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Building and design defects observed in the residential sector and the types of damage observed in recent earthquakes in Turkey

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Abstract

Turkey is situated in a very active earthquake region. In the last century, several earthquakes resulted in thousands of deaths and enormous economic losses. In 1999, the Marmara earthquake had an approximate death toll of more than 20 000, and in 2011, the Van earthquake killed 604 people. In general, Turkish residential buildings have reinforced concrete structural systems. These reinforced concrete structures have several deficiencies, such as low concrete quality, non-seismic steel detailing, and inappropriate structural systems including several architectural irregularities. In this study, the general characteristics of Turkish building stock and the deficiencies observed in structural systems are explained, and illustrative figures are given with reference to Turkish Earthquake Code 2007 (TEC, 2007). The poor concrete quality, lack of lateral or transverse reinforcement in beam-column joints and column confinement zones, high stirrup spacings, under-reinforced columns and over-reinforced beams are the primary causes of failures. Other deficiencies include weak column-stronger beam formations, insufficient seismic joint separations, soft story or weak story irregularities and short columns. Similar construction and design mistakes are also observed in other countries situated on active earthquake belts. Existing buildings still have these undesirable characteristics, so to prepare for future earthquakes, they must be rehabilitated.

1 Introduction

In Turkey, 70 % of the population is living in the first- and second-degree seismic zones, and 95 % of the buildings are at risk. Losses were experienced in medium-intensity and severe earthquakes for many years. The losses were not limited to rural regions; there were also major financial losses and other intangible damages in urban regions (Erzincan, 1939, 1992; Kocaeli, 1999; Van, 2011, etc.). In those urban regions, most of the structural stock was built from reinforced concrete. Medium-intensity earthquakes in particular can be endured as an ordinary event in developed countries, but they are

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still considered one of the most important natural disasters in Turkey. Because most of the current buildings in Turkey were constructed before TEC 2007, their earthquake-resistance features are insufficient and their structural irregularities pose a danger. Another important point is that many of the buildings that have structural irregularities are high-rise buildings.

Big earthquakes occur in various regions of our country in 10–15 year-periods. Earthquake codes are also overhauled at certain times. Earthquake codes are generally reconsidered in response to application and design errors observed in previous earthquakes and in response to the failure mechanisms observed in damaged buildings. Although earthquakes have tragic consequences for society, they also constitute a unique natural laboratory for civil engineers and architects. In the practice of structural engineering, it is necessary to determine and classify the errors and to prevent them from occurring in future applications. Thus, it is important to study the damages caused by earthquakes and to determine how they occur. It is difficult to understand the damage mechanisms of wholly collapsed buildings after earthquakes. For this reason, it is better to concentrate on moderately and heavily damaged buildings in the technical investigations and damage assessments that are conducted after earthquakes. As a result of investigations in these buildings, the engineering and application errors can be taken into consideration. The collapse of an entire building or the loss of a floor generally occurs as a result of similar errors. In this study, the most frequent errors during the design, construction and usage stages in reinforced concrete buildings and their results after earthquakes are described via visual elements. The observations and the reasons for errors are evaluated with consideration of the TEC 2007. The main principles of earthquake-resistant structural design are presented in the following codes:

- In light earthquakes, structural and nonstructural system elements in buildings should not be damaged.
- In moderate earthquakes, the damage to structural and nonstructural system elements should be at a level that can be repaired.

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– In heavy earthquakes, the partial or whole collapse of buildings should be prevented in order to avoid the loss of life (AFAD, 2001).

The requirements established by earthquake-resistant building codes are designed to provide a minimum amount of safety for buildings. For better earthquake responses and higher safety, it is necessary to satisfy conditions that exceed the regulation conditions. There are no obstacles to providing most of the strength and ductility required by code.

Although earthquake-resistant structural design depends on many uncertainties, it can be done to comply with certain conditions. Reinforced concrete buildings can be damaged heavily or collapse in earthquakes due to failures and errors at the stage of design and production. The failures do not represent defects in the reinforced concrete itself.

In Turkey, it is not uncommon for materials to be stolen or for unqualified materials to be used in construction. When damages, collapses and losses occur after earthquakes due to these activities, it is usually just the building contractors who are convicted. However, the problem is not that simple. The causes of damage in affected buildings (1992 Erzincan, 1994 Dinar, 1998 Adana – Ceyhan, 1999 Gölcük, 1999 Düzce and 2011 Van) can be ordered as follows:

– Load-bearing system error (strong beam – weak column, inadequate size, inadequate bearings or misplacement of bearing elements, etc.)

– Architectural design error (ribbon window, soft story, side discontinuities, incorrect geometrical structuring, etc.)

– Inadequate detailing

– Poor labor (lack of transverse reinforcement, poor placement of concrete, etc.)

– Low-quality materials (low concrete strength, poor reinforcement, etc.)

– Floor effects

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Similar studies were conducted by other researchers like (Bahadır, 2012; Gökten, 1994; Kocyigit, 2002; Kucuk, 2006; Mertol and Mertol, 2002; Unal, 2012).

2 Earthquakes in Turkey

The earthquakes in our country, which is on the Alp-Himalayas seismic belt, are related to the movement of the Africa-Arabian plates in a north-northeast direction. The movement depends on the outward spreading of the Atlantic Ocean. Moreover, the Arabian plate is impelled northward due to continuous sea floor spreading throughout the long axis of the Red Sea, and it is forced to dive under the Eurasian plate. This causes a dense pinch effect in the East Anatolian Region between the Arabian plate and the Eurasian Continent. This pinch sets the principal large faults into motion, such as the North Anatolian fault and the East Anatolian fault. This action has been continuing for millions of years and is the main reason for today's earthquakes.

The North Anatolian fault is 1400–1500 km long. The Anatolian plate between the North Anatolian and East Anatolian faults moves west at 13–27 mm per year and proceeds toward the Crete Region by curling to the left in the west. The plate motions in and around Turkey are shown in Picture 1.

An ancient ocean connecting the Atlantic Ocean and the Mediterranean Sea to the Indian Ocean began disappearing due to the northward movement of the Arabian plate. Thus, the Arabian continent and Eurasian continent started to collide. Anatolia is in this collision zone. The collision causes the continental crust in the east of Anatolia to be thickened, and this thickening still continues. East Anatolia has risen approximately 2000 m over the course of several million years. Approximately 5 million years ago, the North Anatolia fault and East Anatolia fault combined in Karliova as a result of pinching of the Arabian plate, which moves northward by 2 m every 100 years. The Anatolian plate has moved west ever since.

The Greek-Aegean geography works against the westward movement of the Anatolian plate. This causes spreading in West Anatolia resembling "a fan formed by pinch-

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ing the end of the broom on the wall," and fields called graben and horst are formed in this region. The residual oceanic crust in the north of the African plate and in the bottom of Mediterranean began to dive under the Eurasian plate in the south of Crete approximately 15 million years ago, and the subsumed crust melted into the mantle and turned into magma. This magma formed a band of volcanic islands in the Aegean Sea through a reincreasing process, and it is known that this process continues to this day. Due to the diving of the African plate under the Anatolian plate and the European continent, the African continent, the European continent and the Anatolian plate will eventually combine after approximately 100 million years. During the diving period, the Anatolian plate will continue to develop many faults and will experience corresponding earthquakes (AFAD, 2001).

2.1 Seismic zones in Turkey

Turkey is in the Alpine-Himalayas region, which is one of the most active seismic zones in the world. Approximately 42 % of the country's surface area is in the first-degree seismic zone. There are five seismic zones in Turkey:

- 1st degree seismic zone: the areas close to subsidences and active fracture faults. The earthquakes occurring here cause substantial loss of life and property.
- 2nd degree seismic zone: these are the areas in which earthquakes cause less damage than in the first-degree seismic zone.
- 3rd degree seismic zone: shocks cause less damage in these areas.
- 4th degree seismic zone: shocks cause little or no damage in these areas.
- 5th degree seismic zone: there are few shocks in these areas, or no shocks are not felt (Picture 2).

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3 Overview of Turkish structural stock and earthquake damages

3.1 Overview of structural stock in Turkey

Research revealed that 70 % of the building stock in big cities such as İstanbul, Ankara and İzmir is illegal and that 50 % of the building stock in the country is illegal.

The Ministry of the Environment and Urban Planning stated that "half of the buildings constructed before 2002 are non-resistant, and for this reason, there are a great number of buildings that need recruitment and reinforcement" (Pampal, 2000).

According to inventory studies by the Ministry of Environment and Urban Planning (2008), there are approximately 1 million buildings in Istanbul alone, and no engineering services were found in 400 000 of them. Engineering services in others were found to be inadequate for mitigating disaster risks. First, it is necessary to identify buildings with problems and to improve them. However, Istanbul has a building stock of 1 400 000. One-by-one reinforcement of buildings will not be sufficient to eliminate urban risks in building areas exposed to danger (Anonymous, 2013).

At the Izmir Symposium on Reducing Disaster Risk (2009), it was determined that the building quality in Izmir was 3 % good, 52 % medium and 45 % poor/bad, according to studies carried out in 3 pilot regions (1490 buildings) by the Izmir Chamber of Civil Engineers. This study was to serve as a model for structural stock in Izmir. In the study, it was found that 60 000 (8.2 %) of the 725 000 buildings in Istanbul will endure heavy damage and that approximately 70 000 (9.5 %) will endure moderate damage in the case of an earthquake.

The situation in central and eastern Turkey is similar. On Sunday, 23 October 2011, at 13:41 LT, destructive ground motion occurred with an epicenter at Tabanlı Village in the Ercis district of Van city. Destruction and damage to buildings occurred in Van city and other Ercis district centers and villages, and losses of life and property were experienced because of the building damage. The Van-Ercis earthquake caused severe destruction in the center of Ercis district due to the building stock of the region, but similar levels of destruction were not observed in the center of Van city (AFAD,

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2001). Following the earthquake on the 23 October, another earthquake occurred on Wednesday, 9 November 2011, at 21:23 LT, with an epicenter in the Edremit district of Van city. In total, 644 citizens lost their lives in the earthquakes on the 23 October and the 9 November; 1966 citizens were injured and 252 citizens survived wreckages. In the first week of the earthquake, 187 earthquakes with magnitudes of 4.0–4.9 and 13 earthquakes with magnitudes greater than 5 occurred in the region, and approximately 180 aftershocks occurred every day for the first month after the earthquake. According to data from the National Seismological Observatory, which is managed by the prime ministry disaster and emergency management presidency, the amount of energy released as a result of an earthquake is very high. The energy of the main shock of the earthquake on 23 October was equal to 33.2 times the energy of the Hiroshima atom bomb, and if aftershocks are taken into consideration, the energy released was equal to 37 atom bombs (AFAD, 2001).

3.2 Earthquakes in Turkey during the last 25 years and the types of damage observed in reinforced concrete buildings after the 2011 – Van Earthquake

Every earthquake is unique in terms of the characteristics of the ground motions and the responses of buildings to those ground motions. The calculations of civil engineers include many assumptions related to the design of the structures. First, the concrete material used is not homogeneous, and the properties change over time. Cracking occurs in reinforced concrete elements, which complicates the engineering assumptions related to concrete. In addition to these uncertainties about materials, the determination of dead and live loads for building has its own uncertainties and assumptions. When building design is discussed only in terms of vertical load, the work of a civil engineer is relatively easy. However, the possibility of earthquake loads must be considered, as many uncertainties are introduced. The most effective method to prove the validity of an engineering problem that includes such uncertainties and assumptions is to perform experiments. However, doing full-scale, three-dimensional experiments with buildings is expensive and difficult. In experimental studies, experiments are gener-

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ally performed on samples that are modeled, usually in two dimensions, by minimizing a certain rate. From an engineering point of view, it is possible just a few years are after completion of a building to understand whether the building is resistant against vertical loads. In this way, an assessment can be made about the accuracy of the assumptions made in the design of the building. However, for earthquake calculations, it is necessary to wait for the occurrence of an earthquake before the assumptions about earthquake-resistant structure design and the calculations of earthquake loads can actually be tested and validated. For this reason, it is important to examine the causes of damage and collapse after an earthquake in order to revise the earthquake code, review current engineering assumptions, reorganize the code for building construction and, above all, raise awareness among civil engineers and architects.

3.2.1 Short column behavior

The most important defects observed in damaged structures after earthquakes are errors related to have the wrong architecture or the wrong load-bearing system. It is very difficult and sometimes impossible for a civil engineer to convert an existing structure to an earthquake-resistant one. The first category of architectural design errors commonly encountered after earthquakes is the formation of short column behavior.

If there is a difference between the lengths of columns on the same floor, it will cause a negative damage mechanism during an earthquake (Picture 3). Ribbon windows that are made from column to column in the halls of structures such as schools and hospitals and the presence of which are not considered in static projects can cause brittle fractures in columns.

Short columns form in structures mainly because of brick walls that are not load-bearing. Brick walls (partition walls) are considered to be dead loads in structural analysis. However, brick walls that are accepted as non-load bearing contribute to load transfer in the frame under horizontal loads as diagonal struts. They increase the rigidity of the frame and limit horizontal displacement (Fig. 1). In some situations, ribbon windows can be made in the upper parts of brick walls, just under the beam. This sit-

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uation is frequently encountered in the illumination of halls in dormitories and training centers as well as in basements. However, partial brick walls extending from column to column will prevent the horizontal translation of columns and cause the formation of short columns (Fig. 2).

As shown in Fig. 3, the shearing force that occurs in a short column is expressed as the ratio of the sum of moments at the top and bottom of the short column to the height of short column (Eq. 1).

$$V = (M_a + M_{ü})/l_k \quad (1)$$

M_a and $M_{ü}$ are the moments of the top and bottom parts of the column where the wall is not placed, and l_k is the length of the short column. As shown in Eq. (1), the shearing force in a short column is inversely proportional to the length of the short column, l_k. Accordingly, when l_k is short, the shearing force is high. The height of the short column is generally kept to 40–50 cm for lighting applications. Shearing forces due to earthquakes may cause great damage if adequate preventions are not taken in the short columns.

The walls are generally not considered in the model during the project stage. Structural frames with infilled walls and the model of the structure are shown in Fig. 4. In other words, the contribution of the rigidities of the infilled walls to structural behavior in the design stage is not taken into consideration. However, the rigidity of these elements is expected to affect structural behavior during an earthquake if the structure has both symmetric and asymmetric plans (Güney and Boduroğlu, 2006).

The following provisions and explanations related to short columns are given in the Turkish Earthquake Code. Short columns may be formed due to the bearing system or due to voids left between columns in infilled walls. If the formation of short columns is not prevented, the shearing force that will be experienced can be calculated as a function of lateral reinforcement as follows:

$$V_e = (M_a + M_{ü})/l_n \quad \text{and} \quad V_e \le V_r \quad (2)$$

$$V_e \le 0,22 \times A_w \times f_{cd} \quad (3)$$

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The moments in Eq. (2) are calculated at the bottom and top of the short column as $M_a \cong 1.4 M_{ra}$ and $M_{ü} \cong 1.4 M_{rü}$ and l_n is taken to be the length of the short column. However, the calculated shearing force will provide the conditions given in Eqs. (2) and (3). Throughout the short column, the minimum lateral reinforcement and settlement conditions described by the confinement zones of columns in TEC-2007 are applied. The lateral reinforcements are maintained for story height in columns that become short columns by remaining between infilled walls (Fig. 5).

Examples of damage that occurred in reinforced concrete buildings due to window bays left in infilled walls and column edges are shown in Picture 4 and Picture 5.

3.2.2 Damages depending on soft story–weak story irregularities

Some geometric and structural configurations addressed in the TEC-2007 in 2007 are described as irregularities, and it was determined that it is necessary to avoid the design and construction of irregular buildings due to the negative behaviors of these building during earthquakes. Irregularities are grouped into two categories: plan and verticality irregularities. There are three types of irregularities observed in verticality: strength Irregularity between adjacent floors (weak floor–B1), rigidity irregularity between adjacent floors (soft story–B2) and discontinuity of vertical elements of load-bearing system.

For one of two perpendicular earthquake directions in a reinforced concrete building, the rate of average relative displacement of any floor to that of the upper floor is defined as the rigidity disorder coefficient (η_{ki}) (Fig. 6). When this coefficient is greater than 1.50, then a B2 type irregularity is present in the structure (Isik, 2006).

$$\eta_{ki} = \frac{(\Delta_i)_{ort}}{(\Delta_{i+1})_{ort}} > 1.50 \quad (4)$$

In the calculation of average relative floor displacement, an additional eccentricity equivalent to ±5 % of the floor size in the earthquake direction should be considered.

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Equation (4) should be calculated for both the x and y directions, which are perpendicular to each other and belong to the ith and $(i+1)$th floors of the building. An operation should be carried out according to greater value.

According to TEC-2007, to determine whether there is a floor irregularity in a building, an η_c-criterion is applied. The criterion is described as follows, and an operation should be carried out according to the minimum value of η_c:

$$\eta_c = \frac{A_{ef,i}}{A_{ef,i+1}} \tag{5}$$

$$A_{ef} = A_c + A_s + 0,15A_m \tag{6}$$

Here, for the earthquake direction under consideration, the following parameters are defined:

$A_{ef,i}$ = sum of the effective cross-sectional areas on a floor

A_c = sum of the cross-sectional areas of columns on a floor

A_s = sum of the cross-sectional areas of partitions on a floor

A_m = sum of the cross-sectional areas of infilled walls on a floor.

The above operations are repeated for the x and y directions of the building, which are perpendicular to each other, and an operation is carried out according to Table 1 by considering the minimum η_c.

When damage to buildings caused by earthquakes has been investigated, it was found that the buildings that had few or no masonry-infilled walls, typically on the ground floor, experienced greater damage at the ground floor than on the upper floors (Tezcan et al., 2007). Masonry-infilled walls are definitely not considered by engineers in the internal force calculations for reinforced concrete load-bearing systems. Because masonry-infilled walls are not allowed to minimize the inner cross-section demands of the main load-bearing system elements such as column-beam-partitions by taking

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a share of the horizontal earthquake load, the designs for load-bearing systems are less than adequate. However, masonry-infilled walls play a large role in decreasing the horizontal displacements of the floor on which they are installed. The resistance of a ground floor lacking infilled walls against horizontal displacement is less than the resistance of upper floors that have many infilled walls. For this reason, a floor that has a rigidity discontinuity in the horizontal direction is called a soft story. If the height of the ground floor is greater than the upper floors, it creates a soft story irregularity. In order to provide space for the various commercial functions of shops, restaurants and banks, the ground floors that do not have infilled walls and/or have relatively high floor height are the focal point of earthquake damage in multi-story buildings (Tezcan et al., 2007).

Damage mainly occurs in a soft story when there are masonry-infilled walls on the upper floors but not on the ground floor. Because earthquake damage is caused by the lack of masonry-infilled walls, it is necessary to include masonry walls in the models used to calculate horizontal displacement. Otherwise, earthquake damage might occur unexpectedly due to excess horizontal displacement in a weak story. Accordingly, the rigidity provided by infilled walls should absolutely be taken into consideration during the calculation of horizontal displacement and especially during the determination of the elastic first natural pulse period.

Plastic hinges in soft story behavior occur when the lateral translation rigidity in the upper floors above the ground floor is greater and when the lateral rigidity of the ground floor is low. Horizontal earthquake forces cause great strain in the ground floor and increase the lateral displacement, causing plastic hinges to form. The structure cannot benefit from the extra rigidity provided by infilled walls, and an inconvenient situation develops that negatively affects the strength and translation of the ground floor. If non-load-bearing walls are not installed in the structure, the growing lateral translations will impair structure stability. If there is a discontinuity in the shear walls and if these regions are necessary for construction, transition zones should be reinforced adequately. When the ground floor rigidity is lower than that of other floors, large increases are

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observed in the strength of structure when horizontal loads are applied. Lateral translations increase in the structure, and plastic hinges occur in the columns. Sudden rigidity changes in the ground floor and non-elastic behavior can cause severe damage at the top points of the ground floor columns, and such damage is not desirable according to earthquake-resistant structure principles. Moreover, large lateral translations that occur in ground floor columns will cause second-degree moments (Isik, 2006).

In Picture 6, many examples are shown of soft story and weak story damages observed in the 2011 Van earthquake.

Many buildings were heavily damaged in the 17 August 1999, Kocaeli earthquake. Those buildings did not have masonry-infilled walls on their ground floors in order to leave space for commercial uses on Izmit and Golcuk Street and especially on AdapazarıInonu and Cark Street (M = 7.4). The upper floors of the incomplete five-floor building shown in Picture 7 were designed as residential floors, and the ground floor was designed for shops. This building, in which the ground floor was quite high and lacked masonry-infilled walls, was heavily damaged and collapsed due to weak story syndrome.

According to the η_c – weak story and η_k – soft story criteria in the TEC-2007, weak story and soft story irregularities were not found in this building. Instead, the building collapsed due to the weakness and softness of the ground floor. In this case, the TEC-2007could not identify a weak or soft story in the building. Thus, the design engineer and the assignors were misled as to the threat to human life because the building was shown through rose-colored glasses.

3.2.3 Damage caused by reinforcement detailing errors

Reinforcement detailing errors were frequently encountered in structures, especially those constructed before the establishment of the 1998 Earthquake Code. After the 2011-Van earthquake, typical reinforcement defects were observed in most of the heavily damaged buildings. Discontinuities in column lateral reinforcements in the joint re-

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gion and inadequate length of the reinforcement interlocking sections were the most common detailing errors.

It was observed that hooks were attached to the ends of the longitudinal reinforcements in old reinforced concrete structures. In Picture 8, a heavily damaged column element can be seen. The building, which was used as a primary school, contained many negative application and detailing defects. Beam lateral reinforcements were applied adjacent to the end region, and they were not integrated with concrete. Hooks were attached to the ends of the beam longitudinal reinforcements and continued at the end region, except for the lateral reinforcement. Column lateral reinforcements did not continue at the integration region of the column and beam. Moreover, the interlocking length of the column longitudinal reinforcement was inadequate. For these reasons, the reinforcements were pulled away from the integration region (Caycı, 2012).

According to the earthquake code, non-deformed reinforcement steel cannot be used except for stirrup and tie reinforcements that are combined with slab reinforcement. This decision was accepted in earthquake regulations in 1998. Non-deformed construction steel was used in many of the old buildings in Turkey's structural stock that were constructed before the 1998 earthquake code. The adherence of non-deformed steel with concrete is weak. Under reversible-repeatable loads, adherence is required by reinforcements that are consistently exposed to tensile stress and compressive stress to transfer the load to the concrete. After losing adherence, reinforcement steel will be pulled away from the concrete, and the column or beam elements will be removed, especially from the column-beam integration points (Picture 9).

According to the Turkish Standarts 500–2000 standard, the ore length (generally the corbelled additional length) is determined by the quality of the steel and concrete used. The ore length for ST 420 steel and C20 concrete (or corbelled additional length) should not be less than approximately 62 times the steel diameter. Accordingly, the ore length for Ø14 steel should be at least 90 cm, and for Ø16 steel, it should be at least 100 cm. For Ø18 steel, it should be at least 115 cm. Despite this, in applications, the ore lengths are kept short by disregarding the above rule and minimizing the amount

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of steel used. This weakens the joints of columns between floors. Thus, the columns break off during earthquakes if they are connected to sub-floors, and the building collapses.

According to the TEC 2007, the stirrups and ties will be rearranged in all earthquake zones, in the columns of all reinforced concrete systems that have high ductility or normal ductility, in the column-beam integrations, in the end zones of partitions and in beam confinement zones such as special earthquake stirrups and special earthquake ties. Stirrup hooks that satisfy the earthquake code are shown in Fig. 7.

At both ends of the special earthquake stirrups, corrugated hooks with a 135-degree angle must be present. The length of hooks must not be less than 10Ø and 100 mm in planiform bars and not less than 6Ø and 80 mm in ribbed bars, as measured from the last point of tangency.

Making stirrup hooks with 90° angles is straightforward. Bending stirrup hooks to an angle of 135° increases the production time and labor costs. Because the importance of the correct angle has not been well understood, defective manufacturing has been very common.

A concrete core wall under axial pressure is forced to undergo lateral deformation due to the Poisson effect. The stirrups resist the lateral deformation of the concrete core wall by using longitudinal reinforcements as structural bearings. The prevention of lateral translation results in an increase in compressive strength in the concrete core wall. Moreover, the ductility of elements also increases significantly.

Another contribution of stirrups to element strength is that they shorten the bending length of the longitudinal reinforcement. Bending load will increase due to shortening of the bending length of the longitudinal reinforcements under axial pressure.

When stirrup hooks are made with a 90° angle, the reinforcement hook only bonds with the concrete core wall. The concrete core wall first flakes off by cracking because it is not exposed to the confinement effect under repeatable loads. In this case, the stirrup hook will easily open outwards because the shell concrete confinement of the stirrup is removed. The opening of the stirrup will result in loss of the winding effect on the

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concrete core wall, which is forced to expand outwards. Moreover, because the buckling length of the longitudinal reinforcement is increased, buckling in the reinforcements can easily occur. Element ductility and bearing capacity will decrease. In Picture 19, various examples are given for the opening of stirrups and the buckling of longitudinal reinforcements.

According to the Turkish Earthquake Code, special confinement zones must be placed at the bottom and top of each column. The length of each confinement zone must not be less than the maximum column cross-section, 1/6 of the column clear headroom or 500 mm. In the confinement zones, lateral reinforcements with less than Φ8 diameter must not be used. In this zone, the stirrup range in the longitudinal direction must not be more than 1/3 of the smallest cross-sectional dimension or more than 100 mm, and it must not be less than 50 mm.

In columns with stirrups, the minimum total lateral reinforcement area in the confinement zone should be calculated based on the given conditions in Equations 7 and 8 when $N_d > 0.20 A_c \times f_{ck}$ in this calculation, the core size of the column b_k is taken into consideration separately for each direction.

$$A_{sh} \geq, 30 s b_k \left[\left(\frac{A_c}{A_{ck}} \right) - 1 \right] \left(\frac{f_{ck}}{f_{ywk}} \right) \quad (7)$$

$$A_{sh} \geq, 075 s b_k \left(\frac{f_{ck}}{f_{ywk}} \right) \quad (8)$$

s: Longitudinal reinforcement interval, spiral reinforcement step

b_k: Core size of column

A_c: Gross cross-sectional area of column or partition end zone

A_{ck}: Column concrete core wall area within the measurement taken out-to-out of confinement reinforcement

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f_{ywk}: Characteristic yield strength of lateral reinforcement

f_{ck}: Characteristic cylinder compressive strength of concrete.

In the zone between the confinement zones at the bottom and top of the column (the middle zone of column), a lateral reinforcement with less than Ø8 diameter must not be used. A stirrup, tie or spiral interval throughout the column must not be more than half of the smallest cross-sectional dimension or more than 200 mm. The horizontal distance between stirrup levers and/or ties, *a*, must not be more than 25 times the diameter of the stirrup.

In the TEC-2007, although confinement zones are defined at the end points of columns and beams and although it was determined that the stirrup frequency in the confinement zones should not be less than 10 cm, it was observed that such confinement zones were not formed in reinforced concrete elements damaged in the earthquake (Picture 10). It was observed that the stirrup frequency at the end zones of the columns and beams was approximately 20–30 cm. Plastic hinges are formed at the column ends due to strong beam-weak column formation under reversible loads. Because of the infrequent stirrups under excess spinnings caused by the ductility request and because hooks with 90° adequate ductility were not obtained, correspondingly heavy damage was observed in many of the buildings. Inadequate concrete quality also contributed to these failures. In Picture 11, examples of column damage and stirrup frequency are given.

The most constraining zones within the framework under earthquake loads are column-beam integration zones. These zones must withstand both the reversible moment and the axial load resulting from the column and reversible moments that the connected beams are exposed to. The increased frequency of stirrups used in the confinement zones of columns in these zones must be continued exactly. Moreover, columns and beams connected in this zone should provide enough overlapping length to provide longitudinal reinforcements. The decisions related to overlapping length for longitudinal reinforcements at integration zones in the TEC-2007 are given in Fig. 8.

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The stirrups in the confinement zones of columns should be continued with the same frequency in the column-beam integration zone. These zones are the most important ones in terms of both the bending moment and the shearing forces under reversible-repeatable loads. The stirrups increase the spin capacity of the element joint by increasing ductility and contribute to shear strength. However, the continuity of column stirrups in integration zones is difficult to maintain during manufacturing, and it is a detail that has not been included until recently. Most people do not understand the importance of it. Examples of damage in column-beam integration zones are given in Picture 12.

3.2.4 Damage caused by weak column–strong beam issues

All modern earthquake codes specify that columns should be stronger than beams. Thus, the plastic hinges under horizontal earthquake loads will form first on beams and then on the columns at the same node. Before system stability is lost, the number of plastic hinges formed on beams is greater than the total number of hinges formed by the same hinges on the column. Thus, more hinges form in the system, and the ductility of the system increases due to greater translation of the system and a greater amount of energy consumed. Moreover, the axial load-bearing level of the beams is lower than that of columns, so the ductility of the beams under bending moments will be more than that of columns. Thus, the stability loss of the columns will cause the collapse of the structure. However, even when hinges form in all of the beams, the building may not collapse. Repair or reinforcement after an earthquake by adding hinges to columns is difficult and may be economically impossible, but repair and reinforcement of a damaged building by adding hinges to beams is easier. For these reasons, the TEC-2007 states that "in a load-bearing system constituting of only frameworks or a combination of partitions and frameworks, the total bearing capacity moments of the columns integrating at each column-beam node point (Fig. 9) must be at least 20 moments in cross-sections of the column surfaces of beams integrated at that node

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point".

$$M_{ra} + M_{rü}) \geq 1.2\left(M_{ri} + M_{rj}\right) \quad (9)$$

(M_{ra}: bearing capacity moment calculated by taking f_{cd} and f_{yd} of the column under the node point as a basis

$M_{rü}$: bearing capacity moment calculated by taking f_{cd} and f_{yd} of the column above the node point as a basis

M_{ri} and M_{rj}: bearing capacity moments calculated by taking f_{cd} and f_{yd} at the beam ends as a basis.

This equation will be applied separately in each earthquake direction, and it will give unfavorable results for both directions of the earthquake. In the calculation of column bearing capacity moments, N_d axial forces that result in the smallest moments for the direction of the earthquake will be taken into consideration.

The reason for the condition determined in Eq. (9) is that beams should be more ductile than columns because ductility decreases as axial load increases. By taking this reality into consideration in the code, it is possible to ensure the formation of plastic hinging on more ductile beams, which is unavoidable in big earthquakes (increase in curvature with tension reinforcement yielding under constant moment). This condition is determined by Eq. (9) as "Columns should be stronger than beams" (Ersoy and Özcebe, 2004).

Although it was stated in the earthquake code that columns should be stronger than beams and that hinging occurred in columns investigated after earthquakes, it was observed that the damage to beams was limited and that in some cases there was no damage. Several examples of this type of damage, which is called strong beam-weak column damage, are given in Picture 13. As the damage becomes progressively worse, hinges occurred in the columns, decreasing the horizontal stability and causing the collapse of building in the way that floors collapse one after another, which is called a

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"pancake" collapse. Pancake collapses are the most common type of collapse (Picture 14).

One of the most commonly encountered problems in damage investigations after earthquakes is that the buildings do not have adequate lateral rigidity or an adequate layout of shear walls. Ductile behavior is desired in a structure under earthquake loads, and all structure systems should have adequate rigidity. More horizontal translation will cause damage to elements such as non-load bearing partition walls, and this will increase the economic losses. Moreover, second-degree forces will cause more movement of the elements. For these reasons, the TEC 2007 imposes restrictions on the horizontal displacements that occur in floors under horizontal loads.

For each earthquake direction, the maximum value (δ_i) max of calculated effective relative floor translation in columns or partitions on the *i*th floor of a building must satisfy the following condition:

$$\frac{(\delta_i)_{\max}}{h_i} \leq 0.02 \tag{10}$$

If this condition is not satisfied on any of the floors of a building, the earthquake calculation will be repeated by increasing the rigidity of the load-bearing system. However, even if the condition is satisfied, the use of nonstructural brittle elements (frontal elements, etc.) under effective relative floor translation will be confirmed by the calculations.

Buildings must have adequate shearwalls in both directions in order to have adequate lateral rigidity. However, the construction of shearwalls is not sufficient to rehabilitate old buildings. The building shown in Picture 15 was constructed near Van Lake.

Picture 16 shows examples of earthquake damage and collapse observed in reinforced concrete buildings with insufficient lateral rigidity.

Picture 17 shows examples of damage in non-bearing brick-infilled walls as a result of excess translation of buildings.

Walls with upper sides that are clear and supported at the bottom, such as vertical corbels or garden walls, frequently collapse under horizontal loads because the gable

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walls are not combined with overhead beams in garrets. The collapse of these walls does not pose a problem in terms of the veering of the building. However, the collapse of these walls on the street or in other buildings might put people in danger. Picture 18 shows examples of gable wall damage.

In the buildings shown in Picture 19, damage occurred due to the displacement of load-bearing system elements. All columns in the building are positioned vertically relative to the street, and there are no columns parallel to the street. It is difficult to know the angle of the earthquake that affected the building, but the component of the earthquake that is parallel to the street caused heavy damage in the building.

3.2.5 Inadequate lateral rigidity

Rigidity

A structure must have adequate rigidity to minimize the second-degree moments as much as possible and to reduce the non-structural damage in earthquakes that correspond to the serviceability limit state. The most important criteria for structural rigidity under the effect of horizontal loads are the rigidity of the element and the translation of the floor relative to a lower floor in the structure.

For rigidity, the positions of the vertical bearings and their dimensions in both directions are very important, more so than the geometry of the structure. In a reinforced concrete structure, because the columns and partitions behave more rigidly as vertical load-bearing elements than infilled walls, it is sufficient to take these elements into consideration in the calculation. In the calculation of rigidity for vertical load-bearing elements, the properties of the materials used in the elements (elasticity modulus), the cross-sectional dimensions of the element and the supporting types of element ends must be considered.

Relative floor translation is related to displacements that might occur in an earthquake, and the structural rigidity in the earthquake code defines the displacement difference between two floors as $i = d_i - (d_i - 1)$. For one of the two perpendicular earth-

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quake directions, if the ratio of the maximum relative floor translation in any floor to the relative translation in the same direction in that floor (which is defined as the Torsion Irregularity Coefficient) is greater than 1.2, then it is described as a torsion irregularity. To prevent torsion irregularities, attention should be paid to the displacements of partitions and rigid load-bearing elements in order to increase the torsional rigidity of the system (Aktan and Kıraç, 2010).

Displacement of partitions in the plan

In reinforced concrete structures, a torsional moment occurs due to the geometry of the system in constructional elements or non-symmetrical loadings. The Torsional Rigidity of the system changes according to the cross-section of the partition and its displacement in the plan.

The partitions should have buckling stability and symmetric rigidity and should be secure against toppling in the basement. When placing partitions in the plan, it is important to provide the distribution of expected plastic strains accurately in the building plan. Otherwise, some partitions will be compressed more, and some of them will be compressed beyond their capacities. In order to provide adequate rigidity in a highly partitioned construction, at least three partitions should be used in which the system lines do not pass on a point. In order to eliminate torsion formation in a building experiencing an earthquake, a conflict between mass and rigidity centers is required. Earthquake forces affect the center of mass in a floor. If the center of mass does not conflict with the rigidity center, the construction will turn around the center of rigidity. When the horizontal force affecting the center of mass is moved to the center of rigidity, as is the F_y force, a torsional moment with a value of $M_z = F_{ye}$ will affect the center of rigidity. In fact, if the earthquake load affects the center of rigidity directly, the torsional moment will not occur because the construction will produce equal translation in the direction of the earthquake force. Because the torsional effect on the floor is the product of the shearing force affecting the vertical elements and the moment arm, the torsional effect in the partition or framework with the largest moment arm will be higher.

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To decrease the torsional effect on partitions, it is necessary to arrange the partition systems in an ideal way. This is done as follows:

1. In order to provide maximum torsional rigidity in the construction, shearwalls should be distributed around the construction. In Fig. 10, suitable partition placements with high torsional rigidity are shown.

2. Shearwalls should be arranged in the floor plan so that most of the floor loads are transferred to the basis as axial force. In this way, the reinforcement required for the bending moment in the partition is reduced.

3. In multi-story buildings, putting the earthquake-resistance into few partitions exposes the basis system to a big earthquake effect at a few points. This situation should be avoided because it requires an expensive heavy basis system.

4. Shearwalls should be placed in both directions (Aktan and Kıraç, 2010).

Picture 20 shows examples of damage that occurred during earthquakes due to the placement of partitions in the plan.

3.2.6 Damage that depends on the Daubling Effect

Earthquake joint

Many building bylaws allow adjoined constructions (attached). These attached constructions are not well-protected against earthquakes. They transfer earthquake force to each other, and they collide as a result of oscillation. The last type of construction, called a street corner, is generally damaged very heavily (Turk, 2011).

A new building that is built next to an old one is separated with a joint. There is an attempt to prevent collisions of the two buildings due to different oscillations in the earthquake, which would cause the buildings to damage each other (the daubling effect). When a new construction site has bossage or big spaces in the plan and/or in

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the vertical direction, it is deblocked into symmetric and rectangular spaces as much as possible. Blocks are constructed with a joint between them. Theoretically, the joint width between blocks should be greater than the sum of the maximum horizontal displacements of both constructions:

$$d \geq \Delta_a + \Delta_b \tag{11}$$

If one of the constructions is old, its displacement is generally not known. In this case, it is recommended to take the joint width as a minimum:

$$d \geq 0,02\frac{H(\text{cm})}{3} \tag{12}$$

For example, if $H = 15$ m, then the joint width in the construction will be at least

$$d = 0,02\frac{1500}{3} = 10\,\text{cm} \tag{13}$$

The basis of blocks separated by an earthquake joint should be separate (Turk, 2011).

Picture 21 shows examples of the types of damage that occur when there is not an adequate earthquake joint between attached buildings.

3.2.7 Damage observed in non-load-bearing construction elements

The construction elements that do not have load-bearing properties in buildings but cause maximum damage during an earthquake are the partition walls. Partition walls are built to separate the usage areas physically within a building and to protect internal volumes in the edge axis from the outside, and they are generally made of hollow bricks in our country. However, the outer walls that are called sandwiched walls due to climatic conditions in East Anatolia Region are made of two-row hollow bricks, generally with polystyrene foam and glass wool between the bricks to provide heat insulation.

In two-row brick walls that are not connected to each other mechanically throughout the wall gaps, diagonal cracks and big out-of-plane motions have been observed. For

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this reason, there have been many cases of buildings with no damage in load-bearing components but with severe loss of life and property due to wall damage (Picture 22) (Celebi et al., 2011).

3.2.8 Damage depending on the suitability of materials used in load-bearing elements

In the concrete that is generally used for construction, segregations and reinforcement placement defects are observed. Such defects were observed in all investigated buildings (Picture 23).

4 Results

The most important causes of damage during earthquakes have to do with building not being constructed according to modern codes and standards.

The only way to construct earthquake-resistant buildings is to be aware of the requirements and to avoid the errors of the past. It is necessary to have good communication and collaboration between architects, civil engineers, geologists, urban and regional planners and related professional organizations for the construction of earthquake-resistant buildings.

It is impossible to predict earthquakes, but it is possible to construct earthquake-resistant buildings. Visual values such as aesthetics and artistic value should be secondary to safety in a construction project.

The construction of earthquake-resistant buildings is the joint responsibility of engineers and architects, and it necessitates the cooperation and systematic study of occupational fields.

The most important thing during an earthquake is the strength of the building. For this reason, no matter how perfect the calculations are, if the manufacturing and construction processes are not performed carefully, the building will not perform as expected in

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the event of an earthquake. Thus, it is necessary to instill the control mechanisms that are currently lacking in our country as soon as possible.

Because earthquakes affect buildings as horizontal loads, an adequate number of shearwalls should be placed to increase the lateral rigidity and decrease translation during the construction of a load-bearing system. Architects and engineers should investigate and understand the factors that cause damage in earthquakes.

In the education of architects and engineers, earthquake and earthquake-resistant construction design should be considered important, and previous graduates should have their education updated via seminars or courses.

In Turkey, what is said after every earthquake is the same as what was said after previous earthquakes. The defects and errors that cause damage do not change. If we do not want to experience the same negative consequences, we should identify the problems correctly and take the necessary precautions.

Three causes of earthquake damage can be identified:

- Design errors (soft story, inadequate lateral rigidity, short column, strong beam-weak column integration, irregularities in vertical and horizontal directions, etc.)
- Construction stage errors (poor workmanship, low strength of materials, inadequate transverse reinforcement–stirrup usage, defective and inadequate interlocking length, lack of control)
- Errors in usage (cutting column, soft story formation by complete or partial removal of walls, etc.)

In the strongest earthquakes, especially those that cause permanent damage to buildings, it is necessary to obey the criteria given in codes for the controlled distribution of damage in construction in order to avoid a collapse. The following suggestions should be taken into consideration in construction to avoid damage and collapse:

(a) The construction system selected should be as simple as possible and should be a system that can be easily understood by everybody involved in the project.

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(b) Various load-bearing components of the building may have the property of sharing horizontal loads.

(c) Plastic deformations should be received by all components without loss of stability under vertical and horizontal loads. A capacity design should be used, and the maximum horizontal translation should be enhanced within the limits given in code.

When a designer pays attention to these details, the required conditions can generally be provided for the elastic and ductile behavior of a building.

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Table 1. Operations according to the weak story criterion η_c.

η_c	Operations
$0.80 \leq \eta_c < 1.00$	No irregularity
$0.60 \leq \eta_c < 0.80$	Increase floor shearing force by dividing by 1.25 η_c.
$0 \leq \eta_c < 0.60$	Increase the number and/or size of the columns, partitions and infilled walls of the ground floor until a value of $\eta_c/0,60$ is reached.

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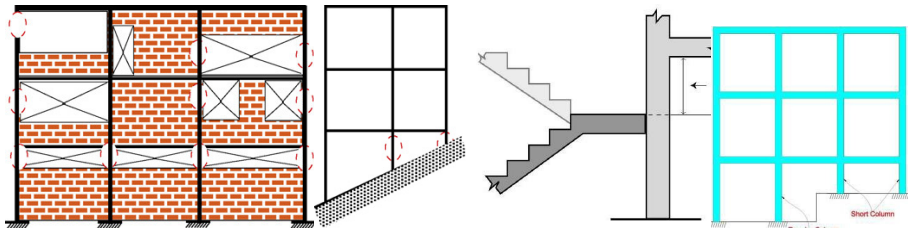

Figure 1. Short column formations in structures due to various causes.

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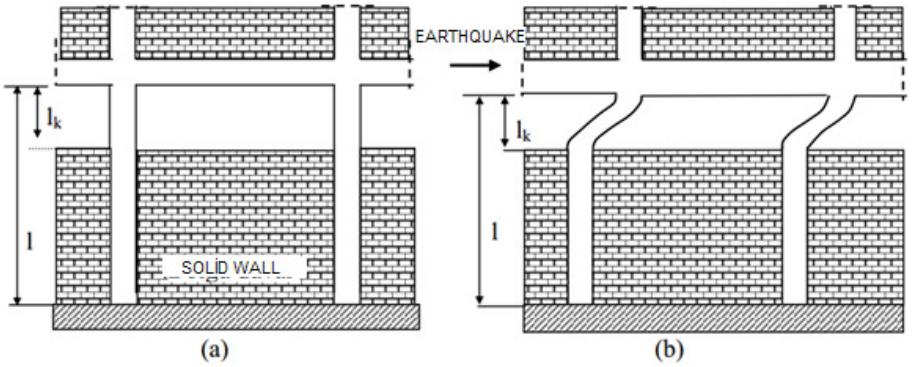

Figure 2. (a) Structural frames with infilled walls, **(b)** Short column behavior (Cağatay, 2007).

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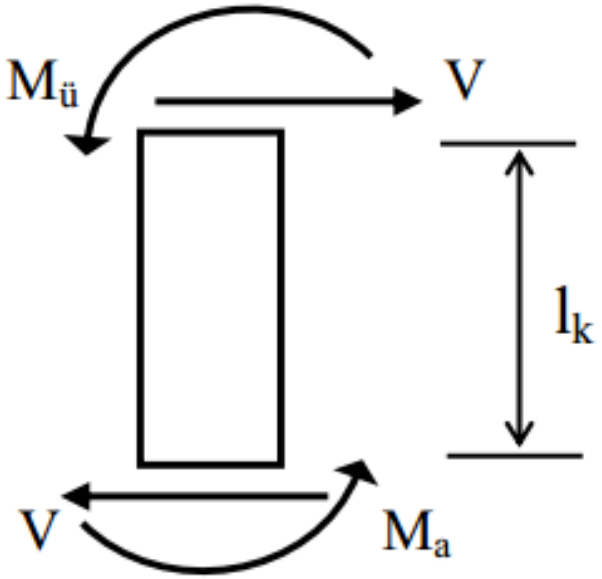

Figure 3. Moments affecting short columns and shearing force (Cağatay, 2007).

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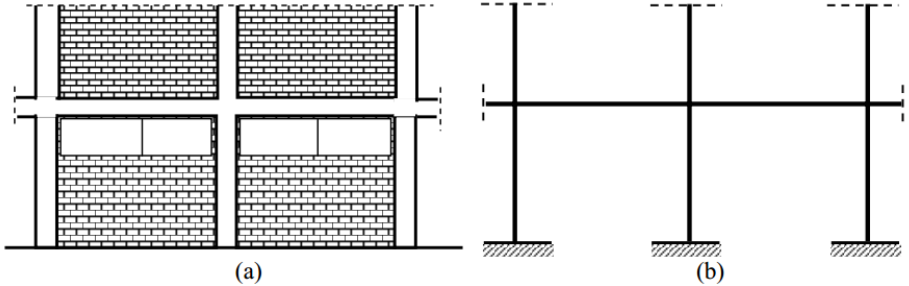

Figure 4. (a) Structural frames with infilled walls. **(b)** Model of the structure (Cağatay, 2007).

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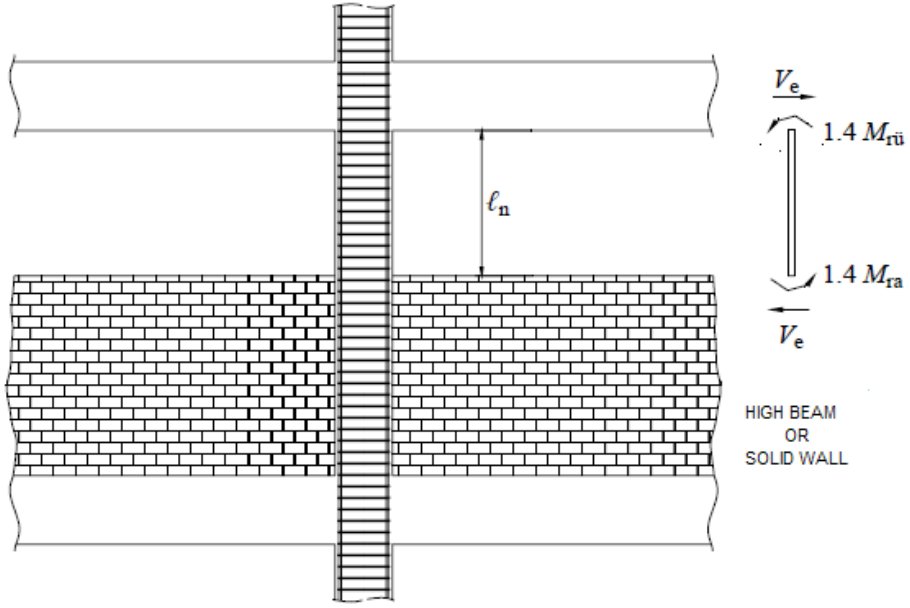

Figure 5. TEC-2007.

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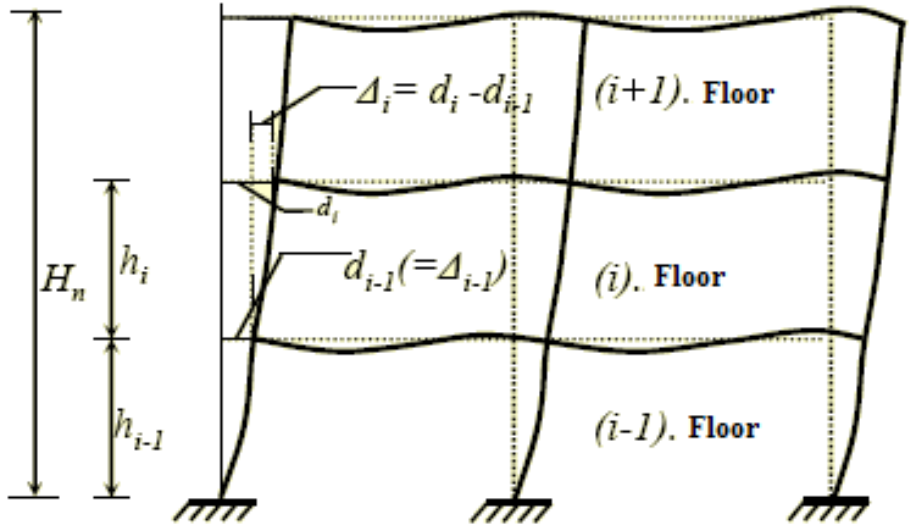

Figure 6. Calculation of relative floor translations.

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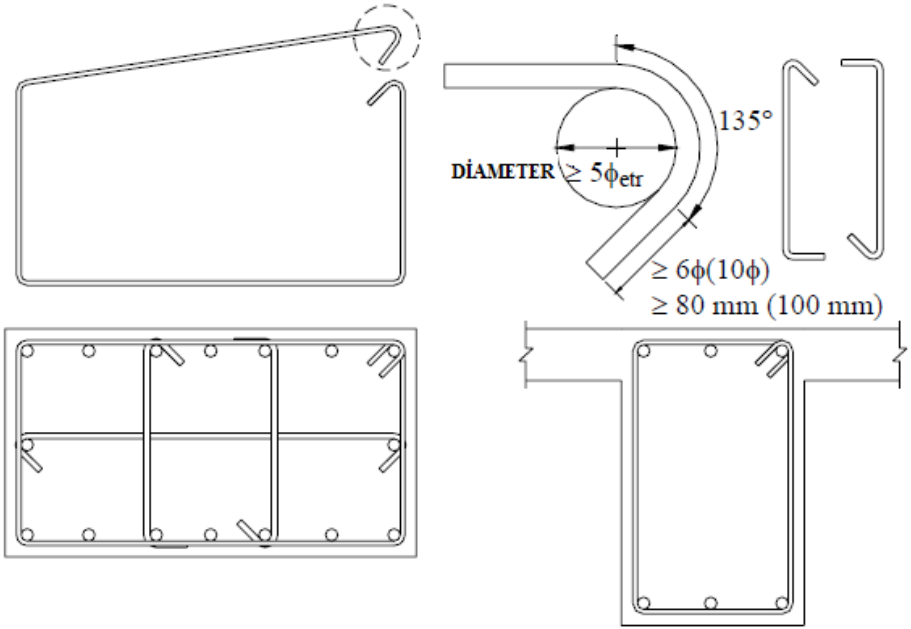

Figure 7. Stirrup hooks designed according to earthquake code (TEC-2007, 2007).

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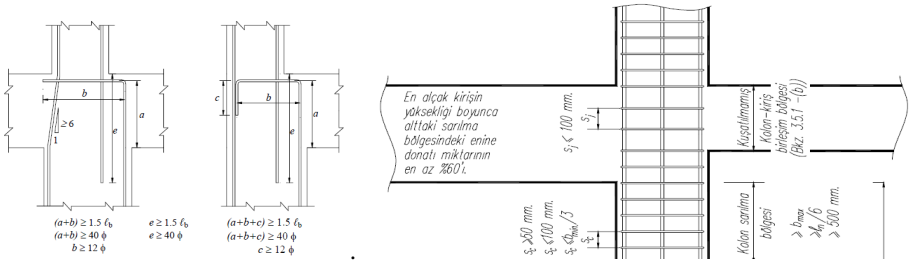

Figure 8. TEC-2007.

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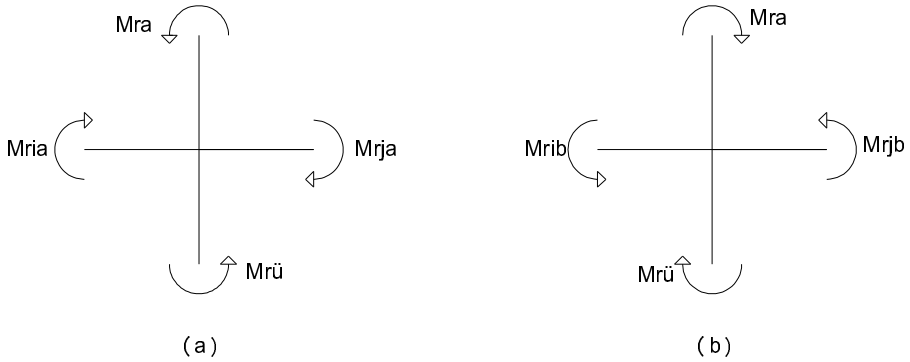

Figure 9. Bearing capacity moments affecting the node point (Ersoy and Özcebe, 2004).

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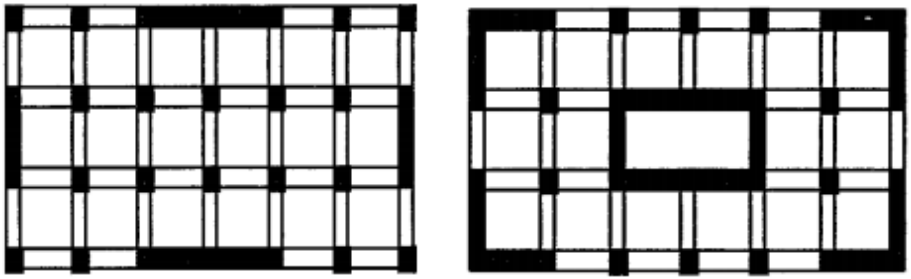

Figure 10. Suitable partition placements (Aktan and Kıraç, 2010).

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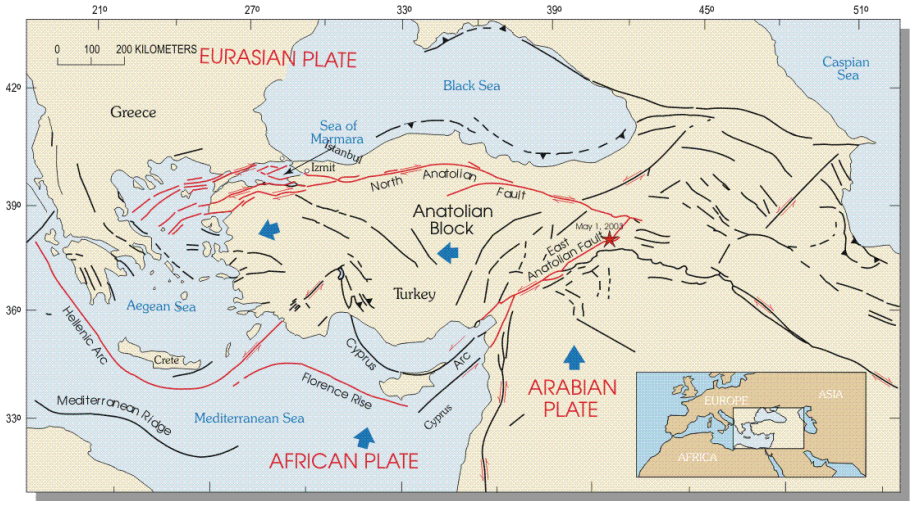

Picture 1. Plate motions in Turkey and the surrounding region (USGS, 2011).

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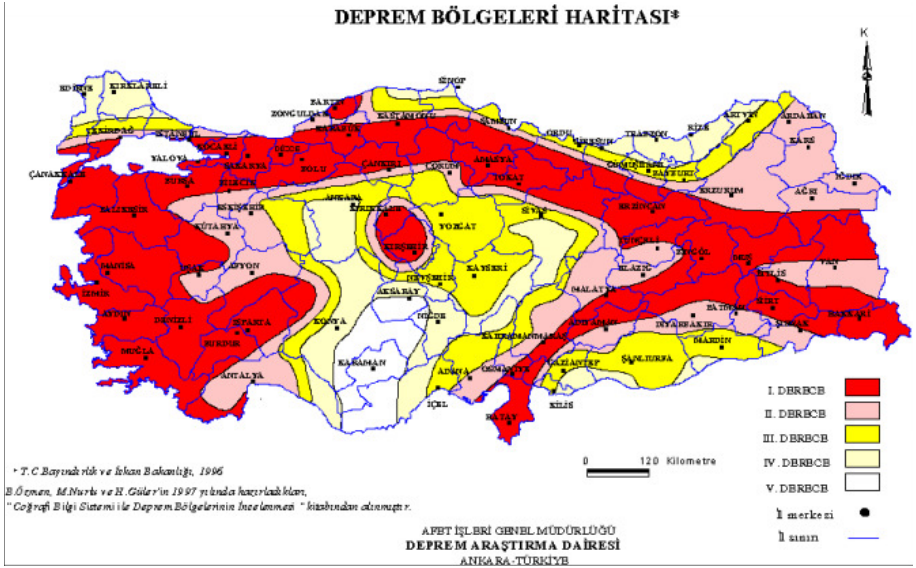

Picture 2. Map of seismic zones in Turkey.

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Picture 3. Several short column failures observed after earthquakes (Isik, 2006).

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Picture 4. Examples of damage caused by the formation of short columns (Middle East Technical University, 2011).

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Picture 5. Short column formation caused by ribbon windows (Middle East Technical University, 2011).

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Picture 6. Examples of damage related to soft-story and weak story irregularities (source: Ahmet Topçu).

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Picture 7. Heavy damage in the weak story of a five-floor building in Adapazari (17 August 1999, Kocaeli Earthquake, M = 7.4) (Isik, 2006).

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Picture 8. A reinforced concrete structure that was heavily damaged at the integration region during the 19 May 2011, Simav Earthquake (Caycı, 2012).

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Picture 9. Pulling away of longitudinal reinforcements.

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Picture 10. Damage caused by reinforcement detailing errors (source: Erdal Camcı).

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Picture 11. Damage caused by excess stirrup intervals in the confinement zones of columns (İKÜ-CE-2011/01, 2011).

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Picture 12. Damage to column-beam integration zones (İKÜ-CE-2011/01, 2011).

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Picture 13. Examples of strong beam-weak column damage.

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Picture 14. Full collapse of a building as a result of plastic hinges in the bottom and top zones of columns in a reinforced concrete structure in Van-Ercis (Onen et al., 2011).

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Picture 15. Buildings that do not have adequate shearwalls.

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Picture 16. Damage observed in buildings with insufficient lateral rigidity.

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Picture 17. Damage observed in non-bearing brick-infilled walls.

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Picture 18. Gable wall damage (www.dramaistanbul.com.tr).

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Picture 19. Earthquake damage in a building with columns placed in the same direction.

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Picture 20. A building that collapsed due to contributions of lateral translation and torsion (Anonymous, 2013).

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Picture 21. Reinforced concrete multi-story building that collapsed by colliding with adjacent buildings due to a daubling effect (Inel et al., 2011).

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Picture 22. Damage in the outer walls of a reinforced concrete building in Van city due to separations in the walls (Celebi et al., 2011).

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Picture 23. Concrete and aggregate samples in a single-story reinforced concrete building that collapsed in the Van Organized Industrial Site (İKÜ-CE-2011/01, 2011), and a concrete cross-section displayed in the wreckage (Kızılkanat et al., 2011).

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