EVOLUTION OF SEISMIC BUILDING DESIGN AND CONSTRUCTION PRACTICE IN TURKEY

(To be submitted to "The Structural Design of Tall Buildings")

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Abstract: A study of building design and construction practice in Turkey since the early 1940s showed that substantial changes to the Turkish earthquake code typically followed major earthquakes that killed many people and destroyed buildings and infrastructure. Key changes to the Turkish earthquake code paralleled similar changes in the US seismic codes. Three key changes should be made to the Turkish earthquake code: (1) spectral maps similar to those recently introduced in the United States should replace the earthquake zonation map of the 1997 earthquake code; near-field factors should be included in the spectral maps, (2) the use of frames of high ductility should be mandated in regions of Turkey prone to moderate-to-severe earthquake shaking, and (3) the strength reduction factors for frames of nominal ductility should be substantially reduced to values that can be supported by rigorous analysis and testing. The excellent performance of the retrofitted Adapazari City Hall during the August 17, 1999, earthquake is a strong endorsement for reinforced concrete shear-wall retrofitting. Such conventional methods of retrofitting are likely the most promising and cost-effective means of reducing the vulnerability of the large inventory of non-ductile moment-frame buildings in Turkey.

Keywords: Turkey, earthquake, codes, design, construction

1. Introduction

Shortly following the destructive Kocaeli earthquake on the North Anatolian fault that struck north-eastern Turkey on August 17, 1999, the Pacific Earthquake Engineering Research (PEER) Center dispatched a reconnaissance team to the epicentral region to learn first hand about the damage to and performance of buildings, bridges, and industrial infrastructure. The geographic region that was impacted by the earthquake was somewhat narrow banded and centered on the North Anatolian fault, and stretched from Istanbul in the west to Duzce in the east. Figure 1 shows the fault line and the length of the fault that ruptured on August 17, 1999. Much of Turkey's industrial infrastructure is concentrated in this band and very little industrial and building construction pre-dates World War II. Damage to building construction was severe and widespread [Sezen et al., 2000].

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Immediately following the earthquake, much was reported in the press about the perceived poor quality of building construction in Turkey. Although the general quality of *residential* building construction was poor by current US standards, the quality of much of the *engineered* reinforced concrete 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. [2000] can be attributed to the use of non-ductile details and not poor quality construction. The widespread use of non-ductile details in *modern* reinforced concrete building construction, and the failures and collapses of such modern construction in the earthquake, led the authors to study the evolution of seismic design practice in Turkey.

Major earthquakes in Turkey have led to substantial changes in the practice of seismic design and construction. Fifty-seven destructive earthquakes have struck Turkey in the twentieth century, most occurring along the 1500-km-long North Anatolian fault. The largest earthquakes on this fault occurred in 1939, 1943, 1944, 1966, 1967, 1992, and 1999 (two earthquakes), resulting in more than 90,000 deaths, 175,000 injuries, and the destruction of 650,000 residential and office buildings. The M 7.9 Erzincan earthquake of December 27, 1939, in northeastern Turkey, was the largest of these earthquakes. The city of Erzincan was devastated and approximately 32,000 people died. Following that earthquake, the Turkish Ministry of Public Works and Settlement formed a committee to prepare a seismic zone map. The formation of this committee was the first step toward developing regulations for the seismic design of buildings in Turkey. Table 1 lists key events in the evolution of seismic codes and design practice in Turkey. Major revisions to the earthquake code have typically followed large earthquakes and substantial loss of life and property.Section 2 of this paper describes the evolution of seismic design and construction of buildings in Turkey from 1940 to the present day. Because reinforced concrete is the most common building material in Turkey, emphasis is placed on reinforced concrete design and construction. A brief comparison of current seismic codes in the US and Turkey is presented in Section 3. The efficacy of a conventional retrofit of a non-ductile moment-frame building is illustrated in Section 4, which describes the response of the Adapazari City Hall during the August 17, 1999, earthquake. This building was retrofitted with reinforced concrete shear walls after sustaining damage in the 1967 earthquake.

2. Design and Construction Practice

2.1 Introduction

Two codes influence the design and construction of reinforced concrete buildings in Turkey: *Specification for Structures to be Built in Disaster Areas* [Ministry, 1975, 1997] and TS-500, *Building Code Requirements for Reinforced Concrete* [Turkish, 1985]. The *Specification* is termed the "earthquake code" in this paper and has included procedures for calculating earthquake loads on buildings since 1940. The *Building Code Requirements for Reinforced Concrete* is termed the "building code" in this paper. The building code presents requirements for the design and detailing of reinforced concrete components but does not include ductile detailing requirements for use in seismic design. The following two sections present summary information on the building code and earthquake code.

2.2 Building Code Requirements for Reinforced Concrete

The building code is similar to ACI-318 [ACI, 1999] but does not address design and detailing for earthquake loads. Early editions (e.g., 1969) of the building code were based on allowable stress design. Major changes were introduced into the code in 1981 and 1985. The latest (1985) version of the code permits both allowable stress and strength design. Earthquake effects are considered using two load combinations, U,

$$U = G + P + E \quad \text{and} \quad U = G + 0.9E \tag{2-1}$$

where G is the dead load effect, P is the live load effect, and E is the earthquake effect. Earthquake loads were calculated following the procedures of the seismic code of the time. However, the building code does not contain any seismic detailing requirements, and the engineer is referred to the earthquake code for such information.

2.3 Earthquake Codes

2.3.1 Introduction

Although the earthquake codes were not widely used in Turkey in the period from 1940 to 1999 [Gulkan, 2000], it is instructive to document the evolution of the code. Such documentation is presented below. More information is presented in Sezen et al. [2000]. Many of the important changes in the Turkish earthquake code paralleled similar changes to US earthquake codes [ATC, 1995b].

2.3.2 Period from 1940 to 1953

The first seismic design code for buildings was published in 1940, one year after the destructive Erzincan earthquake. The 1940 code was similar to the Italian seismic code of that time [Bayulke, 1992; Duyguluer, 1997], and the base shear, V, was calculated as the product of a lateral force coefficient, C, and the weight of the building, W, namely:

$$V = CW \tag{2-2}$$

The value of C was set equal to 0.10 regardless of location in Turkey. The base shear force was distributed over the height of the building using a uniform load pattern.

An earthquake zonation map for Turkey was prepared in 1942 and promulgated in 1945. The map identified all provinces in Turkey [Duyguluer, 1997] and three seismic zones: first degree (hazardous); second degree (less hazardous); and no hazard. No earthquake analysis was required for the no-hazard zone. The inter-zonal boundaries followed administrative boundaries.

The 1947 code utilized the 1942 maps. The values of the lateral-force coefficient, C, were set equal to 0.10, 0.05, and 0.00, in the first-degree, second-degree, and no-hazard zones, respectively. Allowable stresses were increased in the 1947 code by 25% for component checking using earthquake load combinations.

In 1949, the zonation map was drawn and appended to the code. The lateral-force coefficients were further reduced from the values in the 1947 code to between 0.02 and 0.04 in the first-degree zone, and 0.01 and 0.03 in the second-degree zone. The specific value assigned to C was a function of soil and construction type. Duyguluer [1997] noted that the "...proper coefficient was to be established by the design engineer in charge in accordance with the soil formation at the construction site and the constructional characteristics of the building, and approved by the supervising agency." The weight of the building was calculated as

$$W = \sum_i w_i = \sum_i (g_i + np_i) \tag{2-3}$$

where w_i is the weight of the i^{th} floor, g_i is the dead load of the i^{th} floor, n is a live load coefficient (equal to 0.33 for houses, 0.5 for commercial buildings, and 1.0 for high-occupancy buildings), and p_i is the live load of the i^{th} floor. Allowable stresses were increased by 50% for component checking using earthquake load combinations, rather than the 25% increase of the 1947 code.

The 1953 code introduced load combinations for earthquake effects. Stresses, U, for earthquake design were calculated using

$$U = G + P + E + 0.5J \tag{2-4}$$

where J is the wind-load effect. No minimum requirements were established for detailing reinforced concrete components.

2.3.3 Period from 1954 to 1967

In the 1961 revision of the earthquake code, the procedure for calculating the lateral-force coefficient, C, was changed to read

$$C = C_0 n_1 n_2 \tag{2-5}$$

where C_0 is a coefficient that varies with building height, and n_1 and n_2 are coefficients that vary with building material, soil condition, and earthquake zone. Figure 2 shows the variation of C_0 with height; for heights greater than 40 m, C_0 was increased by 0.01 for every 3.0 m above 40 m.

In 1963, the earthquake zonation map was substantially revised and the number of zones was increased to four: Zone 1 (first degree), Zone 2 (second degree), Zone 3 (third degree), and Zone 4 (no hazard). The four zones were defined on the basis of the maximum expected shaking using the Modified Mercalli Intensity (MMI) scale. In Zone 1, shaking greater than or equal to MMI VIII was expected; in Zone 2, shaking equal to MMI VII was expected; in Zone 3, shaking equal to MMI VI was expected; and in Zone 4, shaking less than or equal to MMI V was expected. Figure 3 is the 1963 earthquake zonation map for Turkey. The interzonal boundaries shown in this figure continued to follow administrative boundaries and it was possible to move directly from a first-degree zone (maximum shaking) to a no-hazard or out-of-danger zone (minor shaking).

2.3.4 Period from 1968 through 1971

The 1968 earthquake code was substantially changed from earlier codes. Modern concepts relating to spectral shape and dynamic response were introduced. The design base shear, *V*, of (2-2) was calculated using the weight estimate of (2-3) and a lateral force coefficient, *C*, that was defined as

$$C = C_o \alpha\beta\gamma \tag{2-6}$$

where C_0 was a seismic zone coefficient and equal to 0.06, 0.04, and 0.02 for Zones 1, 2, and 3, respectively; α was a soil coefficient equal to 0.80 for rock, 1.00 for sand, gravel, and hard clay, and 1.20 for "...loose soil containing water and poorer soils..."; β was an importance factor equal to 1.50 for critical, high-occupancy, or historically important buildings, and 1.00 otherwise; and γ was a dynamic coefficient. The dynamic coefficient introduced spectral shape into the Turkish seismic code for the first time, and was calculated as $0.5 / T$ for period T greater than 0.5 sec and 1.00 for T less than or equal to 0.5 sec; the minimum value of γ was 0.3. The fundamental period could be calculated as

$$T = 0.09\frac{H}{\sqrt{D}} \tag{2-7}$$

where H was the height of the building above the foundation in meters, and D was the width of the building in the direction under consideration.

The base shear was distributed over the height of the building using the following equation

$$F_i = V\frac{w_i h_i}{\sum_i w_i h_i} \tag{2-8}$$

where h_i is the height of the i^{th} floor above the foundation. Equation 2-8 served to replace the uniform load profile of earlier codes with a first mode (inverted triangle) profile that is used in current US codes such as the 1997 Uniform Building Code [ICBO, 1997].

Geometry and detailing requirements for reinforced concrete components were introduced in the 1968 code. Minimum dimensions were specified for beams (150 mm x 300 mm [width x depth]), columns (the smaller of 0.05 times the story height and 240 mm), and shear walls (0.04 times the story height and 200 mm). The code did not specify a minimum stirrup (for beams) and tie (for columns) spacing but required that "...sufficient transverse reinforcement shall be provided..." and "... where beams frame into columns, the spacing of stirrups and column ties shall be half the spacing at the mid-regions of these members, within a distance not less than the effective depth of the deepest member framing into the joint. Column ties shall be continued within the story beams..." The addendum to the 1968 code included requirements for the use of shear walls. Specifically, the code wrote that if the height of a building exceeded a threshold value (12 m in a first degree zone, 15 m in a second degree zone, and 18 m in a third degree zone), shear walls "...extending along the height of the building shall be provided to transfer lateral earthquake loads to the foundation."

2.3.5 Period from 1972 through 1996

In 1968, the Ministry of Reconstruction and Resettlement embarked on a project to update the earthquake zonation maps based on new information on geologic structure, plate tectonics, historical seismicity, and earthquake occurrence. Zones were defined on the basis of maximum observed earthquake shaking in the period 1900 through 1970, measured in terms of the Modified Mercalli Intensity, namely, Zone I for MMI greater than or equal to IX; Zone 2 for MMI equal to VIII; Zone 3 for MMI equal to VII; Zone 4 for MMI equal to VI, and Zone 5 for MMI less than or equal to V.

The earthquake zonation map was updated in 1972 and the earthquake code was revised in 1975. Key changes to the zonation map included an increase in the number of seismic zones from 4 to 5. Important additions to the seismic code included new methods for calculating earthquake loads on buildings and ductile detailing requirements for reinforced concrete. The lateral force coefficient of the 1975 code was defined as

$$C = C_o KIS \tag{2-9}$$

where C_o was a seismic zone coefficient and equal to 0.10, 0.08, 0.06, and 0.03, for Zones 1, 2, 3, and 4, respectively; K was a coefficient related to the type of framing system, I was an importance factor (identical to β in the 1968 code), and S was a spectral coefficient. Values of K for different framing systems are presented in Table 2. The spectral coefficient was calculated as

$$S = \frac{1}{|0.8 + T - T_o|} \le 1.0 \tag{2-10}$$

where T and T_o are the fundamental periods of the building and soil column, respectively. Figure 4 presents spectral shapes for soil types I through IV, respectively. Soil types were classified on the basis of blow counts or shear wave velocity. Shear wave velocities for soil types I through IV were set at greater than 700 m/sec for I, 400 to 700 m/sec for II, 200 to 400 m/sec for III, and less than 200 m/sec for IV. The fundamental period was taken as the smaller of the value calculated using (2-7) and

$$0.07N \le T \le 0.10N \tag{2-11}$$

where N is the number of stories in the building above the foundation and "... the value of the coefficient ... shall be determined by interpolation between the values of 0.07 and 0.10 according to the degree of general structural flexibility."

Geometry and detailing requirements for reinforced concrete components were modified in the 1975 code. The 1975 code provided information on minimum details for columns. The minimum rectangular column dimension was limited to 250 mm or 0.05 times the story height; the maximum column width-to-depth ratio was 3.0. The minimum and maximum longitudinal rebar ratios were 0.01 and 0.035, respectively. Columns were divided into three regions as shown in Figure 5: *confinement* regions at each end of the column clear height, *middle* region, and *beam-column* joint regions. The *confinement* region was defined as the height not smaller than 0.167 times the column clear height or

450 mm, measured from the slab soffit or beam top surface. The volumetric ratio of transverse reinforcement, ρ , in this region was set at 0.12 times the concrete compressive strength divided by the rebar yield strength. One-hundred and thirty-five degree hooks were required on ties in confinement regions; the minimum tie diameter was 8 mm, and the minimum and maximum tie spacings were 50 mm and 100 mm, respectively. In the middle region, tie sizes were based on satisfying gravity and earthquake forces that were calculated using (2-9). The maximum tie spacing, s_1 in Figure 5, was the smaller of 200 mm and 12 times the diameter of the longitudinal rebar.

2.3.6 Period from 1997 to 2000

A new earthquake zonation map that is shown in Figure 6 was included in the 1997 earthquake code. Response-spectrum and linear and nonlinear dynamic analysis procedures were also introduced in this edition of the seismic code. The lateral force coefficient *C* of the 1975 code was replaced by

$$C = \frac{A(T)}{R_a(T)} = \frac{A_0 IS(T)}{R_a(T)} \tag{2-12}$$

where *A* is the spectral acceleration coefficient; *T* is the fundamental period; and the effective ground acceleration coefficient A_0 is 0.4, 0.3, 0.2 and 0.1 for the first four seismic zones, respectively. The fifth seismic zone is assumed to have no earthquake hazard. The importance factor varies between 1 and 1.5, but is equal to 1.0 for ordinary structures. The spectrum coefficient, *S*, which defines the design acceleration spectrum, is given by three equations in the short-period, constant acceleration, and constant velocity ranges, respectively. These ranges are delineated by spectrum characteristic periods, which vary as a function of soil type. The maximum spectral amplification is 2.5, which is identical to that of US codes [ICBO, 1997] for 5% damping. The seismic load reduction factor R_a in this code is similar to the response modification factor in U.S. codes, except that the seismic load reduction factor reduces linearly from the maximum value of *R*, which is tabulated in the code, to 1.5 at zero period. The maximum value of *R* depends on the assumed ductility (high or normal) of the system and varies between 3 and 8.

Reinforced concrete buildings are classified as systems of either high or nominal ductility based on the detailing of the components. Detailing requirements are more stringent for systems with high ductility. The detailing requirements for columns of both high and nominal ductility levels are most similar. The minimum cross-section dimensions are 250 mm by 300 mm. Information on the transverse reinforcement requirements along the height of a column are shown in Figure 7. All hoops are must have 135-degree seismic hooks at both ends. Cross ties may have 90-degree hooks at one end. The sum of the column strengths at a joint must exceed 120% of the sum of the beam strengths at that joint. The shear strength of a column must exceed the shear force associated with the plastic moment in the column. The only major provision that is not applicable for columns of nominal ductility level is the spacing of transverse reinforcement along the confinement zones (Figure 7), which is required to be half the spacing in the column middle region. For columns in frames of nominal ductility, the maximum spacing of the transverse reinforcement between the confinement zones was increased by a factor of 2 over the spacing shown in Figure 7.

2.4 Summary Remarks

From discussions with expert academicians and design professionals in Turkey, it is clear that little use was made of the Turkish earthquake code for the engineering of commercial and residential construction in the epicentral region prior to the August 17, 1999, earthquake. Prior to the adoption of the 1997 Turkish earthquake code, the design professional was permitted to design non-ductile reinforced concrete buildings in Turkey. The use of construction details for ductile response, although clearly presented in the earthquake code prior to 1997, was not mandatory. The design professional had two options for the analysis, design, and detailing of reinforced concrete moment-frame buildings: (1) calculate seismic design forces from the earthquake code using the force reduction factors for non-ductile moment resisting frames (Table 2), and design and detail the frame using the building code (that has no provisions for ductile detailing), or (2) calculate seismic design forces from the earthquake code using the force reduction factors for ductile moment resisting frames, and design and detail the frame using the earthquake code and its corresponding detailing requirements for high ductility. The choice of design option was likely driven in part by cost, as is often the case in the United States when code-permissible options are presented to an owner. The costs of the increased section sizes associated with the use of the larger lateral-force-coefficients for non-ductile moment resisting frames were likely substantially smaller than the costs associated with providing the ductile details of the Turkish earthquake code. The reinforcement details described in the 1975 earthquake codes were seldomly observed in buildings inpected by the authors after the August 17, 1999, earthquake, supporting that option 2 was selected more frequently by the engineer.

Much was written in 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 non-ductile detailing in regions exposed to earthquake shaking.

3. Comparison of United States and Turkish Seismic Codes

Figure 8 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 1997 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 amplification factors that must be applied in the United States if the site of the building is located within 15 km of a major active fault. Putting these factors aside, the elastic spectral demands of the two current codes are most similar.

Table 3 presents values of the response modification factor in the 1997 UBC (=R) and the 1997 Turkish earthquake code (=R_a) for different framing systems. The values are most similar. The reduction of the factor from the maximum value of R_a in the short-period range to 1.5 at zero period in the Turkish earthquake code is a substantial improvement over US codes that assume that R is period-independent, because ductility cannot be used to reduce strength demands in the short-period range [ATC, 1995a].

Figure 9 presents lateral-force-coefficient spectra (C in equation 2-2) for the 1975 and 1997 Turkish codes and the 1997 UBC for reinforced concrete moment-resisting frames on rock and soft soil sites. Such frames were chosen for the purpose of comparison because reinforced concrete moment-resisting frames are the most common seismic framing system in Turkey for building construction. To construct the spectra for the 1975 Turkish code, K was taken as 0.80 and 1.50 for ductile and non-ductile reinforced concrete moment frames, respectively; C_0 was set equal to 0.10; these allowable stress design spectra were scaled to the strength level by multiplying the spectral ordinates by 1.4. To construct the spectra for the 1997 Turkish code, A_0 and the importance factor were set equal to 0.40 and 1.0, respectively; and R was set equal to 4 and 8 for reinforced concrete moment-resisting frames of nominal and high ductility, respectively. To construct the spectra for the 1997 UBC, soil types S_B and S_E were assumed for the rock and soft soil sites, respectively; near-field factors were not considered; the importance factor was set equal to 1.0, and R was set equal to 3.5 and 8.5 for ordinary moment-resisting frames (OMRF) and special moment-resisting frames (SMRF), respectively. (The OMRF and SMRF of the UBC correspond approximately to frames of nominal and high ductility in the Turkish earthquake code, respectively.)

For modern reinforced concrete moment-resisting frames of high ductility (the SMRF in the US), the ordinates of the 1997 Turkish lateral-force-coefficient spectra exceed those of the 1997 UBC for both rock and soft soil sites. Recognizing that the prescriptive details of the 1997 UBC and the 1997 Turkish code for frames of high ductility are most similar, the performance of buildings designed to either code should be similar if the construction standards are similar and the fundamental period exceeds 0.2 sec. For short-period buildings and identical construction standards, buildings designed and detailed according to the 1997 Turkish earthquake code will suffer much less damage than buildings designed and detailed using the 1997 Uniform Building Code [ICBO, 1997].

For information, consider the ordinates of the lateral-force coefficient spectra for the 1975 Turkish earthquake code for frames of nominal ductility on both rock and soft soil sites that are presented in Figure 9. If buildings in the epicentral region of the August 17, 1999 earthquake had been designed in accordance with an earthquake code, the 1975 code would likely have been used. The ordinates are less than or equal to 0.22. The intensity of earthquake shaking necessary to yield and potentially fail non-ductile reinforced concrete moment-frame buildings designed for such modest levels of lateral force is likely 0.10g or smaller. Ground accelerations considerably larger than 0.10g were measured throughout the epicentral region during the August 17, earthquake and so the widespread damage is not unexpected.

4. Seismic Performance of the Adapazari City Hall

4.1 Introduction

There is scant information in the literature on the performance of retrofitted non-ductile reinforced concrete moment-frame buildings during severe earthquake shaking. The Adapazari City Hall, which was retrofitted in 1967, was located in downtown Adapazari where many buildings either collapsed or suffered substantial damage during the August 17, 1999 Kocaeli earthquake [Sezen et al. 2000].

The construction of Adapazari City Hall started in 1959 and was completed in 1964. The City Hall was a five-story reinforced concrete moment-frame building with no basement because the water table is only 1 to 2 m below grade. The tapered footing foundations, which are interconnected by grade beams, are located on relatively soft soil deposits [Anadol et. al, 1972]. The building has a clear height of 17 meters above grade. The plan footprint is 14.2 m by 40 m with 13 frames in the transverse direction and 3 frames in the longitudinal direction. A typical floor plan is presented in Figure 10; all dimensions in this figure are in meters. The two shear walls in the transverse (y) direction on lines 5 and 6 were not placed symmetrically in plan and created a stiffness eccentricity. Anadol et al. [1972] reported that the concrete strength was 16 MPa (2350 psi) and the yield strength of the smooth reinforcing bars was 220 MPa (32 ksi).

Resistance to lateral forces in the original building was provided by 2 shear walls and 35 columns. The depth of the columns varied between 300 and 700 mm, and the width varied between 230 and 400 mm. Typical floor framing consisted of 370 mm deep joist system infilled with lightweight bricks, and 370 mm deep beams varying in width from 400 to 1000 mm. No ductile details were used in any structural components because such details and knowledge were not widely available in the early 1960s. The lightweight brick infill walls in the building, that ranged between 100 mm and 150 mm in thickness, were not designed to resist lateral or gravity loads.

4.2 1967 Akyazi earthquake: damage and retrofit

Adapazari was 36 km to the east of the epicenter during 1967 Akyazi earthquake. The shaking intensity in Adapazari was estimated as MMI VII. The City Hall was heavily damaged by this earthquake, with damage being concentrated in the shear walls and columns of the lower stories. No ties were observed in one column (A1 of Figure 10) [Anadol, 1972]. Damage to stairwells, slabs, and beams was also reported at several floors, and large shear cracks were observed in many of the infill walls [Anadol, 1972].

The earthquake damaged City Hall building was retrofitted in the 8 months following the earthquake. Retrofit work included repair and strengthening of damaged slabs, beams, columns and shear walls, and the addition of new beams and full-height shear walls. All columns were typically retrofitted by the addition of transverse reinforcement and an increase in the column size by 120 to 200 mm. New transverse perimeter shear walls were added on Grids 1 and 13, and transverse interior shear walls were added on Grids 8, 9, and 10 to match existing walls on Grids 4, 5 and 6. Short longitudinal walls were placed in the four corners of the building. Figure 11 shows the locations of the new reinforced concrete shear walls. In this figure, B.A. PERDE is Turkish for reinforced concrete shear wall. Existing perimeter longitudinal beams were retrofitted and new perimeter longitudinal beams were added. The retrofit solution adopted in 1967 substantially stiffened the building and eliminated the plan eccentricities in the original building. Summary information from Aytun [1972] is presented in Table 4. Figure 12 is a photograph of the City Hall building.

4.3 1999 Kocaeli earthquake and aftershock

Sezen et al. [2000] report some of the damage in Adapazari that was documented by the reconnaissance team following the August 17, 1999, M7.4 Kocaeli earthquake. The North Anatolian fault ruptured within 10 km of the City Hall and devastated many reinforced concrete moment-frame buildings in the immediate vicinity of the City Hall, but the retrofitted City Hall sustained negligible structural damage. The non-structural components and contents in the City Hall suffered only modest damage. Because the City Hall was one of the few buildings in downtown Adapazari that suffered little to no structural damage, it was used as an Emergency Crisis Center to direct response and recovery after the earthquake.

The solid line in Figure 13 presents the 5% damped pseudo-acceleration response spectrum prepared using the east-west ground acceleration history recorded a few kilometers from the City Hall during the August 17, 1999, earthquake. The peak ground acceleration for this recorded ground motion was 0.41g. The spectral acceleration at 1 second, the approximate period of the City Hall building, was 0.4g. Because the building sustained little-to-no structural damage, and assuming that the strength of the building was no greater than 15 percent of the weight (calculated by doubling the allowable-stress-design lateral-force coefficient of 0.07), the intensity of shaking at the site of the building was likely substantially less than that indicated by the solid line in the figure. Parenthetically, the elastic spectral demands associated with the earthquake shaking in Adapazari were large and it is not surprising that many non-ductile reinforced concrete moment-frame buildings in this city collapsed because the spectral demands greatly exceeded the lateral strength of these buildings that likely did not exceed 5 to 10 percent of their weight.

On November 11, 1999, an aftershock of the August 17 earthquake produced substantial shaking in Adapazari. The dashed line in Figure 13 is the 5% damped pseudo-acceleration response spectrum prepared using the east-west ground acceleration history recorded at the site described in the previous paragraph. The zero-period acceleration for this motion was 0.35g. Additional damage to structural and non-structural components in the City Hall was reported.

The lack of damage to the structural frame of the City Hall building, and the almost complete destruction of reinforced concrete moment-frame buildings around the City Hall clearly show that reinforced concrete walls are an effective means of retrofitting older reinforced concrete construction in Turkey. The strategic placement of new structural walls can reduce the plan and vertical irregularity of an existing building, thereby substantially improving its performance. The addition of a modest number of structural walls to the original City Hall building more than doubled the strength and substantially increased the stiffness of the building. The walls protected the vulnerable non-ductile columns and joints from the gross damage and collapse that was observed in adjacent buildings of a similar age and newer that had not been retrofitted.

5. Summary Remarks

A study of building design and construction practice in Turkey since the early 1940s showed that substantial changes to the Turkish earthquake code typically followed major earthquakes that killed many people and destroyed buildings and infrastructure. Such a pattern is not unique to Turkey: similar patterns are observed in the United States, Mexico, and Japan. Key changes to the Turkish earthquake code, including indirect consideration of dynamic response and introduction of ductile detailing for reinforced concrete, paralleled similar changes in the US seismic codes.

The design and detailing requirements of the current (1997) Turkish earthquake code [Ministry, 1997] are most similar to those of the current US earthquake codes such as the 1997 Uniform Building Code [ICBO, 1997]. Putting aside issues related to characterization and mitigation of ground failure such as lateral spreading and liquefaction that were prevalent in the epicentral region, three key changes should be made to the Turkish earthquake code: (1) spectral maps similar to those recently introduced in the United States should replace the earthquake zonation map of the 1997 earthquake code, and near-field factors should be included in the spectral maps, (2) the use of frames of high ductility should be mandated in regions of Turkey prone to moderate-to-severe earthquake shaking, and (3) the strength reduction factors for frames of nominal ductility should be substantially reduced to values that can be supported by rigorous analysis and testing.

The excellent performance of the retrofitted Adapazari City Hall in the August 17, 1999, mainshock and the November 11, 1999 aftershock, by comparison with the poor performance of most reinforced concrete moment-frame buildings in the immediate vicinity of the City Hall is a clear endorsement for reinforced concrete shear-wall retrofitting. The introduction of a modest number of strategically placed ductile structural walls into an existing non-ductile reinforced concrete moment-frame building will protect the vulnerable non-ductile components and substantially improve the performance of the building. Such conventional methods of retrofitting are likely the most promising and cost-effective means of reducing the vulnerability of the large inventory of non-ductile moment-frame buildings in Turkey.

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FIGURE CAPTIONS

Figure 1	Line of rupture in the 1999 Izmit earthquake
Figure 2	Variation of force coefficient C_0 with building height in 1961
Figure 3	1963 earthquake zonation map of Turkey [IAEE, 1966]
Figure 4	Spectral shapes in 1975 Turkish earthquake code
Figure 5	Column details from 1975 Turkish earthquake code [Ministry, 1975]
Figure 6	1997 earthquake zonation map of Turkey [Ministry, 1997]
Figure 7	Column details from 1997 seismic code
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Figure 9	Lateral-force-coefficient spectra. (a) rock site; (b) soft soil site
Figure 10	Plan view of Adapazari City Hall [Anadol, 1972]
Figure 11	1967 shear wall retrofit of City Hall [Anadol, 1972]
Figure 12	Adapazari City Hall following the November 11, 1999 aftershock
Figure 13	Acceleration spectra for the August 17, 1999 mainshock and November 11, 1999, aftershock