3 Reinforced Concrete Frame and Wall Buildings

3.1 Introduction

One of the missions of the Pacific Earthquake Engineering Research (PEER) Center is to develop procedures and guidelines for performance-based earthquake engineering. The damage resulting from the Izmit earthquake gave the reconnaissance team a unique opportunity to study response limit states for selected buildings impacted by the earthquake. This chapter summarizes the key observations of the reconnaissance team regarding residential and commercial reinforced concrete construction in Turkey. Construction practices are described and the responses of moment-resisting frames and the behavior of shear walls are summarized. In the epicentral region, the two most widely used framing systems for residential and commercial construction are reinforced concrete moment-resisting frames and shear walls, and most of the loss of life and damage in the Izmit earthquake was a result of the poor performance of reinforced concrete buildings.

3.2 Construction Practice

The quality of the construction of residential and commercial buildings in the epicentral region varied widely. Both individuals and registered contractors undertake building construction work. Commercial construction is typically built by registered contractors and was generally of better quality than residential construction. Anecdotal evidence suggests that the level of inspection by regulatory officials of building construction work undertaken by registered contractors and individuals was limited prior to the earthquake.

The reconnaissance team found evidence of both extremely good and extremely poor commercial construction. One example of excellent quality construction is the building of Figure 3-1 that was located in central Yalova. Residential construction quality also ranged from excellent to poor. Although construction work by registered contractors was generally of better quality than that by individuals and homeowners, the quality of contractor-completed construction was often poor by U.S. standards. Figure 3-2 shows a completed shear wall in a multistory apartment building in Yalova. The vertical and horizontal rebar in the shear wall can be seen on the exterior face of the wall. Although buildings in Yalova suffered damage in the earthquake, the apartment building was not damaged despite the poor quality of the concrete evident in the photograph.

Figure 3-1 High quality apartment construction, Yalova

An example of residential apartment construction by a homeowner in a village outside of Gölcük is shown in Figure 3-3. The shoring is composed of cut and trimmed tree limbs. (In most instances, conventional steel and timber shoring is used for apartment construction over four stories.) Timber planks are used to form columns, beams, and slabs. Photographs of the second floor of the apartment building are presented in Figure 3-4. Slabs typically span in one-direction and are approximately 100 mm in thickness. Beams span 2 m to 4 m, range in depth up to 500 mm, and are typically 200 mm wide. Bent-up rebar can be seen in Figure 3-4. Transverse ties with 90° hooks are used. The beam rebar details are nonductile. 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 3-4. 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. Transverse reinforcement with 90° degree hooks is typical. Joint shear reinforcement is not provided. The column rebar details are nonductile.

Figure 3-2 Poor quality construction of a shear wall in an apartment building in Yalova

Figure 3-3 Homeowner apartment building construction

Smooth rebar is commonly used for reinforced concrete construction in the epicentral region. The yield strength of such rebar is approximately 275 MPa. Smooth rebar is used because it is less expensive and more readily available than deformed rebar, and is also easier to bend and cut on site. The strength and quality of the concrete varied widely as noted above. Concrete is typically batched on site for low-rise residential and commercial construction, and standard quality control procedures such as slump tests are rarely used. Low-strength concrete was identified in a number of damaged buildings visited by the reconnaissance team. Some samples were weak enough to crush by hand.

The reconnaissance team was surprised by the volume and type of residential construction. Specifically, there was much unoccupied new residential construction, and there were many incomplete single- and two-story additions to existing construction. Local experts explained that homeowners often added stories to existing apartments or constructed new multistory apartments as a hedge against inflation. The quality of such construction was often poor; it is highly likely that much of this construction was neither engineered nor approved by the local jurisdiction.

Figure 3-4 Typical gravity framing including beam and column details

3.3 Moment-Resisting Frame Construction

3.3.1 Typical Framing Systems for Residential Construction

Residential buildings in the epicentral region typically range in height between two and seven stories. Sample two- and four-story buildings are shown in Figures 3-5a and 3-5b, respectively. Because federal agencies limit the building first-story footprint-to-plot ratio, cantilever construction in the form of beams or asmolen floor framing (see Section 3.3.2) is often employed at the second-floor level to maximize the gross floor area of the building. Cantilever construction can be seen in both buildings of Figure 3-5.

Figure 3-6 presents photographs of a three-story building that was under construction at the time of the earthquake. A plan of the second-floor framing is shown in Figure 3-7. The column and beam orientation shown in Figure 3-7 suggests that the framing system is much stiffer and stronger in the direction perpendicular to the street (parallel to the *y*-axis of Figure 3-7), assuming that similar rebar are used in all beams and columns in this building. The ratio of total column area to plan footprint in this building is 1.3%.

a. Two-story building

b. Four-story building

Figure 3-5 Typical framing systems in epicenter region

A plan of the roof framing of a five-story moment-resisting frame building is shown in Figure 3-8. The column orientations and locations are such that there are no moment-resisting frames of more than one bay in either direction of the building in the fifth story. Such framing likely possesses limited strength and stiffness that, if coupled with nonductile reinforcement details, results in a vulnerable building in the event of earthquake shaking.

a. north-south elevation

b. east-west elevation showing fault rupture

Figure 3-6 Three-story moment-resisting frame east of Gölcük

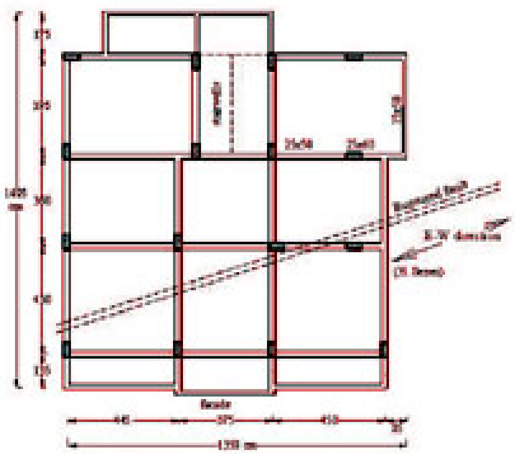


Figure 3-7 Floor plan for the three-story building shown in Figure 3-6

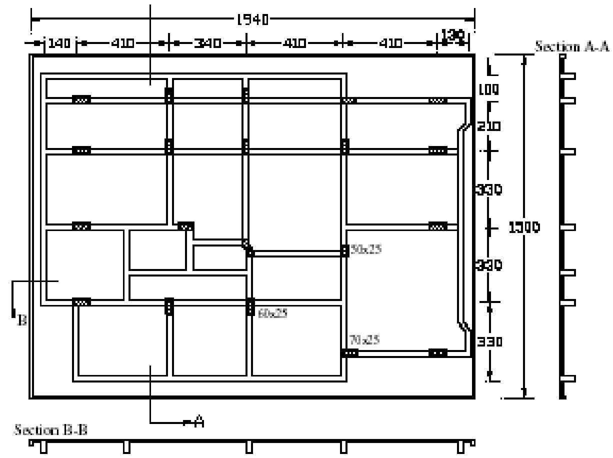


Figure 3-8 Plan of roof framing for a five story apartment

3.3.2 Typical Construction Details

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. Most columns in such construction are blade columns with an aspect ratio of approximately 3. Column plan dimensions range between 150 mm x 500 mm to 250 mm x 800 mm. 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° hooks. The spacing of transverse ties is typically 200 to 250 mm along the clear height of the column.

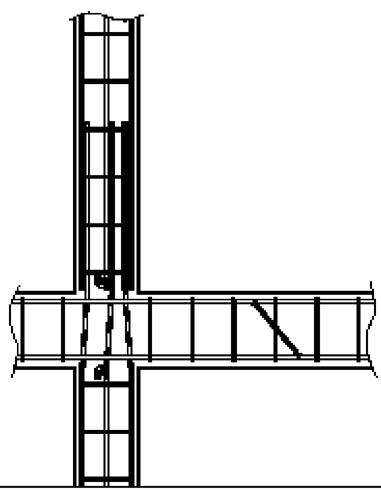


Figure 3-9 Typical modern beam and column rebar details

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° hooks. The spacing of transverse ties is typically 200 to 250 mm along the clear length of the beam. 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, do not resist shear force if the loads are reversed due to earthquake shaking.

Information on column and beam rebar details is provided in Figure 3-9. 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° hooks. No transverse reinforcement for the purpose of confinement is provided in the hinge, joint, or splice regions. Bent-up beam rebar is shown in the typical section.

Figure 3-10 Hollow clay tile block used for infills

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. Photographs of hollow clay tile block are shown in Figure 3-10.

Common slab systems include the one-way and asmolen slabs. One-way slabs range in thickness from 80 mm to 120 mm, and span distances up to 4 m. For longer spans and heavy loadings, the one-way, or asmolen, joist system is typically used. This system is composed of one-way joists that are formed by hollow clay tile blocks; the slab between the joists is cast directly atop the blocks. The thickness of the asmolen slab is usually 300 mm (200 mm block and 100 mm slab) or 320 mm (250 mm block and 70 mm slab). Figure 3-11 shows the underside of three asmolen floor systems.

Figure 3-11 Asmolen floor system in a four-story building

3.4 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 apartment buildings were hard hit, although many had been constructed in the past 20 years. 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 second stories, the use of nonductile details, and poor quality construction.

Figure 3-12 shows the collapse of six moment-resisting frame buildings in a village on the outskirts of Gölcük. Every moment-resisting frame building on this street collapsed, and 122 people died in these buildings. Nonductile details were observed in every (collapsed) moment-frame building on this street.

Figure 3-12 Collapse of moment frame buildings, Gölcük

Figure 3-13 Collapse of moment-frame and wall buildings, Adapazari

Similar collapses were common in the epicentral region. Figure 3-13 shows the extent of the destruction in Adapazari, a major city approximately 10 km from the line of rupture. Many of these 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, glass panels and not hollow clay tile infill walls 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.

3.4.1 Moment-Frame Buildings Straddling or Adjacent to Line of Rupture

Not all moment-frame buildings that straddled the line of rupture collapsed. The fault ran directly beneath the building shown in Figure 3-6, but this building suffered only modest damage despite approximately 1 m of horizontal offset beneath the foundation. The lack of damage can be attributed to the stiffness of the building's 1-m-deep raft foundation. Three hundred meters from the building of Figure 3-6, a moment-frame school building that straddled the fault collapsed completely (Figure 3-14). Figure 3-15 shows collapsed and damaged moment-frame buildings on the Gölcük naval base. The horizontal offset of the fault beneath these buildings was on the order of 4 m. Fault rupture beneath a five-story building (Figure 3-16) located approximately 200 m east of the naval base caused a partial collapse.

Figure 3-14 Collapsed school building that straddled the line of rupture

Figure 3-15 Collapsed and damaged moment-frame buildings on the Gölcük navy base

a. Collapsed building

b. View of failed first story

Figure 3-16 Collapsed building that straddled the line of rupture

"Near-field" is the common term used to define the zone within 5 to 10 km of a major fault. Earthquake shaking in the near field is often severe, as illustrated by recorded ground motions obtained from the Northridge (1994) and Kobe (1995) earthquakes. Widespread collapse of older construction is to be expected in the near field due to the intensity of the earthquake shaking. Although many moment-frame buildings in the near field of the Izmit earthquake suffered gross damage or collapsed, some fared surprisingly well. Shown in Figure 3-17, the four-story moment-frame building is sited within 2 m of the line of rupture and yet suffered no visible damage despite 1.2 m of horizontal offset and 2.35 m of vertical offset on the fault. Figure 1-2 shows another view of the same building. The reasons for such good performance of this 10- to 30-year-old frame building with masonry infill are unknown.

Figure 3-17 Undamaged building located within 2 m of the line of rupture

3.4.2 Variability of Moment-Frame Building Response

Figure 3-18 is a photograph of two six-story nonductile moment-frame buildings in Gölcük. One of the buildings collapsed completely, whereas the building immediately adjacent 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 similar plan footprints and common construction details. It is likely that the two buildings were nearly identical and that the same contractor constructed both buildings. 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 limit states 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 substantial difference in performance.

a. lightly damaged six-story building

b. collapsed six-story building

Figure 3-18 Variability of building response

3.4.3 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. As noted in Section 3.3.2, these walls are 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. Figures 3-19a and 3-19b show complete and partial damage to hollow clay tile walls in four- and thirteen-story buildings, respectively. The four-story building was under construction at the time of the earthquake; the thirteen-story building was constructed in the early 1970s.

a. four-story building

b. thirteen-story building

Figure 3-19 Varying degrees of damage to infill masonry walls

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 3-20 illustrates the distribution of damage to infill walls in two buildings, one near Gölcük, and one in Degirmendere. In these buildings, the lateral stiffness of the masonry infill walls is likely of the same order or greater than that of the moment-frames. For these buildings not to collapse following the failure of the infill walls the moment-frames must have possessed significant strength and some limited ductility.

a. infill wall damage in Gölcük **b. infill wall damage in Degirmendere**

Figure 3-20 Damage to infill masonry walls

Figure 3-21 shows two views of a collapsed apartment building in Gölcük. The first two stories of this building failed completely but damage in the upper three stories was limited. The long infill walls in the upper three 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 three 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.

a. view of front face of building

b. view of infill wall perpendicular to sidewalk

Figure 3-21 Collapsed apartment building in Gölcük

The first two stories of the building in Figure 3-22 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 which the moment-frame is both flexible and weak by comparison with the upper stories (Figure 3-23). 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 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.

Figure 3-22 Failure of two stories of moment-frame building with infill masonry

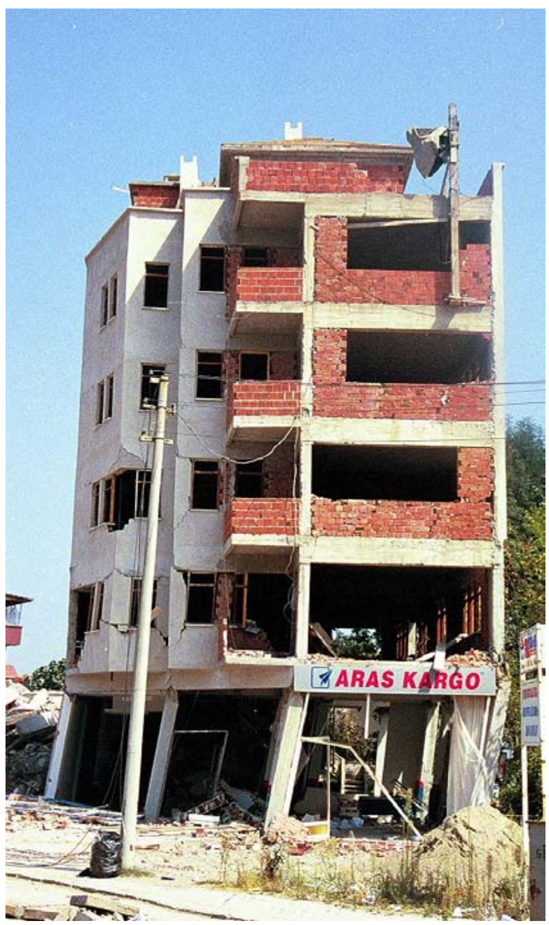


Figure 3-23 Formation of a soft and weak story

3.5 Response of Moment-Frame Components

Previous sections of this discussion on moment-frame construction have focused on the response of moment-frame systems. This section of the report addresses the response of the components of moment-frames, namely, beams, columns, beam-column joints, and slabs.

3.5.1 Beams

The reconnaissance team observed little damage to interior moment-frame beams because columns were generally weaker than beams. One type of beam damage is shown in Figure 3-24. 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 beam-column joints.

Figure 3-24 Damage to a nonductile reinforced concrete beam

Figure 3-25 Failure of lap splices in a moment frame conncetion

3.5.2 Columns

The majority of moment-frame component failures were in columns and were due to (a) the use of nonductile details and unconfined lap splices, (b) excessive beam strength, and (c) 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 photograph of the exposed lap splice of Figure 3-25 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° hooks were used in this column; no cross ties were present. The 90° hooks on the ties opened during the earthquake, and the limited strength and confinement afforded by the ties were lost.

Figure 3 –26 Typical transverse reinforcement details in columns

a. view of damaged blade column
b. transverse tie details in blade column

Shear reinforcement was lacking in most damaged columns observed by the reconnaissance teams. The transverse tie details of Figures 3-26, 3-27, and 3-28 were common, namely smooth rebar, widely and often unequally spaced ties (200 to 250 mm), and 90° hooks. The wide spacing of the ties resulted in shear failures (Figure 3-26), buckling of longitudinal rebar (Figure 3-27), and poor confinement of the core concrete (Figure 3-28).

Figure 3-27 Shear failure of a moment-frame blade column

Figure 3-28 Lack of transverse reinforcement in moment-frame column

Shear failures in short columns were common. A typical example of such a failure is a school building in Adapazari, shown in Figure 3-29. The damaged column of Figure 3-29b is shown in Figure 3-29a in a blue circle.

Figure 3-29 Shear cracking in short columns

a. building elevation

b. diagonal cracking in column

Column-infill masonry wall interaction resulted in severe damage to and failure of many moment-frame columns. Consider the building east of Gölcük that is shown in part in Figure 3-30. The infill hollow clay tile masonry on each side of the central column in this figure failed completely and the column hinged at each end. The column to the left of the central column was captured approximately 1 m above the floor by the residual hollow clay tile. The shear cracks that were observed in this captive column formed in the column at the top face of the remaining infill masonry.

Concentrated damage at the ends of moment-frame columns was observed throughout the epicentral region. Examples of such damage are presented in Figures 3-31, 3-32, and 3-33. Large rotations at the ends of the columns (Figure 3-31) produced severe cracking and loss of concrete. (Note the relative proportions of the columns and the beam in this figure.) Out-of-plane deformations in the column of Figure 3-32a led to loss of cover concrete in the hinging zone. The transverse ties in this column were widely spaced (200 mm) and composed of smooth rebar with 90° hooks. Such connection details have limited rotation capacity. The beam and slab framing (Figure 3-32b) lost seating on the column. The first-story columns of a collapsed building are shown in Figure 3-33; the second-story of the building is to the left of the columns. Note the smooth failure surface on the top right side of the columns.

Figure 3-31 Concentrated damage at ends of moment-frame columns due to excessive drift

a. damage from out-of-plane deformation

b. unseating of beam-slab system from column

Figure 3-32 Damage and failures at ends of moment-frame columns

3.5.3 Beam-Column Joints

Typical damage to beam-column joints is shown in Figures 3-34, 3-35, and 3-36. The collapse of a building in Adapazari (Figure 3-35) was due to the failure of beam-column joints. Much of the framing (Figure 3-35a) is essentially intact but many of the beam-column joints are heavily damaged. One of the damaged joints is shown in Figure 3-35b. Beam rebar anchorage in the joint is inadequate and no transverse ties are present in the joint.

Figure 3-36 is a photograph taken in a building under construction in Adapazari at the time of the earthquake. Severe damage in the beam-column joints is evident, but horizontal transverse ties in the joints maintained the integrity of the joints. (See Section 3.7.3 for a more complete description of this building.)

Figure 3-33 Damage to moment-frame columns

Figure 3-34 a. Damage to moment-frame beam-column joints

Figure 3-34 b. Damage to moment-frame beam-column joints, reinforcement in joint

Figure 3-35 a. Building collapse due to failure of beam-column joints

Figure 3-35 b. Damage to one beam column joint

Figure 3-36 is a photograph taken in a building under construction in Adapazari at the time of the earthquake. Severe damage in the beam-column joints is evident, but horizontal transverse ties in the joints maintained the integrity of the joints. (See Section 3.7.3 for a more complete description of this building.)

Figure 3-36 Damage to a new moment-frame beam-column joint

3.5.4 Asmolen Slab Floor System

The asmolen slab floor system, described in Section 3.3.2, is commonly used in the epicentral region. In this type of construction, the hollow clay tile, which is used as permanent formwork, is not positively attached to the slab or the joist-beam framing.

Figure 3-37 Typical damage to asmolen floor systems

Damage to such systems was widespread. Figure 3-37 shows typical damage to the asmolen floor system. Deformation of the joist-beam framing led to sections of the hollow clay tile formwork dislodging and falling to the floor below. Although failure of the hollow clay tile blocks in the floor system is not considered structural damage, falling tile blocks constitute a hazard to life.

3.6 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 deformations of the nonductile framing system in the elastic range. Story collapses were not observed in buildings containing a substantial number of shear walls, but it should be noted that shear walls were not widely used in the epicentral region.

3.6.1 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 building under construction in Figure 3-38 and the apartment buildings of Figure 3-39, performed well. Shear walls were used as the lateral load-resisting system in both the transverse and longitudinal directions of the building in Figure 3-38. This building suffered minor damage to the infill walls. The lateral force-resisting systems in the two apartment buildings shown in Figure 3-39 included moment-frames along the major axis of the buildings and shear walls along the minor axis. These buildings were located in a residential area near Gölcük where all of the nearby moment-frame apartment buildings collapsed (see Figure 3-12).

Figure 3-38 Shear wall building under construction at the time of the earthquake

Figure 3-39 Undamaged apartment building in Gölcük

The building shown in Figure 3-40 experienced damage at the stiffness discontinuity in the shear wall. The fault ruptured directly beneath this building. The limited damage in this instance constitutes excellent performance.

Figure 3-40 a. Shear wall building damaged due to fault rupture

Figure 3-40 b. Close up of damage to a shear wall

The reconnaissance team toured a number of buildings that would be classed as dual wall-frame systems in the United States. However, because design provisions for such systems did not exist in Turkey prior to 1997, these buildings would have been designed as either shear walls or moment-resisting frames. The most significant damage observed by the team in a dual wall-frame building is shown in Figure 3-41a. The wall and first-story exterior columns shown (Figure 3- 41b) failed and shortened. These components displaced out of the plane of the wall, as seen in Figure 3-41b.

Figure 3-41 a. Collapse of dual wall-frame five story building, Adapazari

Figure 3-41 b. Close-up of failure of the shear wall and perimeter columns

Figure 3-42 a. Damaged wall-frame building due to ground failure and wall rotation

Figure 3-42 b. shear wall settlement

Another example of damage to beams and columns in a dual wall-frame building is shown in Figure 3-42. No cracks were observed in the shear wall, but the right end of the wall settled approximately 0.5 m (Figure 3-42b) 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 3-43). These walls or blade columns were detailed similarly to regular moment-frame columns with light transverse reinforcement with 90° hooks and no cross ties. The failures shown are similar to those observed in moment-frame columns.

Figure 3-43 Damage to short wall / blade column

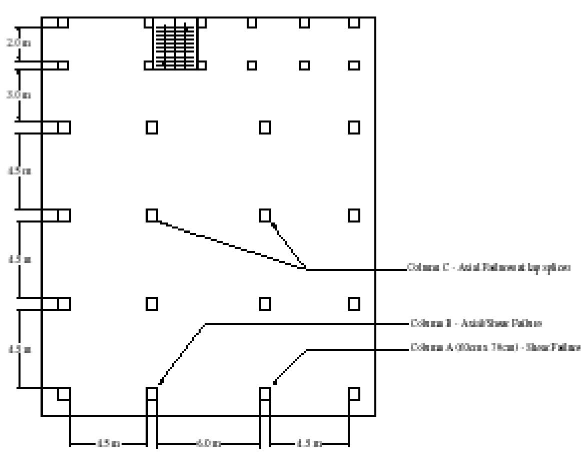
3.7 Performance of Selected Buildings

Modern standards for the seismic evaluation of buildings (FEMA 1997) dictate decisions regarding system response using information on component response. For the system performance level of collapse prevention, system failure is linked to the first failure of a component (typically measured in terms of deformation demands or demand-capacity ratios). Such correlation of system and component response is misleading and often overly conservative if the seismic and gravity load-resisting systems are redundant. One objective of the reconnaissance team was to gather information related to limiting states of response of building systems, with an emphasis on the limit state of collapse prevention. The following sections describe in some detail the performance of four buildings: A through D. The first three buildings (A, B, and C) sustained severe damage to critical components but did not collapse. The fourth building (D) performed poorly but as a result of ground failure.

3.7.1 Building A

Figure 3-44 Front elevation of Building A

Figure 3-45 Rear elevation of Building A showing intact infill masonry walls



3.7.1.1 Description

Building A, shown in Figures 3-44 and 3-45, was located at the eastern outskirts of Gölcük. Much of the first story of this moment-frame building (not seen in Figure 3-44) was located below grade. The grade level sloped down from the front to the back of the building. A sketch of the first-floor plan of the building is shown in Figure 3-46. Most of the hollow clay tile infill masonry failed during the earthquake but some remained intact at the rear of the building in the sixth story (see Figure 3-45).

3.7.1.2 Component Failures

Structural damage was concentrated in the first-story columns at the front of the building Figure 3-47) and around the stairwell at the rear of the building (Figure 3-48). Nonductile detailing was evident in each damaged component viewed by the reconnaissance team.

Figure 3-47 Damage to first story columns

Figure 3-48 Damage at the rear stairwell

The staircases in the rear stairwell were cast integrally with the exterior columns. The landings were located approximately 1 m below the beam-column joints (Figure 3-48). No transverse reinforcement was present in these joints. The lateral support provided by the landings and the staircases resulted in short column construction and led to shear failures immediately above the landings. Figure 3-48 shows severe damage to the staircases that suggests that the staircases resisted significant lateral forces during the earthquake via strut action. The lateral stiffness of the staircases is evinced by the damage they suffered but likely was not included in the earthquake analysis of the building (which is also common practice in the United States).

The distribution of damage to columns in the first story is shown in Figure 3-46. Figures 3-49, 3-50, and 3-51 show column failures. Nonductile detailing is evident, including widely spaced perimeter transverse ties with 90° hooks and no cross ties, and lap splices located at the floor level with no confining transverse reinforcement.

Figure 3-49 Shear failure of Column A (See Figure 3-46)

Figure 3-50 Axial failure of column B (See Figure 3-46)

Figure 3-51 Axial failure at lap splice in Column C (See Figure 3-46)

3.7.1.3 System Response

A comprehensive performance-based evaluation methodology should be able to predict distributions of damage similar to that identified above assuming an accurate characterization of earthquake shaking. The performance of Building A brings into question the procedures currently adopted in the United States for system evaluation for the performance level of collapse prevention. (In this report, “collapse” is defined in terms of the failure of the gravity load-resisting system.)

As shown in Figures 3-46 and 3-47, the first and third rows of columns were badly damaged but the second row of columns suffered no significant damage. All columns in the first three rows were the same size; rebar in the first and third rows of columns were essentially identical. If the interior columns in the first row failed initially, conventional approaches would suggest that lateral forces were redistributed to other stiff components (including the second row of columns) and gravity loads were transferred to the undamaged columns in the first and second rows. The increase in the gravity and earthquake effects should have been greater on the second-row columns than on the third-row columns, yet the columns in the third row failed and the columns in the second row were undamaged. New knowledge regarding the transfer of lateral loads and gravity from failed components to other components of a building frame is needed to obtain accurate estimates of building performance.

Although several columns in the first story of the building failed in shear and axial compression, the building did not collapse. Clearly system response cannot be judged on the basis of the most highly loaded (forces or deformations) component in the building, as is the practice in FEMA 273, *NEHRP Guidelines for the Seismic Rehabilitation of Buildings* (FEMA 1997). The gravity load-resisting system of the building did not collapse for a number of reasons that include (a) frame action in the stories above the damaged columns and (b) residual axial-load capacity in the heavily damaged columns.

After the columns in the first row failed in shear and shortened, the slab and beam framing deflected in the shape of a catenary (see the sag in the floor slabs in Figure 3-44) and gravity loads were carried to the adjacent undamaged columns by axial tension in the beams and slabs. Vierendeel truss action in the upper stories also likely transferred gravity loads to adjacent undamaged columns. Provision for such redundancy in framing systems would reduce the likelihood of building collapse and substantially uncouple system-level response from component-level response. The catenary and Vierendeel truss mechanisms may be very effective in stabilizing the structure when interior columns are lost. To ensure that beams and slabs are able to maintain catenary deflections, bottom reinforcement should be continuous through any columns that may fail under lateral loads.

Recent studies (Moehle et al. 2000) have shown that columns heavily damaged in shear are still capable of supporting axial loads. Residual axial strength in these columns would reduce the need to redistribute gravity loads as described in the previous paragraph. The failed columns in the first row were squat so that after failure in shear, the upper segments of the columns bore on the lower segments, albeit not concentrically. (Contrast this behavior with that described earlier for narrow columns; see Figure 3-32). The core concrete in the failed columns in the third row continued to carry gravity loads after the earthquake because the cores of the columns remained partially intact. The use of transverse reinforcement in the amount needed to keep the core of a column intact at large deformation would further reduce the likelihood of building collapse.

3.7.2 Building B

3.7.2.1 Building Description

Building B (Figure 3-52) was located 5 km east of Gölcük. The six-story reinforced concrete frame building was unoccupied at the time of the earthquake. The footprint of the building was approximately 12 m by 16 m.

Figure 3-52 Elevation of Building B

3.7.2.2 Component Failures

A corner column in the third story of Building B failed during the earthquake, as seen in Figure 3-52. Part of the roof framing of the adjacent building can be seen immediately below the failed column, indicating that the third-story column failed due to the impact of the adjacent building. Figure 3-53 shows more details of the failed column and the roof slab of the adjacent building.

Figure 3-53 Details of damage to third-story column of Building B

3.7.2.3 System Response

Although one column in the third story of the building was completely destroyed due to impact of the roof slab of the adjacent building, the building did not collapse. The gravity load-resisting system in the building did not collapse because of frame action in the stories above the destroyed column. The performance of Building B also raises a number of questions once again about the procedures currently adopted in the United States for system evaluation for the performance level of collapse prevention. The performance of Building B raises the questions: (1) should component loss be accounted for in the design of the building for the performance level of collapse prevention? and (2) If so, how?

The loss of one or more components in a moment-frame building can substantially modify the magnitude and behavior of the remaining components. Assuming that the location(s) of the failed component(s) are known, nonlinear methods of analysis can be used to evaluate the forces and deformations in the damaged building frame. Two challenges with such analysis are (1) identifying the number and locations of components to be removed from the mathematical model and (2) including the effects of column failure and load redistribution.

Procedures for selecting the number and locations of components to be removed from the mathematical model have not been developed. The number and locations will vary as a function of the earthquake histories used for analysis and evaluation. Demand-to-capacity ratios (deformations for ductile actions and forces for nonductile actions) could perhaps be used to identify combinations of components for removal from the mathematical model. Two approaches could be used to assess system response following the failure of selected components: (1) remove the components from the mathematical model before analysis and (2) remove the components from the model during the analysis when deformations or forces, or demand-capacity ratios exceed a threshold value. Approach 1 is more conservative than Approach 2. Approach 1 could be used with nonlinear static or dynamic analysis. Approach 2 would be used only with nonlinear dynamic analysis.

The rapid loss of a column or beam can lead to a dynamic amplification of the gravity loads that are transferred to adjacent components. Procedures for calculating the amplification factor are not available at this time. Studies are very recently completed at PEER by Rodgers and Mahin on steel moment-frame buildings and by Elwood and Moehle on nonductile reinforced concrete moment-frame buildings to evaluate the effect of component failure on system response.

3.7.3 Building C

3.7.3.1 Building Description

Building C, shown in Figure 3-54, was located near the Adapazari city center. The five-story reinforced concrete frame building included a high retail space in the first story. The retail space shown in Figure 3-55 included a mezzanine level on the west side of the building. The building was not occupied at the time of the earthquake.

Figure 3-54 Elevation of Building C

3.7.3.2 Component Failures

Most of the first-story columns connected to the mezzanine level failed in shear (Figure 3-55). The mezzanine level reduced the clear length of the columns, resulting in shear failures before the moment capacities of the columns could be developed. Damage to the stair framing and short shear wall connected to the mezzanine level can also be seen.

One of the failed columns at the rear of the building had a very steep shear crack (Figure 3-56) that suggested that the column was carrying high axial loads. Following a more detailed inspection of the building, the reconnaissance team concluded that the column was part of a two-story addition, which was separated from the five-story building by an expansion joint. No plausible explanation for the steep shear crack is proposed. The beam-column joints at the top of the first-story columns on the eastern façade were damaged but did not fail because transverse reinforcement was provided in the joint region; see Figure 3-57. The beam-column joint at the north-east corner of the first story was heavily damaged, as shown in Figure 3-58, but continued to carry gravity loads.

Figure 3-55 View of retail space in the first story of the Building C

Figure 3-56 Shear crack in the first story column in the rear of Building C

Figure 3-57 Damaged bean-column joints at top of first story columns, Building C

Figure 3-58 Damaged beam-column joint at top of first-story corner column, Building C.

3.7.3.3 System Response

The residual drift in the first story of the building was approximately 300 mm to the east. Although the building to the west of Building C overturned due to bearing failure of the soils beneath its 1-m-thick foundation, soil deformation and failure did not appear to contribute to the damage in Building C.

To predict incipient collapse of a building or to evaluate buildings for the performance level of collapse prevention, new information on how gravity loads are supported in buildings with severely damaged or failed components is needed. The severely damaged interior columns of Figure 3-55 could support little or no gravity load. This observation suggests that much of the gravity load in the building must have been distributed to the perimeter first-story columns by Vierendeel truss action in the upper stories. Many of these perimeter columns suffered damage to their beam-column joints, but the use of transverse reinforcement prevented joint failure and gravity load resistance was maintained.

Although the residual drift of the first story of the building adjacent to the front sidewalk was approximately 5%, the P - Δ effects did not lead to collapse of the building. Three factors probably contributed to the stability of the building. First, the shear wall near the stairwell (see Figure 3-55), although heavily damaged, likely had significant residual lateral stiffness and strength. Second, the axial loads in the columns were low as a percentage of $f_c'A_g$. Third, the residual drift at the rear of the five-story building was much less than 5% and the framing at the rear of the building may have partially stabilized the building.

3.7.4 Building D

3.7.4.1 Building Description

Building D was a six-story moment-frame building located in the center of Adapazari. An elevation of the building is shown in Figure 3-59. Based on similar construction of the same age in Adapazari, the foundation for Building D was probably a mat or raft with a thickness of approximately 1 m.

Figure 3-59 Elevation of Building D

3.7.4.2 System Response

Building D is an example of poor system performance that was not accompanied by component damage or failure. This building suffered little damage as a result of the earthquake shaking but could not be occupied because it settled more than 1 m due to liquefaction and bearing failure of the supporting soils (see Figure 3-60). It is likely that this failure of the supporting soils limited the shaking experienced by the building. Services and utilities to the building were destroyed and ingress and egress were most difficult. The poor performance of this building underscores the need to explicitly account for soil and foundation behavior in performance-based earthquake engineering. Although the building would have satisfied the performance level of collapse prevention (as defined in FEMA 273 [FEMA 1997] and Vision 2000 [SEAOC 1995]), it would not have satisfied the egress requirements of the life-safety performance level. To achieve performance beyond collapse prevention, site improvements to avoid liquefaction would have been required.

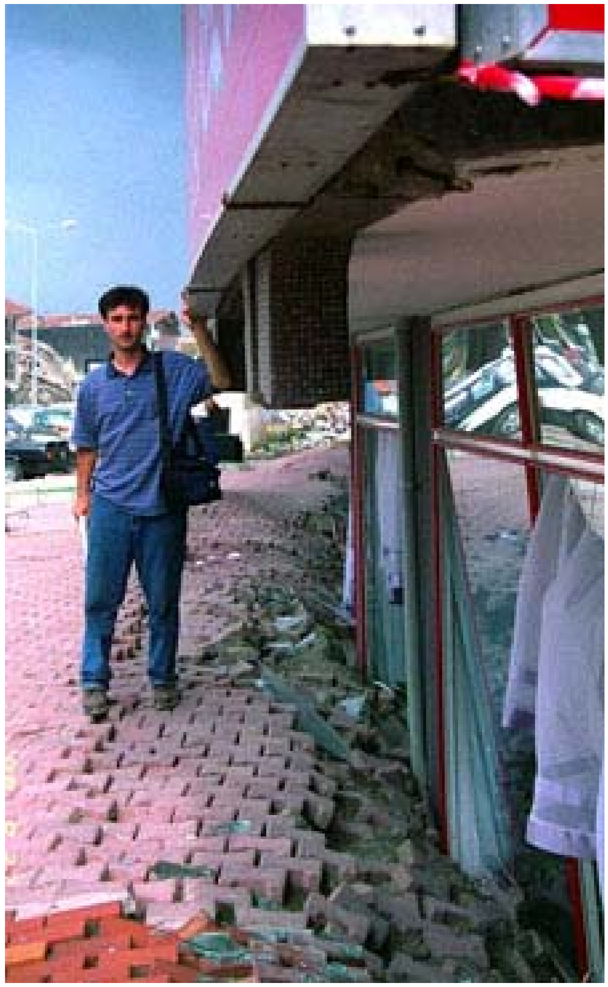


Figure 3-60 Settlement of Building D due to liquefaction and soil-bearing failure

3.7.5 Summary Remarks

The purpose of the discussion of selected buildings is to identify issues relating to building performance that must be addressed in the development of guidelines and tools for the implementation of performance-based earthquake engineering.

At the time of this writing, building (system) response is often judged on the basis of the most highly damaged component in the building. Clearly, this approach, although conservative, is neither accurate nor cost effective. Poor behavior of one or two random components does not necessarily lead to poor system behavior, although poor behavior of one or two key components may lead to system collapse if mechanisms for redistribution of gravity loads do not exist in a building.

Much remains to be learned about the collapse of buildings and the design of buildings on soils prone to liquefaction or failure. Research on the following topics is needed to improve analysis, evaluation, and design procedures to ensure with high confidence and low cost that buildings will not collapse.

1. Triggers for axial load failure of ductile and nonductile reinforced concrete columns under combined loadings based on large- or full-scale test data.

2. Mechanisms for redistribution of gravity loads in the event of component(s) failure, and characterization of gravity-load amplification effects due to component failure.

3. Analytical tools for predicting component strength and stiffness loss under combined loadings based on evaluation of large-scale experimental data.

4. Procedures for eliminating components from mathematical models to simulate component failures.

5. Large-scale 3-D earthquake simulator testing of buildings with weak and brittle components to validate the analysis, evaluation, and design procedures developed in 1 through 4 above.

Editors note: This text follows as closely as possible from the paper copy of Chapter 3 "Reinforced Concrete Frames and Wall Buildings" by H.Sezen et al. ***Structural Engineering Reconnaissance of the Kocaeli (Izmit) Turkey Earthquake of August 17 1999****. Berkeley: Pacific Earthquake Engineering Research Center, (PEER Report 2000-09), December 2000. A few figures have been substituted where color originals did not exist. Minor text and layout modifications were made. Higher resolution images of all images are available through EQIIS image database http://nisee.berkeley.edu/eqiis.html by searching under the* "Izmit (Kocaeli), Turkey earthquake, Aug. 17, 1999"
C. James, 2001