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PERFORMANCE OF SCHOOL BUILDINGS IN BINGÖL DURING THE 1 MAY 2003 EARTHQUAKE

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SUMMARY

An earthquake of magnitude 6.4 (Mw) struck the city of Bingöl in eastern Turkey on 1 May 2003, resulting in 168 deaths and extensive damage to private and public buildings. A US team of researchers and engineers from Purdue University, the University of Kansas, and WJE Associates Inc. sponsored by the National Science Foundation (NSF) was on the ground within a week of the earthquake to survey the damage. The US team collaborated with researchers from the Middle East Technical University (METU). A joint report sponsored by the NSF (US) and TUBITAK (Turkey) summarizes the results of their observations. This paper focuses on the damage to 23 reinforced concrete schools and 4 dormitories. The sample includes 2, 3 and 4-story moment-frame structures, some with only masonry infill walls and others with both masonry infill walls and reinforced concrete shear walls. The geotechnical and geological observations indicated that any variation in the shaking at the different schools was not likely to have been affected strongly by the soil conditions and noted the absence of foundation failures. The quality of construction materials, workmanship, detailing practices and inspection level can be considered uniform throughout the entire building sample. These observations provide a unique opportunity to study the structural performance of the schools where various levels of damage were observed. The noted damage ranged from light to complete collapse of the first story. The number of stories, type of the structural system, ratio of reinforced concrete column and wall areas to the overall floor area, and proximity of the schools to the epicenter were selected to be the major study parameters. Regardless of the school type, total collapse was prevented if reinforced concrete shear walls were present because they compensated for the vulnerability of the captive columns effectively shortened by masonry walls. In the schools with heavy damage, captive columns were found to be the main cause of poor performance. The salient feature of the structural damage in Bingöl was defined by the varying interaction, ranging from good to very bad, of the tile masonry with reinforced concrete frames.

INTRODUCTION

The earthquake of moment magnitude 6.4 (USGS and KOERI) occurred 10 km north of Bingöl at 03:27 am (local time) on 1 May 2003. The epicenter of the earthquake was at 38.94N- 40.51E (ERD-Ankara) (Figure 1). The depth of the quake was estimated to be at 6 km. As of 1:32 pm local time on 2 May 2003 aftershocks

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between magnitudes 2.8 and 4.3 had been recorded. The magnitudes of the possible aftershocks can be expected to be as high as 5.1 within one week after the main event. The strongest record of the earthquake was registered at Bingöl. The peak ground accelerations (PGA) of the three components of this record are 0.55g (NS), 0.28g (EW) and 0.47g (UP). The duration of the strong motion was 17 s. High vertical acceleration indicates that the station at Bingöl was very close to the source of the quake. The official number of fatalities is 168.

Immediately following the 1 May 2003 earthquake in Turkey, a US reconnaissance team composed of researchers and practicing engineers under sponsorship of the US National Science Foundation (NSF) set out to join a Turkish team sponsored by the Scientific and Technical Research Council of Turkey (TUBITAK) so as to study the impact of the event. The NSF team sponsored under Grant No. 0334950 was led by researchers from Purdue University, working closely with researchers from the University of Kansas, and engineers from WJE Associates Inc. in California. The TUBITAK team was led by researchers from the Middle East Technical University (METU) in Ankara. This team consisted of faculty and graduate students in structural engineering, geotechnical engineering, and geological science. A joint report sponsored by the National Science Foundation (US) and TUBITAK (Turkey) summarizes the results of this collaboration and can be found at http://www.anatolianquake.org. The main objectives of the overall effort were to document: (a) the damage to reinforced concrete buildings and (b) the geotechnical and geological aspects of the earthquake. The teams also paid some attention to mosques, masonry and nonengineered structures. The group concentrated its efforts in the vicinity of Bingöl.

Figure 1. The Bingöl earthquake (M_w=6.4)

GEOTECHNICAL CONDITIONS OF BINGÖL

Most of the city of Bingöl is built on top of an alluvial terrace, approximately 40 to 60 m above the current level of the Capakçur River that flows through the middle of the urban area. North of the river, all the buildings' foundations lie on this old (Pleistocene) alluvial deposit. The deposit can de described as a GP, a brown, poorly graded rounded gravel with small amounts of sand and traces of clay and silt, with maximum size of the particles of 1 to 2 m. The deposit is dense to very dense. Fig. 2 shows the terrace and thickness of the deposit. Cuts excavated in this terrace are stable with angles of 40^o to 50^o.

The south part of Bingöl is built on a terrace at the same elevation as the north. In this terrace most of the subsurface materials are similar to those found in the north. Because of the high permeability of the alluvial deposits, it is expected that the water table is very deep below the city. Observations of water outflow at the toe of the river slopes provide evidence in support of a deep water table, although it is expected that some subsurface water can be found perched at higher elevations due to local soil conditions.

Towards the southwest, and as the topography rises, the buildings are founded on moderately weathered bedrock or on stiff colluvial deposits, which locally can be several meters thick. The colluvial deposits can be classified as CL, brown stiff clay, with variable percentages of sand or gravel, which in many cases can be described as sandy or gravelly clay.

Because of the geological and geotechnical characteristics of the soils in the area, it is not likely that there was any amplification of the ground motions. Furthermore it is expected that the ground motions in the area were quite uniform. The only exceptions perhaps are areas close to the edge of very tall slopes where some amplification might have occurred; for example, buildings on top of the slopes near the river.

Several buildings were examined in the north and south sides of Bingöl to determine whether there were signs of distress, settlement, excessive deformations, or any other indication of foundation damage. The most damage observed was light damage to the lateral basement walls, even in buildings with severe damage. The damage was invariably concentrated in the structure above the basement.

The observations suggests that differences in damage to buildings from place to place in Bingöl due to the 1 May 2003 earthquake were a result of characteristics of the structures, not of foundation conditions or gross ground deformation of any kind.

Figure 2. View southwest over the Capakçur River at western edge of Bingöl. Gravel in steep bank on right of view. Volcanic bedrock in shadow in steep bank on left of view. Contact slopes toward viewer.

DAMAGE EVALUATION SCHEME

The damage rating of the reinforced concrete system used in this survey aims to group the buildings with similar damage patterns rather than to define their damage states in absolute terms. Inclined cracking of columns is a very dangerous type of damage in the case of insufficient transverse reinforcement and improper detailing. Therefore, the structures with inclined cracks observed on their columns were rated to be severely damaged. The shear and flexure cracks on beams, spalling of concrete on columns and hairline cracks on shear walls were the most common damage patterns in the moderately damaged structures. The lightly damaged structures were the ones with only hairline cracks on beams.

The masonry infill wall damage of the buildings was also rated in three levels which can be defined as follows:

- Severe damage: Wide cracks on walls and their boundaries.
- Moderate damage: Cracks on walls and their boundaries, flaking of large pieces of plaster.
- Light damage: Hairline cracks on walls, flaking of plaster.

DAMAGE SURVEY OF THE SCHOOLS IN BINGÖL

The team visited 27 schools and dormitory buildings in Bingöl and its close vicinity, one school in Ilicalar (20 km away from Bingöl) and one school in Sancak (25 km away from Bingöl) between May 13 and May 17 (Fig. 3). The complete list of the schools is given in Table 1.

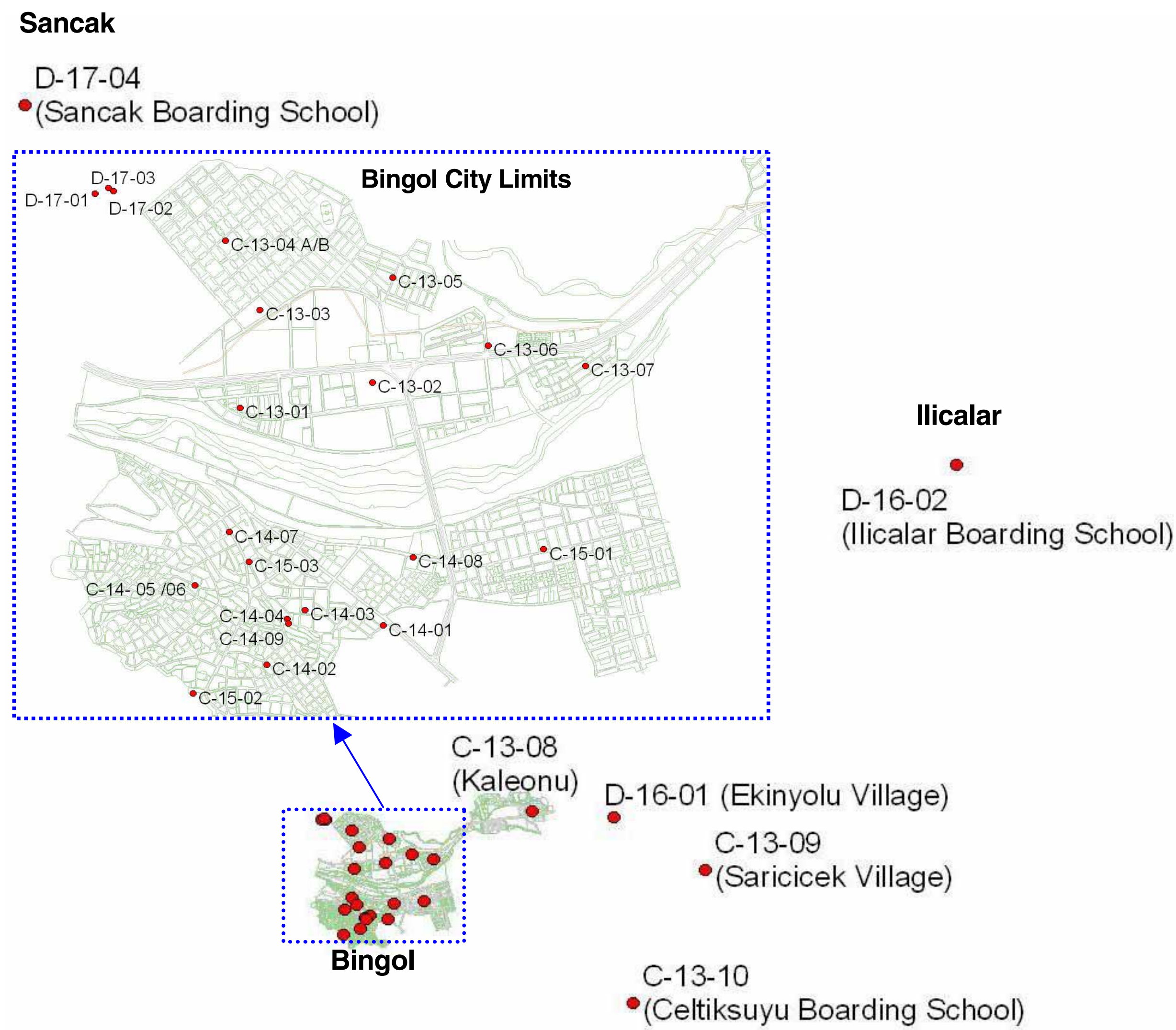

Figure 3. The location of the school and dormitory buildings surveyed.

Table 1. List of the school and dormitory buildings visited

School ID	Name	Location	GPS N	GPS E	Type	Position
C-13-01	75. Yil Ilkogretim Okulu	Bingöl	38 53.665	40 29.565	Frame	Independent bldg
C-13-02	Anadolu Ogretmen Lisesi	Bingöl	38 53.745	40 30.749	Frame	Dependent bldg
C-13-03	Rekabet Kurumu Lisesi (Building B)	Bingöl	38 54.001	40 29.660	Dual	Dependent bldg
C-13-04A	Mustafa Kemal Pasa Ilkogretim Okulu Building A1	Bingöl	38 54.242	40 29.512	Dual	Dependent bldg
C-13-04B	Mustafa Kemal Pasa Ilkogretim Okulu Building A2	Bingöl	38 54.242	40 29.512	Dual	Dependent bldg
C-13-05	Sehit Mustafa Gundogdu Ilkogretim Okulu	Bingöl	38 54.105	40 30.247	Frame	Independent bldg
C-13-06	Kazim Karabekir Ilkogretim Okulu	Bingöl	38 53 866	40 30.663	Frame	Independent bldg
C-13-07	Vali Kurtulus Sismanturk Ilkogretim Okulu	Bingöl	38 53.789	40 31.091	Frame	Independent bldg
C-13-08	Kaleonu Ilkogretim Okulu	Bingöl	38 54.486	40 32.994	Frame	Independent bldg
C-13-09	Saricicek Koyu Ilkogretim Okulu	Saricicek	38 53.556	40 36.300	Frame	Independent bldg
C-13-10	Celtiksuyu Ilkogretim Okulu	Celtiksuyu	38 51.587	40 34.855	Frame	Independent bldg
C-14-01	Karaelmas Ilkogretim Okulu	Bingöl	38 52.905	40 30.179	Frame	Independent bldg
C-14-02	Fatih Ilkogretim Okulu	Bingöl	38 52.776	40 29.664	Masonry	Independent bldg
C-14-03	Mehmet Akif Ersoy Ilkogretim Okulu	Bingöl	38 52.964	40 29.837	Frame	Independent bldg
C-14-04	Ataturk Lisesi	Bingöl	38 52.934	40 29.758	Frame	Independent bldg
C-14-05	Vali Guner Orbay Ilkogretim Okulu (Main Building)	Bingöl	38 53.054	40 29.352	Frame	Independent bldg
C-14-06	Vali Guner Orbay Ilkogretim Okulu (2nd Building)	Bingöl	38 53.054	40 29.352	Frame	Independent bldg
C-14-07	Ataturk Ilkogretim Okulu	Bingöl	38 53.236	40 29.507	Frame	Independent bldg
C-14-08	Bingöl Lisesi (Building B)	Bingöl	38 53.139	40 30.317	Dual	Dependent bldg
C-14-09	Bingöl Imam Hatip Lisesi (Building B)	Bingöl	38 52.917	40 29.763	Dual	Dependent bldg
C-15-01	Sarayici Ilkogretim Okulu	Bingöl	38 53.159	40 30.894	Frame	Independent bldg
C-15-02	Murat Ilkogretim Okulu	Bingöl	38 52.681	40 29.337	Frame	Independent bldg
C-15-03	Bingöl 100.Yil Ilkogretim Okulu (Building B)	Bingöl	38 53.133	40 29.593	Dual	Dependent bldg
D-16-01	Ekinyolu Koyu Ilkogretim Okulu	Bingöl	38 54.374	40 34.564	Frame	Independent bldg
D-16-02	Ilicalar Yatili Ilkogretim Bolge Okulu Dormitory Bldg	Ilicalar	38 59.581	40 41.250	Dual	Independent bldg
D-17-01	Merkez Cumhuriyet Kiz Yatili Ilkogretim Bolge Okulu Boys' Dormitory Building	Bingöl	38 54.411	40 28.941	Dual	Independent bldg
D-17-02	Merkez Cumhuriyet Kiz Yatili Ilkogretim Bolge Okulu Girls' Dormitory Building	Bingöl	38 54.419	40 29.021	Dual	Independent bldg
D-17-03	Merkez Cumhuriyet Kiz Yatili Ilkogretim Bolge Okulu School Building	Bingöl	38 54.430	40 28.999	Dual	Independent bldg
D-17-04	Sancak Yatili Ilkogretim Bolge Okulu Dormitory Building	Sancak	39 05.235	40 23.452	Dual	Independent bldg

The structural system of the schools can be grouped as:

- RC Moment resisting frame systems (17 buildings)
- RC Dual systems (11 buildings)
- Masonry (1 building, not surveyed)

School Buildings with RC Moment-Resisting Frame System

Of the 17 buildings in this category, 16 had the same column layout (Fig. 4 and 5). As the floor plan indicates, the lateral load resisting system in these buildings can be categorized as regular in plan. The majority of the columns were aligned in regular bays, and most of the beams framed into columns. The dimensions of the columns in the buildings were typically 0.3m x 0.5m. The orientation of the columns was the same in all buildings, with the exception of a corner column in building C-13-01. The locations of the masonry infill walls varied depending on the use of the space in these schools. The exterior masonry walls were typically thicker than the interior walls.

The only school building with a different column layout was C-13-02. The school complex was a combination of two separate buildings. The separation afforded by the expansion joint between the two buildings was not sufficient to avoid pounding between the two structures. The floor plan of the northern building is shown in Fig. 6. All the columns shown in the figure have dimensions of 0.2m x 0.5m.

The total column area of buildings with moment-resisting frames was approximately 1% of the floor area, regardless of the number of floors. Consequently, the performance of the structures during the earthquake was significantly influenced by the number of floors. The level of damage of the lateral load resisting system with respect to the number of floors can be categorized as follows:

- 5 two-story schools: 4 moderately damaged, 1 lightly damaged
- 11 three-story schools: 3 collapsed, 6 severely damaged, 2 moderately damaged
- 1 four-story schools: 1 severely damaged.

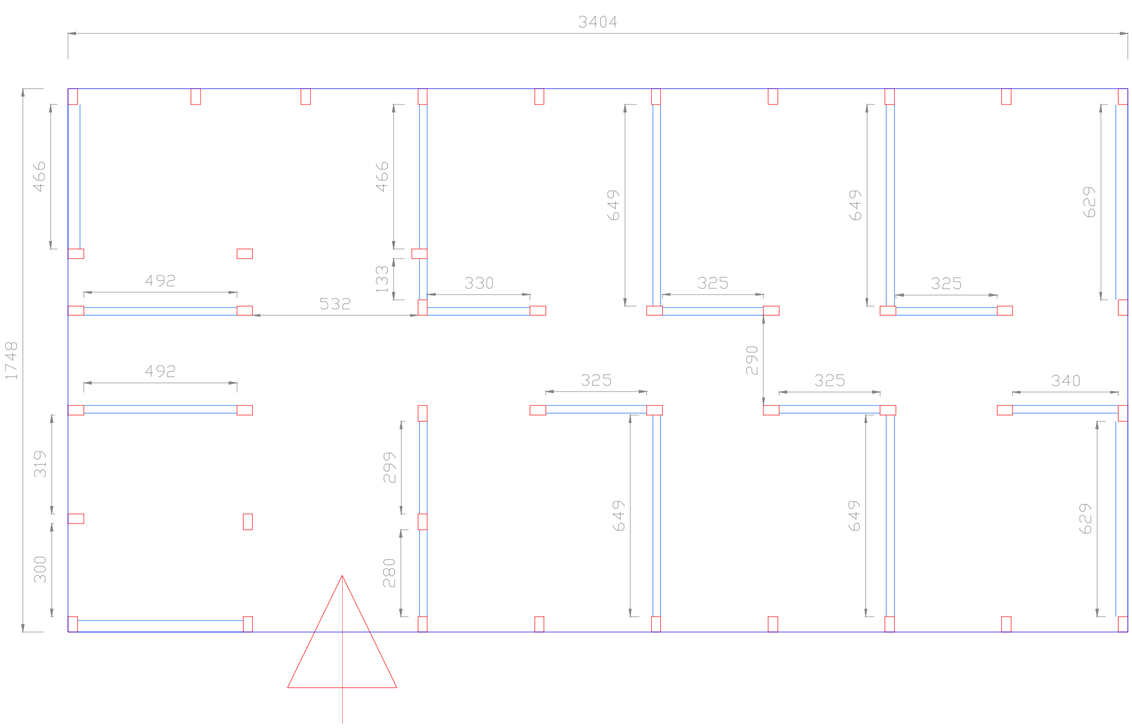

Figure 4. Typical school plan without shear walls. All the walls shown in the drawing refer to those occupying a full span. Walls with openings are excluded. All the columns have dimensions of 0.3m x 0.5m. The arrow indicates the entrance to the building. Dimensions are in cm.

Figure 5. School buildings with typical floor plan without shear walls

Damage to the masonry walls was rated separately. The three- and four-story buildings typically sustained severe masonry wall damage (Table 1). There were several construction and structural design deficiencies commonly observed in the school buildings. In most of the structures surveyed, the quality of construction practices was uniform. Specific problems noted were:

- Use of unwashed aggregate,
- Use of aggregates with large maximum size (up to 10 cm),
- Use of undeformed bars,
- Inadequate preparation of cold joints.

One of the most common structural problems observed in these buildings was the presence of captive columns, which made the structures vulnerable with respect to column shear failures. In almost all the schools, openings for small windows in the furnace room and restrooms were placed adjacent to columns. The exterior rectangular columns were oriented with the strong axis resisting moments in the short direction of the building layout in Fig. 4. Therefore, windows on the exterior walls in the long direction of the building exposed columns to shear forces acting perpendicular to their weak axis for bending (Fig. 7). It was observed also that crushing of the masonry walls in the upper corners created captive columns (Fig. 8).

In the school buildings that were visited the detailing of structural members was inadequate with respect to requirements of modern seismic codes. Lack of confinement in plastic hinge regions of the columns was observed to be one of the most significant causes of damage. Even though the spacing of the stirrups was reduced in the end regions of some columns, the amount of transverse reinforcement provided was not sufficient to prevent shear failures, particularly in the case of captive columns (Fig. 9). Another detailing deficiency commonly observed was the inadequate anchorage of the free ends of the stirrup reinforcement.

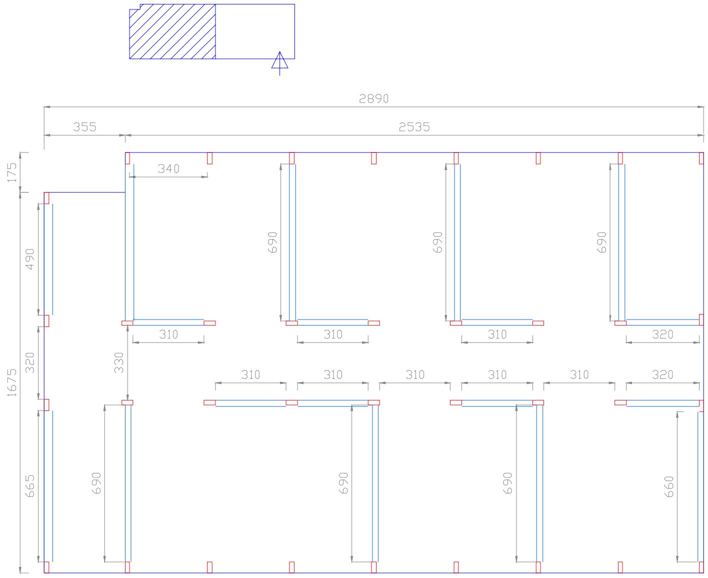

Figure 6. The floor plan for C-12-02. The structural system is a moment-resisting frame. The columns are 0.2m x 0.5m. The school building comprises two independent structures separated by an insufficient expansion joint. Only the shaded part in the upper figure was surveyed. The arrow indicates the entrance to the building. The dimensions are in cm.


Figure 7. Shear failure of captive columns created by the small windows of the furnace room in building C-14-03.

Figure 8. Shear failure of captive columns as a result of crushing of upper corner of masonry walls in building C-14-01.

Figure 9. The shear failure of the corner column in building C-13-09 (collapsed). The spacing of the transverse reinforcement is 10 cm at the top 30-cm portion of the columns. The ends of the stirrups were not anchored properly.

School and Dormitory Buildings with Dual Systems

The schools with dual systems surveyed can be categorized into four groups.

1. Buildings C-13-04A and C-13-04B

These buildings were part of the same school complex comprising five different structures separated by expansion joints. Buildings C-13-04A and C-13-04B had a similar lateral load resisting system, shown in Fig. 10. The only difference between the buildings was the location of masonry walls. The total shear wall area of the structure in the longitudinal and transverse directions was 1.4 and 2.0 % of the floor area, respectively.

There were no indications of structural damage in the buildings, and the masonry walls were only lightly damaged.

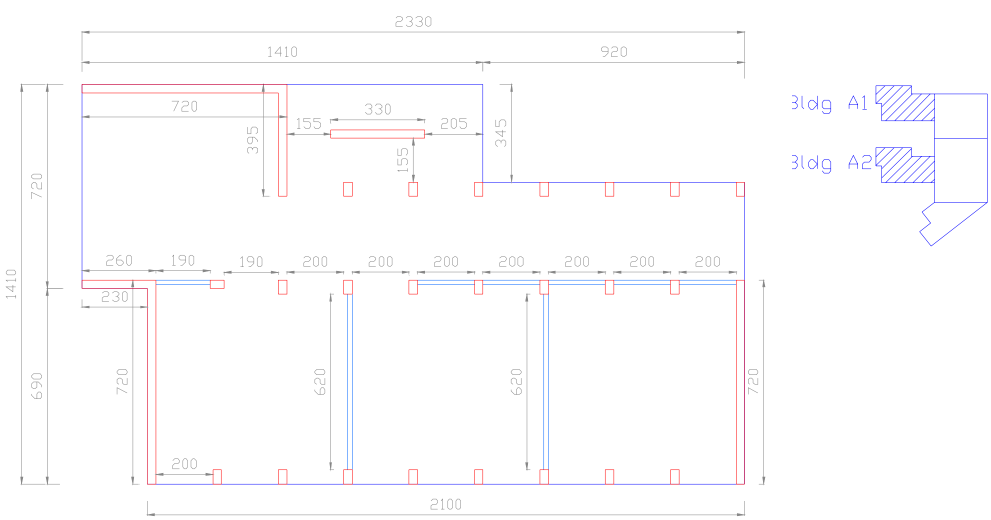

Figure 10. Structural floor plan for building C-13-04A. All the columns have dimensions of 0.3m x 0.5m. The thickness of the reinforced concrete and masonry walls were 0.3m and 0.16m, respectively. Dimensions are in cm.

2. Buildings C-13-03, C-14-08, C-14-09 and C-15-03

Each one of these buildings is one structure of a three-structure complex that conforms to the plans for a typical high school building commonly used by the Ministry of Education of Turkey. Each of the buildings is a four-story structure. Although the buildings C-14-09 of C-15-03 are smaller than the other two, the structural plans of all four are similar. The main difference is that the smaller buildings have two fewer bays in the longer direction. The total column area was 1.5 % of the floor area for all the buildings. The ratio of shear wall area to area of the floor was not uniform. Building C-14-08, which had the smallest ratio of shear wall to floor area, had a wall area of 0.7% and 0.4% of the total floor area in the two principal directions (Fig. 11). The highest ratio of wall to floor area was found in building C-13-03 (Fig. 12).

The most severe damage in this group was observed in C-14-09. A cold joint in one of the shear walls in the structure initiated a horizontal crack along the joint during the earthquake. Although the rest of the structural members did not show signs of damage, the building was classified to be severely damaged because of the damage to the shear wall. The masonry walls of the building did not suffer any severe damage.

In building C-14-08, damage to the structural system consisted of hairline cracks in the shear walls. There was no damage observed in the columns and a few beams had severe flexural cracks. The structural system of this building was rated as moderately damaged. The masonry walls were separated from the structural frame because of crushing of the bricks at the edge of the walls. There was no partial or full collapse of the masonry walls.

Buildings C-13-03 and C-15-03 with higher shear wall ratios than the other two (ratio of wall to floor area) had only moderate damage to their structural systems and the masonry walls.

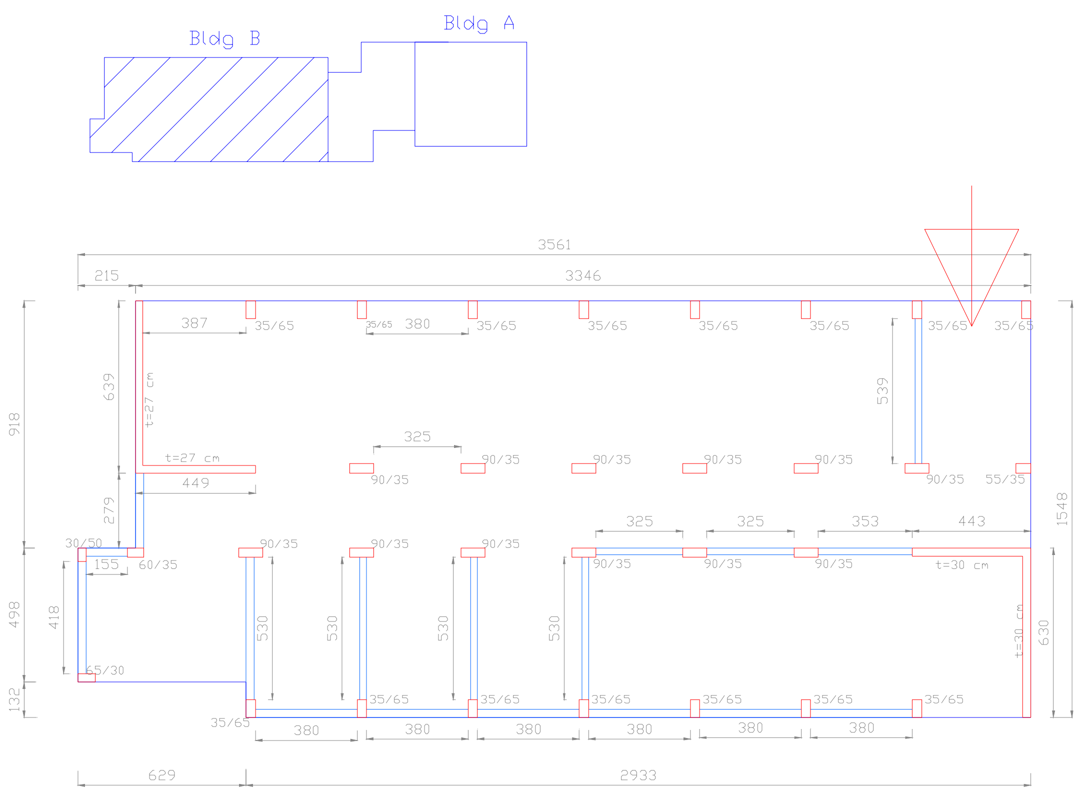

Figure 11. Structural floor plan for building C-14-08. The thickness of interior and exterior masonry walls were 0.25m and 0.3 m, respectively. The arrow shows the entrance to the building. Dimensions are in cm.

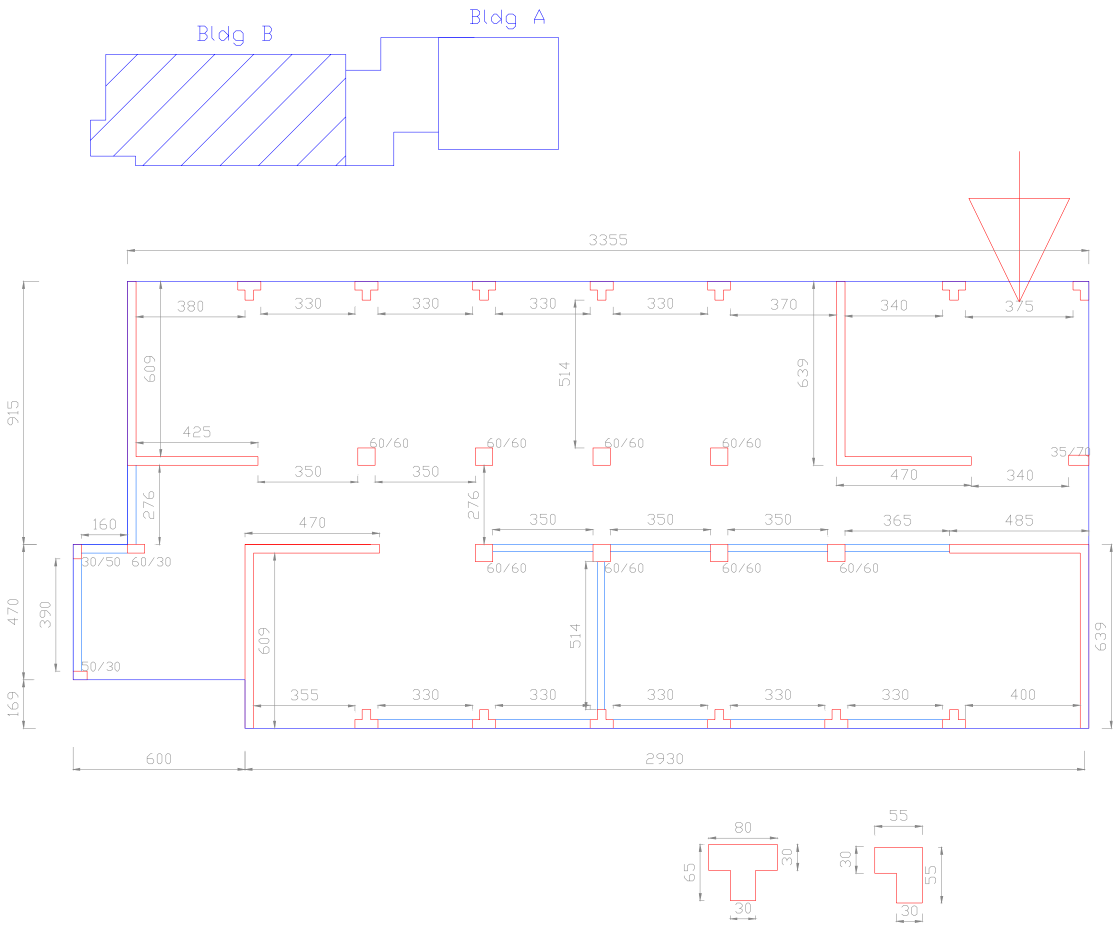

Figure 12. Structural floor plan for building C13-03. The thickness of the shear walls was 0.3m. The masonry wall thickness was 0.25m for the interior walls and 0.3m for the exterior walls. The arrow shows the entrance to the building. Dimensions are in cm.

3. Building C-17-03

Building C-17-03 had another structure adjacent to its west end. Although the buildings were separated by an expansion joint, the gap provided between the two was very small. Building C-17-03 had 4 stories. The total column area was 1.1% of the floor area. The area of shear walls was 0.8 and 1.0% of the floor area in the longitudinal and transverse directions respectively. The structural floor plan of the building is shown in Fig. 13.

Damage to the structural system and the masonry walls were both rated as moderate. There were no inclined cracks observed in the columns or shear walls. There was some local damage to members in the form of spalling of concrete cover likely as a result of construction deficiencies.

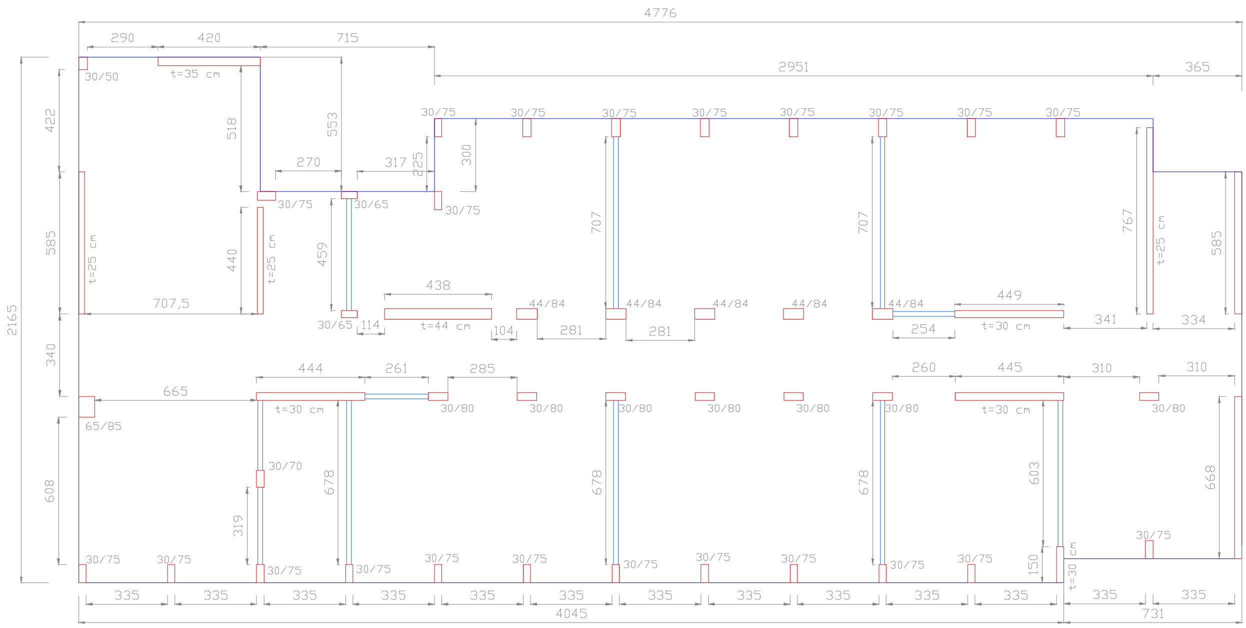

Figure 13. Structural floor plan for building C-17-03. Masonry wall thickness was 0.19m. Dimensions are in cm.

4. Dormitory Buildings (D-16-02, D-17-01, D-17-02 and D-17-04)

These buildings did not have any other structures adjacent to them. They had identical floor plans shown in Fig. 14, and all of them were four-story structures. The structural system of these schools had a column area of 0.7% of the floor area, and wall areas of 1.0% and 1.5% of the floor area in the two principal directions.

Of the four dormitory buildings that were surveyed, the structural systems of two of them, D-17-01 and D-17-02, were rated as severely damaged because of the inclined cracks on the captive columns. There were also inclined hairline cracks on the shear walls. Some of the beams had flexural and shear cracks, and damage was commonly observed in beams that framed into other beams as opposed to columns.

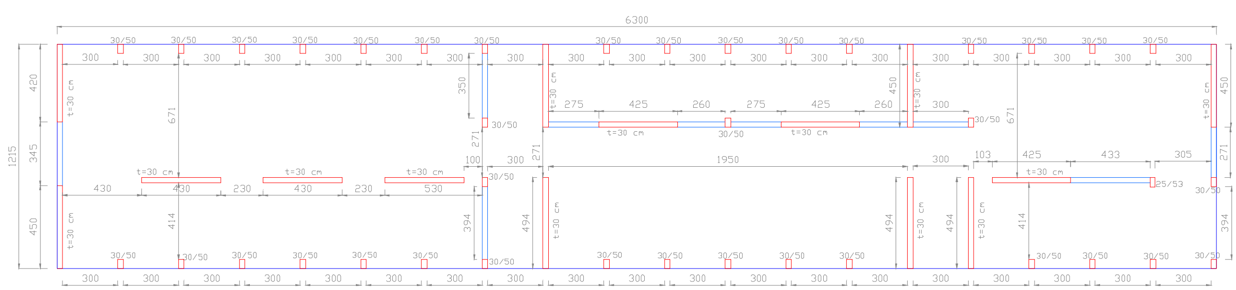

Figure 14. Structural floor plan for dormitory buildings. The masonry wall thickness was 0.3m. Dimensions are in cm.

Because buildings D16-02 and D-17-04 had inclined hairline cracks on shear walls and shear and flexure cracks on beams the damage to them was rated as moderate.

The most striking damage in these buildings was the collapse of the free standing masonry walls separating the sleeping units in the upper levels. These walls were not included in the damage rating because they were unattached to the structural system. However, they presented a serious hazard to the students living in these dormitories because in some cases the walls collapsed on the beds (Fig. 15). Fortunately the collapse of the walls did not result in any fatalities because almost all the beds so affected were unoccupied at the time.

Figure 15. Collapse of the free standing masonry walls onto the beds in the dormitory buildings.

COMPARISON OF THE PERFORMANCE OF THE FRAME AND DUAL SYSTEMS

For the purpose of comparing the performance of both groups of school buildings, the data from the damage assessments were organized using the procedure proposed by Hassan and Sozen [1]. The wall and column indexes in this procedure are defined as follows:

Column index (CI): Half the total column area at the base level divided by the product of the floor area and number of floors above the ground level.

Wall index (WI): sum of the area of reinforced concrete shear walls and a tenth of area of masonry walls in a given horizontal direction divided by the product of the floor area and the number of floors above the ground level.

The wall index is calculated for both main horizontal axes of the buildings, and the smaller of the two is taken as the wall index for the given building. The wall and column indexes calculated for the school and dormitory buildings in Bingöl are given in Table 2. The correlation between the damage category of the buildings and the wall and column indexes are presented in Fig. 16. As the figure shows, the damage level tended to decrease as the wall and column indexes increased.

Building damage observations indicate that the performance of dual systems was satisfactory. Even though some of the dual system buildings were rated as severely damaged because of the damage associated with captive columns and cold joints; observed damage to the masonry walls indicate that the reinforced concrete walls were effective in controlling lateral drifts. Buildings with moment-resisting frame systems did not perform well during the earthquake. Although the quality of construction is quite uniform for all the buildings, frame systems were more vulnerable to damage associated with deficiencies in construction practice. The flexibility of moment frame buildings resulted in larger drift demands than those in buildings with dual systems, which caused severe damage and in many cases the collapse of the structure due to loss in gravity load capacity. Further, the damage level of infill masonry walls in moment frame buildings that were severely damaged supports the conclusion regarding the role of structural walls in controlling drift demands. The shear damage to columns was very severe in buildings with moment resisting frames.

Based on the damage assessment of these buildings, a boundary for the minimum column and wall indices for satisfactory performance is shown in Fig. 16.

Table 2. Damage state and structural information of the school buildings

	Building Number	Damage to RC	Damage To Masonry	No. of Stories	Floor Area (m^2)	RC Wall Area (m^2)		Masonry Wall Area (m^2)		Column Area (m^2)	CI (%)	Min. WI (%)
						EW	NS	EW	NS			
Moment Resisting Frame (Schools)	C-13-07	Light	Moderate	2	589	0.00	0.00	16.84	11.92	6.45	0.27	0.10
	C-13-02	Moderate	Moderate	2	528	0.00	0.00	19.49	6.50	5.40	0.26	0.06
	C-13-05	Moderate	Light	2	585	0.00	0.00	18.69	11.71	6.45	0.28	0.10
	C-13-06	Moderate	Light	2	589	0.00	0.00	15.74	11.02	6.45	0.27	0.09
	C-14-06	Moderate	Moderate	2	595	0.00	0.00	15.99	10.69	6.45	0.27	0.09
	C-14-04	Moderate	Moderate	3	595	0.00	0.00	12.49	11.41	6.45	0.18	0.06
	C-14-05	Moderate	Moderate	3	595	0.00	0.00	7.39	17.13	6.45	0.18	0.04
	C-13-01	Severe	Severe	3	595	0.00	0.00	15.99	10.57	6.45	0.18	0.06
	C-14-01	Severe	Severe	3	595	0.00	0.00	15.74	9.41	6.45	0.18	0.05
	C-14-03	Severe	Severe	3	595	0.00	0.00	5.38	12.74	6.45	0.18	0.03
	C-14-07	Severe	Moderate	3	595	0.00	0.00	15.99	8.95	6.45	0.18	0.05
	C-15-01	Severe	Severe	4	595	0.00	0.00	14.91	7.31	6.45	0.14	0.03
	C-15-02	Severe	Severe	3	595	0.00	0.00	14.36	10.57	6.45	0.18	0.06
	D-16-01	Severe	Severe	3	595	0.00	0.00	14.82	7.67	6.45	0.18	0.04
	C-13-08	Collapsed	Collapsed	3	595	0.00	0.00	18.34	7.39	6.45	0.18	0.04
	C-13-09	Collapsed	Collapsed	3	595	0.00	0.00	15.99	9.25	6.45	0.18	0.05
	C-13-10	Collapsed	Collapsed	3	595	0.00	0.00	15.99	9.25	6.45	0.18	0.05
Dual System (schools)	C-13-04A	None	Moderate	3	281	3.93	5.51	1.90	1.98	3.45	0.20	0.49
	C-13-04B	None	Moderate	3	281	3.93	5.51	2.53	3.01	3.45	0.20	0.50
	C-13-03	Moderate	Moderate	4	524	7.49	5.55	3.28	8.49	7.99	0.19	0.31
	C-14-08	Moderate	Severe	4	523	3.62	2.54	9.00	9.35	7.94	0.19	0.17
	C-15-03	Moderate	Moderate	4	396	3.15	4.46	4.45	3.67	5.74	0.18	0.23
	D-17-03	Moderate	Moderate	4	895	7.41	8.81	0.98	9.72	9.78	0.14	0.21
	C-14-09	Severe	Moderate	4	381	3.22	3.57	2.97	5.10	5.89	0.19	0.23
Dual System (Dorms)	D-16-02	Moderate	Moderate	4	765	8.00	11.1	5.41	4.08	5.68	0.09	0.28
	D-17-04	Moderate	Moderate	4	765	8.00	11.1	5.41	4.08	5.68	0.09	0.28
	D-17-01	Severe	Moderate	4	765	8.00	11.1	5.41	4.08	5.68	0.09	0.28
	D-17-02	Severe	Moderate	4	765	8.00	11.1	5.41	4.08	5.68	0.09	0.28

A good example of the different types of response experienced by different structural systems is illustrated in Fig. 17. Before the earthquake, there were three buildings at the site of Celtiksuyu Boarding School, located approximately 10 km to the south east of Bingöl. The buildings were the teachers' apartments (C-13-11), the student dormitory and the school building (C-13-10). The figure shows the state of the three structures after the earthquake. Both the school building (right of the picture) and the student dormitory (center of the picture), with moment-resisting frames, collapsed with a death toll of 84 (mostly children). The apartment building for the teachers, with a dual structural wall – moment frame resisting system, survived without damage.

CONCLUSIONS

The observations in the school buildings showed that structural walls improve the behavior of reinforced concrete systems drastically. Especially in the structural design of school buildings using reinforced concrete frames such as those found in Bingöl, structural walls are recommended.

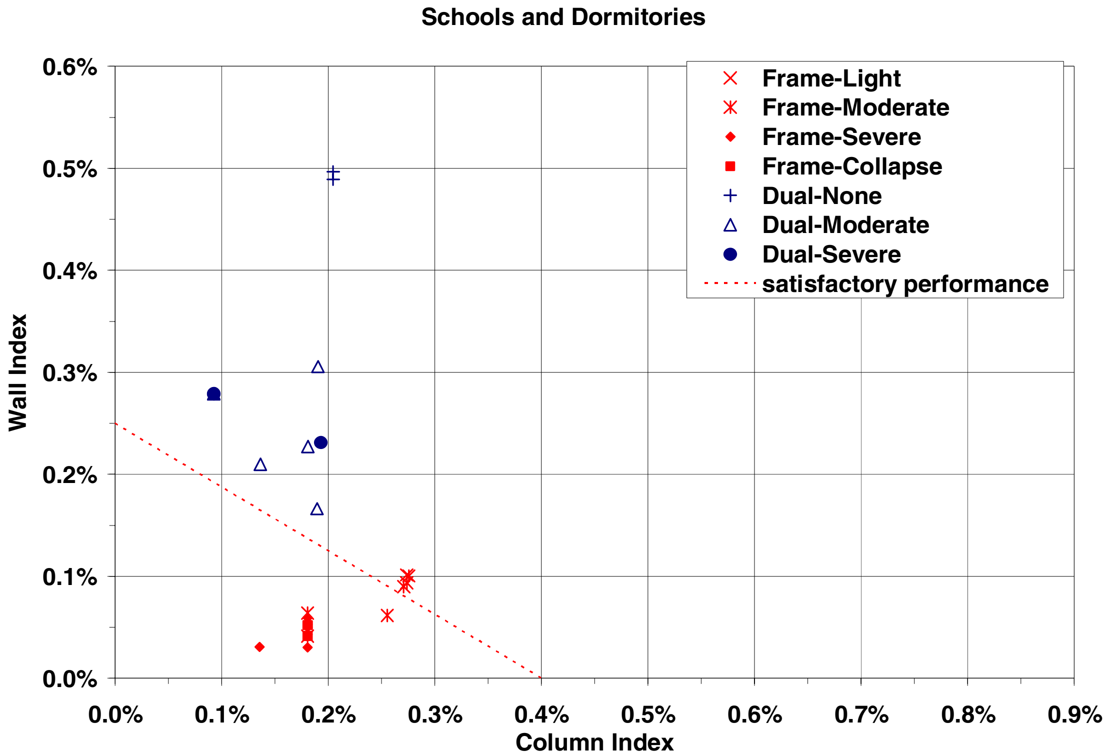

Figure 16. Correlation between the structural performance and the wall and column indexes defined by Hassan and Sozen (1997).

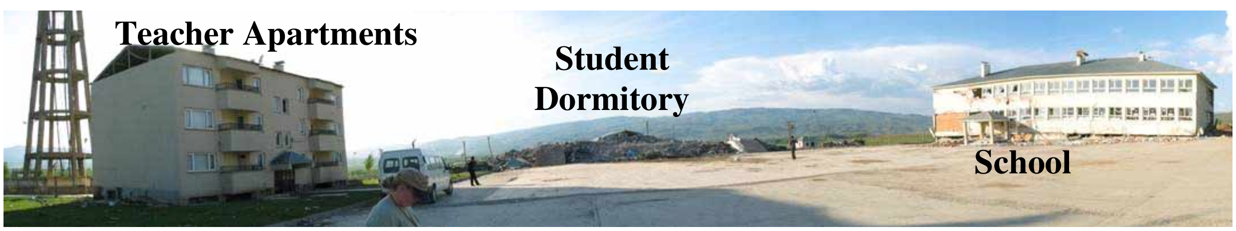

Figure 17. Comparison of the performance of the moment-resisting frame (MRF) and dual system buildings at the same school site. The two MRF buildings, the school and the dormitory, collapsed whereas the dual system building on the left survived without damage.

Acknowledgments

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