17 AUGUST MARMARA EARTHQUAKE AND THE PRECAST CONCRETE STRUCTURES BUILT BY TPCA MEMBERS

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Introduction

An earthquake of magnitude 7.4 on the Richter Scale occured on 17 August 1999 in the Marmara Region of Turkey. The epicenter of the earthquake is approximately 12 km. southeast of the city of Gölcük. It was originated at a depth of 17 km. and caused a fault movement of 120 km.(1) The assessment done in the following days of the earthquake, indicated that the Marmara earthquake, (officially called Kocaeli earthquake) in fact, comprised of 5 different tremors. It is stated that instead of accepting it as a single earthquake of 7.8 magnitude, it was possible to consider it as 5 different quakes with 7.0 magnitude each.

Field studies revealed that the stress distribution reached IX and X levels according to EMS 1992 (European Macroseismic Scale). It is verified both by the assessment report and the field studies that in destruction levels as such, multi-storey reinforced concrete frames are subjected to damage beyond repair, i.e. % 65 and % 50.

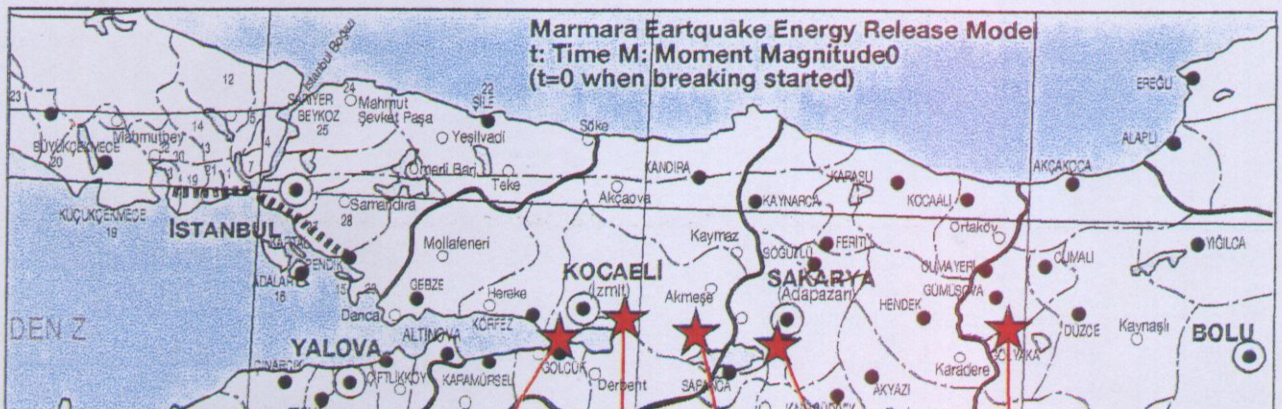


***Figure 1** Kocaeli (Marmara) Earthquake*

Statistical data about the earthquakes that took place in Turkey during the last 12 years (including the Marmara Earthquake) is given in Table 1.

Table-1 *Destructive earthquakes in Turkey in the Last 10 Years*

Location	Magnitude	Seismic Moment	Greatest Acceleration
ERZİNCAN 13.03.1992	6.8	$2.94X10^{18}$	0.50 g
DİNAR 01.10.1995	6.0	$3.80X10^{17}$	0.30 g
CEYHAN 27.06.1998	5.9	$2.94X10^{17}$	0.22 g
KOCAELİ 17.08.1999	7.4	$3.78X10^{19}$	0.32 g

Marmara earthquake which occured as a result of the movement of North Anatolian Fault Zone effected an area of 240 km. long, strecthing from Düzce to Avcılar in İstanbul. This area is densely populated with industrial developments. Especially the section between İzmit and Adapazarı contains almost all types of precast concrete industrial buildings.

WORKS UNDERTAKEN BY TPCA AFTER THE EARTHQUAKE :

The following action plan is prepared in connection with the precast concrete buildings that are effected by the earthquake in the region.

1. Survey of the buildings that are built by the members of Turkish Precast Concrete Association.
2. Accumulation of the follwing data and documents of the precast concrete buildings that are damaged or collapsed by the earthquake :
   2.1. Engineering projects.
   2.2. As-built material information.
   2.3. Collection of geological and soil information.
3. Obtaining the acceleration records.
4. Field investigation of damaged/collapsed buildings.
5. Identifying the causes of damage/collapse.
6. Reporting to the members.
7. Informing the public.

INVENTORY OF BUILDINGS BUILT BY TPCA MEMBERS IN THE EARTHQUAKE REGION

Results of the survey carried out by TPCA concerning the numbers and the condition of the precast concrete structures built by member firms are given in Table 2.

Table-2 *TPCA Regional Precast Concrete Building Survey*

Location	Completely Damaged	Partly Damaged	Undamaged	Total	Share Damage Bld.
AVCILAR	-	-	54	54	0.-
İZMİT	1	5	235	241	2.50
GÖLCÜK	-	-	35	35	0.-
YALOVA	-	1	49	50	2.0
ADAPAZARI	16	8	74	98	24.50
BOLU	-	-	2	2	0.-
DÜZCE	-	-	1	1	0.-
TOTAL	**17**	**14**	**450**	**481**	

Table 3 summarizes the detailed analysis given in Table 2.

Table-3 Buildings built by TPCA Members

Total No. of bldgs.	481	% 100
Completely Damaged bldgs.	17	% 3.50
Partly Damaged bldgs.	14	% 3.00
Undamaged bldgs.	450	% 93.50

SITE INVESTIGATION AND DESIGN REVIEWS

Data collected in the earthquake region and the field surveys indicated that precast concrete buildings built by TPCA members were most effected by the earthquake in Adapazarı region. There fore, it is decided that further studies are concentrated in Adapazarı Industrial Zone which contained the highest number of damaged/collapsed precast concrete buildings.

Priority is given to the analysis of soil conditions and it is found out that water saturated sand-silt is the predominant soil formation. This type of soil is identified as group D, and classified as Z4 soil. (Fig.2)

Analysis of damaged/collapsed buildings showed that there are four distinct groups in terms of building types and the types of collapse. A typical building for each group is selected as a case study. Then, the structural design, site conditions and material properties of each case study is examined and evaluated.

The buildings are coded as A, B, C and D. For each building the loads that caused collapse and the final service loads are calculated. In

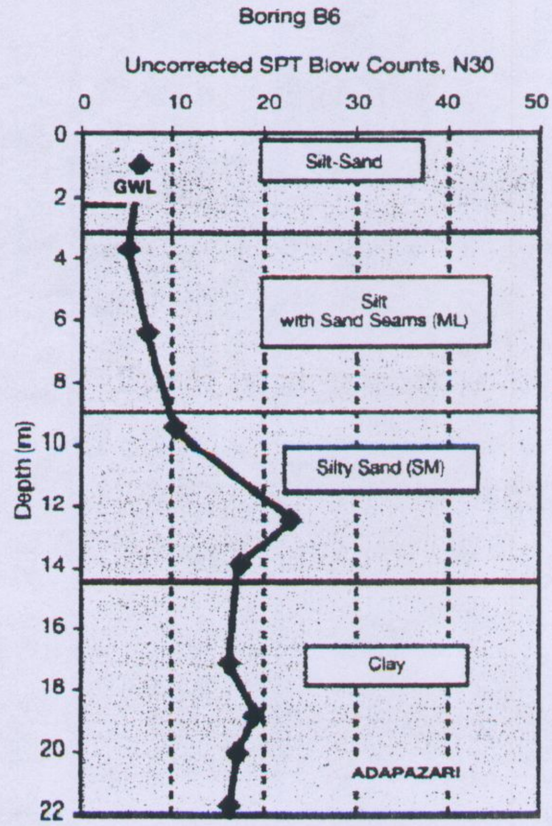


Figure 2 : Borehole log in Adapazarı

the calculations, the earthquake codes, both existing and the one in construction year, are considered.

BUILDING TYPE A :

The basic module is 20.00 m x 7.50 m and the height is 7.00 m. In both directions, hinge joints (with double pins) are used for columns and beams. Site investigations revealed that :

a) It collapsed completely due to breakage and tilting.
b) Direction of collapse is perpendicular to the main axis beams and the building collapsed after the inner columns broke off.

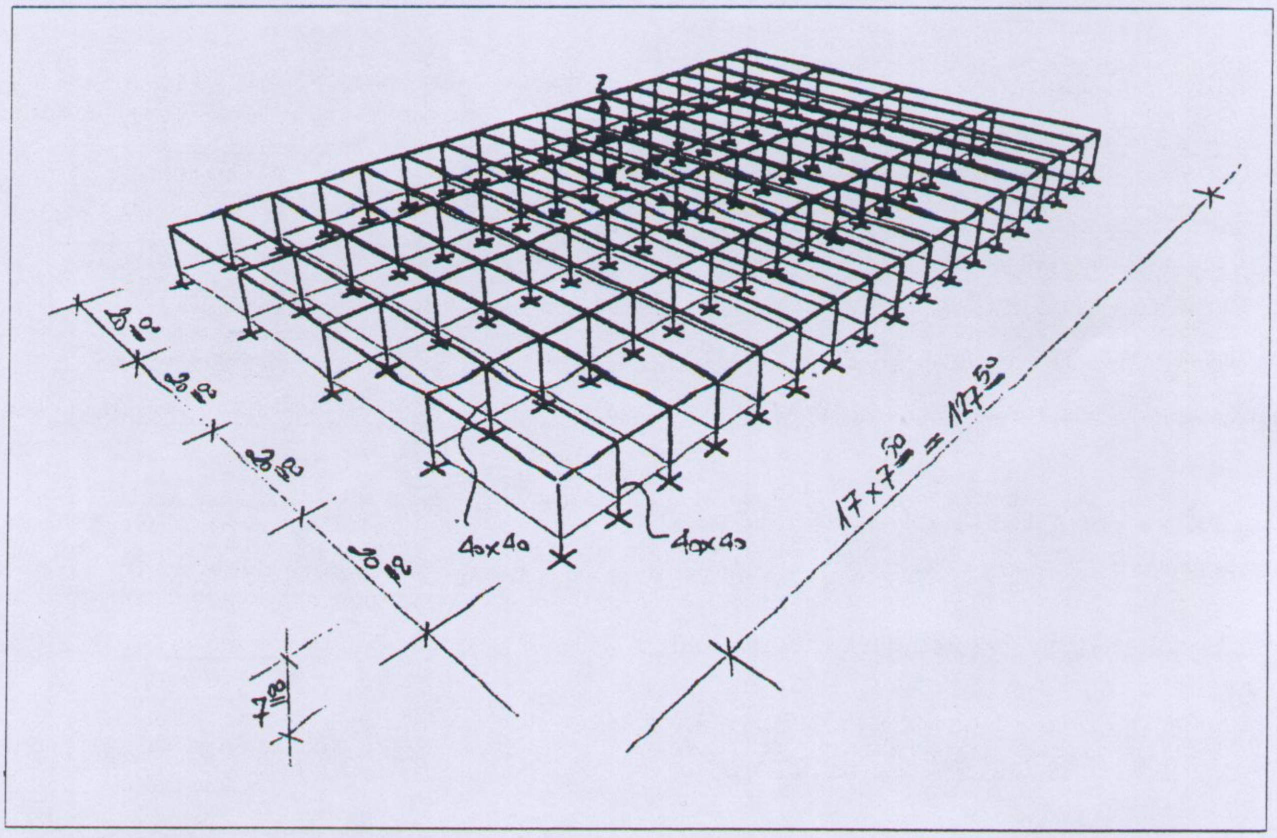


Figure 3: *Building Type A.*

c) Column sections are measured 40 cm x 40 cm.
d) Column reinforcements are placed asymmetrically.
e) All connections are hinge joints; roof beams being connected with double pins.
f) There isn't any deformation or failure in column-socket connections.

Both design and construction of the building was done in 1998. So, structural design is reviewed in complience with 1997 Earthquake Code.

The analysis proved that the main reason for collapse was "inadequate lateral rigidity". Following factors are identified as other reasons :

1. Non-complience with the Code.
2. Improper selection of design loads.
3. Not conforming with displacement limits.
4. Underrating the soil characteristics.
5. Asymmetrical positioning of column reinforcements.

Table-4 *Building Type A 1. Mode Displacements*

Column	Under Ultimate Loads (cm)	Under Service Loads (cm)	Max. Permitted by Code (cm) (0.0035xh)
PC Direction	4.47	5.57	2.45
PC Direction	3.25	4.45	2.45
IC Direction	4.47	5.57	2.45
IC Direction	4.90	6.34	2.45

PC: Peripheral Column IC: Inner Column

Table-5 Building Type A Column Dimensions

Column	As Built Size (cm)- Reinforcement (%)	acc. to Code (*) Size(cm) Reinforcement (%)
Peripheral	40 x 40 Pt→ % 2.60	50 x 50 Pt→ % 1.20
Inner axis	40 x 40 Pt→ % 2.60	55 x 55 Pt→ % 1.40

(*) Code 1997; For A_0=0.40 g, Z_4 and R=5

Table-6 Building Type A Damage Ratio

	Damaged	Undamaged	Total	Damage Ratio
No. of Bldgs.	4	-	4	% 100

Photograph 1 Column in Building Type A

BUILDING TYPE B :

As illustrated in Fig. 4., the basic module in this building is again 20.00 m x 7.50 m and its height is 7.00 m Connection detail is similar to building type A, but in this case, a groove is introduced. The building has a mezzanine floor and is built before 1997. Following points are noted in the site investigation :

a) Collapse was a result of breakage and tilting.
b) Collapse started with inner columns giving away and direction of collapse was perpendicular to the main beams.
c) Column sections are 40 cm x 40 cm and 35 cm x 35 cm
d) All connections are hinge type connections. Roof beams are connected with a single pin.
e) Column beam and column gutter beam connections failed during the earthquake and some of the elements fell down.
f) No deformation or failure have been observed in column-socket connections.

After the site investigation, the structural design is reviewed. Noting that the design was done in 1996, it had to comply with the 1975 Code. Analysis of the design showed in this case too, that inadequate lateral rigidity was the main cause of collapse. Following points, contributing to the collapse, have been identified.

1. Displacement limits of the 1975 Code are not complied with.
2. Inability of the peripheral beams to stand lateral forces due to poor connection details.

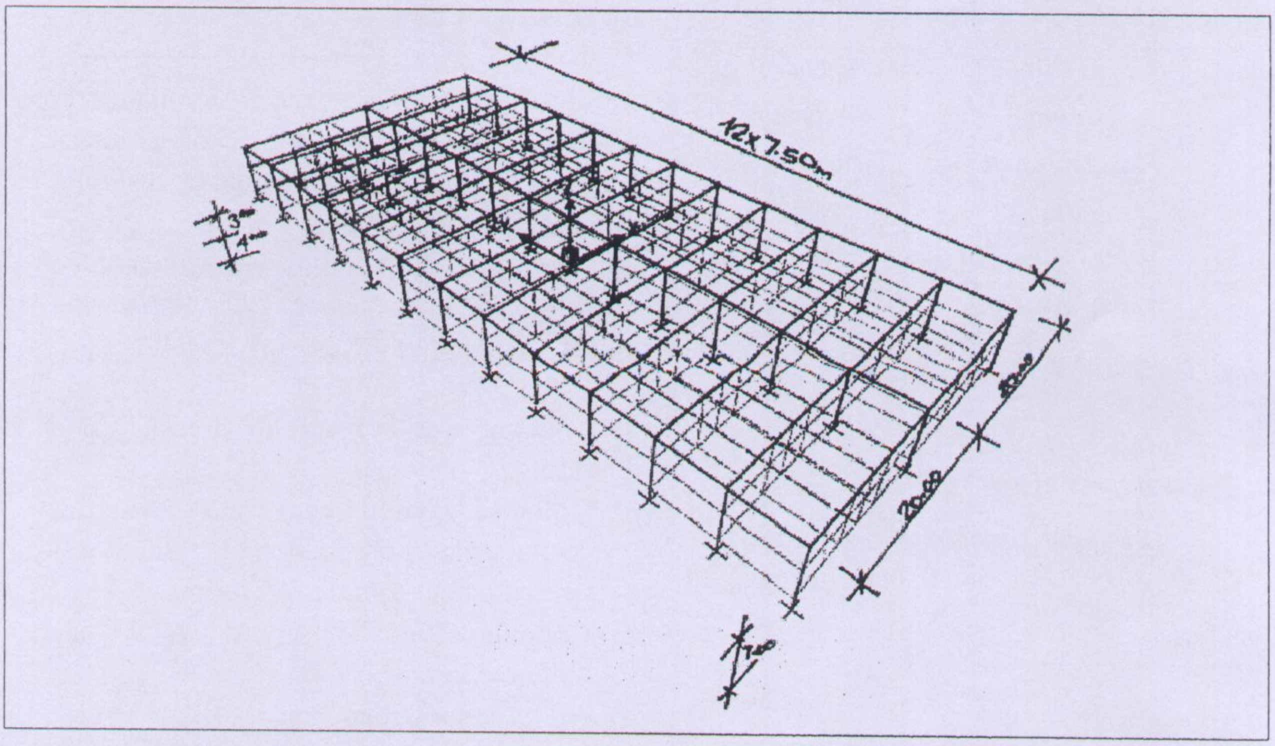


Figure 4 Building Type B

Table-7 Building Type B 1. Mode Displacements

Column	Under Ultimate Loads (cm)	Under Service Loads (cm)	Max. Permitted by Code (cm) (0.0025xh)
LC X axis	5.60	6.90	1.75
LC Y axis	3.70	4.60	1.75
SC X axis	1.70	2.10	0.75
SC Y axis	1.50	1.90	0.75

LC: Long columns SC: Short columns

Table-9 Comparison of Horizontal shear under equivalent seismic loads

	Existing Design	1997 Code (*) Requirements
X axis	0.07	0.114
Y axis	0.07	0.122

(*) Code 1997; For V_1=WA(T_1)/Ra(T_1)
Ra(T_1)=R=5 A_0=0.40 g
Soil class $Z_4 \rightarrow T_x$=1.82 s T_y=1.68 s.

Table-8 Building Type B Column Dimensions

Column	As Built (cm)	Min. Dimension(*) acc. to Code (cm)
Peripheral	35x35	50x50
Inner axis	40x40	55x55

(*) Code 1997; For A_0=0.40 g, Z_4 and R=5

Table-10 Building Type B Damage Ratio

	Damaged	Undameged	Total	Damage Ratio
No of Bldgs.	5	3	8	% 62

Photograph 2 Building Type B

BUILDING TYPE C :

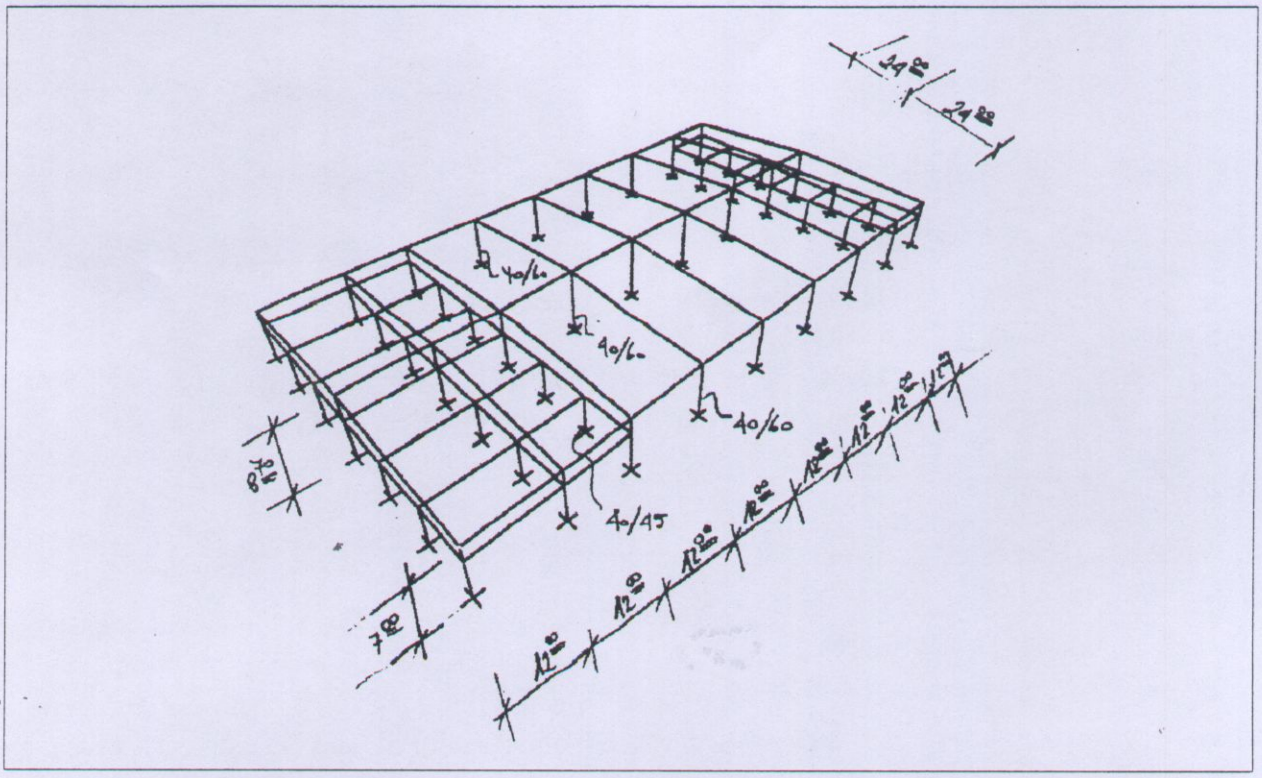


Figure 5 : Building Type C

In this case study, the basic module is 24.00 m x 12.00 m. Building height is 7.00 m around the periphery and 8.70 m along the central axis. (See Fig.5)

The system is composed of hinge joints in both directions (beam to beam connections by tie-rods and beam to column connection with concrete corbel with a pin.) Site analysis indicated the following :

a) The building collapsed due to breaking and tilting
b) The direction in which the building collapsed is perpendicular to the main beams.
c) Column dimensions are 40 cm x 60 cm in the main space and 40 cm x 45 cm over and under the mezzanine floor.
d) Tie rods are used in beam to beam connections and concrete corbel with a pin is used in beam to column connection.
e) The columns failed in the direction where they were narrow.
f) The building collapsed shortly after it was erected. Therefore it was subject to its own weight only.
g) It is observed that in some of the beam to socket connections, mortar fill was not in place.

Both the design and construction was done in 1998. So the structural design is reviewed in accordance with the 1997 Earthquaake Code. (Refer to Tables 11-12-13-14) The main reason for the damage was inadequate lateral rigidity as in the previous cases and following errors have ben identified :

1. Non-complience with the 1997 Earthquake Code.
a) Incorrect load assumptions.
b) Not confronting with the displacement limits
c) Underrating the soil characteristics.

Table-11 *Building Type C 1. Mode Displacements*

Column	Under Ultimate Loads (cm)	Under Service Loads (cm)	Max. Permitted by Code (cm)
PC X axis	3.44	4.22	2.45
PC Y axis	5.45	6.24	2.45
IC X axis	3.43	4.22	3.05
IC Y axis	11.48	13.25	3.05

PC: Peripheral Column IC: Inner Column

Table-12 *Comprison of Horizontal shear under equivalent of seismic loads.*

	Existing Design	1997 Code Requirements
X axis	0.100	0.200
Y axis	0.057	0.113

(*) Code 1997; For V_1=WA(T_1)/Ra(T_1) Ra(T_1)=R=5
A_0=0.40 g
Soil class $Z_4 \rightarrow T_x$=1.93 s. T_y=0.86 s.

Table-13 *Building Type C Column Dimensions*

Column	As built (cm)	Min. Dimensions acc. to code (cm)
Inner Peripheral	40x60	55x55
Inner Central	40x60	70x70

Table-14 *Building Type C damage ratio*

	Damaged	Undameged	Total	Damage Ratio
No. of Bldgs.	1	-	1	% 100

Photograph 3 *Building Type C*

2. Lack of design check for the weaker direction.
3. Inadequate reinforcement.

BUILDING TYPE D :

As illustrated in Fig.6, the layout is based on a 20.00 m x 7.00 m module. Maximum height of the building is 8.0 m. The system is composed of L and Y elements which are connected at points where moment is zero. Following points are observed during the studies on site:

a) Horizontal elements in both directions fell down due to failure of the connecting equipment.
b) Columns which were disconnected from the beams managed to stand erect.
c) Inner colums are broken down into two.
d) Collapse or snapping and falling occured along the shorter side of the columns.
e) Column dimensions are all 25 cm x 50 cm.
f) At the time of the earthquake, the weight of the building included only that of the structural frame and roofing.
g) No deformation or failure is observed in column to socket connections.

Structural design projects could not be obtained for this case study. Calculations are

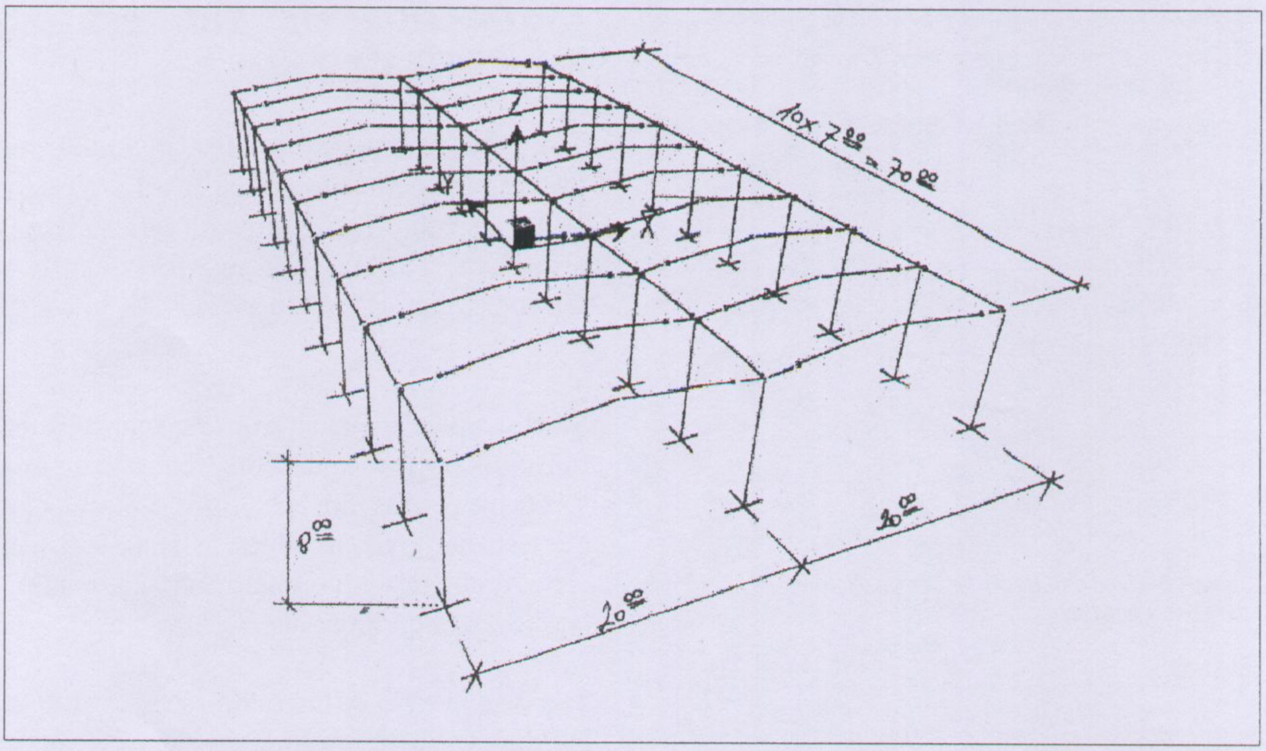


Figure 6 *Building Type D*

carried out for the system under critical loads and following conclusions are reached. (Tables 15-16-17)

1. The main reason leading to collapse was "inadequate lateral rigidity". This was a consequence of :

   a) Not complying with the 1997 Code.

   b) Insufficient performance of connecting equipment

   c) Weakness of the system against lateral forces.

Table-15 *Building Type D 1. Mode Displacements*

Column	Under Ultimate Loads (cm)	Under Service Loads (cm)	Max. Permitted by Code (cm)
PC X axis	7.75	10.17	2.80
PC Y axis	3.36	4.40	2.80
IC X axis	12.92	16.95	2.80
IC Y axis	5.60	7.34	2.80

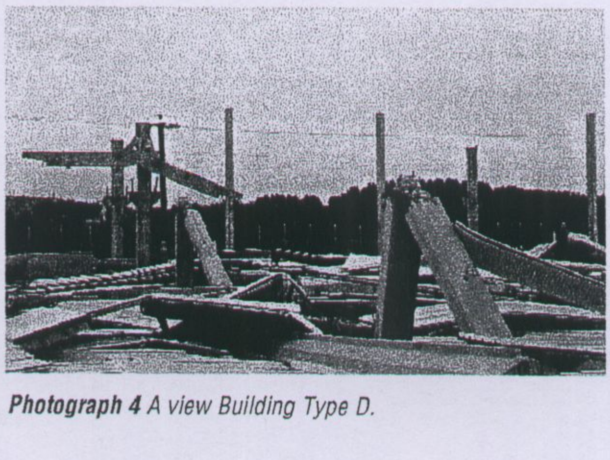
PC: Peripheral Column IC: Inner Column

Table-16 *Building Type D; Column Dimensions*

Column	As Built (cm)	Min. Dimension acc. to Code (cm)
Peripharel	25x50	55x55
Inner Central axis	25x50	60x60

Table-17 *Building Type D; Damage Ratio*

	Damaged	Undamaged	Total	Damage Ratio
No. of Bldgs.	6	2	8	% 67



Photograph 4 *A view Building Type D.*

Table-18 Results of the Mechanical Analyis for Reinforcing Steel

	Sample No.	Diameter (mm)	Creep Strength (Kg./mm2)	Tensile Strength (Kg./mm2)
A	991C	18	45.14	52.18
	992C	18	55.12	58.96
B	996C	12	-	77.59
	997	12	-	75.39
C	989C	22	26.25	41.68
	990	22	27.82	41.68
D	993C	14	49.35	58.17
	991C	14	50.92	58.38

Table-19 Analysis of Chemical Composition of steel used in the sample buildings

ELEMENT	A	B	C	D
C	0,17200	0,44300	0,10500	0,17800
Si	0,23600	0,26000	0,12500	0,27200
Mn	0,56800	1,64000	0,64400	1,82000
P	0,00594	0,00692	0,00512	0,00512
S	0,02420	0,10500	0,03340	0,08540
Cr	0,05180	0,12000	0,04970	0,13300
Mo	0,00416	0,01190	0,00100	0,00100
Ni	0,06950	0,09300	0,09590	0,04340
Al	0,00200	0,00200	0,00200	0,00200
Co	0,00100	0,00100	0,00100	0,00100
Cu	0,03900	0,13800	0,09260	0,06490
Nb	0,00200	0,00200	0,00200	0,00200
Ti	0,00100	0,00105	0,00100	0,00100
V	0,00100	0,00100	0,00100	0,00100
W	0,00500	0,00500	0,00500	0,00500
Pb	0,00200	0,00200	0,00200	0,00200
Sn	0,00300	0,00300	0,00300	0,00300
Mg	-	-	-	-
Sb	0,023360	0,023390	0,02000	0,00886
Fe	-	-	-	-
	0,28529	**0,75831**	**0,23524**	**0,51555**

EVALUATION OF THE PERFORMANCE OF MATERIALS

The site investigation also included the testing of concrete and steel. In all 4 types of buildings compressive strength of concrete is tested with "SCHMIT HAMMER" and it is observed that BS 25 and BS 30 standarts are met.

Steel samples are taken from the four selected building types and both chemical and mechanical tests are undertaken. Results of the tests are given in tables 18 and 19. There were problems with steel, both in building type B and building type C.

The carbon ratio of the reinforcement steel in building type B was higher than the max. value of 0.45. This increased its brittleness and reduced its capacity for bending. In fact, in the tests ultimate strength was reached before creep started.

In building type C, although S 420 steel was specified in the design, the samples yielded questionable values of creep and ultimate strength.

GENERAL FINDINGS AND THE PROBLEMS CONCERNING THE PRECAST CONCRETE BUILDINGS COLLAPSED IN THE MARMARA EARTHQUAKE

1. In all the buildings collapsed in the earthquake, the main cause of collapse is INADEQUATE LATERAL RIGIDITY.
2. The reason for inadequate lateral rigidity is the non-complience with the displacement limits of 1975 Code which is 0.0025 x h, and of 1997 Code which is 0.0035 x h.
3. In the buildings designed and erected

***Photograph 5** An undamaged precast concrete structure*

after 1997, the Earthquake Code of 1997 is ignored.

4. The S coefficient (building dynamic coefficient) in the 1975 Code is taken below 1.0 in the design of the collapsed buildings.
5. Columns of the collapsed buildings are all designed asymmetrically (in terms of their forms and reinforcements) and they have collapsed along the weaker direction.
6. Structural design of the frame does not match with load distribution. Especially in the frames with two spans, the columns in the middle, take more load than they are designed for. Such design methods should be revised.
7. In two of the building types investigated (type B and D) column-beam connections were either poorly detailed or poorly executed.
8. Concrete quality, in all building types either satified the requirements or was even better.
9. It is observed that there are problems with steel. Obviously, there is a need for more frequent chemical and mechanical tests. National standart TS 708 should also be revised in terms of welding and bending properties.
10. Concentration of damaged buildings in certain locations proved once again how important it is to identifty the correct soil class. When the amplifying effect of alluvial soil is considered, this becomes a very crucial design criteria.

CONCLUSIONS :

In the earthquake that occured on 17 August 1999 in the Marmara Region of Turkey, some single storey, reinforced precast concrete buildings with hinge joints are either damaged or have collapsed. This damage is neither a result of the design method, nor of the materials and the construction technology used. Incorrect applications are to be blamed for this result. The common problem in all the collapsed buildings is ignoring the existing design rules and requlations set by the Codes. The technical teams of Turkish Precast Concrete Association observed that the buildings that have been designed and produced in complience with existing Earthquake Code, have suffered very little or no damage. Even in areas where devastating effects of the earthquake is mostly felt, there was no damage in the buildings where only the column dimensions were correctly selected. (Photo. 5)

In single storey, reinforced precast concrete structures, the design criteria controlling the column dimensions is very important. Any design, ignoring the existing codes, and hence, not limiting the lateral displacement is nothing but a wrong design.

It is a fact that some single storey reinforced precast concrete structures with hinge joints were damaged in the earthquake. This is a very common and widely used system in Turkey and in many cases the buildings and projects are hardly controlled. Although, it is true that monolithic precast concrete frame structures perform much better under lateral forces, it should also be emphasised that despite all incorrect applications and negative views, many correctly designed, properly detailed and carefully produced single storey, reinforced precast concrete buildings with hinge joints continued their service during and after this earthquake.

The general critique aimed at this type of building technology should therefore, first question if the design, production and building requirements are properly fulfilled or not.

Approached in this framework, serious design and implementation problems will come out as the first and predominant factors for failure.

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