GEOTECHNICAL ASPECTS OF THE SPITAK EARTHQUAKE

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INTRODUCTION

In March 1989, the authors visited the regions in Armenia devastated by the December 7, 1988 earthquake. During this visit damages were surveyed with a special focus on the geotechnical aspects of the earthquake. Also, while in Armenia, with the assistance of their Armenian colleagues, the authors retrieved relevant geotechnical data, boring logs, maps and soil samples that are now being utilized in the evaluation of the effects of the Armenian earthquake.

Damages and field observations of interest to geotechnical engineers were extensive, including: ground motion intensities; site effects; liquefaction; retaining structures; pipelines; bridge abutments; embankments; and soil and rock slope failures. The scope of this paper is limited to three of these areas of interest: ground motions, site effects and liquefaction. The paper summarizes the initial evaluations of the authors using presently available geotechnical data.

GROUND MOTIONS

Unfortunately, except for one location, the ground motions within the epicentral region could not be recorded during the earthquake. In Ghoukasian, located 34 kilometers from the new fault that ruptured near Spitak, a strong-motion record with its three components were made by the Institute of Geophysics in Leninakan. Dr. Armen Der Kiureghian of the University of California at Berkeley has been successful in digitizing the two horizontal components of this record. The local soil profile at this station is not well established but it is believed to be rock. Figure 1 shows the acceleration time-history of component 1 of the Ghoukasian record. The record indicates that the peak ground acceleration in Ghoukasian was approximately 0.21 g. The bracketed duration (acceleration > 0.05 g) for this record is about 9 seconds.

In addition to the Ghoukasian record, Dr. Der Kiureghian has obtained a digitized record of the seismograph located in Yerevan approximately 75 kilometers from the fault. The peak ground acceleration of this record is 0.06 g.

It is of interest to compare these limited recordings of the peak ground acceleration with values predicted using the attenuation relationships of Donovan & Bornstein (1978) and Algermissen & Perkins (1976) applicable to the Western region of the United States. Table 1 shows this comparison which indicates quite a good agreement between the recorded (in Ghoukasian and Yerevan) and predicted acceleration data. Also in Table 1 are estimates of the peak ground accelerations on rock in Spitak, Leninakan and Kirovakan computed based on the two attenuation relationships referred to earlier.

During their trip to Armenia, the authors visited the cemeteries in Spitak, Leninakan and Kirovakan and observed the responses of grave markers to the earthquake ground motions. On the hill in Spitak's cemetery, almost all of the grave markers had either toppled or permanently translated relative to their bases. Figure 2 shows typical observations. Of particular interest was that all the markers that were looked at, that had translated but not toppled, also had rotated counterclockwise about their vertical axes from initial positions of facing East to final positions of facing 15° - 30° Northeast as shown schematically in Figure 3. This is attributed to the twisting moment from the skewed horizontal inertial force acting on the blocks while they rocked without toppling. A shaking table test conducted on model blocks confirmed this tendency of blocks to rotate for as long as the direction of the horizontal base motion is not perpendicular to the rocking axis of the block as shown in Figure 4. Based on these model tests it is also concluded that the direction of the axis perpendicular to the faces of the markers in their final positions in the cemeteries is the direction of the predominant motion during the Armenian earthquake. Coincidently, this is nearly perpendicular to the new thrust fault near Spitak. This suggests a strong azimuthal variation of the ground motions during the Armenian earthquake. A similar observation was made by Arnold (1975) investigating the 1971 San Fernando earthquake data.

In Leninakan the cemetery is in the valley on the old lake bed. Except for a few grave markers that had toppled, most showed no movements relative to their granite bases.

In Kirovakan, approximately 20 kilometers from the fault, the cemetery is similar to that in Spitak and most likely founded on rock. In this cemetery, there was surprisingly no evidence of any permanent displacements of the markers. This suggests that rock motions in Kirovakan were significantly smaller than Spitak. At the present, these observations of the responses of the grave markers in the three cemeteries are being evaluated analytically using the mathematical formulation of Tso & Wong (1989) and shaking table tests on model blocks. Table 1 summarizes the initial results of the calculated limits on the peak ground accelerations in the three cemeteries. These estimated ranges for the peak ground acceleration compare well with the values estimated using the attenuation relationships shown in Table 1.

In summary, Spitak, because of its proximity to the fault and the epicenter, must have experienced extremely large ground motions (0.4 - 0.9 g) on rock, which caused the total devastation of the city. In Leninakan, the ground accelerations on rock were significantly smaller than Spitak, most likely about 0.2 g. However a number of grave markers founded on soil did topple indicating that the ground acceleration on soil most likely were larger (0.25 - 0.4g). This poses the question of what were the site effects, if any, in Leninakan, where damage was extensive, and Kirovakan, where damage was comparatively small. The following section addresses this question.

SITE EFFECTS

The influence of soil conditions upon the ground motions specifically in Leninakan, was evaluated analytically using the equation of motion governing propagation of shear waves through soils. The computer program SHAKE was employed to solve for the motions at the ground surface of a typical soil profile using component 1 of the Ghoukasian record as input at outcropping of rock. Figure 5 shows the soil column that was analyzed. The depth to bedrock was assumed to be about 200 meters as was suggested to the authors by their colleagues in Armenia. A 300 meter soil column, analyzed using a larger shear wave velocity for the additional 100 meters to account for increased overburden pressures, yielded very similar conclusions as the 200 meter profile. Also selected analyses showed that the presence of a 6 meter thick tuff at shallow depths had insignificant influences upon the computed ground responses. Hence, all subsequent analyses were made for 200 meters and did not include the tuff layer. The range of values for the shear wave velocities of the subsurface soils were estimated from limited information that is available on the clays in Leninakan. Following the earthquake a team of Japanese scientists conducted shallow cone penetration tests and concluded that the shear wave velocity of the sandy clay was about 250 meters per second. Also, estimates of the shear wave velocities were made based on the strength properties of the clay provided to the authors by Professor Mestchyan ($c = 0.8$ kg/cm^2, $\phi = 10^o$). It is noted that the range of values of the shear wave velocities shown in Figure 5 are comparable with those estimated by Borcherdt (1989). As noted in Figure 5, slightly larger values of shear wave velocities were used for the deeper clays to account for the increased overburden pressure with depth.

In the dynamic site response analyses, component 1 of the Ghoukasian record was scaled to 0.2g as suggested by the results shown in Table 1. In the analyses, strain compatible moduli and damping ratios were calculated using the normalized relationships developed for the Mexico City clays reported by Seed et.al. (1988).

Figure 6 shows the response spectrum of the input rock acceleration of the Ghoukasian record for 5% damping. In Figure 7, this spectrum is normalized with respect to the peak ground acceleration and is compared with spectra established statistically by Seed et. al. (1974) using recorded rock motions. It is noted that the Ghoukasian record is quite consistent with Seed's spectra. The relatively high spectral values in the Ghoukasian record in the period range of 0.8 to 1.3 seconds may be indicative of presence of soil at the site of the seismograph in Ghoukasian.

Figure 8 shows the ground surface response spectra (5% damping) computed using the four combinations of the limits of the shear wave velocities shown in Figure 5. In Figure 9 a similar comparison of these ground surface spectra normalized with respect to the ground acceleration is made with Seed's statistically arrived spectra for deep soil (greater than 250 feet). A reasonably good agreement is observed between the range of computed spectra for ground surface motions in Leninakan and the range established using other ground motions recorded on similar profiles. The fundamental period of the soil column analyzed was computed to range between 2 and 2.5 seconds. This can also be observed by dividing the computed response spectra for ground surface with the spectrum of rock motion. This has the effect of eliminating the influence of the peaks in the input rock spectrum upon the spectra at ground surface. Figure 10 shows the resulting ratios of response spectra which confirm the 2 to 2.5 seconds range of the fundamental period of the Leninakan soil column. It is noted that Borcherdt (1989) reports this identical range (2 to 2.5 seconds) for the fundamental period of the soil profile in Leninakan evaluated based on analysis of aftershock records.

Figure 11 shows a typical acceleration time-history computed from the dynamic site response analysis. It is interesting to note that the duration and the predominant period of the computed ground surface motion is not significantly different than the input record. This indicates that although the soil profile is deep (200 meters) the shear wave velocities are indicative of medium stiff soils and the primary effect is therefore amplification of the rock motion spectrum by about a factor of 1.5 to 2 in the period range of importance (0-1.5 seconds) in Leninakan. The computed short duration of about 10 seconds (Acceleration>0.05g) and a predominant period of about 0.3 to 0.5 seconds of the ground surface motions in Leninakan also explain why lifelines, which are relatively flexible structures, performed remarkably well during the earthquake. Also, from the dynamic response analyses the calculated peak ground accelerations ranged between 0.22g and 0.31g which is in agreement with the range calculated based on the observations of the responses of the grave markers in Leninakan.

The result of this initial evaluation of the influence of the soil conditions upon the ground motions in Leninakan raises the following question: to what extent such possible site effects were responsible for the massive devastation experienced in Leninakan?

From Figure 8 it is apparent that the answer to the above question depends largely upon the building type and its associated natural period. The following explanations are proposed for four types of buildings built in Leninakan.

Stone Buildings. These structures have stone bearing walls, are typically less than 4 stories high and are of older construction that is believed to be of good quality. These buildings are most likely very stiff with periods less than 0.3 seconds. In this period range, the site condition had insignificant influence upon damage and the most likely reason for their relatively good performance even though the spectral acceleration in this range was probably as high as 0.6g is that they had the strength capacity.

Composite Frame and Stone Buildings. These buildings are mostly 5 stories high, built with exterior stone shear walls and a concrete framing cast within the exterior walls and within the interior of the buildings. These buildings probably have fundamental periods of about 0.3 to 0.5 seconds. In this period range the site most likely amplified the elastic spectral accelerations by a factor of 1.5 to 2.0 from a value of 0.6g on rock to as high as 1.2 g. Over 62% of these buildings either collapsed or were heavily damaged (Der Kiureghian, 1989). The site condition for these buildings may have further intensified the damages that they would have experienced if founded on rock.

Precast Frame and Panel Buildings. These buildings are constructed of precast beams, columns and panel floors connected at their joints by welding and then filled with concrete. The buildings were designed for MSK intensity VIII. In addition, allowance was made for reduction in the design lateral forces for ductility. In effect, as Der Kiureghian (1989) states these buildings were designed to resist lateral forces equal to about 2.5 to 5 percent of their weight. For these buildings, with probable fundamental periods between 0.5 to 1.0 seconds, the influence of the site was to amplify the elastic spectrum by a factor as much as 2.0 to spectral acceleration values as large as 0.8 g, thus inflicting the devastation where 95% of these buildings either collapsed or were heavily damaged (Der Kiureghian, 1989).

Precast Panel Buildings. These buildings are constructed with precast wall and floor panels connected to cast-in-place concrete frames. The fundamental period of these buildings is estimated by Der Kiureghian (1989) to be about 0.35 seconds. At this period the maximum amplification is about 1.5 resulting in spectral acceleration values as large as 1.0 g. The excellent performance of these buildings in Leninakan is indicative of their strength capacities.

In summary, in Leninakan the general influence of the soil conditions is that the elastic spectral accelerations were most likely amplified by a factor of 1.0 to 2.0 depending on the period range considered. The effect of this amplification upon the extent of the building damages varies for different building types.

In Kirovakan, which is closer to the fault and the epicenter than Leninakan, building damages were significantly less. Also most of the buildings that collapsed were located in a particular region in the southwestern part of the city. These observations suggest that either buildings in Kirovakan are of better quality - a contention made to the authors during their visit in Armenia - and/or the site conditions are different than those found in Leninakan. For example, the observed better performance of buildings in most of Kirovakan can be explained if the soils in those regions are softer than those encountered in Leninakan. At such sites the spectral accelerations within the period range of interest (less than 1.0 seconds) will be significantly lower than calculated for Leninakan and may even be less than the spectral accelerations on rock (Seed et. al., 1974). Conversely, in Kirovakan in the region where building collapse was intense, if the soils are stiffer or more likely bedrock is shallower, then the site in that region can be responsible for amplification of the ground motion leading to the collapse of the short period buildings as experienced in Leninakan. These explanations are speculative and illustrate the importance of the need for acquisition of reliable field geotechnical data on shear wave velocities and bedrock elevations. Such information is essential in order to be able to explain what happened during the Spitak earthquake and to mitigate damage from future earthquakes.

LIQUEFACTION

During one of the field excursions near the village of Nalband, the authors observed presence of sand boils in the agricultural plains. This evidence that liquefaction did occur during the Armenia earthquake is of particular importance because it does provide an explanation as to why the highway embankment that is founded on these sands collapsed during the earthquake. The embankment was immediately reconstructed since it provided a major link between Spitak and Leninakan. Figure 12 shows the sand boils. A sample of the liquefied sand was retrieved and tested in the geotechnical laboratory at Northeastern University. In Figure 13 the grain size distribution of the liquefied sand is compared with ranges of typical sands that have liquefied in Japan. It is not surprising that the test result confirms the susceptibility of the sand to liquefaction. What is surprising is why, to the authors' knowledge, to date, other evidences of liquefaction have not been identified.

There are many analytical procedures that have been developed to evaluate liquefaction potential.

Yegian and Vitelli (1981) developed a procedure that employs earthquake magnitude and distance to describe the earthquake intensity. This approach is specially useful when reliable measurements of acceleration is not available. Employing this procedure a relationship was established between maximum distance of soils where liquefaction would have been expected during the Armenia earthquake and the density of the sands. Figure 14 shows this relationship together with the farthest distance that liquefaction has been observed for a magnitude 6.8 earthquake (Ambraseys, 1988). The regions shown in Figure 14 are approximate since certain assumptions had to be made regarding the water table and the depth to liquefiable soil. Nevertheless, the results do suggest that if there were loose to medium dense sands within 30 to 40 kilometers of the epicenter, those sands should have liquefied, as was the case at the isolated site that the authors identified. It is likely that there were more evidences of liquefaction along Pambak river near Spitak extending to Kirovakan but were not detected. The railway station in Nalband also collapsed during the earthquake. The cause for the collapse of this embankment has not yet been investigated although liquefaction is a speculative reason.

CONCLUSIONS

The earthquake of December 7,1988 inflicted devastating damages to cities, towns, and villages located within 50 kilometers of the epicenter. Many aspects of the damages are of interest to geotechnical earthquake engineers. This paper addressed the topics of ground motions, site effect and liquefaction. The data available to date is limited yet general conclusions using this data can be made which can help the investigations directed at understanding why so much destruction took place and what can be done to safeguard against a similar calamity. To this end the following conclusion and recommendations are made:

Analyses of the ground motion data and response of grave markers indicate that Donovan & Bornstein (1978) and Algermissen & Perkins (1976) acceleration attenuation models can provide reasonable predictions of the peak ground accelerations on rock during the Armenia earthquake.

Dynamic Response Analyses of soil profiles, using reasonable values of input soil and rock parameters, can provide insight to the relative influence of the site upon building damages, compared to other factors including the intensity of the rock motion, type of building and strength capacity. The site may have had some effect upon the ground motions. This was not necessarily the primary reason for the extensive damage suffered by the different types of buildings. In Kirovakan, for the most part of the city, except where damage was intense, the site may have influenced favorably by deamplifying the intensity of motion within the period range of 0.0 to 0.5 seconds. This is predicated upon the assumption that the soils in Kirovakan are generally softer than in Leninakan. It may be that in future studies of the site effects during the Armenia earthquake the real question that needs to be considered should be why Kirovakan escaped the massive destruction similar to that experienced in Leninakan.

During the earthquake, liquefaction of sands occurred in at least one location and is the most plausible cause for failure of the highway embankment near Nalband.

There are important lessons to be learned from the Armenia earthquake. A thorough evaluation of these geotechnical aspects of the earthquake can provide a rational and reliable basis for the selection of criteria for seismic designs in Armenia.

It is imperative that to provide proper input for such evaluations, geophysical and geotechnical field explorations be performed in selected areas including in the regions where cities and towns are being relocated.

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Table 1: Peak Ground Accelerations (PGA) on Rock

December 7, 1988 Spitak Earthquake

Site	Distance from Source km	Recorded (PGA)	Calculated (PGA)		
			Donovan & Bornstein (1978)	Algermissen &Perkins (1976)	Cemetery Blocks
Ghoukasian	34	0.21g 0.19g	0.16g	0.20g	-
Yerevan	75	0.06g	0.06g	0.06g	-
Spitak	5	-	0.50g	0.55g	0.40-0.90g (rock)
Leninakan	32	-	0.16g	0.20g	0.25-0.40g (soil)
Kirovakan	20	-	0.23g	0.30g	<0.25g (rock)

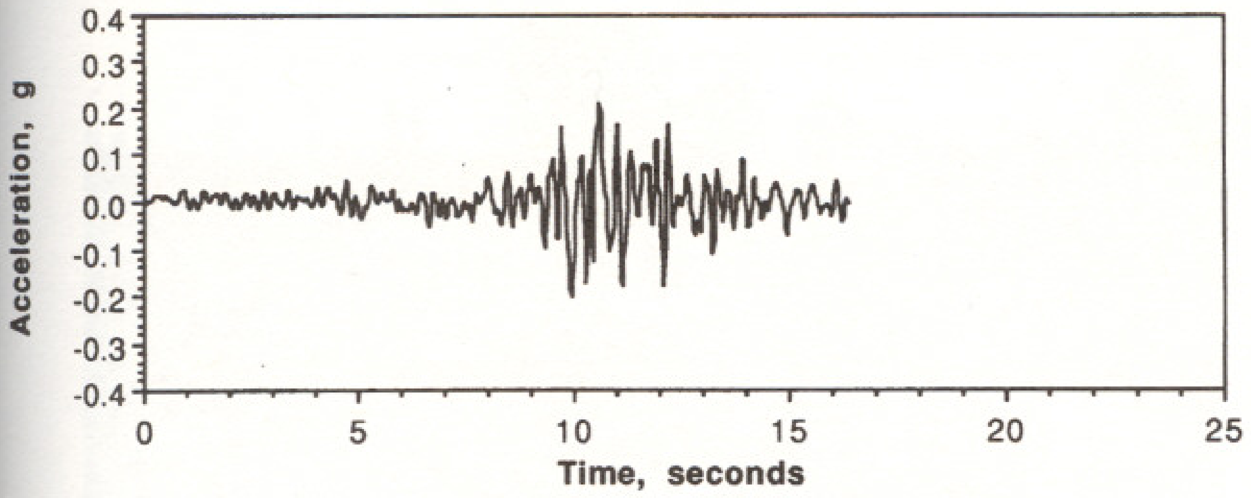

FIGURE 1: Recorded time-history in Ghoukasian (Der Kiureghian, 1989)

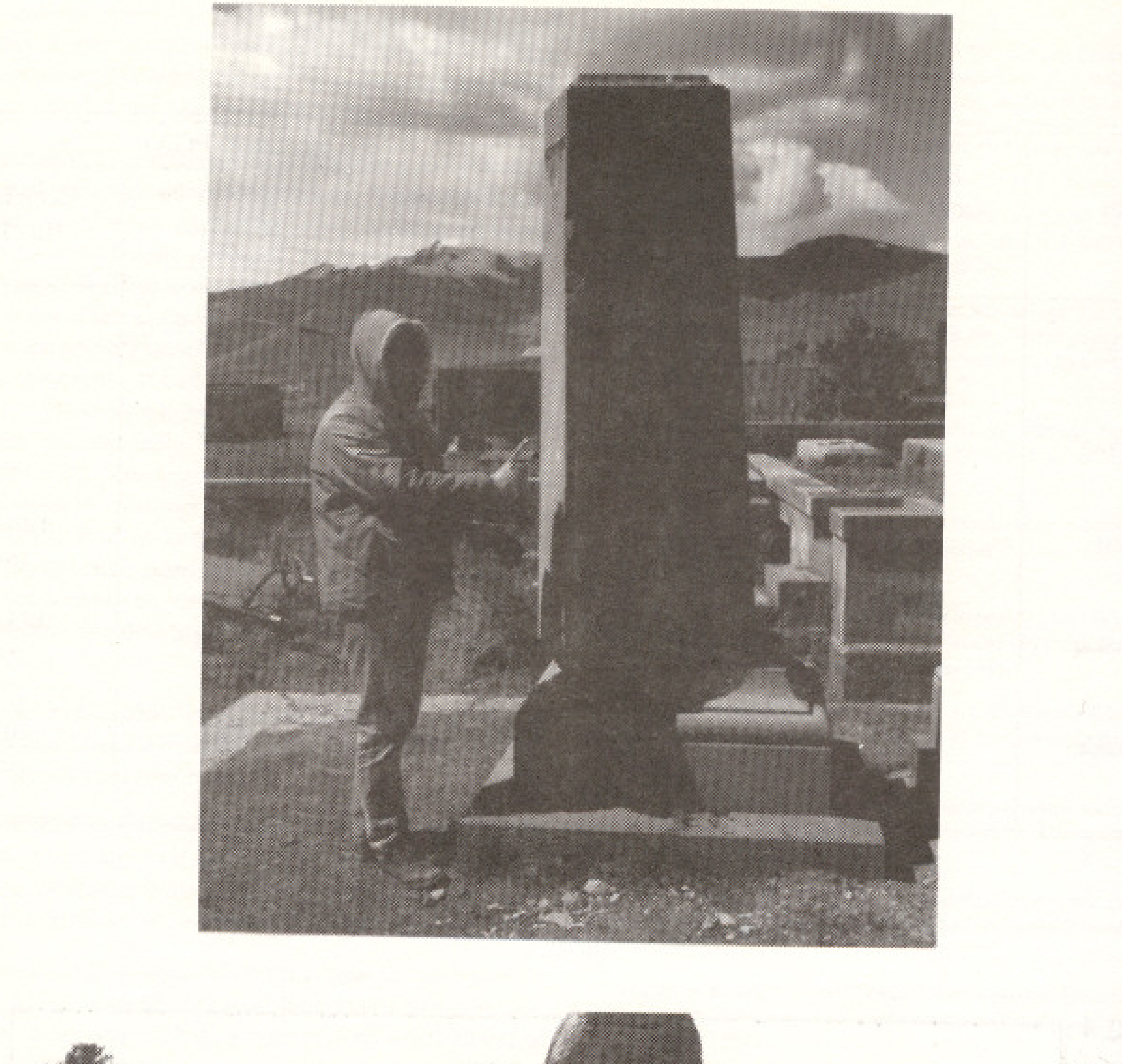

FIGURE 2: Grave markers in Spitak cemetery

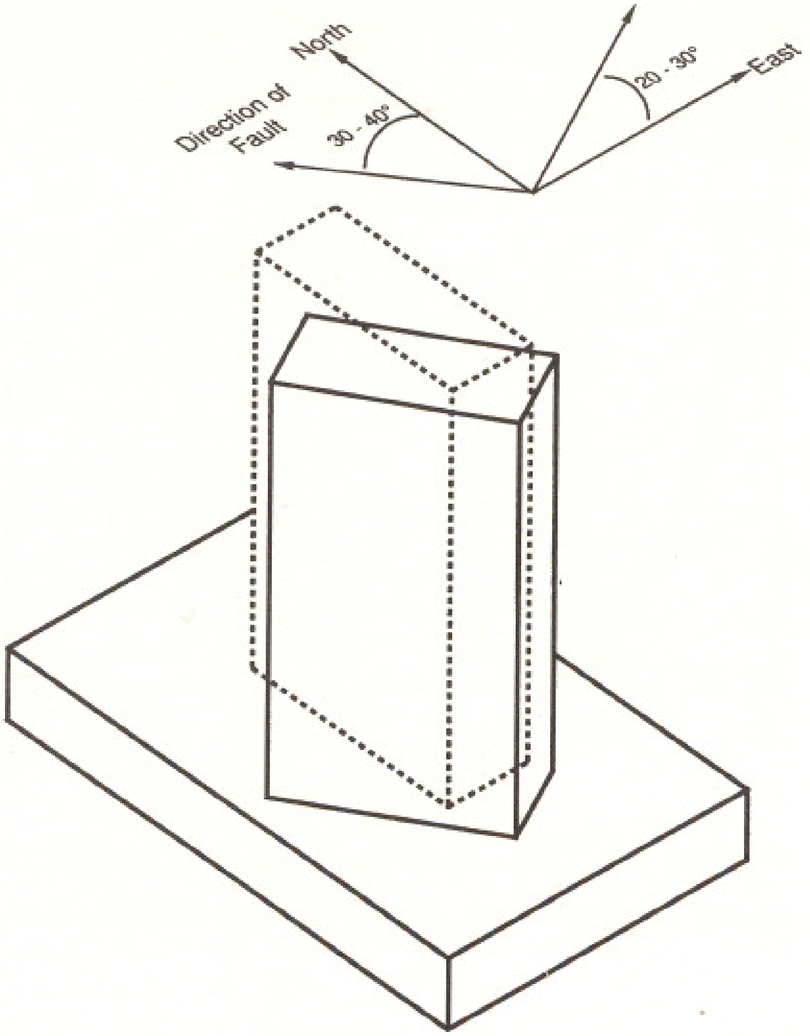

FIGURE 3: Illustration of the response of grave markers in Spitak cemetery

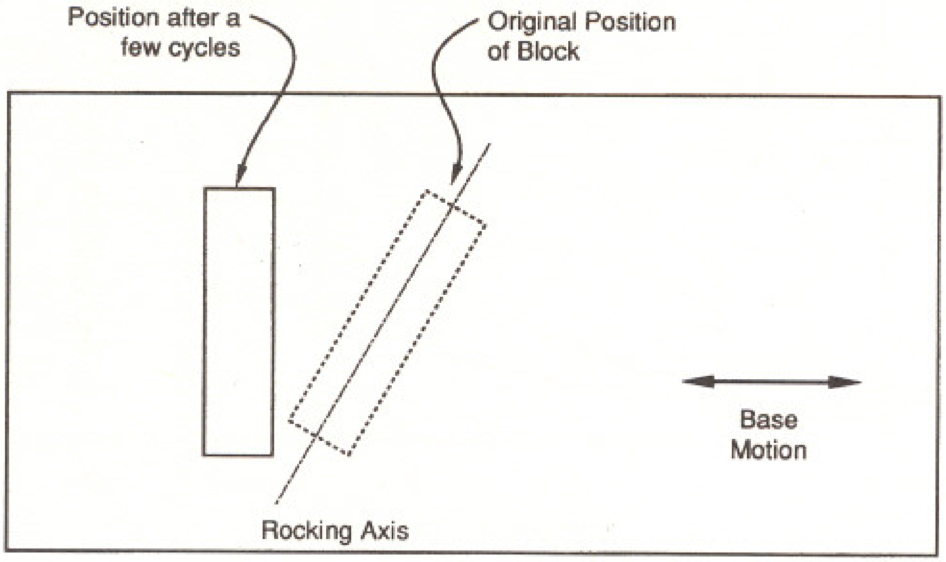

FIGURE 4: Schematic illustration of the shaking test on blocks

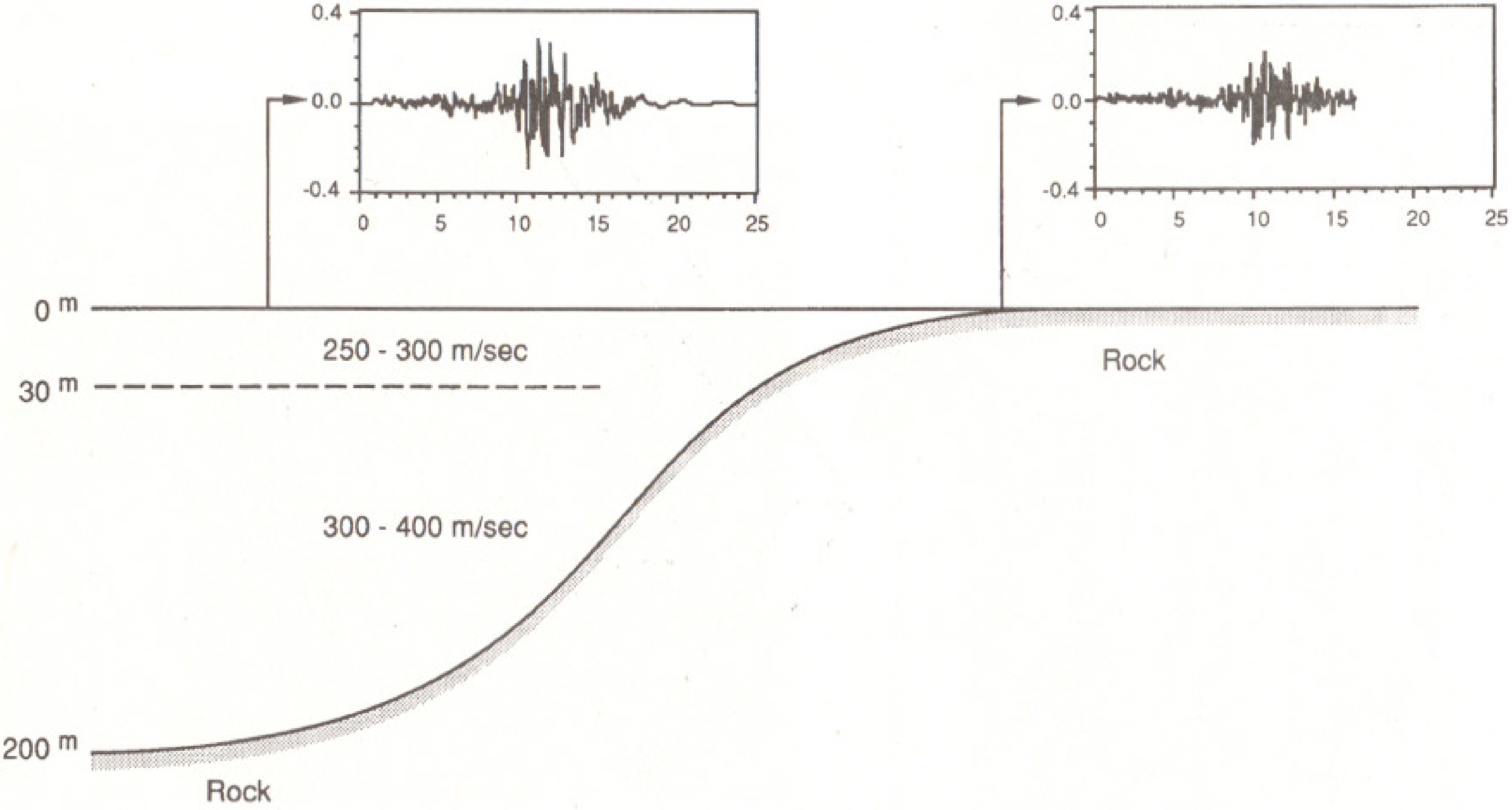

FIGURE 5: Soil profile for Leninakan used in dynamic response analysis

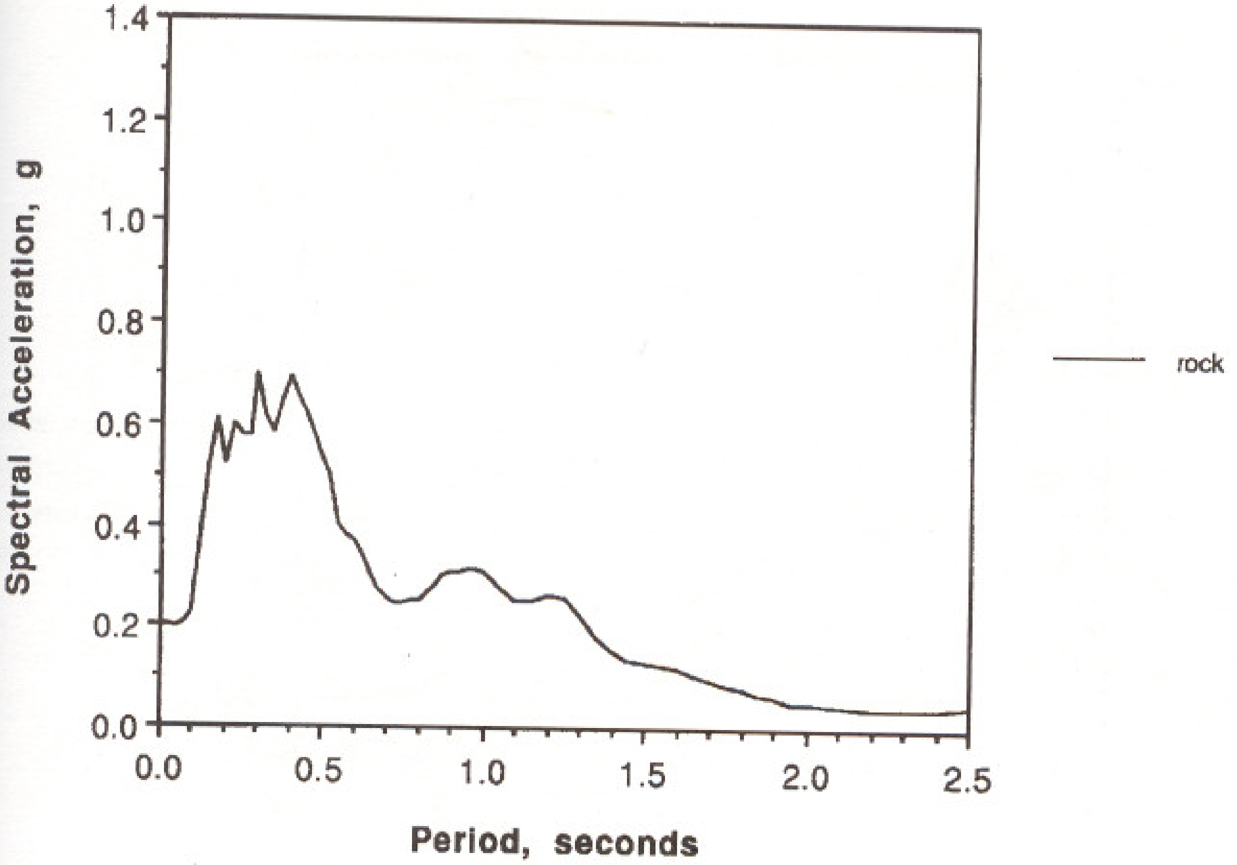

FIGURE 6: Response spectrum of the Ghoukasian record

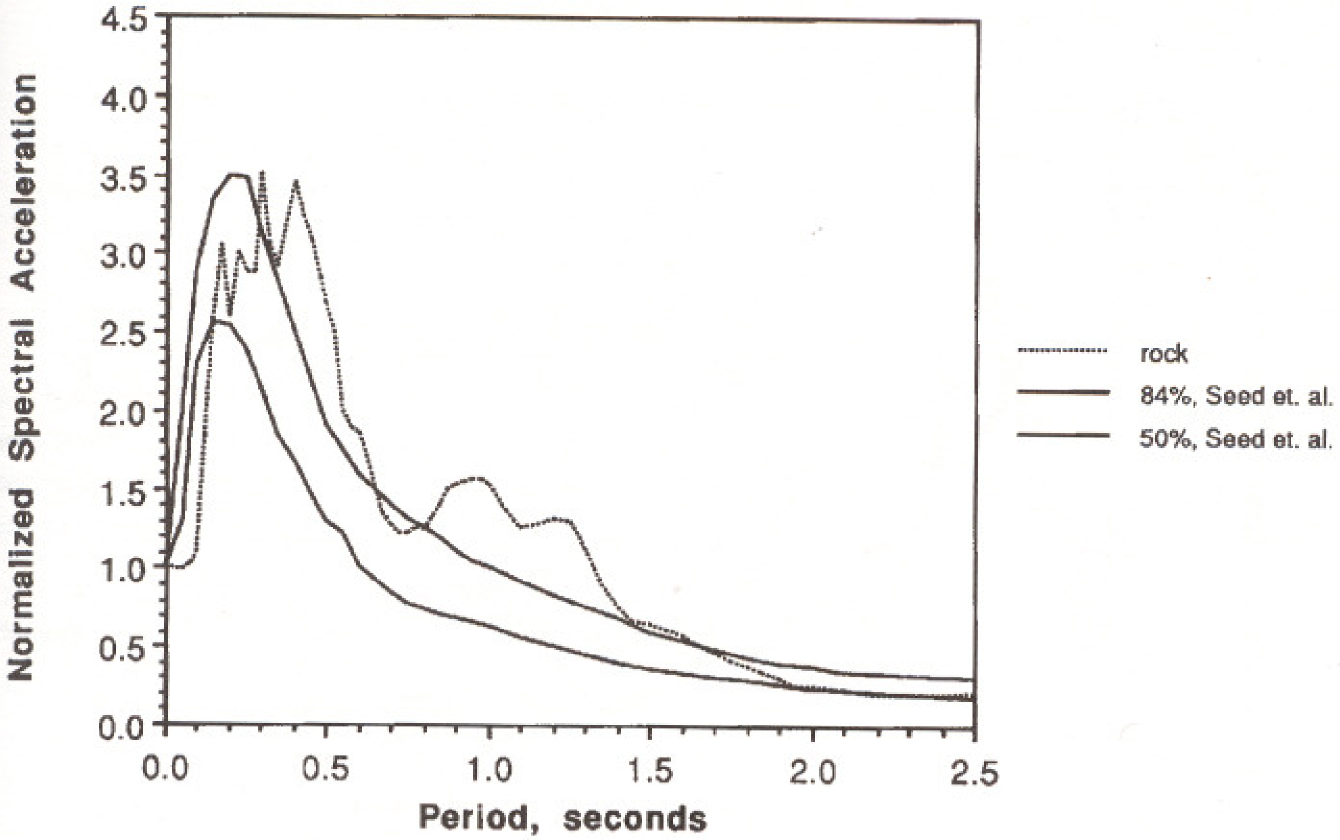

FIGURE 7: Comparison of normalized rock spectrum with Seed et. al's data for rock Deposits

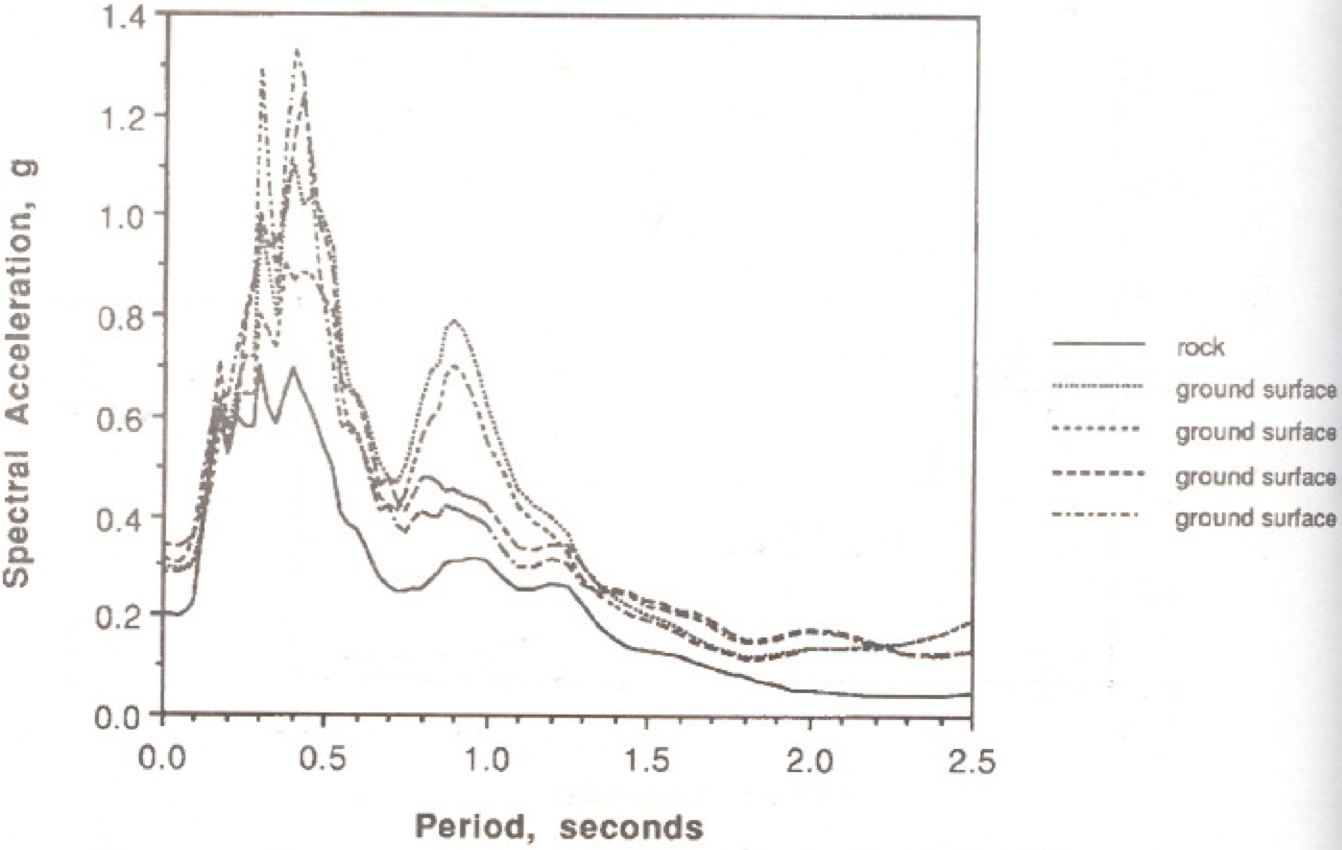

FIGURE 8: Acceleration response spectra (5% damping)

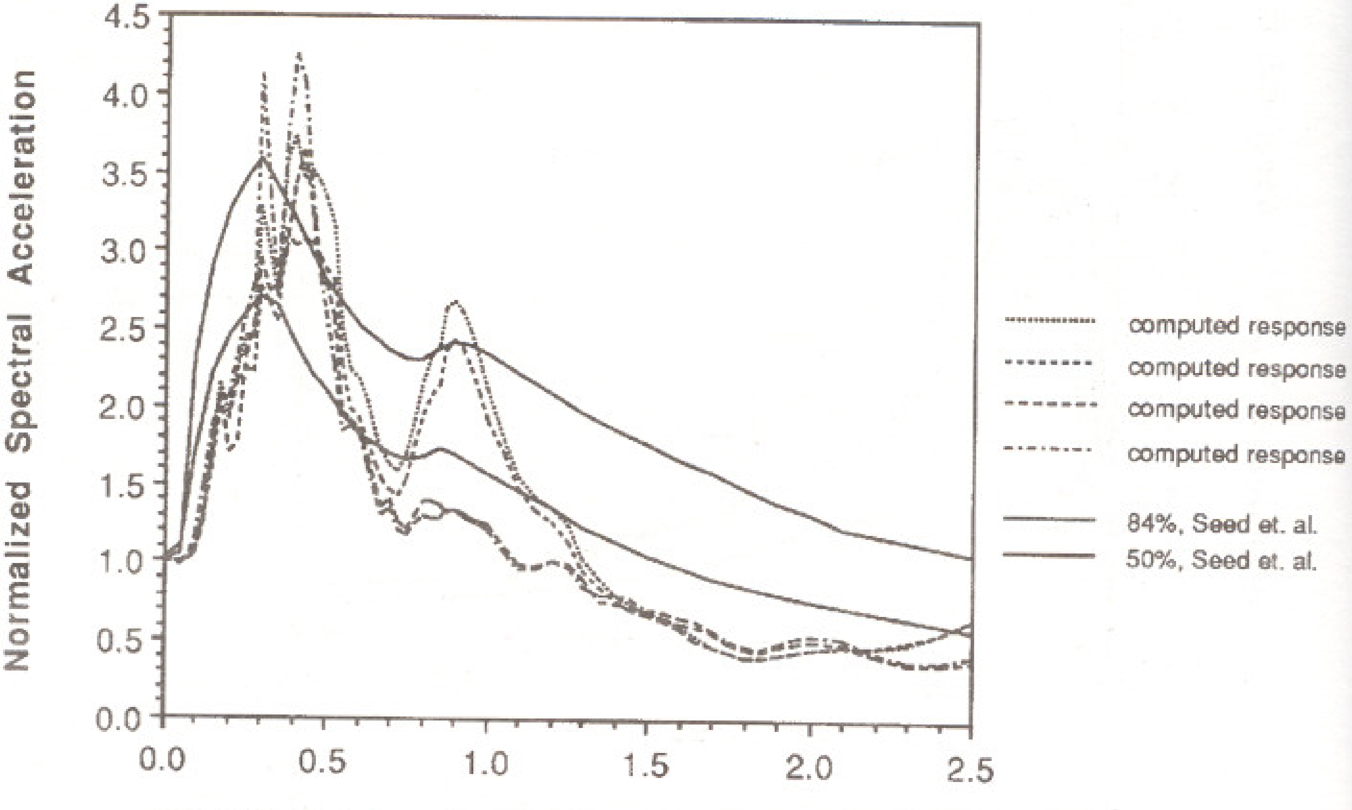

FIGURE 9: Comparison of normalized computed spectra with Seed et. al.'s data for deep deposits

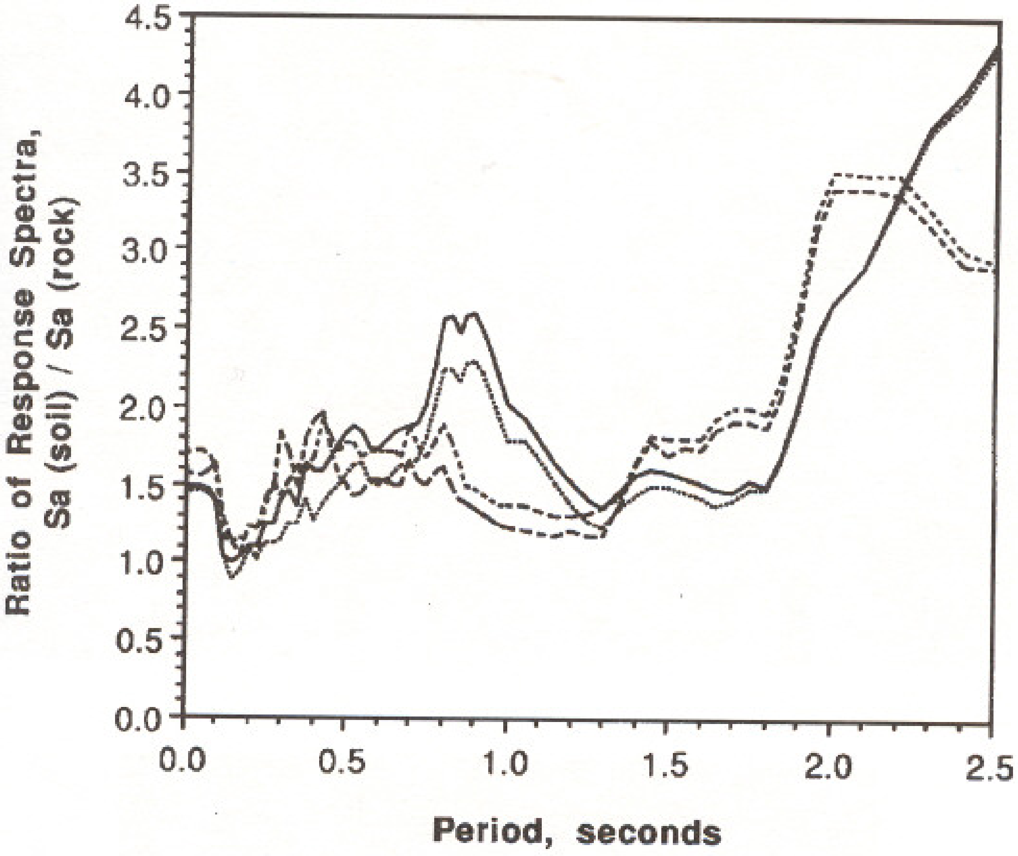

FIGURE 10: Ratio of response spectra (5% daping)

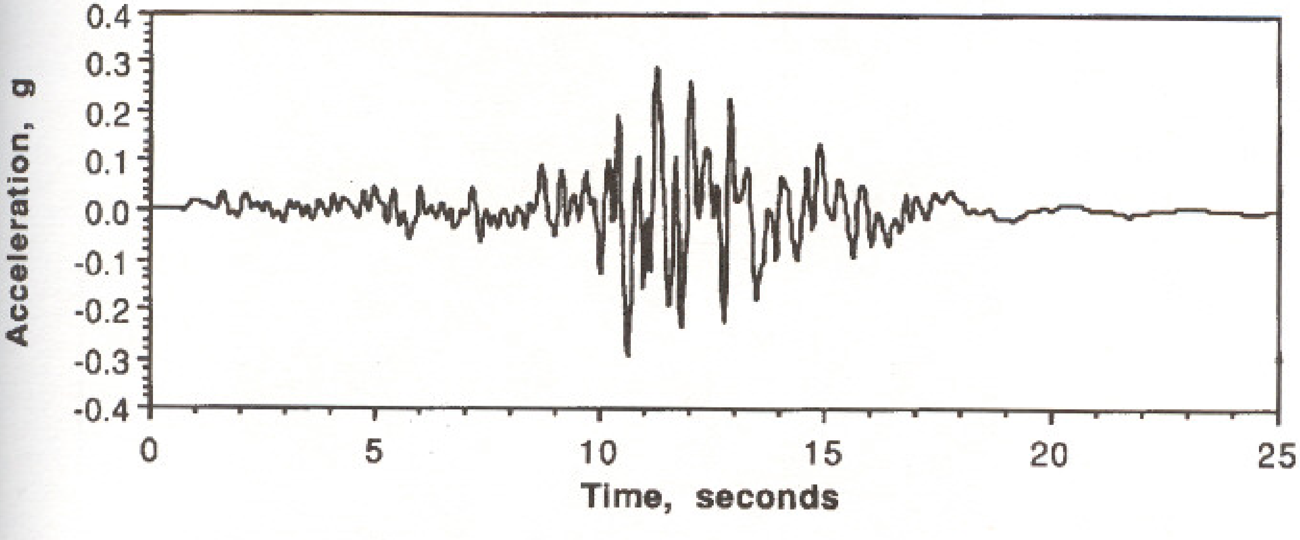

FIGURE 11: Computed time-history for ground surface in Leninakan

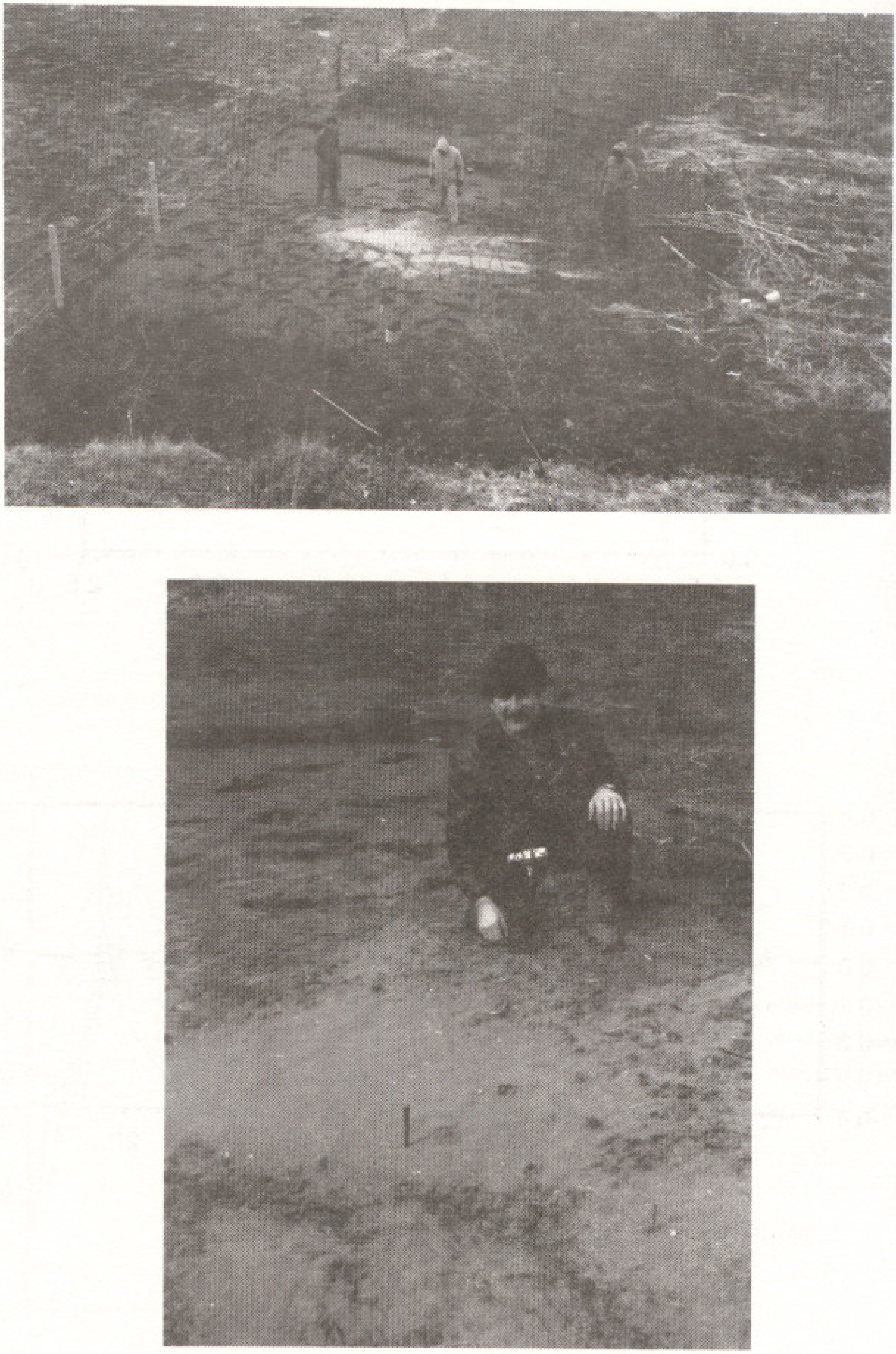

FIGURE 12: Sand boils near Spitak

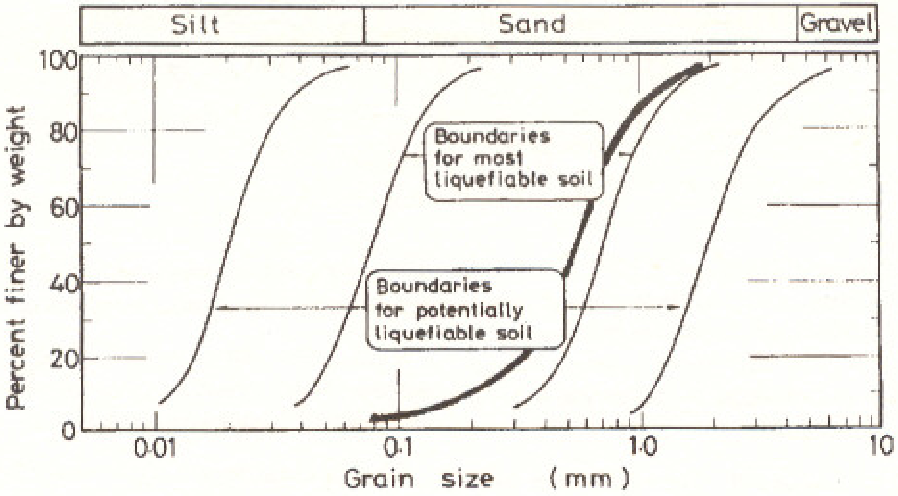

FIGURE 13: Grain size distribution curve of liquefied sand sample from Spitak

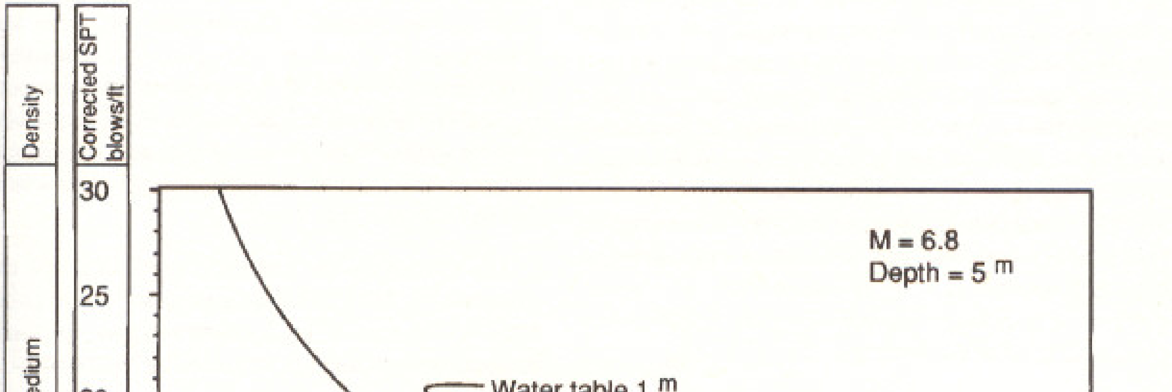

FIGURE 14: Maximum epicentral distance as a function of liquefiable sands