Performance of reinforced concrete structures

The 1992 Erzincan Earthquake

by Murat Saatcioglu and Michel Bruneau

A Richter magnitude 6.8 earthquake struck the city of Erzincan in eastern Turkey on March 13, 1992. The epicenter of the earthquake was 7.7 km (4.8 mile) north of the city, resulting in extensive structural damage and casualties in spite of its relatively moderate magnitude. Approximately 300 buildings, mostly reinforced concrete, either completely collapsed or were damaged beyond repair. An additional 500 buildings suffered various degrees of structural and non-structural damage. The official dead toll was more than 525 people, and the number of seriously injured was about 700 people.

Fig. 1 — A typical apartment building with a soft story.

Fig. 2 — Red Crescent commercial center with strong beams and weak columns.

The city of Erzincan is located near a number of active fault lines, all of which have ruptured during the last four decades. More than 20 earthquakes of a catastrophic nature have been recorded. The last major earthquake, with a Richter magnitude of 8.0, occurred on December 27, 1939, killing 32,000 people, and leaving 230,000 people homeless. After the 1939 earthquake the city was rebuilt using structural systems similar to those used in Europe and North America. Performance of these structures during the 1992 earthquake is of significance, since similarly engineered structures exist in different parts of the world with a likelihood of being subjected to similar magnitude earthquakes.

The authors conducted a 4-day reconnaissance visit to the disaster area shortly after the earthquake. Where applicable, comparisons were made between the local design practice and the requirements of the ACI 318-89 Building Code.[1]

Structure types and soil conditions

The majority of building structures in Erzincan consist of reinforced concrete frames with unreinforced brick masonry infills. Some reinforced concrete shear wall-frame interactive systems are also used, especially in newer, medium-rise buildings. Most reinforced concrete structures are in the low to medium-rise range with less than seven stories. Industrial buildings are mostly cast-in-place or precast concrete. The few bridges that exist in the city are single and multiple-span reinforced concrete railroad overpasses.

The city is located in a valley 1200 to 1500 m (3900 to 4900 ft) above sea level, between two mountain ranges that peak at about 3500 m (11,500 ft) high. A river runs through the city in the east-west direction, parallel to the mountains. The city is built on soft river-born sediments, which is believed to be a couple of hundred meters deep in the central part of the city. The soil borings conducted by the State Water Works Directorate indicate a reduction in the depth of the alluvial sediments as the mountains are approached in the north. However, no boring has ever reached bedrock in central Erzincan.[2] Although it showed significant seasonal variation, the average water table was reported to be only a few meters in certain parts of the city, shortly before the earthquake.[2]

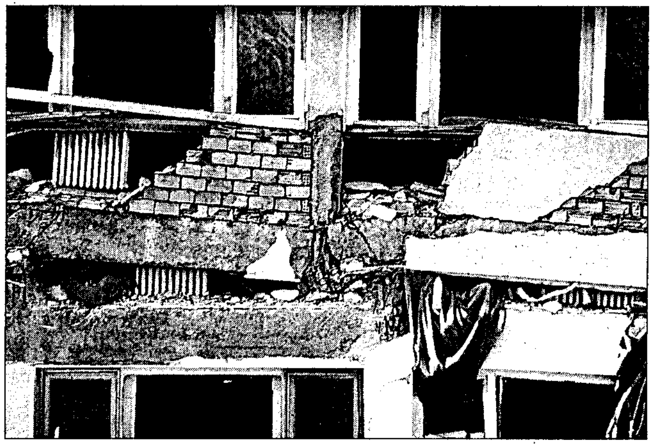

Fig. 3 — Military hospital with strong beams and weak columns.

Fig. 5 — Example of lack of transverse reinforcement and resulting brittle behavior in columns.

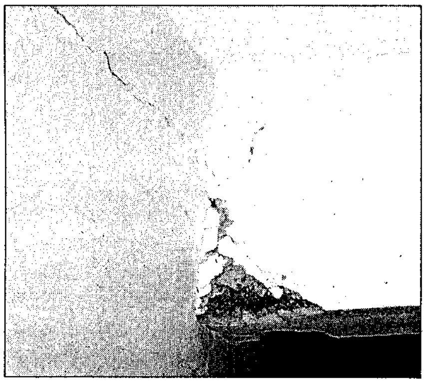

Fig. 4 — Crushing of beam cover concrete in the Officers' Club.

Structural performance

Reinforced concrete frame structures

Effects of masonry infills: The predominant form of construction in Erzincan consists of low-rise reinforced concrete frames with non-bearing unreinforced brick masonry infill walls. The masonry walls are used both for exterior enclosure and interior partitioning. These walls contributed significantly to the lateral stiffness of buildings during response. In many cases, the masonry walls were able to control lateral drift enough that the reinforced concrete components remained essentially elastic. This was especially true in low-rise buildings, older buildings where the ratio of wall to floor area was very high, and buildings located near the firm soil close to the mountains. Most low-rise reinforced concrete buildings that survived the earthquake were of this type.

When structural response and deformability demands exceeded the threshold for elastic response, however, crushing and diagonal cracking in brick masonry walls occurred. In some of the older buildings, where higher quality solid bricks were used, the elastic limit was not exceeded. Most brick crushing observed was associated with high deformability demands in buildings, coupled with the use of low grade hollow bricks produced for non-bearing purposes. The failure of the brick infills placed the burden of seismic resistance on reinforced concrete elements, which were called upon to resist these forces without a significant loss of strength. Performance of these elements depended on the design and detailing practices employed.

The requirements for design of reinforced concrete structures in Turkey are specified in the Turkish Standards Institute's TS 500 building code.[3] The seismic provisions are in Part III of the Earthquake Research Institute's "Specifications for Structures To Be Built in Disaster Areas,"[4] referred to as the Seismic Code. The Seismic Code was published in 1975, and contains provisions similar to those included in Appendix A of ACI 318-1971 "Building Code Requirements for Reinforced Concrete."[5] The seismic design base shear specified in the Seismic Code for ductile moment resisting frames with unreinforced masonry infills, located in a high seismic zone and on soft soils, is approximately 60 percent higher than that specified by the 1976 edition of the Uniform Building Code[6] for comparable conditions. The relatively high design base shear required by the Seismic Code may explain the survival of some of the reinforced concrete buildings with masonry infills that responded elastically during the earthquake.

Soft stories: Many reinforced concrete frame buildings in Erzincan have soft stories. Generally, the first floors are devoted to commercial areas enclosed with glass windows, resulting in soft stories (Fig. 1). This is especially true in the business and commercial districts of the city, although apartment buildings with first floor stores are common throughout the city. Heavy masonry exterior and partition walls start immediately above the commercial floor, significantly increasing mass above the soft story. During the earthquake, the presence of soft stories increased deformation demands on the first-story columns. If these columns were not designed and detailed for large cycles of inelastic deformation cycles, column failures occurred, sometimes resulting in total structural collapse.

Strong beams, weak columns: A common type of reinforced concrete slab system used in Turkey consists of hollow concrete masonry units cast with cast-in-place reinforced concrete to form a concealed waffle slab. Consequently the slab thickness and the depth of the attached beams are significantly higher than a typical two-way reinforced concrete slab system. Columns remain relatively small in size and weaker in flexure than the attached beams at the joints. This results in strong beams and weak columns at beam-column connections, forcing plastic hinges to form in the columns.

Fig. 2 illustrates the Red-Crescent Business Centre in downtown Erzincan, which clearly had deep and strong beams relative to the supporting columns, and suffered column failures followed by a complete loss of a story. Another example of such failure was observed in a military hospital (Fig. 3), which again illustrates column failures and a loss of a complete story. The same structural system was used in the Social Security Hospital, a wing of which completely collapsed, killing more than 80 people.

An examination of Turkish codes revealed that neither TS 500 nor the Seismic Code had any provision against the use of strong beams and weak columns. The strong-column weak-beam behavior, as stipulated in the ACI 318-89[1] building code, was observed in the north wing of the New Police Headquarters. This building's beams developed flexural cracks wide enough to signify potential yielding of the longitudinal tension reinforcement near the columns. The columns maintained their elastic behavior without any sign of distress. The Officer's Club behaved similarly, despite the heavy damage observed in its unreinforced brick infill walls, which may be viewed as an indication of high drift demand. Structural integrity and strength was maintained by columns performing well, while hinging occurred at the ends of beams. Fig. 4 illustrates crushing of cover concrete at the end of a beam.

Column confinement: The presence of soft stories and strong beams, and a high earthquake response in general, put a tremendous burden on the lower story columns, especially beyond the threshold of elastic response for the infill walls. High deformability demands in columns could not be met in most cases due to lack of concrete confinement. Most structural damage observed was in the first story columns in the form of concrete crushing and reinforcement buckling. In almost all of the cases, virtually no confinement reinforcement was found.

Observations indicated that a four bar arrangement was quite common, especially in apartment buildings. Column ties in most cases consisted of 6 mm (1/4 in.) plain bars with 90 degree bends used as hoop steel, with a spacing of 200 to 500 mm (8 to 20 in.) This practice violates the Seismic Code, which calls for confinement reinforcement at the ends of columns of ductile moment resisting frames. The minimum diameter for confinement reinforcement is specified to be 8 mm (0.3 in.) and the maximum spacing of closed hoops is to be 100 mm (4.0 in.). The hoops are required to have 135 degree hooks at the ends and are to be extended at least 10 bar diameters into the column core.

Columns with longitudinal bars between the 4 corner bars did not have any cross ties. There were cases where as many as 9 bars were used along the side of a rectangular column without any cross tie, although the TS 500 Code requires adequate support of longitudinal reinforcement by interior hooks and/or cross ties, without a specific guideline. Fig. 5 illustrates column failures due to lack of concrete confinement.

Column shear failure: Shear failure in columns was one of the common

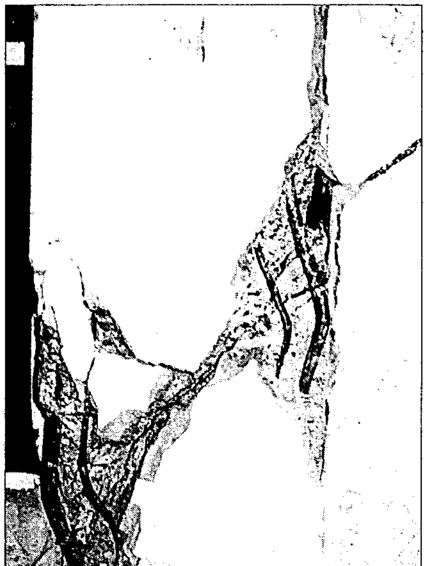

Fig. 6 — Diagonal tension failure in a column.

Fig. 7 — Shear failure in a rectangular column.

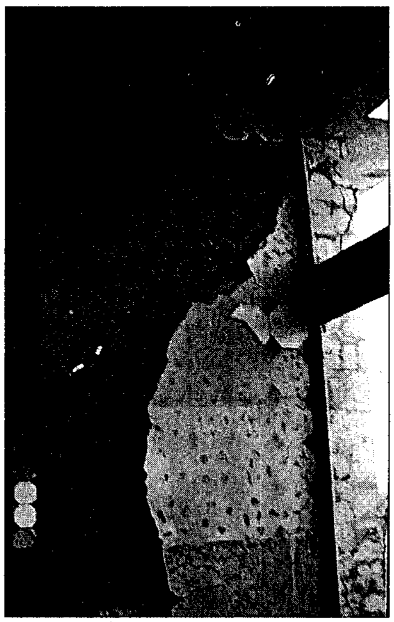

Fig. 8 — Shear failure in a rectangular column.

modes of structural failure observed. The failures were attributed to lack of transverse reinforcement, coupled with the use of low grade reinforcing steel. The minimum code specified steel grade of 220 MPa (32,00 psi) was used extensively in Erzincan. Fig. 6 shows two of the many diagonal tension failures observed throughout the city. Similar cracks were observed in wide rectangular columns, which attracted high shear forces. The rectangular columns of a commercial building and an apartment building (Fig. 7 and 8) illustrate typical diagonal cracks observed in wide columns.

Another group of columns that suffered shear failure was the captive columns. Extensive use of masonry walls produced unintended supports along columns, resulting in short column behavior. This was typical in columns of the Police College. Fig. 9 illustrates a typical short column supported by masonry walls above and below a window opening, reducing shear span and increasing shear vulnerability. It may be of interest to note that the Turkish Seismic Code does not allow window openings above filler walls unless proper recognition of short column behavior is given during design.


Fig. 9 — Short column behavior observed in the Police College.


Fig. 10 — Example of poor joint detailing.

Beam-column joints: Poor behavior of monolithic beam-column joints was observed in many buildings. Lack of joint reinforcement was evident in almost all the exposed joints, although the Seismic Code after 1975 called for seismic design of beam-column joints. The code requirements for joint design are very similar to those specified in ACI 318-89. Fig. 10 illustrates examples of buildings where joint reinforcement was completely missing.

Lack of good detailing practice and quality of concrete: A number of detailing deficiencies were observed in the damaged structures. One common problem was the lack of proper anchorage of beam reinforcement in the beam-column connection. Similar observations were made in the Military Hospital and the Red-Crescent Business Centre.

Some detailing deficiencies were observed in columns. While Fig. 11 shows inadequate splice length in columns. Poor detailing of column reinforcement is shown in Fig. 12 where the longitudinal reinforcement did not have lateral support in a joint with a beam offset. Column hoops with 90 degree bends were common in all exposed columns, although the Turkish Seismic Code called for 135 degree hooks, well anchored into the core concrete.

The quality of concrete was acceptable in most cases. The minimum concrete strength specified by the Seismic Code was 22.5 MPa (3260 psi), except for low occupancy buildings. The quality of concrete was poor especially in old apartment buildings. Concrete placement problems were observed in some buildings. Low quality concrete and workmanship are evident in some of the photographs shown, especially in Fig. 10.

Shear-wall frame interactive systems

Few buildings contained shear walls, although the north wing of the new police headquarters had shear walls in both orthogonal directions. The shear walls controlled lateral drift during the earthquake, preventing structural and non-structural damage. The masonry infills of this wing survived the earthquake without any visible damage. Reinforced concrete frames behaved in a satisfactory manner with some signs of beam hinging at the ends, while the columns maintained their integrity and appeared to have responded elastically. The south wing of the same building, separated by an expansion joint, did not have any shear walls. This wing developed extensive damage of unreinforced masonry, and also some structural damage at the roof level. Fig. 13 and 14 illustrate both wings of the building after the earthquake.

A condominium complex in the northeast was under construction at the time of the earthquake and had shear walls in both directions. Although these buildings were under construction, they were complete in terms of the structural framing as well as ma-

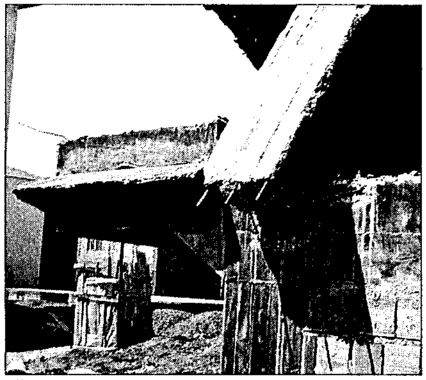
Fig. 11 — Inadequate splice length in columns.

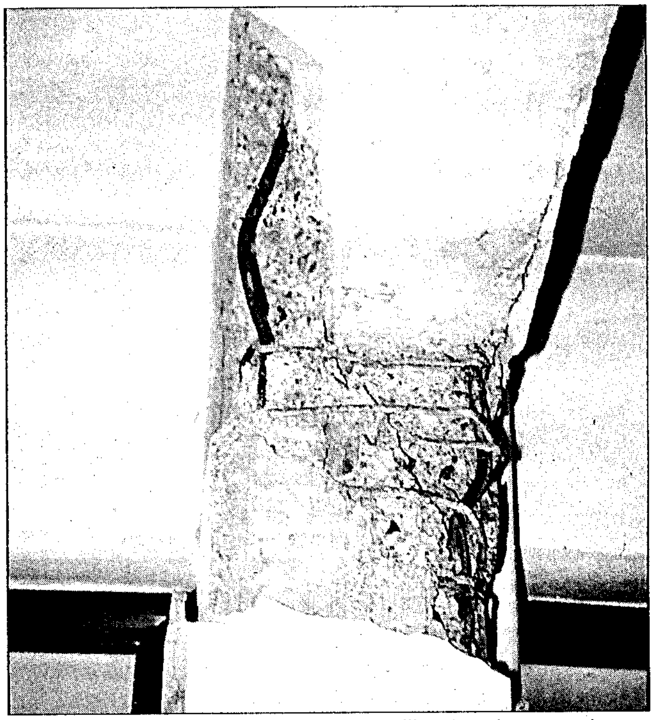

Fig. 12 — Lack of reinforcement detailing in a beam-column joint with beam offset.

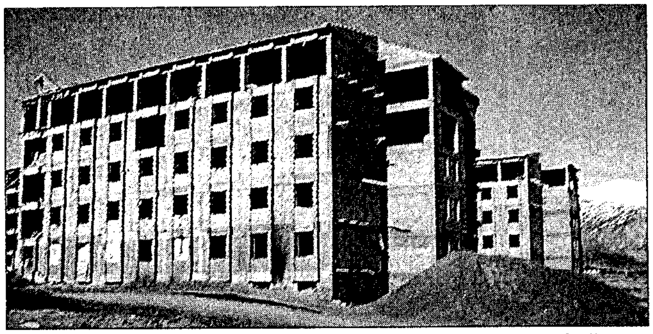

Fig. 15 — Condominium buildings with frame-shear wall interactive systems after the earthquake.

sonry enclosure and partitions. Therefore, practically all the mass of the building was in place during the earthquake. All of these buildings survived the earthquake without any sign of structural or non-structural damage (Fig. 15).

Precast industrial buildings

Two industrial complexes with precast concrete elements were found in Erzincan. One consisted of a pair of single story frame buildings with precast columns and beams forming the framing system, which was enclosed with unreinforced brick masonry. No sign of damage was observed in these buildings. The other was a new animal slaughter house consisting of two precast buildings. Long span precast girders and supporting prefabricated concrete columns formed the framing system. One of the two buildings had concrete cladding in both directions. This building survived the earthquake without any damage. The second building, only meters away from the first building, was not enclosed by cladding. It suffered connection failures, and collapsed completely (Fig. 16).

Concrete bridges

Only one of the concrete bridges suffered damage during the earthquake, essentially due to the settlement of the abutments. The bridge was a three-span, cast-in-place railroad overpass with two transverse beams supported by three columns each. The columns were 800 mm (32 in.) in diameter and 12 m (39 ft) in height. Fig. 17 shows the overall view of the bridge. The settlement at the north abutment led to the tilting of the retaining wall, which bared against the north columns. These columns developed cracking at the ends wide enough to signify yielding.

Fig. 13 — North wing with shear walls of the New Police Headquarters after the earthquake (on the left).

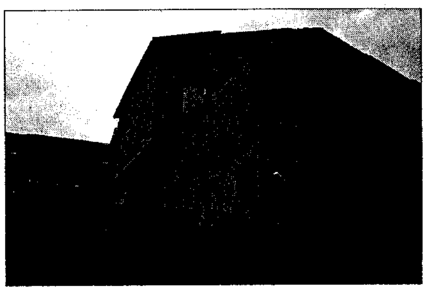

Fig. 14 — South wing without shear walls of the New Police Headquarters after the earthquake (on the right).

Summary and conclusions

The recent earthquake in Erzincan has demonstrated once again the poor performance of brittle structural systems during strong earthquakes. Amplification of ground motion due to the soft soil conditions of Erzincan, the close proximity of the epicenter, and widespread use of soft stories increased ductility demands on structural components.

Some of the low-rise buildings with unreinforced masonry infills remained elastic during the earthquake, and hence performed well. Reinforced concrete frames of these buildings were not damaged significantly, even if they were designed and built without any seismic considerations. Those that responded beyond the threshold for elastic behavior of unreinforced masonry suffered irreparable damage of reinforced concrete columns and beam-column connections in most cases, sometimes leading to total structural collapse. The cause of most of the structural damage observed can be at-

Fig. 16 — Total collapse of a precast industrial building.

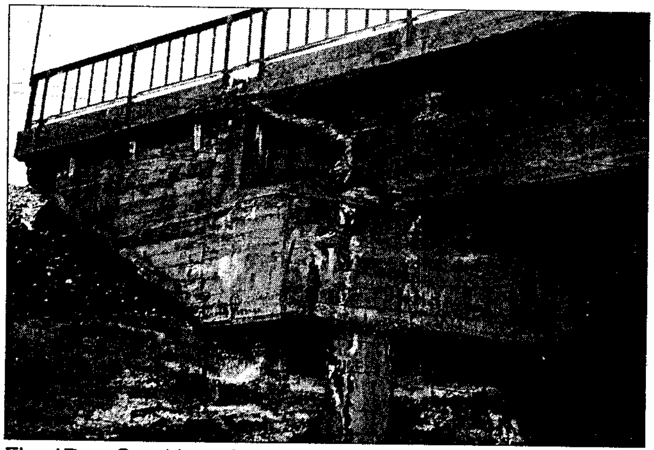

Fig. 17 — Cracking of the bridge piers of a three-span reinforced concrete bridge on the south-west highway.

tributed to a lack of seismic design practice in the region, including lack of column confinement and shear reinforcement, lack of joint reinforcement and detailing, and poor workmanship and quality of materials in general.

Although the seismic provisions of the local building code were comparable to those of the ACI 318 Building Code at the time, lack of code enforcement for new buildings explains the non-complaint seismic designs observed in the region. For older buildings, the observed deficiencies largely reflect the incomplete state-of-knowledge at the time of their construction. Few shear-wall frame interactive systems that existed at the time of the earthquake performed remarkably well. A requirement for shear walls in every building could be an alternative approach to prevent similar disasters in the future, if the enforcement of seismic provisions of the building code cannot be accomplished.

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Selected for reader interest by the editors.

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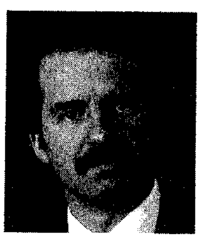

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