2 Evolution of Seismic Building Design Practice in Turkey

2.1 Introduction

This chapter describes the practice of seismic design and construction of buildings in Turkey from 1940 to the present. Because reinforced concrete is the most common building material in Turkey, emphasis is placed on reinforced concrete design and construction.

Two codes influence the design and construction of reinforced concrete buildings in Turkey: TS-500, *Building Code Requirements for Reinforced Concrete* (Turkish 1985), termed the "building code" in this report, and *Specification for Structures To Be Built in Disaster Areas* (Ministry of Public Works and Settlement 1975, 1997), termed the "seismic code."

The building code presents requirements for the proportioning and detailing of reinforced concrete components, and is similar to ACI-318 (ACI 1999) except for the detailing of earthquake effects, which is not covered by the building code. Summary information on the building code is presented in Section 2.3.

Since 1940, the seismic code has included procedures for calculating earthquake loads on buildings. In 1968, restrictions on component sizes and rebar details were introduced for the design of ductile components. Earthquake loads for buildings are calculated using the seismic code similar to U.S. practice in which earthquake loads are calculated using the Uniform Building Code (ICBO 1997). Additional information on the various editions of the code, from 1940 through 1997, is presented in Section 2.4.

The following sections of this chapter present information on major earthquakes in Turkey in the 20th century (Section 2.2), the requirements of the Turkish building code for reinforced concrete (Section 2.3), and Turkish seismic design codes (Section 2.4). A comparison of U.S. and Turkish codes is presented (Section 2.5). Summary remarks are presented in Section 2.6.

2.2 Major Earthquakes in Turkey in the 20th Century

Major earthquakes in Turkey have led to substantial changes in the practice of seismic design and construction. Fifty-seven destructive earthquakes struck Turkey in the 20th century, most occurring along the 1500-km-long North Anatolian fault (see Chapter 1, Figure 1-1). 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.

Table 2-1 lists key events in the evolution of seismic codes in Turkey. Destructive earthquakes have usually resulted in revisions to the codes. In this table and hereafter in this report, "ductile detailing" refers to the use of reinforcement details that provide ductile response in components.

The M7.9 Erzincan earthquake of December 27, 1939, in northeastern Turkey, was the largest earthquake in Turkey in the 20th century. 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 2-1 Key events in the evolution of seismic design codes in Turkey

Year	*Event*	*Code development*
1939	Erzincan earthquake (M7.9)	
1940	Committee formed to develop a seismic zonation map for Turkey	First seismic code published
1942		Earthquake zone map prepared; map promulgated in 1945
1943	Tosya earthquake (M7.2)	
1944	Gerede earthquake (M7.2)	Seismic code revised
1947		Seismic code revised
1949		Seismic code revised
1953		Seismic code revised
1958	Ministry of Reconstruction and Resettlement established	
1961		Seismic code revised
1963		Earthquake zone map revised
1966	Varto earthquake (M7.1)	
1967	Adapazari earthquake (M7.1)	
1968		Seismic code revised
1975		Seismic code revised; ductile detailing introduced
1992	Erzincan earthquake (M6.9)	
1997		Seismic code revised; ductile detailing required
1999	Izmit earthquake (M7.4) Düzce earthquake (M7.2)	

2.3 *Building Code Requirements for Reinforced Concrete*

The *Building Code Requirements for Reinforced Concrete* provide general proportioning and detailing procedures for reinforced concrete components. Early versions (e.g., 1969) were based on allowable stress design and were similar to other building codes. Major changes were introduced into the code in 1981 and 1985.

The latest version of the building code (1985) permits calculations using both allowable stress design and strength design. For designs in which earthquake loads are considered, stresses for calculations are made using the two following load combinations, U,

$$U = G + P + E \tag{2-1}$$

$$U = G + 0.9E \tag{2-2}$$

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 given in the seismic code of the time. However, the building code did not contain any special seismic detailing requirements, and the designer was referred to the seismic code for such information.

2.4 Evolution of Turkish Seismic Design Codes

2.4.1 Years 1940 to 1953

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

$$V = CW \tag{2-3}$$

The value of C was set equal to 0.10 regardless of location. 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 listed all provinces in Turkey (Duyguluer 1997). Three seismic zones were identified in the map: first degree (hazardous); second degree (less hazardous); and no hazard. No earthquake analysis was required for the no-hazard zone. The interzonal boundaries followed administrative boundaries. According to Duyguluer, the zonation of a province or region was based on the observed or projected intensity of earthquake shaking.

The 1947 code utilized the 1942 maps. The values assigned to C were established on the basis of seismic zone. In first-degree zones, C was set equal to 0.10; in second-degree zones, C was set equal to 0.05. Allowable stresses were increased by 25% for component checking using earthquake load combinations.

In 1949, the zonation map was drawn and appended to the revised code. The coefficients were further reduced to between 0.02 and 0.04 in the first-degree zone, and to between 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

$$W = \sum_i w_i \tag{2-4}$$

and

$$w_i = g_i + np_i \tag{2-5}$$

where w_i is the weight of the floor, g_i is the dead load of the 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 floor. Allowable stresses were increased by 50% for component checking using earthquake load combinations, rather than 25% per the 1947 code.

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

$$U = G + P + E + 0.5J \tag{2-6}$$

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

2.4.2 Years 1954 to 1967

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

$$C = C_0 n_1 n_2 \tag{2-7}$$

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 conditions, and earthquake zone. Figure 2-1 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. Tables 2-2 and 2-3 list values for n_1 and n_2. In Table 2-2, soil type 1 is "hard and monolithic rock," soil type II is "sand, gravel, and compact soils...," and soil type III is "less strong soils other than mentioned" (IAEE 1966).

Table 2-2 Values of n_1

Soil Classification	*Building Type*	
	Steel	*Reinforced Concrete*
I	0.6	0.8
II	0.8	0.9
III	1.0	1.0

Table 2-3 Values of n_2

Earthquake zone	n_2
First degree	1.0
Second degree	0.60
Third degree	0.60

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 2-2 is the 1963 earthquake zonation map for Turkey. Because the interzonal boundaries shown in this figure continued to follow administrative boundaries, it was possible to move directly from a first-degree zone (maximum shaking) to a no-hazard or out-of-danger zone (minor shaking).

2.4.3 Years 1968 through 1971

The 1968 seismic code was substantially different from earlier codes. The 1968 code changed the procedures for calculating earthquake demands on building components, introduced requirements for detailing reinforced concrete components, and introduced modern concepts relating to spectral shape and dynamic response. The design base shear of Equation 2-3 was calculated using the weight estimate of Equation 2-5 and a lateral force coefficient, *C*, that was defined as

$$C = C_o \alpha \beta \gamma \tag{2-8}$$

where C_0 is a seismic zone coefficient and equal to 0.06, 0.04, and 0.02 for Zones 1, 2, and 3, respectively; α is 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 is 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, which is calculated as 0.5/*T* for fundamental period, *T*, greater than 0.5 sec but not less than 0.3, and 1.00 for *T* less than or equal to 0.5 sec. A coefficient, γ, introduced spectral shape into the Turkish seismic code for the first time. The code wrote that the fundamental period could be calculated as

$$T = 0.09\frac{H}{\sqrt{D}} \tag{2-9}$$

where *H* is the height of the building in meters above the foundation, and *D* is 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

$$F_i = V\frac{w_i h_i}{\sum_i w_i h_i} \tag{2-10}$$

where h_i is the height of the floor above the foundation. Equation 2-10 served to replace the uniform load profile of earlier codes with a load profile similar in shape to the typical first mode shape in a building.

Geometry and detailing requirements for reinforced concrete components were also introduced in the 1968 code. Minimum dimensions were specified for beams (150 mm x 300 mm [width times 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 minimum spacing for beam stirrups and column ties, 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."

2.4.4 Years 1972 through 1996

The earthquake zonation map was updated in 1972 and the seismic code was revised in 1975. Key changes to the zonation map included an increase in the number of 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. Information on earthquake effects and analysis, design, and detailing are presented below.

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 (Duyguluer 1997). 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 1 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 lateral force coefficient of the 1975 code was defined as

$$C = C_o KIS \tag{2-11}$$

where C_o is 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 is a coefficient related to the type of framing system, I is an importance factor (identical to β in the 1968 code), and S is a spectral coefficient. Values of K for different framing systems are presented in Table 2-4. The spectral coefficient was calculated as

$$S = \frac{1}{|0.8 + T - T_o|} \tag{2-12}$$

where T and T_o are the fundamental periods of the building and soil column, respectively. Figure 2-3 presents spectral shapes for soil types I through IV, respectively. Soil types were classified on the basis of blow counts or shear wave velocity, and values for T_o were set for each type. 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 Equation 2-9 and

$$0.07N \le T \le 0.10N \tag{2-13}$$

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."

Table 2-4 Structural type coefficient, K, from 1975 code

<table>
<tr><th colspan="2">Structure Type</th><th>Fillerwall type[1]</th><th>K</th></tr>
<tr><td colspan="2">All building framing systems except as hereafter classified</td><td>-</td><td>1.00</td></tr>
<tr><td colspan="2">Buildings with box systems with shear walls</td><td>-</td><td>1.33</td></tr>
<tr><td rowspan="6">Buildings with frame systems where the frame resists the total lateral force</td><td rowspan="3">a. ductile moment-resisting frame</td><td>a</td><td>0.60</td></tr>
<tr><td>b</td><td>0.80</td></tr>
<tr><td>c</td><td>1.00</td></tr>
<tr><td rowspan="3">b. nonductile moment-resisting frame</td><td>a</td><td>1.20</td></tr>
<tr><td>b</td><td>1.50</td></tr>
<tr><td>c</td><td>1.50</td></tr>
<tr><td colspan="2" rowspan="3">Shear wall systems with ductile frames capable of resisting at least 25% of the total lateral force</td><td>a</td><td>0.80</td></tr>
<tr><td>b</td><td>1.00</td></tr>
<tr><td>c</td><td>1.20</td></tr>
</table>

1. Filler wall types: a = reinforced concrete or reinforced masonry walls; b = unreinforced masonry block partition walls; c = light partition walls or prefabricated concrete partition walls

Geometry and detailing requirements for reinforced concrete components were modified in the 1975 code. Minimum dimensions were specified for beams (200 mm x 300 mm [width times depth, = B x D]), columns (the smaller of 0.05 times the story height and 250 mm), and shear walls (0.05 times the story height and 150 mm). Minimum reinforcement ratios and sizes were set for beams (minimum stirrup diameter of 8 mm and minimum stirrup spacing of B or 0.5D) and shear walls (ρ = 0.0025, 0.0020 for horizontal and vertical reinforcement, respectively; maximum rebar spacing of 300 mm or 1.5 times the wall thickness). Figure 2-4 shows sample detailing requirements for beams and shear walls. Minimum floor slab thicknesses were set at 100 mm. Infilled joist slab construction (termed "asmolen" construction) was permitted only in buildings taller than 12 m if shear walls were used as the lateral force-resisting system.

The 1975 code provided much 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 2-5: *confinement* regions at each end of the column clear height, a *middle* region, and *beam-column* joint regions. The *confinement* region was defined as the distance 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

$$\rho = 0.12\frac{f_c}{f_y} \tag{2-14}$$

where f'_c and f_y are the concrete compressive strength and rebar yield strength, respectively. Hooks of 135° 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 gravity and earthquake forces (calculated using Equation 2-11). The maximum tie spacing, s_1 in Figure 2-5, was the smaller of 200 mm and 12 times the diameter of the longitudinal rebar.

2.4.5 Years after 1997

The earthquake zonation map was updated (Figure 2-6) and the seismic code revised in 1997. In addition to the equivalent static load method (Equation 2-3), the mode superposition method and linear and nonlinear dynamic analyses were introduced for the seismic design of buildings. The lateral force coefficient was replaced by $A(T)/R_a(T)$, where A is the spectral acceleration coefficient calculated as

$$A(T) = A_o IS(T) \tag{2-15}$$

The effective ground acceleration coefficient, A_0, is 0.4, 0.3, 0.2, and 0.1 for the first four seismic zones, respectively. Note that the fifth seismic zone was specified to have no earthquake hazard. The importance factor, *I,* is 1.0 for ordinary structures and varies between 1.0 and 1.5. 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, T_A and T_B, which vary as a function of soil type. The maximum spectral amplification is 2.5. The seismic load reduction factor 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 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. Transverse reinforcement requirements for beams are presented in Figure 2-7. These requirements apply for frames of both high and nominal ductility.

The detailing requirements for columns of 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 2-8. All hoops must have 135° seismic hooks at both ends. Cross ties may have 90° hooks at one end. The sum of the column strengths at a beam-column 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 moments 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 2-8), which is required to be half the spacing in the column middle region. Lap splices of column longitudinal rebar should be made in the middle third of the column. If column rebar are spliced at the bottom of a column, the splice length is increased to 125% or 150% of the development length of the bar in tension, depending on the number of bars being spliced. For columns in frames of nominal ductility, the maximum spacing of the transverse reinforcement between the confinement zones is increased by a factor of 2 over the spacing shown in Figure 2-8. For shear walls, the minimum wall thickness is the smaller of 0.067 times the story height and 200 mm.

2.5 Comparison of United States and Turkish Codes of Practice

Figure 2-9 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 this figure do not include near-field amplification factors, N_a and N_v, that must be applied if the site of the building is within 15 km of a major active fault. Putting these factors aside, the spectral demands of the two current codes are very similar.

Figure 2-10 presents the lateral force coefficient spectra (C in Equation 2-3) 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 "allowable-stress-design" spectra for the 1975 Turkish code, K was taken as 0.80 and 1.50 for ductile and nonductile reinforced concrete moment frames, respectively; C_0 was set equal to 0.10. The ordinates were then increased by 40% to construct "strength-design" spectra to facilitate comparison with the 1997 Turkish seismic code and the 1997 UBC. To construct the spectra for the 1997 Turkish code, A_0 and the importance factors 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 code, respectively.)

For modern reinforced concrete moment-resisting frames of high ductility (the SMRF in the U.S.), the ordinates of the 1997 Turkish lateral-force-coefficient spectra exceed those of the 1997 UBC for both rock and firm soil sites. Recognizing that the prescriptive details of the 1997 UBC and the 1997 Turkish code for frames of high ductility are similar, the performance of buildings designed to either code should be similar if the standards of construction are comparable.

Table 2-5 presents values of R in the 1997 UBC and the 1997 Turkish codes for different framing systems. These values are similar for each type of framing system. Further review of the two codes indicates similarities in most other regards. Because the linear-elastic acceleration response spectra (Figure 2-9) are similar in both codes for the regions of highest seismicity, buildings designed and constructed in accordance with these two codes should perform equally if the construction quality is similar.

Table 2-5 Response modification factors in current seismic codes

Lateral force-resisting system	*Country*	
	1997 Turkey[1,2]	*1997 USA*[2]
Reinforced concrete shear wall	6	5.5
Reinforced concrete moment-resisting frame	8	8.5
Steel eccentrically braced frame	7	7
Steel moment-resisting frame	8	8.5

1. Framing systems of high ductility
2. 1997 codes in Turkey and USA

2.6 Summary Remarks

Revisions to the practice of earthquake engineering in Turkey have generally followed major, damaging earthquakes. This trend is not unique to Turkey because changes in design practice in Japan, Mexico, and the United States have followed major earthquakes in those countries.

Provisions for special detailing of reinforced concrete moment-resisting frames for ductile response were introduced in Turkey in 1975. Such requirements were similar to those introduced in the United States in the early 1970s. However, the construction of buildings with ductile details was not mandated as it was in California in the 1970s. Rather, buildings could be constructed without special details for ductile response (frames of nominal ductility) or ductile details (frames of high ductility). Because it was cheaper to construct stronger buildings without special details for ductile response (nonductile detailing) than weaker buildings with ductile detailing, nonductile moment-resisting frame construction was most prevalent in Turkey up to the time of the Izmit earthquake.

The current codes of practice in Turkey and the United States are similar in terms of strength and detailing requirements. However, two key changes to the Turkish *Specification for Structures To Be Built in Disaster Areas* are recommended:

1. A factor that accounts for the close proximity of a structure to a fault (i.e., a near-field factor) should be included in the design force equation.

2. Special details for ductile component response and the use of rules for ductile system response should be mandatory in moderate and severe seismic zones, regardless of the lateral forces used for design.

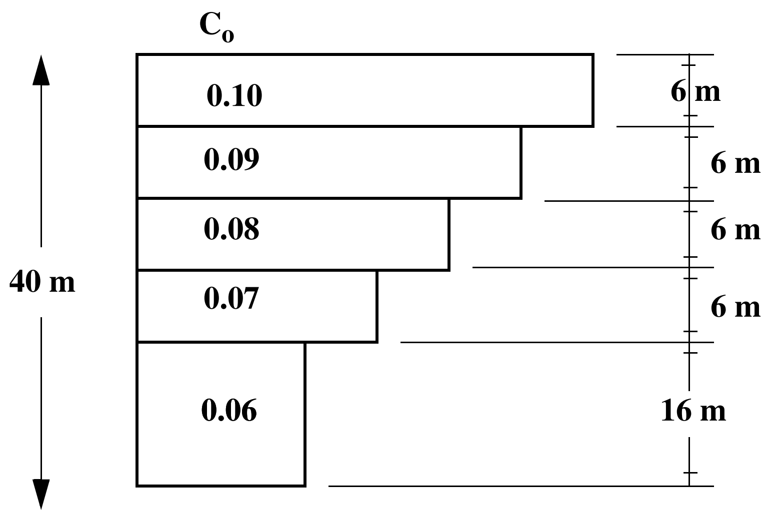


Figure 2-1 Distribution of coefficient C_0 with height above grade in 1961 seismic code

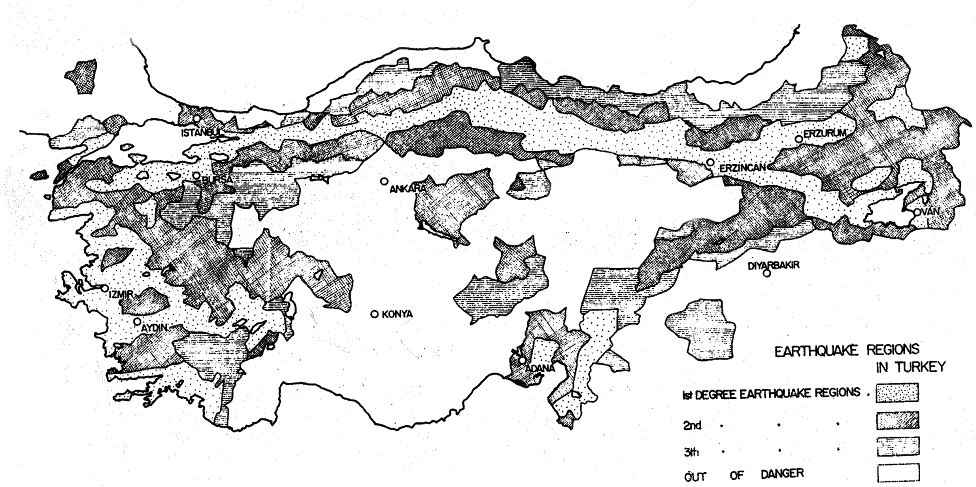


Figure 2-2 1963 earthquake zonation map (IAEE 1966)

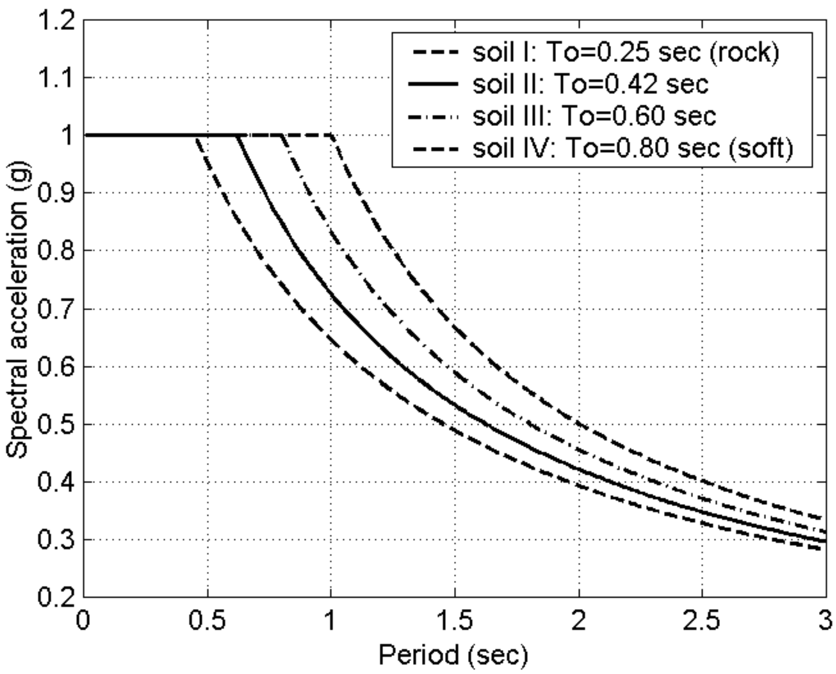


Figure 2-3 Spectral coefficients, *S*, from 1975 seismic code

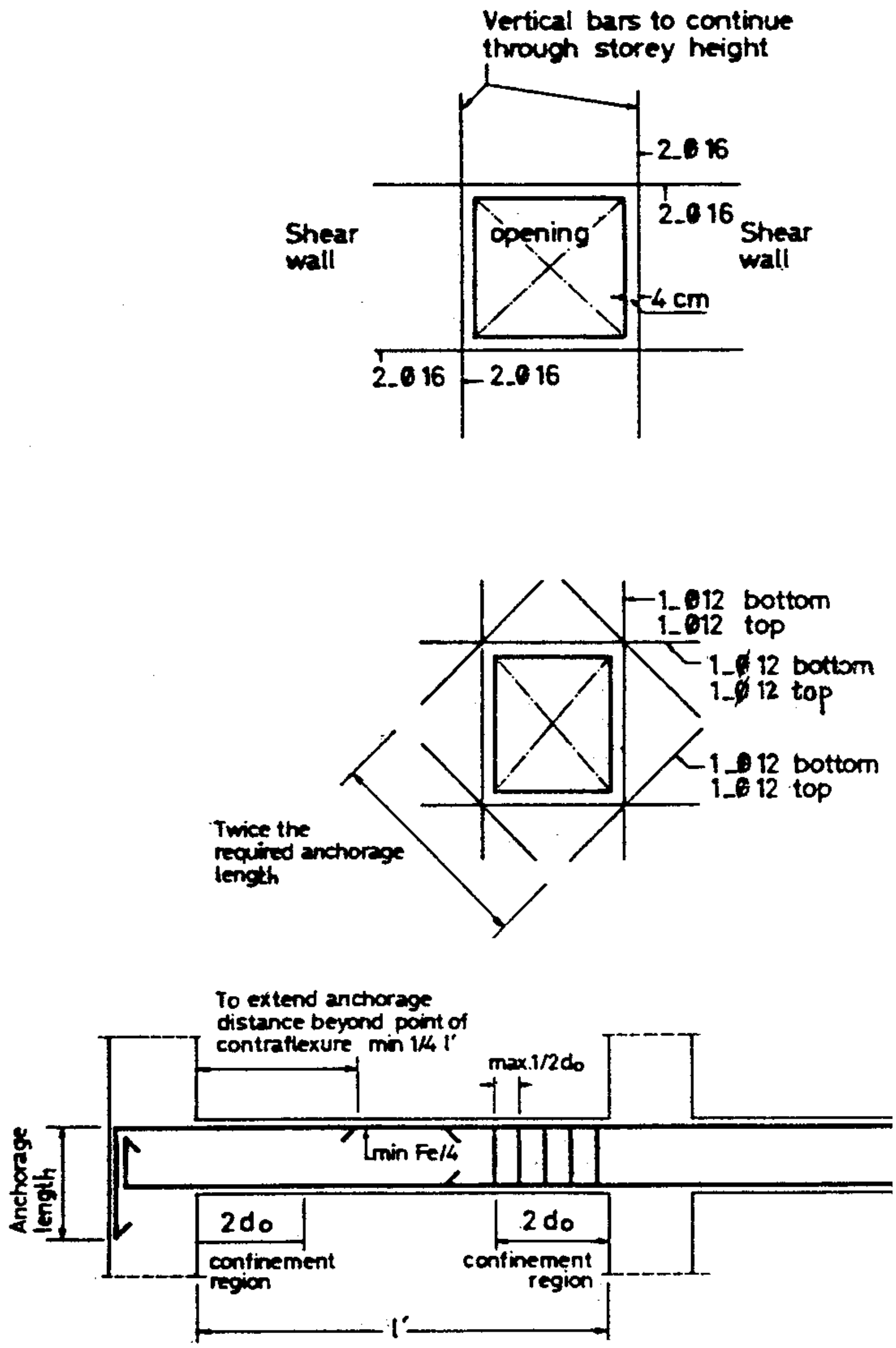


Figure 2-4 Detailing requirements for beams and shear walls from 1975 seismic code

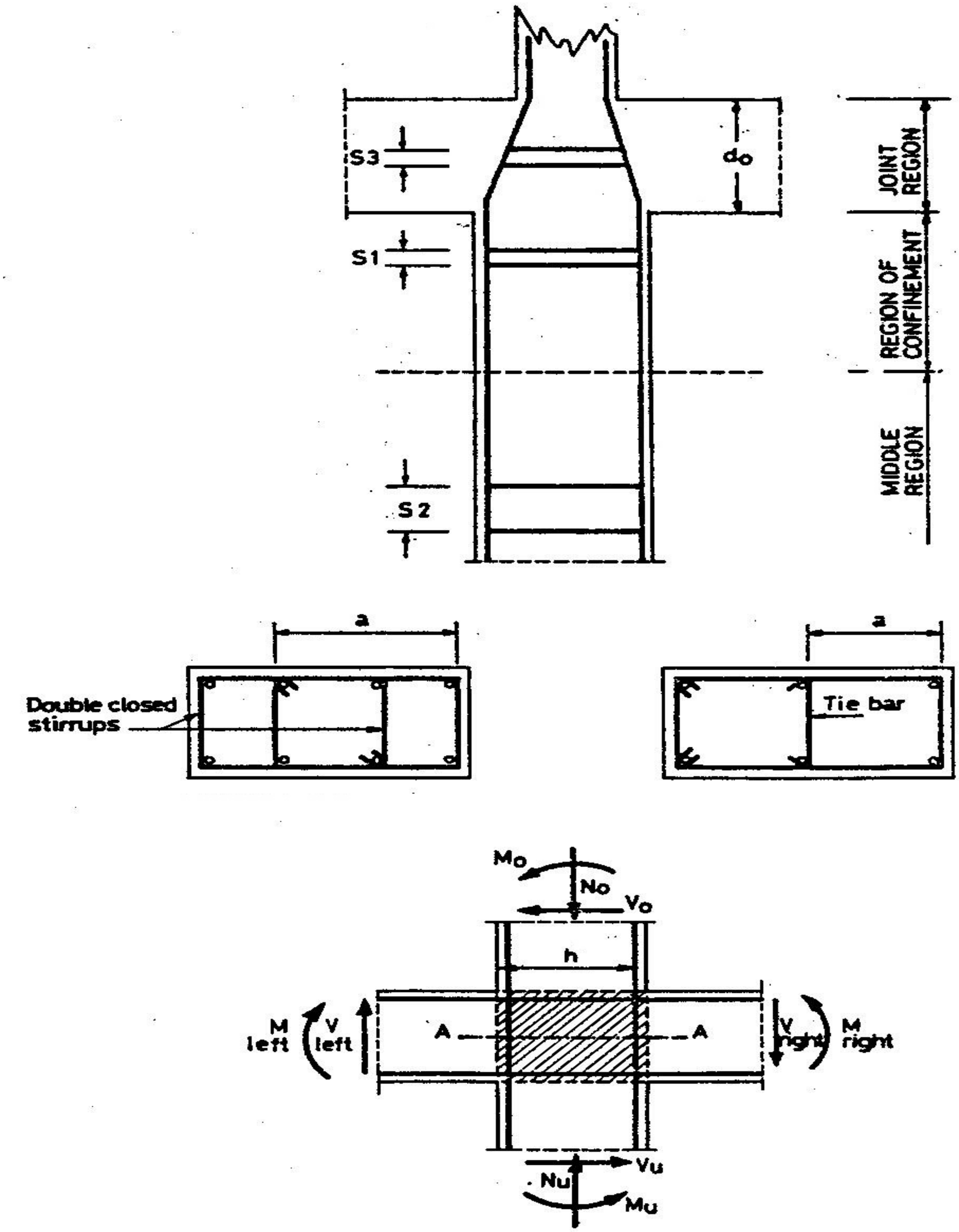


Figure 2-5 Detailing requirements for columns from 1975 seismic code

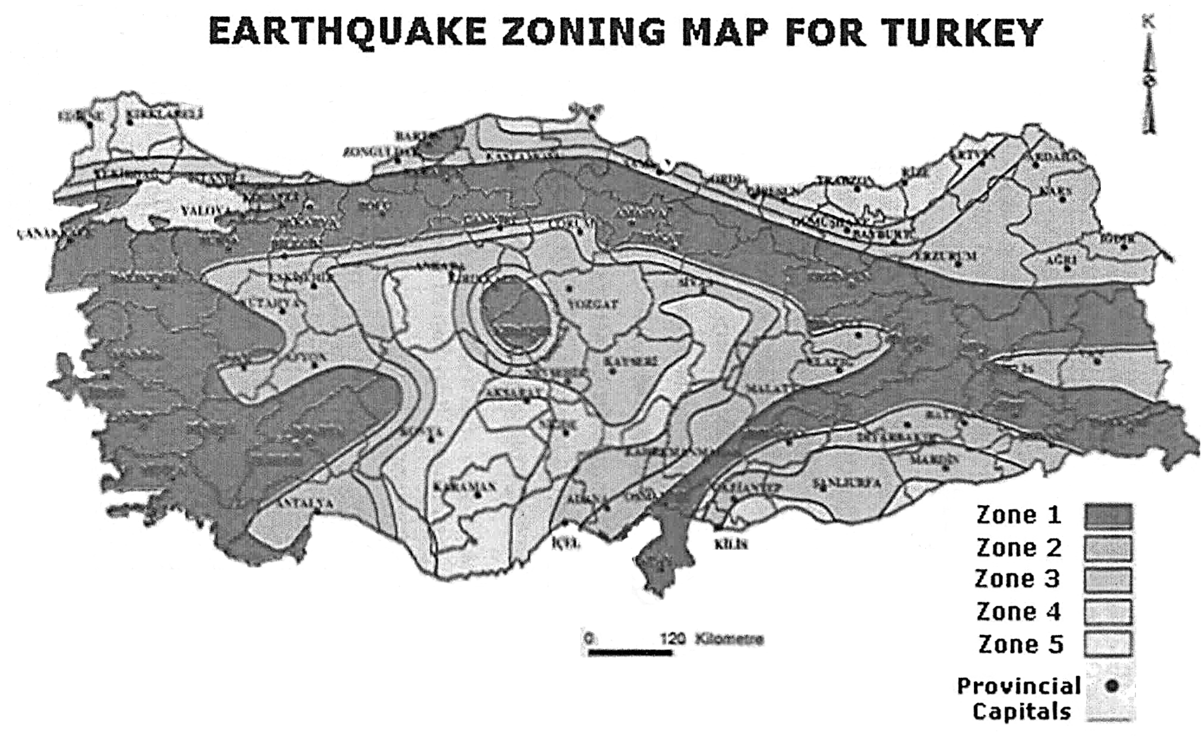


Figure 2-6 Earthquake zonation map of Turkey in the 1997 seismic code

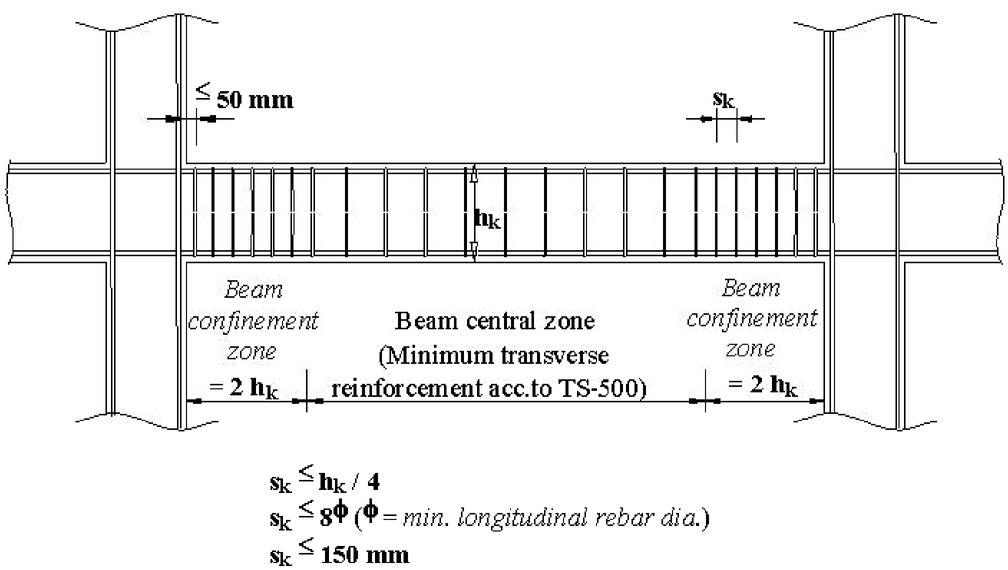


Figure 2-7 Transverse reinforcement requirements for beams in the 1997 seismic code

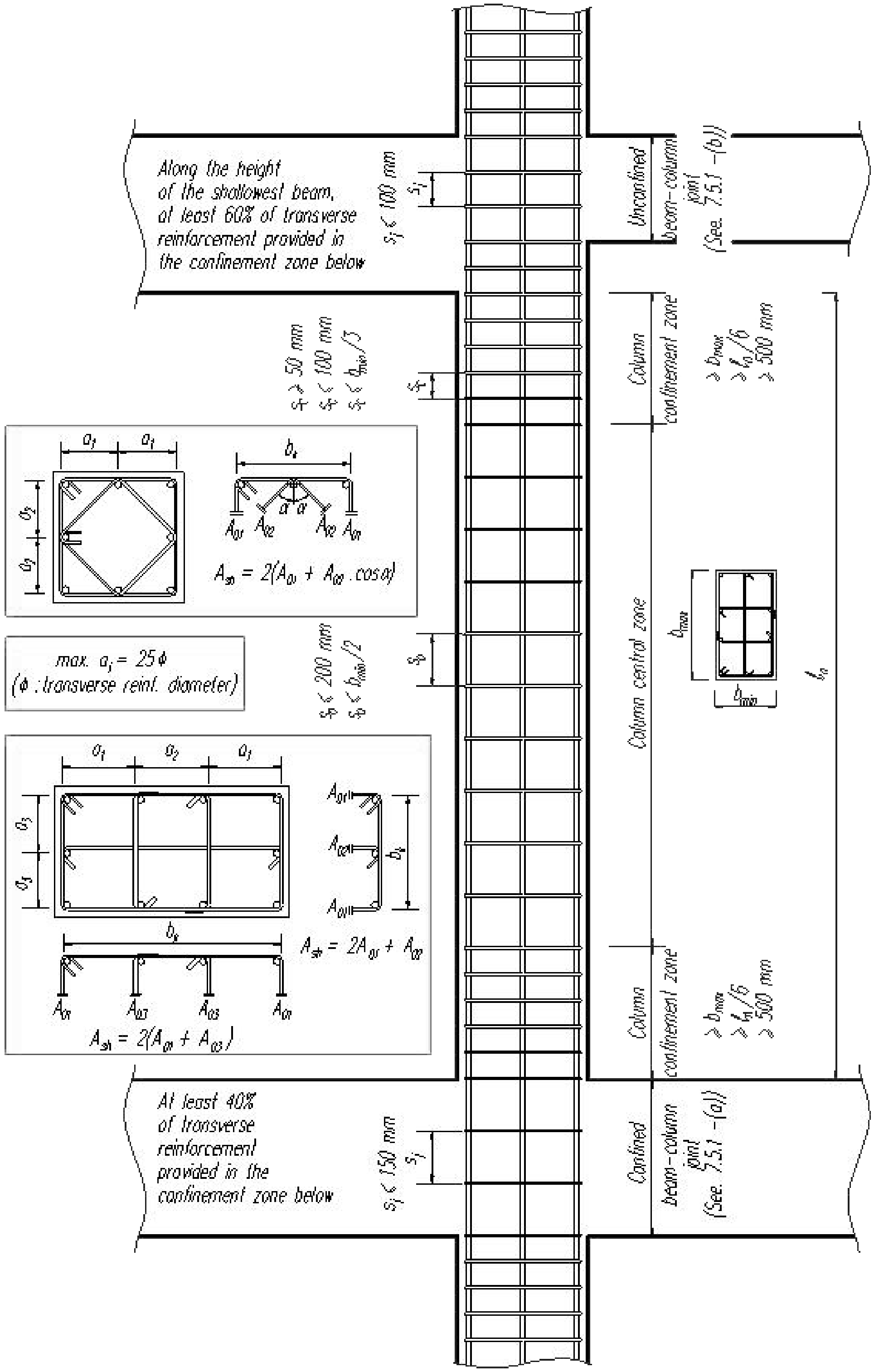


Figure 2-8 Column confinement zones and detailing requirements in the1997 seismic code

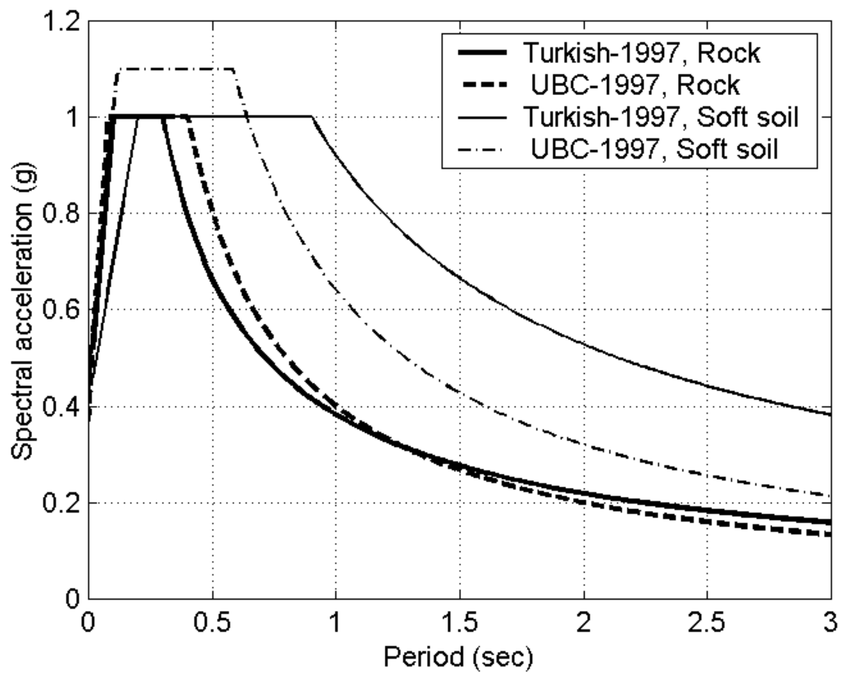
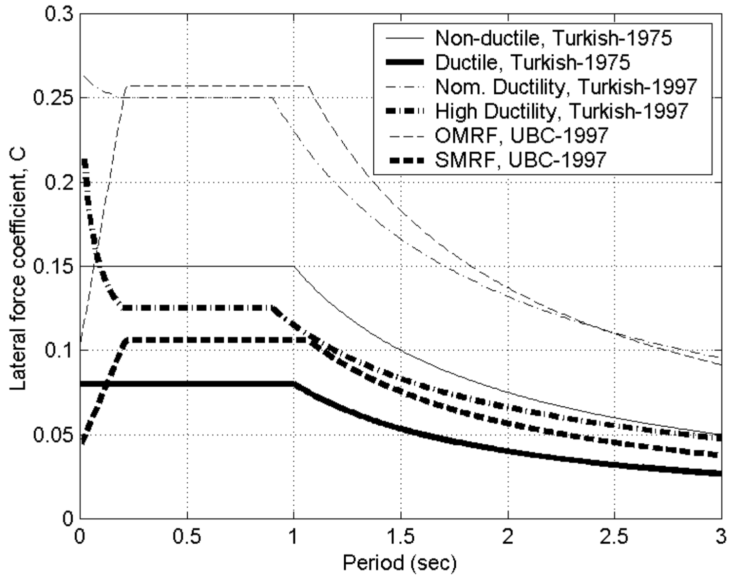
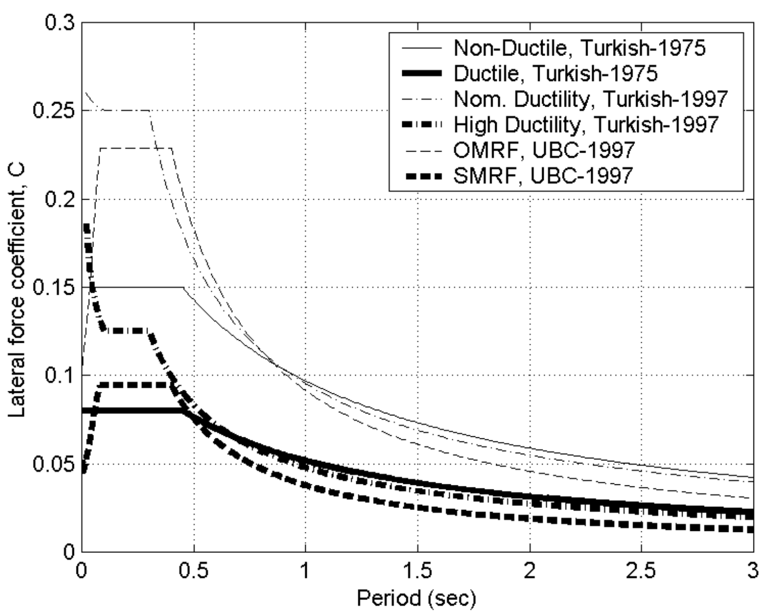


Figure 2-9 Comparison of elastic response spectra from the 1997 UBC and the Turkish seismic codes



a. soft soil sites



b. rock sites

Figure 2-10 Comparison of lateral force coefficient, *C,* in the 1997 UBC, and 1975 and 1997 Turkish seismic codes