Performance of structures during the 1992 Erzincan earthquake

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A large number of engineered buildings were severely damaged and collapsed during the Richter magnitude 6.8 earthquake that struck Erzincan, Turkey, on March 13, 1992. The building inventory of the city contained structures similar to those built worldwide at the time of their construction. The study of the effects of this earthquake on structures is of particular significance to Canada, since an earthquake of approximately the same magnitude is likely to occur in many parts of the country, where similar structural systems are used. This paper reports on the findings of a 4-day reconnaissance visit conducted by the authors. A brief description of seismological aspects of the earthquake is presented, including characteristics of the ground motion. The emphasis is placed on structural performance during the earthquake, and possible causes of observed structural failures and collapses.

Key words: building code, concrete, earthquake, Erzincan, masonry, precast concrete, reinforced concrete, seismic design, structures, structural design, steel, unreinforced masonry.

Nombreux édifices conçus et dimensionnés par des ingénieurs furent détruits ou endommagés lors du tremblement de terre de magnitude Richter 6.8 qui frappa la ville d'Erzincan, en Turquie, le 13 mars 1992. La plupart de ces édifices (et autres structures) présents à Erzincan lors du séisme étaient de conception semblables à d'autres ailleurs à travers le monde construits aux mêmes époques. L'étude des conséquences de ce séisme sur ces structures est de grande importance pour le Canada puisque des tremblements de terre de puissance semblable sont attendus dans plusieurs régions du pays où des systèmes structuraux similaires sont également utilisés. Cet article rapporte les observations et conclusions formulées suite à une visite de 4 jours sur le terrain par les auteurs. Une brève description du contexte séismologique et des caractéristiques particulières des enregistrements obtenus est présentée. Toutefois, l'emphase, dans cet article, porte tout particulièrement sur le comportement des structures durant le tremblement de terre, et la description des nombreuses causes de leur ruine totale ou partielle.

Mots clés : code du bâtiment, béton, tremblement de terre, Erzincan, maçonnerie, béton précontraint, béton armé, conception séismique, structures, conception structurale, acier, maçonnerie non-renforcée.

Introduction

On March 13, 1992, an earthquake with a magnitude of 6.8 on the Richter scale struck Erzincan, a city of 100 000 population in eastern Turkey. In spite of its relatively moderate magnitude, this earthquake is of great engineering significance, since the epicentre was only 7.7 km north of the city. The full non-attenuated intensity and damaging power of the earthquake was felt in the city, destroying about 300 buildings and damaging approximately another 500. The number of fatalities was over 525, and seriously injured was near 700. The unofficial death toll, based on the estimates of the survivors in the region, was in excess of 3000.

Opportunities to study the seismic performance of structures located so close to the epicentre are rare, especially in a city like Erzincan, which has a unique history of construction practice. Indeed, following the 1939 Richter magnitude 8.0 Erzincan earthquake, which destroyed 6600 of the 7200 buildings in the region, and badly damaged all the others, the city was slowly rebuilt using building forms and structural systems similar to those used during the same period in Europe and North America. A review of their performance during this earthquake is undoubtedly instructive of how similar buildings are likely to perform in eastern and western Canada, where earthquakes of similar or greater magnitudes are likely to occur.

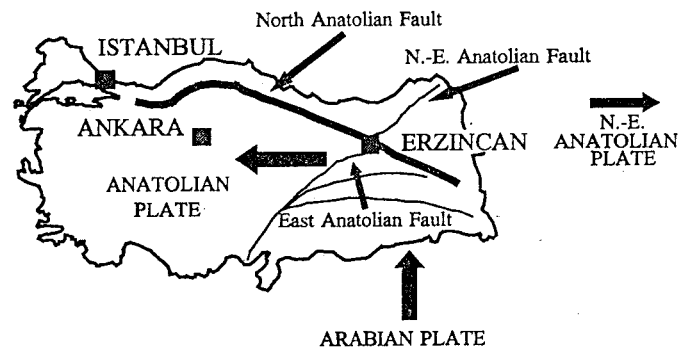This paper reports on the findings of a 4-day recon-

FIG. 1. Tectonic structure of Turkey (adapted from Gulkan, 1992).

naissance visit by the authors to the disaster area, conducted in cooperation with the Ministry of Reconstruction and Resettlement of the Republic of Turkey, and the Middle East Technical University, Ankara, Turkey. It is intended to provide a brief description of seismological and geological features of the region, to analyze the characteristics of the recorded near-source ground motions, and to report on the observed structural damage. It is also intended to evaluate seismic performance of structures in the affected area from a Canadian perspective. Lessons learned and re-learned from this earthquake are believed to have direct implications on seismic resistant design practice.

Seismological and geological aspects

Tectonic structure and history of earthquakes
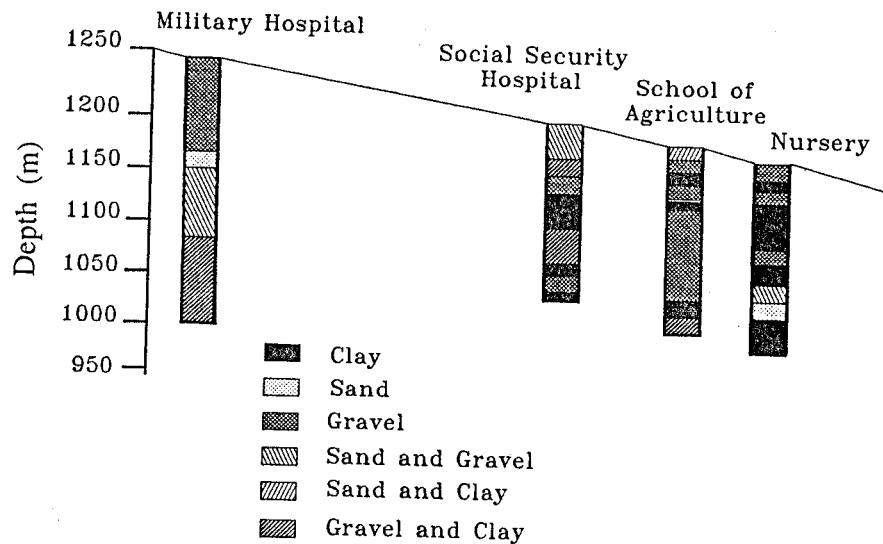
Erzincan is one of the eastern cities of the Republic of

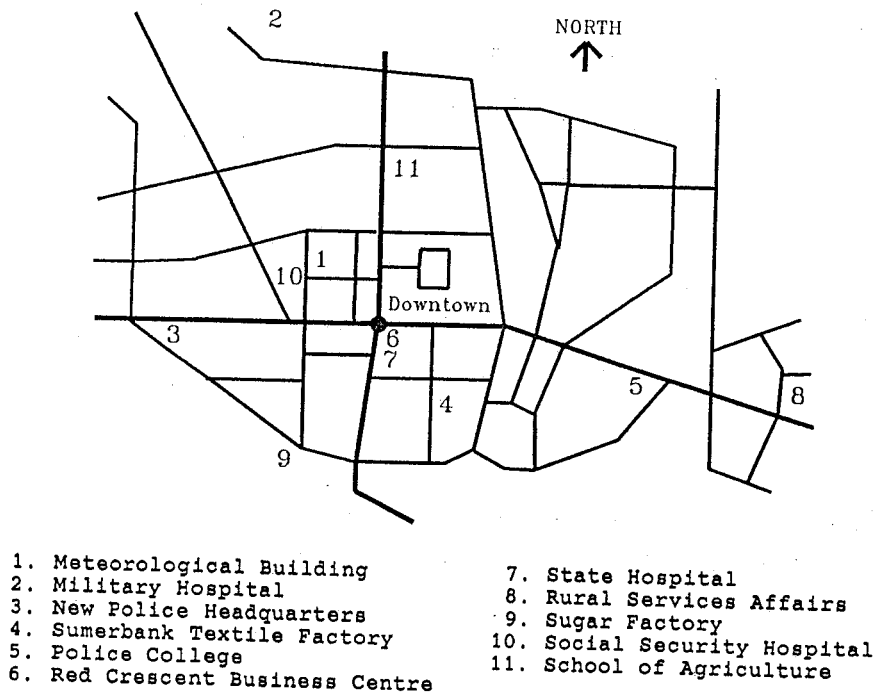FIG. 2. Soil borings conducted by the State Waterworks Directorate (adapted from Gulkan, 1992).

1. Meteorological Building
2. Military Hospital
3. New Police Headquarters
4. Sumerbank Textile Factory
5. Police College
6. Red Crescent Business Centre

7. State Hospital
8. Rural Services Affairs
9. Sugar Factory
10. Social Security Hospital
11. School of Agriculture

FIG. 3. Erzincan city map.

Turkey, situated in the vicinity of a number of active fault lines, all of which have ruptured during the last four decades. Figure 1 illustrates the tectonic structure of the region. The city, while straddling the North Anatolian fault, also happens to be situated immediately southeast of the intersection of the North and North Eastern Anatolian fault lines. The area is affected by subduction of the Arabian Plate, which causes westward and eastward movements of the Anatolian and North Eastern Anatolian Plates, respectively. The average rate of movement along the North Anatolian fault is estimated to be 20 mm/year (Gulkan 1992).

Over 30 damaging earthquakes were recorded in Erzincan within the last 1000 years. About 20 of these were of catastrophic nature. The most recent earthquake prior to the March 13, 1992, event occurred on November 18, 1983. This earthquake had a magnitude of 5.6, with peak ground accelerations (PGA) of 0.07g and 0.04g in the N–S and

vertical directions, respectively. Although there was no fatality, a number of engineered structures were lightly damaged. It may be of interest to note that the majority of buildings that were damaged in 1983 collapsed during the 1992 earthquake.

The great Erzincan earthquake of December 27, 1939, with a Richter magnitude of 8.0, was one of the strongest and most damaging earthquakes ever recorded worldwide, killing 32 000 people and leaving 230 000 homeless. The surface fracture observed after the earthquake extended about 350 km. Similar devastating earthquakes have reportedly occurred in or near Erzincan in 1888, 1782, 1584, 1457, and 1047.

Soil conditions

Erzincan is located in a valley between two mountain ranges running in the east–west direction. The Karasu (Black
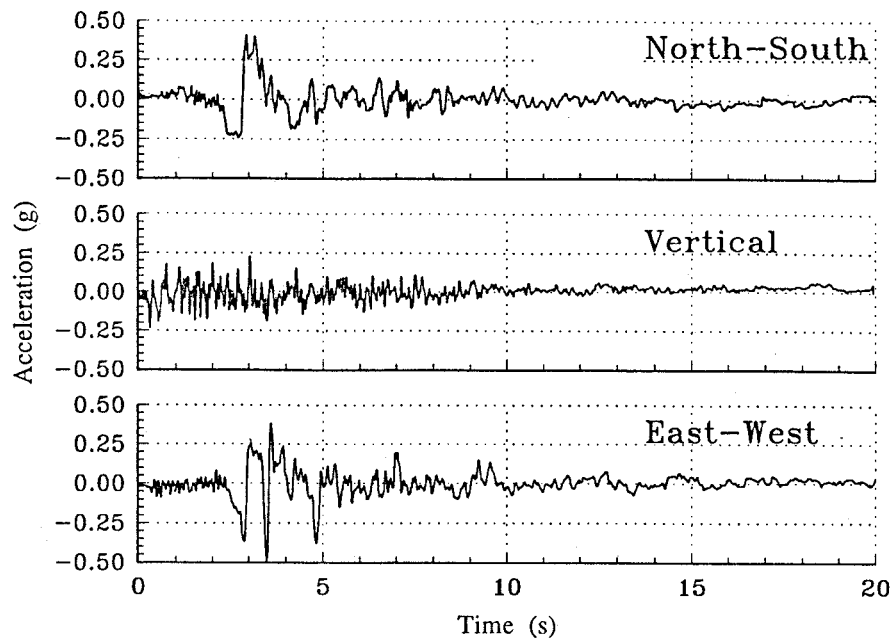
FIG. 4. Strong motion accelerograms.

Water) River runs through the city and is fed by many small creeks. The mountain peaks reach up to 3500 m altitude, while the low points of the river bed vary from 1200 to 1500 m above sea level. The city is underlaid by thick alluvial soil, typical of river-born sediments. The depth of the alluvial sediments increases rapidly with distance from the mountains, and is believed to be a couple of hundred metres in the central part of the city. Figure 2 illustrates the results of soil borings done by the State Water Works Directorate, at the locations identified on the city map depicted in Fig. 3. Apparently, no boring has ever reached bedrock in central Erzincan.

The average water table in Erzincan reportedly varies from 40 m in the north to 18 m in the south, although these figures are expected to show significant seasonal variations. In March 1992, just before the earthquake, the ground water level was reported to be only a few metres in certain parts of the city (Gulkan 1992).

Surface faulting, settlements, and liquefaction

Although no surface faulting was observed by the authors, the preliminary field reconnaissance report submitted by Gulkan (1992) mentions a 200 m long discontinuous ground rupture observed by one of the visiting teams at the northernmost part of the city, near the Military Hospital. This was alleged to be an authentic fault trace, since it laid on the extension of the previously formed ground rupture that had occurred during the great 1939 Erzincan earthquake.

Two instances of ground settlement were observed in the form of embankment settlements. One of them was along the eastern highway leading to Erzurum. At approximately 15 km east of the city the roadway was cracked both longitudinally and transversely, and the underlying sand was settled. At the same location, the railway running parallel to the highway, about 500 m north, suffered a slight offset which was rapidly fixed. The other settlement was observed at both ends of the bridge on the southwest highway leading to Kemah. The bridge is only a couple of kilometres from the city, overcrossing the railway. The abutments on either side settled significantly, sliding and tilting the retaining wall
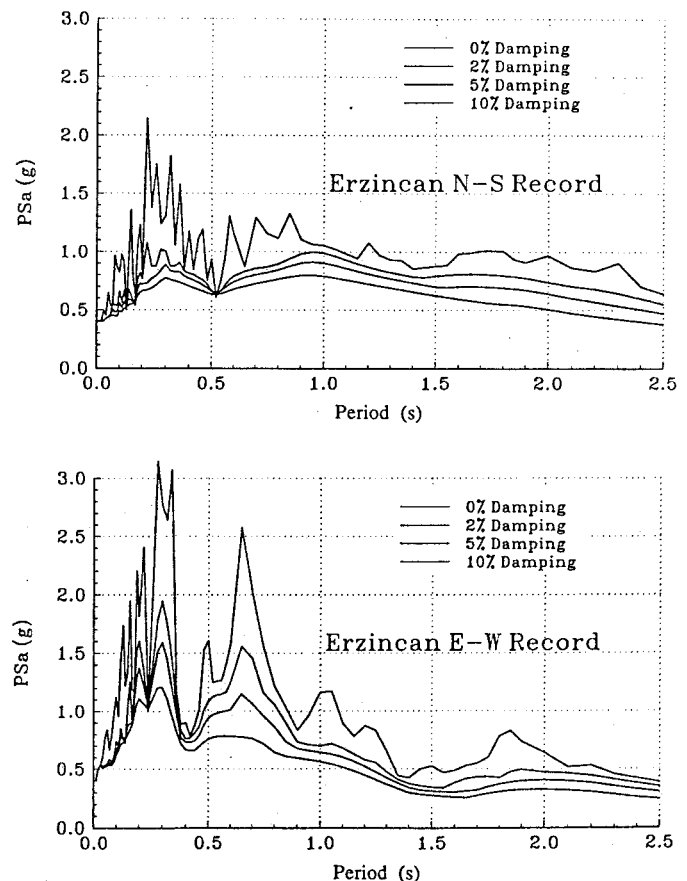

FIG. 5. Linear elastic response spectra.

at the north end, and also causing separations from the bridge deck at either ends. Other settlements were reported by Gulkan (1992) in the vicinity of the mineral water facility, 8 km east of the city, and at Mertekli diversion facility on Firat River. Some minor cases of liquefaction and landslides have also been reported following earlier field visits conducted by others (Gulkan 1992).
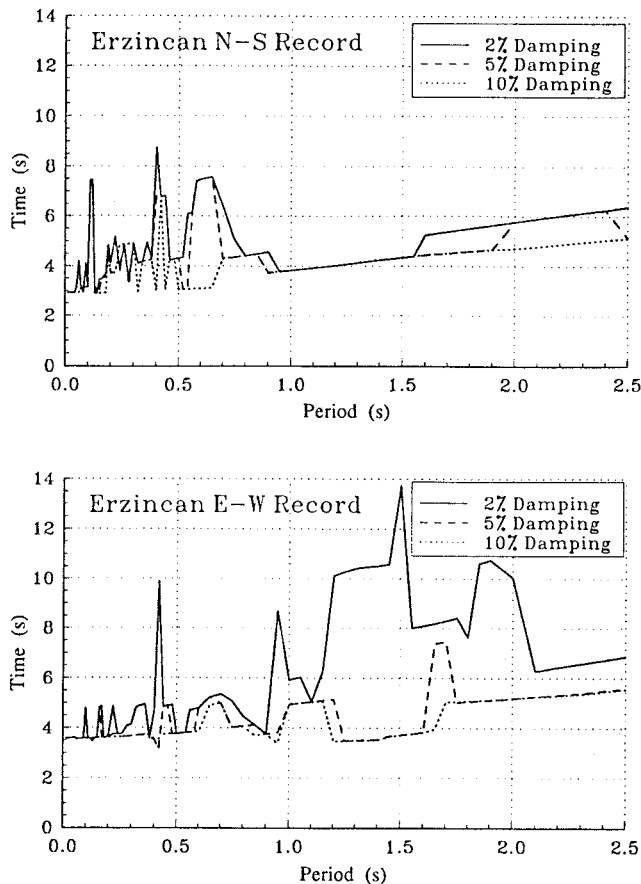
FIG. 6. Time at which the maximum elastic response (PSa) is obtained.

FIG. 7. Nonlinear response spectra for N–S direction based on elasto-perfectly plastic hysteretic model.

Characteristics of the ground motion

A single strong motion instrument was present in Erzincan at the time of the main shock. It was located in the Meteorological Services Building, approximately in the centre of the town, hence on the aforementioned deep soil deposit. The three acceleration time histories recorded by this instrument along orthogonal axes were obtained by the authors, and are plotted in Fig. 4. It is noteworthy that these records have been base and instrument corrected, but not filtered. From these records, the epicentral and hypocentral distances have been estimated to be 7.7 and 28 km respectively. The values of horizontal PGA recorded, 0.39g and 0.49g in the N–S and E–W directions respectively, are compatible with those anticipated at such a close epicentral distance from an earthquake of this magnitude as per the existing applicable attenuation relationships (Naeim 1989).

The duration of strong motion for this earthquake is highly sensitive to the calculation method adopted. The Trifunac–Brady durations (defined as the time interval between the 5% and 95% contributions to the integral of the square of the accelerogram — an approximate measure of input energy) are 10.51 (N–S) and 9.45 (E–W) seconds, but it is noteworthy that if the time interval between the 10% and 90% contributions are used instead, these strong motion durations drop to 5.53 (N–S) and 5.89 (E–W) seconds. The relatively short Trifunac–Brady duration of this earthquake emphasizes that the most of the energy of this earthquake is released intensively in its few early seconds. This can be visually confirmed by inspection of the ground acceleration time histories, where several large pulses of acceleration are
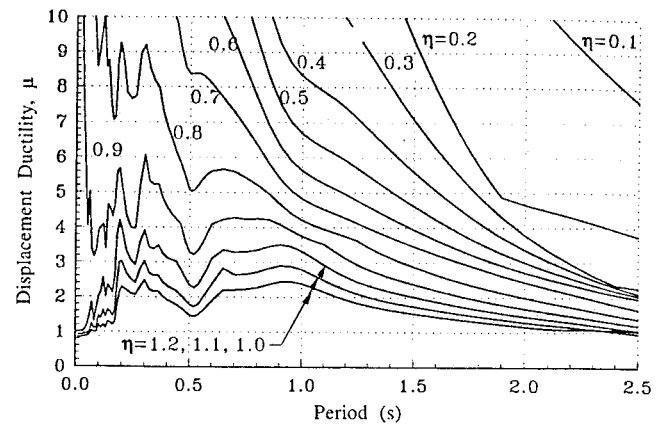
observed early in the record. Such damaging acceleration pulses, often present in near-source records, are known to translate into large energy demands on structural systems. Local soil conditions may be held partly responsible for the presence of these pulses but, in the absence of accelerogram-records on rock, and until further geotechnical investigations are conducted, this can only be speculated.

The bracketed durations (defined as the time interval between the first and last peaks greater than 0.05g) are 15.06 (N–S) and 14.45 (E–W) seconds. By comparison, this is still about half the average bracketed duration of strong motion expected for an earthquake of this magnitude. Thus, generally speaking, the structures in Erzincan have been subjected to fewer cycles of potentially damaging ground motion than normally anticipated from an earthquake of this magnitude; however, the severity of these cycles, as demonstrated below, was rather brutal.

Linear elastic response spectra have been generated by the authors and are presented in Fig. 5. Very large pseudo-accelerations (PSa) are observed throughout the period range for the N–S record, and for periods less than 0.8 s in the E–W direction. The dominant influence of the aforementioned pulses on the severity of the above spectra is eloquently expressed by Fig. 6, where the time at which the maximum elastic response is obtained is plotted for the same periods considered in the PSa spectra. For most periods, these maximums are generally reached at approximately 5 s after the onset of the ground motion, i.e., shortly after the end of the pulses. It is noteworthy that PGA beyond the pulses do not exceed 0.14g and 0.20g in the N–S and E–W directions respectively.

Unusually large ductility demands are known to develop in the inelastic range when the duration of pulses is long relative to the period of the structure (Mahin and Bertero 1981). Hence, inelastic response spectra are needed to illustrate the destructive effect of pulses, whereas elastic spectra alone can be misleading. A nonlinear inelastic response spectrum has thus been derived by the authors for the N–S component of excitation, assuming an elasto-perfectly plastic hysteretic model and 5% damping (Fig. 7). In that figure, displacement ductility demands, μ, can be read for each structural period as a function of the strength ratio, η, as defined as
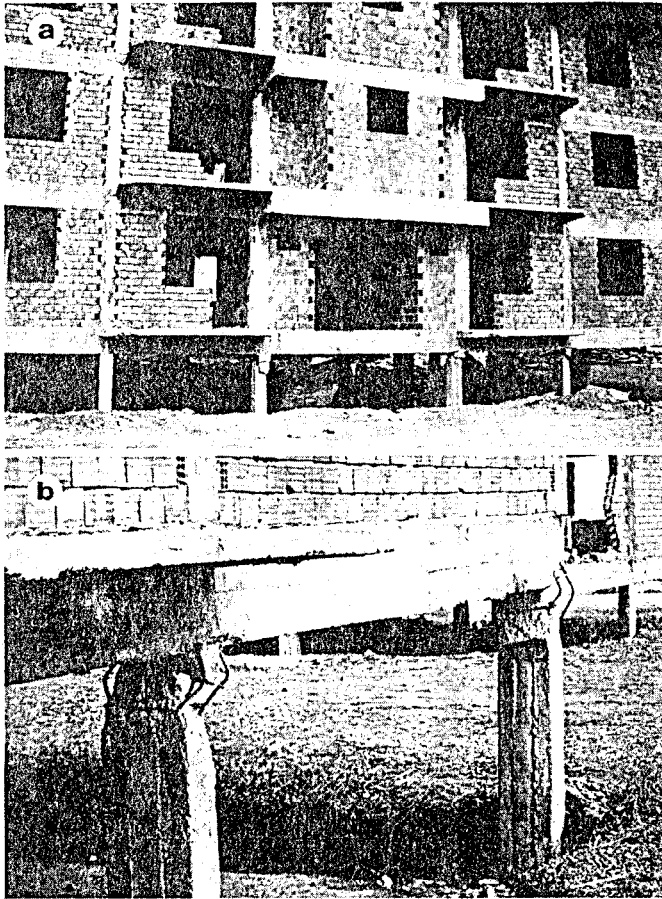
$$[1] \quad \eta = \frac{R_Y}{m\ddot{a}_g}$$

FIG. 8. (a) A reinforced concrete frame building with a soft storey, under construction; (b) close-up view of first-storey column failures.

FIG. 9. (a) A typical apartment building with a soft storey; (b) close-up view of column damage in the first floor.

where R_Y is the yield strength of the system, m is its mass, and $\ddot{a}_g$ is the PGA of the acceleration record. Clearly, from this inelastic spectra, ductility demands rapidly grow to unreasonable levels for progressively weaker systems and smaller periods. This observed high sensitivity of strength and ductility trade-off at the lower periods is typical of pulse-dominant near-source accelerograms. It also illustrates how slight improvements in strength, due to partition walls or infills for examples, translates into considerable reductions of ductility demand for this particular earthquake. The consequences of this will be evident throughout the following.

Characteristics of the building inventory

The predominant structural system used for buildings in Erzincan consists of reinforced concrete frames with unreinforced masonry infills. This structural form is used for all building heights and occupancy, from single-storey commercial to multistorey residential and office buildings. While the majority of buildings in Erzincan are lower than four storeys high, most new buildings are in the four- to six-storey range. The second most widely used type of construction, particularly for low-rise buildings, is unreinforced masonry (URM) bearing walls (mostly brick, rarely concrete masonry) supporting reinforced concrete slabs. Frame–shear wall interactive systems are also used in the new medium-rise buildings, although this practice is not common. The few industrial buildings that exist in the city are either reinforced concrete (cast-in-place or precast) or steel frame structures.
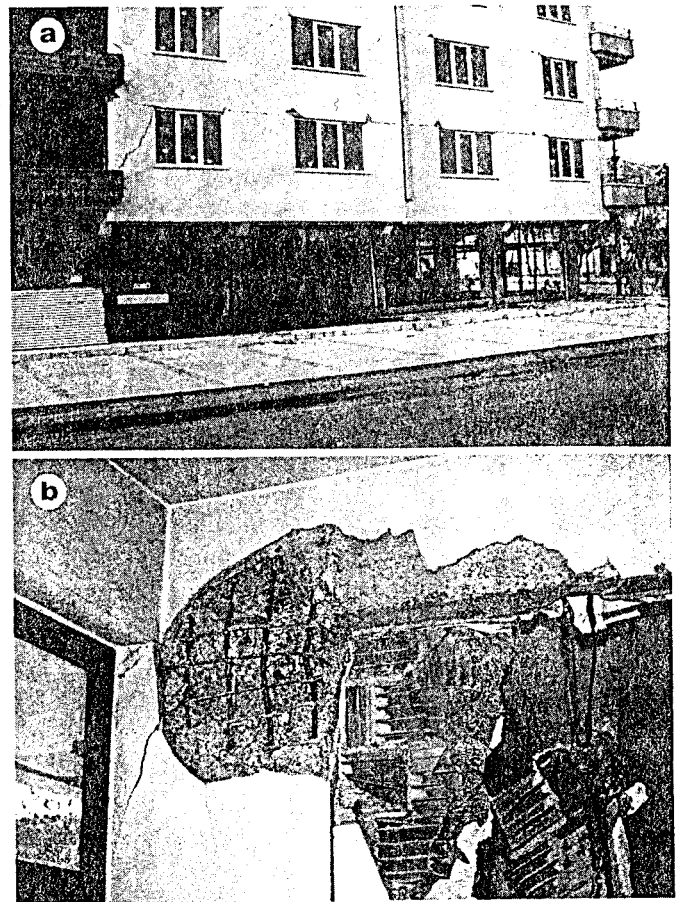
Many single-family houses in the outskirts of the city and the surrounding villages are built using locally made mud masonry units. The seismic performance of these various types of construction is reviewed in the following sections.

Reinforced concrete frame structures

A typical reinforced concrete frame building in Erzincan consists of a regular, symmetric floor plan, with square or rectangular columns and connecting beams. The exterior enclosure as well as interior partitioning are done by non-bearing unreinforced brick masonry infill walls. In many cases double layers of brick masonry, with insulation in between, are used for the exterior walls. These walls contributed significantly to the lateral stiffness of buildings during the earthquake and, in many instances, controlled the lateral drift and resisted seismic forces elastically. This was especially true in low-rise buildings, older buildings where the ratio of wall to floor area was very high, and buildings located close to the firm soil near the mountains, especially when the masonry walls started from the ground level and continued along the building height. Where structural response and deformability demands were high, the masonry walls were not able to remain elastic. In such buildings, typical diagonal cracks and crushing of brittle hollow bricks were observed. Failure of individual brick units triggered out-of-plane failures of infills in some cases. Once the brick infills failed, the lateral strength and stiffness had to be provided by the frames alone, which then experienced significant inelasticity in the critical regions. At
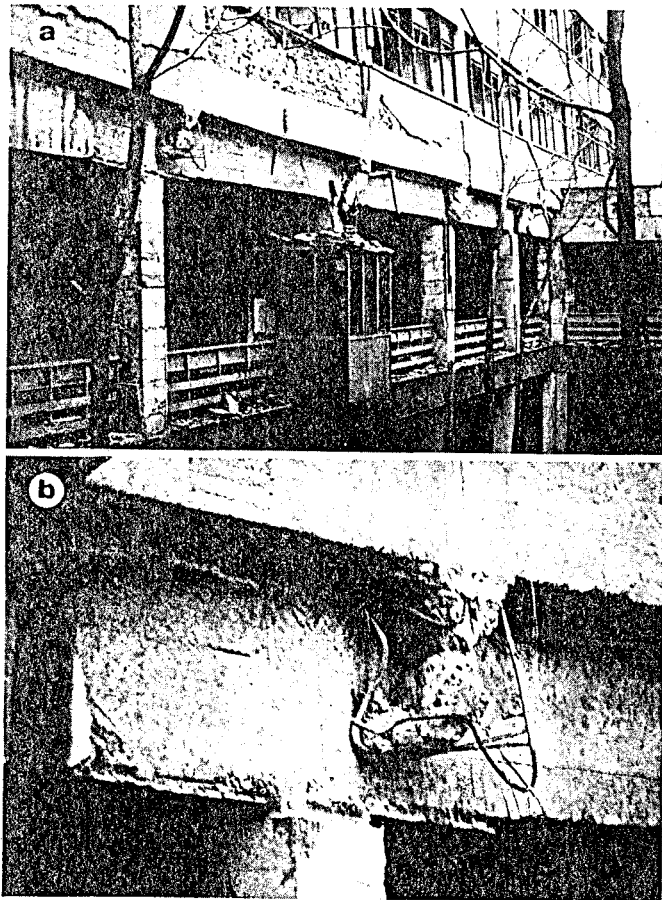
FIG. 10. (a) Example of a commercial centre with strong beams and weak columns which suffered the loss of a complete storey; (b) close-up view of column failures.
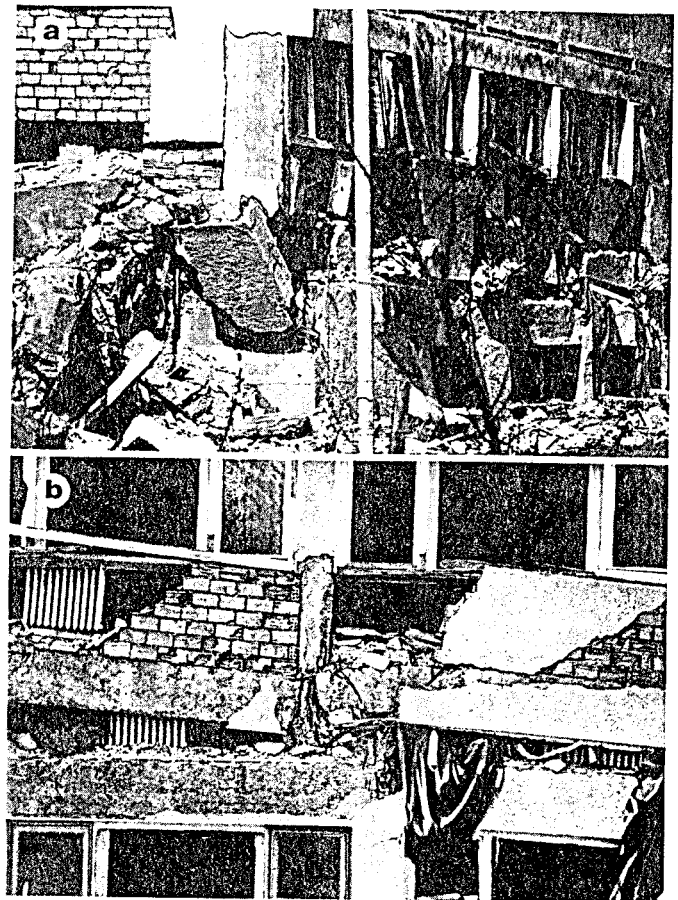

FIG. 11. (a) Military Hospital with strong beams and weak columns; (b) close-up view of a complete storey failure.

this stage the ability of reinforced concrete columns, beams, and beam-column joints to sustain deformation demands depended on how well the seismic design and detailing requirements were followed both in design and in construction.

The Turkish design requirements for reinforced concrete buildings are specified in TS 500 "Building code requirements for reinforced concrete" (Turkish Standards Institute 1985a), with seismic design provisions specified in Part III of "Specifications for structures to be built in disaster areas" (Earthquake Research Institute 1975), from now on referred to as the Seismic Code. A seismic resistant reinforced concrete structure must conform to both the provisions of TS 500 and the seismic design and detailing provisions outlined in the Seismic Code. The Seismic Code was published in 1975, and includes design requirements similar to those outlined in most modern building codes at the time. Seismic design base shear specified in the Turkish Seismic Code is given in [2].

$$[2] \quad F = C_0 KSIW$$

where C_0 = seismic zone coefficient ($C_0 = 0.1$ for highest seismic zone where Erzincan is located); K = coefficient related to structural type; S = dynamic coefficient for the structure; I = building importance coefficient ($I = 1.5$ for post-disaster and high-occupancy buildings; $I = 1.0$ for other buildings); and W = total weight of building.

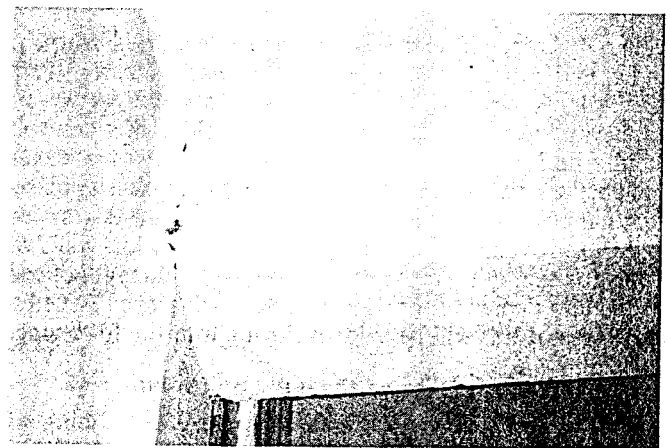The dynamic coefficient S is a function of the fundamen-

FIG. 12. Cracking in the critical region of a beam in the New Police Headquarters.

tal period, T, and the predominant period of the underlying soil, T_0, in seconds, as given by [3]. T_0 ranges between 0.2 for bedrock and 0.9 for soft clay. T_0 is specified to be 0.7 for soft and deep alluvial layers with a high water table, which is probably representative of the soil condition in the central portion of Erzincan.

$$[3] \quad S = \frac{1}{0.8 + T - T_0}$$

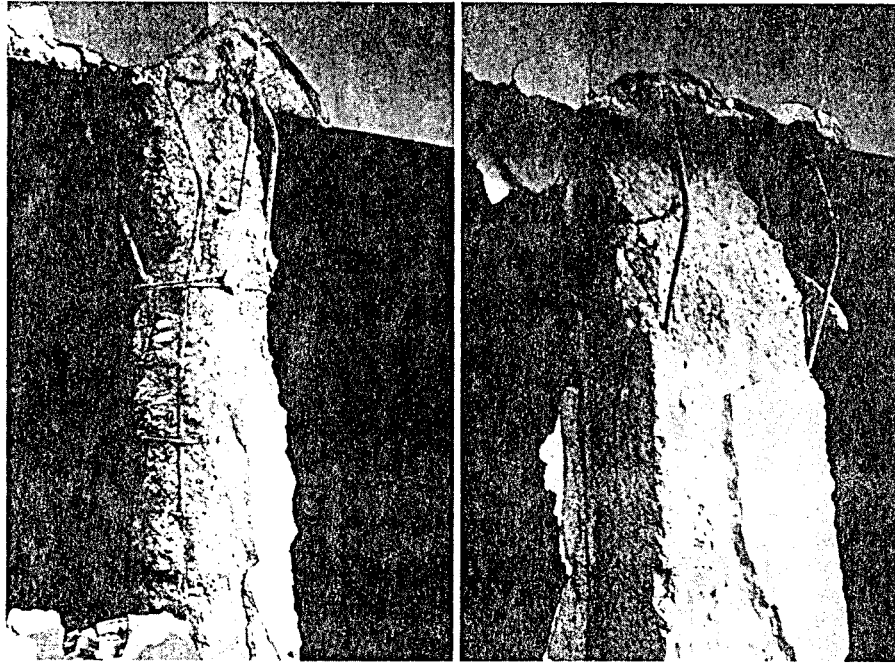The structural coefficient K for ductile moment resisting

FIG. 13. Column failures in the southeast residential area, caused by lack of transverse reinforcement.

FIG. 14. (a) Lack of column confinement and buckling of longitudinal reinforcement in the first storey of a commercial building in centre town; (b) lack of column confinement in Sumerbank Textile Factory.

frames is 0.8 for buildings with unreinforced masonry partitions, the type of building construction most commonly encountered in Erzincan. It may be of interest to note that ductile moment resisting frames are assigned two other values of structural coefficient K, depending on the type of partitions used. Those with reinforced concrete or reinforced masonry partitions are assigned a value of 0.6, whereas others with light and sparse partitions or prefabricated concrete walls are assigned a value of 1.0. Masonry buildings, which form the second most common building type in Erzincan, are assigned a K value of 1.5. The country is divided into five earthquake zones, with Erzincan being in

the most severe one. Comparison of seismic design base shears between the 1975 Turkish Seismic Code and the 1975 National Building Code of Canada (NBCC 1975) indicates that for ductile moment resisting frames with unreinforced masonry infill walls in the low period range, located in a similar site as the city of Erzincan, the base shear obtained from the Turkish Seismic Code is approximately 43% higher than that obtained from NBCC 1975. On the contrary, a similar comparison for unreinforced masonry buildings reveals that design base shears from the two codes are comparable.

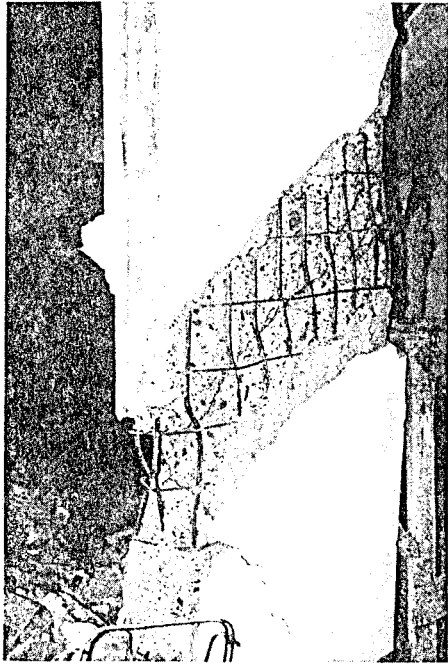The design provisions of the Turkish Seismic Code for

FIG. 15. Example of a rectangular column where no interior hoops or crossties were used to support the longitudinal reinforcement.

FIG. 16. Example of a reinforced concrete frame building with well-confined interior circular columns and poorly confined exterior square columns (courtesy of Profs. Ersoy and Tankut).

moment resisting ductile frame components call for confinement reinforcement at the ends of beams and columns by closed hoops, cross ties, and spirals, as well as for the design of beam-column joints, with considerations of joint shear and joint reinforcement. These provisions resemble those of the North American reinforced concrete building codes developed at the time, namely, ACI 318-1971 Appendix A (American Concrete Institute 1971), and CSA A23.3-1973 Chapter 19 (Canadian Standards Association 1973). Apparently, special seismic provisions for reinforced concrete did not exist in the Turkish Seismic Code prior to 1975, nor did they exist in ACI code and CSA standard prior to 1971 and 1973, respectively. The seismic provisions and building code developments in Turkey and Canada do not show wide discrepancies. Hence it is reasonable to expect similarities between buildings constructed in Erzincan and regions with similar seismic activity in Canada, during the same period, while recognizing differences in construction practices of the two countries. One major difference is perhaps the widespread use of unreinforced masonry as partition walls in Turkey, which is not as common in Canada except when used as exterior unreinforced masonry walls.

Most framed buildings in Erzincan suffered damage due to lack of seismic resistant design practice and poor reinforcement detailing. This was not surprising for buildings constructed prior to the Seismic Code of 1975. However, many recently constructed buildings were also damaged. The following sections are devoted to the discussion of design and detailing flaws observed, and the consequential structural damage, with relevant examples.

Soft storeys

A large number of residential and commercial buildings were built with soft storeys at the first-floor level. First storeys in Erzincan are often used as stores and commercial areas, especially in the central part of the city. These areas are enclosed with glass windows, and sometimes with a single masonry infill at the back. Heavy masonry infills start immediately above the commercial floor. During the earthquake, the presence of a soft storey increased deformation demands very significantly, and put the burden of energy dissipation on the first-storey columns. Many failures and collapses can be attributed to the increased deformation demands caused by soft storeys, coupled with lack of deformability of poorly designed columns. Figures 8 and 9 illustrate examples of apartment buildings with soft storeys.

Strong beams and weak columns

The observed behaviour of most frame structures indicated strong beams, remaining elastic, and weak columns suffering compression crushing or shear failure. In many cases relatively deep beams were used with flexible columns, contributing to the strong-beam weak-column behaviour. Figure 10 illustrates the failure of a complete storey in a commercial building with strong beams and weak columns. Another example of this type of failure is the Military Hospital shown in Fig. 11, for which one wing collapsed, and another lost a storey due to column failures.

The examination of TS 500 and the Turkish Seismic Code did not reveal any provision against the use of strong beams and weak columns. Further compounding this problem was the occasional use of a slab system which consisted of hollow concrete masonry units cast with cast-in-place reinforced concrete to form a concealed waffle slab, increasing the strength and stiffness of the beams significantly. This type of slab system, with 350 mm slab thickness, was used, for example, in the Social Security Hospital, a wing of which collapsed killing more than 80 people.

Strong-column weak-beam behaviour, as stipulated in the Canadian Standard CSA A23.3-M84 (Canadian Standards Association 1984) for ductile frames, was observed in only one building: the north wing of the New Police Headquarters, where some light shear walls were also used. In this wing of the building, beam yielding was developed as evidenced by the crack pattern, while the columns behaved elastically. Figure 12 shows a typical beam column subassembly of this building.

Lack of column confinement

Most of the structural damage observed in frame buildings was concentrated at column ends. The structures used in Erzincan called for high ductility demands on columns.
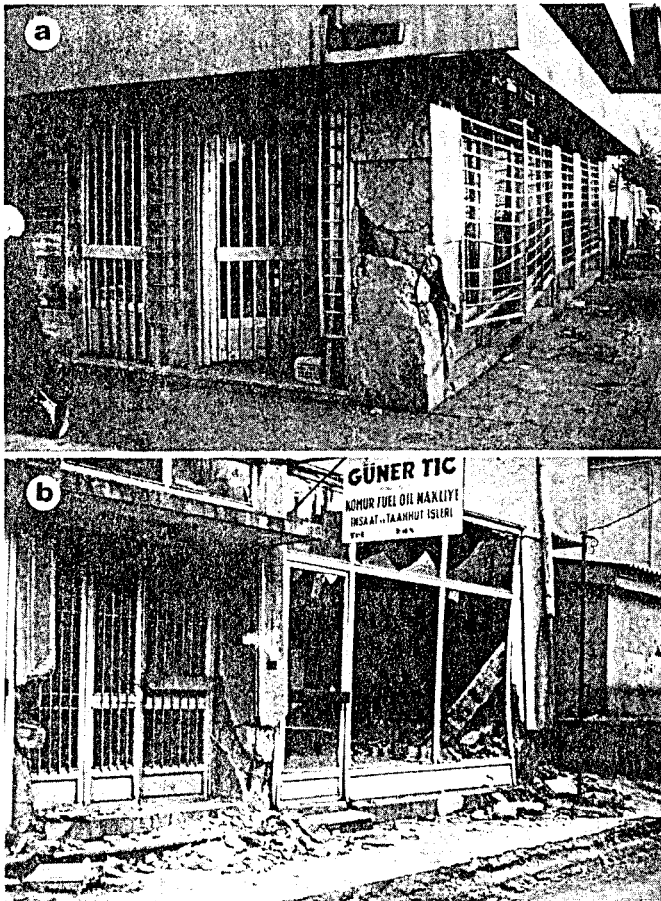
FIG. 17. Typical column shear failures in (a) a bank and (b) a commercial building in centre town.

Unfortunately, confinement reinforcement virtually did not exist in these members. Measurements taken from some of the columns indicated widespread use of 6 mm plain bars as hoop steel with 90° bends, placed at approximately 200 mm to as high as 500 mm spacings. It should be noted that the Turkish Seismic Code calls for confinement reinforcement at the ends of columns of ductile moment resisting frames. The minimum bar diameter for confinement steel is specified as 8 mm and the maximum spacing of closed hoops as 100 mm. The hoops are required to have 135° hooks, well anchored into the core concrete by extending the ends at least 10-bar diameter. These requirements are similar to those specified in CSA A23.3-M84 for ductile detailing. However, the confinement requirements were not followed in the damaged columns, some of which may have been designed prior to the 1975 Seismic Code. Most of these columns failed by premature crushing of concrete due to the lack of confinement reinforcement, followed by buckling of longitudinal reinforcement. Figure 13 illustrates typical column failures of this type in apartment buildings. Additional examples of column failures, resulting from lack of confinement, are illustrated in Fig. 14 for a first-storey column of a commercial building, and a column of a textile factory, which suffered similar column damages throughout the structure. It may be of interest to note that square columns with 4-bar arrangements and 1% longitudinal steel were very common in the apartment buildings investigated.

The longitudinal column reinforcement consisted typically of 16 mm diameter plain bars with no crossties. In some of
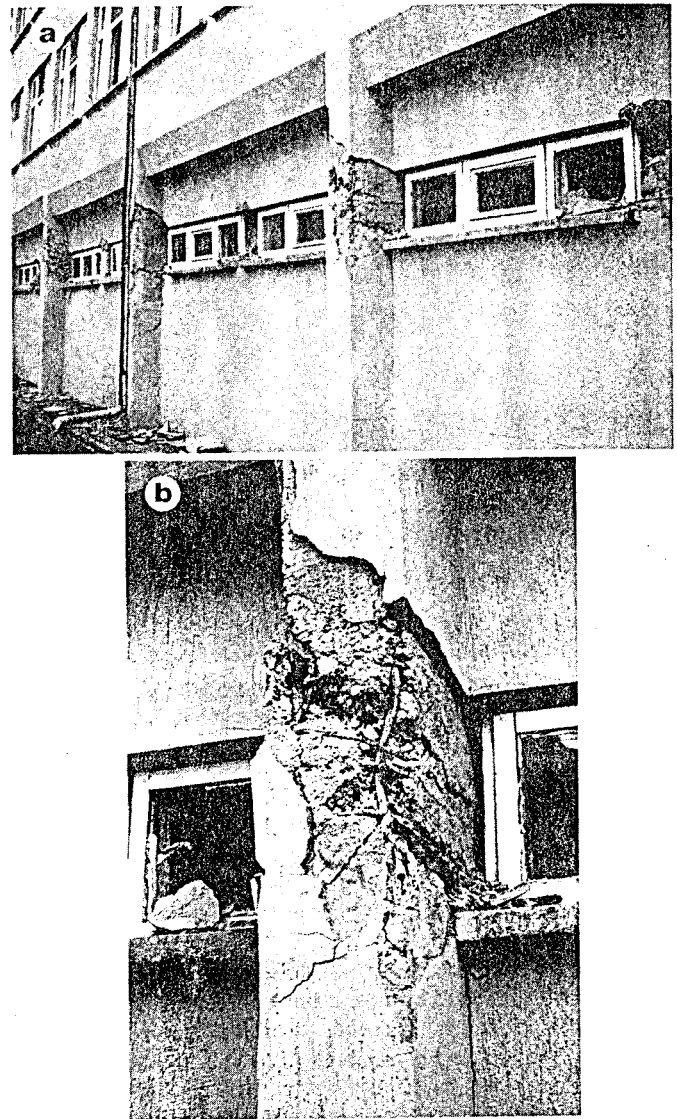

FIG. 18. (a) Short columns and (b) close-up view of short column behaviour in one of the buildings of the Police College.
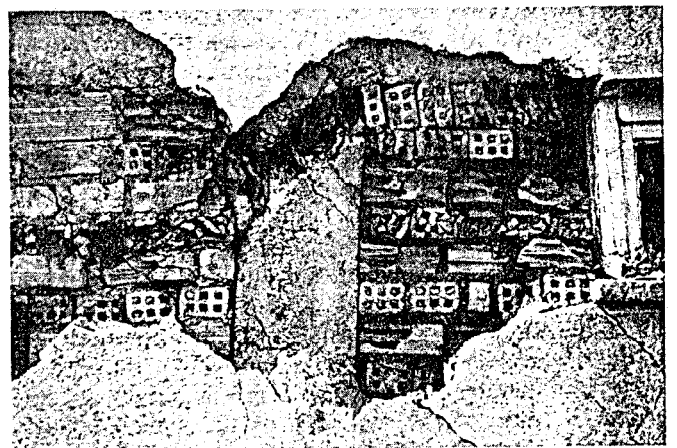

FIG. 19. Shear failure in a column supported by brick masonry.

the rectanglar columns, as many as 9 longitudinal bars were used with perimeter hoops only. Figure 9b, 14b, and 15 include examples of columns without the crossties. The current edition of TS 500 specifies that interior bars be
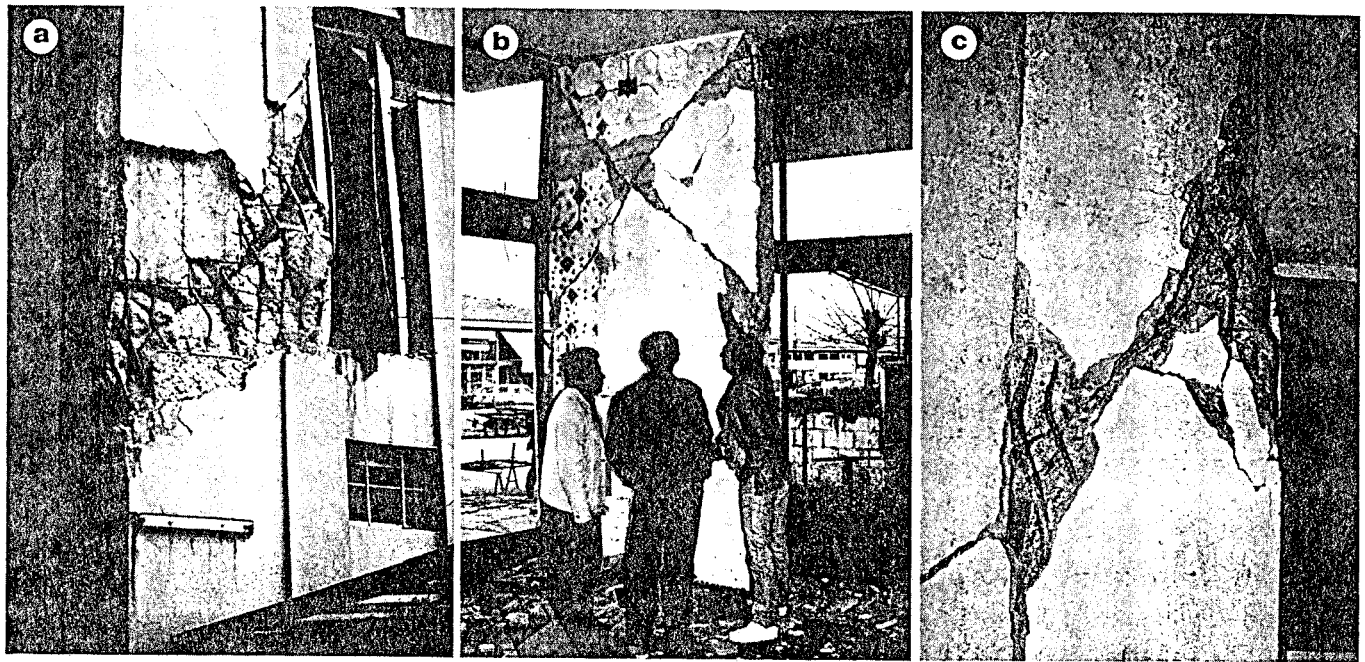
FIG. 20. Lack of shear capacity in rectangular columns: (*a*) multistorey office building in centre town; (*b*) two-storey commercial building on east–west highway, downtown area; (*c*) Sumerbank Textile Factory.

supported by adequate number of interior hoops and (or) crossties. If this is interpreted as each longitudinal bar must be supported by transverse reinforcement, then the Turkish provisions are more stringent than the equivalent requirement of CSA A23.3-M84, which permits leaving every other bar unsupported.

In one of the multistorey buildings, spirally reinforced circular columns and square columns were used together. The spiral columns showed superior performance as expected, while the square columns, lacking confinement steel, failed along with the supported slabs. Figure 16 illustrating this behaviour, clearly shows the importance of column confinement on seismic resistance of buildings.

Column shear failures

There were many cases of diagonal tension failures of first-storey columns. Figure 17 illustrates typical diagonal tension failures in a bank building and a commercial building. These failures are attributed to lack of sufficient transverse reinforcement. Compounding the problem was the use of relatively low grade steel. The minimum code-specified yield strength for all plain bars used in Erzincan is 220 MPa.

Another reason for column shear failures was high shear forces attracted by short columns, captive columns, and wide columns. Widespread use of masonry walls, providing unintended supports, resulted in captive columns in many buildings, increasing shear vulnerability of these members. Figure 18 illustrates the captive columns behaving as short columns in one of the buildings of the Police College. Incidentally, the Turkish Seismic Code does not permit window openings above the filler walls, as they lead to short columns. However, exceptions are made with proper recognition of short column behaviour. In some cases diagonal shear failures occurred in columns fully supported by brick masonry. Figure 19 includes one such example, illustrating local crushing of brick masonry and resulting

unsupported segment of the column behaving as a short column.

Some of the wide rectangular columns, attracting higher shear forces, also developed diagonal tension failures. These columns were responsible for resisting the total base shear in a given direction, and could not withstand the applied force with the shear capacity provided. Figures 15 and 20 show examples of such failures in apartment, office, commercial, and factory buildings.

Poor joint behaviour

Joint shear failures were the second most common cause of frame failure after the columns. Although the Turkish Seismic Code after 1975 called for seismic design of monolithic beam-column joints, virtually no joint reinforcement was observed in the buildings. This was true not only in the older buildings but also some of the recently built structures. Examples of joint failures are illustrated in Fig. 21. Observations made on the exposed beam-column joints of the collapsed wings of the Military Hospital and the Red Crescent Business Centre, shown in Fig. 22, indicate that no transverse reinforcement was used in the joints.

Lack of good detailing practice

A number of detailing deficiencies were observed in the damaged structures. One common fault was the anchorage of beam reinforcement in exterior and interior joints. Figure 23 illustrates an exterior joint of one of the buildings of Sumerbank Textile Factory where the beam reinforcement was anchored into the cover concrete outside the column cage. These bars lost their anchorage as soon as the cover concrete spalled off. Similar anchorage problems were observed in the Military Hospital as well as the Red Crescent Business Centre.

Other detailing deficiencies were observed relative to column reinforcement. Figure 24 illustrates a beam-column joint with an offset. The column reinforcement outside the

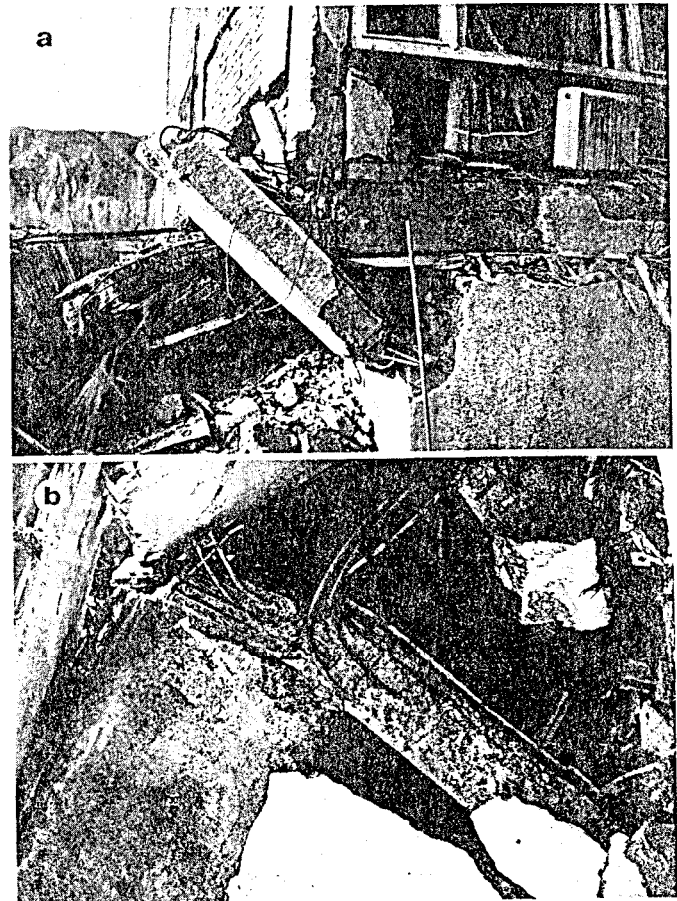FIG. 21. Examples of poor joint detailing.

FIG. 22. Failures attributed to lack of joint reinforcement: (a) Military Hospital; (b) Red-Crescent Business Centre.

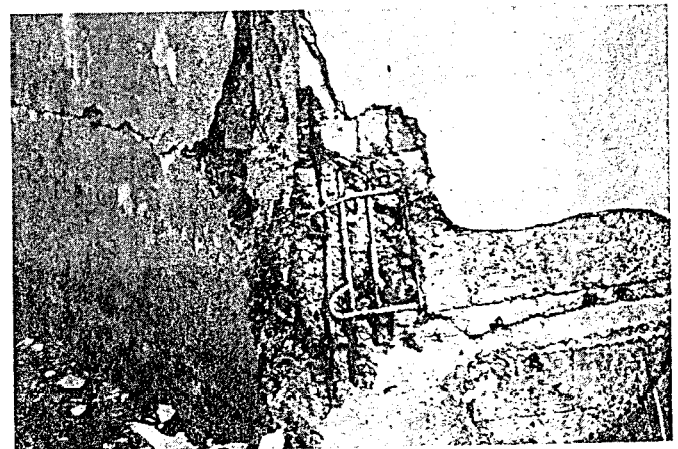FIG. 23. Anchorage of beam reinforcement in joint cover concrete.

joint was not supported laterally, and hence buckled upon spalling of the cover. Figure 25 illustrates lack of continuity between the columns and footings of the Sumerbank Textile Factory, where the column reinforcement was terminated at the surface of the footing. Insufficient splice length was also observed in the second-storey columns of a building that was under construction at the time of the earthquake. The resultant column failure is illustrated in Fig. 26. The use of 90° hooks for the hoop steel was another common problem, although 135° hooks has been required by the Turkish Seismic Code after 1975 for ductile columns. Opening of these hoops under lateral confinement pressure was the source of many failures.

Concrete quality

The Turkish seismic provisions include a minimum specified cylinder strength of 22.5 MPa for concretes used in all buildings except those designed for low occupancy. In most cases the quality of concrete appeared to be acceptable. Some placement problems were observed. However, there was no clear evidence of structural failures that may be directly attributed to concrete quality.

Behaviour of masonry infills

The masonry infills used in Erzincan were largely built out of unreinforced brick masonry. The use of concrete masonry was limited. The brick units used were either solid or hollow brick. The solid bricks were used essentially as load bearing units, although they were occasionally used as infills in some of the older buildings as well. Many different types of hollow, multicell bricks were used both as load bearing and non-load bearing units. The solid brick infills performed well, showing essentially elastic response, while
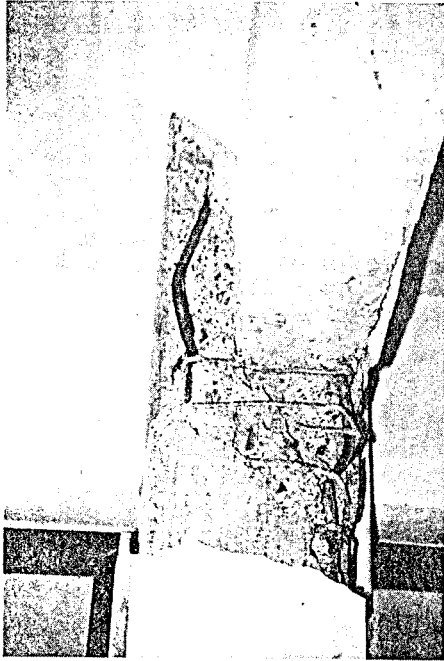
FIG. 24. Lack of reinforcement detailing in a beam-column joint with beam offset.
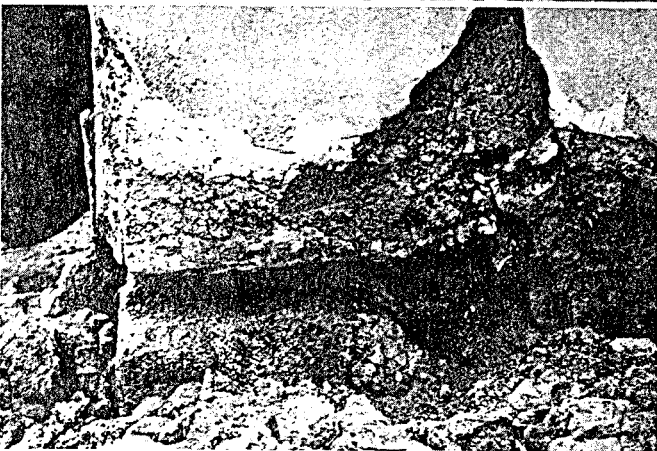
FIG. 25. Lack of continuity in column reinforcement, Sumerbank Textile Factory.

FIG. 26. Inadequate splice length in column reinforcement.

contributing to the lateral strength and stiffness of the structure. The frames of this type of infills also performed well with little structural damage. The non-bearing hollow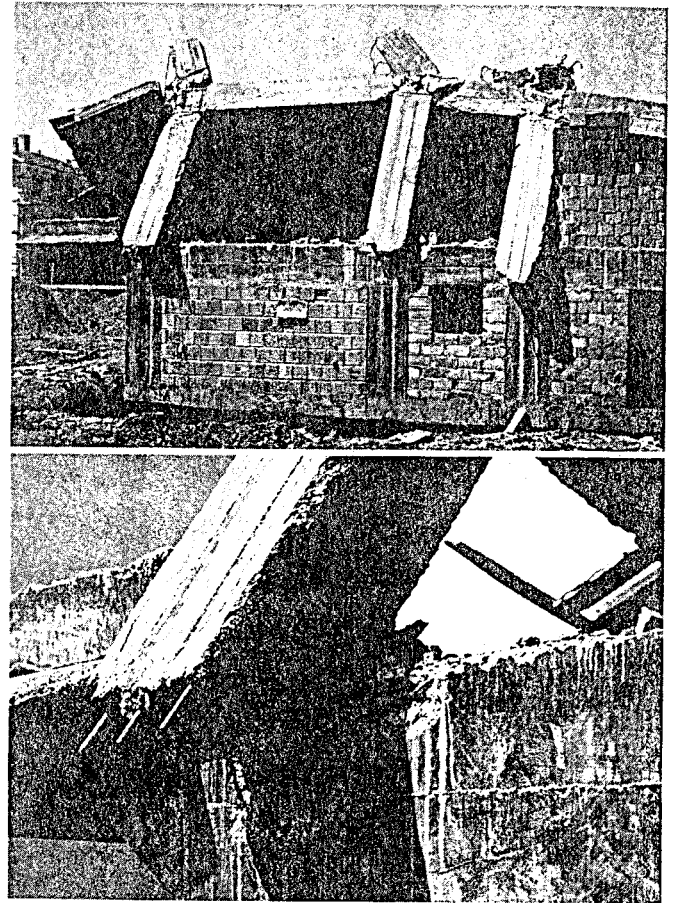 bricks did not have the strength to withstand diagonal compression generated during the earthquake, and high deformability demands of frame structures. Therefore they developed diagonal tension and compression failures, as shown in Fig. 27. Crushing of the cells of the hollow brick units leads to the separation of the infills from the frame, as shown in Fig. 28. In one of the buildings of the Sumerbank Textile Factory, this separation was followed by out-of-plane movement of the infill, as illustrated in Fig. 29.

Unreinforced concrete masonry infills were used in some of the commercial buildings as well as the State Hospital. Their performance appeared to be slightly better than those of the brick masonry. Figure 30 shows the concrete infills used in the State Hospital.

Concrete shear wall–frame interactive systems

Only a limited number of reinforced concrete shear wall–frame interactive systems were found in Erzincan. A new condominium complex in the northeast had a number of buildings with shear walls placed in two orthogonal directions. These buildings, shown in Fig. 31, were under construction at the time of the earthquake, and did not suffer any damage. Another building that contained reinforced concrete shear wall–frame interactive system was the north wing of the new Police Headquarters. Ironically, the south wing of the same building, separated by a construction joints, did not contain any shear walls. As a result, the north wing showed a far superior response with no appreciable damage, while the south wing experienced extensive failure of unreinforced brick infills. Figure 32 illustrates the two

FIG. 28. Crushing of hollow brick masonry and separation of infills from the frames.
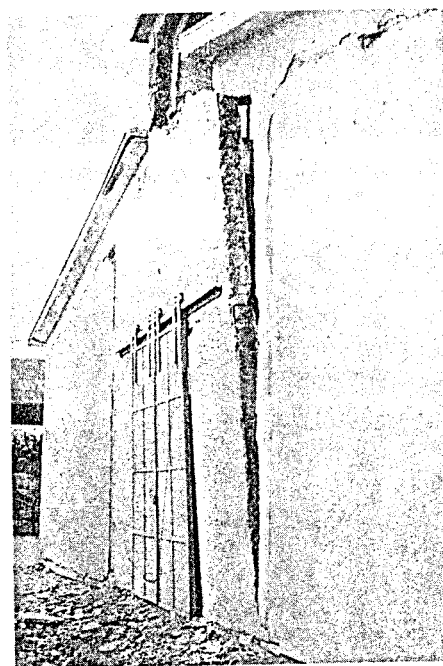
FIG. 29. Out-of-plane movement of a masonry wall in Sumerbank Textile Factory.
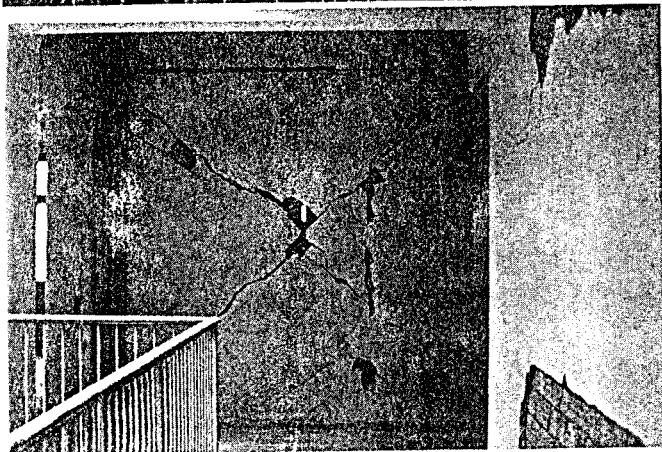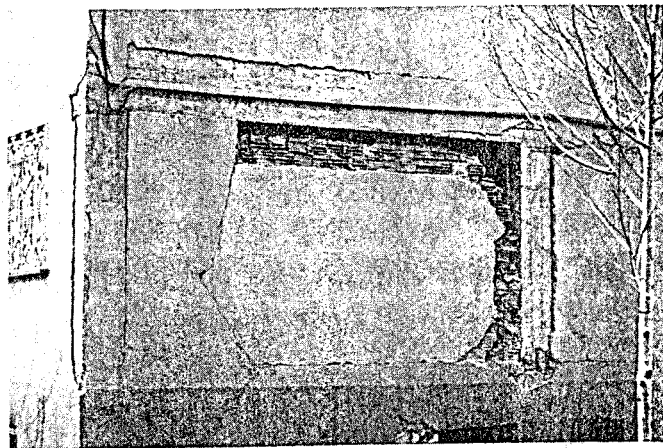
FIG. 27. Typical diagonal tension and compression failures in masonry infills.

wings after the earthquake. This comparison and the observed behaviour of the shear wall buildings that were under construction clearly indicate the superior response of shear wall structures during the earthquake.

Another multistorey building that contained reinforced concrete shear walls was the west wing of the Red Crescent Business Centre. In this building the lower part of the shear wall was supported in the same plane by an adjacent shear wall. The wall suffered a sliding shear failure at the plane of discontinuity, as shown in Fig. 33.

The State Hospital is another example of a reinforced concrete shear wall building. The central wing of the building had coupled shear walls in the N–S direction, and a thick stone masonry wall with rubble infill, running in the E–W

direction. The stone masonry, shown in Fig. 34, was damaged at the base, but controlled the drift in this direction, thereby saving the structure. In general, the shear wall structures behaved remarkably well during the earthquake.

Precast industrial buildings

Precast concrete columns, beams, and walls were used in two industrial complexes. One of them was the newly constructed animal slaughter house that consisted of two nearly identical buildings. The buildings were positioned side by side and both consisted of prefabricated columns providing support to long-span precast concrete girders. One of the buildings was also enclosed by precast concrete wall panels in two orthogonal directions. This building survived the earthquake without any damage, while the open building, i.e., without the cladding, collapsed, as shown in Fig. 35.

A pair of industrial precast buildings were also under construction at the time of the earthquake along the side of the main east–west highway. These buildings consisted
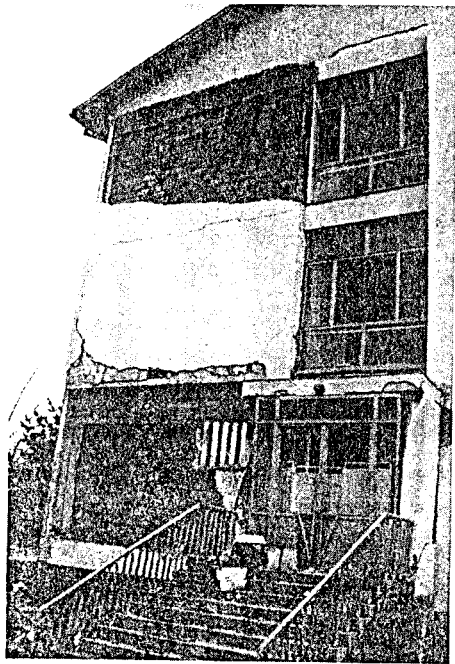
FIG. 30. Concrete block masonry infills used in the State Hospital.
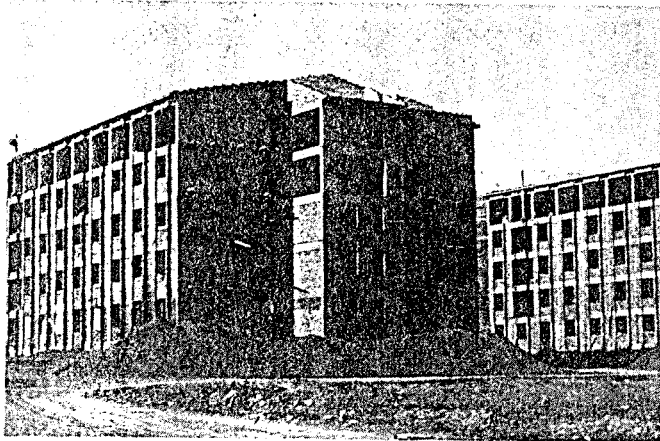
FIG. 31. Condominium complex in the northeast, utilizing frame–shear wall interactive systems.

FIG. 32. New Police Headquarters: (*a*) north wing with shear walls; (*b*) south wing without shear walls.

FIG. 33. Sliding shear in the shear wall of the Red Crescent Business Centre.

of precast columns and beams, with brick masonry infills. They survived the earthquake without noticeable damage.

Concrete bridges

Only one of the bridges in the area suffered damage during the earthquake. This was a three-span reinforced concrete railroad overpass located on the southwest highway, leading to Kemah, just outside the city limits (Fig. 36). Each span was supported by three main girders that were resting on transverse beams monolithically built with three supporting piers. The middle span was 12 m long and the middle piers were 6 m tall. The piers had an 800 mm diameter circular cross section. The settlement of the abutments at both ends left separations that caused the bridge to be closed to traffic (Fig. 37*a*). The settlement at the north abutment led to the sliding and tilting of the retaining wall, which accidentally beared against the north piers, applying a horizontal force in the longitudinal direction (Fig. 37*b*). This resulted in flexural cracking of the piers at the ends, wide enough to signify yielding.
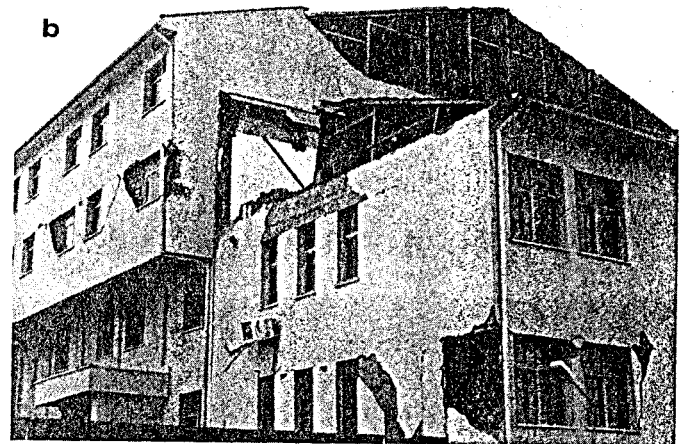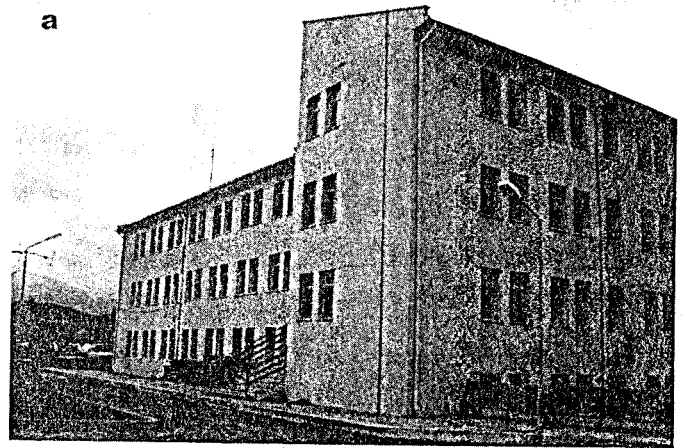
Unreinforced masonry structures

Unreinforced masonry (URM) structures have long been recognized as the most damage-prone during earthquakes. Unfortunately, as masonry is the most universally available
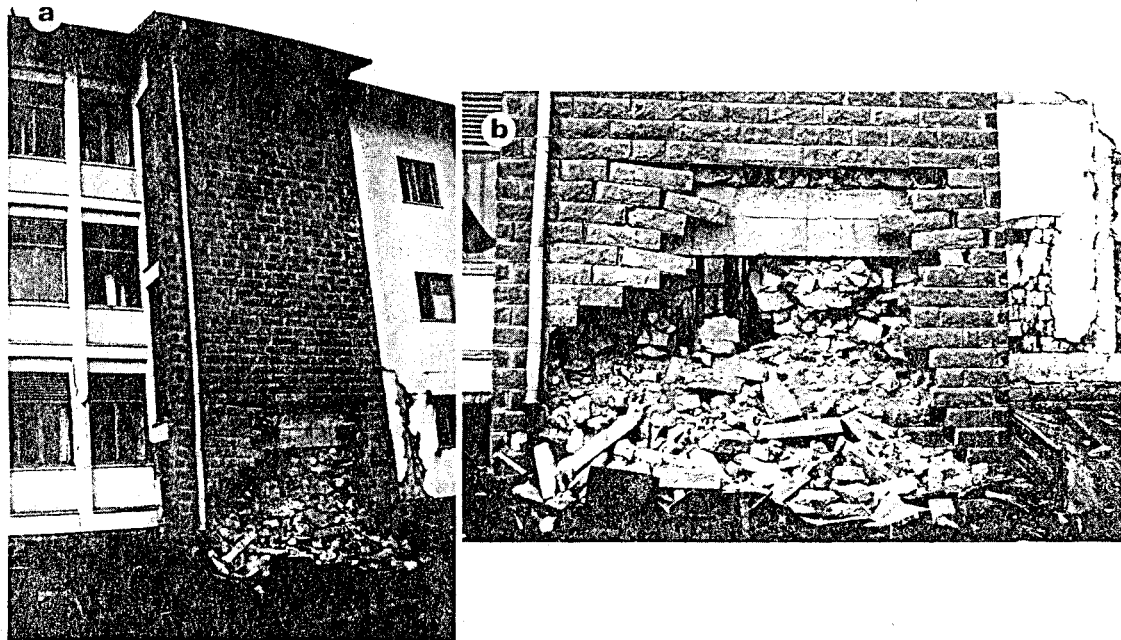
FIG. 34. (a) Stone masonry shear wall of the State Hospital; (b) close-up view of the wall.
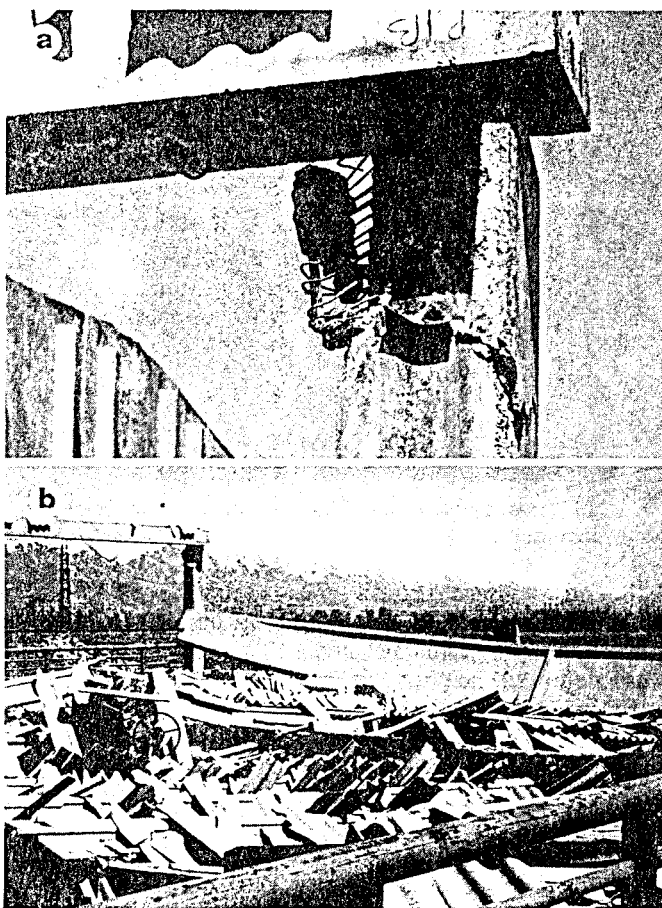

FIG. 35. Precast concrete animal slaughter house: (a) beam-column connection; (b) overall view.
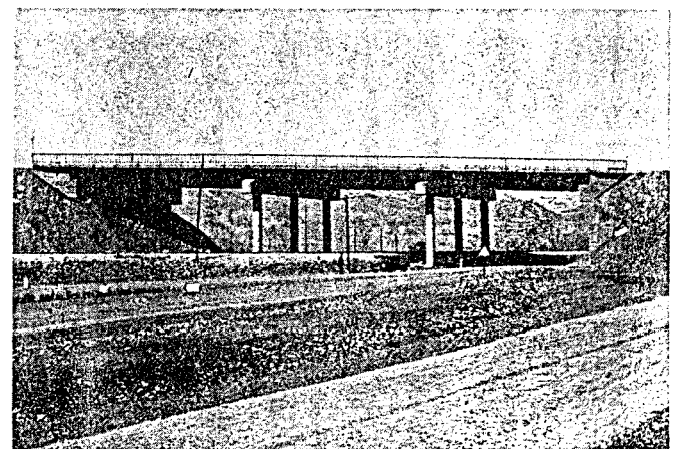

FIG. 36. Three-span reinforced concrete bridge on the southwest highway leading to Kemah.

of URM construction types and practices varied greatly in Erzincan.

Mud brick construction of low quality was used widely in the poorest neighbourhoods on the outskirts of the city; while some have collapsed (Fig. 38) or suffered major damage, others survived somewhat intact. The individual seismic performance of these non-engineered dwellings is an intricate function of wall thickness, internal subdivisions, roof mass, nature of the continuity with adjacent dwellings, etc. Even though these dwellings are of no engineering significance, it was sad to see their resourceless occupants rebuilding with exactly the same techniques and materials following the earthquake, often re-using surviving parts of severely damaged walls, in spite of the demonstrated poor seismic performance and high life-safety hazards of these constructions.

The URM construction type most commonly used in Turkey consists of reinforced concrete slab floors poured on top of URM bearing walls. The high in-plane rigidity of the floors allows a good distribution of seismically induced
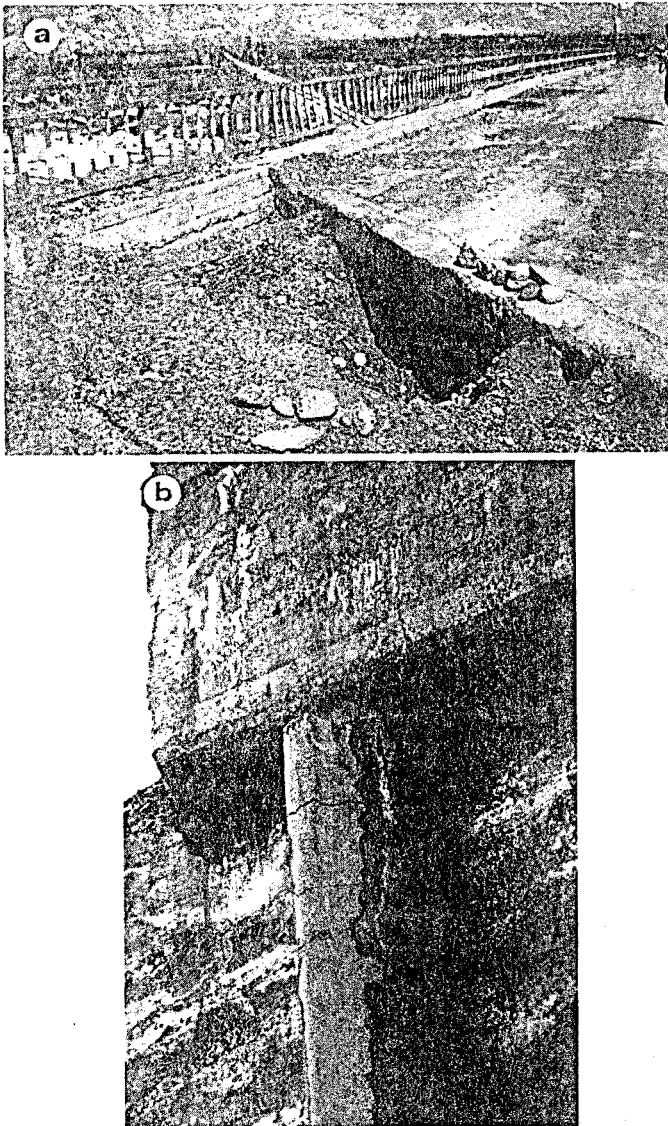
and economical construction material, it was also widely used throughout Erzincan. Hence, catastrophic failures of such structures were observed, but surprisingly, the survival of numerous URM buildings was also noted. The spectrum

FIG. 37. (*a*) Settlement of west abutment and (*b*) pier cracking of the bridge on the southwest highway.

FIG. 38. Example of collapsed mud-brick masonry construction.
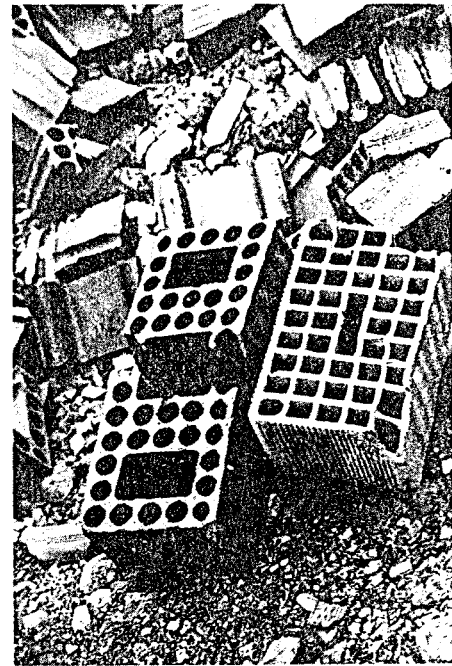
FIG. 39. Examples of types of non-bearing masonry brick commonly encountered.

forces to walls as a function of their respective rigidity, but reliance on the brittle URM to resist lateral forces ensures that once the strength threshold is exceeded, severe damage and (or) dramatic collapse is likely. In-plane, out-of-plane, and combined in-plane and out-of-plane failures have all been observed, as normally expected from URM buildings.

Brick quality has a major influence on the in-plane performance of such structures. This is well demonstrated in cases where non-bearing masonry has been substituted, as a cost-cutting measure or out-of-ignorance, for the bearing masonry normally required for this type of construction. In Turkey, a variety of brick-masonry units are available, and the Turkish standards TS 4377 (Turkish Standards Institute 1985c) and TS 705 (Turkish Standards Institute 1985b) differentiate between bearing and non-bearing units and their corresponding type of applications, the latter being allowed for partitioning purposes only.

Two types of non-bearing masonry were commonly encountered: multi-cell clay blocks with large rectangular holes and with smaller circular holes, manufactured in Erzincan and Tokat, Turkey, respectively (Fig. 39).
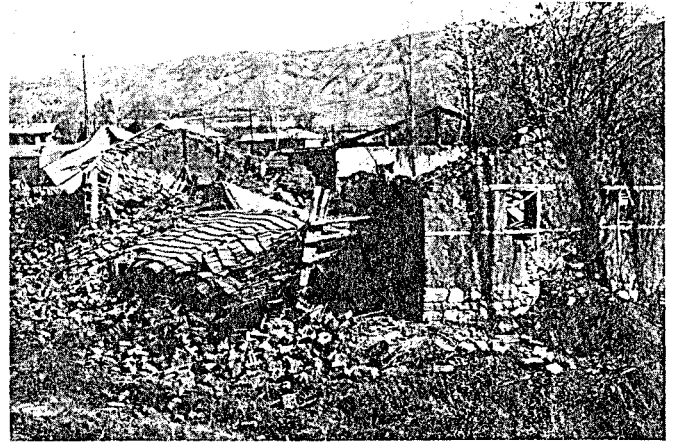
Although the usage of both types is improper in a bearing wall application, even such a slight variation in shape had repercussions on seismic behaviour. This was most forcefully illustrated in the neighbouring town of Uzumlu, 30 km east of Erzincan, where an entire residential complex of URM buildings was under construction at the time of the earthquake. Although the seismic intensity in this town was already less than in Erzincan, being attenuated by distance, a number of those new URM buildings collapsed. While others have alleged that a local soil effect was solely responsible for the damage sustained by the buildings built on the valley side of the Uzumlu complex, the authors believe that, in light of the severe difference in seismic performance observed over only a few hundred feet and a subjective evaluation of the local topography, such a soil effect can only be, at best, partly accountable for this damage. A closer observation revealed that, invariably, the walls built of the rectangular-cell URM blocks were severely damaged. This can be attributed to the poor resistance of these URM walls
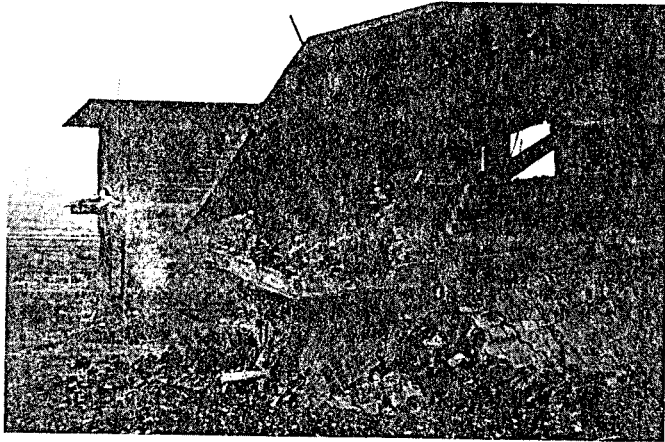
FIG. 40. Collapse of a URM building due to diagonal crushing of non-bearing masonry brick.
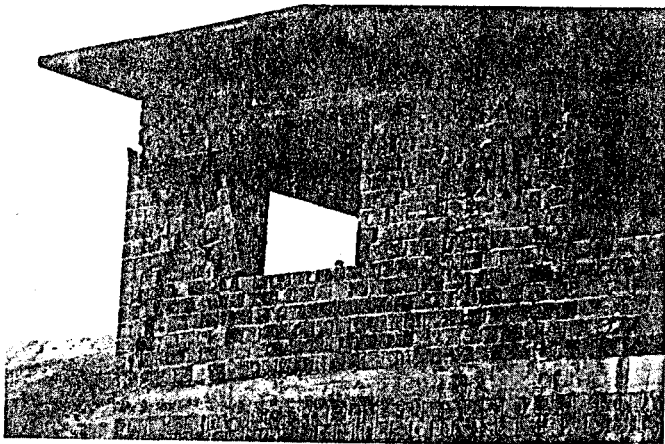

FIG. 41. Large diagonal cracking in URM walls built with circular-cell non-bearing masonry blocks.

FIG. 42. Example of older residential URM masonry buildings which have survived intact, next to a new building collapsed while under construction.
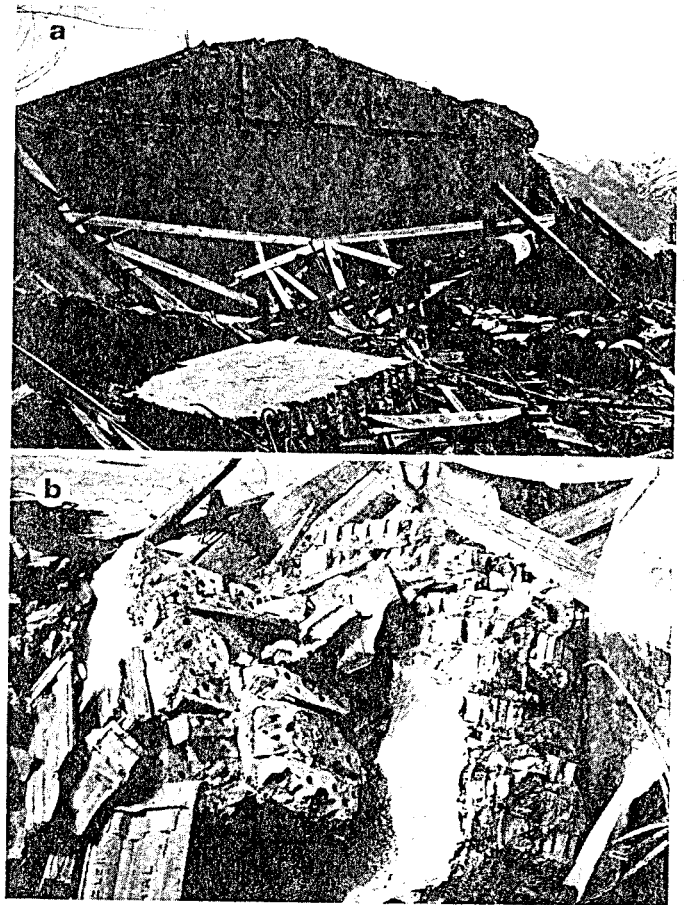

FIG. 43. (a) Completely collapsed warehouse due to out-of-plane failure of near-solid brick masonry at the Rural Services Affairs; (b) close-up view of the brick masonry.

to seismically generated compressive diagonal stresses. The very thin cell walls of these bricks proved unstable as soon as the compressive capacity was exceeded, producing an effective "delamination" accompanied by a rapid drop in strength, often leading to overall collapse (Fig. 40). By comparison, very large diagonal cracking was observed to occur in walls built with circular-cell masonry blocks; a lower void ratio and thicker average brick-cell walls both contributed to an enhanced block stability under compression, and allowed some structures to survive in spite of very large diagonal in-plane cracking (Fig. 41).

In-plane failures dominated in this case. The out-of-plane behaviour, being dependent on height-to-thickness (h/t) ratios, appears adequate for this type of URM construction. The connectivity bond forces at the reinforced concrete floors, the very low relative story heights, and the wide bricks used, all partly explain why very few out-of-plane failures were observed.

In spite of these numerous failure, many residential URM masonry structures have been observed to survive intact this earthquake (Fig. 42). This excellent behaviour is largely attributable to the high structural redundancy of those older structures; the typical living area of residential apartments in Erzincan averages 70 m^2 per unit, subdivided by masonry-block partitions into two or three bedrooms, living room, kitchen, and rest room.

A number of other URM structures were also present in Erzincan, and their performance was similar to that observed worldwide in similar circumstances. For example, an URM warehouse completely collapsed at the Rural Services Affairs (Fig. 43). Constructed of 325 mm thick masonry walls of near-solid bricks coated by more than 25 mm of stucco, for a height-to-thickness ratio of 11.5, the out-of-plane failure of this practically solid-wall rectangular enclosure occurred, largely as a consequence of the absence of anchorage at the
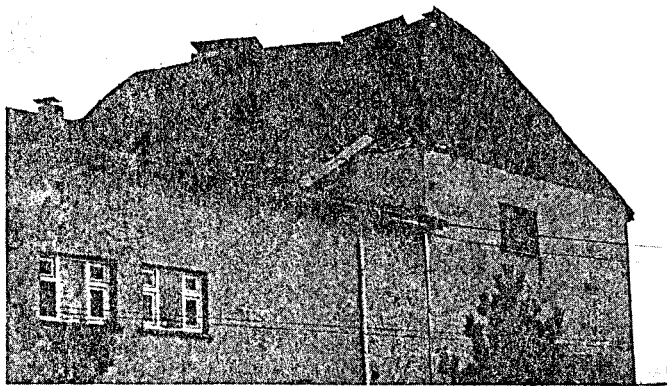
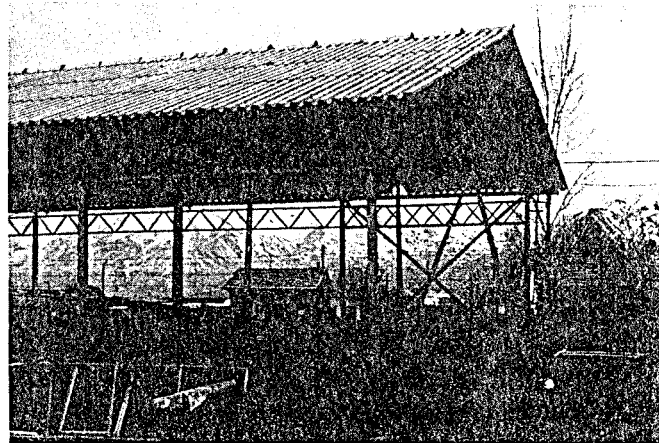FIG. 44. Typical out-of-plane failure of un-anchored gable roofs.

FIG. 45. Open-air steel warehouse of light corrugated steel roofing supported by steel trusses and columns which survived the earthquake (belonged to Rural Services Affairs).
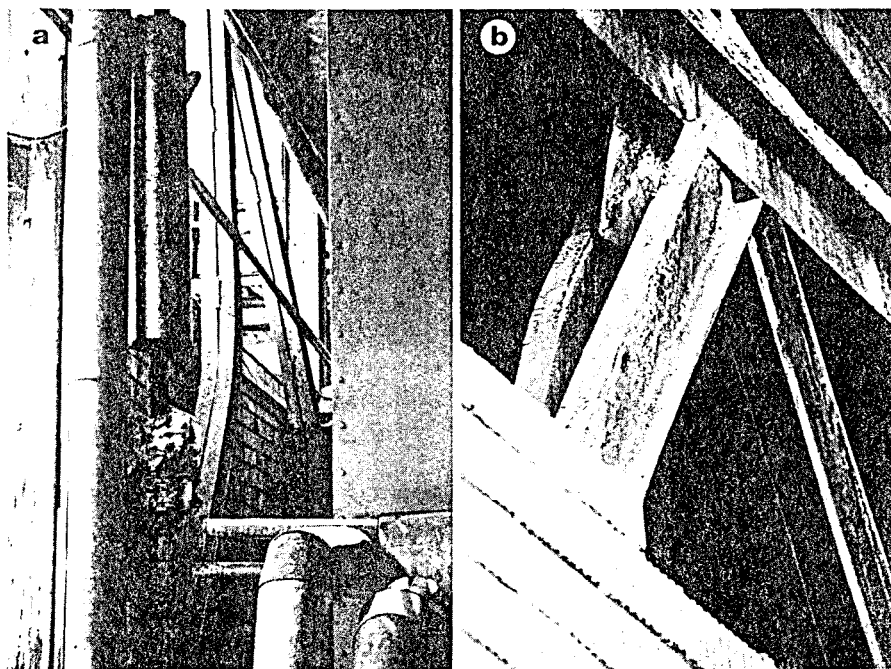

FIG. 46. (a) Buckling of both members of an inverted-chevron braced frame; (b) rupture of a brace-to-gusset weld which initiated a localized buckling of the remaining channel of the built-up section from the gusset-plate connection to the first batten plate (east building of the Sugar Factory).

roof level. Also consequently to similar lack of anchorage, the out-of-plane failure of gable roofs, made of various type of masonry units, was extensive throughout Erzincan (Fig. 44).

Double layer URM walls constructed sandwiching insulation (or saw-dust used as insulation) also performed poorly: the presence of insulation prevented bond between the layers, leading to increased height-to-thickness ratios and out-of-plane damage. For similar reasons, other cavity walls performed equally poorly, irrespective of the type of masonry used and sometimes in spite of the presence of a few metal anchors or transverse cross-blocks intended to act as anchors.

Steel structures

Steel, being by far the most expensive construction material in Erzincan, has been used rather sporadically in con-struction; only a few structures relying on steel for their lateral load resistance have been identified during this recon-naissance visit. Most of these were undamaged by the earth-quake, including a few water silos and tanks nearly empty when the earthquake struck, a telecommunication self-supporting tower, approximately 40 m tall, on which numer-ous large microwave dishes were attached, and a large steel roof (and columns) over the seating area of an open-air stadium. The few types of steel structures which were damaged by this earthquake will be reviewed following.

Industrial open-air warehouses

A number of open-air steel warehouses were found throughout the industrial district of Erzincan. Although all would be classified as non-ductile frames as per the ter-minology and detailing requirements of the 1990 edition of the National Building Code of Canada (NBCC 90) and the
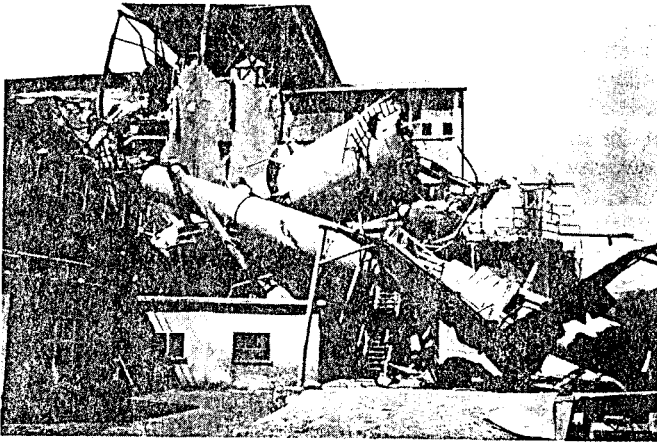
FIG. 47. Collapse of a tall steel chimney which buckled near its base and fell on the adjacent building (Sugar Factory main building).

CSA-S16.1-M89 (Canadian Standards Association 1989) standard, all but one survived.

One such survivor, of particular interest, belonged to Rural Services Affairs (Fig. 45). It had a light corrugated steel roofing supported by steel trusses and columns. Four equally spaced inverted-chevron braced frames and two X-braced frames were used in the N–S and E–W directions respectively. As all bays on the north side of the warehouse were used as loading docks, no X-braced frames were located on that side. Braces of the inverted-chevron and X-braced frames had slenderness ratios (KL/r) of 246 and 207 respectively. It is noteworthy that the maximum slenderness ratio for ordinary compression members allowed by the Turkish steel code (TS-498) is 250, slightly more liberal than the 200 limit specified by the CSA-S16.1-M89. Eccentricities of up to 125 mm were observed between member axes at the gusset plates connecting the diagonal members to the other structural components. Welded connections were used throughout, and the very poor quality of workmanship in the executions of those welds could be easily appraised from simple visual inspection.

Inverted-chevron braced frames have been severely penalized in recent editions of codes on account of their propensity to induce a plastic hinge and large axial forces in the beam where the braces meet when one brace buckles at a lateral load level less than required to yield the other brace (Khatib et al. 1988). For example, the Canadian Standard CSA-S16.1-M89 explicitly excludes inverted-chevron braced frames from the *ductile braced frame* category. Clearly, in the case at hand, the presence of a steel-truss roof to resist heavy snow loads provided a more than adequate capacity to resist any resulting unbalanced vertical loads imparted to this system by brace buckling.

This frame survived without any visible brace buckling or other form of damage. Truly, this excellent behaviour is largely attributable to the very low mass (lightweight roof) supported by the structural system. Yet the contrast between the seismic performance of this steel frame and that of other neighbouring structures essentially serving the same purposes is striking: the aforementioned Rural Services Affairs URM warehouse which collapsed was only 10 m away, and numerous totally or partially collapsed manufacturing warehouses, made of reinforced-concrete frames with URM infills, were roughly 30 m away.
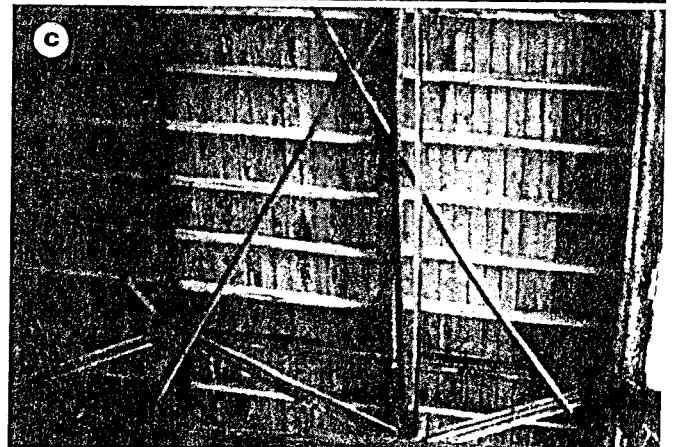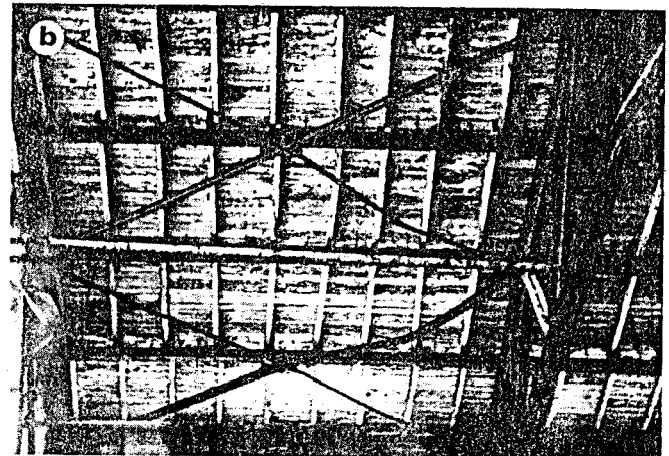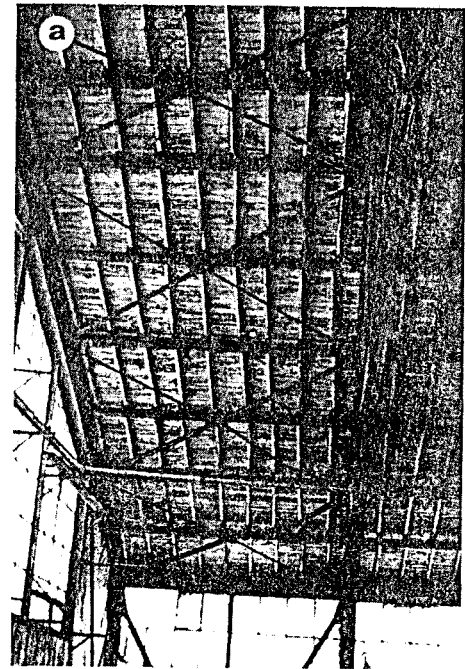

FIG. 48. Examples of horizontal roof bracings (*a* and *b*) buckled and (*c*) ruptured at their connections (Sugar Factory main building).

It should not be construed by the above that the authors condone the careless detailing of steel structure. At best, this provides some evidence that imperfect steel structures can potentially be forgiving, particularly if they are lightweight. The behaviour of un-braced open-air industrial steel
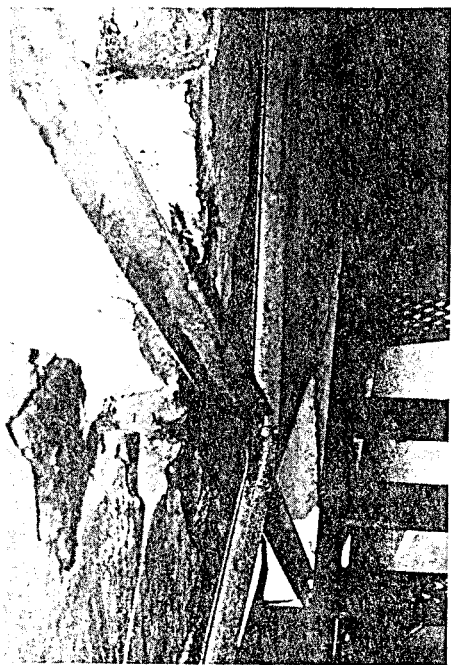
FIG. 49. Typical buckling of diagonal members of non-ductile braced frames of the lateral-load-resisting structural system (Sugar Factory main building).

warehouses was far less consistent. The Sumerbank textile complex owned two such structures, conceptually identical but having different layout, span, number of spans, and structural members sizes. One survived intact, the other totally collapsed. At the time of the earthquake, heavy compacted cotton ballots were stacked high in the warehouse that collapsed, and it was conjectured by the site technical staff that this content overturned, fell, and pushed laterally the columns into failure. The columns were sufficiently stiff to survive the earthquake without damage, but the base connections were the weak point and failed instead, either by combined tension and shear of the bolts or by rupture of the welds of the details. The survival of the other warehouse would then logically be attributed to it being nearly empty at the time of the earthquake. Simple dynamic analysis calculations made subsequently to on-site measurements confirm the credibility of this explanation.

Sugar Factory

Significant damage was suffered by the Sugar Factory's steel buildings, apparently the only steel structures of considerable size in Erzincan. These multistorey plant buildings were designed by a German firm and constructed in 1956. The main plant consists of two taller buildings linked by a lower-rise structure. A separate industrial building exists approximately 30 m to the east of the main plant.

Damage to the separate east building consisted principally of out-of-plane failure of URM infills, buckling of both members of an inverted-chevron braced frame at the first story, and rupture of a brace-to-gusset weld which initiated a localized buckling of the remaining channel of the built-up section from the gusset-plate connection to the first batten plate (Fig. 46). The building remained standing after the earthquake. It is noteworthy that, as predicted by theory (Khatib *et al.* 1988), the floor beam in the inverted-chevron braced-bay plastically deflected at its midspan; however, the non-composite concrete floor slab was capable of indepen-

dently spanning the bay's short width, remaining essentially unaffected by the steel beam's behaviour.

Damage to the main building was more extensive. A tall steel chimney buckled near its base and fell on the east end of the main plant building (Fig. 47). It was difficult to assess how much of the damage in the main plant was attributable solely to the earthquake (i.e., without chimney impact) other than at the location of the impact. However, as the main plant consisted of interconnected industrial buildings of various height, enough discontinuities existed to presume that some damage at the west end occurred inconsequentially of the chimney collapse. For example, many of the horizontal roof bracing buckled or ruptured at their connections in all the structures, even in the west building (Fig. 48). As for the lateral-load-resisting structural system, consisting essentially of non-ductile braced frames, numerous diagonal members buckled (Fig. 49), some quite severely. No evidence of tension yielding of the diagonals was observed. Similarly, no connection was seen to have ruptured, with the exception of one diagonal double-angle member for which the steel area between the tension-tip of an angle leg and the hole of the rivet holding the two angles together ruptured in tension when post-buckling hinging occurred at mid-span of the brace.

It is noteworthy that many of the remaining frames had riveted semirigid connections (or better) which conferred a considerable excess strength and redundancy to the structure. These buildings would meet, without difficulty, the NBCC 1990 requirement that when any member of a ductile structural system yields, the remaining members of the structure shall be capable of resisting 25% of the design seismic force, even though the structural system used in this case would be categorized as a non-ductile braced frame system. However, in view of the rather modest damage observed in the braced frames, and added stiffness compatibility issues, it appears that this reserve strength and ductility present in the structure has not been fully mobilized during the earthquake.

Conclusions

The extensive structural damage which occurred in Erzincan forcefully demonstrates the potential catastrophic seismic performance of non-ductile structural systems. However, in some instances, the behaviour was surprisingly satisfactory, if enough overstrength was present to ensure elastic response. This was observed in most of the reinforced concrete frame structures with URM infills that survived the earthquake. URM infills used in these structures added considerable strength and stiffness to the system, ensuring elastic behaviour of the frames. Beyond the elastic limit, seismic survival of the buildings depended heavily on ductility of the structural components. In reinforced concrete frames, the seismic damage could be attributed to the brittle failures of columns and beam-column joints. Lack of column confinement, inadequate column shear capacity, and poor reinforcement detailing can be listed as the primary causes of damage, especially when the column ductility demands were high due to the presence of soft storeys and strong beams.

Reinforced concrete shear wall buildings demonstrated superior performance, although in one case diagonal tension and sliding shear failures were observed. The behaviour of URM structures varied greatly, from undamaged to total collapse, principally as a function of material quality, structural layout, and wall to floor area. Steel structures in Erzin-

can were rare, and most survived the earthquake without damage.

In light of the observed damage in Erzincan, decision makers should be fully aware of the catastrophic nature of non-ductile structural systems, when weighting options in seismic risk mitigation strategies. This is particularly important in Canada because of the large inventory of existing structures built prior the enactment of ductile design guidelines. Furthermore, the refusal to recognize that earthquakes can cause considerable damage to modern buildings of reinforced concrete and steel, if improperly designed or constructed, can be a major seismic hazard in itself. Lack of appreciation from contractors and labour, even from engineers, of the importance of details in ensuring satisfactory seismic performance of structures is largely a reflection of the public's lack of earthquake awareness and preparedness.

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