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Building damage from the Marmara, Turkey earthquake of August 17, 1999

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Abstract

The objective of this paper is to provide a brief overview of damage as observed immediately following the earthquake. Detailed studies of structural seismic performance, conducted in the time elapsed since August 1999, are not the subject of this paper, but rather the object of other papers presented in this Special Issue of the Journal. Damage to reinforced concrete, masonry, and steel structures, is described. The mode the failure presented include: foundation failures; soft stories; strong beams and weak columns; lack of column confinement and poor detailing practice; buckling and fractures of steel members; and non-structural damage. Some general lessons learned from this earthquake are also formulated.

Introduction

Beyond all other considerations, the true tragedy of the Marmara earthquake is that over 17,000 people were killed by the collapse of their homes. The primary function of a building is to shelter its occupants from a potentially harmful environment, whether they be threats due to predators, frequently occurring climate-related inconveniences and dangers, or rare but potentially devastating natural hazards. The collapse of thousands of buildings during an earthquake is, above all, a societal failure to recognize that latter important role of the built environment, a failure that can be attributed to one or more breakdowns in the chain of events needed to provide any community with a satisfactory level of earthquake resilience. That sequence of events requires, among many important steps, acknowledgment of the earthquake risk and the need for earthquake preparedness, implementation and enforcement of a comprehensive code for the design and construction of new earthquake-resistant buildings, identification of the hazards posed by buildings designed and constructed prior to the enactment of effective seismic codes providing an adequate level of protection, and formulation of a fiscally responsible seismic retrofit policy that will be compatible with societal expectations often only best expressed following a major earthquake.

Unfortunately, many of those important steps were not taken in Turkey prior to the August 17, 1999 Marmara earthquake. The latest 1998 Turkish building code, and its earlier 1975 edition to some extent, embody much of the latest knowledge on how to design effective earthquake-resistant buildings. Unfortunately, most existing structures were not built in compliance with these codes, and one may speculate that the extent of structural damage and number of casualties would have been greatly reduced had enforcement been more effective. However, this obviously would not have affected the thousands of buildings constructed prior to the enactment of effective seismic-design regulations, which would have remained seismically vulnerable.

Nonetheless, in spite of the above, some important lessons for design practice can be learned from the damage observed during this earthquake. The objective of this paper is to provide a brief overview of damage as observed immediately following the earthquake. Detailed studies of structural seismic performance, conducted in the time elapsed since August

1999, are the object of other papers presented in this Special Issue of the Journal.

Past earthquake history and damage

Given Turkey's past seismic history, the extensive damage suffered by reinforced concrete buildings during the Marmara earthquake is neither surprising, nor unexpected. Similar types of damage were observed to a lesser extent in many prior earthquakes throughout Turkey. For example, 30,000 died in Erzincan during the 1939 Richter Magnitude 8 earthquake, further east on the same Anatolian fault responsible for the Marmara earthquake. Following that earthquake, which totally devastated the city, Erzincan was reconstructed a short distance away from the abandoned ruins, further along the fault. Although most of the buildings that collapsed during the 1939 earthquake were of unreinforced masonry, in 1992, a Richter Magnitude 6.8 earthquake struck Erzincan again, with over 500 people killed by the collapse of numerous reinforced concrete buildings of similar construction as those damaged by the 1999 Marmara earthquake (Gülkan 1992, Bruneau and Saatçioglu 1993, EERI 1993, among many). However, because these and many other earthquakes were in more remote, less populated regions of Turkey, the message from earlier reconnaissance visits apparently did not resonate to the same degree.

Building characteristics and building codes

The predominant structural system used for buildings in urbanized Turkey consists of reinforced concrete frames with unreinforced masonry infills. This structural form is used for all building heights and occupancy, from single-story commercial to multistory residential and office buildings. Frame-shear wall interactive systems are also used in new buildings. Industrial buildings are either reinforced concrete (cast-in-place or pre-cast) or steel frame structures. A typical reinforced concrete frame building in Turkey consists of a regular, symmetric floor plan, with square or rectangular columns and connecting beams. The exterior enclosure as well as interior partitioning is of non-bearing unreinforced hollow clay tile masonry infill 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 on firm soil. Once the masonry 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 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 building codes and specifications have addressed seismic provisions since 1975. The 1975 provisions have been described extensively by Bruneau and Saatçioglu (1993), and closely resemble the North American reinforced concrete building codes developed at the time. The provisions in effect at the time of the earthquake (the 1996 edition of the code, enacted in 1998) were available at the web site (http://www.koeri.boun.edu.tr/earthqk/MP.htm). These latter provisions contain a base shear design equation very similar to that found in the 1988 Uniform Building Code (UBC), with the exception that seismic zone numbering is reversed in the Turkish code, the more severe seismic regions being labeled Zone 1 (instead of Zone 4 in the UBC). Allowing for differences attributable to the Limit States Design format of the Turkish code (compared to the Allowable Stress Design basis of the UBC), both codes have generally similar strength reduction factors, R. The Turkish code also requires that reinforced concrete shear walls be included in non-ductile structural systems.

Structural damage

The damage to reinforced concrete buildings from this earthquake can be attributed to one or more of the following factors.

Foundation failures

Foundation failures were observed for many buildings with large settlements, and in some cases, entire structures overturned. This effect was most pronounced in Adapazari.

An example of severe settlement is shown in Figure 1. Taking the distance from balcony to balcony as a reference, one may observe the short-distance left between the second floor balcony and ground level.

Figure 1. Damage due to foundation failure.

Figure 2. Ground heave was extensive all around this building.

This provides a qualitative measure of the magnitude of settlement that occurred. Figure 2 also shows the extent of ground heave all around the building as the soil under the building is pushed outward during settlement.

Examples of overturned buildings are shown in Figures 3 and 4. In the latter case, force vectors are superimposed on the building to illustrate its significant resistance to lateral loads (Figure 4b). As shown in that figure, the resultant weight of the structure, schematically located at the center of gravity of the above-ground portion of the toppled building, can be decomposed into its components perpendicular and parallel to the building's original vertical axis. Calculations indicate that the building, in its new leaning 'position,' resists a statically-applied force approximately equal to 0.9 g perpendicular to its original vertical axis. This resistance is essentially provided by the numerous partition walls that increase lateral strength.

Foundation failures generally occurred as a result of soil liquefaction or bearing pressure failures. Note that geotechnical evaluations of site conditions are apparently often not conducted in Turkey, except for important structures.

Soft stories

A large number of residential and commercial buildings were built with soft stories at the first-floor level

Figure 3. Overturned building due to foundation failure.

(a) (b)

Figure 4. Overturned building due to foundation failure.

(a soft-story is a floor that is structurally significantly more flexible and weaker than the others). First stories are often used as stores and commercial areas, especially in the central part of cities. 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 story increases deformation demands very significantly, and puts the entire burden of energy dissipation on the first-story structural elements, as opposed to distributing the burden along the entire height of the building. Many failures and collapses can be attributed to the increased deformation demands caused by soft stories, coupled with lack of deformability of poorly designed columns. Examples of soft-story failures can be seen in Figures 5 to 9. The timber building shown in Figure 5, having a wide obstruction-free ground floor, has nearly collapsed and clearly illustrates how signific-

Figure 5. Failure of the soft first story in a timber building.

Figure 8. Failure of the soft first story.

ant damage can concentrate in the soft-story of such buildings.

Soft-story buildings having open street facade and solid back-walls tend to collapse toward the street as a result of the torsional plan eccentricity. Story sway is greater on the more flexible facade side, resulting in greater displacement and ductility demands on the more vulnerable columns on the facade side (Figure 8). This was particularly evident on a commercial street where nearly all the buildings collapsed towards the street (Figure 9).

Soft-story failures combined with ground failures were also observed along the waterfront in Gölcük. As shown in Figure 10, the water of the Marmara Sea submerged parts of the waterfront as a result of ground lateral spreading. Some buildings located along the waterfront (e.g., Figure 11) were thus impacted by both geotechnical and structural deficiencies.

Strong beams and weak columns

In most frame structures, the beams were strong and remained elastic, and the columns were weaker and suffered damage and failure in the form of compression crushing, plastic hinging, or shear failure. In many cases, relatively deep beams were used with flexible columns, contributing to a strong-beam weak-column behavior. This undesirable behavior is illustrated in Figure 12 and 13. In Figure 12, for example, the column depths are sizeable along the building alley sides, but they are narrow and thereby less able to resist by flexure the seismically-induced horizontal forces parallel to the front street, particularly when compared to the beam depth and strength at the story above. Not surprisingly, the building collapsed swaying in the direction of weakest column flexural strength.

Whenever damage develops in columns without ductile details, strength and stiffness degradation will be further precipitated by the presence of axial forces. Excessive column damage not only means loss of lateral load resistance but also loss of gravity load resistance. Hence, the wisdom of the strong-column weak-beam alternative, promoted by buildings codes.

Figure 6. Failure of the soft first story.

Figure 7. Failure of the soft first story.

Figure 9. Buildings in this commercial area collapsed towards the street due to open facades (soft story) and walls on back side adding torsional eccentricity.

Figure 10. Marmara Sea submerged parts of the waterfront as a result of ground lateral spreading.

Figure 11. Some waterfront buildings were impacted by both geotechnical and structural failures.

Lack of column confinement and poor detailing practice

Most of the structural damage observed in frame buildings was concentrated at column ends. Unfortunately, confinement reinforcement was virtually nonexistent in these members, making them unable to maintain the required ductility. A number of detailing deficiencies were observed in the damaged structures. This included lack of anchorage of beams and columns reinforcement, insufficient splice lengths, use of 90° hooks, poor concrete quality, less than full height masonry infill partitions, and frequent combinations of many of the above. These errors were often compounded by geometric irregularities such as eccentric beam-to-column connections that induced severe torsion in short perpendicular stub beams (Figure 16).

Column damage varied as a function of column geometry and detailing. Examples in Figure 14 and 15 illustrate shear failure in short captive columns 'trapped' between partial height infills, and flexural failure in non-ductile reinforced concrete plastic hinges, respectively.

The construction sequence adopted in residential buildings in some of Turkey's new developing semi-urban districts is partly responsible for some significant failures due to lack of column reinforcement anchorage. Typically, residential buildings are constructed over a large number of years, one story at the time. Families do not wait for completion of the entire building prior to occupancy, and usually elect to live in the lower completed stories. Additional stories are added as the need arises. Figure 17 illustrates a one-story building during construction. Note the column vertical reinforcement extended above the roof line. Infill walls will be added to the first story and residents will move in, and this reinforcement will be left extending above the slab until the day (maybe years ahead) when construction will resume to add another story. As a result of this practice, and because smooth bars are frequently used, anchorage of the column bars in the slab is often deficient, and may have contributed to many collapses. Figure 18 shows an example of such bar pull-out in a building on the verge of collapse.

Miscellaneous

A number of buildings sitting directly on the fault were also destroyed by the relative movements of the fault. However, numerous buildings immediately adjacent to the fault survived without structural damage. For residential buildings, this is generally attributable to the additional strength unintentionally provided by the infills, as indicated previously. Industrial buildings similarly located, however, survived on the merit of their explicit design. For example, an industrial complex being constructed 100 feet from the fault had very well confined columns. Extensive ground settlements induced large structural permanent displacements to parts of the structure, and the columns developed plastic hinging with considerable permanent deformation and visible concrete spalling, but collapse was prevented in spite of this damage as the spacing of the column's transverse reinforcement was adequate to ensure ductile behavior (Figure 19).

Damage to steel structures

Steel, being by far the most expensive construction material in Turkey, has not been widely used in construction, so that typically only industrial structures rely on steel for their lateral load resistance. Some industrial equipment/structures were damaged

(a)

(b)

(c)

Figure 12. Damage due to strong beams and weak columns.

by this earthquake, and a few collapsed. Typical causes for collapses include failure of anchor bolts at column bases and structural instability under overturning forces. Other evidence of damage include fracture of brace connections, buckling of braces, and local buckling in concrete filled steel hollow pipes used in wharves. A comprehensive review of the seismic performance of industrial facilities is beyond the current scope, but some comments are appropriate here to illustrate that steel structures also require ductile detailing for superior seismic performance.

An interesting comparison between ductile and non-ductile steel details for nearly identical structures is shown in Figures 20 and 21. In Figure 20, the anchor bolts of the tank supports of the NUH concrete producing facility yielded in tension, and concrete of the 2.8 m deep pedestals spalled under the compression impact. Although this is not a ductile detail, the braces would be in partial compliance with the AISC seismic design provisions. The 2,400 mm long braces are $2L80 \times 80 \times 8$ back-to-back 10 mm from each other with batten plates at the 1/3 points, resulting in mem-

Figure 13. Damage due to strong beams and weak columns.

bers having slenderness KL/r of 97 and b/t of 10; note that the AISC limits for ductile concentrically braced frames are 101 and 7.35 respectively for Grade 50 steel, 120 and 8.7 for Grade 36 steel. Connections were also stronger than A_gF_y of the members. Hence, these braces partially met (intentionally or not) the AISC 1997 requirements for ductile concentrically braced frames (DCBF). Inelastic action developed in the braces, allowing ductile structural response through dissipation of hysteretic energy in a stable manner, with the exception of the deficient anchor bolts detail and somewhat premature local buckling, and the braced frame survived. Global brace buckling, and local buckling at the brace mid-length plastic hinge (Figure 20d) illustrate this ductile plastic response. Also typical of braced-frame with Chevron brace configurations, buckling of the diagonal in compression occurred prior to yielding in tension; this led to force redistribution within the structure and resulted in plastic hinging at mid-span of the horizontal member and slight permanent vertical deformations was visible at that point. This typical mechanism of Chevron braced frames is well known and explained in greater detail elsewhere (Bruneau et al., 1997).

By comparison, structures serving the same purpose at the Lafarge concrete plant suffered excessive damage. In this case, all members of the braced towers were hollow structural circular steel members, 150 mm in diameter and approximately 3 mm thick, for a resulting D/t slenderness ratio of 50, greatly in excess of the limit of 26 imposed for Grade 50 steel braces by the aforementioned AISC seismic provisions. However, braces did not buckle in this particular case. Because the lower brace was connected to the tower leg 16″ above the base, the tower behaved as a rigid braced structure supported on a 16″ tall moment frame, and the tower leg buckled at that location under the combined axial and flexural stresses resulting from this localized severe eccentricity (Figure 21). Steel locally fractured in some locations as a result of the large strains induced by this collapse. Note that the most severely damage tower was estimated to be 85–90% full (85 m^3 out of a 110 m^3 capacity) at the time of the earthquake, while the other towers were approximately 65% and 90% full. Incidentally, a gravel and sand bin (approximately 25% full) adjacent to these towers, overturned and collapsed. It was supported on six legs, each tied with a 24 mm diameter bolt to a concrete base. The legs buckled on one side, and the anchor bolts pulled out of the foundation or tore out of the leg as the bin collapsed (Figure 21c). Note that damage or collapse of elevated reservoirs was common throughout the affected area (Figure 22).

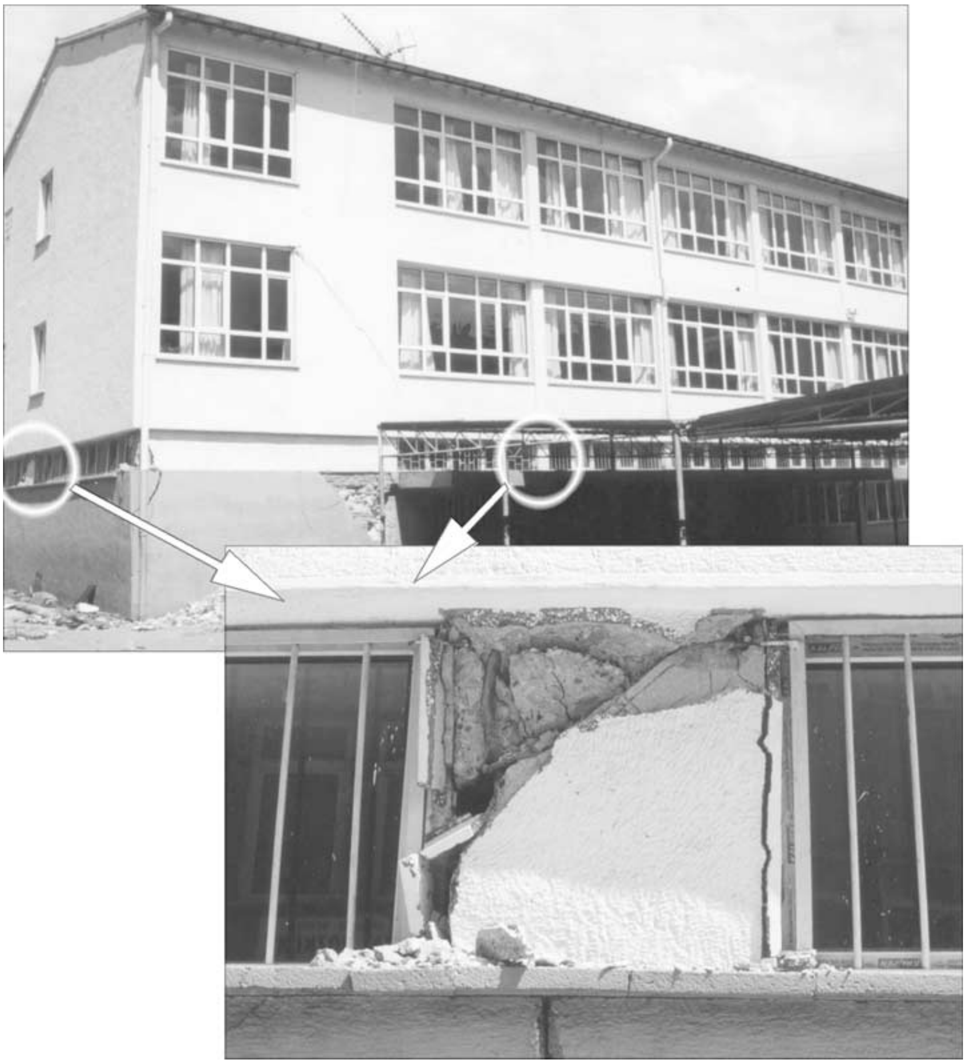


Figure 14. Damage due to short (trapped) columns.

It is noteworthy that some new industrial facilities located in close proximity to the fault survived without structural damage. One such example is the Hyundai plant, designed in 1997 in dual compliance with the latest Turkish and Korean seismic provisions. This structure consists of long span moment resisting frames, with heavily stiffened deep haunches at the end of beams, welded to heavy steel square box-columns built from 1/2″ to 1″ plates, suggesting that design was accomplished in awareness of the numerous fractures observed following the Northridge problem, and the recommendation that haunches be used to ensure that plastic hinges be located away from the face of the columns. Nonstructural damage was sufficiently extensive to stop production, and the plant manager estimated it would take approximately one month before completion of repair and facility inspection by the Hyundai engineers. Although the author was not allowed to take photos during the visit to this plant, examples of damage included sliding of large 4′–5′ diameter 1.6″ thick ventilation pipes following rupture of the 12 bolts connection to the floor, and collapse of cable trays in complex configurations unbraced against twisting.

Figure 15. Damage due to flexural failure in non-ductile plastic hinges.

Other construction types, nonstructural damage and seismic retrofit

Damage to buildings of other construction materials was also observed throughout the affected area. For example, many industrial facilities of pre-cast concrete collapsed as a result of failures at the beam to column connections (Figure 23). Damage to unreinforced masonry buildings was also sometimes observed. For example, the building shown in Figure 24 suffered severe shear failure of its corner wall, as a result of bi-axial seismic action. The number of such buildings was, however, small in proportion to the overall building inventory.

In the presence of such an overwhelming amount of structural damage and collapse, the seismic hazards ensuing from nonstructural damage tend to be forgotten. These should not be overlooked as they may result in extensive injuries and casualties. For the school building shown in Figure 25a, out-of-plane failure of an unreinforced masonry gable due to inadequate anchorage to its backing could have killed or maimed many children had this earthquake occurred at a time when the school yard was being used. Likewise, failure of the plaster wall finishes inside the corridors of the same school (Figure 25b) could also have hurt many small children.

As noted earlier, at the time of the earthquake, seismic awareness in Turkey had not reached a level that made it possible to focus on the seismic hazard posed by the structures designed without due attention to earthquake-resistant design. This may explain why, to the author's knowledge, only one building in the severely affected region had apparently been seismically retrofitted prior to the earthquake. The Sakarya 'province' government building, shown in Figure 26, was allegedly retrofitted by the addition of shear walls some years prior to the earthquake. This building sustained damage to its infill walls at the ground story during the 1999 earthquake and was subsequently repaired. However, it remained operational following the earthquake, which is significant considering that it was literally surrounded by collapsed and severely damaged buildings, such as the one shown in Figure 27.

Damage from aftershocks

It is noteworthy that a number of buildings, weakened by the major initial shock, collapsed during the numerous aftershocks. Because the population was generally left free to re-enter severely damaged buildings to retrieve their personal possessions, these additional collapses added unnecessary casualties and injuries. For example, during the author's earthquake reconnaissance visit, individuals were seen removing miscellaneous furniture (even wood doors) from the dangerously sloping second and third floor of buildings that had completely lost their first story and exterior partition walls.

Furthermore, additional extensive damage, losses and casualties occurred on November 12, 1999, when another earthquake of Richter Magnitude 7.2 struck approximately 115 km east of Izmit, Turkey. Because the town was mostly evacuated after the August 17 event, there were far fewer life losses compared to the large number of structural collapses during this second event. However, this second major earthquake is significant in that it clearly shows that major events may occur closely spaced in time and space to each other (similar to what occurred in the United States when three earthquakes, each of Richter Magnitude

Figure 16. Damage due to eccentric beam-to-column connections.

Figure 17. One story building during construction with column bars extended above roof slab to accommodate possible construction of other stories in future years.

Figure 18. Example of bar pull-out due to deficient anchorage.

greater than 8, struck the New Madrid area from 1811–1812). The concept of a return period is only true in an average sense, and it would be a major error for the government and population to complacently believe the fallacy that a long period of seismic quiescence inevitably follows a major earthquake. The same urgent need to promptly implement seismic hazard mitigation measures remains following an earthquake, and, if anything, authorities should capitalize on the sudden increase in earthquake awareness and receptiveness to sweeping measures to improve the overall level of earthquake preparedness.

The November earthquake also illustrates the misconception that structures that have survived a first earthquake have thus been 'proof-tested' against future earthquakes. First, the same event never repeats itself twice, and a different ground motion 'signature,' with greater peak-ground-accelerations or velocities, is possible even for a second earthquake of similar magnitude. Second, subsequent earthquakes could be of greater magnitude, or simply have their epicenters located closer to a particular building. Hence, the need to assess the seismic adequacy of existing buildings should not be preempted as a result of prior satisfactory seismic performance, and buildings found to be seismically deficient should still be retrofitted if deemed necessary. However, past seismic performance can be most valuable to calibrate the engineering evaluations of seismic adequacy for those buildings that were instrumented prior to an earthquake and for which response data is available.

Lessons learned and conclusions

The most important lesson with implications for practice is that seismic resistance is not inherent to buildings that are able to resist gravity loads. Explicit consideration of lateral loads, together with ductile detailing, are required to ensure seismic survival and control structural damage. While this may seem obvious to many, the situation still remains that earthquake resistant design is not mandatory in many parts of the world exposed to a significant earthquake risk. Major parts of the United States, particularly east of the Rockies, could be used to illustrate this point. The complacent ignorance of the seismic threat that existed in Turkey and that resulted in the poor implementation of existing seismic codes, is not so different from that which impedes efforts to implement seismic codes on the basis of costs or other arguments in parts of the United States. In Turkey, which is a far less litigious society than the United States, the government now finds itself unable to use the Act-of-God argument to

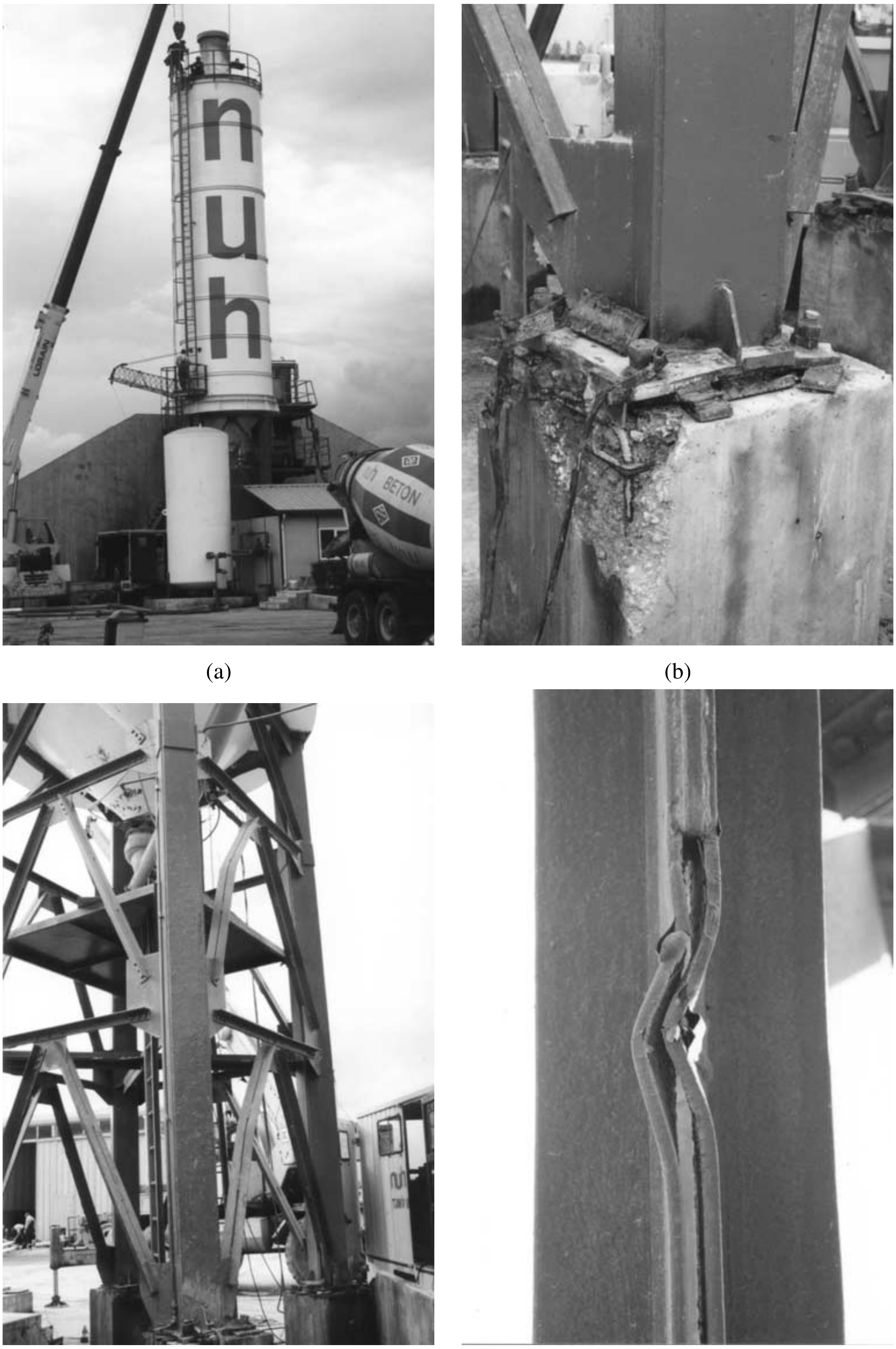
(a)

(b)

(c)

(d)

Figure 19. Industrial facility near the fault: (a-b) global views of damage due to large ground settlements; (c-d) leaning column with limited damage at base as a result of ductile details (spacing of transverse reinforcement is indicated by thumb and index fingers).



(a) (b)

(c) (d)

Figure 20. Damage to steel structures at NUH concrete plant: (a) global view; (b) spalled pedestal; (c) global buckling of braces; (d) local buckling at brace mid-length.

(a) (b)

(c)

Figure 21. Damage to steel structures at Lafarge concrete plant: (a) global view; (b) buckled leg; (c) overturned sand bin.

Figure 22. Overturned reservoir.

Figure 23. Damage to a precast concrete building.

Figure 24. Damage to an unreinforced masonry building.

(a) (b)

Figure 25. Nonstructural damage to a school building.

Figure 26. Undamaged Sakarya ‘province’ government building retrofitted by shear walls prior to earthquake.

Figure 27. Typical building adjacent to retrofitted Sakarya ‘province’ government building.

defend its past inaction, a position that is now recognized to be indefensible in the United States in light of the extensive knowledge that exists on how to perform earthquake-resistant design of buildings. In that perspective, many non-ductile structures likely to suffer severe damage in future earthquakes exist in Eastern North America, and the potential for enormous losses, in lives and properties, is real.

While there may exist in many buildings infill walls and other such nonstructural elements that may contribute to increase the threshold of damage, consideration of this contribution in terms of strength, ductility, and benefit to seismic response, is currently difficult and, at best, uncertain. The development of reliable models to quantify this impact should be pursued aggressively through future research.

Finally, the importance of a sound geotechnical foundation and the benefits of seismic retrofit have been clearly illustrated by this earthquake. The latter is particularly important to support efforts to formulate seismic retrofit policies to mitigate future earthquake-induced damage and ensure building performance compatible with societal expectations.

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