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Local Soil Effects and Liquefaction in the 1978 Thessaloniki Earthquakes

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SYNOPSIS The geotechnical aspects of three earthquakes that struck the city of Thessaloniki, Greece, on May 25, June 20 and July 4, 1978 are presented. General background information on the observed damage, seismic history and geology of the area is followed by detailed description of soil profiles, structural characteristics and accelerograms of ground motions recorded at three sites. Acceleration spectra are then examined and compared in order to assess the degree to which local and regional geology and soil-structure interaction affected the recorded motion. Finally, the possibility of liquefaction having taken place in a 6 m-thick saturated loose layer of silty sand, under the monumental 'White Tower', is investigated. Conclusions are drawn in the light of the current state-of-art of assessing liquefaction potential of soils.

INTRODUCTION

It is well recognized that the damage caused by earthquakes may be influenced in a number of ways by the characteristics of the soils in the affected area. These can be categorized into two broad groups: those in which the soil acts as a vibration transmitter, thereby modifying the intensity, frequency content and spatial distribution of ground shaking and therefore the structural damage; and those where there is a failure of the soil itself (usually in the form of liquefaction of saturated loose sandy layers) resulting in large permanent movements of the ground surface. A number of destructive earthquakes offering evidence of this direct or indirect relationship between soil conditions and earthquake damage have been well documented and widely publicized (e.g., Ohsaki, 1966; Rosenblueth, 1960; Seed, 1969; Seed et al, 1972; Kuribayashi et al, 1975; Tezcan et al, 1977; Tezcan et al, 1978). A related phenomenon, 'soil-structure interaction' implies that structures on soft soil undergo foundation motion which is generally different from the "free-field" motion and may include an important rocking component in addition to the lateral/vertical translational components; this rocking component may be significant for tall structures (ATC, 1978). Evidence on the relative importance of such an interaction has been presented, among others, by Housner (1957), Crouse et al (1975) and Valera et al (1977).

There is still discussion and disagreement regarding the role and significance of these effects. The major question of practical significance is whether a *single design spectrum* is appropriate for all sites and all buildings in an area or *several site-dependent spectra* should rather be specified on the basis of the local geology and building-foundation characteristics. In the literature the existence of such "soil effects" during actual earthquakes has been often either overemphasized or negated. The latter point of view has been primarily based on the lack of conspicuous soil effects in the San Fernando earthquake records. For instance, Crouse (1976) concluded that the data from six sites in Los Angeles offer evidence that soil-structure interaction and local site conditions did not contribute significantly to the character of the recorded motions. On the other hand, Valera et al (1977) found overwhelming evidence of the major importance of soil-structure interaction on the motions recorded at the Humbolt Bay nuclear power plant station in California during the 1975 Ferndale earthquake.

It is evident that a major factor contributing to the continuing debate has been the scarsity of detailed field observations of performance during earthquakes. To meet this apparent need, the paper documents and analyses five accelerograms and response spectra from ground motions recorded at three sites in Thessaloniki during the June 20 and July 4, 1978 earthquakes. Qualitative correlation is attempted between observed concentration of damage in some areas of the city and subsoil conditions. An interesting case study of possible liquefaction is finally presented.

BACKGROUND INFORMATION AND SEISMIC HISTORY

Although the tectonics of the Eastern Mediterranean region "are too complicated to be fully understood" (Papazachos, 1974), the earthquakes that have shaken the city of Thessaloniki in the last two centuries seem to have originated from faults associated with the subduction zone separating the "Saros" and "Rodopean" lithospheric blocks (Fig. 1). Although not very frequent the strong shocks (say, $M \geq 6$) in this area (epicenters portrayed in Fig. 2) seem to occur in groups with respect to time. This rather peculiar "clustering" of strong earthquakes is evident from Table 1, which lists in chronological order the events with magnitude not less than 6 (on the Richter scale) that occurred within a 200 km radius from the city since 1900. For example, three very shallow shocks with magnitudes ranging from 6.2 to 6.9 took place in the period September 26, 1932 to May 11, 1933, originating from an area located about 100 km east of the city. No other major shocks ($M>6$) occurred in the same area until 1978, when the three aforementioned shocks took place.

Thus, the May 24, 1978 earthquake marked the beginning of a new "cluster" of relatively strong shocks that was

LIST OF EARTHQUAKES WITH M ≥ 6 SINCE 1900

Date		Location Lat.	Long.	Focal Depth (km)	M	I_o
1902	July 5	40.75	23.25	11	6.5	IX
1903	Nov. 25	42	20.25	6	6.5	VIII
1904	Apr. 4	41.75	23	15	7	IX-X
	Apr. 4	41.75	23.5	18	7.5	X
	Apr. 19	42	23	8	6	VII-VIII
1905	Oct. 8	41.75	23	19	6.5	VIII
	Nov. 8	40.25	24.5	14	7.5	X
1931	Mar. 7	41.3	22.3	17	6	VIII
	Mar. 8	41.3	22.5	4	6.7	X
1932	Sept. 26	40.5	23.9	6	6.9	IX-X
	Sept. 29	40.9	23.3	13	6.2	VIII
1933	May 11	40.5	23.8	21	6.3	VIII
1947	June 4	40	24	80	6	V
1954	Aug. 3	40.5	25	35	6	--
1970	Apr. 16	40.7	23.4	20	6	VI-VII
1978	June 20	40.8	23.5	22	6.5	VII-VIII

TABLE 1

followed by numerous before and after-shocks. The characteristics of the three major earthquakes, such as the magnitude, focal depth, epicental intensity I_o and peaks of recorded "ground"accelerations at three locations in the city are presented in Table 2.

The villages and towns in the epicentral region suffered wisespread damage corresponding to MKS Intensities VII-VIII, as shown in Fig. 3. Ground failure phenomena were clearly observed in the form of *sand boils* (postulated to be due to liquefaction)and extensive *surface faulting*.

The city of Thessaloniki, located only 20 to 25 km from the three epicenters, suffered very little overall damage, presumably because of the rather strict lateral force requirements of the existing seismic code ; the base shear coefficient for the city varies from 0.06 to 0.12, depending on the quality of the foundation soil but independent of the natural period of the structure. Nonetheless, concentration of heavier damage in some regions of the downtown area was conspicuous. Thus, the shaded area shown on the map of Fig. 4 experienced a total collapse of a nine-story reinforced concrete building, partial collapse of some pre-World War II buildings and severe structural damage of other modern, reinforced concrete buildings.

Since most of the downtown city, near the harbor, is founded on a rather loose deposition of debris extending 5 to 8 meters below ground surface and underlaid by an alluvium deposit of variable thickness and quality (see Figs. 9-10) it is generally believed that the extent of damage and the quality of foundation soil are somehow related.

STRONG GROUND MOTION RECORDS

Four strong accelerograms are available from the earthquakes of June 20 and July 4. They were recorded at the basements of two buildings and a church; their location is shown in Fig. 4 as A, B and C, respectively. Only the accelerograph of building A had been installed

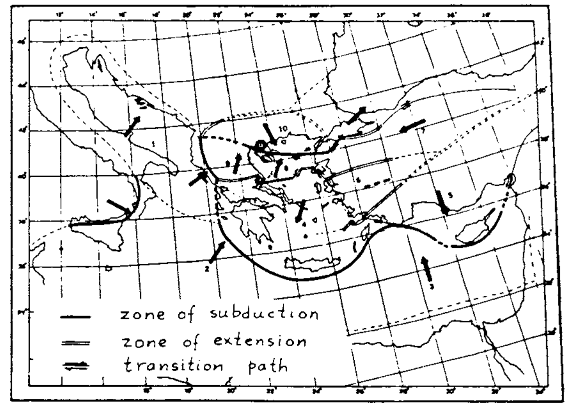


Fig. 1

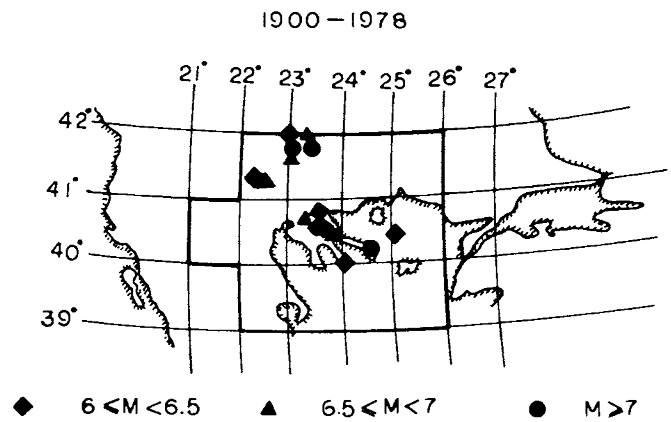


Fig. 2

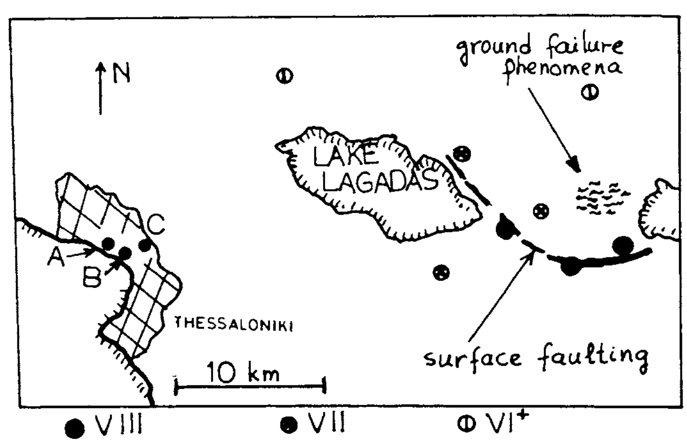


Fig. 3

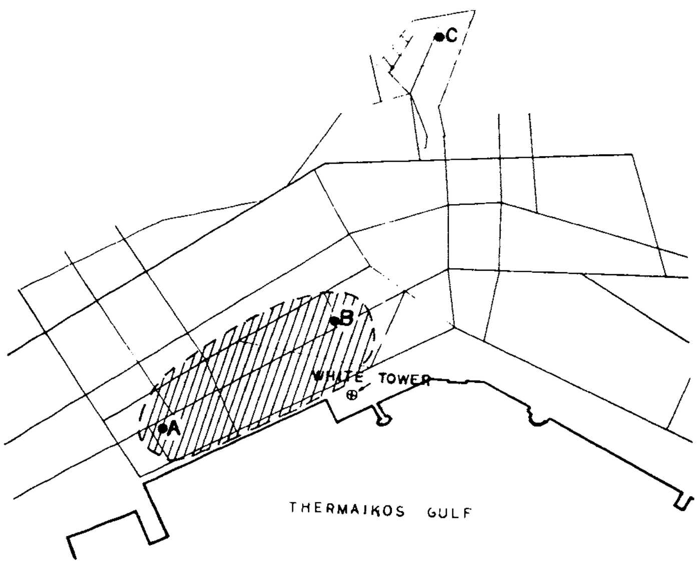


Fig. 4. Map of Thessaloniki showing locations of sites of interest

prior to the June 20th event and, thus, recorded the ground motion during both earthquakes. The accelerographs of Buildings B and C, installed by the Institute of Earthquake Engineering and Engineering Seismology of the University "Kiril & Metodig" of Skopje, Yugoslavia, following the strong earthquake of June 20, recorded only the July 4 event. Table 2 lists the available records and Figs. 5-7 depict the two horizontal components of each accelerogram. The soil profiles at the three sites and the characteristics of structures A, B and C are described next.

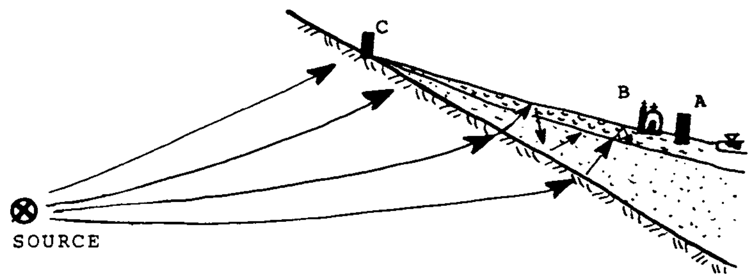


Fig. 8 Schematic Geologic Section

1978 EARTHQUAKE RECORDS WITH PEAK ACCELERATIONS					
Date of event			25 May	20 June	4 July
Magnitude			5.8	6.5	5.0
Focal depth (km)			10	22	—
Epicentral Intensity			V -VI	VIII	VI -VII
RECORDS	A	E - W	-	13.95*	10.50
		N - S	-	15.20	12.33
		U - D	-	13.9	5.50
	B	E - W	-	-	24.06
		N - S	-	-	16.88
		U - D	-	-	9.66
	C	E - W	-	-	10.59
		N - S	-	-	10.98
		U - D	-	-	4.37
* *all accelerations are in percent of g*					

TABLE 2

TOPOGRAPHY AND SOIL PROFILES

The city of Thessaloniki is built along the northern coast of Thermaikos Gulf extending amphitheatrically on hills about 200 m tall, 2 km inland. Fig. 8 shows the topography of the city and the epicentral region and indicates the location of the three sites of interest.

Of these, A and B are in the downtown area, a few hundred meters from the city harbor. The exact soil profile of the two sites is not known (no geotechnical investigation appears to have ever been made). However, fortunately enough, soil conditions have been explored at several nearby sites, located within about 100 m from A and B. By combining information from these borings, the profiles shown in Figures 9 and 10 have been "constructed"; they represent our best estimates of the actual soil profiles at A and B.

Overall, the two profiles are not very different from each other. They, basically, consist of several layers of silty sand and sandy clay of a total thickness 10-15 meters. These are underlain by a stiffer deposit of marly clay and overlain by several meters of debris and compacted fill. The depth to bedrock could not be determined from the actual borings, which were only extended to a depth of about 25 m. Due to the very steep slope of the bedrock, it is believed that the thickness of the soil deposits exceeds 100 m at near-harbor sites like A and B. However, stiff rock-like soil is encountered at much shallower depths.

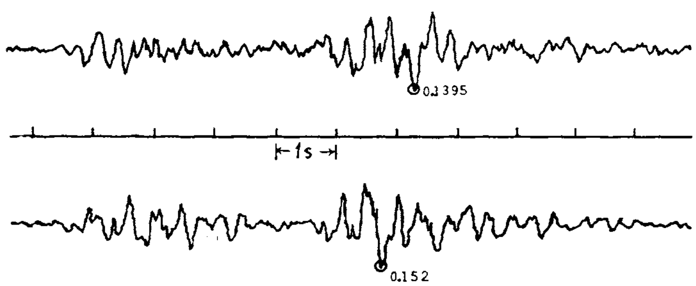
The building of site C is founded on rock outcropping; consequently, no profile is presented.

DESCRIPTION OF STRUCTURES

Only the structures at sites A and B are described herein, assuming that no soil-structure interaction effects have influenced the motion recorded at the basement of 26 Gregoriou Auxentiou Street building, at site C, which is founded on bedrock.

Building A is a 10-story, 42 m-high reinforced concrete hotel founded at an elevation of about 6 m below the surface. Its foundation consists of combined footings. Resistance against lateral forces is provided primarily by 20-40 cm thick reinforced concrete shear walls and additionally by beam-column frames. Fig. 11 shows the structural framing system of the building in plan and elevation and the plan of the foundation. The strong motion accelerograph on which the two motions were recorded had been installed in the basement, at the location indicated in this figure. It is also noted that additional stiffness against lateral forces is provided (at least during not very strong shaking) by the heavy partition brick-walls. Using the empirical formulae recommended by Tassios & Gazetas (1979) for buildings of this type, the "effective" fundamental period of the hotel during the two earthquakes is estimated at about 0.70 to 0.90 seconds.

Structure B is the Greek Orthodox Church of Agios Konstantinos and Elene. It is located near the harbor



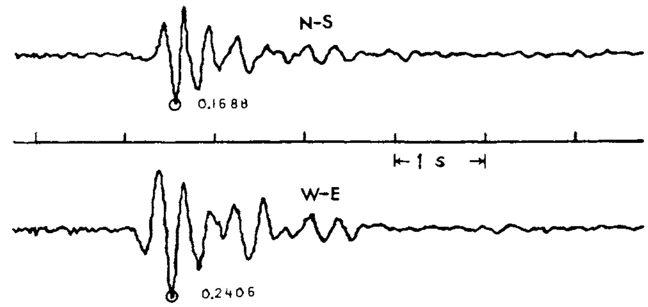


Fig. 6 Accelerogram at site B

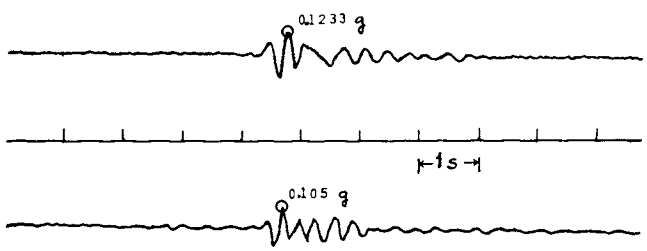


Fig. 5 Accelerogram at site A

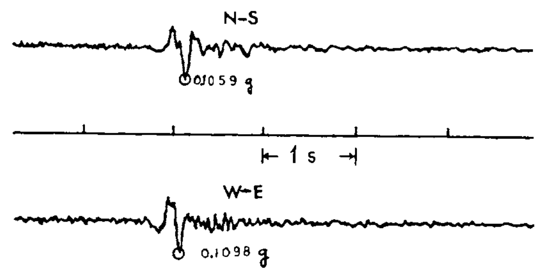
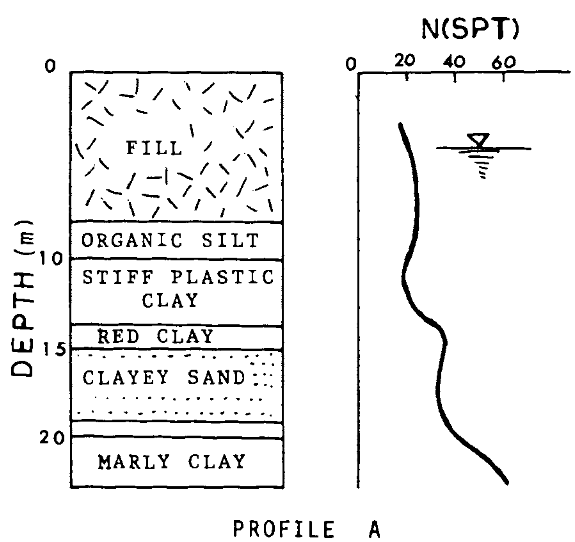


Fig. 7 Accelerogram at site C



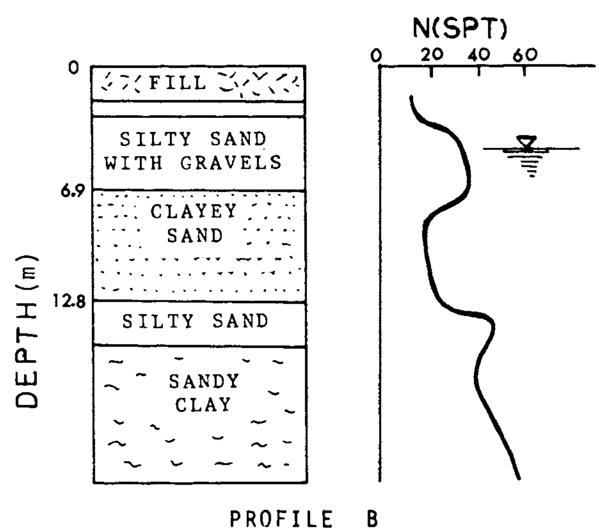


Fig. 9 Soil Profile at site A

Fig. 10 Soil Profile at site B

about 1 km away from building A, in the most heavily affected area by the shocks. Fig. 12 portrays the plan of the main floor and a vertical cross-section of the church. It is evidently a very light structure, compared with the massive building A. It is supported on spread footings and its basement, in which the accelograph had been installed, extends to a depth of 2.50 m beneath the surface and is separated from the ground through a light, 15 cm-thick floating slab. Notice also the large distances between the supporting columns. Clearly such a structure can only exert a minor influence on basement motions; thereby, as a first approximation, soil-structure interaction effects on the accelerogram of July 4, 1978 can be neglected.

ANALYSIS OF RECORDS: GEOTECHNICAL ASPECTS

Characteristics of Accelerograms and Comparison

The NS and EW components of the four strong motion records have been sketched in Figs. 5-7; the vertical motions are omitted for brevity, but have also been studied. Table 2 lists some characteristics of the two earthquakes and the four accelerograms. The key features of these records are briefly discussed next.

The motion at A during the June 20 earthquake (hereafter denoted by A1) consists mainly of two groups of "waves" with periods in the range of 0.25 and 0.50 seconds. The two groups seem to be separated by a "quiescent" interval of approx. 3 sec., a peculiarity attributed to the source mechanism of the earthquake. The horizontal acceleration reaches a peak of about 0.16 g in the second group of waves. The vertical motion is much richer in high frequency components and has a peak acceleration of about 0.13 g. The duration of strong shaking is about 9 seconds.

The July 4 record at A (denoted by A2) is much shorter (duration of strong shaking ≈ 3 seconds) and consists of only one group of "waves". The intensity of motion is apparently quite high for an M ≈ 5 earthquake at R = 25 km; the peak acceleration is about 0.13 g, not very different from the peak of the A1 motion. The "effective" period of the horizontal motions is of the order of 0.30 sec. Much greater participation of high frequencies appears in the vertical component, whose peak reaches only 0.055g, approximately.

The motion B2, recorded in the aforementioned church during the July 4 earthquake, shows a peak acceleration of 0.241 g, the largest peak of all the recorded motions. Otherwise, this record bears a great similarity with the A2 record: duration ≈ 3 seconds, average "effective" period ≈ 0.30 seconds.

The record C2, of the basement motion of the 26 Gregoriou Auxentiou building during the July 4 event, does not share many common features with records A2 and C2. It is much shorter in duration (≈ 1.5 sec), richer in high frequencies and exhibits the smallest peaks in all three components of the recorded acceleration. The largest peak is ≈ 0.11 g. Note that site C is closer to the epicenter than sites A and B (R_C ≈ 23 km vs. R_A ≈ R_B ≈ 25 km) but, as previously mentioned, A and B are underlain by deep alluvium, whereas C is on outcropping rock.

Comparison of Response Spectra

Response spectra are known to convey in a simple graphical form the most meaningful information about a ground motion; i.e., its effect on simple one-dof oscillators. Although such spectra tend to suppress detailed information in the higher frequency range, they are

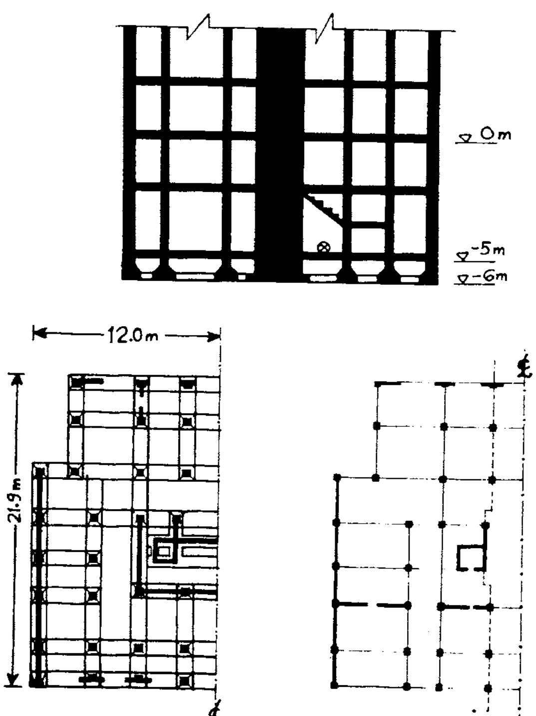


Fig. 11 Structure A

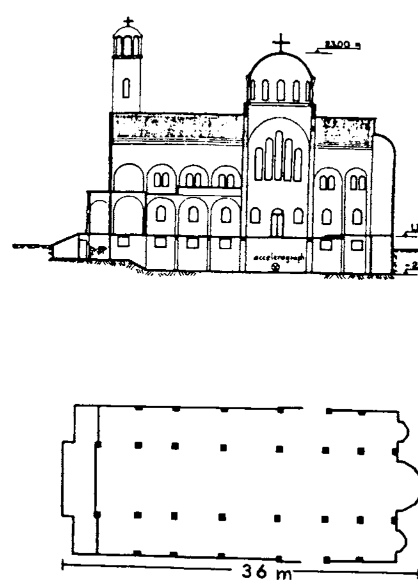


Fig. 12 Structure B

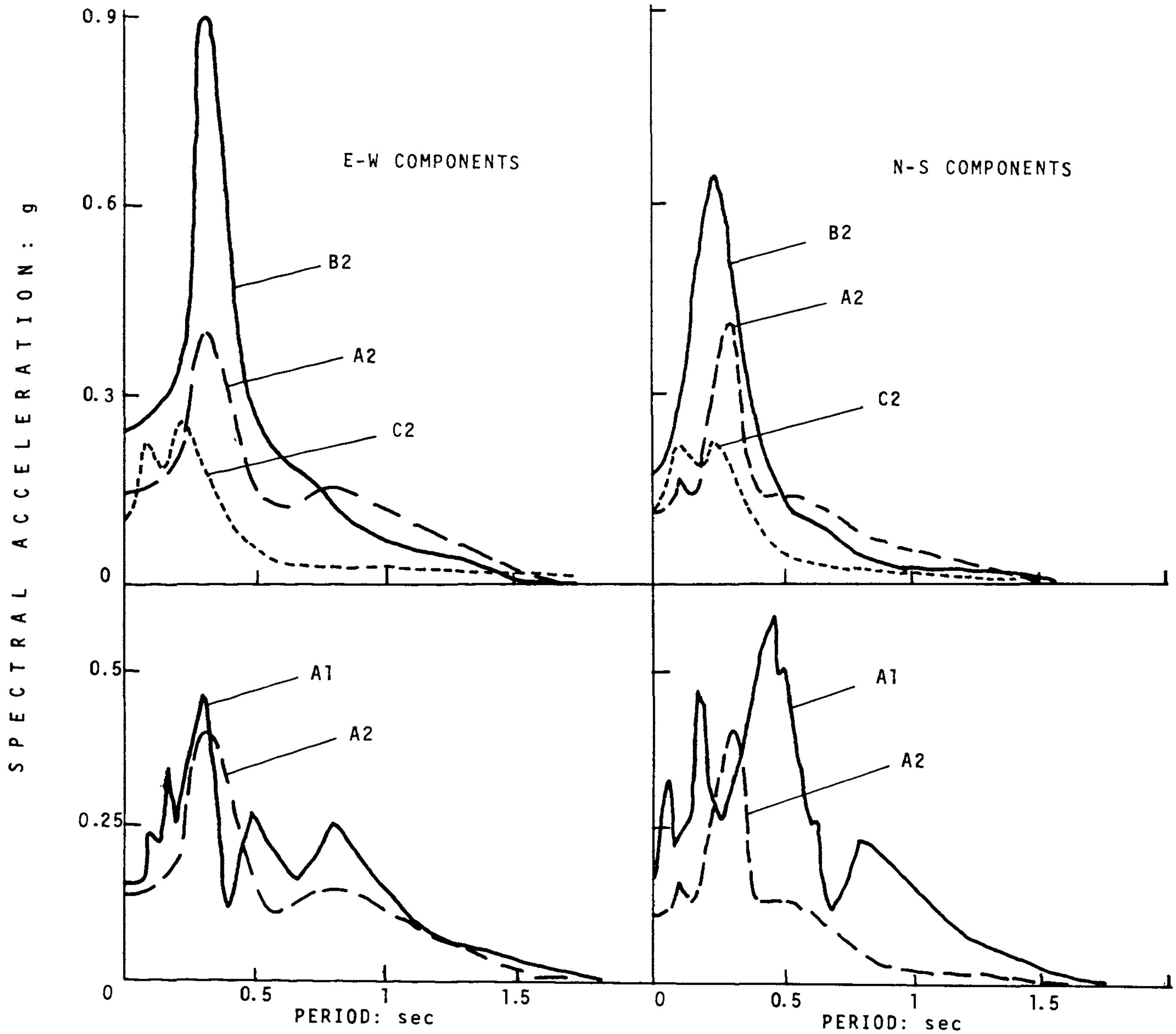


Fig. 13 Comparison of response spectra of horizontal motions

excellent tools in describing the seismic threat at a site and in comparing the characteristics of ground motion records.

The 5 per cent damped acceleration spectra, $S_a(T)$, for the NS and EW components of the four recorded motions are shown in Fig. 13. For clarity of the presentation and easier comparison Fig. 13a plots the spectra of the records from the July 4 earthquake, i.e. A2, B2 and C2, while Fig. 13b shows the spectra of the hotel basement motions during both events, i.e. A1 and A2 The following conclusions may be drawn regarding the influence of soil-structure interaction and the effect of local soil conditions and topography on the recorded motions:

Soil-Structure Interaction

It is true that to ascertain with confidence the significance of soil-structure interaction in modifying a basement motion one must know the free-field motion at the particular site. A comparison between basement and free field motions would then determine the relative importance of interaction, primarily responsible for whatever differences are observed. In our case such records are not available. Nonetheless, B2 can be considered as a reasonable approximation to a free-field motion at site B, for the reasons stated previously (light superstructure, very flexible "floating" basement slab). Moreover, notice that sites A and B have the same epicentral distances and are underlain by alluvial soils of similar overall characteristics, as discussed previously (Figs. 9 and 10). In addition, since the two sites are very close to each other and the harbor, their topography is very similar (see Fig. 8). Therefore motions A2 and B2 may be considered as being the hotel 'basement' and 'free field' motions and their large differences are primarily attributed to soil-structure interaction effects.

In particular, it should be noticed on Fig. 13a that in

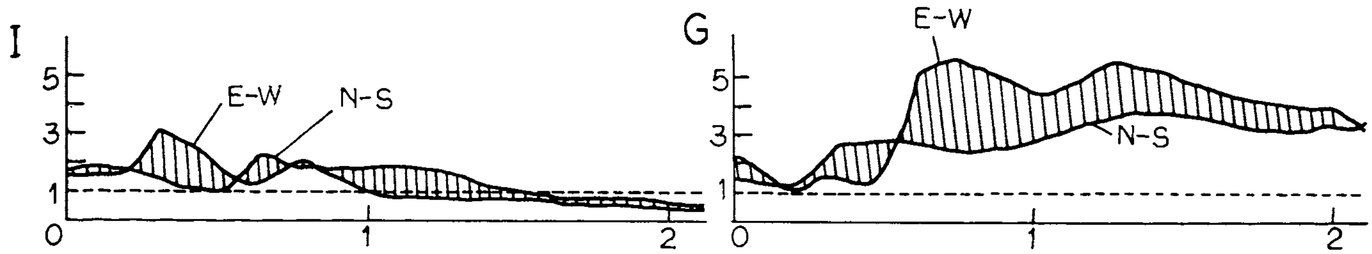


Fig. 14 Ratios of response spectra

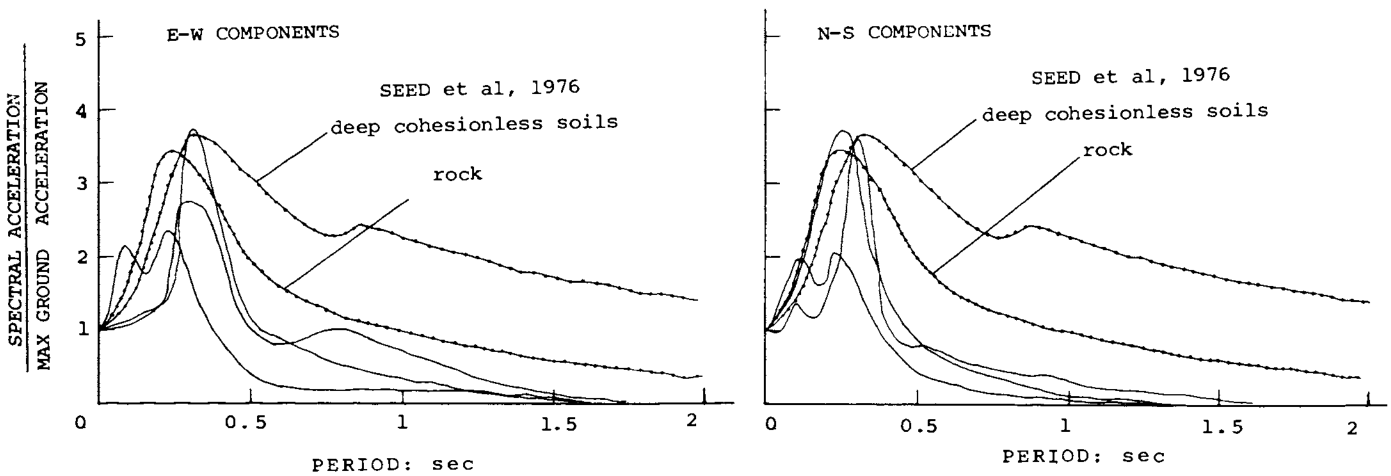


Fig. 15 Comparison of recorded spectra with the 85% percentile site-dependent design spectra of Seed et al, 1976.

the low and medium period range, motion B2 leads to 1.5 to 2 times larger spectral acceleration values than motion A2 does - possibly the result of suppression of high frequencies by the massive structure at site A. Similarly, peak accelerations at the two sites are at a ratio 1.58. In the higher period range ($T \geq 0.6$ sec) the above trend is reversed with the A2 motion resulting in higher spectral values. Notice, moreover, that the A2 spectra exhibit a second major peak at $T \approx 0.60$ - 0.80 seconds, not seen in spectra B2. Recalling that the fundamental period of the building lies also in this period range, one may possibly contribute this peak to structural resonance.

Thus, soil-structure interaction seems to have played an appreciable role in modifying the basement motion of the hotel building. However, on the basis of the available information alone it is not possible to quantitatively assess the exact degree of soil-structure interference. Nor is it possible to completely rule out that soil filtering or other factors have also contributed, to some extent, to the observed differences. Nevertheless, some participation of interaction is unquestionable.

Local Soil Conditions - Topographic/geologic features

The great discrepancies (in intensity, frequency content and duration) between the motions B2 and C2, as evidenced both in the accelerograms (Figs. 6-7) and the response spectra (Fig. 13), can possibly be attributed to different epicentral distances, soil-structure interaction,influence of local soil conditions and topographic geologic features.

Since site C is closer to the epicenter, one would expect there stronger, not much weaker, shaking compared with that at site B. Therefore, different epicentral distances could not possibly explain the observed differences. Soil-structure interaction should also be excluded from consideration for the reasons discussed in preceding sections. It appears that both soil conditions and the topography/geology of the region could have influenced the motions at sites B and C.

Indeed, in site B sedimentary rock is more than 30-50 m beneath the surface and is overlain by softer alluvial layers of various compositions and properties, as discussed previously. It is, thus, quite likely that filtering of seismic waves through the soil layers influenced the resulting surface motion at B. Site C is on an outcropping rock and no similar filtering could have taken place. An examination of the two response spectra shows that especially low frequencies are significantly amplified in B2 relative to C2; clearly resonance phenomena in the soil deposit at B have had at least some contribution to such an amplification.

Topographic and geologic features such as hills and alluvial valleys are known to cause changes in amplitudes of traversing waves. There may be focusing and magnification at some locations on the surface and amplitude reduction at others. In particular it has been observed (Wojcik, 1979) that the existence of a rock-alluvium interface dipping in the direction of incoming seismic waves may lead to "trapping" of wave energy and magnification of motion on the surface of the alluvium. For instance, the heavy destruction of Scopje, Yugoslavia, in the 1963 earthquake is attributed to such a geology (Poceski, 1969). As Fig. 8 shows schematically, similar "trapping" of wave energy is quite likely to have occurred in Thessaloniki. Notice in this figure the location of sites B and C; evidently site B (as well as site A) would "attract" much of the "trapped" wave energy, while site C would not be influenced by the phenomenon.

In conclusion, although reliable quantitative analyses of the differences in motions between sites B and C is not quite possible with the existing data, there is ample evidence that *characteristics of local soil deposits and topographic/geologic features share the responsibility for the observed much stronger shaking in the downtown near the harbor area.* The heavier structural damage in this part of the city (Fig. 4) lends further support to this argument.

Summary - Design Considerations

Fig. 14 summarizes the differences in response spectra from the ground motions. Fig. 14a plots as a function of period the ratio

$$I = \frac{S_a \text{ at B}}{S_a \text{ at A}}$$

which offers an indication of soil-structure interaction effects and Fig. 14b plots the ratio

$$G = \frac{S_a \text{ at B}}{S_a \text{ at C}}$$

which indicates the combined effect of local soil conditions and geology/topography of the region.

From a practical viewpoint, it is interesting to compare the spectral shapes of the recorded motions with the design shapes recommended by Seed et al, 1976, for different site conditions. Fig. 15 portrays the comparison. The Seed et al spectra corresponding to the upper 84% percentile have been used and their "performance" is found to be quite satisfactory: their "deep cohesionless soils" and "rock" curves seem to envelope the recorded spectra.

LIQUEFACTION UNDER THE WHITE TOWER OF THESSALONIKI

This section investigates the possibility that a loose, saturated layer of silty sand did liquefy during the 1978 earthquakes. This layer exists in the subsoil of the monumental White Tower of Thessaloniki, as revealed by two separate geotechnical explorations of the site, in connection with an underground construction nearby. A typical soil profile of the site is shown in Fig. 8. The silty sand layer extends approximately from 6 to 12 meters below the ground surface. Its resistance during standard penetration testing was consistently less than 10 blows/foot with an average of about 6, indicating a fairly loose soil. Unfortunately, no laboratory testing results are available for a reliable assessment of the performance of the layer during the 1978 earthquakes.

Preliminary analyses suggest that liquefaction of such a layer is highly probable during an M = 6.5 & R = 25 km earthquake. Following Seed (1976), for a corrected SPT value $N_C \simeq 1.1\ N \simeq 7$ blows/foot, a cyclic stress ratio $\tau/\sigma_v' \simeq 0.09$ could cause soil liquefaction during such an earthquake. On the basis of the preceding discussion of the recorded motions, the peak ground surface acceleration

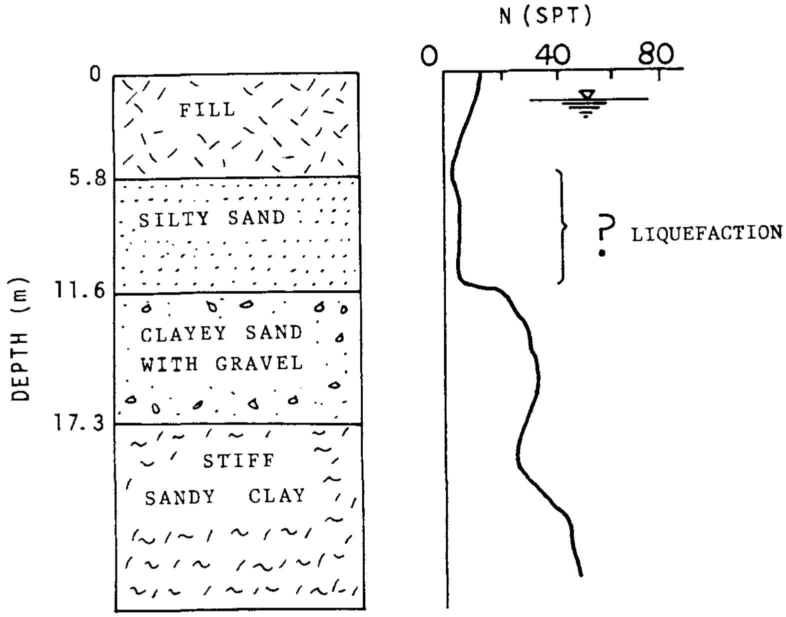


Fig. 16 Soil Profile by the White Tower of Thessaloniki

during the June 20 earthquake is expected to have reached at least 0.20 g. Therefore, the estimated induced cyclic shear stress ratio in the layer is (Seed, 1976):

$$0.65 \frac{a_{max}}{g} \frac{\sigma_v}{\sigma_v'} r_d \simeq 0.65 \times 0.20 \times 1.7 \times 0.92 \simeq 0.20$$

This value is more than two times the aforementioned cyclic shear resistance ratio, suggesting that liquefaction may have occurred.

Nevertheless, there has been absolutely no indication of any such ground failure. No sand boils or settlement of any kind have been observed on the free surface. The White Tower did not suffer any apparent settlement or tilting and, in fact, resisted very successfully all motions. It is noted in passing that this 500-year-old Tower is a massive cylindrical structure, 15 m in diameter and 32 m in height; its 2.5 m-thick exterior wall is composed of stone connected with very strong Centonite type mortar. It apparently is a very strong structure, having resisted numerous earthquakes in its five-century life.

t seems that one may safely argue that such a heavy structure would have experienced at least some settlement, had extensive liquefaction taken place in the supporting soil. On the other hand, the possibility that some loss of strength did take place in a limited volume of soil can not be excluded. Moreover, since neither the exact grain size distribution nor the dynamic properties of the layer (other than SPT values) are known, it would be presumptuous to condemn the current state-of-the-art of predicting liquefaction potential of a site.

Nonetheless, the authors feel that while the bulk of research on liquefaction of soils in the last years has investigated the dynamic behavior of soil samples experimentally or vertical 1-dimensional soil columns analytically, much has still to be learned about:

1. The influence of a liquefied volume of soil on

the behavior of the supported structure; this task requires both detailed field observations and two or three-dimensional analyses of the soil-foundation-structure system excited by vertical and non-vertical seismic waves.

2. The effect of spatial variability of soil properties on the "propagation" of liquefaction. A recent study by Fardis (1979) probabilistically accounted for such a variability, in both vertical and horizontal directions; it concluded that excess pore pressure redistribution affects the horizontal variation of soil stiffness in such a way that soil layers tend to either liquefy completely or not liquefy at all. A similar mechanism may have "saved" the White Tower of Thessaloniki, by preventing isolated pockets of liquefied soil from expanding.

Meanwhile, a thorough geotechnical exploration and laboratory testing program of the subsoil of the Tower would certainly help resolve some of the issues raised by this preliminary investigation.

CONCLUSION

Comparisons of accelerograms and response spectra from the basement motions of three structures during the June 20 and July 4, 1978, Thessaloniki earthquakes indicate that soil-structure *interaction*, wave *filtering* through soft alluvial deposits, and wave *focusing* and magnification from geologic/topographic features may qualitatively explain the wide differences (in duration, intensity and frequency characteristics) between the records. The vulnerability of a structure due to liquefaction of the supporting soil may be not realistically assessed with available empirical procedures which are based on standard penetration test measurements and yield information on the performance of a small volume of soil.

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