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Campano-Lucano Earthquake, November 1980, Italy, Strong Motion Data Related to Local Site Conditions

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SYNOPSIS. When the 23 November 1980 earthquake struck the south of Italy a network of strong motion recorders was in operation. The earthquake triggered 21 stations. The network is composed by 168 stations spread all over the country following statistical criterion, see Iaccarino and Zaffiro, 1973. The paper presents the results of the records obtained at 6 of 21 stations together with the available geological information of the sites where the stations were located.

INTRODUCTION

Nowdays is generally accepted that the ground motion at a given location must be regarded as the result of the three fundamental phenomena: source mechanism, travel path, local conditions. The majority of the worldwide available strong motion data rarely can be correlated with the condition of the recording stations as far as the foundation dimensions and soil characteristics are concerned.
In Italy, since the Friuli 1976 earthquake, was decided by the CNEN - ENEL joint study commission to correlate any station of the permanent strong motion network to the geological conditions and, as soon as the station is activated by an earthquake, to investigate the geotechnical properties of the soil including shear waves measurements by the cross-hole technique.
For the campano - lucano earthquake only the geological information of the enviroment around the stations are available and described hereafter.

DESCRIPTION OF THE STATIONS

Fig. 1 represents the stations of the permanent network triggered by the main shock. As can be observed the acceleration treshold of about 0.01 g was exceeded up to distances more than 140 km from the source.
The strong motion instruments have been located in small houses used by the national electric company as transformer stations.
The basament of the instrument is a pillar (see fig. 2) completely detached from the base slab to avoid the influence of the soil-structure interaction.

As far as the geological site conditions are concerned, the geological cross section (fig.3, 4,5,6,7,8) and the description of the soil below the stations given in table 1 are obtained by a surface geological survey, that is, by the observation of the outcropping soil all aound the stations.
A more refined and detailed geotechnical investigations will be perfomed at the most interesting stations in a near future.

DATA PROCESSING

Table 2 presents the maximum values of the corrected accelerations and velocities for all the stations triggered by the main shock and their hypocentral distances.
The numerical processing of the strong motion records was performed following basically the procedure described by Trifunac and Lee, 1973. Few changes have been introduced to improve the baseline correction at low frequency range (Basili, Brady 1978).
In particular the choice of the values of high-pass cut-off and roll-off frequencies for the mentioned baseline correction was aimed:
1) to save the information contained in the low frequency part of the signal
2) to process the data "at the best" of the available routine.

A previous analysis of the accelerograms led to include in the filtered signal periods of at least 5 seconds and therefore to fix for the cut-off frequency the value of 0.25 Hz and for the termination frequency the value of 0.03 Hz. No particular effort has been devoted to the correction of the high frequency range since

the energy content above 10 - 15 Hz seems to be irrelevant.
The spectra are corrected with error less than 5% from 0.066 Hz to 5 Hz, above 5 Hz the error is greater.
Also the response spectra have been computed for 0.02 and 0.05 damping ratios and each have been normalized to the maximum acceleration of corrinspondig component of motion.
Fig. 3,4,5,6,7,8 present the normalized response spectra of the six stations closer to the most destroied area, the same figures show comparison of normalized response spectra with the normalized U.S.R.G. 1.60 spectra.
Also the low pass band filtered husid ratios (Basili, Gorelli, Muzzi 1981) for the E-W components at the above six records have been computed and presented in Fig. 9.

CONCLUSIONS

From the observations of the accelerogram, response spectra and low pass band filtered husid ratios the following observations can be outlined:

- The earthquake have generated a ground motion of very large duration with the characteristic of a multiple shock event.
- The response spectra can be classified among the broad-band type spectra having sigificative amplifications factors from roughly 0.5 Hz up to 10 - 15 Hz.
- The comparison with the R.G. 1.60 response spectra confirm what assessed before. In fact the R.G. 1.60 spectra, wich have been developed on a statistical basis and rapresents the smoothed average + 2 S.D. shapes, are completaly enveloped by some component of motion and sometime considerably exceded.
- The local conditions played some important role for those stations located far enough from the source not to be influenced by the source mechanism itself. An evident example is the response of the Brienza station (see fig. 2) located about 40 Km from the hypocenter which showed a sharp amplification around 5 - 7 Hz.
  The station is on a slope over an alluvial deposit.
- From the analysis of the L.P.B.F.H.R. it can be observed how the energy associated with the several ranges of frequencies are not generally indicating a concentration of energy in narrow bands of frequency but rather a prevalent distribution betwenn 2.5 Hz and 10 Hz. The two stations Bagnoli Irpino and Calitri showed a prevalent concentration of energy (70% ÷ 80%) within the low frequency range 1 Hz - 3.5 Hz. Given the proximity to the epicenter (hypocentral distance ≃ 28 Km) it could be argued about the possibility that these records carry the prevalent information of the source mechanism.

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TABLE 1 - ACCELEROGRAPHIC STATIONS (FROM BERARDI ET AL, 1981)

STATION DENOMINATION	COORDINATE		INSTRUMENT SENSIBILITY (cm/g)			GEOLOGICAL DESCRIPTION OF THE SOIL BELOW THE STATIONS
	LATITUDE N	LONGITUDE E	N-S	VERT.	E-W	
ARIENZO	41° 01' 43"	14° 28' 00"	1.85	1.82	1.84	Cinerities, Piroclastic and conoid materia (5 m), campanian ignimbrite, overlying limstones of campano-lucana platform.
AULETTA	40° 33' 37"	15° 33' 30"	1.90	1.90	1.81	Lacustrine and deltaic poligenic conglomerates (140 m), presumably stiff soil conditions.
BAGNOLI IRPINO	40° 40' 15"	15° 04' 10"	1.79	1.80	1.85	Limestones and dolomitic limestones of campano-lucana platform. Rock Site.
BENEVENTO	41° 07' 17"	14° 47' 44"	1.69	1.78	1.77	Terraced fluvio-lacustrine deposits consisting of sands, clayey sands and poligenic conglomerates (50 m), overlying the "Ariano formation" (pliocenic terrigenous complex).
BISACCIA	41° 00' 47"	15° 32' 33"	1.72	1.84	1.76	"Variocolour shales". Stiff soil conditions.
BOVINO	41° 15' 02"	15° 30' 35"	1.85	1.77	1.79	Pliocenic sands and sandstones with levels of poligenic conglomerates and sandy clay (25 m) overlying poligenic conglomerates.
BRIENZA	40° 28' 27"	15° 38' 06"	1.80	1.78	1.79	Poligenic conglomerates with sandy matrix, probably having a fluvio lacustrine origin (25 m), overlying flyschoid materials. Soft soil cond
CALITRI	40° 55' 01"	15° 26' 19"	1.80	1.92	1.87	Sandstones and yellowish, reddish grey sands (30 m) overlying marls and bleu-grey silts. Rock Site.
GARIGLIANO	41° 15' 32"	13° 49' 36"	6.76	6.80	7.48	Allufioum complex mostly consisting of sandy clays, fine and coarse grained sands and silt.
GIOJA SANNITICA	41° 19' 18"	14° 26' 51"	1.75	1.82	1.86	Slope detritus (35 m) overlying coarse grained sandstones, interbedded by clays and silty marls.
LAURIA	40° 02' 53"	15° 50' 08"	1.80	1.80	1.98	Limestones of "Campano-Lucana platform".
MERCATO S. SEVERINO	40° 47' 29"	14° 45' 51"	1.86	1.71	1.83	Recent alluvia mostly made of sands, pebbles and palustrine clays (70 m) overlying the "campanian ignimbrite".
RIONERO IN VULTURE	40° 55' 46"	15° 40' 10"	1.66	1.67	1.75	Subaerial dark tuffs, usually layered.
ROCCAMONFINA	41° 17' 19"	13° 58' 49"	1.81	1.81	1.80	Slope detritus (50 m) overlying the latitic domus M. Santa Croce - M. Lattani.
SANNICANDRO G.	41° 50' 02"	15° 34' 16"	1.75	1.85	1.90	Limestones and dolomitic limestones
SAN GIORGIO LA MOLARA	41° 16' 36"	14° 55' 40"	1.79	1.79	1.89	Arenaceus marly flysch made of sandstones and marls.
SAN SEVERO	41° 41' 02"	15° 23' 10"	1.90	1.95	1.95	Yellowish sands, interbedded by conglomerates and clays ("Serracapriola sands")
STURNO	41° 01' 21"	15° 07' 02"	1.70	1.76	1.88	Silty clays and marls, interlayered by marly limestones and quartz sandstones.
TORRE DEL GRECO	40° 48' 04"	14° 23' 08"	1.80	1.85	1.81	Leucititic lava (25 m) overlying vesuvian piroclastic materials
TRICARICO	40° 37' 15"	16° 09' 25"	1.84	1.90	1.99	Ligth yellow or pink grainstone interbedded by marls and levels of silt and sandstone.
VIESTE	41° 52' 43"	16° 09' 52"	1.76	1.73	1.77	Limestones and white marly limestones finelly stratified with flint lists and nodules

TABLE 2 - MAXIMUM VALUES OF THE CORRECTED ACCELERATIONS AND VELOCITIES-HYPOCENTRAL DISTANCES

STATION DENOMINATION	MAXIMUM ACCELERATION (g/10)			MAXIMUM VELOCITIES			HYPOCENTRAL DISTANCES (Km)
	N-S	V	E-W	N-S	V	E-W	
ARIENZO	0.27	0.21	0.37	3.9	3.6	2.9	78.5
AULETTA	0.56	0.31	0.59	4.6	4.2	5.6	31.5
BAGNOLI IRPINO	1.37	0.90	1.72	20.7	14.4	32.2	28.9
BENEVENTO (°)	-	-	0.50	-	-	-	61.2
BISACCIA	0.94	0.54	0.80	18.9	10.1	14.5	34.2
BOVINO	0.45	0.28	0.47	4.5	2.8	3.6	57.2
BRIENZA	2.15	1.52	1.57	11.2	6.8	8.6	47.5
CALITRI	1.57	1.58	1.71	26.9	19.3	28.4	27.3
GARIGLIANO	0.39	0.21	0.33	8.2	3.7	6.7	136.5
GIOIA SANNITICA (*)	0.20	-	-	-	-	-	95.4
LAURIA (*)	0.20	-	-	-	-	-	94.0
MERCATO S.SEVERINO	1.08	0.50	1.39	8.5	5.2	13.1	49.7
RIONERO IN VULTURE	0.98	0.73	0.95	14.1	9.2	8.2	40.9
ROCCAMONFINA (*)	-	-	0.30	-	-	-	126.8
SANNICANDRO G. (°)	0.40	-	-	-	-	-	122.0
S.GIORGIO LA MOLARA (*)	0.25	-	-	-	-	-	67.4
S. SEVERO	0.25	0.10	0.21	2.3	1.6	2.1	103.7
STURNO	2.19	1.63	2.96	36.2	24.1	61.8	37.9
TORRE DEL GRECO	0.61	0.35	0.40	5.4	5.6	5.7	80.0
TRICARICO	0.47	0.23	0.35	6.3	4.1	5.4	76.7
VIESTE	0.36	0.15	0.31	2.4	2.6	2.8	144.1

(*) Not digitizable

(°) Records usable after special photographic treatment

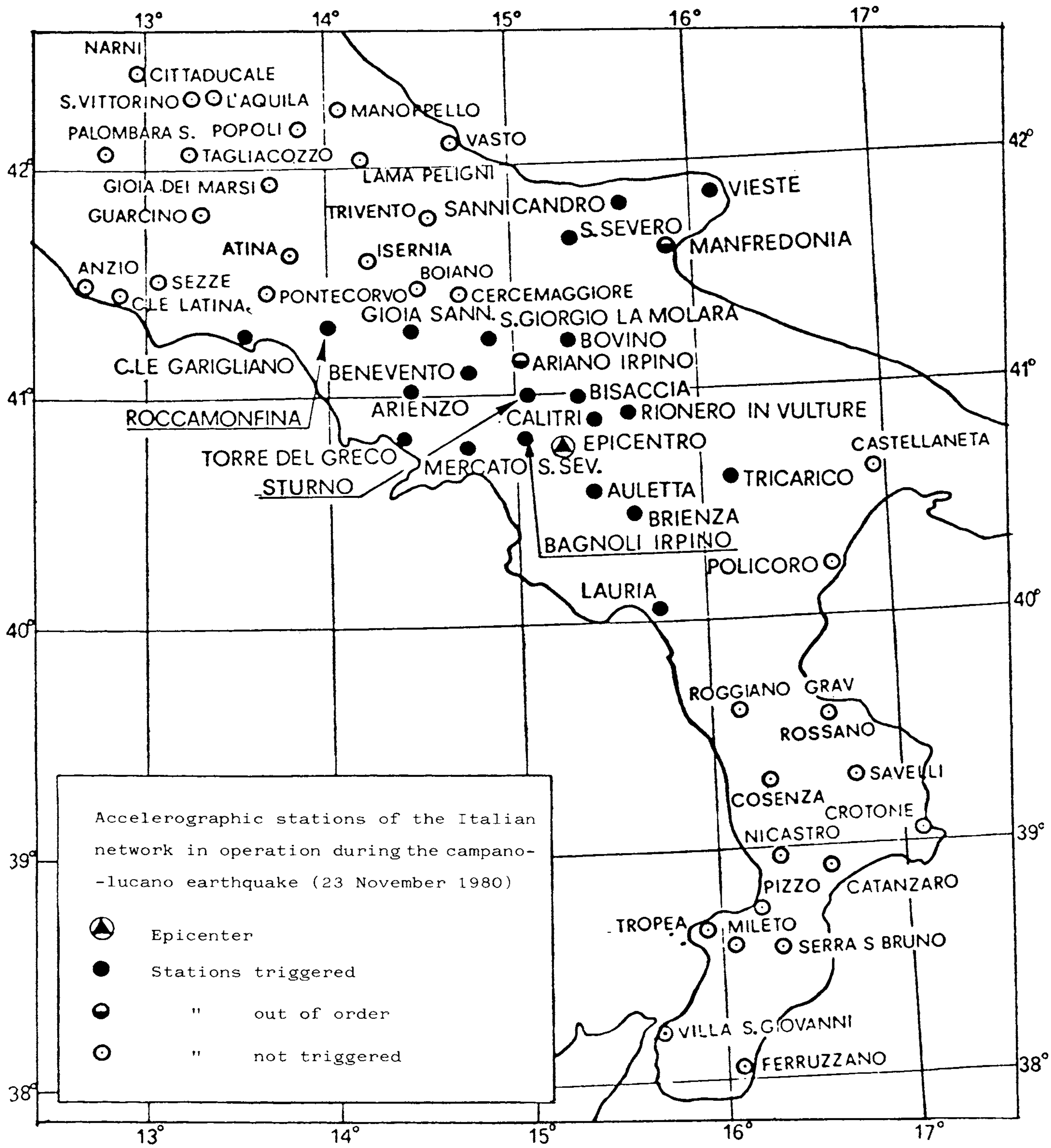


Fig. 1 - Map of the accelerographic stations in operation and triggered during the earthquake

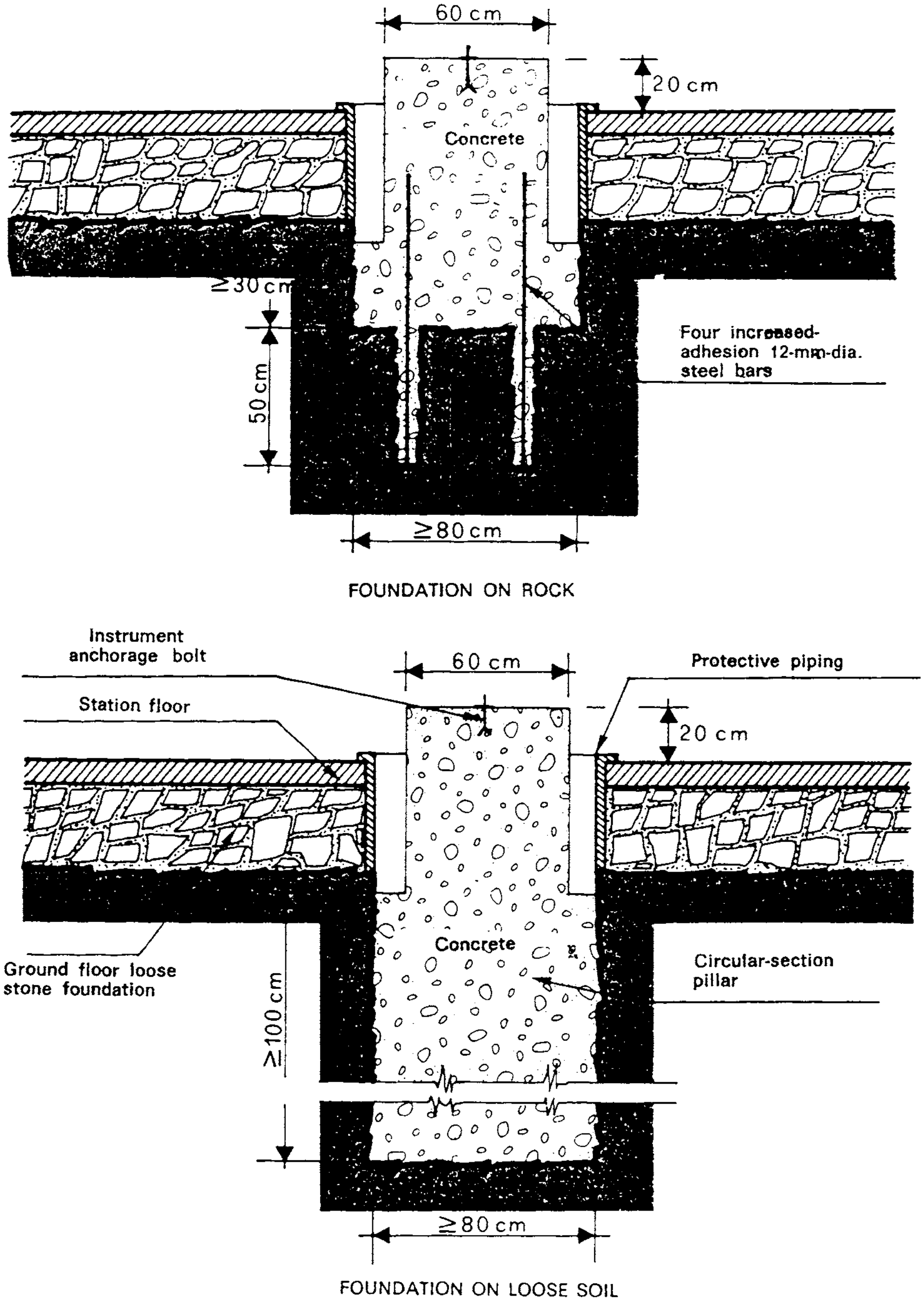


Fig. 2 - Foundation of the accelerograph anchorage pillar
(from CNEN-ENEL Commision on seismic problems..., 1976)

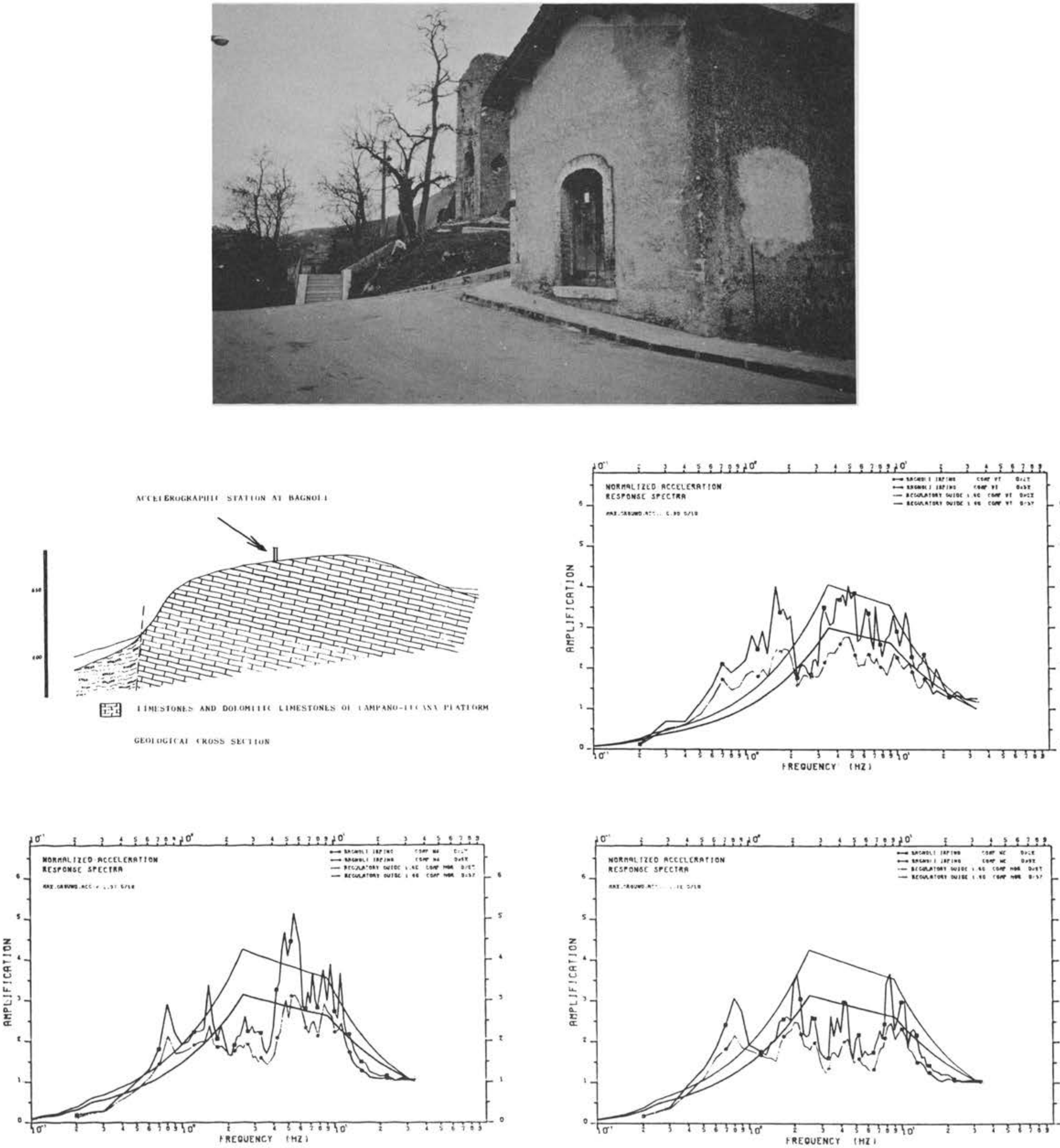
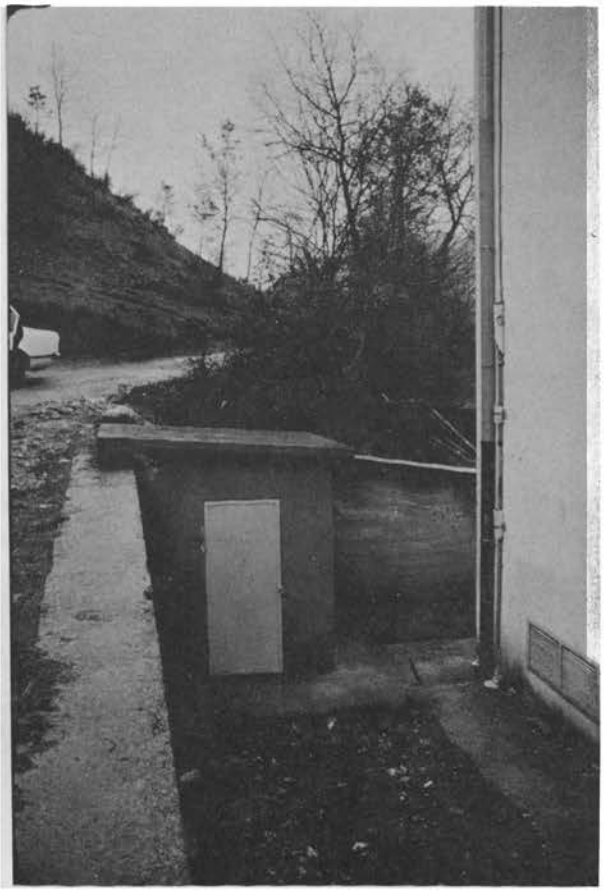
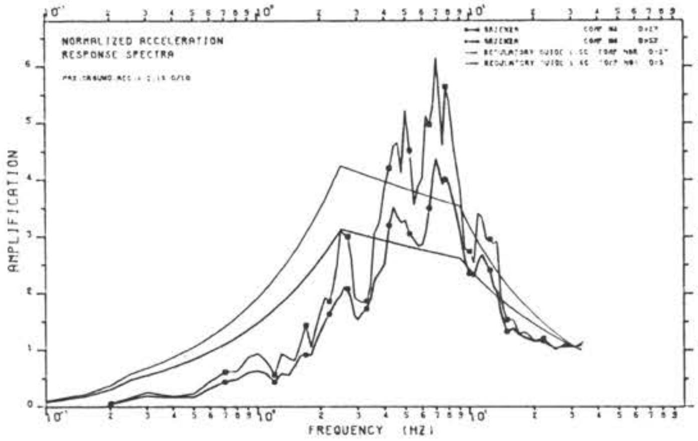
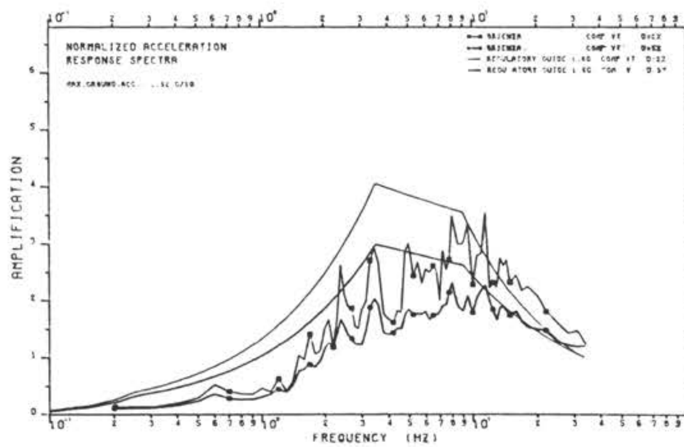


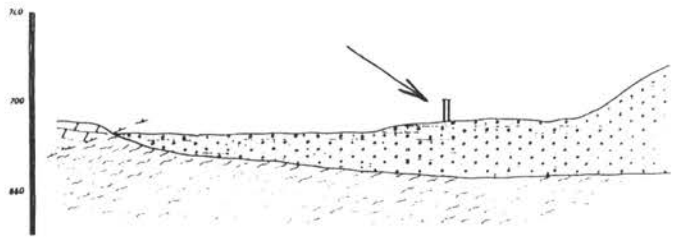
Fig. 3 - General view of the local condition, geological cross section and response spectra of the accelerographic station Bagnoli Irpino.

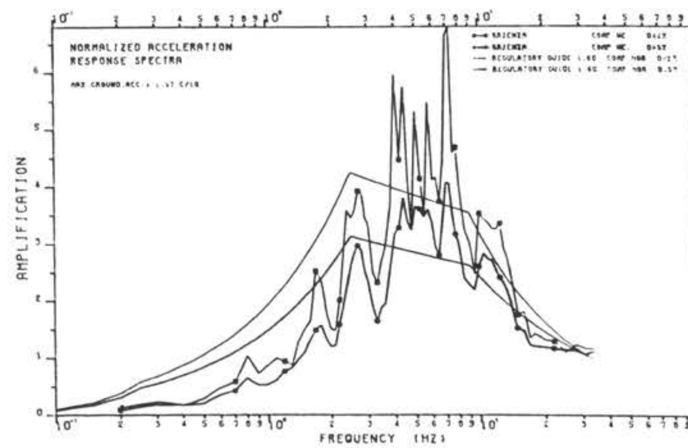






ACCELEROGRAPHIC STATION AT BRIENZA





POLIGENIC CONGLOMERATES WITH SANDY MATRIX, PROBABLY HAVING A FLUVIO-LACUSTRINE ORIGIN, OVERLYING FLYSCHOID MATERIALS.

GEOLOGICAL CROSS SECTION

Fig. 4 - General view of the local condition, geological cross section and response spectra of the accelerographic station Brienza.

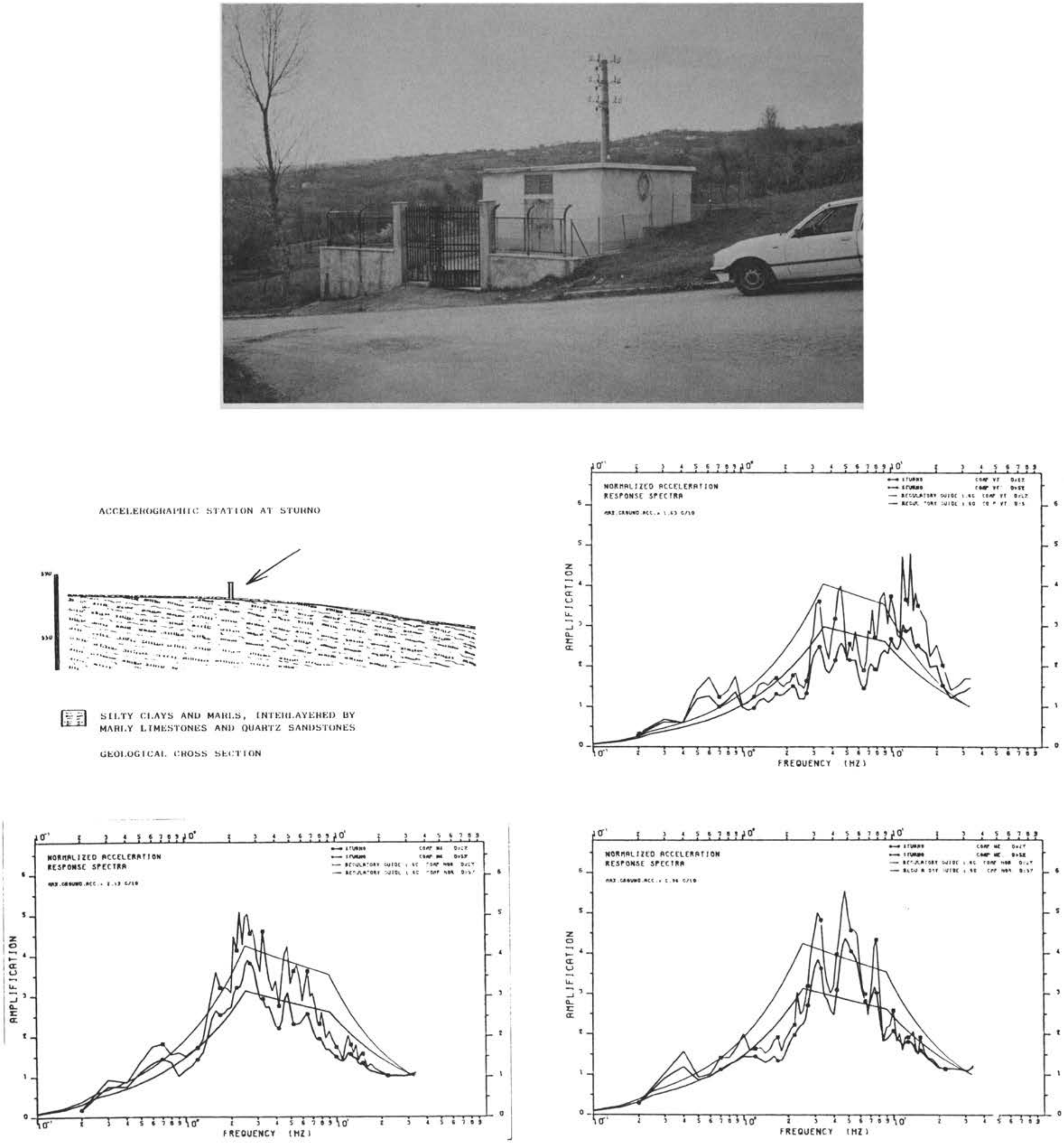
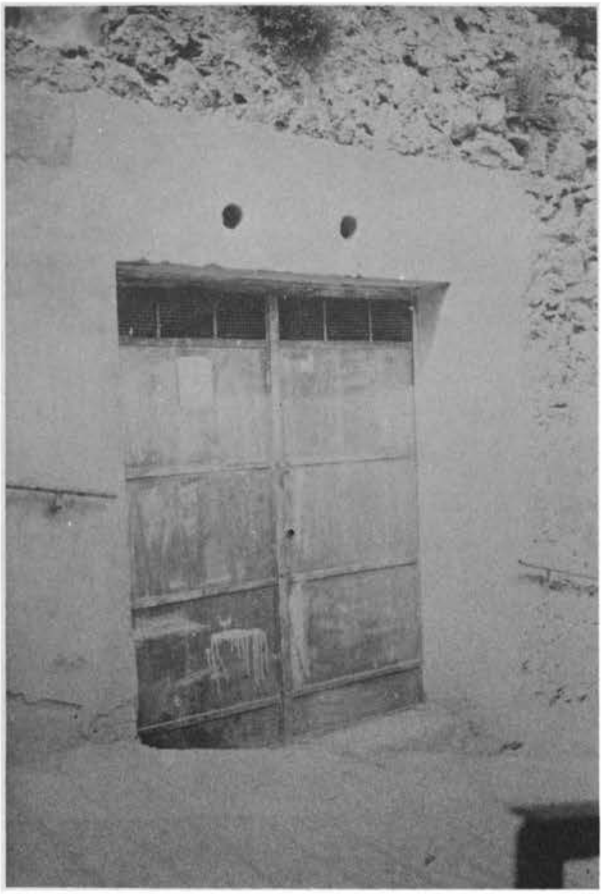
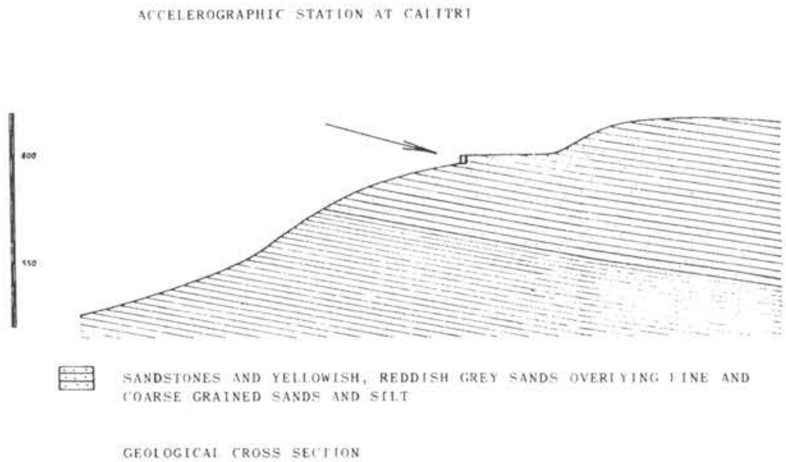
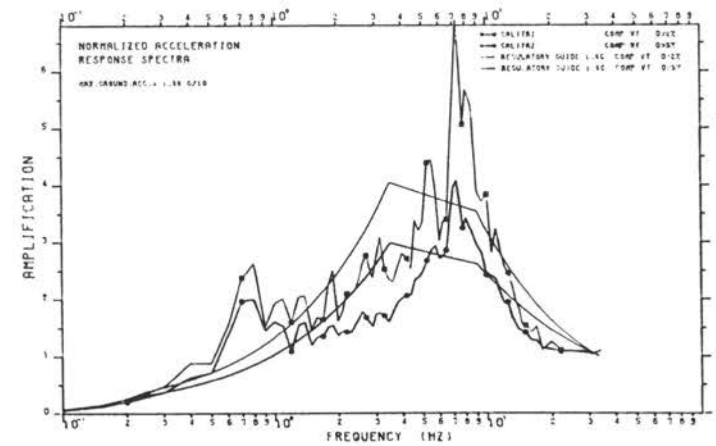
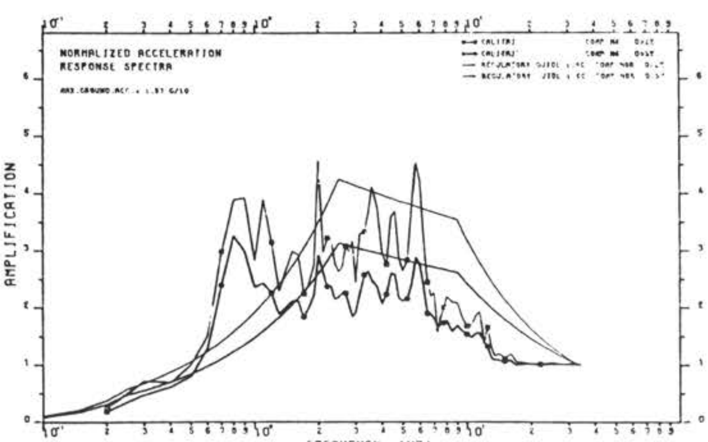


Fig. 5 - General view of the local condition, geological cross section and response spectra of the accelerographic station Sturno.









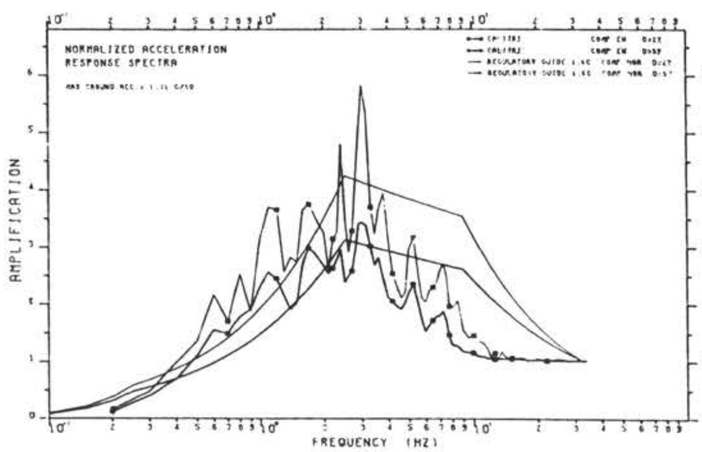


Fig. 6 - General view of the local condition, geological cross section and response spectra of the accelerographic station Calitri.

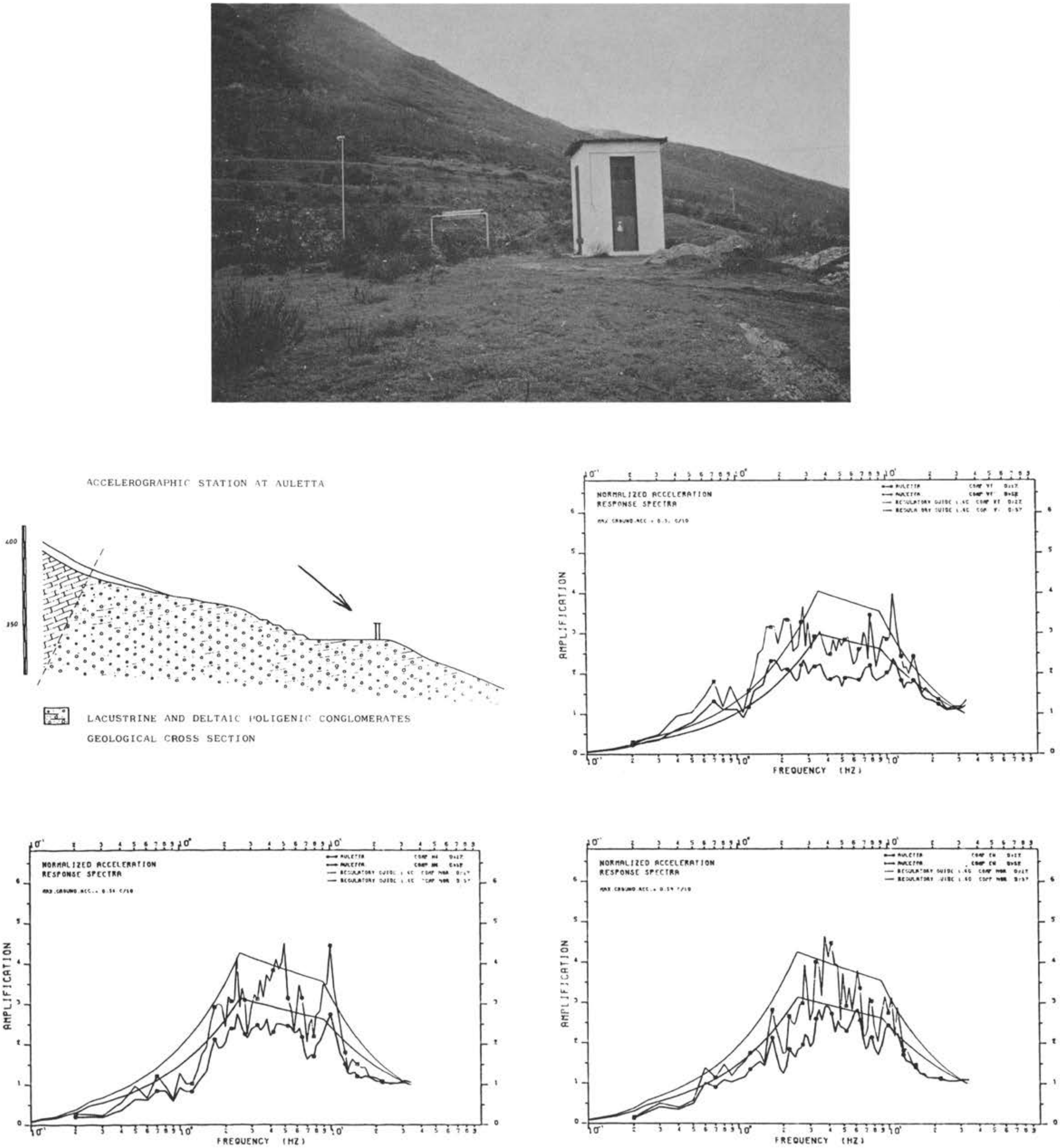
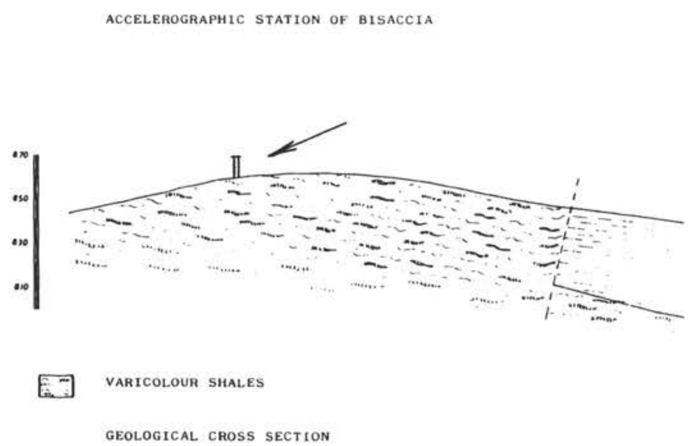
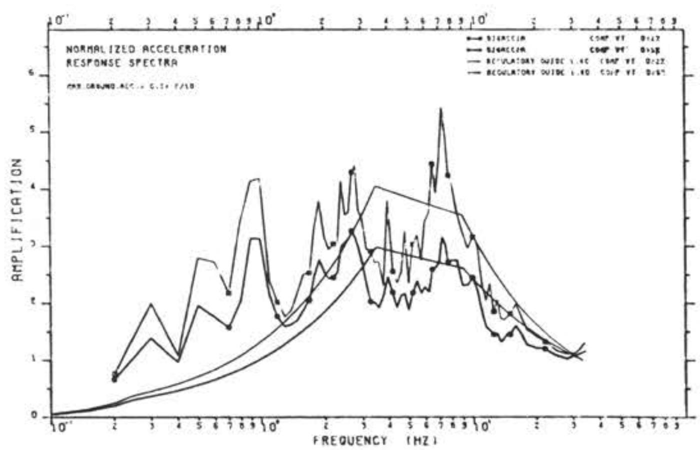
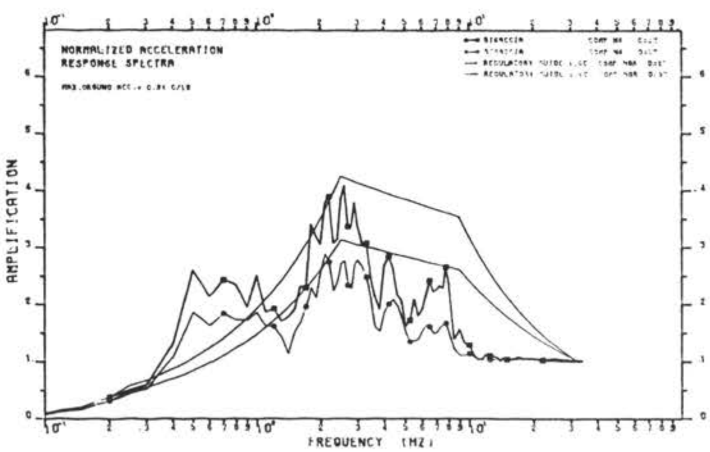


Fig. 7 - General view of the local condition, geological cross section and response spectra of the accelerographic station Auletta.







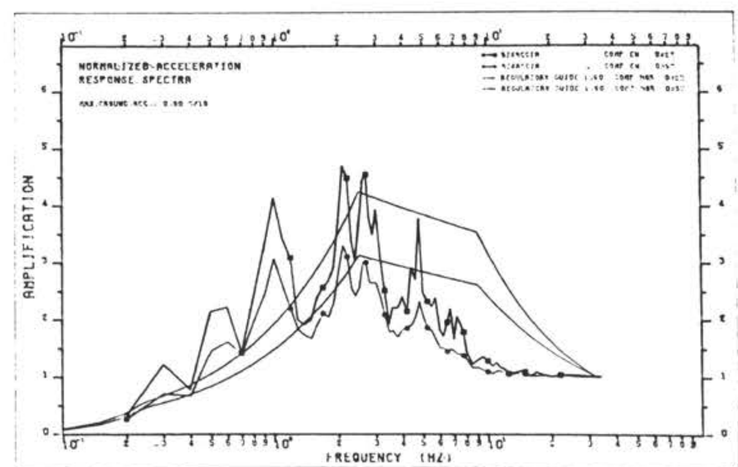


Fig. 8 - General view of the local condition, geological cross section and response spectra of the accelerographic station Bisaccia.

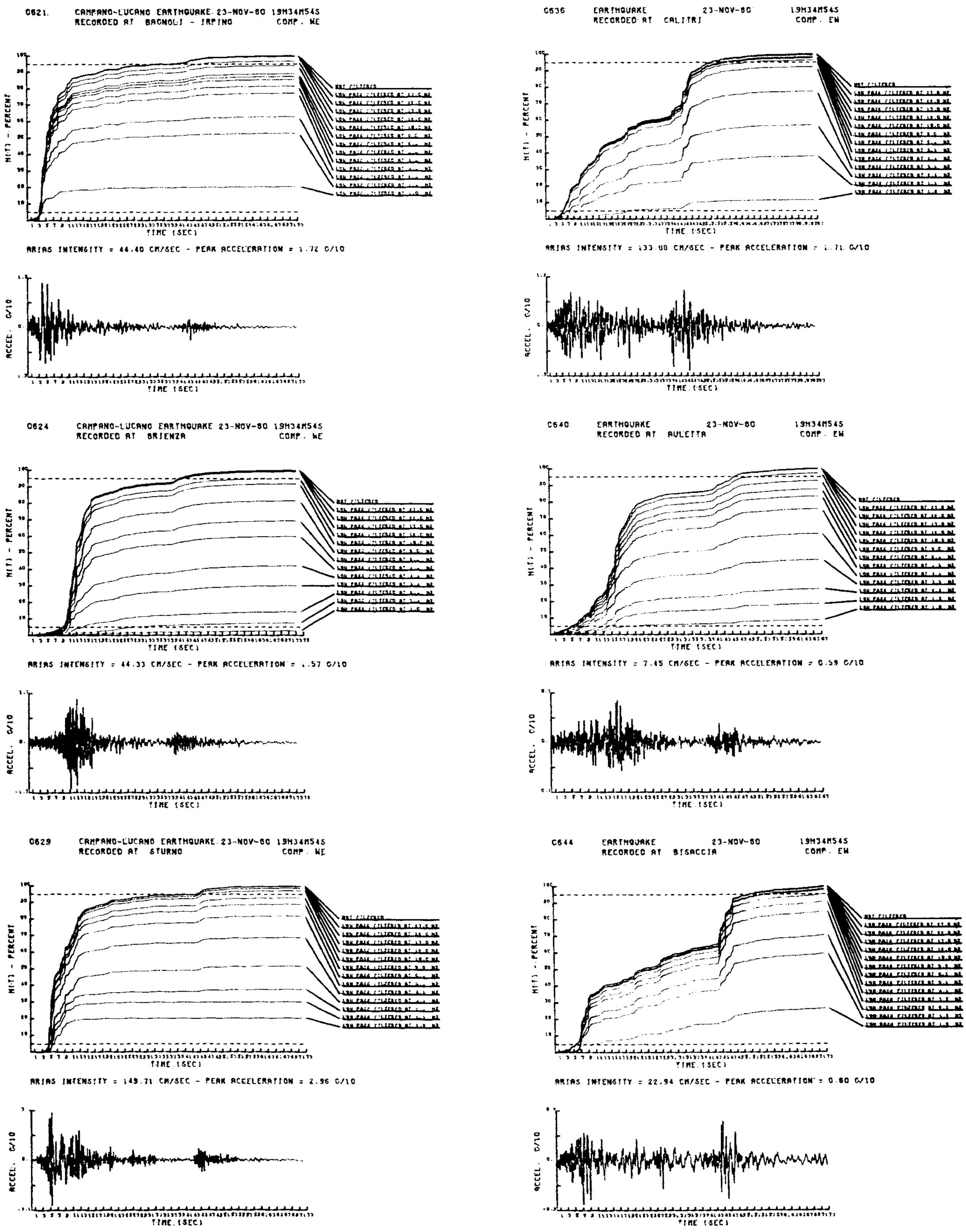


Fig. 9 - Low pass band filtered husid ratios for the E-W component of the accelerograms recorded at Bagnoli Irpino, Brienza, Sturno, Calitri, Auletta and Bisaccia accelerographic stations.

Moderator's Report on
Case Histories in Geotechnical
Earthquake Engineering by
K. Arulanandan, Professor
University of California, Davis,
California.

The papers presented to this session of the conference can be broadly grouped under two categories: firstly, those which add to the state of the art in earthquake engineering and secondly, those describing observed earthquake motions and the effects associated with them.

I. State of the Art Aspects of Earthquake Engineering

(a) The paper by Bhandari, R. K. M. on "Dynamic Consolidation of Liquefiable Sands" demonstrates through the measurement of SPT values before and after dynamic consolidation that dynamic compaction is an effective method of increasing the resistance to liquefaction.

(b) Iyengar, M. considers "Improvement Characteristics of a Liquefiable Soil Deposit by Pile Driving Operations". He found that SPT values increased and lateral deformation of piles decreased with densification of the soil.

(c) Guru Rau, S. N. and R. K. Varma, in the paper titled "Planning Instrumentation Monitoring in Dams" state that many significant characteristics of reservoir induced seismicity and other seismogenic features were noted during the 30,000 distinct seismic events with magnitude 1.0 to 7.0 that were recorded at the Koyna Dam, India, since 1963. These observations were made as part of an extensive study of river valley projects in India using installed seismographs. Seismic instrumentation of the Sedawgyi Dam project located across a main regional fault is also described.

(d) Thomas Vladut, on "Geomechanics of Reservoir Induced Seismicity", considers that the phenomenon is a consequence of the difference in stress generated by weight modification and flow. This difference in stress could, in some cases, exceed the ultimate strength of the rock. Particularly in a fissured medium, the tensile strength could be exceeded, causing hydraulic fracture and releasing the potential energy accumulated by deformation under the storage weight. This mechanism is different from the accepted elastic rebound theory where fault lines are basically the cause of seismicity. Twenty cases of induced seismicity were studied where knowledge of the seismicity before and after dam construction was available. It was concluded that induced seismicity may be due either to hydraulic fracture or to elastic rebound, depending on the geological conditions of the site.

(e) "Wave Propagation at the Surface of Clay Deposits due to Vertical Impact" by Lefebvre, G., M. Veber and J. G. Beliveau considers the following:

Five sites consisting of soft sensitive clay deposits were chosen for the study. Particle velocities, resulting from impact of a free falling mass dropped from different heights, were recorded at the ground surface and at depths of 0.5 and 1m at four distances from the point of impact. The data was interpreted using the theory of wave propagation in an elastic half-space. The limited number of tests indicated some trends which confirm this theory and which could be summarized as follows:

1. The amplitude of the particle velocity is a function of the linear momentum of the falling mass at impact.

2. The major portion of the energy is in the form of Rayleigh waves.

3. The effect of the depth of the water table is instrumental in affecting both the level of motion and the corresponding attenuation. The lower the water table, the larger are the particle velocities at the surface and the larger is the corresponding attenuation.

4. The larger the motion, the larger is the attenuation coefficient.

5. Larger particle motions are measured along the crest of a slope rather than perpendicular to the crest.

II. Case Histories of Earthquake Motions and the Study of their Effects

Characteristics of observed earthquake motions, the purpose of the studies and the major findings of each one of the papers in this category are listed in Table 1.

The first three papers all deal with the Tangshan earthquake of July 1976 and the observations made.

(a) "Mechanism of Surface Faulting and its Seismic Effect" by Wang, Z. Q., S. T. Zhao and Z. L. Huang. This paper attempts to give evidence for surface faulting in terms of the geometry of fault propagation. It considers the geometry of the break, the strike of surface faulting in relation to the epicenter, the trend of aftershocks and the damage that occurred.

(b) "Source Mechanism and Seismic Effect of Tangshan Earthquake", A Preliminary Study on the Tangshan Active Fault Zone by Fang Hong-qi, Mio Xin-kwan and Zhao Shu-dong. This paper, by analyzing the seismic mechanism of the Tangshan earthquake and the pattern of movement, provides support to the elastic rebound theory for the causation of earthquake.

(c) "Observation and Analysis on Building Settlement due to Tangshan Earthquake", by Huang Xiling. Significant settlements and tilts of buildings were observed during the 1976 Tangshan earthquake. These are described in detail in this paper. The cause of settlement of a building which had been built over a layer consisting of a 3 m thick hydraulic fill surface soil underlain by organic silty clay 15 m thick was investigated.

Table 1. Summary of Earthquake Case Histories

Time Of Occurrence	Place	Magnitude	Maximum Acceleration	Duration	Purpose	Finding
July 1976	Tangshan	7.8	0.2 g	--	To study the nature and cause of faults	1) Surface faulting can be estimated from the geometry of the propagation of the fault in the vicinity of structures. 2) Tangshan earthquake is shown to be due to an active fault zone. This fault is assessed to be a strike-slip fault.
					To propose an analysis to predict settlement of building erected on clays and to compare predictions with observations.	The analysis is capable of predicting settlements qualitatively.
March 1979	Enmedio Island Mexico	7.6	0.3 g	15 sec. strong motion shaking	Signs of liquefaction occurred in Zone 2, but not in Zone 1, both underlain by same deposit. Why?	1) Seed's mtd. of predicting liquefaction using SPT values is inadequate to explain this. 2) Rahman et al. analysis for the prediction of pore pressure history explains this to be due to the differences in overburden pressures.
June 1976	Ohgishima Island near Tokyo		0.036g EW 0.026g NS		To present the details of instrumentation to monitor pore pressures and ground motions.	Pore pressure did not rise since the earthquake is too small.
Sept. 1980	Owi Island near Tokyo Bay (Mid-Chiba E.Q.)		0.065g EW 0.095g NS		To present the details of instrumentation to monitor pore pressures and ground motions.	Pore pressure build-up took place only during one cycle including the peak. Soil did not liquefy (pore pressure=16-21% of confining pressure).
1976	Friuli	A series of small earthquakes are described Magnitude 4-6			To investigate the usefulness and validity of a new numerical method proposed to describe an earthquake motion.	Numerical Model appears to reveal more information hidden in an E.Q. record than the response spectra.
1978	Thessaloniki E.Q.	6.5	0.1-0.24g	3-9 sec. strong motion shaking	1) To study the influence of soil conditions and soil-structure interaction on ground motions. 2) To study the liquefaction potential.	1) Qualitatively explains that soil-structure interaction modifies the motion. 2) Reliability of liquefaction potential potential using SPT values and small samples are questioned.

Static and dynamic shear strengths of this clay were determined from laboratory tests, with different initial static shear stresses. These tests revealed that when the initial shear stress $\geq 0.6\ \sigma_f$ (σ_f = static strength), the dynamic strength is approximately $1.1\ \sigma_f$. The earthquake induced shear stress distribution in the foundation soil was also determined using Seed's simplified procedure and the code method used in PRC. (a_{max} = 0.2g). From this, a zone where cyclic shear stresses of $1.1\ \sigma_f$ occurred was located and considered to be the plastic zone. It is evident from this that the larger the extent of the plastic zone, the larger the settlement of foundations would be. These analytical predictions were in accordance with the observed phenomena during the earthquake.

This paper presents an analytical approach to predict qualitatively the settlements of buildings founded on clayey soils.

The next two papers, both by the same authors, deal with an earthquake which caused liquefaction on Enmedio Island, Mexico in March 1979.

(d) "Liquefaction of the Enmedio Island Soil Deposits" by A. Jaime, L. Montanez and M. P. Romo. On March 14, 1979, a 7.6 magnitude earthquake shook the Enmedio Island. The maximum ground surface acceleration recorded was about 0.3g. The duration of shaking was 35 seconds, the strongest shaking lasting for 15 seconds. Material at the site consisted of dark grey, loose to medium dense, fine sand. The thickness of this layer varied from 1 to 6 m. The ground water was at an elevation of 0.0m when the earthquake occurred. For the purpose of this study, the area under observation was divided into two zones.

During the seismic event, cracks 5 to 10 cm wide and sand boils 0.3 to 1.5 m in diameter developed in Zone 2. No sand boils were observed in Zone 1.

SPT values were obtained in both zones. The values of $\bar{N}^1_{min}$, $\bar{N}^1_{max}$, $\bar{N}^1_{av}$ and the associated value of $\tau/\bar{\sigma}_o$ for each zone are given below.

PARAMETER	ZONE 1 X	ZONE 2.+
$\bar{N}^1_{min}$	13	15
$\bar{N}^1_{max}$	30	31
$\bar{N}^1_{av}$	22	24
$\tau/\bar{\sigma}_o$	0.23	0.26

These values were used to analyze the susceptibility to liquefaction of the dark grey sand layer in both zones, following the simplified procedure proposed by Seed et al. Seed's liquefaction chart indicated that soil in both zones was susceptible to liquefaction whereas only soil in Zone 2 actually liquefied.

The reasons for the soil in Zone 1 not liquefying were addressed in the second paper on this topic.

(e) "Observed and Predicted Liquefaction of a Sand Stratum", by A. Jaime, M. P. Romo and L. Montanez. To explain the non-liquefaction of soil in Zone 1, cyclic triaxial tests were carried out on soil obtained from this zone. It was shown from these tests that the stress required to cause liquefaction was lower than that induced by the earthquake. In addition, the authors carried out a computation based on the numerical model proposed by Seed, Martin and Lysemer (1975) and a computer program developed by Booker, Rahman and Seed (1976) to evaluate the influence of overburden pressure on the pore pressure generation and dissipation. Analysis showed that in Zone 2 the pore pressures developed were equal to the confining pressure during the mid period of the duration of strong shaking of 15 seconds, whereas in Zone 1, the time required to develop pore pressures equal to the confining pressure was about 12-15 seconds. The difference in the time required to develop the pore pressure necessary to cause liquefaction is considered to be due to the difference in the overburden pressure. The conclusion is that the earthquake must continue to deliver energy to the layer of liquefied material for a few seconds after initial liquefaction is reached.

If (1) the SPT values used in the analysis are reliable, (2) the in situ conditions (e.g., relative density and state of anisotropy) are correctly evaluated, and (3) the field conditions can be duplicated exactly in the laboratory, then the conclusions reached by the authors are valid. Other measurements, such as shear wave velocity, state of anisotropy, and in situ porosity are therefore needed to substantiate their conclusions.

(f) "Measurements of Insitu Pore Water Pressures During Earthquakes" by K. Ishihara. This paper gives an evaluation of pore pressure generation and horizontal accelerations at two sites, one in its intact form and one in compacted form, on Ohgishima Island near Tokyo. The author's approach is very useful in assisting us to verify the current empirical and numerical procedures.

The intact and the compacted sites were shown to have the same relative density and yet the standard penetration values are shown to be different. This finding invalidates the Gibbs and Holtz's relationship between Dr and N as a function of overburden pressure.

It would be useful to obtain data like Shear Wave Velocities, insitu densities and the anisotropy states at the two sites by other available non-destructive tests (cone penetration resistance and pressure meter tests), to convincingly establish that the two site conditions before and after compaction are, in fact, different with respect to density, fabric, k_o conditions, etc. The proper documentation of the conditions of the sites by various methods is considered to be valuable in adding to our knowledge on the proper evaluation of liquefaction potential.

(g) "Pore Water Pressure Rises During Earthquakes", by K. Ishihara. In this paper, Ishihara shows that during the 1980 earthquake of M=6.1, the pore pressure buildup in a site with N values between 2-10 was essentially caused by the one-cycle application of shear stress involving the peak. The shear stress after the peak had exerted no influence on the pore water pressure build up. It would have been instructive to compare the shear strain developed with the threshold shear strain of 10^{-4} as suggested by Dobry et al. This would require the knowledge of the shear wave velocity at the site.

It would seem appropriate also to compare the pore pressures developed with those predicted by the numerical method of Booker, Rahman and Seed. Such a study may provide a basis for the validity of the numerical procedures. Verification of the observed result by physical modeling is another useful approach.

(h) "Local Site Behavior in the 1976 Friuli Earthquake" by M. Basili, V. Gorelli and F. Huzzi. The response of a site is currently considered, by several investigators, to be a function of three parameters: (1) source mechanism, (2) propagation process and (3) local site conditions. By analyzing these parameters, relationships have been obtained between duration, soil conditions, and response spectra. The authors argue that discrepencies in the relationships obtained by different investigators may be due to the lack of homogeneous soil conditions, lack of proper documentation and uncertainty of the source mechanism.

The authors analyzed ground motions obtained at 2 sites: (1) a hard rock outcrop and (2) an alluvial deposit 20 to 25 m thick underlain by sloping bed rock. The motions were analyzed by means of Husid Ratios of Low-Pass Filtered accelerograms.

Husid ratio versus time plots showed a general pattern of a flat portion followed by a steeply sloped portion and finally another flat portion. The duration associated with the steep portion which represents the duration of strong shaking was found to be independent of cut off frequency. From this, it was concluded that duration of strong shaking is related to the time of rupture along the fault, as previously suggested by Dobry et al (1978). It was also noted that the distribution of energy over frequency intervals was uniform for the hard site and non-uniform for the soft site. The energy distribution of the Ancona 1972 earthquake, with a peak acceleration of 0.6 g, was considered and it was shown that an ordinary building would have stored only 5% of the total energy per unit weight in two seconds at the end of the ground motion. The authors claim that the effective peak acceleration should therefore be taken considerably lower than 0.6 g.

Finally, the authors state the past records could be interpreted more meaningfully in terms of "behavior" of a specific location rather than in terms of site conditions.

(i) "Local Soil Effects and Liquefaction in the 1978 Thessaloniki Earthquakes", by George Gazetas and John Botsis. Accelerograms at the basement of two structures, one a high rise building laid on a spread footing and the other a shallow church building, were recorded. Soil conditions under both structures were assumed to be practically the same. The response spectra of these two motions were compared in order to study the soil-structure interaction. The motion obtained at the basement of the church building was assumed to be the same as that of a free field, i.e., the church building is too small in size to have any effect on the ground motion. The peak spectral acceleration of the high rise building site was found to be very much lower than that of the church building site. This is explained to be due to the soil-structure interaction.

In support of this statement, one could cite the recent shake table test of a tall steel frame building carried out in Berkeley. This test showed a reduction in lateral forces, and thus the spectral acceleration, due to the vertical motion that takes place when the building is not rigidly attached to the soil foundation in the vertical direction. Another argument to support this statement is that due to the formation of a local plastic zone at the base, there could be dissipation of energy leading to lower spectral acceleration values.

The observation made would have been conclusive if accurate soil profile data were available at both sites. Finally, it would be interesting to see if this observation agrees with the behavior predicted by current analytical procedures.

A liquefaction analysis of another site was carried out based on standard penetration measurements. The author concludes that the vulnerability of a structure due to liquefaction of the supporting soil may not be realistically assessed with available empirical procedures which are based on standard penetration measurements and yield information of the performance of a small volume of soil.

SUMMARY

Among the papers which contributed to the state of the art in earthquake engineering, some useful observations have been made.

Dynamic compaction and pile driving operations have both been shown to increase the susceptibility to liquefaction of soil.

An interesting alternative to the elastic rebound theory of induced seismicity is given. Actual cases of reservoir induced seismicity are cited and help to explain this mechanism.

Field tests were carried out to try and verify the theory of wave propagation, an important aspect of earthquake engineering. Certain trends helped to confirm this theory.

Case histories of some recent earthquakes have also been reported, giving valuable information to help our understanding of seismic phenomena.

Through excellent observational data from the 1976 Tangshan earthquake, engineers have provided an explanation of surface faulting and source mechanism. Guidelines have also been given for estimating the dynamic strength of clay in terms of the static strength required to prevent plastic deformation and building settlement.

Observation and testing at Enmedio Island, Mexico, during March 1979 provided evidence that SPT data is not only unreproducible but also insufficient to explain the behavior of some sites during an earthquake.

Information obtained from Ohgishima Island near Tokyo in June 1976 showed that sites with the same relative density could produce different SPT data. This makes the accuracy and validity of the Holtz and Gibbs chart questionable. Measurement of soil properties must be made using all available methods: cone penetrometer, pressuremeter, cross-hole technique and other non-destructive tests. Correlations must then be established between actual field observations and the parameters obtained by these tests.

Investigators of the 1976 Friuli earthquake have suggested that the evaluation of site response be based upon the energy associated with different frequencies rather than a classification of "hard or soft" sites.

The Thessaloniki earthquake of 1978 provided evidence that soil-structure interaction can modify the ground motion. It was also pointed out here that SPT measurements may not realistically assess the vulnerability of a building due to liquefaction of the underlying soil.

In conclusion, it should be stated that, although a fair amount of observational data has already been obtained from various recent earthquakes, there is a great need for more complete data from well instrumented sites. In addition to this, the current analytical procedures need to be verified under controlled laboratory conditions. One way of achieving this is to carry out earthquake simulation studies in the centrifuge.