Performance of Reinforced Concrete Components and Buildings during the August 17, 1999 Kocaeli (Izmit), Turkey Earthquake

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Abstract

A large number of reinforced concrete buildings were heavily damaged or collapsed during the 7.4 magnitude earthquake that struck northwestern part of Turkey on August 17, 1999. Recorded peak ground accelerations were relatively low (0.3g-0.4g) compared to the magnitude of the structural damage, and the elastic acceleration response spectra from the recorded motions were comparable with the elastic design spectra specified in the current Turkish seismic code. A brief description of seismic code requirements is presented and compared with observed details. Other than the damage caused by liquefaction and poor soil conditions in some regions, major causes for large destruction were due to weaknesses and vulnerability of typical three- to six-story nonductile reinforced concrete buildings and their components to seismic loads. These weaknesses include: reinforced concrete columns with insufficient confinement and lateral reinforcement, 90-degree hooks at the end of column ties, short columns, poor detailing in beam-column joint regions, strong-beam and weak-columns, less infill walls in the first stories leading to soft stories, not having proper moment resisting frame system due to irregular column orientation, and poor quality of construction. Buildings with shear wall components performed well.

Keywords: 1999 Turkey earthquake, earthquake damage, reinforced concrete, seismic behavior

Introduction

On August 17, 1999, a Mw 7.4 earthquake occurred along the 1500-km-long North Anatolian fault in northwestern Turkey. The epicenter of the earthquake was near Izmit, 90 km east of Istanbul (Figure 1). Following the earthquake, the Pacific Earthquake Engineering Research Center dispatched a reconnaissance team to the epicentral region to learn first hand about the damage to and performance of building infrastructure. The geographic region that was impacted by the earthquake was somewhat narrow banded and centered around the fault, and stretched from Istanbul in the west to Gölyaka and Düzce in the east. Damage to building construction was severe and widespread (Sezen 2000a, Aschheim 2000, Scawthorn 2000). Estimates for economic losses were around six billion US dollars. The official death toll was over 17,200, with some 44,000 people injured and thousands left homeless. Some 77,300 homes and businesses were destroyed, and 244,500 were damaged. The majority of deaths and injuries were in the cities of Gölcük , Adapazari, and Yalova.

The peak ground accelerations (PGA) recorded in the region affected by the earthquake are shown in circles in Figure 1. An acceleration time history recorded at the Yarimca (YPT) station across the Izmit Bay from Gölcük is shown in Figure 2. The secondary shaking clearly seen in Figure 2 at 40 seconds appears in most acceleration time histories from the Izmit earthquake, however, when integrated this high frequency shaking has very little effect on the velocities and displacements. Elastic response spectra at 5% damping from the YPT station, SKR station in Adapazari (PGA 0.41g), IZT station in Izmit (PGA 0.32g) and the DZC station to the east of Gölyaka (PGA 0.37g) are shown in Figure 3. Figure 3 also presents 5% damped linear elastic acceleration response spectra for rock and soft soil sites calculated using the provisions of the 1997 *Uniform Building Code* (ICBO, 1997) and the Turkish *Specification for Structures to be Built in Disaster Areas* (Ministry, 1997) for the regions of highest seismicity in each country. The Uniform Building Code (UBC) spectra shown in the figure do not include near-field factors. Putting these factors aside, the elastic spectral demands of the two current codes are most similar. Considering that the spectra shown in Figure 3 is calculated using the strongest ground motions recorded in the region, lateral earthquake forces on most structures designed following the 1997 Turkish seismic code (Ministry, 1997) would be smaller than the code design forces. However, the details observed by the authors and the widespread damage to the reinforced concrete (RC) buildings in the region suggest that in the design of most commercial and residential buildings either older building codes, which allow nonductile detailing of RC building components, or no code provisions were followed. The vast majority of collapsed and severely damaged RC buildings were constructed within the last 25 years after the introduction of ductile design requirements in Turkish seismic codes (Ministry, 1975). A detailed investigation of evolution of seismic building design and construction practice in Turkey was provided by Sezen et al. (2001a and 2001b).

Liquefaction and settlement due to poor soil conditions caused widespread damage in certain areas in the epicentral region. Subsidence of a large area around Gölcük, caused flooding and extensive damage to buildings close to the shoreline. However, most of the loss of life and damage in the Izmit earthquake was a result of the poor performance of RC buildings. In the epicentral region, RC moment-resisting frames and shear walls are the two most prominent framing systems used for residential and commercial construction.

Construction Practice

The quality of the construction of residential and commercial buildings in the epicentral region varied widely. The reconnaissance team found evidence of extremely good and extremely poor construction. Although the general quality of residential building construction was poor according to US standards, the quality of much of the engineered RC commercial construction (e.g., office buildings) was reasonable and not poor. Many of the failures and collapses of engineered commercial construction observed by the team and reported in Sezen et al. (2001a) can be attributed to the use of nonductile details and not poor quality construction.

Photographs of the second floor of a residential apartment building under construction in a village outside of Gölcük are presented in Figure 4. This figure shows a typical example of reinforcing details and formwork in residential apartment construction by a homeowner. Transverse reinforcement with 90-degree hooks are used. The beam rebar details are non-ductile. Beam bent-up rebar can be seen in Figure 4b. Blade columns (long and narrow in plan) are routinely used in apartment buildings to enable the builder to construct the columns within the thickness of the wall. Some column details can be seen in Figure 4b. Vertical rebar are spliced at the floor level. Typical splice lengths are approximately 1 m and no additional ties are provided in the splice region. Joint shear reinforcement is not provided. The column rebar details are nonductile. Smooth rebar is commonly used for reinforced concrete construction in the epicentral region. The yield strength of such rebar is approximately 275 MPa (Kurama 2000). Smooth rebar is less expensive and more readily available than deformed rebar, and is easier to bend and cut on site than deformed rebar. Low-strength concrete was identified in a number of damaged buildings visited by the reconnaissance team.

Interior and exterior infill walls are constructed of either hollow clay tile or lightweight gas-concrete blocks. The hollow clay tile block is more widely used than the gas-concrete block and is extremely brittle. The block infill is not reinforced nor is it anchored to the structural framing with masonry ties. Block infill walls are built in contact with the structural framing and add significant stiffness and strength to the framing system. A photograph of hollow clay tile blocks are shown in Figure 5.

Typical Framing Systems for Residential Moment-Resisting Frame Construction

Many apartment buildings in the epicentral region are constructed with a tall first story for commercial (shop) occupancy. Typical story heights range between 3.5 to 4.5 m in the first story and 2.8 to 3.0 m in the upper stories. Residential buildings in the epicentral region typically range in height between two and seven stories. Figure 6a presents photograph of a three-story moment resisting frame building that was under construction at the time of the earthquake. A plan of the second floor framing is shown in Figure 6b. The column and beam orientation shown in Figure 6b would suggest that the framing system is much stiffer and stronger in the direction perpendicular to the street assuming that similar rebar are used in all beams and all columns. Note that ten of the columns are stronger in the x-direction (parallel to the street), whereas five columns are stronger in the y-direction. The column orientations and locations are such that all of the moment-resisting frames include one or more columns with their weak axis perpendicular to the frame direction. Such framing likely possesses limited strength and stiffness, which if coupled with nonductile reinforcement details, results in a vulnerable building in the event of earthquake shaking.

Behavior of Moment-Resisting Frame Construction

Moment-resisting frame construction fared poorly during the Izmit earthquake. According to official estimates, more than 20,000 moment-frame buildings collapsed, and many more suffered moderate to severe damage. Three- to seven-story RC apartment buildings were hard hit, although many had been constructed in the past 20 years. These apartment buildings were likely designed and detailed to comply with the requirements of the 1975 code (Ministry, 1975) for construction in a first-degree seismic zone. Many of the collapses are attributed to the formation of soft first stories that formed as a result of differences in framing and infill wall geometry between the first and upper stories, the use of nonductile details, and poor quality construction in some cases.

Figure 7 is a photograph of two six-story nonductile moment-frame buildings in Gölcük. One of the buildings collapsed completely, whereas an adjacent building suffered only superficial damage in the form of minor cracking in the first-story columns. Much of the first story of the collapsed building was intact. Careful examination of the first stories in both buildings showed that the buildings had a similar plan footprint and common construction details. It is likely that the two buildings were nearly identical and that both buildings were constructed by the same contractor. Both buildings were probably subjected to similar levels of earthquake shaking, yet one building remained in the elastic range and performed well, while the other collapsed. This raises many questions regarding the limit state for nonductile moment frames. Small differences in the strength of these nonductile buildings caused by the use of different construction materials and different construction practices and workmanship could account for the difference in performance.

Role of infill walls in response of moment-frame buildings

Hollow clay tile and gas-concrete masonry infill walls are widely used in the epicentral region. Many of the buildings were constructed with hollow clay tile infill in the frames perpendicular to the sidewalk. Because the buildings often housed shops and commercial space in the first story, instead of hollow clay tile infill walls, glass panels were placed between the first story columns adjacent to the sidewalk, but tile infill was used in the upper stories. Such an arrangement of tile infill created stiffness discontinuities in these buildings, which may have contributed to their collapse. These walls are generally unreinforced and nonductile. The walls abut the frame columns but are not tied to the frame. The high in-plane stiffness of the masonry infill that is developed by diagonal strut action can dictate the response of the more flexible moment-resisting frame. Damage to infill masonry walls was concentrated in the lower stories of buildings because of higher demands on the strength of the moment frame-infill wall system.

Figure 8a and 8b are views of a collapsed apartment building in Gölcük. The first two stories of this building failed completely, but damage in the upper four stories with unbroken glass windows was limited. The long infill walls in the upper four stories have significant elastic strength and stiffness—probably much greater stiffness and strength than the moment-resisting frame. If the infill walls in the upper four stories of the building are indicative of the infill in the failed stories, the first- and second-story infill walls likely played an important role in the collapse of the building. The brittle fracture of the first- and second-story infill masonry walls would have overloaded the nonductile first- and second-story frame columns, resulting in a complete failure.

The first two stories of the building in Figure 9 collapsed. The infill masonry walls and moment-frame construction in the third and fourth stories (first and second stories of the collapsed building) suffered major damage. Damage in this building reduced with increased height above the sidewalk. Failure of the infill masonry in the first and second stories of the building likely precipitated the collapse of the building.

Irregular placement of infill masonry walls can produce discontinuities of stiffness in moment- frame buildings. Consider the building in Figure 10 in which the moment frame is both flexible and weak by comparison with the upper stories. In the first story of this building, infill masonry walls are present in the back face of the building and in the two faces perpendicular to the sidewalk. The front of the building was open in the first story. The lateral stiffness of the building was likely large in the direction perpendicular to the sidewalk and much smaller parallel to the sidewalk. Deformation is concentrated in the first story of this building parallel to the sidewalk, due to the weakness and flexibility of the moment frame and to the lack of infill masonry in the front of the building. The first-story columns in this building were badly damaged and likely close to failure.

Detailing and Response of Moment-Frame Components

Previous sections of this paper on moment-frame construction have focused on the response of moment-frame systems. This section of the paper addresses the response of the components of moment frames, namely beams, columns, beam-column joints**,** and slabs, and follows with discussions of each of these components in turn. Based on the authors observations, information on column and beam rebar details are provided in Figure 11. Bent-up rebar is shown in the typical beam section. Corner column rebar are spliced above the floor slab with lap lengths of 40 to 70 bar diameters. Side-face column rebar are either spliced per corner rebar or terminated above and below the joint with 180-degree hooks as shown Figure 11. No transverse reinforcement for the purpose of confinement is provided in the hinge, joint, or splice regions. In general, typical 80-mm to 120-mm thick RC floor slabs performed well. Vulnerabilities and description of the damage to the commonly used joisted floor slab framing system are provided in Sezen et al. (2001a).

a) Beams

Typical beam spans ranged between 3 and 5 m. Beam depths and widths ranged between 200 to 250 mm and 500 mm to 600 mm, respectively. Transverse ties are smooth rebar of 6 to 10 mm diameter with 90-degree hooks. Bent-up longitudinal rebar, often used for reasons of economy to provide shear resistance to gravity loads and to increase negative moment-resistance for gravity loads at supports, however do not resist shear force if the loads are reversed due to earthquake shaking.

Little damage to interior moment-frame beams was observed by the reconnaissance team because columns were generally weaker than beams. One type of beam damage is shown in Figure 12. The building in this figure suffered a partial story collapse because the fault ruptured beneath the building. The beams shown in the frame elevation were forced to accommodate the partial collapse and were badly damaged at the beam-column connection due to slip of the smooth longitudinal beam rebar. In many cases, beam bottom rebar was inadequately anchored in and through the beam-column joint.

b) Columns

Column plan dimensions range between 150 mm x 500 mm to 250 mm x 800 mm. Most columns in residential construction are blade columns with an aspect ratio of approximately 3. The longitudinal rebar ratio ranges between 1% and 2%; 12 to 16 mm diameter smooth rebar are generally used. Transverse ties are smooth rebar of 6 to 10 mm diameter with 90-degree hooks. The spacing of transverse ties is typically 200 to 250 mm uniform along the clear height of the column. Table 1 shows the column dimensions and details observed by the reconnaissance team, and compares with the seismic code requirements given in the ACI 318, *Building Code Requirements* (ACI, 1995), TS-500, *Building Code Requirements for Reinforced Concrete* (Turkish 1985), and the 1975 and 1997 Turkish Seismic Codes (Ministry, 1975, 1997). From discussions with expert academicians and design professionals in Turkey, it is clear that little use was made of the Turkish seismic code for the engineering of commercial and residential construction in the epicentral region prior to the August 17, 1999, earthquake. Prior to adaption of the 1997 seismic code, the design professional was permitted to design nonductile RC buildings in Turkey. The design professional had two options for the analysis, design, and detailing of RC moment resisting frames: (1) calculate seismic design forces from the seismic code (Ministry, 1975) using the force reduction factors for nonductile moment resisting frames, and design and detail the frame using the building code TS-500 (Turkish, 1985) (that has no provisions for ductile detailing), or (2) calculate seismic design forces from the seismic code using the force reduction factors for ductile moment resisting frames, and design and detail the frame using the seismic code and its corresponding detailing requirements for high ductility. A detailed study of building design practice in Turkey, comparison of Turkish and US building codes, and substantial changes to the Turkish seismic codes are presented in Sezen et al. (2001b).

Along with Table 1, Figure 13 presents information on the transverse reinforcement requirements along the height of a column from the 1975 and 1997 Turkish seismic codes. Table 1 clearly shows that the code provisions were not followed in detailing RC columns, especially in the plastic hinge zones at the end of columns. In damaged columns, the authors did not observe a tie spacing of less than 100 mm or 135-degree hooks at the end of column ties as required by all codes except for the Turkish building code, TS-500, which in general does not include ductile detailing requirements for columns.

The majority of moment-frame component failures were in columns and were due (a) to the use of nonductile details and nonconfined lap splices, (b) to excessive beam strength, and (c) to interaction between the columns and the infill masonry walls. Lap splices in moment-frame columns were typically made immediately above the floor framing or the foundation. The exposed lap splice of Figure 14a is from a moment-frame building in Adapazari. The lap splices in this column were approximately 35 bar diameters in length and were located in a plastic hinge zone. Widely spaced transverse ties with 90-degree hooks were placed in this column; no cross ties were present. The 90-degree hooks on the ties opened during the earthquake and the limited strength and confinement afforded by the ties were lost.

The transverse tie details of Figure 14b were common, namely smooth rebar, widely and often unequally spaced ties (200 to 250 mm), and 90-degree hooks. The wide spacing of the ties resulted in shear failures (Figures 14b and 14c), buckling of longitudinal rebar (Figure 14c), and poor confinement of the core concrete (Figure 14d). Concentrated damage at the ends of moment-frame columns was observed throughout the epicentral region. An example of such damage is presented in Figure 15. Large rotations at the ends of the columns produced severe cracking and loss of concrete. Note the relative proportions of the columns and the beam in this figure.

c) Beam-column joints

Typical damage to beam-column joints is shown in Figures 16. The collapse of this building in Adapazari was due to the failure of the beam-column joints. Much of the framing (Figure 16a) is essentially intact but many of the beam-column joints are heavily damaged. A view of one of the damaged joints is shown in Figure 16b. Beam rebar anchorage in the joint is inadequate and no transverse ties are present in the joint region.

The photograph of a building (Figure 17) that was under construction in Adapazari at the time of the earthquake shows severe damage in the beam-column joints, but the horizontal transverse ties maintained the integrity of the joints.

Shear-Wall Construction

Buildings constructed using shear walls as the primary lateral load-resisting system performed quite well in the 1999 Izmit earthquake. Some buildings with a dual wall-frame lateral load-resisting system were damaged because the shear walls were not sufficiently stiff to keep the displacements of the nonductile framing system in the elastic range. Story collapses were not observed in buildings containing shear walls, but it should be noted that shear walls were not widely used in the epicentral region.

Behavior of Shear-Wall Construction

Outside Istanbul, few buildings in western Turkey are constructed with shear walls as the primary lateral load-resisting system. However, those wall buildings in the epicentral region, such as the apartment buildings shown in Figure 18, performed well. The lateral force-resisting systems in the two apartment buildings shown in Figure 18 included moment frames along the major axis of the buildings and shear walls along the minor axis of the buildings. These two buildings were located in a residential area near Gölcük where all of the nearby moment-frame apartment buildings collapsed. It should be noted that, in the epicentral region, many moment frame buildings without shear walls collapsed because of short column effect created by small window openings as seen in the first story of these buildings.

The reconnaissance team toured a number of buildings that would be classed as dual wall-frame systems in the United States. The most significant damage observed by the team in a dual wall-frame building is shown in Figure 19. The wall and first-story exterior columns shown (Figure 19b) failed and shortened. These components displaced out of the plane of the facade as seen in Figure 19b. Another example of damage to beams and columns in a dual wall-frame building is shown in Figure 20. No cracks were observed in the shear wall, but the right end settled approximately 0.5 m due to bearing failure of the supporting soils. Although the shear wall was likely sufficiently stiff to protect the nonductile frame, the rotation at the base of the shear wall and settlement of the footings beneath the moment-frame columns contributed to the failure of the first-story columns.

Blade columns or short shear walls were often constructed near stairwells (Figure 21). These walls or blade columns were detailed similarly to regular moment-frame columns with light transverse reinforcement with 90-degree hooks and no cross ties. The failures shown are similar to those observed in moment-frame columns.

Conclusions

Much was written in the aftermath of the aftermath of the August 17, 1999, earthquake about the poor quality of residential and commercial construction in the epicentral region. The detailing and quality of much residential construction, perhaps much of it not rigorously engineered, was poor by modern US and Turkish standards. The reconnaissance team also documented many collapses of commercial reinforced concrete moment frame construction that most probably was carefully engineered. The primary reason for most of these collapses was not due to poor construction quality but rather the continued use of nonductile detailing in regions exposed to earthquake shaking. Shear reinforcement was lacking in most damaged columns observed by the authors. In contrast with the code design provisions, common use of 90-degree hooks at the end of column reduced the lateral strength and confinement of columns. Short columns, poor detailing in beam-column joints, strong-beam weak-columns, and use of inconsistent unreinforced infill walls were among other reasons for the widespread destruction in the region affected by the earthquake. For the most part, buildings with blade columns or shear walls survived with limited or no damage.

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Table 1 Observed and code specified column detailing and dimensions

	ACI 318-95	Turkish 1975		**Turkish 1997**	**Observed**
		Ductile Turkish 1975	**Non-ductile TS500-1985**		
min width (W)	= 300 mm = 0.4*D	= 250 mm = H/20 = D/3	= 250 mm	= 250 mm	typical: 250 mm common: 200 mm
min depth (D)	= 300 mm	= 250 mm	= 250 mm	= 300 mm	typical: 400-600 mm
? (longit.)	0.01 = ? = 0.06	0.01 = ? = 0.04	0.008= ? =0.04	0.01 = ? = 0.04	typical: 0.01 to 0.02
column middle zone: tie spacing hooks	= 150 mm = 6*$(d_b)_{long.}$ 135^o hooks	s_2 = 200 mm s_2 = 12*$(d_b)_{long.}$ s_2 = D/2 90^o or 135^o hooks	= 200 mm = 12*$(d_b)_{long.}$	s_u = 200 mm s_u = W/2 135^o hooks	typical: 150 to 250 mm all 90^o hooks no 135^o hooks
column end zones: tie spacing hooks	= 100 mm = W/4 135^o hooks	s_1 = 100 mm s_1 = 50 mm 135^o hooks	135^o hooks	s_c = 100 mm s_c = 50 mm s_c = W/3 135^o hooks	typical: 150 to 250 mm 100 mm or less not observed all 90^o hooks no 135^o hooks

W: minimum member dimension (width), D: longer member dimension (depth), H: column height,
?: longitudinal reinforcement ratio, $(d_b)_{long.}$: longitudinal bar diameter,
s_1, s_2, s_c, s_u : shown in Figure 13