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SOIL AMPLIFICATION STUDIES FOR TYPICAL SOIL PROFILES IN THE BOSTON AREA

Internal Study Report No. 15

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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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The first section of this report describes the criteria used in the selection of typical profiles in the Boston Basin for preliminary soil amplification analysis. Selections from literature studies of typical soil profiles were chosen to correlate the dynamic response of such profiles on the basis of their fundamental periods. Of particular importance are deep profiles which include clay. Specific questions are clarified and investigated through preliminary analysis. Points needing clarification are: how variation of thickness of the profile affects response; how variation of shear modulus for the same profile influences prediction; how the nature of the top layer affects results for the several types of profiles; and how considering or ignoring the flexibility of the shale and weathered rock layers underlying the clay and of the bedrock affect the results. The second section of this report summarizes the conclusions of these studies; presents the model selected for the production amplification studies; and describes the results for many typical profiles.

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<u>List of Internal Study Reports</u>

1. R.V. Whitman, "Preliminary Work Plans and Schedules," August, 1971.

2. E.H. Vanmarcke and R.V. Whitman, "Background for Preliminary Expected Future Loss Computations," October, 1971.

3. P.J. Trudeau, "Identification of Typical Soil Profiles in the Boston Basin Area," November, 1971.

4. J.M. Biggs, "Comparison of Wind and Seismic Forces on Tall Buildings," December, 1971.

5. R.V. Whitman, "Contribution to State-of-the-Art Report of the Earthquake Committee of the IABSE-ASCE Tall Buildings Committee--Economic and Social Aspects," March, 1972.

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7. C.A. Cornell and H.A. Merz, "Analysis of the Seismic Risk on Firm Ground for Sites in the Central Boston Metropolitan Area," January, 1972.

8. R.V. Whitman, J.W. Reed, P. Marshall, "1967 Caracas Venezuela Earthquakes," May, 1972.

9. R.V. Whitman, E.H. Vanmarcke, "Damage Statistics from Japanese Earthquakes, May, 1972.

10. E.H. Vanmarcke, J.W. Reed, and D. Roth, "Evaluation of Expected Losses and Total Present Cost: Preliminary Sensitivity Analysis," July, 1972.

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13. J.W. Reed, and R.V. Whitman, "San Fernando Earthquake Damage Statistics," August, 1972.

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Section 1

SELECTION OF TYPICAL PROFILES IN BOSTON BASIN FOR PRELIMINARY SOIL AMPLIFICATION ANALYSES

After the typical soil profiles in the Boston Basin had been identified (Internal Report #3), it was deemed appropriate that some further simplified profiles should be chosen for some preliminary amplification analyses, which would clarify important factors to be taken into account in the final analyses. In order to make the most intelligent selection, an effort was made to locate in the literature studies on similar profiles as well as previous analyses made at MIT for Boston Profiles. Thus analyses made by Seed (References 10, 11, 12) as well as here at MIT (References 1, 2, 4, 8, 14, 15) were examined, and an effort was made to correlate the dynamic response of such profiles on the basis of their fundamental periods. Some trends of similar behavior for similar periods were noticed (namely analogous variation of the ratio of response spectra at the top and the bottom depending on the predominant values of damping) as was expected, but the differences were too large depending on the variation of the elastic constant and damping with depth, the nonuniformity of the soil profiles and, more decisively, on the nature of the top part of the profile. The data was rather limited and led to little more than some qualitative trends rather than definite results.

Nevertheless what has been suggested by R.V. Whitman was further verified by this study: namely, that apart from damping the two main factors which affect soil amplification for a given profile are the depth and softness of the main (deep) part of the subsoil (as expressed by its fundamental period) and the nature (namely rigidity) of the top part of the profile.

This finding is further stressed by studies made by L. Ayestaran at MIT on idealized soil profiles of different depth and softness with a relatively shallow soft layer at their top.

It was, therefore, realized that from the 5 main typical identified profiles (See Internal Report #3), those of particular importance for the amplification studies would be mainly the deep profiles which include clay (i.e. types no. 3, 4 and 5 in Report #3). These profiles, although covering a rather small part of the total Boston Metropolitan area, are located at vital points including large parts of the center of Boston and the south-eastern part of Cambridge. Preliminary and approximate estimates have shown that the fundamental period of these subsoils would be in the range of 0.3-1.5 secs and that they would present amplification functions with rather high values. Thus the dynamic characteristics of these profiles, which suggest significant amplifying effects for motions important to structures for which a rational design would be required (i.e. modern high-rise relatively flexible structures) and their existence in areas where exactly such structures can be expected dictate that the amplification effects of these subsoils be investigated most closely.

On the other hand it was realized that from the shallow profiles (profile types 1 and 2) only the ones containing organic material and peat would have significant amplifying effects. For shallow profiles of up to 40 ft depth consisting only of relatively dense outwash sands and gravels, their stiffness and depth suggest that rather insignificant amplification effects should be expected. To verify this thesis some approximate calculations were made which gave an estimated fundamental period of 0.05-0.20 secs and an amplification value $A(\omega)$ not greater than 2.5. (For these estimates elastic and damping constants were predicted using the methods suggested by Hardin and the calculations were made by the method suggested by Madera (Ref. No. 8) for a layer with uniform properties--the value of shear modulus taken for depth $D = 0.63H$ where H = the thickness of the layer as Dobry (Ref. 3) suggests.)

For shallow profiles (types 1 and 2) containing organic silts or peats with significant thickness (more than 15 ft) no similar prediction would be made. The softness of these layers suggests a probable effect of some significance. The available information on the properties of these soils is insufficient and the methods to predict the necessary variables (namely shear modulus and damping) for such soils are not trustworthy. Therefore reliable predictions cannot be made for these soils at this time.

For all these reasons, after discussion with the other members of the Soils Division working on the project, it was decided that amplification analyses should be started and be concentrated with the deep profiles. The aim of these first computations would be to clarify several questions. Points needing clarification from the beginning in order to establish an intelligent program of investigation are:

(1) How variation of thickness of the profile (and the corresponding variation of elastic constants) affects the response.

(2) How variation of shear modulus for the same profile (i.e. uncertainty with respect to G) influences prediction.

(3) How nature of top layer affects the results for the several types of profiles.

(4) How consideration or ignoring of the flexibility of the shale and weathered rock layers underlying the clay and of the bedrock affect the results.

In order to facilitate the answer to these questions it was decided that initial runs should be made on the clay profiles without overlying sand or peat. This procedure was deemed rational since these upper layers were deposited on the underlying clay (nevertheless an examination of sections of these subsoils revealed a rather erratic interface between the clay layers and the overlying sands and organic soils). (Ref. 6).

Three different profiles of 50, 100 and 160 ft. of clay were chosen to be analyzed, in order to cover the range of depths (see Fig.1.1). The variation of shear modulus (shear wave velocity) for small strains with depth was estimated by the method suggested by Hardin and checked back by Seed's data. The necessary soil properties were taken from those for the MIT campus subsoils for which extensive information is available (Ref. 7). Average soil properties were taken and a range of variation was estimated depending on the amount of overburden. This variation is presented in Fig.1.2(level A to B for Case 1; A to C for Case 2; A to D for case 3).

The clay layers in this area are underlain by a dense glacial till which varies through the area from as little as 5 ft up to 50 ft or more. Private communication with local geologists (Professor Ronald Hirschfeld of MIT and Mr. Vincent Murphey of Weston Geophysical Research) revealed that a best estimate of the shear wave velocity for this layer and a shallow upper zone of weathered argillite would lie in the range between 1500 and 2000 ft/sec. They also suggested that a good estimate of the shear wave velocity of the bedrock (Cambridge argillite) would be 6000 ft/sec. It was decided, therefore, to make analyses of the profiles underlain by a 50 ft layer of till-weathered rock (with C_S = 1500 ft/sec) which in turn was underlain by either rigid or flexible bedrock (6000 ft/sec) and of the same profiles without this layer, directly underlain by either rigid or flexible bedrock.

For the computations the program DYALS, which permits adjustment of modulus and damping with respect to strain, would be used (with some checks by DYFALS which is more accurate). As strain adjustment curves the average of the variations presented in the last report prepared by Shannon and Wilson and Agbabian-Jacobien Associates (Ref. 16) would be used. As input for the analysis the artificial earthquake Aguirre #3 was chosen as having equally significant magnitudes in a wide range of frequencies. It was normalized to approximately 0.05g (a figure judged appropriate as an average value for Boston).

BASIC PROFILES FOR PRELIMINARY ANALYSES

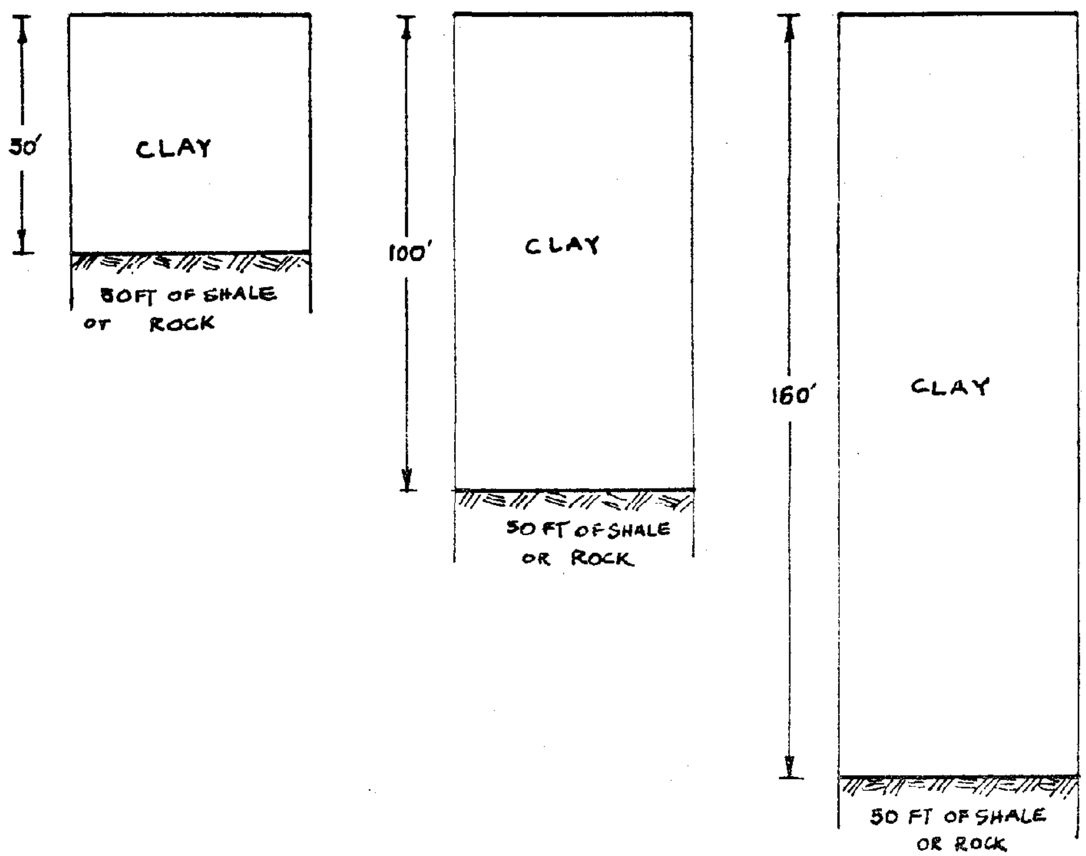

FIGURE 1.1

SHEAR WAVE VELOCITY FOR PROFILES

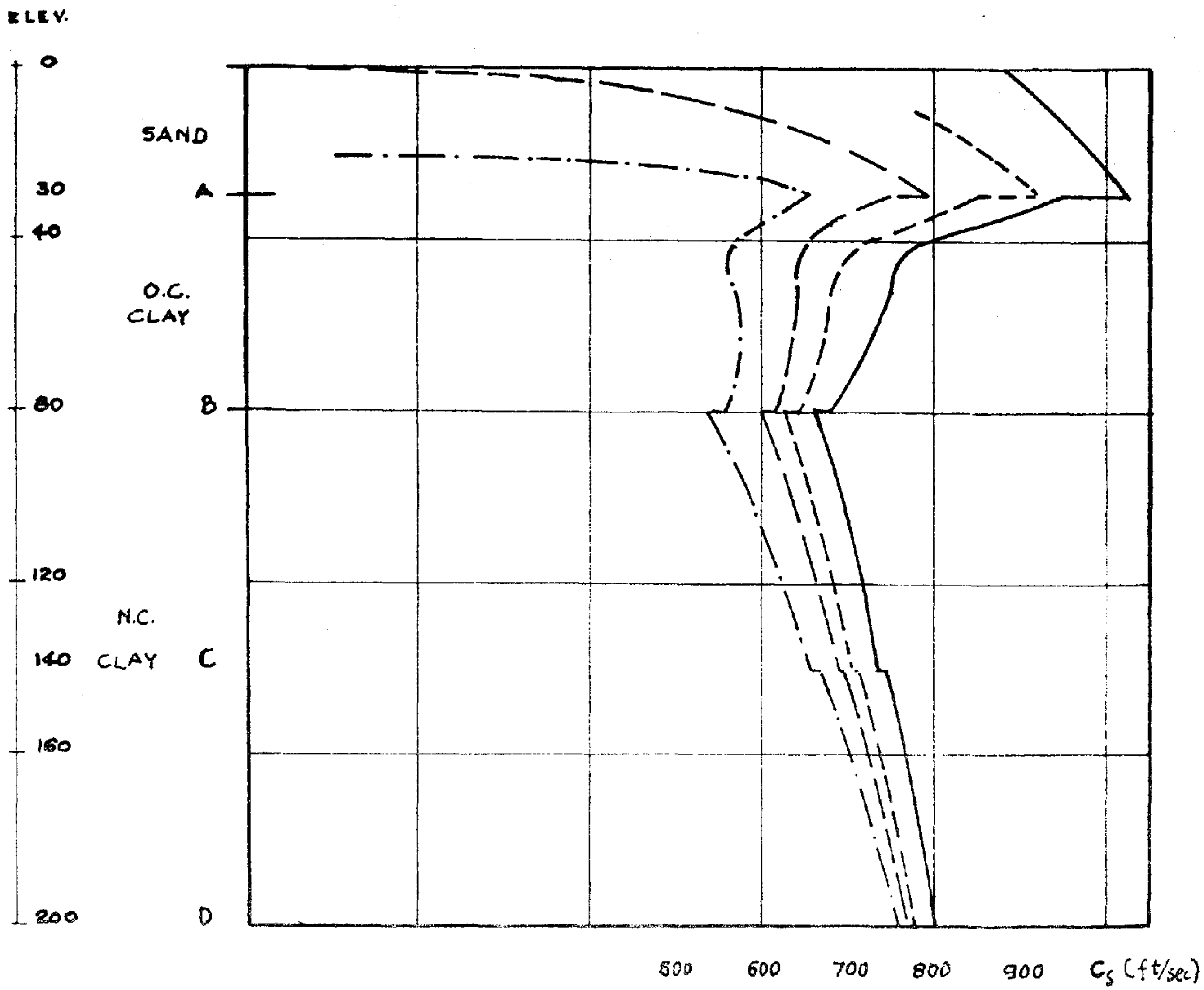

FIGURE 1.2

Section 2

AMPLIFICATION ANALYSES

1. INTRODUCTION

In the previous section the background information necessary for the amplification studies was described. Several specific questions and unclear points, which needed answers by means of some preliminary analyses, were also cited. Section 2 summarizes the conclusions of these studies, presents the model selected for the production amplification studies, and describes the results for many typical profiles (namely those not including organic material in their upper parts). A short discussion of these results is presented at the end of the report.

2. PRELIMINARY STUDIES

The main problems that had to be clarified by these studies were the effects on the ground response of such factors as the rigidity of the base rock, the existence of an intermediate layer of the nature of hard till or weathered argillite between the base rock and the soft soil, and the use in the analysis of the discrete theoretical model (DYALS) or the continuous model (DFALS) (see section 1).

To facilitate the answers to these problems a single profile of 100 ft clay was chosen for these studies. The upper 50 ft of the profile consists of O.C. Clay and the lower 50 ft of N.C. Clay. The variation of the zero-strain C_S is presented in Fig.2.1(C_S was estimated by Hardin's formula for a typical overburden; it is therefore an average of the band of variation of C_S with depth as presented in Fig.1.2 between the corresponding elevations). The choice of this simplified profile without any overlying layers of sand or organic soil was based on reasons discussed in section 1 (i.e. simplicity, geology of deposition etc); the depth of 100 ft was deemed appropriate as being an average between the extreme cases of very shallow clay profiles and very deep ones and very representative of Cambridge and Central Boston sites (Case IV in report #3).

To investigate the influence of the factors described above this basic soil profile was analyzed for the following conditions:

1. Basic profile underlain by rigid rock ($C_S = \infty$)
2. Basic profile underlain by hard rock of finite rigidity (C_S=6000 ft/sec)

3. The same profile underlain by a 50 ft layer of hard till or weathered rock (with C_S = 1500 ft/sec), which is in turn underlain by rigid rock.
4. Basic profile of condition 3 underlain by flexible rock (C_S = 6000 ft/sec).

For the cases involving the intermediate layer (C_S = 1500 ft/sec) the thickness of 50 ft was chosen to investigate its influence as an extreme since this layer is typically thinner. For each of the above 4 cases analyses were made using both the discrete model (computer program DYALS) and the continuous (computer program DYFALS).

As input motion the artificial earthquake Aguirre #3 was used normalized to 1/5 of its maximum acceleration i.e. approximately 0.053g. This value was deemed appropriate for Boston as an average for a return period of 100 years (see Internal Study Report #2); furthermore artificial Aguirre #3 was picked because it includes a wide range of important frequencies.

A series of amplification analyses were made for each case with values of shear wave velocity and damping coefficients adjusted each time according to estimated strain levels at several elevations of the profile by means of the previous analysis. For this adjustment the curves giving the variation of shear wave velocity and damping vs strain suggested in reference 16 were used. Tables 1- 6 show the shear wave velocities and damping values used in the analysis after the final strain adjustment for each case.

The ground motion response for all of the above cases is presented in Figs. 2.2 - 2.5, by means of pseudoacceleration response spectra at the top of the profile.

Figures 2.2 and 2.4 suggest that both the discrete model (DYALS) and the continuous (DYFALS) give results in very close agreement in the cases of rigid rock, whereas figures 2.3 and 2.5 show that DYFALS results in considerably higher response than DYALS in the case of flexible rock.

It must be mentioned here that the most accurate model is DYFALS which uses the Fourier transform to solve directly the differential equations involved. It is thought that the main reason for this discrepancy is the way that the discrete model (DYALS) handles the non-flexibility of the base rock by simply introducing an additional damping coefficient depending on the impedence ratio ($\frac{\gamma r \cdot Cr}{\gamma_S \cdot C_S}$). Notice that the difference is most significant in the case of flexible rock overlain by the intermediate shale layer (Case 4 above).

The effect of the rock rigidity can be best evaluated by comparing the spectra for cases 1 and 2 (profiles without the shale layer) (Fig.2.6) or for cases 3 and 4 (profiles including shale layer) (Fig.2.7). There is a maximum difference of approximately 10% between the response spectra of the two profiles without the shale if we compare the ones computed by DYFALS (see Fig. 2.7). (The difference is somewhat higher if the DYALS spectra are compared; yet the approximate handling of rock flexibility by DYALS must be borne in mind again.) For the profiles with the shale layer (Cases 3 and 4) the comparison appearing in Fig. 2.7 gives very close agreement between the spectra for the rigid and flexible rock cases as computed by DYFALS but significant differences for the spectra as computed by DYALS. The differences are generally higher than those suggested by Seed et al (Ref. 11), who studied the effect of base rock characteristics on amplification for several mainly sandy profiles. Yet there is always the possible explanation that the damping strain curves used by Seed for sandy materials are much steeper in the region of interest than the curves used in the present studies and therefore the higher strains for the rigid rock case would introduce higher damping values resulting in smaller response.

Figs. 2.8 & 2.9 help us to assess the effect of the intermediate shale layer on the response. It can be seen that there are rather significant differences depending on the existence or absence of this layer and particularly in the lower frequency region. It was thought that some of the difference might be caused by the low value of damping used for the "shale" layer (2%) the properties of which are so uncertain. Nevertheless the use of higher damping (6%) that would apply for a soil-like material as it apparently is, reduced the difference only slightly.

3. SELECTION OF MODEL FOR ANALYSIS

The above preliminary studies showed that the differences in the several cases examined were not small enough to allow the use of one of the cases (probably the simpler in terms of subsoil information necessary and computations involved) indiscriminately for all conditions. On the other hand it was realized that the examination of all these cases for each of the profiles identified in Report #3 would complicate the overall problem immensely. Furthermore the purposes of the general Boston Quake project and the uncertainties involved in the amplification studies (uncertainties with respect to soil and rock properties, necessity of examination only of a number of "typical profiles," computational uncertainties etc) would contradict such an examination for each profile or an effort to introduce each time the exact geometric characteristics and the properties of the hard intermediate layer and of the rock. It was therefore decided that the analyses should proceed with the discrete model (DYALS), with rigid rock base and without considering the intermediate till or weathered argillite sublayer. This decision was based largely on the fact that DYALS tended to underestimate response whereas consideration of rigid rock tended to overestimate it; the combination of these effects would therefore probably lead to better agreement of the more realistic conditions of flexible rock and analysis through continuous model (compare Figs. 2.2 - 2.5).

4. RESPONSE OF CLAY AND SAND-CLAY PROFILES

After the above preliminary (background) studies and decisions, the production analyses for the different profiles followed. In addition to the 100 ft clay profile two more clay profiles were analyzed; one with 50 ft of O.C. Clay (representing the typical thickness of O.C. Clay in the area) and one of 160 ft of clay representing the approximate upper bound of the thickness of clay layers to be encountered in the area. Shear wave velocities were taken again as average values from the graph presented in section 1 (Fig. 2.2), and the analyses were made for the conditions chosen above (DYALS, rigid rock, no intermediate layer). The ground response for all three clay profiles is presented in Figs. 2.10, 2.11, 2.12 in terms of pseudo acceleration response spectra at the top and ratios of response spectra for the top and base. Naturally the major peaks in both diagrams correspond to the natural periods of the profiles. It can be seen that there is a definite trend of decrease of the peak S_a with increasing depth (fundamental period). In addition it is noticeable that the peaks of ratio of response spectra do not vary much out of the range between 4 and 5. Some more discussion on these values is presented under the next heading.

Following the amplification studies of the simple clay profiles analyses were made for sand-clay profiles. Three profiles consisting each of a top 50 ft sand layer underlain by clay strata of thickness 50, 100 and 160 ft corresponding exactly to the clay profiles, which were previously considered, were analyzed. The shear wave velocity for the sand layer was estimated by Hardin's formula as described in section 1, and for the adjustment of C_S and damping the curves presented in this same report were used. The resulting ground response expressed in terms of pseudoacceleration response spectra and ratios of response spectra for the top and the base, appear in Figs. 2.13-2.16. An additional profile of 20 ft sand underlain by 30 ft of O.C. Clay was analyzed and the corresponding response is presented in Fig. 2.10).

A careful examination of the response of these profiles reveals how close it is to the response of clay layers of the same depth (Figs 2.10, 2.12, and 2.16).

Generally it can be noticed that the trend of variation of the response with the thickness of the profile is very consistent, independently of whether the profiles are simply clay or consist of sand and clay layers. This result was rather to be expected since the stiffness of the sand layer is very similar to the stiffness of the upper part of the O.C. Clay.

5. CONCLUSIONS AND EVALUATION OF FIRST RESULTS

One important result of these first studies is the very close behavior of simple clay profiles to sand-clay profiles of the same thickness. This conclusion will allow future analyses to treat these two different profiles as one case, and it will reduce therefore significantly the amount of work and complication due to variation of subsoils in Boston (see Internal Study Report 3).

Another result which may help considerably the methodology and sequence of future work is the fact that for all the examined profiles the value of maximum ratio of response spectra is very similar (4.0-5.0) suggesting that these profiles could probably be treated only in terms of relation of their depth to their fundamental period.

It must be mentioned here that this value of 4-5 of the peak ratio of response spectra is rather higher than expected. Possible reasons for this might be the fact that the base rock was considered rigid, the accelerations involved were rather small, and less significantly the nature of the artificial earthquake used. Nevertheless, an examination of several amplification studies revealed that such values were not uncommon for similar conditions (references 2, 4, 8, 14, 15). On the other hand in many cases studied at the University of California at Berkeley, this ratio was rather in the vicinity of 2.9-3.0. A possible explanation for their low values is that in most of these cases the profiles were sandy, involving therefore higher damping values. Thus in most of the cases in reference 11 where the profiles are predominantly clayey the peak ratio varies from 2.5 to 3.0 but in one mainly clayey profile this ratio is as high as approximately 4.5. Furthermore, it seems that the curves of variation of damping value with strain used in these studies done at Berkeley, are generally higher (and steeper) than those used in our analysis (see section 1).

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SHEAR WAVE VOLOCITY FOR CLAY PROFILES

(FOR ZERO STRAIN)

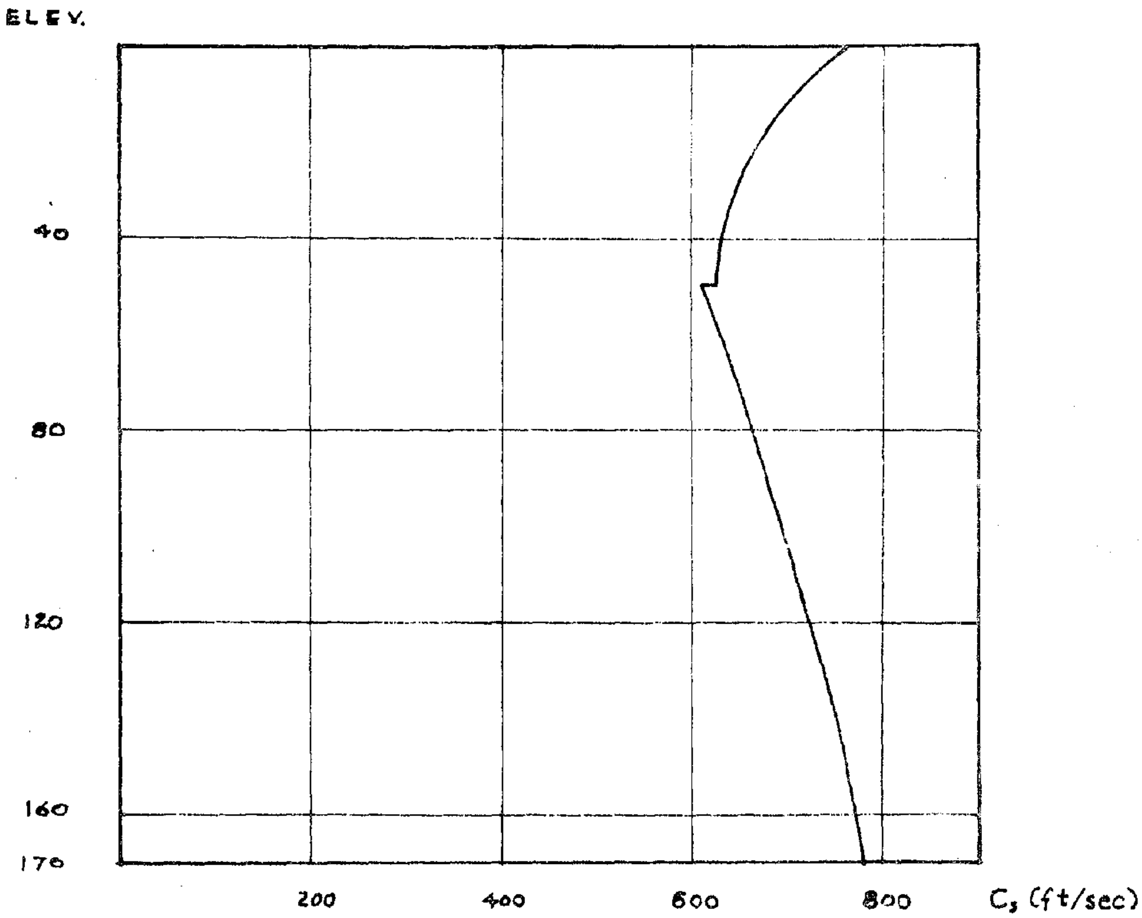


FIGURE 2.1

SHEAR WAVE VELOCITIES AND DAMPING COEF. FOR FINAL ANALYSES

(After Strain Adjustment)

PROFILE: 100 FT OF CLAY + FLEXIBLE ROCK (Case 2)

DYFALS I

SOIL	ELEV.	C_s (ft/sec)	D (%)
	0		
	10	650	5.5
O.C. CLAY	20	580	6.0
	30	530	7.0
	40	510	7.0
	50	500	8.0
	60	500	8.5
N.C. CLAY	70	490	8.5
	80	500	8.5
	90	530	8.0
	100	620	7.5
ROCK		6000	

DYALS

SOIL	ELEV	C_s (ft/sec)	D %
	0		
	10	650	5.5
O.C. CLAY	20	585	5.9
	30	535	6.9
	40	515	6.9
	50	505	7.9
	60	500	7.9
N.C. CLAY	70	440	8.4
	80	505	8.3
	90	530	8.0
	100	620	7.5
ROCK		6000	

DYFALS II

SOIL	ELEV.	C_s (ft/sec)	D %
	0		
	10	620	7.0
O.C. CLAY	20	600	7.0
	30	550	9.7
	40	520	9.7
	50	490	10.3
	60	480	10.3
N.C. CLAY	70	500	10.2
	80	520	10.2
	90	540	10.0
	100	610	10.0
ROCK		1500	

TABLE 2.1

SHEAR WAVE VELOCITIES AND DAMPING COEFS. FOR FINAL ANALYSES

(After Strain Adjustment)

PROFILE: 100 FT OF CLAY + RIGID ROCK

SOIL	ELEV.	DYALS		DYFAS	
		C_s (ft/sec)	D %	C_s (ft/sec)	D %
O.C. CLAY	0	650	5.5	650	5 5.5
	10	580	6.0	580	6.0
	20	530	7.0	530	7.0
	30	510	7.0	510	7.0
	40	500	8.0	500	8.0
N. C. CLAY	50	500	8.5	500	8.5
	60	490	8.5	490	8.5
	70	500	8.5	500	8.5
	80	530	8.0	530	8.0
	90	620	7.5	620	7.5
	100				
ROCK		∞		∞	

TABLE 2.2

SHEAR WAVE VELOCITIES ANA DAMPING COEFS. FOR FINAL ANALYSES

(After Strain Adjustment)

PROFILE: 100FT OF CLAY + 50FT OF SHALE AND RIGID ROCK (Case 3)

SOIL	ELEVATION	DYFALS		DYALS	
		C_s (ft/sec)	D %	C_s (ft/sec)	D %
O.C. CLAY	0				
	10	620	7.0	620	7.0
	20	600	7.0	600	7.0
	30	550	10.0	545	10.2
	40	520	10.0	520	10.2
N.C. CLAY	50	490	10.6	490	10.2
	60	480	10.5	485	10.5
	70	500	10.5	505	10.3
	80	520	10.5	520	10.3
	90	540	10.2	540	10.2
	100	610	9.7	610	9.8
SHALE		1500	2.0	1500	2.0
	150				
ROCK		6000			

TABLE 2.3

SHEAR WAVE VELOCITIES AND DAMPING COEF. FOR FINAL ANALYSES

PROFILE: 100 FT OF CLAY + 50 FT OF SHALE — FLEXIBLE ROCK (Case 4)

SOIL	ELEV.	DYFALS		DYALS	
		C_S (ft/sec)	D %	C_S (ft/sec)	D %
O.C. CLAY	0				
	10	650	5.0	650	5.0
	20	605	5.0	645	5.0
	30	680	6.0	605	6.0
	40	650	6.0	600	6.0
	50	580	7.0	580	7.0
N.C. CLAY	60	550	8.0	560	7.8
	70	560	8.0	570	8.0
	80	580	8.0	590	8.0
	90	660	7.0	665	7.0
	100	670	7.0	675	7.0
SHALE		1500	2.0	1500	2.0
	150				
ROCK		6000		6000	

TABLE 2.4

SHEAR WAVE ANALYSES AND DAMPING COEF. FOR FINAL ANALYSES

50 FT OF CLAY – RIGID ROCK

SOIL	ELEVATION	C_s (ft/sec)	D_i %
O.C. CLAY	0		
		740	6.5
	10		
		640	7.5
	20		
		620	9.0
	30		
		580	9.0
	40		
		580	9.0
	50		
ROCK		∞	

160 FT OF CLAY – RIGID ROCK

SOIL	ELEVATION	C_s (ft/sec)	D_i %
O.C. CLAY	0		
		710	7.0
	10		
		630	7.0
	20		
		600	9.0
	40		
		540	10.5
N. C. CLAY	70		
		560	10.5
	100		
		620	10.0
	130		
		680.	9.8
	160		
ROCK	∞		

TABLE 2.5

PROFILES WITH RIGID ROCK (WITH OR WITHOUT SHALE LAYER)

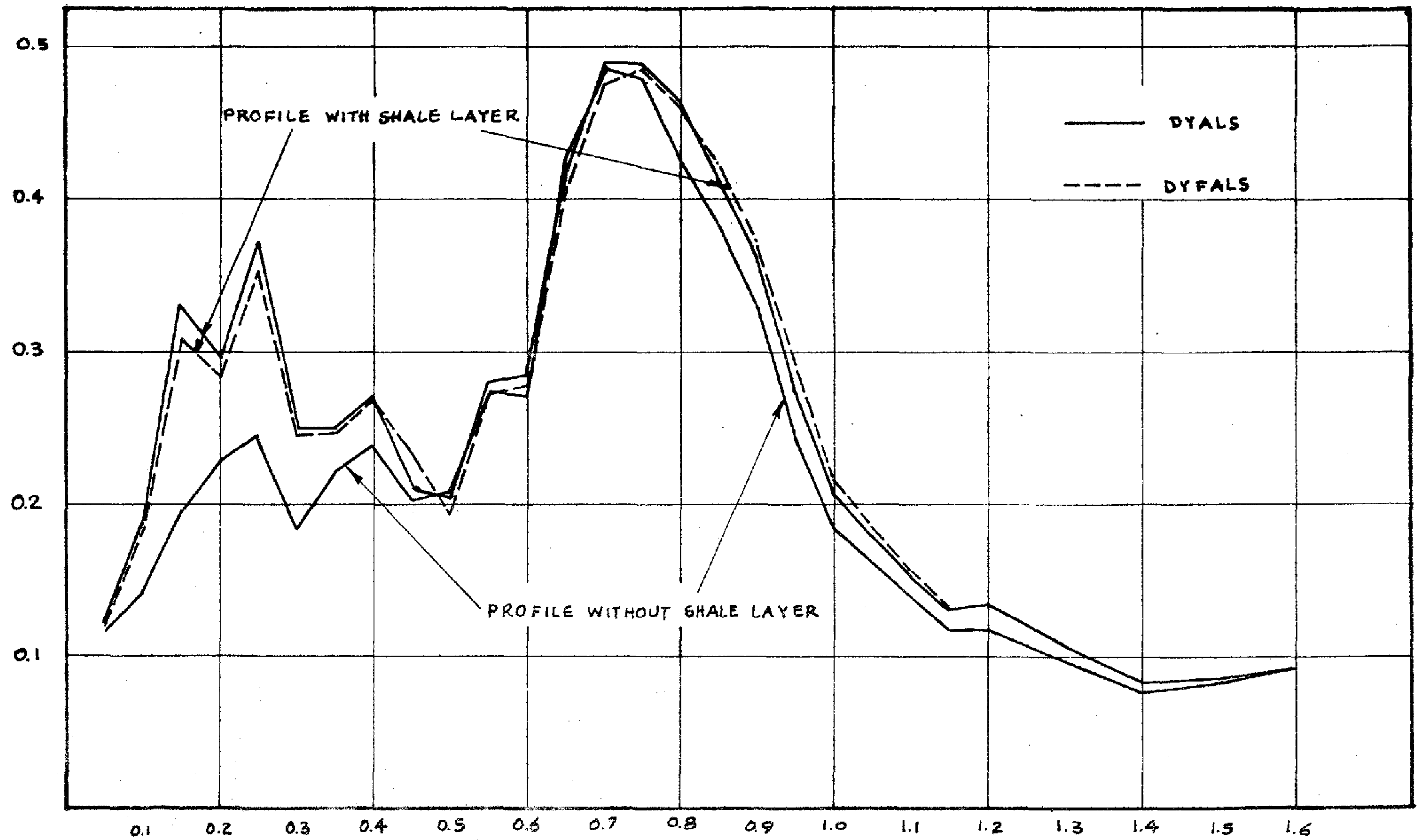
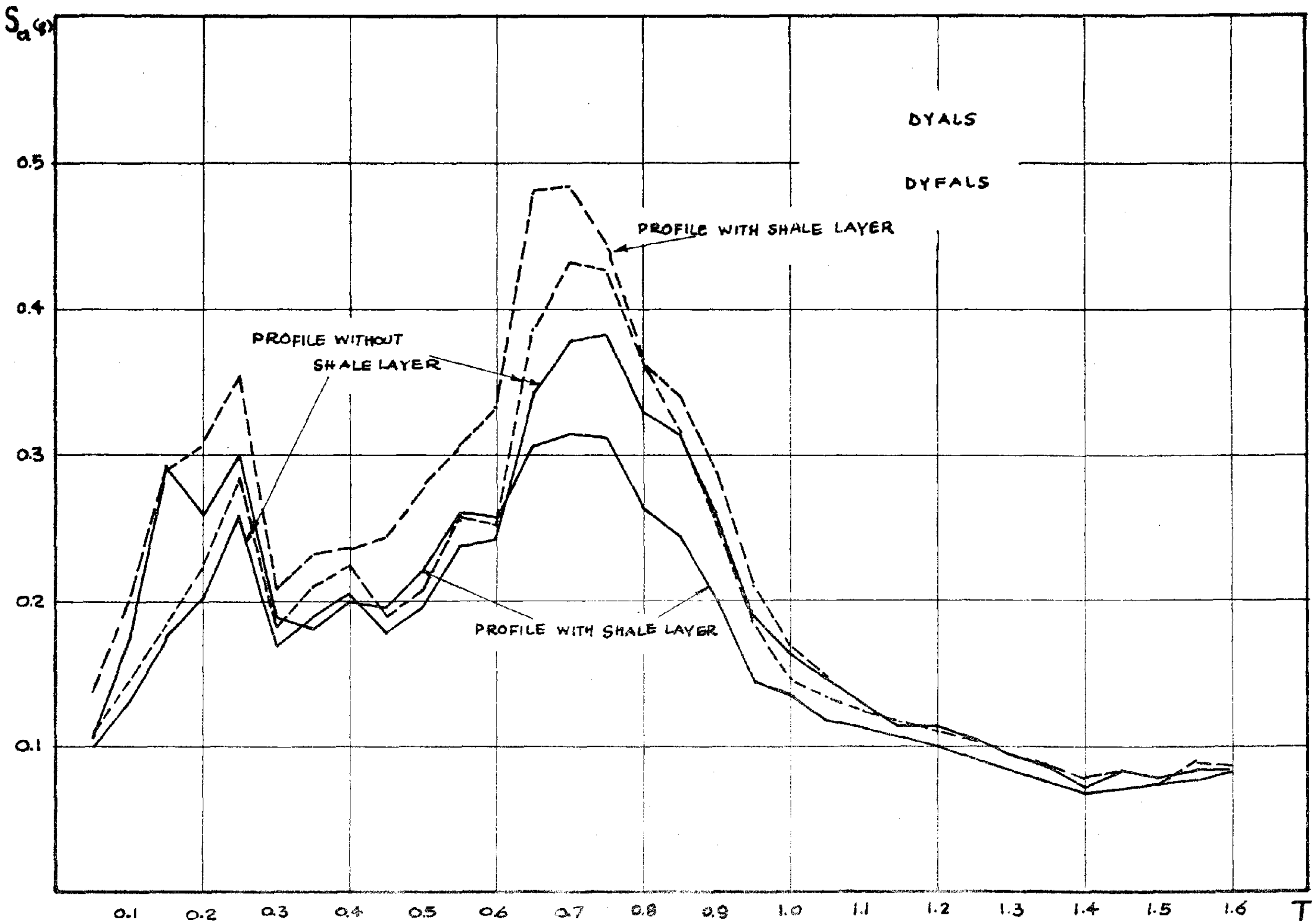


FIGURE 28

PROFILES WITH FLEXIBLE ROCK (WITH OR WITHOUT SHALE LAYER)



F I G U R E 2.9

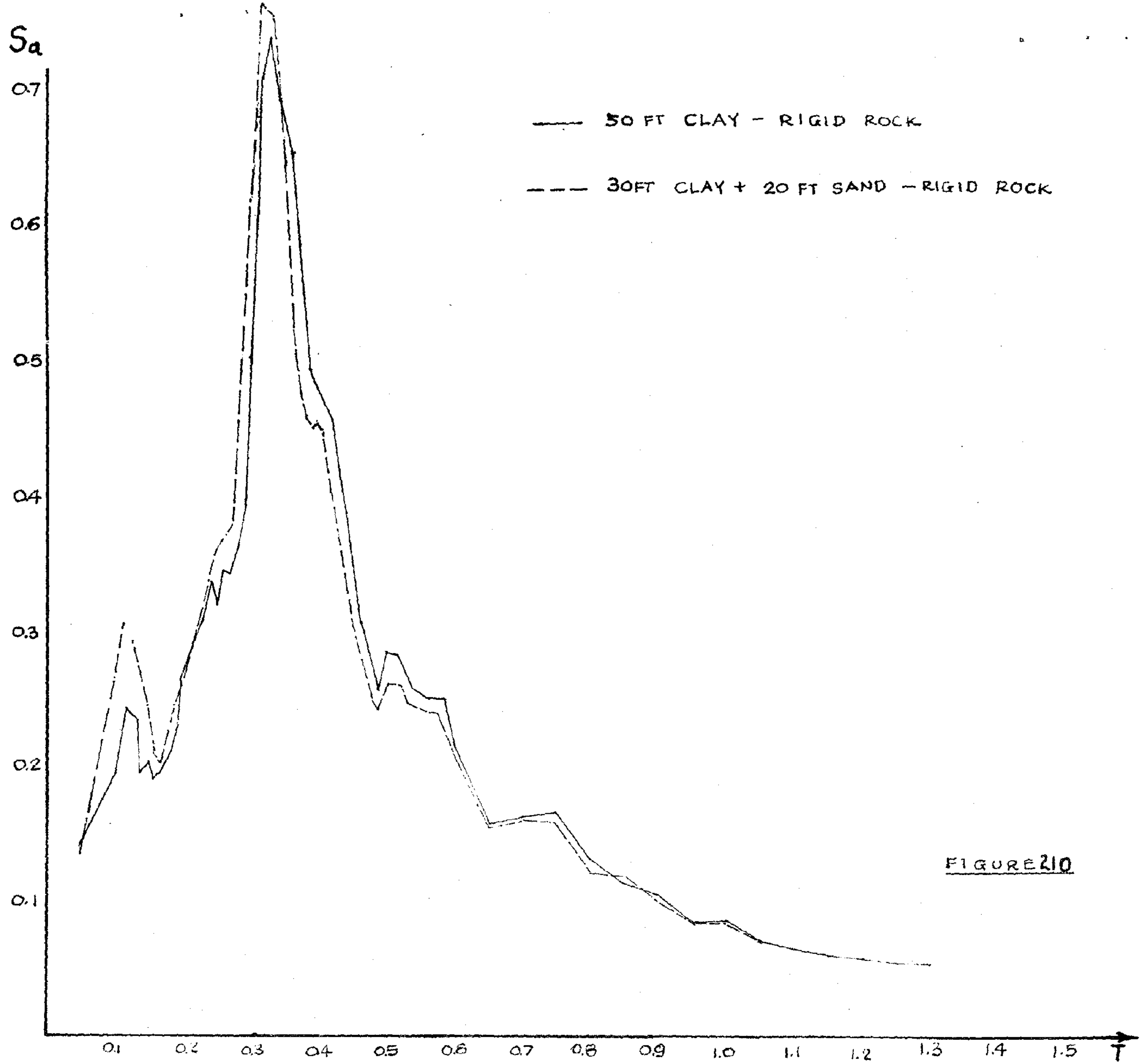


FIGURE 2.10

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160 FT OF CLAY — RIGID ROCK

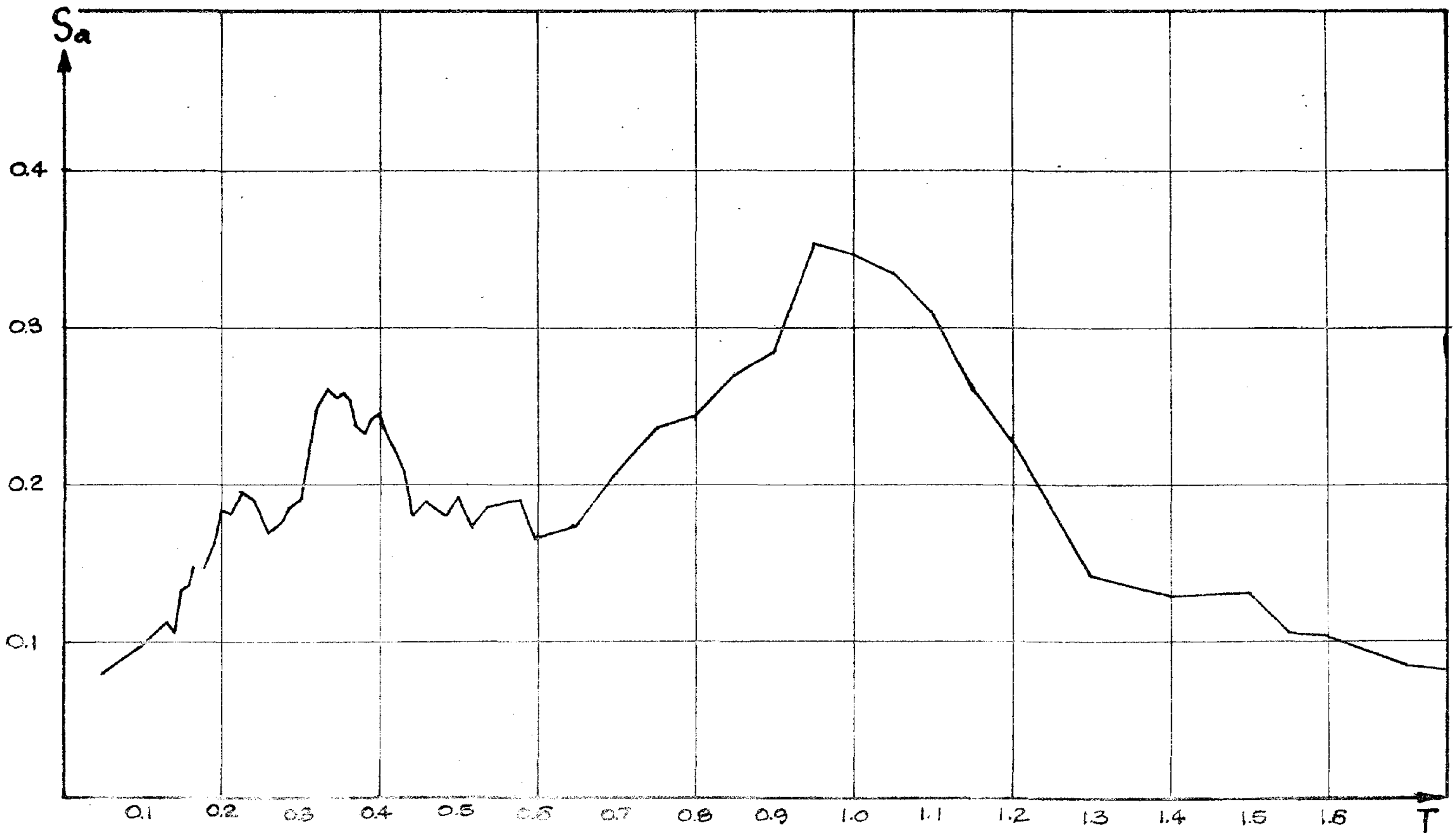

FIGURE 21

RATIOS OF RESPONSE SPECTRA

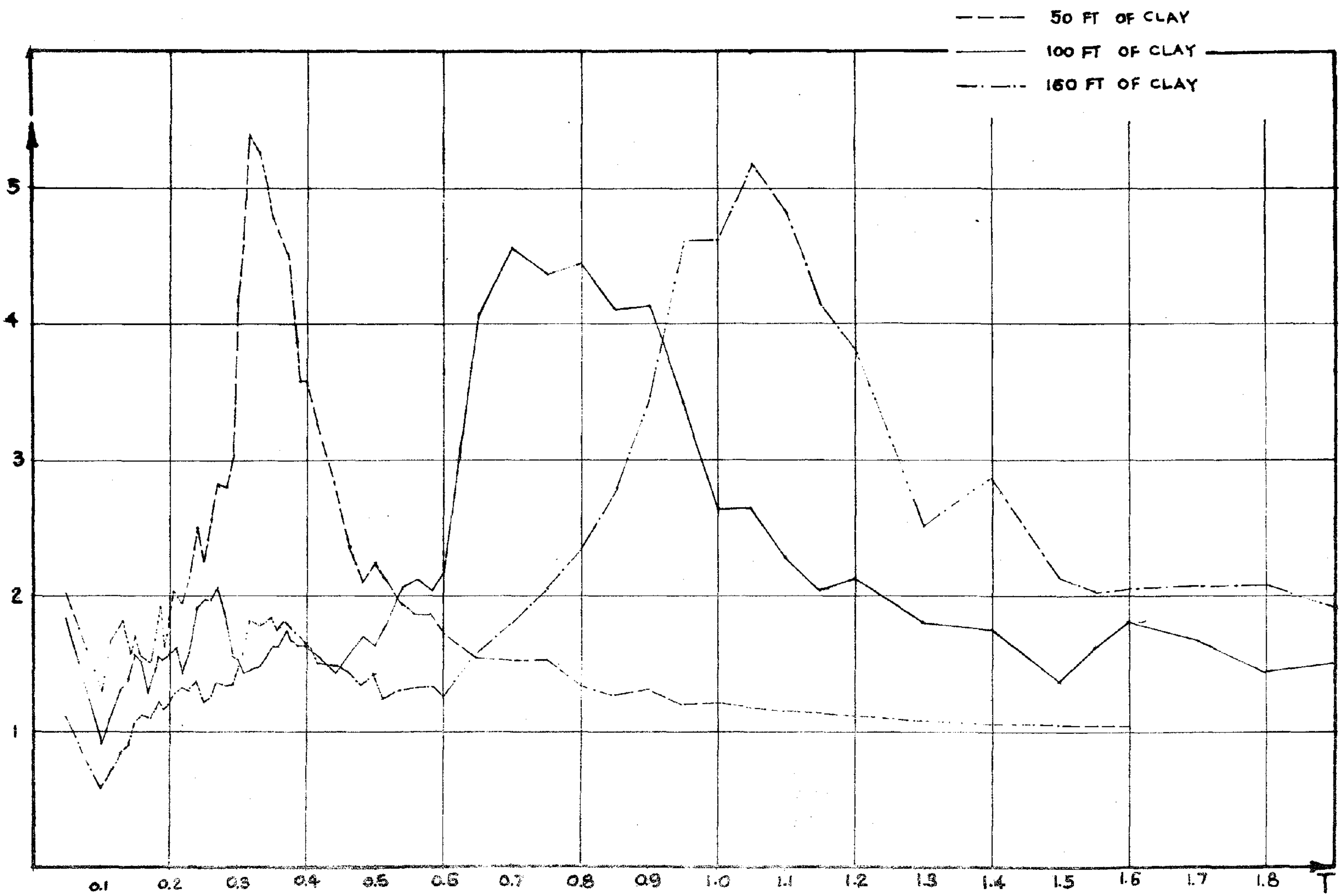

FIGURE 2.12

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50 FT CLAY + 50 FT SAND - RIGID ROCK

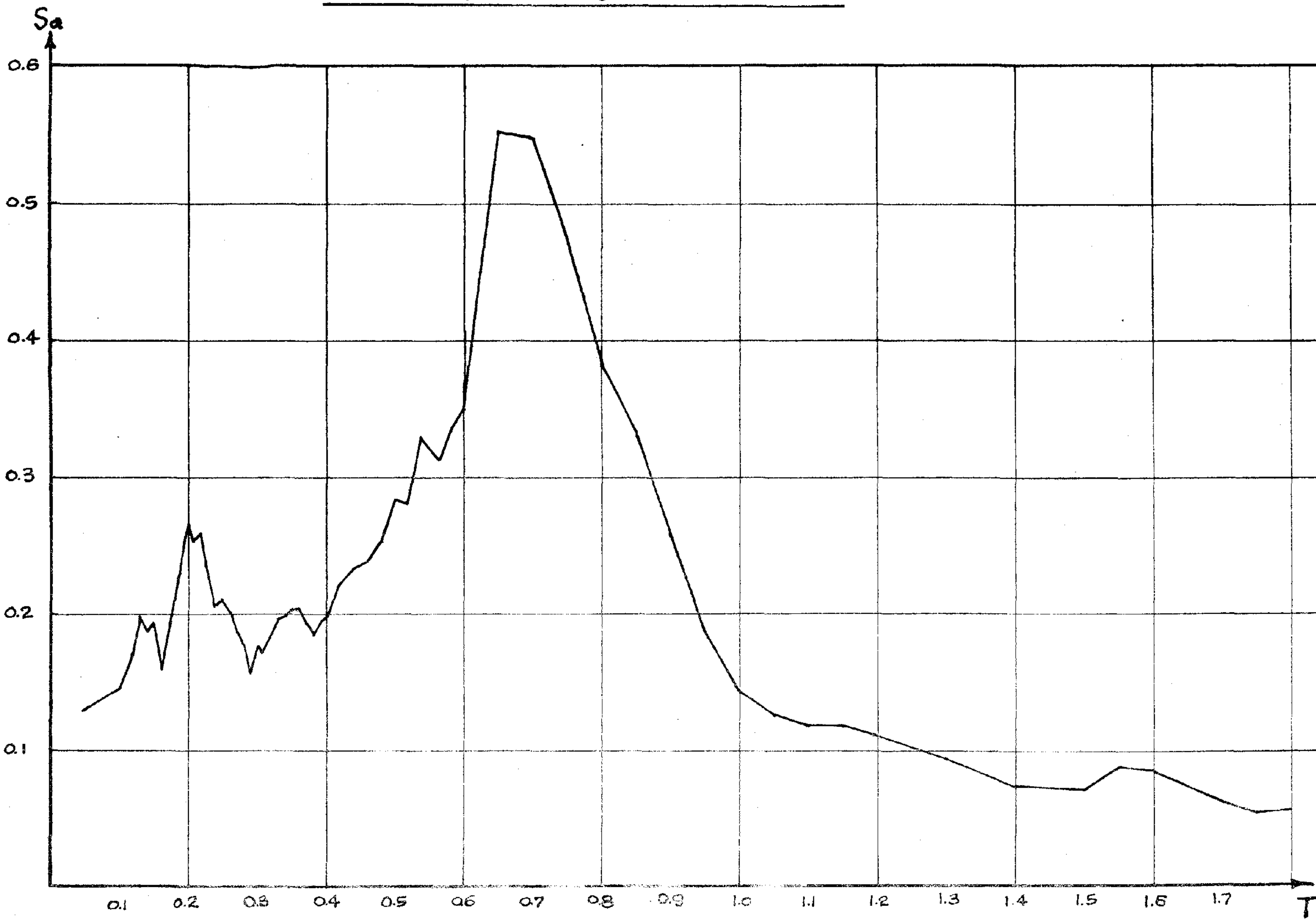

FIGURE 2.13

100 FT OF CLAY + 50 FT OF SAND

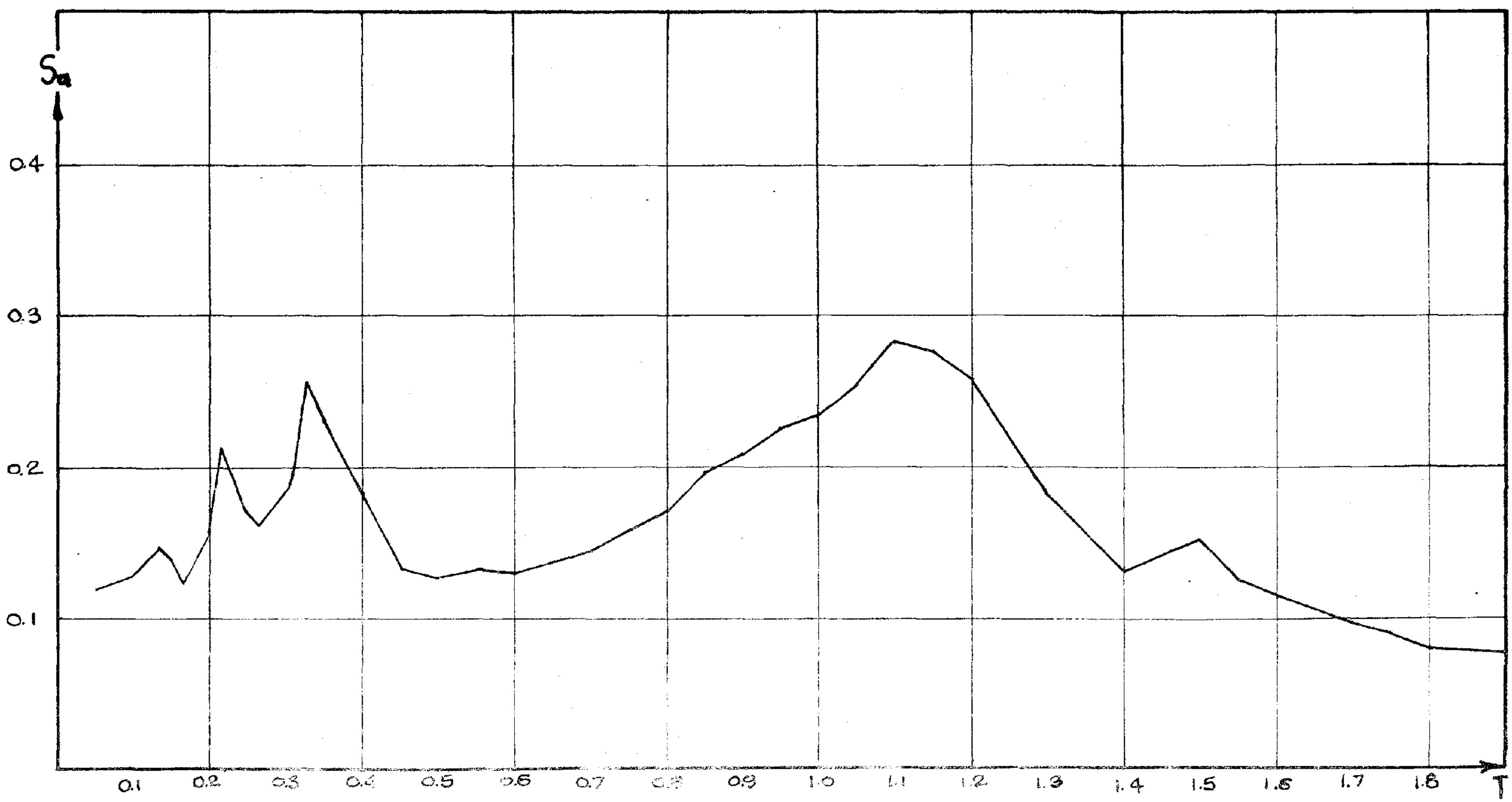

FIGURE 2.14

180 FT OF CLAY + 50 FT OF SAND

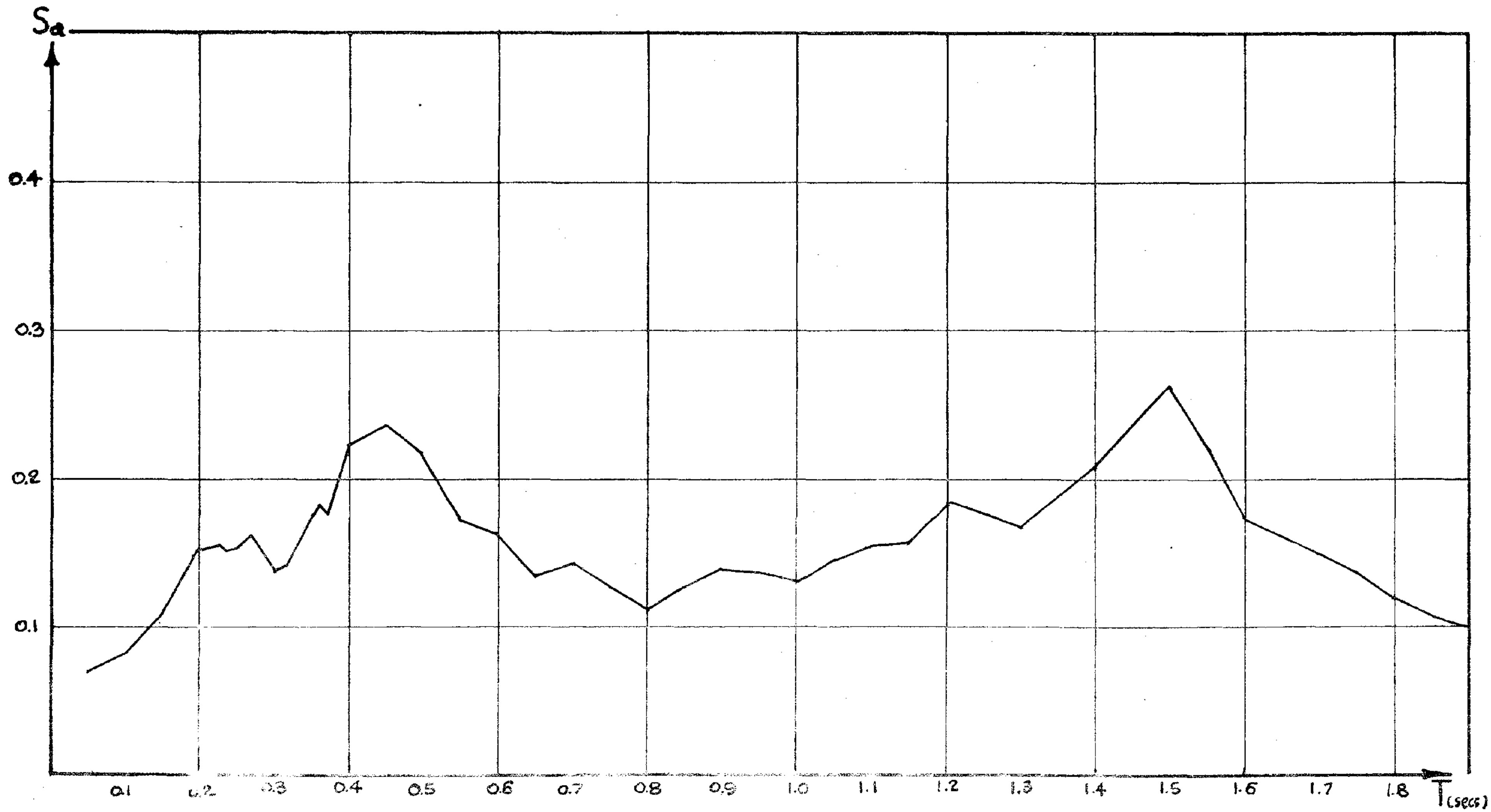

FIGURE 2.15

RATIOS OF RESPONSE SPECTRA

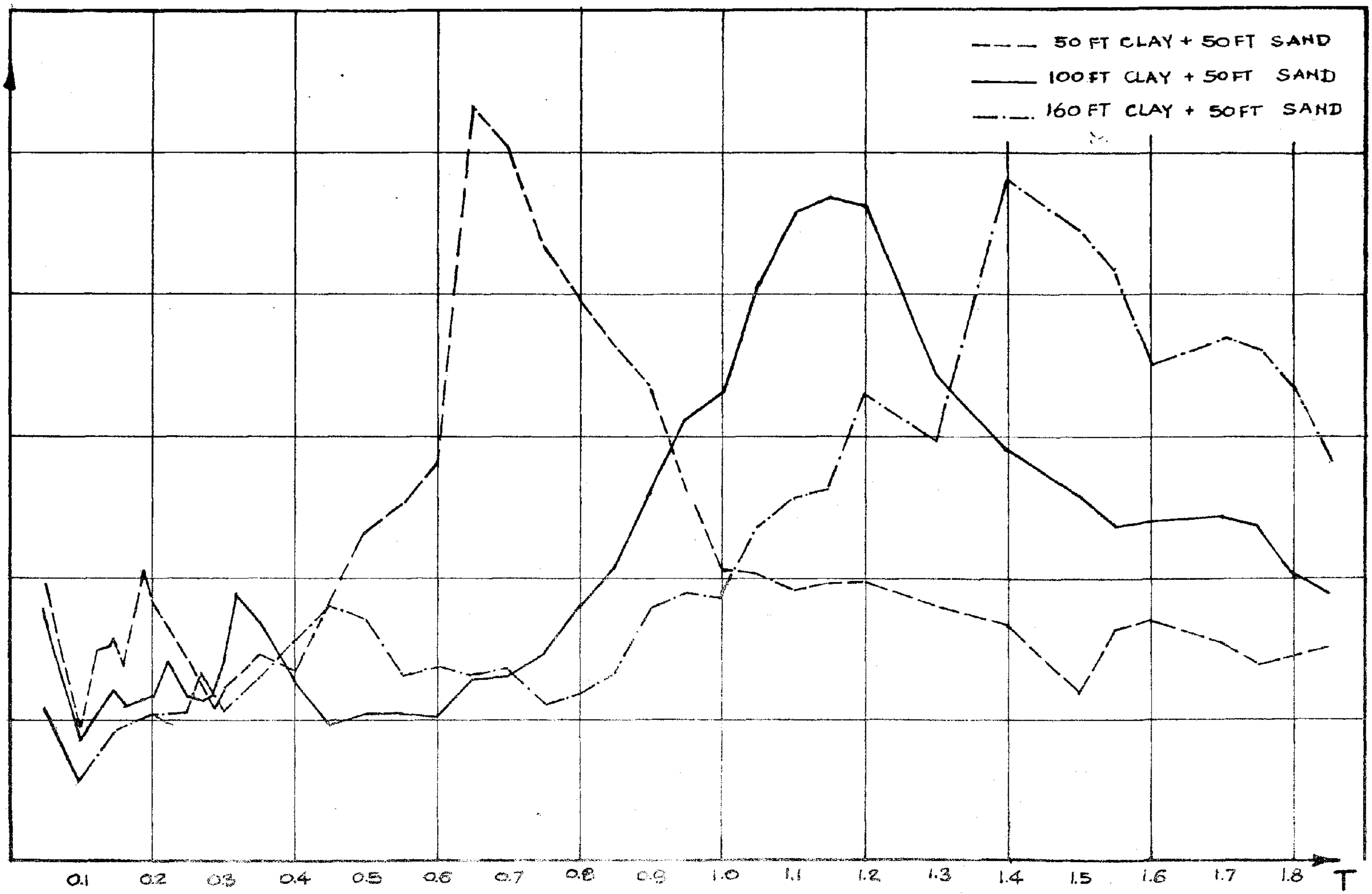

FIGURE 2.16