FRIULI EARTHQUAKES 1976
STRONG MOTION ACCELEROGRAPH RECORDS

by

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SUMMARY

This paper presents the basic tectonic and seismological characteristics of Friuli as well as the data of recorded accelerograms of the earthquakes that occured on May 6. and September 15, 1976 and their stronger aftershocks.

The earthquakes which occured during 1976 in Friuli originate from the known seismogene zone Friuli-Carnia. In this zone, which is the place of intersection of the regional longitudinal faults with the transversal fault zone connected to the valley of the Taligamento River, expressive geological distructions occured during the latest geological history.

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1. INTRODUCTION

Structure design criteria for zones of high seismic activity should be defined from the data concerning ground motion and response of structures during strong earthquakes. The strong motion phenomenon involves almost always numerous questions which cannot be answered exactly due to the lack of instruments capable to record the earthquake intensities and response of structures. Without such a record, the damage and behaviour of structure during strong earthquakes cannot be compared to the seismic design criteria nor proper decisions concerning rational repair and reconstruction could be made. Special difficulties arise when earthquakes do net create large visible damage. In such a case, the usefulness of the structure should be checked by special and expensive inspection.

The only procedure for obtaining such data is installation of strong motion instruments. Therefore, in 1973, Yugoslavia was covered by a strong motion instrument network consisting accelerographs and seismoscopes (Fig. 1), which was realised by the Institute of Earthquake Engineering and Engineering Seismology at the University "Kiril and Metodij" in Skopje.

In Yugoslavia the Friuli earthquakes were felt in an area with a radius of 250km from the epicentres. As it is evident in Fig. 1, this area had been covered by a large number of accelerographs and seismoscopes which were in an operating state during the earthquakes. Within few days after the earthquake of May 6. three temporary accelerographs were additionally installed at the following locations: Breginj, Kobarid and Robic (Fig. 2), which were most severely affected by the earthquake. A number of aftershocks have triggered these instruments.

The earthquakes of May 6. and September 15, 1976 that occured in Friuli, northeast Italy, close to the Italian-Yugoslav border, and the numerous aftershocks have caused enormous loss in goods and human lives, particularly in Italy. In Yugoslavia, these earthquakes created considerable damage in material properties, especially in the north-western part of Slovenia (Posocje), where some villages as Breginj, Podbela and others were completely destroyed.

2. THE YUGOSLAV STRONG MOTION INSTRUMENT NETWORK

One of the largest strong-motion instrument networks in Europe is the accelerograph and seismoscope network installed in Yugoslavia. This network of instruments provides basic information required for predicting the dynamic response of various types of structures, building Code´s improvement, understanding of the ground effects, as well as for better investigation and perceiving of the consequencies caused by earthquakes.

The distribution of the strong-motion instruments on the territory of Yugoslavia is based on previous studies of the geological-tectonic structure and the seismicity of the country. According to these studies it has been concluded that, in the first stage of realization of this project, the instrument network for recording of strong earthquakes should contain 136 accelerographs, 150 seismoscopes and a certain number of accelerographs for recording of maximum acceleration.

The selection of detail locations (Fig. 1) for installation of these instruments makes possible to obtain records on: 1) bedrock, 2) on the surface of a characteristic soil (alluvial and deluvial sediments), 3) on structures (multistorey buildings, dams, bridges etc.).

In this way, the designed network enables obtaining of such data by measuring the radiation pattern of the seismic waves, the effects of the soil conditions and the response of structures.

The Yugoslav network of strong motion instruments mainly contains two types of instruments: accelerographs SMA-1, produced by Kinemetrics, USA, and seismometers of WM-1 type, manufactured by the Astronomic-Geophysical Observatory of Ljubljana, Yugoslavia.

3. SEISMICITY AND TECTONICS OF FRIULI

The region of Friuli-Carnia which includes the area from the east of Veneto to the Yugoslav border and from the Austrian border to the Adriatic Sea, is well known seismic zone which during this century has been affected by strong earthquakes with magnitudes M = 5.0, several times.

This region is subdivided into the Carnic Range, Tolmeco Alps, Julian Alps in Italy, Carnic Pre-Alps, Julian Pre-Alps in Italy and the Friuli plane with Gorizia and Trieste Karst.

Carnic Pre-Alps and Julian Pre-Alps region with an adjucent part of the Friuli plane for the pleistoseistic part of the destructive May 6, 1976 earthquake, as well as September 15, 1976 events and the subsequent series of aftershocks with strong evidence of migration of foci and a specific energy release. Aftershock series of both events show strong variation in numbers, magnitude distribution, variation of coordinates in space and possibly in foci mechanisms.

The tectonics of the Friulian Alps is rather complex and varies strongly from zone to zone. The more conspicious structural elements that crop out in this region show immediately their predominantly east-west direction, in accordance with the general course of the Southern Alps. Eastwards, this direction is inclined towards NW-SE, which becomes striking and dominant in some parts of the Julian Pre-Alps and the Gorizia-Trieste Carst, where the orientation is typical Dinaride (Martines, 1975).

According to Ribaric (1977) the seismicity of this region is as follows:

Friuli-Carnia region is subjected to strong earthquakes which can be traced back for centuries. Some of the events reached a magnitude well over M = 6. Earthquakes in the Friuli-Carnia zone east from Dolomites (Veneto zone) are mainly concentrated around a triabgle formed by Tolmezzo zone, Tramonti di Sopra and Gemona zone. Other typical earthquake zones in this region are the zones of Cividale (Cedad) Montreale and some unequally distributed zones west of Udine (Videm).

To the west of Friuli-Carnia region in Veneto, there are several seismogene zones with potentials of producing seismic events with intensities 9°MCS or bigger (region Elago di S.Croce, Gansiglio, Liedolo). The south-western part of the North-Eastern Italy is also seismic, most of the events being attributed to the seismic activity in the Earth´s crust beneath the valley of Po river.

To the east, in Yugoslavia, typical seismogene zones in continuation with the west-east directed faults of the Friuli Pre-Alps and Julian Pre-Alps are found in the region of Tolmin, whereas southwards this direction changes into north-west-southwest (Dinaride direction). Zones of Ljubljana, Idrija, Ilirska Bistrica-Klana and Rijeka are typical earthquake origin areas with possible magnitudes over 6,5 but however with a small possibility of occurence. As evident from the time distribution of strong shocks in the region, calm periods exist, however they are lately followed by strong seismic motions (Fig. 3). After the series of 1959 events, no stronger earthquakes were recorded in the period from 1963 to 1975. Finally, the energy release culminated in the May 6, 1976 earthquake and the subsequent earthquakes of the fall 1976 and string 1977 (Fig. 4).

Similar situation is noticed after the April 1939 event in Moggio Udinese. After 15 year calm period, stronger seismic activity was observed, culminating in Tolmezzo in April 26, 1959 earthquake.

In the period from 1950 to June 18, 1975 21 earthquakes have been recorded in a zone with radius of 25 km from the supposed epicenter of the May 6, 1976 earthquake. In spite of the fact that the data about epicentral coordinates are possibly far from being accurate, it is evident that in general, the active seismic periods in this region are followed by calm periods, in support to the C.F.Richter's definition of sporadic seismicity (1971).

The picture of irregularity of occurence of stronger earthquakes is also persistent in the case of larger time intervals and areas under consi-eration and it is valid for this particular region, too.

4. STRONG MOTION ACCELEROGRAPH RECORDS

In the period from May 6. through September 15, 1976 32 accelerograph records and 6 seismoscope records have been obtained from the Yugoslav strong motion network.

Table 1. gives adata of the more important recorded earthquakes with maximum acceleration lartger than 5% g.

EARTHQUAKE DATA TABLE 1.

Accelero-graph Station	Date	Time H	Time M	Epicentre Coordinates		M	Epic. Dist. App.km	Max. Rec. Acc.(g)
Ljubljana INFIM	May 6,76	20	00	46.31 N	13.31 E	6.2	100	0.040
Ljubljana ZIMK	May 6,76	20	00	"	"	6.2	100	0.022
Breginj	June 8,76	12	14	46.28 N	13.27 E	4.3	25-30	0.103
Kobarid	Sept.11,76	16	31	46.32 N	13.18 E	5.4	35-40	0.101
Breginj	Sept.11,76	16	31	"	"	5.4	25-30	0.171
Kobarid	Sept.11,76	16	35			5.5	35-40	0.099
Breginj	Sept.11,76	16	35			5.5	25-30	0.123
Robic	Sept.11,76	16	35			5.5	30-35	0.057
Robic	Sept.13,76	18	55			4.1	30-35	0.050
Breginj	Sept.15,76	03	15	46.32 N	13.16 E	5.9	25-30	0.525
Kobarid	Sept.15,76	03	15	"	"	5.9	35-40	0.126
Robic	Sept.15,76	03	15	"	"	5.9	30-35	0.105
Breginj	Sept.15,76	04	39			4.6	25-30	0.071
Kobarid	Sept.15,76	09	21	46.33 N	13.17 E	5.7	35-40	0.143
Breginj	Sept.15,76	09	21	"	"	5.7	25-30	0.419
Robic	Sept.15,76	09	21	"	"	5.7	30-35	0.088

4.1 The Earthquake of May 6, 1976

Fig. 5 shows the isolines of the May 6, 1976 earthquake. As it can be seen this earthquake was felt on a larger territory of Yugoslavia. The same figure shows also the state of the strong motion instrument network in Yugoslavia during the earthquake. A total number of 5 accelerographs and 7 seismoscopes had been installed in the area where the earthquake was manifested with I $\geq$ VI degrees according to ~~the MCS~~ scale.

The earthquake of May 6, 1976 activated 2 accelerographs and 4 seismoscopes altogether. Both activated accelerographs are located in Ljubljana on a distance of cca 100 km from the epicenter of the earthquake. The peak acceleration of the earthquake recorded in Ljubljana was 0.04 g (Fig. 4). Table 1. shows the basic data of this earthquake obtained from the records of these accelerographs and the seismological data obtained from the records of the Yugoslav seismological stations in Ljubljana, Zagreb and Skopje. This earthquake has also been recorded by two seismoscopes installed inTolmin at a distance of cca 40 km and two in Ljubljana at a distance of cca 100 km from the epicentre. Fig. 7 shows the seismoscope records obtained inTolmin and Ljubljana.

On one of the seismoscopes, type WM-1 manufactured by Astronomic Geophysical Observatory in Ljubljana, installed in Ljubljana a record with double amplitude of 1.8 sm was obtained. If we consider that the natural period of the seismoscope is 0.75 sec. damping 10% from the critical, and the sensitivity of the instrument of about 6.0 sm/rad. the obtained value of the acceleration spectrum of Sv = 17.6 sm/sec corresponds to the intensity of an earthquake of 7 degrees according to the MCS scale (Ribaric 1977). The evaluated intensity in Ljubljana of 6-7 degrees completely corresponds to this record, which, as it can be seen on Fig. 7 has an irregular shape. The instrument was installed in a one-story buolding with foundations in a relatively bad soil conditions. The composition of the undersurface structure of the terrain is the following: clay, marshy soil with clay intercalations and high ground water table (h = 0.5 + 1.0 m.). On the same instrument was obtained a record from the earthquake of September 15, 1976 at 09 hour and 21 min. The value of the acceleration spectrum of this record was Sv = 3.9 sm/sec. which is several times smaller than that of May 4, 1976, although the magnitude was M = 5.7 in comparison to 6.2 of the eartquake of May 4, 1976.

4.2 Some Bigger Aftershocks During the period May 6, to September 15, 1976

Considerable number of aftershocks with magnitude of M = 3.6 to M = 5.5 which occurred during the period from May 10 to September 15, 1976 have been recorded by the additianally installed accelerographs in Breginj, Kobarid and Robic.

Theble 2. gives data on the location and number of triggering of each accelerograph separately, during this period, as well as the maximum recorded acceleration of these aftershocks.

TABLE 2.

Instrument Location	Type of Soil	Approximate Epicentral Distance	Number of triggering	Maximum recorded acceleration	
				Horiz.	Vert.
Breginj	Soft glacial morene deposits h=5.10 m. above limestone	25-30	4	0.171	0.052
Kobarid	Soft -Dolomite with 0.5 to 2.5 proluvial deposits	35-40	8	0.101	0.058
Robic	Rock -Hard massive limestone (Dolomite)	30-35	4	0.057	0.025

The maximal acceleration at Breginj and Kobarid was caused by the aftershocks of September 11, 1976 at 6.31 p.m. with M = 5.4 and at Robic by the aftershock of September 11, 1976 at 4.35 with M =5.5. Seismoscope records of the above aftershocks are not available.

The remaining instruments located on an area in radius of 100 km from the epicenter of these aftershocks were not triggered. All instruments were in a good working condition controlled -ach month.

4.3 The Earthquake of September 15, 1976

The earthquake which occured on September 15, 1976 was the strongest one after the earthquake of May 6, 1976. Both strong earthquakes at 3.15 a.m. (M = 5.9) and at 9.21 a.m. (M = 5.7) as well as the numerous aftershocks which occured at the same day caused great material damage in Italy as well as in Posocje (Yugoslavia). The occurence of these earthquake was a surprise both for the seismologists and earthquake engineers because such events in this part of Europe are not known in the seismological history.

These earthquakes were also recorded by the accelerographs installes at Breginj, Kobarid and Robic, and the seismoscopes at Tolmin and Ljubljana.

Five earthquakes were recorded at Breginj, six at Kobarid and three at Robic, all in September 15, 1976.

TABLE 3.

Location of accelerograph	Epicentral distance in (km)	Magni-tude	Maximal acceleration (g)		
			N-S	V	E-N
	Earthquake of September 11, 1976 at 3.15 a.m. (GMT)				
Breginj	25-30	5,9	0.487	0.198	0.525
Kobarid	35-40	5,9	0.108	0,088	0,126
Robic	30-35	5,9	0.105	0.049	0.074
	Earthquake of September 15, 1976 at 9.21 a.m. (GMT)				
Breginj	25-30	7,7	0.166	0.126	0.419
Kobarid	35-40	5,7	0.143	0.065	0.109
Robic	30-35	5.7	0.088	0.046	0.058

The records of the earthquake of 3.15 a.m. (M = 5.9) and 9.21 a.m. (M = 5.7) which were recorded by all the three accelerographs at the same time are of special interest Table 3. shows the data of these earthquakes.

As it can be seen from the above table, the instrument at Breginj recorded two very important records (Figs. 8 and 9) with maximum acceleration of 0.525 g i.e. 0.419 g, which is of special interest for the earthquake engineers. Further more, because the instrument is located at a distance of cca 25-30 km from the epicenter in a structure which didnot suffer significant structural damage due to these earthquakes, while all other structures in Breginj were completely destroyed. At the same time, at Kobarid and Robic, which are at almost the same distance from the epicenter, acceleration of 0.126 g and 0.109g, i.e. 0.105 g and 0.088 g was recorded.

5. REZULTS FROM THE ANALYSIS

In the Institute of Earthquake Engineering and Engineering Seismology in Skopje has been developed a standard procedure for strong motion data processing, taking as base the standard procedure developed at CALTECH - Pasadena, aimed to obtain more exact information. Some changes have been made due to use of different equipment for digitalization and calculation.

Applying this procedure several records of the Friuli earthquakes obtained in Yugoslavia, which are of interest for the engineering practice, have been analysed. The seismological characteristics of these records are presented in Table 3.

Fig.10 through Fig.13 illustrate the response spectra of the Friuli Earthquakes of September 15, 1976, taken at Breginj, Kobarid and Robic.

All these investigations add to the fact that more studious investigations of the mechanism of these earthquakes, of the radioation pattern of the seismic waves, of the acceleration attenuation depending on the epicentral distance, as well as of the effect of the local soil conditions on the acceleration amplification, is necessary.

ACKNOWLEDGEMENT

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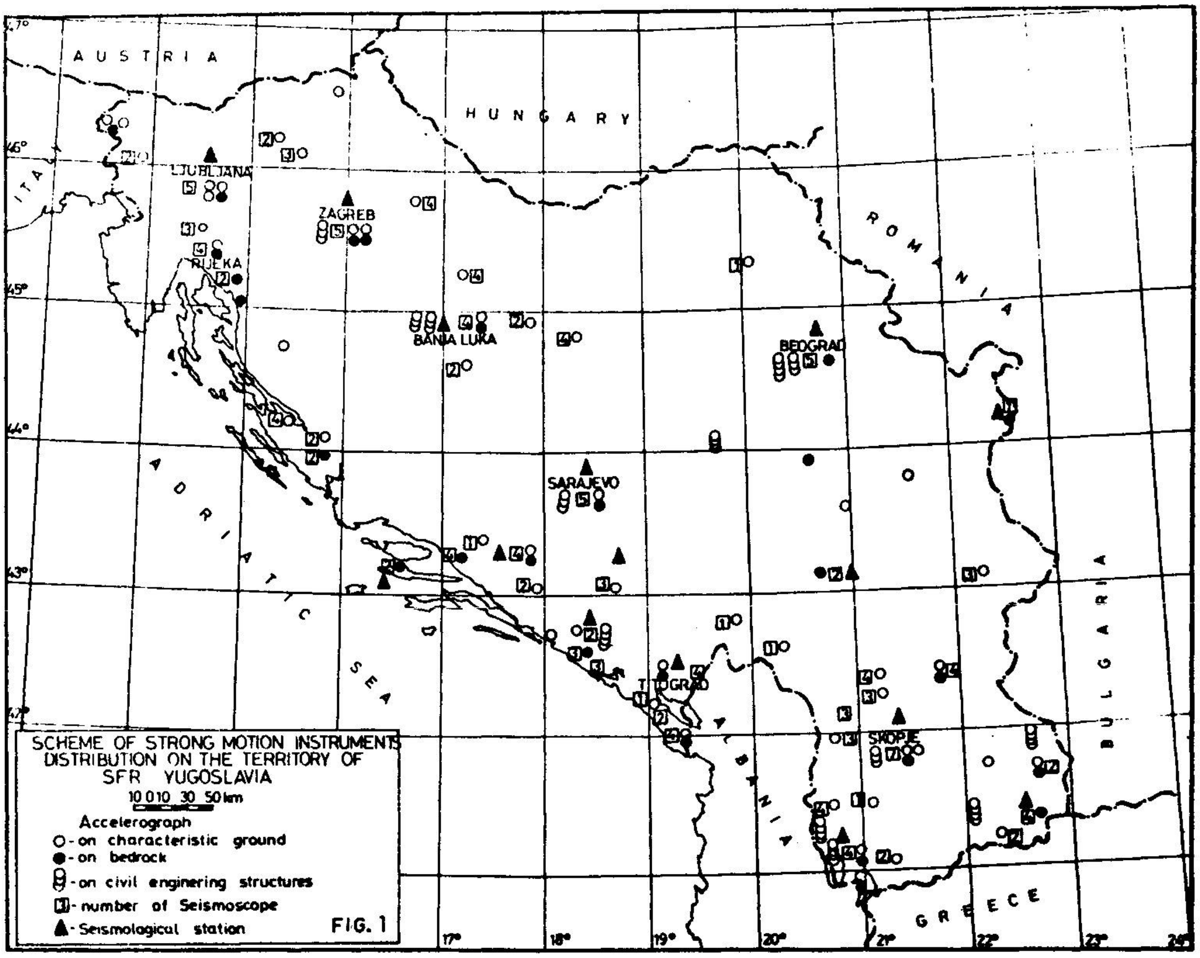

SCHEME OF STRONG MOTION INSTRUMENTS DISTRIBUTION ON THE TERRITORY OF SFR YUGOSLAVIA
10 0 10 30 50 km
Accelerograph
- on characteristic ground
- on bedrock
- on civil enginering structures
- number of Seismoscope
- Seismological station
AUSTRIA
HUNGARY
ROMANIA
BULGARIA
GREECE
ALBANIA
ADRIATIC SEA
LJUBLJANA
ZAGREB
RIJEKA
BANJA LUKA
BEOGRAD
SARAJEVO
TITOGRAD
SKOPJE
17°
18°
19°
20°
21°
22°
23°

FIG. 1

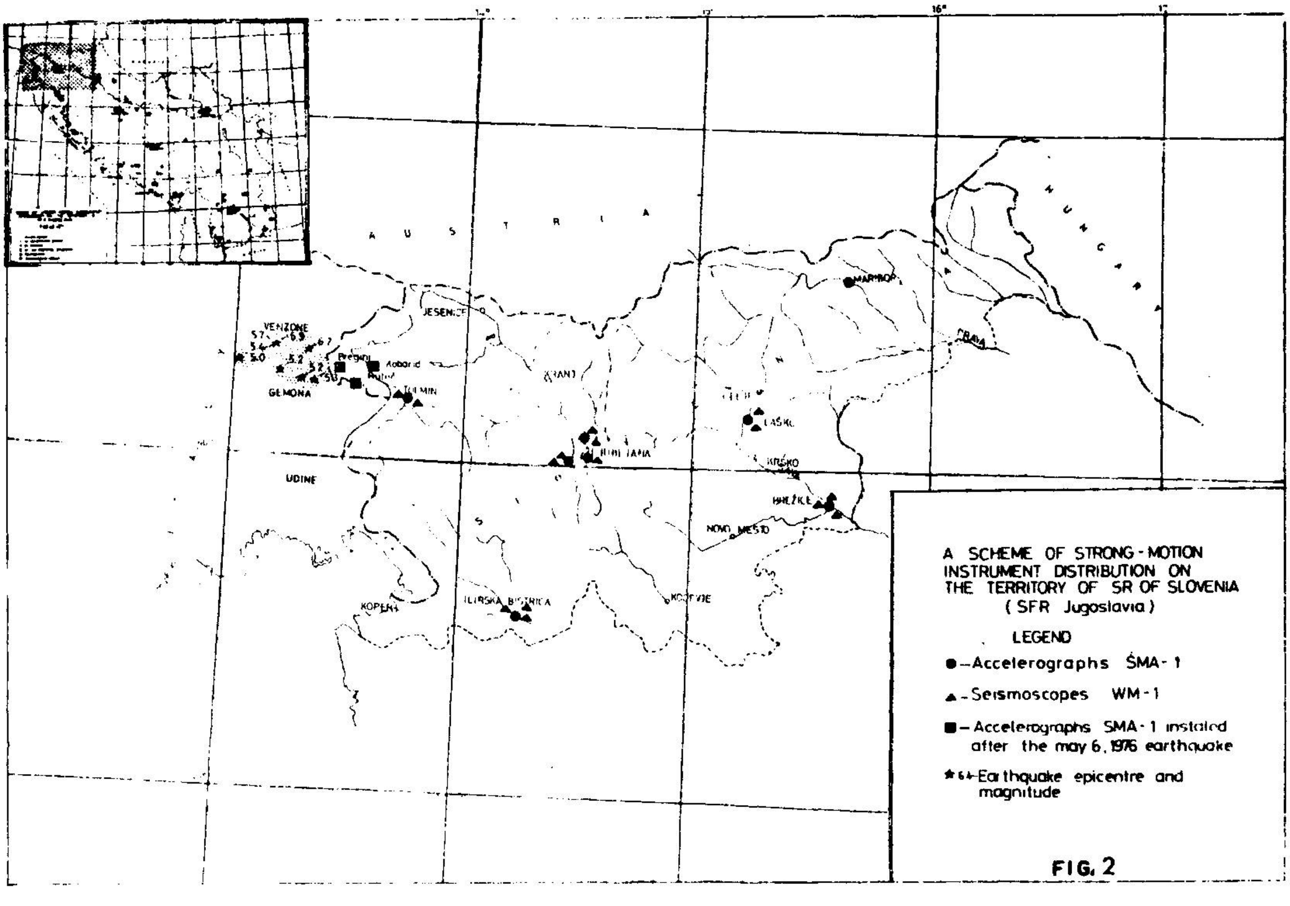

A SCHEME OF STRONG-MOTION INSTRUMENT DISTRIBUTION ON THE TERRITORY OF SR OF SLOVENIA (SFR Jugoslavia)
LEGEND
●–Accelerographs SMA-1
▲-Seismoscopes WM-1
■–Accelerographs SMA-1 instaled after the may 6, 1976 earthquake
★6.4-Earthquake epicentre and magnitude
AUSTRIA
HUNGARY
VENZONE
GEMONA
UDINE
TOLMIN
JESENICE
KRANJ
LJUBLJANA
CELJE
LAŠKO
MARIBOR
KRŠKO
BREŽICE
NOVO MESTO
KOPER
ILIRSKA BISTRICA
KOČEVJE
16°

FIG. 2

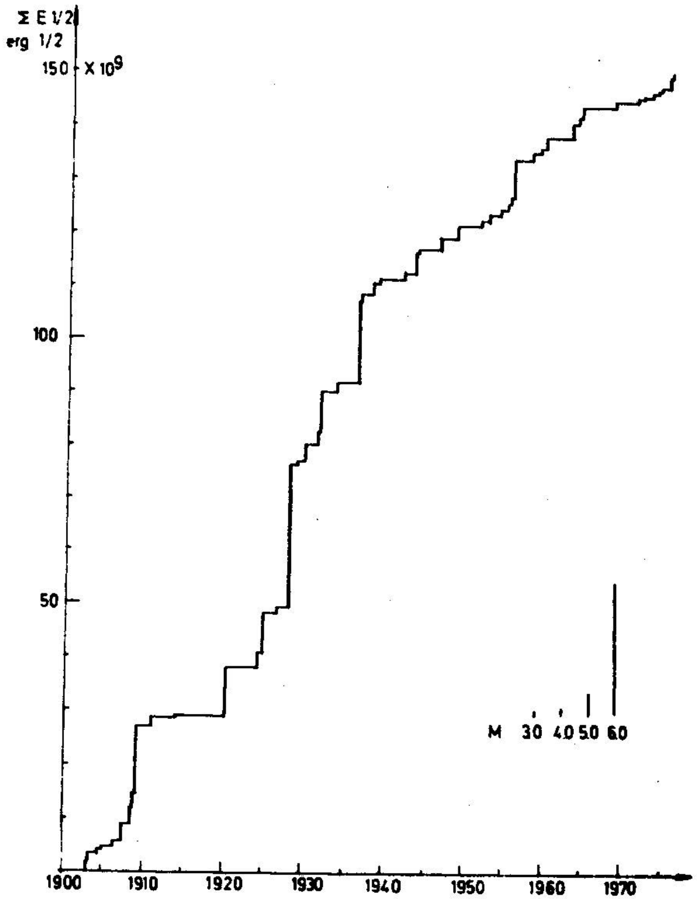


Fig. 3 STRAIN RELEASE CURVE (1900 - 1976)
(After D.Cvijanovic)

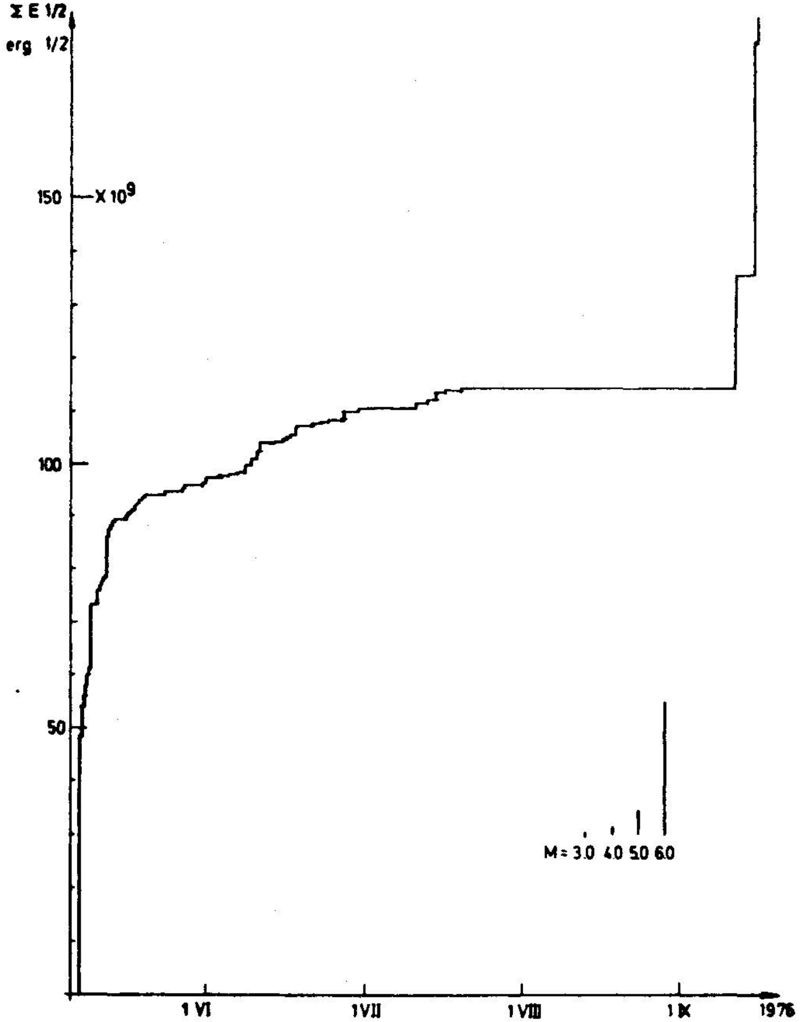


Fig. 4 STRAIN RELEASE CURVE (May 1st to Sept. 15, 1976)
(After D. Cvijanovic)

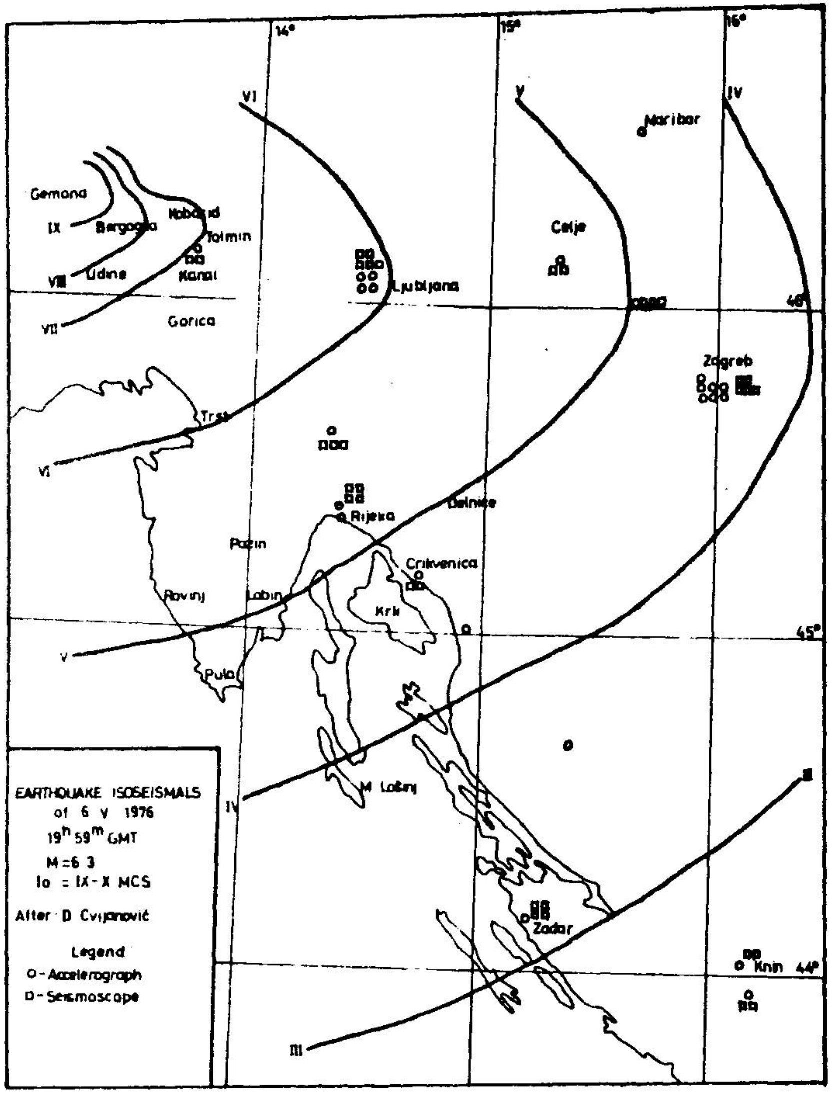

14°
15°
16°
46°
45°
44°
IX
VIII
VII
VI
V
IV
III
Gemona
Bergogna
Kobarid
Tolmin
Udine
Kanal
Gorica
Ljubljana
Celje
Maribor
Zagreb
Trst
Rijeka
Delnice
Pazin
Crikvenica
Rovinj
Labin
Krk
Pula
M Lošinj
Zadar
Knin
EARTHQUAKE ISOSEISMALS
of 6 V 1976
$19^h 59^m$ GMT
M = 6 3
Io = IX-X MCS
After D Cvijanović
Legend
o - Accelerograph
□ - Seismoscope

FIG 5

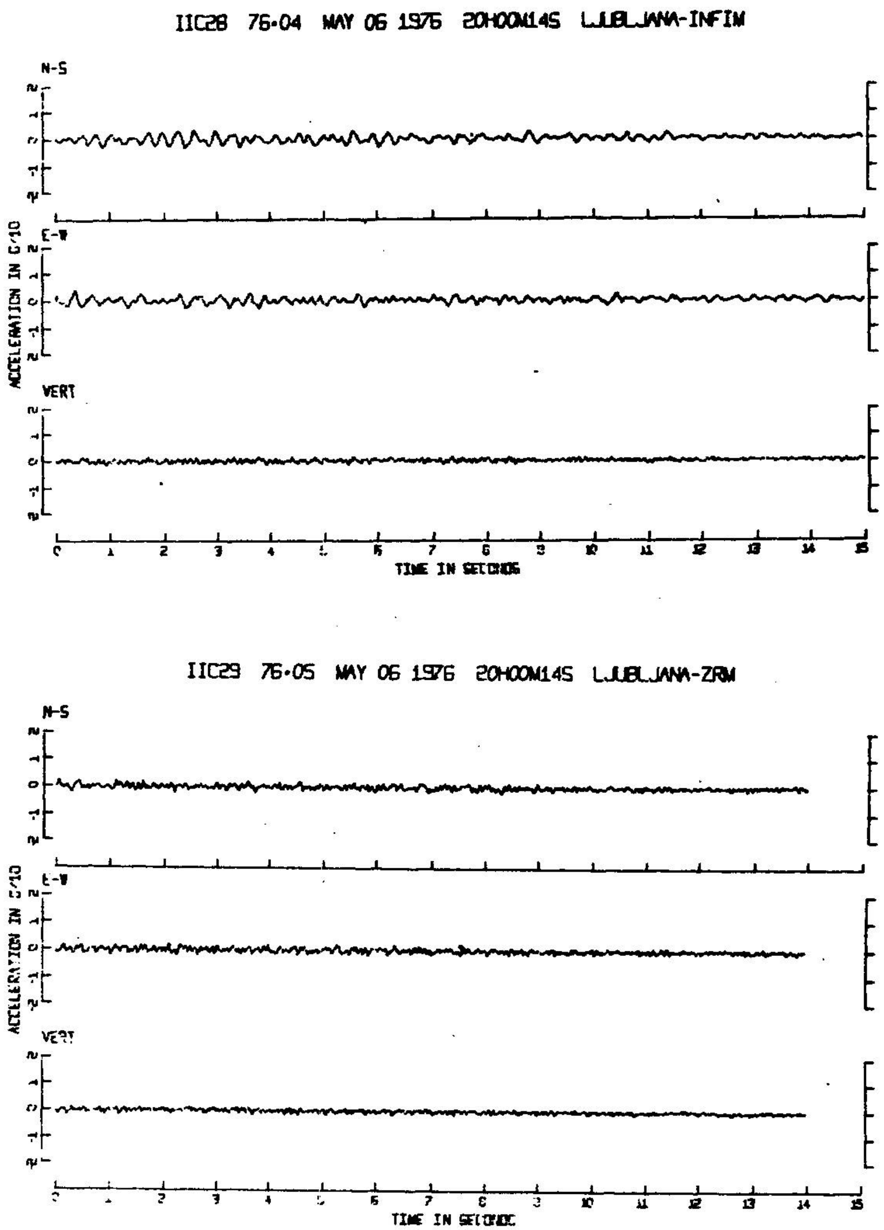

IIC28 76-04 MAY 06 1976 20H00M14S LJUBLJANA-INFIM
N-S
E-W
VERT
ACCELERATION IN G/10
TIME IN SECONDS
IIC29 76-05 MAY 06 1976 20H00M14S LJUBLJANA-ZRM
N-S
E-W
VERT
ACCELERATION IN G/10
TIME IN SECONDS

FIG-6

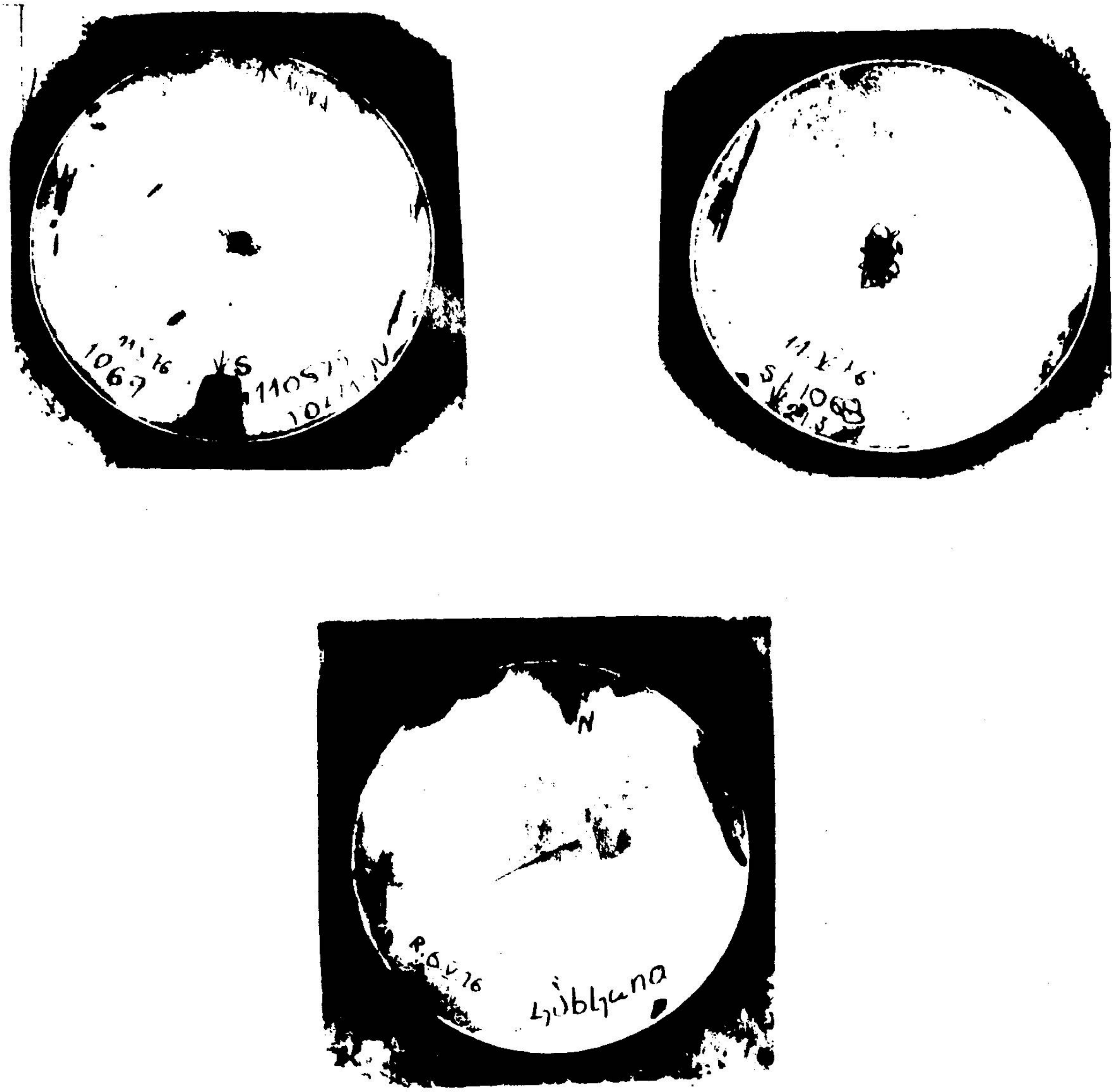
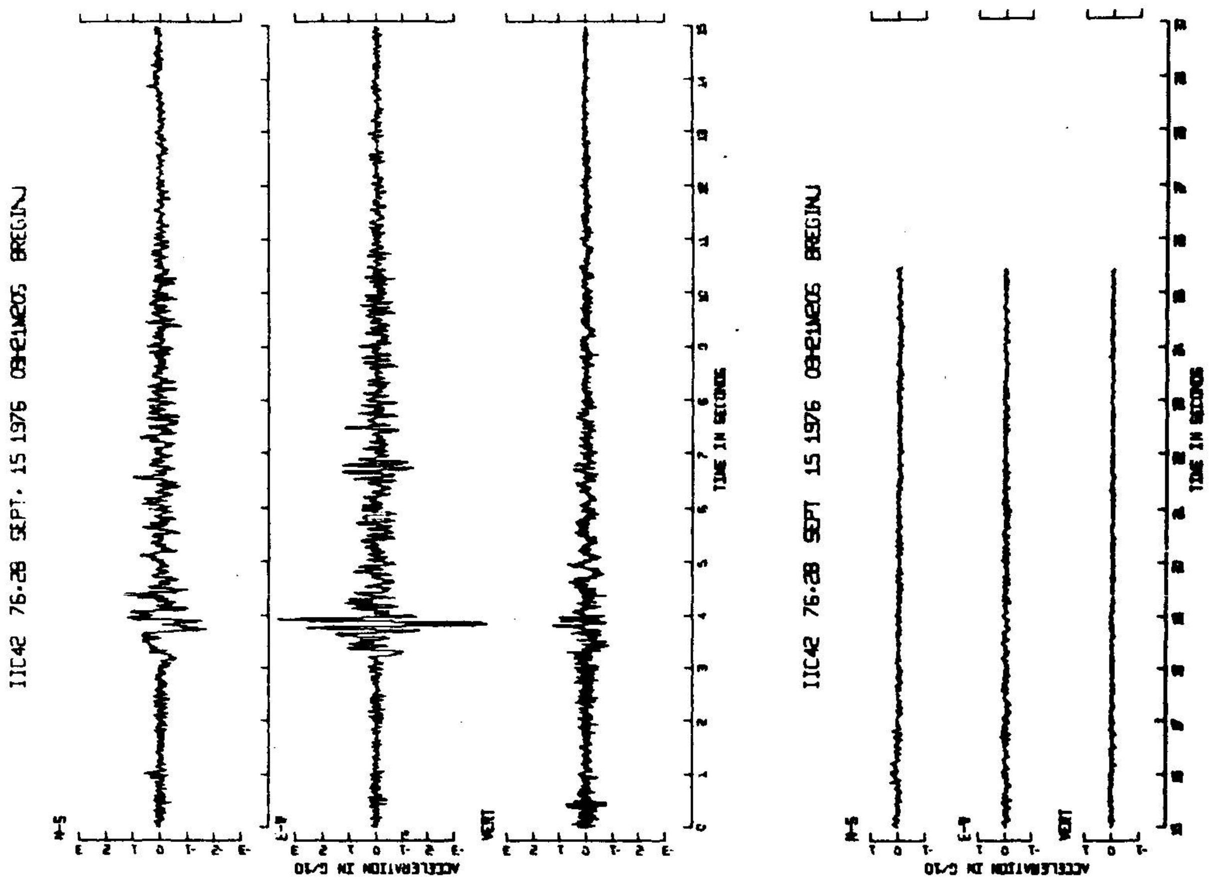
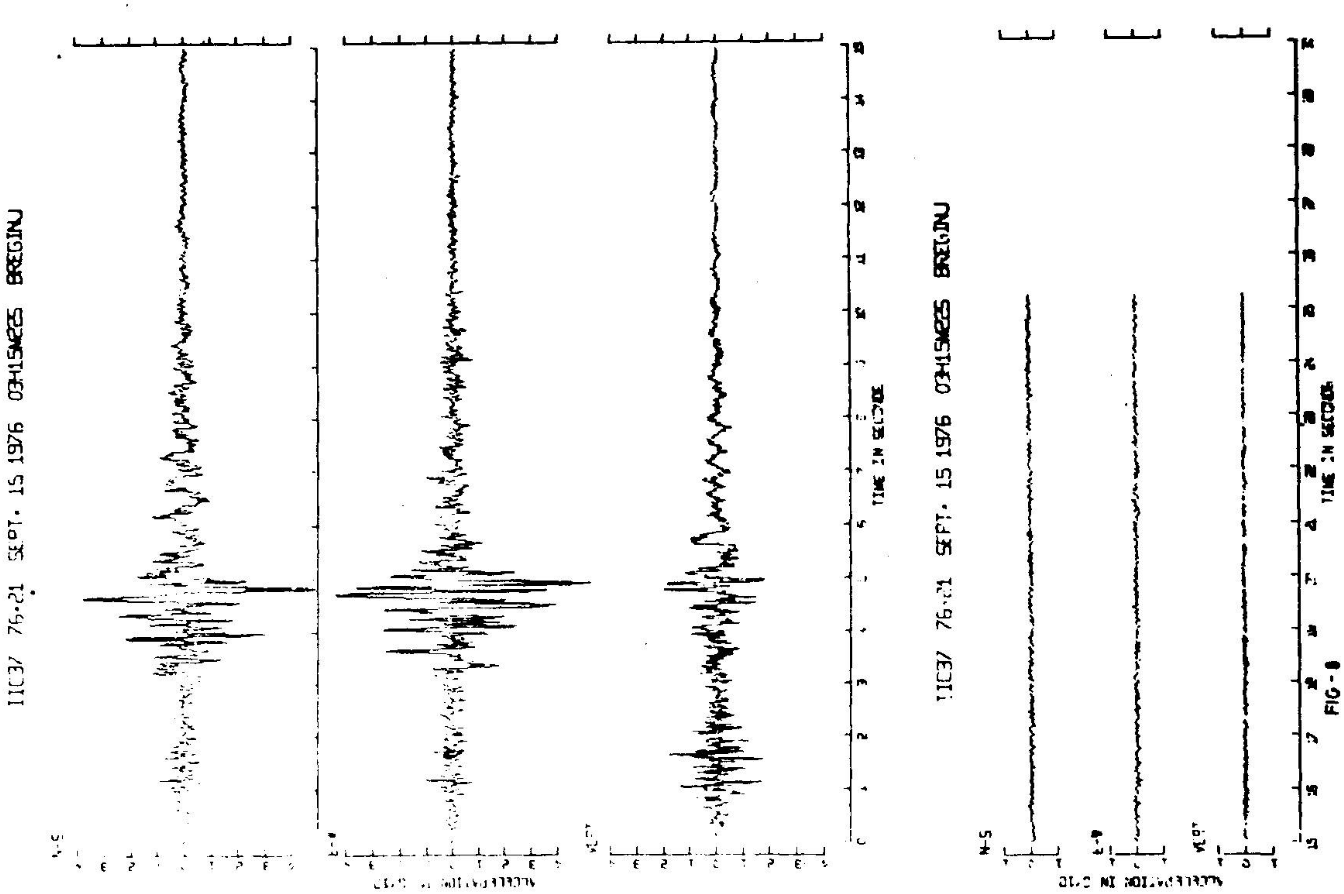


Fig. 7 FRIULI EARTHQUAKE, May 6, 1976
Recordings of Seismoscopes
(1) Tolmin - Town Council
(2) Tolmin - Fire House
(3) Ljubljana - 98 Pokljukarjeva Street



IIC42 76-28 SEPT. 15 1976 03H21M20S BREGINJ
N-S
E-W
VERT
ACCELERATION IN G/10
TIME IN SECONDS

FIG-9



IIC37 76-21 SEPT. 15 1976 03H15M22S BREGINJ
N-S
E-W
VERT
ACCELERATION IN G/10
TIME IN SECONDS

FIG-8

FRIULI EARTHQUAKE, ITALY, SEPT. 15, 1976, 03h 15m, RECORDED IN BREGINJ - YUGOSLAVIA
RELATIVE DISPLACEMENT AND PSEUDO ACCELERATION RESPONSE SPECTRA
Damping 2, 5 and 10 percent of critical

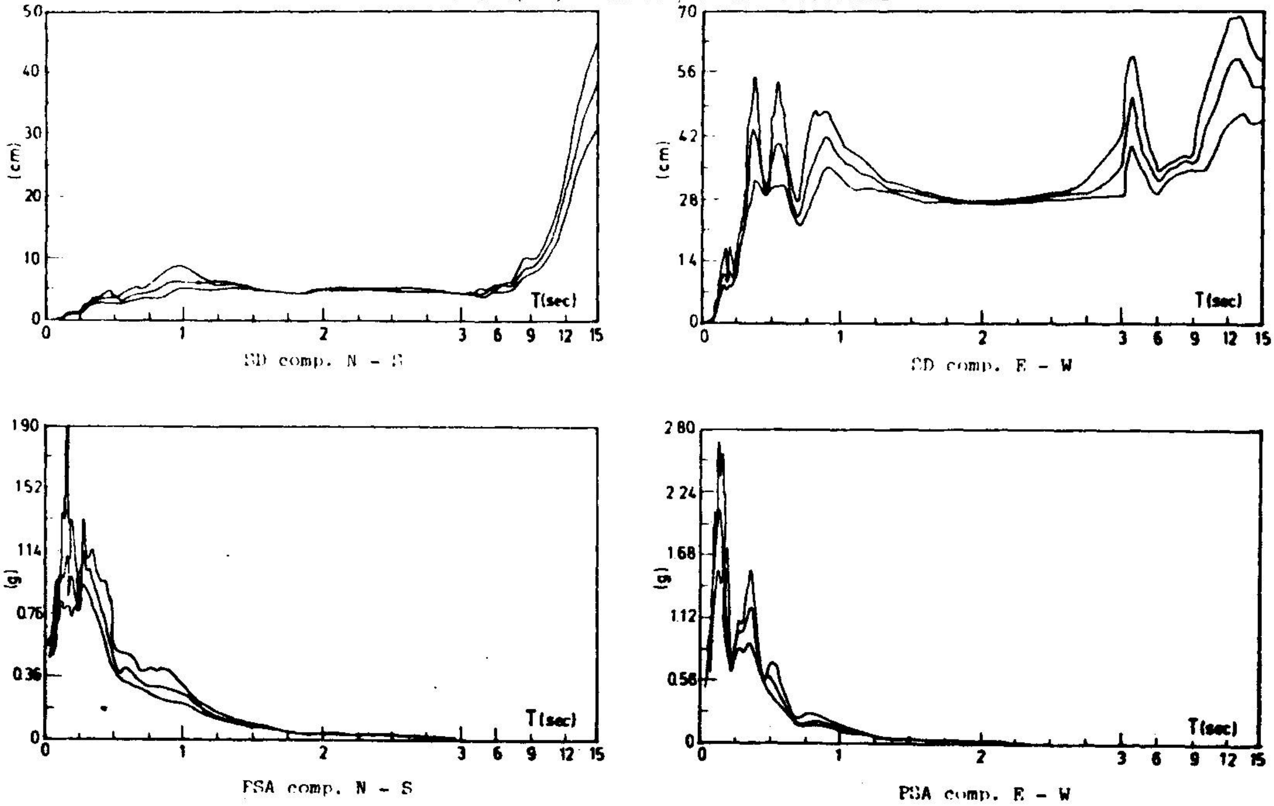


Fig. 10

FRIULI EARTHQUAKE, ITALY, SEPT. 4 1976, 00h 21m, RECORDED IN BREGINJ - YUGOSLAVIA
RELATIVE DISPLACEMENT AND PSEUDO ACCELERATION RESPONSE SPECTRA
Damping 2, 5 and 10 percent of critical

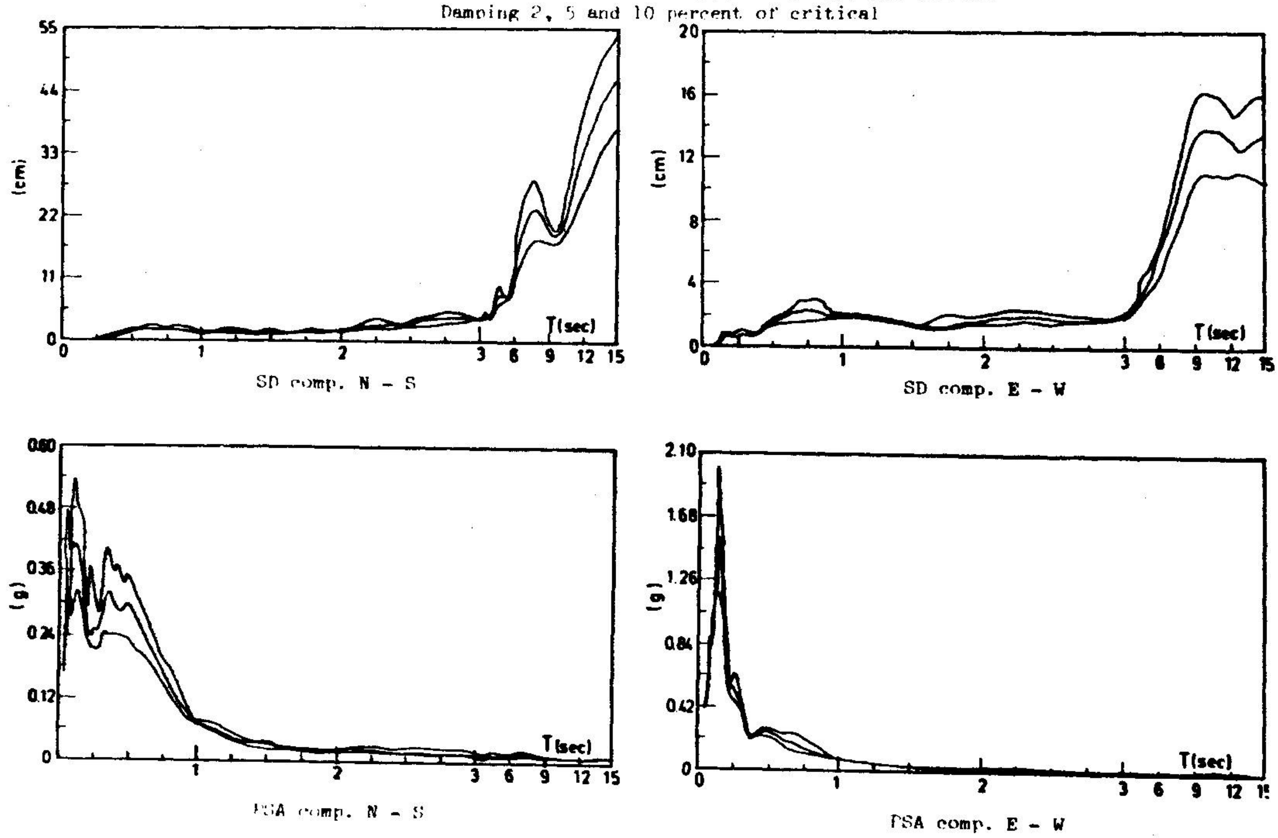


Fig. 11

FRIULI EARTHQUAKE, ITALY, SEPT. 15, 1976, 09h 21m, RECORDED IN KOBARID - YUGO[illegible]
RELATIVE DISPLACEMENT AND PSEUDO ACCELERATION RESPONSE SPECTRA
Damping 2, 5 and 10 percent from critical

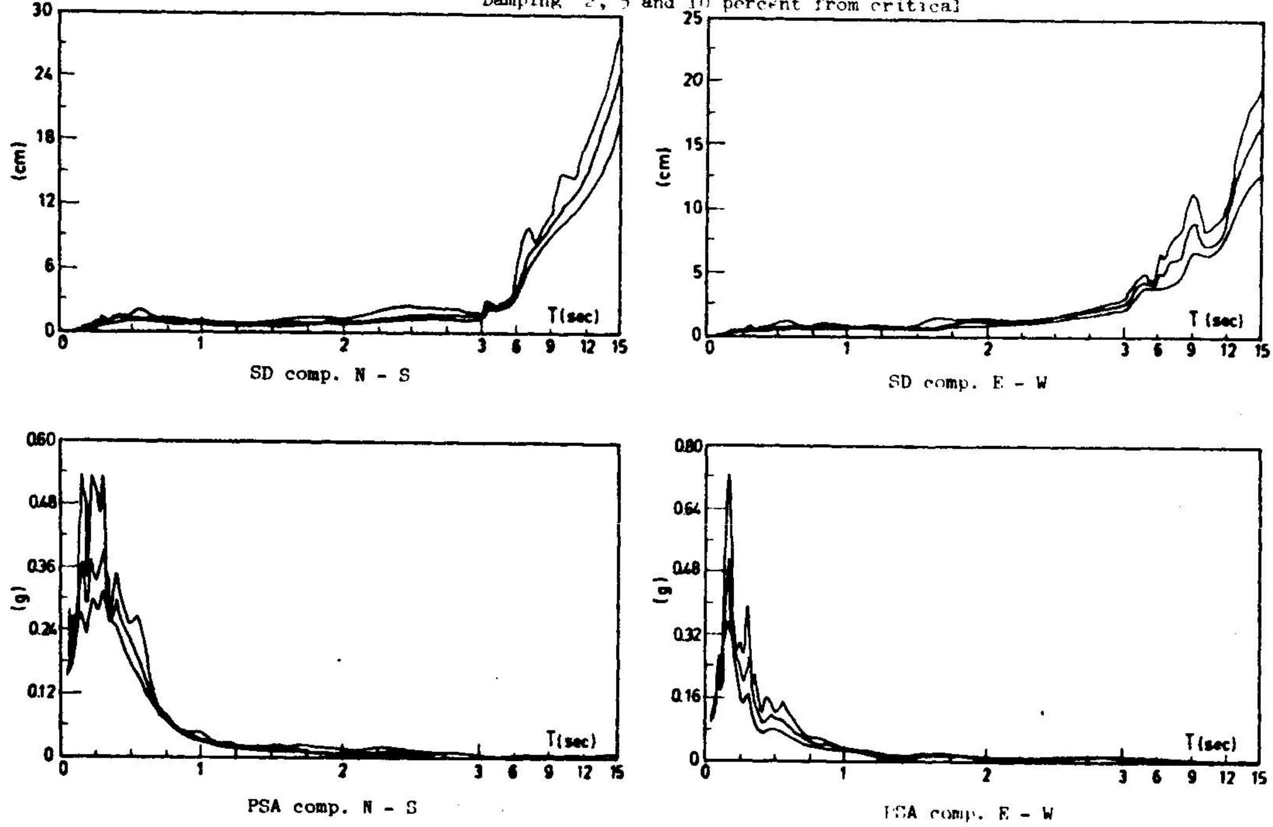


Fig. 12

FRIULI EARTHQUAKE, ITALY, SEPT. 15, 1976, 03 h 15m, RECORDED IN ROBIC - YUGOSLAVIA
RELATIVE DISPLACEMENT AND PSEUDO ACCELERATION RESPONSE SPECTRA
Damping 2, 5 and 10 percent of critical

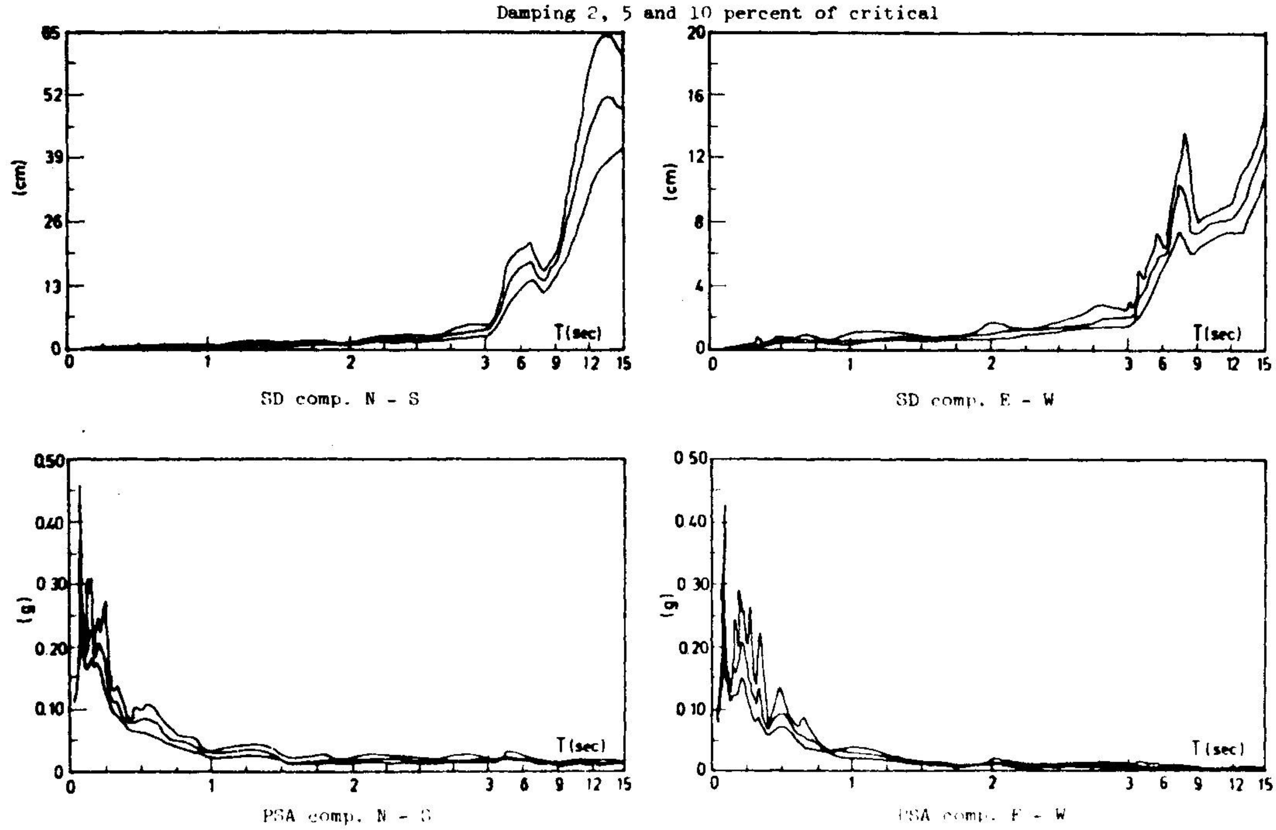


Fig. 13

STRUCTURAL BEHAVIOUR OF THE DAMAGED BUILDING DURING THE FRIULI EARTHQUAKES BETWEEN MAY 6 AND SEPTEMBER 15, 1976

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ABSTRACT

Between May 6th and September 15th, 1976, Friuli was hit by 18 earthquakes with a magnitude of M ≤ 3.8. The expansion of structural damage caused by the further strong shocks in September was investigated in those buildings (1) which had been closely examined during the first stay in the earthquake area (2).

Additional building damage depended to a large extent on behaviour during the first earthquake load as well as on the different kinds of repairs carried out in the meantime.

ABSTRAIT

Entre le 6 mai et le 15 septembre 1976 le Friaul a été ébranlé par 18 secousses sismiques de magnitude M ≤ 3.8. Les expansions des dégâts causés aux bâtiments par les séismes forts en septembre ont été examinées aux édifices bien connus à l'occasion de la première investigation dans la zone sismigue.

Les dégâts additionels causés aux bâtiments dépendaient considérablement du comportement pendant le premier chargement sismique ainsi que des différentes mesures de réparation réalisées entre-temps.

AUSZUG

Zwischen dem 6. Mai und dem 15. September 1976 wurde das Friaul von 18 Erdstössen mit Magnituden von M ≤ 3.8 betroffen. Die Ausweitung der Bauwerksschäden infolge der weiteren, starken Erdbeben im September wurden an den Bauwerken untersucht, die bereits nach dem Erdstoss vom 6. Mai 1976 sehr genau bekannt waren.

Die zusätzlichen Gebäudeschäden waren in sehr starkem Masse vom Bauwerksverhalten während der ersten Bebenbelastung wie auch von den in der Zwischenzeit durchgeführten unterschiedlichen Reparaturmassnahmen abhängig.

1. BACKGROUND

On May 6th, 1976, the Friuli region of Northern Italy was hit by a severe earthquake (Magnitude M = 6.5, measurement of energy released). The epicentral intensity I_o reached the IX to X mark on the XXII-part MSK Scale (Medvedev-Sponheuer-Karnik). This shock had a devastating effect on the buildings in the area.

At the beginning of June 1976, a group of Swiss engineers visited the damaged area. The results of their investigations and the evaluation of the knowledge accumulated is contained in an extensive report (3) with an amply documented description of damage. The geophysical characteristics of the earthquake, the causes of building damage and the behaviour of various building parts under earthquake load have been described in a further paper (2).

A considerable number of aftershocks followed the May 6th earthquake. Within the first two months, 150 aftershocks with epicentral intensities I_o = IV (MSK) were observed.

On September 11th, 1976, after a long period of relative calm, the area was hit by a further earthquake with an epicentral intensity of VII (MSK). This damaging earthquake was exceeded on September 15th by two further strong shocks with intensities almost equal to that of the May 6th earthquake. The earthquake at 4.00 a.m. reached an intensity of VIII (MSK) and that which followed, just after 10.00 a.m., IX (MSK). In order to supplement the impressions gained during the first damage investigations, the area was revisited after the two September earthquakes.

During the second stay in the Friuli earthquake area (1), which lasted from Friday, September 24th until Sunday, September 26th, 1976, the following aspects were investigated and evaluated:

- the process of damage expansion on previously damaged structures,
- the effects of various repair measures on the behaviour of buildings during renewed earthquakes.

In order to distinguish the damage and destruction caused by the two series of earthquakes of May and September from each other, only those buildings were examined which had been closely observed during the first stay. Almost all of the medieval buildings in the centres of the villages and towns were so badly destroyed by the May earthquake (see also (4) that inspection for further damage was pointless.

2. BEHAVIOUR OF PREVIOUSLY DAMAGED STRUCTURES DURING FURTHER STRONG MOTION EARTHQUAKES

At first glance, it seemed that all the buildings left standing after the May earthquake had been completely destroyed by the severe shocks in September. In particular, many houses in the centres collapsed, though there were only a few deaths since the majority of the population was living in tents and mobile homes.

Only a small part of the structural damage studied can be attributed with certainty to any one of the large number of shocks. Already during the first field trip, building damage was analysed which, apart from the first strong shock, had been subjected to a series of aftershocks with intensities up to I_o = VII (MSK).

Earthquakes of this force can cause noticeable damage to intact structures (see description of damage of the seismic intensity scale MSK 1964). Previously damaged and therefore weakened building elements are strained to an even greater extent by such earthquakes. After the strong earthquakes of September 11th and 15th, 1976, the investigations were further handicapped by this uncertainty. It is, therefore, apparent that the comparison of the two investigations cannot establish in detail to what extent the further damage was caused by the numerous weaker aftershocks or by the three strong shocks of mid-September. Various observations and information obtained from the inhabitants themselves showed, however, that most of the further destruction was caused by the severe earthquakes and that the influence of the aftershocks, in contrast, can be ignored. It has, therefore, been assumed in the following conclusions that the changes observed during the second stay can be attributed in the main to the strong earthquakes of September 11th and 15th.

2.1. Behaviour of Unchanged Structures

Buildings were affected to very different extents during the first earthquake. Some of the buildings, because of their construction type, suffered no obvious or, at the most, light damage, while others were widely destroyed to the point of partial or total collapse. In any event, damage always results in a weakening of the structure and altering of the vibration behaviour and, therefore, less satisfactory behaviour under renewed earthquake stress. In most cases, the structural response under further earthquakes will be higher (Figure 21). For the observation of damage expansion, a series of buildings were examined which had remained unchanged after the first earthquake.

The buildings which were undamaged or which suffered only minor plaster cracks, showed no further damage or, at the most, single wall cracks. Their supporting structure withstood this earthquake load and can be considered sufficiently earthquake-proof against similar earthquakes. The building with severely cracked brick walls suffered in most cases even more masonry damage, due to their weakened condition (Figures 1 and 2).

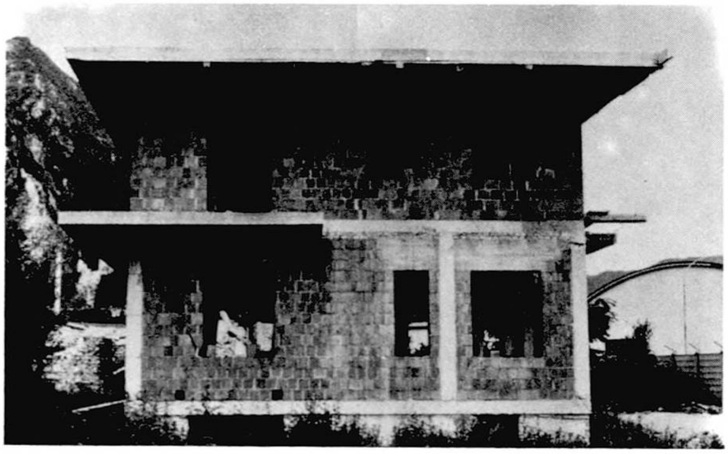


Figure 1: (after May 6th): Uncompleted building in Gemona with groundfloor open on three sides and living floor above. The supporting framework of the groundfloor consists of reinforced concrete columns, partitioned on one of the narrow sides by brick walls. The upper floor is exclusively brick. Due to the earthquake load, the building rotated around the partitioned transverse wall in the groundfloor. The fixing points of the columns were badly damaged in the cellar and groundfloor ceilings. The brick walls on the transverse side on the groundfloor were slighthly cracked: the upper floor remained undamaged.

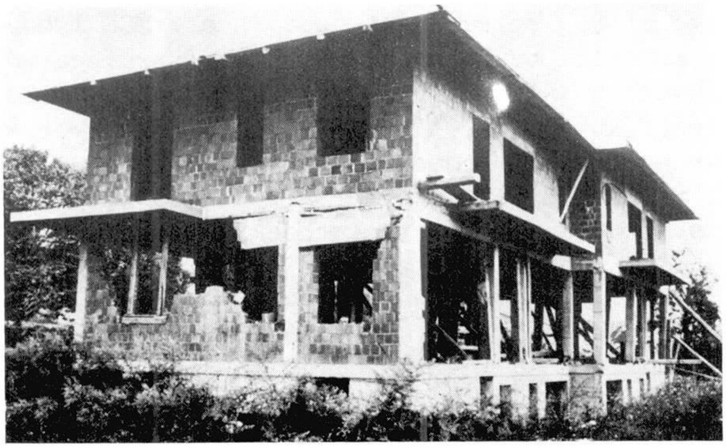


Figure 2: (after September 15th): Uncompleted building in Gemona after renewed earthquake stress. The reinforced concrete columns in the groundfloor were completely destroyed at their fixing points,connection being maintained by the reinforcement only. The massive wooden struts in the groundfloor saved the building from colapse during the second strong earthquake. The already damaged brick wall in the groundfloor was additionally strained due to the strenghtening effect of the wooden struts in the open groundfloor area. The wide brick window jamb was broken out and the reinforced concrete window supports shaken from their rests.

In the case of skeleton structures, this damage could lead to overstrain and, therefore, to damage to the supporting structure. Also badly cracked masonry in non-structural walls means a deterioration of the carrying capacity and a greater danger of building collapse. The carrying capacity was also considerably lessened through the destruction of energy-absorbing building parts such as mortar joints (Figures 3 and 4), whereby the structure experienced reduced damping and therefore suffered greater deformation under renewed earthquake stress.

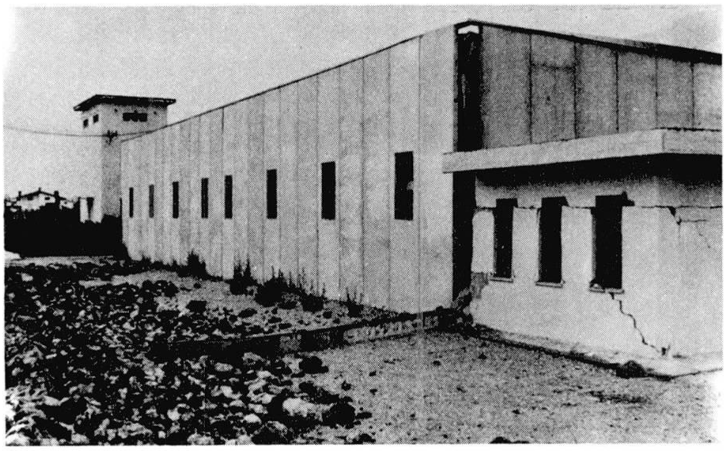


Figure 3: (after May 6th): Prefabricated shed with brick annex in Gemona. The supporting framework of the shed is undamaged. The concrete elements of the side wall, on the other hand, were slightly caved in by the impact of the rigid annex. The plastic material of the wall joints was torn by the large deformation. The roof of the annex was shorn off and pushed over at window level by the impulse load of the impact due to insufficiently wide joints.

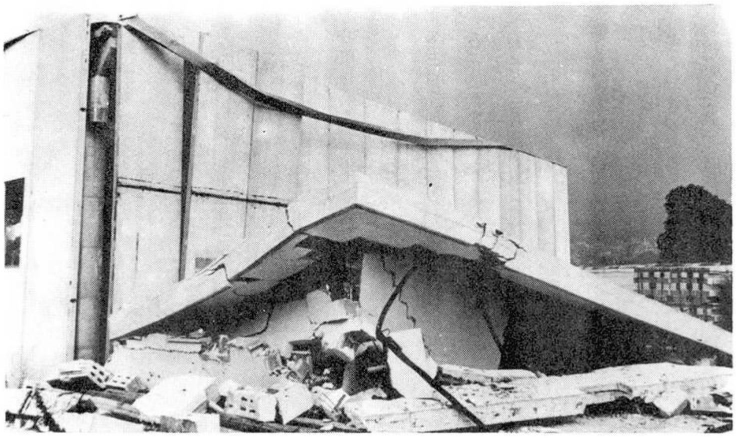


Figure 4: (after September 15th): Prefabricated shed with collapsed annex. The renewed earthquake load caused damage of the supporting framework of the shed at the fixing points of the columns and rests of the bars. The façade slabs of the side wall were severely caved in by the renewed impact: the annex itself collapsed. The increased displacement, as compared with the first earthquake, can be traced back to the reduced structural rigidity due to the destruction of one of the concrete block walls.

Those buildings with badly destroyed brick walls or damage to the supporting structure showed a greatly reduced carrying capacity under renewed earthquake load. The buildings experienced further extensive deformation of the damaged parts which often led to local collapse (Figures 5 and 6) or to the collapse of the entire building (Figures 7 and 8).

Figure 5: (after May 6th): Residential building with workshop near Artegna with a reinforced concrete cellar and two floors in concrete blocks. The side walls by the entrance incurred gaping X-shaped diagonal cracks due to the high shear stress. These cracks led to partial collapse of the masonry. The highly strained window jambs on the longitudinal façade were badly cracked.

Figure 6: (after September 15th: Residential building with workshop near Artegna after renewed earthquake stress. The side wall in the middle floor collapsed under further earthquake load. The door and window jambs on the longitudinal walls shattered and partially collapsed. The shear resistance of these concrete block walls was minimal due to failure of the mortar joints to prevent the layers from shifting.

Figure 7: (after May 6th): Residential building with shop near Artegna. Reinforced concrete frame building with brick partitioning in the two upper storeys In the groundfloor, open on three sides, only the rear longitudinal wall and the staircase are braced. The destruction was concentrated on the groundfloor were a rotating motion of approximately 40 cm maximum displacement occured around the staircase core. The brick walls of the staircase were badly destroyed, while those of the upper floors remained undamaged.

Figure 8: (after September 15th): Wreckage of the residential and business house near Artegna. The groundfloor of the building collapsed during the earthquake of September 11th. Through the impact, the undamaged living storeys also collapsed. The destroyed fixing points of the reinforced concrete columns in the groundfloor as well as the cracked brick walls of the staircase were not enough to carry the renewed strong earthquake stress. The experts had hoped, despite the doubts of the owner, to be able to repair the building.

2.2. Behaviour of Repaired Buildings

After the first strong earthquake, many of those buildings with only minimal or light damage were repaired. Priority was given to those buildings urgently required to keep life-lines open, such as the hospital in Tolmezzo, as well as to industrial and trade buildings in order to resume production and secure employment. The reconstruction work was interrupted by the September earthquakes, thus according the opportunity of judging the merits of the various repair measures and to plan further reconstruction from the standpoint of a threat of further strong earthquakes.

The nature of the repair work was, corresponding to the variety of building types and earthquake damage, very diverse. Therefore, large differences, corresponding to the measures taken, were to be seen after renewed earthquake stress.

2.2.1. Surface Repairs:

Façade and plaster cracks, and even small masonry cracks, were superficially repaired by patching up the plaster and repainting or repapering the walls. Such surface repairs, however, only covered up the weaknesses produced by the cracks in the masonry. Under renewed earthquake load, the same kind of façade damage occured and, due to the existing weakness, the most extensive damage was to be found where the masonry cracks had not been repaired.

2.2.2 Restoration:

Where damage was limited to only a part of the supporting structure or to cracked brick walls, that part was restored as far as possible to its original condition (Figure 9). The object was not only to make the structure reusable but also to reestablish the original carrying capacity against earthquake load. These measures were not intended to improve the carrying capacity, since the existing protection against earthquakes was considered adequate. Naturally, the renewed earthquake load was concentrated on the same building parts and the same damage occured (Figure 9). The repair work which had been undertaken, however, did have the effect of preventing more extensive destruction and offered, therefore a sufficient protection against collapse.

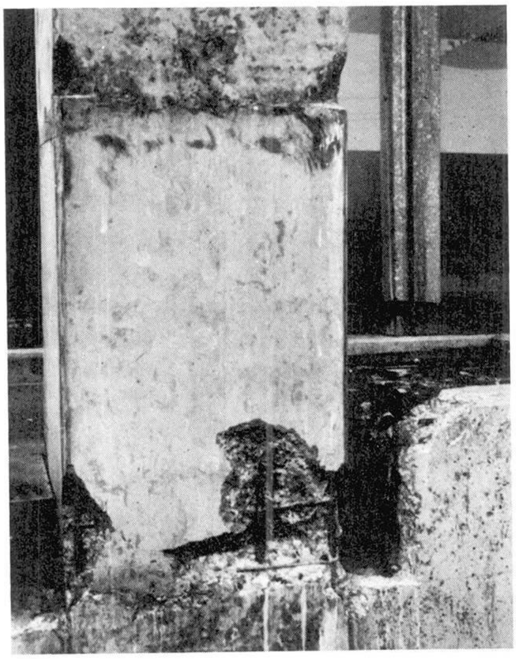


Figure 9: (after September 15th): Foot of a column in the Fantoni furniture factory in Rivoli di Osoppo. By repairing the cracked column foot it was hoped to restore the original carrying capacity. The renewed earthquake load caused the same damage as the first shock, the repaired fixing points of the columns were again cracked and the concrete crumbled in the most highly stressed areas.

2.2.3. Improvement of the Carrying Capacity:

In many cases, an attempt was made to improve the carrying capacity of those structures which were extensively damaged during the first earthquake and which remained unusable for a long period due to complicated repair work. By doing so, the danger of collapse or material loss due to unusability caused by renewed earthquakes should be reduced. The carrying capacity of a structure can be improved in various, distinctly different ways ,which can be applied either single or combined.

The increase in the carrying capacity of the most highly strained building parts results in an increased resistance to renewed earthquake load. This increase in carrying capacity can be attained by replacing the supporting structure with more solid material. In most cases, however, an enlargement of the cross-section of the building parts is a more obvious solution (Figure 12). At the same time, though, the rigidity behaviour is changed and a renewed earthquake will, therefore, put other building parts under a heavier strain.

By altering the vibration behaviour on the other hand, the strain on individual building parts can be changed and heavily strained building parts can be relieved at the expense of other less highly strained parts. Apart from the damping of the structure (see Figures 3 and 4), it is the rigidity and mass behaviour of the structure which above all strongly influence the vibration capacity under earthquake load. Through the walling up of openings in the rear wall of a curved building (Figure 10), its rigidity behaviour was fundamentally altered. Hence, the vibration capacity of the building was so improved that the building parts which had been most strained during the May earthquake suffered no damage during the renewed shocks. Accordingly, the repairs that were made considerably contributed to the improvement of the carrying capacity. The improvement of the vibration capacity provides the most reliable protection against new earthquake damage and makes it possible to achieve optimal strengthening taking the construction type and building material used into consideration.

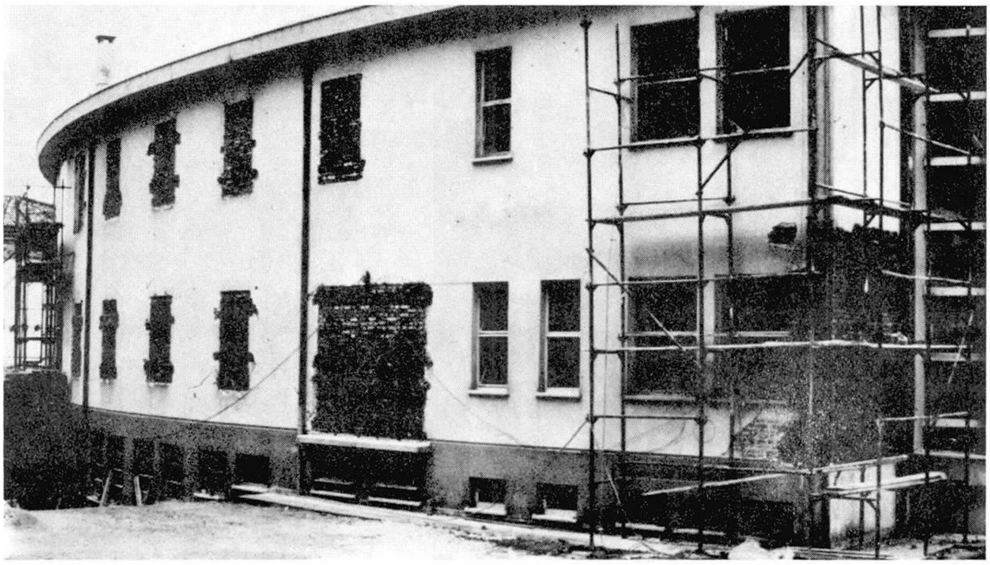


Figure 10: (after September 15th): Business building SBUELZ in Tricesimo. To strengthen the carrying capacity, the door and window openings were walled in. The vibration behaviour of the building was so improved that, during renewed earthquake stress hardly any further damage occured as opposed to the destruction which occured during the earthquake of May 6th.

A lessening of the earthquake load is to be expected, above all, by the reduction in weight of the non-supporting building elements. The reduction in weight of secondary building elements and installations has the effect of reducing almost proportionately the strain on the supporting structure and, therefore, increasing the carrying capacity during renewed earthquake stress. In an industrial building (Figures 11 and 12), for example, the heavy reinforced concrete façade slabs were replaced by walls of lighter steel. Not only the strengthening of the columns but also the lessening of the earthquake load contributed considerably to the fact that the building remained undamaged during the September shocks.

It is obvious that the civil engineer must incorporate earthquake load in the calculation and construction of buildings to be newly erected and in the repair of damaged structures. As demonstrated by the positive behaviour of the newer buildings in the area of Tolmezzo, designed in accordance with the now valid Italian earthquake code, which offer sufficient safety for the life of the occupants. At the same time, however, it should be noted that it is accepted that building damage will occur and the protection of the inhabitants is guaranteed during a limited number of strong earthquakes only. If it is required that a building (for example hospitals, utilities) will be able to continue functioning after an earthquake, then the normal earthquake building code is insufficient. A dynamic analysis of the building behaviour should be made and the structures should be designed in accordance with the loads actually occuring during an earthquake. An earthquake risk study can serve as a basis for the selection of the appropriate earthquake input parameters.

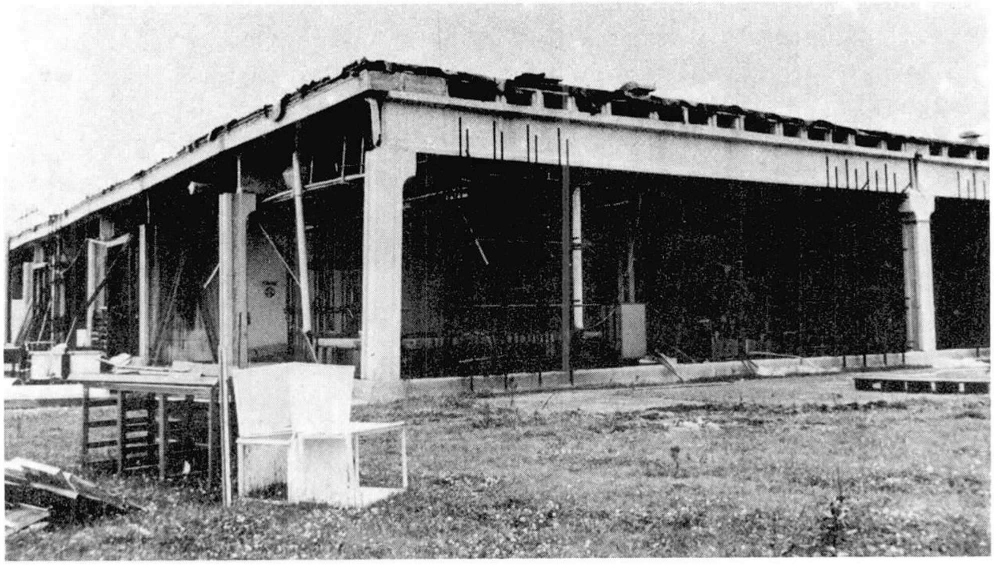


Figure 11: (after May 6th): Factory building near Artegna. Severe displacement of the prefabricated reinforced concrete framework together with distortion of the foundation and tilting of a corner column. A large number of the only lightly secured reinforced concrete façade elements were thrown off.

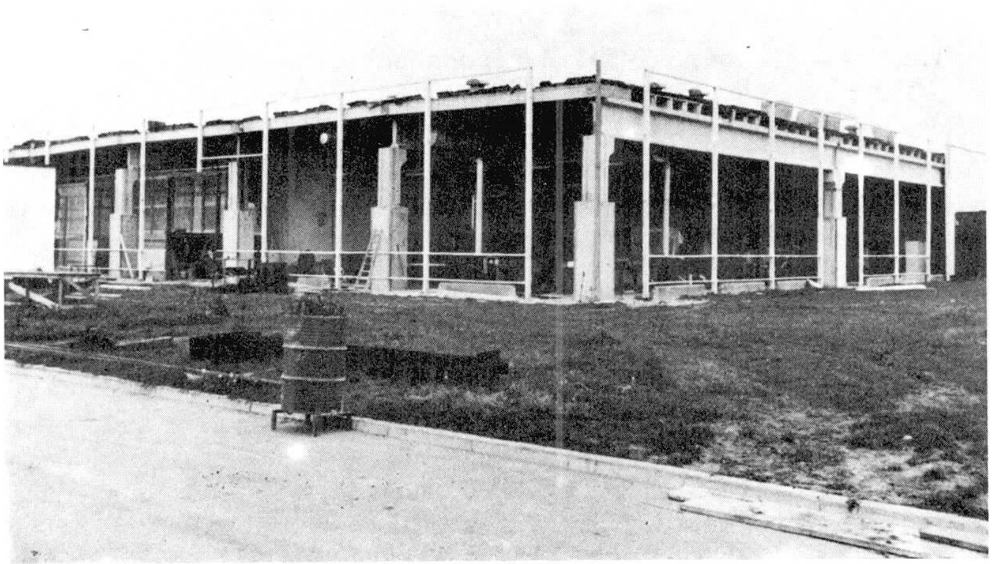


Figure 12: (after September 15th): Factory building near Artegna, after repair, without further earthquake damage. In order to increase the carrying capacity, the concrete columns were surrounded with a 15 cm strong concrete sheath. The earthquake load was reduced by replacing the heavy concrete façade elements by a light metal façade.

3. A FEW GENERAL REMARKS ON EARTHQUAKE BEHAVIOUR OF BUILDINGS

A closer examination of the effects of the earthquake on various types of structures reveals that much of the damage originates from just a few basic structural defects. We have tried to derive several ground rules for the design of structures. Although these conclusions can also largely be confirmed by observation of other earthquakes, the damage described here is peculiar to this particular earthquake and epicentral region.

The main cause of the extensive damage is certainly the fact that the effects of an earthquake did not have to and therefore were not taken into considerations in the design of the buildings. If engineers had only visualized that their structures would have to undergo the displacement, velocity and acceleration of earthquakes, the few selfevident consequences in the layout of the structures would have avoided most of the damage even without a proper earthquake design.

3.1. Structural Elements

3.1.1. Walls

Walls and partitions consisting of brick and masonry are both rigid and brittle. Where the walls are insufficiently strong or where many openings exist, the masonry walls are no longer able to absorb horizontal forces. When overstrained, these walls crack mostly crosswise under 45° (Figure 13), the crack spreading, either along the mortar joints or in the bricks. Because of the brittle nature of masonry constructions, the cracks widen, joints gape open or the walls concerned even collapse. In a skeleton construction, the reinforced concrete framework can in certain cases continue to uphold the weight of the buildings. For purely masonry construction, on the other hand, at least a partial collapse of the building is unavoidable.

Figure 13: Dwelling house with workshop near Artegna. Basement in reinforced concrete and the upper stories in masoning without framing. Typical diagonal cross cracks in a wall of the ground floor.

3.1.2 Reinforced Concrete Columns

Since reinforced concrete columns are, in general, considerably more flexible than walls, the latter carry partically the entire earthquake force. However, in open constructions the entire stress is carried by the columns. The free-standing reinforced concrete columns of one-storey storage sheds were mostly strong enough to absorb the stress without being destroyed, in many cases even without incurring lasting cracks. By partial stiffening, for instance by means of an annex or installations or heavy rigid upper floors, higher stress results, which usually leads to plastic deformation at both ends of the columns. Hence, the reinforcement, overstressed by tension, can buckle as a result of the alternating action (Figure 14). The related cracking of the concrete and the buckling cannot be significantly reduced even by means of closely spaced stirrups.

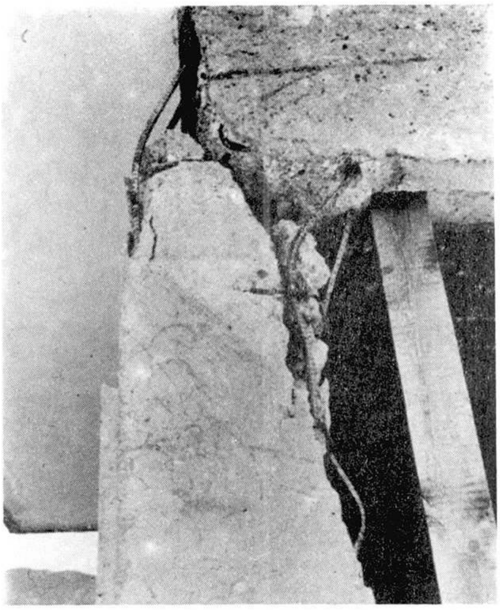


Figure 14: Shearing-off of a column at its connection with the crossbrain due to insufficient stirrups.

The ability of the resulting plastic joint to rotate is, however, increased and a sheer failure prevented.

In the event that it is impossible to design the columns, taking actual earthquake forces into consideration, then at least the plastic deformation of the columns in all directions must be guaranteed. The movements should not be hindered by any secondary elements. An improved building method for the prevention of collapse could, therefore, be to shape the columns in a manner that the plastic hinges, necessary to absorb energy, are formed in the crossbeams.

3.2. Structural Systems

3.2.1. Open Ground Floors

Open or only slightly stiffened ground floors, mostly for commercial use, are particularly vulnerable. The locally severe destruction in the area of such weak spots caused the collapse of entire buildings or made their demolition necessary, even with otherwise only minor damage, because restoration would have been too difficult (Figure 15).

Greatly differing conditions of rigidity in a supporting structure result in local weak spots which are the first to be overstrained in an earthquake and plastically deformed. Hence the stronger parts of the building are no longer irreversibly deformed and energy absorption is limited to the weaker building parts. Consequently, an evenly distributed plastification of the entire structure is necessary to ensure that destruction remains within acceptable limits.

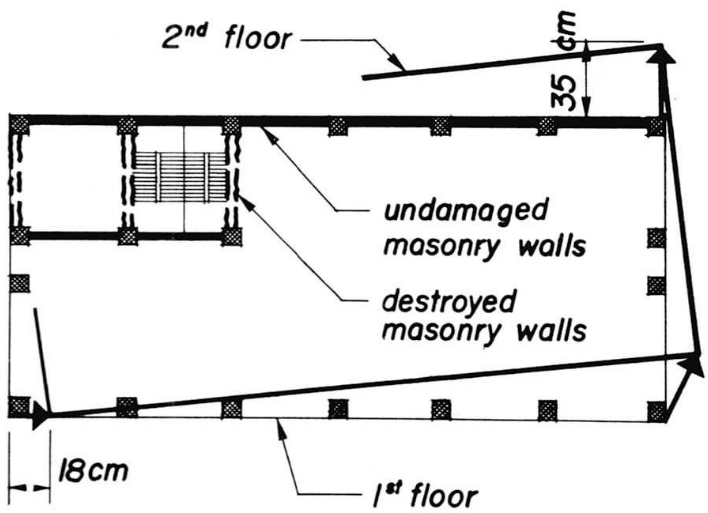
3.2.2. Torsional Action

From the point of view of the structural system, many symmetrical structures suffered damage due to the additional twisting motion of the building around its vertical axis. As a result of the superimposed movement, some parts of the building are relieved whilst others are considerably more deformed than they would be due to translational movements only. Torsional loads are caused by the unsymmetrical layout of the structural system, but also by contingencies arising in the rigidity and execution of partitioning walls and additional fittings (Figure 15). The consequences of torsional strain can only be met by appropriate consideration in the design including provision for sufficient torsional rigidity of the building. Due to the incalculable influence of secondary elements, which are not designed to carry vertical loads, an asymmetry in the ground plan can hardly be excluded.

Figure 15:

Three-storey dwelling house with shop in the ground floor near Artegna. Reinforced concrete frame with brick partitioning walls in the upper floors and mostly open ground floor. Plastic hinges at bottom and top of the ground floor reinforced concrete columns caused large deformations. No damage in the upper floors.

The ground plan of the dwelling house with the open ground floor shown before. Twisting of the building around the staircase stiffened with masonry walls.



3.2.3. Attached Buildings

Severe damage could be located in structures composed of building sections with greatly differing rigidity due to diverse types of construction (for instance, reinforced concrete frame and pure brick) or which varied considerably in their design (Figure 16). This damage occured because the individual deformation of each component was obstructed.

This problem can be overcome by arranging the joints as to divide the structure into sections, each with its own clearly distinct vibration behaviour. The joints should be made adequately wide since numerous uncertainties make an exact calculation impossible. It must be taken into consideration that, for example, the deformation usually provided for in a homogenous supporting structure can turn out to be considerably larger due to the formation of cracks or plastification. An adequate freedom of movement, therefore, allows for greater plastification and a larger capacity to carry earthquake stress.

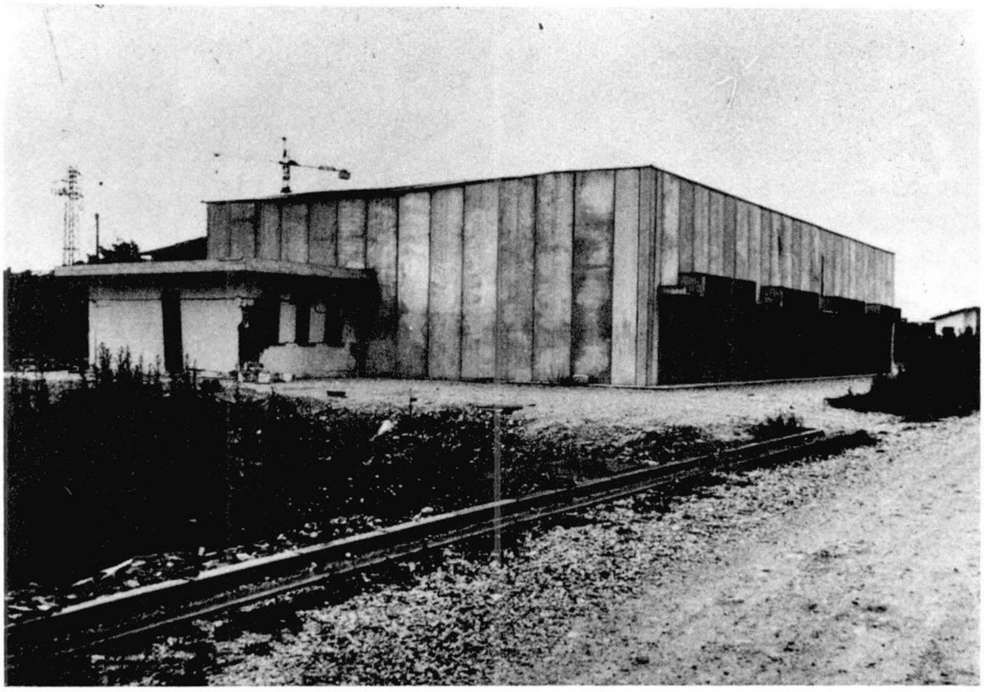


Figure 16: Prefabricated storage shed with brick annex in Gemona. Side wall panels slightly caved in by impact with rigid annex. Upper part of the annex shorn off and pushed over by the impact force due to insufficiently wide joints.

3.2.4. Special Structures

Special structures (e.g., bridges and water-towers) (Figure 17), because of their unusual form and distribution of mass, necessitate a dynamic analysis which takes the vibration behaviour of the structure and the real properties of an anticipated earthquake into consideration.

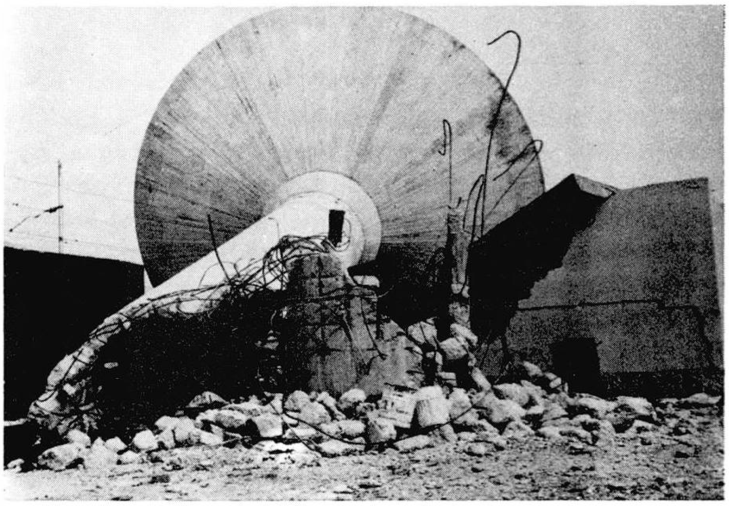


Figure 17: Overturned water tower belonging to the Italian State Railways in Gemona station. Foot of the shaft completely destroyed and concrete shattered.

3.2.5. Secondary Structural Elements

All the components and fixtures, in particular dividing walls, attached façade slabs, covering, pipes and other fittings, which form part of a structure, influence the response of the supporting structure (Figure 18). These secondary structural elements are generally not included in the analysis of the supporting structure and, therefore, not designed against earthquake forces. They can, even when subjected to only slight movements, suffer damage which produces an increasing alteration in the vibration behaviour. It cannot be predicted whether this influence will prove to be positive due to greater absorption of energy or negative due, for example, to added torsional motion. As far as possible, in order to ensure that secondary elements survive earthquake loads without substantial damage, they should be analysed and designed together with the supporting system.

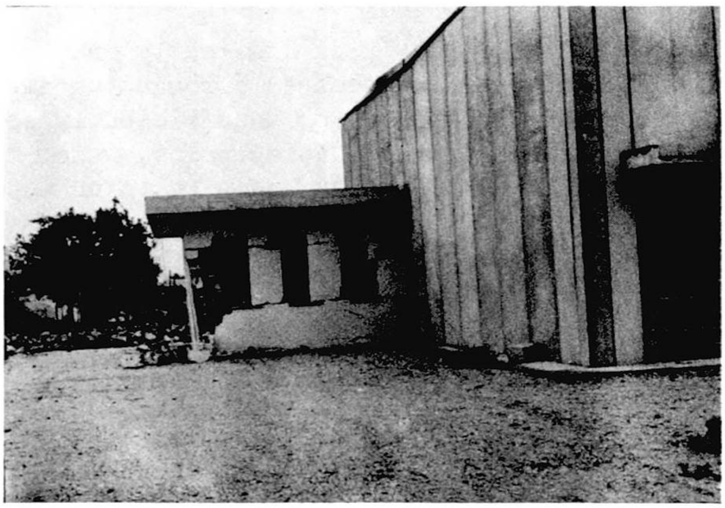


Figure 18: Prefabricated storage shed in Gemona as shown before. The supporting frames and the wall panels generally not damaged because of energy absorption in the material of the joints between the single panels.

3.3. Joints and Supports

3.3.1. Joining of Structural Elements

If prefabricated structures are designed only in accordance with the Standards laid down for earthquake forces or these forces are overlooked altoghether, then the result is greatly underdimensioned connections of the structure elements Load bearing connexions should be properly designed against the expected dynamic forces. Purely friction-type joints are no longer sufficient to transfer the forces that arise, even from only weak earthquake loads (Figure 19).

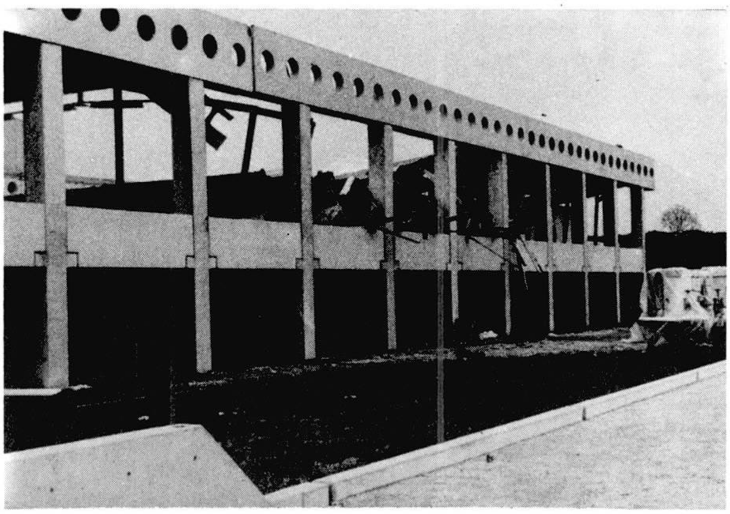


Figure 19: Heavily damaged prefabricated factory shed near Osoppo with roof girders fallen down. Friction joints insufficient to provide structural stability.

3.3.2. Fixation of Secondary Structural Elements

Building parts (such as prefabricated façade slabs and dividing walls and fittings, particularly machines, storage racks and pipes), which are not part of the supporting structure, are usually either directly or indirectly connected to it. Due to the action of the earthquake, much damage occured through the displacement or collapse of façade slabs which were unconnected or insufficiently secured (Figure 20). The actual displacement occuring at the fixation point, which can be considerably larger than the one of the ground shock, must be taken into consideration in the fixation of secondary elements.

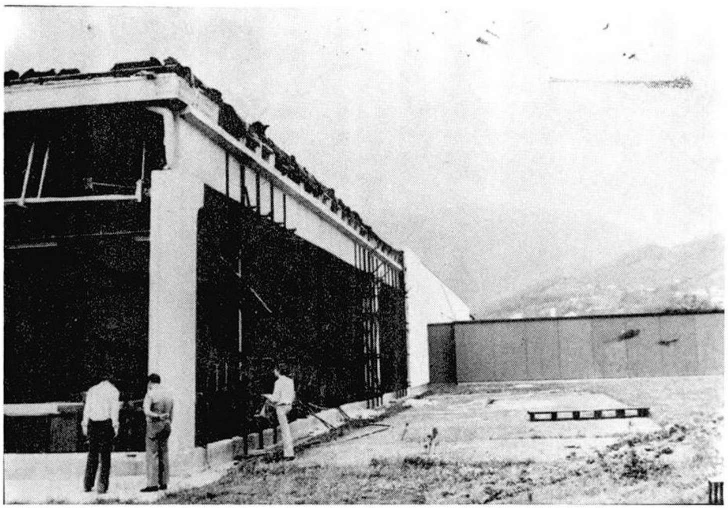


Figure 20: Prefabricated factory shed near Artegna. Wall panels fallen out during the earthquake because of insufficient fixation.

4. MERIT AND LIMITS OF EARTHQUAKE DESIGN SPECIFICATIONS

The region at the southern foot of the Alps hit by the May 6th earthquake has been known for centuries as an earthquake area. However, in the major part of the epicentral area, no laws existed for the design of structures. Such laws applied only for new buildings in a small part of the area.

The Italian State Administration has enacted special regulations for earthquake-prone areas and has repeatedly brought them up-to-date. Using the Code, design earthquake loads are determined by statical or dynamic analysis. An average horizontal acceleration will result, which is about 7 percent of the gravity acceleration g. Comparision of horizontal design accelerations given by the Code with those produced by an earthquake with the Intensity IX (Figure 21) shows large discrepancies.

The Code values are significantly smaller because it is assumed that strong energy absorption will occur due to inelastic behaviour of materials and elements. But this means that the structure must be capable of absorbing the appropriate energy. Consequently, plastic deformation and therefore damage or maybe even collapse can result.

On present day standards, this is not good enough. Originally, Codes were drawn up merely to prevent the collapse of a structure and thus save lives. Today, our more highly developed society demands that at least life-lines (i.e., hospitals, water supplies, electricity, etc.) continue to function after an earthquake. As a result, it is imperative that a Code be introduced incorporating design specifications which distinguish between the various functions for which structures are built.

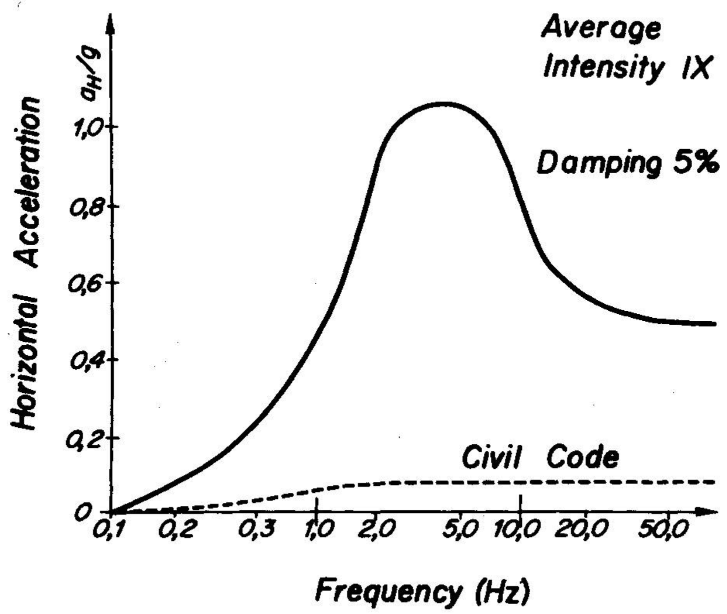


Figure 21: Horizontal ground acceleration for earthquakes of intensity IX (MSK) (U.S. Atomic Energy Commission, WASH 1255). The dashed line gives the disign ground acceleration according to the conventional aseimic building code.

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BEHAVIOUR OF LARGE PANEL BUILDING DURING THE ROMANIA EARTHQUAKE OF MARCH 4,1977

By

Miodrag Velkov *

Summmary

The experience gathered from the failure of several prefabricated buildings caused by the Agadir earthquake of 1960 could give only few data concerning the behaviour of precast structures during earthquakes.However,a general conclusion is made that precast structures suffer more damage than the monolythic structures.

During the Romanian earthquake of March 4,1977 which affected one third of the whole territory of Romania,the behaviour of different prefabricated large panel systems could be observed.In this structural systems over 120.000 apartments have been constructed.

Due to the fact that the behaviour of these systems is considered favourable as compared to other structural systems,more systems were analysed in detail in order to define their behaviour during earthquakes .

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Our experience gathered from the failure of several prefabricated buildings caused by Agadir earthquake of 1960 could give only few data concerning the behaviour of precast structures during earthquakes. However a general conclusion is made that precast structures suffer more damage than monolythic structures.

We could say that up to the Romania earthquake there were almost no data about the behaviour of precast structures during strong earthquakes. During this event, the behaviour of the entire precast system in a wider range could be verified for the first time, this specially refering to large panel systems which in romania have been applied as mass construction during the last twenty years. The large panel systems constructed on the territory of the whole country amounting 120.000 - 150.000 appartments(out of which Bucharest has 75.000, Ploesti 12.000 and Kraiova 6000 - 8000 appartments) give a good possibility for analysis of their behaviour including parameters like : earthquake intensity, frequency content, soil conditions, height of the building, types of members, connections and so forth.

The earthquake epicenter of the March 4,1977 earthquake was on the slope of the Karpathian chain, at a depth of about 100 km and a magnitude of 7.2 according to Richter scale.

The failures and damage due to the earthquake effect were experienced on an area of 80.000 km^2 which is 1/3 of the whole Romanian territory. Also, some distruction is evident in Bulgaria along the River Danube. The earthquake was felt in Yugoslavia, too, with different intensities(fig.1).

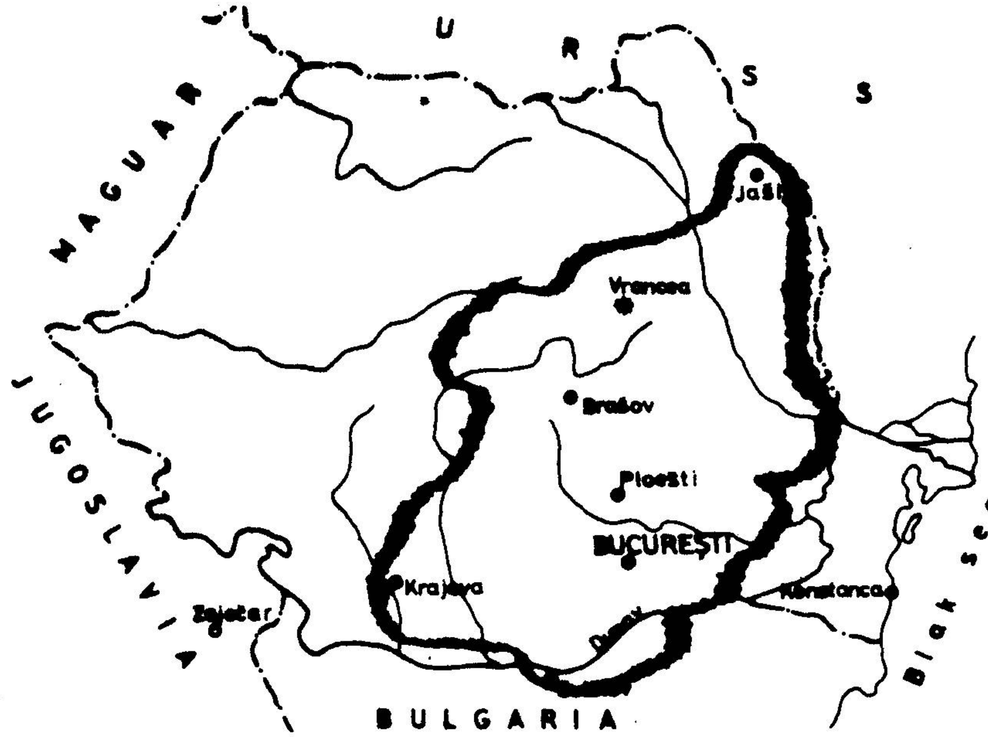


Fig. 1 Map of the Romania showing the region affected by the March 4,1977 earthquake

The geological,geophysical and geomechanical characteristics of the territory as well as the large energy released in the epicenter clarify the destruction and damage of such a vast area.

In Bucharest,a ground surface acceleration of 0.20 g(component N-S) was recorded by SMAC instrument,while at a distance of about 700 km in Nis,Yugoslavia, a ground acceleration of 0.04 g, E-W component was recorded by SMA-1 instrument. Interesting to be mentioned here is the frequency content of this earthquake if it is compared to some other earthquakes(Fig. 2).

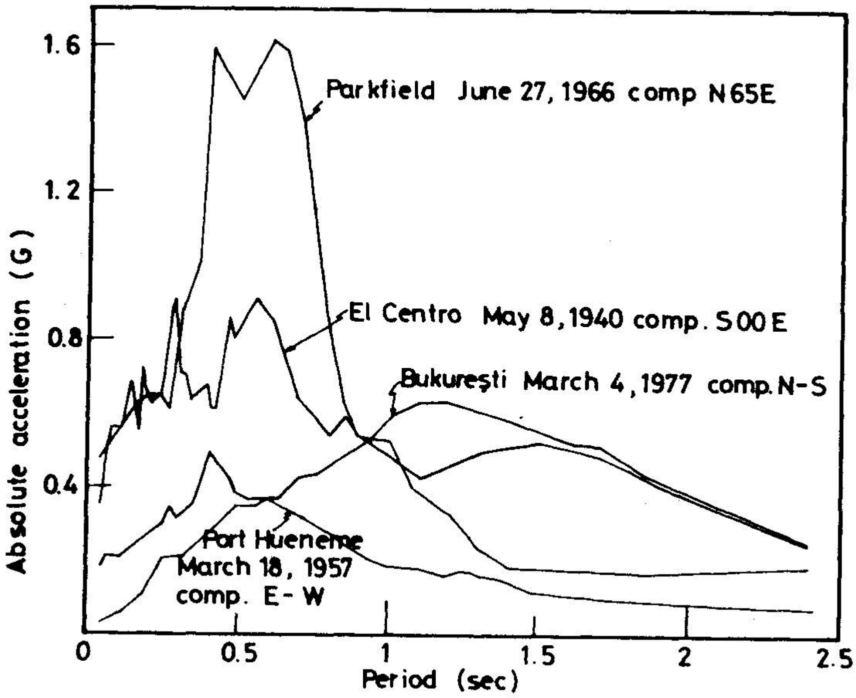


Fig. 2 Absolute acceleration response spectra for damping 5% of critical

The behaviour of different structural systems during the earthquake could be summarized in general,as:

Slender reinforced concrete frame structures with brick masonry infilled walls constructed between 1930 - 1940 ,without earthquake resistant design requirements,with low quality characteristics of concrete,insufficient percentage of reinforcement and unfavourable structural composition.About 30 structures of this type failed while a lot of them were badly damaged.

Structures constructed during tha last twenty years in monolythic reinforced concrete sysrems :bearing walls,infilled frame systems and composite systems consisting of frames,shear walls and bearing walls.These systems give relatively good performance showing different types of damage which are mainly cracks which correspond to their postelastic behaviour.It should be mentioned here that the first Codes of Romania were enforced after the Romania earthquake of November 10,1940,while the contemporary regulations based on spectral analysis were brought in 1963.

The experienced behaviour of the precast structural systems was satisfactory above all expectations, in spite of the different qualities of construction of different types of structures.

The large panel structures were introduced in Romania about twenty years ago. A priority was given to these systems during the last ten years, so today, about 75.000 apartments have been constructed in Bucharest in this system of 5-11 storey height. In Ploesti, which according to the seismic zoning map is included in zone of higher seismic intensity, the number of stories is limited to 5 stories.

According to the European practice Romania has adopted the "two-way " system. Usually, all the panel walls both internal and facade ones are bearing walls. In Bucharest,, there is a panel system constructed 15 years ago, of eight storeys, the external walls of which are not bearing walls. The foundation structure as well as the basement are monolythic. An exception to this is a ten storey building in Bucharest which has precast basement on monolyth foundations. The first slab above the basement was constructed differently, both monolythic and precast, however in Ploesti it is almost always monolythic structure.

The connections of panels both horizontal and vertical are usually wet connections placed in concrete in situ with welded anchor reinforcement, which is a characteristic of European systems.

Structural systems are mainly designed and analysed according to the existing aseismic regulations, applying static methods for definition of the static values while the stresses are defined for ultimate stress state.

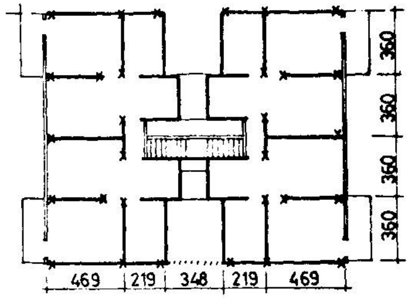
The principal structural characteristics of the systems used are as follows:

1. Two way system of eight storeys and nonbearing facade panels. These structures have no basement and the prefabricated system is placed on monolyth foundations. It was constructed in series some 15 years ago in Bucharest. It is solved with monolythic horizontal and vertical joints, welded reinforcement of vertical panels and monolyth slabs above the last precast structure. Fig.3

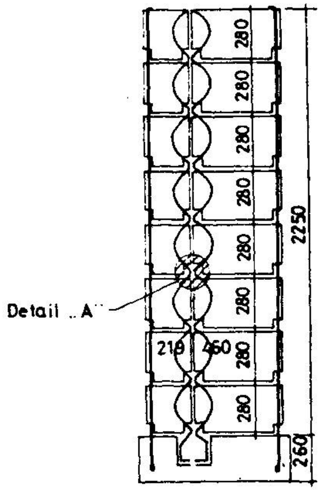
2. Two way system of ten storeys with basement. It is a precast basement structure on monolythic foundations. The system is constructed in series in Bucharest and its use will continue in future, no matter of the recent earthquake event. Fig.4

3. Many structures in Bucharest have 5 storeys and monolythic basement. Fig. 5. gives details of some members and connections of this system. It should be mentioned here that all panel systems in Bucharest have shear base coefficient of 7-9%, up to ultimate state.

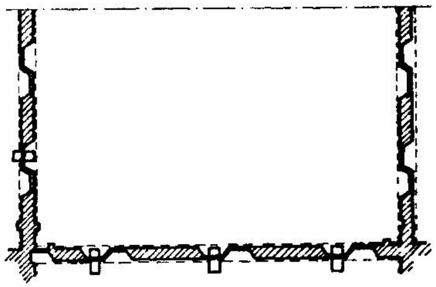
4. In Ploesti which is closer to the epicentral region, construction of large panel structures is limited to 5 storeys. They always have monolyth basement with monolyth floor slab above it. The walls are in two-way system. Fig.6. gives details of the most frequent type of system used. It should be noticed here the enlarged section of the monolythic column in order to increase its shear strength. This system was previously constructed without this enlargement for shears. The base shear coefficient is 15%.



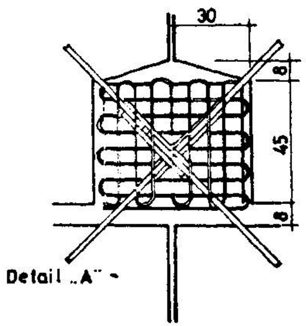
PLAN



SECTION

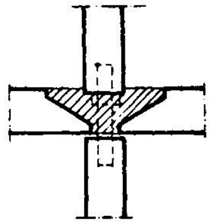


Horizontal panel

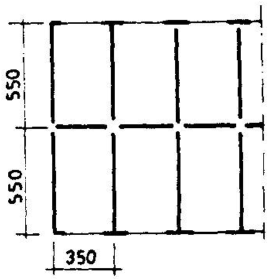


Horizontal joint

Fig.3



Horizontal joint



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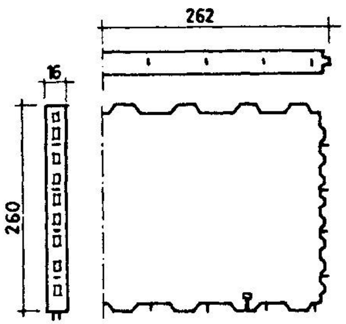

Vertical panel

Fig. 4

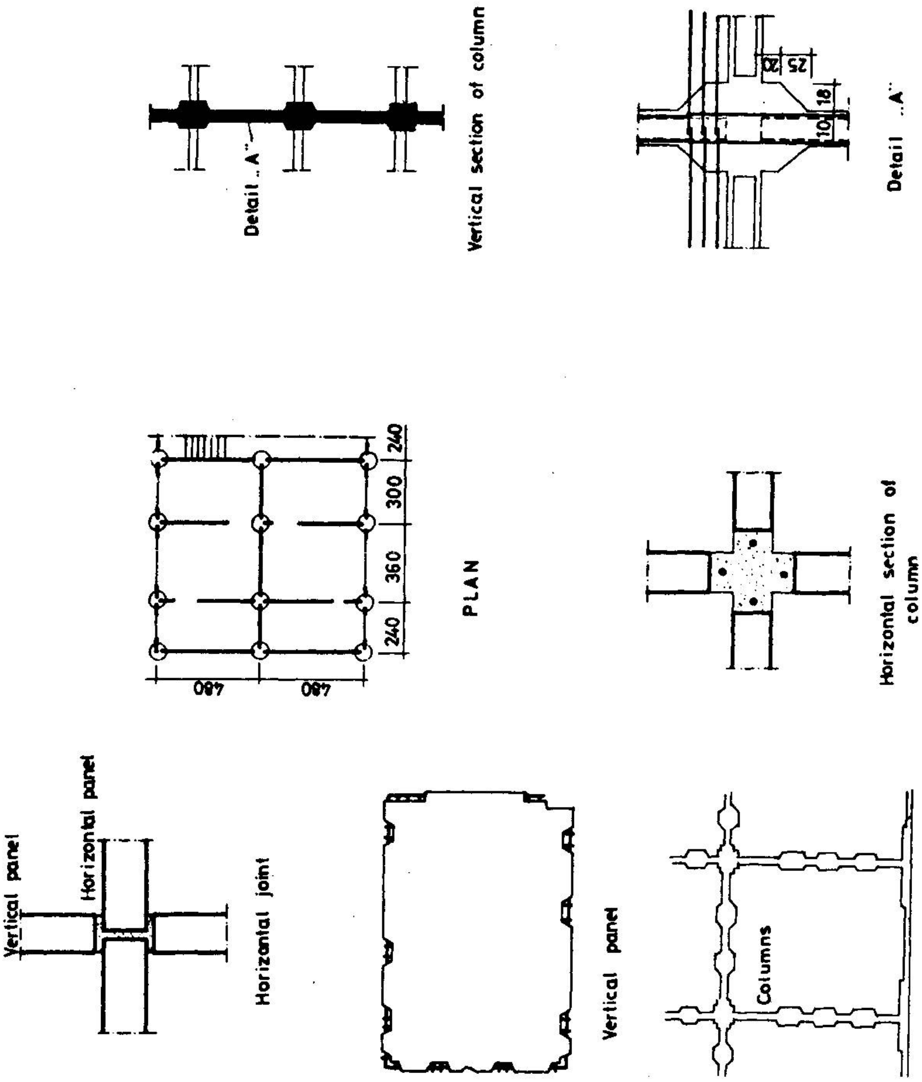

Fig.6

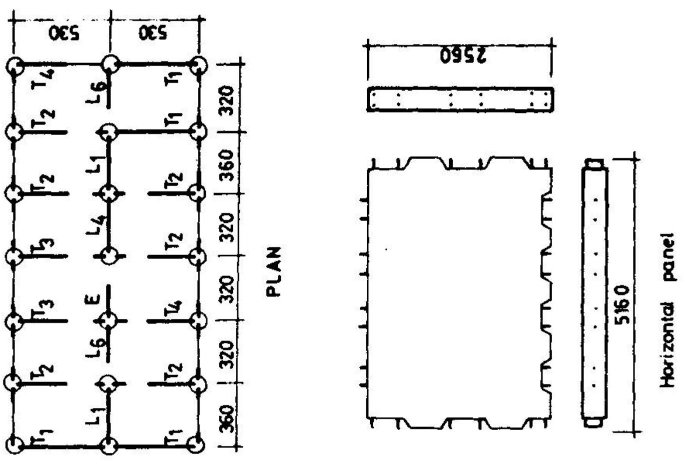

Fig. 5

In Kraiova,buildings with same design,same number of storeys and similar solutions are constructed.The plan of reinforcement distribution is given in Fig. 7 These structures have a shear base coefficient of 10-12 % up to the yield point in bending of the reinforcement.

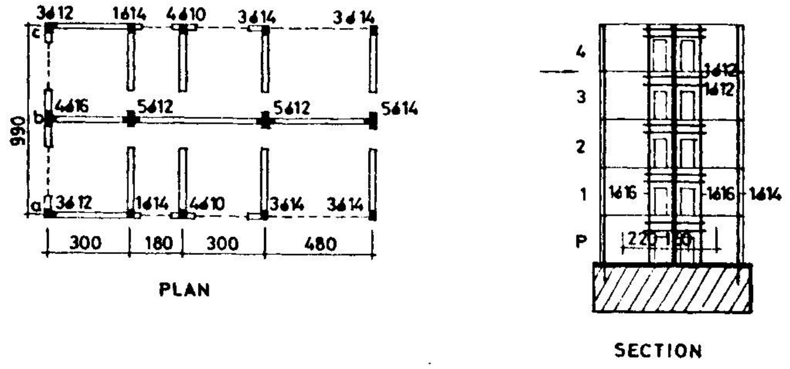

Fig.7

5. In Kraiova a large panel system is both prefabricated and constructed.It is a 5 storey system of box type,i.e.complete room.Each apartment consists of 3,5 boxes monolythically connected in sity along the edges by welding and placed concrete.Each corner is then prestressed by vertical cables of 12 t (wire of 7Ø4 mm.) along the height of the building.The prestressed cables are then anchored to the monolythic basement walls which were constructed in situ together with the foundation. Fig.8

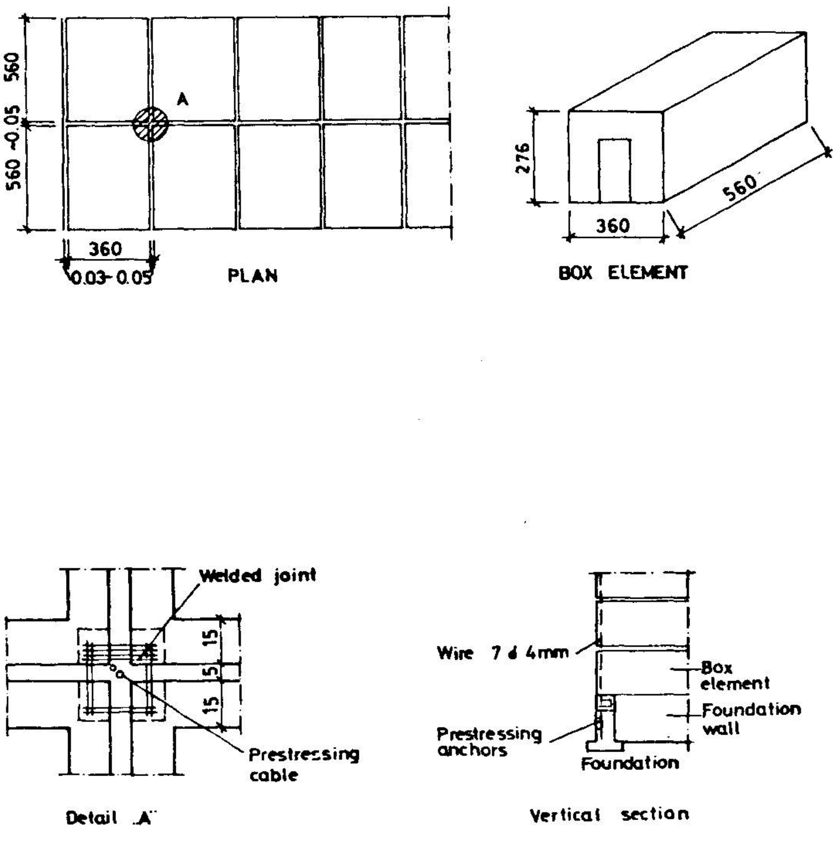

Fig.8

In all Romanian towns all the large panel structures behaved very well and generally speaking ,they did not suffer any significant structural damage.

The overall performance of members within the structural system was qualified as:

- Damage of the foundation structure has not been observed.

- Horizontal panels performed as horizontal rigid diaphragms,without damage.

- In vertical panels there are no observable cracks.Exception is the same building where several longitudinal internal panel walls (without openings)developed fine vertical cracks.

- Also,in some structures on the first and second floor,there are shrinkage cracks in joints in the contacts between the concrete placed in situ and the panels which specially refers to the vertical joints especially in flanged joints.The order of these cracks is from 0.1-0.3 mm. rarely bigger than that. The cracks are mainly concentrated on the first and less on the second floor and as a rule in the intermediate infilled panel walls.

- The horizontal joints occasionally develop cracks close to the place where vertical cracks appear and stretch 1 - 2,0 m. from the contact edges towards the middle of the room.

It should be mentioned here that such cracks in the vertical joints close to horizontal cracks are observed on much smaller number of structures,regardless -he system and the location(Bucharest,Ploesti and Kraiova).

- Sometimes,very fine cracks appear in the connection with a prefabricated staircase.

- Some interesting case of Kraiova should be mentioned here,namely in some structures the reinforcement is anchored to the belt course at the level of the floor slab above the basement.In such a structure ,there was a case of opening of a joint under the first slab in the place where the column reinforcement was anchored.

The satisfactory performance of the system,as compared to other systems,during the March 4,1977 earthquake can be explained by:

- High base shear coefficient as compared to the predominant natural dynamic characteristics of structures,soil conditions and the type and intensity of the earthquake motion - frequency content.

- Sufficient number and favourable distribution of the panels in the two-way system.

- High level of the cast in place of connections,the required length,which provides sufficient monolythic effect regardless the bad quality of construction.

- The whole building worked as a box system with capacity for energy dissipation in the ground at the soil-foundation level.

- Possibilities for bigger damping of the whole system due to joints.

- Energy dissipation in the fine cracks on the contact in vertical and partly in horizontal joints in the zones of shrinkage cracks.

- The quality of concrete is much better than in the case of monolythic structures even if there are some faults in the cast in place joints and welding of reinforcement.

CONCLUSIONS:

The large panel system is extensively used all over Europe ,today.The satisfactory performance of these structures during the Romania earthquake would only contribute to wider application of this system,even for taller buildings in seismic zones.

However,these conclusions should not be generalized since real behaviour of structures during earthquakes depends upon the earthquake intensity and frequency content,the soil conditions and the structural parameters.

Having in mind that,for the first time such a big zone covered by large panel systems with over 150.000 apartments ,was affected by strong earthquake ,an international research project which would investigate the behaviour of these structures during strong earthquakes is necessary,which will enable elaboration of recommendations and instructions for aseismic design of large panel systems in future.

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NONLINEAR SEISMIC BEHAVIOUR OF MAGURELE BUILDING - BUCHAREST, DURING THE EARTHQUAKE OF MARCH 4, 1977

Boris Simeonov *

SUMMARY

During the March 4,1977 earthquake in Romania,the office building IFIN-Magurele underwent moderate damage.For the purpose of definition of a solution for strengthening of the building an elastic and inelastic response analysis was carried out.

The analytical results have shown a good correlation with the real behaviour of the building during the earthquake.

From the analysis carried out a conclusion was made on which elements should be strengthened and their ductility improved.

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1. INTRODUCTION

On March 4,1977 the territory of Romania was subjected,for about 100 seconds,to a severe earthquake with an origin at Vrancea on the arch of Carpati,between the 45th and 46th parallel.The intensity of this earthquake can be compared to the most severe Moldavian earthquakes during the last 500 years(1471,1620,1802.. 1940,1977),and the magnitude was estimated as M=7.2.

Although the earthquake epicenter was 170 km to the north of Bucharest,the city suffered extensive damage.The Office building IFIN-Magurele,a twelve story building,located approximately 10 km to the south of Bucharest was selected for an extensive investigation between IZIIS-Skopje and I.P.Carpati-Bucharest.In fact, this building consists of RC structural walls and during the earthquake suffered moderate damage.

The objective of this joint study was to identify the characteristics of the strength,deformation and the energy processes for such type of structures as a general property of the system,and through this to make suggestions for the most rational measures for repair.

As a result of the joint study,a report was published [1] ,which brought conclusions for the general behaviour of the structure and decisions on the specific aspects of repair.

2. STRUCTURAL SYSTEM AND DESIGN CRITERIA

The Office building within the Nuclear Physics Institute at Magurele is a reinforced concrete building,square in plan of 26 m side and 44 m height,with basement,ground floor and 10 stories (Fig.1).The main structural system consists of reinforced concrete walls,with and without openings,and floor slabs of precast parts and cast in in place parts.Most of the walls have thickness of 25 cm. The main reinforcement is placed at the ends of the sections of walls and ranged from 0.46% to 0.90%.The horizontal reinforcement consists of two layers of bars 10 mm at 20 cm.The yield stress of reinforcement used was 3600 kp/cm^2,and the specified strength of concrete 200 kp/cm^2.

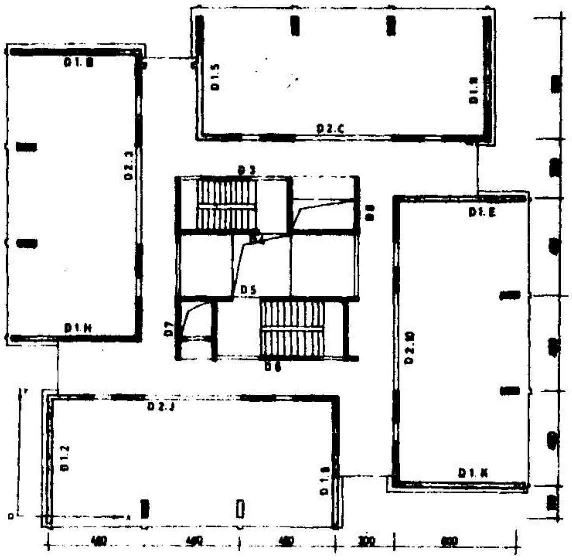

Fig. 1 Typical floor plan

The building was designed by the Design Center of the Ministry of Education in Bucharest and it was analysed for lateral loads according to the earthquake regulations at the time of design(p.13-70).The review of this results showed that the seismic load corresponded to VII degree of intensity,with a shear base factor of $c = 2.62\%$ .After the earthquake,at the Design Institute "Carpati"-Bucharest,an analysis was performed and all member forces were obtained,with the conclusion that all reinforced concrete sections are sufficient and consequently the first design was correctly prepared.

3. GROUND MOTION AND STRUCTURAL DAMAGE

Ground motion of March 4,1977 was recorded in Bucharest at the Institute INCERC. The three components of the record(Fig.2) identified an earthquake with characteristics very different from those which we know as classical (earthquakes in California,Japan and USSR). The characteristics of this earthquake for Bucharest can be synthesized as follows:

- the maximum values of the ground motion for the N-S direction were: a_m = 208,2 cm/sec^2 (0,21 g), v_m = 74.9 cm/sec and d_m = 25,4 cm/sec.
- the mode shape of the predominant pulses has a period larger than 1.0 sec.
- the response acceleration spectra has an upper ceiling with periods of 0.5 to 2.0 and a lower ceiling with periods larger than 3.0 sec.These three characteristics made the conditions of the structures rather unfavourable with periods larger than 0.5-0.6 sec (semi-rigid and flexible structures).The Magurele building has an initial period of vibration of 0.7 sec,according to the design,but during the earthquake the period increased due to the decrease of rigidity and put the structure in an unfavourable condition regarding the periods.

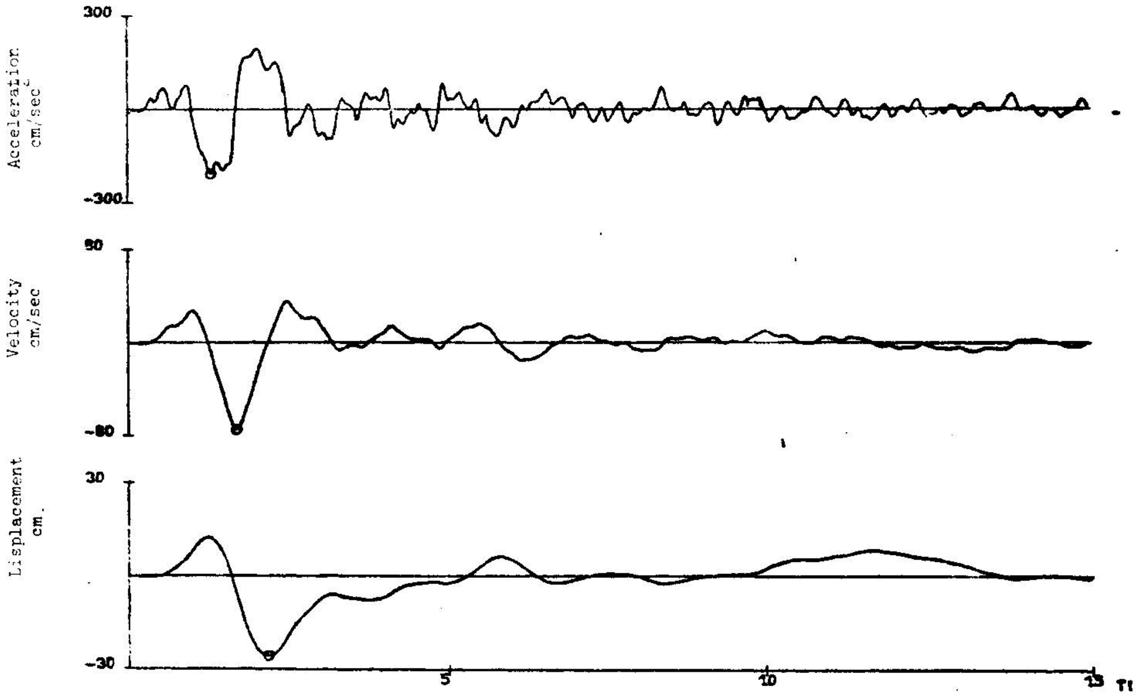

Fig.2 Record of the earthquake of March 4,1977 - INCERC

During the earthquake the main structural system suffered moderate damage. Many cracks in corridor walls and wing walls ($D_2.3$ and $D_1.5$ in Fig.1) were observed and less in core walls (D_3 to D_8 in Fig.1). Cracks in walls and deep beams were inclined, whereas flexural cracks at the ends of the shallow beams appeared. Also, some cracks in the floor slabs, near the openings, were observed. Cracks in walls with openings were distributed over all height of the building, whereas in walls without openings were limited to the bottom part only.

4. STRUCTURAL MODELING

The elastic analysis was performed for the whole building, taking into account all the walls in both directions, by application of the TABS [2] programme. The walls with openings are treated as frames with rigid zones, while the walls without openings act as single columns fixed at the base. For the elastic dynamic analysis twelve modes and viscous damping of 4% of the critical in each mode of vibration were taken. The masses are concentrated at the level of floor slabs and are determined on the basis of 1.3 tons on a square meter of gross area.

The inelastic dynamic analysis was carried out applying the DRAIN-2D [3] programme. For the analysis the walls D_{1-5} and D_{2-3}, treated separately as plane structures with masses concentrated at the nodes, have been chosen. The wall D_{1-5} is divided into twenty members with lower height in the lower part, and above the fifth storey the members have the storey height (Fig.3). Plastic hinges can be formed at both ends of the members during the dynamic action of the earthquake. The wall D_{2-3} is treated as a three-span frame (Fig.4), consisting of two types of members, columns and beams. The ultimate capacity of beam and column sections was calculated using computer programs, and given as interaction diagram M-N for the columns and yield moment M_y for the beams.

The viscous damping is taken to be 4% of the critical and, based upon the periods for two modes of vibration, the damping as proportional to the mass and rigidity was calculated.

The data from the accelerogram obtained at INCERC have been digitized at IZIIS and used as input for elastic and inelastic dynamic response analysis. In total 875 steps of 0.008 sec, or 7 sec. are taken into account for the time history of the building behaviour.

5. ELASTIC ANALYSIS

The elastic analysis for the whole building was conducted supposing that no collapse of the structural members as a result of the earthquake will occur, i.e the behaviour of the members will be linear. In addition to the earthquake effect, the building was analysed for vertical load and for statical horizontal forces equivalent to the shear base factor of $c= 8\%$.

The envelope of the maximum floor displacements is shown in Fig.5. The considerable differences between the displacements point out the need for inelastic dynamic analysis, since these displacements cannot be reached in the elastic behaviour of the structural members.

During investigation of the ability of the building to withstand the March 4, 1977 earthquake, the ratio between "the maximum moment at a section from the gravity and seismic loading, divided by the ultimate mument capacity of the section" was required [4]. These relations are shown in Fig.6 as "ductilities" for the beams of the wall D_{2-3}. The maximum moments from the earthquake are considerably larger than the flexural capacity of the sections, which results in the very high "ductilities".

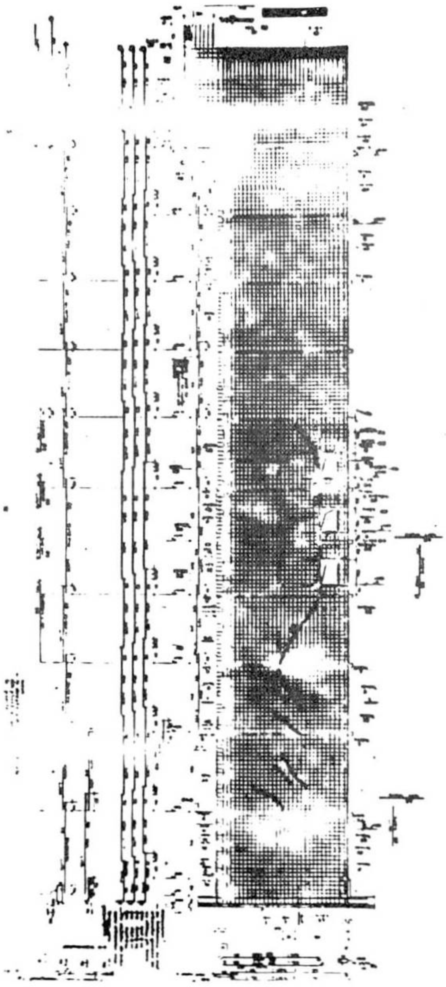

Fig.3 Elevation of wall D_{1-5}

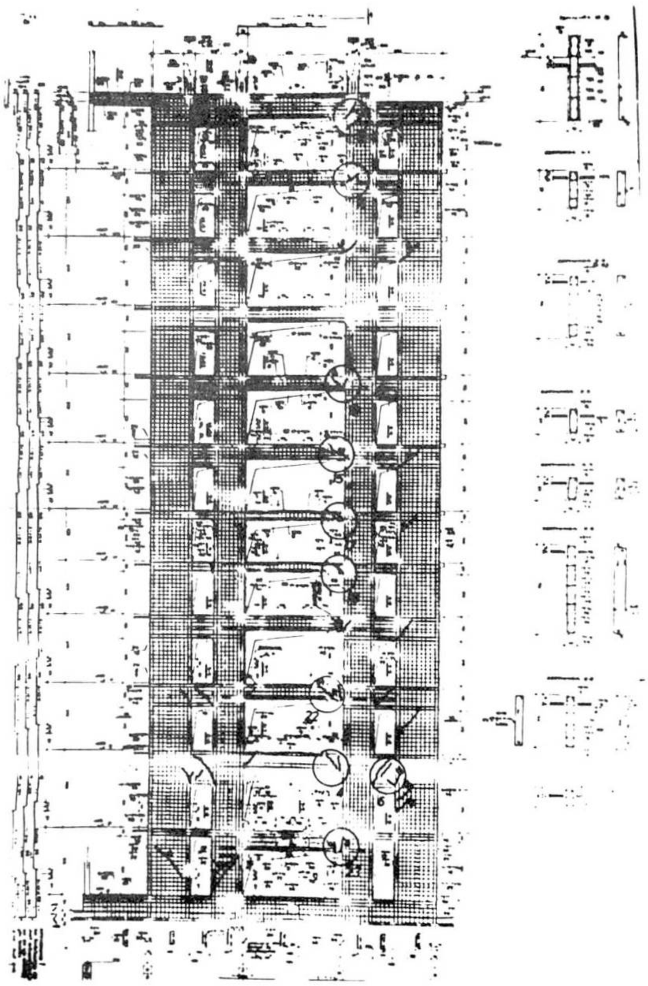

Fig.4 Elevation of wall D_{2-3}

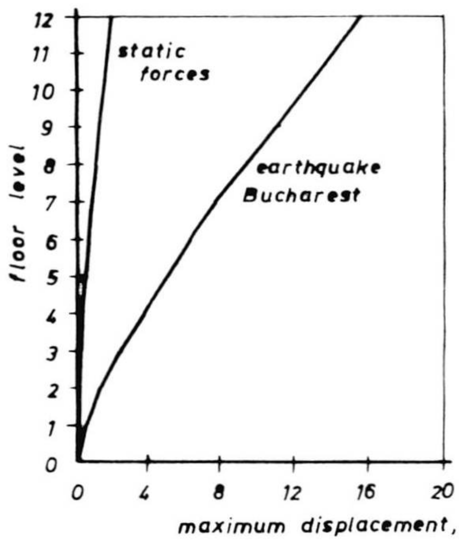

Fig.5 Envelopes of maximum elastic floor displacements

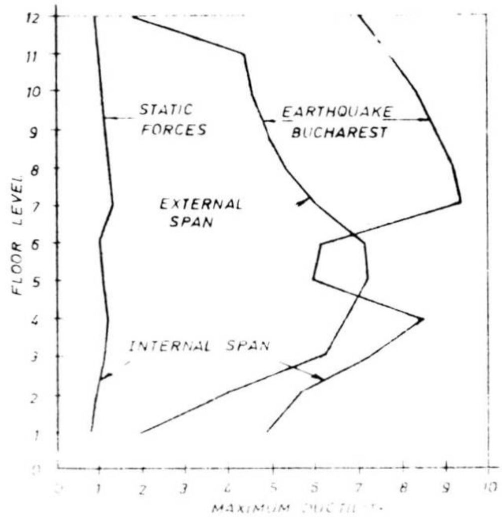

Fig.6 Envelopes of maximum beam ductilities, D_{2-3}

6. INELASTIC ANALYSIS

The inelastic dynamic analysis of the walls D_{1-5} and D_{2-3} is based upon the bilinear histeresis relationship moment-rotation of the wall members,in which the deformations are concentrated at the ends of the members.The version of the DRAIN-2D programme used do not posess the option for stiffness degrading of the beams due to previous yielding,but it was pointed out that the influence of degrading stiffness is not significant [5] .

Before the dynamic action,the gravity load has been applied.For the applied acceleration the dynamic response of the walls for the following values is required:

- horizontal displacement of the nodes
- vertical displacement of the nodes
- rotation of the nodes
- member forces
- plastic rotation at the end of members.

These values are obtained for each 0.2 sec of the adopted time duration of the earthquake.At the end of the analysis,the maximum values for the member forces and the plastic rotations in all members are obtained.

The fig.7 illustrates the maximum storey shears obtained from elastic and inelastic analysis.Although direct comparison is not possible,due to the separate treatment of the walls in the inelastic analysis,it could be stated that the shear forces obtained by the elastic analysis are too high.

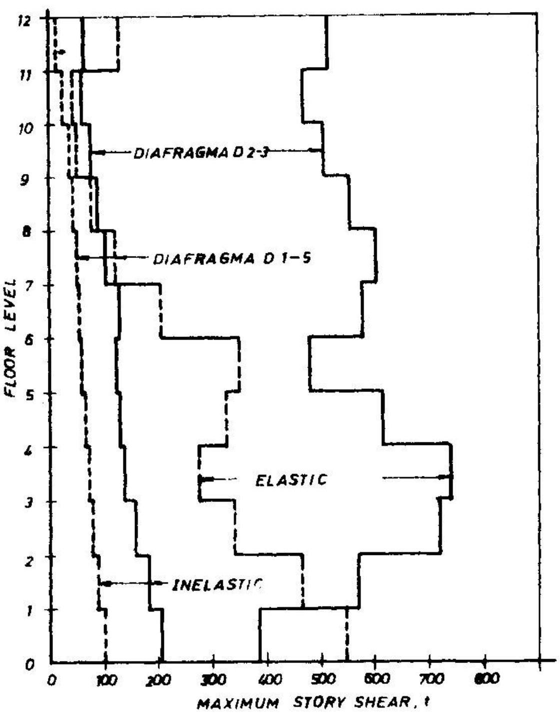

Fig.7 Envelopes of maximum storey shears

The time history of the displacement at the top of the walls D_{1-5} and D_{2-3} is shown in Fig.8.It could be said that the maximum elastic and inelastic displacements were similar.

In the Fig.9 the plastic rotation at the base of the wall D_{1-5} and at the bases of the external and internal column of the wall D_{2-3} are shown.While in the wall D_{1-5} plastic rotations are observed only at the first three stories,in the wall D_{2-3} plastic rotations occured along the whole height,especially at the coupled beams.

The Figs. 10 and 11 show the change of the moments and the axial forces at the base of the columns in D_{2-3} and sometimes the axial forces are larger than those corresponding to the balance point.

Fig 12 illustrates the location and the sequence of forming of plastic hinges in the wall D_{2-3}.The plastic hinges indicated did not occur simultaneously,but generally in groups concentrated at a floor level.

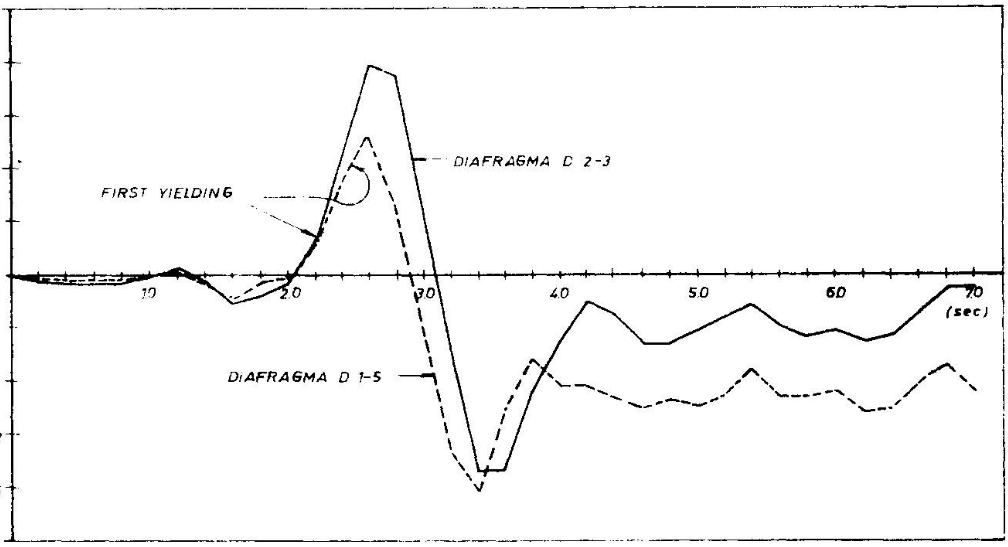

Fig.8 Time history of horizontal displacements at the top of the structure

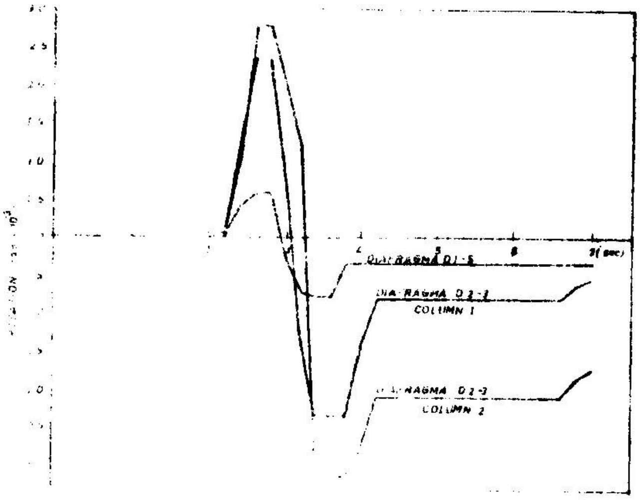

Fig.9 Time history of plastic hinge rotations at base of D_{1-5} and D_{2-3}

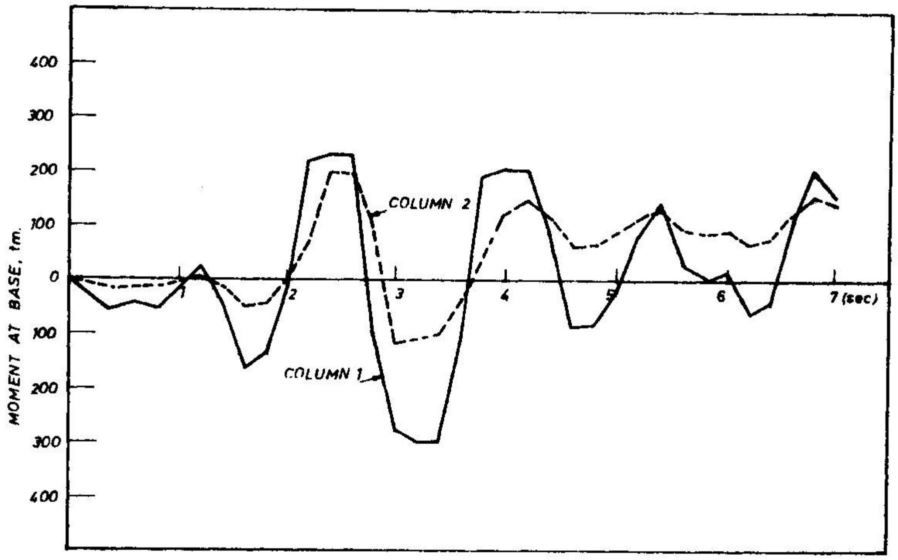

Fig.10 Time history of bending moment at base of D_{2-3}

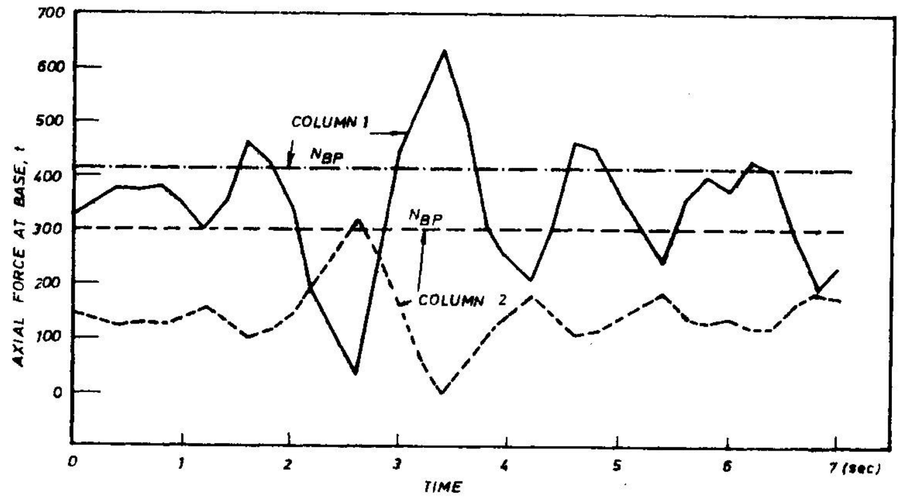

Fig.11 Time history of axial forces at base of D_{2-3}

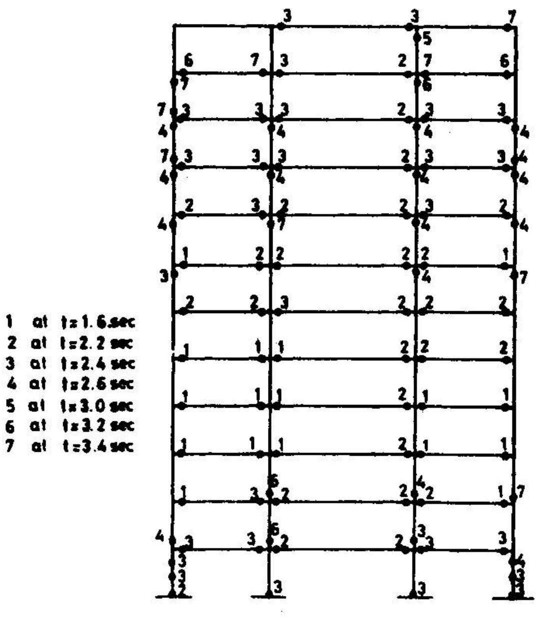

Fig.12 Sequence and location of plastic hinges,D_{2-3}

7. CONCLUSIONS

In summary of the analysis carried out the following conclusions can be drawn:

- The structure of the building was designed according to the Earthquake Regulations in Romania,for an intensity of VII degree,which is not in accordance to the actual intensity and other dynamic characteristics of the March 4,1977 earthquake.

- The most important structural members of the building are the shear walls,frame walls and the core.The specific behaviour of these members points out to the large stresses in the floor slabs,which can be proved by the observed damage. For mixed systems the slab is important structural member as well as the vertical members.

- The comparison of the story shears from the elastic and inelastic response shows that the latter are considerably smaller,which means that the structure acts nonlinearly and requires a source of damping ,not only viscous,but also through inelastic deformations.

- The coupling beams of the walls with openings are very suitable members for energy dissipation and the suggestion for strengthening of the building was to make them good elasto-plastic dampers.

- The analysis carried out clearly indicate that the superstructure of the building should be strengthened ,but it should not be more than the capacity for overturning moment at the joint of the superstructure and infrastructure.

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EFFECTS OF VRANCHA EARTHQUAKE OF 4 MARCH 1977 ON THE TERITORY OF BULGARIA

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SUMMARY

The spectral characteristics (predominant periods,Fourier transforme spectra,response spectra) of Romanian earthquake on 4 March 1977 are analysed in respect to explain the couses of destruction of buildings located 200-500 km from epicentra.

Effects of the earthquake on masonry buildings ,reinforsed concrete frame,large panel and industrial buildings are analysed.

SOMMAIRE

Les characte ristiques spectrales (periodes predominants,spectre de transformation de Fourier,spectre de response) du tremblement de terre a Roumanie de 4.III.1977,sont analysies. Ces characteristiques sont utilisees comme explication des facteurs qui ont produit les degats des batiments et des constructions,situes a environ 200-500 km de l' epicentre.

Les effets du tremblement de terre sur des maconries des batiments fait on beton arme monolitique et prefabrique,sont analyses.

ZUSAMMENFASSUNG

Die spectraliche Charakteristiken (die dominierende Perioden, das Fourierisches Transformationspektrum,das Responspektrum) des Rumanischen Erdbebens vom 4 Marz 1977 sind analysiert. Diese Charakteristiken sind notwendig um die Zertorungsgrunde der Bauwerken, die 200-500 m vom Epizentrum entfernt sind,festzustellen.

Der Einfluss des Erdbebens uber Mauer-und stahlbetonwerke (monolithische und aus Fertigbauteile) ist beschrieben.

1. INTRODUCTION

The earthquake of 4 March 1977 with epicenter Vrancha-Romania, focal depth of 110 km and magnitude 7,3 had an effect on the terito ry of Bulgaria with intensity IV-VIII according to MSK-64 scale. This earthquake colapsed few new reinforce concrete buildings, heavily damaged hundreds and craked towsends. Many tall buildings in Sofia, located 450 km from epicenter, were significantly craced and some of them had to be repared. Three R.C. frame buildings in Svistov on Danube bank with flexible first storey were complitely colapsed and kiled 138 habitans.

The earthquake have had an strong effect over the all teritory of Bulgaria damaged many old and new modern residential and industrial buildings built by monolitic reinforce concret and precast panels and elements. This is the reason the effects from the earthquake to be analysed and conclusions for improvment of the earthquake resistant designing to be suggested.

2. SPECTRAL CHARACTERISTICS OF VRANCHA EARTHQUAKE

The specific mechanism of Vrancha earthquake with focal depth of 110 km had generated seismic waves with long predominant periods 1-2,4 sec which strongly effected the tall and flexible buildings up to 500 km.

On the accelerograme recorded in Bucharest (INCERC), at distance 170 km from epicenter, can be seen tipical sinusoidal character of the motion after 18 sec for both NS and EW components (fig.1). The accelerograme in Nish (453 km from epicenter) (fig.2) contain similar long period motion combined with short period waves specified by the geological conditions.

The Fourier transform spectra from Vrancha earthquake for Bucharest [1] and Nish [2] records and experimentaly determined period [3] at right bank of Danube river (300 km from epicenter) are given on (fig.3). It is evident that specific deep geological conditions with short natural predominant periods for surface layers generate aditional short motions (fig.2). Well known filtration of the short period waves with distance is not observed in this case. There are some regions on the teritory of Bulgaria which generate short period motions and others one-long periods. This fact was confirmed by the behaviour of the structures with diferent natural periods located at different ground conditions.

The response spectra for Bucharest record NS componen, El Centro 1940 NS component and Svishtov NS probable component at 0% critical damping are given on (fig.4). It is evident that Vrancha earthquake effect significantly the flexible structures with natural periods T > 1 sec. On this bases can be explaned the large numbers of the demages in flexible structures. In some specifc regions smal buildings of one-two storeys had been heavyly damaged

The isoseismal map of Vrancha earthquake on the teritory of Bulgaria (fig.5) is influenced significantly by the geological conditions-specialy on alluvial deposits at the rivers valey. The most affected area-right bank of Danube is characterised by alluvial deposits and loessoidal surface layers of thikness 5 to 30 m.

3. EFFECTS OF THE EARTHQUAKE ON THE BUILDINGS

The eartquake of 4 March 1977 damaged mainly nondesigned for earthquakes buildings and structures. The buildings designed for earthquake intensity VII or VIII got nonstructural cracks only in infilling walls, joints and so on.

3.1. One-two storeys brick masonry buildings

In the most effected area-Svistov (point 1 on fig. 5) one storey brick masonry buildings did not get any cracks. Two storey very old buildings (fig. 6) with wooden floors did not suffured significantly. Some of them got small cracks (fig. 7)Some old buildings from last century (fig. 8) not designed for earthquakes but well built with steel conections between walls were damaged mainly in connections between timber roofs and masonery. In some other areas (point 2 on fig. 5) one and two storeys masonry buildings were heavy damaged. Those buildings had been built with low strength bricks and mortars (mud or lim-sand), timber floors and roofs non conected well with walls(fig. 9).

The damages of this type buildings in diferent areas depend of geological conditions and specialy of predominant periods of surface layers.

3.2. Reinforce concrete frame buildings with flexible first storey

The most suprising effect of this eartquake was the total colaps of three reinforce concrete buildings without shear walls in first floor. This colapse can be explained by specific response spectra with maximum at $T > 1$ sec (fig. 4), sinosoidal exitation in both directions, large horizontal displacements and additional influence of $P-\Delta$ effect on the bending moments into the columns.

The fifth storey office building (fig. 10) is tipical building with flexible first storey. The existing of shear walls at the stairs only develop additional rotational effects and reserve partly only first two storeys at the stairs (fig. 11). The cilindrical columns of the building are well designed (fig. 12) but connections between girders and columns did not forme space resisting frame for horizontal exitation. The small length of down girders steel bars into columns (fig. 10 b) was not capable to bear suficient bending moments from the earthquake. Similar construction had the sixt storeys public residential building wich was colapsed on the same way (fig. 13).

The nine storeys apartment building with shops (without shear walls) into first storey was colapsed as previous two buildings (fig. 14). Another two nine storeys buildings with the same construction but with masonry shear walls in first storey have resisted very well to the earthquake (fig. 15).

One more example of distortion this type construction is the total colapse of industrial bunker constructed like renforce concrete box suported on four R.C. columns (height-6m).

Many reinforce concrete frame buildings and package lift-slabs buildings with infiling masnry walls were cracked mainly in walls never maind they were not designed for earthquake.

3.3. Large panel and precast buildings

Many large panel buildings were effected by the earthquake but of the reason of low natural periods ($T \approx 0,4$ sec) they did not get any damages. Not designed for eartquake large panel buildings in some regions were lightly cracked in horizontal joints and corners of the doors.

The damages in many industrial buildings were mainly from bed conections between roof trusses and columns (fig.16) between roof ,wall panels and columns.

4. CONCLUSION

The specific spectral characteristics of Vrancha earthquake of 4 March 1977 effects mainly flexible structures. In some regions with rigid ground conditions the exitation was strong on small rigid buildings. Many tall buildings in Sofia (about 20 storeys) located on aluvial ground were cracked and some had to be repared. This super long distance effect of Vrancha earthquake is influenced from specific mechanism of the earthquake and deep geological conditions of respective regions. The buildings with flexible first storey have to be designed taking into consideration the large displasiment of the ground, long distance effect and resonance from sinosoidal waves. In the regions with longe distance effects this type of construction have to be avoid.

Large panel and precast constructions non designed have resisted very well to this earthquake. Special attantion have to paid to the quality of realization on place of the joints between separate elements.

Earthquake resictance designed and well built constructions, according to the normes, have resisted very well to the earthquake.

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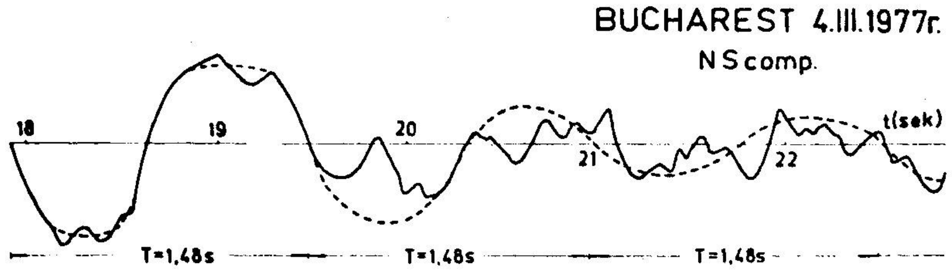


fig. 1

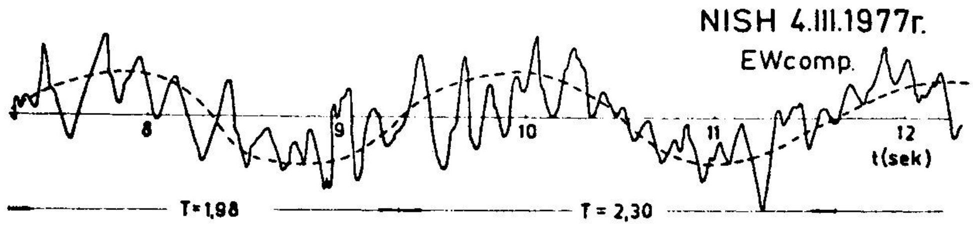


fig. 2

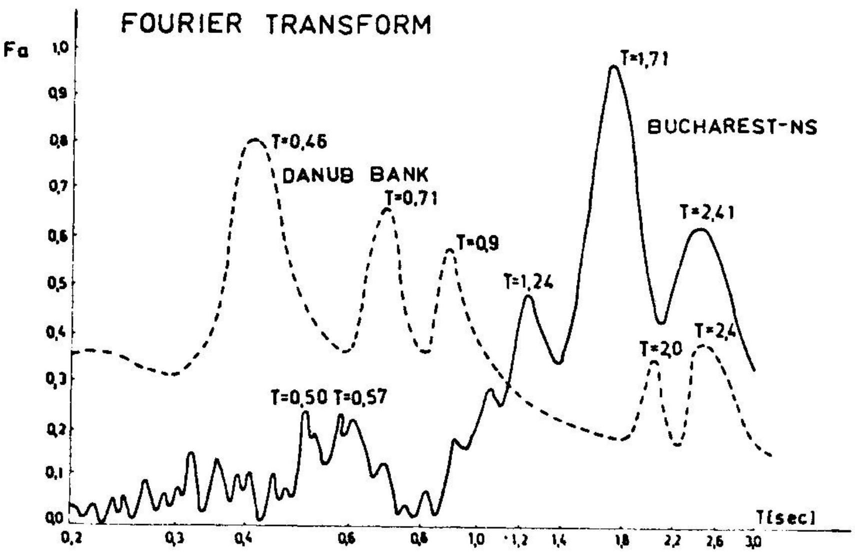
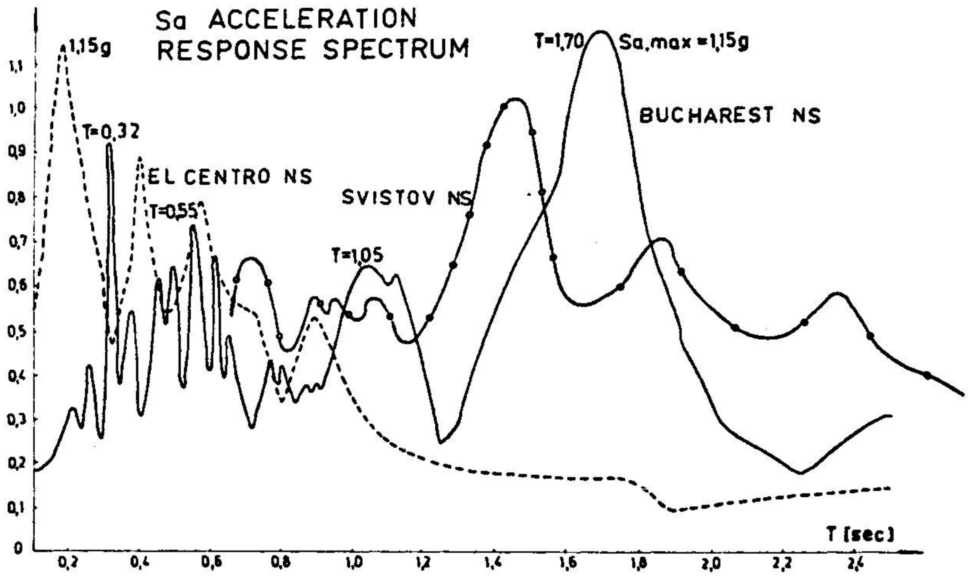


fig. 3


Sa ACCELERATION
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1,15g
T=0,32
EL CENTRO NS
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T [sec]

fig. 4

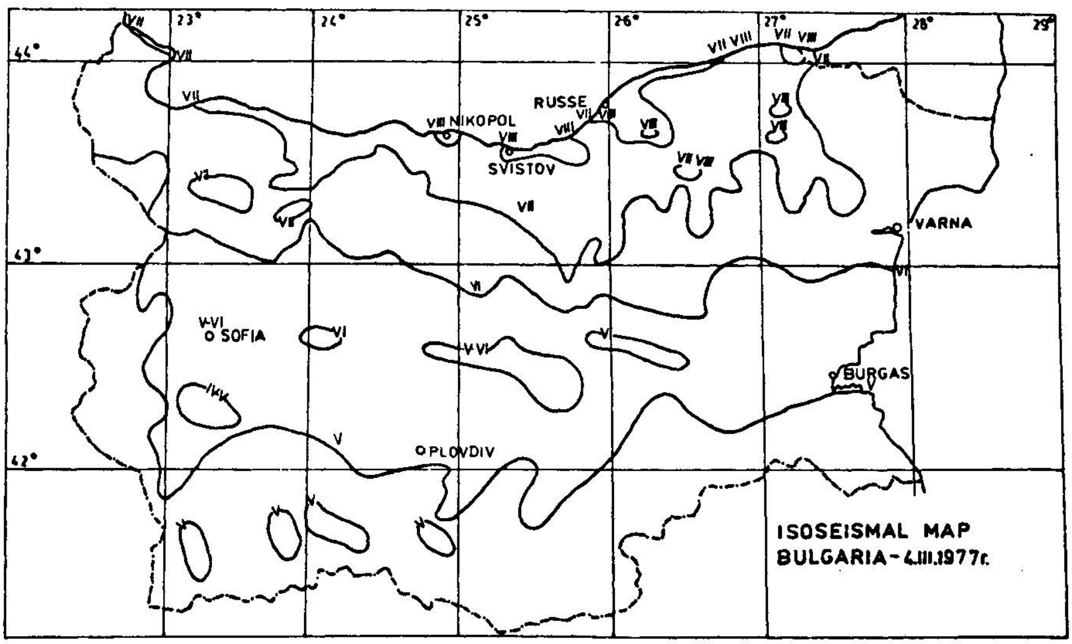
NIKOPOL
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fig.5

fig. 6

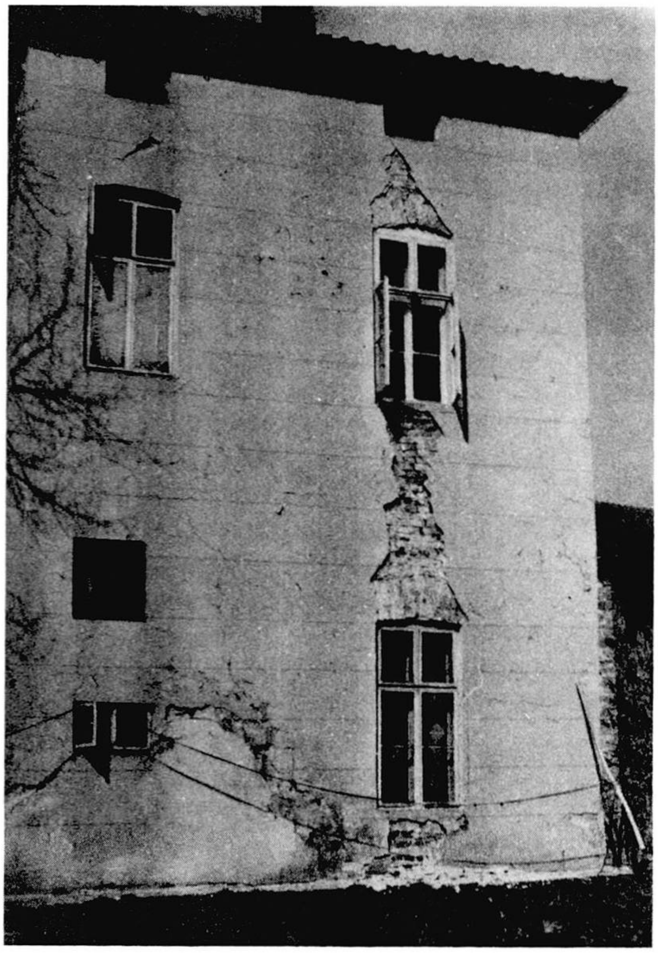

fig. 7


fig. 8


fig. 9

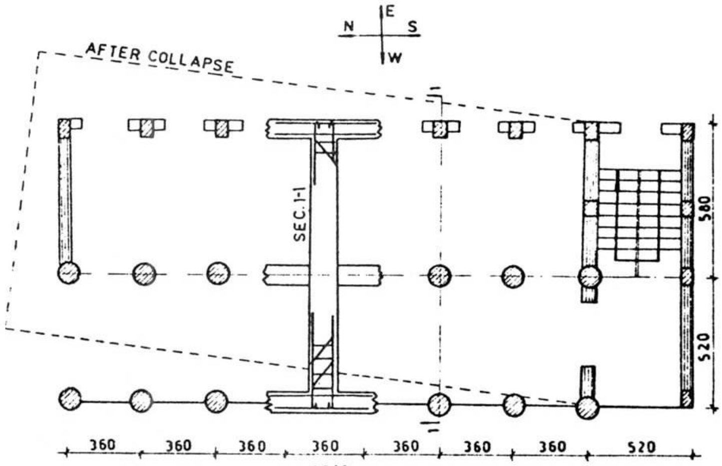

fig.10

fig. 11

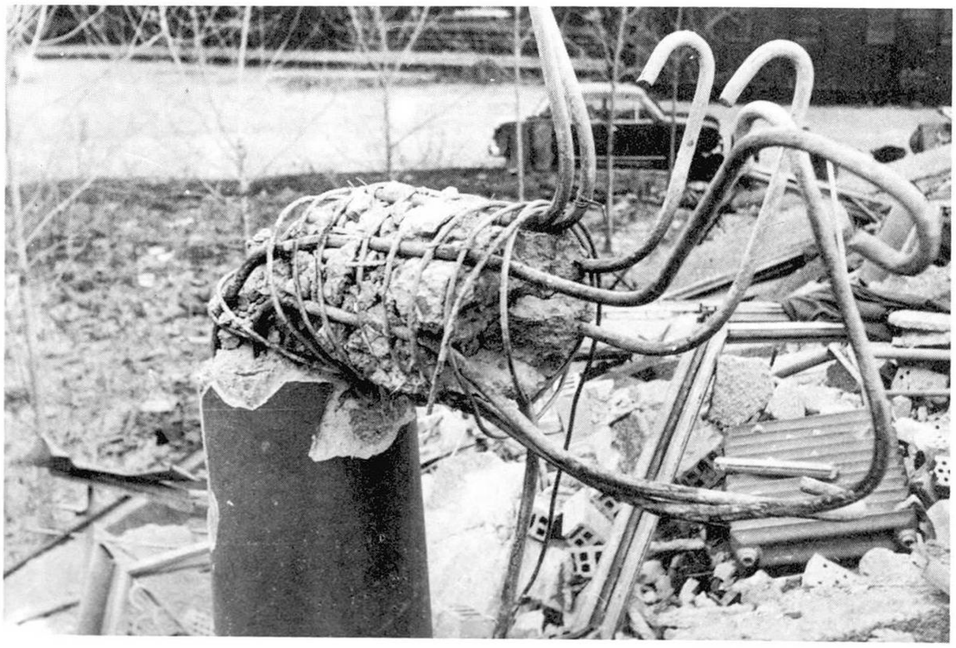

fig. 12

fig. 13

fig. 14

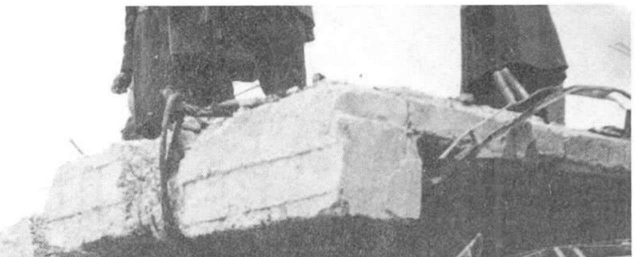

fig. 15

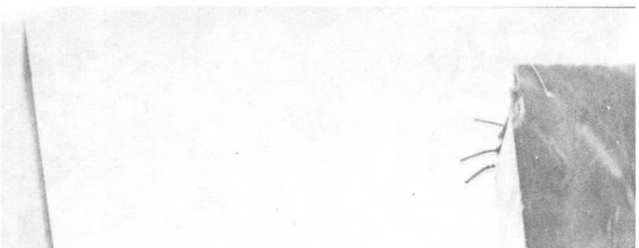

fig. 16

BEHAVIOUR OF BRICK MASONRY BUILDINGS DURING EARTHQUAKES

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SUMMARY

The characteristics and behaviour of brick masonry buildings in Turkey with reference to earthquakes, the measured dynamic properties of several masonry structures, and "Earthquake Resistant Design Code" provisions for brick masonry structures in Turkey are presented and the behaviour of several brick masonry buildings during Nov. 24, 1976 Çaldıran and March 25, 1977 Palu (eastern Turkey) earthquakes are compared with Code provisions. The importance of bearing wall length/floor area ratio and external wall opening ratio is studied and their importance for efficient earthquake resistant is stated.

1. INTRODUCTION

Brick masonry is very common and widely used construction system in Turkey. Approximately 70 % of all the new housing construction is in brick masonry. However the quality and strength of bricks available in Turkey prevents the construction of brick masonry houses higher than four stories. With the rapid growth of cities and the rise of land prices, the trend is to build high rise buildings, at least in large cities and they are all in reinforced concrete. Reinforced masonry is not used to construct high rise buildings since the same amount of steel is required for reinforced concrete construction and cement is readily available. In spite of that considerable number of brick masonry structures are still being built in smaller towns and even at the outskirts of large cities. Considering the high seismic activity of Turkey, the earthquake resistant design and construction of brick masonry structures is still a main topic of concern among Turkish civil engineering circles.

2. CHARACTERISTICS OF BRICK MASONRY STRUCTURES IN TURKEY

2.1 Bricks

In Turkey there are two obligatory standarts covering the production of clay bricks [1], [2]. The dimensions of standart bricks are 19x9x5 centimeters. The handmade bricks are produced by very primitive methods, dimensional, strength and other properties of hand made bricks are non-uniform. The temperature of burning is generally lower than factory made bricks which are stronger and dimensionally uniform. Hand made bricks are burned in simple kilns where control of temperature and rate of burning is impossible. Thus they show very inferior qualities. Even the quality and strength of factory made bricks are lower than corresponding bricks produced in Europe. Since the factory made bricks require considerable investment, they are relatively more expensive. In 1975, while factory made bricks costs 0.40 Turkish Liras (2.9 US cents), hand made brick costs 0.25 Turkish Liras (1.8 US cents). The usage of hand made bricks is very wide, perhaps 60 to 70 % of all the bricks used in Turkey are molded by hand and burned in simple kilns.

2.1.1 Compressive Strength of Turkish Bricks

Based on the brick compressive strength tests carried out at the Directorate of Building Materials of Ministry of Reconstruction and Resettlement the following brick compressive strengths are observed[3]:Table-1

TABLE-1
Compressive Strength of Bricks Tested

Brick type	Hand Made Solid	Factory Made Solid	Factory Made Hollow Load Bearing	Block Filler Brick
Average Compressive Strength	55 kg/cm2	236 kg/cm2	188 kg/cm2	44 kg/cm2
Variation of Strength	33 %	19 %	20 %	25 %

These values are very low in comparison with the brick strengths available in European countries and United States. These low values prevents the construction of high rise brick buildings even for purely vertical loads, because of the generally high safety factors used in brick masonry construction requires very thick walls in the lower floors of a high rise building.

2.1.2 Compressive Strength of Brick Masonry

Although there is a widespread usage of brick masonry in Turkey, the number of tests made on brick masonry walls to determine their mechanical properties are very few. In one of these tests carried out by Tolunay [4] the following results were obtained: Table-2

TABLE-2

Mechanical Properties of Brick Masonry In Turkey

	Factory Made Brick	Hand Made Brick
Brick Modulus of Elasticity	$E = 74\ f'_B$	$E = 112\ f'_B$
Brick Wall Mod. of Elasticity		
Lime Mortar 1:3	$E = 35\ f'_W$	$E = 45\ f'_W$
Cement-lime-sand Mortar 1:2:8	$E = 97\ f'_W$	$E = 97\ f'_W$
Brick Wall Compressive Strength		
Lime Mortar 1:3	$f'_W = 0.20\ f'_B$	$f'_W = 0.27\ f'_B$
Cement-Lime-sand Mortar 1:2:8	$f'_W = 0.27\ f'_B$	$f'_W = 0.28\ f'_B$

In this table f'_W is the brick wall ultimate compressive strength and f'_B is the brick ultimate compressive strength. These ratios are derived from a very limited number of test specimens and their usage is not very dependable, they can only be used if the brick compressive strength is known with sufficient accuracy.

2.1.3 Shear Strength of Brick Masonry

During the earthquakes brick masonry structures are subjected to loads that develope shear and diagonal tension stresses in the walls. Thus these properties of brick masonry gain importance, sometimes it is more important than the brick or masonry wall compressive strength. The diagonal tensile strength of brick masonry is expressed in formulae of the following form:

$$\tau = \tau_o + \mu\sigma$$

In this formula τ is the diagonal tensile strength (shear) of the wall, τ_o is the shear strength without any vertical stress and can be accepted as the adherence between brick and mortar, σ is the vertical stress in the wall and μ is a coefficient of friction. According to tests made by Yorulmaz and Atan[5] in Turkey μ and τ_o values are dependent on brick and mortar type. They give the following values for μ and τ_o: Table-3

TABLE-3

τ_o and μ Coefficients For Shear Strength of Brick Masonry

Brick Type	Load Bearing Hollow Brick		Gas Beton (YTONG)		Filler Block Brick	
	Cement Mortar	Cement-lime Mortar	Cement Mortar	Cement-lime Mortar	Cement Mortar	Cement-Lime Mortar
μ	0.43	0.41	0.68	0.69	0.34	0.21
τ_o(kg/cm2)	5.14	4.02	1.52	1.53	3.64	5.28

One other important property of brick masonry is the adherence between mortar and brick. Tests carried out by Erdülek [6] show that this is also a property which depends on brick and mortar properties. Table-4

TABLE-4

Adherence Between Brick and Mortar (kg/cm2)

Mortar Type / Brick Type	1:4 Cement:Sand	1:1:4 Cement:Lime:Sand	1:2:4 Cement:Lime:Sand
Hand Made	1.6-0.03	1.9-0.45-0.09	1.6-0.02-0.03
Factory Made Solid	1.5-1.06	1.8-0.64-0.31	3.32-0.82-0.03
Factory Made Hollow	2.[illegible]7-1.8	3.23-2.23-1.69	3.37-0.67-1.57

In these tests three different mortar types were used but their average tensile strength were in the order of 4 kg/cm2 (compressive strength of roughly 40 kg/cm2). These values correspond to values obtained from sample bricks which were water saturated, as it is, and oven dried. It seems that the values corresponding to bricks tested as they are should be taken as the case which represents the actual conditions. Although relatively scattered values for has been obtained in tests, it seems that a value not greater than 1 kg/cm2 for could be accepted as the probable value.

Considering the results of these two researches the shear strength of brick masonry walls in Turkey can be expressed as follows:

$$\tau = 1.0 + 0.45\,\mu$$

Taking into account the average dimensions of and loads on brick masonry the vertical stresses coming to ground floor walls of one to four stories high brick masonry structures are 1.40, 1.55, 2.50 and 3.60 kg/cm2; with these vertical stresses, the shear strength of brick masonry walls in

Turkey can vary between 1.63 and 3.62 kg/cm2.

2.2 Mortars

In Turkey the type of mortar which is commonly used is cement reinforced lime mortar. Mortar strengths are generally very low, in view of the low brick strengths, the use of relatively weak mortars is justifiable since the usage of high strength mortars do not bring a considerable rise in brick masonry strengths. The most widely used mortars contain lime plus some amount of cement. The usual lime sand ratio is 1 to 3 and if cement is added then the cement lime ratio is aproximately again 1 to 3. Again the number of tests on various mortar ratios are not very much in Turkey. The strength properties of the mortars used in various tests [4], [5], [6] are given in Table-5

TABLE-5

Mortar Strengths in Turkey

Mortar Ratio Lime:Cement:Sand	[4] 1:0:3	2:1:8	0:1:4	[6] 1:1:4	1:2:4	[5] 0:1:4	1.5:1:8
Comp. Strength (kg/cm2)	7.0	10.0	-	-	-	117-177	33-47
Tensile Strength (kg/cm2)	2.5	3.0	4.8	3.8	3.8	26-44	8-10

As seen from table above, the variation in mortar strengths is very high, the mortar strengths given in reference 5 are for mortars using sand with precisely determined gradation curves, while those taken from Reference [4] could be considered as being closer to the one which can be expected in actual construction conditions. In many cases low mortar strengths have been the cause of extensive earthquake damage.

2.3 Brick Masonry Design Code

It is sad fact that there is not a specific design code with regard to brick masonry in Turkey. Lately there has been an attempt to formulate a standart for the design of brick masonry construction but it is not published as yet. There are some guidelines as to the allowable compressive stresses which can be used in masonry walls with respect to their being stone or clay brick and the kind of mortar used as lime, cement reinforced lime and cement mortar. Table-6. In case of stone masonry these values are decreased by 20 percent.

TABLE-6

Allowable Compressive Strengths for Masonry

Wall Type	Brick Wall Lime Mortar	Brick Wall Lime and Cement	Brick Wall Cement Mortar
Allowable Compressive Stress kg/cm2	5 kg/cm2	8 kg/cm2	10 kg/cm2

2.5 Earthquake Resistant Design Code Provisions for Brick Masonry Buildings

In the absence of any brick masonry design and construction regulations in Turkey, it had been felt to incorporate many provisions concerning brick masonry into the earthquake resistant design code of Turkey [7]. In the following parts some of the important provisions of the code with respect to earthquake behaviour will be given.

2.5.1 Building Height Limitations

According to the earthquake zoning map of Turkey, the country is divided into 5 seismic danger regions. In zones I and II brick masonry houses can be only two stories high (ground floor and 1st floors), in zone III brick masonry buildings can be three stories high (ground, 1st and 2nd floors) and in zone IV brick masonry houses can be four stories high (ground, 1st, 2nd and 3rd floors). In zone V which is taken as seismically inactive, there is no height limitation for brick masonry houses. However as already explained above the quality of bricks available in Turkey prevents construction of brick masonry houses higher than four stories. The code besides limiting the height of brick masonry construction, also specifies the minimum brick wall thickness of each story.

For two story brick masonry houses the ground floor wall thickness must be 1.5 brick size (29 centimeters), while the upper floor must at least be 1 brick thick (19 cm.). For three story high brick masonry construction, the ground floor wall thickness must be at least 1.5 brick size (29 cm.), while the upper two stories must be at least of 1 brick thickness (19 cm.). For four story high brick masonry houses, the ground and first story walls must have a thickness of 1.5 bricks (29 cm.) and the rest would have a thickness of 1 brick. For single story high brick construction the minimum wall thickness is one brick (19 cm.). If these construction have basements, the basement walls should be at least 50 centimeters thick stone masonry.

2.5.2 Openings in Walls

This is the most detailed part of the code [7]. It is based on the assumption that solid walls between openings are the lateral load carrying elements and their size effects the safety of brick masonry construction during earthquakes.

In the code the total length of the openings in an external wall should not be greater than 40 % of the length of that wall.

The maximum size of window or door openings should not be greater than 3.00 meters.

The solid wall between the corner of the building and the first window or door opening on that wall should at least be 1.50 meters in I and II degree earthquake zones and 1.00 meters in III and IV degree earthquake zones. If the building is less than 7.50 meters high, these solid wall lengths could be reduced to 1.00 meters and 0.80 meters in the respective earthquake zones.

The solid wall length between two openings (door or window) while being not less than 1/4 of the length of larger opening, should also be at least 0.80 meters in I and II earthquake zones and 0.60 meters in III and IV degree earthquake zones.

2.5.3 Lintel Beams

While the parts of lintel beams resting on the walls should extend at least 0.20 meters beyond on each side of the opening, these parts should also be greater than 15 % of the span of the lintel beam. This provision of the code in practice enforces the placing of a continous lintel beam at the top level of the door and window openings all around the building.

3. DYNAMICAL CHARACTERISTICS AND EARTHQUAKE BEHAVIOUR

3.1 Dynamic Characteristics

Brick masonry structures because of their large sized walls are stiff structures and their periods of vibration are usually low. Period measurements made at the Earthquake Research Institute show that their period of vibration lie usually in between 0.05-0.15 seconds depending on the height of the building. Table-7

TABLE-7

Periods of Vibration of Brick Masonry Buildings

Building Description	Height H (m)	Length L (m)	Width W (m)	Period (sec.)	H/W	Period (sec.)	H/L
One story Brick masonry	2.50	7.80	6.30	0.093	0.40	-	0.32
One story Brick masonry	2.50	9.76	4.55	0.054	0.55	-	0.26
One story Brick masonry	2.50	8.00	6.80	0.045	0.37	-	0.31
One Story Stone masonry	2.50	10.97	9.90	-	0.25	0.042	0.23
Three story Brick masonry	-	-	-	0.073	-	0.049	-
Four story Brick Masonry	11.00	17.25	15.00	0.16	0.67	-	0.64
Two story Brick masonry with basement	9.00	21.00	14.25	-	0.63	0.065	0.43
Two story Brick masonry w/o basement	6.50	21.00	14.25	0.036	0.45	0.045	0.31
Four story Brick masonry with basement	12.00	26.00	24.00	0.164	0.50	0.156	0.46
Three story Brick masonry	7.80	19.70	9.75	0.126	0.80	0.156	0.40

Same as above Heavily damaged in earthquake, with fewer wall opening	7.80	19.70	9.75	0.096	0.80	0.390	0.40

Based on these relatively limited number of brick masonry buildings, the following relation between the period of vibration (T) and the building story number (N) may be suggested:

$$T = 0.05\ N$$

However this relationship needs further investigation. In this form the T value is slightly greater than the actual value of period of vibration. Brick masonry structures due to their rigidity will exhibit very low damping values, probably, never more than 2 %. But in case of earthquake damage the damping may go up as much as 10 %. Comparison of the last two data on Table-7 indicates that very large changes in the period of vibration of damaged masonry structures should be expected.

Significance of low periods of vibration of brick masonry buildings points out that these buildings will be subjected to spectral accelerations almost equal to the maximum ground acceleration. Considering the shear strength of brick masonry, the lateral force which will cause the cracking of brick walls could be established. Once the brick wall cracks its resistance to shear stresses is provided by the friction along the cracks and thus its shear strength after cracking is highly dependent on the vertical loads. Tests must be carried out to determine the shear strength of brick masonry in the cracked state.

3.2 Earthquake Behaviour

Brick masonry is a highly brittle material and under earthquake loads it breaks very easily and even an earthquake of intensity V MSK would be sufficient to have cracking in the walls. Higher intensities of earthquake motion will increase the level of damage. In this part of the paper an attempt will be made to explain the behaviour of brick buildings during earthquakes.

Under the action of lateral forces (wind and earthquake) the longitudonal walls will transfer the lateral loads to the roof or floor slab (if there is one) or to the roof truss and this element will in turn transfer the loads to end walls. Figure-1. Thus the end walls will be subjected to shear forces and when the shear stress exceeds the shear strength of the wall, failure in the form of diagonal tension cracks occurs. These cracks will make an angle of 45 degrees with the horizontal.

Since the earthquake forces will act in the two principal directions of the building, the corners of the building will be very critical. Figure-2

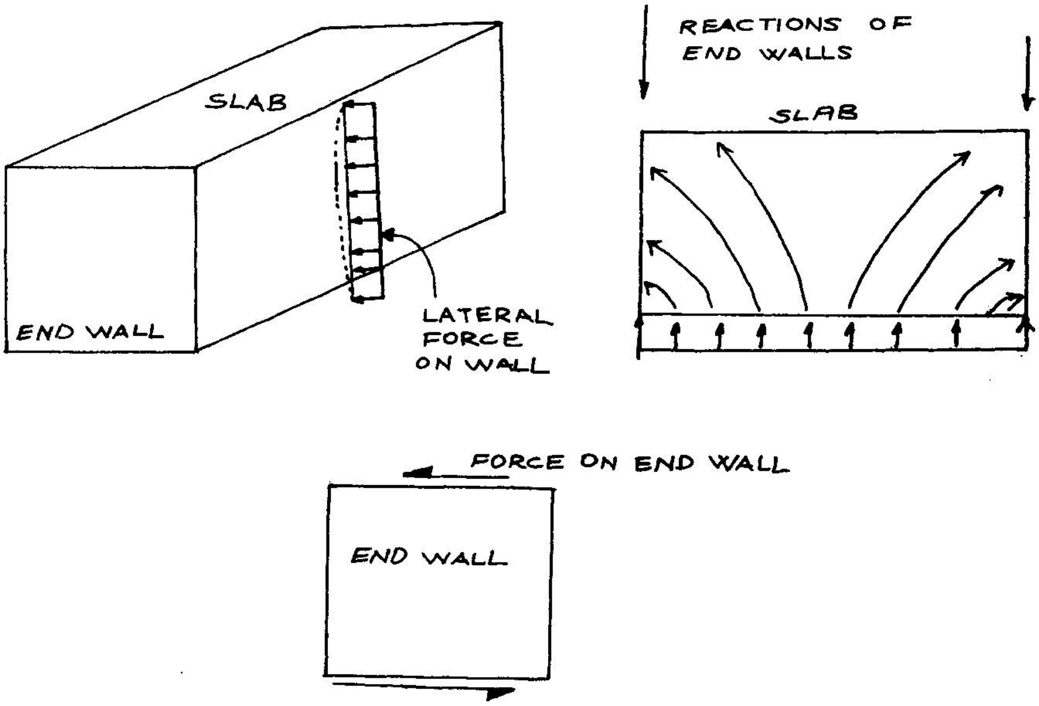

Figure-1 State of Loading on Brick
Walls Due to Lateral Forces

In fact such kinds of corner failures have been observed many times in Turkey. Figure-3. In case when there are no slabs of reinforced concrete or the roof truss is not rigid enough to hold the two cross walls together this form of damage should be expected.

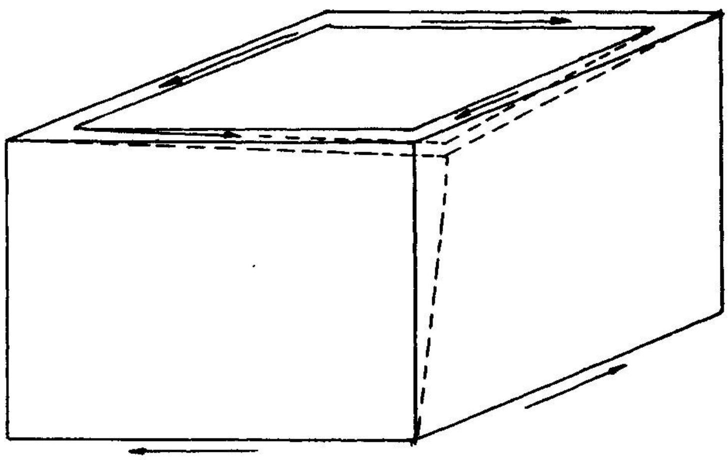

Figure-2 Corner Behaviour of Masonry Buildings.

Figure-3 Corner Damage due to Earthquake

Due to action of shearing forces on the walls of brick masonry, various forms of diagonal tension failures occur. However the walls are also subject to compressive stresses due to the weight of the wall and weight of upper stories. Thus the diagonal tension cracks will deviate from 45° and the angle between the cracks and vertical direction will be smaller than 45° degrees. Apart from the angle of cracks, the extension of the

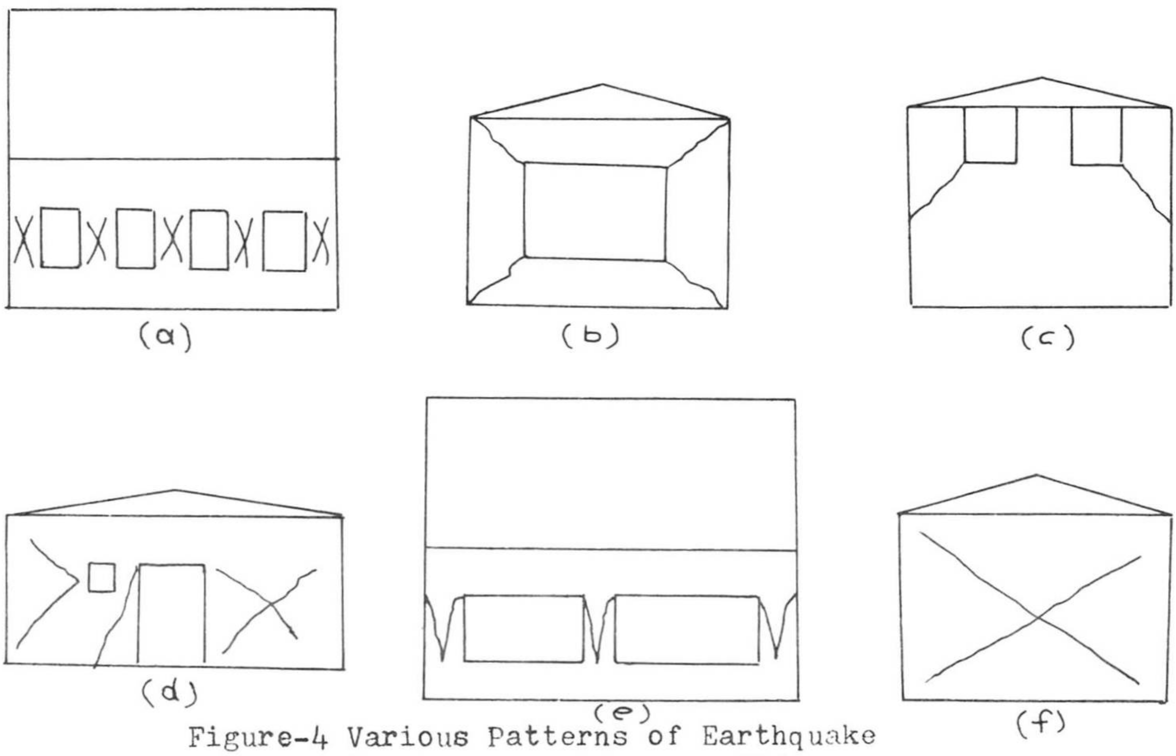

Figure-4 Various Patterns of Earthquake Cracking in Masonry Buildings of the cracks will be influenced by the height to width ratio of the wall and location and size of the window and door openings. Figure-4.

This initial diagonal cracking of the wall, if the earthquake continues at a high intensity, will result in the decrease of vertical load carrying capacity of the wall and vertical cracks due to vertical loads will begin to appear in addition to the existing cracks due to diagonal tensile stresses in the wall. This process will eventually lead to the complete collapse of the wall. Figures-5,6,7 show this kind of heavily damaged brick masonry walls

Figure-5 Typical Failure of Brick Masonry

Figure-6 Typical Failure of Brick Masonry

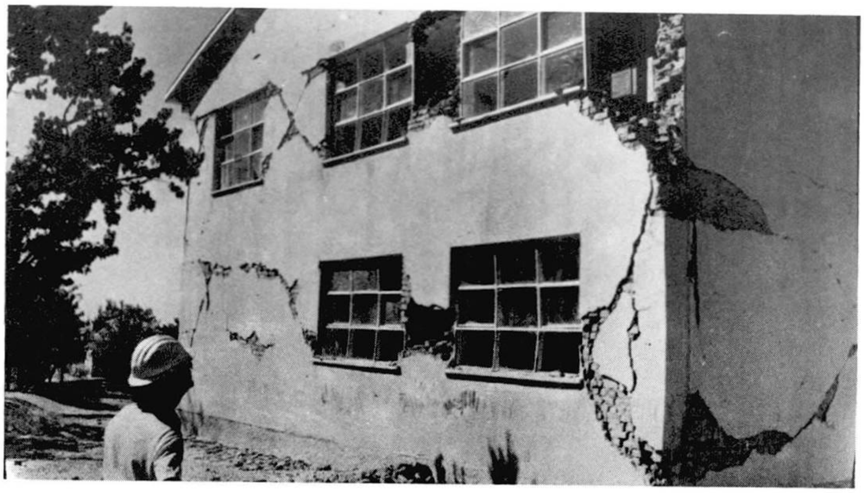

Figure-7 Typical Failure of Brick Masonry

4. COMPARISON OF CODE PROVISIONS AND EARTHQUAKE BEHAVIOUR

The earthquake resistant design code provisions for brick masonry buildings generally is composed of certain rules for wall openings with respect to the total external walls, placement of lintel beams, the foundations and certain restrictions on the height of brick masonry buildings. Although earlier editions of the earthquake resistant design code of Turkey had also stressed on the importance of wall opening ratios, there had been no systematic emprical research on the effectiveness of wall opening ratio as an earthquake resistant feature for brick masonry buildings, using data from buildings damaged in actual earthquakes.

After November 24, 1976 Çaldıran and March 25, 1977 Palu earthquakes [8] [9] a number of masonry buildings were investigated and the effectiveness of wall opening ratio as an earthquake resistant design feature for brick masonry buildings were tested. The results of the analysis of five buildings are given in reference [10]. Here, only a brief summary of the result will be presented.

The window and door opening ratios of two principal axes of these buildings, along with the wall length (cm)/floor area (m2) ratio is as given in Table-8. On this table x-axis of the building corresponds to the lon-

TABLE-8

Opening and Wall Ratios of Earthquake Damaged Buildings

Building	External Wall Opening Ratio %	Wall/Floor Area Ratio cm/m2	Damage	Remarks
Çaldıran Staff Housing				
x-axis	41-37	26	No damage	I = IX
y-axis	0	31	No damage	(Figure-8)
Muradiye Staff Housing				
x-axis	54-53	17	Heavy damage	I = VII
y-axis	0	26	No damage	(Figure-9)
Muradiye Junior High School				
x-axis	15	20	Slight damage	I = VII
y-axis	53-37	12	Heavy damage	(Figure-10)
Palu Monopoly Office				
x-axis	42-35	21	Damage	
y-axis	0-16	24	No damage	I = VI
Palu Local Administration Building				
x-axis	55	25	Heavy Damage	I = VI
y-axis	29	30	No damage	(Figure-11)

Figure-8 Çaldıran Staff Housing

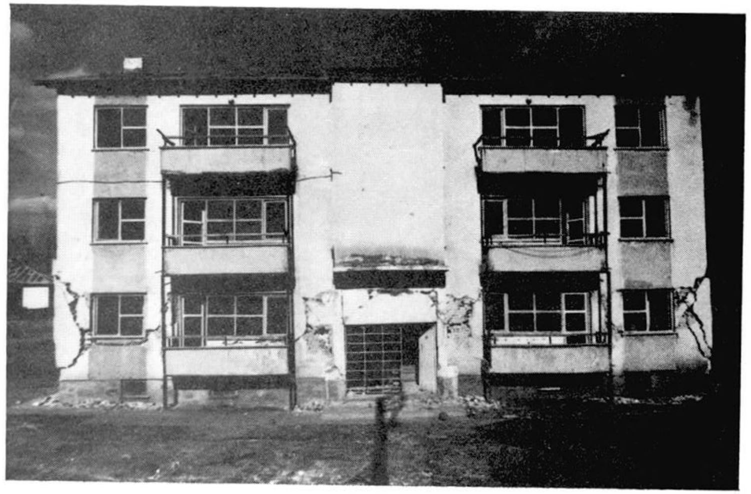

Figure-9 Muradiye Staff Housing

Figure-10 Muradiye Junior High School

Figure-11 Palu Local Administration Building ger plan dimension of the building, while y-axis coincides with the shorter dimension of the building. Wall length/floor area ratio is currently advocated as an earthquake resistant design feature in Japan[11].

From these observed damage and properties of buildings, it can be seen that an external wall opening ratio more than 40 % causes damage in the walls.

On the other hand, buildings with wall/floor ratio less than 25 cm/m2 are also damaged heavily. This points out the fact that large sized rooms in masonry houses could lead to damage in case of earthquakes.

The number of observed cases is few thus the impressions should be taken with caution. However, they could still be taken as useful parameters in the design of earthquake resistant brick masonry houses.

5. SUGGESTIONS AND CONCLUSION

As it is known brick masonry is not a construction system desired and used for earthquake resistant construction however certain economical conditions enforce its usage, specially for one to two stories high houses which are being built in considerable numbers in Turkey. The brittle nature of the brick masonry walls and rapid cracking of walls during earthquakes necessitates certain counter measures. Since the rapid shear cracking weakens the vertical load carrying capacity of walls thus increasing the possibility of collapse, measures are required to prevent extensive cracking and crack propagation. This is usually achieved by providing lintel beams of more ductile nature and of higher strength such as reinforced concrete at certain levels of the walls.

Another critical condition occurs at the corners of external walls, under the action of earthquake forces, the cross walls tend to displace each other outwards at the corners. This could be prevented by providing vertical reinforced concrete columns at the corners. This detail is widely followed and commonly executed in Turkey even in one story high houses. Presence of reinforced concrete slabs at floor and roof levels will prevent outward displacement of corners by exerting a vertical restraint.

External wall opening and wall length/floor area ratios seem to be two important amprical parameters which could be used to improve earthquake behaviour of brick masonry structures. Research work, both analytical and experimental, could be carried out to provide more rational basis to these parameters.

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CONSIDERATIONS CONCERNING EARTHQUAKE RESPONSE ANALYSIS OF ROCKFILL DAMS

RÉMARQUES SUR L'ANALYSE DE LA RÉPONSE SISMIQUE DES BARRAGES EN ENROCHEMENTS

BETRACHTUNGEN ÜBER EINER UNTERSUCHUNG DER ERDBEBENSEINFLUSS AUF DIE STEINDÄMME

Radu Priscu*, Dan Stematiu**, Lucian Ilie**

SUMMARY-RESUMÉ-ZUSAMMENFASSUNG

The earthquake response of rockfill dams is strongly influenced by different assumptions concerning fill behaviour and excitation mechanism. Starting from the results obtained for some Romanian dams, the paper emphasizes the influence of material dynamic properties and nonsynchronous excitation along the dams foundation. Also, it is shown the earthquake behaviour of a large rockfill dam during the Vrancea earthquake which hit Romania on 4-th of March 1977.

La détermination de la réponse au séisme des barrages en enrochements est en étroit rapport avec les hypothèses concernant le comportement des enrochements et le mode d'introduction de l'excitation sismique. À partir des résultats obtenus pour quelques barrages roumains on souligne l'influence des proprietes dynamiques des matériaux et du nonsynchronisme de l'excitation au long de l'emprise. On y expose également le comportement d'un grand barrage en Roumanie au récent tremblement de terre Vrancea, 4 mars, 1977.

Die Bestimmung des Einflusses eines Erdbebens auf die Steindämme ist direkt von einigen Voraussetzungen eingegriffen, betreffens des Steinschüttungskörpers und der Einführung der Erdbebenserregung. Ausgehend von den Ergebnissen bei den rumänischen Talsperren, in der Arbeit wird der Einfluss der dynamischen Eigenschaften der Werkstoffe und der Non-Synchronismus der Erregung längs der Grundrechtsbreite hervorgehoben. Gleichfalls, wird das Verhalten eines grossen Steindammes während des Erdbebens in Rumänien, von 4 März 1977 gezeigt.

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CONSIDERATIONS CONCERNING EARTHQUAKE RESPONSE ANALYSIS OF ROCKFILL DAMS

R.Prisco, D.Stematiu, L.Ilie

In order to evaluate the rockfill dams stability at seismic loads it is necessary to know the value and the distribution of the accelerations induced into their bodies. The analysis of the earthquake response provides both kinds of data, yet the values obtained depend on the hypotheses concerning the behaviour characteristics of the materials as well as on the seismic excitation mechanism.

Current dynamic analyses accept the linear elastic behaviour of the fill materials. The earthquake analyses comprising the elasto - plastic [1] or viscous - elastic [2] behaviour of the rockfill body are yet at a preliminary stage as plastic and viscous deformation characteristics under dynamic loads are not entirely known. With an earthquake analysis in the elastic range the dam response is directly dependent on the Young's modulus and Poisson's coefficient. The deformation process, supposedly elastic, is characterized by dynamic deformation moduli dependent on the initial stress state and on the amplitude of the dynamic stresses [3] . Their values obtained by laboratory and field tests may be 2 to 20 times as big as that of static deformation moduli. As a result the dynamic response is greatly different if static or dynamic deformation moduli are taken into account.

The seismic load is usually introduced under the form of the inertia forces, the excitation accelerations being the same throughout the dam body and equal to ground acceleration. Under this form the excitation is called "synchronous" and confirms the hypothesis that the seismic wave velocity within the foundation ground and the dam body is infinite. As a matter of fact the seismic wave velocity has a finite value of 800 to 4000 m/s, according to the nature of the foundation ground. The earthquake displacements and accelerations will be different alongside the dam base, with a phase lag resulting from the finite propagating time of the wave front. Considering the dam body receives different excitation, according to the distribution of the accelerations along the foundation base, the excitation mechanism is called "nonsynchronous". In the case of rockfill dams the dam-foundation contact exposed to the earthquake waves has high values, up to hundreds of meters, which emphasizes the part played by the nonsynchronous character of the excitation in evaluating the seismic response.

As a result of some dynamic analyses carried out for some Romanian dams, both the effects introduced by the dynamic properties of the dam fill and the above mentioned excitation mechanism are pointed out bellow.

1. THE INFLUENCE OF THE DEFORMATION MODULUS ON THE EARTHQUAKE RESPONSE

During the deformation process of the rockfill body elastic movements of small value and reversible character, as well as remanent movements of an important value and with plastic and creep character do occur [4] . That is why in evaluating the rockfill dam displacements several deformation moduli ocuur each of them characterizing a certain type of loading.

For dam movements during construction sequences and reservoir impounding the analysis implies the consideration of global elastic moduli (E_s) when it is beeing carried out in the elastic range and an instantaneous loading is accepted; in the elasto-plastic range the analysis is carried out using zoned elastic moduli($E_{s,z}$) and construction sequences and reservoir impounding are properly simulated.

In the case of seismic loads, the perturbing effect of the induced accelerations and the initial stress state overlap. The value of connection forces between rockfill blocks depends on the value of the contact forces and, implicitly, on the stresses which actuating on the rock contacts. Provides the seismic action is moderate, the connections among the rock blocks are not affected and the rockfill behaves as a continuous and elastic body. The deformation modulus, called dynamic (E_d), has much higher values in this case.

Fig.1 shows the dependency of the dynamic and static moduli ratio (E_d/E_s) with the principal stresses in the dam body as it came out from the experiments carried out by HAYASHI, FUZIWARA and KOMEDA[3].

Thus in evaluating the earthquake response some serious errors occur and are due not only to the acceptation of the elastic behaviour of the dam body but mainly to the use of static deformation moduli, global or zoned, instead of dynamic ones. With a view to illustrating the deformation modulus influence on the dam response two Romanian dams have been analyzed in various hypothesis: Lotru (H=124 m, clay core rockfill) and Bolboci (H=51 m, reinforced concrete face rockfill).

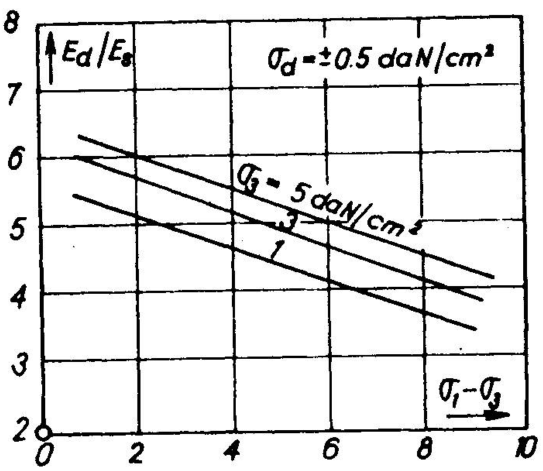

Fig. 1 The correlation between E_d/E_s ratio and the principal stresses in the dam body (after Hayashi, Fuziwara and Komeda).

Global elastic, zoned elastic and dynamic deformation moduli have been succesively considered. The zoned elastic moduli have been determined by simulating the construction sequences and the reservoir filling. For the deformation moduli the hyperbolic relations suggested by Duncan [5] have been admitted. The value of the tangent modulus depends on the internal friction angle and the stress level. Nonlinear stress-strain behaviour was approximated in the finite element analysis by assigning modulus values to each element consistent with the values of stress in that element. The analysis are performed using a step-by-step or incremental analysis procedure. During each step or increment the relationship between stress and strain for each element is assumed to be linear. Thus, at the end of the analysis the displacements,stress-es and deformation moduli in the dam body have been obtained. The dynamic moduli have been determined with approximation in accordance with the static noduli previously determined, using the diagrams shown in Fig.1. The contours of the deformation moduli in dams cross sections for the three hypothesis considered may be followed in figures 3 and 4.

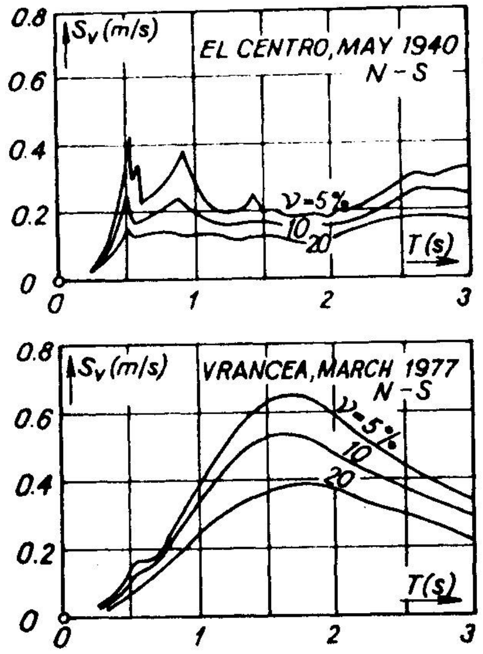

Fig. 2 The velocity spectra of the earthquakes El Centro and Vrancea, scaled to 0,1 g.

The earthquake behaviour has been determined by response spectrum analyses, for two earthquakes considered to be characteristics : EL CENTRO, May 1940 and Vrancea, March, 1977. The corresponding velocity spectra, scaled at the maximum acceleration of 0,1 g are presented in figure 2. One should notice that while the El Centro earthquake spectrum shows peaks within the range of quasirigid structures (with the first vibration periods of 0.3...0.5s), the Vrancea earthquake, that has recently occured in Romania, shows peaks within the period range of 1.5s.

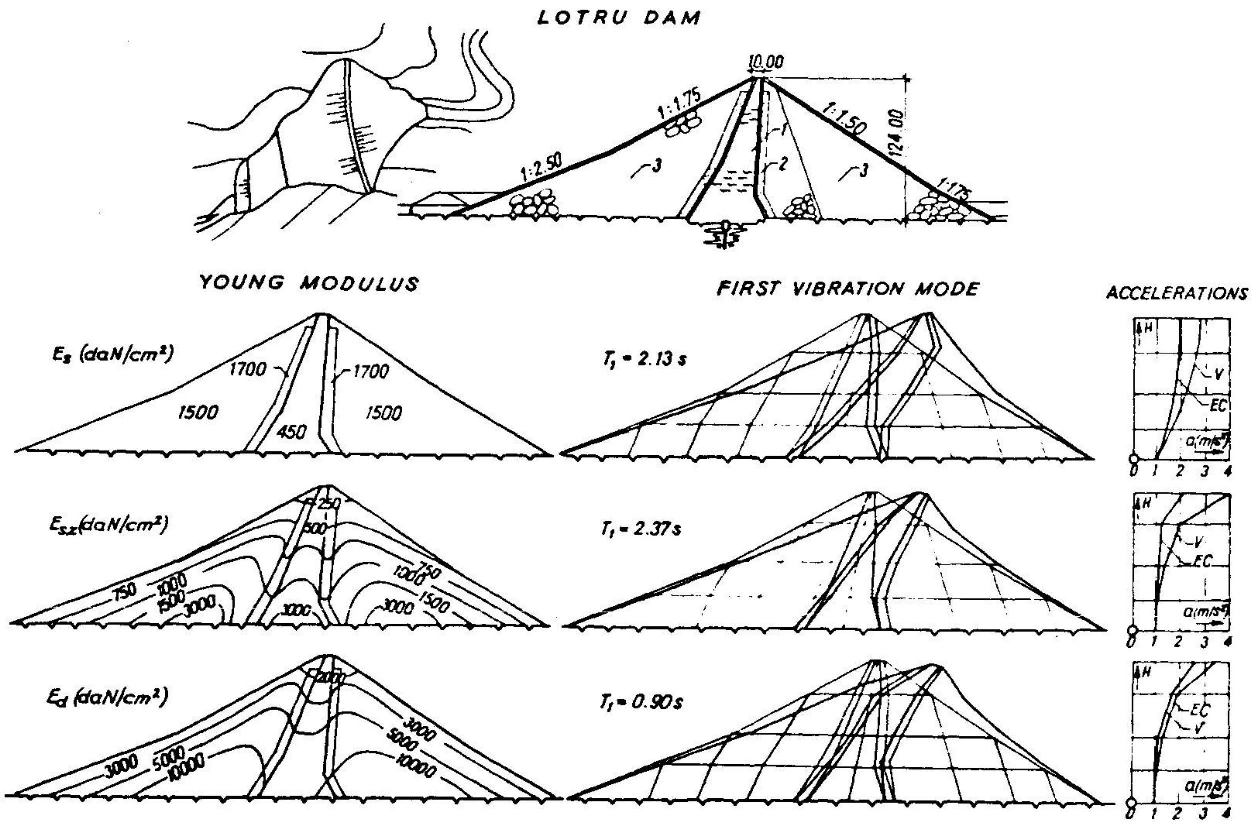

Fig. 3 Earthquake response of the Lotru dam: 1-clay core, 2-tranzition zone; 3-rockfill.

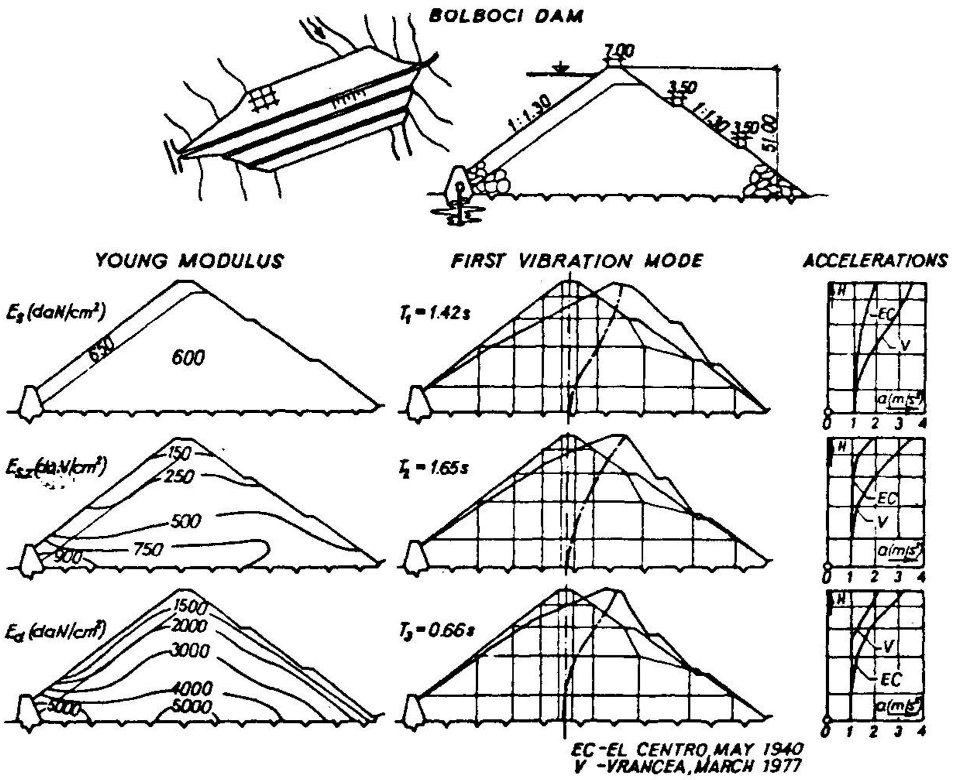

Fig. 4 Earthquake response of the Bolboci dam.

The analysis results are shown in figures 3 and 4. For both dams not only an important change of the vibration fundamental period is obvious but also the shape of the first vibration mode according to the deformation modulus used. In the case of dynamic moduli the fundamental period is about 2.5 times less and the displacements corresponding to the first mode shape are more reduced in the lower half of the dam height, presenting increases twoards the crest. As a result, the distributions of the induced accelerations into the dam body by the two considered eartquakes,for a damping of 10% of the critical one, is different in its turn. While in the case of static moduli, the Vrancea earthquake,a slower one, leads to higher values of the response accelerations,in the case of the dynamic moduli the situation is reverse. At the same time in the case of dynamic moduli, the distribution over the dam height shows important increases in the upper third of the dam only, as it has been noticed according to the field records of the earthquake response of some rockfill dams [6] .

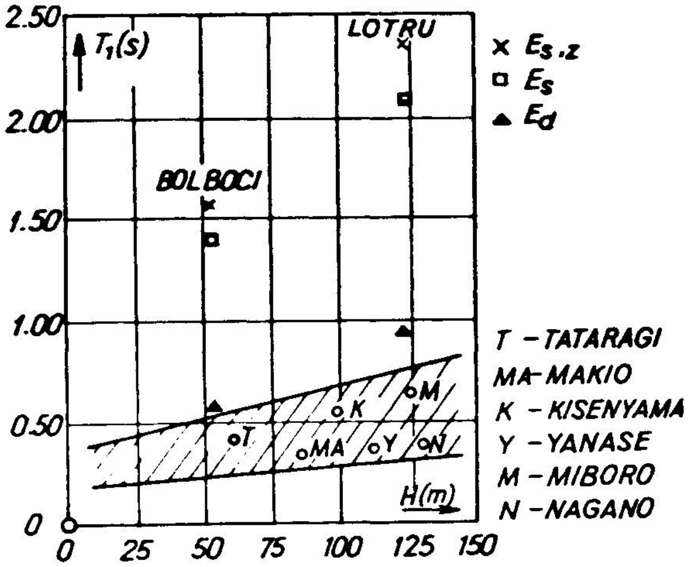

Fig. 5 The first vibration periods determined for Lotru and Bolboci dams comparatively with those measured for several Japanese rockfill dams (after Okamoto).

According to the above presented results as well as to other similar analyses it is evident that the use of the static moduli, zoned or not, the dynamic analysis leads both to erroneous evaluation of the response and to a wrong choice of the type of earthquake (slower or faster) acting upon the rockfill dams which are to be studied.

2. THE EFFECT OF THE NONSYNCHRONOUS CHARACTER OF EXCITATION

As it has been shown in the case of nonsynchronous exitation the excitation accelerations in the dam body are different from one point to onather according to the seismic accelerations of the dam base. By neglecting the dam-foundation interraction the seismic accelerations along the dam base are dependent on the position of the seismic wave. The motion equations have, then, the formula [8] :

$$[M]\{\ddot{\delta}\} + [C]\{\dot{\delta}\} + [K]\{\delta\} = - [M]\,[R]\,\{\ddot{u}\} \qquad (1)$$

in which $[M]$, $[C]$ and $[K]$ are the mass, damping and stiffness matrices, $\{\delta\}$, $\{\dot{\delta}\}$ and $\{\ddot{\delta}\}$ are the displacement velocity and acceleration vectors along the dynamic degrees of freedom, and:

$[R]$ is a matrix of the influence coefficients;

$\{\ddot{u}\}$ - the vector of the seismic accelerations in the dam base.

The term R_{ij} of the matrix $[R]$ stands for the displacement of the (i) - degree of freedom, when a unitary displacement of the (j)-foundation point takes place. The matrix $[R]$ is a rectangular one with the number of rows equal to the dynamic degrees of freedom and the number of columns equal to the number of the foundation degrees of freedom.

The solution of the equations of motion, Eqs (1), can be obtained by direct integration. The integration is carried out by the step by step procedure, modifying the $\{\ddot{u}\}$ vector at each time step according to the position and the magnitude of the seismic wave.

The influence of the nonsynchronous character of the excitation on the earthquake response of the Lotru dam, previously presented, may be followed in figure 6. The same El Centro and Vrancea earthquakes scaled up to a maximum acceleration of 0,1g have been considered; their accelerograms are figured in Fig.6 as accelerations of the point E of the foundation. The damping was this time considered to be 20%. The deformation moduli are the dynamic ones and the seismic wave velocity in the foundation ground is of 1200 m/s. The induced accelerations into the dam body are represented as accelerograms over a period of 4.5s,herein considered to be characteristic. The full line marks the accelerations obtained in the synchronous excitation hypothesis and the dotted line marks the accelerations obtained in the nonsynchronous excitation hypothesis.

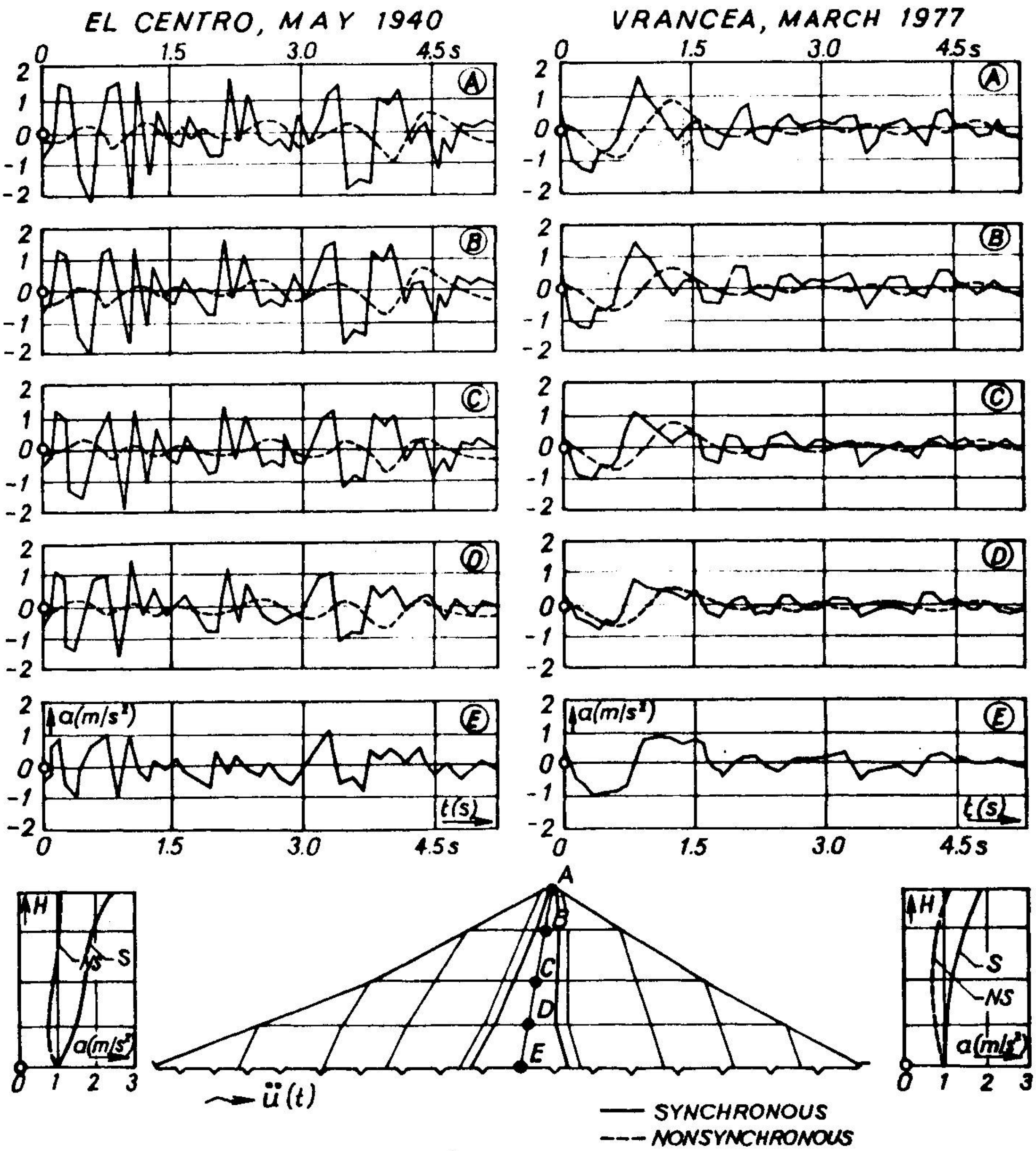

Fig. 6 The influence of the nonsynchronous character of the excitation on the earthquake response of Lotru dam.

Mention should be made of the much lower values of the accelerations obtained in the nonsynchronous excitation hypothesis and the tendency to damp down the response within the range of high frequencies. The distribution of the accelerations over the dam height at its peak shows a fairly constant value in the case of nonsynchronous excitation and a quite important amplification towards the crest in the case of synchronous excitation. There is no evidence of the increase of the response in the upper third of the dam which is obvious in the measured earthquake responses for some existing dams [6] . The discrepancy is due to the fact that actually the behaviour of the embankment towards its crest is no longer elastic.

Acording to the results presented as well as to some other similar analyses it follows that in the case of embankment dams with bread foundation surfaces the effect of the nonsynchronous character of the excitation is important, leading to a reduction of the response and thus emphasizing the capacity of such dams to withstand earthquakes.

3. THE BEHAVIOUR OF THE LOTRU DAM DURING THE VRANCEA EARTHQUAKE, MARCH, 1977.

Of the two dams under consideration, Lotru and Bolboci, it is only the former to have faced the recent earthquake in Romania , the latter being still at the design stage.

The magnitude of 7.2 places the Vrancea earthquake, March 4-th 1977, among the strongest earthquakes to have occured in Europe in the last few decades. The earthquake focus was located at about 100 km depth in Vrancea region, an area with a well known high earthquake activity. From the point of view of the dam shaking it is interesting to point out that on March 4-th 1977 as well as in the previous earthquakes the maximum intensities were felt towards South-South West of the epicenter. In Fig.7 a sketch of the most affected zone within the Romanian territory is presented; it also shows that Lotru dam, located at about 200 km away from the epicenter, was outside the impact zone (the earthquake intensity at the site was estimated to be VI on the MM scale).

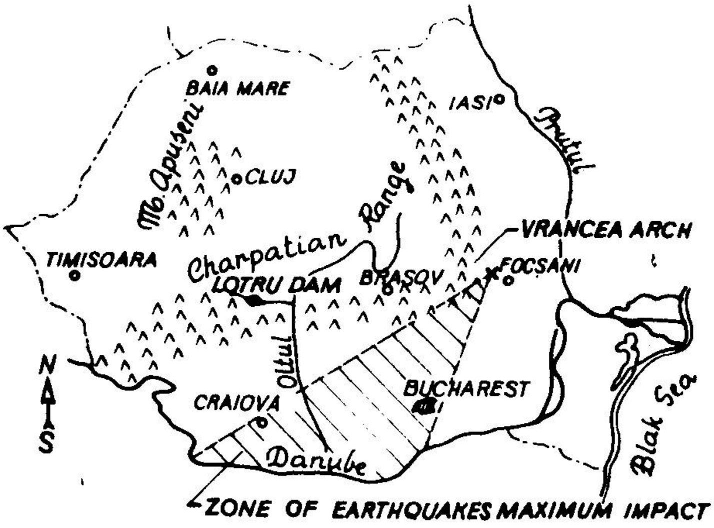

Fig. 7 The Lotru dam location as to the maximum impact zone of the Vrancea earthquake, March 1977.

Unfortunately this dam as well as other important dams in Romania were provided with seismic recording equipment for induced earthquakes only. During the earthquake the seismic devices ran out of the previously set measuring range so that no recordings of the precise data about the excitation and the dam response could be obtained. A engineering team checked out the state of the dam after the earthquake and established the dam behaved normally, that there was no evidence of deformations, unreversible displacements, cracks or increase of seepage. Besides this dam had been designed to withstand an earthquake with a seismic acceleration of 0.05 g which was not surpassed by the 4-th of March 1977 earthquake. The analyses presented in the paper point out the fact that, by the effect of the nonsyncronous character of the excitation the induced accelerations are not higher than the base ones, and that the dam stability is not affected, even for earthquake accelerations up to 0,1g.

4. CONCLUDING REMARKS

- The above-mentoned analysis of the rockfill dam earthquake response, refers to the effect of the deformation moduli and the excitation mechanism upon the induced accelerations.

- The evaluation of the response in the hypothesis of the elastic behaviour of the rock-fill body can lead to results similar to those recorded in the field if the deformation dynamic moduli are considered. Systematically, the induced accelerations are fairly constant along the two thirds of the dam height and amplified on the upper third.

● The analyses carried out for two earthquakes with different characteristics show an important influence of the site spectrum characteristics upon the dam earthquake response.

● The introduction of the non-synchronous character of the excitation leads to a lowering of the induced accelerations, thus pointing out the strength resources of the embankment dams subject to earthquakes.

● The rock-fill dam mathematical models of the dynamic analysis lead to correct evaluations of the earthquake response inasmuch as laboratory and field tests can provide the dynamic characteristics of the foundation ground behaviour as well as of the rock-fill body.

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SURFACE EFFECTS ON SEISMIC WAVES AT MOUNTAIN SITES

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Summary

An impulse motion, propagating orthogonally towards a free surface, within a continuum homogeneous material, doubles its amplitude in the proximity of the surface. An earthquake excitation propagating upwards, vertically, in the same continuum, is affected by an amplification near the surface, up to the same order, if the surface is horizontal.

In presence of valleys or mountains, a variety of reflections and refractions occurs, and in general the intensity of the shaking at the surface is further more amplified.

To evidence such geometric effects from those due to material discontinuities, an investigation is carried out on two sites of Friuli - Northern Italy earthquake of '76'. Comparison with the vertical profile of maximum acceleration in monodimensional propagation put in evidence the amplification likely to be ascribed to the geometric effect.

The relevancy of a deep embedment as a measure to reduce the seismic shaking is suggested.

Resumé

Une impulsion qui se propage orthogonalement vers une superficie libre, à l' intérieur d' un continu homogène, redouble son amplitude dans les environs de la superficie.

Un tremblement de terre qui se propage verticalement en haut, dans le même continu, ressent une amplification du même ordre dans le environs de la superficie si celle-ci est horizontale.

En présence de vallées ou montagnes, l' intensité de la secouse à la superficie est ultérieurement amplifiée parce qu' il y a un certain nombre de réflexions et de réfractions des ondes incidentes sur la superficie.

Pour mettre en évidence ces effets géometriques entre ceux dûs à la discontinuité du Friuli, la région la plus frappée par le tremblement de terre du Nord - Italie du 1976.

Des comparaisons avec le profil des accélérations maximales pour le cas monodimensional mettent en évidence l' amplification due aux effets géometriques.

En outre, on suggère l' importance de profondes fondations comme mesure pour réduire les effets du sisme.

1. INTRODUCTION

In wave propagation theory a surface effect exists, according to which, in the general case , an impulse motion, propagating orthogonally towards a free surface, doubles its amplitude in the proximity of the surface itself. Here, in fact a complete reflection occurs, and the impinging and the reflected waves sum up. Such "surface effect" can be felt up to some depth, near the surface, depending on the shape and the duration of the impulse.

As to a seismic harmonic wave vertically propagating in a horizontally stratified soil, the amplification may be greater than two, and develops in the upper L/4 stratum, where L is the wavelength of the wave type under examination. For instance, in a sand with a void index 0. 8 , a confining pressure of 0. 5 Kg/cm^2, and under a shear wave motion of predominant frequency 2 cps, this amplification applies in the last 20 m. In a homogeneous rock with shear wave velocity 2000 m/sec, with a predominant frequency 5cps, this applies in the upper 100 m.

Effects of this nature are further complicated, and even hidden by the soil's peculiar stratigraphy and by the presence of inclined surfaces, hills or valleys.[8,10,15,19] In the latter case, assuming again vertically propagating seismic waves, reflection and refraction occur along the sides of the surface discontinuity, and the resulting combination gives rise, in general, to an amplification relative to the bedrock motion and even to the free field motion at the nearby flat country. This was in theory observed for valleys by [3,16,22]. But as to authors' knowledge, no precise experimental evidence has been collected and analysed so far, in particular for hilly sites, where the amplification is likely to be greater than for valleys.

During the northern Italy earthquake of '76', a mountain region - Friuli- was affected, and the damage distribution could confirm the above rule. See for instance the large damage concentration at Buia and Gemona, two hilly sites, relative to the less intense shaking in the nearby plane. But this could also be described to the different soil characteristics under the two sites and the plane. [4, 21]

The same problem of filtering the material effects from the geometric effects arose for a proper understanding of the earthquake records, after that a network of strong motion recorders was installed in the region.
In this research, by applying a suitable input motion at the base of the hill through a numerical model - see fig. 1 - , the authors tried to derive a map of amplification factors for all the surface points of the hill - see figs. 2,3,4 . The vertical profile of acceleration at a single point, compared with the profile obtained for horizontally layered soils of the same vertical stratigraphy - i. e. obtained in a monodimensional propagation problem - has allowed for separating geometric from material effects.

The main conclusions so far collected are as follows :

1. Along the hill surface the maximum ground acceleration may be considerably greater than ground acceleration in a monodimensional propagation problem. It is, besides, always greater than the acceleration at the surface of the nearby plane.
2. The acceleration map seems subject to great dispersion, while the velocity map may be more appropriate for zoning purposes.

3. As to the input motion for construction design purposes, the need of a proper choice of foundation level is exasperated in regard to flat surface constructions. For instance, if L is the wavelength of top soil deposit, an embedment of L/10 may reduce in several cases the earthquake effects by a factor of two.

2. REVIEW OF THE NUMERICAL MODELS FOR SOIL AMPLIFICATION ANALYSIS IN BI - TRI DIMENSIONAL PROBLEMS

2.1 Numerical Models

A complete analysis of a soil deposit subjected to earthquake effects should take into account ground motions which vary from point to point and propagate in any direction in the soil deposit.[12] In the great majority of cases the information about the excitation is provided in the form of accelerograms recorded at one or several locations and this is not sufficient to define completely the characteristics of the incoming seismic waves. For instance, it is not possible to determine the propagation direction of the waves. In practice, analytical techniques are confined to the hypothesis of vertical propagation of shear waves from a given horizontal layer at some depth in the soil deposit.

The past years have seen a considerable development of finite element techniques to solve dynamic problems both for soil deposit and for soil-structure interaction analysis, taking advantage of the flexibility and ease with which the finite element method can be adapted to different geometries, which are usually met in soil mechanic problems [13]. But, the use of finite elements to simulate an infinite space calls for suitable boundary conditions to avoid spurious reflections from the model contour [7]. Three techniques, of increasing sophistication, have been proposed up to now - see Fig. 5.

Models shown in figure 5a and 5b have both rigid base and respectively rigid or free lateral boundaries. With these models the motion is applied simultaneously on the entire rigid boundary but the absorption of the waves impinging on the boundary is not allowed. To have satisfactory results, avoiding spurious reflections, it is necessary to resort to very large meshes that often reach the limits of computer storage availability and execution time. It is possible to overcome in part this drawback with models as in figure 5c, having lateral transmitting boundaries and rigid bedrock, [14].

Some standard tests have been developed by the writers, considering a horizontally stratified soils, i.e., a rigorously monodimensional wave propagation problem. The theoretical solution of the problem was also available [20]. Figure 6 shows the comparison between the latter solution and that of a model as in figure 5c.

The agreement can be considered satisfactory; in fact even the lateral acceleration profiles are in good agreement with the theoretical solution, confirming the correct absorption of energy due to the transmitting boundaries. Little differences are visible close to the ground surface where the propagation mechanism is of the Rayleigh type and relevant lateral absorption is not properly simulated by the model.

The limit of assuming a horizontal rigid bedrock, inherent in this technique, implies the assumption that the seismic excitation at some depth consists of vertically propagating shear waves.

A more sophisticated model, which avoids this assumption, has been presented by Ayala et al. [1] - see figure 5d.

In this model "active" boundaries allow free transmission of waves - both for outgoing and incoming waves - as would occur if the discrete domain were continuous, thus providing a consistent representation of an infinite space. Besides, an earthquake source simulation has been included, to generate the seismic waves at the boundary of the mesh. [18]

A few results up to now confirm the relevance of such "active" boundaries in the absolute motion of a surface point. Limited influence, on the other hand, is shown as to the amplification between the motions of two different layers.

The drawback of this model for soil amplification analysis is that ..... it is too advanced; as a matter of fact the input data, in general, are not available on an experimental basis.

2. 2 Checks of numerical models

In recent times checks of these analyses were possible when records have been collected simultaneously in nearby sites where layering and material properties of the entire zone were known. Checks of this kind have been presented since 1971, [11], based on San Fernando earthquake records. Two indipendent methods were presented as giving self-consistent spectral estimates at three sites in agreement with records.

As proof of this agreement a figure was exhibited where the results were reported in a logarithmic scale. It is interesting to note now that the same comparison, reported in a linear scale - see figure 7 - instead of a logarithmic one, should be ascribed as unsatisfactory, on the base of a qualitative judgement.

Only recently, using Shake [20] or Flush [14] techniques ,i.e., models of the kind of fig.5c more satisfactory results have been presented [17] [23] .

3. THE SURFACE EFFECT

The propagation of elastic disturbances in layered media is considered, each layer being continuous, isotropic, homogeneous and linear-elastic. In a single layer, the equilibrium for small strain is expressed by the classical Navier equations :

$$\rho \frac{\partial^2 u_i}{\partial t^2} = (\lambda+\mu)\frac{\partial \theta}{\partial x_i} + \mu \Delta^2 u_i + \rho F_i \qquad i = 1,2,3 \qquad (1)$$

where :

$u_1\ u_2\ u_3$ are the displacements in a space-fixed rectangular coordinate system x_1, x_2, x_3,

t is time

$\theta = \text{div}\,\bar{u} = \varepsilon_1+\varepsilon_2+\varepsilon_3$ cubical dilatation;

F_1, F_2, F_3. are the unit volume forces in the directions of the coordinates, which will be considered zero in our problems ;

λ and μ are the Lamè constants ;

ρ is density.

In the numerical models considered in the present research, the effect of internal friction is introduced into the equations of motion by replacing an elastic constant, such as μ by $\mu+\mu'\frac{\partial}{\partial t}$ in the same equations.This is equivalent to stating that stress is a linear function of both the strain and the time rate of change of strain. For simple harmonic motion such effect leads to replacing μ by the complex rigidity $\mu\, e^{2i\beta}$ where β is the so called fraction of the critical damping for the element.

Notice that this representation of internal friction is not backed by adequate experiences, in particular in the range of high frequencies where the dissipation is greater. In fact, if a modal analysis is applied to the entire system, the normal modes of vibration result are damped by a damping factor proportional to the frequency of the mode, precisely:

$$v_i = \frac{\beta}{2}\,\omega_i$$

where v_i is the damping relative to the critical one for the i^{th} mode of vibration and ω_i is the ith frequency. So the high frequency content of the response might be underestimated in such a model.

In the same subject notice that if the Poisson ratio is a real number, the above assumption implies that the P and S waves have the same attenuation factor.

Under these limits the picture of the maximum acceleration during one Friuli ground shaking, at two different sites have been derived numerically. The input motion was applied at the base of the hill, i.e., at the free surface of the flat country. It was based on two records respectively, collected in the neighbourhood.

The computer code applies this motion simultaneously to all points at free surface which are far enough from the hill to be undisturbed by the hill's presence, [14]. Therefore monodimensional propagation of seismic waves is assumed for such points.

In order to understand the "surface effect" in the shown analyses let us first consider a homogeneous non dissipative, elastic soil. The following simple theorem will be proved in the Appendix.

"If the amplitude of a harmonic displacement component, at some depth , is A, then the amplitude of the same component at the surface is

$$\frac{2\cos\varphi}{1+\cos 2\varphi}\,A\,,$$ where :

V is the wave velocity for the wave type under examination;

A is an arbitrary fixed amplitude;

ω is the harmonic motion frequency

H is the depth.

$$\varphi = \frac{H\,\omega}{V}$$

The above results apply both to compressional and to shear waves.

Figure 8 shows the amplitude transfer function under examination. As evident, this is always greater than one, being one in practice only for :

$$\varphi = \frac{H\omega}{V} \leq \frac{\pi}{20} \qquad \text{or} \qquad H \leq \frac{L}{20}$$

where L is the wavelength for the wave under examination. Therefore, the presence of the surface effect is not a mere question of rigidity of the upper layer, but mainly a matter of the ratio between the depth of the soil deposit and the wavelength.

The above-mentioned result can be worked out further on in terms of earthquake intensity

Let $S_0(\omega)$ be the power spectral density for acceleration at the depth and $S_H(\omega)$ be the analogous quantity at free surface.

Let moreover the earthquake intensity be evaluated as

$$I_0 = \left[\int_0^{\infty} S_0(\omega)\, d\omega \right]^{\frac{1}{2}}$$

and analogously at the free surface.

To derive the ratio between I_0 and I_H, a hypothesis need be assumed as to the frequency distribution of the energy of the earthquake, i.e. , on the function $S_0(\omega)$. If for instance, as in figure 9, S_0 is of a parabolic type, then :

$$I_H \simeq 2\, I_0$$

The importance of this solution is related to the experiences so far collected on the simultaneous registration of the earthquake motion at surface and underground, see for instance [2] . In particular in the paper [6] , where all the available experiences have been reviewed, no singular case is reported at variance with the above rules. On the average, the above-mentioned ratio appears to be two.

As it was previously mentioned such effect can be further on amplified by the presence of non horizontal deposit : see figure 10, which confirms the observations already presented in Fig. 2, for the Buia hill.

Aknowledgment. Research developed under the financial support of ENEL and CNR GEODINAMICA.

APPENDIX

Vertical propagation of harmonic shear waves through th system shown in fig. 11 will cause only horizontal displacements

$u = u(x,t)$

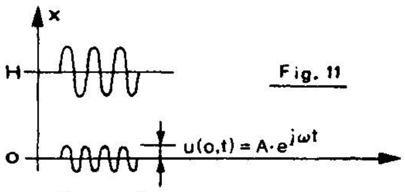

Fig. 11

which must satisfy the wave equation [5, 9] :

$$\rho \frac{\partial^2 u}{\partial t^2} = G \frac{\partial^2 u}{\partial x^2} \qquad (1)$$

Let assume an harmonic motion at the layer $x = o$, $u(o,t) = A \cdot e^{j\omega t}$
General solution of Eq. 1 is :

$$u = B_1\, e^{j\omega(t-\frac{x}{v})} + B_2\, e^{j\omega(t+\frac{x}{v})} \qquad (2)$$

The coefficients B_1 and B_2 can be related by imposing the boundary condition for $x = H$:

$$\gamma = \left[\left(\frac{\partial u}{\partial x}\right)\right]_{x=H} = 0 \quad (3)$$ where γ is shear strain

Defining $\varphi = \frac{\omega H}{V}$ where V is the wave velocity it is immediate to obtain as a consequence of equation (3) :

$$B_2 = B_1\, e^{-2j\varphi}$$

At $x = 0$ the displacement function $u(o,t)$ is according to Eq. 2

$$u(o,t) = (B_1 + B_2)\, e^{j\omega t}$$

and, by the definition :

$$A = B_1 + B_2 = B_1(1 + e^{-2j\varphi})$$

Relating the coefficient B_2 with A the transfer frequency function takes the expression :

$$\frac{u(H,t)}{u(o,t)} = \frac{2\, e^{-j\varphi}}{1 + e^{-2j\varphi}}$$

and finally with algebrical operations results :

$$\left|\frac{u(H,t)}{u(o,t)}\right| = \frac{2\cos\varphi}{1 + \cos 2\varphi}$$

which represents the frequency transfer function versus φ, in the case of homogeneous non dissipative linear elastic half space.

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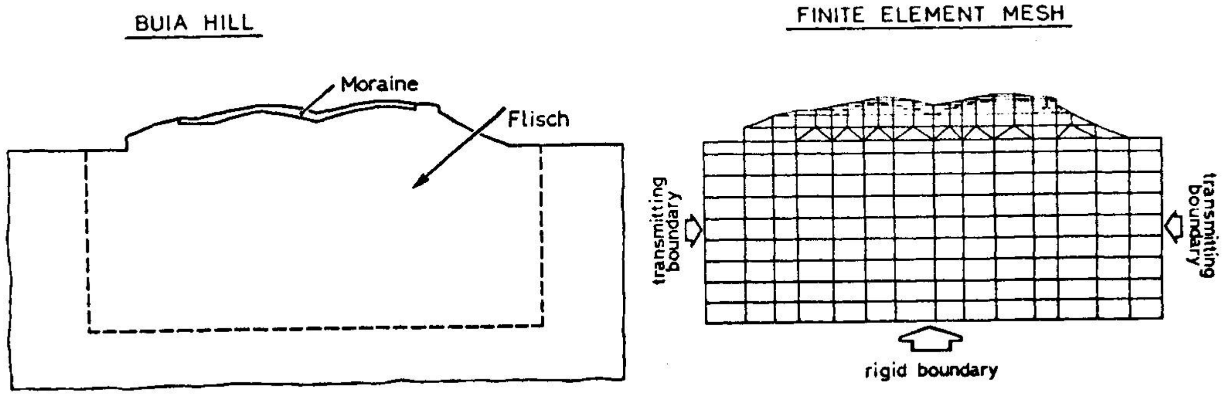

FIG. 1 - Buia hill. Geometry and finite element mesh

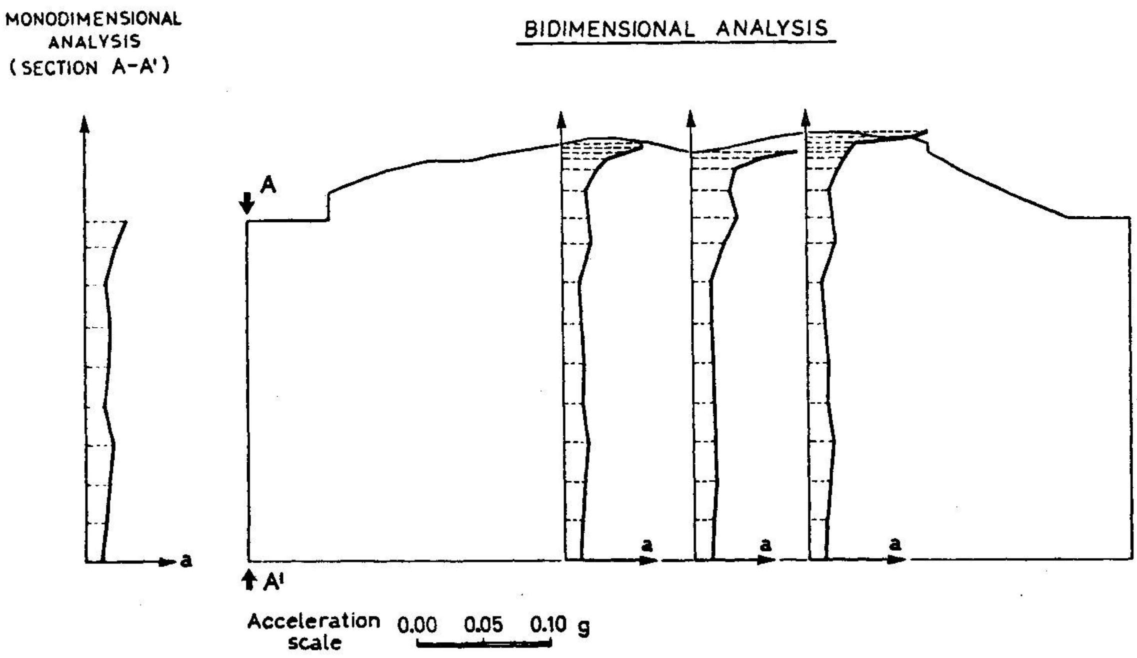

Fig. 2 - Buia hill. Comparison of accelerations obtained with monodimensional and bidimensional analysis

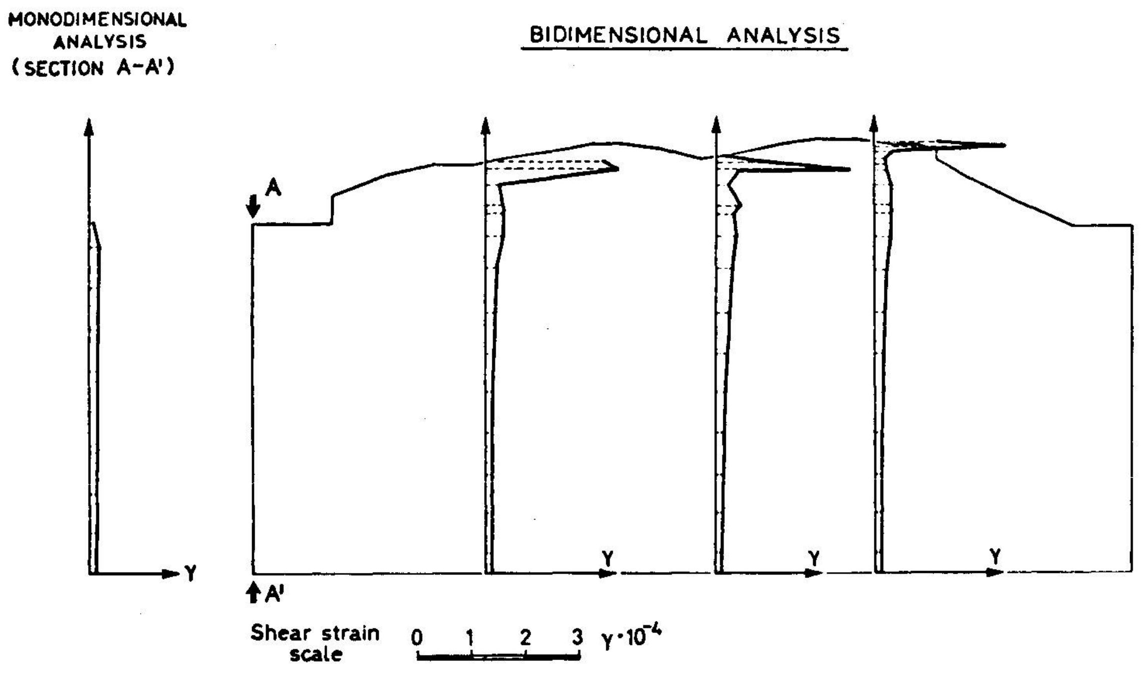

Fig. 3 - Buia hill. Comparison of shear strains obtained with monodimensional and bidimensional analysis

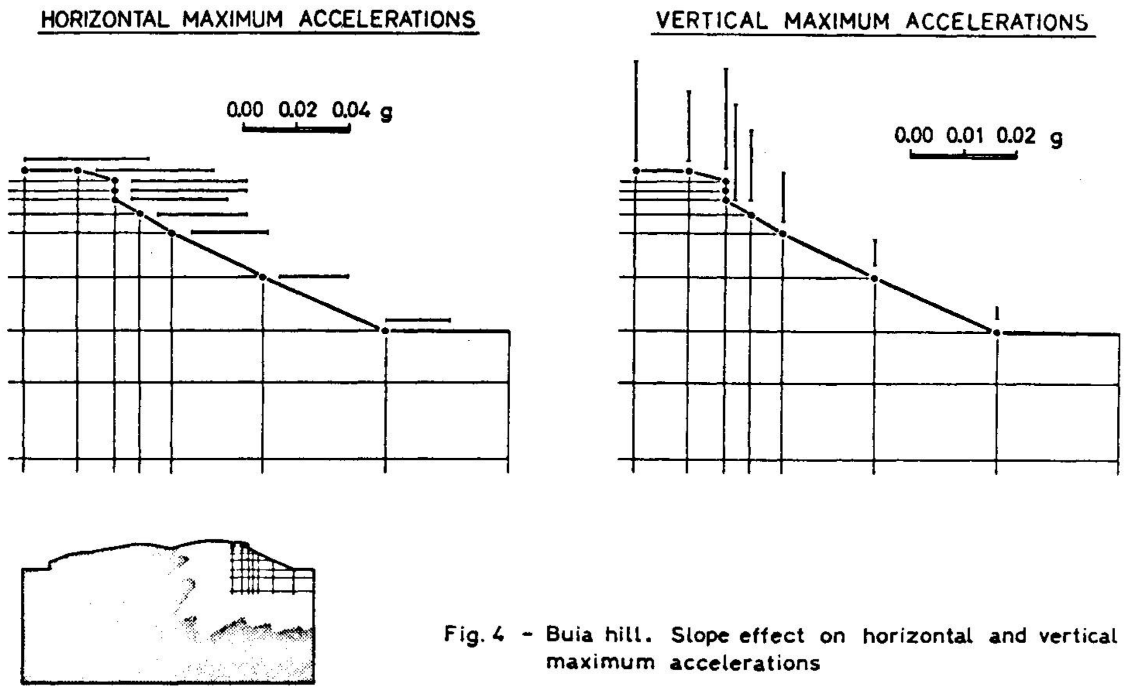

Fig. 4 - Buia hill. Slope effect on horizontal and vertical maximum accelerations

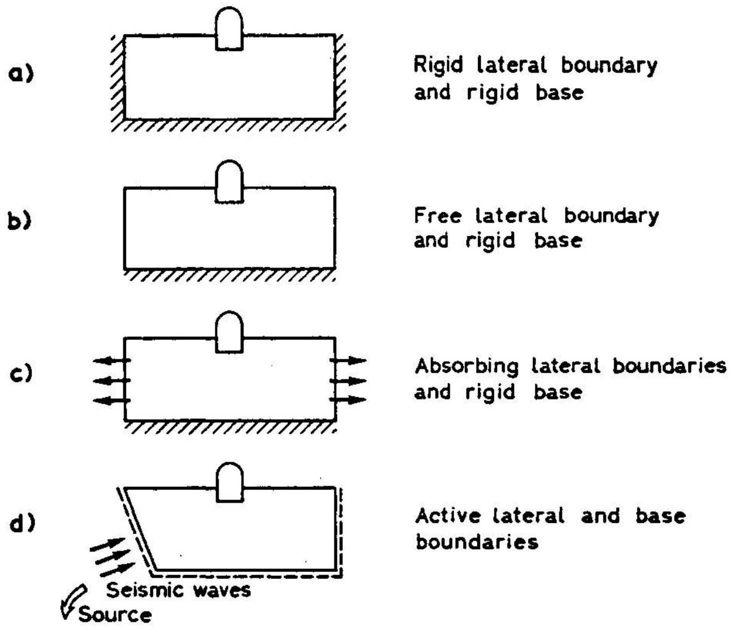

Fig. 5 - Different analytical model for earthquake response of soil deposits

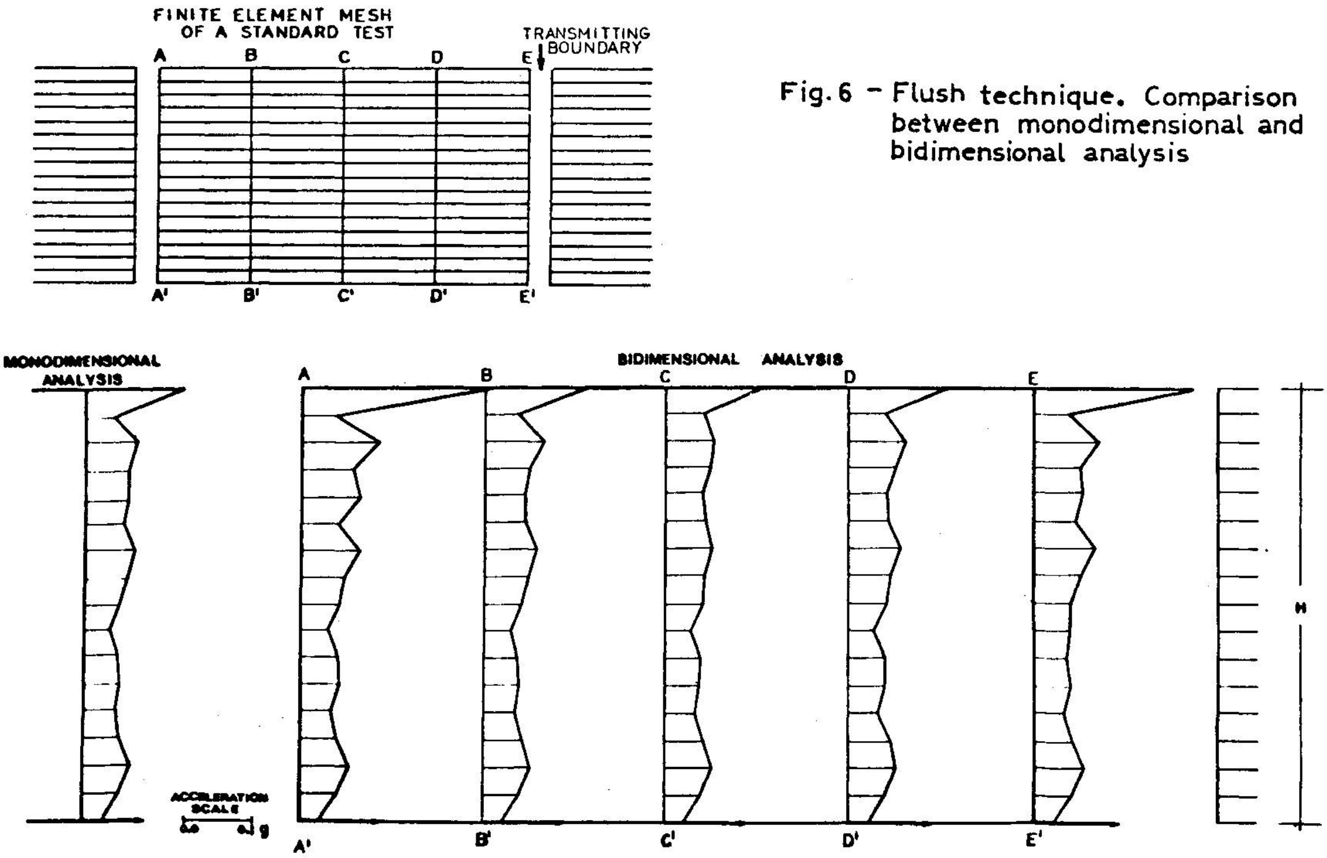

Fig. 6 – Flush technique. Comparison between monodimensional and bidimensional analysis

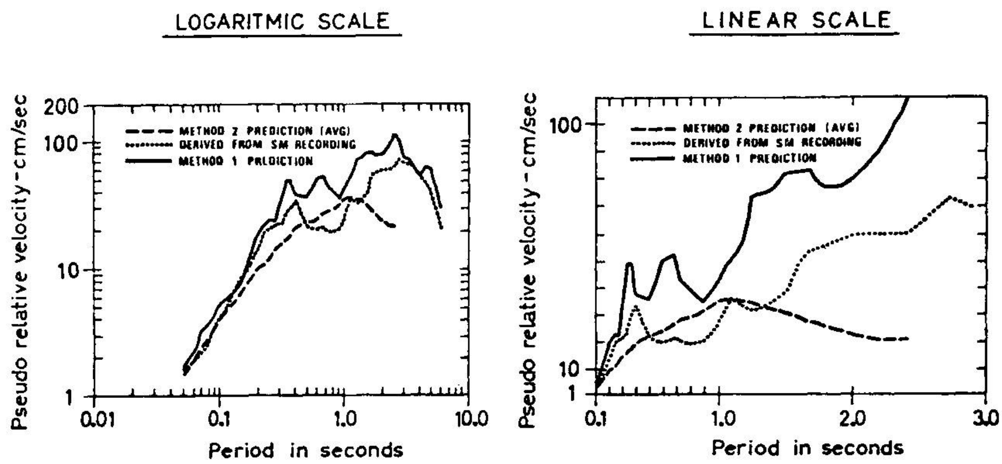

Fig.7 - Calculated relative site transfer function and comparison of spectral prediction methods - Hays et al. results [11]

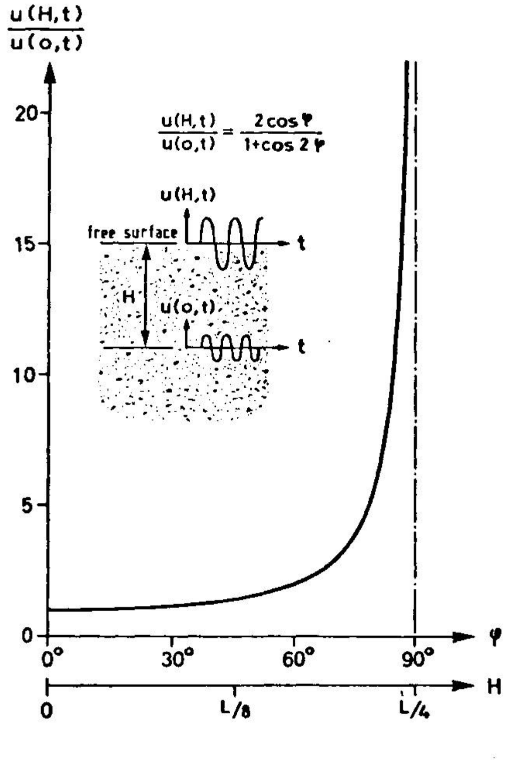

Fig. 8 - Frequency transfer function

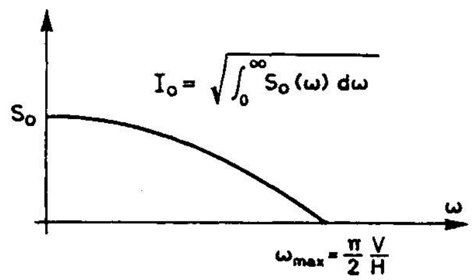

Fig. 9 - Assumption for power spectral density of seismic acceleration at some depth

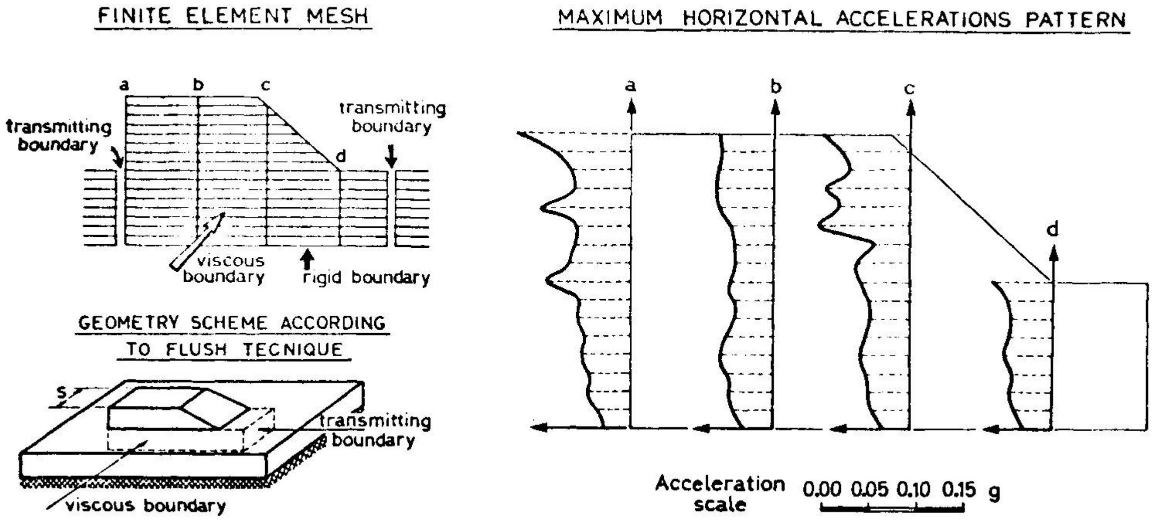

Fig. 10 - Hilly site. Geometric effect on accelerations

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QUANTITATIVE APPRAISAL OF DESTRUCTIVE EARTHQUAKES

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SUMMARY

Strong motion instruments have not captured so far variations of strong ground motion in the epicentral region of destructive earthquakes. Patterns of this variation may be recognised on field measurements of permanent sets.

Localised deformation is responsible to a great extent for the damage in the epicentral region. This mode of failure correlates well with a sliding block type of field measurements.

INTRODUCTION

There are few records of ground motion in the epicentral areas of destructive earthquakes. In the few cases where the epicentral area was equipped with strong motion instruments, the obtained records were too few to pick up the variation of ground motion. On the other hand, the strong motion in the epicentral region produces permanent sets of deformation; this deformation leads to geometric instabilities along collapse mechanisms or produces a vector field of permanent displacements. Both types of deformation may be quantified on simple structures. Bearing this use in mind the simple structures may be termed field indicators.

Each of the field measurements is associated with large uncertainty and is not amenable, on its own, to detailed analysis. A set, however, of identical field indicators scattered in a large area will show distinct patterns. The measurements may be correlated with ground motion characteristics and then with damage or directly with damage inflicted on more complicated structures.

This presentation is concerned with direct correlations of the field measurements with damage on engineering structures. The first part is a discussion of the set of simple structures that have the characteristics of field indicators. In the second part correlations are indicated between the field measurements and the inflicted damage. All the illustrations were obtained in the epicentral region of the May 6, 1976 Friuli earthquake.

FIELD INDICATORS

Among the instruments designed to record ground motion, the seismoscope may be classified as a field indicator. In general, a seismoscope is looked upon as a simple structure of a single natural period. $T_o \triangleq .75$ sec. and structural damping $\lambda = 10\%$; the response of the structure to the ground motion is recorded in two dimensions and the maximum response gives a point on the elastic response spectrum. Many simple structures of essentially one degree of freedom may be found in the field. Their behaviour is usually considered in terms of a linear pendulum (in either shear or rocking mode) but their response to ground motion is measured from their permanent deformation. Therefore, field measurements on this type of indicators give lower bounds of ground motion. There are other structures however which can absorb large deformation without getting unstable. These structures respond, beyond a stress threshold, to different levels of strong ground motion. Their mechanical analogue is a sliding block with some interface strength characteristics. The strength characteristic imposes a triggering threshold much higher than the one used by strong motion instruments (.01g) and limits their use as field indicators to the epicentral region only. Their measured response is not affected by built-in elastic constants and therefore they offer a measure of damage associated with concentrated excessive dislocations.

In the epicentral region of the May 6, 1976 Fruili earthquake an extensive network of field indicators was identified with the small distribution transformers of the electricity supply network. These transformers possess instrument characteristics: they are of uniform size, sitting on concrete pads inside uniform cabins (Fig. 1, 2, 3). A log is kept for each cabin at the regional ENEL centre in Udine. The log registers the exact dimensions of the transformers and the exact location of the cabins (an accurate location map is also available.) During the earthquake the transformers moved and left clear traces on the concrete platforms. The distribution network does not necessarily follow the population density, since transformers were allocated to isolated consumers i.e. farms. It is thought that a uniform density network of field indicators could have been selected from the network of distribution trans-

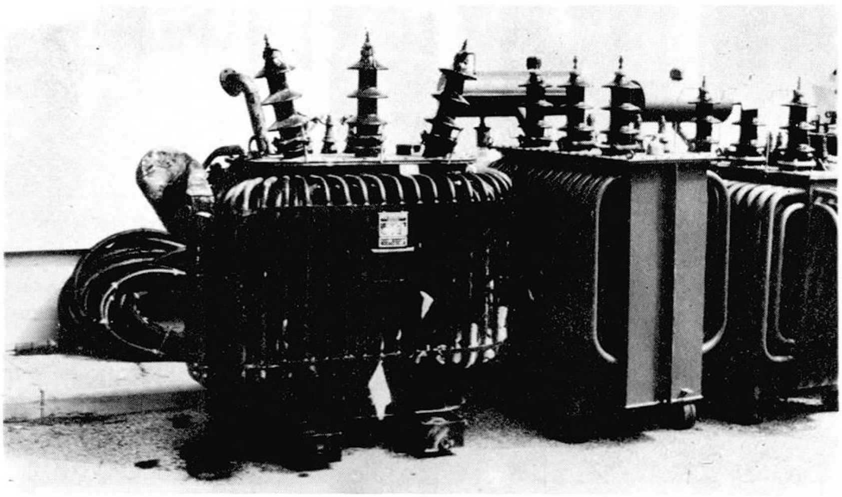


Fig 1. Damaged and new distribution transformers.

Fig 2. Distribution cabin at Bilerio.

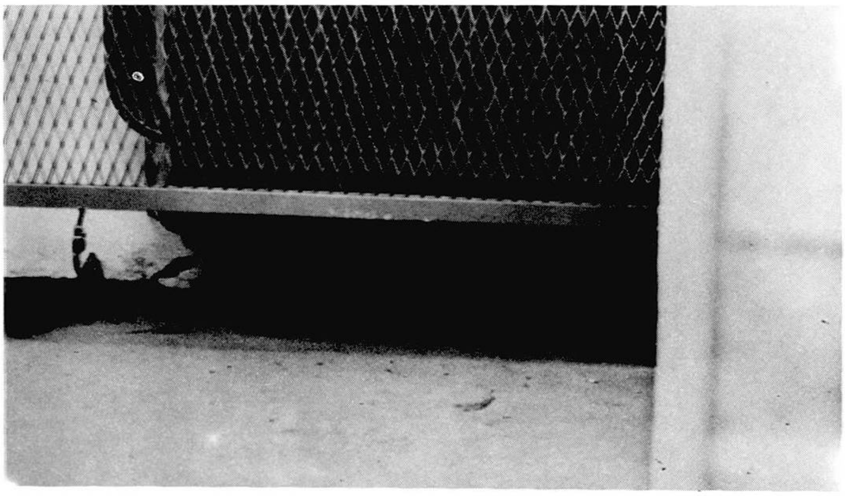


Fig 3. The transformer inside the cabin has moved during the earthquake; one of the back wheels fell into the oil drainage well.

formers in the earthquake area. A systematic recording of the slip vectors of the transformers in the network would have accounted for the scatter of the parameters for each individual station and would have revealed patterns with respect to the variation of geology and topography.

Field measurements in the epicentral area of the May 6, 1976 earthquake were restricted in a zone running NW-SE The slip vectors of the transformers that were studied in this zone are shown on Fig. 4. The transformers at the NW end of the zone surround the nearest strong motion instrument that recorded the main shock at Diga de l' Ambiesta. The measurements have been described elsewhere (1,2). The transformers, in general, moved in a N-S direction. Exceptions were found on recent alluvium where motion was in both directions. Displacements were also larger on recent alluvium.

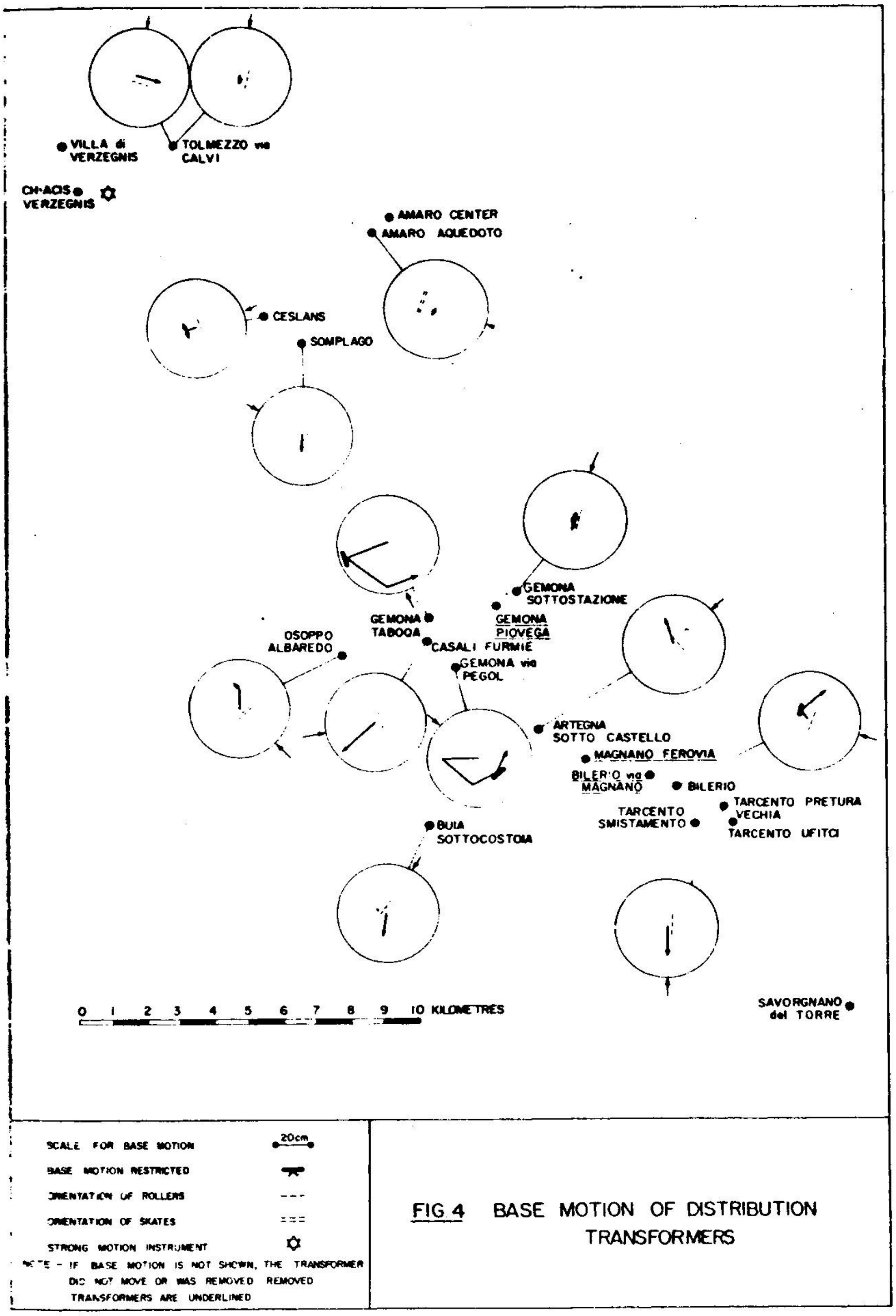


FIG 4 BASE MOTION OF DISTRIBUTION TRANSFORMERS

CORRELATION WITH DAMAGE

By and large, damage in the epicentral area may be attributed to excessive dislocations. The dislocation, necessary to produce collapse varies from structure to structure. This type of damage may be correlated to the slip vectors of sliding blocks in the area. The illustrations below are taken from the epicentral region of the May 6, 1976 Friuli earthquake and parallels are drawn to the few field measurements obtained from the distribution transformers in the same area.

Natural slopes were unstable enough to be triggered by relatively small horizontal dislocations. Of special interest were small landslides, like the one by the road at Cornino (Figure 5); similar landslides might have been responsible for foundation failures in the badly hit villages on alluvial slopes.

The transformers slip varied on alluvial slopes (e.g. Gemona) but was not as

Fig 5. Small landslide by the road at Cornino.

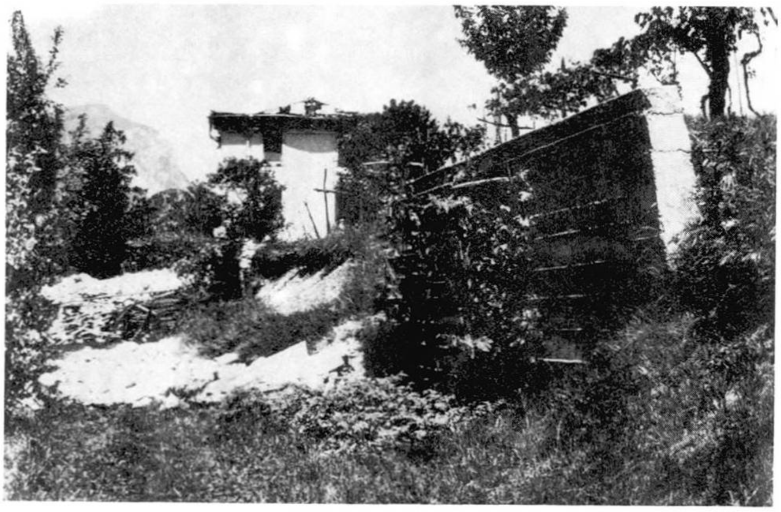


Fig 6. Failure of masonry retaining wall at Gemona.

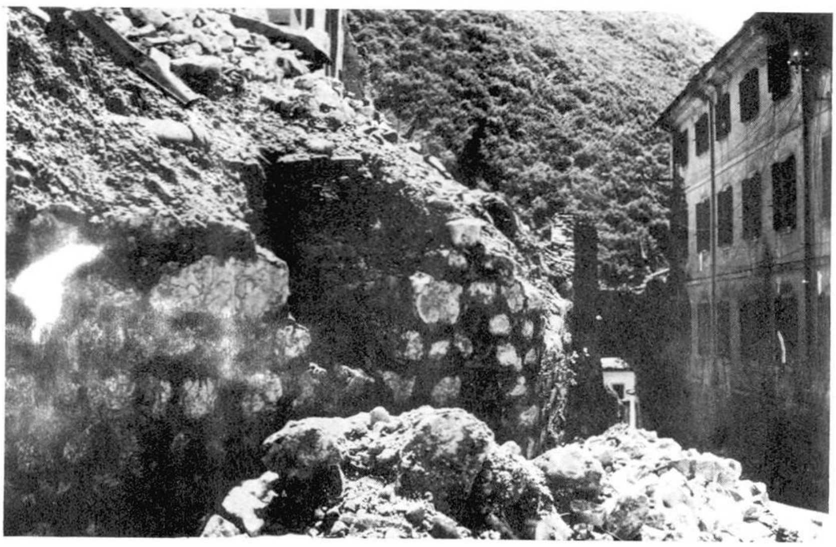


Fig. 7. Duomo Gemona; Failure of the retaining wall.

large as in the valley. The slip vectors, however, were large enough to cause the failure of retaining walls. Figure 6 was taken from Gemona. The low strength masonry wall could not resist the slip vector as did the concrete wall sitting next to it.

Figure 7 shows the failure of the retaining wall at Duomo, Gemona. Was this failure responsible for the collapse of the southern wall of the Duomo?

Figure 8 freezes an early stage of collapse of a masonry building at Gemona. This mode of failure looks similar to the failure of the mass behind a retaining wall and indicates that failure was initiated by the large rotations of the window-frame. The same mode of failure could be interepreted as the result of large vertical component of motion. This component of motion, although sensed by the transformers, could not be decoupled from the rocking mode.

An unfinished brick building in Gemona moved on its concrete foundation like the transformers in the region (Fig. 9).

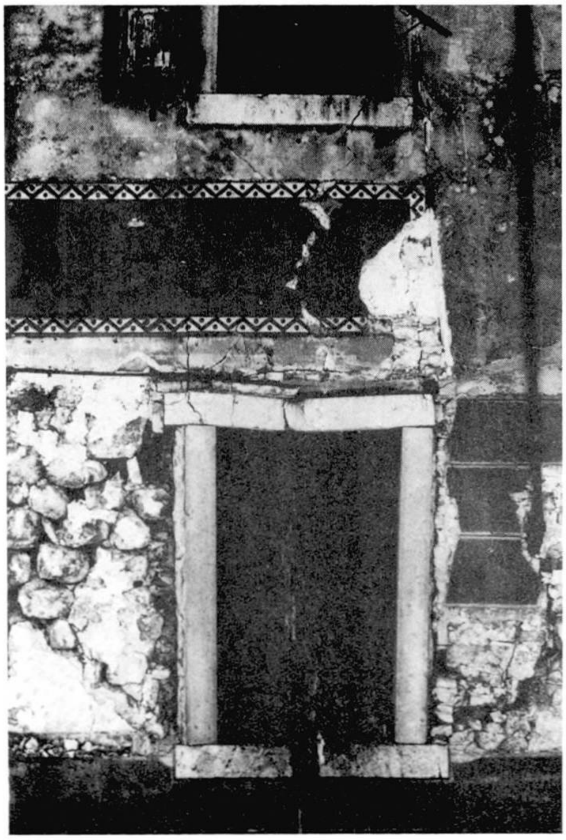


Fig. 8 Collapse mechanism of masonry wall at Gemona.

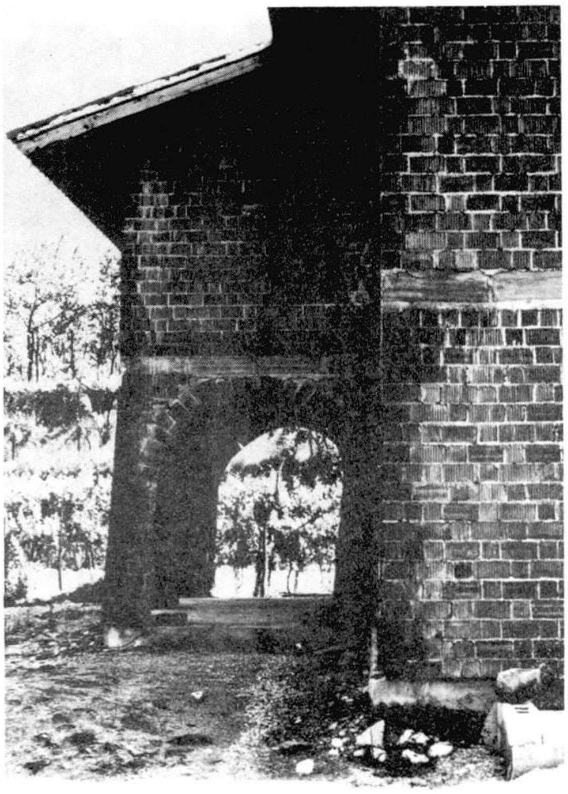
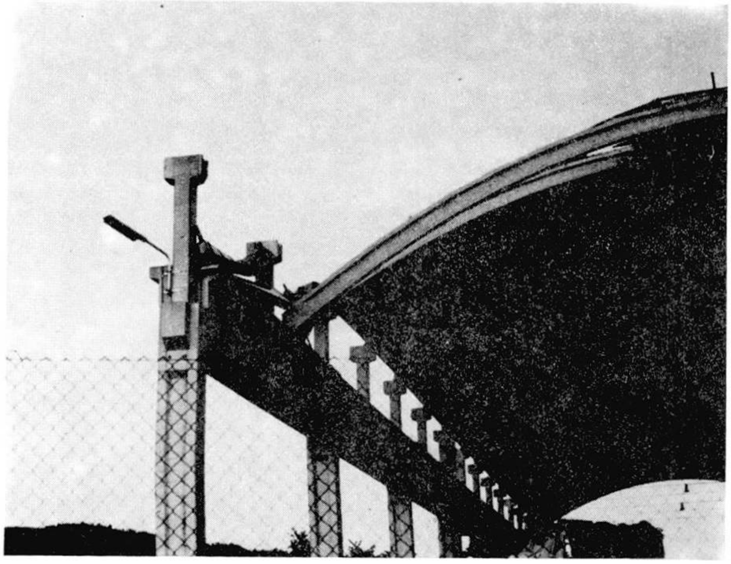


Fig. 9 Gemona; the building moved back and forth on the concrete base.

Figure 10 shows the mode of collapse of an industrial hangar. In this case the relatively small slip vector measured on the ground was amplified by the elastic deformation of the column.

Dislocations along joints in prefabricated buildings may be critical in their earthquake performance as shown on Figure 11. The building is in Majano across the road from another building that collapsed. A variation of the joint strength in one direction could have set up the collapse mechanism.



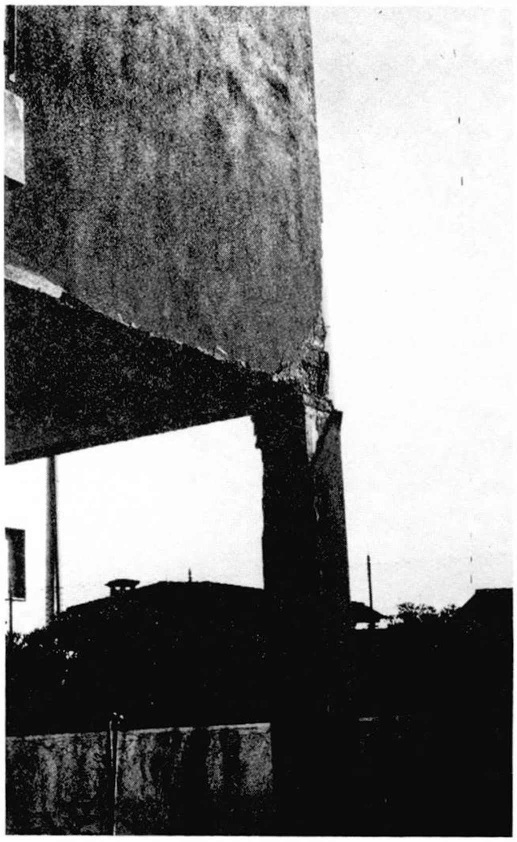


Fig. 10 (above) Industrial hangar at Magnano; collapse mechanism.

Fig. 11 (right) Slip at the joint of a prefabricated building at Majano.

On the recent alluvium the transformers showed large displacements. The same large dislocations could be seen along the supports of the prestressed bridge across the Tagliamento River (Figure 12).

Fig. 12 Prestressed bridge over the Tagliamento River; slip of the supports.

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THE MODELLING OF SPECIAL WATER FILLED STRUCTURES UNDER SEISMIC LOADS

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ABSTRACT

Special water filled structures (water-towers) need special rules for the design against earthquakes. The effects of water, the plastification of the structure and the soil-structure-interaction have to be taken into account. The corresponding design rules are discussed considering a watertower, destroyed during the Friuli earthquake of May 6, 1976.

ABSTRAIT

Les réservoirs d'eau sur hautes piles exigent des méthodes de calculation spéciaux pour le design seismic. Les effects de l'eau, la plastification de la structure et l'interaction du sol avec la structure doivent être considérés. Les règles correspondentes pour le design seismic sont traitées à l'exemple d'un réservoir d'eau qui a été détruit pendant le tremblement de terre en Friuli du 6 mai 1976.

AUSZUG

Wasserbehälter auf hohen Sockeln (Wassertürme) verlangen nach besonderen Berechnungsmethoden für die seismische Auslegung. Insbesondere sind die Wassereffekte, das Plastifizieren der Struktur sowie die Bodenflexibilität in Erwägung zu ziehen. Am Beispiel eines während des Friauler Erdbebens vom 6. Mai 1976 zerstörten Wasserturmes werden die wichtigsten Bemessungsregeln beleuchtet.

1. INTRODUCTION

Among the buildings destroyed during the Friuli Earthquake of May 6, 1976 was a watertower at the Gemona railway station, which apparently had not been sufficently designed against earthquake forces (Figure 1).

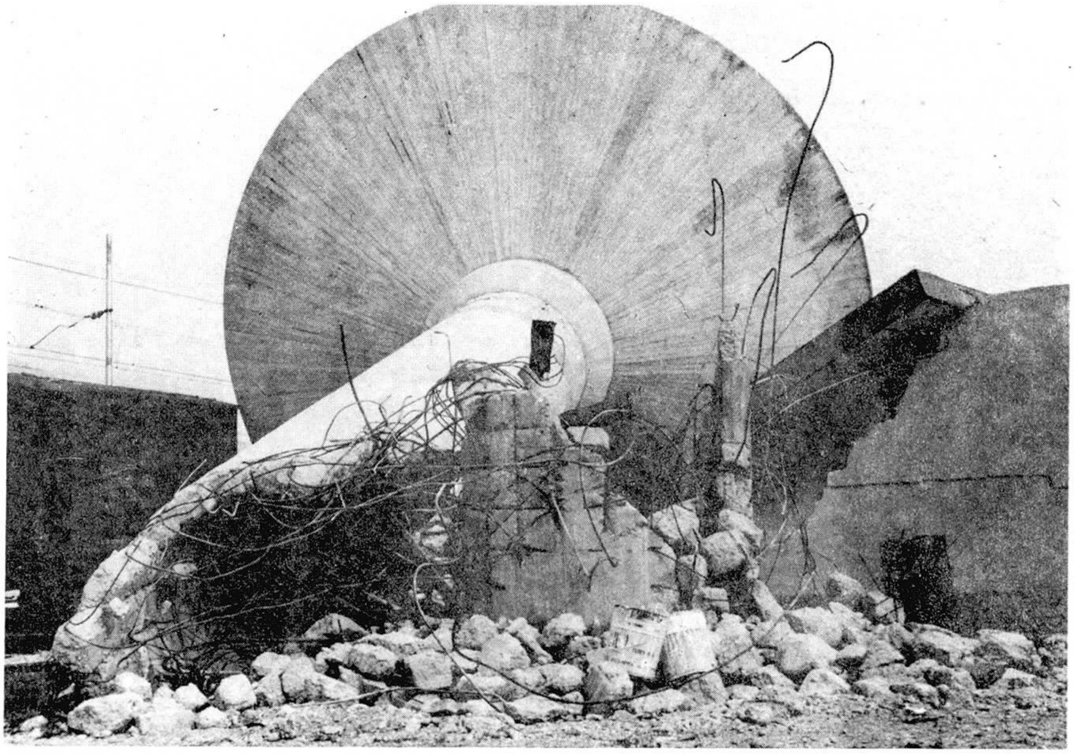


Figure 1: Destroyed watertower at Gemona railway station

Buildings of this type are of special interest because they cannot be designed by normal rules of a building code. Moreover, they belong to lifelines which demand special design considerations.

The purpose of this paper is to discuss very simple rules for modelling and designing such watertowers from a practical point of view, and besides give an answer to why the watertower at Gemona railway station failed.

Taking the hydrodynamic effects into account, Housners method [1] is expanded to conical and composite containers. Structural nonlinearities as plastic flow and P-Δ-effect are considered too.

2. DESIGN PROBLEMS

Some of the most important questions a designer of a watertower has to answer are the following:

a) How is the water pressure on the container wall to be calculated? Has the water-structure-interaction to be taken into consideration?
b) Which is the effect of the water on the frequencies of the whole structure and on the bending moment at the base of the tower?
c) How has plasticity to be taken into account; which model would be appropriate?

d) How would the vertical force (weight of water and container) influence the response and the failure?
e) Does the nonsymmetric opening (door) at the base of the tower produce torsion, resulting in stiffness-degrading of the column?
f) Which is the influence of the foundation flexibility on the response?
g) Which form of seismic input would be appropriate to this type of problem?

3. BASIC CONCEPT FOR DESIGN

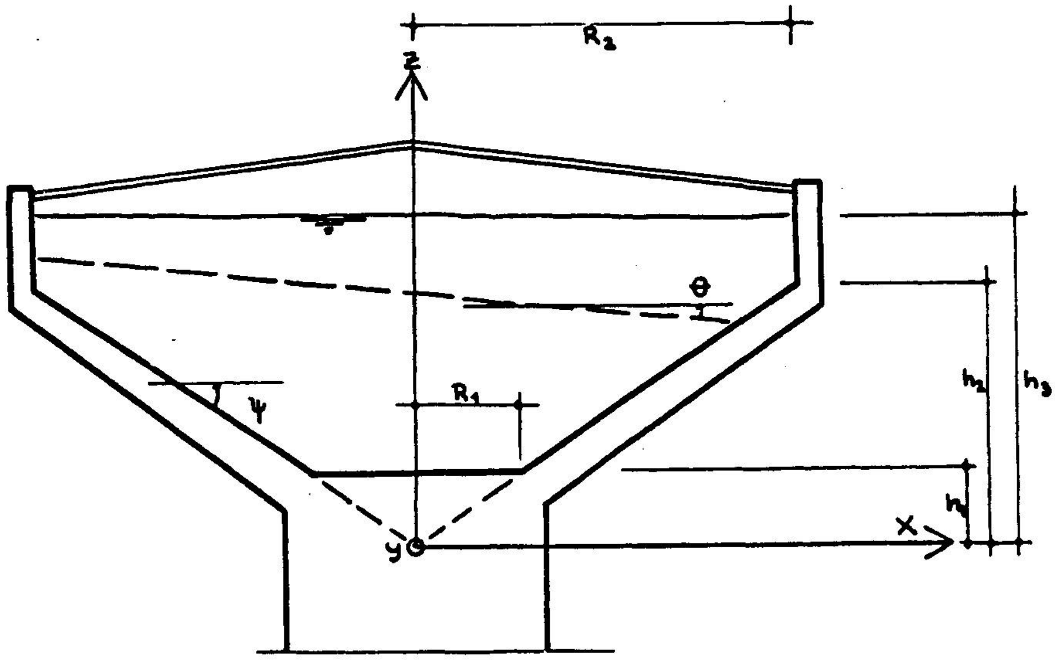
The watertower has to be designed in such a manner that it will withstand the specified earthquake without being fully destroyed but it may be severely damaged. In other words the behaviour of the structure during the specified earthquake may go into the plastic range, however the structure's ultimate resistance should not be exceeded.

For ease of handling the design by the Structural Engineer it should be tried to decouple the water-structure-interaction problem and to solve the hydrodynamic problem by a simple method.

4. HYDRODYNAMIC FORCE

As mentioned before it will be tried to decouple the hydrodynamic problem from the determination of the overall structural response. This can be accomplished if

- the container walls are assumed to be rigid,



h_1 = 0.74 m R_1 = 1.30 m
h_2 = 2.54 m R_2 = 4.55 m
h_3 = 2.89 m $tg\psi$ = 0.558

Figure 2: Geometry of the container of the considered watertower (Gemona railway station)

- the water is considered to be incompressible,
- Housner's concept [1] of added masses and springs for modelling the water is employed

It may be shown that in the present case the first two assumptions lead to reasonable results. It is recommended to use the added masses concept since until now there exists no exact direct solution of the hydrodynamic problem in containers of the given shape (Figure 2).

In the following, the methods to determine these added masses and springs are discussed. Housner [1] destinguished the pressures on the container wall in respect to impulsive and convective pressures. In the present case, where part of the container is of conical shape (Figure 2), the convective pressures are of special importance.

4.1. Convective pressures

When the walls of the fluid container are subjected to horizontal accelerations, the fluid itself is excited into oscillations and this motion excerts pressures on the walls of the container. To examine the first mode of vibration of the fluid, constraints to be provided by horizontal, rigid membranes, free to rotate are considered as in [1] and as shown in Figure 2 (the rigid membranes are inclined by the angle). Following the same procedure as in [1], a velocity field which satisfies the boundary conditions at the rigid wall is assumed which means that the water particles move parallel to the wall. Such a velocity field is found in the conical part of the cylinder to be of the form

$$\begin{aligned} u &= \tfrac{1}{3} b^2 \dot{\theta}_{,z} + \tfrac{1}{a} r \dot{\theta} \\ v &= -\tfrac{1}{3} xy\, \dot{\theta}_{,z} \\ w &= x \dot{\theta} \end{aligned} \tag{1}$$

and in the cylindrical part (from [1])

$$\begin{aligned} u &= \tfrac{1}{3} b^2 \dot{\theta}_{,z} \\ v &= -\tfrac{1}{3} xy\, \dot{\theta}_{,z} \\ w &= x \dot{\theta}\,, \end{aligned} \tag{2}$$

where r, b, x, y, and θ are defined in figures 2 and 3, $a = \mathrm{tg}\,\psi$, a dot means differentiation with respect to time and $(\ )_{,z} = \partial(\ )/\partial z$. u, v, w are the components of the velocity $\underline{v}$. The equations (1) differ from the equations (2) only in the component u, where in (1) the second summand is due to the conical shape of the container.

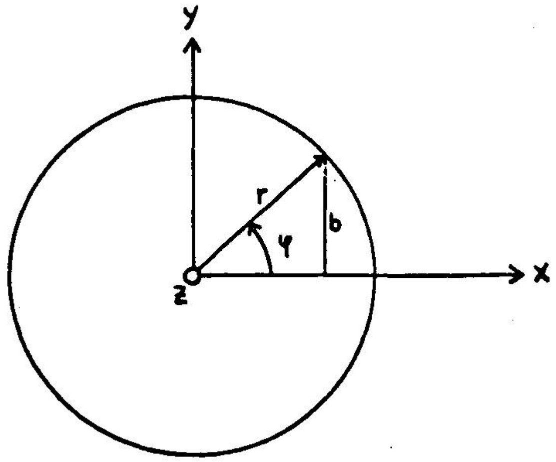


Figure 3: Plan view of the container (z = const.)

Assuming the distinct solutions θ_1 and θ_2 in the conical part and the cylindrical part of the container respectively, and by applying Hamilton's Principle

$$\delta \int_{t_1}^{t_2} (T - V)\, dt = 0,$$

T : kinetic energy of the fluid,
V : potential energy of the fluid,

the following set of differential equations and boundary conditions for the unknown θ_1 and θ_2 are derived:

$$
\begin{aligned}
&(2\alpha - \beta_{,z})\,\dot{\theta}_1 - 2(\gamma\,\dot{\theta}_{1,z})_{,z} = 0\,, && h_1 \leqq z \leqq h_2 \\
&\varepsilon\,\dot{\theta}_2 - \gamma\,\dot{\theta}_{2,zz} = 0\,, && h_2 \leqq z \leqq h_3 \\
&\dot{\theta}_1 \quad \text{finite or} && z = h_1 = 0 \\
&\dot{\theta}_1 = 0\,, && z = h_1 > 0 \\
&\dot{\theta}_1 = \dot{\theta}_2\,, && z = h_2 \\
&\beta\,\dot{\theta}_1 + 2\gamma\,(\dot{\theta}_{1,z} - \dot{\theta}_{2,z}) = 0\,, && z = h_2 \\
&\gamma\,\ddot{\theta}_2 + g\,\varepsilon\,\theta_2 = 0\,, && z = h_3
\end{aligned}
\qquad (3)
$$

where

g : gravity acceleration

$$\alpha = \left(\frac{1}{4} + \frac{1}{a^2}\right)\pi r^4$$

$$\beta = \frac{\pi}{2a} r^5$$

$$\gamma = \frac{2\pi}{27} r^6$$

$$\varepsilon = \frac{\pi}{4} r^4 .$$

Solving for $\dot{\theta}_1$ and $\dot{\theta}_2$:

$$\dot{\theta}_1 = (a_1 z^{\lambda_1} + a_2 z^{\lambda_2})\, C \sin \omega t$$
$$\dot{\theta}_2 = (\coth \mu z + a_3 \sinh \mu z)\, C \sin \omega t . \qquad (4)$$

These equations are quite general and may be taken to solve similar problems for containers with different shape.

With the dimensions of Figure 2,

$$\omega = 3.22 \ \sec^{-1}$$

$$a_1 = 1.293 \qquad a_3 = -0.6438$$

$$a_2 = -0.3920 \qquad \mu = 0.4038 .$$

The pressure p in the fluid is given by

$$\text{grad } p = -\rho \dot{\underline{v}} ,$$

which leads to the horizontal force P_x on the wall. P_x is calculated in the considered example as

$$P_x = -89.66\, \rho \pi \omega C \cos \omega t .$$

The moment M exerted on the wall turns out to be

$$M = -733.2\, \rho \pi \omega C \cos \omega t ,$$

so that the elevation of the applied force P_x is given by

z = 8.18 m.

This is an important result because it shows that containers of conical shape lead to much higher moments than comparable cylindrically shaped containers. Consequently, in the case of conical and combined containers as in the considered example, application of Housner's added masses for cylinders would lead to unsatisfactory results. For this example, the added mass and spring may easily be calculated. By comparing the exerted force on the wall and the kinetic energy of the added mass with the respective amounts of the fluid, the added mass m_1 and the spring k are given by

$$m_1 = 0.602\ m_o \qquad m_o\text{: mass of water}$$
$$k = 4.78 \cdot 10^5\ \text{kg/sec}^2.$$

The respective figures for a cylindrical container of equal volume and radius as for the considered combined container are

$$m_1 = 0.545\ m_o$$
$$k = 7.33 \cdot 10^4\ \text{kg/sec}^2$$
$$w = 1.33\ \text{sec}^{-1}.$$

As mentioned, the combined container leads to a significantly higher frequency for the first mode of the fluid than for a comparable cylindrical container.

Since an exact solution of the hydrodynamic problem in the case of the considered combined container is not available as of today, a direct check of accuracy is not possible. However, a comparison of the frequency of a rectangular conical container with the exact solution given by Troesch [2] shows, that the velocity field (1) leads in this case to the exact frequency.

4.2. Impulsive Pressures

No model exists in the sense of Housner's concept [1] , which allows to calculate the impulsive pressures for combined containers. Therefore it is suggested to choose an added mass, rigidly fixed to the container, calculated for a comparable cylindrical container following Housner's formulas. Since the added mass will be slightly overestimated by this procedure, the design proves to be conservative.

5. STRUCTURAL RESPONSE

5.1. Force-Deflection-Relationship

Considering now as an example the watertower at Gemona station (Figure 4), the structural response taking into account nonlinear effects and soil flexibility is discussed.

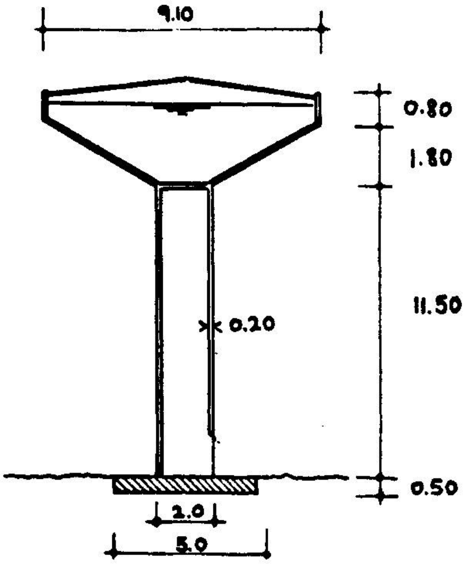


Figure 4: Watertower at Gemona railway station

First, the momentum-curvature-relationship of the reinforced concrete column is evaluated. As may be seen (Figure 5), the existing normal force increases the ultimate moment at ductile failure considerably.

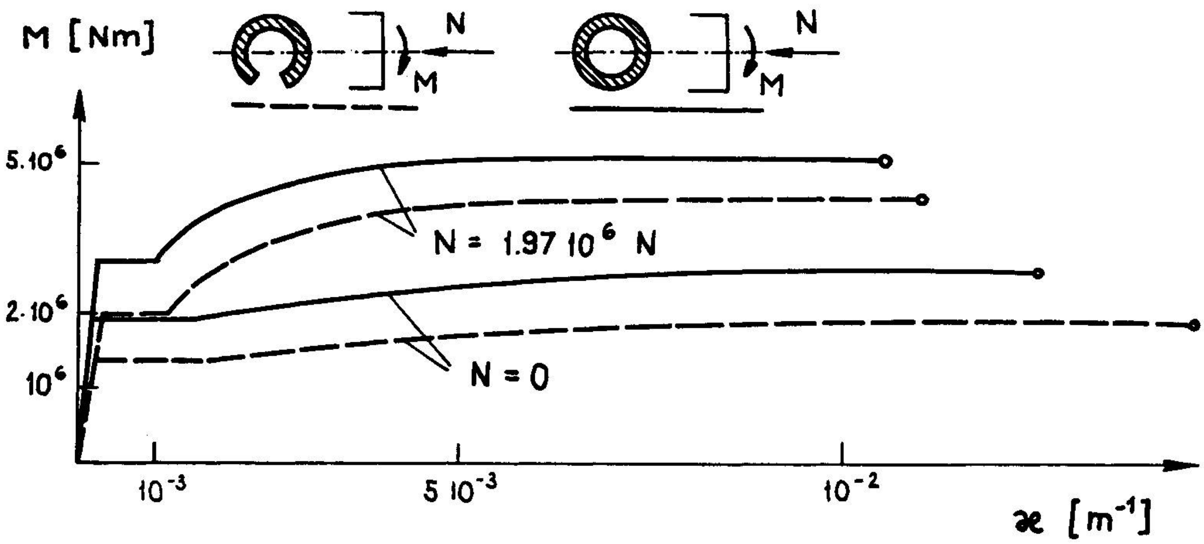


Figure 5: Momentum-curvature-relationship for the reinforced concrete column. The normal force $N = 1.97 \cdot 10^6$ N is the result of water and structure weight.

To get the deflection δ of the column due to the horizontal force F (Figure 6), the curves of Figure 5

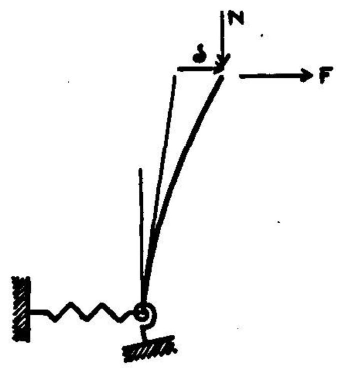


Figure 6: Structural model to calculate the force-deflection-relationship

have to be integrated over the length of the column for different forces F. This leads to the nonlinear force-deflection-relationship indicated in Figure 7.

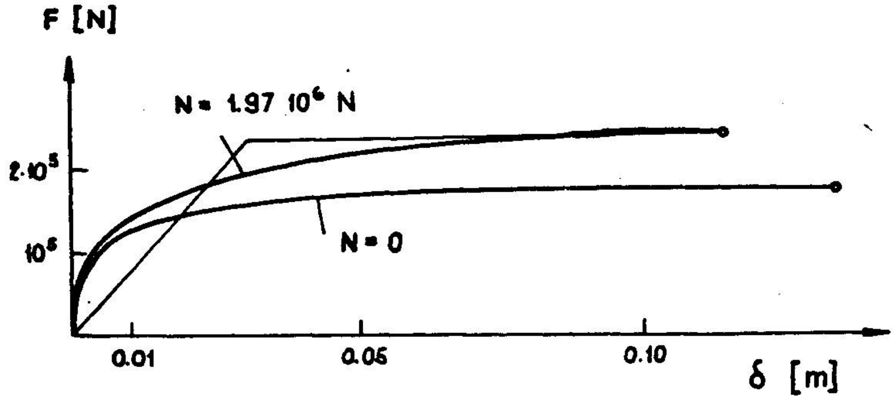


Figure 7: Force-deflection-relationship for the structural system of Gemona watertower

While the P-Δ-effect, which reduces the ultimate force F by approximately 10%, cannot be neglected, the decrease of stiffness due to the shear force may be disregarded.

Consequently, the vertical force influences the ultimate bearing capacity of the structure considerably. On the other hand, stiffness degrading which appears under high cyclic loading above failure deformation, has not to be taken into account here because the response above this limit is not considered.

For a simplified elastic analysis, an appropriate linear stiffness of the column may be chosen as indicated in Figure 7.

5.2. Elastic Analysis

An elastic analysis of the water-structure-system can be performed to study the coupling mechanisms of water and structure. For the purpose of determining and discussing the response of Gemona watertower a two-degree-of-freedom-system (Figure 8) is chosen.

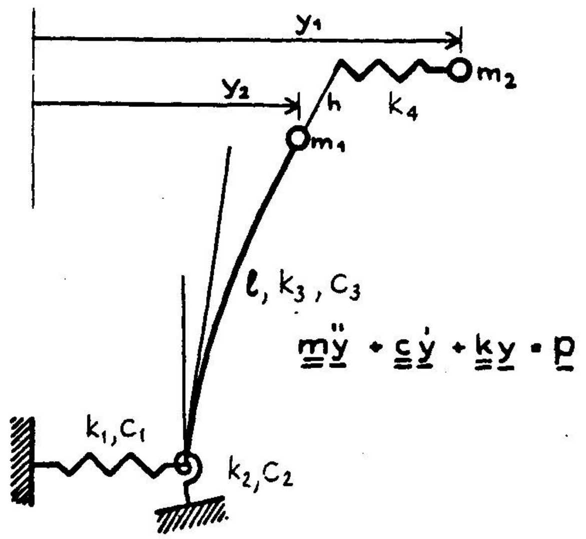


Figure 8: Structural system of Gemona watertower for elastic response analysis; m_1 is the sum of the weight of the structure and Housner's added mass for impulsive pressure; m_2 and k_2 account for the added mass-spring-system resulting from the convective pressure in the composite combiner.

For a cylindrical container with flat bottom, h would be near zero, while in the present case h is not negligible and influences the response considerably. The input parameters for an elastic analysis are (see Figure 8):

$k_1 = 1.56\ 10^9$ kg/sec^2 $\quad C_1 = 26$ %
$k_2 = 9.52\ 10^9$ kg/m^2/sec^2 $\quad C_2 = 7$ %
$k_3 = 4.10\ 10^6$ kg/sec^2 $\quad C_3 = 7$ %
$k_4 = 4.78\ 10^5$ kg/sec^2

$l = 13.9$ m $\quad m_1 = 1.32\ 10^5$ kg
$h = 5.84$ m $\quad m_2 = 4.61\ 10^4$ kg.

The soil stiffness and damping parameters were derived from usual elastic half-spare theory, and the column stiffness k_3 was reduced by considering the fact, that the centre of gravity of mass m_1 lies above the upper end of the column.

A response spectrum analysis was performed, using a provisional response spectrum of ENEL, recorded at Tolmezzo (Italy) during the Friuli earthquake of May 6, 1976. Local soil conditions at Gemona were considered by introducing an increasing factor of 1.4, which leads to a maximum free field acceleration $a_o = 0.43$ g.

The eigenfrequencies of the system were found to be

$f_1 = 0.425$ Hz
$f_2 = 1.02$ Hz

while

$P_{x\,max} = 7.8\ 10^4$ N
$M_{max} = 4.1\ 10^6$ Nm
$\delta_{max} = 0.072$ m.

Two important conclusions may be drawn from these results and the respective mode shapes: A strong coupling between the water and the structure mode is noticed. Thus, the water influences the response of the whole system considerably. This fact is in direct connection with the parameter h. If h vanishes as e.g. for cylindrically shaped containers, this coupling effect becomes insignificant.

Secondly it is shown that the Gemona watertower had to fail because the calculated maximum moment at the base exceeds the ultimate moment by about 20 %.

In this example the rocking spring contributes to the response of the whole system by the amount of 10 % and therefore may not be neglected, while the horizontal spring turns out to be of little influence.

5.3. Torsion

A nonsymmetrical opening (door, 2.0 x 0.6 m) at the base of the watertower gives rise to the question if torsion may impair the bearing capacity of the structure. Considering a twisting moment which results from the maximum horizontal force exerted on the structure, it can be shown, that the ultimate bending moment (Figure 5) is reduced by about 3 % and the overall stiffness of the column (Figure 7) by 6%. Furthermore, a comparative elastic response spectrum analysis shows that the deflections due to twisting of the lowest part of the column is of no significance. Therefore it is concluded that torsional effects usually can be ignored.

5.4. Nonlinear response

In paragraph 5.2. a strong mode coupling was found. This mode coupling leads to both moment and horizontal force loading at the top of the column. If the moment was absent as e.g. for cylindrical containers, one might use the simplified bilinear force-deflection-relationship of Figure 7 directly and integrate step by step over time by applying e.g. the linear acceleration method. This method is easy to apply and gives reasonable results. However, to get the nonlinear response of watertowers with conical containers, this method is not acceptable. In this case, where the moment at the top of the column is not absent, at every time step the moment-curvature-relationship of Figure 5 has to be integrated over the length of the column to give its deflection for a given pair of M and F at every time step. Alternatively a nonlinear finite element procedure may be chosen.

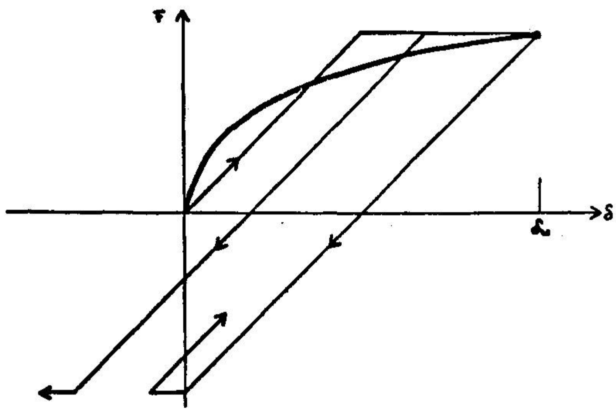


Figure 9: Simplified bilinear hysteretic behaviour of the Gemona watertower column.

6. CONCLUSION

In evaluating the response of watertowers it is possible to decouple the hydrodynamic problem from the determination of structural response by using the added mass concept. Fluid oscillations produce much higher moments in conical containers than in cylindrically shaped conbiners with flat bottom. Therefore it is recommended to construct only watertowers with vertical walls and flat bottom in seismic active zones.

In evaluating the seismic structural response, the weight of the water-structure-system and the rocking mode are to be considered while the twisting moment due to small nonsymmetries, and the shear force may usually be neglected. An elastic response spectrum analysis may be performed to evaluated the coupling behaviour of the modes. However, this method is not recommended for design purposes. Since the ultimate strength of the column is equal to the bearing capacity of the whole system and the design shall be such that the ultimate resistance is reached, a nonlinear analysis is required. This nonlinear analysis can be performed easily by small computer routines in case of a cylindrical containment with flat bottom; in case of a conically shaped containment a nonlinear finite element analysis would be appropriate.

Up to date, usual building code recommendations cannot be taken as design basis. Therefore, it is recommended to incorporate in the codes rules for the design of watertowers as developped in this paper.

ACKNOWLEDGEMENT

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