

SOIL AMPLIFICATION EFFECTS IN THE ADAPAZARI,  
TURKEY, EARTHQUAKE OF 1967

by

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SYNOPSIS

A soil amplification study is performed to explain the total collapse of two reinforced concrete apartment buildings during the 22 July, 1967, earthquake in Adapazarı, Turkey. Soil data is based on a series of borings, in situ measurements and laboratory testing. Results of the non-linear shear wave propagation analysis illustrate that the predominant period of the soil is very close to the natural periods of the collapsed structures.

The base rock accelerations amplified at the surface by about four to five times and developed such an intensity of severe shaking that the moment resistance capacities of the columns are much exceeded due to the quasi-resonance condition.

INTRODUCTION

It was very interesting to observe that only six apartment buildings had totally collapsed in a city of thousands of low rise buildings during the Adapazarı, Turkey, earthquake of 1967. The epicenter of the earthquake was reported to be at 55 km southeast of the city (40° 36' N - 31° 00' E). The magnitude was  $M = 7.1$  and the intensity was  $I_0 = VIII$  (MM). The focal depth was estimated to be  $h = 10$  km. Detailed information on the above was reported earlier by Yarar and Tezcan (1969).

In this presentation, only two major apartment buildings, S. Bayraklar and A. Akkoç (numbered 1 and 3 respectively in the earlier report), are studied from the viewpoint of soil amplification. For the purpose of collecting data about the soil, several bore holes were drilled penetrating to depths of about 50 meters. The soil formations below 50 meters are obtained from bore holes drilled previously for the purpose of determining aquifer thicknesses in the area. In situ and laboratory testing of soil samples provided valuable information about the soil conditions and soil properties. Additional layering information and P-wave velocities of various layers are obtained from seismic refraction surveys.

Following a nonlinear shear wave propagation analysis and determination of the yield capacity of the structures, it appeared that the reason for total collapse was the soil amplification arising from a condition of

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Collapse of the 5 storey S. Bayraklar Building, Adapazarı, Turkey, 1967

pseudo resonance. The induced lateral loads were much higher than those assumed in the design. Basically, both of the structures collapsed due to insufficient column stiffness and ductility. By contrast, smaller, stiffer buildings in the vicinity and other parts of the city had survived the earthquake with minor or no damages.

A similar pseudo resonance condition had occurred in the Gediz, Turkey, earthquake of 1970, which was reported earlier by Tezcan and Ipek (1971), Whitman et al (1974), and recently by Tezcan, et al (1976). Structural data for the two selected buildings and average soil conditions are reviewed, and a soil amplification study is presented below.

#### SOIL CONDITIONS

Typical soil conditions near the two buildings and the average soil properties for the top five soil layers are given in Table 1. Further data used in the soil amplification analysis is summarized in the inset at Fig. 1. Shear wave velocities listed in this figure are based on the maximum shear modulus of the Hardin-Drnevich (1972) empirical formula. In order to account for the disturbance of soil samples, the shear modulus obtained from the empirical formula is multiplied by 2.5. Laboratory measurements of wave velocity and damping ratio utilizing the resonant column technique are listed in Table 2. Laboratory test data of wave velocities are in close agreement with the results of the empirical formula after a disturbance factor of square root of 2.5.

#### STRUCTURAL DATA

Typical transversal frames and the structural data for both buildings were given in the earlier report by Yazar and Tezcan (1969). The results

Table 1. Average Soil Properties

Layer	Description	w <sub>n</sub> (%)	N	w <sub>L</sub> (%)	w <sub>p</sub> (%)	γ <sub>n</sub> (t/m <sup>3</sup> )	s <sub>u</sub> (kg/cm <sup>2</sup> )	φ
1	grey, sandy silty clay	29.0	9	44.0	28.3	1.93	0.34	--
2	yellow-grey, silty sand	20.0	35	--	--	2.11	--	33°
3	grey silty clay	45.9	17	75.1	33.5	1.74	0.65	--
4	grey silty fine sand	23.6	33	--	--	2.04	--	30°
5	grey silty clay	34.0	25	73.0	34.0	1.88	1.30	--

w<sub>n</sub> : natural water content  
 N : Standard Penetration Test Blow Count (blows/ft)  
 w<sub>L</sub> : liquid limit  
 w<sub>p</sub> : plastic limit  
 s<sub>u</sub> : shear strength (undrained for clayey soils)  
 φ : friction angle (drained for sandy soils)  
 γ<sub>n</sub> : unit weight

Table 3. Storey Shears Comparison

Storey No.	A. Akkoc Building T = 1.15 seconds				S. Bayraklar Building T = 1.26 seconds			
	W	φ	Vel	Vpl	W	φ	Vel	Vpl
5	38	1.00	40.9	8.86	45	1.00	49.0	11.5
4	50	0.89	72.1	15.60	66	0.86	84.5	19.8
3	57	0.71	97.1	20.98	66	0.62	105.3	24.7
2	57	0.47	117.4	25.40	66	0.39	126.5	29.6
1	57	0.18	129.2	27.98	66	0.15	140.8	33.2
Sum	259	α=1.37	129.2	27.98	309	α=1.36	140.8	33.2
%	100	--	49.8	10.80	100	--	45.6	10.7

W = storey weight  
 φ = modal vector  
 α = participation factor  
 Vel = storey shears due to 1940 El Centro earthquake, 3% damping  
 Vpl = storey shears sufficient to develop the first plastic hinge at columns

Table 2. Resonant Column Test Results at Two Different Strain Levels

Boring	Depth (m)	Measured V <sub>p</sub> (m/sec)		Calculated (1) V <sub>s</sub> (m/sec)		Damping Ratio D %	
		5 x 10 <sup>-3</sup> %	3 x 10 <sup>-2</sup> %	5 x 10 <sup>-3</sup> %	3 x 10 <sup>-2</sup> %	5 x 10 <sup>-3</sup> %	3 x 10 <sup>-2</sup> %
1	19.40-19.60	620	590	230	218	4.2	6.8
2	30.05-30.50	770	710	285	263	4.8	7.4
2	34.00-34.30	850	790	315	292	3.6	6.5

(1) Poisson's ratio μ = 0.42

Table 4. Interior Columns at Basement

Parameter	Variable	A. Akkoc Building	S. Bayraklar Building
Column Size	b/d	50/23 cm	50/30 cm
Material, concrete	f'c	200 kg/cm <sup>2</sup>	200 kg/cm <sup>2</sup>
Material, steel	f <sub>y</sub>	2400 kg/cm <sup>2</sup>	2400 kg/cm <sup>2</sup>
Reinforcement	p = p'	0.008	0.0078
Axial Force due to DL	P <sub>DL</sub>	43.0 ton	69.3 ton
Moment due to DL	M <sub>DL</sub>	0.36 tm	0.38 tm
Plastic Moment	M <sub>p</sub>	7.00 tm	12.88 tm

of dynamic analysis are summarized in Table 3. Both of the frames are subjected to the 1940 El Centro ground motion, and assuming an elastic behavior the root mean squares of the storey shears  $V_{e1}$  are calculated for a critical damping ratio of  $h = 0.03$ . The level of lateral forces at which the first plastic hinge is developed are also computed and listed in column  $V_{p1}$  in Table 3. Structural data for the interior columns of both buildings are given in Table 4.

Plastic moment capacities of these columns are reached at a base shear level of 10.8% for S. Bayraklar and 10.7% for A. Akkoç. The collapse mechanism certainly requires the formation of more than one hinge. The lateral load coefficient, however, for the stage of total collapse will not be much greater than the above coefficients.

On the basis of purely elastic behavior, the root mean square value of the El Centro lateral load coefficients for these two buildings are 49.8% and 45.6% respectively. It may therefore be concluded that if the ground motion peak acceleration was in the order of 0.33 g, similar to that of El Centro, the ductility factors were in the range of  $D = 49.8/10.8 = 4.6$  for the A. Akkoç building and  $D = 45.6/10.7 = 4.26$  for the S. Bayraklar building. For smaller ground motion peak accelerations smaller ductility factors are obtained. It thus appears that the columns of these two buildings possessed a very low degree of ductility. The collapse occurred basically in the direction of smaller moment of inertia axes of the columns.

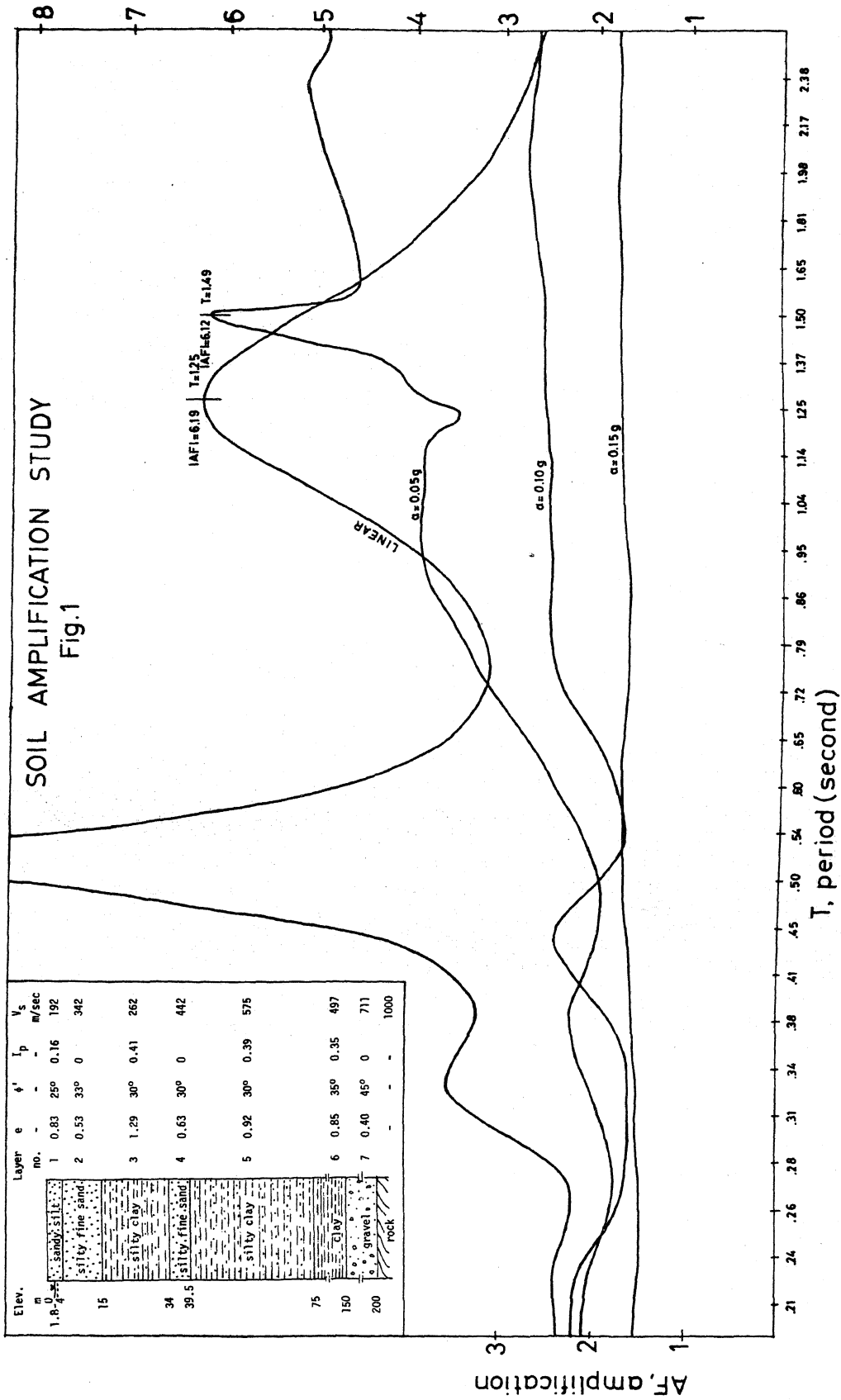
#### SOIL AMPLIFICATION ANALYSIS

Assuming different magnitudes of earthquake accelerations at base rock, the amplitudes of upward propagating shear waves are computed at the surface. The nonlinearity of shear modulus and damping are taken into account as explained by Schnabel, et al (1972) and by Tezcan and İpek (1975).

For small amplitude oscillations the predominant period is very distinct at  $T = 1.25$  seconds as shown in Fig. 1. For higher accelerations, however, the predominant period disappears and the amount of amplification decreases. For a magnitude of  $M = 7.1$  and a distance of about 20 km from the causative fault the base rock acceleration may be estimated to be of the order of  $a = 0.17$  g using the chart supplied by Seed, et al (1972). This acceleration may drop down to about  $a = 0.10$  g at a distance of 55 km from the epicenter. Assuming the amount of amplification at this level of amplitude to be roughly 300%, the peak surface acceleration at the site of the collapsed buildings may be estimated to be of the order of  $a = 0.30$  g, which is much beyond the anticipated level of design coefficients.

#### CONCLUSIONS

1. The soil amplification studies of the two collapsed apartment buildings in the Adapazarı earthquake of 1967 indicate development of a pseudo resonance condition due to soil structure period interaction. It is possible that a base rock acceleration of the order of 0.10 g was amplified at least 300% by the overlying soil layers and this was further amplified by the structure itself to the point of total collapse.
2. The base shear coefficient adopted on the design was only 0.06 g. The lateral load necessary for the formation of the first plastic hinge



was calculated to be on the order of 0.11 g. The structures therefore, yielded at a very premature stage. At the stage of total collapse the lateral loads should have been much higher than 0.11 g. This explanation illustrates the fact that the standard recommendation of the 1967 Turkish Earthquake Code was not adequate to account for the effects of soil amplification. It is encouraging to note, however, that after several examples of developing pseudo resonance conditions in Turkey, the Turkish Earthquake Code has been modified to include soil amplification possibilities.

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