付属資料Ⅵ　農村建築の震害とその原因

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RURAL BUILDINGS IN TURKEY THAT HAVE SUFFERED DAMAGES IN RECENT EARTHQUAKES AND THEIR MAIN CAUSES

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SUMMARY

In this report,the earthquake behaviour of rural buildings of various materials and structural systems in Turkey is briefly presented.Starting with the classification of rural buildings,and their characteristics, observed earthquake damages and their performances are outlined.Post earthquake repair and strengthening essentials for rural buildings are briefly summarized.

INTRODUCTION

Turkey is one of the countries which frequently experince destructive earthquakes.The buildings and dwellings in rural areas are usually very vulnarable to earthquakes.In this report,earthquake behaviour of rural buildings of various materials and structural systems subjected to recent earthquakes is presented.

In Turkey,the most damaging natural hazard is earthquake.During the last 60 years, 333935 houses damaged by earthquakes which is 65.4% of the total number of houses damaged by naturel hazards.A list of destructive earthquakes which occurred from 1925 to 1984 is given by Table 1.

Most of the lives lost,about 92%,in the past earthquakes which occurred due to the collapse of buildings in rural areas constructed by local materials like stone,adobe and brick and with poor workmanship.

Distribution of population,land area,industry and dams with respect to existing seismic hazard zoning map of Turkey is given in Table 2.

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Table 1.

List of destructive earthquakes which occurred during the last 60 years from 1925 to 1984 in Turkey

No.	Place	Date	Local Time	Ms	I-MAX	Number of Heavy Damage	Number of Death	Death / Heavy Damage
1	Afyon-Dinar	1925. 8. 7	8h46m	5.8	IX	2043	3	0.001
2	Izmir-Torbali	1928. 3.31	2h29m	7.0	IX	2000	50	0.025
3	Sivas-Susehri	1929. 5.18	8h37m	6.1	VIII	1357	64	0.047
4	Denizli-Civril	1933. 7.19	22h07m	5.7	VIII	200	20	0.100
5	Erdek	1935. 1. 4	18h20m	6.7	IX	600	5	0.008
6	Kirsehir	1938. 4.19	12h59m	6.7	IX	3860	149	0.039
7	Izmir-Dikili	1939. 9.22	2h36m	6.5	IX	1235	60	0.049
8	Erzincan	1939.12.26	1 h57m	8.0	X-XI	116720	32962	0.282
9	Kayseri-Develi	1940. 2.20	-	6.7	VIII	530	37	0.070
10	Van-Ercis	1941. 9.10	23h53m	6.0	VIII	600	194	0.323
11	Bigadic-Sindirgi	1942.11.15	19h01m	6.1	VIII	1262	7	0.006
12	Niksar-Erbaa	1942.12.20	16h03m	7.0	IX	32000	3000	0.094
13	Adapazari-Hendek	1943. 6.20	17h32m	6.6	IX	2240	336	0.150
14	Tosya-Ladik	1943.11.26	0h20m	7.2	IX-X	25000	2834	0.113
15	Bolu-Gerede	1944. 2. 1	5h22m	7.4	IX-X	20865	3959	0.190
16	Gediz-Usak	1944. 6.25	6h16m	6.2	VIII	3476	21	0.006
17	Ayvalik-Edremit	1944.10. 6	9h28m	7.0	IX	1158	27	0.023
18	Adana-Ceyhan	1945. 3.20	9h58m	6.0	VIII	650	10	0.015
19	Kadinhan-Ilgin	1946. 2.21	17h43m	5.6	VIII	509	2	0.004
20	Varto-Hinis	1946. 5.31	5h12m	6.0	VIII	1986	839	0.422
21	Karaburun-Izmir	1949. 7.23	17h03m	7.0	IX	865	2	0.002
22	Karliova	1949. 8.17	20h44m	6.7	IX	3000	450	0.150
23	Kursunlu	1951. 8.13	20h33m	6.6	IX	3354	52	0.016
24	Hasankale	1952. 1. 3	8h03m	5.8	VIII	701	133	0.190
25	Yenice-Gonen	1953. 3.18	21h06m	7.4	IX	1750	265	0.151
26	Kursunlu	1953. 9. 7	5h58m	6.4	VIII	230	2	0.009

No.	Place	Date	Local Time	Ms	I-MAX	Number of Heavy Damage	Number of Death	Death / Heavy Damage
27	Soke-Aydin	1955. 7.16	9h07m	7.0	IX	470	23	0.049
28	Eskisehir	1956. 2.20	22h31m	6.4	VIII	1440	1	0.001
29	Fethiye	1957. 4.25	4h25m	7.1	IX	3100	67	0.022
30	Bolu-Abant	1957. 5.26	8h33m	7.1	IX	4200	52	0.006
31	Koycegiz	1959. 4.25	2h26m	6.0	VIII	775	0	0.000
32	Cinarcik	1963. 9.18	18h58m	5.9	VIII	230	1	0.004
33	Malatya	1964. 6.14	14h15m	6.0	VIII	678	8	0.012
34	Manyas	1964.10. 6	16h31m	7.0	IX	5398	23	0.004
35	Denizli-Honaz	1965. 6.13	22h01m	5.7	VIII	488	14	0.030
36	Varto	1966. 3. 7	3h16m	5.6	VIII	1100	14	0.013
37	Varto	1966. 8.19	14h22m	6.9	IX	20007	2394	0.120
38	Adapazari	1967. 7.22	18h56m	7.2	IX	5569	89	0.016
39	Pülümür	1967. 7.26	20h53m	6.2	VIII	1282	97	0.076
40	Amasra-Bartin	1968. 9. 3	10h19m	6.5	VIII	2072	29	0.014
41	Alasehir	1969. 3.28	3h48m	6.6	VIII	3702	41	0.011
42	Gediz	1970. 3.28	2302m	7.2	IX	9452	1086	0.115
43	Burdur	1971. 5.12	8h25m	6.2	VIII	1542	57	0.037
44	Bingol	1971. 5.22	18h45m	6.7	VIII	5617	878	0.156
45	Lice	1975. 9. 6	12h20m	6.7	VIII	8149	2385	0.293
46	Caldiran-Muradiye	1976.11.24	14h22m	7.2	IX	9232	3840	0.416
47	Erzurum-Kars	1983.10.30	7h13m	6.8	VIII	3241	1342	0.356
					TOTAL:	315935	57914	

Table 2.

Distribution of Population,Land Area,Industry and Hydraulic Dams With Respect to the Seismic Hazard Zones

Earthquake zone	Population (Percent)	Surface area (Percent)	Big.Industrial Centers (Percent)	Hydraulic dams (Percent)
First degree H.Z I =IX	22	14.8	24.7	10.4
Second degree H.Z I = VIII	29	28.4	48.8	20.8
Third degree H.Z I = VII	24	28.8	12.0	33.3
Fourth degree H.Z I = VI	20	19.4	12.6	27.1
No Hazard Zone I < V	5	8.6	1.7	8.4

Seismic vulnerability of the Turkish rural building stock and seismic risk for rural buildings in Turkey are given in Figures 1 and 2.

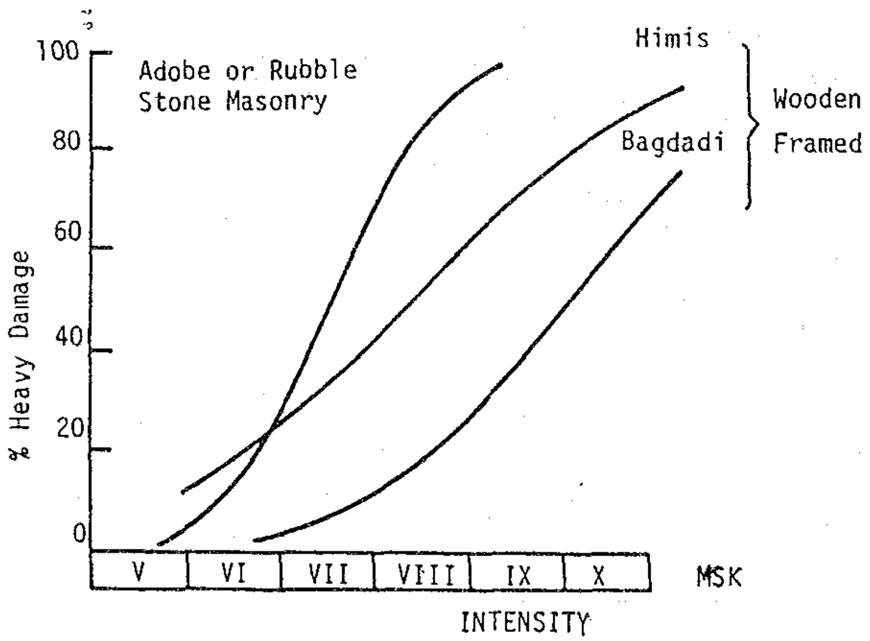


FIGURE 2. SEISMIC VULNERABILITY OF THE TURKISH RURAL BUILDING STOCK

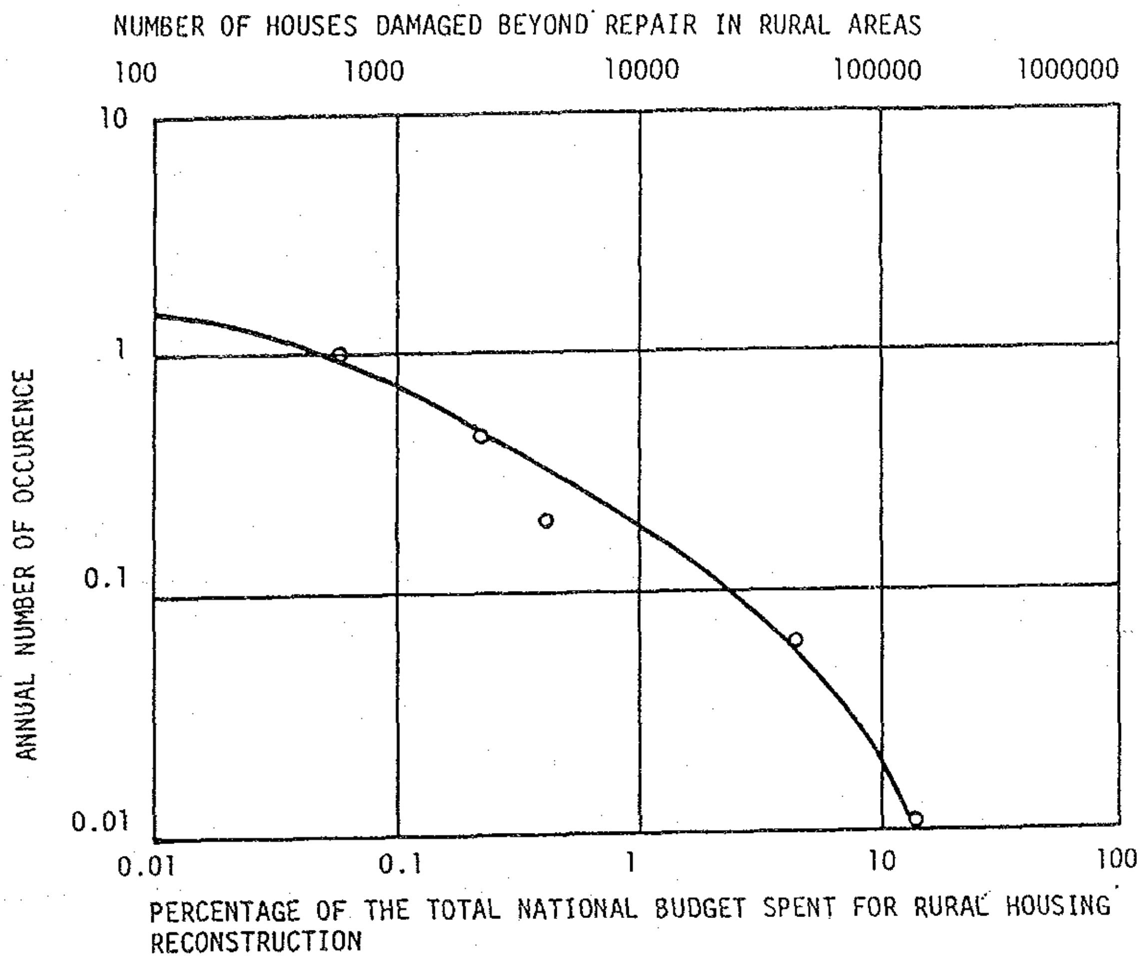


FIGURE 1. SEISMIC RISK FOR RURAL BUILDINGS IN TURKEY

CLASSIFICATION OF RURAL BUILDINGS IN TURKEY

Rural buildings are considered to be the ones which are built in rural or urban areas with local materials,local workmanship and tradition without any engineering input.Since this buildings are not designed to resist earthquake loads,the danages due to the earthquakes increase to larger amounts .

The formation of rural dwellings are effected by the physical surroundings(climate-water resources),the distance to cities and to the production centers of construction materials and transportation facilities, the socio-economic conditions, and the conformity with the traditional structural discipline.

In view of their structural systems,and construction materials rural buildings in Turkey can be divided into two main groups,as block structures and framed structures.

I-Block structures can be considered in two groups.

A - Wood Block Structures:

The walls of wooden block structures are formed from timbers of 20-25 cm of diameter,which are placed on top of each other and connected (nailed) at the corners.This kind of structure has a wide application in the forest houses of Northern Anatolia.

B - Masonry Structures:

These can be subdivided into five groups.

B.1.Stone Masonry Structures

This type of structures in which the walls are made of stones,are generally constructed in the mountainous and rocky regions such as Eastern, South-Western,South and South-Eastern Anatolia.The walls are in general 40-50 cm thick(sometimes 75 cm) and mortar is used between the stones.In the South-Western Regions no mortar is used.The stability of walls are increased by placing horizontal laths(wooden,metal or reinforced concrete) with nearly one meter spacings.

B.2.Adobe Masonry Structures

In general,adobe masonary structures are constructed in regions where timber and stone are scarce.Specially,they exist in the regions of Central and Eastern Anatolia.

B.3.Brick Masonry Structures

In the rural regions near cities,the walls are made of bricks.They are practised mainly in the regions of Marmara,Northern,North-Eastern Anatolia. Iron bars are anchored in brick walls to increase the stability of the walls.

B.4.Other Block Systems

In this case the walls are made of concrete blocks or light weight concrete such as Ytong.

B.5. Mixed System

Combination of the above mention systems are used together to meet the architectural requierements such as window and door oppenings and to increase the stability of the corners.

II.Framed Structures are divided into two depending on the construction material of the frame,such as timber framed and reinforced concrete or steel framed structures.

C.Timber Framed Structures

Depending on the different kinds of filling and covering materials used ,they are subdivided as follows:

C.1.Timber Framed Structures,with filled walls(Himis)

a)Filled with mud and branchs of trees:

Branchs of trees knitted between the vertical elements of the frame,then plastered with mud ,from both sides.

b)Filled with adobe:

The spaces between the vertical elements of the frame are filled with adobe.In Western Anatolia,such as Gediz 70% of the structures are of this type.

c)Filled with stone:

The spaces between the vertical elements oh the frame are filled with stone.Mud can be used as a binding material.

d)Filled with brick:

Bricks are used to fill the spaces between the vertical elements of the frame.Using this system upto three-story hoeses are constructed.

e)Mixed Systems

C.2.Timber Framed Structures with Hollow Walls.

a)Walls covered with wood panels.

In villages near forests,the walls are covered both from the outside and inside with wood panels.

b)Walls covered with lath and plaster("Bagdadi"system).

Stucco is used on wooden laths to cover the spaces between the vertical elements of the frame.

II-Reinforced Concrete System

Very few examples are seen in rural regions.

In Figure 3, the classification of low cost rural dwellings on the basis of structural systems and construction materials are outlined and their percentages in the total building stock are given. Approximately 50% of the total building stock in Turkey can be considered as rural buildings.

CHARACTERISTICS OF RURAL BUILDINGS

1. Adobe Masonary Buildings

In Turkey the most common application of earth as a building material is in the form of unburned clay bricks used in adobe construction. Inlimited applications, walls are built by molding of the clayey earth mortar in 0.8 to 1 meter thick continuous horizontal courses successively. Other applications include the use of adobe block as infill material in wood framed structures. The distribution of adobe buildings in the seismic hazard zones of Turkey is given in Table 3.

Table 3.
Distribution of Adobe Buildings in Seismic Hazard Zones of Turkey

Seismic Regions	Number of Adobe Buildings	Percentage of Adobe Buildings
1. (I > IX MSK)	270000	24
2. (I = VIII MSK)	150000	14
3. (I = VII MSK)	330000	30
4. (I = VI MSK)	230000	21
5. (I < V MSK)	120000	11

Most of traditional adobe houses are single story-structures with a mudmortar stone masonry foundation and a heavy earth roof supported by unsawed logs. Especially in seismic regions, the use of wooden tie-beams under and above the windows and under the roof is common. About 60% of the total adobe building stock have heavy flat earth roofs. The earth roof may have dead weight upto 1.0 ton/m^2.

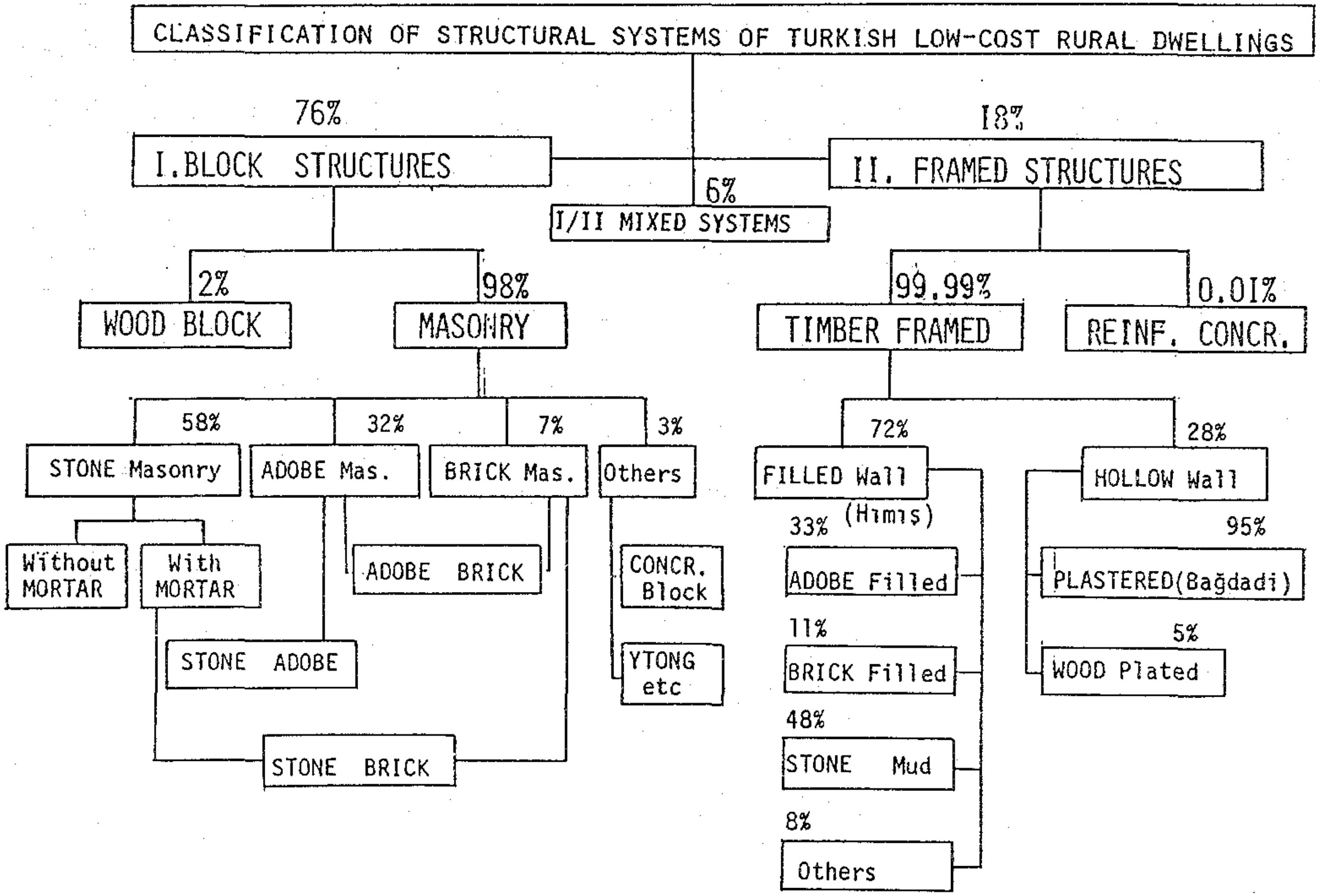


FIGURE 3. CLASSIFICATION OF STRUCTURAL SYSTEMS OF TURKISH LOW COST RURAL DWELLINGS

Compered to existing Turkish design spesifications of adobe structures, the most common errors and deviations encountered in practice are as follows.

-Limited capacity and insufficient bearing area of the roof beams on the walls.

-Increased thickness of the earth layer on the roof.

-Improperly tied at the corners.

-Non-existence or insufficiency of the horizontal tie-beams.

-Too many openings in the walls.

-Poor masonry practises.

-Poor water insulation.

-Poor quality adobe blocks.

2.Stone Masonry Buidings

This type of structures represents a substantial section of the rural housing.They show the poorest earthquake resistance among the building stock,they have been the main source of casualties after earthquakes.

Through the assesment studies on these types buildings,they can be grouped into three as follows.

a-Poor Quality Rural Stone Masonry Structures.

These structures made of rubble or very poorly and randomly sized stone as building material and mud as binding material.The wooden tie-beams do not exist or they are located in the right places.Almost all of these types of structures are single story and have simple rectangular plan with spans and openings.They have flat roofs made of unsawed logs and layer of 30 cm to 50 cm earth.Statistics show that 30-40 lives are lost for each 100 of such houses heavily damaged or collapsed.

b-Medium Quality Rural Stone Masonry Structures.

These are one or two story buildings having simple rectangular plans with small spans and openings.Better qualities of masonry units.Stones are shaped to make angular blocks to fit together.Lime or cement-sand mortar is used.Flat and timber framed roofs with tile or tin corrugated sheet covering are used depending on the climatic conditions and/or traditions.According to statistics,10-15 lives are lost for each 100 of such houses heavily damaged or collapsed.

C-Good Quality Rural Stone Masonry Structures.

These structures have walls made of cut and shaped rectangular stone blocks laid as stable units in proper courses and good quality cement mortar is used as bindings material.Reinforced concrete tie or ring beams are used at the foundation,lintel and roof levels.These structures have suffered little damage consisting of some wall cracking and corner failures even in the epicentral areas of large earthquakes in Turkey.

3-Brick Masonry Buildings.

This type of rural buildings have been widely used in the large cities and outskirts of these cities.Many public buildings such as school buildings ,health centers,etc...built by government are of this type.

In general the brick masonry buildings have stone foundations,their walls are built from brick,and have timber roof trusses with reinforced concrete floor slabs and ceilings.

4-Timber Frame Buildings.

Timber frame buildings have been widely used through the Turkish history.The traditional construction techniques used in timber frame buildings have been very successful.In contrast to stone masonry buildings, they may adequately resist to earthquake forces.But they are susceptible to fires after earthquakes.

OBSERVED EARTHQUAKE DAMAGES AND PERFORMANCES OF RURAL BUILDINGS

Damage statistics of rural structures related to earthquakes since 1966 are shown in Table 3.Only the intensity areas of VIII MSK and larger have been considered.

Table 3.

Earthquake Damage Statistics of Rural Structures.

August 19,1966 Varto Earthquake (MS=6.9,I = IX MSK)

T.S.	T.P.	C.P.	M.D.P.	L.D.P.
Stone Masonry	85%	75%	20%	5%
Adobe Masonry	10%	50%	40%	10%
Other	5%	70%	25%	5%

July 22,1967 Adapazari Earthquake (MS=7.2, I = IX MSK)

T.S.	T.P.	C.P.	M.D.P.	L.D.P.
Wooden Frame	85%	75%	30%	15%
Adobe Masonry	5%	55%	20%	15%
Other	10%	20%	25%	55%

March 28,1970 Gediz Earthquake (MS=7.3,I = IX MSK)

T.S.	T.P.	C.P.	M.D.P.	L.D.P.
Wooden Frame(Himis)	40%	80%	18%	2%
Adobe Frame(Bagdadi)	50%	10%	40%	50%
Adobe Masonry	5%	30%	50%	20%
Other	5%	40%	35%	25%

May 12,1971 Burdur Earthquake (MS=6.2, I =VIII MSK)

T.S.	T.P.	C.P.	M.D.P.	L.D.P
Stone Masonry	40%	55%	30%	15%
Adobe Masonry	50%	35%	35%	30%
Other	5%	40%	40%	20%

May 22,1971 Burdur Earthquake (MS=6.9, I = VIII MSK)

T.S.	T.P.	C.P.	M.D.P.	L.D.P.
Stone Masonry	80%	75%	20%	5%
Adobe Masonry	15%	40%	50%	10%
Other	5%	40%	45%	15%

September 6, 1975 Lice Earthquake (MS = 6.9, I =VIII MSK)

T.S.	T.P.	C.P.	M.D.P	L.D.P.
Stone Masonry	85%	65%	30%	5%
Adobe Masonry	10%	50%	40%	10%
Other	5%	-	60%	40%

November 24,1976 Cald•"ran Earthquake (MS = 7.2 , I = IX MSK)

T.S.	T.P.	C.P.	M.D.P	L.D.P.
Stone Masonry	75%	75%	15%	10%
Adobe Masonry	20%	60%	30%	10%
Other	5%	25%	25%	50%

October 30,Erzurum Earthquake (MS= 7.1, I = VIII MSK)

T.S.	T.P.	C.P.	M.D.P	L.D.P.
Stone Masonry	95%	50%	30%	20%
Adobe Masonry	2%	30%	40%	30%
Other	3%	20%	30%	50%

Legend :
T.S. : Type of structure
T.P. : Percentage of the Type in I>VIII MSK area
C.P. : Percentage of Heavily Damaged or Collapsed Structures
M.D.P. : Percentage of Medium Damaged Structures
L.D.P. : Percentage of Lightly Damaged Structures

Damage statistics within approximately a 20 kilometer radius of epicenter are given in Table 4.

Table 4.

Damage Statistics in the Epicentral Area of Earthquakes

MS	Structural Classification	Percentage of Collapsed Structures	Percentage of Structures Beyond Repair	Percentage of Structures Medium or Light Damage
>	A	80	20	-
6.9	B	50	30	20
6.8	A	50	30	20
6.3	B	20	30	50
6.2	A	20	30	40
5.7	B	5	15	80
5.6	A	10	20	70
4.9	B	-	10	90
4.8	B	-	5	95

A : The adobe and mud-mortar stone masonry structures with heavy earth roofs with no tie-beams.

B : Stone masonry strutures with lime or cement mortar and adobe structures both of them with thin earth or tiled timber roofs.

The earthquake performance of adobe and stone masonry structures are mainly controlled by the weight of the roof and the presence of well constructed tie-beams.

The factors affecting the earthquake performance of adobe masonry buildings can be summarized as:

-Limited bearing area of the roof beams subjected to heavy roof,loads creating bearing and shear failure of adobe walls underneath.

-Insufficient connection of roof beams to the adobe walls.The roof can not act as a rigid diaphragm and allows differential movement among walls leading to the collapse of the roof and/or walls.Out of plane walls behave like cantilever beams.

-Excessive wall openings leaving insufficient wall areas to resist lateral shear ,leading to shear(diagonal tension) failures.

-Lack or improrer construction of tie-beams and/or poor masonry practice of the corner connections of the walls causing seperation of walls at the corners.

-Lack of structural symmetry either in plan or in the amount of openings.

-Deficient bedding of lintels above windows.

-Openings too close to the corners.

-Walls too high or too long between supports.

The factors affecting the *earthquake performance of rural stone* masonry buildings can be listed as.

-Disintegration of the whole structure due to the very poor mortar quality.

-Insufficient connection of roof beams to the bearing walls.The roof can not act as a rigid diagram and allows for differantial movement and contilevered plate action among walls leading to the collapse of the walls and the roof.

-Independently behaving double skinned bearing walls.The walls can not act as one single unit since the inner and outerskins do not have any or enough transwerse connection.At the best the outer skin collapsed outwards due to the plate action.At the worst whole wall and the roof collapses.

-Lack or improrer construction of tie-beams.The in-plane behviour of the walls does not have any ductility.The diagonal tension craks can start and grow very easily leading to seperation and collapse.

-Lack or improrer construction between joining walls.Walls behave independently with out-of-plane cantilever action leading to corner seperation and to eventual collapse.

-Excessive wall openings leaving insufficient wall areas to resist lateral shear and therby causing diagonal-tension type failures.

Earthquake damage statistics for rural masonry buildings are presented in Table 5.

Table 5.

Earthquake Damage Statistics for Rural Stone Masonry Structures.

*Poor Quality Rural Stone Masonry Structures

Damage Classification	Intensity(MSK) V	VI	VII	VIII	IX and Larger
Total Collapse	% 5	%10	%15	%30	%75
Heavy Damage	%10	%20	%30	%40	%20
No or Repairable Damage	%85	%70	%55	%30	% 5

*Medium Quality Rural Stone Masonry Structures

Damage Classification	Intensity (MSK) V	VI	VII	VIII	IX and Larger
Total Collapse	-	-	% 5	%10	%35
Heavy Damage	% 2	% 5	%20	%30	%45
No or Repairable Damage	%98	%95	%75	%60	%20

*Good Quality Rural Stone Masonry Structures
(Satifying Earthquake Code Reqierements)

Damage Classification	V	VI	VII	VIII	IX and Larger
Total Collapse	-	-	-	% 2	%25
Heavy Damage	-	-	%10	%15	%50
No or Repairable Damage	%100	%100	%90	%83	%25

The damage pattern of brick masonry buildings show some differences from the other masonry buildings.Different crack patterns can be observed as shown in Figures 4,5 and 6.

The timber frame buildings show different earthquake performance than than stone masonry buildings.This type of construction in somewhat diminishing due to the fire hazards and earthquake hazards.It is observed that "bagdadi" type houses resist earthquakes than "himis" houses.

CONCLUSIONS

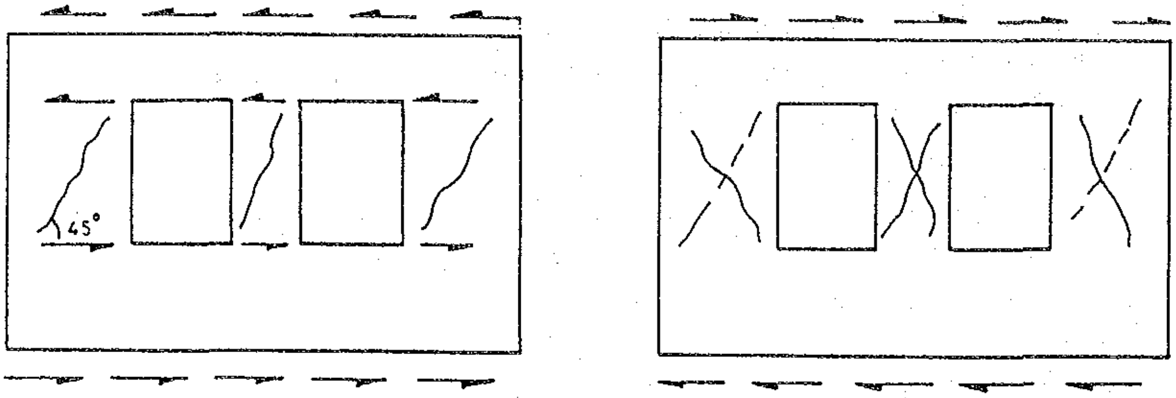
Considering the damage conditions in masonry rural houses in Turkey a rational approach for strengthening or upgrading essentials can be summarized as follows.

i-The integrity of the wall construction should be improved (to prevent failure of individual stones or sections of a wall) independently of the whole wall.

ii-The integrity of the wall-to-wall connections should be increased through realization of a well constructed tie-beams.

iii-The out-of-plane bending strength of the walls should be increased ,so that loads applied perpendicular to them can be transmitted to walls in the plane of these loads.

iv-The roof inertia load should be decreased by decreasing its mass and



(A) (B)

Diagonal Tension Cracking in Brick Walls

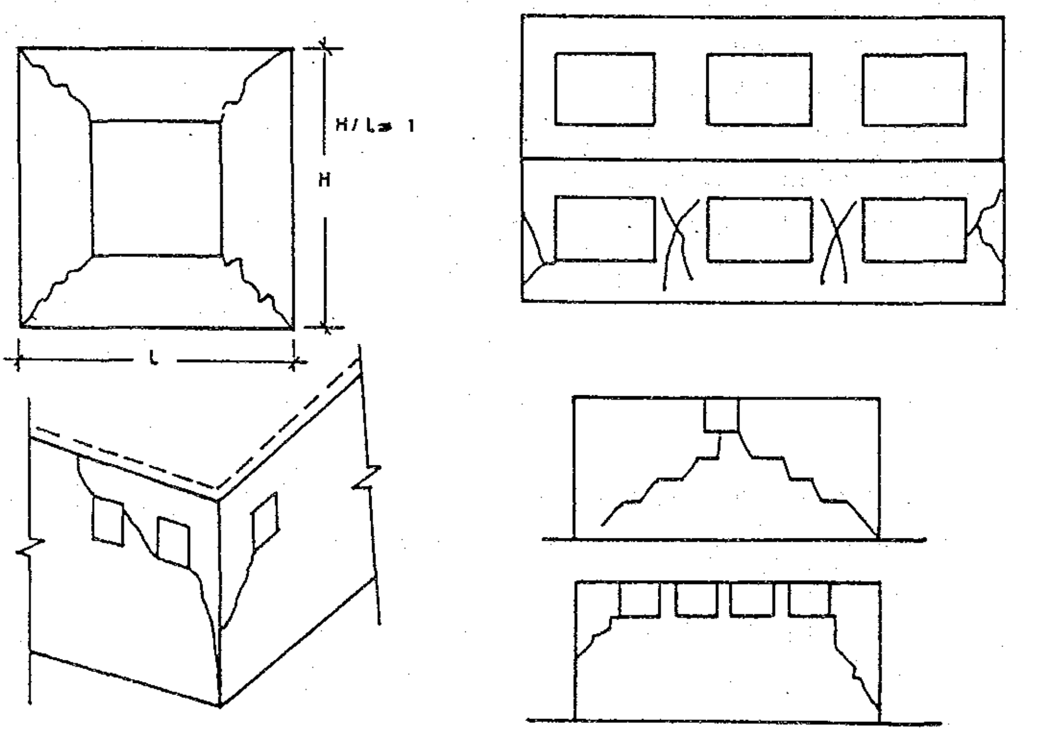
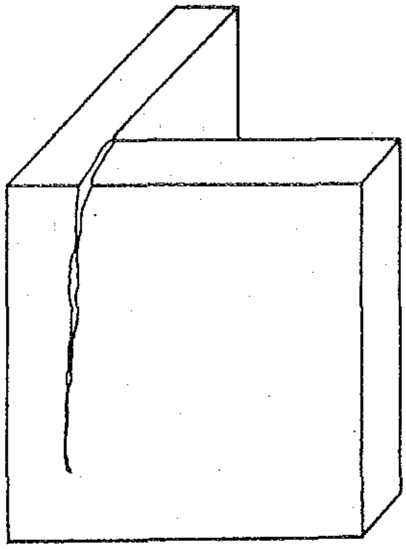
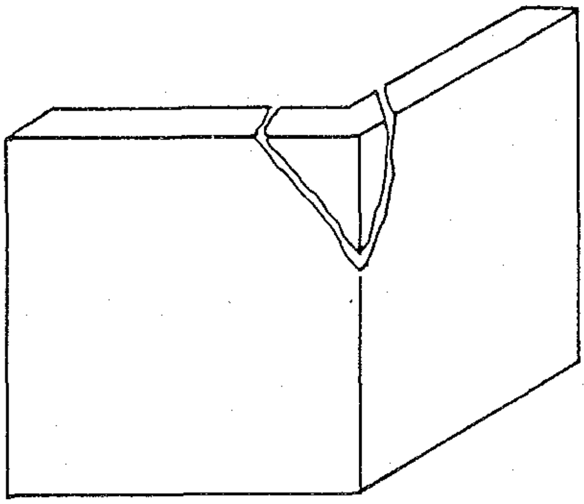


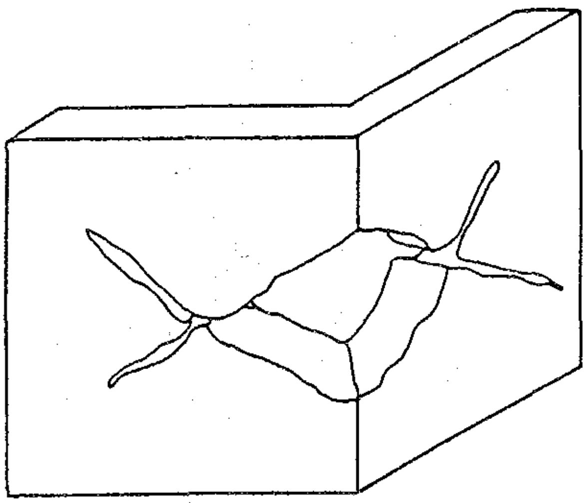
FIGURE 4.Various forms of earthquake crack formation in brick Walls



a) Damage in a Poorly Connected Corner of a Brick Masonry Building

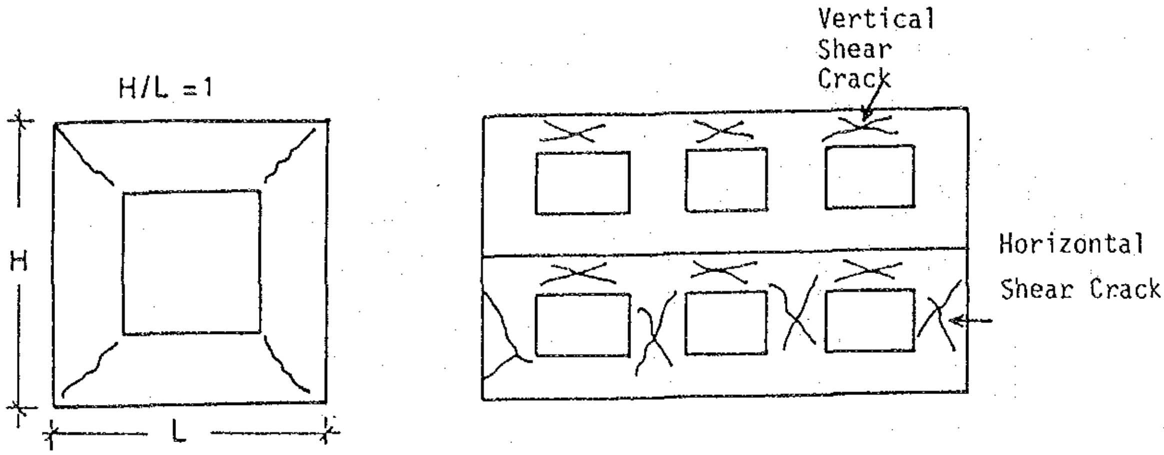


b) Corner Damage in a Masonry Building Having a Flexible Roof System

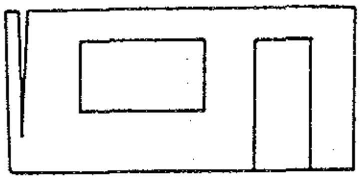


c) Corner Damage in a Brick Masonry Building Due to Large Earthquake Forces

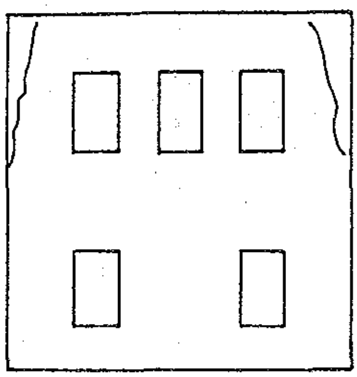
FIGURE 5. Corner damage types in brick masonry building

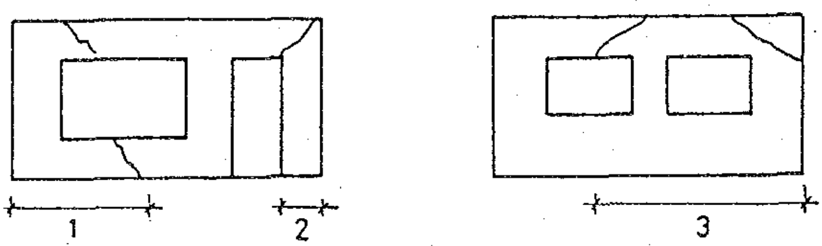


a) Shear Cracking in walls Due to Earthquakes



b) Cracking in walls with
improper Connections





c) Settlement Cracks in Brick Masonry Buildings

FIGURE 6, Cracks in Brick masonry buildings generally reduce the weight of buildings.

v-The roof members should be securely attached to be top of the walls to prevent relative movement.

vi-In some compulsory cases,the sudden collapse of the roof should be avoided through utilisation of columns to support the roof.

Finally,it can be briefly be added some new considerations to accomplish the design procedure to upgrade of a masonry buildings in order to give an example for rural structures or in other words for better understanding stone masonry structures may be classified as box type structures where the primary lateral resistance against earthquakes is provided through the membrane action of the walls and the roof.Thus:

1.The out-of-plane walls should be able to resist their own inertia forces through flexural action as vertical elements.

2.The in-plane walls should be able to resist the lateral inertiaforces developed at the entire roof and that from the top half of the out-of-plane walls through shear action as deep cantilever beams.

3.The roof structure must have sufficient strength and integrity to transmit the lateral inertia forces to the walls appropriately.

Although these conditions can be implemented during the construction phase the same does not hold true for strengthening of an already built one.The total cost of strengthening may even exceed that of newly constructed house,thus making it economically infeasable except for the restoration work of historic monuments.

In summary,for the Turkish case,all activities should be directed to the betterment of the roof structure and avoidance of its premature and birttle collapse during earthquakes for all rural structures within theprimary seismic hazard regions.Once the lives are saved,the rest is the resettlement and reconstruction activities for which spesifically earmarked funds are officially available.

ACKNOWLEDGEMENT

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付属資料Ⅶ　カルディラン地震における建築物の被害

Structural Damage due to the 1976 Çaldıran Earthquake

Nejat Bayülke(*)

1. Introduction

On November 24, the 1976 Çaldıran earthquake occurred near the Turkish-Iranian Border region of Çaldıran, 30 km to the North East of Lake Van. Its Magnitude (M_s) was about 7.1. An important feature of this earthquake was the formation of a ground rupture of nearly 54 km. extending 80 degrees West of North. Nearly ten thousand buildings collapsed or were heavily damaged, causing a total loss of life of 3840 people. This high loss of life was almost completely due to the fact that the local houses had practically no resistance against earthquakes.

The typical masonry houses had rubble stone walls with mud as mortar. They have very heavy flat earth roofs. There were a few reinforced concrete and brick masonry buildings some of which exhibited remarkable behaviour. In this paper emphasis will be on masonry buildings, mud mortar and rubble stone walled and brick masonry with lime and cement mortar walls. First a general description of each type with causes and patterns of damage will be given and followed by a more detailed damage account of a few brick masonry buildings.

2. Typical Local Dwellings

The common type of dwellings in the earthquake region were mud mortar rubble stone masonry houses with flat earth roofs. There were also a few buildings which had adobe walls instead of rubble stone masonry. The walls are constructed with rubble stone collected from fields or river beds. The main rock formation type of the region is of volcanic origin. Thus the rubble stone used in the walls are round stones of andesite and basalt. Since they are collected rather than quarried, they have rounded and moss and dirt covered surfaces which makes them unsuitable for interlocking with each other. Being round and polished or moss covered they slip very easily. The use of mud as mortar was very disastrous as it is seen from the large number of casualties. With time the mud between the stones dries, shrinks and starts falling out, and no bond remains between the stones. At the time of the earthquake rubble stone mud-mortar has a very low shearing strength and this is in the form of dry friction between stones.

Lack of bond stones which extend from one face of the wall to the other face which keep the outer and inner stones and prevent the splitting of the wall into two, is another factor that contributes to the failure of this type of dwelling (Fig. 1).

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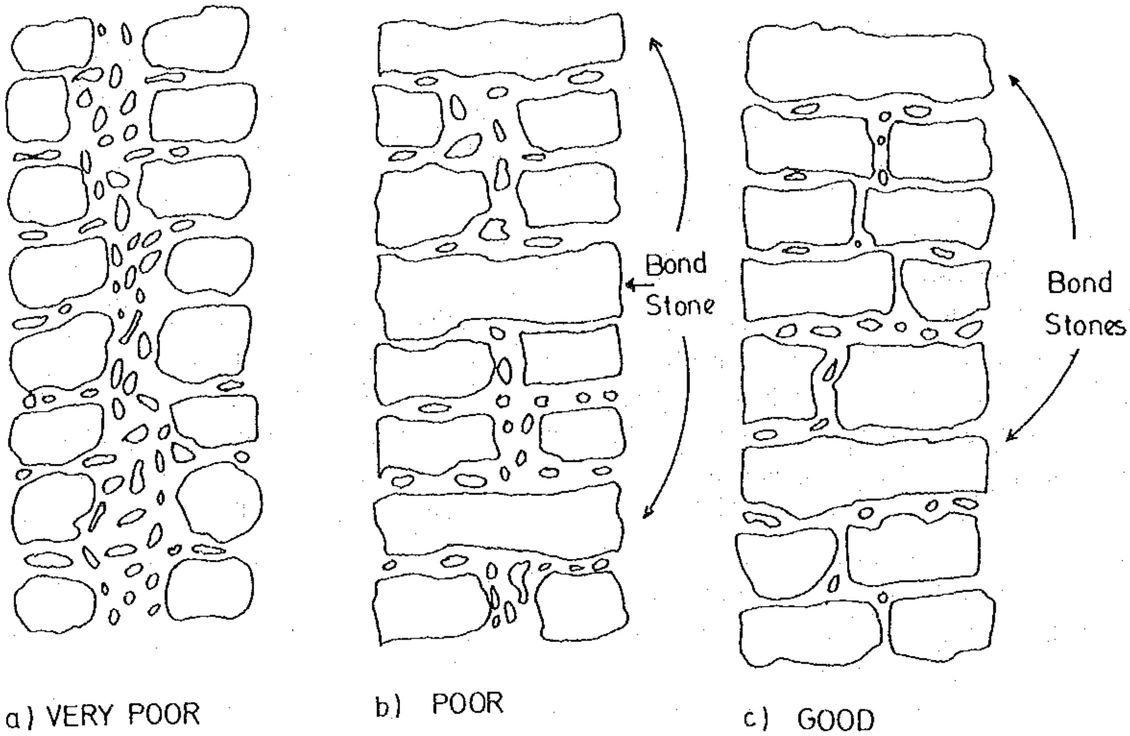

Fig. 1 Typical Rubble Stone Masonry Workmanship in the Region and Suggested Solutions.

Perhaps the factor which is predominantly responsible for the collapse and loss of life in these type of houses is their flat earth roofs. The roof besides being very heavy is also poorly connected to the wall. The thickness of the walls and the flat earth roofs is influenced by the severe winter conditions existing in the region (average monthly temperature in January is -10°C.) and the lack of other practical solutions to the heat insulation problems.

Buildings with adobe block walls generally behave better than the rubble stone masonry houses having the same type of flat thick earth roofs. This may be partly due to the soil being lighter than rock and partly due to the fact that adobe blocks and the mud mortar then they dry up form a homogeneous medium as compared to stone and mud which are two alien materials. When adobe blocks are protected from rain water action, the house has a higher earthquake resistance. This fact was observed in Çaldıran where two adobe houses which were very close to the fault breakage sustained only slight damage (Fig. 2).

There is not adequate quantitative data on the dynamic characteristics of rubble stone and adobe masonry buildings in Turkey. The period of vibration measurements made in two buildings in Muradiye Town, one in adobe building (dimensions 7.5 m×4.5 m height 3.0 m) and one in rubble stone masonry (dimensions 5 m×12 m, height 3.0 m) are shown in Figs. 3 and 4. It appears that these buildings had relatively long periods and damping values for one-story high buildings (Figs. 5 and 6).

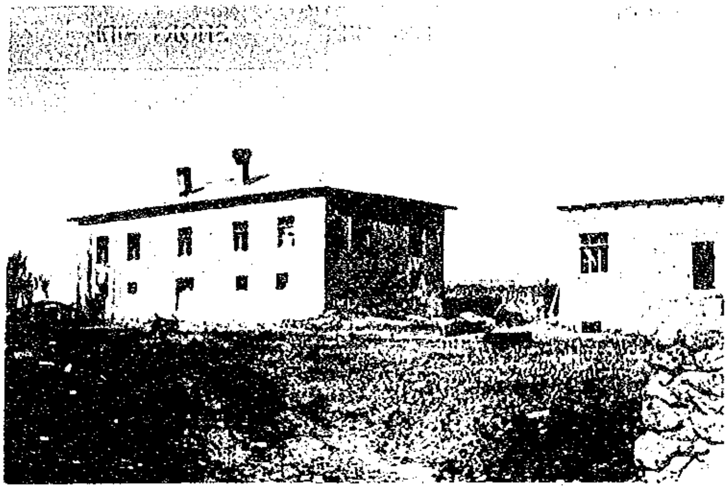

Fig. 2 Undamaged Adobe Houses in Çaldıran.

Fig. 3 Adobe House from Muradiye.

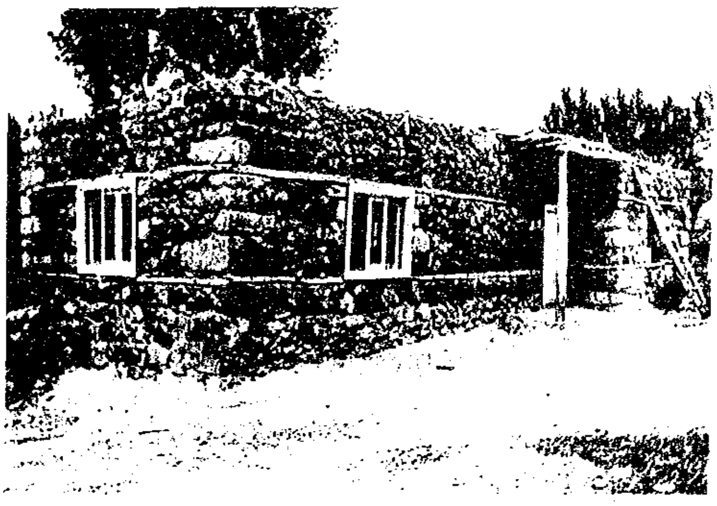

Fig. 4 Rubble Stone Masonry House from Muradiye.

BUILDING	PERIOD (sec.) LONG SIDE	SHORT SIDE	DAMPING (%) LONG SIDE	SHORT SIDE
ADOBE	0.120	0.158	3	5.8
RUBBLE STONE MASONRY	0.140	0.136	–	–

Fig. 5 Period and Damping of Single Story Flat Earth Roofed buildings.

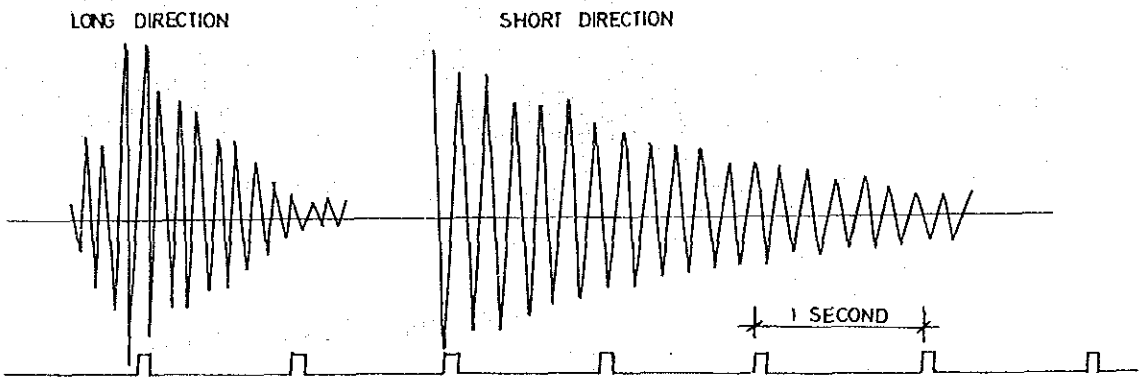

Fig. 6 Forced Vibration Response of the Adobe Building in Figure-3.

3. Roofing Condition

The typical flat earth roof is as shown in Fig. 7. This is a very widely used roofing detail where this type of roofing is adopted. The roof is supported by round tree trunks which were not sawed and which act as roof beams. Their span is usually determined by the size of trees available in the region, but they are rarely more than 4.0 meters. The roof beams are spaced at intervals varying from 25 cm to 50 cm. The beam ends rest on top of the walls only for a depth of 10–15 cm. On top of these beams boards of 2–2.5 cm thickness are placed and are sometimes nailed to the logs acting as beams. On top of these boards tree branches, bushes etc. are placed. The final insulation layer is composed of soil with a thickness sometimes exceeding 50 cm. Such a roofing system usually has a weight of 0.75–1.0 tons/ square meters. Where as an ordinary reinforced concrete slab of 12 cm thickness has a weight of 0.3 tons per square meters.

In order to prevent the rain and snow melt seepage through the earth roof, after every rain, and usually at the start of the rainy season, new finer material is added to the poof and compacted by stone cylinders called 'loğ'. Every earth roof generally has a stone 'loğ' used for compaction of roof soil. Thus over the years the soil layer on top of the roof gets thicker, denser and heavier. With time the roof beams are also affected by moisture, attack of fungi, and creep of timber and start developing large deflections, thus roof beams get weaker. To prevent water accumulation in places where there is large deflections, the roof should be as flat as possible. This is achieved by placing a thicker layer of soil onto the deflected portions of the roof. Thus through these processes with the passage of time the roof gets heavier and the beams weaker.

Another very important weakness of this type of roofing is the connection between the walls and the roof beams. As mentioned before the beam ends rest on the wall for a depth of only 10−15 cm. The only way of transfer of force between the roof and the wall is through friction. If the wall or the roof makes large displacements the roof is highly liable to fall down in the case of an earthquake.

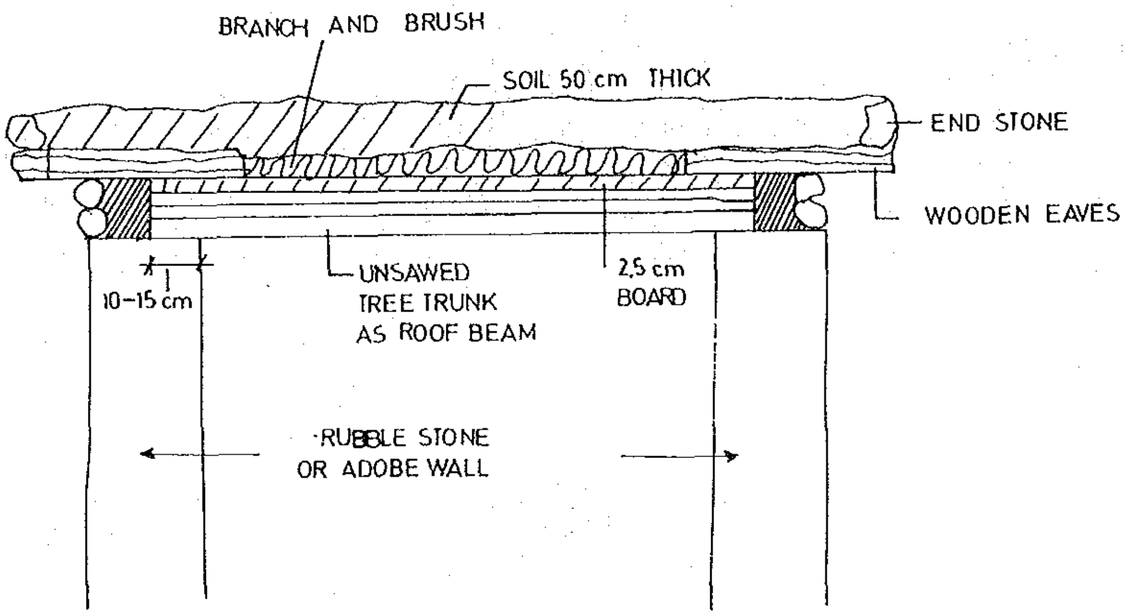

Fig. 7 Typical Flat Earth Roof Cross Section.

4. Failure Mode of Rubble Masonry

Based on observations made in Çaldiran and other latest earthquakes of Turkey, it appears that there are two main factors leading to the damage and total collapse of these type of buidings. One factor is the extreme weakness of walls. When this very weak wall is rapidly destroyed, the heavy roof falls downward killing the people underneath by its weight and by suffocation caused by the thick soil layer on the roof (Fig. 8). Sometimes, although not so often, one end of the roof beams slides from the wall and while one end of the roof falls down first, the other end rests on the opposite wall. This mode of failure is possible when the walls have some strength and the beam ends have a considerably longer support distances on the wall (Fig. 9). This mode of failure of roof is not so very dangerous to the people inside the house.

Rubble stone, mud mortar and flat earth roofed dwellings are usually heavily damaged at intensities of V-VI MSK and are completely destroyed or collapse at intensities of VII-VIII. For this type of building, the intensity step between the initiation of damage and the total destruction is very small. An earthquake of higher intensity occurring in a region where such types of dwellings are common, causes only an increase in the size of the area of total destruction. When there is total destruction of these type of dwellings, it is quite difficult to differentiate higher intensities than say VII MSK, since total destruction can be due to any

Fig. 8 Vertically Collapsed Flat Roof due to the Failure of Supporting Walls.

Fig. 9 Laterally Slided Flat Earth Roof.

intensity greater or equal to VII. Use of other criteria for intensity determination is required.

Damage statistics show that the average loss of life in this type of house is 0.26 person per each heavily damaged or totally collapsed house (Reference-1). In the case of Çaldıran earthquake this ratio approached 0.4 person per destroyed house. Time and season of the earthquake also affects the loss of life. If the earthquake happens in winter or at night, loss of life becomes greater, and this was the reason for the high loss ratio in case of Çaldıran.

5. Improvement of Local Construction

Some improvements to increase the earthquake resistance of these type of houses could be suggested, but they will increase the earthquake resistance only marginally. These improvements could be in the walls and roofing.

In the walls Provisions for the increase of wall shear strength could be taken. As long as mud is used as mortar, the shear strength of the wall can not be improved appreciably.

Fig. 10 Splitted and Collapsed Wall.

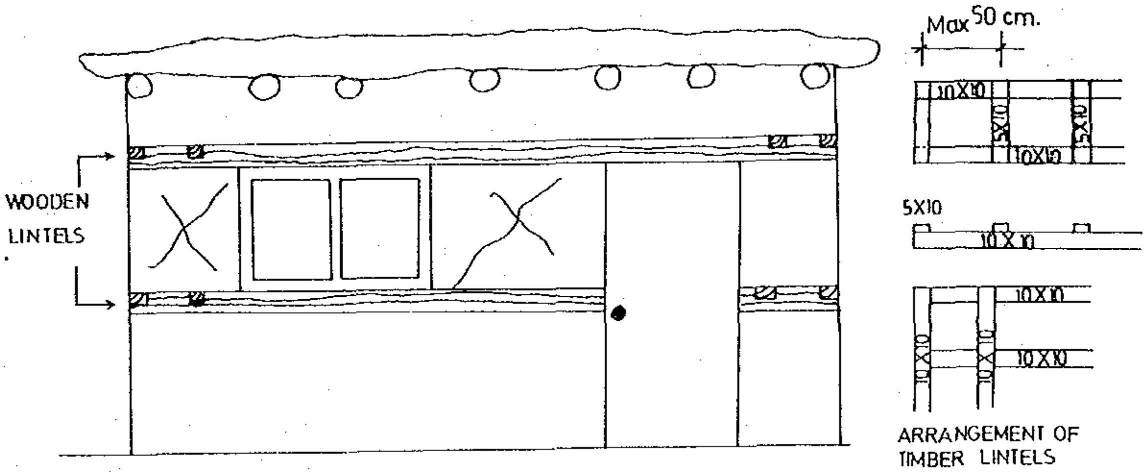

Fig. 11 Wooden Lintels Tozprevent the Extension of Cracks.

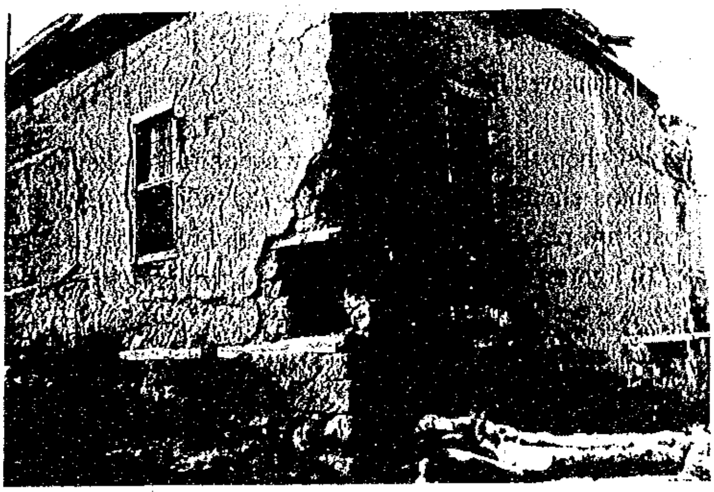

Fig. 12 Lintels to Prevent Corner Damage.

Use of bond stones (Fig. 1) would prevent the splitting of the wall into two (Fig. 10). To prevent the extension and enlargement of shear cracks, provision of continous timber lintels at the top and bottom of window openings would be effective (Fig. 11). At the corners the presence of lintels will prevent or minimize the outward push of crossing walls. This is shown in Fig. 12.Taken in some other place in Eastern Anatolia in an earthquake of intensity V, the presence of timber lintels prevents the extension of damage at the corner. The most effective precaution would be to use cement-lime or at least lime mortars in the wall and to use stones whose largest dimension does not exceed the wall thickness.

In the roof In this respect totally different roofing systems could be employed or efforts could be made to lighten the weight of the flat earth roof. Provision of PVC sheets under the soil layer which is becoming highly popular for this type of roofing, will prevent the moisture attack on timber roof beams and boards as well as making roof impervious. More effective steps could be taken in the connection between the roof beams and walls. To prevent the sliding collapse of roof beams (Fig. 9), the beams should be extended as much as 50 cm outside the walls (Fig. 13). The tree trunks used as roof beams without sawing should at least be given a rectangular shape at their ends and be connected to timber lintels placed on top of the walls (Fig. 14).

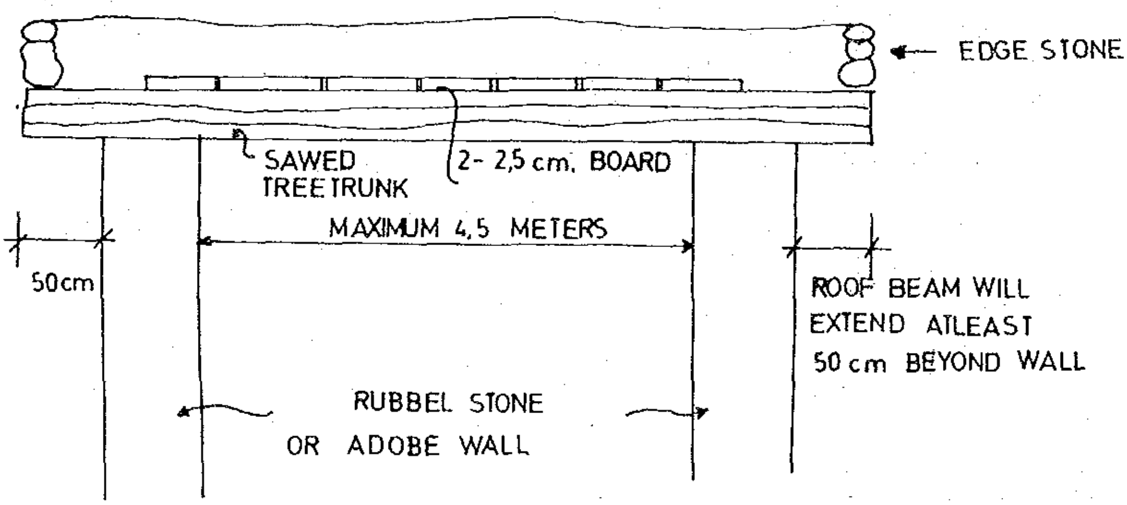

Fig. 13 Improved Flat Earth Roof.

To prevent the collapse of the walls when they are cracked by shear forces generated during earthquakes, special timber frames should be placed inside the window and door openings (Fig. 15). In some cases posts are placed at the corners of the buildings to carry some of the weight of the heavy roof, to lighten the burden of walls and to prevent collapse of heavy roof (Fig. 16).

Completely new roofing system such as timber trussed and sheet metal or tile covered roofs could be encouraged. However the region has no forests. The supply of clay tile requires high fuel consumption and for sheet metal national production of such materials should be increased tremendously, all of which require large nationwide investments. Besides it must be remembered that the use of such a light roofing system would not be efficient against heat losses and light weight expensive insulation materials would be required.

The best solution for decreasing the great loss of life arising from these financially poor houses will be to raise the income of the people in such a way that they will start using better materials (such as cement-lime mortar or steel bars and timber truss and sheet metal covered roofs etc.) and employing better construction technics and systems. The member of people

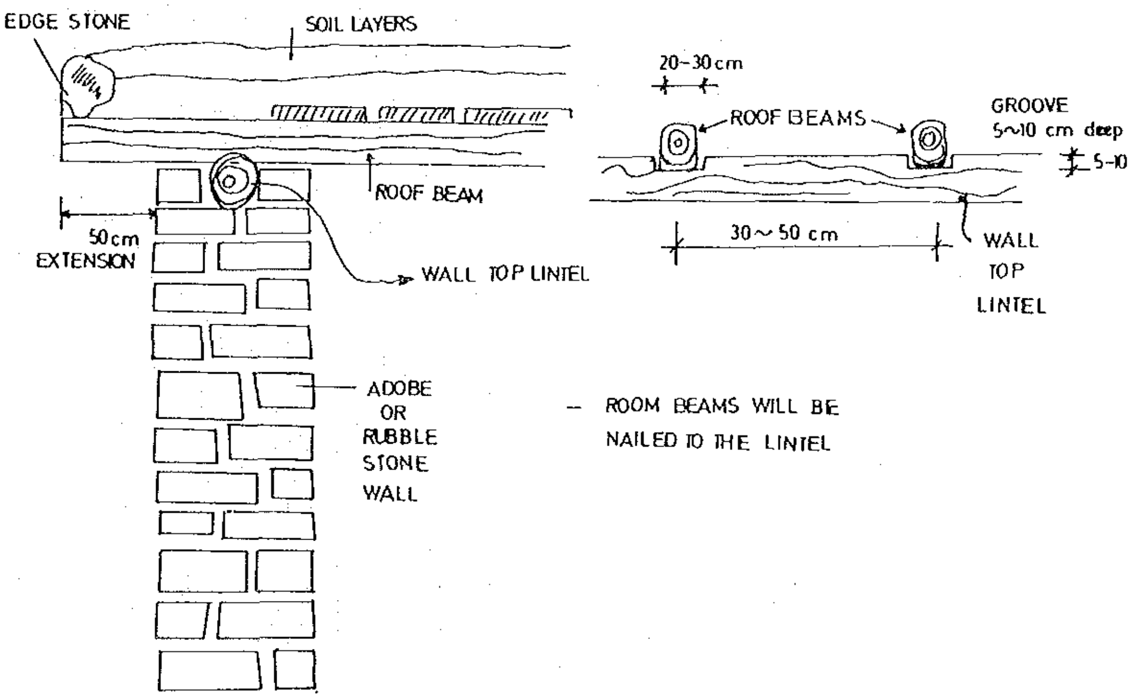

Fig. 14 Improved Roof Beam-Wall Lintel Connection.

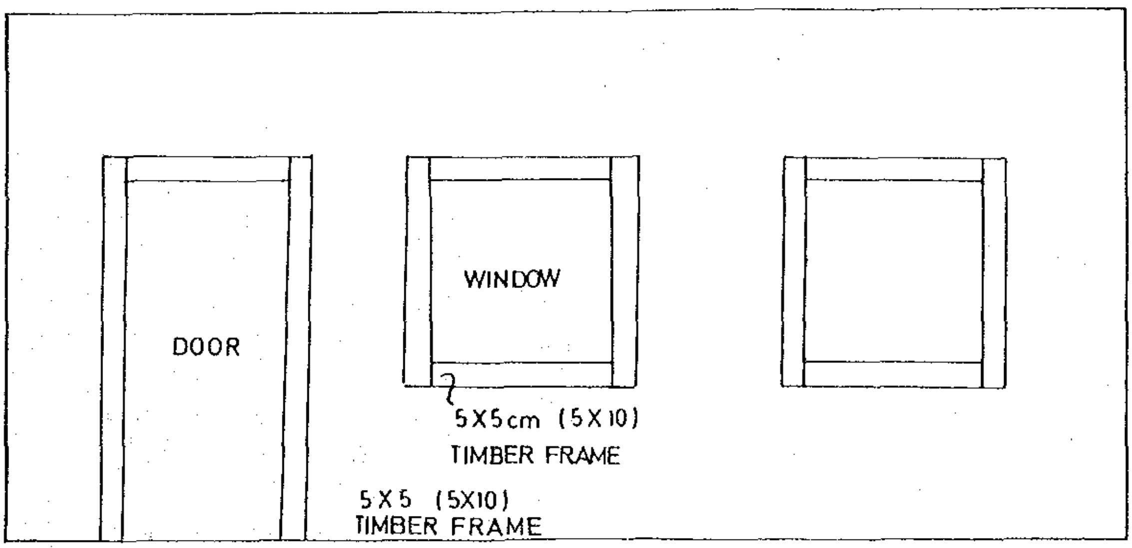

Fig. 15 Timber Frames Inside Window and Door Openings.

being killed in earthquakes decreases in Western Turkey since in that region average per capita income is higher and people can afford for better and safer materials and high costing houses. There is a definite relationship between the income of people and earthquake resistance of the buildings in which they live. So while certain measures could be advised, such as the ones mentioned above, to improve the quality of these existing low income houses for the near future, a main and long term effort should be made to improve the welfare of the people living in the region so that they will build better and more earthquake resistant houses which will eventually minimize the loss of lives. As a side effort, training of qualified workmen and technicians, and establishment of new industries providing cheaper and better construction materials should be encouraged.

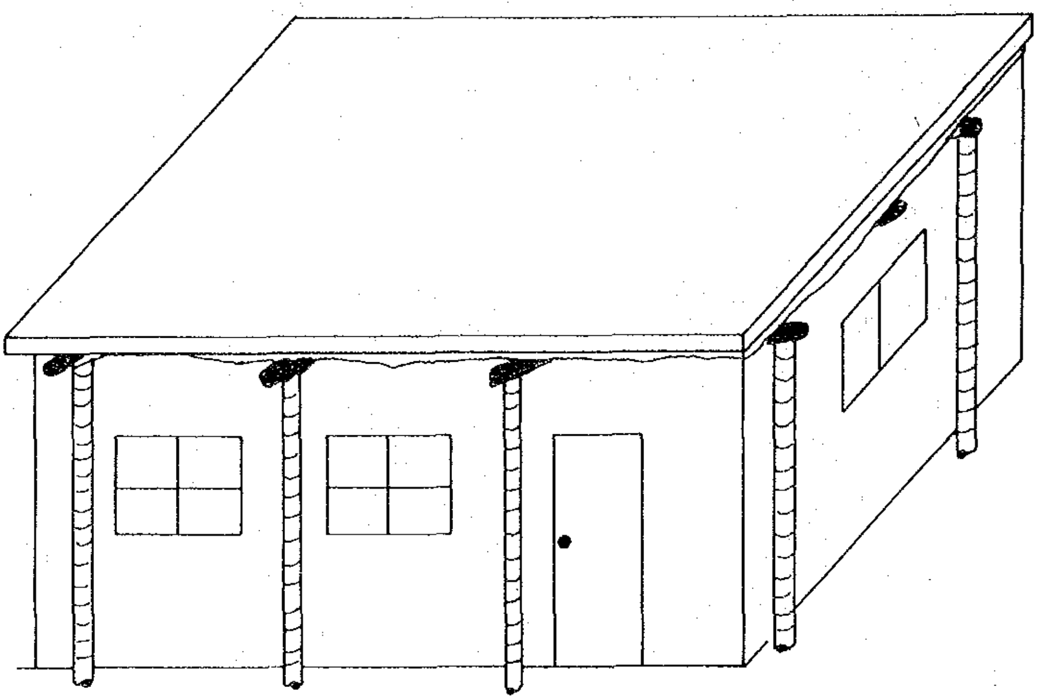

Fig. 16 Roof Beams Supported by Tree Trunks.

6. Engineered Masonry Buildings

In this section the results of damage investigation of several brick masonry buildings will be presented. These buildings had brick masonry walls with cement-lime mortar. They had two or three stories and had reinforced concrete floor slabs. They had been designed as designated 'type' of buildings to be constructed all over Turkey by the civil engineers of Ministry of Public Works. These buildings were particularly studied from the point of establishing and checking two parameters which are considered to be effective in earthquake damage prevention. The floor area/ wall length and window and door opening ratios of three buildings were investigated and were correlated with the degree of damage in the building, and with opening ratio given in the Turkish Earthquake Resistant Design Code (Ref. 5 and also ref. Appendix) which is 0.4. One factor which seemed to be effective in the explanation of the degree of damage was the orientation of the building with respect to the direction of the strong ground motion of the earthquake.

7. The Ground Motion of Çaldıran Earthquake

During November 24, 1976 Çaldıran earthquake a fault trace of nearly 54 km in length was formed. The direction of the fault trace was 80 degrees west northerly, and the slip was right lateral with almost no vertical motion. The ground motion were recorded in Agri (90 km away) and in Van (80 km away) by SMA-1 type accelerographs. However the amplitude of the records were very small, in the order of 5% of g. The earthquake ground motion were also recorded by a WS-100 type seismoscope located in Van PTT Radio-link Station. The seismoscope record (Fig. 17) clearly show a dominant motion roughly in East-Westerly direction which coincides perfectly with the direction of the faulting. If one can assume that the character of the ground motion recorded by the seismoscope is the same as the one which would have been recorded in Çaldıran and Muradiye, then the explanation of the damage to the three buildings which will be made in the following parts could be based on the fact that the weaker direction of these buildings coincided with the strong direction of the earthquake ground motion.

Fig. 17 Seismoscope Record of Çaldıran Earthquake, in Van.

8. Çaldıran Staff Housing (Fig. 18)

This three story brick masonry building had a plan and orientation with respect to the fault as shown in Fig. 19. It is located approximately 300 meters away from the fault.

During the earthquake it was slightly damaged. The dynamical and structural characteristics of this building were as follows :

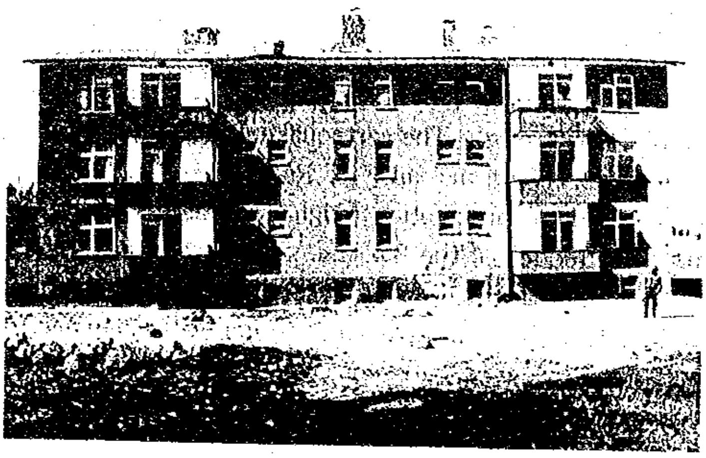

Fig. 18 Çaldıran Staff Housing.

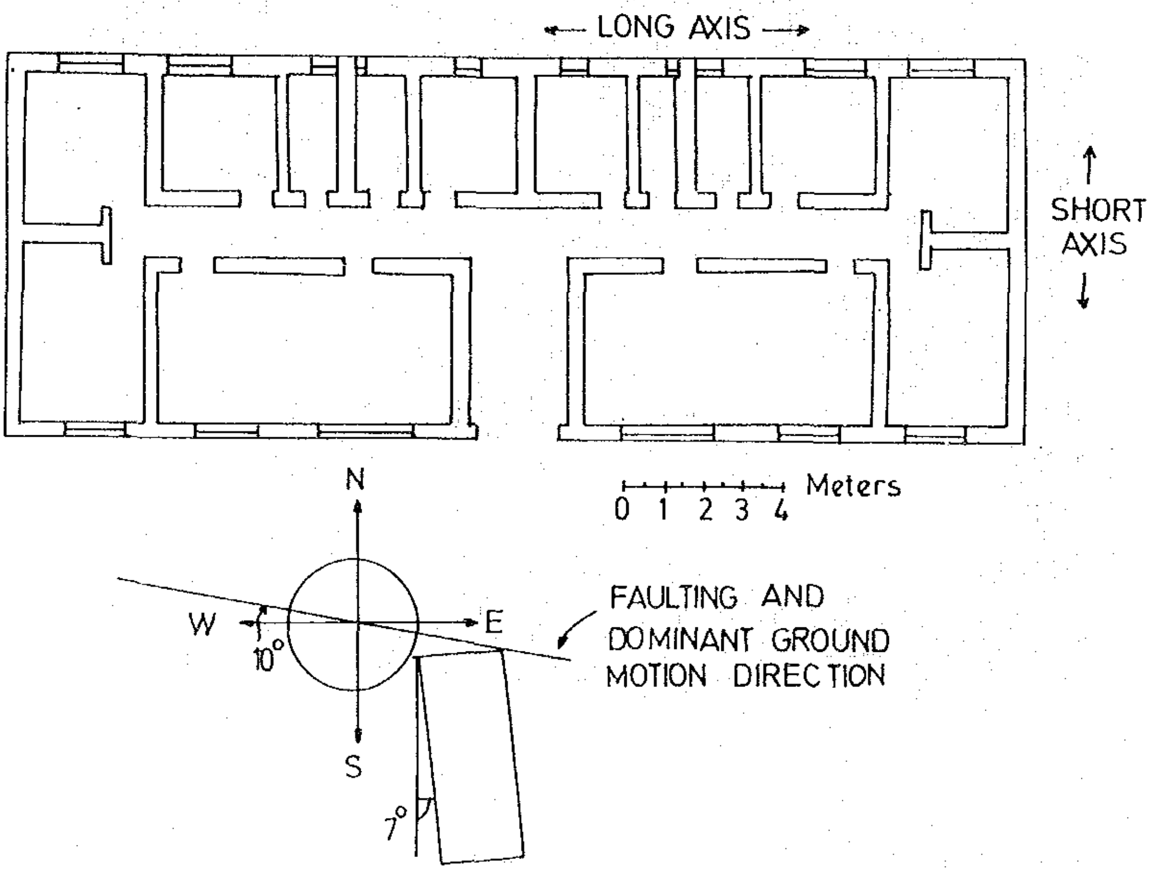

Fig. 19 Plan and Orientation of Çaldıran Staff Housing.

9. Muradiye Staff Housing (Fig. 20)

This was a designated 'type' building constructed all over Turkey. The three story brick masonry building had a different plan and orientation with respect to the Çaldıran fault (Fig. 21). Although it is located in Muradiye town which is approximately 25 km away from Çaldıran town where the intensity was highest (IX MSK). The assigned intensity in Muradiye was only VII MSK (Ref. 1). However this building was more severely damaged than the staff housing in Çaldıran. The structurally 'weak' axis of the building coincided with the direction of the fault in Çaldıran which is taken as the direction of the dominant component of the ground motion. The damage inside the building in the long direction were as severe as the outside (Fig. 22). The dynamical and structural characteristics of this building were as follows :

Fig. 20 Muradiye Staff Housing.

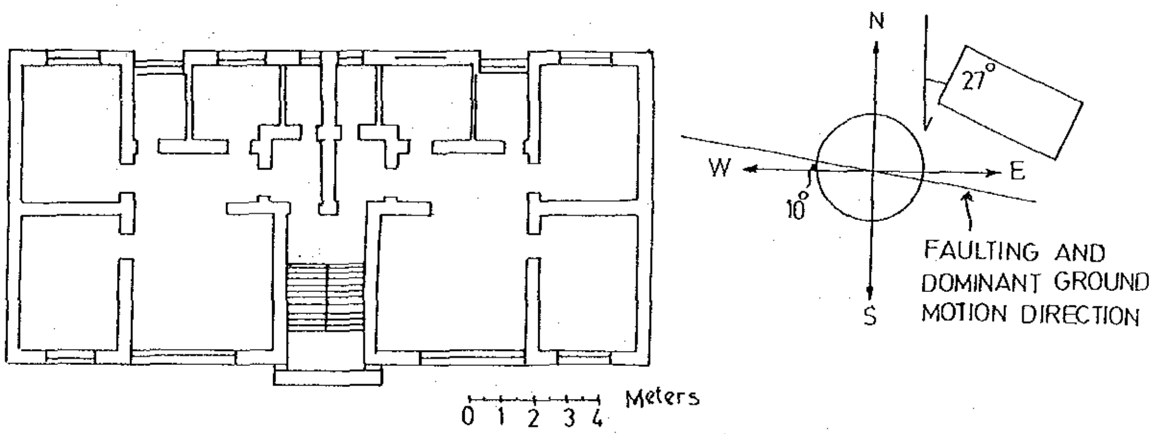

Fig. 21 Plan and Orientation of Muradiye Staff Housing.

Of the two opening ratios given in the long direction, the larger one is for the ground floor and the smaller one is for upper story. The period for undamaged building was measured in an identically the same type building located in Ankara. In the damaged direction of the building (long direction) the vibration period of the building were increased by 3 times due to the degree of damage caused by earthquake.

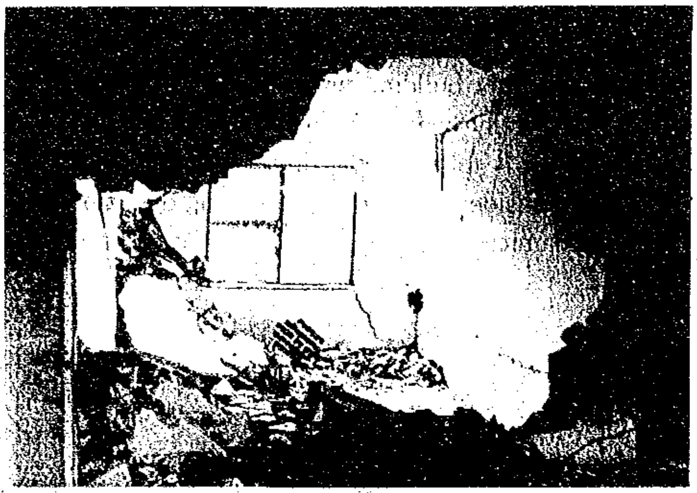

Fig. 22 Inside View of the Muradiye Staff Housing.

10. Muradiye Junior Highschool Annexe

This was another designated 'type' building constructed throughout Turkey (Fig. 23). Its plan and orientation with respect to the fault formed by the November 24, 1976 earthquake is as shown in Fig. 24. This building also sustained considerable damage. The dynamical and structural characteristics of this building is as follows :

It happened that the same type of building had been in the town of Lice and damaged by September 6, 1975 earthquake, magnitude of which was 6.9 (Reference-3). The intensity assigned to town of Lice was Ⅷ MSK. The building in Lice had been more severely damaged than the one in Muradiye. The damaged building in Lice is given in Fig. 25.

Another designated 'type' building present both in Lice and Muradiye were the Gendarmerie buildings (Figs. 26 and 27). As seen while the building in Muradiye had no damage, the same building in Lice was damaged beyond repair. Although the quality of

Fig. 23 Muradiye Junior Highschool Annexe.

construction might have been not the same in both cases, the difference in the level of damage cannot be explained only by construction quality difference. It is mostly a result of different intensities to which the buildings had been subjected.

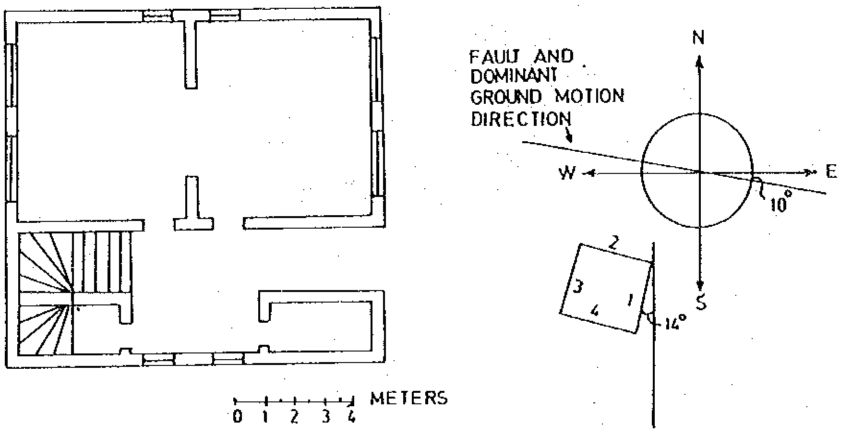

Fig. 24 Plan and Orientation of Muradiye Junior Highschool Annexe.

Fig. 25 Lice Junior Highschool Annexe.

Fig. 26 Muradiye Gendarmerie Building.

Fig. 27 Lice Gendarmerie Building.

11. Conclusion

This earthquake once again showed that poor income buildings have no chance against earthquakes, and their destruction causes a great loss of life. This was not a special earthquake from the point of the damage it caused to regional dwellings and to other buildings and from the size of loss of life.

Within the earthquake region there were quite a number of brick, concrete block and stone masonry buildings built by govenment. With the exception of one collapsed primary school in Çaldıran, none of them caused loss of life, althorgh some of them had heavy damage beyond repair, and damage causing loss of function. Also few locally constructed dwellings survived the earthquake without loss of life because they had light trussed roofing (Fig. 28). These fact point 9ut that use of better construction materials is highly effective in reducing the loss of life if not the damage. Therefore steps must be taken to encourage the people to use better construction materials and practices, and it seems this can be achieved mostly through the improvement of the welfare of the people and they will themselves start building better houses.

Fig. 28 Slightly Damaged Local Dwelling in Muradiye.

The few examples of brick masonry buildings subjected to this earthquake, mentioned above, show that wall opening and floor area/wall length ratios show clearly that they are effective parameters to be employed in the earthquake resistant design of brick masonry buildings.

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付属資料Ⅷ　トルコの建築物の耐震性

EARTHQUAKE PERFORMANCE
OF
BUILDINGS
IN
TURKEY

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September-1985
Ankara-TURKEY

INTRODUCTION

In this study the earthquake behaviour of buildings of various structural systems will be presented. The behaviour of buildings described in this paper are mostly derived from the observations made by the author during more than twenty earthquakes of various intensities which had occured in Turkey within the last fifteen years. Significant structural details of various types of buildings are given first to achieve a better understanding of their earthquake behaviour. Only the type of buildings most commonly built in Turkey are considered. The aim of this paper is to contribute to the studies of expected building damage to be used in earthquake vulnerability studies. The type of buildings covered are reinforced concrete, brick masonry, timber frame, and rubble and adobe masonry. Of these the last two types are built mostly in rural and semi-rural areas, while the others are mostly built in cities and larger towns. Timber frame houses are also built very widely in rural areas of Turkey where forests are abundant.

REINFORCED CONCRETE BUILDINGS AND THEIR EARTHQUAKE BEHAVIOUR

Reinforced concrete buildings are one of the most common construction type in cities. The increaseing cost of land is good incentive to build higher buildings. Since Turkey is an earthquake country the maximum number of stories a birck masonry building can have is limited to two to three stories in allmost 50 % of the country. Thus to make an optimum use of land one has to build high rise buildings and they can be built with only reinforced concrete, steel being too expensive. The characteristics of reinforced concrete buildings in Turkey will be studied under the headings of materiels, workmanship, design and dynamic characteristics.

Materials

The concrete used in allmost all of the reinforced concrete buildings in Turkey has a design cube compressive strength of 160 kgf/cm2. However, this strength is very rarely achieved. There are a number of reasons for the generally poor strength of concrete. The aggregate is mostly unwashed and ungraded riverbed material used es it is. The water cement ratio is usually kept high so as to have good workability in order to have smooth surface appearance and, as a result, some of the cement flows away with water. The concrete is mixed often by hand and after placement it is rarely compacted by vibrators. Curing of concrete after placement is usually observed. Only a few large construction companies do reach the anticipated design strengths. The concrete used in most of the residential buildings is mixed at the job site. Contractors usually do not have sufficient capital to invest in construction machinery and equipment and also labor is cheap and [illegible] is no control of concrete strengths used in a buil[illegible] [illegible] past several investigations were made to dete[illegible] [illegible]ncrete strengths achieved by contractors and the[illegible] were not very promising. Same situation has been ob[illegible] the author who did similar tests. The concrete cylin[illegible] compressive strengths at best is about 100-120 kgf/cm[illegible] is the maximum strength that can be reached under the [illegible]tions described above.

The steel used in the reinforced concrete buildings is denoted as St-I (mild steel) and St-IIIa or b (cold worked or hot rolled high strength alloy steels)' In most of the reinforced concrete apartment houses St-I type is used. This steel has a yield strength of 2.4 ton/cm2 and a strain hardening strength of 3.85 ton/cm2. There may be considerably large variations in yield strengths of steel. The high strength St-III type of steel has a yield strength of 4-4.2 tons/cm2 and it is used in few important government and private construction as main reinforcement or in welded wire fabric used in reinforced concrete slabs, and in shear and retaining walls.

The filler and partition walls are mostly made of clay bricks. Formerly hand made bricks were being used but now factory made hollow bricks with horizontal perforations are used. The compressive strength of solid hand made bricks may be higher than that of factory made hollow bricks. The compressive strength of these bricks may be taken as 20-30 kgf/cm2. In relatively few cases lighweight (gas beton) concrete blocks (YTONG) and cinder blocks are used as filler and partition walls. These are comparatively weaker than clay bricks under compression and shear but they are lighter and, thus, decrease the dead load of buildings.

Workmanship

Workmanship for concrete and formwork is generally poor. The placement of steel and the design details for reinforcements are generally quite improper. Particularly ties are neglected, not all the ties prescribed in the design are placed and they are also placed in wrong places and many of them are not even tied to longitudonal bars with wires. The crowding of ties near column-beam joints is unknown and almost never practiced eventhough it is mentioned in the codes since 1968. The proper development lengths of longitudinal bars are not observed. Due to poor placement of steel it is quite common to observe reinforcing bars not covered by concrete. The columns are plastered and, thus, there is no danger of corrosion. However, concrete with cavities and with very large sized stones is also common resulting in a decrease in strength. The adherence of this usually low strength concrete to steel is also poor.

Design

Up until a few years ago the Turkish Reinforced Concrete Design Code covered only the allowable stress design although the ultimate strength design was also permitted. The ultimate strength design has very recently entered into the curricula of the universities in Turkey. Almost all of the reinforced concrete buildings are designed according to the allowable stress design method. In practice, serious attention is paid to the design of slabs and beams, but columns are designed very superficially. Design against earthquakes is generally not carried out even in large towns and cities which were subjected to severe earthquakes in the past (such as Bolu and Erzincan). During a field work carried out in 1981 in Bolu, it was found out that the municipality which is issuing building permits did not have an engineer to check the designs for earthquake loads. Bolu is a city on North Anatolian Fault with a population of nearly 50 000. While the control organisations lack the necessary personnel to check the design and construction of reinforced concrete buildings, the designers themselves either do not know the proper earthquake resistant design methods or they consider them to be too difficult to handle. In same cases only one frame is analysed eventhough it is not a represantative one. The general superficiality of the designs is due to severe competition of civil engineers to have enough work to maintain their design offices. Since they work at very low rates the designs are not done according to code requirements.

Usual designs result in weak columns and strong girders. The vertical reinforcements in columns are kept to a minimum. The column and beam longitudinal steel percentage is around 0.5-0.8. The axial loads acting on columns are about 35-45 percent of the total axial load carrying capacity. The ties and late[illegible] forcements are ∅ 6 mm bars spaced at 25-30 cm inter[illegible] The allowable stresses for concrete in compressi[illegible] steel in tension are 50-60 kgf/cm2 for a concrete of 160 kgf/cm2 compressive cube strength and 1400 kgf/cm2 for St-I type steel, respectively. They can be increased by 33 percent in case of earthquake loading.

The reinforced concrete slabs are about 10-12 cm thick. Use of filler block joist slabs is also common, in which case the usual slab thickness and beam depth are about 30-32 cm. The use of this kind of flat slabs results in highly flexible buildings against lateral forces. Such buildings are known to have collapsed during the 1967 Adapazarı earthquake and even during construction as experienced in the city of Antakya.

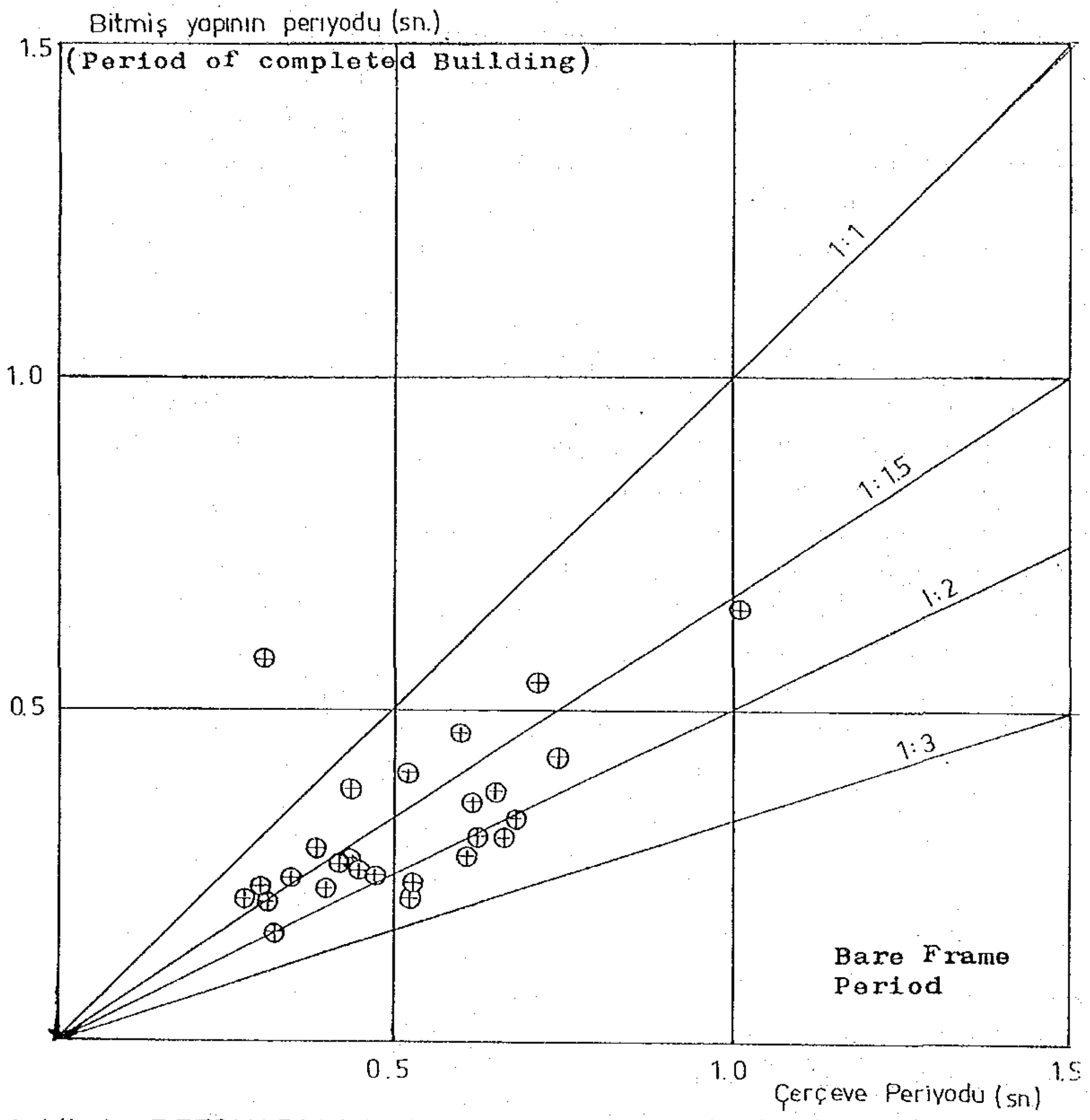
Reinforced concrete buildings are usually of frame type; however during the last 5-6 years, frame buildings have begun to incorporate shear walls as well. This is due to the recent Turkish Earthquake Resistant Design Code accepted in 1975, by which the base shear coefficients were increased from 6 to 10 percent in the 1st degree earthquake zone and 4 to 8 percent in 2nd and 2 to 6 percent in 3rd degree earthquake zones. The increased base shear coefficients and the general increase in the allowable number of stories necessitated the introduction of shear walls to take up the increased lateral forces.

The distance between column axes are usually 3-4 meters. The dead weight of reinforced concrete buildings is about 1-1.2 tons/square meter. Story heights are about 2.80-3.20 meters.

Dynamic Characteristics

The periods of vibration of reinforced concrete buildings in Turkey have been measured very frequently. The well known formula $T = 0.09\, H/\sqrt{D}$ usually gives values smaller than the actually measured values. Reinforced concrete buildings with flat slabs generally have periods about 20 percent higher than those with similar dimensions but with beam-supported slab floor systems do. Also the filler and partition walls greatly alter the dynamic characteristics of the frame at least under low amplitude vibrations. Pure frames, when filled up with brick masonry walls, become more rigid and the period of the completed building decreases about 50 percent as compared the period of the original bare frame. Figure-1.

In 1981 periods of several reinforced concrete buildings in Bolu



Şekil-1 BETONARME YAPILARDA DOLGU DUVARIN PERİYODA ETK

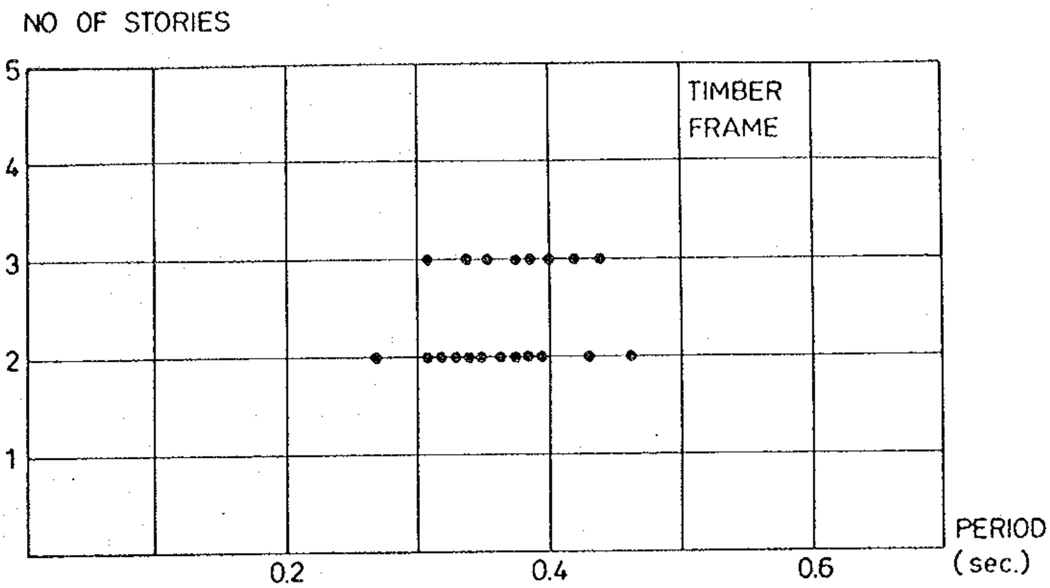
(Influence of Filler Walls on the period of Reinforced Concrete Buildings) were measured. Some of them were bare frames and some were in the completed stage. All data on periods from Bolu are given in Figure-2.There is a large variation in the periods of reinforced concrete buildings. The data show that the period of vibration reinforced concrete buildings is in the range of 0.2-0.5 seconds depending on the number of stories and other internal variations. A few measurments of damping of reinforced concrete buildings under man-excited vibrations showed that damping is about 2-4 percent of the critical damping coefficient. These values of damping and period of vibration should be considered to correspond to the values just at the begining of an earthquake.

Periods of vibration of several buildings in Erzincan had been measured before, after an earthquake damage and after repairs. In the case of an hotel of 4 stories the-pre earthquake period was 0.32 seconds before the earthquake. During the 4.7 magnitude earthquake this building had seperation cracks between its filler walls and the frame and some x-cracking in the walls. After the earthquake its period had been increased to 0.43 seconds. With the plastering of the cracks the period were reduced to 0.37 seconds, apparently there were some fine cracking in the frame as well which had not been repaired. In general the periods of vibration of the buildings were increased by about 5-10 percent by the earthquake damage and the periods were reduced by a somewhat lesser amount with the repairs.

Earthquake Behaviour

A number of indepth earthquake behaviour analysis of reinforced concrete buildings have shown that such buildings designed without any particular concern for earthquake resistant design rules have an equivalent static base shear coefficient of about 0.06 to 0.12. The smaller value usually indicates the lateral static force level at which the first hinging in columns is initiated, while the larger value corresponds to the formation of hinges in almost all of the columns of the building. In other words, the first coefficient can be considered as the elastic limit of the whole building and the larger value as the ultimate lateral force limit.. This could be interpreted as an equal energy ductility factor of about 2.5 (Figure-3). It should be noticed that these are static equivalent base shear coefficients. Those values would probably correspond to a couple of times greater dynamic ground motion peak acceleration values. In the last 10-15 years very few reinforced concrete buildings have actually collapsed in an earthquake in Turkey, while there were many cases of seriously effected reinforced concrete buildings. It seems that the factor which causes exceptionally poor behaviour is the use of low quality and strength concrete which is mostly due to poor workmanship and use of very poor quality aggregates in making concrete, the next important factor being poor structural conception.

A reinforced concrete building designed and constructed as described above would have hairline cracks between its filler walls and frame at an earthquake intensity of V-VI MSK scale. Depending on the type of filler wall material there would be various levels of damage. If the wall material is cinder block or lightweight concrete block it will start to have diagonal tension cracks at this intensity. At a higher intensity earthquake (VII-VIII MSK), the cracks between the frame and the filler wall would be more pronounced and x-cracking would be present in almost all of the ground floor or first story walls. These cracks would reach a few centimeters, and hinging may start in a few columns which would usually be the ones carrying the heaviest vertical axial loads. It is anticipated that most of the reinforced concrete buildings in Turkey would be at a more critical stage when an earthquake of intensity IX-X occurs. The damage at sucn an intensity of shocks would be heavy damage in the filler walls; total hinging at the top and bottom ends ofcolumns, namely the concrete cover would fall down, exposing reinforcements and the longitudinal reinforcements would buckle out, ties would open up. Figure-4 shows the details of this hinging. This damage would be observed at the ground story and would decrease gradually in the upper stories. The intensity of IX-X would damage the building beyond repair, it may even collapse. This anticipated earthquake behaviour should be taken as an average behaviour of reinforced concrete buildings in Turkey. In Figure-5 this damage is presented schematically.



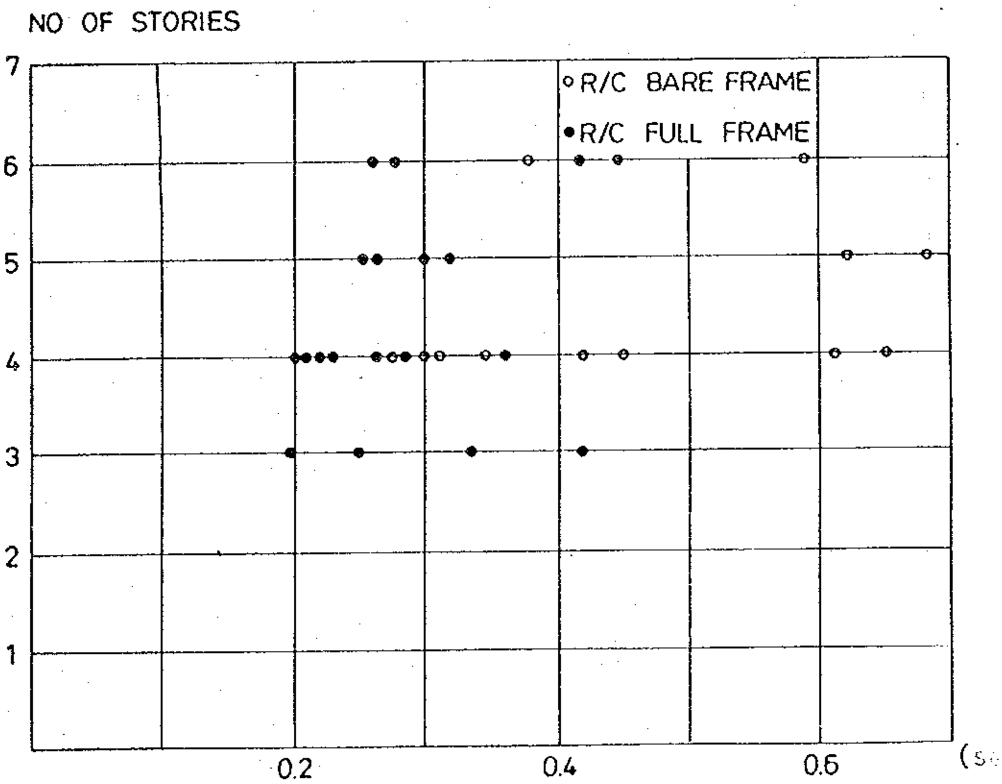


FIGURE - 2 MEASURED PERIODS IN BOLU - 1981

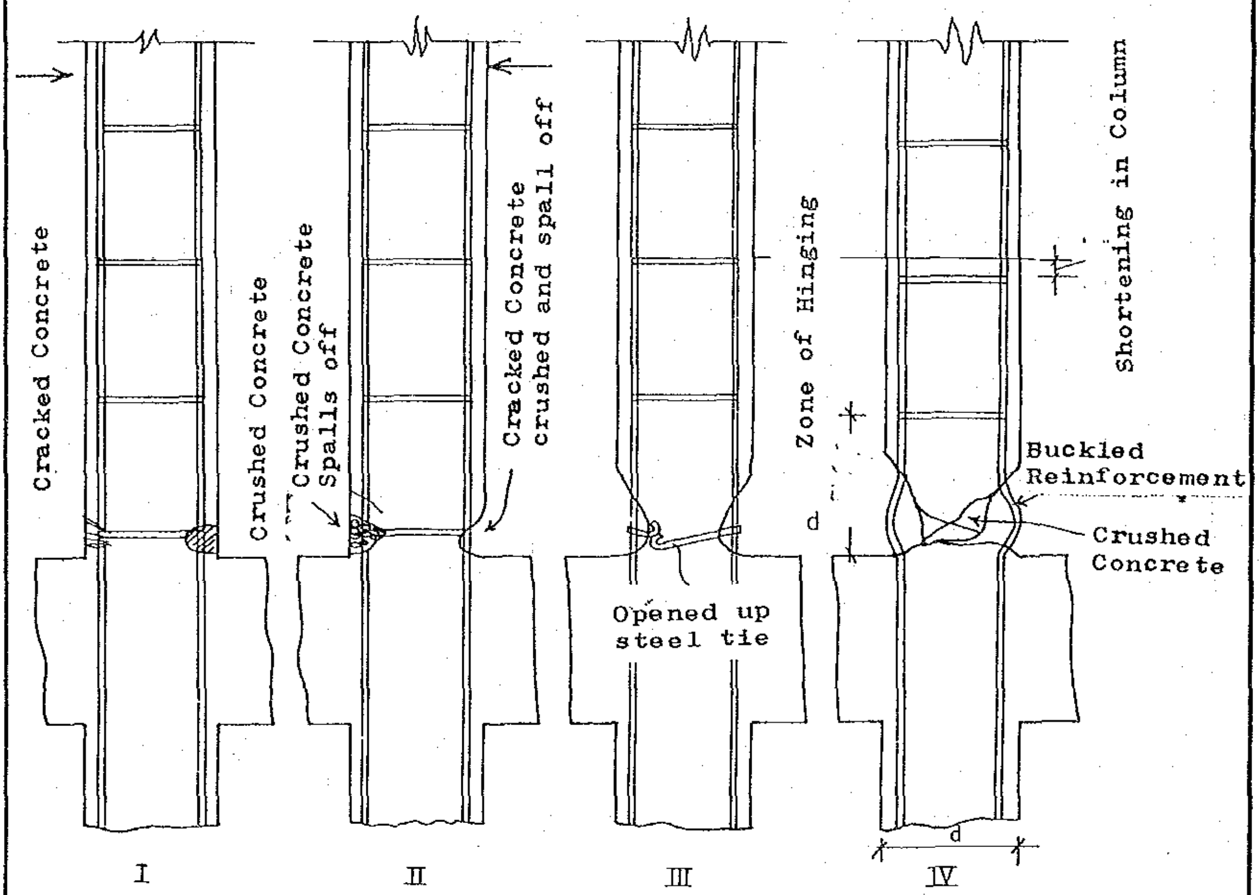


Figure-4 Development of Hinging in C

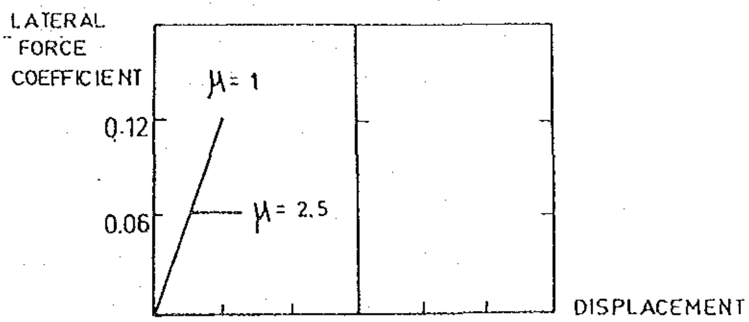


FIGURE-3 EQUAL ENERGY DUCTILITY

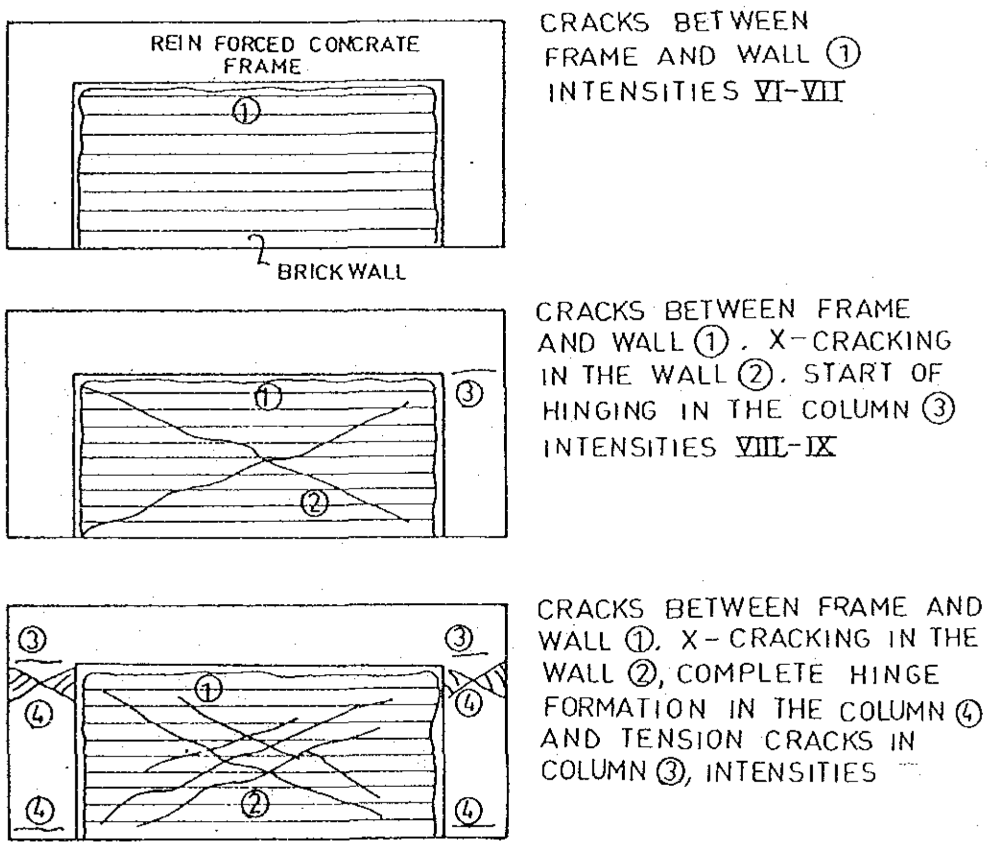


FIGURE-5 DAMAGE LEVELS IN REINFORCED CONCRETE FRAME

There will always be some buildings which will behave better or worse than the average. It seems that the intensities of VIII-IX would be very critical. There were very few reinforced concrete buildings in the regions which had been subjected to the recent large earthquakes in Turkey. Eventhough there had been many damaging earthquakes in the last 50 years, in very few cases a strong earthquake (magnitude about 7.0) occured in a locality where a sufficiently large number of reinforced concrete buildings were located for statistical investigation.

Damage Details

The most commonly observed damage in reinforced concrete buildings are the column damage by hinging as shown on Figure-4. This is a ductile failure with a lot of energy dissipation. As long as more columns with fixed ends than the earthquake energy turn into hinges, eventhough there may be severe hinging in the columns the building will resist the earthquake. Since the columns have low percentage of longitudinal steel this type of tension or moment failure is frequently encountered.

In some cases when the concrete compressive strengths are extremely low, that is below the average value for Turkey compression failure of the column is also observed, as vertical cracks on column face.

Another frequent column damage is the shear failure of the columns. This usually occurs as the distance between column ties are very large and the low strength concrete has also a low shear strength, and there is not sufficient confinement of concrete

Sometimes crowding of the longitudinal bars and insufficient cover of concrete results in bond failure and the concrete cover of the column falls along the whole length of the columns even at very small earthquakes. Figure-6.

...rt column failure of reinforced concrete columns ... ...quently observed.

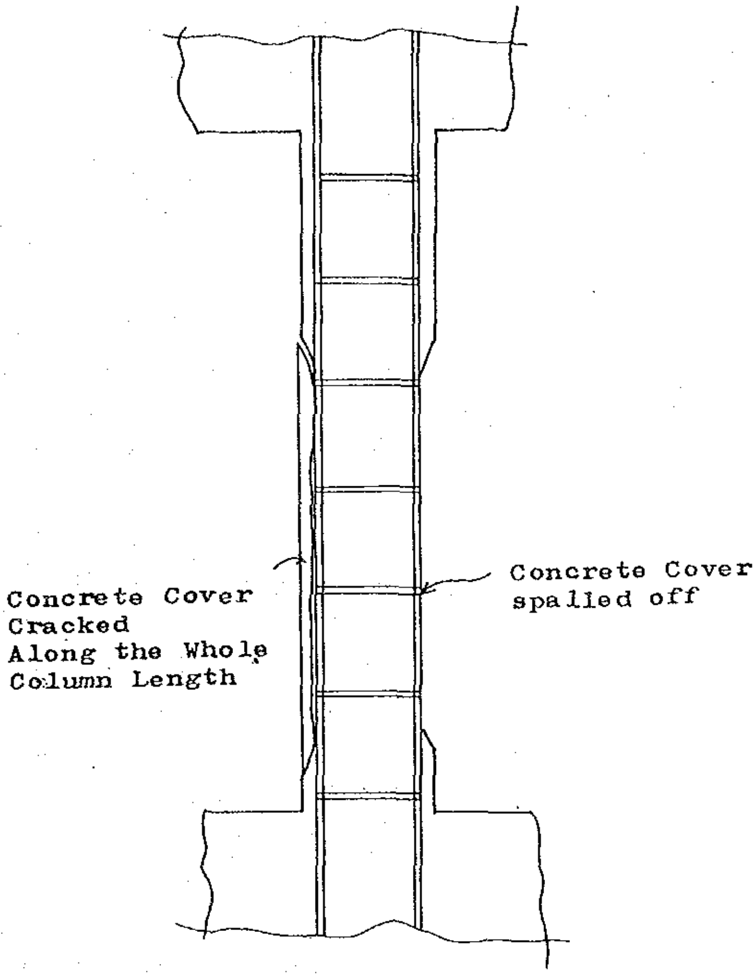


Figure-6 Bond Failure in Columns

One of the most common architectural damage is the collapse of gable walls of reinforced concrete buildings. Such free standing walls create a great hazard for the passerbys in the streets. These gable walls in some cases are built for architectural effect and do not have any functional use.

Stair cases are also frequently damaged. They are basically a diagonal element in a frame. Thus the staircase with its surrounding frame is the most rigid element in the building and they receive a larger percent of the lateral load coming to the structure. This load causes tensile forces along the staircase diagonal and the staircase is easily cracked where it is connected to the floor beams and rapidly become unusable. Also filler walls may collapse and close the stairway to traffic and the evacuation of the damaged building becoms dangerous.

Recently with the building of high rise reinforced concrete buildings and the increased earthquake design force coefficients required requires to have shear walls in reinforced concrete buildings. If the shear wall in a frame type of building is not located at or near the center of building, torsional effects come into play during earthquakes. In fact this factor played an important role in the damage of Adapazarı government house during 1967 Adapazarı Earthquake. That building had an unsymmetrically located stair case with shear wall at its sides.

The irregular lot sizes in older parts of many cities and towns result in buildings of unsymmetrical plan shapes. This also effects the structural framing and the structural framing system would usually create a great opportunity for torsional effects to occur during earthquakes. Even unsymetrically provided filler walls in a building with symmetrical framing system were observed to have created in the framing damages due to torsional effects.

Filler walls within the frames are damaged usually because they are less ductile than the reinforced concrete frame. On the other hand free standing partition walls have a very high probability of collapse and they should surrounded by frames their collapse.

Damage Raito for Reinforced Concrete Buildings

Damage ratios for buildings are needed in case of vulnerability and disaster scenario type studies. There is a need to know the number of buildings which would be damaged at various levels during a given earthquake intensity. There are many examples of such relations for damage ratios for various types of buildings. One such study was carried out by Gürpınar and Et al (1978) for reinforced concrete buildings. In Figure-7 damage ratios calculated from the damage matrises given by Gürpınar is presented. In his original work Gürpınar had developed two damage matrices: one for buildings built according to the Code and the other for buildings built without any concern for earthquakes. In the preparation of the relations given in Figure-7 it is assumed that 20 % percent of the stock of buildings would be built according to the code and the rest would be buildings in which no precautions had been taken against earthquakes. On Figure-8 the damage ratio for reinforced concrete buildings actually observed in three earthquakes of different intensities are given. In these curves heavy damage includes collapsed buildings aswell.

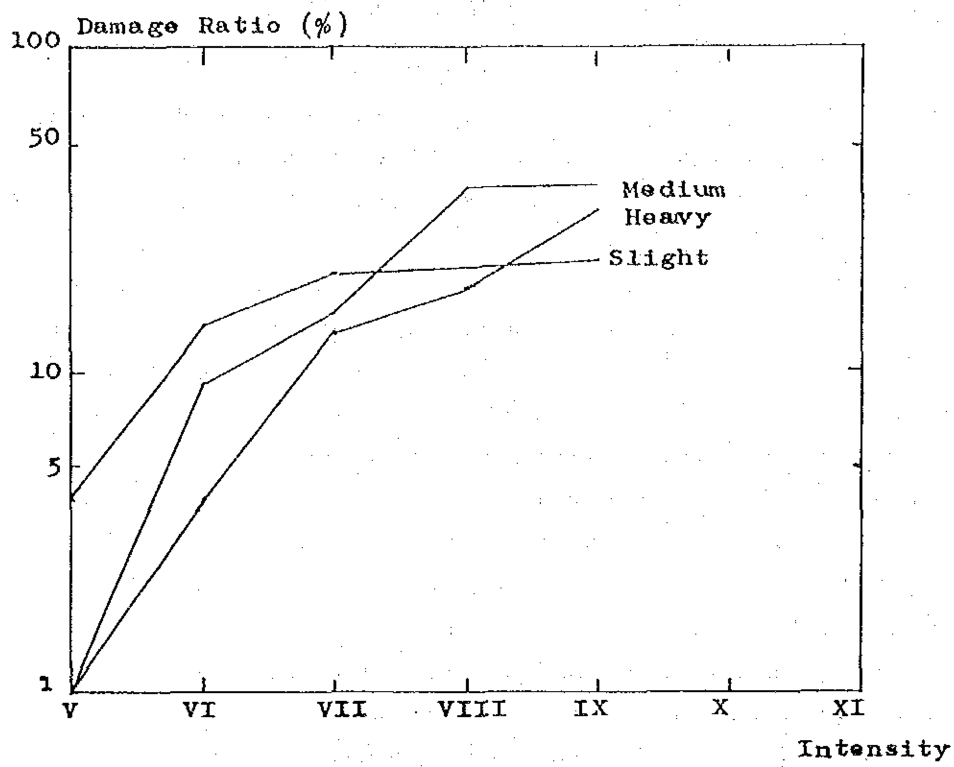


Figure-7 Damage Ratio for Reinforced Concrete Buildings(After Gürpınar et al 1978)

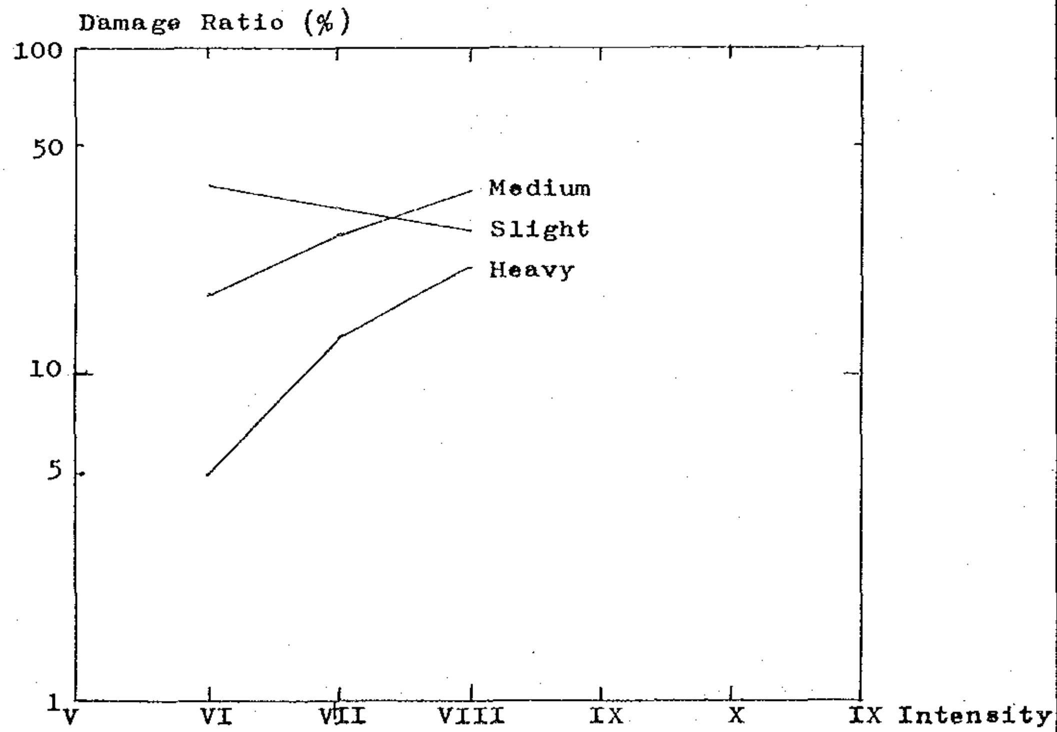


Figure-8 Observed Damage Ratios for Reinforced Concrete Buildings

DÖŞEMELERDE 100 Santim Genişlik için Demir Alanı (cm2)

Mesafe (cm)	mm olarak demir çapı ø 6	ø 8	ø 10	ø 12	ø 14
7.0	4.04	7.18	11.22	16.16	21.99
7.5	3.77	6.70	10.47	15.08	20.52
8.0	3.53	6.28	9.82	14.14	19.24
8.5	3.33	5.91	9.24	13.31	18.11
9.0	3.14	5.59	8.73	12.57	17.10
9.5	2.98	5.29	8.27	11.90	16.20
10.0	2.83	5.03	7.85	11.31	15.39
10.5	2.69	4.79	7.48	10.77	14.66
11.0	2.57	4.57	7.14	10.28	13.99
11.5	2.46	4.37	6.83	9.84	13.39
12.0	2.36	4.19	6.54	9.42	12.83
12.5	2.26	4.02	6.28	9.05	12.32
13.0	2.17	3.87	6.04	8.70	11.84
13.5	2.09	3.72	5.82	8.38	11.40
14.0	2.02	3.59	5.61	8.08	11.00
14.5	1.95	3.47	5.42	7.80	10.62
15.0	1.89	3.35	5.24	7.54	10.26
15.5	1.82	3.24	5.07	7.30	9.93
16.0	1.77	3.14	4.91	7.07	9.62
16.5	1.71	3.05	4.76	6.85	9.33
17.0	1.66	2.96	4.62	6.65	9.05
17.5	1.62	2.87	4.49	6.46	8.79
18.0	1.57	2.79	4.36	6.28	8.55
18.5	1.53	2.72	4.25	6.11	8.32
19.0	1.49	2.65	4.13	5.95	8.10
19.5	1.45	2.58	4.03	5.80	7.89
20.0	1.41	2.51	3.93	5.65	7.69

DEMİR KESİT ALANLARI (cm2)

	1	2	3	4	5	6	7	8
ø 6	0.28	0.57	0.85	1.13	1.41	1.70	1.98	2.26
ø 8	0.50	1.01	1.51	2.01	2.51	3.01	3.52	4.02
ø 10	0.79	1.57	2.36	3.14	3.93	4.71	5.50	6.28
ø 12	1.13	2.26	3.39	4.52	5.65	6.79	7.92	9.05
ø 14	1.54	3.08	4.62	6.16	7.70	9.24	10.73	[illegible]
ø 16	2.01	4.02	6.03	8.04	10.05	12.06	14.[illegible]	
ø 18	2.54	5.09	7.63	10.18	12.72	15.26	17.[illegible]	
ø 20	3.14	6.28	9.42	12.57	15.71	18.84	21.99	[illegible]
ø 22	3.80	7.60	11.40	15.21	19.01	22.81	26.61	30.41

TIMBER FRAME BUILDINGS AND THEIR EARTHQUAKE BEHAVIOUR

Types

The timber frame buildings in Turkey can be classified into several categories. First there is HIMIŞ type. This is a timber frame type with its walls filled up with adobe blocks, stone, clay bricks or concrete blocks. The framing may be formed from well sawn timber or it may be of unsawnround tree trunks. The second type is called BAĞDADİ, Here the inner and outer faces of the basic timber frame are coverede with wooden lathes nailed to the framing system. The center-to-center distance between the lathes is about 5 cm. The inner space between the lathes may be empty, or filled up with adobe, brick, stone (usually large sized pebbles), pine cones or with pieces of tree barks. There is another type of timber frame not so frequently encountered in cities but more frequent in rural areas close to forests. This is called ÇİZEME, DİZEME or DOKU. In this type the wall framing consists of timber logs cut into two. The filler material of the walls is again timber. These three main types of timber frame buildings are shown in detail in Figure-9

Framing

In timber frame type of buildings the elements of the frame may be of various forms. Logs may be used without any treatment and even logs of crooked shape may be employed. Generally the quality of timber is not good in rural areas where as better quality of timber is used in cities. Timber is not dried in kilns and generally has a high water content. It dries naturally after it is placed in buildings and thus, large shrinkage cracks and weakening of nailed connections are very common. Pine wood is ... n type used in building. Chemical treatment of ... of fungi and insects is very rare. The verti... l elements of typical frames are 10x10 cm and 5x10 ... ion. During a field study conducted by the author ... Bolu it was learned that before the 1944 earthquake the ... was to provide diagonal elements only in one direction. The use of cross-bracing was adopted after the 1944 quake. In Figures 10 to 14 various timber frame types observed in North western anatolia are given.

Foundations

The foundations of timber frame buildings are generally poor. If the ground water level is high, there is a need to protect the timber from moisture coming from the soil. In some buildings the timber framing rests on foundation stones. These are uncut large sized rubble stones gathered from the field. Buildings with this kind of foundation behaved very poorly during the 1944 and 1967 earthquakes which had occured on the North Anatolian Fault. Collapse of timber frame houses because of poor foundations should be expected more frequently in rural areas. Timber frame houses built in cities usually are 2 two 3 stories high and have continous foundation walls around. Their collapse during an earthquake due to foundation failure or due to poor connection to foundations seems less probable.

Exterior Finish

Protection of timber against environmental effects is required in order to increase the durability of timber frame houses. Most of the timber frame type of buildings have their exterior surfaces covered by a plaster of lime and sand mortar with some cement and on the whole, they appear as masonry buildings. The DİZEME type timber frame buildings do not have any protection on their surface. Formerly timber frame buildings built in larger cities such as Istanbul, Bursa, etc. had their exterior covered with timber planks and paint. This was the tradition in late XIX and early XX centuries. The preference of a mortar cover is due to the fact that paint is more expensive and less durable as well as the fact that a layer of mortar provides some heat insulation as well.

Architectural Plans

The timber frame buildings have quite similar plans. Each [illegible]
central corridor extending perpendicular to th[illegible]
there are rooms on each side of this corri[illegible]
plan and the detailed scaled plan of a typical timber frame are shown on Figures 14 and 15. In some cases there are balçonies in

in place of one of the frontal rooms. The sizes of rooms and corridors are almost standart. This due to the sizes of the timber generally available.

Dyhamic Characteristics

During the field investigation made in 1981 in Bolu area the periods vibration of several timber frame buildings were measured. They had square or nearly square plan shapes. Their story heights are around 3 meters. Plan areas do not vary greatly and they are about 100 m2 plus or minus 24 m2. Damping of one building was measured to be about 5-6 percent of the critical. Usually because of the nearly square shape of buildings the periods do not vary significantly about the two major axes of the buildings. The measured periods of vibration are given in Figure-2. The average period of two-story timber frame buildings is 0.354 0.051, while for three story buildings it is 0.382 0.039 seconds. There appears to be a noticeable increase in the average period with increase in story number. For practical purposes, timber frame buildings of two and three stories can be considered having periods in the range of 0.3 to 0.4 seconds.

Earthquake Behaviour

There are several factors which govern the earthquake behaviour of timber frame buildings. It seems that age is the most important factor along with the connection details of the frame to the foundation. In Turkey the timber frame buildings are getting older. That is the percentage of timber frame buildings built within the last 10-20 years are very small compared to timber frame buildings of 40 or more years old. Even in forest areas of Turkey people are not building timber buildings as many as in the past.

In timber areas there is also high humidity which accelerates the deterioration of timber. The strength of connections in the frame is effected by the creep and shrinkage in timber. In addition, because the people in Turkey nowadays prefer to live in masonry and reinforced concrete apartment houses, the timber frame buildings which require good maintenance are not looked after properly. They are rather treated as buildings the majority of which will be replaced in the next 10-20 years. An earthquake which may occur in

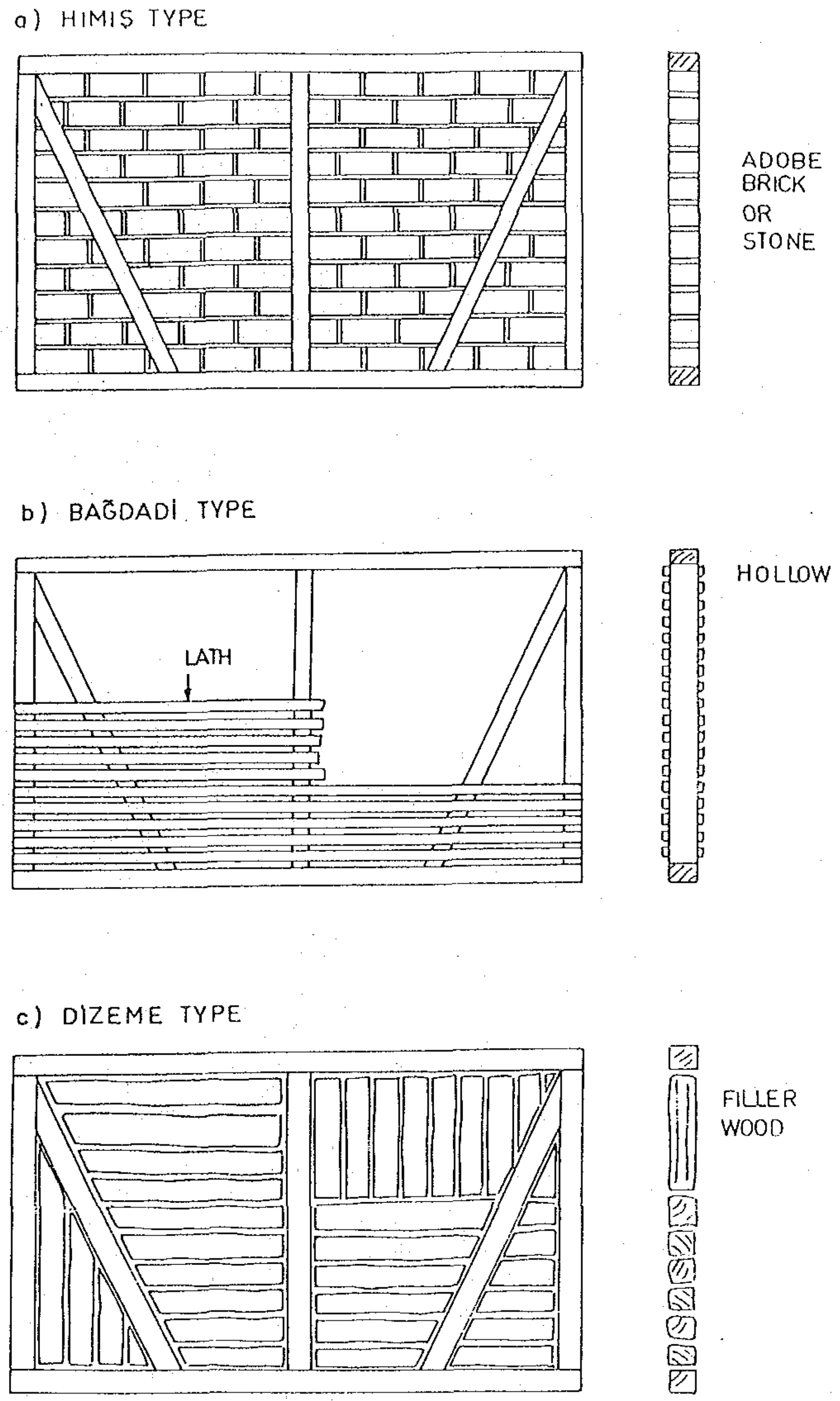
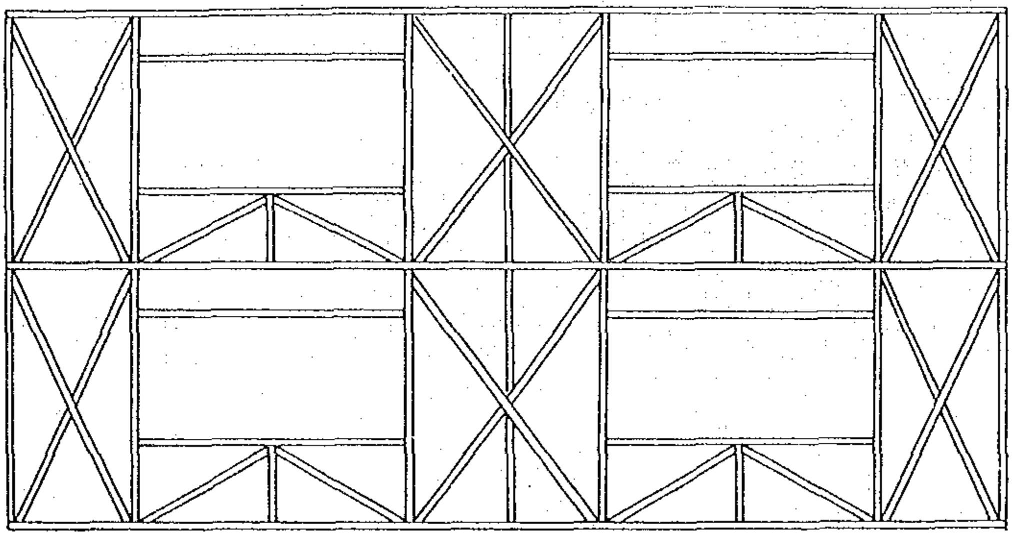
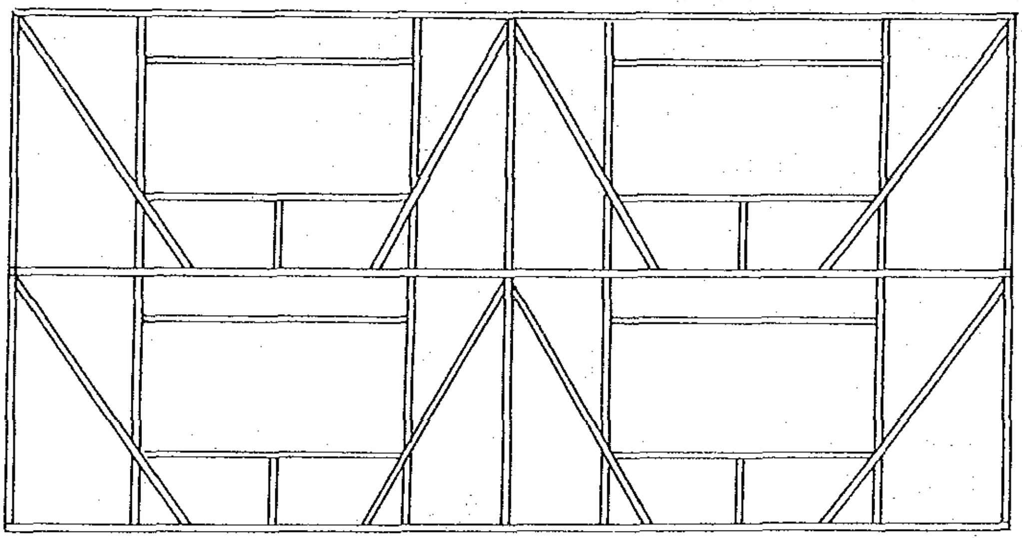


FIGURE-9 TYPES OF TIMBER FRAME

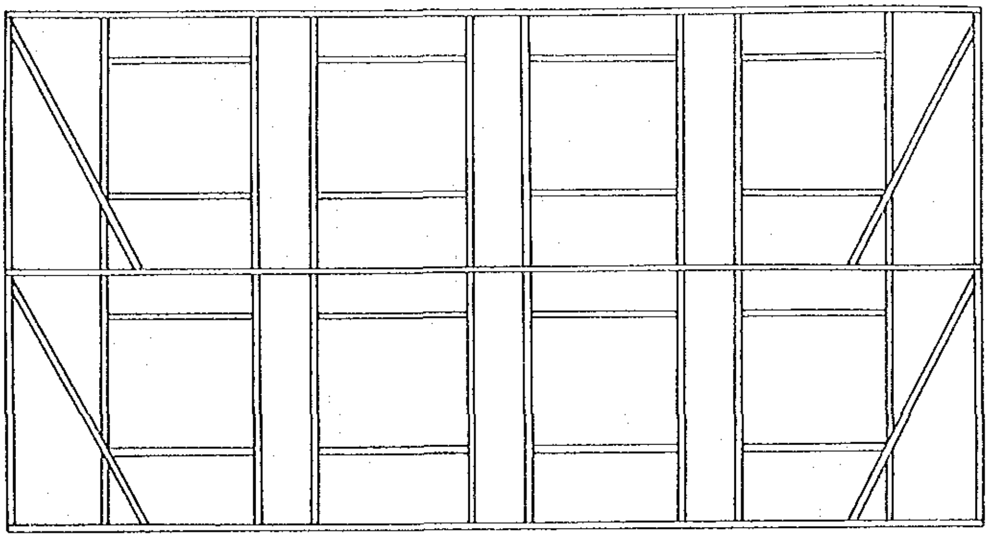


a) FULLY FRAMED TYPE WITH WINDOWS
(APPLIED AFTER 1944 QUAKE)

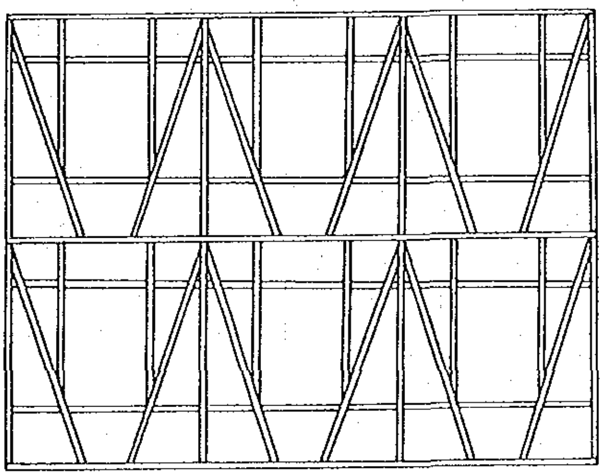


b) SPARSELY FRAMED TYPE WITH WINDOWS.
(PRACTICED BEFORE 1944 QUAKE)

FIGURE-10 TIMBER FRAMES

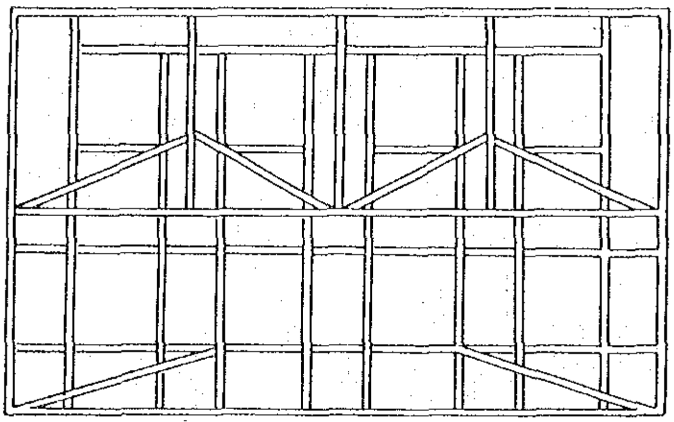


a) SPARSELY FRAMED TYPE WITH WINDOWS

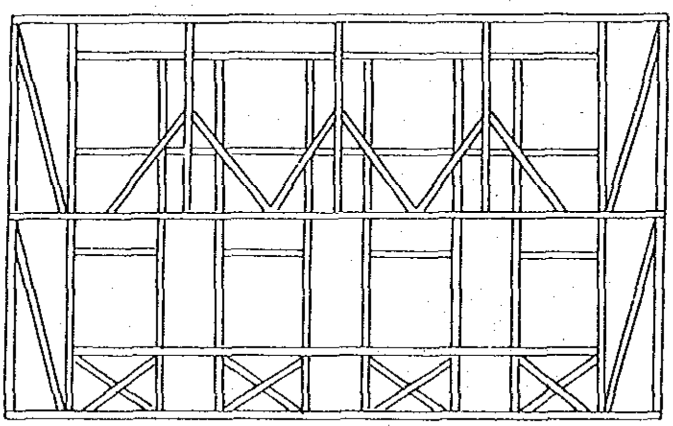


b) FRAME WITH WINDOWS

FIGURE-11 TIMBER FRAMES

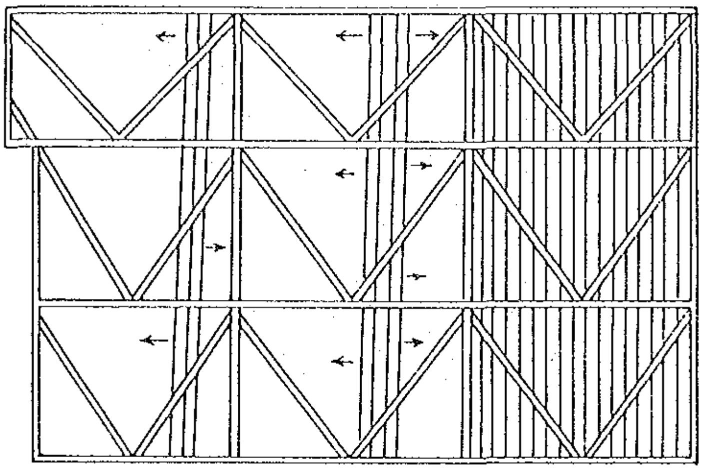


a) VARIOUS FRAMING FOR WALLS WITH MANY WINDOWS

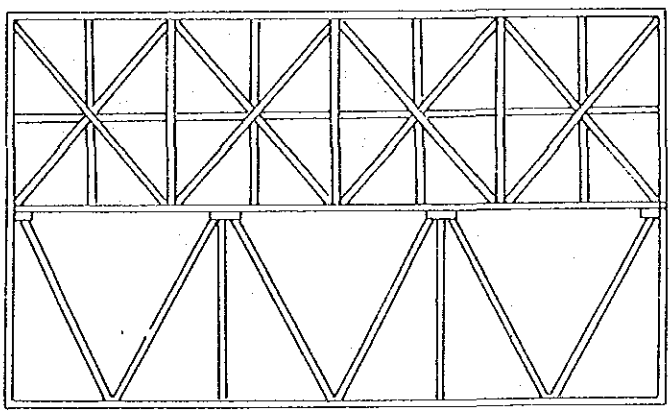


b) VARIOUS FRAMING FOR WALLS WITH MANY WINDOWS

FIGURE-12 TIMBER FRAMES



a) "DİZEME" TYPE WALL WITHOUT OPENING
WITH VERTICALLY PLACED LONGS



b) TIMBER FRAME WITHOUT OPENINGS

FIGURE -13 TIMBER FRAMES

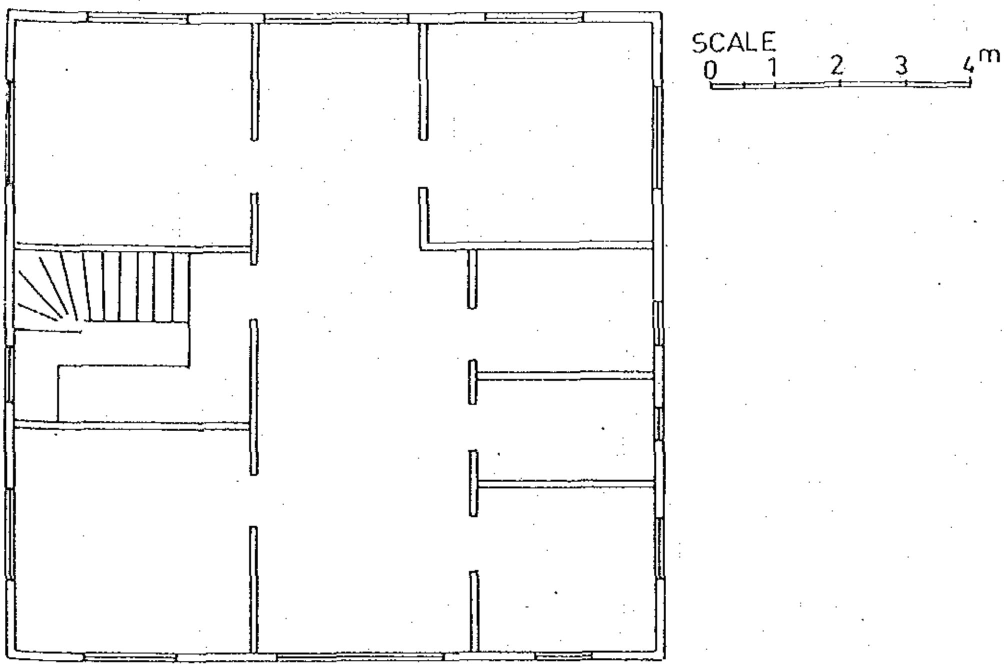


FIGURE -14 PLAN OF TIMBER FRAME BUILDING IN BOLU

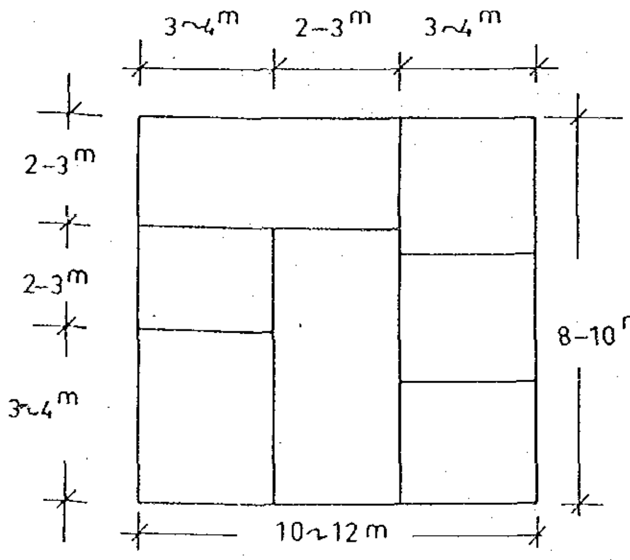


FIGURE 15 TYPICAL FLOOR PLAN FOR TIMBER FRAME BUILDINGS

the mean time may find many neglected timber frame buildings in cities formerly composed of timber frame houses. Another factor of damage which should be considered in case of an earthquake is fire. During the 1970 Gediz Earthquake a whole district of Gediz town was destroyed by fire , while the whole village of Akçaalan (population 2500) burnt down. In urban areas, like large cities Bursa and Kütahya nowadays there are still districts composed of timber frame buildings and there may be few individual timber f frame buildings in old districts converted to reinforced concrete and brick masonry buildings.

The timber frame buildings with brick and stone veneers are also common. Usually these surface walls fall down since they are not connected to the frames. The filler materials in the wall framing also fall down easily. The next stage of damage is the loosening of the connections and breaking up of some of the connections leading to the collapse. Since timber frame bulidings are rather flexible, their lateral deflections during an earthquake may cause secondary moments and whole building may overturn. Single story timber houses without proper foundations may shift or rotate at their base or fall off their foundations and the framing may disintegrate.

It is observed that in earthquakes BAĞDADİ houses behave muçh better than HIMIŞ houses, Both types loose their surface mortar layer very early in the quake. But BAĞDADİ type is generally lighter and the numereous wooden lathes nailed on the framing provide considerably high damping and energy absorption to the structure.

For the damage ratio of timber frame buildings the curves given in Figure- 26 can be used tentatively

BRICK MASONRY BUILDINGS AND THEIR EARTHQUAKE BEHAVIOUR

Materials

In Turkey there are now four kinds of clay bricks used in masonry houses; 1-Hand made bricks: They are fired at low temperatures and they have low strengths. Their compressive strengths are 20-30 kgf/cm2. Their dimensions are 190x90x55 and 220x105x55 mm. With the construction of brick factories all over Turkey their production is on decline. 2-Factory Made Load Carrying Solid Bricks: These bricks can have vertical perforations of 15 % of their cross section area. Compressive strengths are about 200 kgf/cm2. They 190x95x50 mm size. Even the lowest quality ones have a compressive strength of 100 kgf/cm2. 3-Factory made Load Carrying Block Bricks These have vertical perforations. They have two types 190x90x85 and 190x290x135 mm. The area of perforations are 30-35 % in the first and 50-55 % in the second type. The first type has compressive strengths of 100-150 kgf/cm2 while the other type has 60-70 kgf/cm2. The second type is widely used allthough it has more perforations than which is allowed by standarts to be used in load carrying walls. 4-Hollow Block Filler Bricks Although they produced for filler walls of reinforced concrete buildings they are being used as load carrying walls in masonry buildings. They 45-50 % perforations and 190x190x85 and 190x190x135 mm dimensions. Their compressive strength about 20 to 30 kgf/cm2.

The kind of mortar used in masonry is of low strength. The cement, lime and sand ratio of 1:4:8 by volume is the most commonly used mortar mix ratio. This mortar has a 28th day compressive strength of approximately 10 kg/cm2. Using this type of mortar with the above type of bricks results in wall compressive strengths of 0.3-0.75 times the compressive strength of brick. A test carried out by the author shows the relation between brick strength and wall strength in Figure-16. There is not much information about the shear and tensile strengths of walls made from these materials. Strengt in the order of 1-2 kgf/cm2 seem to be probable values kind of masonry construction in Turkey.

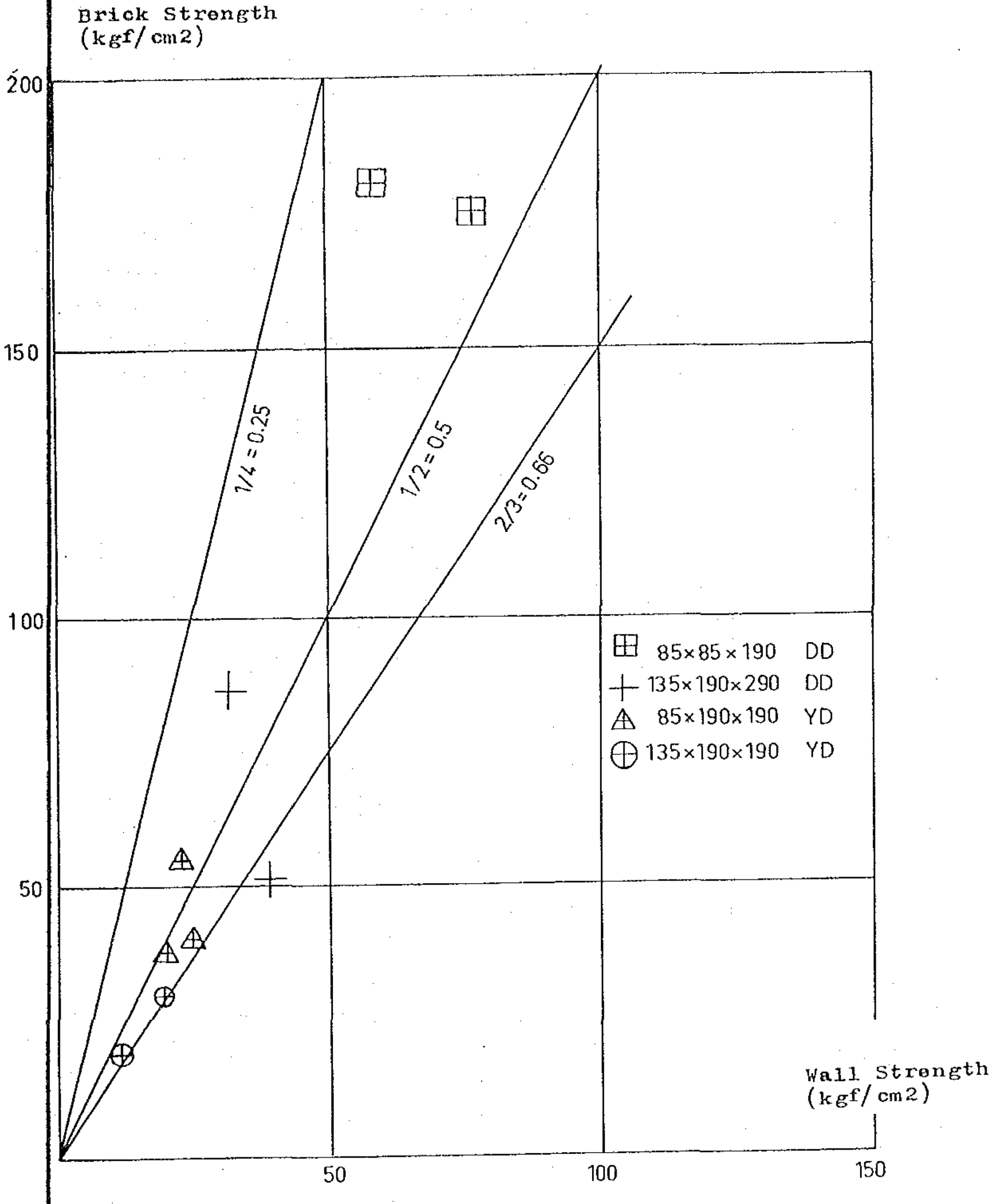


Figure-16 Relation Between Wall Strength and Brick Strength

The earthquake code requires that the strength of the brick to be used should not be less than 50 kgf/cm2.

Characteristics Of Masonry Buildings

Brick masonry buildings can have 2 stories in 1st degree earthquake zones and 3 stories in 2nd and 3rd degree earthquake zones, and 4 stories in 4th degree zones and 5 stories in danger free zones. The minimum allowable wall thickness depends on earthquake zone and the height of the building. There are also many restrictions on the window and door opening sizes, locations, total amount of opening and the amount of walls between two openings and an opening and the wall corner.

In Turkey many of the brick masonry buildings have reinforced concrete columns, lintels and tie beams, and they may appear at first notice as reinforced concrete frame buildings. However, the columns and tie beams present in the building are not designed to carry vertical or lateral loads, rather they are intended as elements which will strengthen the building. Perhaps this type of construction may be considered as an evolution of the traditional timber frame type, with the few reinforced concrete columns and tie beams replacing the traditional timber framing elements. Many of this type of brick masonry buildings are built by local constructors without any formal training other than apprenticeship by an older constructor. This type of masonry buildings can be considered as a kind of reinforced masonry.

There are also many public buildings such as schools, health centers etc built by government out of brick. These are usually one to two stories high with reinforced concrete slabs. Many of the such government buildings of single story height has timber roof trusses without a reinforced concrete slab.

The reinforcement used in these tie beams and columns are at least four longitudinal bars of ø 10-14 mm with ties of ø 6-8 mm bars at intervals of 25-40 cm. The cylinder strength of the concrete used in this type of buildings is around 100 kgf/cm2 at best, not being much different from the strength of concrete used in ordinary reinforced concrete buildings in Turkey.

The usual room size in brick masonry residential buildings is in the order of 3.5-4.5 meters squared, so there are cross walls at every 3-4.5 meters. Wall thicknesses are 1.5 brick in ground floors and 1 brick in the upper stories (29 and 19 cm with new bricks and 35 and 23 cm with pre-1972 bricks). The usual brick masonry building has reinforced concrete slabs of 10-12 cm thickness; the slabs are connected to the walls by tie beams with cross sectional dimensions of wall thickness times 20 cm and with the reinforcement described above. The vertical load coming to the brick walls of a typical brick masonry building can be taken as 3.5 tons/meter per story. This force creates vertical stresses in the ground floor walls of the structure which are, depending on the height of the building and the thickness of the ground floor walls, between 1.9 and 4.5 kgf/cm2. When these values are compared with the wall compressive strengths given in previous section, it is seen that the factor of safety against vertical loads in brick masonry is about 2-10. A more conservative figure of 4-5 seems to be the normal value.

Dynamic Characteristics

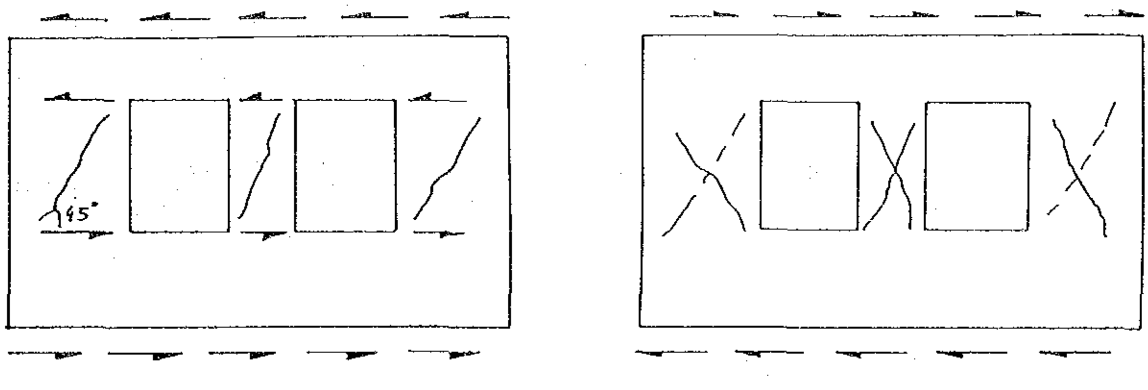
The natural periods of vibration of brick masonry buildings have large variations. As a rule of thumb the period of vibration of a masonry building can be expressed as $T = 0.05\ N$ (N: number of stories). Damping factors are very low, in the order of a few percent. However both damping and the period may increase very significantly with the severity of damage. As high as 300 percent increase in period has been observed in case of a masonry building damaged in Çaldıran in 1976.

Earthquake Behaviour

Brick masonry buildings start to have cracks in their walls at an MSK intensity of V-VI. If the intensity is higher, which may be due to earthquakes of longer duration, the cracks widen and the wall is weakened. The wall begins to loose its vertical load carryin capacity also. The X-shaped cracks due to lateral forces may be followed by vertically oriented cracks due to decreased vertical load carrying capacity of the wall. The occurence of vertical cracking following the deterioration of the brick wall by lateral earthquake forces is seen in buildings having walls with large door and/or window openings. In Figures-17 and 18 various crack formations depending on the shape, width and location of opennings are presented. A relatively 'well' designed brick masonry building can withstand the Intensities of VIII-IX with serious and irreparable damage, but it will probably have to be demolished. The presence of tie beams and columns greatly improves the earthquake behaviour of brick masonry buildings. A 'well' designed brick masonry building should have openings whose total length is less than 40 percent of the total length of the building. The presence of sufficient amount of solid walls between two openings and between an opening and the corner of the building will also increase earthquake resistance of the building.

Over the years it was learned through observations of earthquake damage that the presence of reinforced concrete slab or roof trusses of sufficient rigidity are very effective in tieing all the walls together and decrasing earthquake damage. Poorly connected walls cause damages in the walls as shown in Figure-18. Proper connection of walls at the corners are also important. In Figure-19 types of corner damage in masonry buildings are given.

For the damage ratio of brick masonry buildings relations given in Figures-26 and 27 can be used. While the relation on Figure- 26 applies to well made buildings, the Figure-27 applies to poorly made brick masonry buildings.



(A) (B)

Diagonal Tension Cracking in Brick Walls

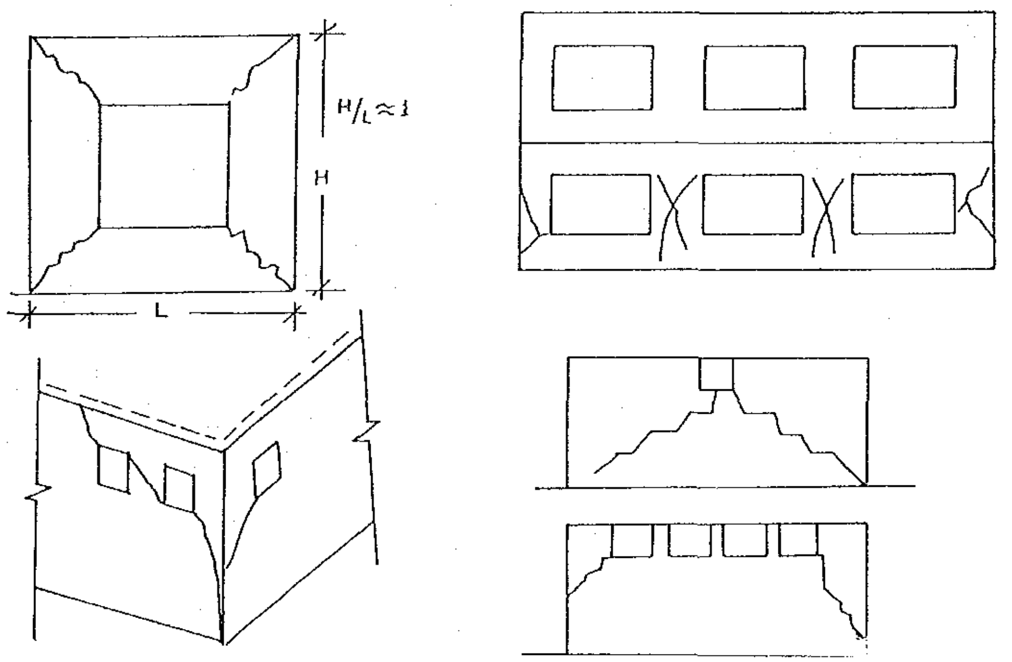
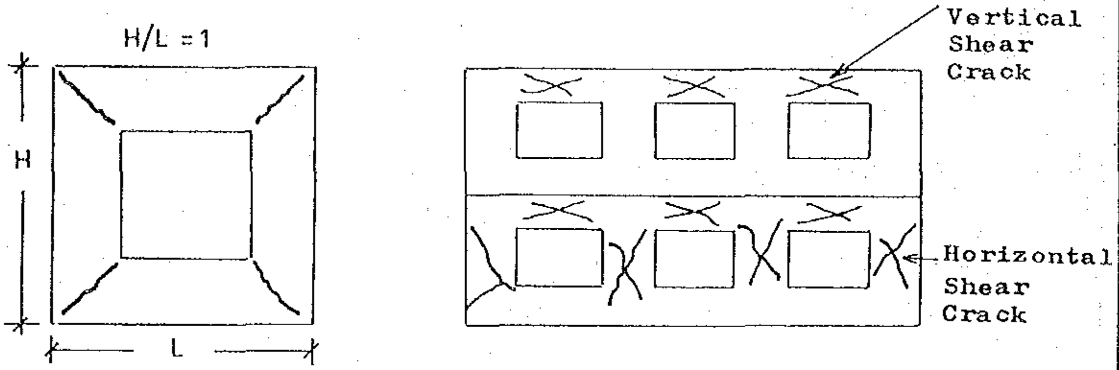
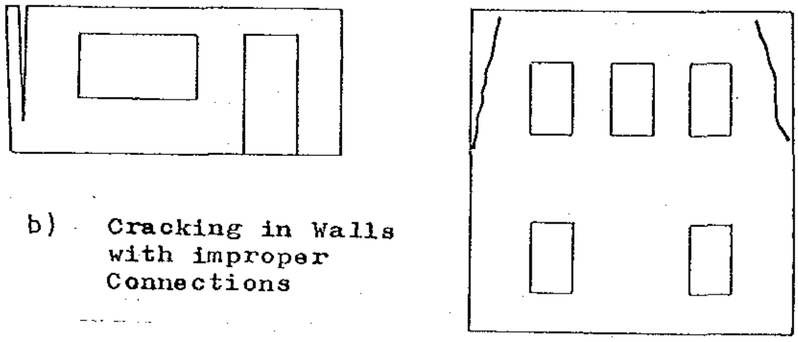


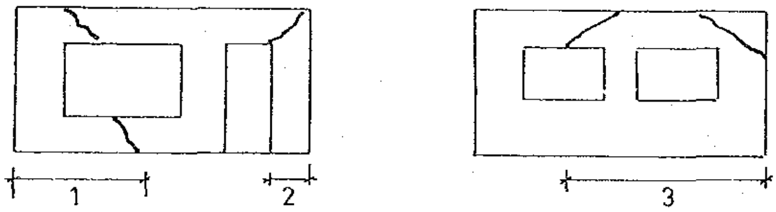
Figure-17 Various Forms of Earthquakr Formation in Brick Walls



a) Shear Cracking in Walls Due to Earthquakes

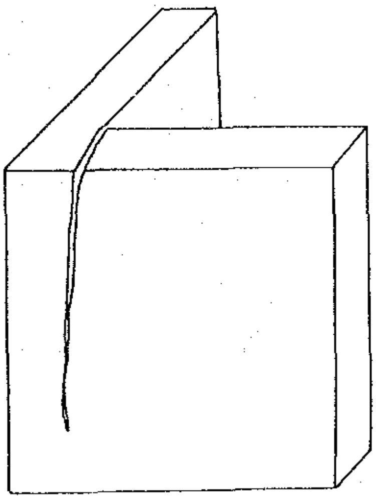


b) Cracking in Walls with improper Connections

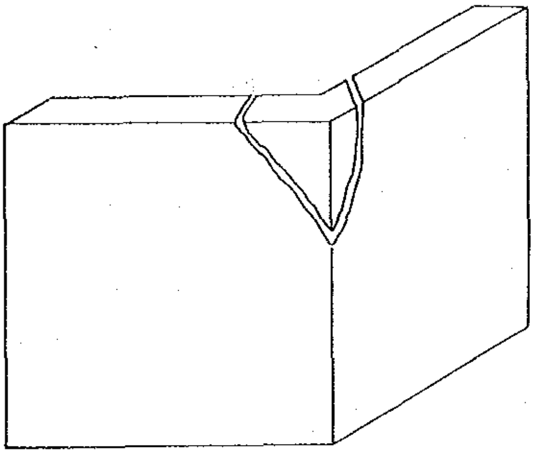


c) Settlement Cracks in Brick Masonry Buildings

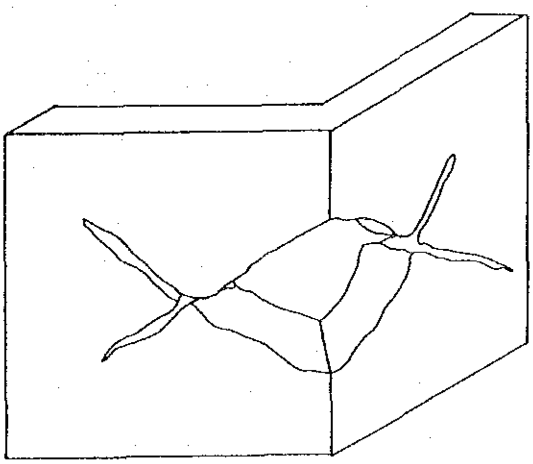
Figure-18 Cracks in Brick Masonry Buildings



a) Damage in a Poorly Connected Corner of a Brick Masonry Building



b) Corner Damage in a Masonry Building Having a Flexible Roof System



c) Corner Damage in a Brick Masonry Building d Large Earth

Figure-19 Corner Damage Types in Brick Masonry Buildings

RUBBLE STONE AND ADOBE MASONRY BUILDINGS AND THEIR EARTHQUAKE BEHAVIOUR

This is the type of rural houses very widely constructed in Central and Eastern parts of Anatolia. The severe climate of the region and the lack of better construction materials dictate this solution to the housing in rural areas. Particularly the rubble stone masonry buildings suffer greatly. In Table-1 the damage and casualty resulting from the four magnitude 6.9-7.1 earthquakes which had occured in Eastern Anatolia in the last 20 years are given. Such an earthquake causes approximately 1700 casualties in rural areas and 300 in urban areas. The average collapsed houses reach to a number of 8000 per earthquake.

Walls

In figure-20 rubble stone masonry wall types are given. If there are horizontally bedded and cracked rock formations relatively good stone work is done. However most of the time the walls are like as given in Figure-20a. The stone is collected from river beds or fields. These are usually round, well polished and sometimes moss covered stones. THe wall is usually made from stones of all sizes. In such cases even if cement reinforced lime sand mortar is used as mortar it is quite difficult to achieve connection between the stones. Use of large sized stones also prevents bonding between stones. This is evidenced from the damage of state built school, medical center etc buildings built by using the same stone but cement,lime and sahd mortar. Allthough such built public buildings do not collapse, they are severely damaged and demolished.

Use of mud as mortar does not provide any connection or bond between stones after the mud has dried out and crumbled. Dried mud usually falls out ou the wall. The only connection between stones are achieved by friction. During an earthquake such walls have almost no shear strength.

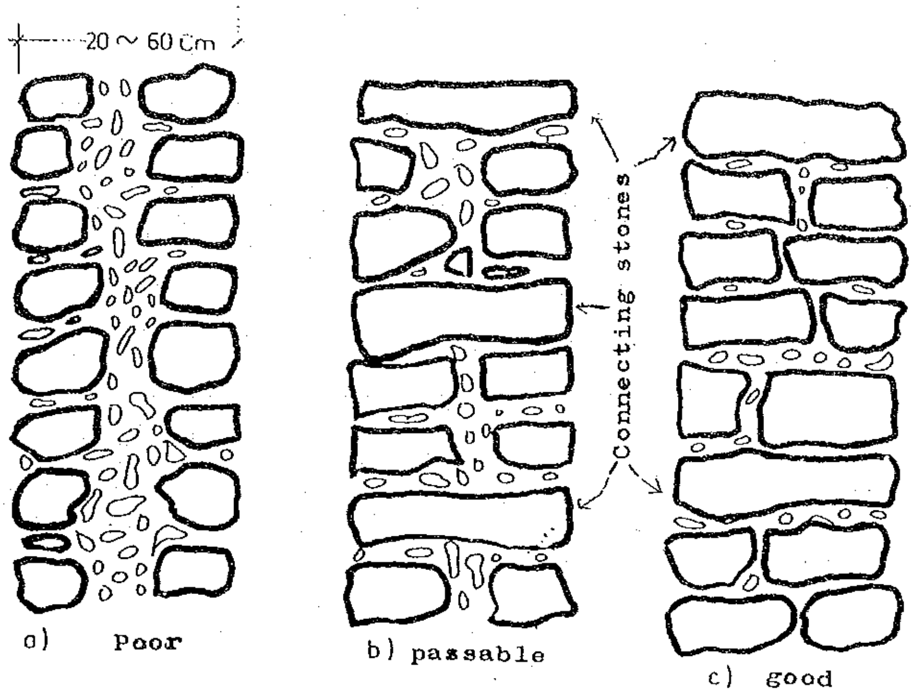


Figure-20 Typical Wall Cross Sections

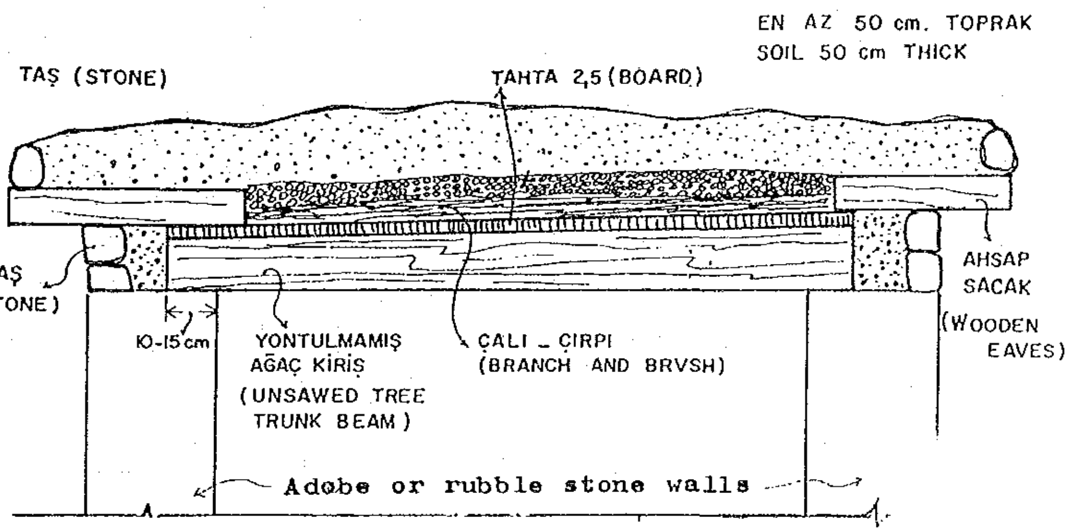


Figure-21 Flat Earth Roof Detail

TABLE -I

Earthquake	19/8/1966 VARTO	22/5/1971 BİNGÖL	24/11/1976 ÇALDIRAN	30/10/1983 HORASAN	AVERAGE
Magnitude	6.9	6.9	7.2	7.1	
Focal Depth	20 km	10 km	10 km	20 km	
Season	Summer	Spring	Winter	Winter	
Time	14:22	18:43	14:22	07:13	
Effected area km2	11 095	3 300	6 400	4 550	6 340
Effected population	225 588	77 471	146 661	92 713	135 583
Population density	20	23	23	20	22
Effected Villages	485	174	256	145	265±154
Area per Village (km2)	23	19	25	31	25±5
Casualties	2394	755	3840	1153	2035±1391
Casualties in Villages	2223 %91	461 % 61	3063 % 80	1153 % 100	1725 % 84
Casualty per Village	4.6	2.7	12	8	6.8±4.1
Casulty Ratio	% 1 1/101	% 0.6 1/168	% 2.1 1/48	% 1.3 1/80	% 1.2 1/81
Collapsed Houses	19078	3752	7486	3241	8389
Damaged Houses	?	5162	8259	7082	6837
Collapsed houses per Village	39	22	29	22	28
Damaged houses per village	?	30	32	49	37

Lack of bond stones extending through the width of the walls results in the wall cross section as given in Figure-2oa. Thus the walls look like sandwich walls with no connection between the inner and outer faces. In many small earthquakes the outer face of the wall seperates from the wall and fall down.

On the other hand adobe masonry walls are usually behaving better than rubble stone masonry walls of mud mortar. Since both the adobe block and the mortar are made of same material there is a continous medium. If the wall is protected from rain and moisture, it does not deteriorate and loose strength. But adobe walls has to be built on high foundation walls since there is heavy snowfall in the region.

The severity of winter conditions, sometimes the temperature in winter goes to -30 centigrades below zero, requires the walls to be thick. The wall thicknes is never less than 50-60 centimeters.

Roofing

In Figure-21 the typical flat earth roof and its connection tothe walls are given. Roof beams are made of unsawn tree trunks. The span depends on the type of trees available in the region. It is usually below 4.0 meters. The roof beams are placed at 25 to 50 cm intervals. The ends of the beams resting on the walls are usually 15 centimeters. On top of the beams planks of 2-2.5 cm thickness are placed. In some regions instead of planks canes are placed. On top of the planks treebranches and bushes are placed. The final insulation layer is the soil layer whose thickness sometimes reaches 50 cm. The weight of such a roof system can be as high as 0.75 to 1.0 ton/m2. On the other the weight of a reinforced concrete slab of 0.12 m thickness has a unit weight of only 0.3 tons/m2. Thus the roofing system is even heavier than reinforced concrete slab.

In order to prevent the seapage of rain and melting snow water, after every rain and usually before the rain season fine materials are added to the roof and the roof is compacted by stone cylinders called 'loğ'. Usually every roof has its own stone cylinder.

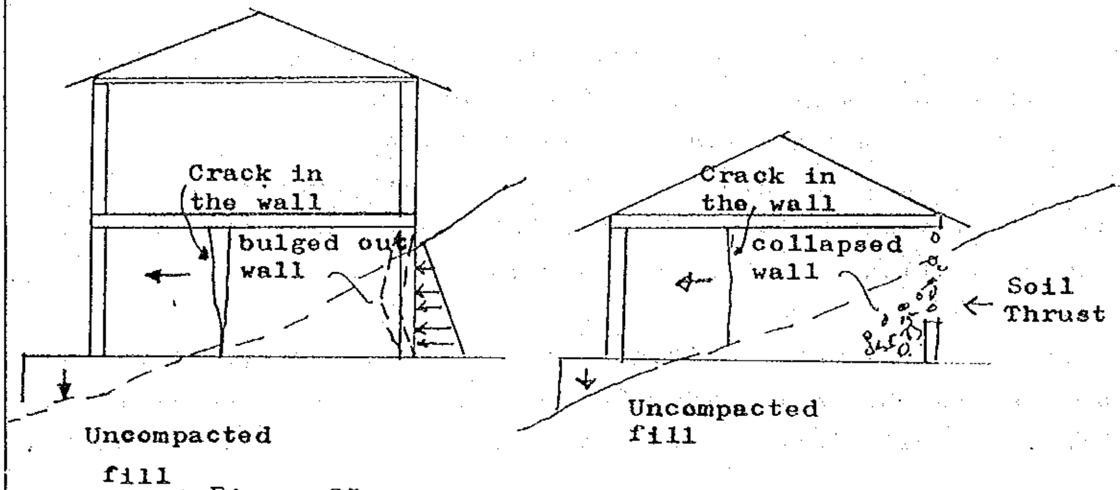


Figure-25

Back wall Damage in Houses on Sloping Ground

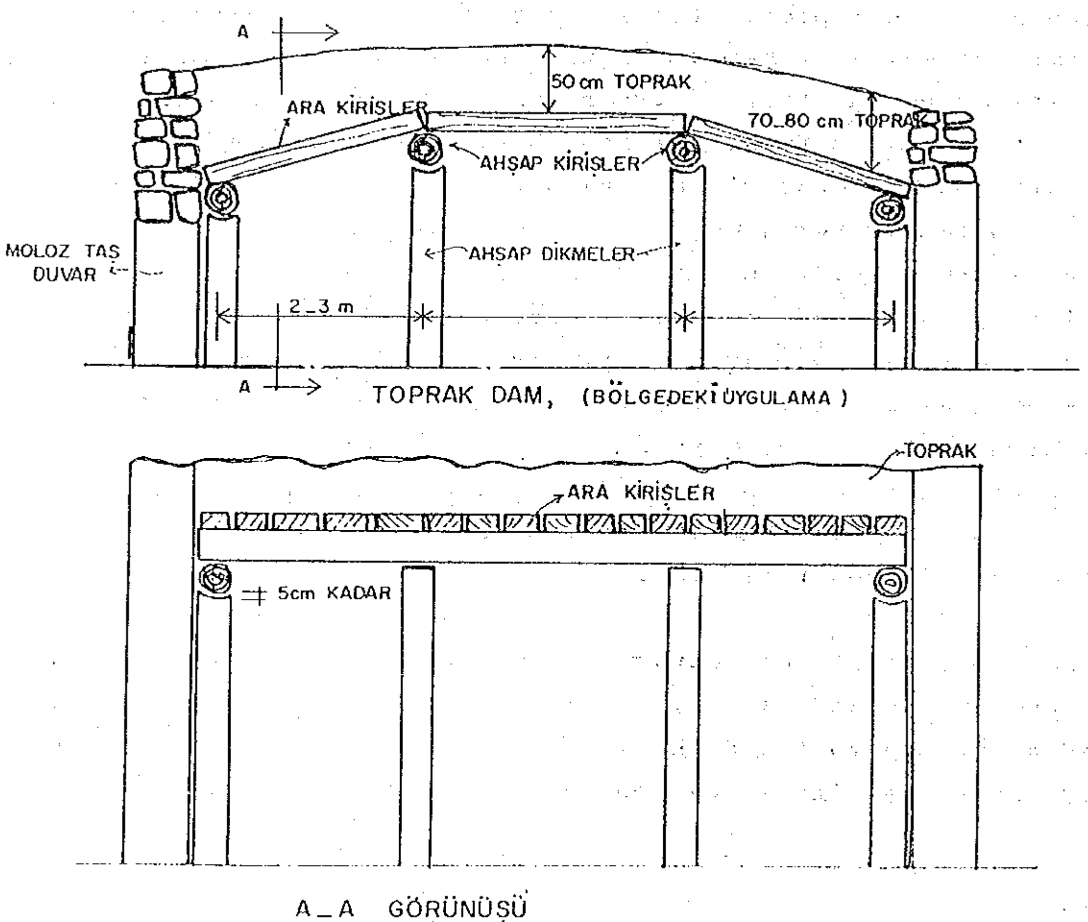


Figure-22 Flat Earth Roof Application in Eastern Anatolia

Thus as the years go by the soil layer on the roof gets thicker, denser and heavier. In the meanwhile the creep in timber increase and the timber roof beams start to sag. To prevent the collection of water in sagging part of the roof more soil is added to these places in order to have a flat roof as much as possible. All these operations weaken the structure.

Another factor which increase the weight of the roofs is the tradition of putting hay on top of the roof. This also gives more heat insulation. On the average 100-200 bundles of hay each weighing between 25 to 30 kg are placed on the roof. This adds 2.5-6.0 tons to the weight of the roof. Approximately 0.1-0.3 ton/m2.

In order to carry this roof load and the heavy snow fall, in the eastern Anatolia the roof beams are supported by timber posts at intervals and the roof is made dome like by putting higher posts in the middle and shorter posts near the walls. Figure-22. This in a way prevents the collection of water in the middle of the roof.

Unfortunately the connection of these posts to the roof beams are not properly done.. The tops of the posts have about 5 cm deep 'saddle' like depressions on top of which the roof beam rests. The post and the beam are not tied together by any metal elements. The beam rests on the post and against a lateral force there is only friction between the pos and the beam. As the beams are resting on the posts near the walls, the length of the beam resting on the wall is kept to a minimum. Sometimes they do not even supported by the walls. Thus the walls are not tied together by the roof and act as inverted pendulums. The roof supported by posts, on the other hand, is also like a pendulum.

Earthquake Behaviour

The period of vibration of single story rubble masonry houses are in the order of 0.10-0.20 seconds. They are relatively rigid structures. Damping ratios are generally high. Before damage damping ratios are about 4 -6 percent. Because of their initial rigidity they are subjected almost to the maximum ground acceleration.

The earthquake damage starts in the walls. Usually the X-cracking observed in brick masonry buildings walls due to shear stresses are not observed. Because the wall does not have a homogeneous structure like the brick wall. There is no continuity like brick-mortar-brick as in the brick walls. Rather the wall elements are at first 'loosened' by the earthquake. The open spaces between the stones are further opened.

As mentioned before, the wall does not have binding stones putting together the external and internal walls it is rather like a sandwich wall with the external and internal walls loosely connected. The damage starts first in the outer wall, the loosened stones start to fall downfollowed by falling down of the in the internal wall.

The beams carrying the flat earth roof are placed only in one direction. Thus the roof acts as if it is one directional slab. The load of the roof is transferred only to two opposite walls. The other walls are neither connected to the roof nor any load is tarnsferred to them. The walls carryihg the roof are also supported by their ends while the other roof behave like inverted pandelums. It is the such unconnected walls that are damaged first. The roof carrying walls, because of the weight of the roof is under somekind of stress and they usually exhibit more shear strength than the other walls. The collapse of the roof carrying walls takes place later during the earthquake.

The addition of new rooms to the already existing houses also creates walls unconnected to each other. In Figure-23 the walls carrying the roof with roof beams and the walls which are not carrying the roof and which are not connected to other walls as well are indicated. Such walls neither carrying the roof beams nor connected to other walls at the corners are the most critical walls of the houses. The damage starts with the seperation of these walls from other.

The presence of a timber or reinforced concrete tie beam in the wall prevents the propagation and enlargement of the cracks formeed in the wall. If the tie beams are not continous at the corners thair effectiveness is reduced. The walls begin to seperate at the corners. Reinforced concrete tie beams are more effective since the timber tie beams loose their strength with time.

In walls without tie beams if there are timber window cases of some strength, which are stronger than the walls of rubble masonry, the window case can carry the weight of the roof even if the rubble masonry wall has disintegrated thus preventing the collapse of the roof. They even prevent the disintegration of the wall to some extent.

The age of the building also has an important part on the level of damage. The deterioration in timber and the crumbling of the mud mortar in the wall all happen with time. While the roof also gets heavier with time.

Perhaps the best pictorial description of the damage to this kind of houses has been provided by Coburn and Hughes. In figure-24 the progressive damage of rubble stone masonry buildings are given. On this figure all kinds of damage described here are presented in pictures.

In villages on sloping ground the soil thrust on walls standing against the slope like a retaining wall sometimes damages the walls as shown on Figure-25. Houses on sloping ground are partially resting on fills. These fills usually settle during earthquakes and the front walls of the houses are damaged.

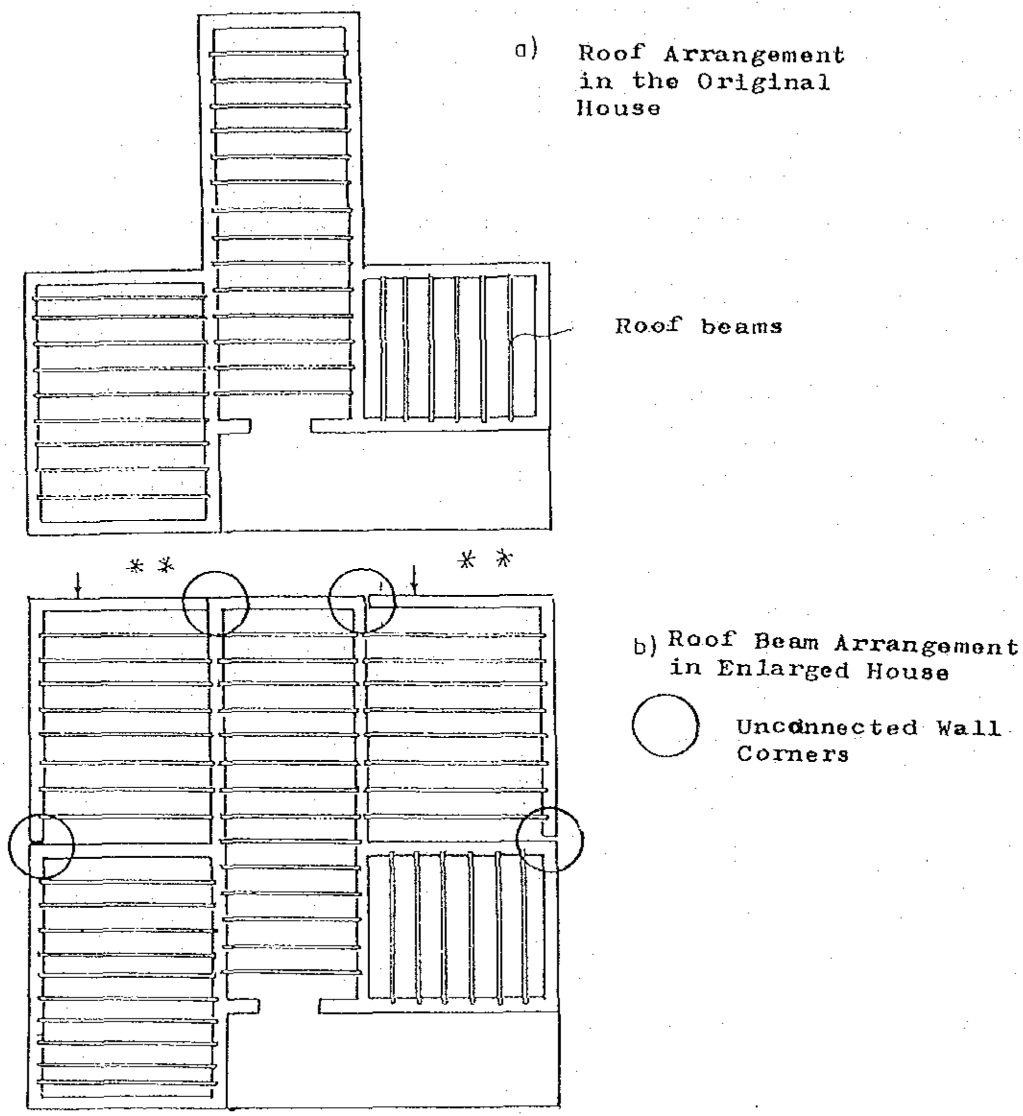


Figure-23 Roof Beam Arrangement of Rural Houses

Damage Ratio for Rubble Stone Masonry Houses

The houses built in rural areas of Turkey include timber frame, BAĞDADİ and HIMIŞ; masonry houses of adobe blocks, bricks, concrete blocks, stone. Their roofing system is quite important in the degree of damage. Houses with reinforced çoncrete slabs or timber roof trusses generally behave better than flat earth roofs. The kind of mortar used in stone masonry houses is also effective in prevention or limiting of damage. For these reasons the rural houses are classified as 'good' and 'bad' ones. The damage ratio of these buildings are given in Figures-26 and 27. Timber frame buildings, brick buildings, stone masonry buildings with cement and lime mortar and with reinforced concrete slabs or timber roof trusses, HIMIŞ buildings of well maintained ones, could be considered as 'good' rural buildings. The rubblestone masonry buildings with flat earth roofs, old HIMIŞ buildings of poor appearance, brick buildings of high walls and with timber roof trussess, brick buildings more than two stories high and with timber floor and roofs should be considered as 'poor' buildings. With these given properties the damage ratios for good and poor rural buildings can be applied.

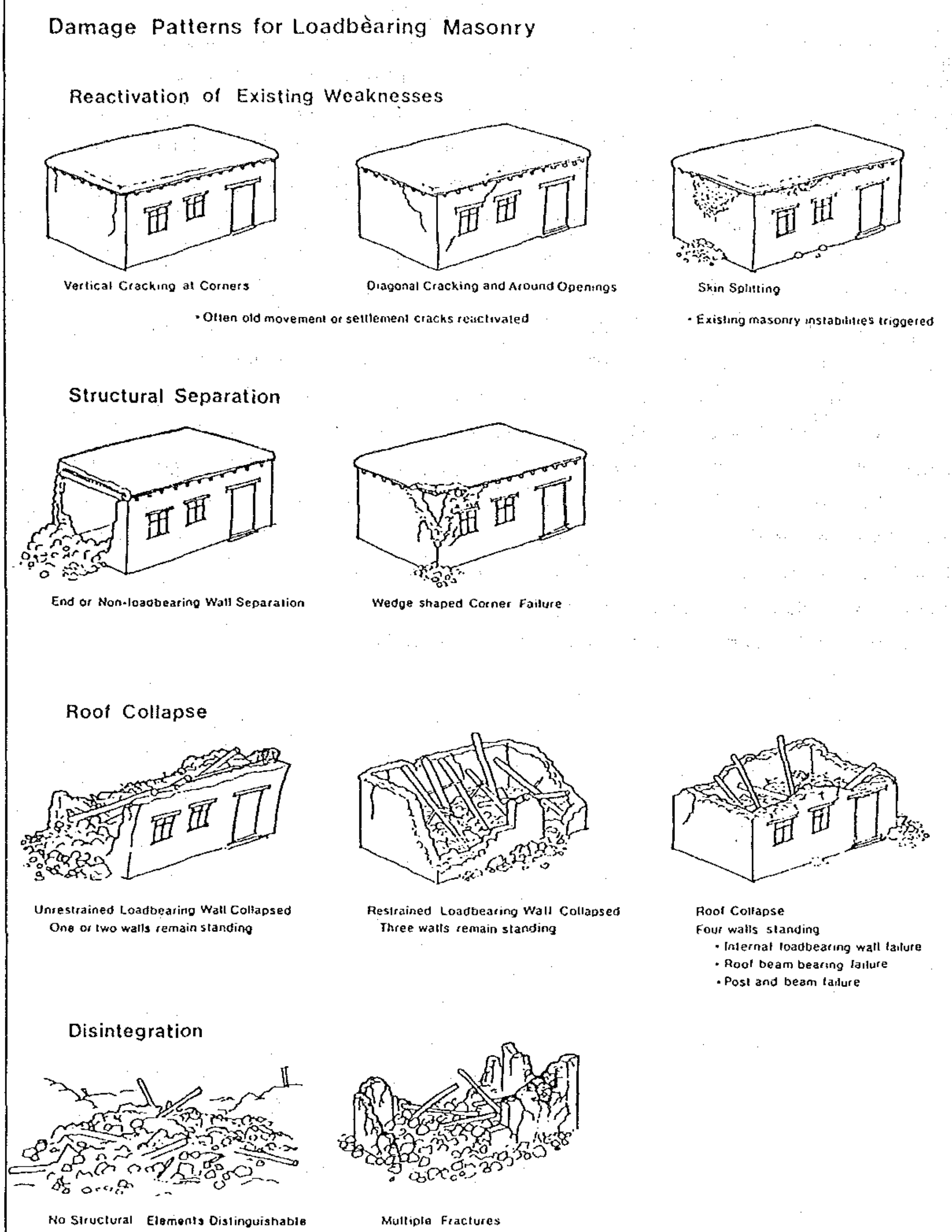


Figure-24 Damage Characteristics of Adobe and Rubble Stone Masonry Rural Houses (After Coburn and Hughes)

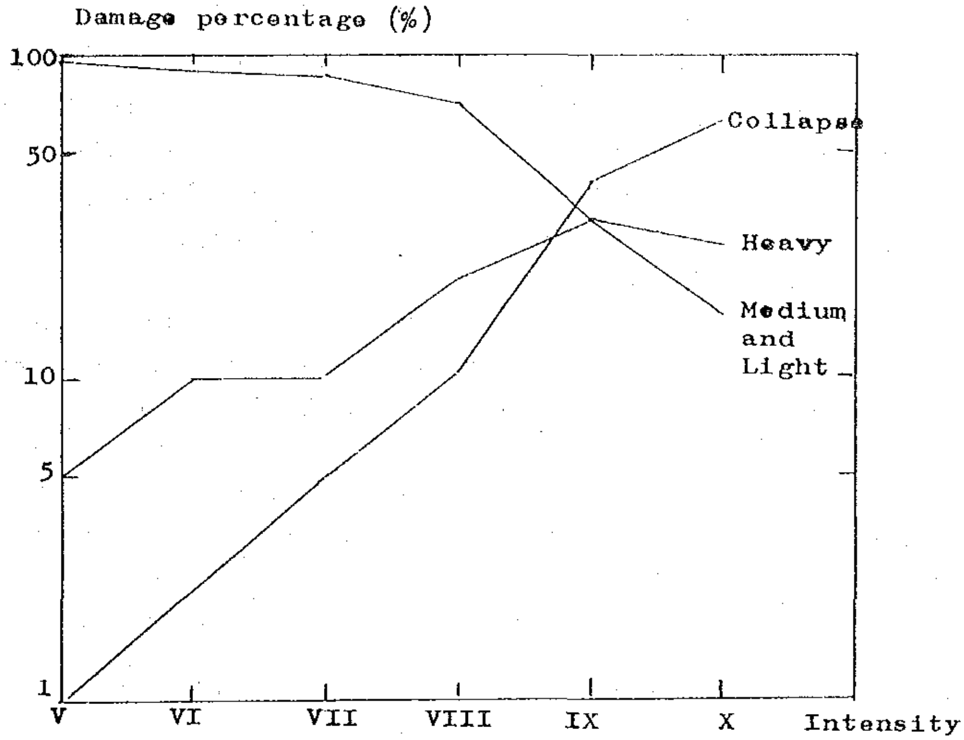


Figure- Damage Ratio for 'Good' Rural Houses

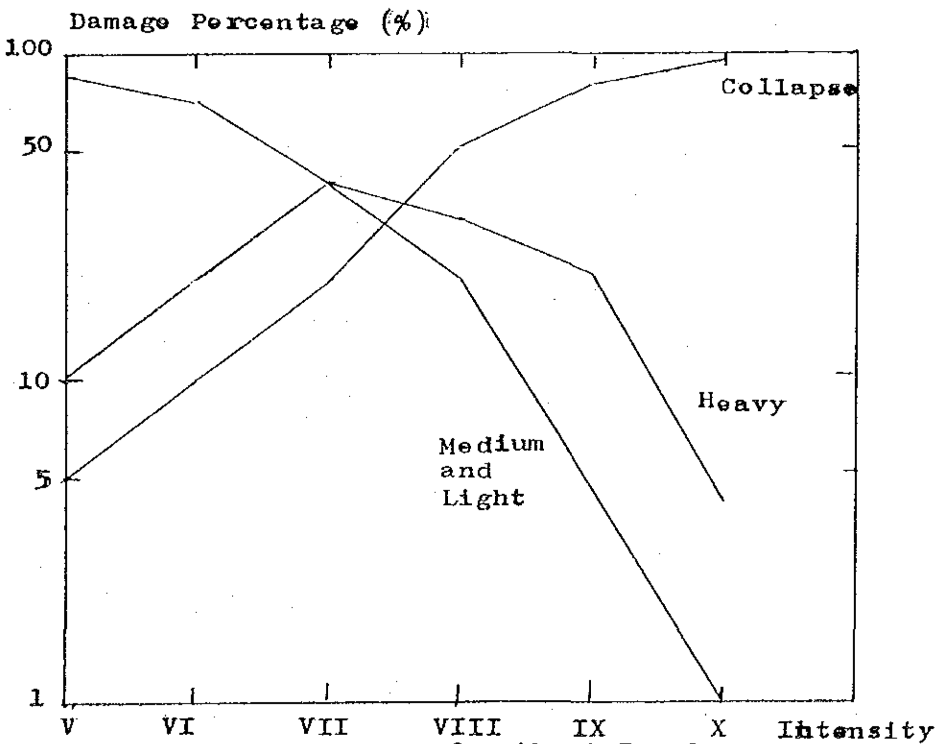


Figure- Damage Ratio for 'bad' Rural Houses

CONCLUSION

This paper should be taken as an attempt to describe the characteriztics of bulidings in Turkey and their earthquake damage and behaviour. The data provided here can be used to estimate the casualties and damage resulting from earthquakes in Turkey and to explain the behaviour of buildings in Turkish Earthquakes. The present condition of the buildings of all types in Turkey are highly vulnerable to earthquake damage and their damage potential could lead to very high casualties both in rural and urban areas. In fact the past earthquakes which had happened ir rural areas always confirmed this high casualty potential. As for the urban areas there is still to be an earthquake to show the potential dangers. The high levels of casualties and damages which had occured in rural areas so far did not create a serious concern for actions to reduce earthquake hazard perhaps a similar disaster occuring more densly populated urban areas could alarm the public.

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付属資料Ⅸ　トルコの建築耐震基準

トルコの建築物は、Ministry of Construction and Settlement (MCS)によって1975年に制定された耐震設計基準（Specitications for structures to be built in disaster areas, Part 1）に準じて設計されることになっている。現在トルコで建設される建築物の半数はこの基準に準じて設計されているようであるが、残りの半数はこの基準に違反しているとのことである。

基準の本文は末尾に添付している。以下にその概要を紹介する。なお、この基準が想定する設計法はいわゆる「存在応力設計法」である。鉄筋コンクリート造の建築物については、この基準の他に"Building Code Requirements for R／C"（TS500）という基準があり、そこでは終局強度型の設計法も許されている。

Specifications for Structures to Be Built in Disaster Areas, Part 1の概要

Section　1　適用範囲

〃　2　建設禁止地区

〃　13　耐震設計の方法

13.2.1　地震力は水平方向の静的な力とする。水平力は建物の主軸方向に作用するものとする。

13.2.3　地震力と風圧力の同時作用は考慮しなくてもよい。

13.3.1　建物を２つに区分する。

class a: 柱、梁、スラブから成る架構に、鉛直方向に耐震壁が連続して配置されるもの

class b: 上記以外のもの及び力学的不整形性をもったもの

13.3.2　動的解析をしなくて良いもの：－

class aで高さ75メートル以下のもの、組積造建築、煙突、塔、高架水槽

13.3.3　動的解析が必要なもの：－

class aで高さ75メートル超のもの、class bのもの

動的解析を行った場合でも、本基準に定める静的地震力の70％を下回る地震力を想定することはできない。

13.3.5　地震種別は４区分とする。→Table 13.1

13.3.6　MCSが指定する建築物の所有者は、強震計の設置に応じなければならない。

13.4　地震力は（13.1）式による。

13.5　地震力の鉛直方向分布は（13.8）式による。

13.6　捩りモーメント：各階の重心と剛心との距離に、水平力の作用方向に直交する方向の建物の最大寸法の５％を加算した偏心量によって生じる捩れモーメントに

耐えられるように設計すること。

13.7　建築物の部分及び付属物の設計用地震力は、(13.1) 式の地震力の 3 倍とする。

13.8　許容応力度（コンクリートの許容応力度は材料強度の⅓、鋼は½）

13.8.1　耐震設計においては、コンクリート及び鋼の許容応力度を33%増しとする。

13.8.3　Class Ⅰ、Ⅱ、Ⅲの地盤では、許容地耐力を33%増しとする。

13.9　擁壁等

Section 14　雑則

SPECIFICATIONS FOR STRUCTURES TO BE BUILT IN DISASTER AREAS

PART: I

GENERAL

Section 1 - Scope of Specifications

1.1. All government or private buildings to be altered, expanded, newly constructed or to undergo major repair work and located in disaster areas as determined by Section 2 of Act 7269 as revised by Act 1051, shall conform to the technical provisions stated in this specification in accordance with Section 3 of Act 7269 as revised by Act 1051.

1.2. The material and labour standards in buildings to be constructed in disaster areas shall conform to the provisions of the Turkish Standards and the "General Technical Specifications" of the Ministry of Public Works.

Section 2 - Land with Restricted Building Construction

2.1. No buildings or dwellings shall be constructed upon, nor shall existing buildings and dwellings located on land restricted for construction as per Section 14 of Act 7269 as revised by Act 1051. Furthermore, no buildings or dwellings shall be constructed upon artificial fills less than 30 years old, unless special subsoil compaction is provided.

2.2. In regions affected by at least one of the hazards of avalanche, rock-fall or land slide, and declared as hazard area in accordance with Section 2 of Act No.7269 as revised by Act No.1051, no buildings or dwellings shall be constructed, nor shall existing buildings or dwellings be repaired.

Section 13 - Method of Analysis for a Seismic Design of Buildings

13.1. Notation

C = Seismic coefficient

C_o = Seismic zone coefficient

D = Dimension of building in metres in a direction parallel to the applied earthquake forces

F = The total lateral force or shear at the base

F_i = Lateral force applied to level "i"

F_t = Concentrated lateral force considered acting at the top of the structure

G_i = Total dead load on "i"th floor

H = Height of the building in metres from foundation base

H_z = Thickness of underlying soil stratum

h_i = Height of "i"th floor above foundation base (metres)

I = Building importance coefficient

K = Coefficient related to structural type

N = Number of storeys in the building

N_{sp} = Number of blows, standard penetrations test

n = Live load factor

P_i = Total live load on "i"th floor

S = Dynamic coefficient for the structure (Spectral Coeff.)

T = Natural period of the building, sec.

T_o = Predominant period of underlying soil stratum, sec.

V_s = Shear-wave velocity, m/sec.

W = Total weight of building

W_i = Weight of "i"th floor.

13.2. General:

13.2.1. This section covers the principles of calculating the lateral forces to be used in the aseismic design of buildings in earthquake zones. Earthquake effects acting on buildings covered by this section shall be considered as lateral static forces applied at each floor level.

Lateral forces shall be assumed to act along the principal axes of the building in each direction, but not simultaneously in both directions. Such forces shall be distributed to the vertical bearing elements in proportion to their stiffnesses.

Where the principal axes of the vertical bearing elements do not lie along the principal axes of the building, the possibility of unfavourable conditions due to skew loading shall be investigated.

13.2.2. Lateral forces, whose principles of calculation are set forth in this section shall be taken as minimum loads acting on the whole of the structure.

13.2.3. In designing the elements, it shall be assumed that the structure is not subject to earthquake and wind loading simultaneously and the more unfavourable of the two loadings shall be considered in the design.

13.3. Definitions and Scope:

13.3.1. In this specification structures shall be considered in two classes as regards their principles of aseismic design are concerned

a) Structures with regular load bearing systems:

Structures where the bearing systems consist of slabs and/or beams with vertical columns, and where columns and shear-walls extend continuously down to the foundation level.

b) Structures with irregular load bearing systems:

Structures not falling within the scope of the definitions given for class (a) structures and structures having discontinuously or irregularly (to distributed) rigidity and mass.

13.3.2. All reinforced concrete or steel frame structures with regular load bearing systems not higher than 75 m above foundation base, all masonry buildings, chimneys, towers and elevated tanks may be designed using the lateral loads stipulated in this section in the absence of a rigorous dynamic analysis. RC/S

13.3.3. Where the structure has an irregular load bearing system or the clear height of the structure above the base level exceeds 75 m, such structures shall be designed against earthquakes using an appropriate and rigorous dynamic analysis.

Such analysis shall comprise the complete dynamic properties of both the structure and the underlying soil. Modal superposition method based on real or idealised spectra, integration of the equation of motions with respect to time etc, or experimental model analysis are acceptable methods of solution. However, the total lateral load values calculated as a result of such dynamic analyses shall not be less than 70% of the lateral load values determined using the method of calculation of this section.

13.3.4. The aseismic design calculations of bridges, three dimensional space structures, gravity type dams, tunnels, underground structures, and of surface load carrying structures such as domes, shell roop and arch dams, are not covered by this specification.

13.3.5. In the calculation of lateral loads acting on structures in accordance with the principles of this specification, four separate soil groups are defined and the characteristics of each group listed in Table 13.1.

13.3.6. Owners of buildings in earthquake risk areas shall permit the installation of the required number of strong motion accelerographs in such buildings as may be designated by the Ministry of Construction and Settlement.

13.4. Calculation of Total Lateral Load:

The total lateral equivalent statical load to be used in the aseismic design of buildings is given by:

$$F = C\,W \qquad (13.1)$$

where C = seismic coefficient, and is to be calculated by

$$C = C_o\,K\,S\,I \qquad (13.2)$$

in which C_o = seismic zone coefficient
K = coefficient related to structural type
S = dynamic coefficient for the structure (spectral coefficient)
I = building improtance coefficient.

TABLE 13.1

SOIL CLASSIFICATION TO BE USED IN DETERMINATION OF PREDOMINANT PERIOD OF VIBRATION

Soil Class	Identification	N_{sp} Number of blows standard penetration test	D_r Relative Compaction %	q_u Unconfined compressive strength kg/cm^2	V Shear Wave Velocity m/sec
I	a) Massive volcanic rocks and deep bedrock, undecomposed sound metamorphic rocks, very stiff cemented sedimentary rocks	-	-	-	
	b) Very dense sand	>50	85-100	-	>700
	c) Very stiff clay	>32	-	>4.0	
II	a) Loose magmatic rocks such as tuff or agglomerate, decomposed sedimentary rocks with planes of discontinuity	-	-	-	
	b) Dense sand	30-50	65-85	-	400-700
	c) Stiff clay	16-32	-	2.0-4.0	
III	a) Decomposed metamorphic rocks and soft, cemented sedimentary rocks with planes of discontinuity	-	-	-	
	b) Medium dense sand	10-30	35-64	-	200-400
	c) Medium stiff clay, silty clay	8-16	-	1.0-2.0	
IV	a) Soft and deep alluvial layers with a high water-table, marshland or ground recovered from sea by mud-fill, all fill layers	-	-	-	
	b) Loose sand	0-10	$\leq$ 35	-	< 200
	c) Soft clay, silty clay	0-8	-	$\leq$ 1.0	

13.4.2. The seismic zone coefficient, C_o, is given below, in Table 13.2:

TABLE 13.2

VALUES OF SEISMIC ZONE COEFFICIENT

Seismic Zone	C_o
1	0.10
2	0.08
3	0.06
4	0.03

13.4.3. The values of K, structural type coefficient, are given in Table 13.3.

TABLE 13.3.

STRUCTURAL COEFFICIENT

Structure Type		K
All building framing systems except as hereinafter classified		1.00
Buildings with box systems with shear-walls		1.33
Buildings with frame systems where the frame resists the total lateral force (see note (2) for filler-wall types a, b and c).		
1. Ductile moment resisting frames (1) (steel or reinforced concrete)	a)	0.60
	b)	0.80
	c)	1.00
2. Non-ductile moment resisting frames	a)	1.20
	b)	1.50
	c)	1.50
3. Steel space frames with diagonal bracing	a)	1.33
	b)	1.50
	c)	1.60
Shear-wall systems with ductile frames capable of resisting at least 25% of the total lateral forces	a)	0.80
	b)	1.00
	c)	1.20
Masonry buildings		1.50
Elevated tanks not supported by a building		3.00

cont'd.

Structure Type	K
Structures other than buildings, towers and chimney stacks	2.00

Notes: 1) See section 6.2 for the definition of "ductile frames"

2) Filler-wall types:

a) Reinforced concrete or partition walls of masonry blocks with horizontal and vertical reinforcement.

b) Unreinforced masonry block partition walls

c) Light and sparse partition walls or prefabricated concrete partition walls.

13.4.4. The spectral coefficient shall be calculated by:

$$S = \frac{1}{|0.8 + T - T_o|} \qquad (13.3)$$

where T = Natural period of the structure in the first mode (sec)
T_o = Predominant period of underlying soil (sec).

The value of S calculated by Eq. (13.3) shall not be taken larger than 1.0 (x)

Note: In all one or two storey buildings the value of S shall be taken as 1.0 and the structural coefficient K shall be not less than 1.0. In masonry buildings S shall be taken as 1.0.

13.4.5. Unless calculated by experimental or theoretical methods, based on valid assumptions, the value of T, the natural period of the structure, shall be calculated by both of the following approximate relations;

$$T = \frac{0.09\ H}{\sqrt{D}} \qquad (13.4)$$

$$T = (0.07 - 0.10)N^{xx} \qquad (13.5)$$

and the less favourable value of T shall be used in Eq. (13.3). In Eqs. (13.4) and (13.5)

(x) Curves obtained from Eq. (13.3) are plotted in Fig. (13.1).

(xx) The value of the coefficient for Eq. (13.5) shall be determined by interpolation between the values 0.07 and 0.10 according to the degree of general structural flexibility.

H = height of structure above base level (metres)
D = dimension of building in a direction parallel to the applied lateral forces (metres)
N = number of storeys above foundation level.

Note: Eqs. (13.4) and (13.5) shall not apply to; industrial structures with large spans, cinemas, sport halls and stadiums, etc., buildings with regular bearing systems but with a height more than 35.0 m above foundation level, chimney stacks, towers, elevated tanks. The natural periods of such structures shall be calculated through a rigorous dynamic analysis where the properties of the soil and the structure (soil-structure interaction) are taken into consideration.

13.4.6. Unless determined by experimental, empirical or theoretical principles based on valid assumptions and geological observations, the value of T_o may be selected from Table 13.4. These values are valid only for the case where the top layer of soil directly above bed-rock or other formations with similar characteristics has a thickness of the order of 50.0 m. Where the thickness of the top layer of soil is greatly different than 50 m, the values of the shear-wave velocity, V_s (m/sec) and the thickness of the top layer, H_z, (metres) shall be determined more accurately by experimental, empirical or theoretical methods. In this case, the value of T_o shall be calculated by the equation $T_o = 4H_z/V_s$. Where the value of V_s cannot be determined accurately for use in the formula given above, the values of V_s given in Table 13.1 may be used.

Where the underlying soil consists of a number of layers with different values of V_s, a separate value of T_o shall be calculated for each and every layer.

Soils that have a V_s value larger than 700 m/sec shall be assumed to be very sound and layers below the depth where this value is exceeded shall not be taken into consideration.

TABLE 13.4

Soil	Class	T_o Predominant Period of soil (sec)	T_o Average (sec)
I	a b c	0.20 0.25 0.30	0.25
II	a b c	0.35 0.40 0.50	0.42
III	a b c	0.55 0.60 0.65	0.60
IV	a b c	0.70 0.80 0.90	0.80

Note: For structures defined below, appropriate and sufficient seismic and subsoil exploration along with laboratory experiments shall be carried out for the accurate solution of soil related problems such as foundation type, bearing capacity and settlements, and also to be able to determine as realistically as possible the predominant period of vibration of the soil layer.

i) Buildings having a height more than 75 meters above foundation level

ii) Industrial structures with large spans, buildings such as theatres, cinemas

iii) Towers, chimney stacks, elevated tanks etc.

13.4.7. The structure importance coefficient I is given in Table 13.5.

TABLE 13.5

STRUCTURE IMPORTANCE COEFFICIENT

Structure Type	I
a) Structures and buildings to be used during or immediately after an earthquake (Post office, fire stations, broadcasting buildings, power stations, hospitals, stations and terminals, refineries, etc.)	1.50
b) Buildings housing valuable and important items (museums, etc.)	1.50
c) Buildings and structures of high occupancy (schools, stadiums, theatres, cinemas, concent halls, religious temples, etc.)	1.50
d) Buildings and structures of low occupancy (private dwellings, hotels, office buildings, restaurants, industrial structures, etc.)	1.00

13.4.8. The value of the seismic coefficient shall in no case be taken less than $C_o/2$.

13.4.9. The total structure weight to be used in the calculation of the total lateral load shall be calculated by

$$W = \sum_{i=1}^{N} W_i \qquad (13.6)$$

where, W_i, weight of "i"th floor, is given by:

$$W_i = G_i + nP_i \qquad (13.7)$$

in which, G_i = total dead load on "i"th floor
P_i = total live load on "i"th floor.

The values of n, the live load factor, are given below in Table 13.6:

TABLE 13.6

LIVE LOAD FACTOR

Type of Structure	n
Warehouses, etc.	0.80
Schools, student housing buildings, stadiums, cinemas, theatres, concert halls, garages, restaurants, commercial establishments, etc.	0.60
Private dwellings, hotels, hospitals, office buildings, etc.	0.30

13.5. Vertical Distribution of Lateral Loads:

13.5.1. Lateral loads F_i to be applied at floor levels of the structure shall be calculated by the relation:

$$F_i = (F - F_t) \frac{W_i h_i}{\sum W_i h_i} \qquad (13.8)$$

in which F = *total lateral load (base shear)*
W_i = *weight of "i"th floor*
h_i = *elevation of "i"th floor above foundation level*
F_t = *concentrated lateral load acting at the top of the structure.*

The value of F_t *shall be calculated as follows:*

$$F_t = 0.004\ F(H/D)^2 \qquad (13.9)$$

Note: 1) The value F_t *shall in no case exceed* $0.15F$

2) Whenever the ratio H/D is equal to or smaller than 3, then F_t *may be assumed as zero*

i.e. for $H/D \leq 3$, $F_t = 0$

13.5.2. For tall structures such as towers, chimney stacks, the total height of the structure may be subdivided into a sufficient number of portions whereby Eqs. (13.8) and (13.9) become applicable.

13.5.3. For elevated tanks, the value of C as calculated by Eq. (13.2) shall be not less than 0.12 nor greater than 0.25, and the total lateral load shall be assumed to act as a single load at the centre of gravity of the tank.

13.6. Horizontal Torsional Moment:

Buildings shall be designed to resist torsional moments due to an eccentricity in each direction calculated as the difference between the centres of mass and stiffness of any floor, plus 5% of the largest plan dimension of the building perpendicular to the direction of the lateral loads.

13.7. Parts and/or Portions of Buldings:

Earthquake loads acting on parts and/or portions of buildings such as parapet walls, chimneys, cantilever parts, and balconies shall be calculated separately. In these calculations the coefficient C as determined for the structure by Eq. (13.2) shall be increased threefold and the lateral load F, determined by Eq. (13.1) shall be assumed to act at the centre of gravity of the part or portion in the more unfavourable direction.

13.8. Allowable Stresses:

13.8.1. In the aseismic design of sections the allowable stresses for concrete and steel may be increased by not more than 33%.

13.8.2. In reinforced concrete structures an increase in bond stresses shall not be permitted. In steel structures allowable stresses for all connections and joints shall not exceed the values for increased allowable stresses. The same shall apply to the design of diagonal wind bracing and stability members.

13.8.3. Whenever the effects of earthquake are considered, the allowable bearing pressures for subsoils may be increased by not more than 33% in soils of class I, II and III. No such increase shall be permitted for class IV soils.

13.8.4. Where the top layer of soil is of class II, III or IV soil, possible settlements and/or differential settlements due to seismic vibrations should be determined in addition to those settlements due to static loads, so as to be incorporated into the calculations.

13.8.5. In foundations bearing directly on class IV soils no increase in allowable stresses for concrete and reinforcing steel shall be permitted.

13.9. Retaining Walls and Sheet-pile Walls:

13.9.1. For the design of retaining walls and sheet-pile walls in earthquake zones with heights in excess of 6.00 m, the characteristics of the soil shall be determined by appropriate laboratory and field testing.

13.9.2. In the calculation of earth pressure, the angle of shear strength shall be decreased by;

6^{o} in 1st and 2nd degree earthquake zones,
4^{o} in 3rd and 4th degree earthquake zones.

Section 14 - Miscellaneous Provisions

14.1. All sections of this specification shall be considered to be interrelated. Furthermore, Part III, "Protection Against Earthquakes" shall be applied in conjunction with Section 4, "Protection Against Fire".

14.2. "Specifications for Buildings to be Built in Disaster Areas" as published in the official gazette No. 12801 on 16/1/1968, is no longer in effect.

14.3. This specification shall be effective as of 9/8/1975.

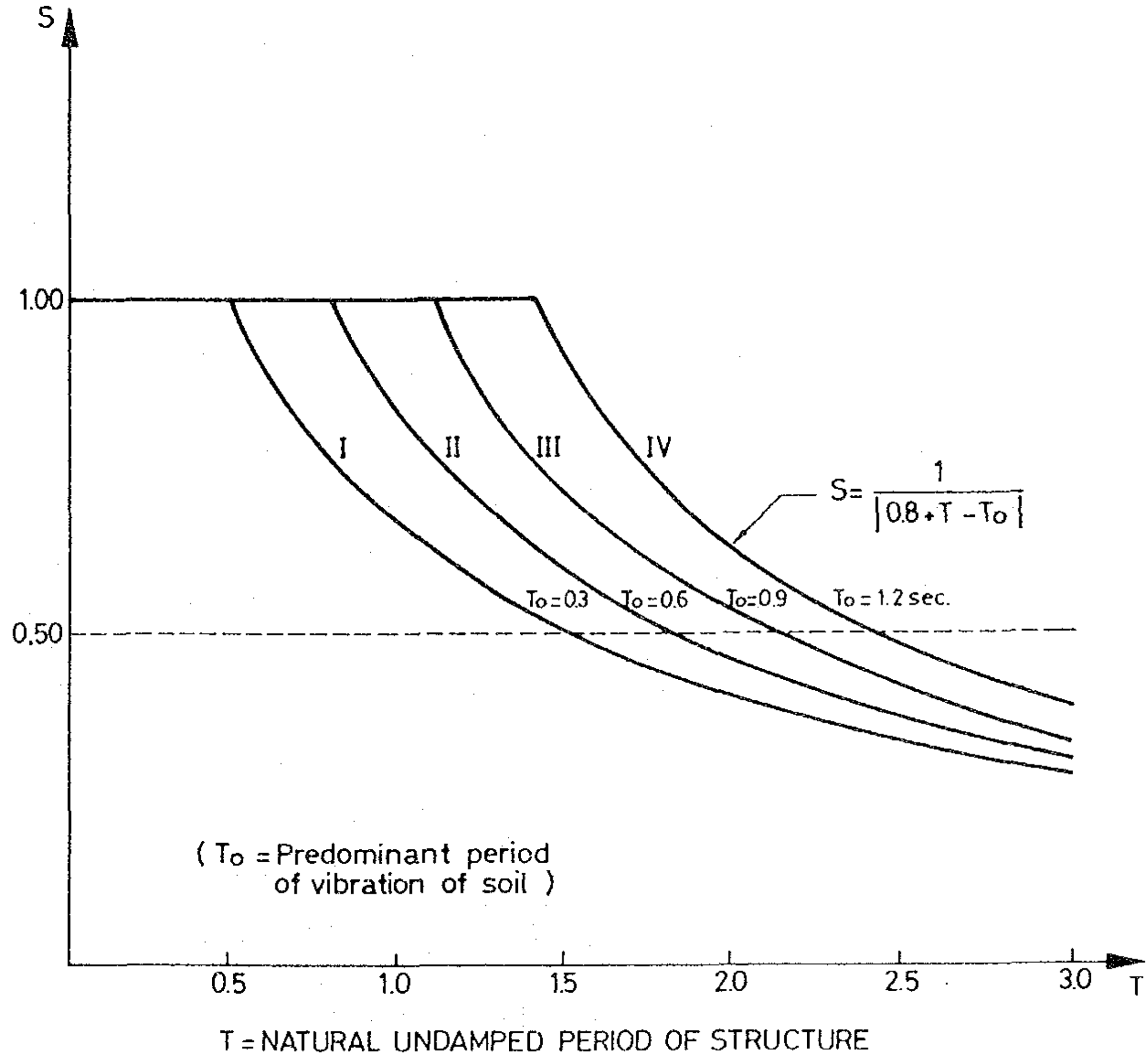
S
1.00
0.50
I
II
III
IV
S= 1 / |0.8+T −To|
To=0.3
To=0.6
To=0.9
To=1.2 sec.
(To = Predominant period
of vibration of soil)
0.5
1.0
1.5
2.0
2.5
3.0
T
T = NATURAL UNDAMPED PERIOD OF STRUCTURE

FIGURE 13.1