Digest 2011, December 2011, 1477-1497

A Comparative Study on Structural Wall Design Approach of 2007 Turkish Earthquake Code[†]

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ABSTRACT

The current version of the Turkish Earthquake Code (TEC-07) was released in 2007 and the capacity design approach for high ductility structural walls was introduced for the first time. The practicing engineers, however, mentioned various problems especially related with the shear design section of high ductility structural walls. Because of this reason, an in-depth comparative study was conducted to examine the differences and similarities of the design approaches of TEC-07, American Reinforced Concrete Code (ACI 318-08) and various studies in the literature that focus on capacity design of structural walls. In addition to this, differences between normal ductility structural walls of TEC-07 and ordinary walls of ACI 318-08 were also examined. It was observed that in the current version of the TEC-07, several clauses related to structural wall design needs further considerations. Several modifications were proposed in order to improve the structural wall design of the current code.

***Keywords:** Structural wall design, Turkish Earthquake Code, High Ductility, Medium Ductility*

1. INTRODUCTION

The current version of the Turkish Earthquake Code (TEC-07) was released in March 2007 [1]. Major changes to the design provisions for structural walls, especially at high ductility level, were incorporated into this new version code and the capacity design approach for structural walls was introduced for the first time. Shortly after its release serious design problems arose, particularly in the design of structural wall dominant buildings. It was a general consensus among structural engineers that it was almost impossible to design buildings with tunnel form systems with the new structural wall design provisions. As a result of these and similar objections, the TEC-07 was updated in May 2007 (two months after its release) and requirements related to the shear design of structural walls were modified. Discussions on the updated version of the code, however, have not come to a

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† Published in Teknik Dergi Vol. 22, No. 3 July 2011, pp: 5487-5508

conclusion. Especially, the necessity of capacity design for structural wall dominated buildings with limited interstory drifts has been questioned and no satisfactory answer has been given to what must the design forces be for foundation design.

It was noticeable that, the revisions made on the TEC-07 regarding the capacity design of high ductility structural walls were mostly inspired by the studies of Paulay [8, 9]. For the design of ductile structural walls, Paulay [8, 9] proposed the calculation of a flexural overstrength factor, which was the ratio of the moment capacity of the wall at its base to the factored bending moment obtained from analysis. The maximum shear force that could occur at the wall base was then computed by multiplying the shear force obtained from the analysis by this flexural overstrength factor. In addition to this, Paulay suggested that the effects of higher modes became more pronounced when the structural walls behave nonlinearly and introduced a new factor to account for higher mode effects. Thus, the magnified shears forces are further multiplied by this additional factor. Paulay also pointed out that the design moments and the magnified shear forces should be distributed along the wall height in accordance with a design envelope. Wallace [11], on the other hand, presented different coefficients for the consideration of the higher mode effects. The design approach adopted in the Turkish Earthquake Code for the high ductility structural walls has considerable similarities with works of Paulay [8].

The capacity design of structural walls in regions of high earthquake risk is addressed in Eurocode as well. In addition to the ordinary structural walls that may be used in low seismicity regions as described by EC2 [6], two additional types of structural walls are defined by EC8 [5]. These walls are named as DC-M and DC-H. Here DC-M represents a medium ductility wall and DC-H stands for a high ductility structural wall. In medium ductility structural walls (DC-M) the design shears may be comparatively lower than those used for high ductility structural walls (DC-H). Both types of structural walls, however, have almost similar detailing requirements. In EC8, the design moments are derived from a design envelope defined along the height of the structural wall. In the current Turkish Earthquake Code, an identical approach is used to determine the distribution of the design bending moments along the wall height. Furthermore, for ductile walls, EC8 also requires the use of a design envelope for shear forces. In EC8 although there is a general consensus on the design envelope of bending moments, studies on the design envelope for shear forces are not yet conclusive [13].

In ACI 318-08, however, the design of special structural walls is performed following a displacement-based approach that involves the estimation of compressive strains at the wall base caused by the earthquake induced wall displacements [2]. The need for special boundary elements at the wall edges is evaluated based on this value. This approach will be discussed more extensively in the following section. The displacement-based approach, which was first introduced in ACI 318-99, allowed considerable reductions in the amount of longitudinal reinforcement used in structural walls, thereby, enabling the flexural overstrength factor to remain at minimum levels [12]. The capacity design of structural walls in shear is not addressed in ACI 318-08; but instead shear stress limits have been set.

The capacity design of structural walls has not been adopted by ACI 318-08, primarily because in the past earthquakes, no noticeable damage has been observed in conventionally designed structural walls without any special detailing [14]. Besides, it has been demonstrated in many experiments that structural walls designed with boundary elements that do not obey the detailing requirements of the TEC-07 and/or EC8, displayed considerably ductile behaviors [10, 16].

Two different design approaches are utilized in different buildings codes for the shear design of high ductility structural walls. The capacity design approach has been accepted by both the TEC-07 and EC8. On the other hand, a variety of problems is still being mentioned related to the structural wall design approach of the Turkish Earthquake Code. Thus, in this study, the design of high ductility structural walls was studied in detail and structural wall design approach of ACI 318-08 and various studies related to the capacity design approach were compared. Besides, a comparison between the TEC-07 and ACI 318-08 for buildings located at low seismicity regions was also presented. By studying the structural wall design provisions of the Turkish Earthquake Code in detail and in comparison to other standards, it was intended to clarify what must be done in order to overcome the shortcomings of our current code.

2. STRUCTURAL WALL DESIGN APPROACHES of TEC-07and ACI 318-08

2.1 High Ductility Structural walls

Flexure Design Approaches

TEC-07 assumes that a plastic hinge occurs along the critical height of a high ductility structural wall and the design and detailing of the wall is performed accordingly. The critical height of a wall is defined in Article 3.6.2.2 of TEC-07 as being equal to the length of the wall (l_w) or one sixth of its height (h_w), whichever is larger. Moreover, it is mandatory to form boundary elements with a minimum length of 20% of the wall length ($0.2l_w$) along the critical height of the wall and 10% of the wall length ($0.1l_w$) elsewhere.

The first step in the design of flexural structural walls to revise the base bending moments obtained from the structural analysis according to the rules defined in Article 3.6.6.1 of TEC-07. In other words, the moment values along the critical height are set equal to the base moment. This way, the plastic hinging along the critical height of the wall was considered. Area of the longitudinal reinforcing bars at the boundary elements must be at least 0.2% of the total cross-sectional area of the wall along the critical height, whereas above the critical height, the minimum reinforcement ratio for boundary elements was defined as 0.1%. Moreover, the minimum reinforcement ratio for the middle regions of the wall and maximum allowed spacing of the longitudinal bars was stated to be 0.25% and 250 mm, respectively.

According to ACI 318-08, the design of high-ductility structural walls (special structural walls) starts with the determination of longitudinal reinforcement requirement for the design axial load and bending moment pairs (N, M). It should be emphasized here that bending moment values taken from the analysis are directly used in this step without any modification. In ACI 318-08, the requirements on the minimum longitudinal bar ratio and maximum bar spacing are 0.25% and 450 mm, respectively. After determining the longitudinal steel requirement and the spacing of the reinforcement, the designer has to make a check to see if boundary elements are required. For this purpose the location of the neutral axis (c) must be determined. The code stipulates construction of the boundary elements along the critical height of the column when the condition stated in Equation 1 is satisfied. (ACI 318-08, Article 21.9.6.2).

$$c \geq \frac{l_w}{600\left(\delta_u / h_w\right)} \tag{1}$$

In Equation 1, δ_u is nonlinear lateral top displacement; h_w is total height; and l_w is length of the wall. If the formation of boundary elements are required, their lengths are defined as the larger of $c - 0.1l_w$ and $c/2$. This design approach is based on the maximum expected lateral displacement structural wall and the corresponding axial strains at the end regions of the walls during an earthquake. Once the condition given in Equation 1 is satisfied, the designer should expect axial strains greater than 0.003 at the end regions of the walls. Thus, concrete in those regions should be carefully confined with transverse reinforcement in order to increase the ductility of the wall.

Shear Design Approaches

According to ACI 318-08, the adequacy of the structural wall cross sectional areas is checked by employing two basic conditions. These two conditions are defined in Equations 2 and 3, respectively. Equation 2 describes the limiting case for the total shear force resisted by all structural walls at each story; whereas Equation 3 establishes a capacity check for one single structural wall. In each case, shear forces are directly obtained from the results of structural analysis without any modification. If either of the two conditions is not satisfied, cross sectional area of the structural wall must be increased (ACI 318-08, Article 21.9.4.4).

$$V_{Tu} \leq 0.66 A_{Tcv} \sqrt{f_c'} \tag{2}$$

$$V_u \leq 0.83 A_{cw} \sqrt{f_c'} \tag{3}$$

In Equations 2 and 3, V_{Tu} is the total shear force acting on a story; V_u is the shear force that has to be resisted by an individual structural wall; A_{Tcv} is total cross-sectional area of structural walls in one single story; A_{cw} is the cross-sectional area of a structural wall; and f_c' is the characteristic compressive strength of concrete (f_{ck} in TS500). With the help of Equations 2 and 3, ACI 318-08 intends to provide a consistent distribution of shear forces in a structural wall system by setting a maximum value for the shear strength of each wall.

Before calculating the shear capacity of a wall, its dominant response mode should first be determined. Depending on the governing deformation state of the wall, the walls were tagged as shear or flexure dominant walls and the strength reduction factor, Φ, is set accordingly. This deformation state of a wall is directly related to its length over height ratio. If a structural wall is not long enough to bend, it will mainly undergo shear deformation and attract high shear stresses. For such a case, Φ factor in Equation 4 is taken 0.6. Otherwise, it can be taken equal to 0.75. Lateral reinforcement ratio to be used in the design of the structural wall is determined from Equation 5 (ACI 318-08, Article 21.9.4.1).

$$\phi V_n \geq V_u \tag{4}$$

$$V_n = A_{cv}(\rho_t . f_y + \alpha_c \sqrt{f_c'}) \tag{5}$$

In Equations 4 and 5, A_{cv} is cross-sectional area of the wall; ρ_t is lateral reinforcement ratio; f_y is characteristic yield strength of shear reinforcement; f_c' is characteristic compressive strength of concrete (f_{ck}); and Φ is the strength reduction factor. Moreover, α_c shall be taken 0.25 for walls which have a height over length ratio (h_w/l_w) less than 1.5. For walls with height over length ratio greater than 2, α_c shall be taken 0.17.

Shear design procedure in TEC-07 initiates with the magnification of shear force results obtained from structural analysis as described in Equation 6 (TEC-07, Article 3.6.6.3, and Equation 3.16).

$$V_e = \beta_v . \frac{(M_p)_t}{(M_d)_t} V_d \tag{6}$$

In this equation, V_e is the design shear force; V_d is the design shear force obtained from structural analysis; β_v is the shear force dynamic amplification factor; $(M_d)_t$ is base bending moment obtained from the analysis; and $(M_p)_t$ is the plastic moment capacity of the structural wall determined by using material characteristic strengths and strain hardening of reinforcement steel. Furthermore, in TEC-07, it is stated that unless a more detailed sectional analysis is carried out, the plastic moment capacity of a structural wall $(M_p)_t$ can be computed to be α times the moment capacity of the same wall determined using design strengths of materials. In the first edition of TEC-07 on March 2007, β_v was stated to be 1.5 for all types of structures and α as 1.4. In the supplements of the same standard (May 2007), however, β_v was redefined to be 1.0 for those structures in which all of the earthquake loads are carried by structural walls; and α was redefined as 1.25. The V_e value calculated this way is used along with Equation 7 to determine the ratio of lateral reinforcement.

$$\rho_t . f_{yd} = V_e - 0.65 . A_{cv} . f_{ctd} \tag{7}$$

In Equation 7, f_{yd} is design yield strength of lateral reinforcement; A_{cv} is gross cross-sectional area of the wall; and ρ_t is lateral reinforcement ratio which must be larger than 0.25%. The spacing of the transverse reinforcement should not exceed 250 mm.

Detailing Rules for Boundary Elements of Structural Walls

TEC-07 and ACI 318-08 have quite similar approaches for determining the amount of reinforcement at the boundary elements. In TEC-07, the transverse reinforcement of the boundary elements along the critical height is defined as by Equation 8 and this amount cannot exceed two-thirds of the confinement reinforcement specified for the ductile columns. Spacing of the reinforcements must be at least half of the wall thickness. The transverse steel spacing cannot be greater than 100 mm but greater than 50 mm at all times.

$$A_{sh} = 0.05\frac{s.b_c.f_{ck}}{f_{ytk}} \tag{8}$$

In Equation 8, s is the spacing of reinforcements; b_c is width of the core of wall; f_{ytk} is the characteristic yield strength of lateral reinforcement; f_{ck} is the characteristic compressive strength of concrete.

Although ACI 318-08 has similar equations with TEC-07 regarding the detailing of reinforcements, it adopts a more conservative approach to be on the safe side. According to ACI 318-08, maximum transverse steel spacing should be the smallest of one-fourth of thickness of the wall; six times the diameter of longitudinal reinforcement used in the boundary elements; and the value expressed in Equation 9. Moreover, maximum spacing is not allowed to exceed one-third of the minimum dimension of the boundary element. On the other hand, transverse steel spacing cannot be smaller than 100 mm.

$$s_o = 100 + \left(\frac{350 - h_x}{3}\right) \tag{9}$$

In Equation 9, h_x is the spacing of the longitudinal reinforcement. Minimum transverse reinforcement area is defined by Equation 10. The parameters in Equation 10 are similar to the ones in Equation 8 (ACI 318-08, Equation 21-5).

$$A_{sh} = 0.09\frac{s.b_c.f_c'}{f_{ytk}} \tag{10}$$

2.2 Normal Ductility Structural Walls

In the guidelines of American Concrete Institute, structural walls having normal or medium ductility are not defined. Instead, there is 'ordinary structural wall' definition which is allowed to be used in the regions that are less prone to seismic hazard. These types of structural walls are also similar to the ordinary walls defined in Eurocode EC2 [6]. Normal ductility structural walls expressed in TEC-07 are in accordance with DC-M type structural walls defined in EC8. American regulations related with loads (ASCE 7-05) [3] defines the response modification factor R as 5 for RC structures with ordinary structural walls. However, it prohibits the utilization of structural systems with ordinary structural walls in high seismic regions. In TEC-07, however, the use of normal ductility structural walls is allowed in all seismic regions together with a response modification factor, R of 4.

In the following paragraphs, the design and detailing requirements of ordinary structural walls of ACI 318-08 and normal ductility walls of TEC-07 will be compared and the differences in their design approaches will be discussed.

Design and Detailing Rules

In TEC-07, design rules for normal and high ductility structural walls are quite similar. Especially the regulations related to the reinforcement detailing for both wall types are almost the same. The main difference in their designs is hidden in their design load calculations. In normal ductility structural wall design, the base bending moments are directly obtained from the structural analysis. On the other hand, design shear forces are determined by amplifying the values obtained from the analysis. This amplification factor was 1.5 in the March 2007 release of TEC-07. It was, however, changed to 2.0 in the May 2007 release. Regardless of their different ductility demands, the shear capacity of normal and high ductility structural walls are calculated in the same way.

On the other hand, in ACI 318-08, design rules for ordinary walls are quite different from the ones for high ductility walls. First of all, forming of boundary elements at the ends of the ordinary walls are not required. Secondly, longitudinal reinforcement requirement of such walls is determined by employing a section analysis using the design axial force and bending moment pairs obtained from structural analysis without any modification. Moreover, the minimum longitudinal reinforcement ratio is stated as 0.15% with a maximum allowed spacing of 450 mm. Likewise, design shear forces are directly obtained from analysis. The cracking capacity of the cross-section is calculated from Equations 11 and 12 by taking the nature of the expected cracking into account (ACI 318-08, Article 11.9.6).

$$\phi Vc_1 = \phi\left[0.27\sqrt{f_c'}.h.d + \left(\frac{N_u.d}{4.l_w}\right)\right] \tag{11}$$

$$\phi Vc_2 = \phi\left[0.05\sqrt{f_c'} + \frac{l_w\left(0.1\sqrt{f_c'} + 0.2\dfrac{N_u}{l_w.h}\right)}{\dfrac{M_u}{V_u} - \dfrac{l_w}{2}}\right].h.d \tag{12}$$

In Equations 11 and 12, l_w is length, d is effective length ($0.8l_w$) and h is thickness of the wall; Φ is the strength reduction factor; and N_u, M_u and V_u are axial load, bending moment and shear force results obtained from the structural analysis, respectively. Equation 11 aims to determine the shear force which leads to principal axial tensile stress of approximately $0.33\sqrt{f_c'}$ at the centroid of the wall cross section. On the other hand, Equation 12 corresponds to a case where a flexural tensile stress of $0.5\sqrt{f_c'}$ develops at a section $l_w/2$ above the section being investigated. The smaller of these two values is taken as the cracking strength of the section under investigation.

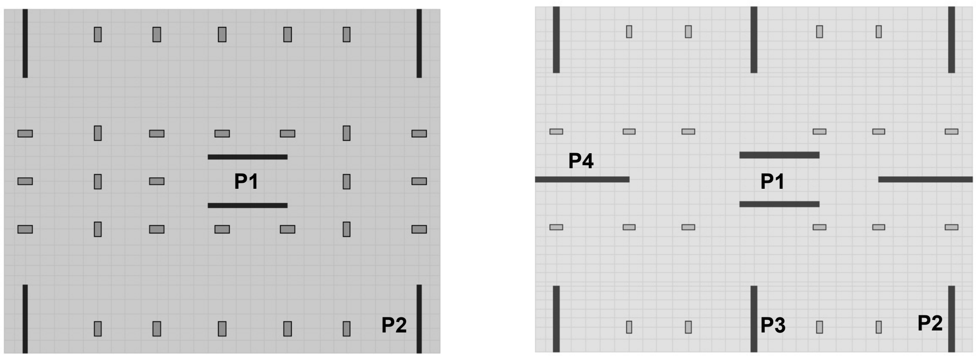
If the design shear force is smaller than $0.5\Phi V_c$ the minimum shear reinforcement requirement is applied. If the design shear force exceeds this target value, the required amount of transverse reinforcement is determined from Equation 13.

$$\phi V_s = \phi \frac{A_{sh}.f_y.d}{s} \tag{13}$$

In Equation 13, A_{sh} is the area of lateral reinforcements; f_y is the characteristic yield strength of steel; d is the effective length; Φ is the strength reduction factor; and s is the spacing of lateral reinforcements. According to ACI 318-08, minimum lateral reinforcement area should be at least 0.25% of the cross-sectional area of the wall with a maximum allowable spacing of $l_w/5$, $3h$ or 450 mm.

3. CASE STUDIES

In this section two buildings, one having five stories and the other having twelve stories, were considered for a parametric study, as shown in Figure 1. Each building was designed with two different lateral load carrying system, i.e. wall only system and dual (wall-frame) system. In every design set, the buildings were assumed to be located on a Z3 type soil (close to Site Class D definition of ASCE 7). In each case, the structural wall design was carried out by considering two different ductility levels, i.e. high ductility/special and normal ductility/ordinary walls. Furthermore, the design calculations were performed for both high and moderate seismic hazard zones. In other words, the sample buildings demonstrated in Figure 1 were designed for sixteen different cases by following the regulations of TEC-07and ACI 318-08 separately for each case. In each design the concrete grade was C30.



(a) Five-story Building - Structural System *(b) Twelve-Story Building - Structural System*

***Figure 1**: Structural Systems of Sample Buildings*

***Table 1**: Dimensions of Structural Walls*

	P1	P2	P3	P4
5 Story Wall only System (1st Degree Seismic Zone)	300 × 6240	300 × 3750	-	-
5 Story Dual System (1st Degree Seismic Zone)	250× 4400	250 × 3750	-	-
5 Story Wall only System (3rd Degree Seismic Zone)	250 × 4400	250 × 3750	-	-
12 Story Wall only System (All Seismic Zones)	300 × 4400	300 × 3750	300 × 3750	300 × 5200
12 Story Dual System (All Seismic Zones)	300 × 4400	300 × 3750	300 × 3750	300 × 5200

All units are in mm

While arranging load carrying systems of the buildings, structural walls were located initially at the most suitable locations such as around the stair openings or between the units on each floor. In order to increase the torsional rigidity of the structural system, additional structural walls were added at the edges of buildings (Figure 1a, structural wall, P2). Although the number of walls was sufficient for the five-story buildings, additional walls were required for the twelve-twelve-story buildings (Figure 1b). Dual systems (Wall-frame interaction) were formed by connecting columns and walls with beams. Dimensions of columns and beams have been arranged in such a way that frames took 25% of the entire base shear force resulting from earthquake loadings. The wall dimensions of five-story buildings at moderate seismic hazard zone were reduced when compared with the wall sizes of the design for high seismic hazard zone. On the other hand, the structural wall dimensions were kept constant for all design cases of twelve-story buildings. Table 1 illustrates the wall dimensions considered in both building systems.

When structural walls are designed by using guidelines of American Concrete Institute, earthquake effects should be calculated by the regulations of ASCE 7 [3]. Even though the earthquake effects considered in TEC-07 are quite similar with ASCE 7, there are some basic differences between them. First of all, their design earthquake spectra are different. In the design spectrum of TEC-07, as type of soil gets softer, the length of the flat plateau increases without any change in its amplitude. In ASCE 7, however, while the plateau gets longer, amplitudes of the spectral accelerations also vary. Unlike TEC-07, ASCE 7 does not have a structural system definition with normal ductility structural walls. Instead, there are structural systems with ordinary structural walls which can only be utilized in regions of low or moderate seismic hazard. Response modification factor (R) is defined as 5 for systems with ordinary structural walls. In ASCE 7, systems with special structural walls, corresponding to the systems with high ductility structural wall definition in TEC-07, R is defined as 6. For dual systems, R is taken as 5.5 for systems with ordinary structural walls and intermediate moment systems and is taken as 7 for systems with special structural walls and moment frames. In such systems, ASCE 7 forces the moment frame to be designed to have a capacity to carry at least 25% of the earthquake forces whereas TEC-07 reduces the value of R according to the ratio shear force resisted by the moment frames. In ASCE 7, earthquake loads which are reduced by the response modification factor in highly seismic regions (for seismic design categories D, E, F which correspond to first and second seismic zones in TEC-07) are amplified 1 to 1.3 times according to the degree of redundancy of the structure. Finally, for some structural components, design forces due to earthquake loads are increased by using overstrength factors. For buildings that are located at highly seismic regions, it is mandatory to use the overstrength factor while designing of structural elements that transfer seismic loads between slabs and structural walls. In addition to these, regulations for the mathematical modeling of reinforced concrete buildings are quite different in ASCE 7 and TEC-07. For instance, while ASCE 7 stipulates the use of cracked stiffness in structural elements, TEC-07 requires utilization of uncracked stiffness for all structural members except coupling beams.

In this study earthquake loads were first calculated for all of the example buildings by using both ASCE 7 and TEC-07. For all cases, the base shear forces determined by using TEC-07 were larger than the ones calculated according to ASCE 7. Thus, in order to eliminate the differences in earthquake load calculations and focus on the differences in design approaches of TEC-07 and ACI 318-08, only TEC-07 was utilized while calculating the earthquake forces. In mathematical models, uncracked sectional properties were used for structural elements as it is stated in TEC-07, Article 3.2.3. On the other hand, while modeling the buildings with structural wall only systems, the bending rigidities of slab elements were reduced by 50% to better reflect the actual behavior. Earthquake loads were calculated utilizing the equivalent static load approach. Buildings were modeled with finite element method in three dimensional systems. In dual systems, bending rigidities of slabs are ignored and beams were modeled with rectangular sections. Structural modeling and analysis were performed by utilizing the commercially available software, ETABS v9.1.6 [7] but design calculations were worked out manually.

4. DISCUSSION OF RESULTS

4.1 Normal Ductility / Ordinary Structural Walls

The design procedures of normal ductility structural walls of TEC-07 and ordinary structural walls of ACI 318-08 show significant differences. As mentioned earlier in TEC-07, there is no difference between the detailing provisions of high and normal ductility structural walls. Formation of boundary elements is a requirement also for normal ductility structural walls. At boundary elements, incorporating at least 8 mm diameter confinement reinforcement must be placed with a maximum of 10 cm spacing. The maximum spacing and minimum reinforcement ratio for the web reinforcement of the structural walls are defined as 25 cm and 0.25% respectively. On the other hand, according to ACI 318-08 formation of boundary elements is only required for special walls but not for ordinary walls. Limitations for the design of web reinforcement are similar with TEC-07, minimum reinforcement ratio is 0.25% but the maximum spacing for lateral reinforcement is allowed to be 45 cm. As well as the differences in the detailing provisions, twice the magnitude of shear force obtained from structural analysis is used as design shear force in TEC-07. In ACI 318-08, the shear force obtained from structural analyses is, however, used as the design shear force without any magnification. Moreover, in order to calculate shear strength of ordinary walls according to ACI 318-08, two possible cases that can cause concrete cracking are considered and the effects of the axial and shear forces together with the moment acting on the wall are taken into account. In order to discuss the effects of such differences on the structural wall reinforcements, the transverse reinforcements (web and confinement reinforcements) of 5.2 m and 4.4 m long structural walls in the Y direction of the twelve story building at the first earthquake zone are presented in Tables 2 and 3, respectively. In both tables the design shear forces calculated according to the TEC-07 and ACI 318-08 are represented as V_e and V_u, respectively and transverse reinforcement areas were calculated for a section with 1 m height.

Table 2 describes the situation when the amount of the web reinforcement of walls is determined according to the magnitude of the shear forces rather than the code limitations. As it can be observed from the results, when TEC-07 design approach was utilized, the shear strength of the wall provided by the minimum web reinforcement became inadequate and therefore a significant increase in the amount of web reinforcement was observed. This situation is caused by the magnification of shear forces according to TEC-07. On the other hand, for the same earthquake loads, the minimum web reinforcement was sufficient to carry the shear forces acting on the walls above the 4th story according to ACI 318-08. For this case, TEC-07 required 1.5 times more transverse reinforcement than ACI 318-08. In TEC-07, 15% of the transverse reinforcement of the structural wall was placed in the boundary elements for confinement. After the 6th floor, the minimum reinforcement was sufficient; the amount of the confinement reinforcement placed in the boundary elements was 10% of the total amount of the transverse reinforcement placed in the whole floor.

***Table 2**: 12 Story Building, Structural Wall only System, P4, Transverse Reinforcements*

	TEC-07		ACI 318-08	
Story	**V_e (kN)**	**Wall + Boundary Element Reinforcement**	**V_u (kN)**	**Wall Reinforcement**
1	3442	∅12/200+∅8/100 (1357 + 1105 mm^2)	1721	∅12/275 (1049 mm^2)
2	3500	∅12/200+∅8/100 (1357 + 1105 mm^2)	1750	∅12/275 (1049 mm^2)
3	3472	∅12/250+∅8/200 (1131 + 603 mm^2)	1736	∅12/285 (1019mm^2)
4	3331	∅12/250+∅8/200 (1131 + 603 mm^2)	1665	∅10/250 (785 mm^2)
5	3143	∅12/250+∅8/200 (1131 + 603 mm^2)	1572	∅10/250 (785 mm^2)
6	2905	∅12/250+∅8/200 (1131 + 603 mm^2)	1452	∅10/250 (785 mm^2)
7	2620	∅10/250+∅8/200 (785 + 603 mm^2)	1310	∅10/250 (785 mm^2)
8	2288	∅10/250+∅8/200 (785 + 603 mm^2)	1144	∅10/250 (785 mm^2)
9	1910	∅10/250+∅8/200 (785 + 603 mm^2)	955	∅10/250 (785 mm^2)
10	1485	∅10/250+∅8/200 (785 + 603 mm^2)	743	∅10/250 (785 mm^2)
11	1020	∅10/250+∅8/200 (785 + 603 mm^2)	510	∅10/250 (785 mm^2)
12	518	∅10/250+∅8/200 (785 + 603 mm^2)	259	∅10/250 (785 mm^2)

Unit of reinforcement spacing is in mm

The transverse reinforcement requirements for the shorter wall (4.4 m) are presented in Table 3. It can be observed from the results that, the minimum reinforcement requirement was adequate for carrying the design shear forces for both codes. The differences in the design of these types of walls are usually caused by the reinforcements placed in the boundary elements. For wall P1, 20% of the whole transverse reinforcement in a story was placed in the boundary elements up to critical height and after the critical height, 10% of the whole transverse reinforcement in a story was placed in the boundary elements. As a result, TEC-07 required 1.35 times more transverse reinforcement than ACI 318-08.

***Table 3**: 12 Story Building, Structural Wall only System, P1, Transverse Reinforcements*

	DBYBHY-07		ACI 318-08	
Story	**V_e (kN)**	**Wall + Boundary Element Reinforcement**	**V_u (kN)**	**Wall Reinforcement**
1	2563	∅10/200+∅8/100 (943 + 1106 mm^2)	1282	∅10/250 (785 mm^2)
2	2293	∅10/250+∅8/100 (785 + 1106 mm^2)	1147	∅10/250 (785 mm^2)
3	2216	∅10/250+∅8/200 (785 + 603 mm^2)	1108	∅10/250 (785 mm^2)
4	2113	∅10/250+∅8/200 (785 + 603 mm^2)	1056	∅10/250 (785 mm^2)
5	1996	∅10/250+∅8/200 (785 + 603 mm^2)	998	∅10/250 (785 mm^2)
6	1849	∅10/250+∅8/200 (785 + 603 mm^2)	925	∅10/250 (785 mm^2)
7	1674	∅10/250+∅8/200 (785 + 603 mm^2)	837	∅10/250 (785 mm^2)
8	1470	∅10/250+∅8/200 (785 + 603 mm^2)	735	∅10/250 (785 mm^2)
9	1237	∅10/250+∅8/200 (785 + 603 mm^2)	619	∅10/250 (785 mm^2)
10	974	∅10/250+∅8/200 (785 + 603 mm^2)	487	∅10/250 (785 mm^2)
11	682	∅10/250+∅8/200 (785 + 603 mm^2)	341	∅10/250 (785 mm^2)
12	351	∅10/250+∅8/200 (785 + 603 mm^2)	175	∅10/250 (785 mm^2)

Unit of reinforcement spacing is in mm

The contributions of concrete strength to the capacity of the structural walls were calculated for some example cross sections according to both codes and the results are tabulated in Table 4. Both of the capacity values for two cracking modes were calculated according to ACI 318-08 and presented separately in the table. All of the results at Table 4 are for the buildings in the third earthquake zone. The value of strength reduction factor, Φ, was taken as 0.75 in the calculations according to ACI 318-08.

***Table 4**: Structural Wall Shear Capacities – Effect of Concrete*

Building System	V_c TEC-07	V_{c1} Equation 11	V_{c2} Equation 12	V_c ACI 318-08
12 Story, Dual System P4, 1. Story	1296 kN	1673 kN	915 kN	915 kN
5 Story, Wall only System P1, 1.Story	914 kN	1105 kN	437 kN	437 kN
5 Story, Wall only System P1, 5. Story	914 kN	1000 kN	1826 kN	1000 kN

As can be seen from the results in Table 4, the contribution of the concrete strength to the shear capacity of the wall in the cross sections at the base of the wall was calculated to be much smaller in ACI 318-08 than TEC-07. This situation is caused by the reason that it is expected that shear cracks (Equation 12) are generally formed by bending especially for the walls under the influence of high shear force and moment. Since this behavior is not taken into account in TEC-07, the contribution of concrete strength to shear resistance of the wall was calculated to be much more than ACI 318-08. At the upper floors, shear cracks are usually caused by axial tensile stresses (Equation 11) in the web section rather than bending (Equation 12). However, in this case contribution of concrete was calculated to be less in TEC-07, since the axial force acting on the wall was ignored.

***Table 5**: 12 Story Building, Dual System, 5.2 m long Wall, Longitudinal Reinforcement*

	TEC-07	ACI 318-08
Story	Wall + Boundary Element Reinforcement (mm)	Wall Reinforcement (mm)
1	28∅10/200+12∅22/200 (11325 mm^2)	30∅22/370 (11405 mm^2)
2	28∅10/200+12∅20/200 (9740 mm^2)	30∅16/370 (6032 mm^2)
3-12	40∅10/200+6∅20/200 (6911 mm^2)	30∅10/370 (2356 mm^2)

The discrepancy in the amounts of lateral reinforcement can be also noticed in the longitudinal reinforcement. In ACI 318-08, the only requirement about the longitudinal reinforcement is the minimum reinforcement ratio of 0.0015 and the maximum spacing between two reinforcement bars to be 45 cm. In TEC-07 minimum longitudinal reinforcement ratio and maximum spacing are determined as 0025 and 25 cm respectively. In addition, up to critical height minimum reinforcement ratio in the boundary elements is determined as 0.2% of gross cross section area of the wall and 0.1% of gross cross section area of the wall for the remaining parts. In order to show the effects of the differences in the minimum reinforcement ratios and the detailing provisions, on the amount of reinforcement, the 12 story building with the dual structural system in the third earthquake zone was designed. Table 5 presents the necessary amounts of longitudinal reinforcement in P4 wall. As it can be observed from the results, TEC-07 requires approximately 2.2 times more longitudinal reinforcement than ACI 318-08.

4.2 High Ductility / Special Structural Walls

Though design of high ductility structural walls according to TEC-07 and ACI 318-08 includes similarities, differences in formulae and detailing provisions stand out. The most striking difference is about boundary elements. TEC-07 requires formation of boundary elements for every situation and along the wall height rather than along the critical height. There are strict requirements about the minimum longitudinal reinforcement ratio and maximum spacing of lateral reinforcement in the boundary elements. The related requirements in ACI 318-08 are more flexible. Formation of boundary elements is associated with how much the wall is strained by the earthquake loads. For example, according to ACI 318-08, it is not necessary to form boundary elements along the critical height for the high ductility walls of five-story and twelve-story buildings in the third earthquake zone. A similar situation is also valid for walls of the twelve-story building in first earthquake zone with dual structural system. In TEC-07, along the critical height, the length of a single boundary element is defined as 20% of the length of the wall. Whereas in ACI 318-08, the length of the boundary elements is determined according to the depth of compression area (depth of neutral axis) caused by the forces acting on the wall (Equation 1). In order to understand the discrepancies caused by these two approaches, lengths of boundary elements and the amount of reinforcements in these regions that are determined according to two codes are tabulated in Table 6.

Table-6 indicates that, for the cases where the structural walls are predominantly under flexural actions (core wall system), the length of the boundary elements determined according to ACI 318-08, are longer than the ones determined according to TEC-07. However, for the case where the moments acting on the wall are relatively small (dual structural system) boundary elements determined according to TEC-07 are longer than ACI 318-08. Another important point about the boundary elements is the amount of lateral reinforcement to be placed in these regions. Wrapping the longitudinal reinforcement in these areas tightly, delays or prevents the buckling of the longitudinal reinforcement due to the high compressive forces occurred during a large earthquake. Moreover, the energy absorption capacity of the structural wall increases, since the concrete is tightly wrapped. Although, calculation of the minimum amount of reinforcement in the boundary elements is emphasized by both codes, it can be observed that ACI 318-08 requires approximately two times more reinforcement than TEC-07, when Equation 8 and 10 are examined in detail. As seen from the results in Table 6, for both five-story and ten-story buildings with core wall system, ACI 318-08 required two times of the amount of reinforcement in the boundary elements determined according to TEC-07. On the other hand, for the dual systems, according to the ACI 318-08 necessity for formation of boundary elements was eliminated, due to the reduced forces acting on the wall. ACI 318-08 does not enforce the use of boundary elements beyond the critical height in all designs In TEC-07, on the other hand, the use of boundary elements beyond the critical height is still compulsory. The designer, however, is allowed to reduce the sizes of the boundary elements to be built outside the critical height.

***Table 6**: Boundary Element Length and Transverse Reinforcement Requirements Along the Critical Height*

		Boundary Element Length		Boundary Element Transverse Reinforcement	
Building System	**Wall**	**TEC-07**	**ACI 318-08**	**TEC-07 (mm)**	**ACI 318-08 (mm)**
5 Story Wall only System	P1	75 cm	90 cm	∅8/100	∅12/100
	P2	125 cm	150 cm	∅8/100	∅12/100
5 Story Dual System	P1	75 cm	80 cm	∅8/100	-
	P2	90 cm	65 cm	∅8/100	-
12 Story Wall only System	P1	120 cm	115 cm	∅8/100	∅12/100

In TEC-07, the shear force obtained from structural analysis is augmented twice with multipliers greater than 1.0. The first multiplier is the M_{pt}/M_{rt} ratio of the wall at the base. Here M_{pt} and M_{rt} are the plastic moment capacity and the factored moment capacity of the wall section at the base, respectively. The second multiplier is called the shear force dynamic magnification factor, β_v, which assumes different values depending on the structural system chosen for design. Furthermore, in the calculation of moment capacity of structural walls, it is stated that under the circumstances where more detailed analysis is not performed M_{pt} can be taken as $1.25M_{rt}$. However, TEC-07 mentions that in calculation of M_{rt} design material properties (f_{cd}, f_{yd}), in calculation of M_{pt} characteristic material properties (f_{ck}, f_{yk}) must be used.

For the case studies considered, the M_{pt}, M_{rt} values were determined by carrying out detailed moment-curvature analysis. These ratios are given in Table 7.

As shown in Table 7, the plastic moment capacity of the wall sections, M_{pt}, is almost 40 percent bigger than its factored moment resistance, M_{rt}. Approximately 15 percent of this difference is due to the material factor used for steel (γ_s=1.15) and 25 percent is due to strength gain resulting from strain hardening. The approach adopted in Equation 6 is originally proposed by Paulay [8]. Paulay in this study directly suggested the use of characteristic material strengths in the calculation of M_{rt}. With the updates of M_{rt} and M_{pt} relationship in TEC-07 in May 2007, this relationship became similar to its original form. This situation, however, causes a conceptual confusion in TEC-07. When M_{pt} is computed by increasing M_{rt} by 25%, the obtained moment value is not the actual moment capacity of the wall. Thus, this approach misleads the structural engineers who want to perform a more detailed sectional analysis.

***Table** 7: M_{pt}, M_{rt} and M_{pt}/M_{pt} values*

Building Type	Story	M_{pt} (kN.m)	M_{rt} (kN.m)	M_{pt}/M_{rt}
5 Story, Wall only System, P1	1st	24800	18700	1.33
5 Story, Dual System, P1	1st	9890	6920	1.43
12 Story, Dual System, P4	1st	15664	11040	1.42

Another problem encountered in the capacity design approach of Equation 6 is the way the shear forces are magnified for different structural systems. Such magnifications are not consistent with the expected building behavior. As an examplary case, base moments and shear forces acting on wall P4 at the twelve-story building with dual and wall only structural systems are presented in Table 8. These values were calculated according to $M_{pt}=1.25M_{rt}$ relationship as in TEC-07.

***Table 8**: Twelve Story Building, V_d and V_e values*

Building System	Story	M_d	M_{rt}	V_d	V_e	V_e/V_d
12 Story Wall only System P4	1. Story	28578	31643	1126	1558	1.38
12 Story Dual System P4	1. Story	7451	11039	880	2444	2.78

In Table 8, M_d and V_d represent the design moment and the design shear force directly obtained from structural analysis, M_{rt} is the resisting moment due to the reinforcement in the wall including the boundary elements and V_e represents the design shear force. According to the results of the structural analyses, the structural wall P4 of the wall-only system was subjected to much more severe earthquake forces than the same wall in the dual system. Because of this, higher amount of longitudinal reinforcement was used in the walls of wall-only system due to larger moments. From the capacity design point of view, walls having a greater moment capacity should be designed for larger shear demands. The situation in this specific case was just the opposite; the design shear forces calculated for the dual systems were larger than the ones for wall-only system. The reason for this is employment of different dynamic magnification factor β_v for different load carrying systems. According to the current version of TEC-07, β_v coefficient was defined as equal to

1.5 for dual systems and 1.0 for wall-only systems. In addition to this, the difference between M_{rt} and M_d was much larger in dual systems due to the 0.2% minimum longitudinal reinforcement ratio requirement for boundary elements. Furthermore, when the original study where the approach defined by Equation 6 was first suggested [8] is carefully examined, it can be seen that the use of β_v is to consider the magnifying effect of higher modes on shear forces. Paulay [8] suggested calculating this value by two different ways as shown below. Equation 14 is for the buildings up to six stories and Equation 15 is for buildings having more than six stories.

$$\beta_v = 0.9 + n/10 \tag{14}$$

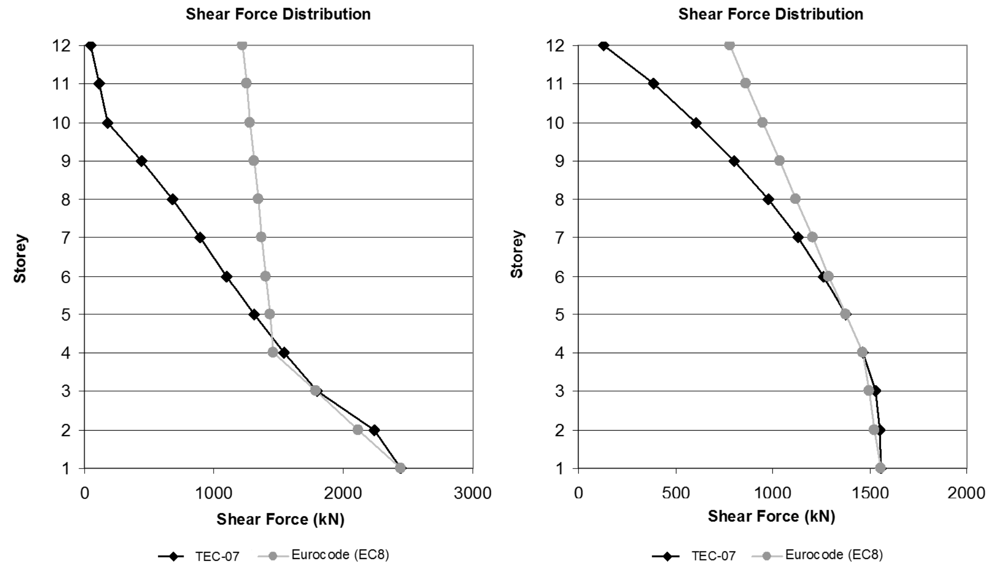
$$\beta \ = 1.3 + n/30 \tag{15}$$

In the above equations n represents the number of stories. Wallace [11] later suggested an upper limit of 15 on the number of stories. He also suggested taking β_v coefficient as 4/3 for the buildings up to 10-story and 5/3 for taller buildings. In both cases, β_v coefficient is greater than 1. Thus, both Paulay's [8] and Wallace's [11] studies showed that the design shear forces for wall only systems were underestimated by TEC-07. Wallace [11] also suggested that β_v factor can be set to 1.0 when dynamic analysis procedures were employed in the determination of the design forces.

The use of Equation 6 in its present form in TEC-07 creates additional problems. During the derivation of this equation, it was assumed that $M_{rt} \leq 1.10M_{dt}$ for all cases in design. This assumption introduces severe problems in the design of the walls; as in many design applications larger wall sizes may simply be used to speed-up the construction, to provide better drift control or to reduce displacements and vibrations due to wind. None of these initiatives originates from a seismic concern. Moreover, when the minimum longitudinal reinforcement requirement of the boundary elements (0.2% of gross cross section of the wall) is applied on such walls a further increase in M_{rt} is inevitable. In such cases it is quite normal that M_{rt} will be much greater than M_{dt}, and hence, Equation 6 would lead to meaningless and unnecessary increase in the flexural overstrength factor.

In the studies where further details of the capacity design approach were investigated [11, 13] and in the codes where this approach was adopted [EC8], an envelope curve approach, similar to moment envelope curve, was proposed for the distribution of shear force at higher stories. To construct the shear force envelope using EC8, the first thing to do is to determine the magnification factor for the base shear force. Unlike TEC-07, the system behavior coefficient and the period of the structure are taken into account together in the calculation of this magnification factor. In the second step, the shear forces obtained from the analysis are magnified within the lower one-third of the wall. The design shear force at the top of the wall is then set equal to 50 percent of the magnified base shear, the least. Finally, within the upper two-thirds of the wall a linear variation of shear force is assumed between the magnified design shear forces at the wall-top and at the level one-third-wall length above the base.

The Turkish Earthquake Code, however, suggests the use of the same shear force magnification factor for the entire wall. For the twelve-story buildings considered in this study, the design shear force envelopes, constructed in accordance with TEC-07 and the EC8, are shown in Figure 2.



(a) Dual System *(b) Wall only System*

(Base shears were calculated according to TEC-07)

***Figure 2**: Shear Force Distribution along the Wall Height*

As can be seen in Figure 2, the capacity design approach adopted in TEC-07 significantly underestimates the shear forces of the structural walls at the upper stories. The nonlinear analyses made especially on dual systems [13, 15] indicated that the moment frame response becomes more dominant in the upper stories of such buildings. It must be emphasized that this change in the system behavior cannot be observed in linear analyses. In order to reduce the damage and to provide a better crack control for the upper story walls, the definition of their design shear force in TEC-07 must evidently be changed.

As mentioned before, ACI 318-08 does not adopt the capacity design approach for the shear design of structural walls because no serious performance deficiency was observed in buildings having structural walls during past earthquakes [12, 14, 17]. Besides, ACI 318-08 follows a displacement based approach in the flexural design of structural walls. The detailing provisions are then decided according to the displacement demand under the design earthquake. In the design, the energy absorption capacities of structural walls are increased as much as possible by paying special attention to detailing of boundary elements. Moreover, tests on structural walls with boundary elements [8, 14] showed that, even though the shear reinforcement of the walls is not designed according to capacity design approach, no significant damage was observed at the walls up to 1% drift ratio. For higher drift ratios although some shear cracks were observed, the walls continued to show a ductile behavior. The tests also showed that, walls can resist shear stresses up to $0.53\sqrt{f_c'}$ and dense lateral reinforcement placed in the walls decreased the deformation capacity of the walls [17]. For these reasons, the shear design approach in ACI 318-08 provides rules for determining the dimensions of structural walls (Equation 2 and 3) to limit the earthquake induced shear stresses and does not magnify the design shear force, which causes an increase in the transverse reinforcement.

5. CONCLUSIONS

This study presented a comparative study on structural wall design provisions of the TEC-07, ACI 318-08 and EC8.Various shortcomings and drawbacks of the structural design approach of Turkish Earthquake Code were discussed. Based on the analyses made in this study following recommendations, to improve the present provisions, were made.

1) Ordinary structural wall design for low earthquake risk regions was not defined in Turkish Earthquake Code. It is the sincere belief of the authors that the design of ordinary structural walls in the low seismic hazard zones should be allowed in the newer versions of the Turkish Earthquake Code. Forming of boundary elements, a must do for structural walls to be built in high seismic zones should not be imposed in the design of these walls. Moreover, results of the structural analysis shall be used as they are, without additional magnification. While calculating the concrete contribution to the shear capacity of structural walls, the effect of axial loads and flexure-shear strength shall be considered. Consequently, the building design will be more economical in regions where earthquakes are less of a threat, especially in those where wind loads are more critical than earthquake loads or for buildings that are constructed utilizing tunnel form systems.

2) It was observed that in the TEC-07 the principles of the capacity design approach were not fully applied in the design of high ductility structural walls. The current form of the code requires modifications in the following points.

- The Equation 6, as it appears in TEC-07, creates several problems in the shear design of high ductility structural walls. In Equation 6, a magnification factor is defined based on the moment capacity of the wall. The code forces the designer to increase the shear forces obtained from the analysis along the wall height, by multiplying them with this magnification factor. Especially, in the case of dual systems, shear forces estimated by this way may be much lower than the ones that the wall might experience during an earthquake. Therefore, a design envelope shall be defined for the shear forces to estimate the design shears at the upper levels properly, in accordance with the provisions of capacity design.

- Another problem associated with Equation 6 is related to the dynamic magnification factor. The use of different magnification factors for different lateral load carrying systems does not reflect the systems behavior properly. The dynamic magnification factor, β_v, shall be the same for all structural systems. The value to be assigned to this factor shall be restudied along with all parameters influencing it.

- The minimum longitudinal reinforcement ratio requirement for the boundary regions artificially increases the moment carrying capacity of the wall sections, thereby increasing the flexural overstrength factor and the design shears. Therefore, the minimum longitudinal reinforcement requirements for the boundary elements are strongly advised to be reconsidered. Furthermore, the necessity of formation of boundary elements beyond the critical height should be reexamined.

- The $M_{pt} = 1.25M_{rt}$ relation for structural walls is not only a source of confusion but also creates a contradiction in the derivation of Equation 6. It is recommended that the formulation shall be revised as $M_{pt} = 1.4M_{rt}$.

3) The authors believe that enforcing the capacity design approach for structural walls at all types of buildings without considering the wall/plan area ratio and the possible story drifts caused by the earthquakes that may occur in the region discourages the use of structural walls by increasing the building costs in countries that need low-cost and rapid solutions against threat of earthquakes. In this respect, adopting a new design approach that account for interstory drift or strain demands would result in more economical and safer designs.

Regardless the adopted design approach, one important lesson that was acquired from past earthquakes in Turkey is that the presence of structural walls in a building greatly improved the system response and prevented total collapse of the building. The authors therefore believe that, in earthquake regions, the code provisions should encourage the use of structural walls in building design. The present Turkish Earthquake Code, on the contrary, discourages the designers from the use of structural walls by artificially increasing the design loads and thus boosting the use of the reinforcement considerably. The writers would therefore strongly suggest that this code approach must be changed as soon as possible in order to mitigate the earthquake risks in the building stock to be built in the coming years.

As a final note, a list of references should be attached to the code to reveal the basis of equations and requirements adopted in the code in order to give further guidance to the users The designers will thus gain an insight about the derivation of these equations and comprehend the ranges of applicability of the code requirements by understanding all the assumptions and experimental results behind them.

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