

**BOĞAZIÇI UNIVERSITY**  
Earthquake Engineering Research Institute

**RESONANT PERIOD EFFECTS  
IN THE GEDİZ, TURKEY EARTHQUAKE OF 1970**

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## RESONANT PERIOD EFFECTS IN THE GEDIZ, TURKEY EARTHQUAKE OF 1970

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### INTRODUCTION

One of the most interesting features of the Gediz, Turkey, earthquake of 28 March 1970 was the damage and partial collapse of some buildings at the Tofas automobile factory located at a distance of about 135 km (85 miles) from the epicentre.

The earthquake had a magnitude of about 7 and the epicentre was located near Gediz, about 225 km (140 miles) south of Istanbul, as shown in Figure 1. Many people were killed and many buildings, mainly poorly constructed dwellings and mosques, were destroyed in the near-epicentral region. This heavy damage occurred within a radius of about 85 km (50 miles) from the epicentre.

Beyond this radius, however, damage was generally minor with the notable exception of the buildings at the Tofas factory near Bursa. Here a one-storey garage and paint workshop, having reinforced concrete frames, partially collapsed. Two large workshop buildings, with steel columns and roof trusses, were also somewhat damaged: bolts of the roof trusses failed, wind bracing collapsed and base plates yielded. By contrast, smaller stiffer buildings at the factory were undamaged and there was little or no damage in Bursa or in other cities at a comparable distance from the epicentre. The damage at the Tofas factory was even more surprising since the structures had been designed to withstand forces comparable to those required by the Uniform Building Code for Seismic Zone 2.

Following the main shock and the collapse of some of the factory buildings, a seismometer was installed at the site. Figure 2 shows the acceleration response spectra computed from the motions recorded during an aftershock (magnitude about 5 and with an epicentre in the same general area as the main earthquake) on 25 April 1970. For any damping ratio, there is a very strong response for structures with periods of about

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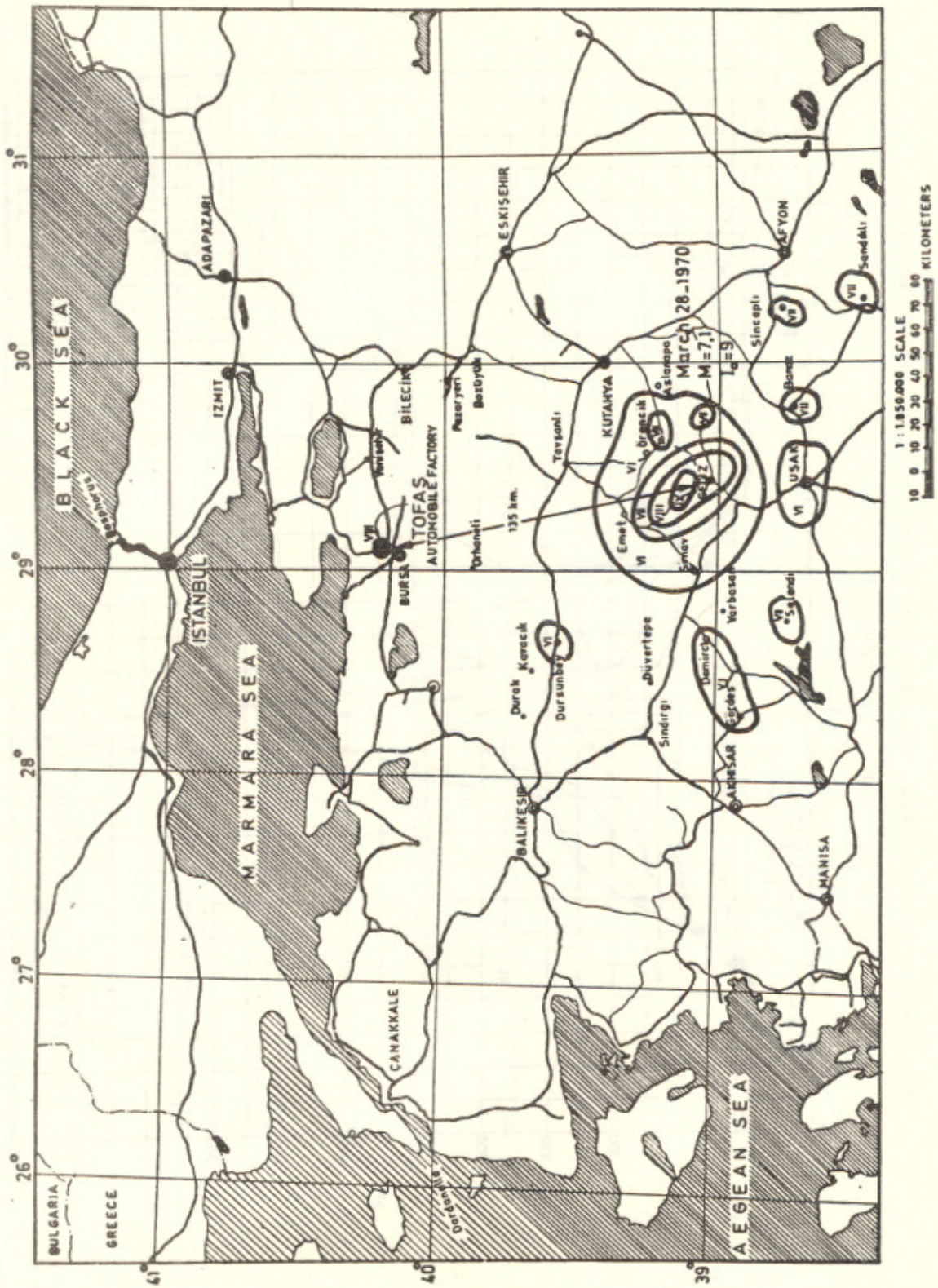


Figure 1. Location of Tofas automobile factory

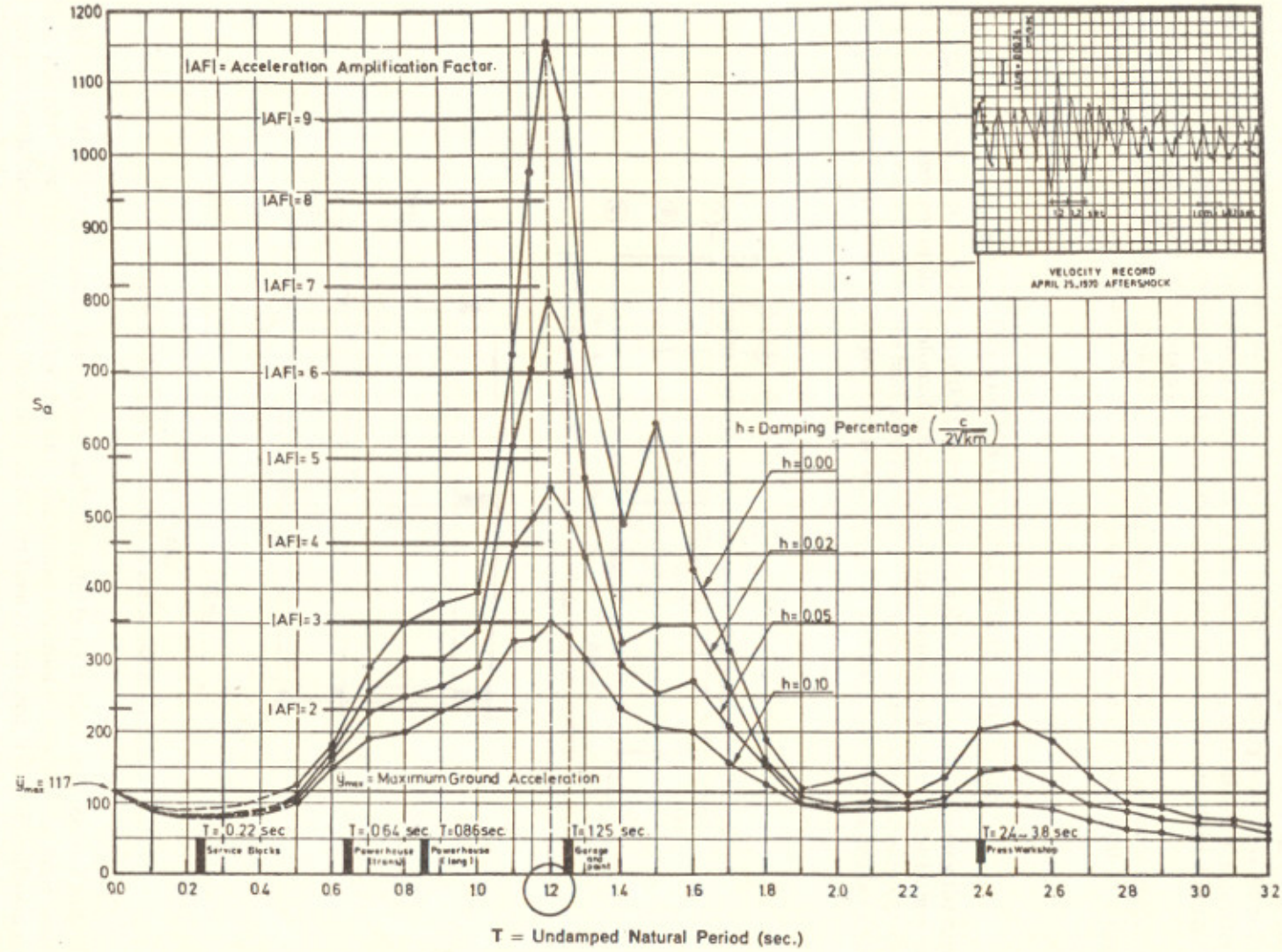


Figure 2. Acceleration and amplification response spectrum curves of the 25 April aftershock records

1.0–1.4 s and a peak response for structures with a period of about 1.2 s. The spectra from another aftershock on 26 April also exhibited a peak between 1 and 2 s, although the peak was not as sharp as in Figure 2.

Based on studies of the damage patterns described above and the characteristics of the structures involved, several investigators<sup>9,22</sup> concluded that the damage at the automobile factory may have been due in large measure to amplification of the earthquake motions by the local soil conditions and the development of a pseudo-resonance condition resulting from the coincidence of the natural periods of the damaged structures and the predominant period of the ground.

The following pages examine the validity of this explanation, and whether response interaction effects of the type described can be anticipated before they occur by appropriate geotechnical investigations and analyses.

### CHARACTERISTICS OF COLLAPSED BUILDINGS

The garage and paint workshop were the subject of considerable investigation following the earthquake.<sup>22</sup> The general arrangement of the reinforced concrete building is shown in Figure 3. The frames and roof of block A had been completed, and the brick exterior walls were in place. The end frame of this block collapsed while other frames were mildly damaged, and all walls toppled. The frames of block B had been poured from

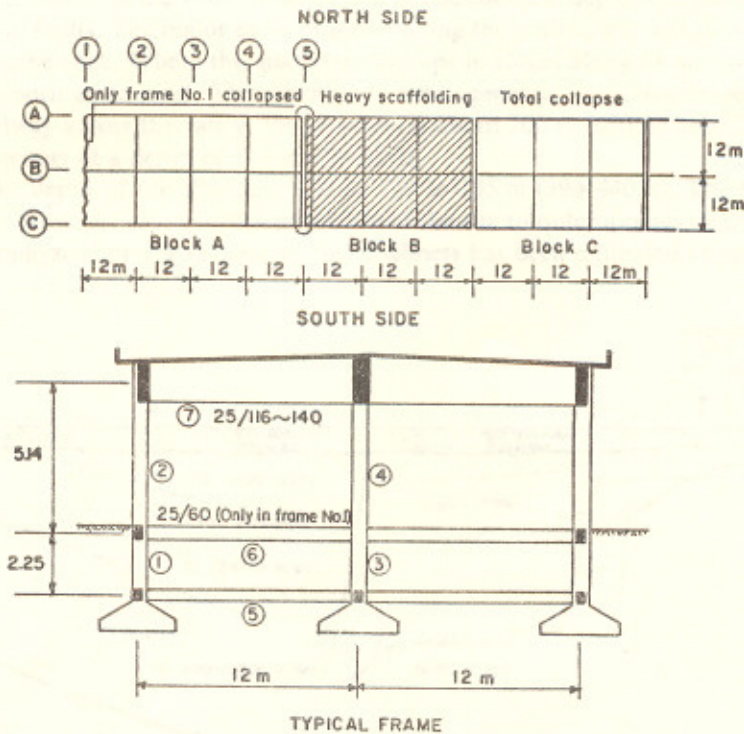


Figure 3. Plan and cross-section of garage and paint shop

3 to 10 days before the earthquake, the erection scaffolding still being in place, and the roof had not yet been added: this part experienced only minor cracking. The frames and roof of block C had been completed and the erection scaffolding removed, but the walls had not yet been constructed. This part collapsed completely.

The building had been designed for a static horizontal force coefficient of 0.06. Analyses made after the earthquake indicated that the horizontal force coefficient necessary to yield a transverse frame was slightly greater than 6 per cent, while a horizontal force coefficient of about 9 per cent would bring a frame to a

fully plastic condition. The requirements for the reinforcing bar details at the beam-column connections at the time of the earthquake were not as strict as those required in the United States beginning with the 1970 Uniform Building Code.

Post earthquake analyses gave the fundamental period of the collapsed building as 1.25 s, essentially at the period showing maximum response on the acceleration spectrum curves from the 25 April aftershock. As indicated in Figure 2, the steel press workshop building which suffered only minor damage had a much longer period, while the concrete service buildings and powerhouse, which were undamaged, had much shorter periods than that producing maximum response in the aftershock.

### SUB-SURFACE CONDITIONS

Soil and geological conditions at and near the site have been established from shallow (7–30 m; 25–100 ft) geotechnical borings, deep (122–127 m; 400–450 ft) drillings for water supply development and various geophysical surveys (resistivity, gravity, reflection and refraction profiles). These studies have been summarized by Tezcan *et al.*<sup>21</sup> and Whitman *et al.*<sup>24</sup>

The Tofas factory lies near the north edge of a broad flat plain. Geologically, the valley has been described as a graben. Figure 4 shows a geological cross-section through the plain along a north-south line several km from the factory. Under the centre of the valley, bed rock is at considerable depth and there is a suggestion of successive, nearly vertical faults. The major earth material filling the graben is a soft rock of the Tertiary period, referred to as Neocene rock. Where this material outcrops in ridges along or across the valley, it is identified in geotechnical borings as marl. The alluvium is heterogeneous, involving lenses and strata of gravel, sand and clay. Midway across the valley, the alluvium is about 200 m (650 ft) deep. The water table across the plain typically occurs at a depth of 5–7 m (16–23 ft).

At the Tofas factory, the depth of the alluvium is from 120 to 135 m (390–440 ft). The thickness of the Neocene rock is uncertain; however, based on the proximity of the site to outcroppings of the Neocene rock and on the basis of the geophysical profile (Figure 4), this thickness has been estimated as being from 100 to

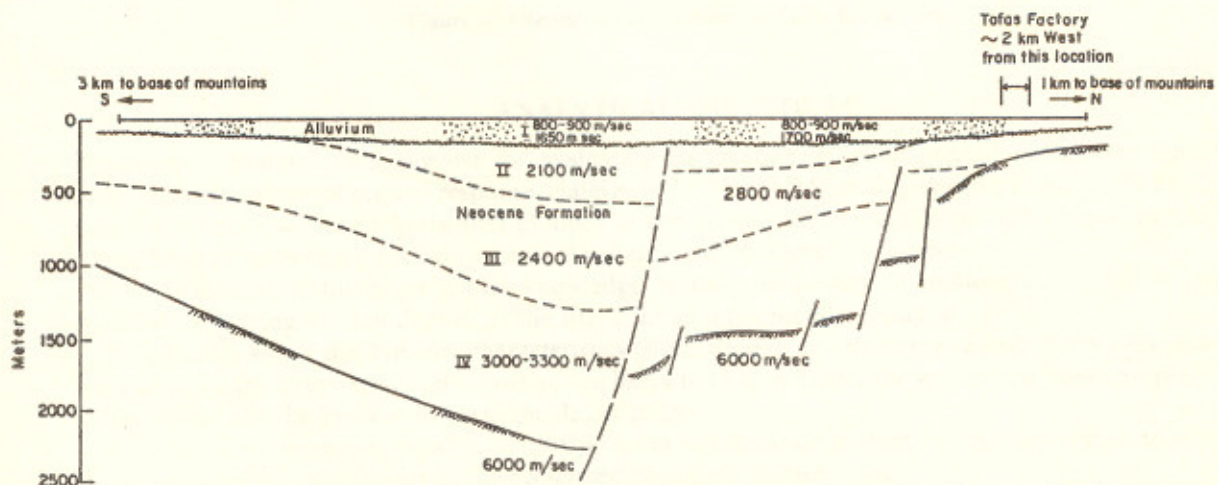
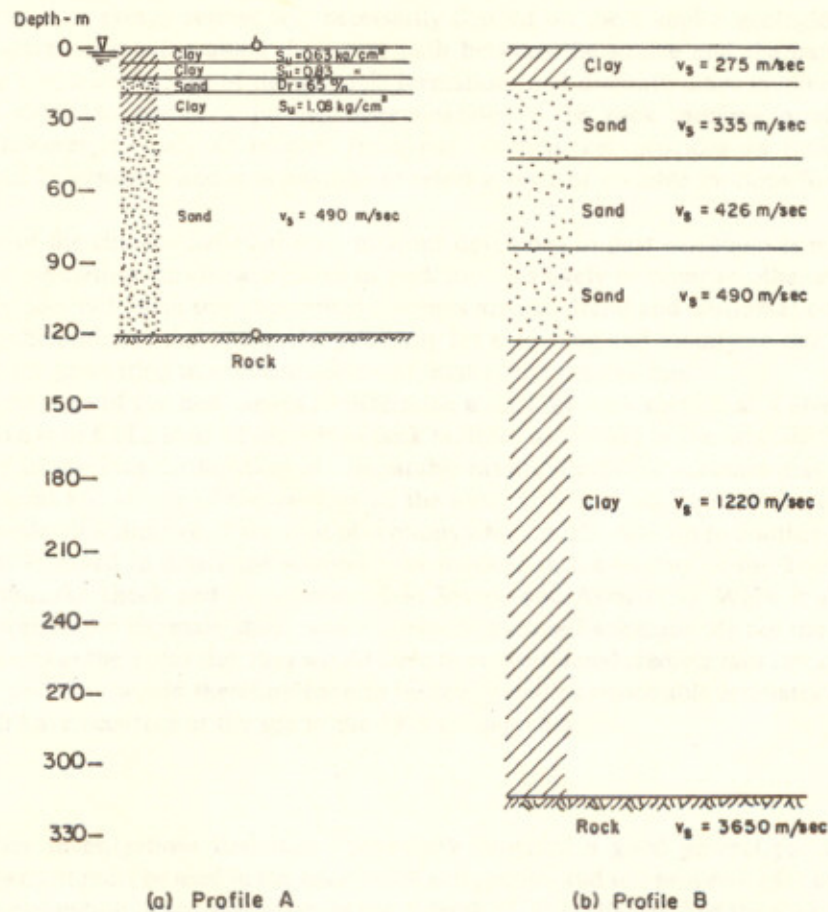


Figure 4. North-south profile through Bursa Plain from reflection studies, and velocities of compressional waves

200 m (330–660 ft). The geotechnical properties of the upper 30 m (100 ft) of the alluvium have been established from blow counts, some unconfined compression tests and resonant column test on 'undisturbed' samples of clay, and the measured seismic wave velocities. Wave velocity data are also available for the underlying soil layers. Details of the soil profile determined by these means are presented in Figure 5.



(a) Profile A (b) Profile B  
Figure 5. Alternative soil profiles for Tofas factory site

### ANALYTICAL PROCEDURE

A procedure finding increasing use for evaluating the characteristics of ground surface motions during earthquakes is the use of ground response analyses.<sup>1, 4, 6, 7, 10-12, 15, 23</sup> Such analyses are based on the assumption that the primary source of horizontal motions at the ground surface is the upward propagation of shear waves from an underlying rock formation. Analytical studies therefore require:

1. A reasonable estimate or actual knowledge of the time-history of motions developed in rock-like material underlying the soil deposit at the site or in an adjacent rock outcrop.
2. A knowledge of the dynamic characteristics (shear moduli and damping ratios) of the soils underlying the site; since the stress-strain relationships are likely to be non-linear, the variation of these properties with strain should also be known with reasonable accuracy.
3. A computer program, capable of incorporating the strain-dependent soil characteristics, to determine the response of the soil deposit to the prescribed base rock accelerations.

The steps involved in evaluating the response of the Tofas factory site using this procedure are outlined below.

#### (a) Rock motions

Evaluation of rock motions developed during earthquakes is one of the greatest sources of uncertainty in analyses of seismic effects. The characteristics of the motions developed in a rock formation at some

distance from the source of energy release will necessarily depend on the complex geological formations encountered by the waves in moving along the travel path between the source and the particular site of interest. It is unlikely that a knowledge of the geologic formations and discontinuities involved will ever be known in sufficient detail to permit a precise determination of the rock motions produced by any given earthquake. However, a study of records from past earthquakes provides an indication of the variations which might be expected and it is possible to select a suite of credible motions for many design purposes.

In fact, knowledge of the characteristics of rock motions developed in past earthquakes is probably the only reliable basis for estimating the characteristics of rock motions likely to occur in other earthquakes at the present time. This does not mean that theoretical concepts are not useful and desirable; on the contrary they can be extremely helpful, but they are useful primarily for extending and modifying our knowledge of field records and not for generating new information—at least at the present time.

In view of these facts, one of the best means to determine a credible rock motion at a given location in a particular earthquake is to find a suite of records of rock motions developed at the same distance from the zone of energy release in previous earthquakes of comparable magnitude. Some selection may be warranted on the basis of a general knowledge of the geology of the areas involved but most of these past records would have to be considered indicative of the type of motions which might develop in another similar event.

This procedure was followed to determine possible rock motions in the vicinity of the Tofas factory site due to the main earthquake shock and the aftershock in March and April 1970. While it should not be imagined that the motions for the main shock and aftershock discussed subsequently are the only possible estimates of rock motions at the Tofas site, they would have to be considered credible motions on the basis of available knowledge, and they would therefore have to be considered as reasonable estimates of the excitation which might well have occurred at the site in the 1970 earthquakes.

#### (b) Soil properties

While the subsurface investigations described above have provided a good general picture of subsoil conditions, some judgement must be used in the selection of soil profiles and soil properties for use in analysis.

One problem is the evaluation of soil properties below a depth of 30 m (100 ft). For the soils in this deeper alluvium it is possible to determine reasonable bounds on their characteristics from available information on the dynamic properties of soils.<sup>16,3</sup> For example, if the soil in this zone was normally consolidated clay or very dense sand its average shear wave velocity would be about 490 m/s (1,600 fps). For a clayey sandy gravel, the shear wave velocity would be expected to be somewhat higher, about 615 m/s (2,000 fps).

The average shear wave velocity of the soils in this zone can also be estimated from the measured values of the compressive wave velocity, using the relationship

$$\frac{v_p}{v_s} = \left( \frac{2-2\mu}{1-2\mu} \right)^{\frac{1}{2}}$$

where  $\mu$  = Poisson's ratio. For sands and gravels  $\mu$  is approximately 0.35 while for saturated clays,  $\mu$  will be closely equal to 0.5. Thus for the combined soils in this zone, the average value of  $\mu$  is likely to be in the range of 0.4–0.45. From the above formula and the measured value of  $v_p = 5,300$  fps, the average shear wave velocity of the soils in this zone would therefore fall in the range of 1,600–2,100 fps. While the measured value of  $v_p = 5,300$  fps is very probably the compression wave velocity of water, it is clear that the compression wave velocity of the soil cannot exceed this value, and thus the computed range of values for the shear wave velocity of the soil probably represents an upper bound on this parameter. Thus it seems reasonable to conclude that shear wave velocities in the range of 1,600–2,000 fps are representative of the soils below a depth of 30 m (100 ft). Corresponding values of shear moduli,  $G$ , were computed from the relationship:

$$v_s = \sqrt{\left( \frac{G \cdot g}{\gamma} \right)}$$



where

$g$  = acceleration of gravity

$\gamma$  = unit weight of soil

$v_s$  = shear wave velocity

This same approach may be used to deduce values of  $v_s$ , and hence  $G$ , for the rock below the alluvium.

The values of shear moduli determined at very low strains in this way were reduced for analysis purposes in accordance with relationships shown in Figure 6 which are generally representative of moduli vs strain relationships for clays and cohesionless soils. Representative damping values for these types of soils are also shown in the figure.<sup>16</sup>

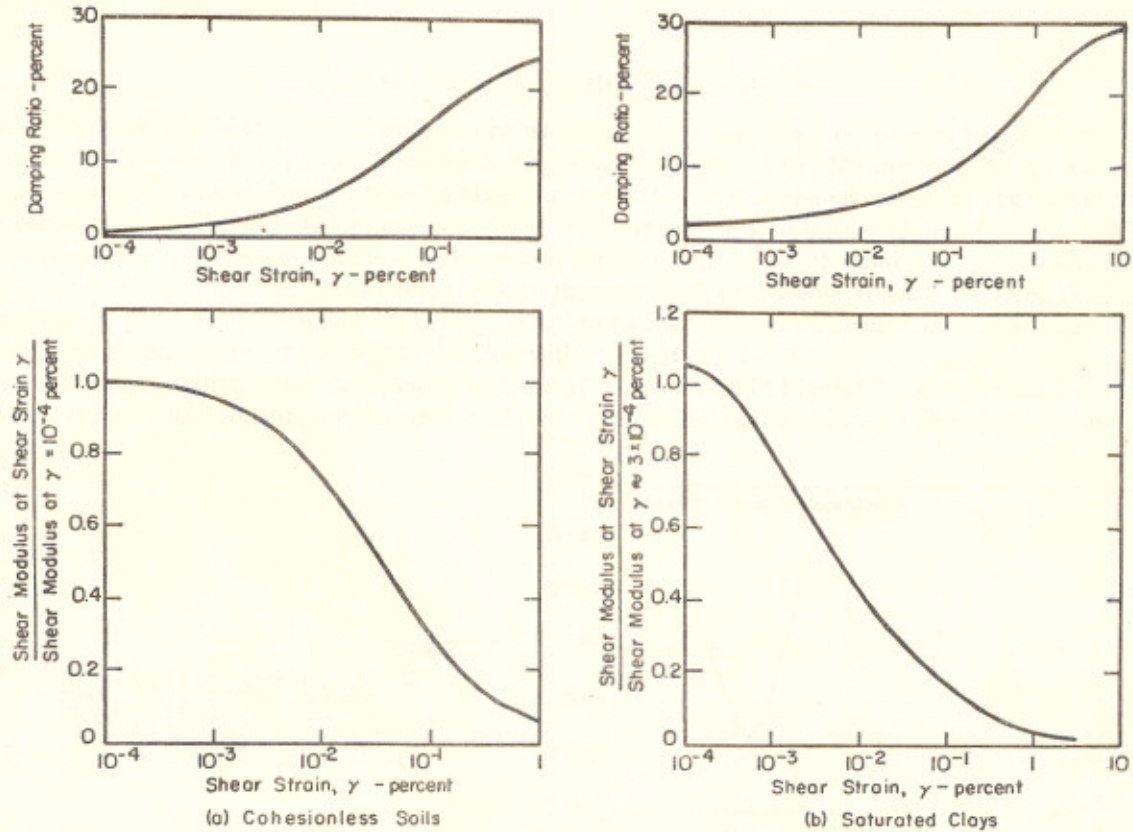


Figure 6. Dynamic characteristics of soils used in ground response analyses

A second problem is selection of the depth to be considered in the soil profile. Two viewpoints are found in practice. One viewpoint holds that many accelerographs providing records used as rock motions are located on materials with a compression wave velocity of about 1,700 m/s (5,500 fps), and hence the soil profile used in analysis should logically be terminated when material with this velocity is reached. The other viewpoint maintains that the profile should be continued until hard rock, with a compression wave velocity of at least 3,600 m/s (12,000 fps), is encountered.

Figure 5 shows two different profiles prepared by different subsets of the writers. Within the alluvium, while the profiles differ in the details of the stratification, the shear wave velocities are very similar. The major difference lies in the treatment of the earth material below the alluvium. In Profile A, rock is assumed to occur at the base of the alluvium, and the rock motions are assumed to be applied at this interface. In Profile B, a stratum of 'clay' representing the Neocene rock is included, and the rock motions are assumed to occur at an outcropping of the base rock. Thus, in Profile A the rock motions are assumed to be unaffected by the overlying alluvium, while in Profile B the rock motions are affected by the alluvium and Neocene rock.

## (c) Computations

Using the data on soil characteristics and rock motions discussed above, analyses were made to compute the ground surface response during the main shock and the aftershock. The computations were made using the wave propagation analysis procedure, and incorporating strain-dependent properties. This involved an iteration procedure since the strains in the deposits were not initially known. Accordingly an estimate was first made of the appropriate soil properties and the response of the deposit to the rock excitation was computed. This computation provided values of the shear strain developed in the different layers, which were then used to determine an improved assessment of appropriate soil characteristics. This procedure was continued until the soil properties used in the computations were compatible with the strains developed in the soils by the earthquake motions. The computations were made using the computer program SHAKE<sup>14</sup> which performs these iterations automatically.

## ANALYSIS OF AFTERSHOCK MOTIONS

For the aftershock analysis, a rock motion record was required at a distance of about 130 km (80 miles) from an earthquake with a magnitude of about 5. The closest available record for these conditions was the accelerogram recorded on rock near Cachuma Dam during the Parkfield earthquake of 1965. The magnitude of the earthquake was 5.5 and the epicentral distance of the recording station was 135 km (83 miles). The record was essentially a sinusoidal motion with a period ranging from 0.9 to 1.5 s and a maximum acceleration of 0.003 g. To allow for the smaller magnitude of the aftershock near Gediz, it was considered that the rock motions at the Tofas factory site might be expected to have a maximum acceleration of 0.002 g and be represented by several cycles of a sinusoidal motion with a period of 1.1–1.3 s.

The results of the analysis to the aftershock motions are summarized in Figures 7 and 8. Figure 7 shows the ground surface motions computed using Profile A and a base rock motion having a period of 1.3 s and a

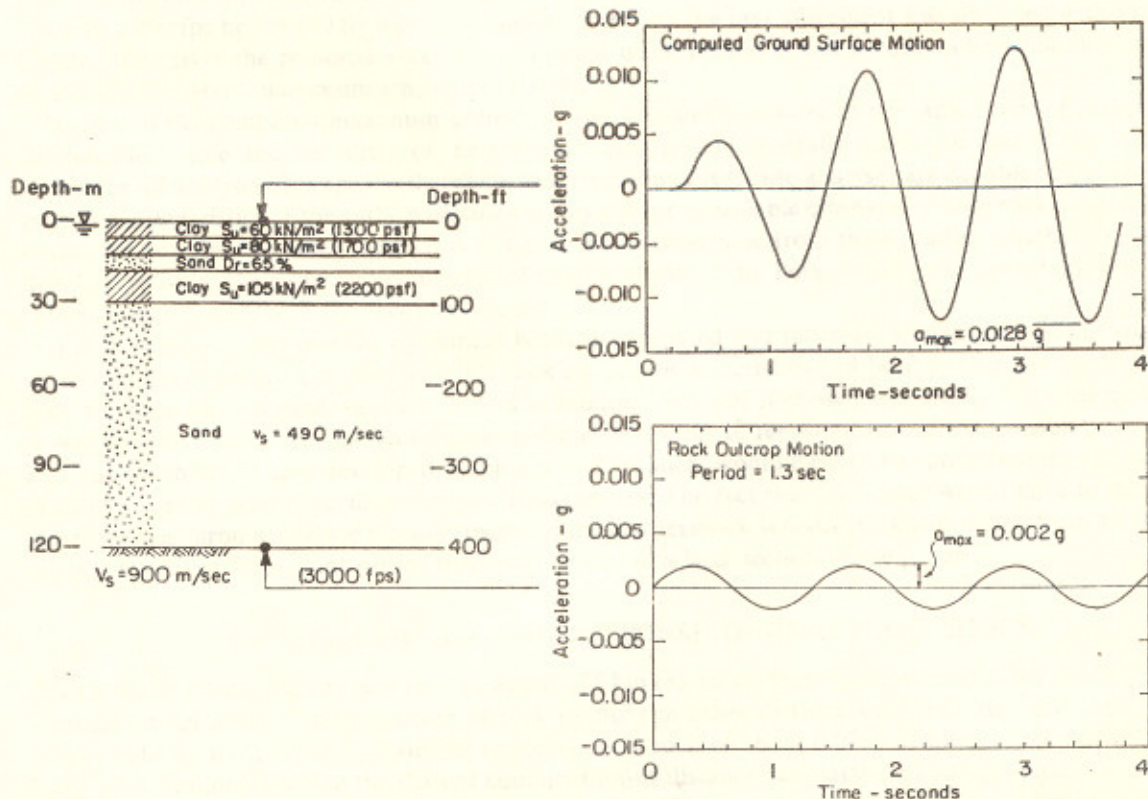


Figure 7. Ground response analysis for aftershock motions

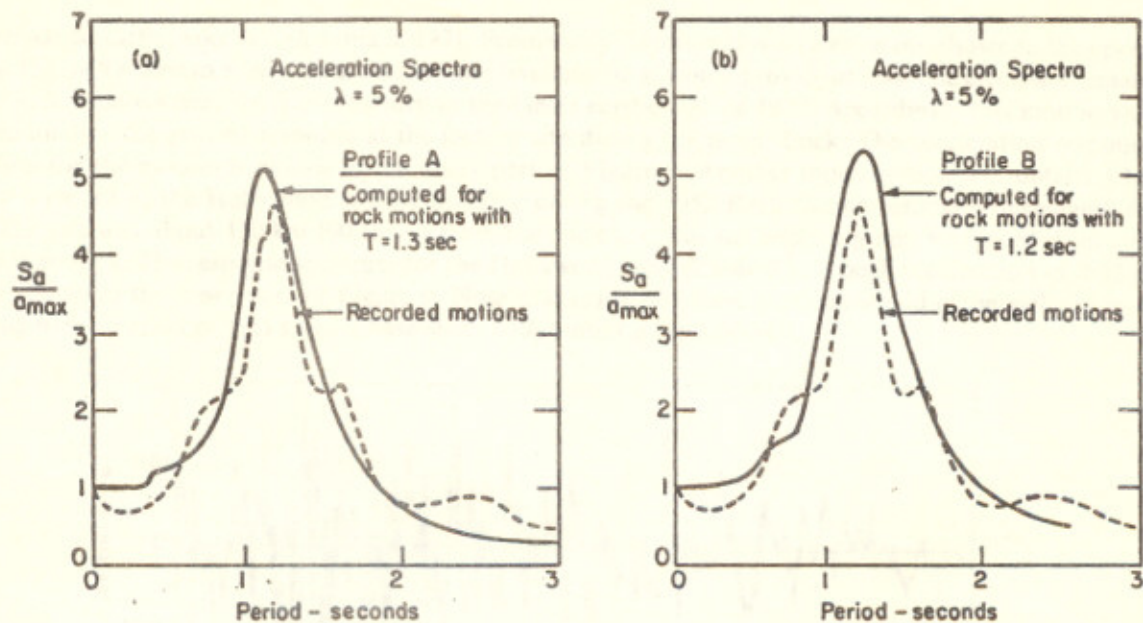


Figure 8. Comparison of acceleration response spectra for recorded and computed aftershock motions

maximum amplitude of 0.002  $g$ . For the strains developed in the soil profile, the damping ratio varied from 0.5 to 3.6 per cent and the maximum ground surface acceleration was computed to be 0.013  $g$ . The acceleration response spectrum for the computed surface motions is shown in Figure 8(a). The results of other computations for different rock motions (periods from 1.1 to 1.3 s) and soil characteristics ( $v_s$  varying from 1,600 to 2,000 fps below 100 ft) were very similar, although the best agreement was obtained with Profile A. Figure 10(b) gives the response spectrum computed using Profile B and a rock outcrop motion having a period of 1.2 s and a maximum amplitude of 0.002  $g$ .

In general the computed maximum ground surface accelerations were of the same order of magnitude as the recorded value and the forms of the response spectra were generally similar to that of the aftershock record for all analyses. It appears, therefore, that the ground response analyses are capable of computing the general characteristics of the surface motions at this site for reasonable estimates of base rock motion and soil characteristics. On the other hand, it is not possible to determine from these studies whether Profile A or Profile B provides the better representation of the soil profile at the Tofas factory site, since both give spectra agreeing equally well with the recorded spectra.

It is fair to point out that the agreement between computed and recorded aftershock motions would not have been nearly so good if other possible rock motion characteristics had been used in the analyses. However, the rock motion used was not selected at random; nor was it chosen specifically to fit the conditions. It was used because it was shown by past earthquake records to represent one of the possible suite of rock motions which could have developed at this site and, as such, would have to be considered in computing the possible range of ground surface motion characteristics. The fact that this range would have to include the rather unusual ground surface motions recorded in the aftershock is distinctly encouraging from an engineering point of view. This point will be discussed further in a later section of the paper.

#### ANALYSES OF GROUND RESPONSE DURING MAIN SHOCK

For the main shock, the site was located about 135 km (85 miles) from the epicentre of an earthquake with a magnitude of about 7. Examination of rock motions in other earthquakes<sup>13</sup> indicates that such an event would be likely to produce a maximum acceleration in rock of about 0.02  $g$ . While it is not possible to find many rock motions meeting the desired combination of distance and earthquake magnitude, a motion was recorded on rock-like formations at San Onofre about 130 km (80 miles) from the ep'central region of the

San Fernando earthquake in California in 1971. Presumably therefore this accelerogram, shown in the upper part of Figure 9 scaled to a maximum acceleration level of  $0.02\text{ g}$ , would provide at least one credible estimate of the rock motions near the Tofas factory in the Gediz earthquake of 1970. Accordingly this motion was used to analyse the ground response at the factory site during the main shock. The acceleration response spectrum for the motion is shown in the lower part of Figure 9. Another input motion was obtained by scaling a record at the Hollywood storage building during the 1952 Kern County earthquake (magnitude 7.1). The site was about 150 km (90 miles) from the epicentre, but the accelerograph was located on soil rather than rock. The response spectrum for the Hollywood record, scaled to a peak acceleration of  $0.02\text{ g}$ , is also shown in the lower part of Figure 9. Note that neither of these motions would cause yielding in a building with a period of 1.25 s and a base shear coefficient at a yield of 0.06.

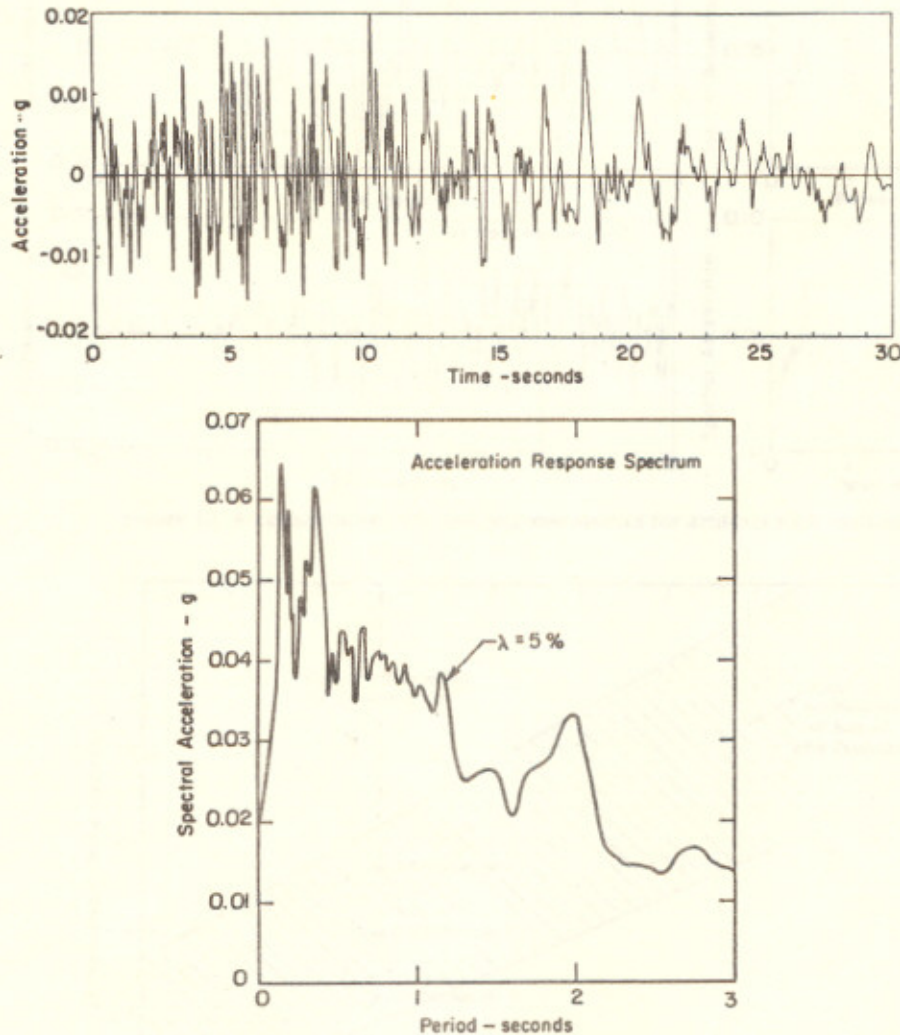


Figure 9. Accelerogram and acceleration response spectrum for San Onofre record of 1971 San Fernando earthquake

Ground response analyses were also made for three artificial rock motion records, as shown in Figure 10. The predominant periods of a number of rock motions, defined as the periods at which the peak response occurs in an acceleration response spectrum, have been presented by Figueroa.<sup>2</sup> The range of predominant periods at different epicentral distances for earthquakes with magnitudes  $> 7$  is shown in Figure 11. These results indicate that at an epicentral distance of about 135 km or 85 miles, the predominant period of rock motions is likely to fall in the range of 0.4–0.9 s. Accordingly three artificial accelerograms were developed

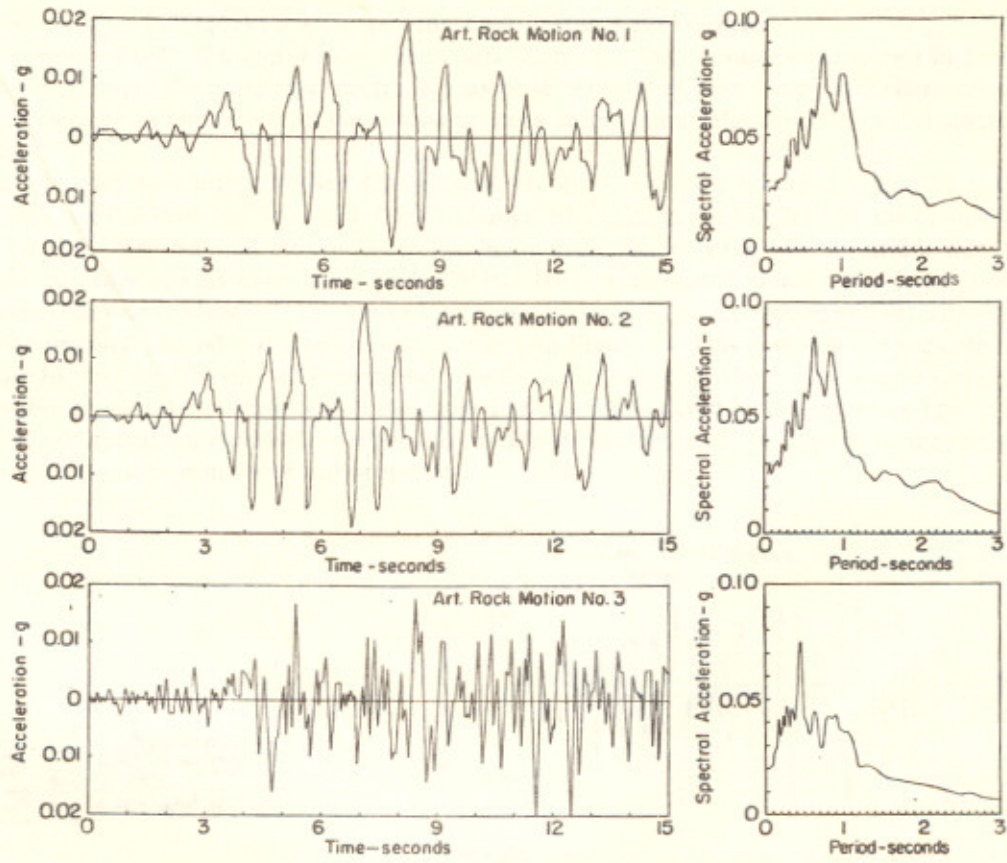


Figure 10. Acceleration records and response spectra for artificial rock motions

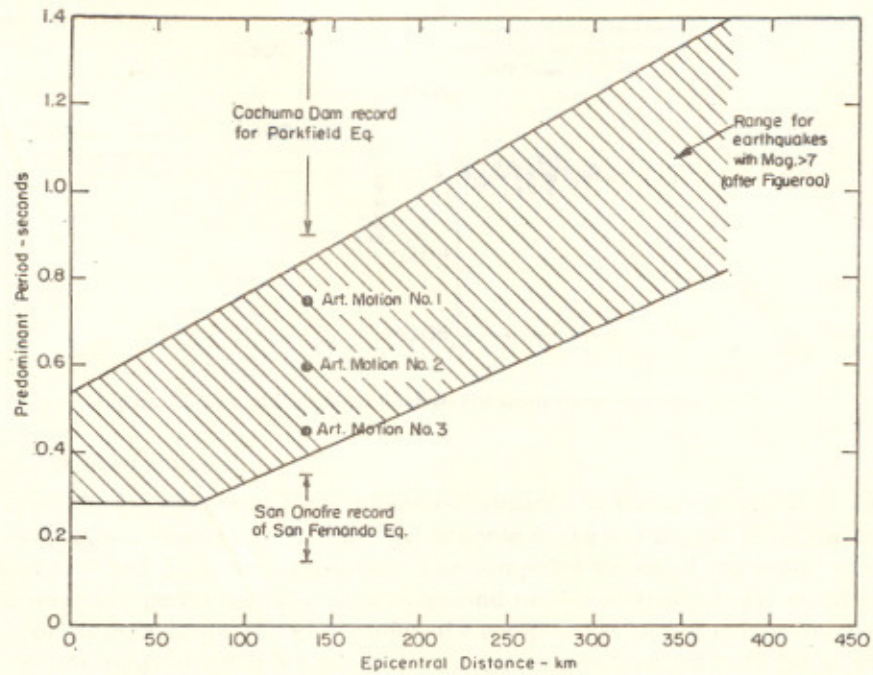


Figure 11. Predominant periods of rock motions used for analyses

by modification of other earthquake motion records, each having a peak acceleration of 0.02 g, but having predominant periods of 0.45, 0.6 and 0.75 s. The accelerograms for these motions are shown in Figure 10 together with the corresponding response spectra. It may be seen that they have the general characteristics of rock motion accelerograms and also meet the criteria for peak acceleration values and spectral characteristics discussed above.

The results of an analysis using the San Onofre rock record as the base rock excitation are shown in Figure 12; in this analysis Profile A was used. For the computed strains in the soil deposit, the damping ratio ranged from 2.2 to 7.1 per cent and the computed maximum acceleration at the ground surface was 0.066 g which is within the range of maximum accelerations (0.046–0.075 g) indicated to have developed during the main earthquake shock on the basis of the observed structural damage. The acceleration and velocity response spectra for the computed ground surface motions are shown in Figure 13. This computation indicates a very strong response for buildings having a fundamental period of about 1.2 s, as shown in Figure 13, and would apparently readily explain the extraordinarily high degree of damage suffered by the garage and paint shops.

A number of additional computations were made to indicate the effect of the various assumptions concerning input motions, soil profile and soil properties.

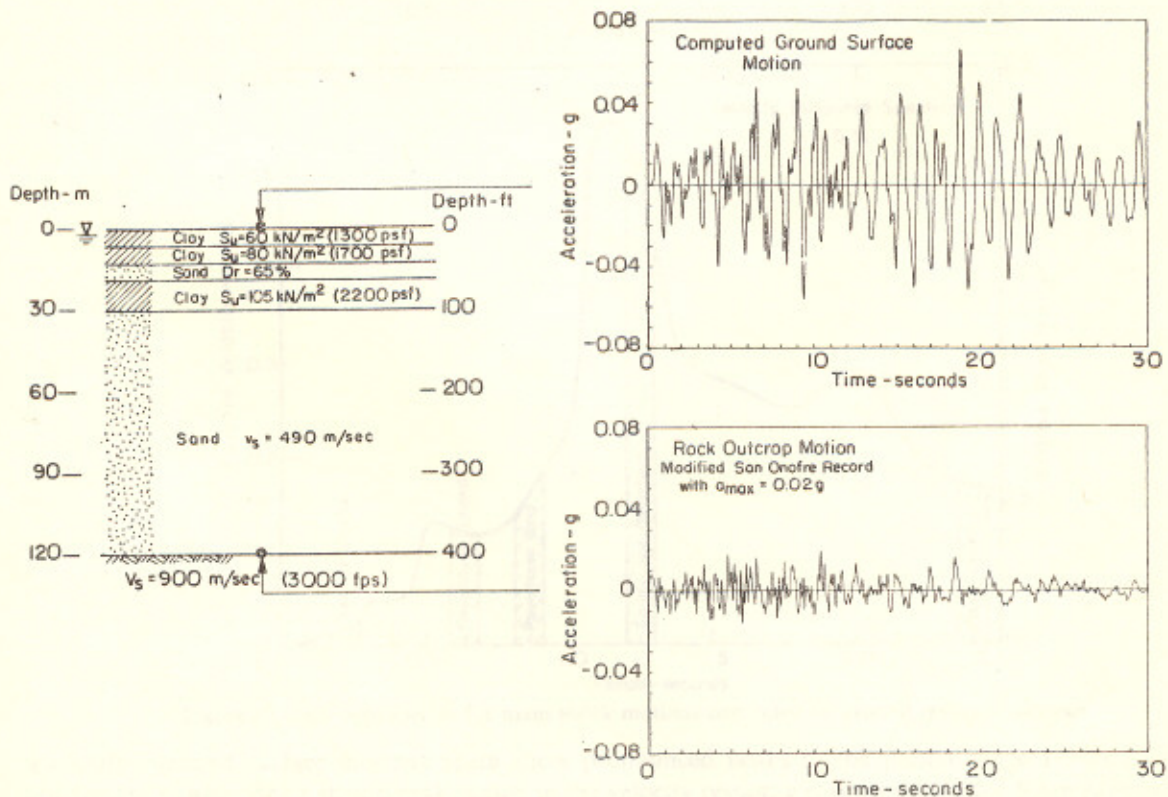


Figure 12. Ground response analysis for main shock motions

(a) *Effects of variation in input motions*

To determine whether the foregoing result was influenced primarily by the characteristics of the base rock excitation, similar analyses were made to determine the response of the soil deposit using the artificial rock motions' records, Nos 1, 2 and 3, as rock excitation. The computed values of maximum ground surface acceleration and the response spectra for the computed ground surface motions are presented in Figure 14. It may be seen that for all three artificial rock motions, the computed values of maximum ground surface acceleration lie within the range 0.058–0.065 g, which is again in good accord with the estimated range (0.046–0.075) indicated by analysis of the structural damage. Furthermore, the response spectra for the

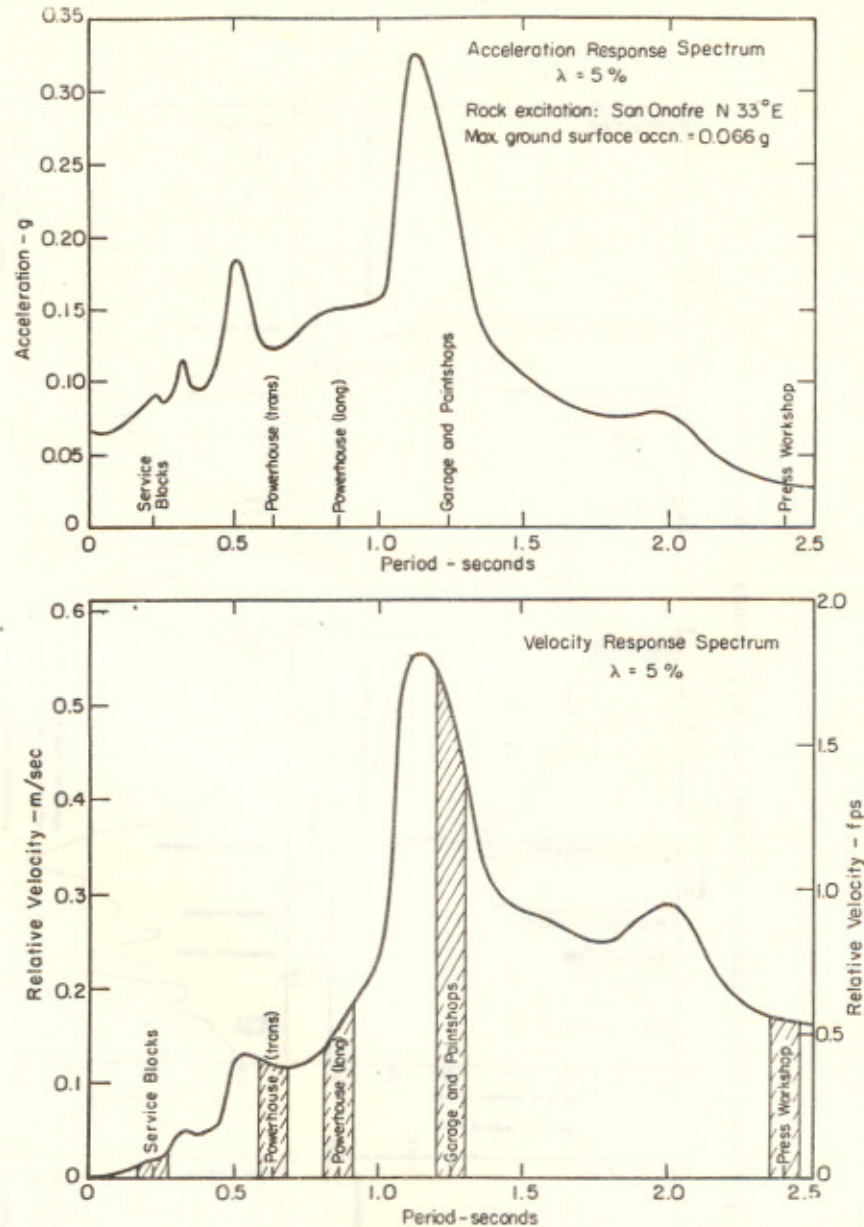


Figure 13. Response spectra for main shock motions computed by ground response analysis

computed ground surface motions again show pronounced peaks in the period range 1.0 to 1.5 s. It is interesting to note that a strong peak occurs in the velocity response spectrum even for Rock Motion No 3, which had a predominant period of only 0.45 s, illustrating that for some sites, generally similar forms of response spectra may occur for widely different forms of rock excitation.

(b) *Effects of variations in properties of alluvium and soil profile*

A series of analyses were made, using Profile A, to determine the extent to which the computed results in Figures 13 and 14 might have been influenced by the choice of characteristics assigned to the soil deposit between depths of 30 and 120 m (100 and 400 ft). For this purpose, the soil in this zone was considered to be either cohesionless or cohesive and to have shear wave velocities ranging from 490 to 610 m/s (1,600–2,000 fps). The San Onofre motion was used as base rock excitation. The ranges of velocity response spectra for the computed ground surface motions are shown in Figures 15 and 16.

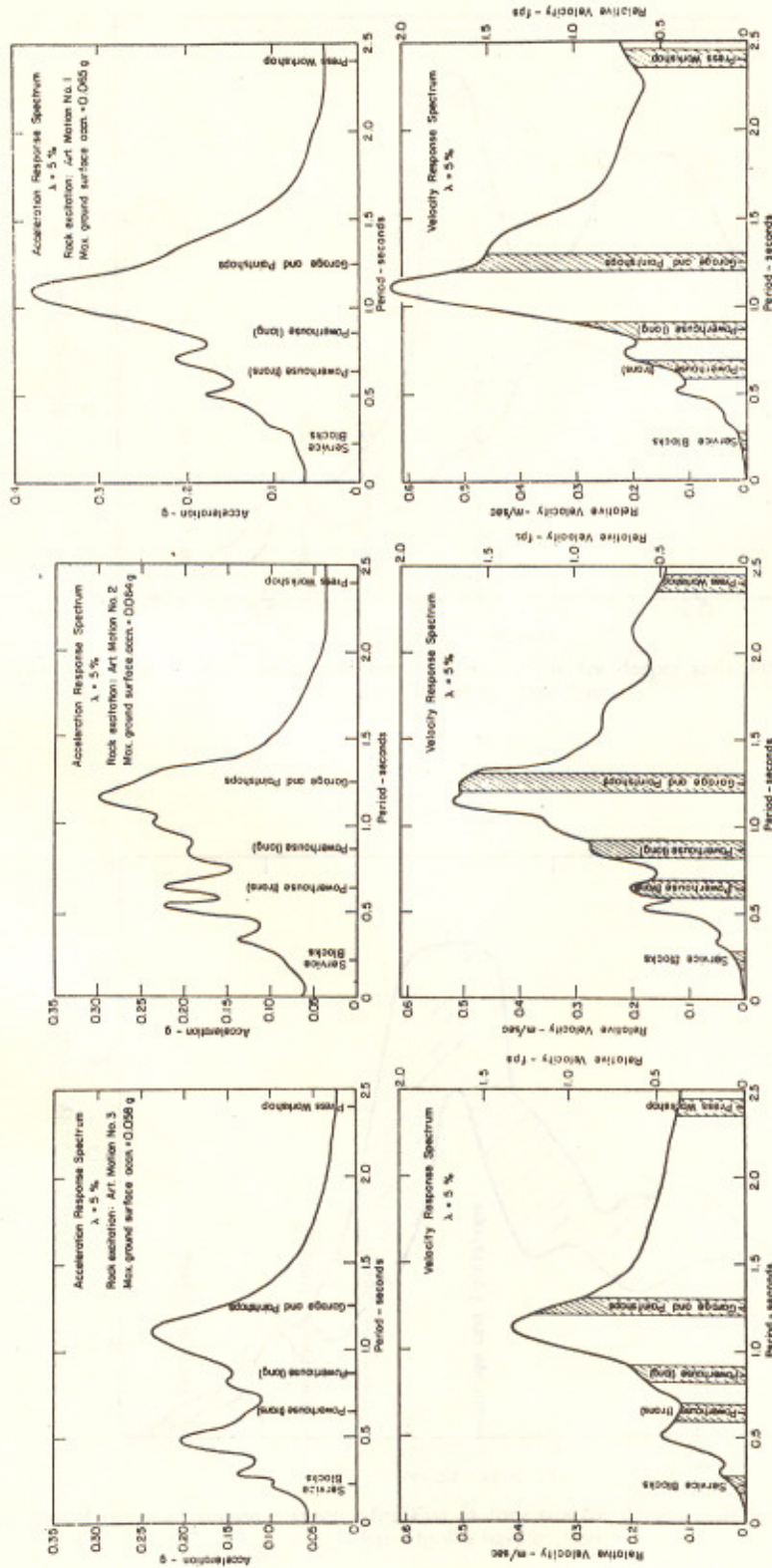


Figure 14. Response spectra for main shock motions computed by ground response analyses



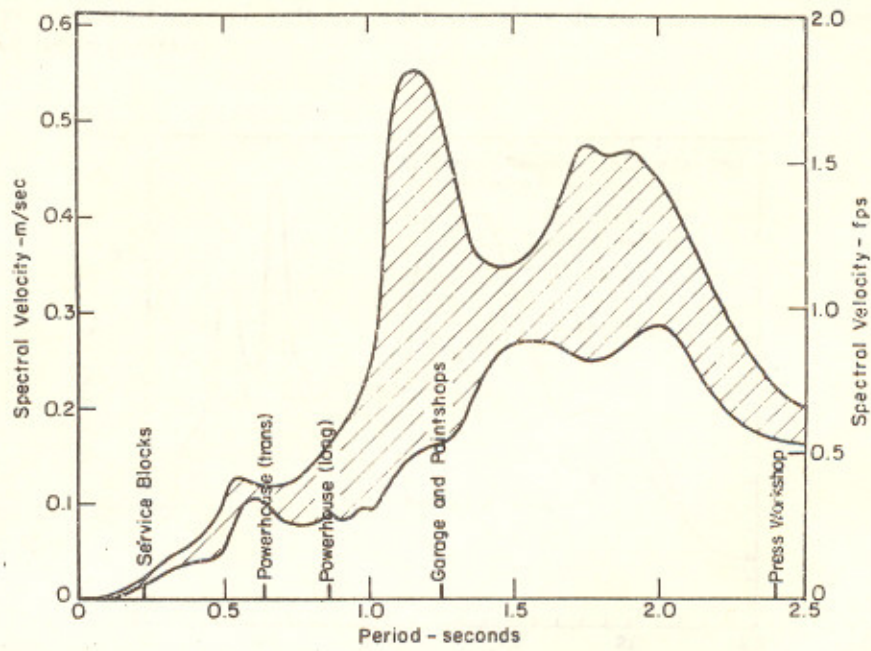


Figure 15. Computed range of response spectra for Fiat factory site for deeper soils with  $v_s = 500$  m/s (1,600 fps) and San Onofre base motion

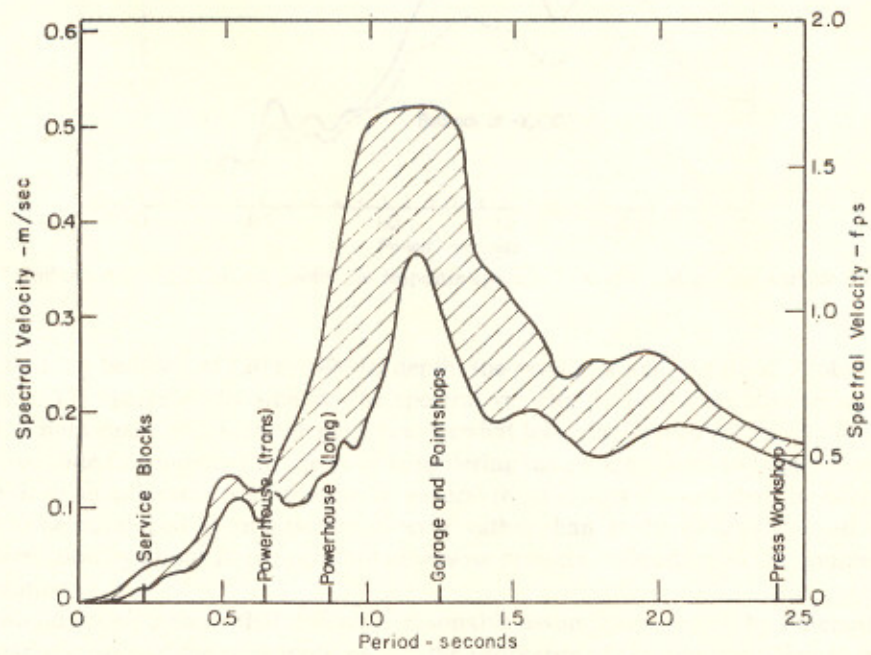


Figure 16. Computed range of response spectra for Fiat factory site for deeper soils with  $v_s = 600$  m/s (2,000 fps) and San Onofre base motion

Another analysis was made for Profile B using the same San Onofre motion as excitation. The results of this analysis are presented in Figure 17, both for conditions where the rock is considered to be at a depth of 120 m (400 ft) and 300 m (1,000 ft).

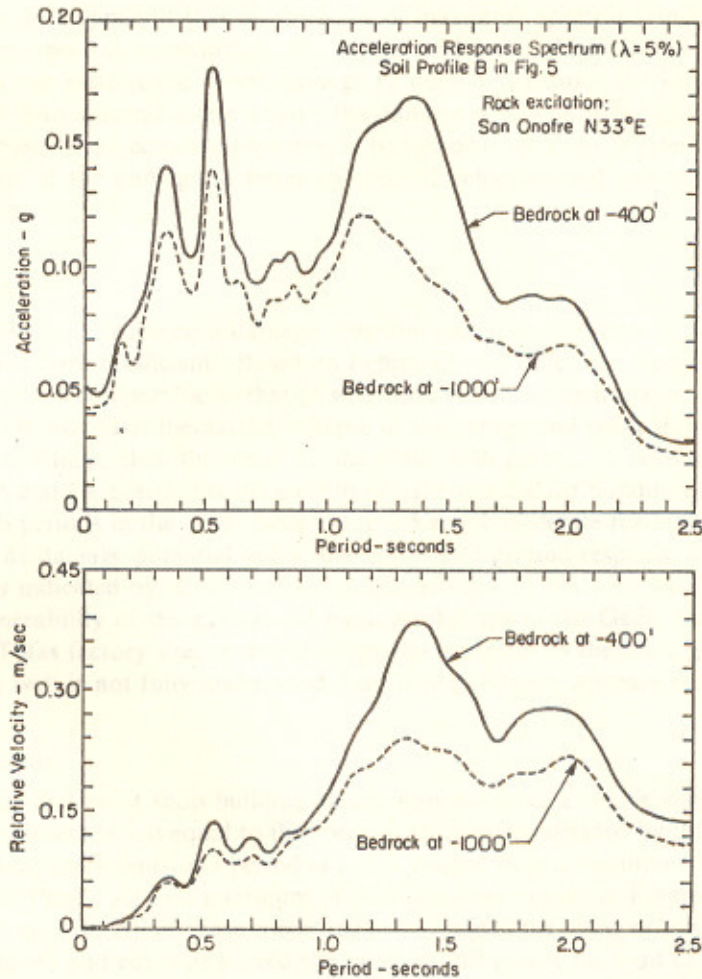


Figure 17. Influence of depth of soil profile on response spectra for computed ground surface motions

It will be seen that for bedrock at 120 m (400 ft) depth, the results of analysis using Profile A (Figure 13) or Profile B (Figure 17) are generally similar, the spectral velocity plot showing a pronounced peak at a period of 1.2-1.4 in both cases, although the peak is somewhat less pronounced for Profile B.

Studies were also made to compare the effects of considering the rock motions to be developed at the base of the soil profile or in an adjacent rock outcrop. In general these results showed that the effect of assuming the rock motion to be developed at an adjacent outcrop rather than at the base of the soil was to reduce the base rock acceleration by about 20 per cent but otherwise produce little effect on the computed frequency contents of the motions.

These studies would seem to show that different reasonable assumptions, made by different investigators concerning the characteristics of the rock motions and the properties of the alluvium, lead to results that are substantially similar in overall effects even though they may differ in detail. All of the analysis showed a strong amplification of motions in the period range 1.1-1.5 s with the greatest amplification being obtained using Profile A.

## DISCUSSION OF RESULTS FOR MAIN SHOCK

While there are some variations between the shapes of the computed spectra shown in Figures 13–17, all of them serve to explain the high degree of damage suffered by the garage and paint workshop in the Gediz earthquake. Also shown on the spectral plots are the fundamental periods of the various structures at the Tofas factory site. It is apparent that on the basis of maximum spectral velocity, and in most cases on the basis of maximum spectral acceleration, the garage and paint shops would be expected to develop the greatest effects of the earthquake. Since damage to ductile structures is likely to be determined more by spectral velocities than spectral accelerations, the damage potential of the garage and paint shops might be expected, on the basis of the computed spectra, to be two or three times greater than that of the other factory buildings. Analysis of the damage in terms of spectral velocities and spectral accelerations may be summarized as follows:

*Spectral velocities*

Seed and Idriss<sup>17</sup> have proposed a damage potential index,  $S_v/C$  (where  $S_v$  is the spectral velocity and  $C$  is the design lateral force coefficient). Based on Figure 13, this index for the garage and paint shop is about 8 m/s (27 fps), which is comparable to that of structures which suffered major damage in the Caracas earthquake of 1967.<sup>19</sup> On this basis the partial collapse of the garage and paint shops is not surprising. If  $C$  was the same for all buildings, then the trend in this index with period, as reflected by the velocity spectra in Figures 13, 14, 15 and 16, exactly fits the pattern of damage: low for buildings with short periods, very high for buildings with periods in the range from 1.0 to 1.5 s and moderate for buildings with very long periods. Thus, based on the damage potential index, the analysis of ground response for a reasonable range of soil characteristics, as indicated by seismic velocities and aftershock analyses, would readily have predicted the much higher vulnerability of the garage and paint workshops in the Gediz earthquake compared to other buildings at the Tofas factory site, or any short period buildings in the same general area. Why this simple index works very well is not fully understood, but its use certainly appears promising.

*Spectral accelerations*

Since the garage and paint shop building clearly approximates a simple single-degree-of-freedom system the base shear coefficient is just equal to the spectral acceleration divided by the acceleration of gravity. The computed base shear coefficients, at a period of 1.25 s, ranged from a maximum of 0.24 (achieved using several rock motions with Profile A) to a minimum of 0.09 (the lower curve in Figure 15). Analysis with Profile B gave a low value of 0.10. All of these computed ratios exceed the value of this ratio required to cause yield of the structure (0.06) and equal or exceed the ratio for full plastic moment capacity (0.09).

The question of how much damage such analyses would predict is more difficult, since the linear response spectrum no longer applies once a structure yields. Since yielding causes the period of a structure to increase, spectral ordinates at periods larger than the elastic period have considerable influence on the extent of damage to be expected once a structure yields. One possible way to judge the degree of damage is illustrated in Figure 18. Here the shaded areas are bounded by the fundamental elastic period, the spectral acceleration corresponding to yield, and the computed spectra; these areas are one measure of the energy available to deform the structure once it begins to yield. Although the spectral ordinates at  $T = 1.25$  s are very different for Profiles A and B (0.25 vs 0.10), the shaded areas are less dissimilar (11.2 vs 6.7 in/s). When the same rock motion is used for both profiles, the sizes of the shaded areas are in very close agreement. Even though the spectral acceleration for Profile B at the elastic period is relatively small (indeed the acceleration response has a valley at this period) the increased spectral ordinates at higher periods mean significant damage once the structure starts to yield. This type of behaviour has previously been noted in the analysis of damage caused by the Caracas earthquake of 1967.<sup>19</sup> Considering that the garage and paint workshop apparently had relatively little ductility, both spectra shown in Figure 18 would predict that collapse might well occur.

The next question is what the computations would predict concerning damage to other buildings at the Tofas factory. For the service blocks, all of the computations give spectral ordinates of 0.07  $g$ –0.08  $g$ . For

the powerhouse, the ordinates range from 0.13 to 0.22 g. These values are affected but little by the choice of soil profile and soil properties. Similarly, all computations give substantially the same spectral ordinates (about 0.03 g) at the period (about 2.5 s) of the press workshop.

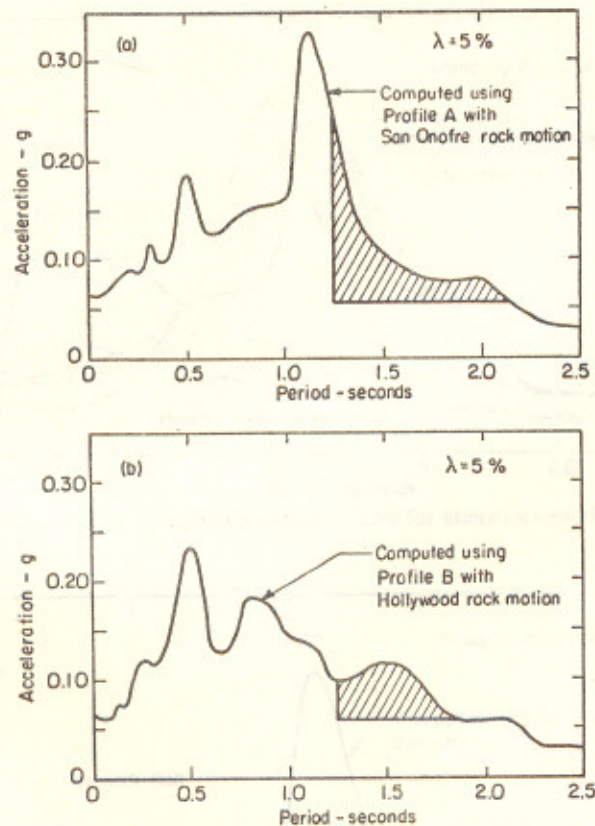


Figure 18. Acceleration response spectra with and without effect of underlying rock

The service blocks and powerhouse were presumably designed for the same base shear coefficient (0.06) as the garage and paint workshop, but the actual yield strength of these buildings is not known. Since there was no damage, if the analyses are correct, the buildings must have considerable extra strength. Such extra strength is not uncommon in many small buildings. Conversely, the press workshop building experienced yielding and damage even though the computed response acceleration was only 0.03 g. Again, the actual yield strength of this building has not been determined.

It is apparent, however, that the damage at the Tofas factory seems to be more closely related to spectral velocities than to spectral accelerations for the known structures.

#### *Prediction of response spectra at Tofas factory site*

It is apparent from the above discussion that, although other reasons may exist for the damage and motion forms observed, the damage which occurred at the Tofas factory site in the Gediz earthquake can be explained adequately on the basis of soil amplification effects and soil-structure response interaction effects during the Gediz earthquake. However, equally important as explaining the observed damage is the question of whether or not it could have been anticipated in advance. Clearly a key aspect of this question is the possibility of predicting the form of the response spectra for both the aftershock and main shock ground surface motions.

The general shapes of response spectra are customarily determined for design purposes either by adopting standard spectral shapes based on statistical studies of past earthquake records or by ground response analyses. The most widely used standard shapes are those proposed by Housner<sup>5</sup> and Newmark and Hall.<sup>8</sup> In Figures 19 and 20, these standard shapes are compared with those computed by ground response analyses

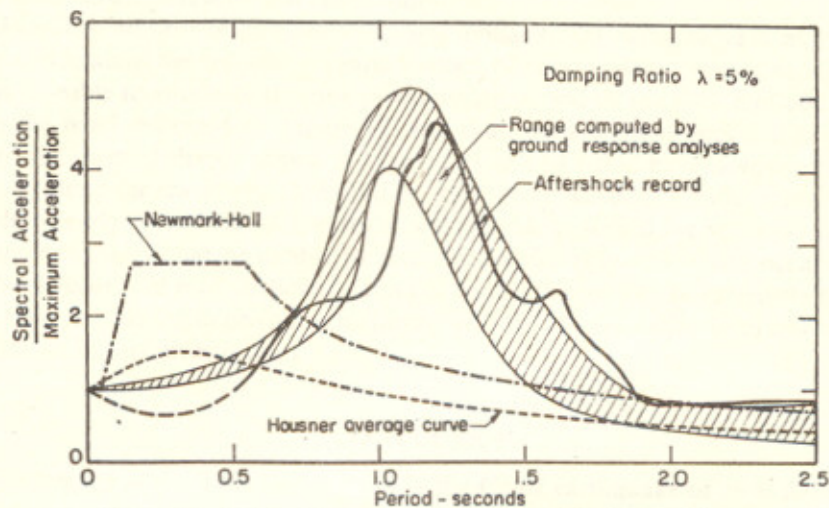


Figure 19. Acceleration response spectra for aftershock motions

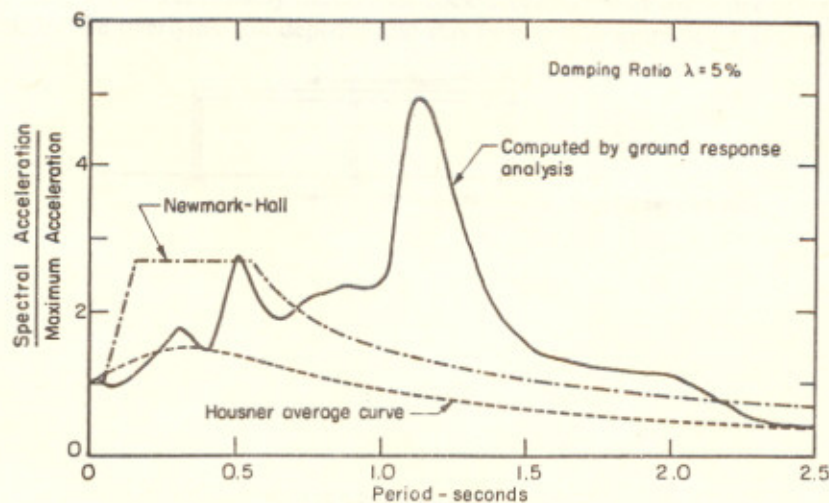


Figure 20. Acceleration response spectra for main shock motions

for this site and with the spectral shape of the aftershock ground motions. It is clear that the spectral shapes of the motions which developed do not conform to the standard shapes which are often used. Of course the standard shapes were intended primarily for use with strong nearby earthquakes and stiffer soil conditions than those at the Bursa site; for such conditions amplification of longer periods by the alluvium would not be expected to be as significant as at Bursa. However, the amplification and response interaction effects at Bursa emphasize that such effects should never be neglected in cases such as that at the Tofas factory site. For such cases it would appear that these effects can be anticipated satisfactorily with the aid of appropriate ground response analyses or through the use of site-dependent spectral shapes.<sup>18</sup> It is also desirable that appropriate safety provisions be incorporated in future earthquake codes<sup>20</sup> to account for the possibility of resonant period interaction effects.

## POTENTIAL USE OF AFTERSHOCK RECORDS

In the analyses described previously, good agreement was obtained between observed response, actual damage patterns and computed ground response for a wide range of rock motion excitations. However, it is possible that some choices of rock motion characteristics, even within the range indicated in Figure 11, could have led to somewhat different results and conclusions.

Since in the absence of records of motions following similar travel paths, there is no reliable basis at the present time for anticipating the frequency characteristics of rock motions within narrow limits, it would normally only be possible to conclude that the predominant period of the rock motions in the main shock would lie within the band indicated by Figure 11, and thus, even if the proper soil profile is known (say Profile A), any of the spectral shapes shown in Figures 13 and 14 could have developed, depending on the geologic conditions along the travel path followed by the seismic waves in moving from the zone of energy release to the site. Clearly there is in such cases a considerable potential for variation in ground response conditions and the designer would be unable to determine which response would occur in any given case. However, aftershock records of rock motions in areas of special interest can be extremely helpful in clarifying questions of this type, in addition to providing an improved basis for evaluating local soil response conditions and characteristics, as in this investigation.

## CONCLUSION

The studies of the damage at the Tofas factory site in the Gediz earthquake of 1970 illustrate the potential effects of soil amplification and resonant period interaction in inducing damaging motions in engineering structures. The probable effects of the earthquake on the garage and paint shop buildings are illustrated schematically in Figure 21. It seems likely that a base rock acceleration of the order of 0.02 g was amplified about 330 per cent by the overlying soil deposit, and this in turn was amplified a further 150-360 per cent

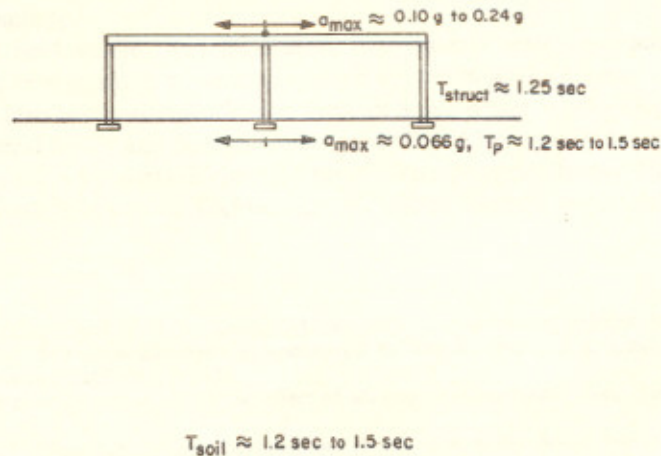


Figure 21. Illustration of amplifying effects of soil-structure system at Tofas factory site

by the garage and paint shop buildings, to the point where major damage occurred. The primary cause of these large amplifications was probably the similarity in values of the predominant period of the base rock motions, the fundamental period of the overlying soil deposit and the fundamental periods of the buildings, all of which were of the order of 1.0–1.25 s.

For other structures in the same area, with different natural periods of vibration, no significant damage occurred because of their much smaller response to the earthquake motions.

The studies also lead to the following conclusions:

1. The damage which developed at the Tofas factory site would not have been anticipated on the basis of the standard spectral shapes often used for design purposes or standard seismic design parameters.
2. The damage patterns could have been anticipated in this case on the basis of soil engineering analyses of site response; although other potential patterns would also be indicated by these procedures, the predictive capability of this method of approach can be useful in determining the potential range of ground response characteristics in many cases.
3. The analysis of soil response and damage at the Tofas factory site provides an example of the usefulness of aftershock motion records in predicting the probable characteristics of ground response during stronger earthquakes. It should be noted in this regard that the rock/soil amplification factor was about 6 during the aftershock and only about 3 during the main shock. Thus some analytical treatment is necessary in translating aftershock records to other excitation levels. In spite of this, aftershock records can do much to clarify uncertainties in soil and rock response characteristics and facilitate interpretation of ground response investigations during earthquakes.

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