A PRELIMINARY REPORT
ON THE ERZİNCAN EARTHQUAKE, MARCH 13, 1992

13 Mart 1992 Erzincan Depremine İlişkin Bir Ön Çalışma

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SUMMARY

The preliminary data collected is being presented here concerning the strong earthquake hit the Erzincan area (Eastern Turkey), on March 13, 1992.
Brief information on geotectonic, seismologic and soil conditions is given. Attention has been paid to the accelerograms recorded in Erzincan and to the specific features of the response spectra. The poor behaviour of **rc** frame buildings has been considered. The limitations in ductility concept and some recommendations for the Code revision as well as recommendations on the rational structural systems for Erzincan type sites were discussed.

INTRODUCTION

A destructive earthquake has occured in the area of the city Erzincan, Eastern Turkey, on March 13, 1992. The coordinates of the epicenter are 39.71°N and 39.57°E. The depth of the epicenter is estimated as 26 to 28 km. The Richter's magnitude of the earthquake is Ms=6.8.

The MM-intensity in Erzincan and surrounding villages was about 9+0,5 degrees.

More than 15000 buildings were damaged during the earthquake, and more than 4000 buildings have collapsed or suffered heavy damages, 534 people died and 2800 injuries 11000 house holds lost their houses and finally 70000 people were affected by Earthquake.

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In this paper a preliminary explanations on the strong motion accelerograms recorded in Erzincan and on the response spectra are being presented. Province of Tunceli, located in the southern part of Erzincan also suffered badly; Almost 1000 houses were damaged, 500 of these entirely collapsed or highly damaged. Observations on the structural behavior of **rc** frame buildings and some of their dynamic characteristics in connection with the response spectra are presented as well. In addion the existance of the similarities of structural response of **rc** frame buildings subjected to the earthquake in Spitak, Armenia, 1988, and Erzincan, Turkey, 1992 are indicated.

GEOLOGICAL, SEISMOLOGICAL AND SOIL CONDITIONS OF THE ERZİNCAN REGION

Turkey lies within the Mediterranean sector of the Alp-Himalayan orogenic system. The Alpine orogeny is produced as a result of the compressional motion between Europe and Africa, whereas the Himalayan orogeny has resulted from the India-Asia collision. The main active faults illustrated in Fig.1 are as follows ,

1- North Anatolian Fault (NAF)-Anatolian Trough
2- East Anatolian Fault (EAF)
3- Western Turkey Graben Complex

The NAF is a morphologically distinct and seismically active right-lateral strike slip fault. It has a well developed surface expression for most of its length of approximately 1000 km. in the mainland. The Anatolian Trough is the westward continuation of the northern strand of NAF.

The EAF is an active left-lateral strike-slip fault which extends from Antakya to Karlıova, the eastern terminal of the EAF zone. It is a fault zone which is about 2-3 km wide and links to the south into the Dead Sea fault system.

The destructive March 13, 1992 Erzincan earthquake is the result of seismic activity in the eastern part of the North Anatolian Fault (Fig.1) and East Anatolian Fault. The most severe earthquake in Turkey experienced in the 20th century had the epicenter also nearby Erzincan, and around 38000 people lost their lives during that earthquake with M_s=8.1 and I=XI occured in December 26, 1939, [1].

Both December 26, 2939 and March 13, 1992, Erzincan Earthquakes resulted due to the seismic activity along the eastern part of NAF.

A geological cut nearby Erzincan is presented in (Fig.2) The shaded areas indicate the alluvial deposits on which the city of Erzincan has been built. As far as the seismic strong motion parameters are concerned, this has a special importance, which will be discussed in the following paragraphs.

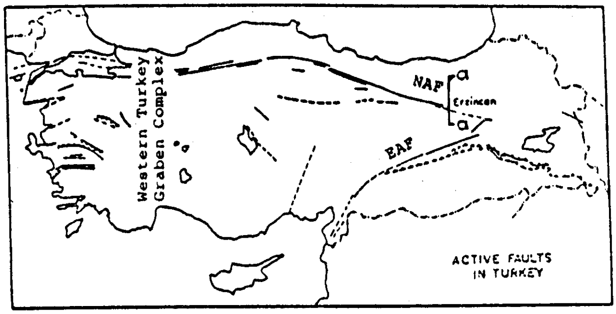

Figure-1

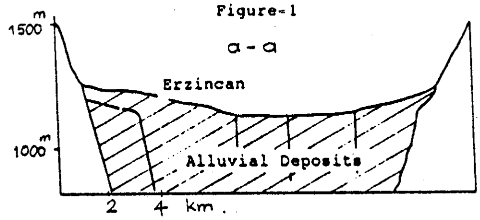

Figure-2

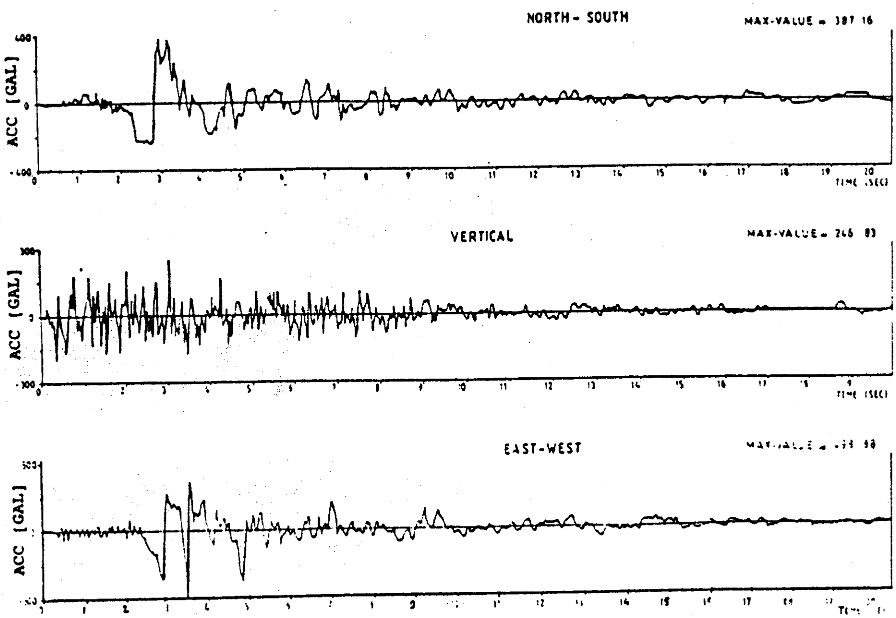

Figure-3

RECORDED ACCELEROGRAMS AND RESPONSE SPECTRA OF THE MARCH 13,1992 ERZİNCAN EARTHQUAKE

Several strong motion accelerograms were instrumentally recorded during the main shock. The recorded, namely, North-South and East-West horizontal components and the vertical component are presented in (Fig.3). The spectrum curves obtained using the digitized data of (Fig.3) which is given in [9] are presented in (Fig.4) (after the courtesy of Dr. Simirnov).

When the recorded accelerograms are analysed it has to be kept in mind that the sensor of the recording instrument was fixed to a concrete foundation placed 120cm away from the building of the Meteorogical Station in Erzincan where the record was obtained during the main shock. The soil-structure interaction perhaps influenced more or less the free surface earthquake motion. It would be expedient to analyse the soilstructure interaction problem in the future to understand the extent of the distortion of the free surface accelerations. However, the building of the Meteorological Station seems to be a relatively rigid, and not a heavy structure. Therefore, the expected distortion of free surface motion, due to the interaction of the building probably is not too large.

The following preliminary results can be achieved after having a visual inspection of the accelerograms and the associated response spectra:

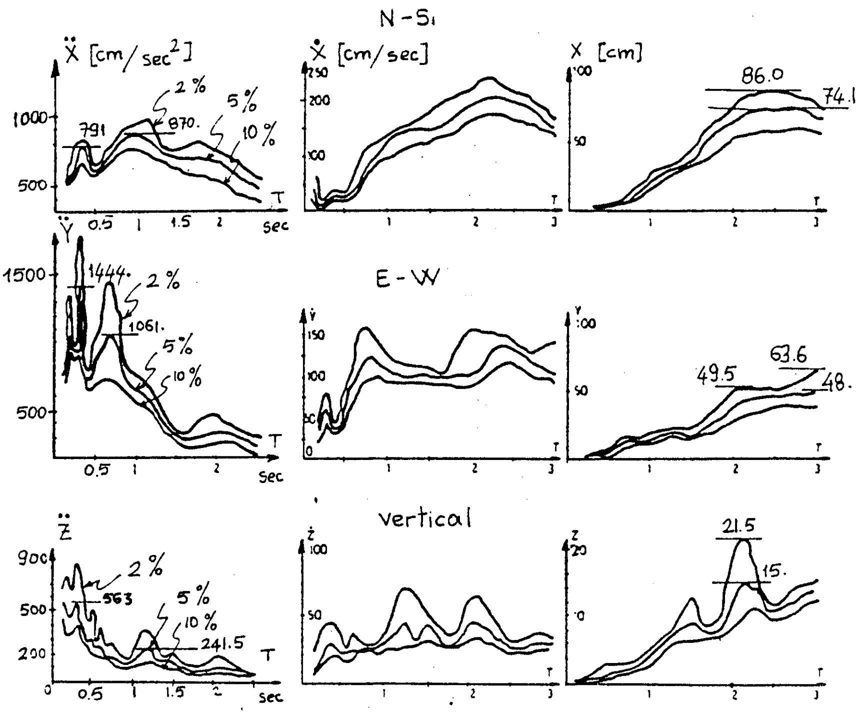

Figure-4

1. Response spectrums of North-South and East-West horizontal components of the main shock have more or less two distinct peaks. One of them is around the period of 0.3 seconds and the others are observed around the period ranges of 1.0. - 1.1 and 0.6 - 0.7 seconds, respectively.
2. The maximum ordinates of the vertical component response spectrum correspond successively to 0.1 sec, at which the highest response is achieved, and to 0.25-0.30 seconds. Some other additional small peaks are observed for all three components at periods around 2 sec and they take even larger values in velocity and displacement spectrums.
3. The spectral peak accelerations and corresponding amplification factors are roughly given in Table 1.
   As it is seen, the peak values of response accelerations are very high. And the amplification factors for critical selected periods are of comparatively high
4. Maximum spectral velocities for horizontal and vertical components are 150-205 cm/sec and 70 cm/sec respectively, corresponding to the range of periods of 0.7-2.0 second.
5. Maximum spectral displacement associated to the period range of 2.0-2.5 sec are 48-74 cm in two horizontal directions for 5% damping and 15.1 cm in the vertical direction. For periods of 0.1 seconds the vertical displacements are, approximately, 0.5-0.7 cm.

There are two important reasons for inspection of accelerograms and response spectrums: It provides better understanding of main features of the ground motion and its correlation with local soil and geological conditions, as well as better understanding of the seismic behavior of structures. All in all, it gives a chance to improve the earthquake engineering concepts.

The preliminary examination of the recorded accelerogram and response spectra permits to make some apriori suppositions. The existance of two peaks could have the following two physical explanations;

Table-1
Some Parameters of accelaration response spectra(5% damped system)

Peak Ground Accelaration $\ddot{X}$ $\ddot{Y}$ $\ddot{Z}$ max [cm/sec^2]		Periods Related to the peak response acceleration [sec]	Peak Response Acceleration $\ddot{X}$-$\ddot{Y}$-$\ddot{Z}$ [cm/sec^2]	Spectral Amplification Factor
X N-S	391	0.3 1.0-1.1 2.0	791. 870. 690.	2.02 2.22 1.76
Y E-W	491	0.3 0.6-0.7 2.0	1444. 1061. 501.	2.90 2.03 1.00
Z Vertical	250	0.3 1.1	563. 241.	2.26 0.97

Hypothesis 1:

The peak value of the acceleration response spectrum curve which corresponds to the period of 0.3 sec for the horizontal component coincides with the natural period of the interaction soil-structure system. The "structure" here refers to the building at which the accelerograph was installed. Probably, the 0.1 second period according to the peak value of the vertical component's acceleration spectrum, coincides with the natural period of the system in the vertical direction.

Hypothesis 2:

The peak value of acceleration response spectrum curve which correspond to the period of 0.3 sec for the horizontal component may reflect the predominant period of the seismic motion of rock under the alluvial soil layers. In both cases the peaks observed on the response curves against the periods 1.0-1.1 and 2.0 sec. for N-S and 0.6-0.7 and 2.0 sec. for E-W components should probably correspond to the predominant periods of the upper alluvial layers.

A comprehensive dynamic analysis should be carried out after having had dependable soil characteristics and the structural parameters of the Meteostation buildings at which the recording was obtained.

Independently, which of the two hypotheses will prove to be correct, it is obvious that relating to the relatively long period range, e.g T 0.6 sec., the Meteostation building behaved as a rigid body.

So the free surface motion recorded in the spectral interval of relatively higher periods could be assumed as not distorted by the building surface motion.

Then, if the Hypothesis 1 is correct, the free surface motion amplitudes in the spectral period range of 0.1-0.3 sec. should be smaller (maybe in 1.5 to 3 times) compared with the recorded amplitudes.

THE SEISMIC BEHAVIOR OF LOCAL STRUCTURES

Table-2 makes an outline of the damaged residential and office buildings in and around Erzincan. An investigation is being carried out for having a detailed scientific classification of the damaged structures using different indicators and parameters.

The damaged structures during the 1992 Erzincan Earthquake consists of two major groups. Adobe houses with a heavy earth roof, stone, mud block hollow brick masonary type one or two storey builidngs like in Davarlı 25 km. to the west from Erzincan where ground rupture observed visually can be considered in the first group. Reinforced concrete structures which do have only moment resistant frames are taking place in the second group of damaged structures. In this paper special attention has been given to the second group of engineering framed structures which are generally three to five storey residential or office buildings with shops at ground levels.

Most of the buildings investigated have one storey basements. Infilling materials used are generally brittle hollow bricks. As it can be easily estimated that there is no need to have relatively big columns for these types of structures. Because the total weight of the structure which controls the earthquake design forces ar relatively small. It is easily encountered in these types of structures with T shape strong beams 20*(50, 70) and weak columns 20*(40, 50, 60) which are supposed to be restricted at least theoretically for the buildings to be constructed in severe earthquake prone areas.

Table.2. The number of damaged apartment buildings and offices during Erzincan Earthquake, 26.III.1992

AREA	Damage rate					
	Heavy or (Collapse)		Medium		Small	
	Apart. buil.	Offi. Shops	Apart. buil.	Offi. Shops	Apart buil.	Offi. Shops
Erzincan-City	1344	825	2881	409	3832	229
75 villages surrounding Erzincan	1469	29	1547	24	2382	44
Üzümlü-town	23	0	30	0	294	13
25 Villages surrounding Üzümlü	406	0	346	2	623	6
TOTAL	3242	854	4.804	435	7131	292

It has to be kept in mind that there are too many framed rc buildings finished or underconstruction in Erzincan which are standing up with minor damages. Even higher structures, 6-7 storeys high, behaved very well against this severe earthquake, from getting damaged in their shear walls around the lifts and staircases.

On the other hand, two storey masonary stuctures with heavy reinforced concrete roofs and partitions made of brittle hollow bricks and many openings in the walls have totally collapsed in Üzümlü village 20 km.

to the East of Erzincan. The following paragrapghs have been devoted to the theoretical verification of these contradictory like special cases as well.

There are several important factors which should be consdiered during the damage assesment process. One of them is the high intensity of the Erzincan 1992 Earthquake. The maximum acceleration recorded in the Meteorological Building is 50% g. This may be even more in some other part of the city. It means that MSK intensity is above IX and may be X degrees. It is inevitable to have cracks, local damages, and inelastic deformations in both structural and nonstructural parts of the reinforced concrete frames if it is subjected to an earthquke with this intensity even if the quality of concrete used is good and the design is satisfactory. Of course the damages occured could be controlled and many human lives could be saved. As soon as the local damages are developed during the earthquake, the dynamic parameters of the structure change drastically. The frequency spectra of the structure changes. In other words, the natural period becomes much longer. Not only the lateral rigidity, but at the same time the strength,the inaelastic energy dissipation capacity of the system degrade. While the bearing capacity of the structure starts to decrease, the seismic loads imparted to the structure may start to increase because of the specific spectral features of the present Erzincan Earthquake. If the measured fundemantel periods of a sample structure is 0.2-0.3 seconds, it easily becomes 0.6-1.0 seconds after the structure having experienced the permanent deformations. Since the acceleration spectrum has the peak values around 0.6-1.1 seconds as it is mentioned above, seismic loads on the damaged structure may become higher than the initial seismic forces, (Fig-4). It has to be kept in mind that the damping ratio for the damaged building should be bigger than the original structure. According to the displacement spectra obtained, the displacements of elastic systems could reach to 70-80 cm. It is obvious that this amount will increase rapidly when the structure undergoes inelastic deformations, (Fig-4). As soon as the relative displacements start to increase by any reason, the second order effects of axial forces namely, p-delta effects, start to influence the stability of the system. Meanwhile, the importance of the vertical earthquake component grows, increasing the axial forces of the columns and hence the p-delta effects as well. Both the axial deformations and the second order effects of axial forces, it is known that has an influence, especially on the fundamental period of the systems, and if the structure is located on a soft soil, like in Erzincan, on the seismic design forces as well [5].

Low quality concrete, poor design, poor detailing and poor construction are the other important factors which should be considered in the process of damage evaluation. Low compressive strenght and poor bond between concrete and mild stell used generally, unsatisfactory and unproperly placed confinements in columns, beams and beam column connections, weak column-strong beam connections are observed as illustrated in (Photos 1-6)

SOME CONSIDERATION ON THE CONTROL OF ASEISMIC DESIGN AND CONSTRUCTION

The reliable dynamic behaviour of a structure can only be achieved by means of a comprehensive detailed analsyses taking into acount any kind of nonlinearity and interaction features. This complicated analyses, it is obvious that, will have only academic importance. Since other procedures to simplify these anlyses at least for ordinary structures are needed One of the concept adapted at this stage is the ductility concept, which is used either directly or indirectly in the present earthquake codes to reflect the nonlineority of the material.

Ductility concept will be applicable if the elastic and inelastic either the displacements experienced or energies dissipated by the structure are equal whehever this is a valid concept one can reduce the actual earthquake forces by the ductility ratio which is easily obtained in the order of 4-6 to find the design load of an elastic simplified analysis which is prefered.

Many **rc** frame buildings which were heavily damaged or collapsed during the Erzincan Eartquake, March 13, 1992, as well as Leninakan Eartquake, December, 7, 1988 [6] and also during other earthquakes [1], [2] are designed practically using the ductility concept.

The poor behavior of these buildings suffered heavy damages can not be explained by means of mulpracticing only but the validity of the ductility concept should also be checked. Because some of the damaged building frames were good in quality and strength point of views.

The seismic ground acceleration to be used in design is taken generally as 0.1 g for the most hazardaus zones. As it is recorded in Erzincan 92 Earthquake the maximum acceleration is around 0.4g-0.5g. which is 4 or 5 times bigger than design accelerations. The maximum acceleration corresponding to the MSK İntensity is also around 0.2 g to 0.4 g and bigger than the design accelerations.

The preliminary time-history analysis of the inelastic response using 2 horizontal components of the strong motion Erzincan-92 earthquake demonstrated that linear and non-linear response maximum displacements are almost equal for EW-component but for the NS component they differ several times. This result is related to a bilinear hysteretic system having an initial natural period 0.3 sec. The results of these investigations for different systems will be published in a separate article.

Cracks and other type of damages in rc frames are almost inevitable during a severe earthquake. They are even desirable from optimization point of view [4], and they can be taken into account through the ductility coefficient whenever it is applicable. Due to the allowable plastic deformations the rigidity of the frames decreases and the natural periods increase. This is another reason to change the seismic forces imparted to the structure and the displacement expectations either.

Fundamental periods of two sets of buildings measured in two directions before and after the earthquakes of Erzincan 83 and Leninakan

88 and their ratios are given in Table 3 where one can observe that the natural periods of some rc frame buildings changed around 3 times. If this is the case not only the acceleration but the displacements will be affected due to the change of lateral stiffness of the structure especially when thay are exicitated by an earthquake with similar spectral features of Erzincan 92 and Leninaken 88 earthquakes.

It is strongly possible, in this case, to experience bigger displacements than the design displacements which way not be equal to elastic displacements and may be big enough to be considered in equilibrium equations. It means that ductility concept will be no longer valid and the second order effects of axial forces may become important and may govern the behavior and the design of the frame.

Table 3 [8], [6]

Earthquake considered	Buildings	Storey	Fundamental Periods before T_x	T_y	after T_x	T_y	Change
Erzincan 83	Hotel Urartu	4	0.15	0.32	0.59	0.43	3.9-1.3
	Municipality Building	5	0.50	-	0.69	0.45	1.4 -
	Foundation Off. Buil.	4	0.30	0.30	0.39	0.40	1.3-1.3
Leninakan 88	RC Frame	9	0.60		1.80		3.0
	RC Frame&Shear Walls	10	0.95		1.60		1.7
	Lift-Slab Structures	16	0.90		1.34		1.5

The maximum displacements observed in Erzincan 92 Earthquake is around 40-74 for the structures with foundamental period of 1 to 2.5 seconds and 5%damping.

In some cases when the horizontal displacements of the columns are large, only a small part of the columns cross section is capable to resist the compressive forces. Therefore the very high compressive stresses led the concrete crushed and/or the longitudinal reinforcing bars buckled and the frame building collapsed even the P-Δeffects were not critical.

After having investigated the heavy damaged or collapsed structures in Erzincan it can be concluded that the ductility approach may not be enough to provide seismic resistance of **rc** frame buildings during an earthquake. In such cases instead of the ductility concept a compehensive dynamic time-history analysis should be performed taking into account all important aspects of the seismic behaviour of this type of buildings. [7].

It is of practical interest to compare the design seismic loads and accelerations defined in Turkish Seismic Code [3] and the response spectra calculated using the recorded strong motion accelerogram during Erzincan-92 Earthquake A simple numerical example is presented here. If a Structure with a 0.3 sec fundamental period is choosen as an example to compare the earthquake forces imparted to the structure according to the recent quick of Erzincan and the design forces given by Turkish earthquake code one will find that the calculated ratios can be in the order 1.444 g/0.06 g=24 and 0.791 g/0.06 g - 13.2 for EW and NS component respectively (see Table 1). The expected changes in elastic displacement of the same structure can easly be found as 0.135 cm. On the other hand the observed displacements of similar damaged structures are around 70 cm. In other words the necessary ductility ratio to justify the design earthquake loads is much more bigger than 4-6 generally provided by the ordinary structures.

Many **rc** framed structures damaged in different levels and collapsed not only during present Erzincan Earthquake but during the Mexico City earthquakes in 1957, 1962, 1986, Bucharest Earthquake in 1977 and Armenia-Spitac earthquake in 1988, more or less due to the same reasons. All these cities are located on deep alluvial deposits which are able to amplify the base rock acceleration. Just because of this spesific acceleration, the frequency content of the larger seismic design forces are imparted into the structures with relatively long periods. Among those type of flexible structures are so-called "soft first storey structures" which can be observed easily in Erzincan. The design codes should have special provisions to take into account the abovementioned increasaed seismic loads during the design stage of the flexible structures. Codes should also have some clear provisions to restrict of having weak columns and strong beams. One can examplify that an ordinary framed structure with four-five stories can properly be designed accourding to the valid codes with weak column and strong beams which may develop lateral storey mechanisms. It is clear that these kind of flexible structures do have lower safety factors against an expected earthquake. Lateral load capacities of this kind of structures should be increased choosing proper design and detailing techniques. This previously discussed subject may be the topic of another paper.

CONCLUSIONS

In this paper short and preliminary data is given for the March 13, 1992 Erzincan Earthquake and for some of the consequences of it. Brief information on geotectonic, seismologic an dsoil condition is provided. Special attention has bee given to the characteristics of the record recieved in Erzincan and the response spectra based on this record. The reasons of local and global collapses of rc framed structures are discussed and the following conclusions and/or recommendations have been achieved;

i- The intensity of the March 13, 1992 Erzincan Earthquake is high enough to cause important structural damages and the design loads predicted in codes are low and/or the expected ductility is very high so that in can not practically be provided.

ii- Because of the specific amplitude and frequency content of this motion, higher earthquake forces are imparted on the structures which were originally flexible or became flexible after having experienced plastic deformatons, in addition to some structures with short periods.

iii- Low quality concrete, high axial stress, poor design detailing and construction, poor bond between concrete and steel, unsatisfactorly build weak column-strong beams and their connections are several common reasons for charecteristics damages or collapses.

iv- The validity of ductility concept is limited, other simplified approaches are needed to take into account both material and geometrical nonlinearities.

v- Existing rc structures should be checked and strengthened.

vi- The mistakes and their consequences observed in detailing and construction techniques are due to the lack of responsibilities. This may be developed by means of checking mechanisms such as administrative provisions or compulsory insurance regulations etc. And imporevement in code requirements is necessary to prevent any kind of local or global premature collapse and high axial stress consentrations. For these purposes usage of structural walls should be encouradged, new simple forms of structural walls like three layered walls should be tried.

REFERENCES

1. A comprehensive study on Earthquake Disasters in Turkey in view of Seismic Risk Reduction. Edited by Yutaka Okta. Hokkaido University. Spporo. Japan.1983.
2. Yarar R., Earthquake Behaviour of Rural Buildings İn Turkey. Report. İstanbul. 1985.
3. Specifications for structures to be Built in Disaster Areas. Turkish Goverment Ministry of Reconstruction and Resettlement. Earthquake Research Institute. Ankara 1975.
4. Eisenberg J.M., The Control of the Mechanisms of Inelastic Deformations and of Local Damages During Strong Earthquakes. Journal " Structural Mechanics and Design". Moscow. N1. 1986.
5. Karadoğan H.F., "Earthquake Analysis of Geometrically Nonlinear Systems", Cento Scientific Programme, Symposium on Earthquake Engineering. Nov 18-20, 1974. Ankara.
6. Eisenberg J.M., Some Lessons of the Armenia, Earthquake December 7, 1988. Earthquake Engineering. VNIINTPI (in Russian) Moscow, Vol. 2, 1991. PP.2-7.
7. Panahshahi N., A. Reinhorn, S. Kunnath, a.a. Seismic Response of a 1.6 RC Scale-Model structure with Flexible Floor Diaphragms. ACI Structural Journal. Vol.88, No 3, May-June, 1991.
8. Bayülke, N., "Depremler ve Depreme Dayanıklı Betonarme Yapılar", Ocak 1989.
9. İmar İskan Bakanlığı Afet İşleri Genel Müdürlüğü Deprem Araştırma Enstitüsü Ankara.

ÖZET

Bu çalışmada Erzincan ve çevresini sarsan 13 Mart 1992 tarihli deprem ile ilgili olarak derlenen ilk bilgiler sunulmaktadır. Bölgenin jeotektoniği, sismolojik ve zemin koşulları konusunda özet bilgi verilmektedir. Erzincan da alınan deprem kayıtları ve onlardan üretilen spektrum eğrilerinin özellikleri üzerinde durulmaktadır. Çerçeveli betonarme yapıların deprem sırasındaki kötü davranışı süneklik kavramının kullanılmasındaki kısıtlamalar, bazı öneriler ve yönetmeliklerin gözden geçirilme zorunluluklarına işaret edilmekte Erzincan ve benzeri bölgeler için araştırılmasına başlanmış değişik bir yapı sisteminden söz edilmektedir.

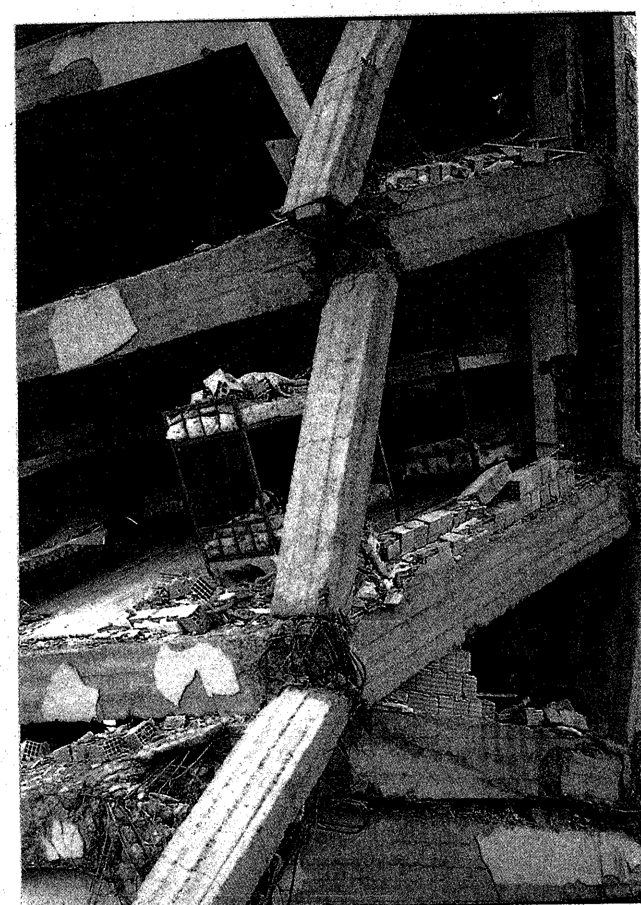

Photo.-1