Paper Presented at

CUREe Open Symposium Sponsored by Kajima
October 1999 Tokyo Japan

Los Angeles Tall Buildings Conference
May 2000

BUILDING PERFORMANCE IN THE 17 AUGUST 1999 İZMIT (KOCAELİ), TURKEY EARTHQUAKE

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ABSTRACT

At 3:02 am on 17 August 1999, a magnitude M_w 7.4 earthquake occurred along the North Anatolian fault in the province of Kocaeli. The epicenter was located southwest of the city of İzmit, approximately 75 km southeast of Istanbul. The region affected is densely populated and includes the industrial heartland of Turkey. Approximately 120,000 residential buildings were heavily damaged or collapsed, and 15,000 deaths were reported. Monetary loss estimates range from 10 to 40 billion USD, or approximately 10 to 15% of the GNP of Turkey. Preliminary observations of structural damage are reported.

SEISMOLOGICAL, GROUND MOTIONS, AND CODE BACKGROUND

The North Anatolian Fault Zone and Observed Fault Rupture

The earthquake was produced by rupture along a branch of the 1300 km-long North Anatolian Fault (NAF) system. The right-lateral strike-slip NAF has been very active in recent years, with seven earthquakes since 1939 exceeding M_S 7.0 (Fig. 1). The fault rupture between 1939 and 1999 has generally progressed from west to east (Fig. 1). Studies prior to the 17 August 1999 earthquake estimated that there was a 12% in 30-year probability of a large earthquake near İzmit (Stein, Barka, and Dieterich, 1997). During the 17 August 1999 earthquake, surface faulting extended 110 km east of Gölcük to nearly Düzce. The distribution of aftershocks suggests that faulting extended west of Gölcük towards Yalova (Fig. 2) for another 50 to 60 km, for a total length of rupture of 150 to 200 km. Ground motion records as well as observations by residents indicate that the fault may have ruptured in stages.

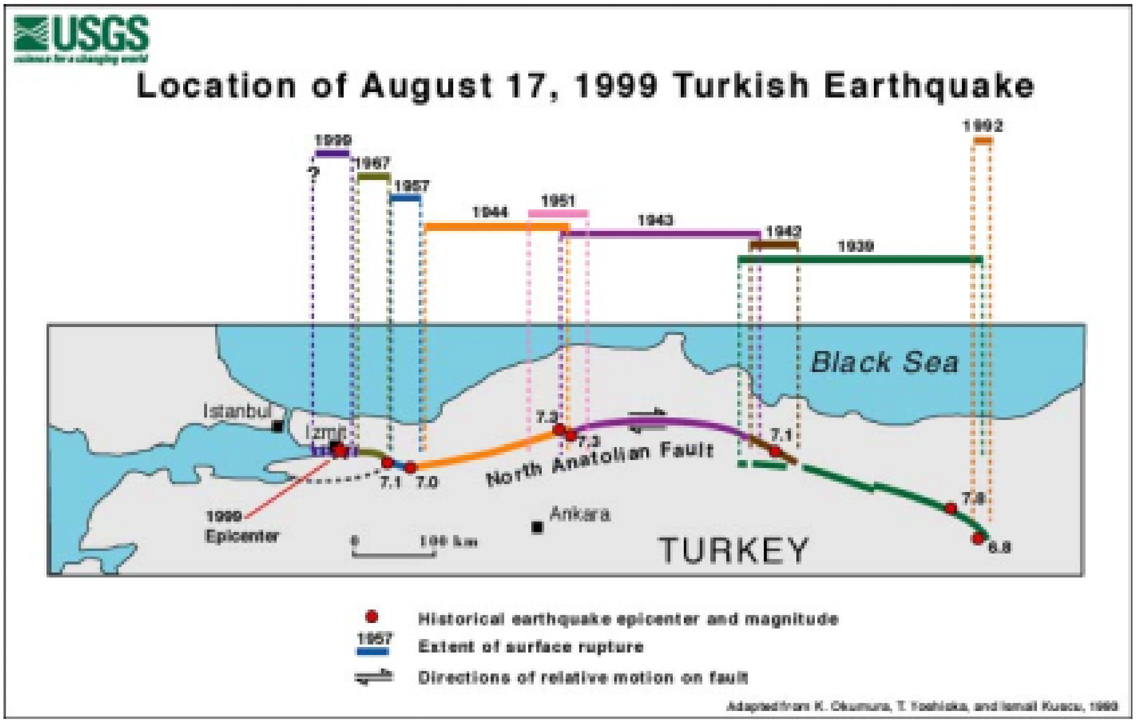


Fig. 1 North Anatolian Fault Activity: 1939 to 1999

The fault offset is predominantly right-lateral strike slip with offsets generally in the range of 2 to 4 meters (Fig. 3). A short dip-slip segment of the fault east Gölcük exhibited vertical offsets of 2 (Fig. 3), inundating the coastal area of Gölcük.

Engineering Characteristics of the Measured Earthquake Ground Motions

Peak ground accelerations measured at 13 stations (Fig. 4) varied between 0.04g to 0.41g. The measured ground motions at the YPT station are plotted in Fig. 5. Acceleration response spectra for the east-west and north-south components of the ground motion records obtained at five stations are plotted in Fig. 6. Spectral amplitudes for the east-west (fault parallel) direction plotted in Fig. 6 are approximately equal to those for the north-south direction (not plotted). Equivalent code spectra for the Turkish Standards (1975 and 1996) and the UBC (1994 and 1997) are also plotted on Fig. 6. The unreduced code spectra for the Turkish Code and the UBC are not substantially different. The relations plotted in Fig. 6 reveal that the spectral demands for the 17 August 1999 earthquake are generally less than those for the unreduced code spectra. In addition, the design level forces for the 1975 Turkish Standard are somewhat less than those for the other cases plotted (no near-field factors or redundancy factor were considered for UBC 97). For buildings with adequate detailing and designed for the code required forces, acceptable performance would be expected (collapse prevention). It is also noted that the 1996 Turkish code uses a variable force reduction factor for low periods.

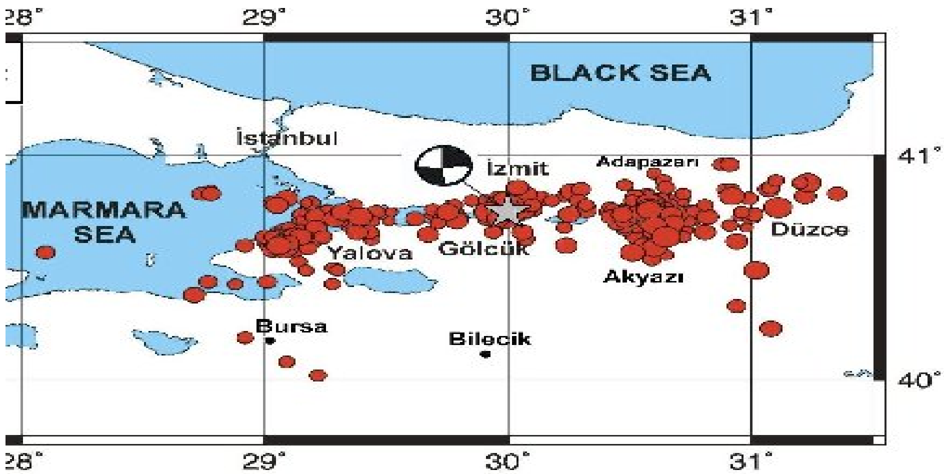


Fig. 2 Aftershocks between 17 August and 9 October 1999 (Kandilli, 1999)

Turkish Code Requirements for Detailing of RC Buildings

The 1975 Turkish Code for reinforced concrete contains detailing provisions and design concepts that are consistent the 1976 version of the UBC. For example, column hoops and crossties with 135-degree hooks, reduced spacing of beam and column transverse reinforcement at member ends, and transverse reinforcement within the joint are required. The maximum horizontal joint shear stress of approximately $0.8\sqrt{f'c}$ MPa is specified; however, a strong-column, weak-beam design is not required. Since the region affected by the earthquake lies within the highest seismic zone in Turkey, the observed performance is not consistent with expectations based on code requirements. Improved details are contained in the 1996 code. Observed damage indicates that the code provisions are not followed and that improvements in construction practice (as well as seismic retrofit) are needed if improved performance is to be achieved in future earthquakes.

Construction Practice and Materials

A majority of the residential construction in Turkey consists of three to seven story reinforced concrete frames with hollow clay tile infill walls. Rectangular column cross-sections (25 cm by 60 cm) with 12-15 mm diameter smooth vertical reinforcing bars are common ($\rho = 0.014$). In some of the newer buildings, square columns (40 cm by 40 cm) with 8 vertical 15 mm diameter deformed reinforcing bars are used ($\rho = 0.009$), typically for corner columns. Column splices are usually located just above the floor slab, consisting of a straight bar extension from below and a hooked bar from above. Transverse reinforcement typically consists of 6 or 8 mm smooth reinforcement spaced at 20 to 25 cm on center. Based on observations from damaged buildings by the reconnaissance team, crossties and joint transverse reinforcement are rare. In one building where joint transverse reinforcement was observed, it did not appear to be used consistently throughout the joints in the building. Concrete quality appeared to vary widely and in many cases the concrete was poorly consolidated.

Hollow clay tiles are used predominantly for infill walls. The use of lightweight white “foam” block has become more common in some newer buildings. In general, the infill walls are placed


(a) Vertical Scarp East of Gölcük (3m) (b) Horizontal Slip West of Adapazarı (4 m)

Fig. 3 Surface Faulting

directly against the narrow side of the column so that the column is contained within the partition wall. This practice results in an irregular layout and orientation of columns; however, a majority of columns are commonly oriented with the long side parallel to the sides of the building. It was common to see about 75% of the columns with the same orientation in residential buildings, whereas a uniform orientation was observed in some commercial/residential construction along main streets and in buildings with parking on the ground level. Therefore, the lateral strength of most buildings is much less for loads creating moments about the weak axis for the majority of the columns in a building.

Use of reinforced concrete walls was observed in some of the more recent construction in Adapazarı and Yalova. In general, the wall cross-sections are quite small (25 cm by 1.2 m) and only light vertical reinforcement is used (22 – 15 mm deformed bars). Horizontal web reinforcement is typically smooth 8-mm bars at 20-cm spacing with 90-degree hooks at each end.

Apparently due to inflation, many buildings in Turkey are constructed piecemeal over an extended period as funds become available. For example, over a period of several years, the foundation is built, reinforcing steel is purchased, columns and floors are constructed, infill walls are placed, and eventually the building is completed. For smaller residential buildings, it appeared few, if any, restrictions are placed on contractors and that the quality of the construction varied widely. As might be expected, larger industrial facilities and high-end commercial buildings (e.g., banks) were generally better engineered and constructed.

OBSERVED PERFORMANCE OF BUILDINGS

A majority of the collapsed buildings were three to five story residential units. First story collapses due to poor structural configuration (soft-story) were common. The soft first-story level was typically created due to a taller first-story height and/or a lack of infill walls to provide for an open commercial area or for parking. Typical examples are given in Figure 7. As noted earlier, the hollow clay tile infill walls are commonly placed within the short column direction; therefore, column orientation and layout appeared to be arbitrary. In many of the buildings, majorities (often 75%) of columns were oriented with the long side parallel to the sides of the building (Fig. 8a). This created a relatively soft, weak building, resulting in column hinging about the weak axis at the top and bottom of the column (Fig. 8b). The use of column splices just above the floor levels exacerbated this problem, particularly at the base of the first story. It was apparent that column hinging was a major cause of first story and multiple story collapses.

Poor beam-column joint performance was observed in many buildings. Reinforcing details for a building under construction just west of Gölcük. In the lower joint shown in Fig. 9, no hoops are used within the joint and the beam longitudinal bars are spliced within the joint outside the column reinforcing cage. As joint deterioration occurs, anchorage of the beam bars is completely lost and vertical load carrying capacity is compromised. Joint ties were used in the upper story joint in Fig. 9; however, this was the only damaged joint where ties were observed. Although some diagonal cracking is observed in the column in Fig. 9, a relatively tight spacing of column hoops was used in this building compared with other buildings, which may have pushed the failure into the joint region (versus a column shear failure).

Poor anchorage of column bars through the joint may have initiated many upper story collapses (weak-column, strong-beam). It was common practice to hook the exterior column bars from above that were lapped just above the joint; however, hooks were not used on the bars along the interior face of the column (Fig. 10). The short lap of the smooth column bars appeared to pull through the joint effectively resulting in a "true hinge" above the floor levels leading to complete collapse of the building. This failure pattern was apparent in many collapsed or near collapsed buildings (Fig. 12).

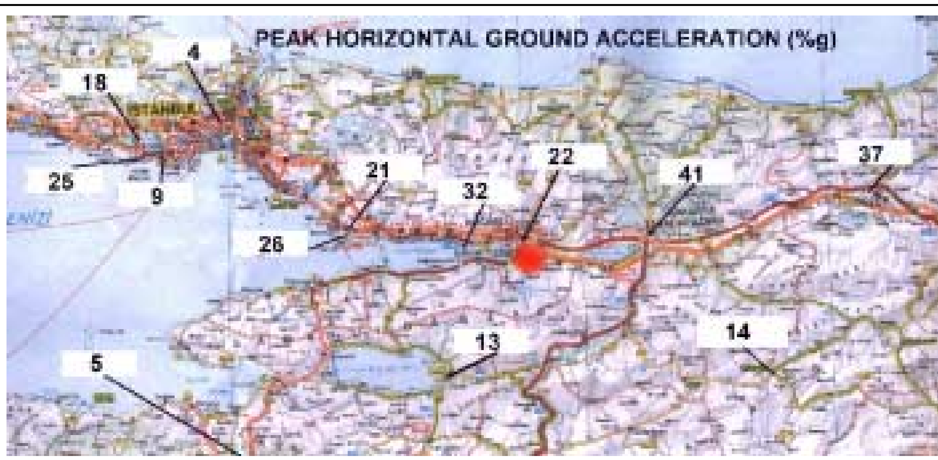


Fig. 4 Peak Horizontal Ground Accelerations (Bogazici web site)

The relatively wide spacing of transverse reinforcement (20 to 25 cm), the lack of crossties, and the use of 90-degree hooks on the hoops (versus 135-degree hooks anchored into the column core) were primary reasons for poor column performance (Fig. 10 and 11). For columns with nearly square cross-sections and adequate anchorage through the joint (strong-beams), insufficient transverse reinforcement was typically provided to resist the column shear developed when the columns reached their flexural capacity. As a result, relatively large diagonal cracks formed within the columns (e.g., see Fig. 13a), leading to "shear" failures and buckling of the vertical reinforcement. Relatively few column compression failures were observed. A compression failure for an interior column in a building west of Gölcük is shown in Fig. 11 (the same building as shown in Fig. 9).

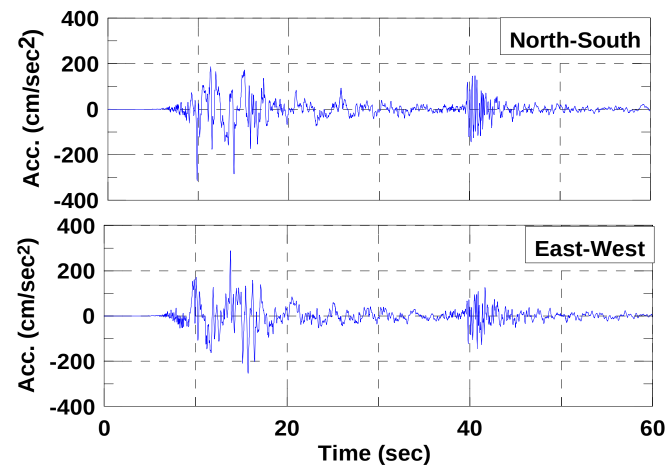


Fig. 5 Measured Accelerations – YPT Station

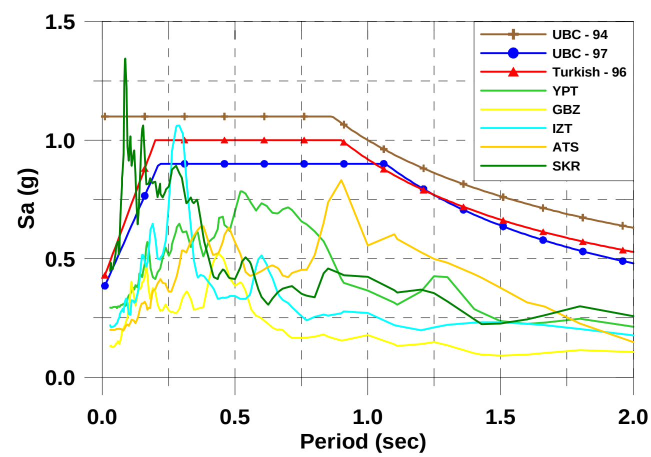
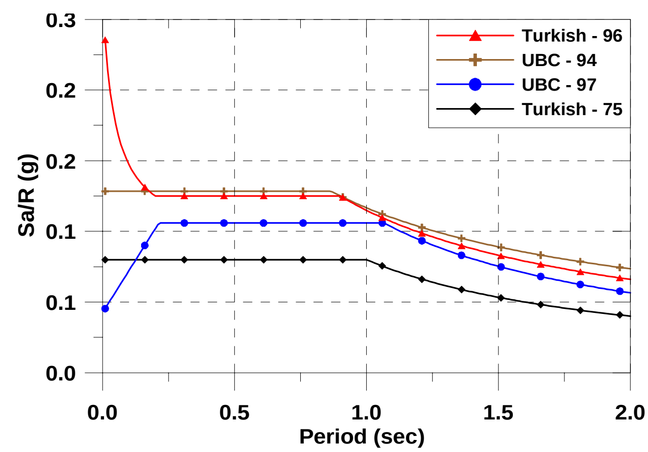


Fig. 6 (a) Elastic acceleration spectra: east-west



Fg. 6 (b) Equivalent code spectra

Many modern buildings were damaged in the city of Yalova (Fig. 13). Primary reasons for the poor performance includes previously mentioned deficiencies (column hinging, poor column details, column shear failures, soft-stories). The building in Fig. 13(a) was situated on the corner of two streets and the two faces of the building facing the streets were fairly open for retail space and parking, leading to a collapse of the first-story. A similar nearby building (Fig. 13b) did not exhibit a first-story collapse even though the axial load-carrying capacity of all the first-story columns along one side of the building had been compromised (e.g., Fig. 14a). Collapse of the building was apparently prevented by the use of relatively short, narrow coupled walls (Fig. 14a,b). A total of eight walls, with cross sections of approximately 200 mm by 1200 mm, were distributed around the perimeter of the building. Although the walls were damaged, the axial load-carrying capacity was maintained and collapse was avoided (as well as in several similar buildings that did not appear as damaged). Relatively deep beams and with short spans (approximately 4.5 m) to the interior columns may also have provided an alternate load path, with the walls providing vertical stability. Consistent with observations from other damaging earthquakes, a few, well-placed structural walls provide substantial life-safety.

Poor construction practices, as well as poor consolidation of concrete were observed in numerous buildings. In Fig. 15, poor alignment was achieved at the splice between the column vertical bars and the vertical foundation bars, and the vertical bars in the foundation were bent over to match

(a) Under construction near Gebze (b) Near collapse in Adapazarı

Fig. 7 Examples Soft-story buildings

(a) Typical placement and orientation of columns (b) Weak axis hinging of column

Fig. 8 Typical floor plan and damage for 3-story residential building the column vertical bars. Significant misalignment of columns between floor levels (1.4 cm/100 cm) was also observed. Poor concrete consolidation, lack of concrete cover, and rusted reinforcement are depicted in Fig. 16 for a seven-story building under construction in Yalova.

Fig. 9 Beam-column details and damage

Fig. 10 Beam-column joint damage – bar slip

Fig. 11 Column compression failure

(a) Complete collapse of building west of Gölcük

(b) Column Hinging

Fig. 12 Impact of column hinging on observed performance

Poor foundation performance was observed throughout the city of Adapazarı which is founded on a former lakebed. The soils for the top 10 m consist of sands and silty sands with low blow counts (uncorrected SPT blow counts of 5 to 10). According to some boring logs available over the Internet, the ground water table is only 2 m below the ground surface. Local residents indicated that the ground bubbled for several minutes and sand boils were observed throughout the city. Liquefaction-induced bearing failures lead to substantial building settlement (Fig. 17), and in some cases, lead to overturning of tall, narrow buildings (Fig. 18).

CONCLUSIONS

A large earthquake occurred due to rupture of the North Anatolian fault in western Turkey. The earthquake resulted in tremendous loss of life and property damage, and it will take decades for the region to recover. Observed damage indicates that a majority of the damage and loss of life resulted from poor structural configuration (buildings with soft first-stories and buildings with strong-beams and weak-columns), poor detailing of beams, columns and beam-column joints (splices, transverse reinforcement, and crossties), poor construction practices, and a lack of inspection. Poor performance of foundations was also observed, leading to substantial damage and loss of use in many structures. The magnitude and scope of damage observed occurred despite the existence of a modern building code, primarily because the code was not followed or enforced.

ACKNOWLEDGEMENTS

The information presented in this paper was collected as part of a reconnaissance mission by a team of researchers from the Pacific Earthquake Engineering Research (PEER) Center. The team consisted of Andrew Whittaker, John Stanton, Atila Zekioglu, Kalid Mosalam, and was joined by Sergio Alcocer, Halil Sezen, and Ken Elwood. During and after the reconnaissance trip, the PEER team collaborated with engineers and scientists from Bogazici University and the Kandilli Observatory in Istanbul, and the Middle East Technical University in Ankara. Considerable information was also collected over the Internet from the web sites of Bogazici University and the USGS (and the associated links). The willingness of researchers to post information on the web is greatly appreciated. Mark Aschheim at the University of Illinois is thanked for providing slides, documentation, and comments about his experiences in Turkey. Opinions and recommendations in this paper are those of the author and do not necessarily represent those of the sponsor or other individuals mentioned.

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(a) Column shear failure (wall in background) (b) Shear failure in coupled wall

Fig. 14 Damage to frame-wall building in Yalova

Fig. 15 Bent column bar at foundation

Fig. 16 Poor concrete placement
In shear wall (Yalova)

Fig. 17 Bearing/settlement failure in Adapazarı

Fig. 18 Liquefaction induced bearing failure
in Adapazarı