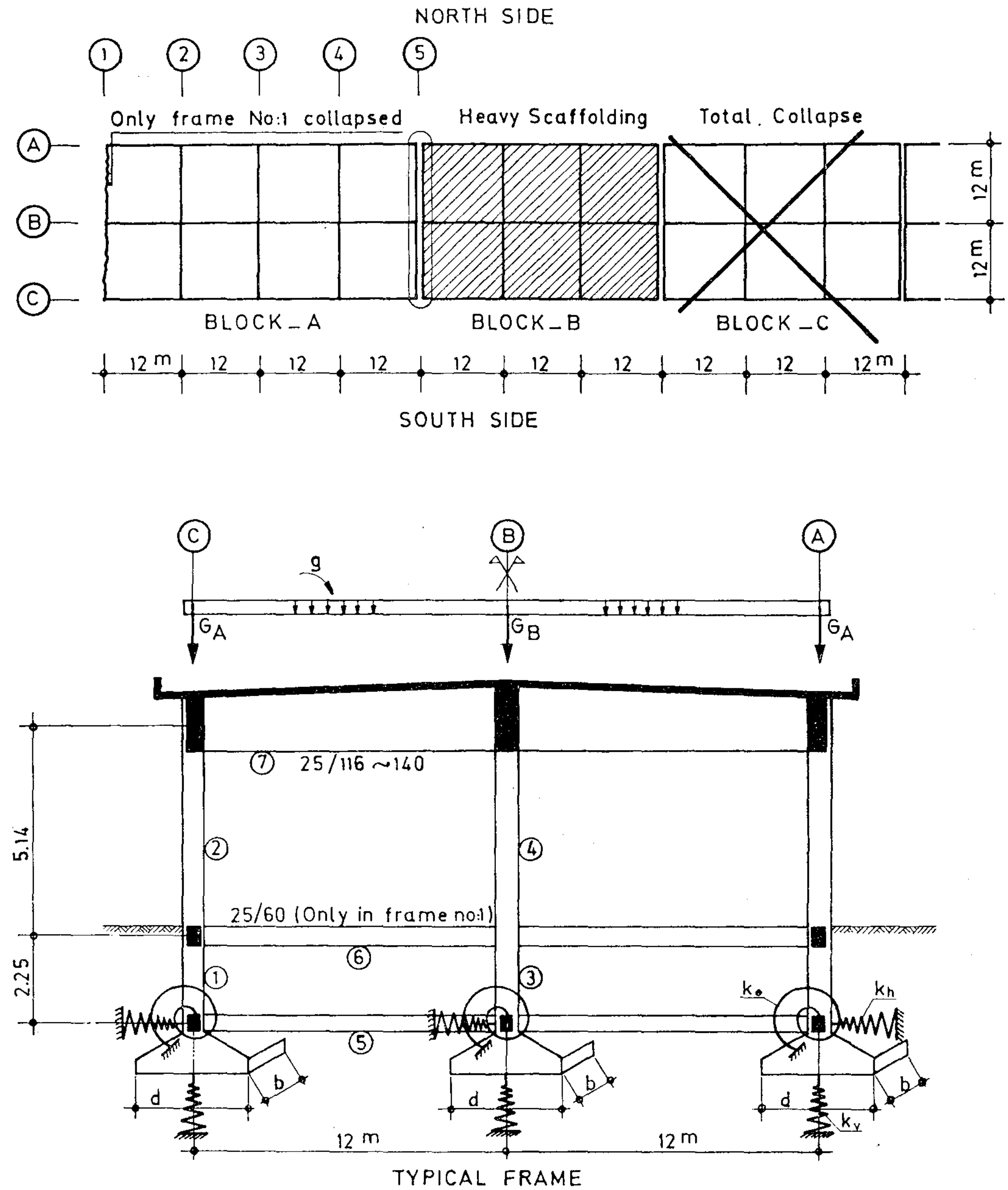

FIGURE 1.2

PLAN AND CROSS-SECTION OF GARAGE AND PAINT SHOP

(from "March 28, 1970 Gediz Turkey Earthquake and its Long Distance Effects", by S.S. Tezcan and M. Ipek, Research Center, Robert College, Istanbul, Turkey, March 28, 1971)

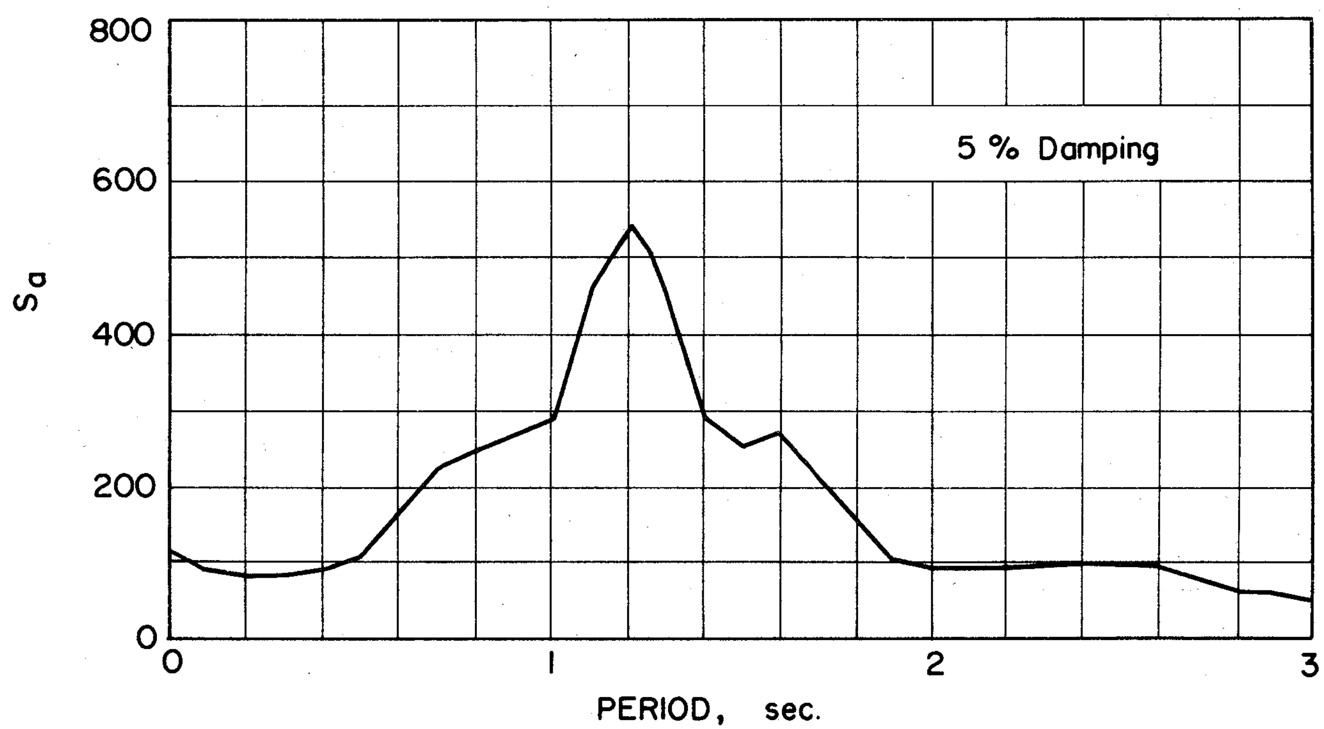

FIGURE 1.3 RESPONSE SPECTRUM FROM AFTERSHOCK RECORDING
(After Tezcan and Ipek, 1971)

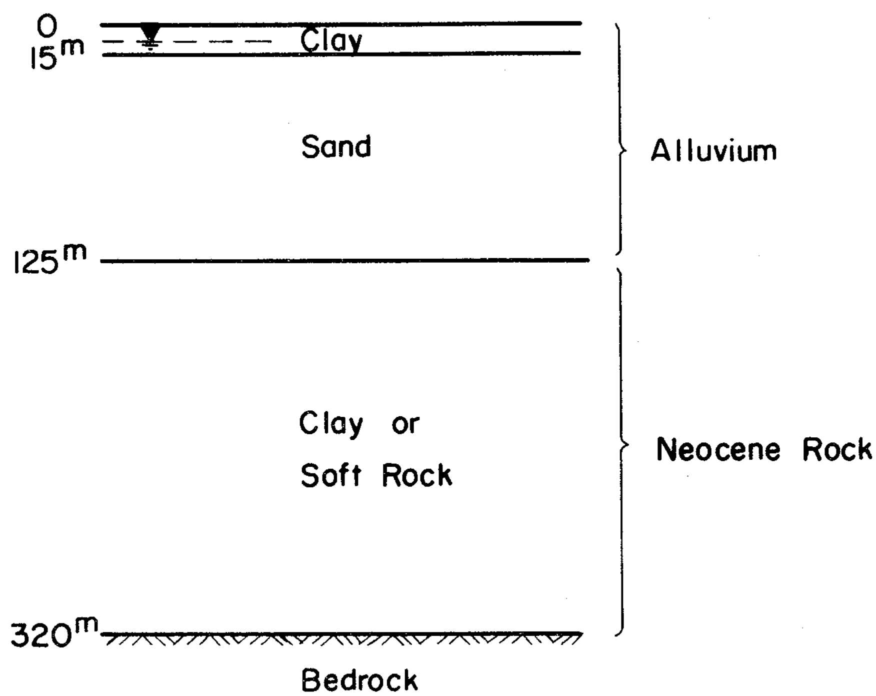

FIGURE 3.1 GEOMETERY OF THE BASIC PROFILE

32

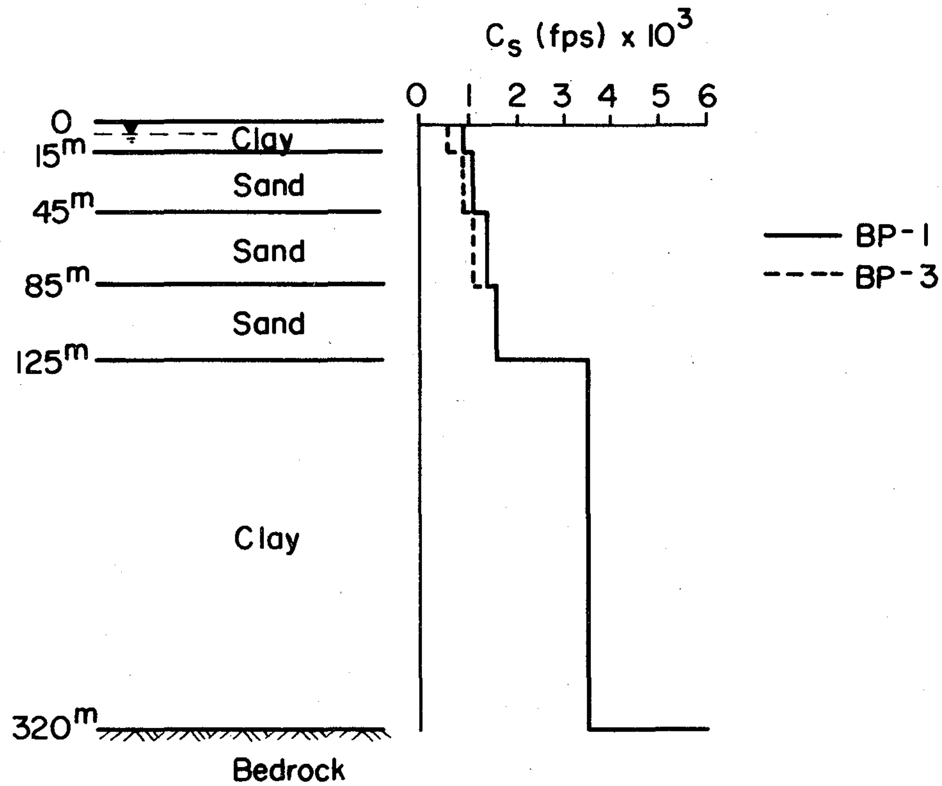

FIGURE 3.2 BURSA PROFILES, BP-1 AND BP-3

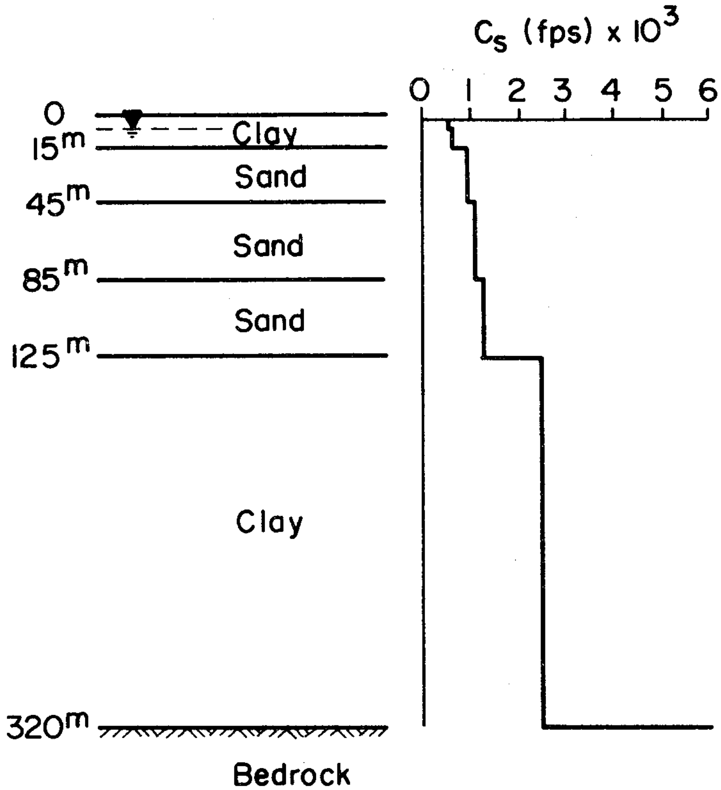

FIGURE 3.3 BURSA PROFILE, BP-4

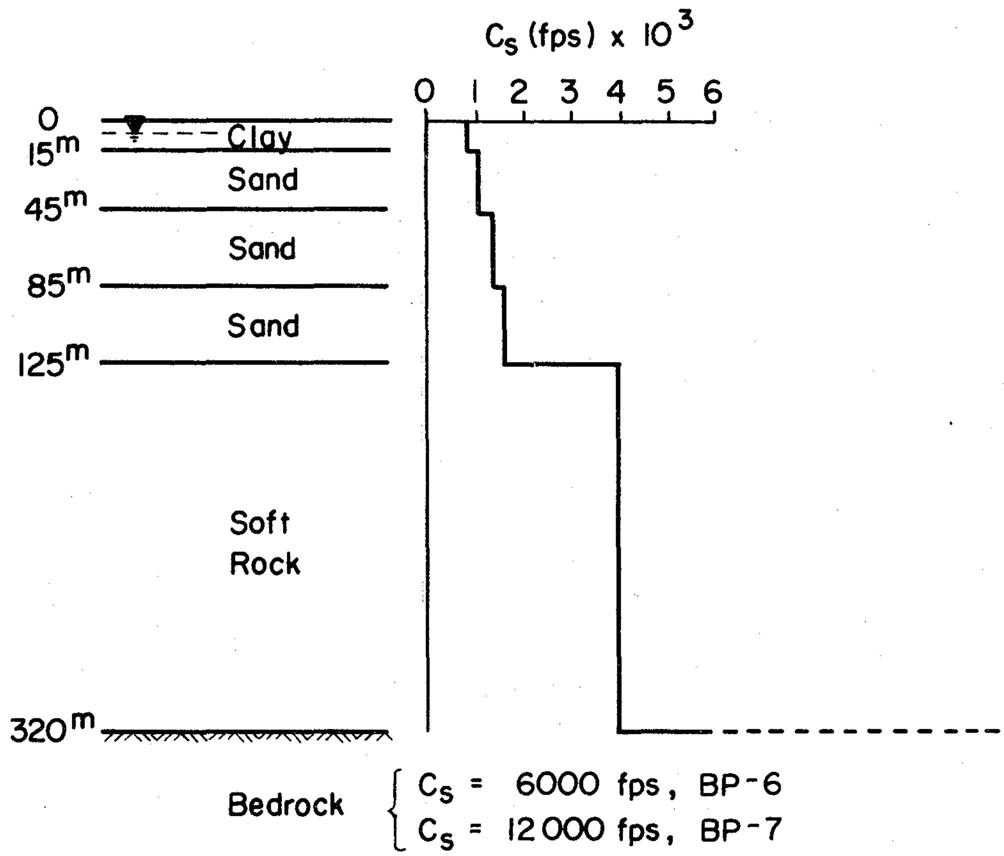

FIGURE 3.4 BURSA PROFILES, BP-6 AND BP-7

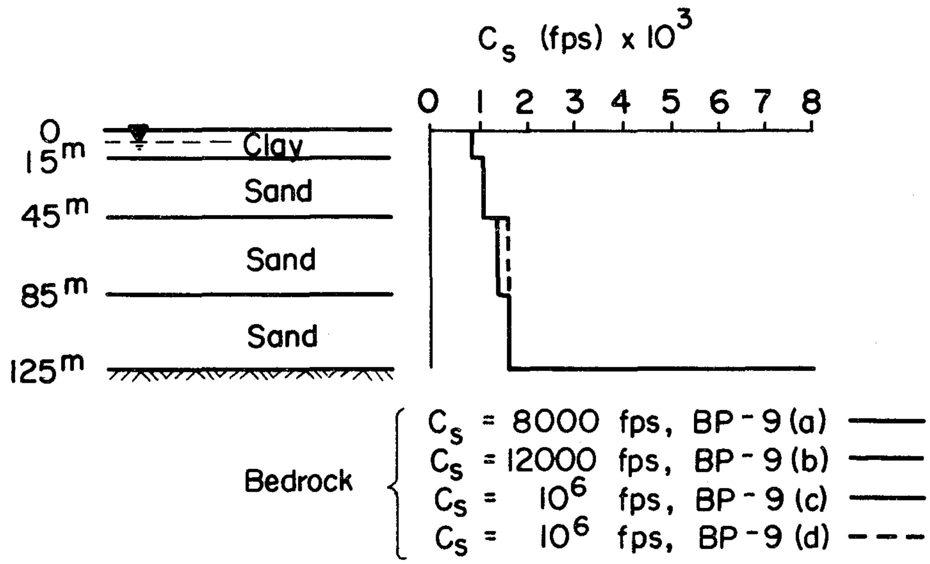

FIGURE 3.5 BURSA PROFILES, BP-9 (a), (b), (c) AND (d)

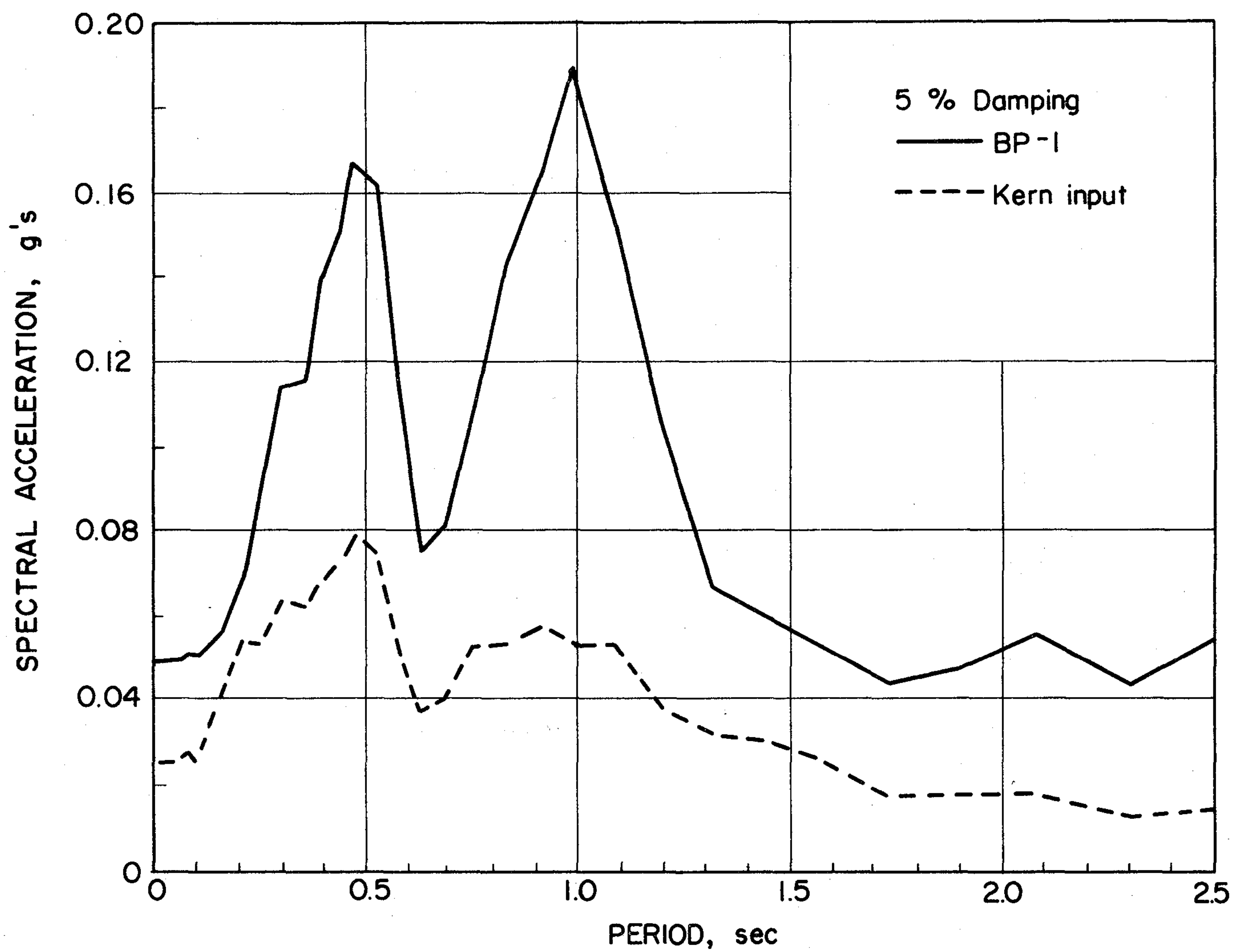

FIGURE 3.6 RESPONSE SPECTRA OF KERN INPUT AND BP - I.

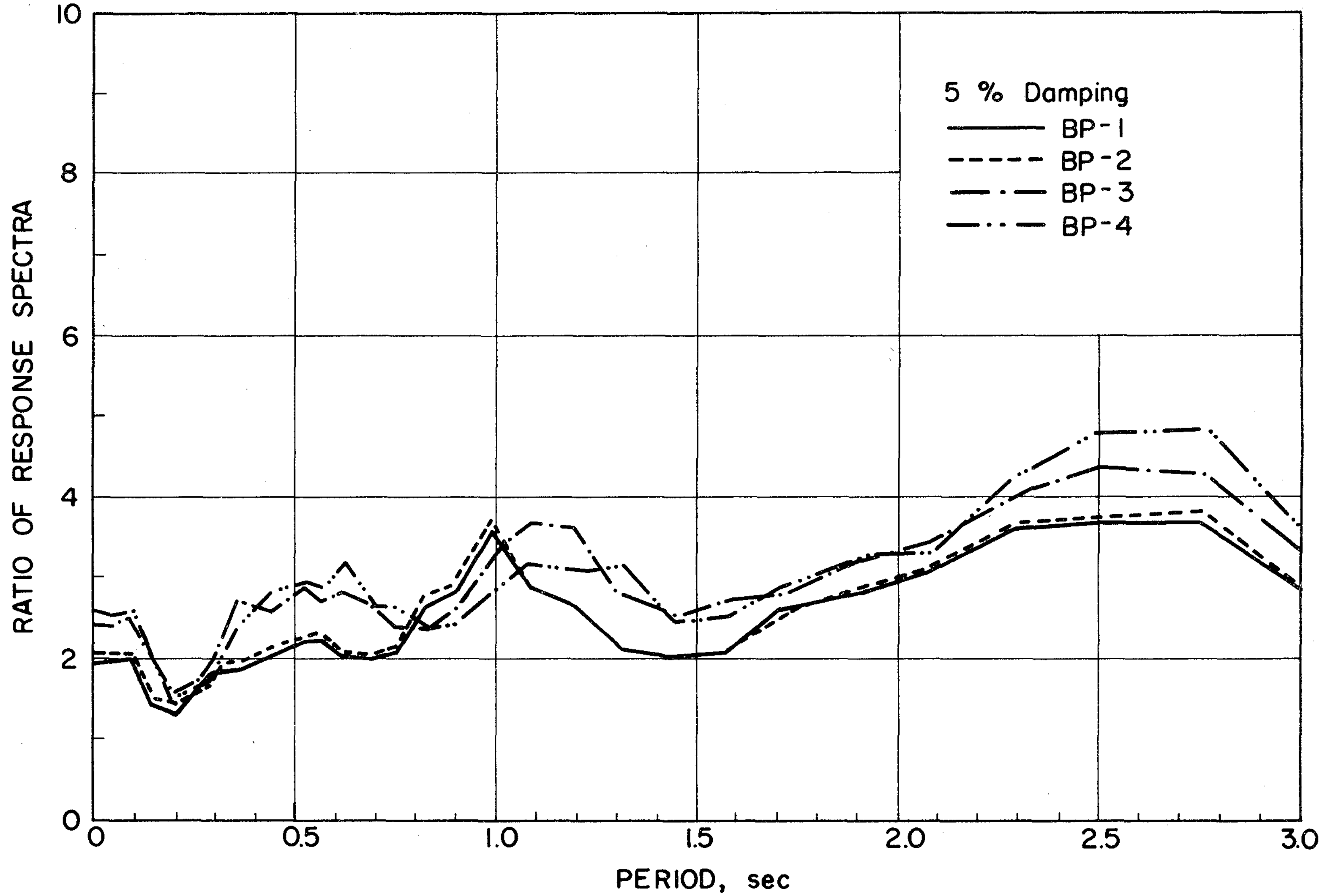

FIGURE 3.7 RATIO OF RESPONSE SPECTRA OF BP-1, BP-2, BP-3 AND BP-4.

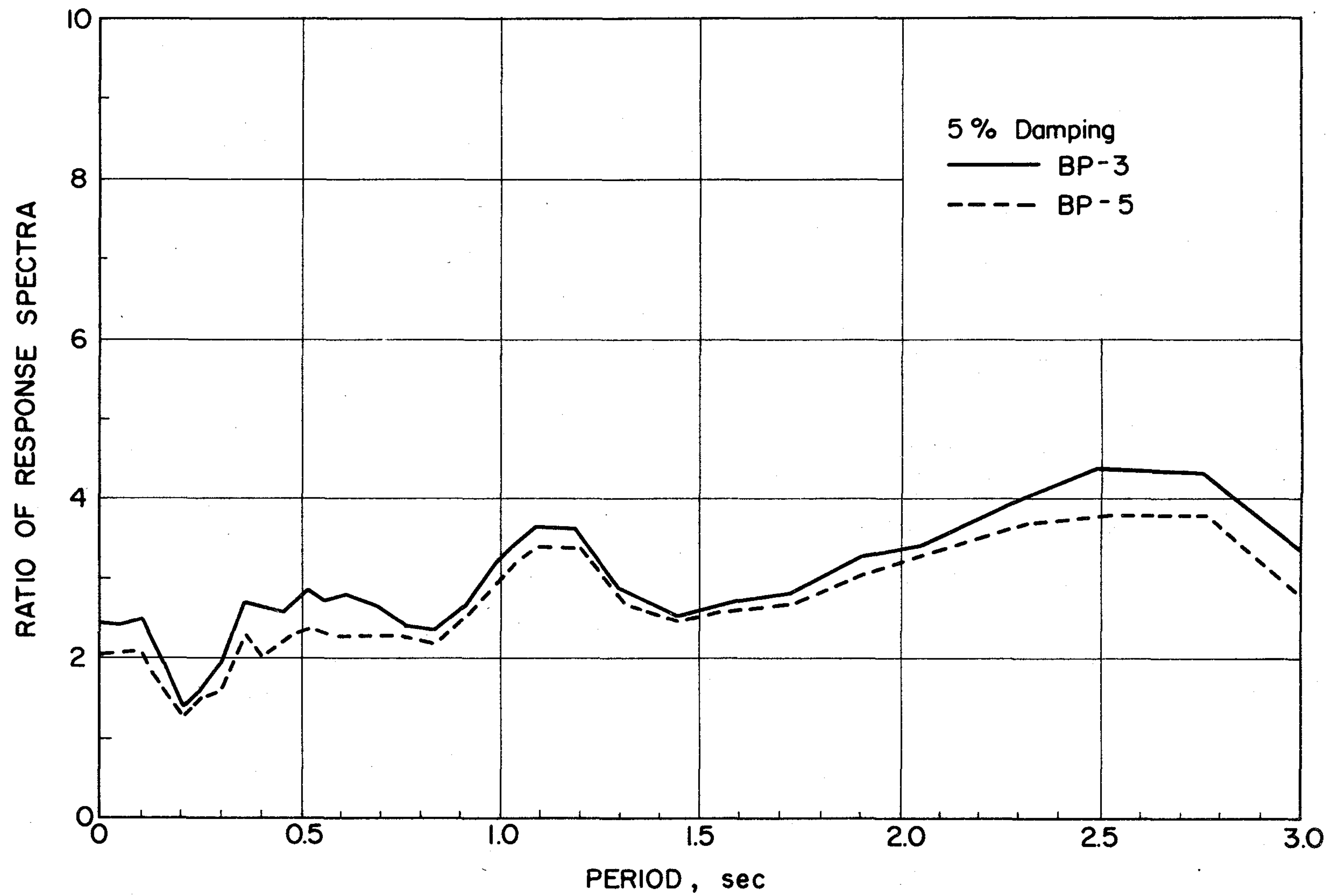

FIGURE 3.8 RATIO OF RESPONSE SPECTRA OF BP-3 AND BP-5

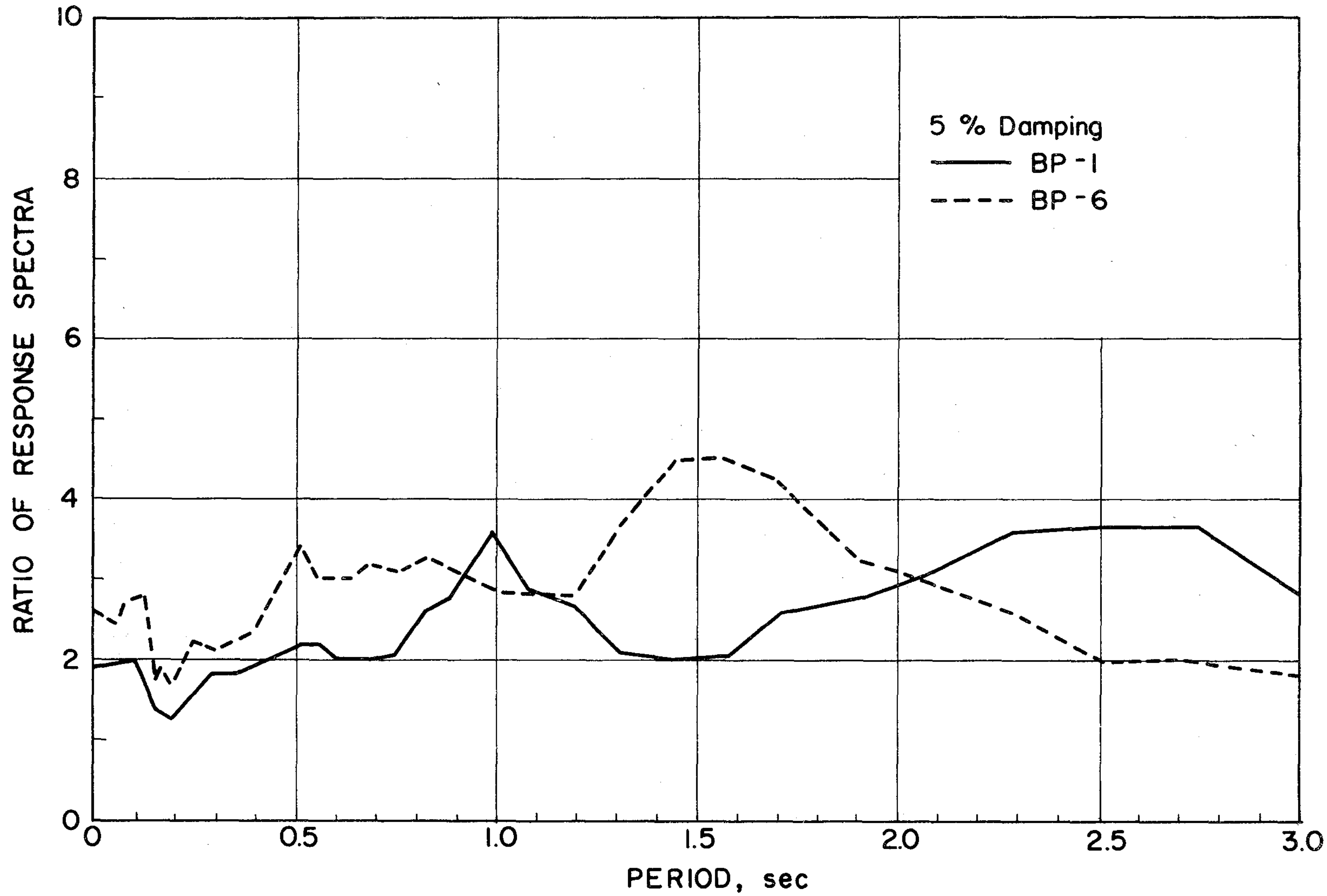

FIGURE 3.9 RATIO OF RESPONSE SPECTRA OF BP - I AND BP - 6

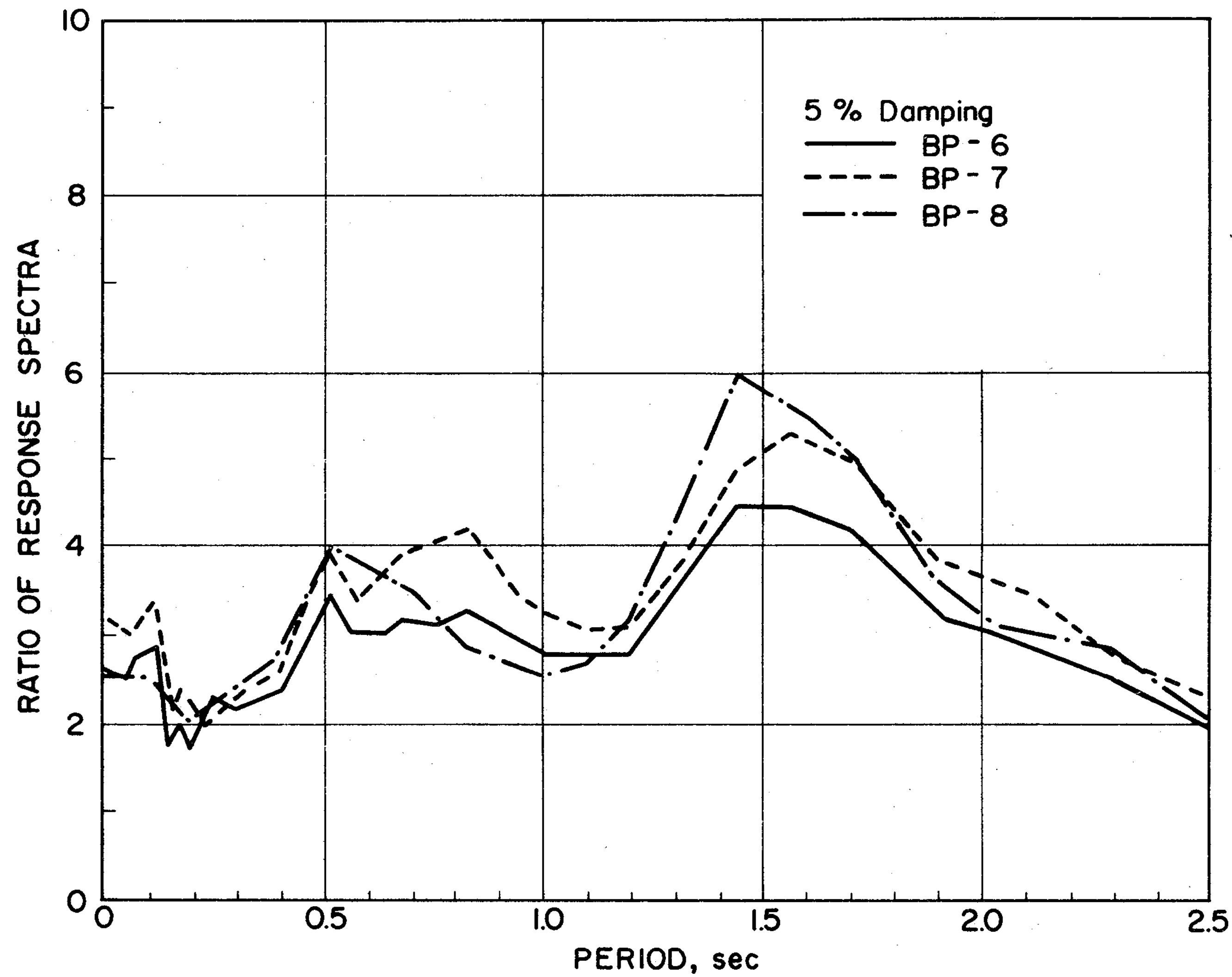

FIGURE 3.10 RATIO OF RESPONSE SPECTRA OF BP-6, BP-7 AND BP-8

40

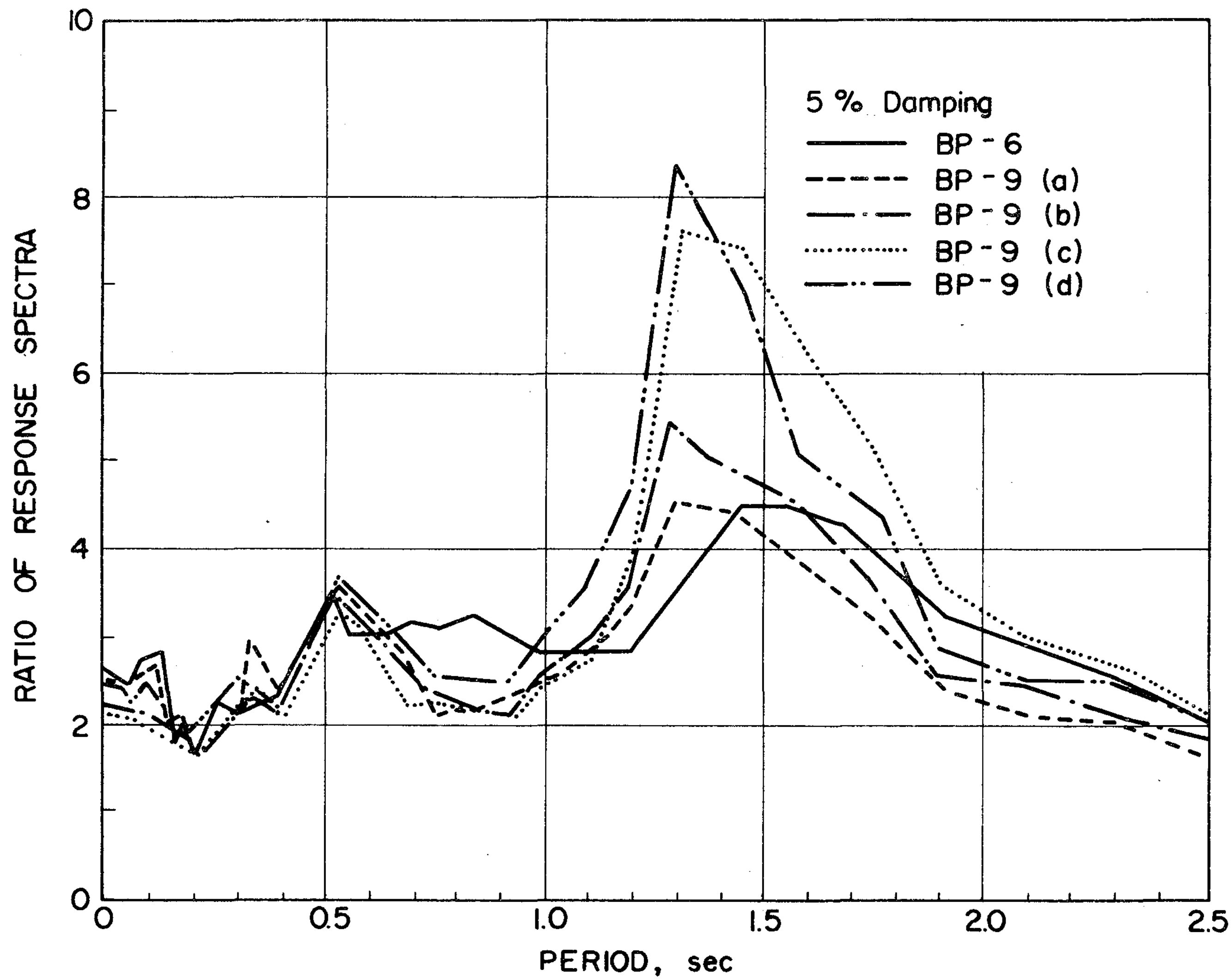

FIGURE 3.11 RATIO OF RESPONSE SPECTRA OF BP-6, BP-9 (a), (b), (c) AND (d)

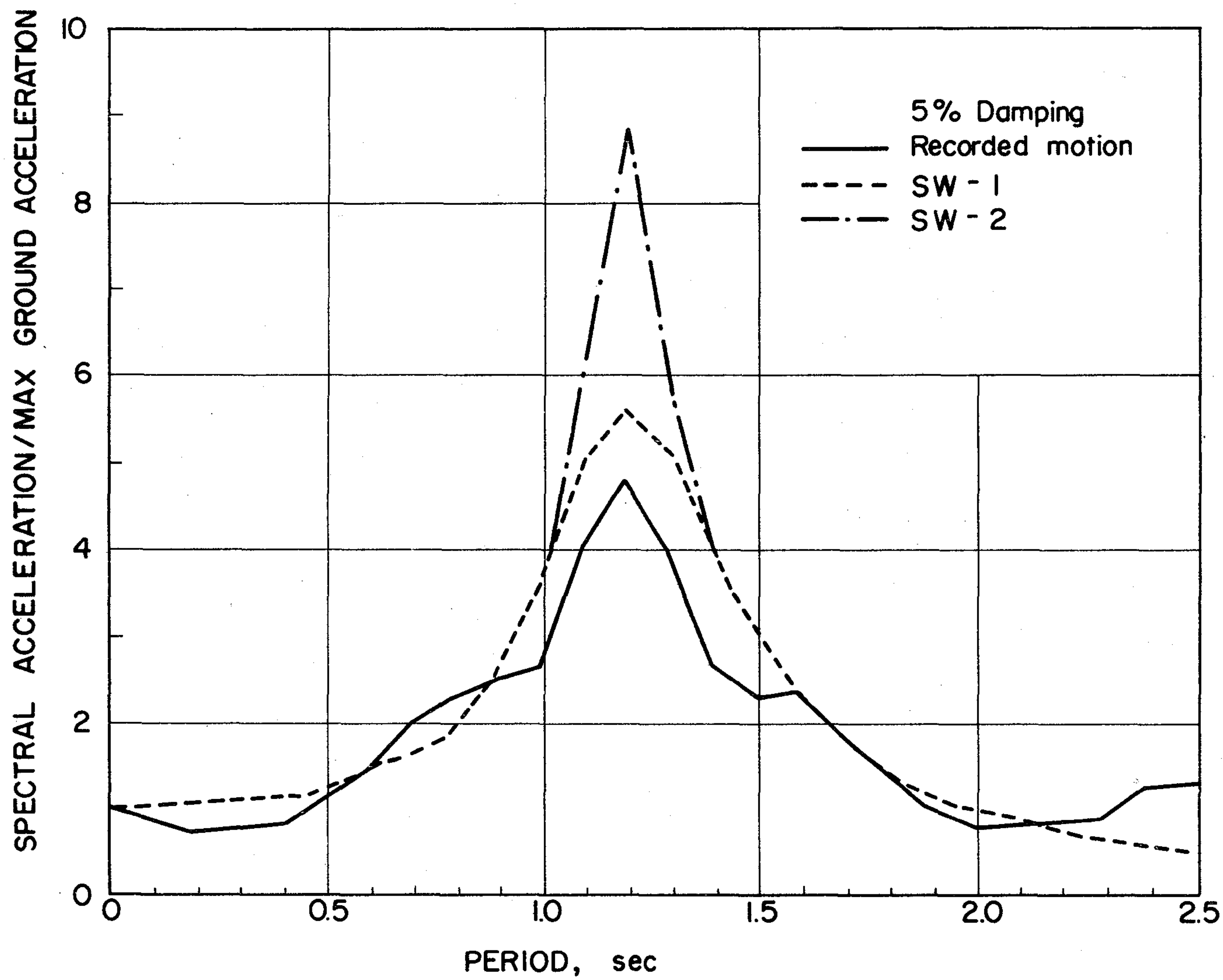

FIGURE 3.12 NORMALIZED RESPONSE SPECTRA OF RECORDED MOTION, SW-I AND SW-2

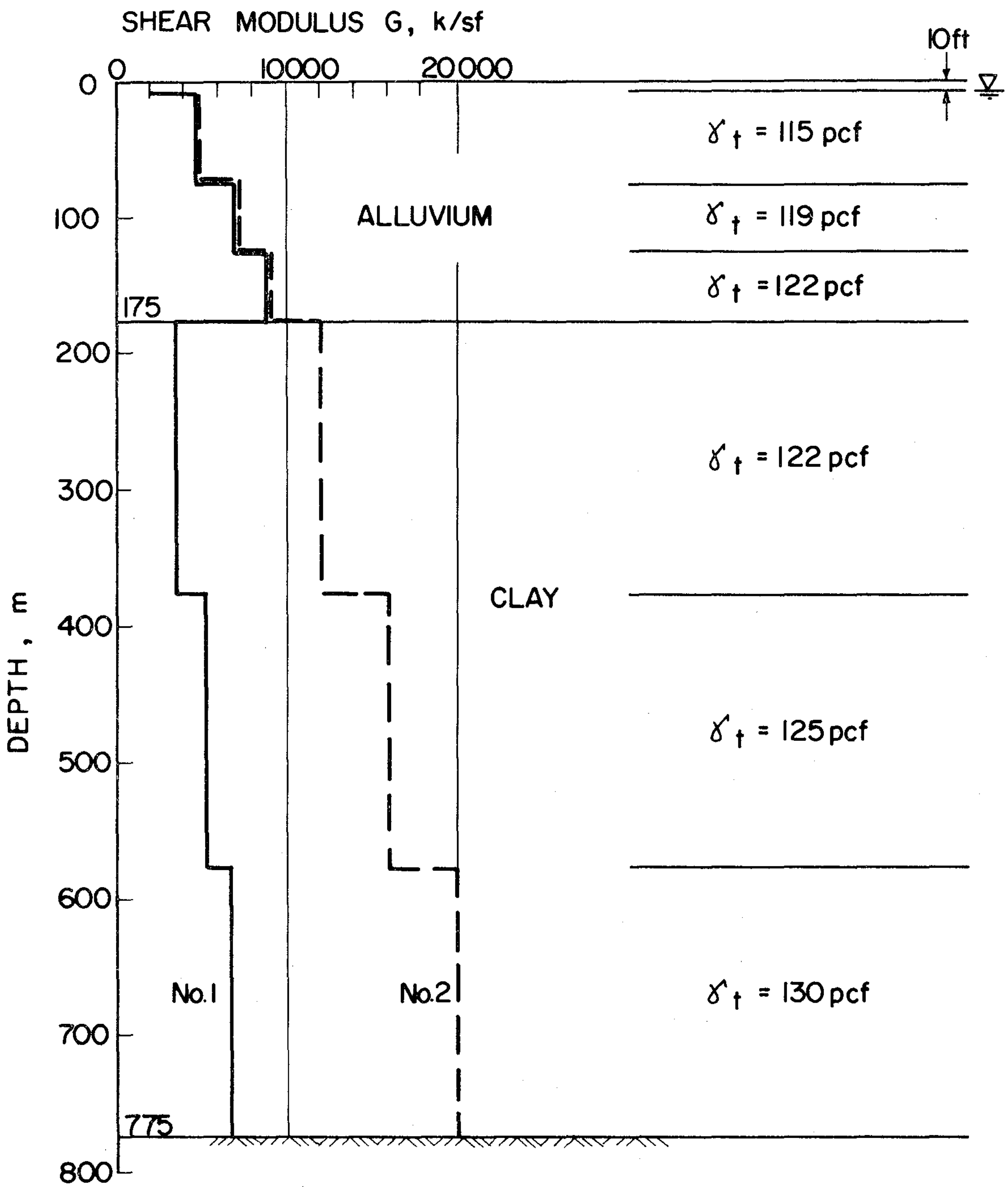

FIGURE A.1 SOIL PROFILES FOR PHASE I STUDIES

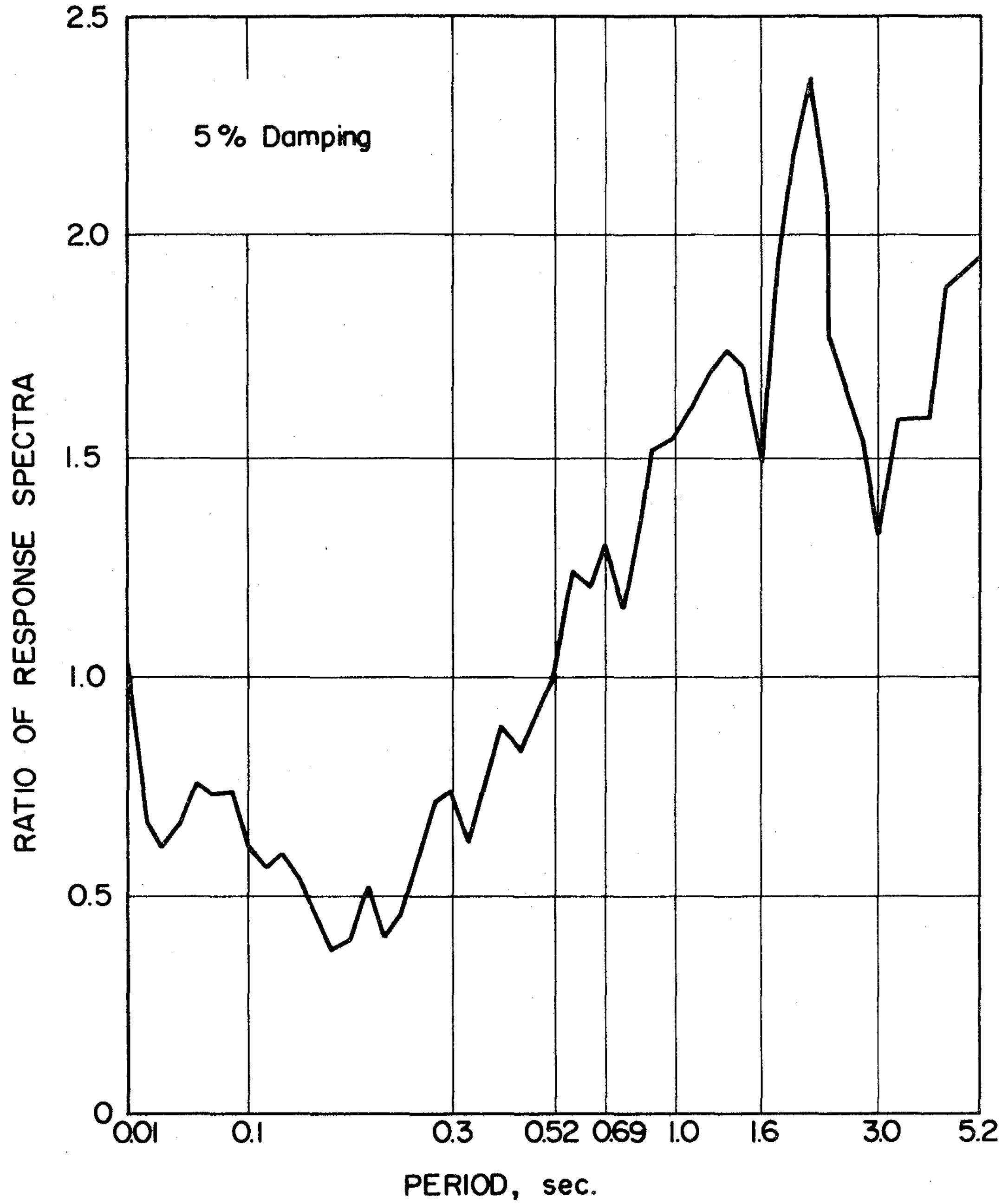

FIGURE A.2 RATIO OF RESPONSE SPECTRA FOR PROFILE 4 AND TAFT INPUT NORMALIZED TO 0.05 g

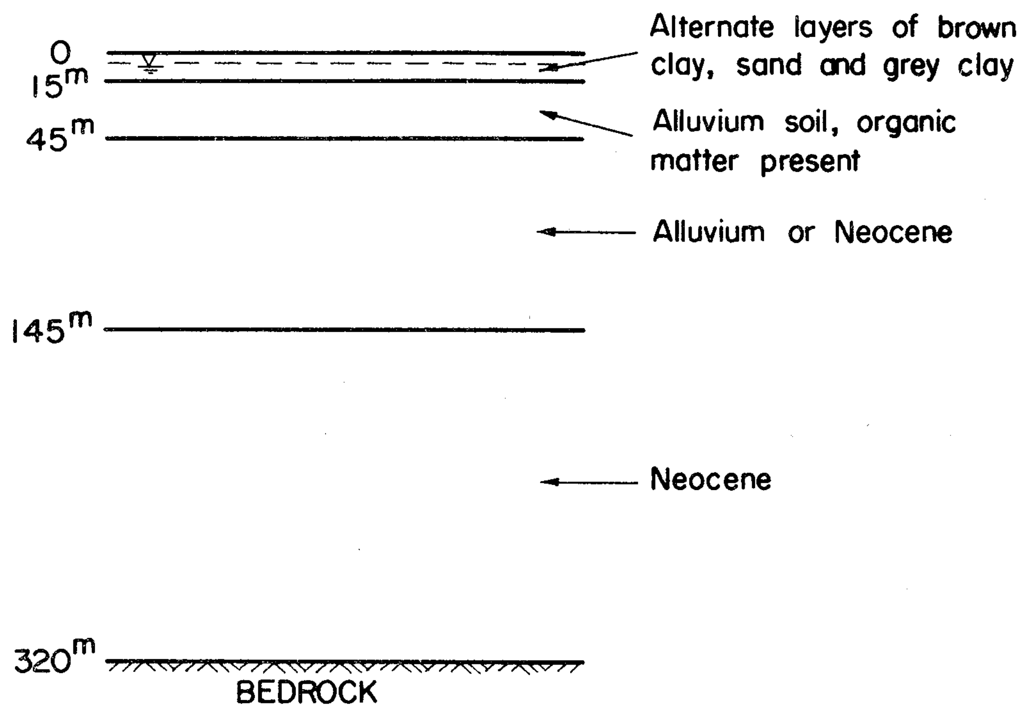

FIGURE B.1 GEOMETERY OF THE BASIC PROFILE

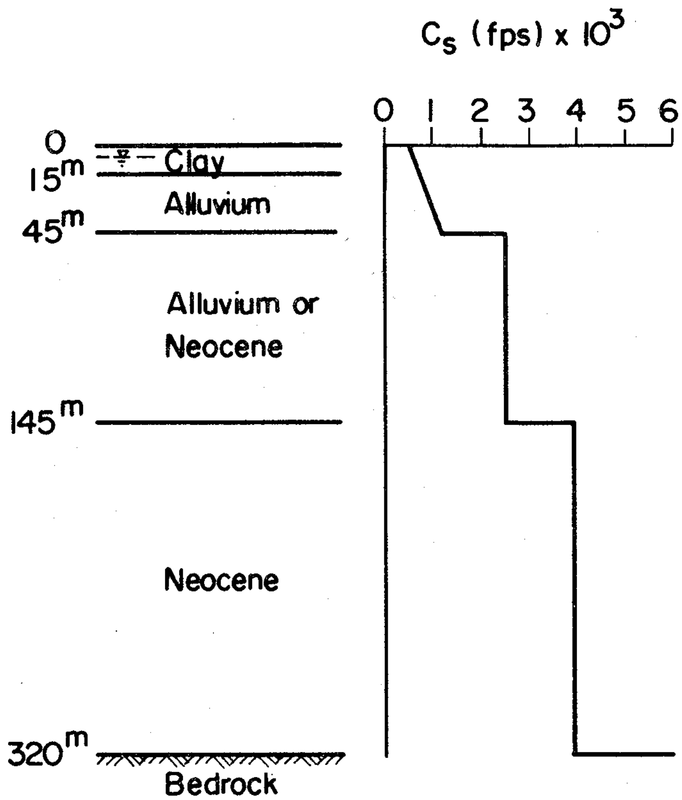

FIGURE B.2 SHEAR WAVE VELOCITY PROFILE I, (C_s - Profile I)

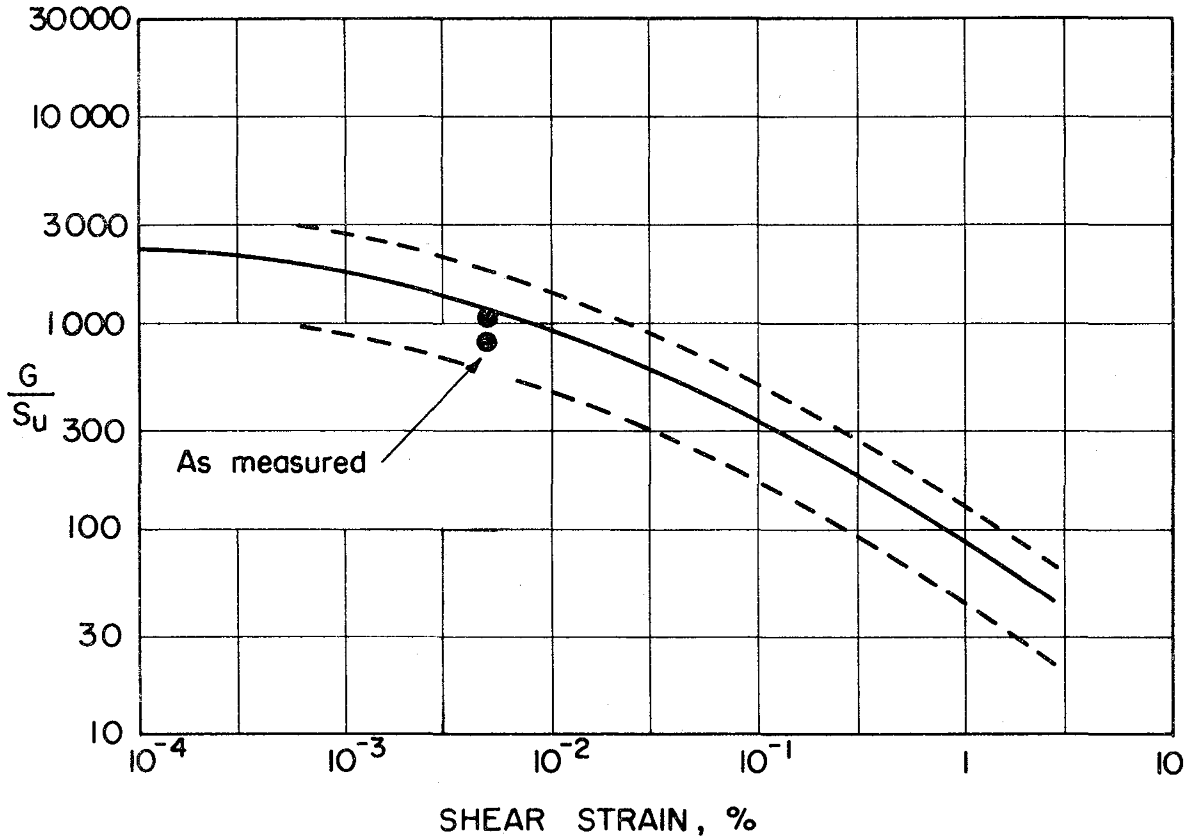

FIGURE B.3 DATA FROM RESONANT COLUMN TESTS
(Ave. curve and scatter band from Seed and Idriss, 1971)

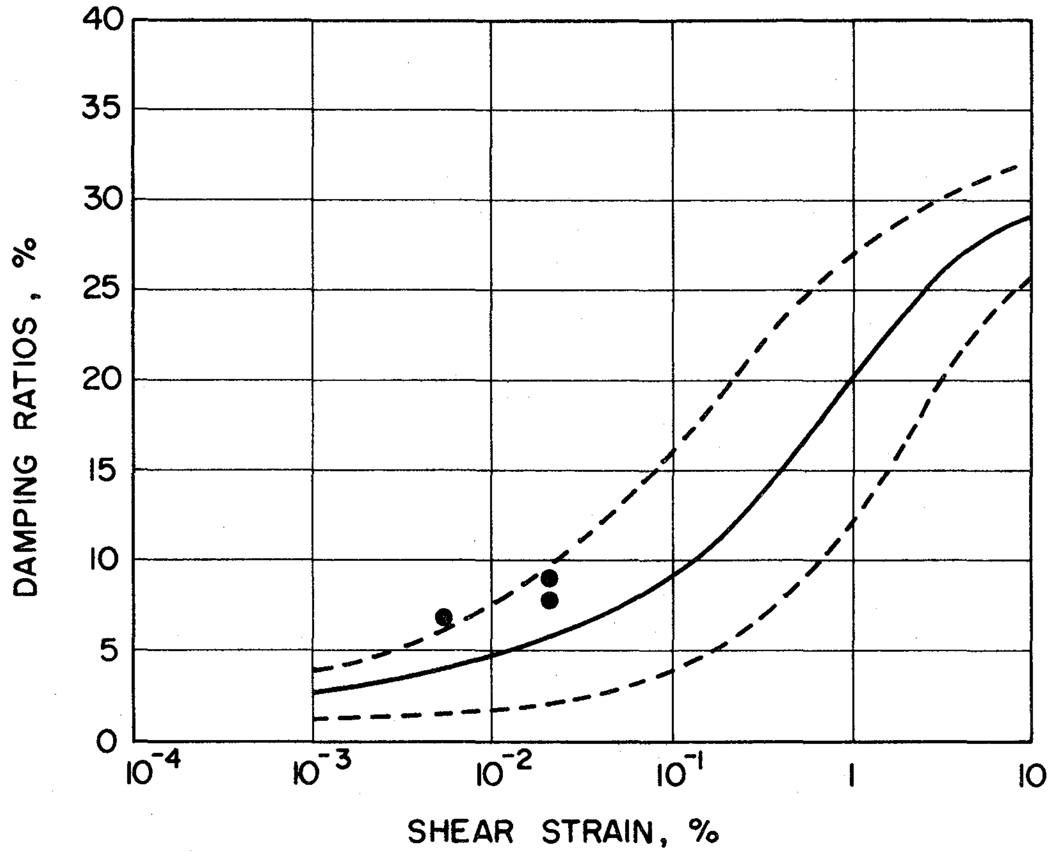

FIGURE B.4 DAMPING DATA FROM RESONANT COLUMN TESTS

(Ave. curve and scatter band from Seed and Idriss, 1971)

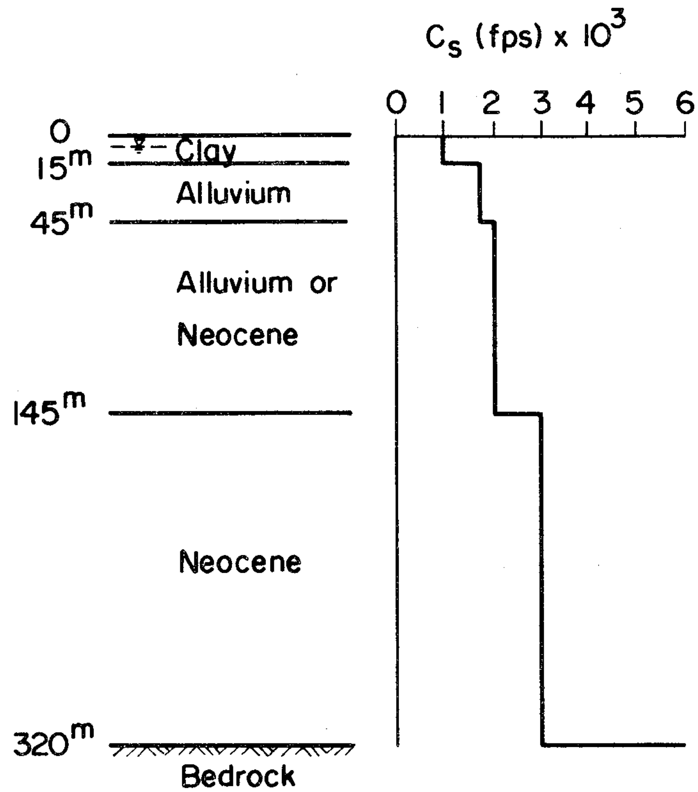

FIGURE B.5 SHEAR WAVE VELOCITY PROFILE 2,
(C_s - Profile 2)

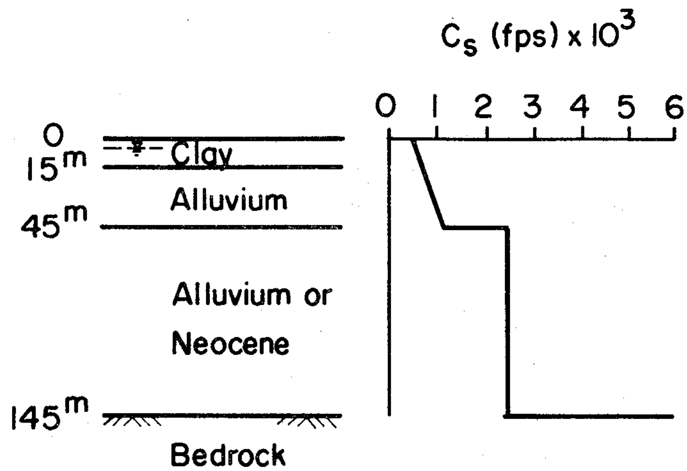

FIGURE B.6 BURSA 3 PROFILE

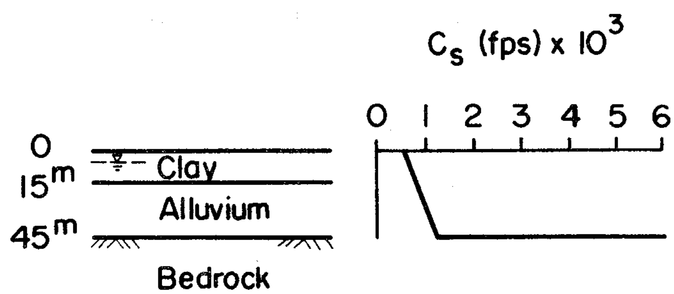

FIGURE B.7 BURSA 4 PROFILE

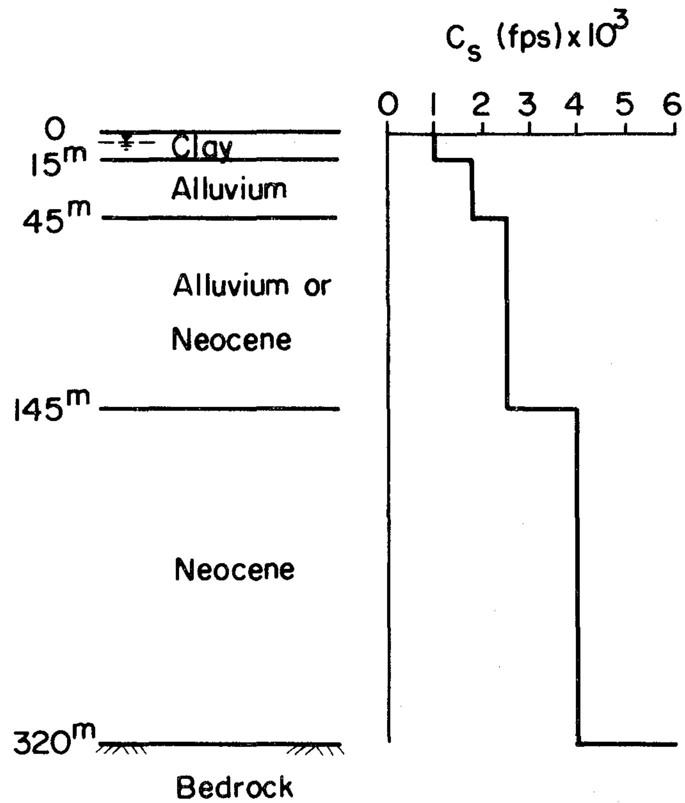

FIGURE B.8 BURSA 5 PROFILE