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Research Applied to National Needs

Internal Study Report No. 55

ELASTIC AND INELASTIC RESPONSE ANALYSIS OF SITE-INFLUENCED GROUND MOTION RECORDS

by

Mishac K. Yegian

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April 1975

Department of Civil Engineering

Massachusetts Institute of Technology

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Any opinions, findings, conclusions or recommendations expressed in this publication are those of the author(s) and do not necessarily reflect the views of the National Science Foundation.

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Chapter 1

INTRODUCTION

Earthquake induced ground motions are greatly influenced by the local soil conditions at a site. The experiences recorded in Japan (Ohsaki, 1969), Mexico City and California (Seed, 1969) affirm the hypothesis that tall and flexible structures founded on deep deposits of "soft"soil experience stronger shakings and consequently greater degree of damage than short and rigid buildings built on the same deposit. Similarly, short and rigid structures built on "firm" ground may experience greater damage than flexible structures built on the same ground. Extensive investigations have been conducted and analytical tools have been developed to determine the influence of local soil conditions upon ground motions. In the literature, we find comparisons made between the recorded motions at a site and the predicted motions obtained from analytical studies. In some cases, the agreement is quite good. However, in others the theoretical analysis fails to yield results comparable to the actual recorded data.

In most countries around the world which are vulnerable to strong earthquakes, building codes have been modified to incorporate provisions for the influence of local soil conditions. However, it has been only very recently that attempts are being made to introduce such "soil factors" in building codes across the U.S. The building code in the state of Massachusetts has now been modified to include provisions for such soil effects. Lateral force requirements are now more stringent for buildings founded on soil types other than "firm" ground.

The work presented herein was an attempt to determine the different dynamic response characteristics of some very basic soil profiles ranging in properties from Boston blue clays to very dense sand and gravel. The purpose of this report, which presents the results of these investigation, is to provide some basic understanding of the range of influences (as determined by available analytical tools) of some soils upon earthquake induced ground motions.

Chapter 2 presents a description of the one-dimensional amplification studies carried out using three types of soils; together with the results of elastic dynamic response analyses of one-degree of freedom systems subjected to the computed site influenced ground motions. Chapter 3 starts with a brief introduction to elasto-plastic dynamic analysis of 1-DOF systems and continues with the presentation of the results of some elasto-plastic dynamic analyses performed using the computed motions. Finally, some conclusions and recommendations are included in Chapter 4.

Chapter 2

ELASTIC RESPONSE ANALYSIS OF SITE-INFLUENCED GROUND MOTION RECORDS

2.1 INTRODUCTION

The results of the studies performed to determine the dynamic response characteristics of one-degree of freedom systems subjected to site-influenced ground motions are presented herein. In order to generate these records three basic soil types were chosen, namely Boston blue clay, medium dense sand and very dense sand and gravel. Profiles of several different depths (50 ft., 100 ft. and 160 ft.) were used. Utilizing these nine profiles, one-dimensional amplification analyses were carried out and site-influenced ground motions were computed.

These computed ground motions were employed to determine the elastic responses of one-degree of freedom systems. The results of these analyses are presented in this chapter in the form of elastic response spectra. Section 2.2 will briefly describe the concepts of elastic response spectrum and ratio of elastic response spectra.

2.2 RESPONSE SPECTRUM AND RATIO OF RESPONSE SPECTRA

A convenient way of interpreting an earthquake record and observing its characteristics such as frequency content and strength is to conduct dynamic analysis of 1-DOF systems using the record in question. The maximum response (acceleration, velocity or relative displacement) of a 1-DOF system with a given natural period and a damping ratio, when plotted on a response vs. period graph, will yield one point on the response spectrum curve. The response spectrum is therefore a plot of all the maxima corresponding to different natural periods of the system having a constant damping.

Frequently, it is very desirable to compare the site-influenced ground record with the record corresponding to an outcrop. This is accomplished easily by dividing the response spectrum at the ground surface by the response spectrum at the outcrop. The resulting curve is known as ratio of response spectra (RRS), the character of which is only slightly changed with changes in the frequency content of the record at the outcrop.

In the analyses, the results of which are presented in this chapter, it was assumed that the stiffness of the 1-DOF system was constant at all levels of relative distortion. Therefore, the response spectra and RRS presented herein correspond only to an elastic analysis. Chapter 3 will present the results of some elasto-plastic analyses which will be described subsequently.

2.3 DESCRIPTION OF PROFILES

2.3.1 Profile designations. Altogether, nine profiles were investigated to assess the influences of soil type and depth of profile upon the computed ground motions. For each soil type (Boston blue clay, medium dense sand and very dense sand and gravel), three soil profiles of varying depths (50 ft., 100 ft., and 160 ft.) were used in the amplification studies. The three Boston blue clay profiles were designated as BBC-1, BBC-2 and BBC-3. The numbers 1, 2 and 3 indicate the depth of the profiles as 50 ft., 100 ft. and 160 ft. respectively. Similarly, the medium dense sand and dense sand and gravel profiles were designated as MDS and DS respectively with the proper numbers following the abreviated profile names to indicate appropriate depths of the profiles.

2.3.2 Material properties. The shear wave velocity of Boston blue clay was obtained from Report No. 6 by P.J. Trudeau. The value of 850 fps for low strains was arrived at as a result of in situ measurements conducted at a site on the M.I.T. campus supplemented with results of laboratory tests using the Hardin oscillator. Figure 2.1 shows the geometry and the material properties of the three profiles (BBC-1, BBC-2 and BBC-3).

The low strain shear modulii for the other two soil types, i.e. the medium dense sand and dense sand and gravel, were estimated from equation 2.1 (Seed, 1971).

$$G = 1000\ K_2 \sqrt{\bar{\sigma}_m} \qquad (2.1)$$

Following Seed's (1971) recommendations for a medium dense sand of relative density of about 75%, the value of K_2 was assumed to be equal to 61. Likewise, for the dense sand and gravel profiles of relative density of about 90% the value of K_2 was assigned to be equal to 120. In all the analyses, the shear velocity of rock for low strains was assumed to be equal to 6000 fps. Figures 2.2 and 2.3 present the medium dense sand and dense sand and gravel profiles respectively.

2.4 ONE-DIMENSIONAL AMPLIFICATION ANALYSIS

The computer program SHAKE-3 was used to conduct the amplification studies and compute ground response records for the nine profiles. The normalized 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. An artificial earthquake record (referred to at M.I.T. as Aguirre), normalized to a peak acceleration of 0.053 g was used to study the nine previously described profiles. Additional runs were also made using the Boston clay profiles with the input record normalized to 0.15 g. These runs are designated with an asterix, i.s. BBC*-1, BBC*-2 and BBC*-3, indicating larger input acceleration. All input records were specified at the outcrop.

Figure 2.4 shows the acceleration response spectra of the Aguirre input record with peak acceleration normalized to 0.053 g and 0.15 g.

2.5 RESULTS AND DISCUSSION

Figures 2.5, 2.6 and 2.7 present the acceleration response spectra, for 5% damping, for the three types of soils investigated: Boston blue clay, medium dense sand and dense sand and gravel, respectively. From these figures, the influences of profile depth and soil stiffness can readily be observed. For example, the greater the thickness of the soil deposit is,the larger is the fundamental period of the profile. The influence of profile depth is especially significant in the case of BBC profiles and less significant in the case of dense sand profiles. In fact, increasing the depth of the Boston clay profile from 40 ft. to 160 ft. increases the fundamental period from about 0.4 to 1.6 seconds. Whereas, in the case of the dense sand profile, the fundamental period increases from 0.2 to about 0.7 seconds. Therefore, one can reasonably estimate the fundamental period of a "dense sand" deposit of up to 160 ft. thick to be in the range of 0.1 to 0.8 seconds. However, for soft deposits of considerable thickness, the estimation of the fundamental period may require more elaborate analysis. The fundamental period of rather shallow (less than 50 ft. thick) soft deposits may be approximately estimated from equation 2.2,

$$T = \frac{4H}{C_s} \tag{2.2}$$

provided some "reasonable" allowance is made for the reduction in the shear wave velocity depending upon the strength of the input record.

Another observation that can be made from Figures 2.5, 2.6 and 2.7 is that the deeper the profile is,the smaller is the peak spectral acceleration. Here again this influence of the profile depth on the peak spectral accelerations is very predominant in the case of the soft deposits and less significant in the case of the stiffer deposit.

In order to reduce the influence of the frequency characteristics of the input record upon the computed responses, ratios of response spectra for the three soil types were computed and are presented in Figures 2.8, 2.9 and 2.10. The periods at peak responses observed from these figures are not very much different from the ones obtained from the acceleration spectra of figures 2.5 to 2.7, probably indicating that, for the profiles studied, the frequency characteristics of the Aguirre record has little influence upon the computed responses within the neighborhood of the fundamental periods of the profiles. A further observation that is made from Figures 2.8 to 2.10 is that, for all the cases studied, the maximum values of the ratios of response spectra are roughly the same and fall between 4 and 5. This can be explained by recalling the two previous observations made from the acceleration spectra of figures 2.5 to 2.7, namely, the deeper the profile, the larger the fundamental period of the profile and the smaller the spectral acceleration. As the depth of the profile increases, the fundamental period of the deposit increases such that the reduced spectral acceleration of the input record at the fundamental period causes a decrease in the peak spectral acceleration at the ground level, thus yielding almost constant peak ratio of spectra for different depths.

Figure 2.8, which presents the results for the three Boston profiles, confirms the validity of the hypothesis that tall (flexible) buildings founded on deep, soft deposits or short (rigid) buildings built on shallow deposits of soft soil experience much larger spectral accelerations than short buildings founded on deep soft deposits or tall buildings founded on shallow soft deposits.

If we choose to define "firm ground"as any deposit of dense sand and gravel with thickness of up to 150 ft., then Figure 2.8 suggests that the response of a 50 ft. deposit of Boston blue clay will not be very much different from that of the "firm ground." However, in the extreme case, if the depth of the profile is about 150 ft., then a flexible structure (with a period of about 1.4-2.0 sec.) founded on Boston blue clay may experience 3 to 4 times the spectral accelerations experienced by a similar structure built on "firm ground."

In order to determine the influence of the strength of the input record upon the computed responses, the analyses for the Boston blue clay profiles were repeated using the Aguirre record normalized to a peak acceleration of 0.15 g. The results of these studies labelled as BBC* are presented in Figures 2.11 and 2.12. From these figures it is apparent that increasing the strength of the input significantly increases the fundamental periods of the deposits. For example, the fundamental period of BBC-3 was about 1.4-1.6 seconds. This can be attributed to the fact that increasing the strength of the input induces larger strains in the soil, thus softening the soil due to reduction in the modulii. Figure 2.12 indicates slightly smaller peak ratios of response spectra than previously computed for BBC profiles. This is due to an increase in the material damping which is a consequence of larger strains caused by the stronger input.

Table 2.1 summarizes the maximum ground accelerations and velocities for all the cases studied and presented in this chapter. The amplification ratios indicated in the table were obtained by dividing the maximum ground acceleration.

Finally Figure 2.15 shows the trend in the reduction of the normalized peak spectral accelerations with increasing depth of profile. Included in this figure are a few points corresponding to some data reported by Seed such as: El centro (1940), Alexander building, SF (1957) and State building, SF(1957).

Chapter 3

INELASTIC RESPONSE ANALYSIS OF SITE-INFLUENCED GROUND MOTION RECORDS

3.1 INTRODUCTION

The dynamic analyses presented in Chapter 2 were repeated using non-linear springs to model the one-degree of freedom systems. This chapter presents the results of these analyses performed using two of the Boston blue clay profiles, BBC-1 and BBC-3. The results are shown in the form of inelastic response spectra. Section 3.2 briefly describes the inelastic analysis performed together with some explanation of how to interpret inelastic response spectra.

3.2 INELASTIC RESPONSE SPECTRA

Dynamic response analysis of a one-degree of freedom system entails the solution of equation 3.1 for u

$$m\ddot{u} + c\dot{u} + Ku = -m\ddot{u}_g \qquad (3.1)$$

in which

m=mass of the system
c=coefficient of viscous damping of the system
K=stiffness of the system
$\ddot{u}_g$=acceleration of the ground
u=displacement of the system relative to the ground.

When the system is elastic, K is constant and the solution for u can be obtained from equation 3.1 by using any of the available numerical techniques such as the so-called constant velocity or linear acceleration or Newmark β methods. The quantity $(\frac{K}{m}) \times |u|_{max}$ which is called the pseudo acceleration may then be computed. The pseudo acceleration is a useful measure of the maximum absolute acceleration and is equal to it when the damping of the system is zero.

If the system is inelastic, then K will be a function of the relative displacement u. In the studies presented in this chapter, the system analysed was assumed to be elasto-plastic, that is, the force in the spring was assumed to be proportional to the relative displacement (K = constant) up to a yield displacement, u_{yield} beyond which the spring force was assumed to be constant and equal to F_{ult}. The solution in this case was again obtained numerically, except that once the computed relative displacement exceeded u_{yield} equation 3.1 was replaced by equation 3.2:

$$m\ddot{u} + c\dot{u} + F_{ult} = -m\ddot{u}_g \qquad (3.2)$$

In order to facilitate the interpretation of the results of the elasto-plastic analyses performed, the concept of ductility ratio defined by equation 3.3 was used

$$\mu = \frac{|u|_{max}}{u_{yield}} \qquad (3.3)$$

For example, a ductility ratio of 1 or less implies an elastic behavior of the system.

Using the computed records of BBC-1 and BBC-3, elasto-plastic analyses of systems with different natural periods were carried out. The ultimate force level was varied in the analyses to obtain yield accelerations for different ductility ratios. The computed yield accelerations corresponding to ductility ratios of 1, 2 and 4 were plotted to obtain the so-called inelastic response spectra. Therefore, an inelastic response spectrum specifies the design yield acceleration together with the corresponding required ductility ratio in the system. The ratio of the inelastic response spectra of the ground to outcrop for a specified ductility ratio then yields the ratio of inelastic response spectra (RIRS).

3.3 RESULTS AND DISCUSSIONS

Figures 3.1 and 3.2 show the inelastic response spectra for BBC-1 and BBC-3 for 5% damping. Also included in these figures are the elastic response spectra(μ=1) for two profiles. From these figures it is readily observed that the greater the available ductility is in a structure, the smaller is the spectral (yield) acceleration experienced. In fact, for ductility ratio of about

4 which is a reasonalbe value for most steel structures. The spectral accelerations in general are smaller than the maximum ground acceleration. Also for ductility ratio of about 4 the depth of the profile has insignificant effect upon spectral acceleration for the period range greater than 0.8 seconds.

Figures 3.3 and 3.4 present the ratios of the inelastic response spectra of ground motion to outcrop for BBC-1 and BBC-3. The peak values in the RIRS (3-4) are somewhat smaller than those obtained from the elastic analysis (4-5).

Chapter 4

CONCLUSIONS

The results of the elastic analyses presented in this report clearly indicate that the characteristics of a site-influenced ground motion record very much depend upon the stiffness of the soil and the thickness of the deposit. For example, the stiffer the soil is, the greater is the maximum ground acceleration. Also, for the same depth of profile, the stiffer the soil is, the smaller is the fundamental period of the profile. Here, it is worthwhile to note that, due to nonlinear effects, the stiffness of a soil deposit is a function of not only the material properties at low strains, but also the strength of the input motions.

The influence of profile depth upon the computed responses is very apparent in all the analyses. In fact, the softer the soil is, the more pronounced is this influence. Increasing the depth of a soft deposit significantly increases the fundamental period and reduces the maximum ground and peak spectral accelerations. However, it appears that the profile depth and the soil stiffness do not have any significant influence upon the peak ratio of response spectra.

Almost identical responses computed for the three dense sand and gravel profiles warrant the definition of "firm ground" as a dense sand and gravel deposit of up to 150 ft. thick. With this definition in mind, the responses of the Boston blue clay profiles may be compared with the response of the "firm ground." For example, the response of a 50 ft. thick Boston clay profile is not significantly different from the response of the "firm ground."

However, the response of a deep deposit of Boston clay may be so different that a building in resonance with a 160 ft. thick layer of Boston clay may experience 3 to 4 times the spectral accelerations of a similar building founded on the "firm ground."

The results of the elasto-plastic analyses show that the higher the available ductility is in a structure, the smaller is the design yield acceleration. For example, the design acceleration for a structure with natural period close to the fundamental period of a 160 ft. thick Boston clay profile (1.6 sec.) can be reduced by a factor of 7 provided that the available ductility ratio is increased from 1 to 4. Finally, for ductility ratios of about 4 the results from BBC-1 and BBC-3 show that the spectral accelerations are less than the maximum ground acceleration which may be amplified from rock by a factor of 2 or less depending upon the size of the input and the depth of the profile.

TABLE 2.1

SUMMARY OF AMPLIFICATION RESULTS

Description of Run	Max. Acceleration (Ground)		Max. Velocity (Ground)	
	Value	Amplific.	Value	Amplific.
INPUT	0.053	1.0	0.163	1.0
BBC-1	0.1054	2.0	0.2608	1.6
BBC-2	0.0657	1.2	0.2900	1.8
BBC-3	0.0522	1.0	0.3520	2.2
DS-1	0.1236	2.3	0.1863	1.1
DS-2	0.1402	2.6	0.2179	1.3
DS-3	0.1039	2.0	0.2889	1.8
MDS-1	0.1249	2.4	0.2522	1.5
MDS-2	0.1016	1.9	0.3494	2.1
MDS-3	0.0812	1.5	0.2695	1.7
INPUT	0.15	1.0	0.461	1.0
BBC*1	0.2215	1.5	0.9185	2.0
BBC*2	0.1566	1.0	0.7712	1.7
BBC*3	0.1021	0.7	0.7721	1.7

* Aguirre #3 0.15 g Max acceleration

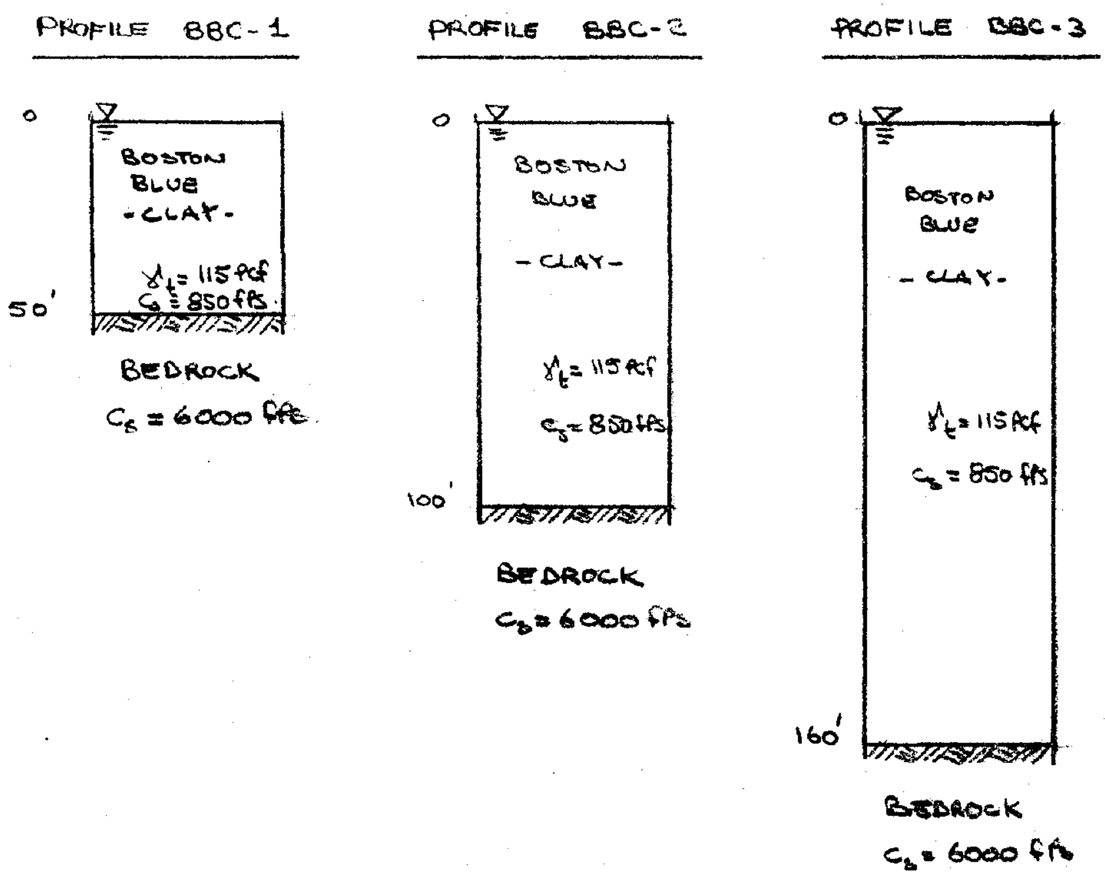


FIGURE 2.1 BOSTON BLUE CLAY PROFILES

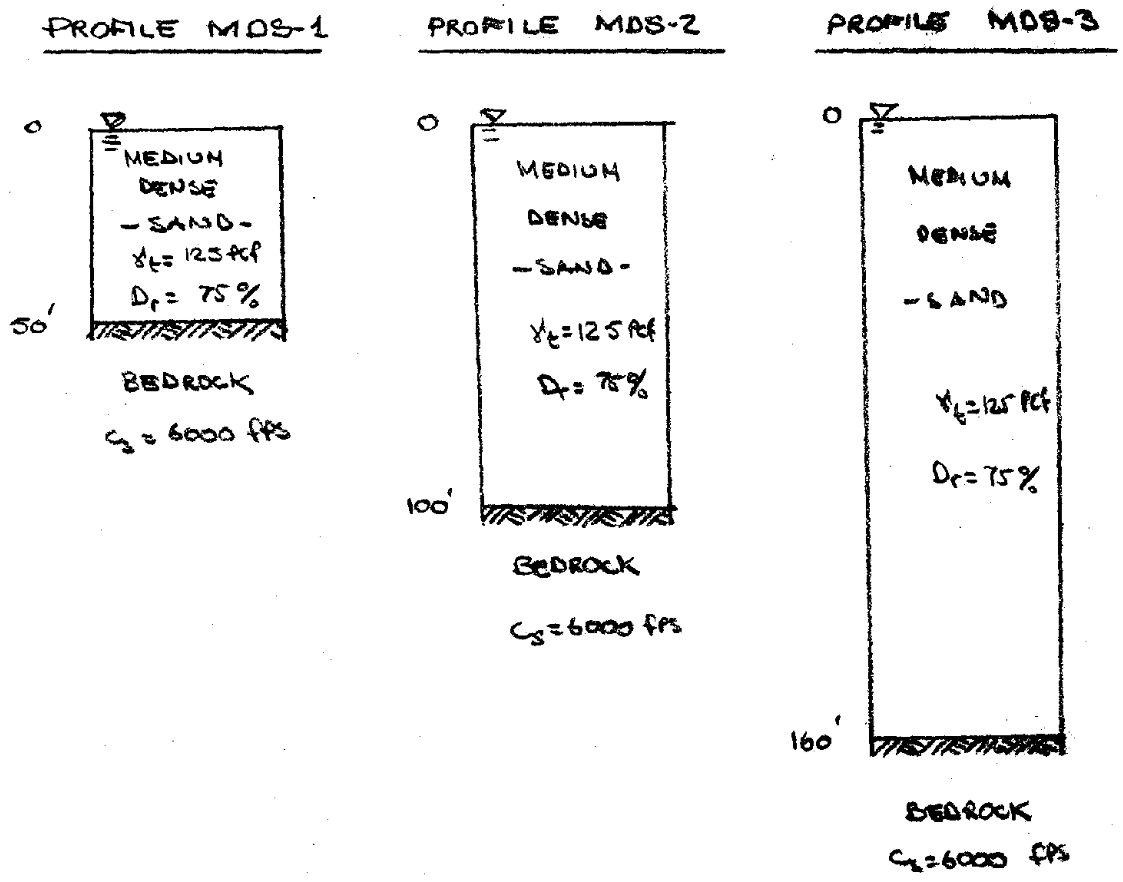


FIGURE 2.2 MEDIUM DENSE SAND PROFILES.

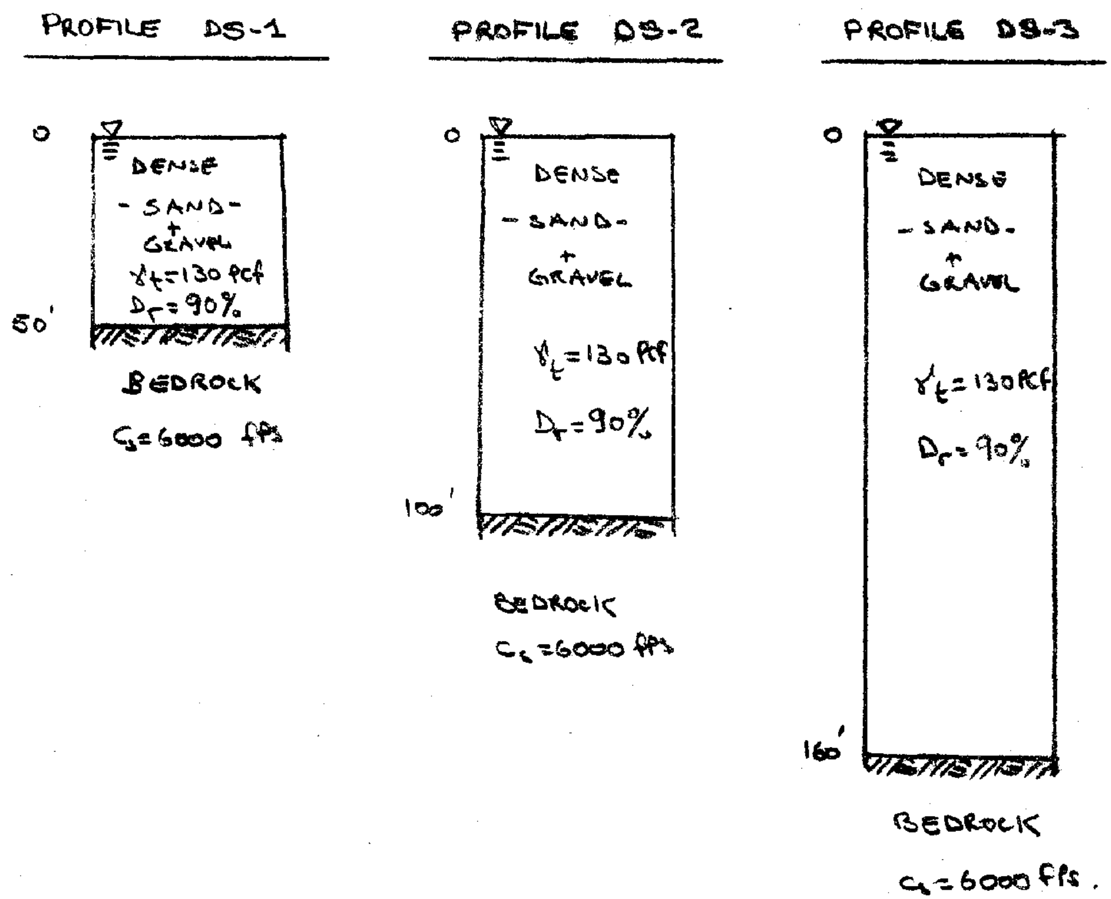


FIGURE 2.3. VERY DENSE SAND AND GRAVEL PROFILES

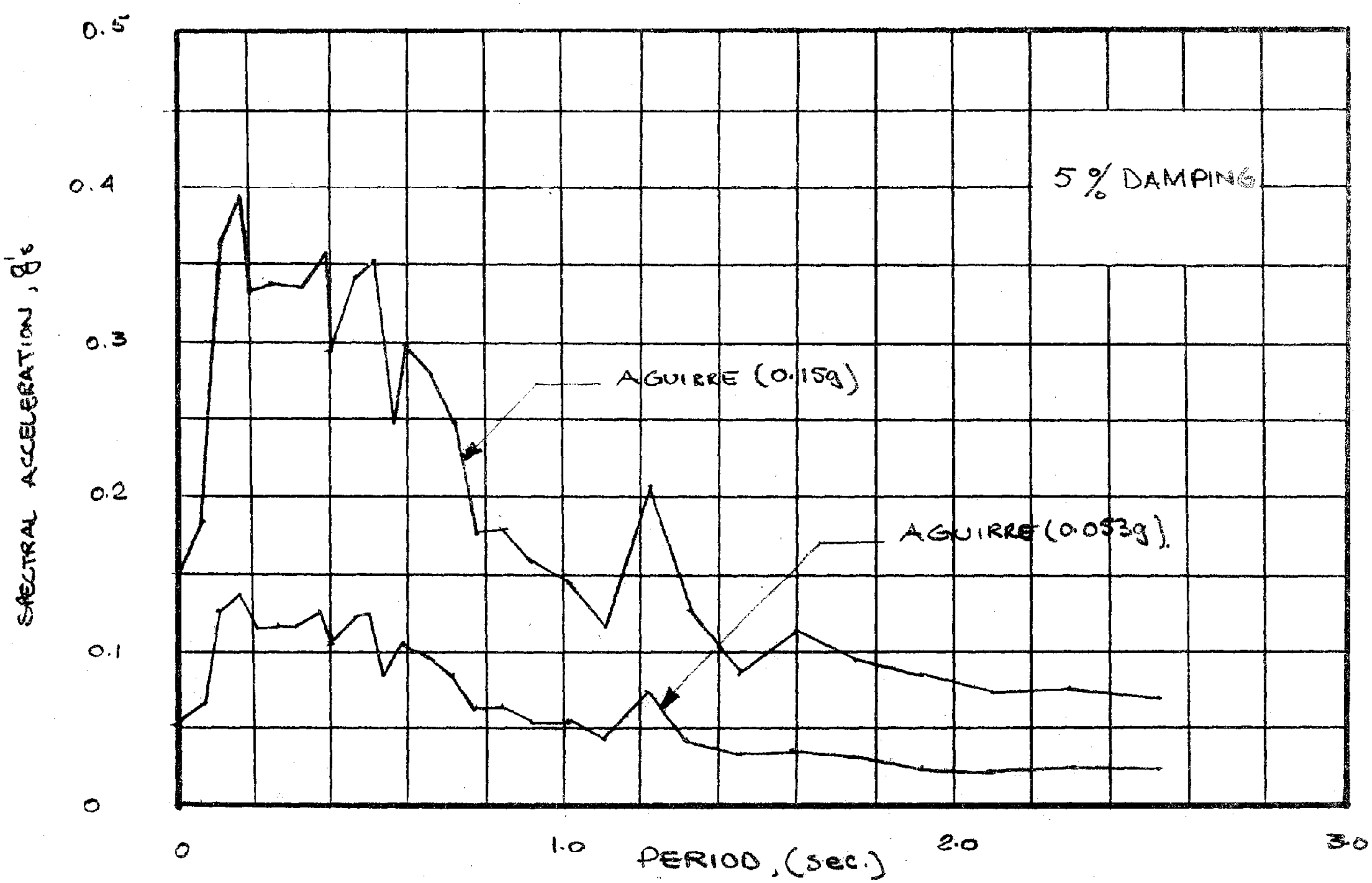


FIGURE 2.4 ACCELERATION SPECTRA OF AGUIRRE RECORD FOR 0.15g AND 0.053g MAXIMUM ACCELERATIONS.

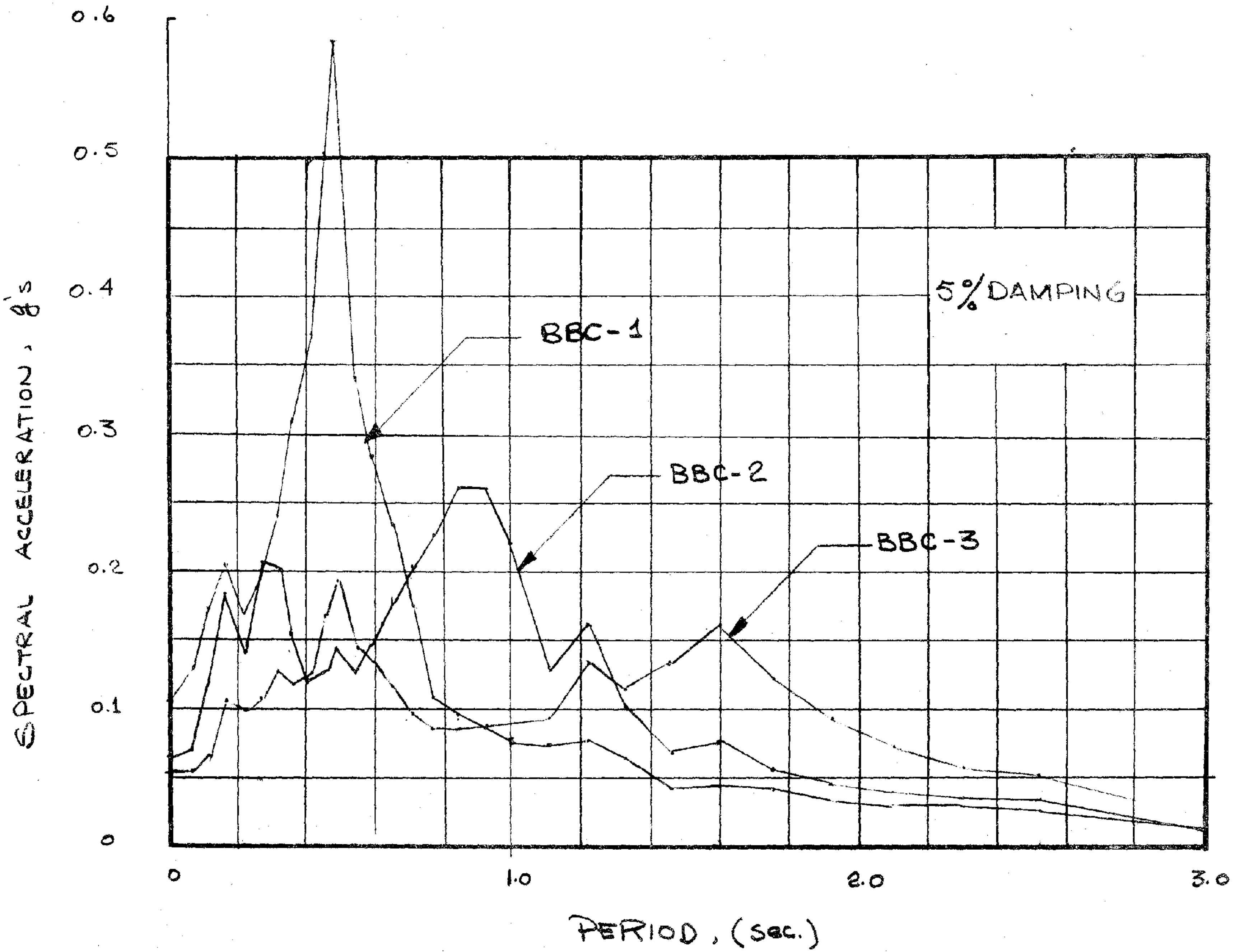


FIGURE 2.5 ACCELERATION SPECTRA FOR BOSTON BLUE CLAY PROFILES

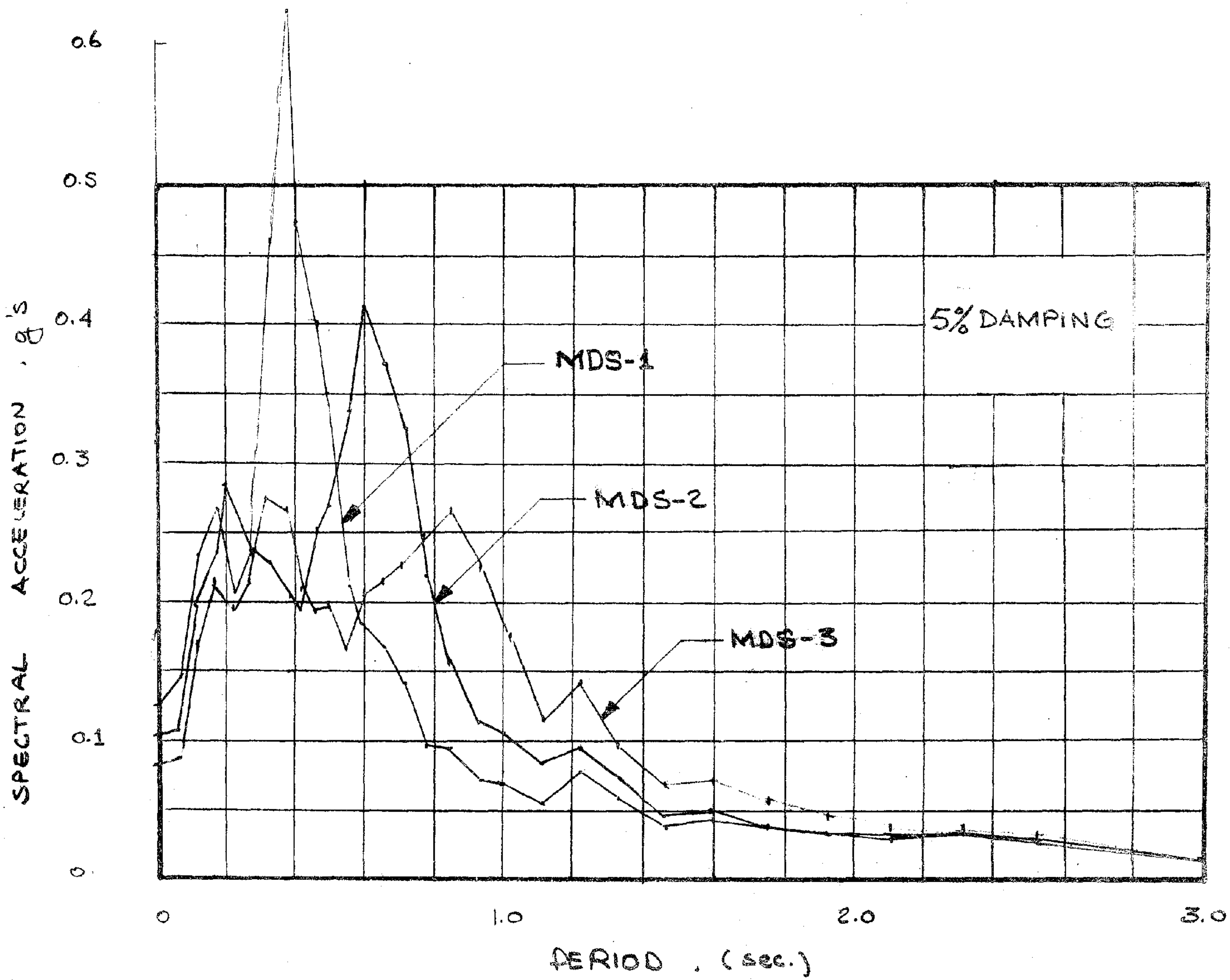


FIGURE 2.6 ACCELERATION SPECTRA FOR MEDIUM DENSE SAND PROFILES

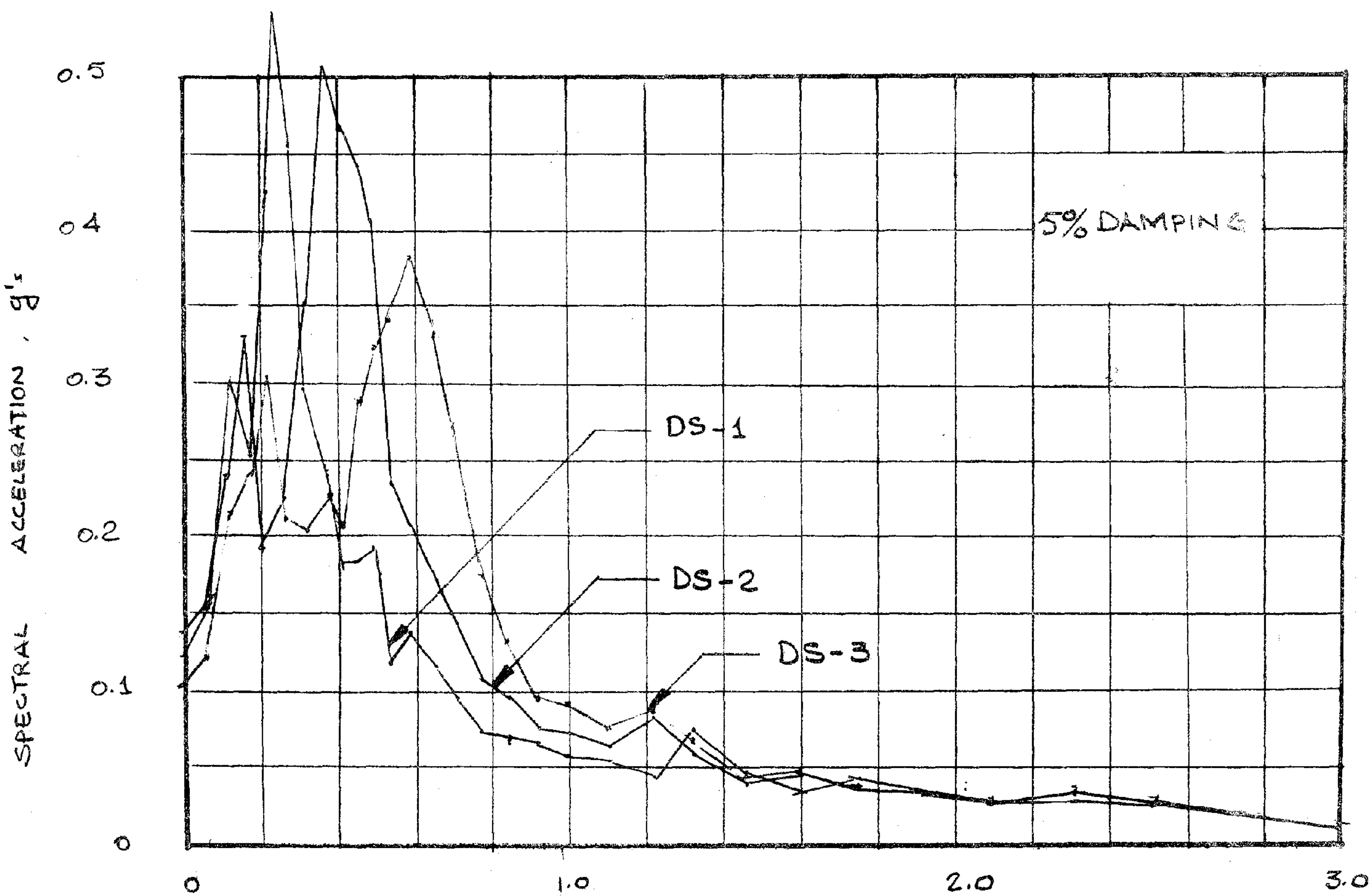


FIGURE 27 ACCELERATION SPECTRA FOR DENSE SAND AND GRAVEL PROFILES.

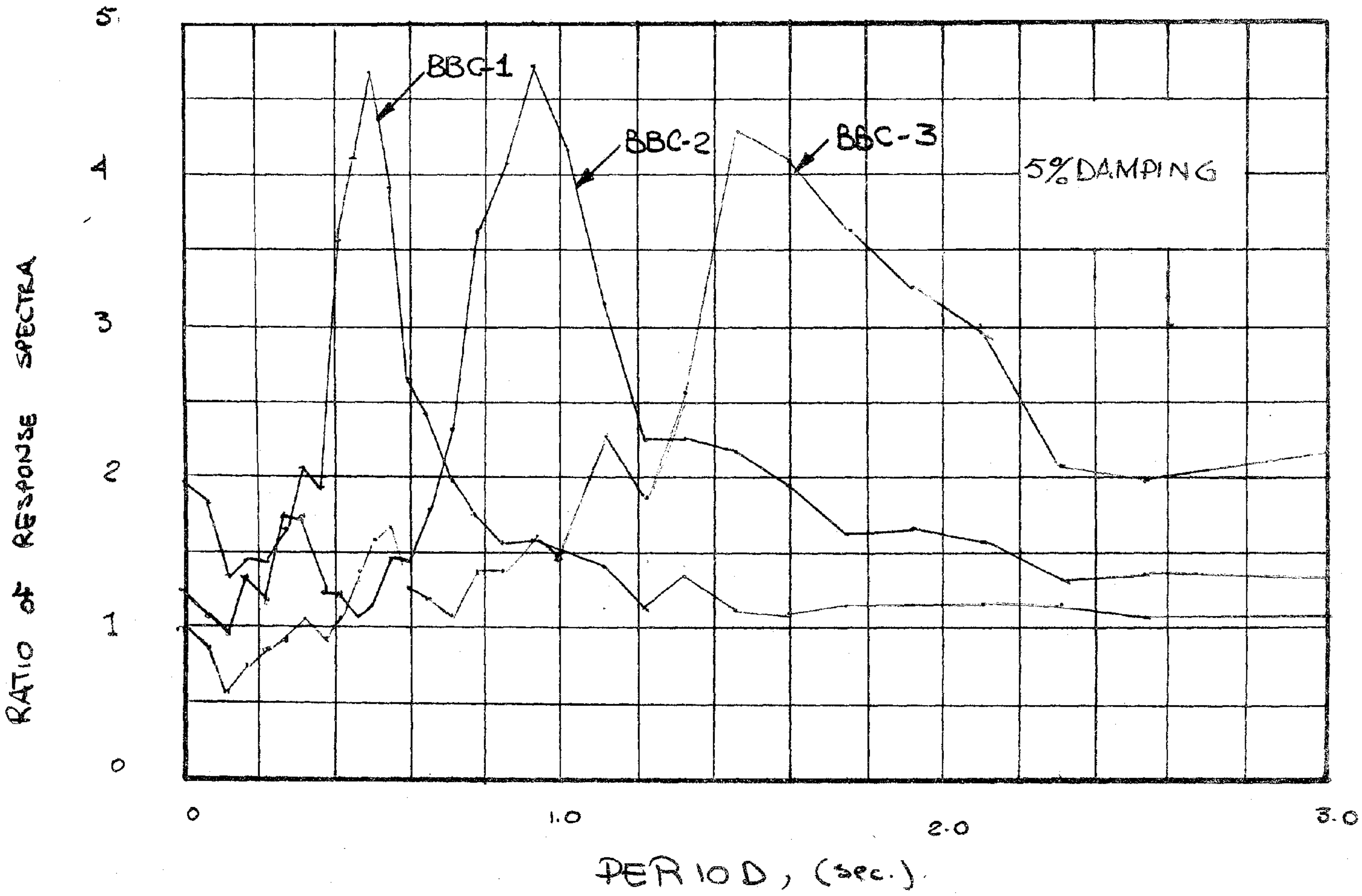


FIGURE 2.8 RATIO OF RESPONSE SPECTRA FOR BOSTON BLUE CLAY PROFILES.

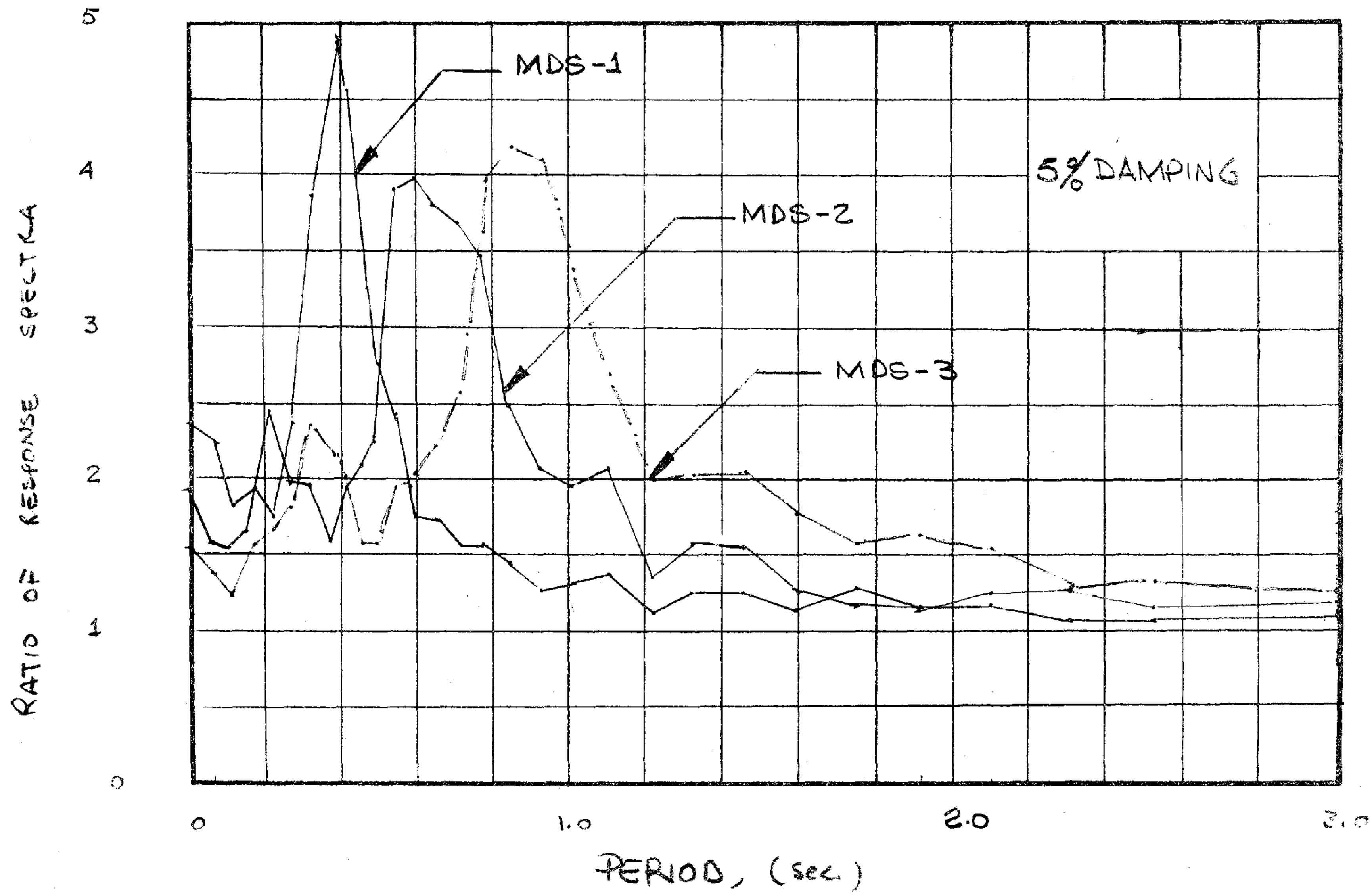


FIGURE 2.9 RATIO OF RESPONSE SPECTRA FOR MEDIUM DENSE SAND PROFILES.

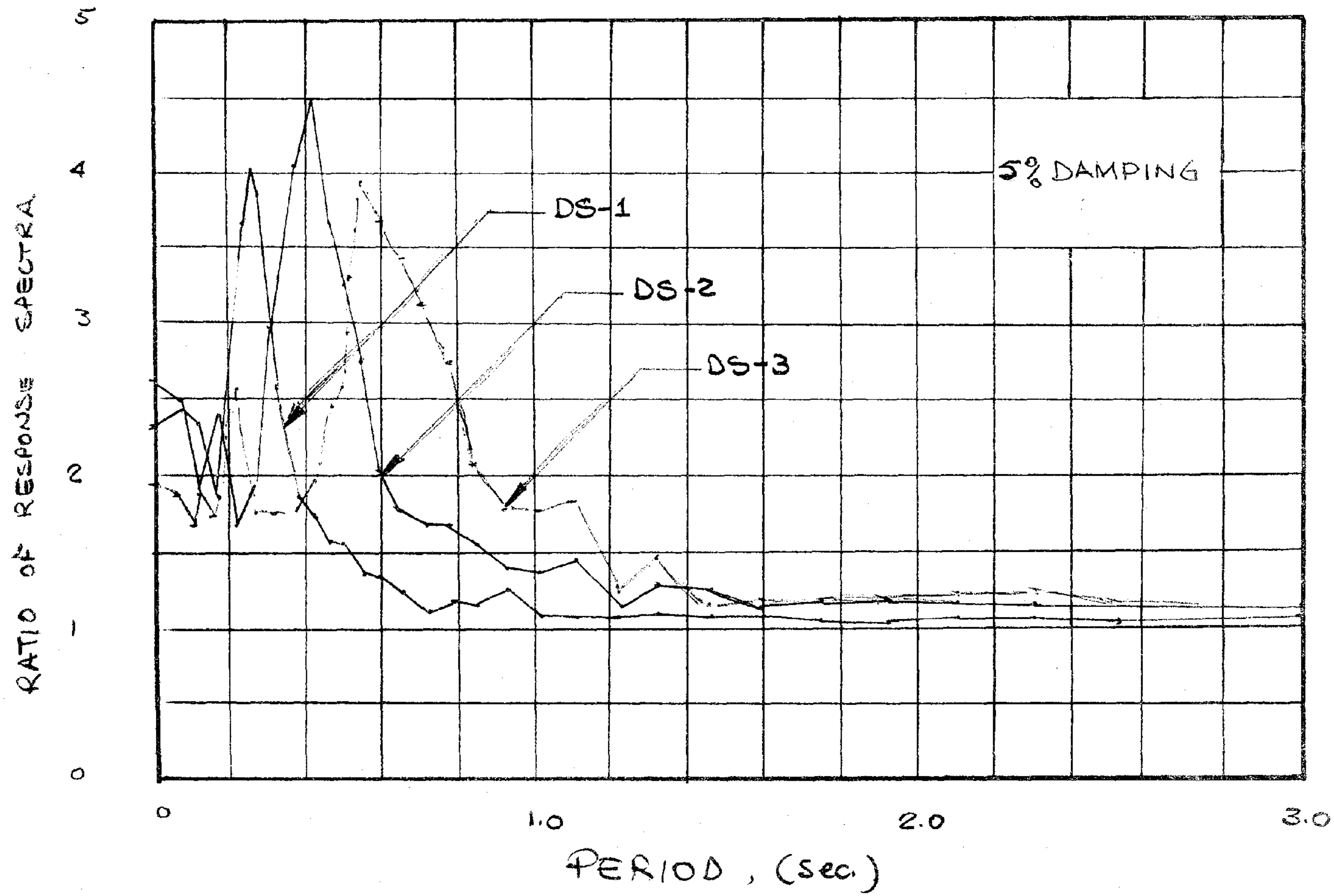


FIGURE 2.10 RATIO OF RESPONSE SPECTRA FOR DENSE SAND AND GRAVEL PROFILES

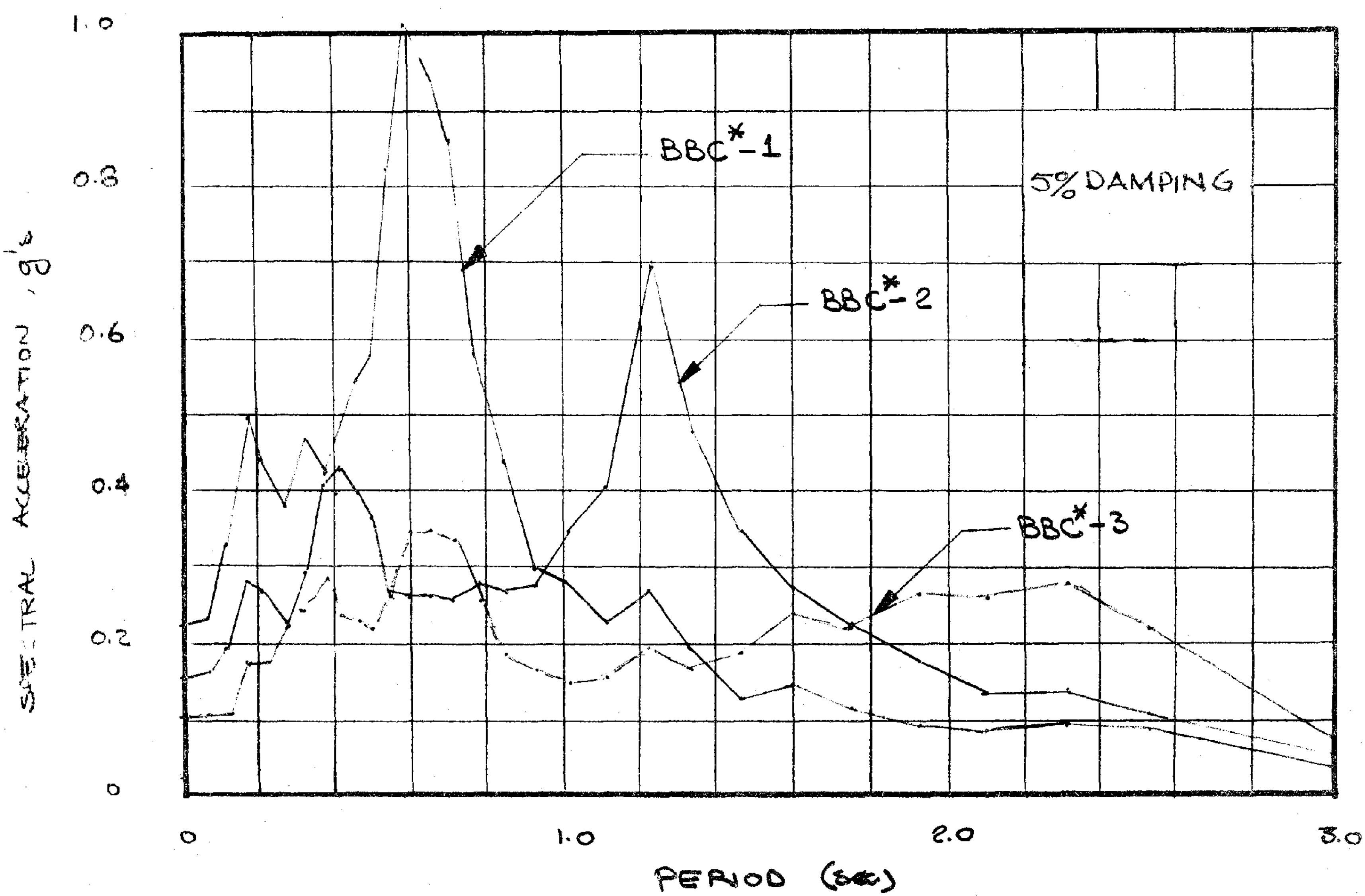


FIGURE 2.11 ACCELERATION SPECTRA FOR BOSTON BLUE CLAY PROFILES WITH INPUT NORMALIZED TO 0.15g.

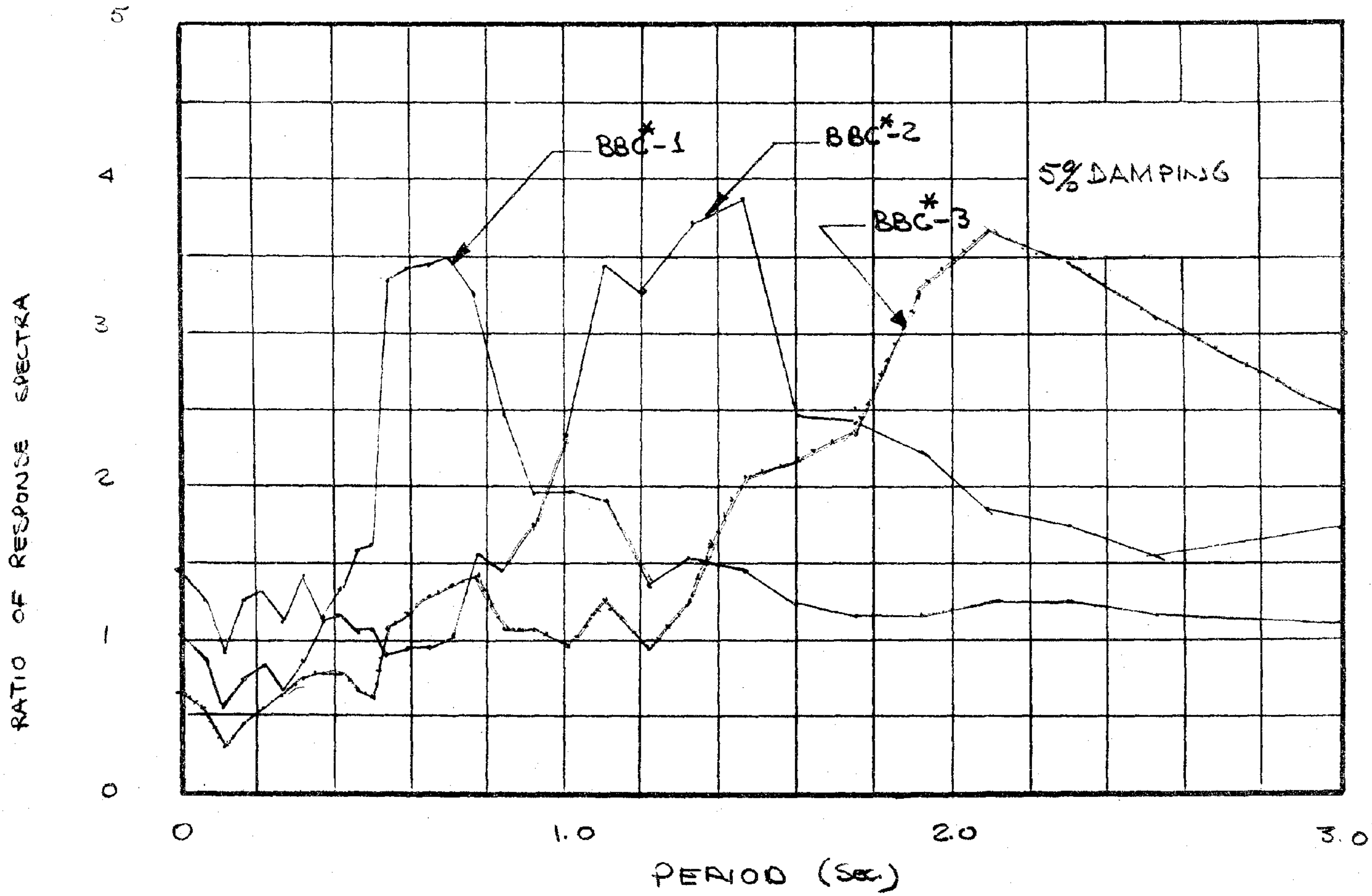


FIGURE 2.12 RATIO OF RESPONSE SPECTRA FOR BOSTON BLUE CLAY PROFILES WITH INPUT NORMALIZED TO 0.15g.

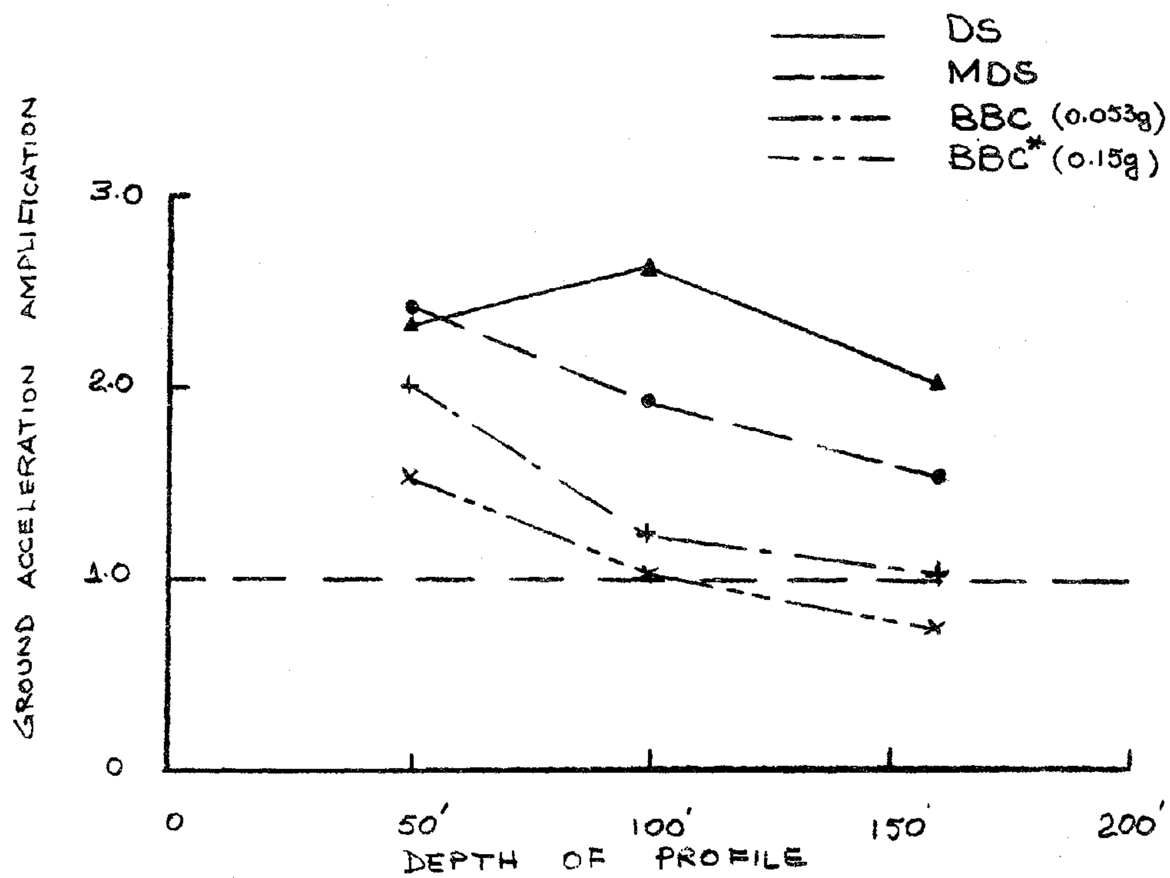


FIGURE 2.13 VARIATION OF GROUND ACCELERATION AMPLIFICATION RATIOS WITH DEPTH OF PROFILE.

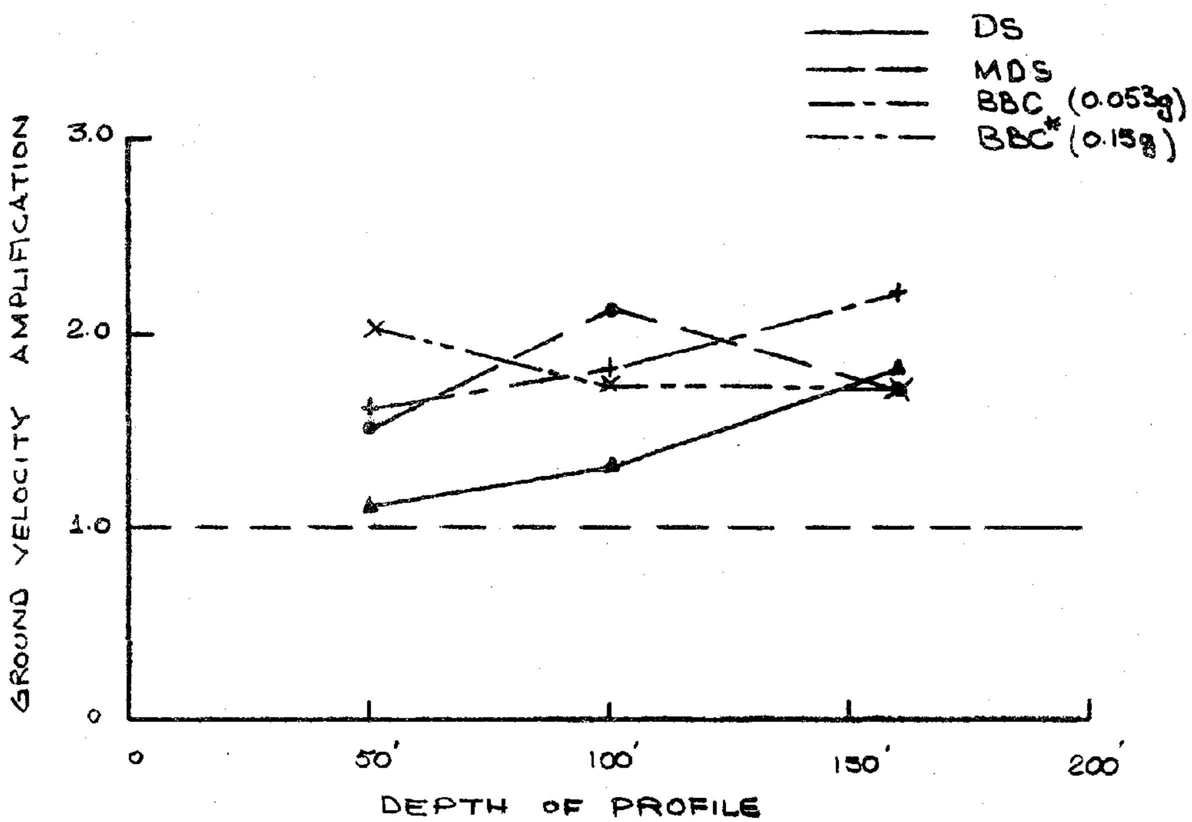


FIGURE 2.14 VARIATION OF GROUND VELOCITY AMPLIFICATION RATIOS WITH DEPTH OF PROFILE

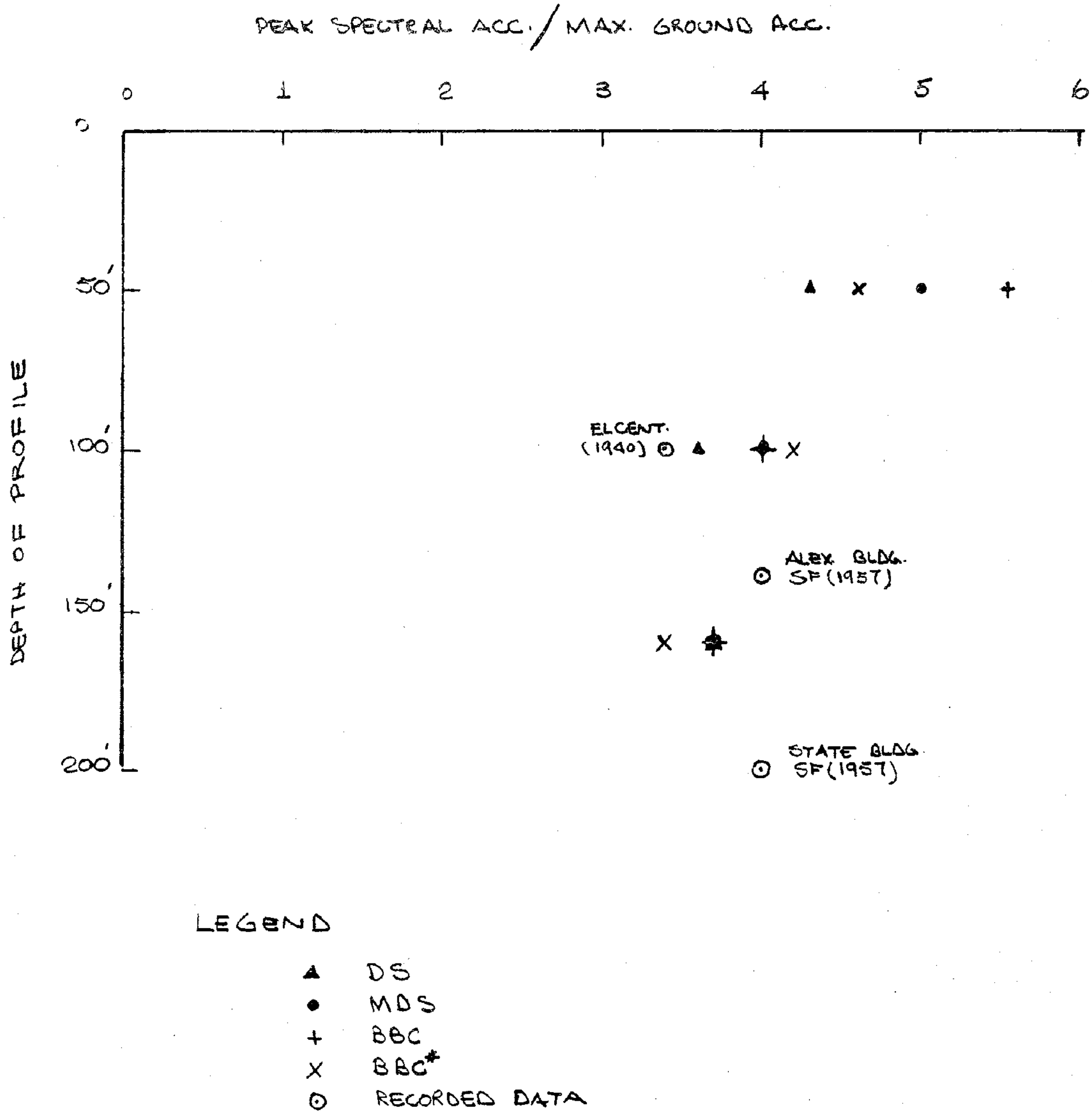
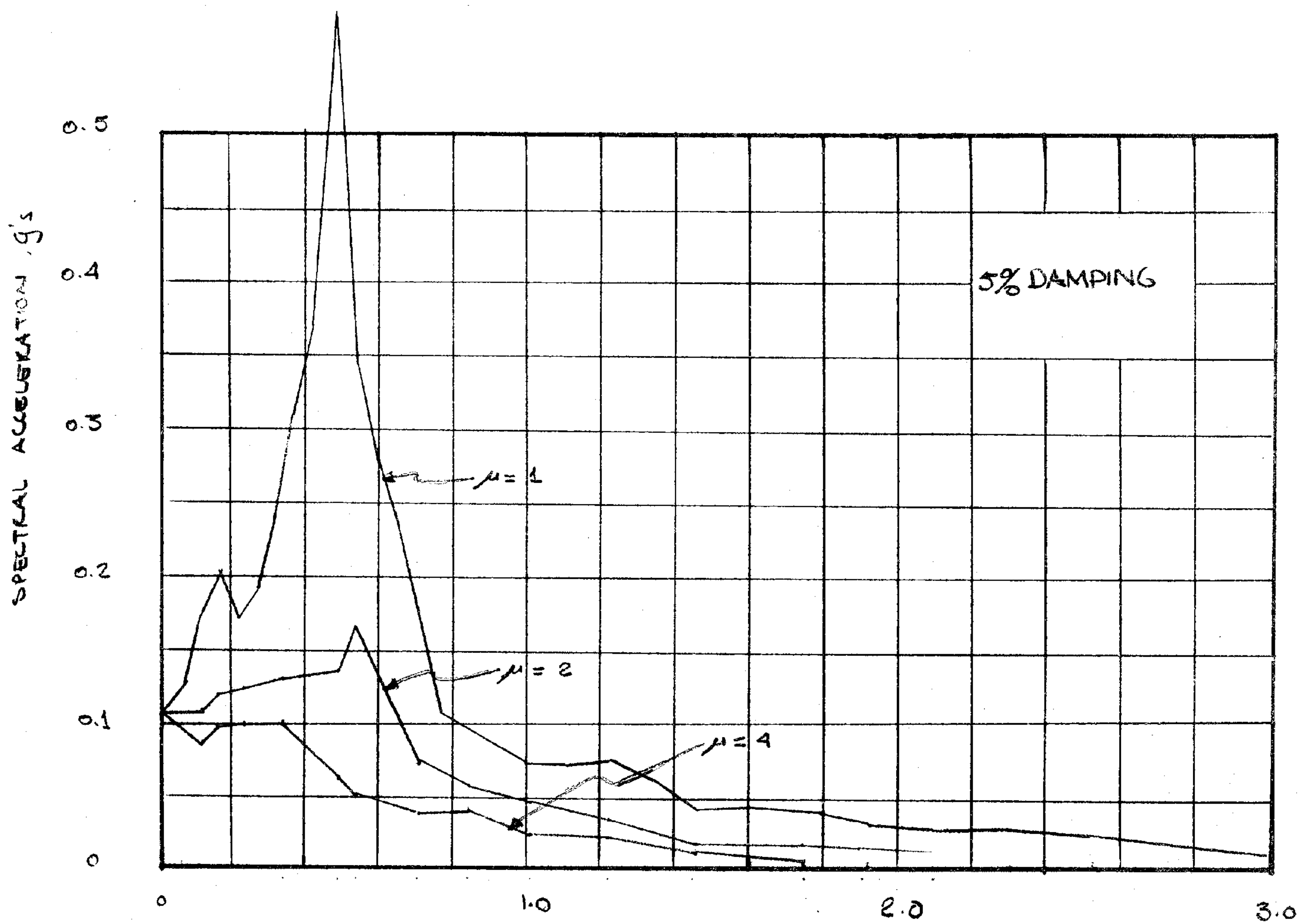


FIGURE 2.15 VARIATION OF NORMALIZED PEAK SPECTRAL ACCELERATION WITH DEPTH OF PROFILE.



FIGURES 3.1 INELASTIC RESPONSE SPECTRA FOR SOFT THICK PROFILE OF BOSTON BLUE CLAY, (BBC-1).

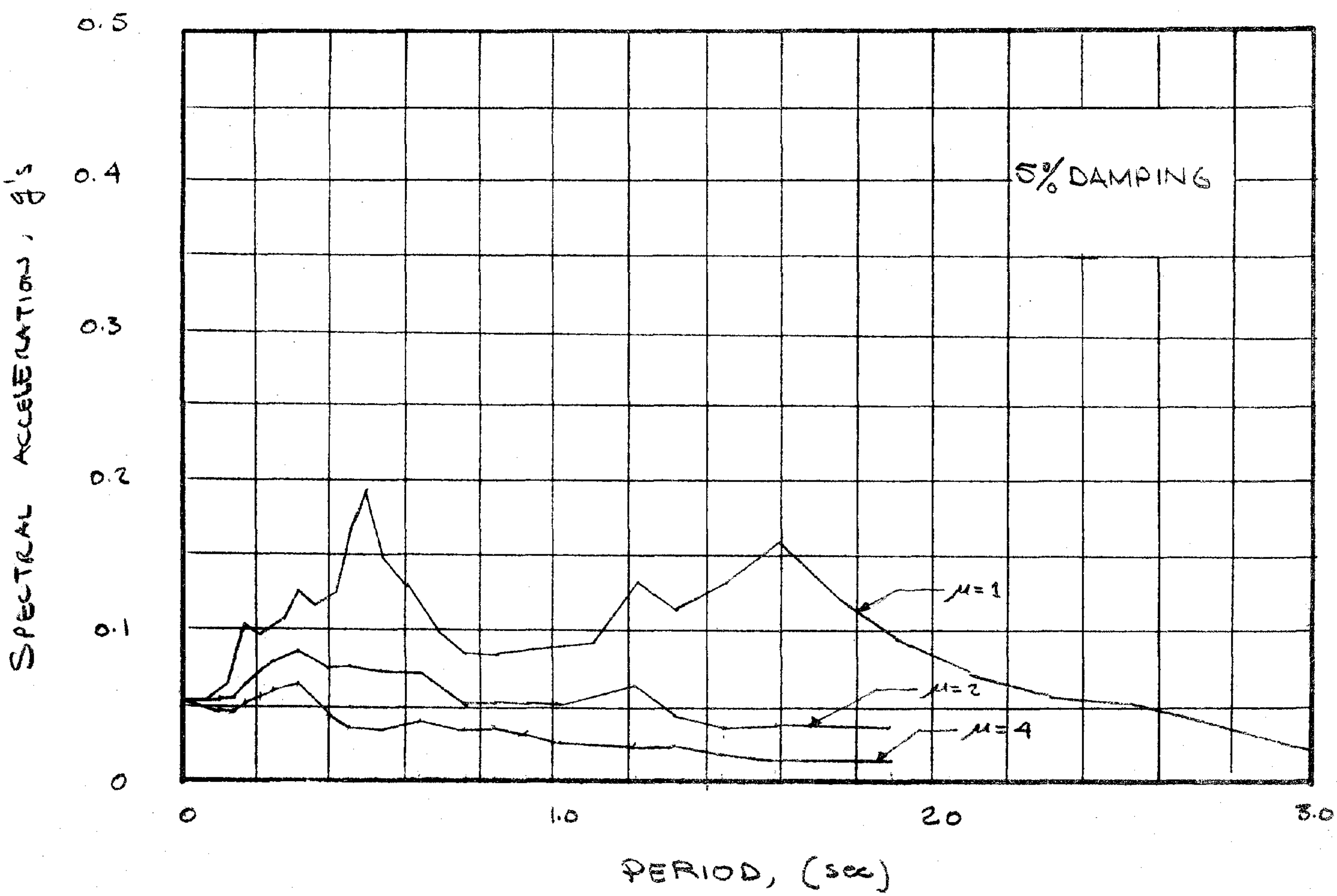


FIGURE 3.2 INELASTIC RESPONSE SPECTRA FOR A 160 FT THICK PROFILE OF BOSTON BLUE CLAY, (BBC-3).

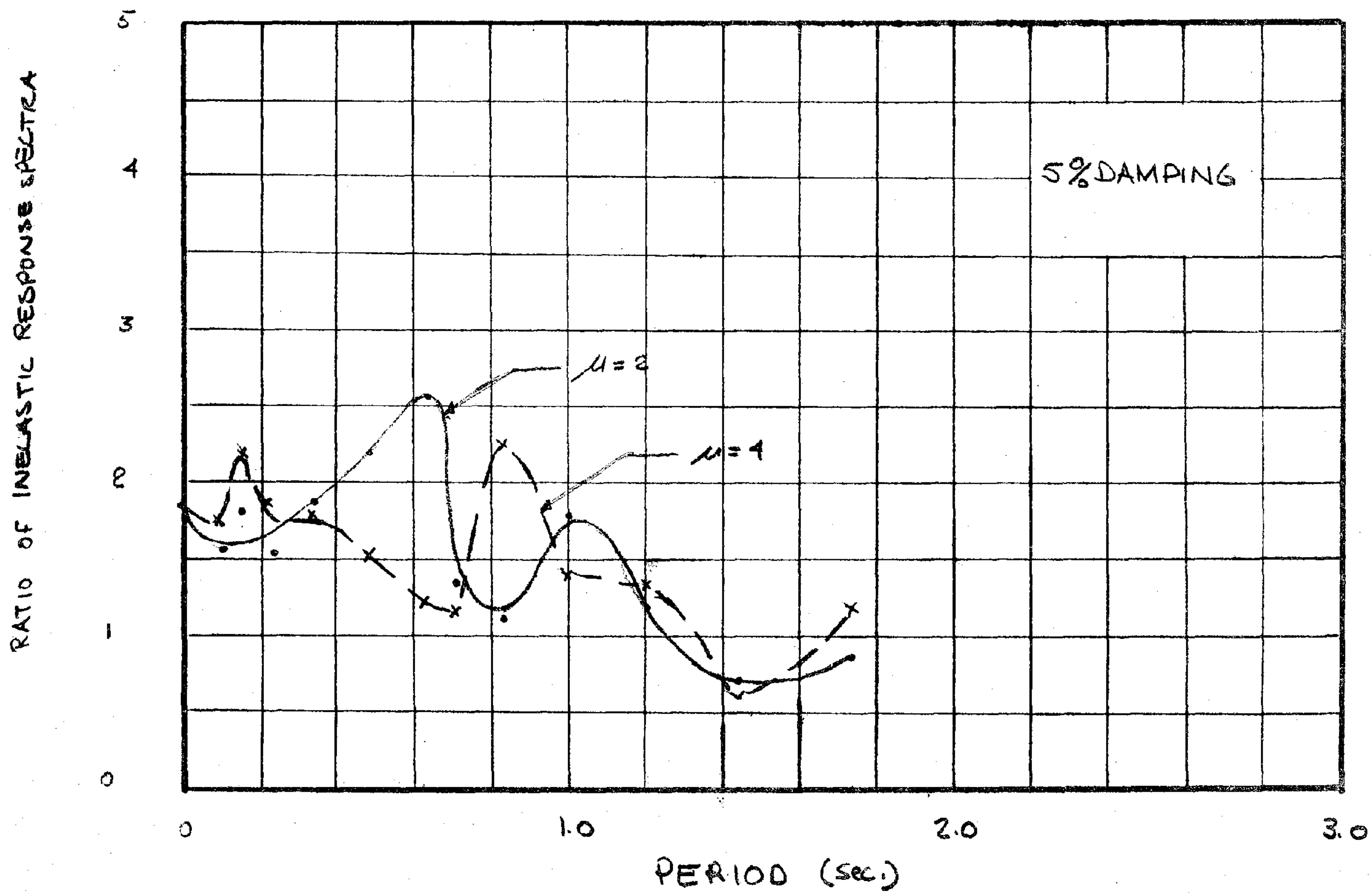


FIGURE 3.3 RATIO OF INELASTIC RESPONSE SPECTRA FOR A 50 FT THICK PROFILE OF BOSTON BLUE CLAY, (BBC-1).

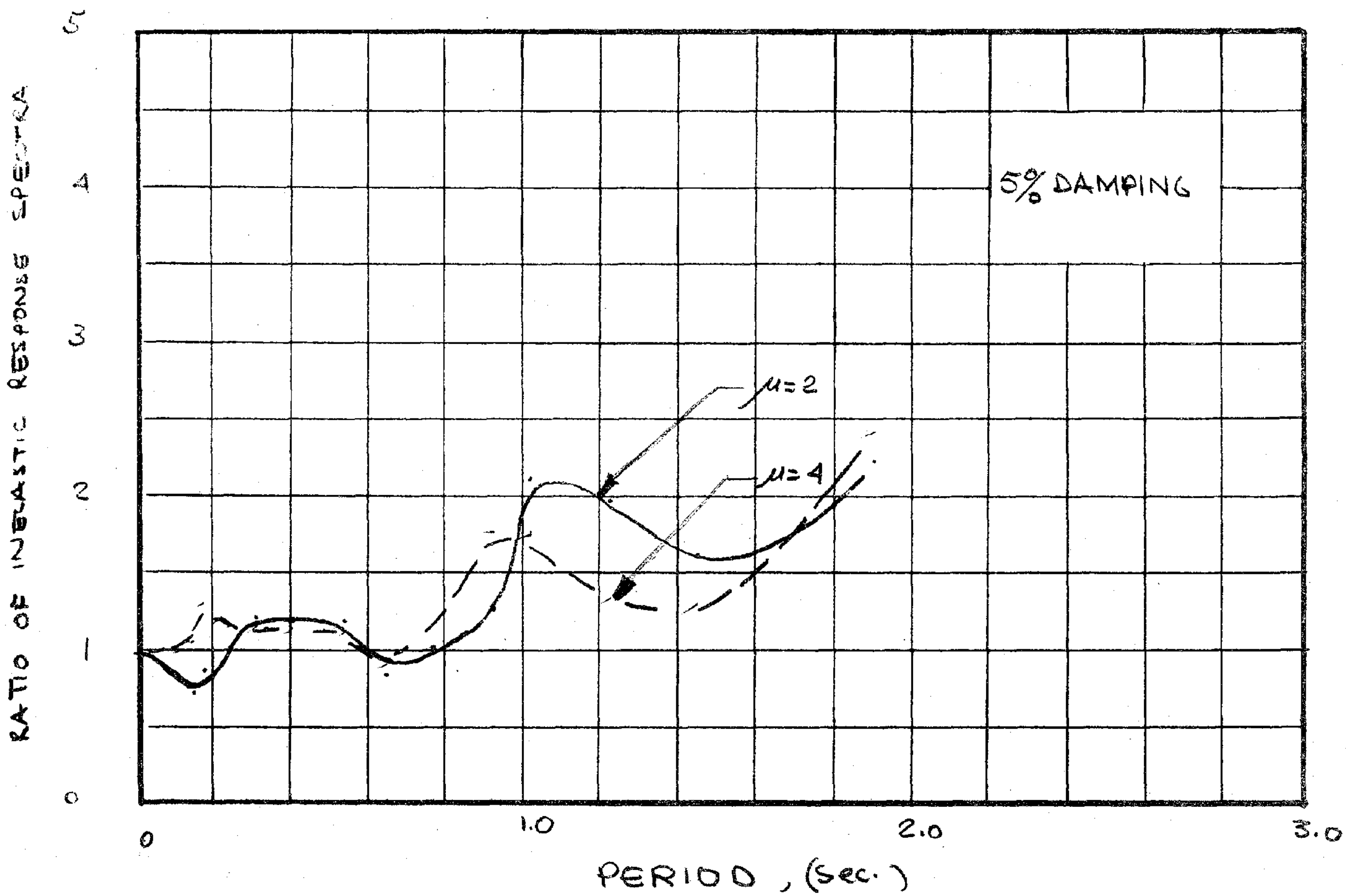


FIGURE 3.4 RATIO OF INELASTIC RESPONSE SPECTRA FOR A 160 FT THICK PROFILE OF BOSTON BLUE CLAY, (BBC-3).