The August 17, 1999, Kocaeli (Turkey) earthquake — damage to structures

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Abstract: The 1975 Turkish code provisions are first reviewed to provide the background for design and detailing of structures prior to the earthquake. The performance of reinforced concrete and masonry structures is described indicating many of the deficiencies in design, detailing, and construction execution. The behaviour of precast concrete structures, steel structures, and industrial facilities is also presented. The provisions of the 1997 Turkish building code are summarized and a description of new construction provides evidence of both excellent and poor construction practice. Some examples of retrofitting of damaged structures soon after the earthquake are also presented.

Key words: seismic design, earthquake, Kocaeli, structures, codes, concrete, precast concrete.

Résumé : Les clauses du Code turc de 1975 sont, en premier lieu, évaluées afin de fournir les bases pour la conception et la documentation de structures construites avant le tremblement de terre. La performance de structures en béton armé et en maçonnerie est décrite, indiquant plusieurs des déficiences dans la conception, la documentation et l'exécution de la construction. Le comportement de structures en béton préfabriquées, de structures en acier et d'installations industrielles est aussi présenté. Les clauses du Code du bâtiment turc de 1997 sont résumées, et un description de nouvelles constructions fournit les preuves de bonnes et mauvaises pratiques de construction. Quelques exemples de remise en état de structures endommagées, suivant le tremblement de terre, sont aussi présentés.

Mots clés : conception paraséismique, tremblement de terre, Kocaeli, structures, code, béton, béton préfabriqué.

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Introduction

The magnitude, M_b, 7.4 earthquake that occurred along the North Anatolian Fault at 3:01 a.m. on August 17, 1999, resulted in over 20 000 deaths, 50 000 injured, and over $30 billion in damage. Information on lifelines and earthquake preparedness can be found in Gillies et al. (2001). Figure 1 illustrates the concentration of damage in regions along the fault line, particularly in Izmit, Adapazari, Sapanca, Golcuk, and Yalova. Although Avcilar, a western suburb of Istanbul, is at a considerable distance from the fault, it also experienced damage due to the presence of soft soil conditions. A total of 140 000 structures collapsed, which represent 7.7% of the building stock in the epicentral region, while 28.6% of buildings suffered light to moderate damage.

This paper describes different types of structural damage that were observed during several separate site visits by the authors. The reasons for the poor performance of some structural systems are discussed and placed into context with the Turkish codes and construction practice.

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Turkish seismic code requirements

Many reinforced concrete buildings that suffered damage during the earthquake were designed in a period when the 1975 edition of the *Specifications for Structures to be Built in Disaster Areas*, which had been issued by the Ministry of Reconstruction and Resettlement of the Government of Turkey (Ministry of Public Works 1975), was in effect. In this code, emphasis is placed on reinforced concrete frame buildings with masonry infills, since this type of structural system dominated the building inventory in the earthquake-stricken areas.

1975 building code

The formula for the design base shear in the Turkish code resembles those in the North American building codes of the era. The design base shear, *F*, is given as

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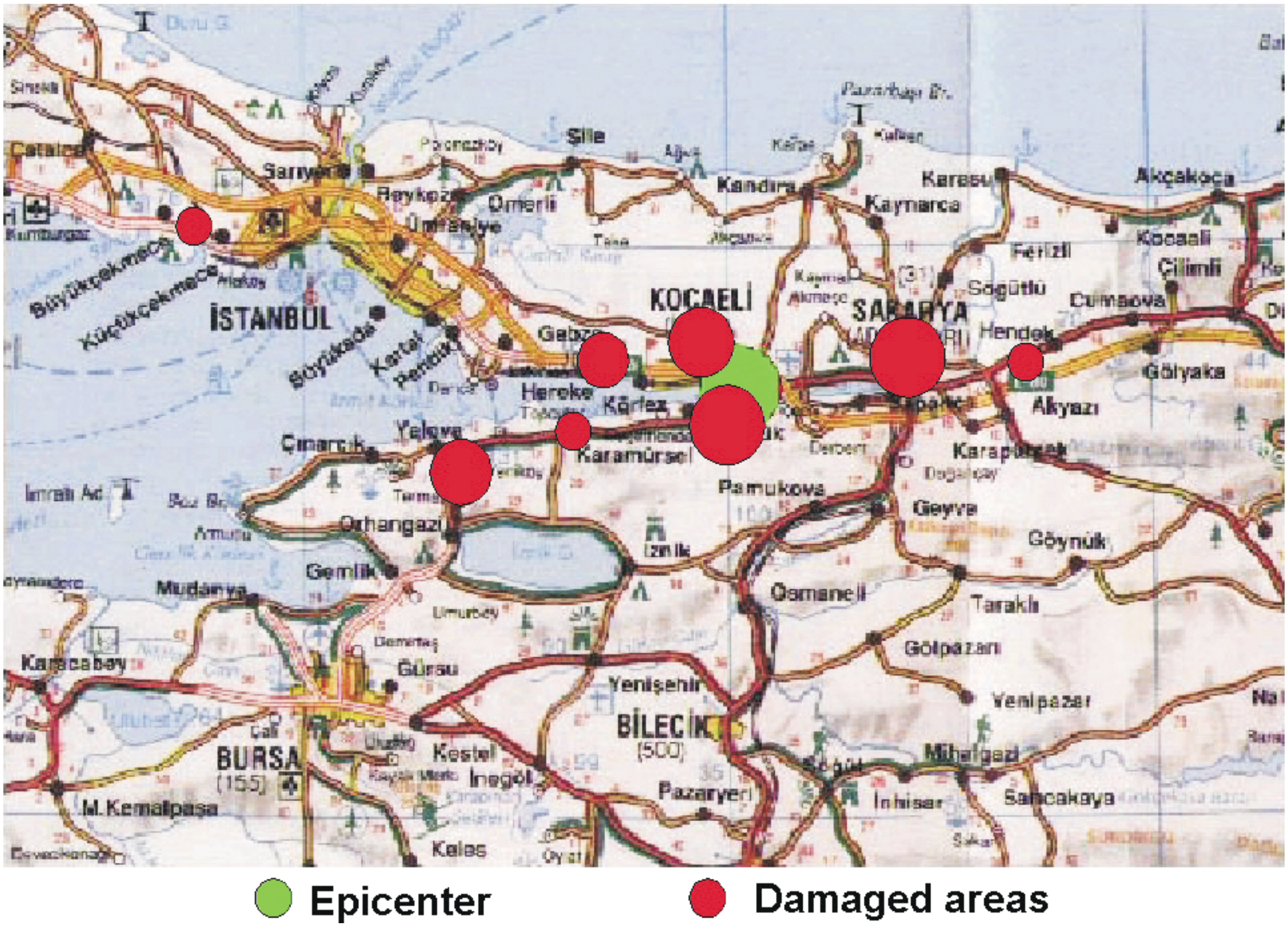
Fig. 1. Fault locations and regions of structural damage.



[1] $F = CW$

where W is the total structural weight and C is the seismic coefficient defined as

[2] $C = C_{\mathrm{o}}KSI$

where C_{o} is the seismic zone coefficient (C_{o} = 0.1 for seismic zone 1, which is the zone of the area affected by the earthquake); K is the coefficient related to structural type (K = 0.8 for ductile concrete frames with unreinforced masonry infills and K = 1.5 for non-ductile frames with unreinforced masonry infills); and S is the spectral coefficient, a function of the fundamental period of the structure and soil type, and is equal to the inverse of the period (T) for soft and alluvial soils with a high water table, similar to the soil conditions encountered in the disaster area. The maximum value of coefficient S is limited to 1.0; I is the importance factor (I = 1.0 for ordinary residential, office, and industrial buildings).

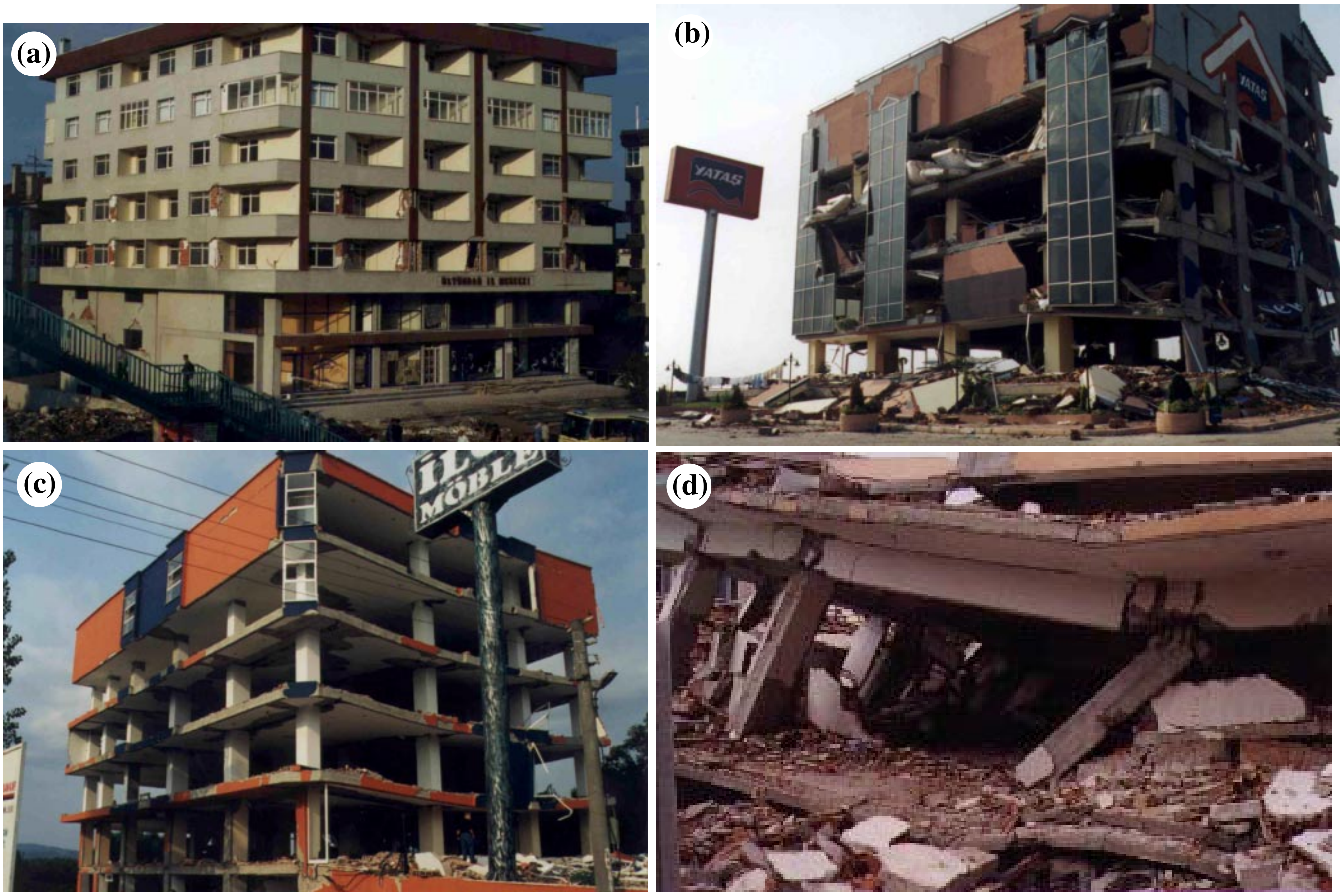
Therefore, for the majority of building structures with fundamental periods of less than 1.0 s, the design base shear varies between 8% and 12% of the building weight, depending on whether the concrete frame is designed to be ductile or non-ductile, respectively.

The specifications provide provisions to incorporate the effects of non-structural components on structural elements to prevent the creation of “short columns,” which have behaved poorly in past earthquakes. The non-structural elements are required to be separated from the frame if the storey drift exceeds 0.25%.

1975 design and detailing requirements

Concrete frame members are required to be confined at the ends, in the potential plastic hinge regions. Contrary to observations of construction practice, the code required columns to be confined by closely spaced transverse reinforcement. The minimum volumetric ratio of rectilinear and spiral transverse reinforcement is specified to be the greater of 1.0% or 0.12 times the ratio of the concrete compressive strength to the yield of the transverse reinforcement. The spiral pitch is limited to one-fifth of the column core or 80 mm, whichever is less. The hoops are required to be placed with a spacing not to be less than 50 mm and not to exceed 100 mm. The minimum tie diameter is specified to be 8 mm. The hoops are expected to have 135° bends with 10 bar diameter extensions into the confined core. The middle portions of the columns between the confined ends are permitted to have reduced amounts of transverse reinforcement, with maximum spacing limits of one-half the column dimension, 200 mm, or 20 times the smallest longitudinal bar diameter, whichever is smaller. Hoops are required at the ends of beams, within a distance of at least twice the beam

Fig. 2. Different levels of contribution of masonry infills to reinforced concrete frame responses: (*a*) limited masonry damage; (*b*) extensive damage to masonry, no apparent distress in frames; (*c*) total masonry damage with some distress in first-storey columns; (*d*) total damage to masonry and structural collapse.



depth, with a maximum spacing limit of one quarter of the beam depth. Beam–column joints are to be designed for joint shear, requiring transverse reinforcement in the joints. Under no circumstances is the joint shear reinforcement allowed to be less than that needed in the column middle region.

The beams are designed for shear under the effects of gravity loads and end moments due to the earthquake. The spacing of transverse shear reinforcement is limited to the beam width or one half of the beam depth. The longitudinal beam reinforcement is required to be continuous at the top such that it is not reduced below one-quarter of the larger amount required at either end near the support. Bottom bars are required at the ends, near the supports, such that the area of bottom reinforcement is not less than one-third of the area of the top reinforcement. Additional requirements are specified pertaining to reinforcement splicing and development.

Performance of reinforced concrete structures

The predominant structural system used in Turkey consists of reinforced concrete frames with masonry infills. Concrete, which is locally available, is generally preferred over other construction materials for economic reasons. The majority of concrete is used for cast-in-place construction, with an increasingly larger percentage being ready-mix concrete. Precast concrete construction is popular for industrial buildings. Concrete shear walls have gained greater popularity only in recent years.

Reinforced concrete frame buildings

The majority of collapses during the earthquake were attributed to the poor performance of reinforced concrete frames and masonry infill walls. Buildings with 4–6 storeys suffered the heaviest damage, inflicting most of the casualties. Structures close to the region of faulting were subjected to very high accelerations and velocities, resulting in very high seismic demands.

Inspection of collapsed and damaged buildings revealed that very little or no aseismic design had been implemented during the design and construction of reinforced concrete frame systems. It has been generally acknowledged that there has been very poor regulatory control over both structural design and construction. It was clear that the structural layouts used were susceptible to very high drift demands due to lack of proper lateral load resisting systems and extensive presence of soft storeys. The high seismic demands became increasingly critical due to the amplification of ground motion by soft soil. The only mechanism of defence for such structures with inadequate lateral load resisting systems is the ability of the structural members to undergo inelastic de-

Fig. 3. Examples of soft storey reinforced concrete buildings.

Fig. 4. Use of "Asmolen" one-way slab system and resulting strong beam and weak columns: (*a*) "Asmolen" concrete slab system and (*b*) strong-beam weak-column connection.

formations without experiencing brittle failures. Unfortunately, all of the frame buildings inspected lacked appropriate seismic design and detailing practices, which could have provided the required ductility and energy absorption. Proper design practices were missing in spite of the seismic design requirements of the 1975 Turkish code.

Causes of damage can be viewed under two categories: (*i*) factors contributing to increased seismic demands and (*ii*) factors contributing to reduced ductility and energy absorption.

Factors contributing to increased seismic demands

Lateral bracing for reinforced concrete frame structures was provided by unreinforced brick and (or) concrete masonry walls. The brick masonry used was often in the form of unreinforced hollow architectural blocks. During the earthquake, these walls were able to participate in lateral load resistance to varying degrees and were often damaged prematurely, developing diagonal tension and compression failures or out-of-plane failures. The degree of lateral load resistance depended on the amount of masonry used and the framing system provided. In contrast to modern moment resisting frames of North American practice, the use of light partitions, such as dry walls, was not common in the earthquake-stricken areas. Instead, masonry was used extensively for interior partitioning, as well as exterior enclosure of buildings, increasing wall-to-floor area ratios. Therefore, in spite of lower strength and expected brittleness of this type of masonry walls, the frames did benefit somewhat from such extensive use of masonry until the threshold of elastic behaviour was exceeded. Beyond the failure of brittle masonry, there was no lateral load resisting system with sufficient stiffness to control lateral drift, thereby resulting in high drift demands on the frame members. Figure 2 illustrates different degrees of masonry failure, resulting in partial damage, severe damage, and collapse of the frame structure.

Most buildings in Turkey are designed to have commercial space at the first-storey level, generally used for stores, as illustrated in Fig. 3. Furthermore, many buildings have a larger floor area above the ground-storey level. Both of these features result in soft storeys at the street level and added mass above the ground-storey level, placing excessive deformation demands on the highly critical first-storey columns.

Another reason for increased seismic demands on structures was the common use of a structural slab system called Asmolen. This is a one-way slab system, consisting of concrete joists with masonry units placed in between the joists, forming a deep structural slab system, as illustrated in Fig. 4*a*. Columns used are usually smaller in size, resulting in flexible and weak vertical elements relative to the adjoining horizontal members at beam–column joints. This system, totally in violation of the strong-column weak-beam design philosophy, places heavy deformation demands on the columns, especially at the first-storey level, increasing storey drifts and forcing hinging to occur in the columns. Figure 4*b* shows column damage resulting from a strong-beam weak-column connection.

Factors contributing to reduced strength and deformability

During the seismic response, the failure of brittle masonry walls placed a heavy demand on the first-storey columns of multistorey buildings. The columns sustained heavy damage mostly because of lack of sufficient transverse reinforcement. The transverse reinforcement consisted of 8 mm diameter smooth reinforcement, generally placed at 300 mm or wider spacing. In some buildings some of the ties were left out as illustrated in Fig. 5. The ties did not appear to be sufficient either in terms of amount or detailing. This resulted in widespread column shear failures as illustrated in Fig. 6. In the majority of cases, the transverse reinforcement was limited to perimeter ties with 90° hooks. Columns that were subjected to heavy axial compression and flexural compression resulted in the crushing of concrete due to lack of confinement. Figure 7 shows additional examples of column damage caused by lack of sufficient transverse reinforcement, this time resulting in compression crushing rather than diagonal tension failures. First-storey column failures accounted for the majority of building collapses and hence a significant portion of the overall casualties.

Fig. 5. Lack of transverse reinforcement in concrete columns.

The lack of transverse reinforcement was also observed in monolithic beam–column connections. Beam–column connections in the majority of buildings did not contain any transverse reinforcement, suggesting that joint shear design was never a consideration in these buildings. Figure 8 illustrates damage to beam–column connections lacking joint reinforcement.

Deformation capacities of some structural elements were impaired because of unintended interference of nonstructural elements with the structure. As masonry walls participated in lateral load resistance of the framing system, short-column effects were created around window and other openings. Columns not designed for the increased shear as-

Fig. 6. Lack of transverse shear reinforcement and resulting diagonal tension failures.

Fig. 7. Lack of column confinement reinforcement and resulting crushing.

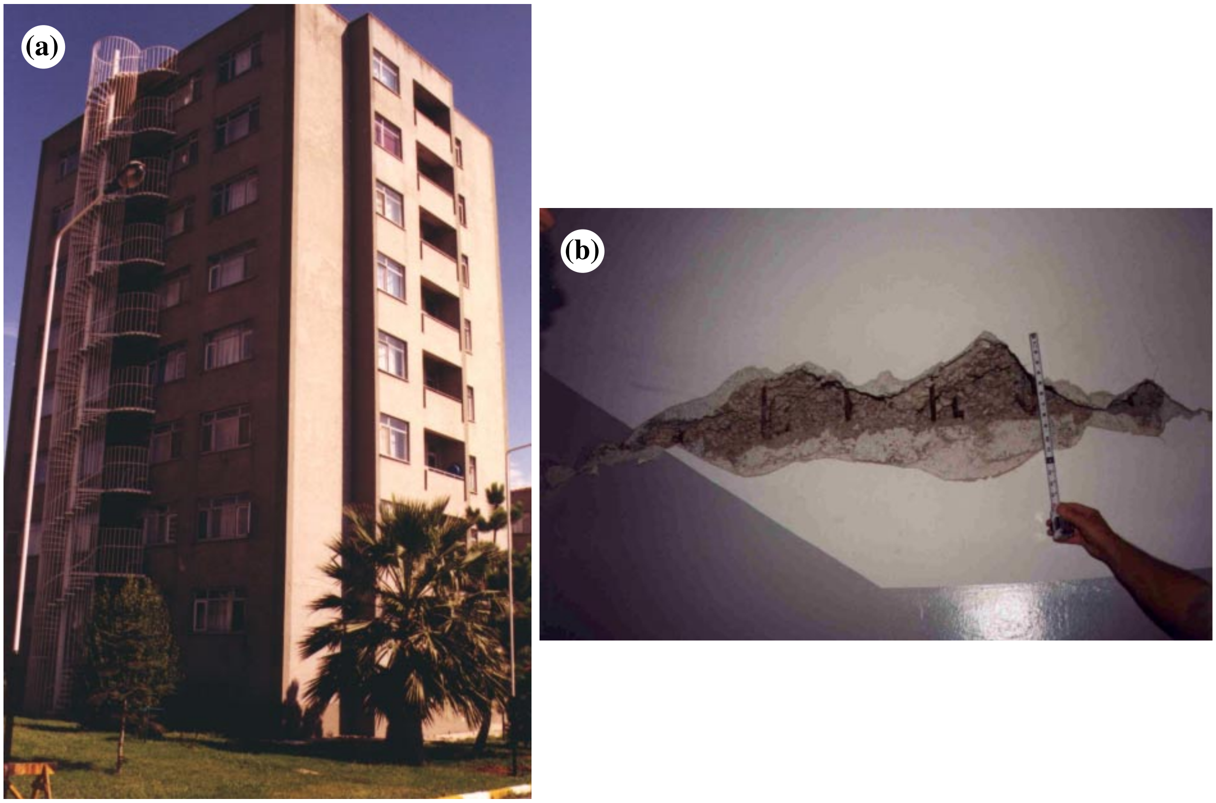
Fig. 8. Lack of joint reinforcement and resulting failures in beam–column connections.

Fig. 9. Interference of non-structural elements with lateral load resisting system: (*a*) short column effect and (*b*) interference of stairway landing slabs.

Fig. 10. Frame shear wall building with lightweight concrete masonry blocks: (*a*) apartment building with narrow shear walls and (*b*) diagonal tension cracks in shear wall.

Fig. 11. Excessive damage caused by shear in older shear walls.

Fig. 12. Frame wall building in Izmit without significant damage: (*a*) exterior view and (*b*) structural wall with concrete placement problem.



sociated with reduced unsupported height suffered brittle shear failures as depicted in Fig. 9*a*. In some buildings, the landing slabs of staircases were connected to columns, and either applied unexpected lateral forces or caused short column effects as shown in Fig. 9*b*.

Additional problems were observed, associated with irregularities in structural elements, reducing deformability of elements. Although most floor plans had symmetric layouts, there were cases where torsional effects created by asymmetry had adverse effects. Columns with plan offsets suffered damage. Two cases of cranked columns were found to survive the earthquake without significant damage, mainly because they were overdesigned, but with some signs of distress, requiring retrofitting.

Reinforced concrete shear wall structures

Use of reinforced concrete shear walls is limited in Turkey, especially in older buildings. A number of buildings were found with walls having cross sections with relatively small aspect ratios, which in some cases resembled elongated rectangular columns. These buildings performed reasonably well. Figure 10 illustrates an apartment complex under construction with lightweight concrete masonry units and this type of walls. These buildings survived the earthquake with minor damage to the structural framing system, but major damage to the masonry. Figure 10*b* shows a shear wall in this complex that developed diagonal shear cracks wide enough to suggest some yielding of the reinforcement, survived the earthquake, and saved the structure. There were other shear wall buildings with older and significantly lower quality concrete. Although the concrete in these walls was damaged extensively, the shear walls did save the structures from collapsing. This is illustrated in Fig. 11.

Properly designed and detailed shear wall structures performed exceptionally well even in regions close to the fault. Figure 12*a* shows the overall view of an apartment building within the Tupras Oil Refinery in Izmit, which performed well except for minor damage at a cold joint in one of these walls due to poor concrete placement (see Fig. 12*b*).

Performance of precast concrete structures

A significant number of precast concrete industrial structures are located in the epicentral area. In general, the precast structures did not fair well during this earthquake. Figure 13 provides an interesting contrast between two structures that totally collapsed and an adjacent structure that remained standing. These three structures are located in the industrial park region of Yeni Sanayi about 2 km from the fault. The two structures that collapsed were still under construction with only bare frames, without any roof diaphragms and without the side panels. These unfinished structures had inadequate connections between the columns and the beams and many of the columns failed at their bases as shown in Fig. 13*c*. The third structure was identical to the ones that collapsed, but the structural wall panels and roof diaphragms were completed. This structure underwent only slight hinging at some columns near their bases, with some evidence of connection distress at one beam–column joint (see Fig. 13*d*).

Fig. 13. Two precast concrete industrial buildings under different stages of construction: (*a*) incomplete building with frame elements only; (*b*) building with bracing walls in place; (*c*) shear distress at base of column; (*d*) distress at beam–column joint.

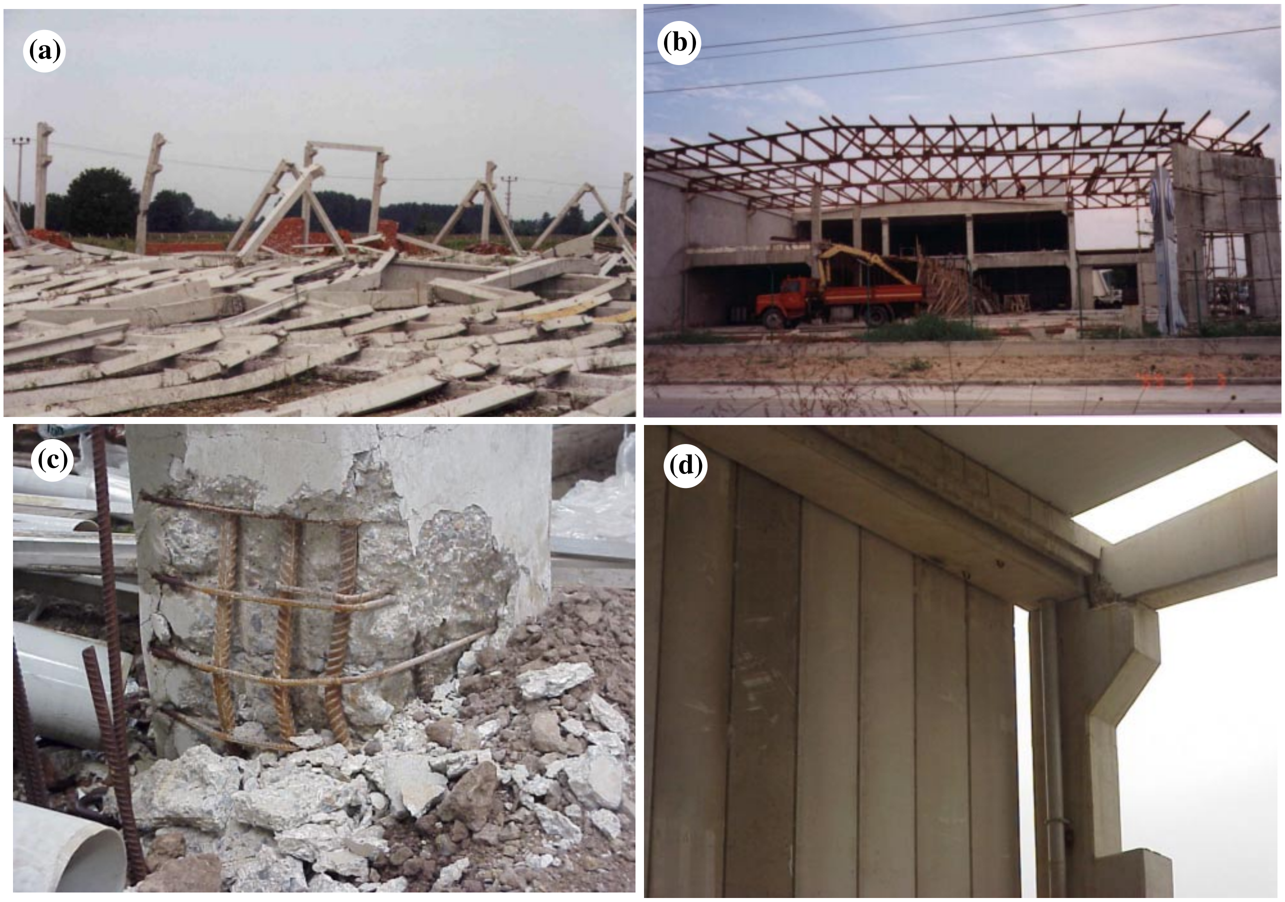


Fig. 14. Typical precast industrial building: (*a*) frame with short cantilevers supporting cranked beam and (*b*) column socket at base.

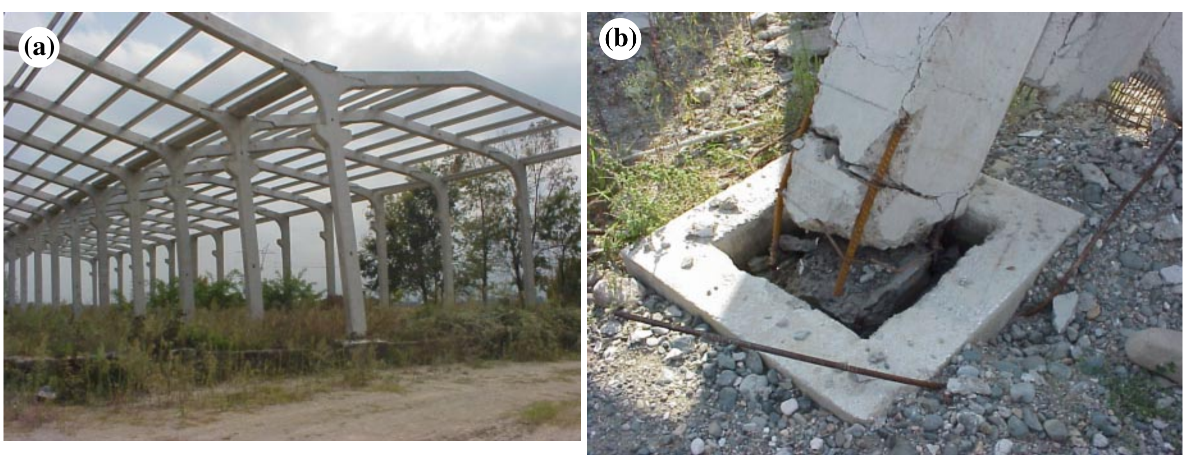


Figure 14*a* shows one form of construction for precast industrial buildings that was very common in Turkey. This construction consists of precast columns with short cantilevers, which support cranked beams and, in turn, support precast stringers. The precast columns are fixed at their bases, since they are grouted into sockets in the foundation perimeter wall footing as shown in Fig. 14*b*. The damage at the base of the central columns is partly due to the lack of diaphragm action in this unfinished structure. Figure 15*a* illustrates the complete collapse of a similar precast structure that was completed before the earthquake. The main reasons for the collapse are the following:

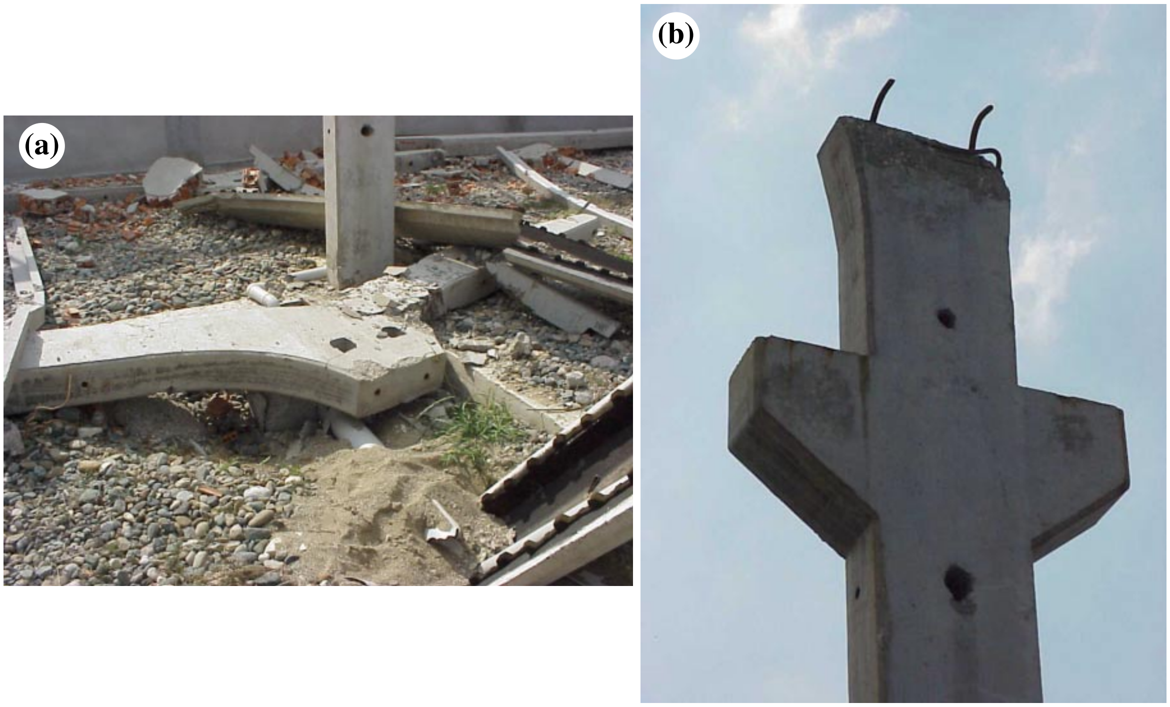
(*i*) The presence of partial height masonry infills led to failure in some columns at the level of the top of the damaged infills where the columns contained splices of the vertical bars.

(*ii*) The columns contained inadequate confinement (see Fig. 15*b*).

Fig. 15. Collapse of completed precast industrial building: (*a*) influence of partial height masonry infills; (*b*) inadequate confinement in columns; (*c*) thin corrugated fibre concrete diaphragm.

Fig. 16. Severe damage to frames of precast warehouse structure: (*a*) large deformation of columns and (*b*) flexural hinging at column base.

Fig. 17. Collapse of industrial building: (*a*) failure of double cantilever and (*b*) failure of connection.



(*iii*) The diaphragm consisted of very thin corrugated fibre-concrete panels with very flexible clip-on fasteners as shown in Fig. 15*c*.
(*iv*) The following connection failures took place: cantilever–column, cantilever–beam, and beam–stringers connections.

Figure 16*a* shows the severe damage to another industrial structure. Figure 16*b* shows the significant flexural hinging that occurred at the base of one of the interior 400 × 450 mm columns in an industrial precast building. The vertical reinforcement consisted of 4–20 mm diameter bars. The column ties consisted of 8 mm diameter deformed bars, spaced at 150 mm. The ties were anchored with 135° bends and had free end extensions of 60 mm. Other columns along the centre line were 260 × 510 mm, which suffered flexural hinging about their weak axis. Another deficiency of this type of construction is the connection between the double cantilever heads to the interior columns. Figure 17*a* shows a double cantilever head, in a building in the same industrial park, which has collapsed as a result of failure of the bolted connection to the column. Figure 17*b* shows the connection bolts for the double cantilever head after the failure. A different structural system in the same industrial park utilized pretensioned long-span tapered beams which fitted into slots formed at the top of the interior and exterior columns. Figure 18*a* shows the failure of a precast industrial building which experienced large lateral drift at the roof level. Figure 18*b* shows an interior column where loss of support for the beam has occurred due to the extremely large lateral drift at the top. Figure 18*c* shows the significant flexural hinging over a height of about 800 mm from the base of one of the interior columns. The 450 × 450 mm column had 2–20 mm diameter vertical bundled bars in each corner. The column ties consisted of 6 mm diameter plane bars at 100 mm spacing. The ties were anchored with 90° bends with 120 mm free end extensions. Although the column behaved in a very ductile manner, the lack of proper diaphragm action and inadequate connections between the beams and the columns contributed to the collapse.

Figures 19*a* and 19*b* illustrate another type of failure that occurred in precast columns within the industrial area between Izmit and Adapazari. The complete failure of this unfinished structure was due to the absence of diaphragm action, failure of the double cantilever to column connections (see Fig. 19*a*) and mid-height column failures (see Fig. 19*b*). Figure 19*c* shows a mid-height failure. The vertical reinforcement of the 260 × 510 mm rectangular columns consisted of 12–15 mm diameter continuous bars and 4–20 mm diameter bars that were curtailed at the top of the failure zone. The transverse reinforcement consisted of 6 mm diameter plain ties at a spacing of 175 mm. The transverse hoops were anchored with 135° bends into the core.

Although there were many examples of precast buildings with poor design, detailing, and construction, there were also large numbers of precast industrial buildings in the epicentral area that performed extremely well, as shown in Figs. 20*a* and 20*b.*

Steel structures

Apart from the special facilities such as oil refineries, there were very few steel industrial structures. Figure 21 shows a one-storey steel structure in an industrial park in the epicentral region. The I-shaped beams and columns were formed with galvanized light-gauge, cold-formed steel. The steel structure had tension-only bracing in the roof and tension-only bracing every fourth bay along the side walls. This structure with its partially completed roof diaphragm did not suffer any damage.

Fig. 18. Failure of industrial precast building: (*a*) evidence of large displacements at tops of columns; (*b*) loss of beam support in column slotted connection; (*c*) severe hinging at base of column.

Industrial facilities

Many industrial facilities suffered damage and interruption of operations as a result of the earthquake. The most significant damage occurred at the Tüpras refinery (Izmit), which supplies 96% of the country's requirement of petroleum products. Three separate fires started due to the earthquake, one of which took several days to bring under control with substantial fire damage. Figure 22 illustrates the extent of tank damage in one of the four tanks, inflicted by the fire. The main fire was caused by a pipe rupture during the earthquake, exposing oil products at extreme temperatures to oxygen. One of the fires started because of the collapse of a 115 m high reinforced concrete chimney that ruptured pipes carrying hot H_2S gas which only needed oxygen to ignite, as illustrated in Figs. 23 and 24. The failure occurred at about one-third of the height from the base due to the additional support provided by connecting pipes at this level. The refinery suffered ruptures of many pipes due to large displacements. Over 60 main pipeline systems were broken. Damage to the liquid storage tanks in the refinery was extensive. About 80% of the tanks suffered from various types of failure such as buckling of the supports of elevated tanks, buckling of the tank walls, and elephant foot buckling of the tank wall near the base, as depicted in Figs. 25–28. Many tanks suffered damage to the floating roof system with the consequence of liquid spillage with associated increased fire hazard. Several tanks were destroyed by the fire. An interesting mode of failure of several tanks is the bulging inwards and outwards of the roof causing internal suction resulting in inward deformation of the walls. One of the tanks that suffered roof damage had a diameter of 27.4 m and a height of 12.8 m and was filled to the 12 m level at the time of the earthquake. Other civil works in the refinery

Fig. 19. Complete collapse of unfinished precast structure: (*a*) failure of double cantilevers; (*b*) mid-height column failures; (*c*) close-up of column failure.

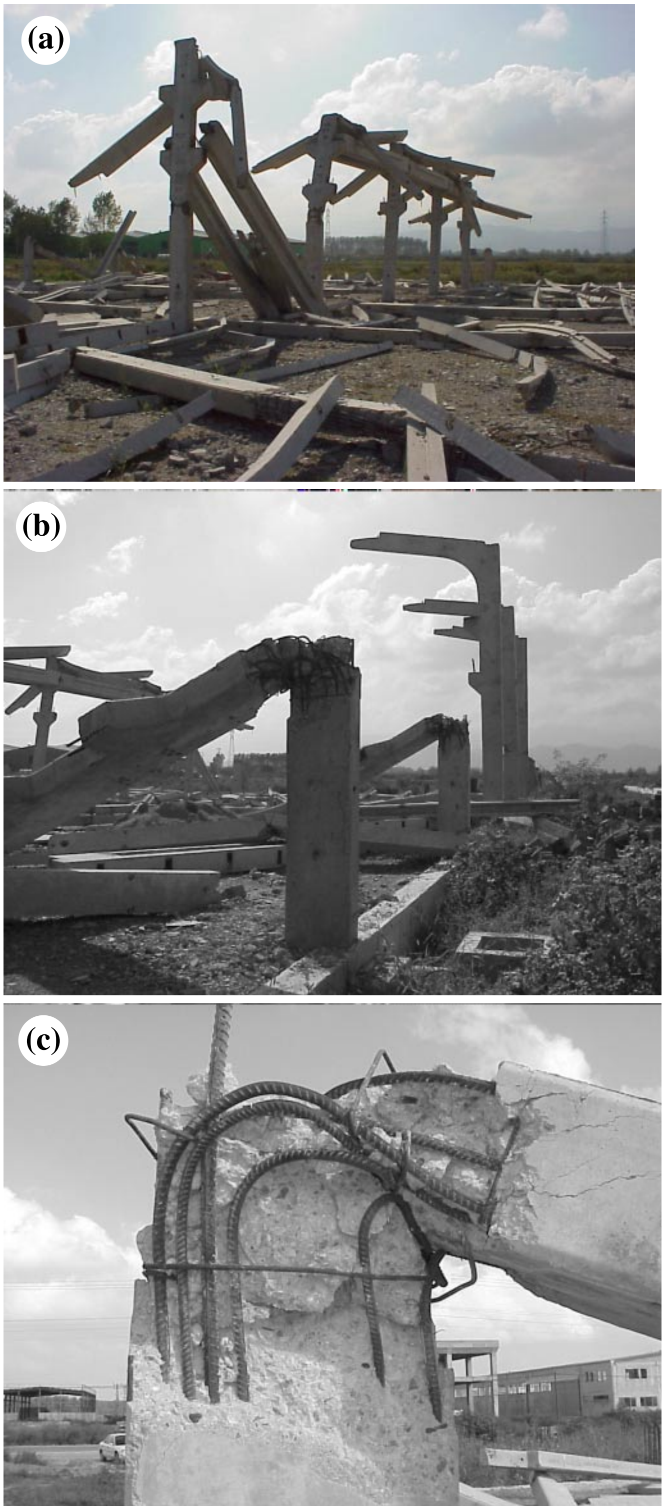


Fig. 20. Examples of precast structures that performed well.

Fig. 21. Cold-formed steel structure.

Fig. 22. Tank damage caused by fire.

facility suffered severe damage, including the port facilities and pipe supports, as shown in Figs. 29 and 30, and the collapse of one of the administration buildings and the heavy damage of others. Other less spectacular damage to industrial facilities includes collapse of structures, partial roof collapse, failure of tanks, and large displacement of equipment. Another industrial facility that suffered minor damage was a fertilizer factory just west of Izmit. The factory survived the earthquake without significant damage, other than minor hairline cracks in some of the concrete walls and minor soil failure at its harbor. The operation had to be stopped, however, because a large kiln, which is shown in Fig. 31, was forced off its support.

Fig. 23. Chimney failure at Tupras Oil Refinery.

New seismic code requirements and construction practice

1997 building code

The 1997 Turkish code (Ministry of Public Works 1997), with subsequent amendments in 1998, distinguishes between four seismic zones, with zone 1 being the highest with an effective ground acceleration coefficient, A_0, of 0.40. All of the damaged areas reported in this paper are located in zone 1. There are two methods for obtaining design force levels, the equivalent seismic load method and the mode superposition method. The design base shear, V_t, is given as

$$[3] \qquad V_t = \frac{A_0 I S(T_1) W}{R_a(T_1)} \geq 0.1 A_0 I W$$

where A_0 is the effective ground acceleration coefficient; I is the importance factor (e.g., 1.5 for post-disaster buildings; 1.4 for schools; 1.2 for sports facilities, cinemas, and theatres; 1.0 for other buildings); $R(T_1)$ is the seismic load reduction factor; $S(T_1)$ is the spectrum coefficient; and W is the total building weight including contributing live loads (e.g., 0.3 times the live load for residential and office buildings).

The spectrum coefficient, $S(T_1)$, depends on the natural period of the building, T_1, as well as the local site class as given.

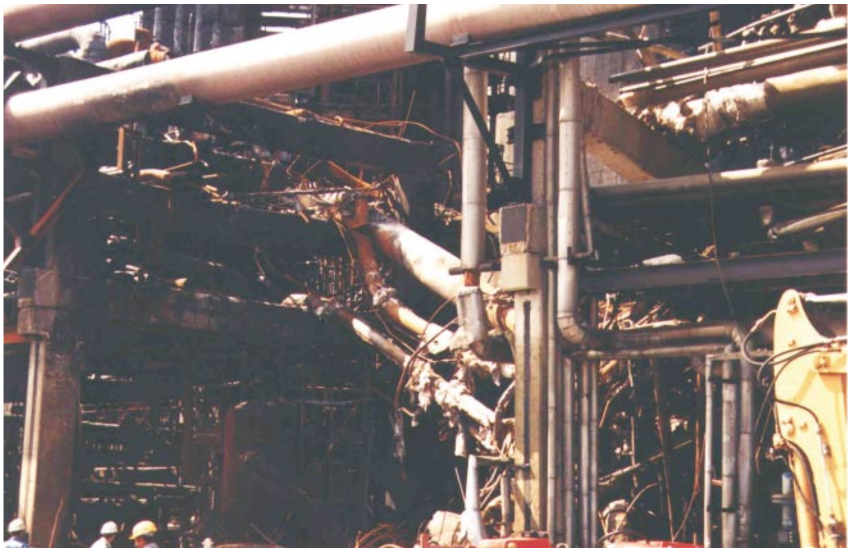
The first natural period of the building is given by

$$[4] \qquad T_1 = C_t H_N^{0.75}$$

where C_t is the coefficient used in calculating the period (0.07 for reinforced concrete frames or eccentrically braced steel frames; 0.08 for steel frames; 0.05 for all other frame buildings).

For structures containing reinforced concrete structural walls, the coefficient, C_t, is calculated from

Fig. 24. Pipe ruptures caused by chimney failure.



[5] $$C_t = \frac{0.075}{(\Sigma A_{wj}[0.2 + (l_{wj}/H_N)^2])^{1/2}}$$

where A_{wj} is the gross section area of the *j*th structural wall in the first storey; H_N is the total height of the building, m; l_{wj} is the effective length of the *j*th structural wall in the first storey, in the direction of the earthquake.

The seismic load reduction factor, $R(T_1)$, is given as

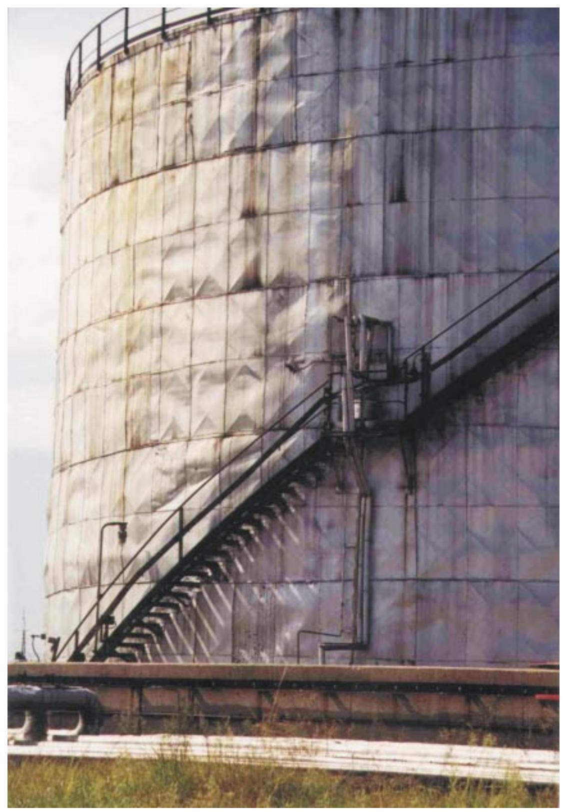
[6] $$R_a(T) = 1.5 + \frac{(R - 1.5)T}{T_A} \qquad (0 \le T \le T_A)$$
$$R_a(T) = R \qquad (T > T_A)$$

where R is the structural behaviour factor.

Table 1 gives the values of the structural behaviour factors, R, for different types of structural systems. The Turkish code distinguishes between nominal and highly ductile levels depending on the reinforcing details.

The 1997 code contains design and detailing requirements for concrete structures that are comparable to those in the North American codes.

Fig. 25. Wrinkling of tank wall.



Shear wall structures under construction in 1999

One very large complex of apartment buildings in Izmit having 10–20 storeys and located about 5 km from the fault sustained no visible damage. Figure 32*a* shows the overall view of the apartment complex and Fig. 32*b* shows a 15-storey structure under construction. Figure 32*c* shows the plan view of the structural walls and Fig. 32*d* shows the foundation mat. The reinforcement detailing in a typical wall at the second-storey level is shown in Fig. 32*e*. The aspects that contributed to the excellent performance include the following:

(*i*) The 700 mm thick, heavily reinforced foundation mat was founded on firm ground (see Fig. 32*d*).

(*ii*) The multiplicity of the symmetrically located shear walls (see Fig. 32*c*) in both principal directions provided excellent drift control, limited torsional effects, and provided redundancy in resisting lateral loads.

(*iii*) The 200 mm thick walls were reinforced with concentrated reinforcement (14–20 mm diameter bars at the base and 11–12 mm diameter bars at the second-storey level) at the ends of the walls confined with 8 mm diameter ties spaced at 200 mm. Two layers of welded wire fabric (6 mm diameter wires with a vertical and a horizontal spacing of 150 mm) provided the uniformly distributed vertical and horizontal reinforcement in the web regions of the walls. One small variation from North American practice was the use of welded wire fabric, the ends of which simply overlapped the confined cage of vertical bars, rather than having the horizontal bars project inside of the confined region.

Fig. 26. Local buckling of steel tank wall.

Fig. 27. Elephant foot buckling of a tank wall.

Fig. 28. Close-up view of the elephant foot type buckling.

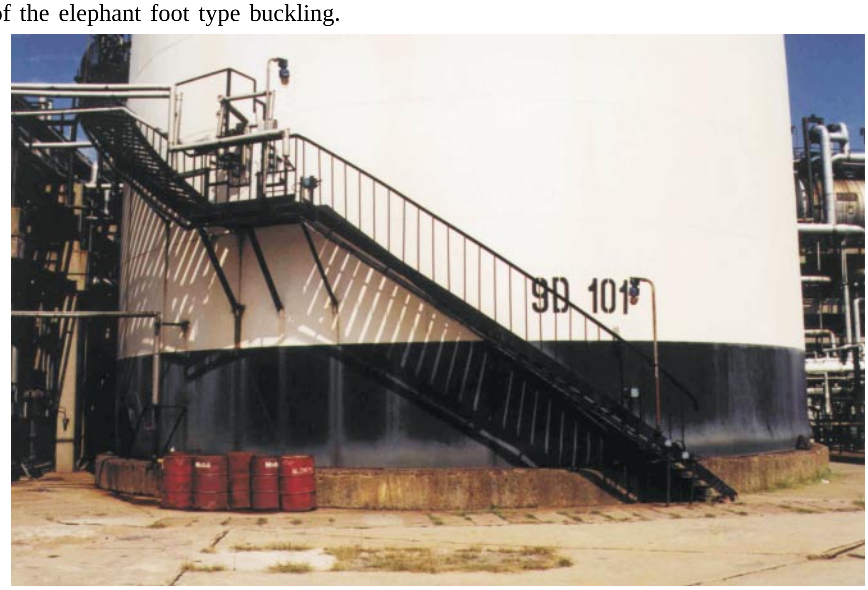


Fig. 29. Dislocated pipes along the seashore.

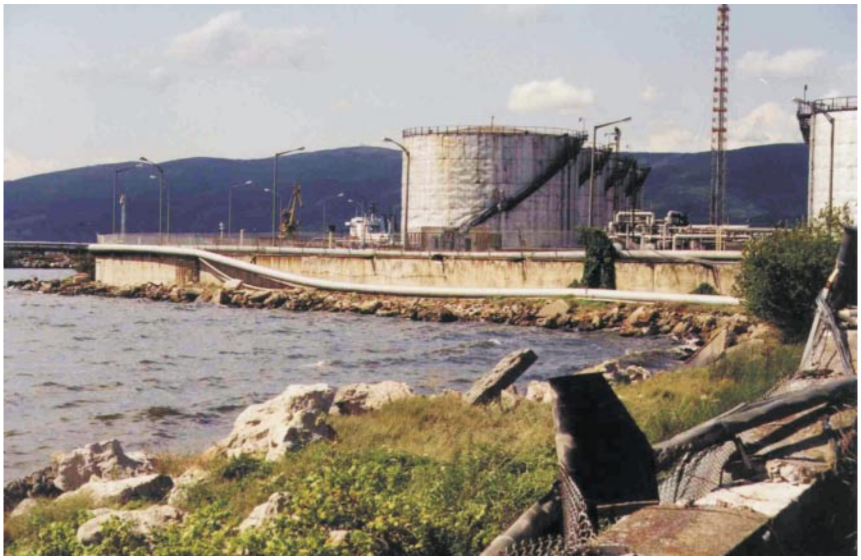


Fig. 30. Failure of pipe supports.

(*iv*) The buildings were not only well designed, but it was evident that construction and material quality control practices were strictly enforced.

Frame wall structures under construction in 1999

Figure 33*a* illustrates the details of a frame wall structure under construction, which had the following deficiencies:

(*i*) The columns had insufficient confinement, ties with 90° bend anchorages, and had lap splices located at the floor slab levels (see Fig. 33*b*).

(*ii*) The shear walls had insufficient confinement at the ends of the walls, 90° bend anchorages for the 8 mm diameter ties (see Fig. 33*c*).

(*iii*) The poor construction control and very small concrete cover provided over the reinforcement is evident in Fig. 33*d*.

The poor detailing in this structure represents numerous violations of the Turkish seismic code requirements.

Frame structures under construction in 1999

A six-storey reinforced concrete cast-in-place frame structure under construction in Yuvacik is shown in Fig. 34*a*. This 3 bay by 4 bay structure has the very common "asmolen" floor slab system. This structure, which is south of Izmit and about 5 km from the fault, suffered structural damage to the columns and joints. The traditional masonry infill had not been placed before the earthquake. The 250 by 600 mm corner column contained 10–15 mm diameter smooth vertical bars. These vertical bars were hooked at the ground floor slab level and the dowel bars coming up from the basement overlapped the vertical column bars. Hinging took place just below the hooks. The column ties consisted of 8 mm diameter smooth bars with 90° bend anchorages and 60 mm free end extensions. The spacing of the ties was 200 mm. Shear distress occurred in the joints of a corner and an edge column. There was no joint shear reinforcement in any of the joints. In addition, insufficient concrete cover had been provided. These details in this structure provide strong evidence that there was insufficient inspection and enforcement of building regulations, despite the well-established seismic code requirements of 1975 and the more stringent requirements of 1997.

Precast concrete structures under construction in 1999

Although there were many examples of precast concrete structures with poor lateral load resisting systems and details under construction in 1999, there were many examples of excellent systems under construction. Figure 35 shows the beam–column connections in a three-storey precast building under construction near the epicentral region. The hollow

Fig. 31. Kiln of a fertilizer factory in Izmit, dislocated during the earthquake.

core slabs are connected to the edge beams by a pour strip, which has perimeter diaphragm reinforcement as well as adequate connection to the beams through protruding stirrups in the pour strip. This design clearly addresses the need for diaphragm reinforcement and adequate connection between the diaphragm and the exterior frame. This form of construction, which is similar to North American practice, has stiffer, stronger diaphragms due to the load bearing requirements on the floors. In contrast, the single-storey precast industrial structures that suffered damage had very flexible, weakly connected roof diaphragms, which were designed for very light gravity loads.

Table 1. Structural behaviour factors, *R*.

Structural system	Nominal ductility	High ductility
1. Cast-in-place reinforced concrete		
Frames	4	8
Coupled walls	4	7
Wall structures	4	6
Frames and walls	4	7
2. Prefabricated reinforced concrete		
Frames with moment connections	3	6
Single-storey hinged frames fixed at base	—	5
Precast structural walls	—	4
Frames with moment connections and cast-in- place walls	3	5
3. Structural steel		
Frames	5	8
Single-storey frames fixed at base	4	6
Braced frames or cast-in-place concrete walls		
Concentrically braced frames	3	—
Eccentrically braced frames	—	7
Reinforced concrete walls	4	6
Frames with braced frames or reinforced concrete walls		
Concentrically braced frames	4	—
Eccentrically braced frames	—	8
Reinforced concrete walls	4	7

Examples of retrofitting

One month after the earthquake there were several examples of retrofitting of damaged structures. Figure 36 shows the retrofit of a corner column of an eight-storey reinforced concrete frame structure in Avcilar, west of the airport in Istanbul. Due to lack of symmetry in the wall system and the torsional eccentricities, the corner column was subjected to large displacements. This column was retrofitted at the ground-storey level with steel angles or channel sections at the corners of the column and with welded steel batten plates. This retrofit enhances the strength and stiffness of the column over its clear height but may lack proper continuity with the column above and the basement level.

Figure 37 shows severe damage to the ground-storey column of a five-storey reinforced concrete frame structure on Darica Road in Gebze, between Istanbul and Izmit. Shear failure occurred at the base of the column just above the lap splice in the vertical reinforcing bars. The 400 by 520 mm column contained 16–25 mm diameter bars with 8 mm diameter ties at 200 mm spacing. The ties had 90° bend anchorages. A similar five-storey structure immediately across the street had similar damage to the ground-storey columns. Figure 38 shows the retrofitted ground-storey columns. The columns were retrofitted by encasing the existing columns with at least 200 mm of reinforced concrete on all sides. The resulting dimensions were 650 by 850 mm for the edge columns and 650 by 1080 mm for the corner columns. The added vertical bars were 14 mm in diameter, while 8 mm diameter ties were added around the perimeter. The added ties were spaced at 100 mm near the top and bottom of the column, and at 150 mm in the central region. Although this retrofit strengthened and stiffened the ground-storey columns, there was no attempt to provide continuity with the structure above and below the ground storey.

Fig. 32. Shear wall structures under construction: (*a*) overall view of apartment buildings; (*b*) 15-storey structure; (*c*) plan view of structural walls; (*d*) foundation mat; (*e*) typical wall reinforcement details.

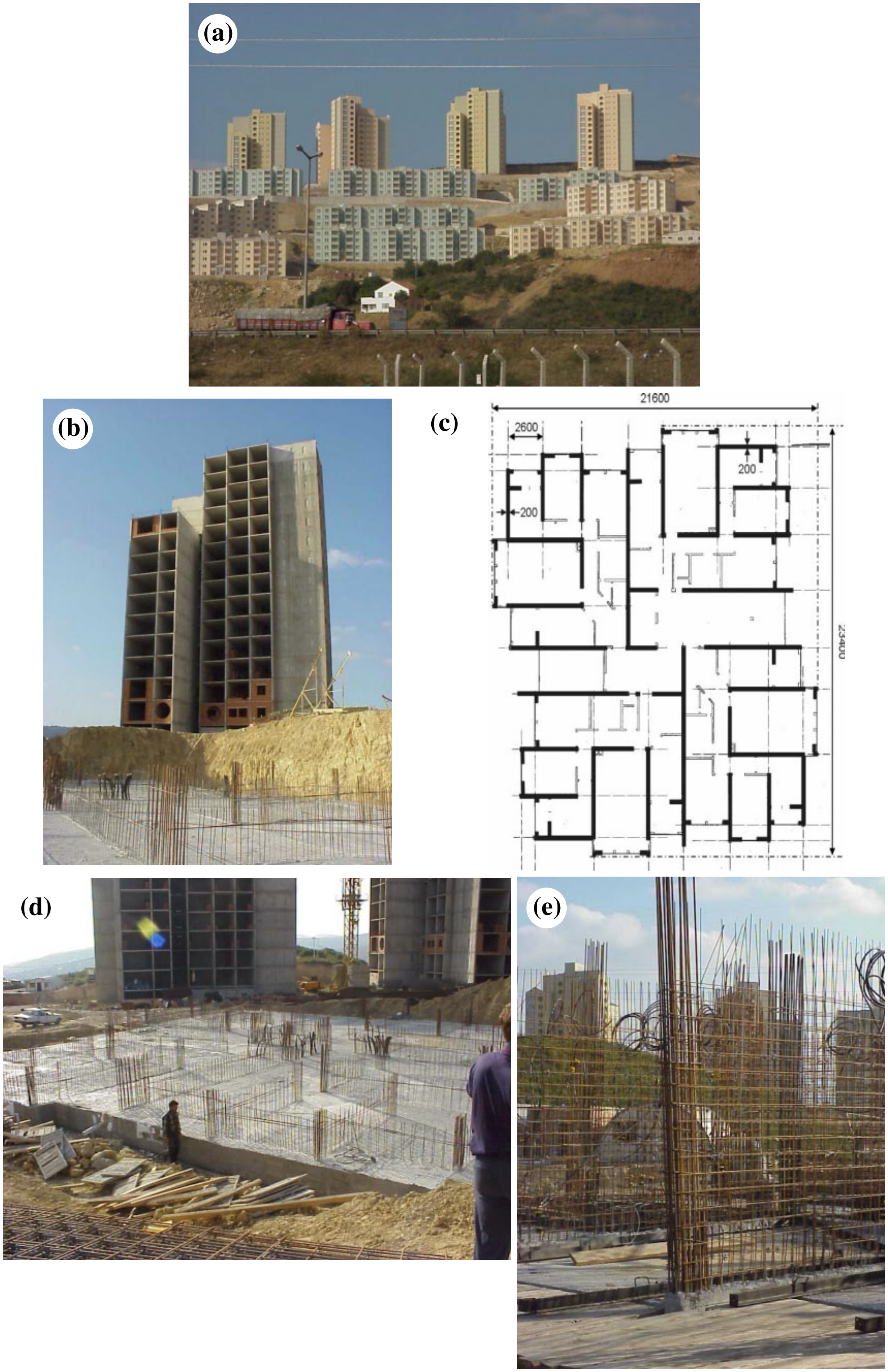


Fig. 33. Frame wall structure under construction: (*a*) overall view; (*b*) column reinforcement details; (*c*) wall reinforcement details; (*d*) insufficient concrete cover.

There were examples where epoxy filling of cracks was observed even for cases of severe shear cracking in columns. It is noted that the epoxy filling of cracks should be viewed as a temporary measure, since the main structural deficiencies had not been corrected.

Conclusions

The 1999 Kocaeli earthquake resulted in the collapses and severe damage to several types of concrete structural systems.

Structural deficiencies

The following deficiencies in the different types of structures were observed:

Concrete frame structures

- Unreinforced masonry infill walls suffered brittle failures and increased the base shear level, demonstrating that non-ductile frames with brittle infill walls are poor lateral load resisting systems for earthquakes.
- Many collapses at the ground-storey level were due to soft storeys resulting from the commercial usage and the reduced dimensions of ground-storey residential buildings.

Fig. 34. Concrete frame structure under construction.

Fig. 35. Precast concrete structure under construction.

- Use of thick slabs resulted in "strong column" – "weak beam" structures that resulted in column failures.
- Presence of partial infills created "short columns" that failed in shear.
- Lack of beam–column joint shear reinforcement resulted in many shear failures in joints.
- Inadequate detailing of structural members.
- Although unreinforced masonry is a poor structural system, the extensive use of masonry infills probably limited the drift sufficiently to limit the response of the frame structures to the elastic range and hence helped to prevent collapse of some buildings.

Fig. 36. Retrofit of corner column.

Fig. 37. Shear failure of ground-storey column.

Precast concrete structures

- Inadequate roof diaphragms and inadequate diaphragm connection in single-storey industrial buildings permitted large relative lateral displacements of frames.
- Complete structural collapse was caused by inadequate beam-to-beam, beam-to-column, and purlin-to-beam connections.
- Full-height and partial-height unreinforced masonry infills resulted in excessive seismic demands on precast concrete columns.

Fig. 38. Concrete columns after retrofit.

- Poor detailing of columns, including inappropriate placement of splices, inadequate confinement reinforcement, and poor details of column ties, resulted in many different types of brittle failures.

Inadequate enforcement of design and construction

It is generally acknowledged by site-visit teams from many countries, including experts in Turkey, that the prime factor that led to poor structural performance was the inadequate and sometimes nonexistent regulatory enforcement of both design and construction.

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