CONVERGENCES AND DIVERGENCES IN SEISMIC CONSTRUCTION STANDARDS SPECIFIED IN EUROPEAN UNION AND TURKEY, UP TO 4 STORY BUILDINGS

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ABSTRACT

The Earthquake codes contain provisions for planning and designing earthquake resistant structures. These codes help the structural engineers to design and create a safe structure also helps to avoid creation the major mistakes.

In this study, the seismic construction recommended by European Union code (EC8) and Turkish seismic code are considered for comparison. The comparisons are made in expressions of the ground condition, response spectra, criteria structural regularity, design of reinforced concrete structure, and many others.

The aim of this study is to better understanding the significance or the necessity for seismic building code provisions, as well as the basic performance requirements of seismic technology for construction building, also to describe and understand the convergences and divergences in seismic construction standards specified in European Union Construction and standards specified in Turkish Seismic Construction.

Several tables and figures are presented to show the convergences and divergences between these codes.

The observations obtained from this study showed that the performance objectives of the Turkish Seismic codes which is very similar to Eurocode8 such as limit the damage structural and nonstructural elements in medium intensity earthquake, and prevent overall or partial collapse of building in high intensity earthquake, as well as their design approaches are very similar, both are aimed at designing safe and economic structures.

Keywords: Seismic construction, Turkish earthquake code 2007, Eurocode8

ÖZET

Deprem kodları planlama ve tasarım depreme dayanıklı yapılar inşa edilmesi için hükümler içermektedir. Bu kodlar güvenli bir yapı tasarımında büyük hataları önlemek için yardımcı olur.

Bu çalışmada, Avrupa Birliği kodu (EC8) ve Türk sismik kodU tarafından önerilen sismik inşaat kurallarının karşılaştırılması irdelenmiştir. Karşılaştırmalar zemin durumuna, tepki spektrumları, kriter, yapısal düzenlilik, betonarme yapı tasarımı, ve diğer ifadeleri içermektedir.

Bu çalışmanın amacı, sismik kod hükümlerinin önemini ve gerekliliğini daha iyi anlatmaktır, Bu çalışma ayrıca sismik temel performansını daha iyi anlamak için esas teşkil etmekte ve Türk deprem yönetmeliği hükümleri ve Eurocode8 arasında karşılaştırma yapmaktadır.

Tez çalışmasında tablolar ve figürler kullanılarak Türk deprem yönetmeliği ve Eurocode8 hükümleri arasında karşılaştırma yapılmaktadır.

Çalışmanın sonucunda Türk deprem yönetmeliği ve Eurocode8 de bulunan hükümlerin yapı elemanları ve yapı elemanı olmayan elemanları orta ölçekli deprem durumlarında benzer davranış içerdiği gözlemlenmiştir. Iki standartta bulunan tasarım hükümlerinin benzer olduğu ve güvenli ve ekonomik yapı tasarımı için benzer hükümler içerdiği gözlemlenmiştir.

Anahtar Kelimeler: Sismik İnşaat, Türk Deprem Yönetmeliği 2007, Eurocode8

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LIST OF ABBREVIATIONS

ASCE:	American Society of Civil Engineering
ANSI:	American National Standards Institute
CEN:	Committee European de Normalization
DCL:	Low Ductility Building Member
DCM:	Medium Ductility Building Member
DCH:	High Ductility Building Member
EC8:	Eurocode8 Design of Structures for Earthquake Resistance
EC2:	Eurocode2 (Design of Concrete Structures)
FEMA:	Federal Emergency Management Agency
HDL:	High Ductility Building Level
ICBO:	International Conference of the Building Officials
NDL:	Nominal Ductility Building Level
SEI:	Structural Engineering Institute
TEC 2007:	Turkish Earthquake Code 2007
TS 500:	Requirements for Design and Construction of Reinforced Concrete Buildings
TS 498:	Design Loads for Building
UBC:	Uniform Building Code

LIST OF SYMBOLS

$(V_{S,30})$	Average shear wave velocity in the upper 30 m of the soil profile
N_{SPT}	Number of blows in the standard penetration test
Cu	Undrained cohesive resistance
$Se(T)$	Elastic response spectrum
$Sve(T)$	Elastic vertical ground acceleration response spectrum
T	Vibration period of a linear single degree of freedom system
Ag	Design ground acceleration on type A ground
AgR	Reference peak ground acceleration on type A ground
TB	Lower limit of a period of the constant spectral acceleration branch
TC	Upper limit of a period of the constant spectral acceleration branch
TD	Value defining the beginning of the constant displacement response range of the spectrum
S	Soil factor
T_{NCR}	Reference return period of the reference seismic action for the no-collapse requirement
P_{NCR}	Reference probability of exceedance in 50 years of the reference seismic action for the no-collapse requirement
n	Damping correction factor with a reference value of η=1 for 5% viscous damping
ξ	Viscous damping ratio of the structure, expressed as a percentage
$Sd\ (T)$	Design spectrum
$SDe(T)$	Elastic displacement response spectrum
a_{vg}	Design ground acceleration in the vertical direction

q	Behavior factor
B	Lower bound factor for the horizontal design spectrum
γ_I	Importance factor
G_{kj}	Characteristic value of dead loads
Ms	Magnitude
A_{Ed}	Design value of return period of specific earthquake motion
ψ_{2i}	Combination coefficient of live load
Q_{ki}	Characteristic value of live load
$\Psi_{E,i}$	Combination coefficient for the variable action I
λ	Slenderness
L_{max}	Larger dimension in plan of the building
L_{min}	Smaller dimension in plan of the building
e_{ox}	Distance between the center of stiffness and the center of mass measured along the x direction, which is normal to the direction of analysis considered
rx	Square root of the ratio of torsional stiffness to the horizontal stiffness in the y direction (torsional radius)
ls	Radius of gyration of the floor mass in plan
I_{ot}	Distance torsional restraints
b	Total depth of beam in central part of I_{ot}
h	Width of compression flange
b_{wo}	Thickness of the web of reinforcement concrete wall
I_c	Length of boundary element

b_w	Width of boundary element
I_w	Wall cross section length
h_s	Clear story height
f_{ctm}	Main value tensile strength of concrete
f_{yk}	Characteristic yield strength
h_w	Depth of the beam
f_{cd}	Design value of concrete compressive strength
μ_ϕ	Value of curvature ductility factor
P	Tension reinforcement ratio
ρmin	Minimum tension reinforcement ratio
ρmax	Maximum tension reinforcement ratio
ρ'	Compression steel ratio in beams
εsy	Design value of steel strain at yield
ρw	Shear reinforcement ratio
d_{Bl}	Diameter of the longitude bars
d_{bw}	Diameter of hoops
l_c	Length of the column
hc	Biggest cross-sectional dimension of the columns (in meters)
b_c	Cross-sectional dimension of column
ω_{wd}	Volume ratio of confining hoops to that of the confined core to the centerline of the perimeter hoop times f_{yd}/f_{cd}

a	Confinement effectiveness factor
b_o	Width of confined core in a column or in the boundary element of a wall (to centerline of hoops)
ρv	Reinforcement ratio of vertical web bars in a wall
N_{Ed}	Axial force from the analysis for the seismic design situation
l_w	Long side of the rectangular wall section
H_w	Total wall height
h_{storey}	Storey height
μ_{φ}	Design value of steel at yield
$A(T)$	Spectral acceleration coefficient
A_0	Effective ground acceleration coefficient
I	Building importance factor
$S(T)$	Spectrum coefficient
$Sae(T)$	Elastic spectral acceleration
g	Gravitational acceleration (9.81 m/s2)
TA,TB	Spectrum characteristic periods
Ed	Load Combinations
G	Dead load
Q	Live load
E_x, E_y	Earthquake in direction to x and y
gi	Total live load of the building at i,th storey
qi	Total dead load of the building at i,th storey
n	Live load participation factor.
N	Number of stories in the structure.

η_{bi}	Torsional irregularity factor of the building at i,th storey
$(\Delta i)_{ave}$	Average storey drift of the building of i,th storey
$(\Delta i)_{max}$	Maximum storey drift of the building of i,th storey
$(\Delta i)_{min}$	Minimum storey drift of the building of i,th storey
A_b	Total area of openings
A	Gross floor area
L_x, L_y	Length of the building at x, y direction
ax, ay	Length of re-entrant corners in x, y direction
Ae	Effective shear area
A_w	Effective of web area of column cross sections
A_g	Section areas of structural elements at any storey
A_k	Infill wall areas
η_{ki}	Stiffness irregularity factor defined at i'th storey of the building
Δ_i	Storey drift of i'th storey of the building
h_i	Height of i'th storey of building [m]
A_p	Plane area of story building
V_t	Total seismic load acting on a building
f_{ctd}	Design tensile strength of concrete
N_d	Axial force calculated under combined effect of seismic load and vertical loads multiplied with load coefficient
A_c	Total cross sectional area of column.
f_{ck}	Characteristic compressive cylinder strength of concrete.

N_{dm}	Maximum axial force caused by combine effects of gravitational and seismic loads.
f_{ctm}	Main value tensile strength of concrete.
f_{yd}	Design value of yield strength of steel.
Vd	Column axial load ratio.
D_{bar}	Diameter of longitudinal rebars.
D_{min}	Smallest dimension of beam cross-section.
h_c	Clear height of the column.
A	Lateral distance between legs of hoops and crossties.
N_d	Axial force calculated under combine effect of seismic loads and vertical loads multiplied with loads coefficients.
A_{sh}	Total area steel of hoops.
A_{ck}	Concrete core area within outer edges of confinement reinforcement.
f_{ywk}	Characteristic yield strength of transverse reinforcement.

CHAPTER 1

INTRODUCTION

Earthquake is a natural event occurring by means of all uncertainty in all over the word with different magnitude and intensity.

Generally earthquake is ground shaking which can be horizontally and vertically or in all directions caused by a sudden movement of rock on the crust of earth which results in a sudden release of energy and creates seismic waves.

The performance of structures during earthquakes depends seriously on the shape, size and geometry of the structures, so the architects and structural engineers should be work together in the planning and design stages to ensure that a proper pattern and design is selected for construction (Shah and Rusin, 2010).

By considering Newton'□s law of movement, the foundation of buildings shakes and moves with the ground but the roofs has a propensity to keep on in its imaginative location in the case of earthquake but since walls and column are joined to the foundation, all of them will move in the same direction (Shah and Rusin, 2010).

Every year more than 300,000 earthquakes occur worldwide, many of these are of small intensity and do not cause any damage to structures; however, earthquakes of larger intensity in the surrounding area of populated areas cause large damages and loss of life. It is estimated that on the average, 15,000 people are killed each year in the world because of earthquakes (Ersoy, 1988).

Earthquake risk in poor countries is large and rapidly growing, because in poor countries, badly constructed concrete frame structure, inadequate planning and methods of emergency reply, planning and lack of information and investments in disaster mitigation, increase the number of deaths in developing area (Oliveira et al., 2004).

For example earthquake occurred in the 1988 at the Armenia and the 1989 at the Loma Prieta earthquake in Northern California were nearly equivalent in their magnitudes and the number of people occupying the affected regions, but the results were very different;

in the California, 62 people died but in Armenia, at least 25,000 people died (Oliveira et al., 2004).

1.1 Importance of the Research, General Objectives

The importance of this research is to acquire knowledge by gathering information about seismic construction, and to know the effect of the seismic standards on the construction building.

The general objectives of this research:

- To describe and understand the convergences and divergences in seismic construction standards specified in European Union Construction and standards specified in Turkish Seismic Construction.
- To better understanding the significance or the necessity for seismic building code provisions, as well as the basic performance requirements of seismic technology for construction building.
- To investigate the role of Eurocode8 and Turkish seismic code on the construction of building in order to satisfy the safety requirements of the construction project, performance of high quality of engineering condition and to build an economic structure.
- To have an in-depth knowledge about the use of these standard codes in the construction buildings to ensure the protection of human losses and to ensure that structures are able to respond without structural damage to earthquake of moderate intensities and, also total collapse during earthquake of heavy intensity.

1.2 Research Question

The research is tried to answer the following questions regarding to comparison seismic construction standards specified in European Union Construction and Turkish Seismic Construction:

- What does the term Earthquake means?
- What are the requirements of seismic standards to construction buildings?
- What are the affecting of these codes on the safety and economy?
- How to design reinforcement concrete building according to Eurocode8 and Turkish seismic code?
- What are the convergences and divergences of seismic construction standards specified in European Union Construction and Turkish Seismic Construction?

1.3 Methodology

The methodology carried out in this research in order to get the above mentioned objectives is as follows:

- Searching and collecting the information commonly about the background of earthquake and especially about Turkish seismic construction and European Union construction.
- Declaration the seismic construction according to Turkish Earthquake Code 2007 and Eurocode8.
- Observation of the Turkish Earthquake Code 2007 and Eurocode8 and their requirements for seismic building.
- Compared Eurocode8 and Turkish Earthquake Code 2007 to find out the convergences and divergences between them.

1.4 Hypothesis

This study tries to compare seismic construction standards specified in European Union construction (Eurocode8) and Turkish seismic construction, to investigate and evaluate the differences and similarities between Eurocode8 and Turkish seismic code, and their roles in the design of the building considering safety and quality.

1.5 Theoretical Approach

Earthquake has effects on buildings indirectly, the ground shaking leads to shaking of building structures and persuades inertia forces on them; therefore earthquake should be considered in design of building construction to a certain permanence of structures and strength with satisfactory degree of protection against seismic waves and its intensity.

Earthquake kills many people in different countries and destroys many construction buildings and structures because of the absence of a proper and sufficient design of construction buildings against earthquake, and due to poor detailing of seismic resisting building. Thus, many seismic codes were published in all around the world.

1.6 Literature Review

In the recent years several researches have been conducted in order compare earthquake standards of different structures such as reinforced concrete buildings, masonry, timber, and steel buildings according to different codes.

Most of these studies employ a similar methodology in trying to achieve the research objectives. The general requirements for seismic construction according to the codes to be studied are compared theoretically, procedural similarities and or differences are highlighted and then the structures are designed as per the design codes.

In the previous year, earthquake design of structures became significant phenomena due to tragedy of earthquakes which caused a big human disaster. These earthquakes show that the buildings have low seismic performance due to the usage of low quality material and workmanship and lack of the design codes. Since then numerous new codes detailing requirements have been introduced to make sure seismic resistance.

1.7 Structure of Chapters

The Structure of Chapters consists of five chapters, they are the following:

Chapter 1: This chapter covers the importance of the research as well as general objective of this research, research questions, a briefly background about methodology and literature review.

Chapter 2: This chapter includes historical and background of earthquake standards.

Chapter 3: This chapter consists of Earthquake Standards for seismic construction according Turkish Earthquake code2007 and Eurocode8.

Chapter 4: The content of this chapter is a Comparison between of Turkish Earthquake Code 2007 and Eurocode8.

Chapter 5: This chapter summarizes the result of this research, presents its conclusion and recommendation.

CHAPTER 2

HISTORICAL BACKGROUND OF EARTHQUAKE STANDARDS

Earthquakes are the Earth's natural means of releasing stress. When the Earth's plates move against each other, stress is put on the upper mantle (lithosphere). When this stress is great enough, the lithosphere breaks or shifts. As the Earth's plates move they put forces on themselves and each other. When the force is large enough, the crust is forced to break. When the break occurs, the stress is released as energy which moves through the Earth in the form of waves, which we feel and call an earthquake (Booth, 2013).

The type of earthquake depends on the region where it occurs and the geological make up of that region (Booth, 2013). There are many different types of earthquakes:

- Tectonic earthquake
  Tectonic earthquake is most common one. These occur when rocks in the earth's crust break due to geological forces created by movement of tectonic plates.
- Volcanic earthquake
  This type of earthquakes occur in conjunction with volcanic activity.
- Collapse earthquakes
  Collapse earthquakes are small earthquakes in underground caverns and mines.
- Explosion earthquake
  Explosion earthquakes result from the explosion of nuclear and chemical devices.

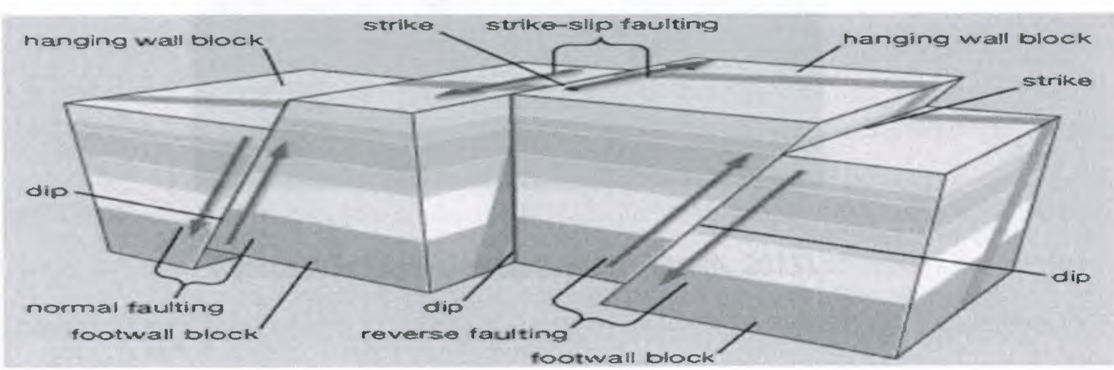


Figure 2.1: Tectonic earthquake (Booth, 2013)

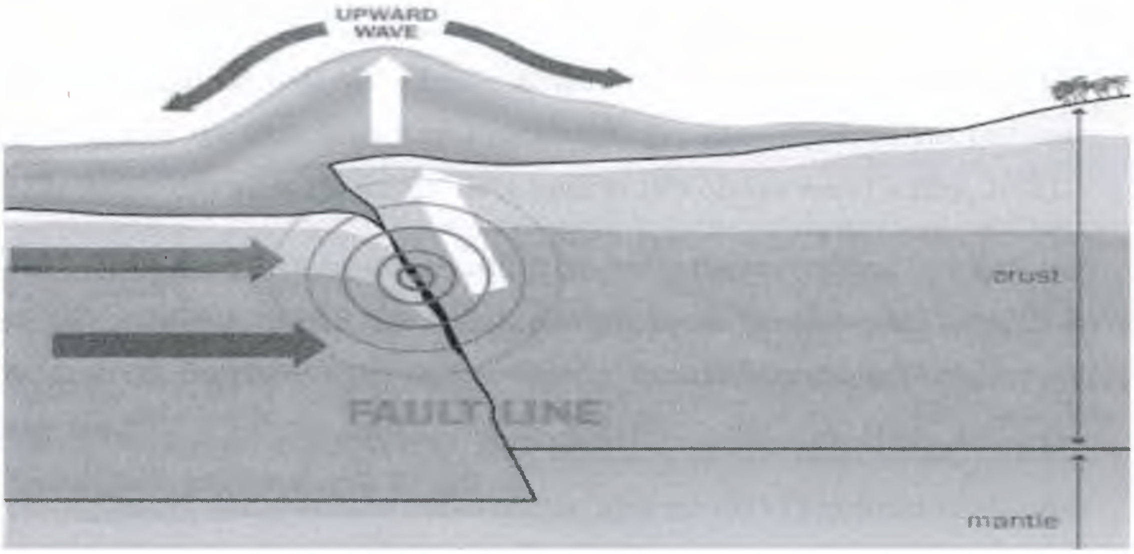


Figure 2.2: Volcanic earthquake (Booth, 2013)

Figure 2.3: Explosion earthquake (Booth, 2013)

2.1 History of Earthquake Standards

The primary official code for seismic design was due to the Japanese Building Ordinance, after the 1923 Great Kanto earthquake. The rules stipulated that buildings must be designed to resist a horizontal force equal to 10% of their mass (Walley, 2001).

In 1927, the Uniform Building Code was first enacted in the International Conference of the Building Officials (ICBO). The seismic provisions were "recommended for addition in the Code of cities placed within an area subjected to earthquake shocks (Anderson and Naeim, 2012).

In the US, seismic design became mandatory just after the 1933 Long Beach earthquake. A seismic design coefficient of 8% of the mass of the structure was suggested, in any case of earthquake or structure characteristics.

In 1943, Los Angeles enacted the first code requirement that related the lateral design force to the flexibility of the building (Anderson and Naeim, 2012).

The most important codes that have been commonly used and tested are the Uniform Building Code (UBC, mostly developed in California but used on many if not most international projects), the Japanese Building rule(sometimes inspired or increased by the Architectural Institute of Japan documents) and the New Zealand seismic design code (recognized to contain higher concepts of ductile seismic reaction), and "Eurocode8" called "Design of Structures for Earthquake Resistance", The Eurocode are common set of building codes in Europe (Anderson and Naeim, 2012).

2.2 Available Codes for Seismic Construction

Seismic design for a building that always considers the specification of earthquake code associated to the location of construction building. Nowadays have many codes related to seismic construction and have a good approach for construction site. In this section, several codes are defined some of the codes are importance in this thesis:

- **FEMA-356,** the abbreviation of The Federal Emergency Management Agency Pre-standard and commentary for the earthquake Rehabilitation of Buildings. It is a code that is used for seismic performance and assessment of an existing building. It is prepared by ASCE American Society of Civil Engineering and SEI " Structural Engineering Institute" and prepared for FEMA Federal Emergency Management Agency Washington, D.C November2000. The NEHRP "National Earthquake Hazards Reduction Program'' Guidelines approved the formal code for the Seismic Rehabilitation of Buildings and the American National Standards Institute(ANSI) of the USA and The guideline are also used by other countries around the world.

- **Eurocode8,** is the abbreviation of The European Standard.Eurocode8 has started in 1975 by the European Committee for Standardization or Committee European de Normalization (CEN). It is a non-profit association whose mission is to develop the European economy in global trading, the benefit of European people and the environment by provide an efficient infrastructure to interest parties for the development, repairs and division of logical sets of standards and specifications. European earthquake regulation is "Eurocode8" called "Design of Structures for Earthquake Resistance"
- **TEC-(2007)** is the abbreviation of The Turkish Earthquake Code 2007.Specification for Buildings to be built in Seismic Zones (2007). After the 1999 Marmara earthquake, which was the most dangerous earthquake of Turkey in the previous century, the requirements have been added to the Turkish earthquake code. 1998 disaster regulation was revised in 2007 in which the new regulation was called Specifications for Buildings to be built in Earthquake Areas. It is used for Turkey and Turkish Republic of Northern Cyprus.
- **UBC** is the abbreviation of "Uniform Building Code" was first enacted by the International Conference of Building Officials (ICBO). The seismic provisions were "recommended for addition in the Code of cities placed within an area subjected to earthquake shocks.

2.3 Previous Studies

In the recent years several researchers have been conducted about comparisons earthquake standards of different structure such as reinforced concrete buildings, masonry, timber, and steel buildings according to different codes.

For purpose of this study, a review of such papers mostly and thesis was conducted and a brief review of these publication is given below:

- Atiyah, (2013). "General Comparison and Evolution of EC8 and TEC-2007 Using STA4-CAD V12.1 In Respect of Cost Estimation" This study compared the general design conditions of Turkish Earthquake code 2007 and Eurocode8. The study focused on the earthquake design of reinforced concrete multi storey buildings which were modeled by using STA4-CAD V12.1 program, And the buildings were designed according to these codes are compared which each other in terms of cost according to the results obtained indicates to the cost is approximately the same.
- Doğangün, & Livaoğlu, (2006). "A comparative study of the design spectra defined by Turkish Earthquake Code, UBC, IBC and Eurocode8 on R/C sample buildings". In this study the design spectra are considered for comparison. The purpose of this study to investigate the divergences of seismic verification according to different codes and different sites for buildings. The divergences in expressions and some significant point for elastic and inelastic spectrum according to these codes that explained before are briefly illustrated in figures and tables.
- SAFKAN, I. "Comparison of Eurocode8 and Turkish Earthquake Code 2007 for Residential RC Buildings in Cyprus". In this study two different seismic design codes are used. These codes are Turkish Earthquake code 2007 and Eurocode8. Two site location have been chosen (Nicosia and Famagusta) and the same structure has been used for the analysis for both places. The study comparison of the inelastic response spectrums, base shear and bending moment value acting to the building by according to the TEC-2007 and Eurocode8.

- LAOUMI2, (2014). Comparative Seismic Study between Algerian Code (RPA99), European Code (EC8) and American Code (UBC97). Second European Conference on Earthquake Engineering and Seismology. Istanbul. In this study the design spectra and ground types are considered for comparison, and show the difference seismic verification according to different codes of a multi-story building, in addition this research explain the difference of elastic and inelastic spectrum.

CHAPTER 3

EARTHQUAKE STANDARDS EC8 AND TEC 2007

3.1 Eurocode8-Design of Structures for Earthquake Resistance

The Euro codes are common set of building codes in Europe. At the moment, they are still in the trial phase. These codes are often used between countries which are members of European Union.

The use of Eurocode8 to make sure the following in an earthquake result:

- To protected human lives.
- To limited damage.
- Structures important for civil protection remain operational (EN 1998-1, 2004).

3.1.1 Fundamental Requirements:

Structures in seismic zones shall be designed and built for the following basic requirements:

- **No-collapse requirement**

  The structure shall be designed and constructed to resist the design seismic action without global or local collapse, so retaining its structure integrity and a remaining load bearing capacity after the seismic event (Bisch et al., 2011).

  This requirement is associated to the protection life under an infrequent event, through the prevention of the local or global collapse of the structure, after the event may present large damages, it may be economically irrecoverable, but it should be able to keep life of human in the evacuation process or through aftershocks (Bisch et al., 2011).

- **Damage limitation requirement**

  The structure shall be designed and built to resist a seismic action having a greater probability of happening than the design seismic action, without the happening of damage and the related limitations of use, the cost of which would be disproportionately high in comparison with the cost of the structure itself (Bisch et al., 2011).

This requirement is associated to the reduction of economic losses in repeated earthquake, the structure should not have perpetual deformations and its elements should keep its original strength and stiffness and so should not need structural repair (Bisch et al., 2011).

3.1.2 Ground Condition

The earthquake vibration at the surface is strongly affected by the underlying of the ground condition and correspondingly the ground characteristic very much influence the seismic response of structure.

The main objectives of the ground investigation are:

- To permit the classification of the soil profile.
- To recognize the probable event of a soil behavior during an earthquake, harmful to the reply of the structure (Bisch et al., 2011).

The building site and the character of the supporting ground should be free from risk of ground crack, slope instability and stable settlements caused by liquefaction or densification in the event of an earthquake.

If the ground research show that such risks do be present, measures should be taken to alleviate its undesirable effects on the structure or the location should be reassessed (Bisch et al., 2011).

There are five types of ground profiles types (A, B, C, D, E), defined by the stratigraphic profiles and parameters is given in Table 3.1.

Table 3.1: Ground Types (EN 1998-1, 2004)

Ground type	Description of stratigraphic profile	Parameters		
		$Vs,30$(m/s)	N_{SPT}(blows/30 cm)	Cu(Kpa)
A	Rock or other rock like geological Formation, containing at most 5 m of weaker material at the surface.	> 800	_	_
B	Deposits of very dense sand, gravel, or very stiff clay, at least several tens of meters in thickness, characterized by a regular increase of mechanical properties with depth.	360 – 800	>50	>250
C	Deep deposits of dense or medium dense sand gravel or stiff clay with thickness from numerous tens to many hundreds of meters.	180 – 360	15 – 50	70 - 250
D	Deposits of loose to medium cohesion less soil (with or without some soft cohesive layers), or of mostly soft to firm cohesive soil.	< 180	< 15	< 70
E	A soil profile containing of a surface alluvium layer with Vs values of type C or D and thickness varying between about 5mand 20m, underlain by stiffer material with Vs > 800 m/s.			
S1	Deposits containing, or consisting a layer at least 10 m thick, of soft clays/silts with a high plasticity index (PI> 40) and high water content	< 100 (Indicative)	_	10-20
S2	Deposits of liquefiable soils, of sensitive clays, or any other soil profile not included in types A – E or S1			

$(V_{S,30})$ Is the average shear wave velocity.

N_{SPT} Is the number of blows in the standard penetration test.

Cu Is the undrained cohesive resistance.

$(V_{S,30})$ this parameter used to select ground types if it is available. When direct information about average shear wave velocity is not available, the other parameters could be used to select the ground type (EN 1998-1, 2004).

In Table 3.1 two additional soil profiles (S1 and S2) are available. For sites with ground situation similar each one of these ground types, special studies for the description of the seismic action are essential.

For these types, and particularly for S2, the possibility of soil failure under the seismic action shall be taken into account. In such event the soil loses its bearing capacity, entailing the collapse of any foundation system before relying on such bearing capacity (EN 1998-1, 2004).

3.1.3 Seismic Action

For every country, the seismic hazard is explained by a zonation map defined by the National Authorities. For this purpose, National territories shall be subdivided by into seismic zones, based on the local risk. By definition, the risk in each zone is assumed to be constant. The reference peak ground acceleration (a_{gR}) is constant. The risk is defined in terms of a single parameter, the value of the reference peak ground acceleration on type *A* ground, a_{gR} (EN 1998-1, 2004).

The reference peak ground acceleration (a_{gR}), for each seismic zone, corresponds to the reference return period T_{NCR} of the seismic action for no-collapse necessity (or equivalently the reference probability of exceedance in 50 years, P_{NCR}) chosen by the National Authorities. A significance factor γI equal to 1.0 is assigned to this reference return period. For return periods other than the reference, the design ground acceleration on type A ground ag is equal to a_{gR} times the significance factor γI ($ag = \gamma . a_{gR}$).

Earthquake motion at a given point on the surface is denoted by an elastic ground acceleration response spectrum, henceforth called an "elastic response spectrum" (EN 1998-1, 2004).

Horizontal elastic response spectrum

For the horizontal components of the seismic action, the elastic response spectrum (EN 1998-1, 2004).

Se(T) is defined by the following expressions, as seen in Figure 3.1

$$Se(T) = ag.S.\left[1 + \frac{T}{TB}.(\eta.2.5 - 1)\right] \qquad 0 \leq T \leq TB \qquad (3.1)$$

$$Se(T) = 2.5\, ag.S.\eta \qquad TB \leq T \leq Tc \qquad (3.2)$$

$$Se(T) = ag.S.\eta.2.5\left(\frac{Tc}{T}\right) \qquad Tc \leq T \leq T_D \qquad (3.3)$$

$$Se(T) = 2.5\, ag.S.\eta.\left(\frac{TcT_D}{T^2}\right) \qquad T_D \leq T \leq 4s \qquad (3.4)$$

Where:

Se(T)	Elastic response spectrum.
T	Vibration period of a linear single degree of freedom system.
ag	Design ground acceleration on type A ground.
TB	Lower limit of a period of the constant spectral acceleration branch.
TC	Upper limit of a period of the constant spectral acceleration branch.
TD	Value defining the beginning of the constant displacement response range of the spectrum.
S	Soil factor.
n	Damping correction factor with a reference value of η=1 for 5% viscous damping.

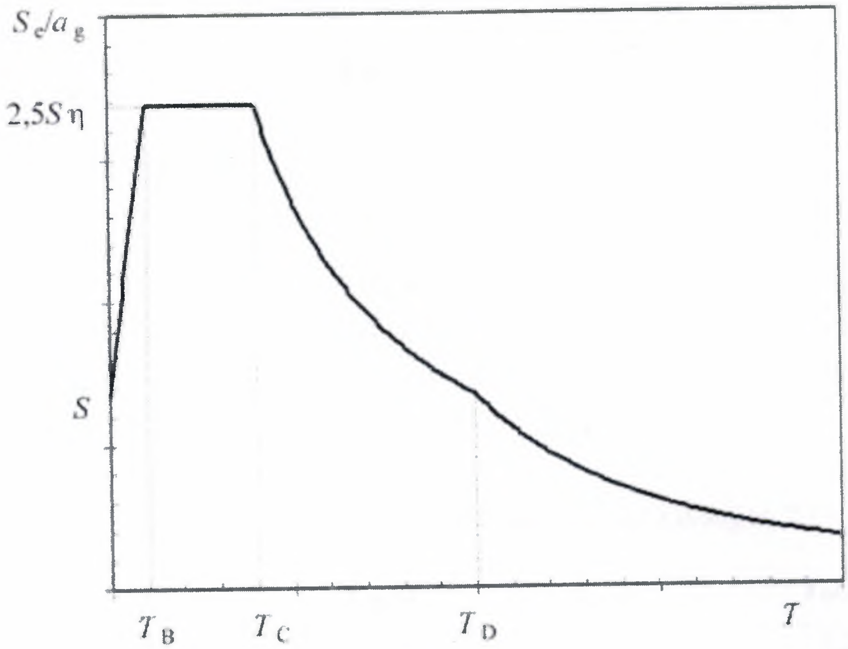


Figure 3.1: Basic shape of the Elastic Response Spectrum (Solomoset al., 2008)

The values of the periods *TB*, *TC* and *TD* and of the soil factor S describing the shape of the elastic response spectrum based upon the ground type.

The values of parameters, *TB*, *TC*, *TD* and *S* for every ground type and type (shape) of spectrum to be used in a country may be found in its National Annex. If the earthquakes that contribute most to the seismic risk described for the site for the purpose of probabilistic risk assessment have a surface-wave magnitude, *Ms*, smaller than 5.5, it is recommended that the Type 2 spectrum is adopted (Laouami and Chebihi, 2014).

For the five ground types *A*, *B*, *C*, *D* and *E* the recommended values of the parameters *S*, *TB*, *TC* and *TD* are given in Table 3.2 for the type 1 spectrum and in Table 3.3 for the type 2 spectrum. Figure 3.2 and Figure 3.3 show the shapes of the recommended type 1 and type 2 spectra, respectively, normalized by ag, for 5% damping.

Table 3.2: Type 1 Elastic Response Spectra (Doğangün and Livaoğlu, 2006)

Ground type	S	$T_B(S)$	$T_C(S)$	$T_D(S)$
A	1	0.15	0.4	2.0
B	1.2	0.15	0.5	2.0
C	1.15	0.20	0.6	2.0
D	1.35	0.20	0.8	2.0
E	1.4	0.15	0.5	2.0

Table 3.3: Type 2 Elastic Response Spectra (Doğangün and Livaoğlu, 2006)

Ground type	S	$T_B(S)$	$T_C(S)$	$T_D(S)$
A	1	0.05	0.25	1.2
B	1.35	0.05	0.25	1.2
C	1.5	0.10	0.25	1.2
D	1.8	0.10	0.30	1.2
E	1.6	0.05	0.25	1.2

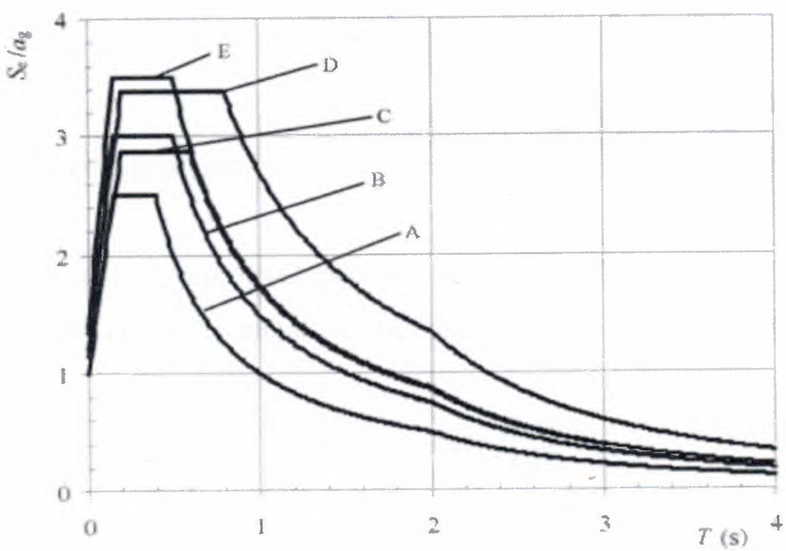


Figure 3.2: Type 1 Elastic Response Spectra for Ground Types A to E 5% damping (Fardis, 2004)

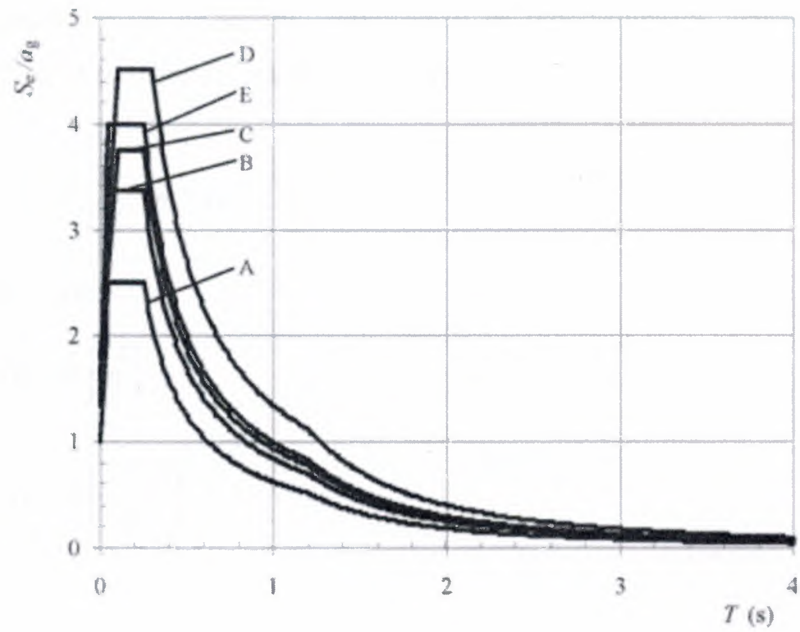


Figure 3.3: Type 2 Elastic Response Spectra for Ground Types A to E5%damping (Fardis, 2004)

For ground types, S_1 and S_2special studies must provide the corresponding values of *S*, T_B, T_C, T_D (EN 1998-1, 2004).

The value of the damping correction factor (η) may be determined by the expression:

$$\eta = \sqrt{\frac{10}{(5+\xi)}} \geq 0.55 \tag{3.5}$$

Where:

ξ : Is the viscous damping ratio of the structure, expressed as a percentage.

The elastic displacement response spectrum, $S_{De}(T)$, shall be achieved by direct transformation of the elastic acceleration response spectrum $S_e(T)$, using the following expression:

$$\mathrm{SDe(T)} = \mathrm{Se(T)}\left[\frac{\mathrm{T}}{2\pi}\right]^2 \tag{3.6}$$

Expression 3.6 should generally be applied for vibration periods not exceeding 4.0 second. For structures with vibration periods longer than 4.0 second, a more complete definition of the elastic displacement spectrum is possible (EN 1998-1, 2004).

Vertical elastic response spectrum (EN 1998-1, 2004).

The vertical component of the seismic action shall be represented by elastic response spectrum, $S_{ve}(T)$, derived using Equation 3.7 to Equation 3.10.

$$Sve(T) = avg.\left[1 + \frac{T}{TB}.(\eta.3.0 - 1)\right] \qquad 0 \leq T \leq TB \qquad (3.7)$$

$$Sve(T) = avg.\eta.3.0 \qquad TB \leq T \leq Tc \qquad (3.8)$$

$$Sve(T) = avg.\eta.3.0\left[\frac{Tc}{T}\right] \qquad Tc \leq T \leq TD \qquad (3.9)$$

$$Sve(T) = avg.\eta.3.0\left[\frac{TcTD}{T}\right]^{2} \qquad TD \leq T \leq 4S \qquad (3.10)$$

Where:

avg Design ground acceleration in the vertical direction.

The values to be ascribed to *TB, TC, TD* and a_{vg} for each type (shape) of vertical spectrum to be used in a country may be found in its National Annex.

The recommended choice is the use of two types of vertical spectra Type 1 and Type 2. As for the spectra describing the horizontal components of the seismic action, if the earthquakes that contribute most to the seismic risk described for the site for the purpose of probabilistic risk assessment have a surface-wave magnitude, *Ms*, not larger than 5.5, it is recommended that the Type 2 spectrum is adopted (EN 1998-1, 2004).

For the five ground types A, B, C, D and E the recommended values of the parameters describing the vertical spectra are given in Table 3.4. These recommended values do not apply for special ground types S1 and S2 (EN 1998-1, 2004).

Table 3.4: Vertical Elastic Response Spectra (EN 1998-1, 2004)

Spectrum	a_{vg}/a_g	T_B(s)	T_C(s)	T_D(s)
Type A	0.90	0.05	0.15	1.0
Type B	0.45	0.05	0.15	1.0

Design spectrum this reduction is accomplished by introducing the behavior factor q. The behavior factor (q) is an approximation of the ratio of the seismic forces that the structure would skill if its response was completely elastic with 5% viscous damping, to the seismic forces that may be used in the design, with a conventional elastic analysis model, still guaranteeing a reasonable reaction of the structure. The value of the behavior factor q may be different in different horizontal directions of the structure, though the ductility arrangement shall be the same in all directions (EN 1998-1, 2004).

For the horizontal components of the seismic action the design spectrum, *Sd(T)*, shall be described by the following equations:

$$Sd(T) = ag.S.\left[\frac{2}{3} + \frac{T}{TB}.\left(\frac{2.5}{q} - \frac{2}{3}\right)\right] \qquad 0 \leq T \leq TB \qquad (3.11)$$

$$Sd(T) = ag.S.\left(\frac{2.5}{q}\right) \qquad TB \leq T \leq TC \qquad (3.12)$$

$$Sd(T)\begin{cases} = & ag.S.\frac{2.5}{q}\left[\frac{Tc}{T}\right] \\ & \geq \beta.ag \end{cases} \qquad Tc \leq T \leq TD \qquad (3.13)$$

$$Sd(T)\begin{cases} = & ag.S.\frac{2.5}{q}\left[\frac{TcTD}{T^2}\right] \\ & \geq \beta.ag \end{cases} \qquad TD \leq T \qquad (3.14)$$

Where:

ag, S, T_C and T_D are as described in the equations before.

Sd (T) Design spectrum.

q Behavior factor.

B Lower bound factor for the horizontal design spectrum.

For the vertical component of the seismic action the design spectrum is given by Equation 3.11 to Equation 3.14, with the design ground acceleration in the vertical direction, a_{vg} replacing a_g, *S* taken as being equal to 1.0 (EN 1998-1, 2004).

For the vertical component of the seismic action a behavior factor q up to 1.5 should commonly be approved for all materials and the structural systems.

The adoption of values for q larger than 1.5 in the vertical direction must be justified through a suitable analysis (EN 1998-1, 2004).

Buildings are classified into four importance classes (γI), based on the consequences of collapse for human life, on their significance for public safety and civil protection in the immediate post-earthquake period and on the social consequences of collapse (EN 1998-1, 2004). The recommended values of γI for significance classes are given in Table 3.5.

Table 3.5: Values of γI for Significant Classes (EN 1998-1, 2004)

Significance classes	**Buildings**	**The recommended value of γ**
I	*Buildings of minor significance for public* safety, e.g. agricultural structures, etc.	0.8
II	Ordinary buildings, not belonging in the other categories.	1.0
III	Building whose seismic resistance is *significance in view of the consequence* related with a collapse, e.g. school, assembly halls, cultural institutions etc.	1.2
IV	Building whose integrity during earthquakes is of vital significance for civil protection, hospitals, fire stations, power plants, etc.	1.4

3.1.4 Method of Analysis

There are four methods of analysis possible for determination of the seismic effects on a structure:

- Lateral force method of analysis.
- Modal response spectrum analysis.

- Non-linear static (pushover) analysis.
- Non-linear time history (dynamic) analysis (EN 1998-1, 2004).

Depending on the structural characteristics of the building

1. Lateral force method of analysis

This type of analysis may be applied to buildings whose response is not significantly affected by contributions from modes of vibration higher than the fundamental mode in each principal direction (EN 1998-1, 2004).

Applies always if:

- They have fundamental periods of vibration *T*1 in the two main directions which are smaller than the following values

$$T \leq \begin{cases} 4Tc \\ 2.0\ s \end{cases} \tag{3.15}$$

Where:

Tc is the upper limit of the period of the constant spectral acceleration branch.

- Building regular in elevation (EN 1998-1. 2004).

The seismic base shear force *F*b, for each horizontal direction in which the building is analysed, shall be determined using the following expression:

$$F_b = S_d(T_1) \cdot m \cdot \lambda \tag{3.16}$$

Where:

Sd Is the ordinate of the design spectrum at period T1

T1 is the fundamental period of vibration of the building for lateral motion in the direction considered

m Is the total mass of the building, above the foundation or above the top of a rigid basement

λ Is the correction factor, the value of which is equal to: $\lambda = 0.85$ if T1 < 2 TC and the building has more than two storeys, or $\lambda = 1.0$ otherwise

The fundamental mode shapes in the horizontal directions of analysis of the building may be calculated using methods of structural dynamics or may be approximated by horizontal displacements increasing linearly along the height of the building (EN 1998-1, 2004).

The seismic action effects shall be determined by applying, to the two planar models, horizontal forces Fi to all storeys (EN 1998-1, 2004).

$$Fi = Fb.\frac{si.mi}{\sum sj.mj} \tag{3.17}$$

Where:

Fi Is the horizontal force acting on storey *i*

Fb Is the seismic base shear

si, sj Are the displacements of masses *m*i, *m*j in the fundamental mode shape

mi, mj Are the storey masses

2. Modal response spectrum analysis

This type of analysis shall be applied to buildings which do not satisfy the conditions given for applying the lateral force method of analysis (EN 1998-1, 2004).

The response of all modes of vibration contributing significantly to the global response shall be taken into account. This requirement may be deemed to be satisfied if either of the following can be demonstrated:

- The sum of the effective modal masses for the modes taken into account amounts to at least 90% of the total mass of the structure;
- All modes with effective modal masses greater than 5% of the total mass are taken into account (EN 1998-1, 2004).

3.1.5 Combinations of the Seismic Action with other Actions

The design value Ed of the impacts of actions in the seismic design state shall be determined in accordance with the following combination (EN 1998-1, 2004):

$$Ed = \Sigma\, G_{kj} + \gamma\, A_{Ed} + \Sigma\, \psi_{2i}\, Q_{ki} \qquad (3.18)$$

Where:

γ_I Importance factor as seen in Table 3.5.

G_{kj} Characteristic value of dead loads.

A_{Ed} Design value of return period of specific earthquake motion;

ψ_{2i} Combination coefficient of live load.

Q_{ki} Characteristic value of live load.

The inertia effects of the design seismic action shall be evaluated by taking into account the presence of the masses associated with all gravity loads (EN 1998-1, 2004). Is showing in the following combination of action:

$$\Sigma\, G_{kj} + \Sigma\, \psi_{Ei} \cdot Q_{ki} \qquad (3.19)$$

Where:

$\Psi_{E,i}$ Is the combination coefficient for the variable action I.

The combination coefficient ($\Psi_{E,i}$) is calculated by the following equation:

$$\Psi_{E,i} = \varphi . \psi_{2i} \quad (3.20)$$

Values for φ and ψ_{2i} can be taken from Tables 3.6 and 3.7 where the building types are summarized in categories; A-G.

Table 3.6: Values of φ for calculating $\Psi_{E,i}$ (Cyprus National Annex EN1998-1:2004)

Type of variable	**Storey**	**Φ**
Categories A-C	Roof.	1.0
	Storeys with correlated occupancies.	0.8
	Independently occupied storeys.	0.5
Categories D-F and Archives		1.0

Table 3.7: Values of Ψ factors for buildings (EN 1998-1, 2004)

Actions	Ψ_0	Ψ_1	Ψ_2
Category A: domestic, residential areas	0.7	0.5	0.3
Category B: office areas	0.7	0.5	0.3
Category C: congregation areas	0.7	0.7	0.6
Category D: shopping areas	0.7	0.7	0.6
Category E: storage areas	0.1	0.9	0.8
Category F: traffic area, vehicle weight ≤30 Kn	0.7	0.7	0.6
Category G: traffic area, 30kN <vehicle weight ≤160kN	0.7	0.5	0.3

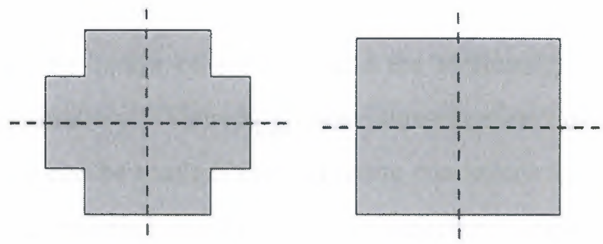
3.1.6 Criteria for Structural Regularity

There are two types of design building criteria should be achieved as possible, which are for regularity in plan and in elevation.

3.1.6.1Criteria for Regularity in Plan

Building regular in plan, it should be satisfied some conditions. These are:

1. The building structure with respect to the mass distribution and lateral stiffness, shall be symmetrically in plan with respect two orthogonal axes (EN 1998-1, 2004; D'Aniello, 2011).



2. The slenderness the ratio between larger and smaller length of the building must be equal or smaller than 4 (EN 1998-1, 2004; D'Aniello, 2011).

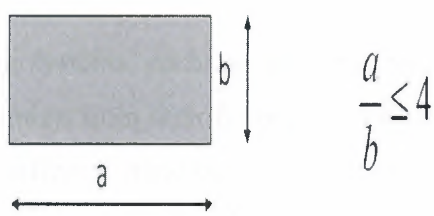
$$\lambda = L_{max} / L_{min} \leq 4 \tag{3.21}$$

Where:

λ Slenderness.

L_{max} Larger dimension in plan of the building.

L_{min} Smaller dimension in plan of the building.



3. The structural eccentricity (e_0) shall be smaller than 30% of torsional radius (r), for every direction of analysis x and y (D'Aniello, 2011).

$$e_{0X} \leq 0.30 r_X, \;\; e_{0Y} \leq 0.30 r_Y \tag{3.22}$$

$$r_X , r_y \geq ls \tag{3.23}$$

Where:

e_{ox} : Distance between the center of stiffness and the center of mass measured along the x direction, which is normal to the direction of analysis considered.

rx : Is the square root of the ratio of torsional stiffness to the horizontal stiffness in the *y* direction (torsional radius).

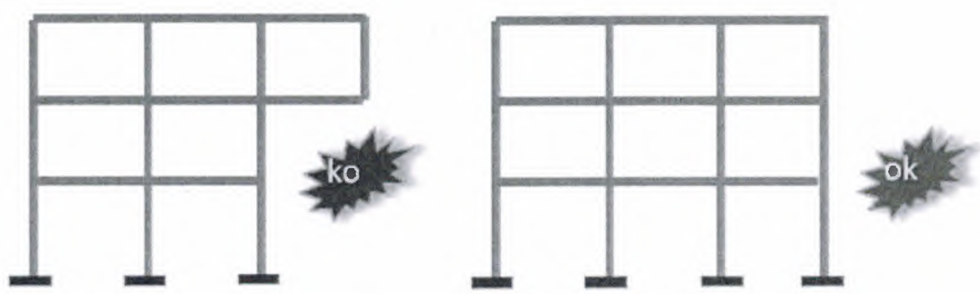
ls: Is the radius of the gyration of floor mass in plan.

4. In multi storey buildings the center of stiffness and the torsional radius can be determined only approximately. Therefore, for classification of structural regularity, a simplification can be made if the following conditions are satisfied:
   - All horizontal load resisting systems, such as structural walls, frames, or cores, run with no interruption from the foundations to the highest point of the building (EN 1998-1, 2004; D'Aniello, 2011).
   - The deflected shaped of the individual systems under lateral loads are not much different. This situation can considered in the case of wall systems and frame systems (EN 1998-1, 2004; D'Aniello, 2011).

3.1.6.2 Criteria for Regularity in Elevation

Building to be categorized as being regular in elevation, it shall fulfill all the circumstances below:

1. All horizontal load resisting systems, such as structural walls, frames, or cores, shall run without the interruption from their footings to the top of the structure or, if setbacks are present at different heights, to the top of pertinent zone of the building (EN 1998-1, 2004).



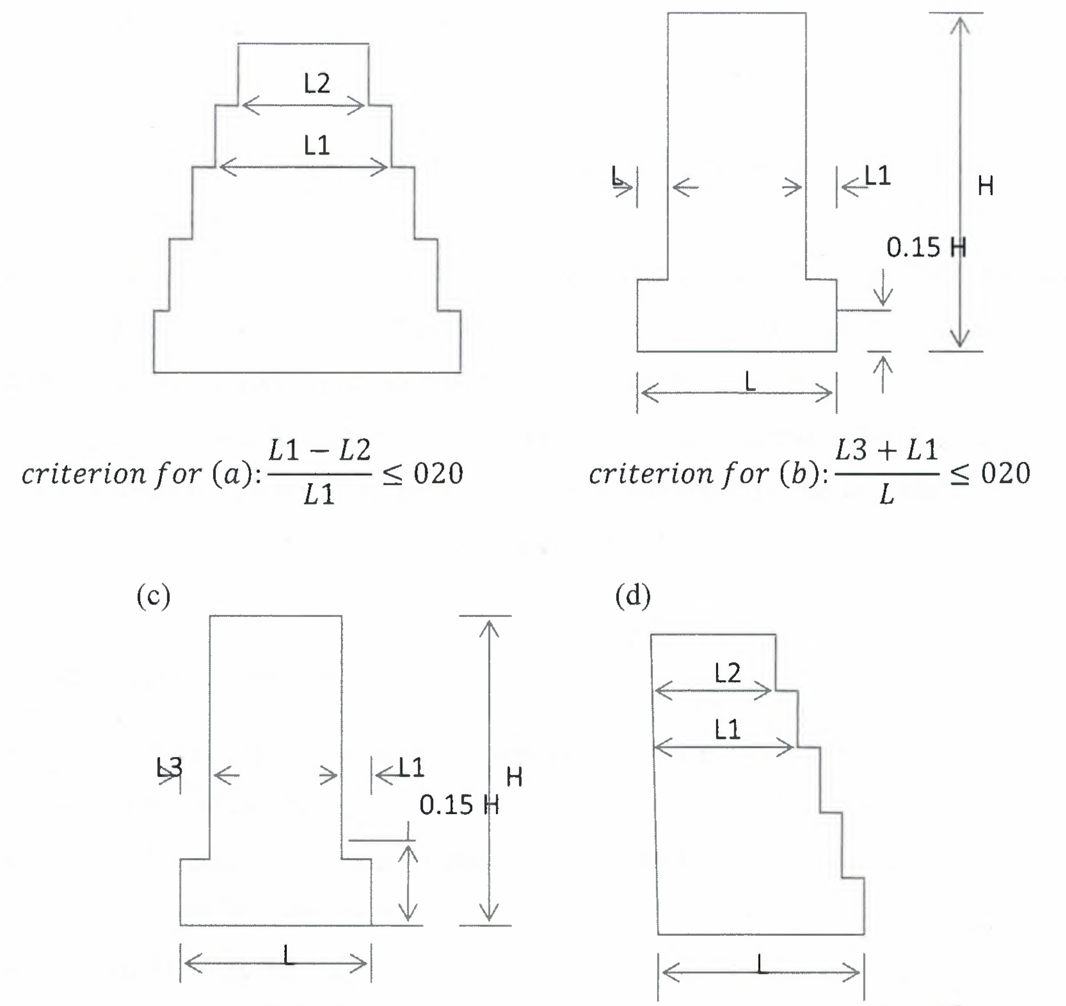
2. Both the horizontal stiffness and the mass of the single stories shall remain constant or decrease regularly, without abrupt changes, from the foundation to the top of a particular structure (EN 1998-1, 2004; D'Aniello, 2011).

3. In frame buildings the ratio of the actual stories resistance to the resistance desired by the analysis should not differ disproportionately between contiguous stories (EN 1998-1, 2004; D'Aniello, 2011).

4. When setbacks are existent, the following extra circumstances apply:

   - *For regular setbacks protection axial symmetry, the setback at any story shall* be equal or smaller than 20 % of the previous plan dimension in the direction of setback as seen in Figure a and Figure b.
   - For an individual setback within the lesser 15 % of the total height of the chief structural system, the setback should be equal or smaller than 50 % of the former plan dimension as shown in Figure c. In this situation the structure of the foundation zone in the vertically projected perimeter of the higher storeys should be designed to resist smallest amount 75% of the lateral shear forces that would development in that region in a similar building without the foundation enlargement (D'Aniello, 2011).
   - If the setbacks do not protect symmetry, in each face the quantity of the setbacks at all storeys shall not be larger than 30 % of the plan dimension at the ground floor above the foundation or above the top of a rigid basement, and the single setbacks shall not be bigger than 10 % of the former plan dimension as seen in Figure 3.4.d (D'Aniello, 2011).

(a) (b)



$$criterion\,for\,(a){:}\frac{L1-L2}{L1}\leq 020 \qquad criterion\,for\,(b){:}\frac{L3+L1}{L}\leq 020$$

$$criterion\,for\,(c){:}\frac{L3-L1}{L}\leq 0.50 \qquad criterion\,for\,(d){:}\frac{L-L2}{L}\leq 0.30$$

$$\frac{L1-L2}{L1}\leq 0.10$$

Figure 3.4: Criteria for Regularity of Buildings with Setbacks (D'Aniello, 2011)

3.1.7 Design of Reinforced Concrete Structures

The reinforced concrete building elements are divided into three types according to their ductility level; low ductility (DCL), medium ductility (DCM) and high ductility level

(DCH). For DCL building elements are not preferred to use in region with seismic risk (EN 1998-1, 2004).

- Low ductility class: corresponds to structures designed and dimensioned according to EC2, completed by the specific rules to enhance ductility.
- Medium ductility class: corresponds to structures designed, dimensioned and detailed according to previous recorded earthquakes, allowing the structure to work in the inelastic domain under cyclic actions, without brittle failures.
- High ductility class: corresponds to structures designed, dimensioned and detailed so that the structural response to seismic action is according to the considered failure mechanism, with a large amount of energy dissipated.

3.1.7.1 Material requirement

- For DCM, in primary seismic element the Concrete class C16/20 shall use as a lower class of concrete.
- For DCM, reinforcing steel of class B or C shall be used in the critical zones of primary seismic elements.
- For DCM, may be used the welded wire meshes if they meet the design of condition.
- For DCM & DCH in critical zones of primary seismic elements, with the exceptions of the closed stirrups and cross-tie, only ribbed bars shall be used.
- For DCH, in primary seismic element a concrete class greater than C 20/25 shall be used (EN 1998-1, 2004).
- For DCH, in critical regions of primary seismic elements, only class C reinforcement steel must be used as shown in Table 3.8. This shows the properties of reinforcing steel classes according to Eurocode2.

Table 3.8: Properties of Reinforcement (EN1992-1-1,2004)

Product form	**Bars and de-coiled rods**			**Wire Fabrics**			**Requirement or quintile value (%)**
Classes	A	B	C	D	E	F	-
Characteristic yield strength $f_{0.2k}$ or f_{yk}(MPa)	400 to 600						5.0
Minimum value of $k = (f_t/f_y)_k$	≥1.05	≥1.08	≥1.15 <1.35	≥1.05	≥1.08	≥1.15 <1.35	10.0
Characteristic strain at maximum force, ε_{uk} (%)	≥2.5	≥5.0	≥7.5	≥2.5	≥5.0	≥7.5	10.0
Bendability	Bend/Rebind test			-			
Shear strength	-			$0,3\ A\ f_{yk}$ (A is area of wire)			Minimum
Maximum Nominal Deviation bar from size (mm) nominal mass ≤ 8 (individual bar > 8 or wire) (%)	± 6.0 ± 4.5						5.0

3.1.7.2 Geometrical restrictions

1-Beam

- To obtain benefit of the favorable outcome of column compression on the connection of reinforcement passing through the beam/column joint, the

$$widthofbeam \leq columnwidth + depthofbeam \qquad (3.24a)$$

$$or$$

$$\leq twicecolumnwidth \qquad (3.24b)$$

- For DCM & DCH, The space between the centroidal axes of two beams is restricted to less than *bc/4* (EN 1998-1, 2004).

  Where:

  bc: The largest cross- sectional element of the column normal toward the longitudinal axis of the beam

- For DCH, the width of the primary seismic beam should be greater than or equal 200 mm (EN 1998-1, 2004).
- For DCH, in primary seismic beams the width to height ratio of the web shall satisfy the following expression below

$$\left(\frac{lot}{b}\right) \leq \left(\frac{70}{(h/b)^{1/3}}\right) \qquad \text{And} \qquad h\,/\,b \leq 3.5 \qquad (3.25)$$

  Where:

  I_{ot} Is the distance torsional restraints.

  b Is the total depth of beam in central part of I_{ot} .

  h Width of compression flange.

2-Column

- For DCM & DCH, the cross sectional dimensions of primary seismic column must be smallest amount 1/10 distance connecting the point of contra flexure and the ending of column, if the inter storey drift sensitivity coefficient Θ is bigger than 0.1 (EN 1998-1, 2004).
- For DCH the cross sectional dimension of primary seismic column shall be higher than or equal 25 cm (EN 1998-1, 2004).

3- Ductile walls

- For DCM & DCH, the thickness of the web of wall (b_{wo}) should be greater than of clear storey height (h_s) divided by 20, or by a minimum of 0.15m (EN 1998-1, 2004).

$$bwo > hs/20 \tag{3.26a}$$

$$or$$

$$bwo \geq 0.15m \tag{3.26b}$$

- For DCH, random opening, not commonly given to form coupled walls, must be avoid in primary seismic walls, if their influence is also not important or accounted for analysis, dimensioning and detailing (EN 1998-1, 2004).
- For DCM the width of the boundary element (b_w) should be higher than or equal (0.20)m if :-
- The length of boundary element $Ic \leq max\begin{cases} 2\ bw \\ 0.2\ Iw \end{cases} \quad then\rangle$ (3.27)

  The width of the boundary element $bw \geq hs/15$ (3.28)

- The length of boundary element $Ic > \max\begin{cases} 2bw \\ 0.2Iw \end{cases} \quad then\rangle$ (3.29)

  The width of the boundary element $bw \geq hs/10$ (3.30)

Where:

b_{wo} Is the thickness of the web of reinforcement concrete wall.

Ic Is the length of boundary element.

bw Is the width of boundary element.

Iw Is the wall cross section length.

h_s Is the clear story height.

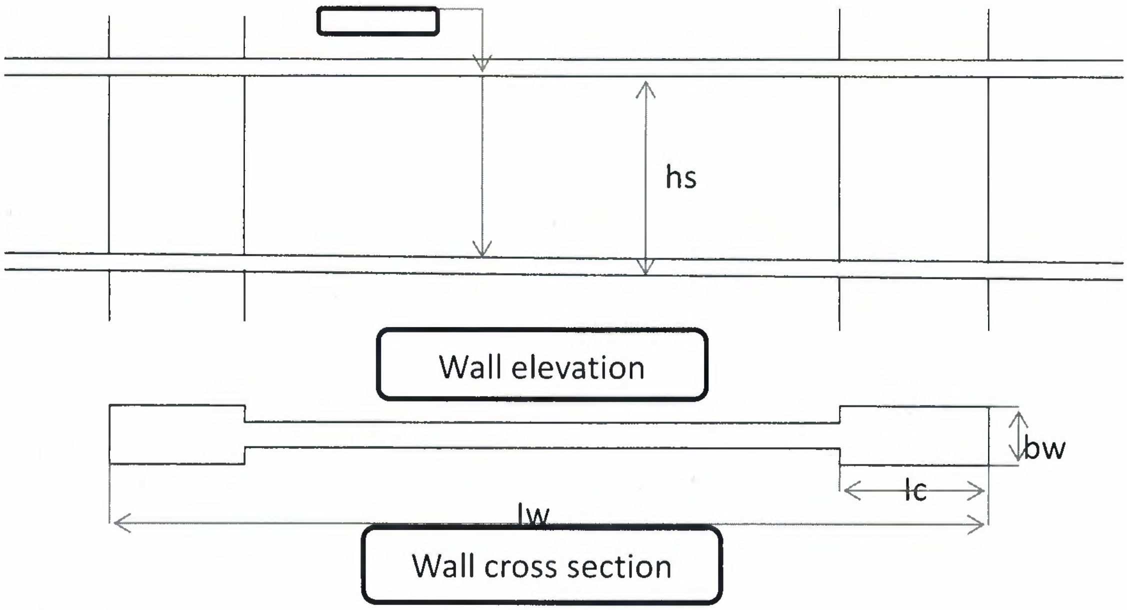


Figure 3.5: Minimum thickness of Wall Boundary Elements (EN 1998-1, 2004)

3.1.7.3 Reinforcement Conditions

1- Beam Reinforcement Conditions

Beam reinforcement conditions are explained in the Table 3.9.

Table 3.9: Generals Rules of EC8 Beams Reinforcement Design (Bisch et al., 2011)

	DCH	DCM
"critical region" length (1)	$1.5h_w$	h_w
- *Longitudinal bars (L):*		
ρ_{min}, tension side (2)	$0.5f_{ctm} / f_{yk}$	
ρ_{max}, critical regions(3)	$\rho' + 0018f_{cd} / (\mu_\varphi\ \varepsilon_{sy,\ d} f_{yd})$	
$A_{s,min}$, top & bottom	2Ø14 (308mm^2)	-
$A_{s,min}$, critical regions	$0.5A_{s,top}$	-
$A_{s\ min}$, top – span	$A_{s,top-supports} / 4$	
$A_{s\ min}$, supports bottom	$A_{s,\ bottom\ span} / 4$	-
d_{bL} / h_c – bar crossing interior joint(4)	$\leq \dfrac{6.25(1+0.8\nu d)}{(1+0.75\frac{\rho'}{\rho max})}\dfrac{fctm}{fyd}$	$\leq \dfrac{7.5(1+0.8\nu d)}{(1+0.5\frac{\rho'}{\rho max})}\dfrac{fctm}{fyd}$

d_{bL} / h_c – bar crossing exterior joint[4]	$\leq 6.25(1+0.8vd)\dfrac{fctm}{fyd}$	$\leq 6.25(1+0.8vd)\dfrac{fctm}{fyd}$
- Transverse bars (w)		
I- Outside critical regions[5]		
Spacing (s_w)	$\leq 0.75d$	
ρ_w	$0.08\sqrt{(f_{ck}(\text{Mpa}) / f_{yk}(\text{Mpa}))}$	
II- In critical regions[5]		
d_{bw}[6]	$\geq 6\text{mm}$	
spacing s_w	$\leq \min\{6d_{bL}, h_w/4, 24b_w, 175\text{mm}\}$	$\leq \min\{8d_{bL}, h_w/4, 24b_w, 225\text{mm}\}$

(1) For beams supporting non continue (cut-off) vertical elements, the critical length shall be $2h_w$.

Where:

h_w Is the depth of the beam.

(2) f_{ctm} is the main value tensile strength of concrete, and f_{yk} is the characteristic yield strength.

(3) f_{cd} is the design value of concrete compressive strength, μ_ϕ is the value of curvature ductility factor that agrees to the basic value, q_o, of the behavior factor used in the design as: $\mu_\varpi=2q_o-1$ *if* $T \geq T_C$ *or* $\mu_\varpi=1+2(q_o-1)T_C/T$ *if* $T<T_C$. $\varepsilon_{sy,d}$ is the design value of steel at yield, and f_{yd} is the design value of yield strength of steel.

(4) h_c is the depth of column in the bar direction, d_{bL} is the diameter of the longitude bars and $v_d = N_{Ed} / A_c f_{cd}$ is the load ratio of column axial , for the algebraically lowest value of the axial load due to the design seismic action plus concurrent gravity (compression: positive).

(5) The first hoop shall be $\geq$ 50mm from the first beam end section.

(6) $d_{bw}$is the diameter of hoops.

2- Column Reinforcement Conditions

Columns reinforcement conditions are explained in the Table 3.10.

Table 3.10: Generals Rules of EC8 for Columns Reinforcement Design (Fardis, 2008)

	DCH	DCM
"critical region" length (1)	max{$1.5h_c$, $1.5b_c$, 0.6m,$l_c/5$}	max{h_c, b_c, 0.45m, $l_c/6$}
- Longitudinal bars (L):		
ρ_{lmin}	*0.01*	
ρ_{lmax}	*0.04*	
Symmetrical cross-sections	$\rho=\rho'$	
At the corners(2)	One bar along each column side	
Spacing between restrained bars	$\leq$ 150mm	$\leq$ 200mm
Distance of unrestrained bar from nearest restrained	$\leq$ 150mm	
- Transverse bars (w):		
Outside critical regions:		
Spacing s	min{$20d_{bL}$, h_c, b_c,400mm}	min{$12d_{bL}$, $0.6h_c$, $0.6b_c$, 240mm}
Within critical regions:		
d_{bw} (4)	$\geq$ {6mm, $0.4(f_{ydL}/f_{ywd})^{1/2}$ $d_{bL,max}$}	$\geq$ {6mm, $d_{bLmax}/4$}
Spacing s	min{$6d_{bL}$, $b_o/3$, 125mm}	min{$8d_{bL}$, $b_o/2$, 175mm}
$\omega_{wd,mim}$ (5)	0.08	-
$\alpha\omega_{wd}$ (6)	$30\ \mu_\phi\ v_d \varepsilon_{sy,d} b_c/b_o - 0.035$	
In critical region at the column base:		
$\omega_{wd,mim}$ (5)	0.12	0.08
$\alpha\omega_{wd}$	$30\ \mu_\phi\ v_d \varepsilon_{sy,d} b_c/b_o - 0.035$	-

(1) If $(l_c/h_c) < 3$, the entire length of the column shall be considered as a critical regions and shall be reinforced accordingly.

Where:

l_c is the length of the column.

hc is the biggest cross-sectional dimension of the columns (in meters).

b_c is the cross-sectional dimension of column.

(2) As minimum one intermediate bar shall be supplied between corner bars along every column side, to make sure the integrity of the beam-column joints.

(3) d_{bw} is the diameter of the hoops.

(4) ω_{wd} is the volume ratio of confining hoops to that of the confined core to the centerline of the perimeter hoop times f_{yd}/f_{cd}.

(5) α is the confinement effectiveness factor, computed as $\alpha = \alpha_s.\alpha_n$, where $\alpha_s=(1\text{-}s/2b_o)$ for hoops and $\alpha_s=(1\text{-}s/2b_o)$ for spirals : $\alpha_n=1\text{-}\{b_o/((n_h\text{-}1)h_o)+h_o/((n_b\text{-}1)b)\}/3$ for rectangular hoops with n_b legs parallel to the face of the core with length b_o and n_h legs parallel to the one with length h_o.

(6) Index c represent the full concrete section and index o is the confined core to the middle of the perimeter hoop, $b_o$is the smaller face of this core.

3-Ductile Shear-Wall Reinforcement Conditions:

Ductile shear wall reinforcement conditions are explained in the Table 3.11.

Table 3.11: Generals Rules of EC8 for Ductile Shear-Wall Reinforcement Design (Bisch et al., 2011)

	DCH	DCM
"critical region" length [1]	$\geq$ max (l_w, $H_w/6$) $\leq$ min ($2l_w$, h storey) if $\leq$ 6 storey $\leq$ min ($2l_w$, 2h storey) if > 6 storey	
boundary elements:-		
a) In critical regions		
- length of lc from the edge $\geq$	$0.15l_w$, $1.5b_w$, length over which $\varepsilon_c > 0.0035$	
- thickness $b_w$over $\geq$	0.2m; $h_{st}/15$ if $l_c \leq$ max ($2b_w$, $l_w/5$), $h_{st}/10$ if $l_c >$ max($2b_w$, $l_w/5$)	
- vertical reinforcement:		
$\rho_{w,min}$	0.5%	
$\rho_{w,max}$	4%	
confining hoop (w)[2]:		
$d_{bw} \geq$	6mm, $0.4(f_{yd} / f_{ywd})^{1/2}$ d_{bL}	6mm
spacing $s_w \leq$	$6d_{bL}$, $b_o/3$, 125mm	$8d_{bL}$, $b_o/2$, 175mm

$\omega_{wd} \geq$	0.12	0.8
$\alpha\omega_{wd} \geq$[3]	$30\ \mu_\varphi\ (v_d + \omega_v)\varepsilon_{sy,d} b_c/b_o - 0.035$	
b) over the rest of the wall height	In parts of the section where $\varepsilon_c > 0.2\%$: $\rho_{v,min} = 0.5\%$; elsewhere 0.2% In parts of the sections where $\rho > 2\%$:- - distance of unstrained bar in compression zone from adjacent restrained bar ≤ 150mm; - hoops with $d_{bw} \geq$ max (6mm, $d_{bL}/4$) & spacing $s_w \leq$ min ($20d_{bL}$, b_{wo}, 400mm) beyond that distance.	
Web:		
Vertical bars (v) :		
$\rho_{v,min}$	Where in the section $\varepsilon_c > 0.2\%$: 0.5%; elsewhere 0.2%	
$\rho_{v,max}$	4%	
$d_{bv} \geq$	8mm	
$d_{bv} \leq$	$b_{wo}/8$	
spacing s_v	min($25d_{bw}$, 250mm)	min($3b_{wo}$, 400mm)
horizontal bars (h) :		
$\rho_{h,min}$	0.2 %	max (0.1%, $25\rho_v$)
$d_{bh} \geq$	8mm	-
$d_{bh} \leq$	$b_{wo}/8$	-
spacing $s_h \leq$	min { $25d_{bh}$,250mm }	400mm
axial load ratio vd $N_{Ed} / A_c f_{cd}$	≤ 0.35	≤ 0.4

(1) l_w is denoted as the long side of rectangular wall section, H_w is defined as the total wall height, h_{storey} is defined as the storey height.

(2) For DCM: If below the maximum axial force in the wall from the analysis for design seismic action plus agreeing gravity the wall axial load ratio $v_d = N_{Ed}/A_c f_{cd}$ satisfies $v_d \leq 0.15$, the DCL rules may be functional for the confining reinforcement of the boundary elements, these DCL rules apply also if this value of the wall axial load ratio is $v_d \leq 0.2$but the value of the q used in the design of the structure is not bigger than 85% of the q-value permissible when the DCM confining reinforcement is used in boundary elements.

(3)μ_φ is the value of the curvature ductility factor that corresponds as: μ_φ=2qo-1 if T≥TC or μ_φ=1+2(qo-1)TC/T if T<TC, to the product of the basic value qo of the

behavior factor times the value of the ratio M_{Edo} / M_{Rdo} at the base of the wall $\varepsilon sy,d= f_{yd}/E_s$,ωvd is the mechanical ratio of the vertical web reinforcement.

3.1.8 Foundation Tie-beams

The minimum value of a cross sectional width of tie beams ($b_{w,min}$) should be equal 250 mm. and the minimum value of across sectional depth of tie beams ($h_{w,min}$) should be equal 400 mm for the buildings with up to three storeys, or equal 500 mm for those with four storeys or more above the basement. As seen in Appendix 4

The minimum longitudinal reinforcement ratio ($\rho_{b,min}$) of foundation tie beams at the top and bottom shall not be less than 0.4% (EN 1998-1, 2004).

The minimum thickness of foundation slabs (t_{min}) shall not be less than 200 mm, and a minimum reinforcement ratio ($\rho_{s,min}$) of foundation slabs at the top and bottom shall not be less than 0.2% (EN 1998-1, 2004).

3.2 Turkish Earthquake code 2007

The Turkish Earthquake code 2007 Specifications for Buildings to be built in Earthquake Zones (2007). It is used for Turkey and Turkish Republic of Northern Cyprus. After the 1999 Marmara earthquake, which was the most dangerous earthquake of Turkey in the previous century, the requirements have been added to the Turkish earthquake code. 1998 disaster regulation was revised in 2007 in which the new regulation was called Specifications for Buildings to be built in Earthquake Areas (Soyluk and Harmankaya, 2012).

3.2.1 General rules of the Turkish Earthquake code are as follows:

- To prevent any damage in structural and nonstructural elements in the case of low intensity earthquake.
- To limit the damage structural and nonstructural elements to reparable levels in medium-intensity earthquakes.
- To prevent overall or partial collapse of building in the case of high intensity earthquake in order to avoid loss of life (TEC, 2007).

In this specification the design earthquake considered corresponds to high intensity. For buildings with building importance factor of I=1 in accordance with table 3.12, the probability of exceedance of the design earthquake in a period of 50 years is 10%.

Table 3.12: Building Importance Factor (TEC, 2007)

Purpose of Occupancy or Type of Building	Importance Factor (I)
<u>1. Buildings required to be used after the earthquake and buildings containing hazardous materials</u> a) Buildings required to be used immediately after the earthquake (Hospitals, health wards, dispensaries, firefighting buildings and facilities, transportation stations and terminals, PTT and other telecommunication facilities, power generation and distribution facilities, governorate, county and municipality administration buildings, first aid and emergency planning stations) b) Buildings containing or storing toxic, explosive and flammable	1.5

materials, etc.	
2. Intensively and long-term occupied buildings and buildings preserving valuable goods a) Schools, other educational buildings and facilities, hostels and dormitories, prisons, military barracks, etc. b) Museums	1.4
3. Intensively but short-term occupied buildings Sport facilities, theatre and concert halls, cinema, etc.	1.2
4. Other buildings Buildings other than above defined buildings. (Hotels, residential and office buildings, building-like industrial structure, etc.)	1.0

An official seismic hazard zonation map for Turkey was prepared recently by the Ministry of the Public Works and Settlement considering the latest knowledge of earthquakes which divides Turkey into five seismic zones according to their seismic activity, as seen in Figure3.6, where the places with warm colors considered as high seismic activity (Zoellick, 2012).

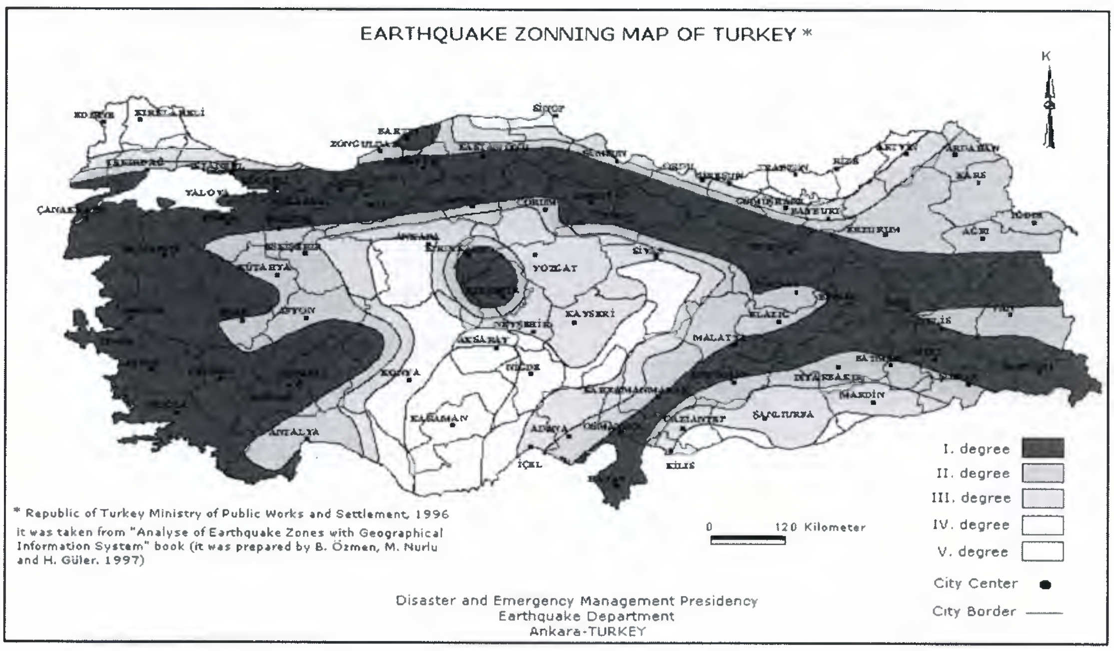


Figure 3.6: Seismic Hazard Zonation Map of Turkey (Zoellick, 2012)

3.2.2 Ground Condition

Soil groups and local site classes shall be consider because the bases of determination of the local soil circumstances are given in Table 3.13 and Table 3.14, also the value of soil parameters in Table3.13 should be considered as standard values given for help only in determining the soil groups (TEC, 2007).

Soil investigation based on suitable site and laboratory tests and mandatory to be behavior for all buildings by total height greater than 60 m in the first seismic zone and second seismic zone, and buildings in all seismic zones by building important factors of I=1.5 and I=1.4 (TEC, 2007).

During every part of seismic zones, Group (D) soils according to Table 3.13.

With water table less than 10 m from the soil surface shall be investigated and the resulted shall be known to recognize whether the liquefaction probable exists, by using suitable analytical methods based on in-situ and laboratory tests (TEC, 2007).

Table 3.13: Soil Groups (TEC, 2007)

Soil Group	Description of Soil Group	Standard Penetration (N/30)	Relative Density (%)	Unconfined Compressive Strength (kPa)	Drift Wave Velocity (m / s)
(A)	1. Massive volcanic rocks, un weathered sound metamorphic rocks, stiff cemented sedimentary rocks	-	-	>1000	>1000
	2. Gravel, very dense sand.	> 50	85-100	-	>700
	3. Silty clay and hard clay.	> 32	-	> 400	> 700
(B)	1. Soft volcanic rocks such as tuff and agglomerate, weathered cemented sedimentary rocks with planes of discontinuity	-	-	500-1000	700-1000
	2. Gravel, dense sand.	30-50	65-85	-	400-700
	3. Silty clay, very stiff clay.	16-32	-	200-400	300-700
(C)	1. Highly weathered soft	-	-	< 500	400-700

	metamorphic rocks and cemented sedimentary rocks with planes of discontinuity				
	2. Gravel and Medium dense sand.	10-30	35-65	-	200-400
	3. Silty clay and Stiff clay.	8-16	-	100-200	200-300
(D)	1. Soft, deep alluvial layers with high ground water level.	-	-	-	< 200
	2. Loose sand.	< 10	< 35	-	< 200
	3. Silty clay and Soft clay.	< 8	-	< 100	< 200

Table 3.14: Local Site Classes (TEC, 2007)

Local Site Class	**Soil Group according to Table 3.13 and Topmost Soil Layer Thickness (h1)**
Z1	Group (A) soils. Group (B) soils with h1 < 15m
Z2	Group (B) soils with h1 > 15m. Group (C) soils with h1 < 15m
Z3	Group (C) soils with 15 m < h1 < 50m. Group (D) soils with h1 <10m
Z4	Group (C) soils with h1 > 50 m. Group (D) soils with h1 > 10m

3.2.3 Seismic Action

The spectral acceleration coefficient $A\ (T)$ shall be considered as the foundation for the determination of seismic loads is shown by Equation 3.31. The elastic spectral acceleration $S_{ae}\ (T)$, which is defined as ordinate of the elastic acceleration spectrum defined for 5% damped rate, elastic acceleration spectrum is equal to spectrum acceleration coefficient times the acceleration of gravity, g, as given by Equation 3.32 (TEC, 2007).

$$A\ (T) = A_0\, I\, S(T) \tag{3.31}$$

$$S_{ae}(T) = A\ (T)_g \tag{3.32}$$

Where:

A_0 Effective ground acceleration coefficient.

I Building importance factor.

$S(T)$ Spectrum coefficient.

g Gravitational acceleration (9.81 m/s2).

The effective ground acceleration coefficient (A_0), is indicated in Table 3.15.

Table 3.15: Effective Ground Acceleration Coefficient (TEC, 2007)

Seismic Zone	A_0
1	0.4
2	0.3
3	0.2
4	0.1

The spectrum coefficient *S(T)*, depending on the building natural period, *T* and the local site conditions (TEC, 2007).

$$S(T) = 1 + 1.5\frac{T}{TA} \qquad 0 \leq T \leq TA \tag{3.33}$$

$$S(T) = 2.5 \qquad TA \leq T \leq TB \tag{3.34}$$

$$S(T) = 2.5(\frac{TB}{T})^{0.8} \qquad TB < T \tag{3.35}$$

Spectrum characteristic periods, T_A and T_B, are identified in Table 3.16, dependent on local site classes defined in Table 3.14.

Table 3.16: Spectrum characteristic Periods (TEC, 2007)

Local Site Class	T_A *(second)*	*TB (second)*
Z1	*0.10*	*0.30*
Z2	*0.15*	*0.40*
Z3	*0.15*	*0.60*
Z4	*0.20*	*0.90*

In case where the requirements specified before are not met, spectrum characteristic periods defined in Table 3.16 for local site class Z4 shall be used.

In required cases, elastic acceleration spectrum can be determined through special investigations by considering site conditions and local seismic. But spectral acceleration coefficients corresponding to acquired acceleration spectrum ordinates shall in no case be less than those determined by Equation 3.31 based on relevant characteristic periods indicated in Table 3.11 (TEC, 2007).

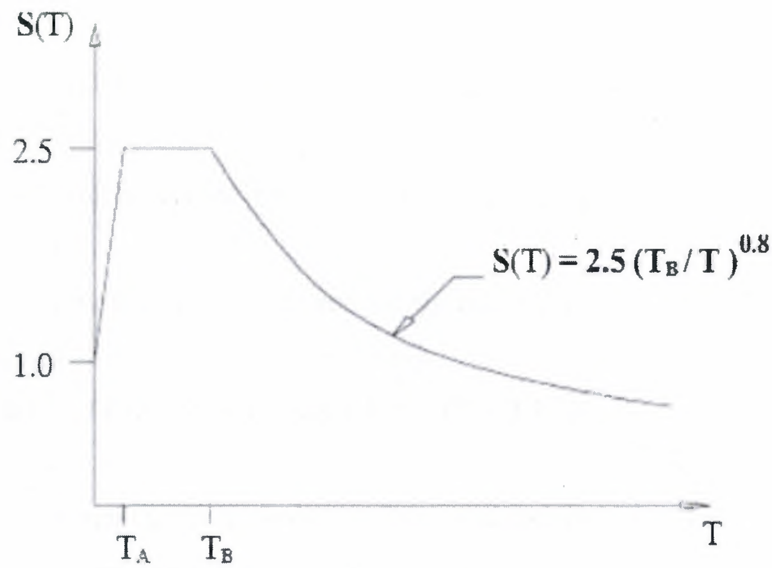


Figure 3.7: Design Acceleration Spectrums (TEC, 2007)

In order to consider the specific nonlinear behavior of the structural system during earthquake, elastic seismic loads to be determined in terms of spectral acceleration coefficient shall be divided to below defined Seismic Load Reduction Factor to account for. Seismic Load Reduction Factor, shall be determined by Equations 3.36 or 3.37 in terms of Structural System Behavior Factor, R, defined in Table 3.17 for reinforced concrete structural systems and the natural vibration period T (TEC, 2007).

$$Ra(T) = 1.5 + (R - 1.5)\frac{T}{TA} \qquad 0 \leq T \leq TA \qquad (3.36)$$

$$Ra(T) = R \qquad TA < T \qquad (3.37)$$

Table 3.17: Structural Systems Behavior Factors (TEC, 2007)

BUILDING STRUCTURAL SYSTEM	**Systems of Nominal Ductility Level**	**Systems of High Ductility Level**
1. CAST-IN-SITE REINFORCED CONCRETE BUILDINGS		
1.1. Buildings in which seismic loads are fully resisted by frames.	4	8
1.2. Buildings in which seismic loads are fully resisted by coupled structural walls.	4	7
1.3. Buildings in which seismic loads are fully resisted by solid structural walls.	4	6
1.4. Buildings in which seismic loads are jointly resisted by frames and coupled structural and / or solid walls.	4	7
2. PREFABRICATED REINFORCED CONCRETE BUILDINGS		
2.1. Buildings in which seismic loads are fully resisted by frames with connections capable of cyclic moment transfer.	3	7
2.2. Single storey buildings in which seismic loads are fully resisted by columns with hinged upper connections.	-	3
2.3. Prefabricated buildings with hinged frame connections in which seismic loads are fully resisted by prefabricated or cast in situ solid structural walls and / or coupled structural walls.	-	5
2.4. Buildings in which seismic loads are jointly resisted by frames with connections capable of cyclic moment transfer and cast-in-situ solid and / or coupled structural walls.	3	6

3.2.4 Analysis Methods

Methods to be used for the seismic analysis of buildings and building-like structures are:

1. Equivalent Seismic Load Method
2. Mode – Superposition Method.
3. Time Domain Method.

1. EQUIVALENT SEISMIC LOAD METHOD

Total Equivalent Seismic Load (base shear), Vt, acting on the entire building in the earthquake direction considered shall be determined by Equation 3.38 (TEC, 2007).

$$Vt = \frac{WA(T1)}{Ra(T1)} \geq 0.10\, Ao\, I\, W \tag{3.38}$$

Where:

Vt	In the Equivalent Seismic Load Method, total equivalent seismic load acting on the building (base shear) in the earthquake direction considered
$T1$	The first natural vibration period of the building.
W	Total building weight.
A	Spectral Acceleration Coefficient
Ra	Seismic Load Reduction Factor
Ao	Effective Ground Acceleration Coefficient
I	Building Importance Factor

Total building weight (W), to be used in Equation 3.38 as the seismic weight shall be determined by Equation 3.46.

Total equivalent seismic load determined by Equation 3.38 is expressed by Equation 3.39 as the sum of equivalent seismic loads acting at storey levels:

$$Vt = \Delta FN + \sum_{i=1}^{N} Fi \tag{3.39}$$

Additional equivalent seismic load, ΔFN, acting at the N'th storey (top) of the building shall be determined by Equation 3.40 (TEC, 2007).

$$\Delta FN = 0.0075\, N\, Vt \tag{3.40}$$

Excluding ΔFN , remaining part of the total equivalent seismic load shall be distributed to stories of the building (including N'th storey) in accordance with Equation 3.41 (TEC, 2007).

$$Fi = (Vt - \Delta FN) \frac{wiHi}{\sum_{j=1}^{N} wjHj} \tag{3.41}$$

Where:

Fi Design seismic load acting at i'th storey in Equivalent Seismic Load Method.

wi Weight of i'th storey of building by considering Live Load Participation Factor.

Hi Height of i'th storey of building measured from the top foundation level.

2. MODE SUPERPOSITION METHOD

In this method, maximum internal forces and displacements are determined by the statistical combination of maximum contributions obtained from each of the sufficient number of natural vibration modes considered (TEC, 2007).

3. ANALYSIS METHODS IN TIME DOMAIN

Artificially generated, previously recorded or simulated ground motions may be used for the linear or nonlinear elastic analysis of buildings and building-like structures in the time domain (TEC, 2007).

Artificially Generated Seismic Ground Motions

In order to use artificially generated ground motions, at least three seismic ground motions shall be generated in accordance with the following properties:

- The duration of strong motion part of the acceleration shall neither be less than 5 times of the first natural vibration period of the building or less than 15 seconds (TEC, 2007).
- Average of spectral acceleration values of simulated seismic ground motion corresponding to period zero shall not be less than $A_o g$.
- Average of spectral acceleration values to be recalculated for each simulated acceleration record with 5 % damping ratio shall not be less than 90 % of elastic spectral accelerations $Sae(T)$ (TEC, 2007).

3.2.5 Load Combination

The design value Ed of the effects of action in the seismic design situation shall be determined in accordance with the following combination:

$$E_d = G + Q \pm E_x \pm 0.3E_y \tag{3.42}$$

$$E_d = G + Q \pm E_y \pm 0.3E_x \tag{3.43}$$

Where:

G Dead load

Q Live load

E_x, E_y Earthquake in direction to x and y.

Or in the case of unfavorable result

$$E_d = 0.9G + Q \pm E_n \pm 0.3E_y \tag{3.44}$$

$$E_d = 0.9G + Q \pm E_y \pm 0.3E_n \tag{3.45}$$

The seismic weight of the structure shall be determined by given equation:

$$W = \Sigma\, g_{i,N} + \Sigma\, n\, q_{i,N} \tag{3.46}$$

Where:

gi Is the total live load of the building at i,th storey.

qi Is the total dead load of the building at i,th storey.

n Is the live load participation factor.

N Is the number of stories in the structure.

Live load participation factor (n) is given in Table 3.18. In industrial buildings, n=1 shall be taken. Shall be considered 30% of snow load in the calculation of roof weight for seismic load (TEC, 2007).

Table 3.18: Live Load Participation Factors (TEC, 2007)

Purpose of Occupancy of Building	***n***
Depot, warehouse, etc.	0.8
School, dormitory, car park, cinema, sport facility, restaurant, shop, etc.	0.6
Residence, hotel, office, hospital, etc.	0.30

3.2.6 Irregular buildings

Design and construction of irregular buildings must be avoided since unfavorable seismic behavior. There are two types of irregularity buildings as follows:

3.2.6.1 Irregularities in plan

A1-Torsional irregularity

The case where Torsional Irregularity Factor (η_{bi}), which is defined for any of the two orthogonal earthquake directions as the ratio of the maximum storey drift at any storey to the average storey drift at the same storey in the same direction, is greater than 1.2 as shown in Figure 3.8 (TEC, 2007).

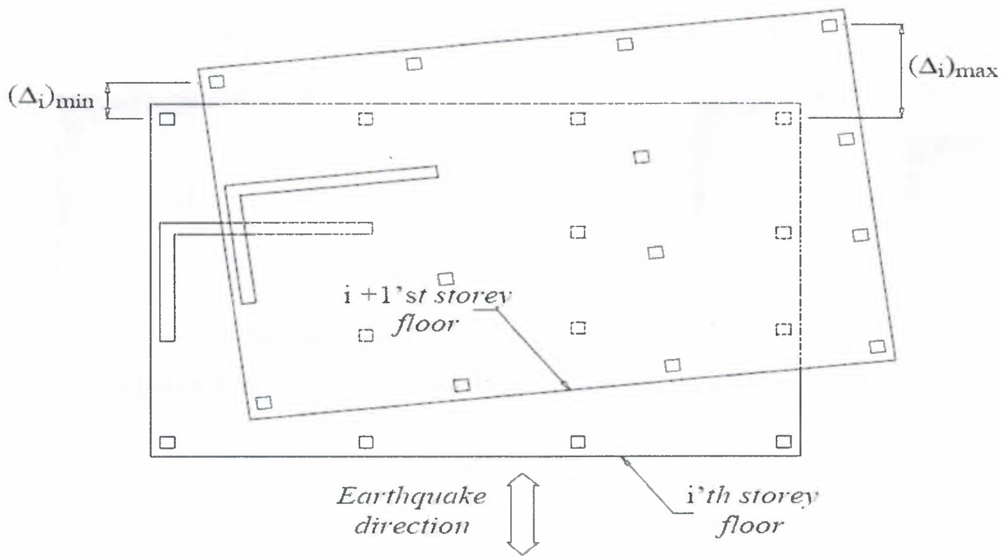


Figure 3.8: Type A1 Torsional irregularity (TEC, 2007)

In the case where floors behave as rigid diaphragms in their own planes:

$$\eta bi \;=\; [\,(\Delta i)max \,/\, (\Delta i)avg] \;>\; 1.2 \qquad (3.47)$$

$$(\Delta i)avg = \frac{1}{2}[(\Delta i)max + (\Delta i)min] \qquad (3.48)$$

Storey drifts shall be calculated by considering the effects of ± %5 additional eccentricities.

Where:

η_{bi}: Torsional irregularity factor of the building at i,th storey.

$(\Delta i)_{ave}$: Average storey drift of the building of i,th storey.

$(\Delta i)_{max}$: Maximum storey drift of the building of i,th storey.

$(\Delta i)_{min}$: Minimum storey drift of the building of i,th storey.

A2-Floor Discontinuities

The floor discontinuities are three cases:

- The case where the total area of the openings those of stairs also elevator shafts exceeds (1/3) of the gross floor area, as shown in Figures 3.9 (TEC, 2007).

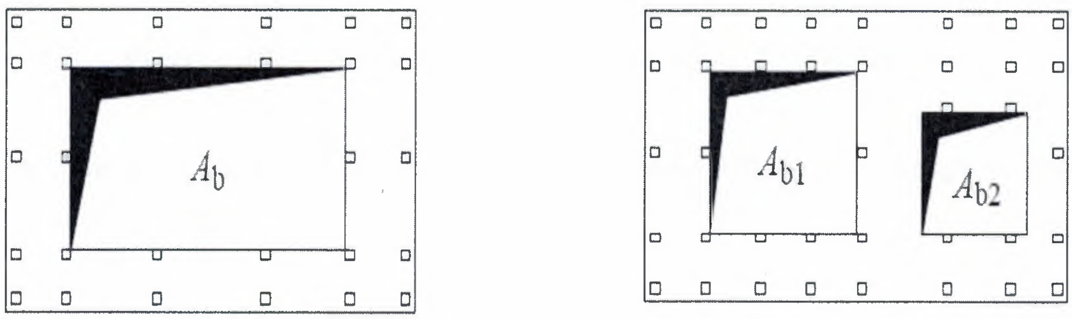


Figure 3.9: Type A2- Floor Discontinuity Cases I (TEC, 2007)

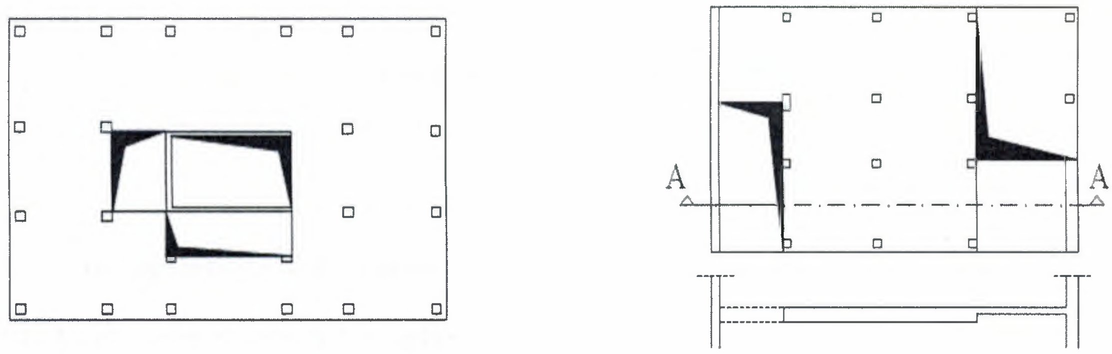
$$Ab = Ab1 + Ab2 \tag{3.49}$$

$$Ab/A > 1/3 \tag{3.50}$$

A_b: Total area of openings.

A: Gross floor area.

- The cases where local floor openings make it difficult the safe transference of seismic loads to vertical structural elements, as shown in Figure 3.10 (TEC, 2007).



SectionA-A

(a) Type A2 -Irregularity-II (b) Type A2 -Irregularity- II and III

Figure 3.10: Type A2- Floor Discontinuity Cases II (TEC, 2007)

- The cases where abrupt reductions in the in-plane stiffness and strength of floor (TEC, 2007).

A3-Projections in plan

The cases where projections beyond the re-entrant corners in both of the two main directions in plan exceed the total plan dimensions of the building in the respective directions by more than 20% as shown in Figure 3.11 (TEC, 2007).

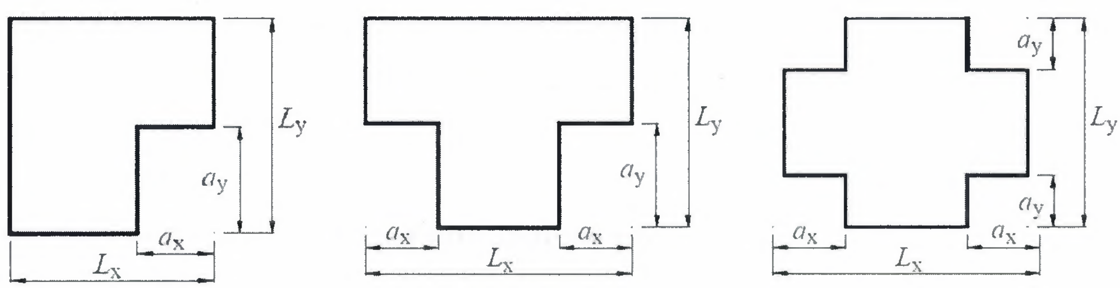


Figure 3.11: Type A3- Irregularity (TEC, 2007)

$$ax > 0.2Lx \tag{3.51}$$

$$ay > 0.2Ly \tag{3.52}$$

Where:

L_x, L_y Is the length of the building at x, y direction.

ax, ay Is the length of re-entrant corners in x, y direction.

3.2.6.2 Irregularities in Elevation

B1- Interstorey Strength Irregularity (weak storey)

In reinforced concrete buildings, the case where in each of the orthogonal earthquake directions, Strength Irregularity Factor (η_{ci}), which is defined the ratio of the effective shear area of any storey to the effective shear area of the storey directly above, is less than 0.8 (TEC, 2007).

$$\eta ci = \frac{(\sum Ae)i}{(\sum Ae)i + 1} < 0.8 \tag{3.53}$$

Definition of effective shear area in any storey:

$$\sum Ae = \sum Aw + \sum Ag + 0.15 \sum Ak \tag{3.54}$$

Where:

Ae Effective shear area.

A_w Effective of web area of column cross sections.

A_g Section areas of structural elements at any storey.

A_k Infill wall areas.

B2- Interstorey Stiffness Irregularity (Soft Storey)

The case where in each of the two orthogonal earthquake directions, Stiffness Irregularity Factor (η_{ki}) , which is defined as the ratio of the average storey drift at any storey to the average storey drift at the storey immediately above or below, is greater than 2.0 (TEC, 2007).

$$\eta ki = \frac{(\Delta i/hi)ort}{(\Delta i + 1/hi + 1)ort} > 2.0 \tag{3.55}$$

Or

$$\eta ki = \frac{(\Delta i/hi)ort}{(\Delta i - 1/hi - 1)ort} > 2.0 \tag{3.56}$$

Where:

η_{ki} Stiffness irregularity factor defined at i'th storey of the building.

Δ_i Storey drift of i,th storey of the building.

h_i Height of i,th storey of building [m].

Storey drifts shall be calculated by considering the effects of ± %5 additional eccentricities.

B3-Discontinuity of Vertical Structural Elements

The cases where vertical structural elements (columns or structural walls) are removed at some stories and supported by beams or gusseted columns underneath, or the structural walls of upper stories are supported by columns or beams underneath as shown in Figure 3.12 (TEC, 2007).

These conditions are related to buildings with irregularities of type B3 are given below:

(a) In all seismic zones, columns at any storey of the building shall in no case be permitted to rest on the cantilever beams or on top of or at the tip of gussets provided in the columns underneath (TEC, 2007).

(b) In the case where a column rests on a beam which is supported at both ends, all internal force components induced by the combined vertical loads and seismic loads in the earthquake direction considered shall be increased by 50% at all sections of the beam and at all sections of the other beams and columns adjoining to the beam.

(c) In the case where both ends of a structural wall rest on columns underneath, all internal force components induced at such columns by the combined vertical loads and seismic loads shall be increased by 50%. In reinforced concrete buildings with such irregularity (TEC, 2007).

(d) Structural walls shall in no case be permitted in their own plane to rest on the beam span at any storey of the building (TEC, 2007).

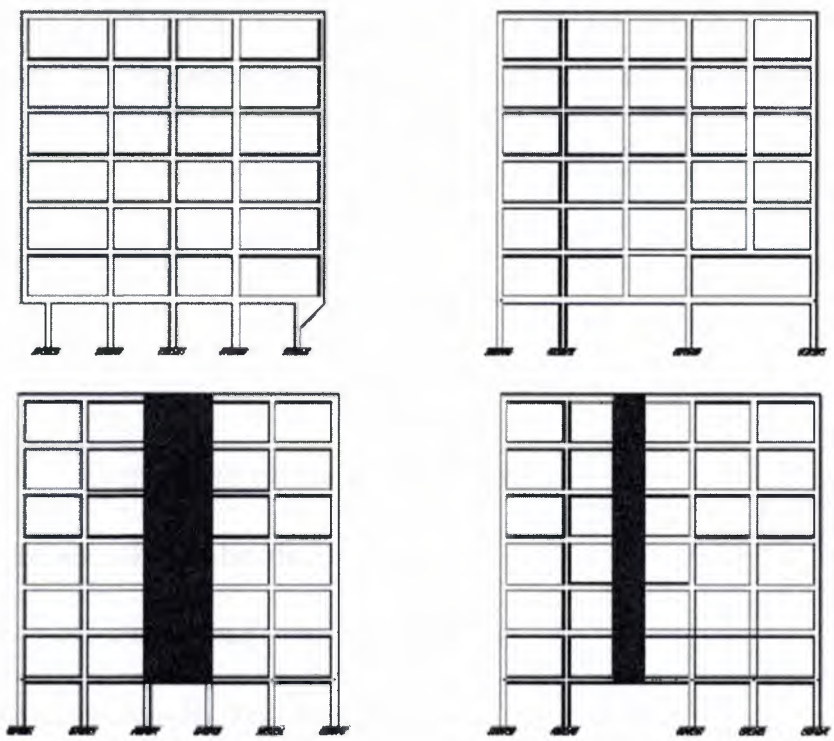


Figure 3.12: Type B3- Discontinuities of Vertical Structural Elements (TEC, 2007)

3.2.7 Design of Reinforced Concrete Structures

The reinforced concrete building elements according to their ductility level are divided into two types; nominal ductility level (NDL), high ductility building level (HDL) (TEC, 2007).

3.2.7.1 Material requirement

- The concrete strength C20 quality should be used or higher strength concrete (TEC, 2007).
- Concrete shall be produced by concrete quality control requirements indicated in TS-500, in all seismic zones.
- Concrete shall be placed by using vibrator (TEC, 2007).

- If reinforcing steel with strength greater than S420 shall not be used in reinforced concrete structure. But sometimes may be used in flat slabs in the slabs of joist floor, in peripheral external walls of basements, in the webs of structural walls of building during which complete seismic loads are resisted with such walls of full building height satisfying together of the condition given by equations below :

$$\sum Ag \Big/ \sum Ap \geq 0.002 \tag{3.57}$$

$$Vt \Big/ \sum Ag \leq 0.5\, fcdt \tag{3.58}$$

Where:

A_g Gross section area of column or wall end zone.

A_p Plane area of story building.

V_t The total seismic load acting on a building.

f_{cdt} The design tensile strength of concrete.

- The rupture strain of reinforcement less than 10% shall not be used (TEC, 2007).

3.2.7.2 Geometrical restrictions

1- Beam

- The beam web width shall be smallest amount 250 mm.
- The height of the beam shall not be less than 300 mm also three times slab thickness and more than 3.5 time the beam web width or ¼ of clear span length (TEC, 2007).
- Limitations specified above in relation to beam width and height are not applicable to reinforced concrete or prestressed /prefabricated beams with hinge connections to columns, to coupling beams of coupled structural walls, and to the

secondary beams which are connected to frame beams outside the beam-column joints (TEC, 2007)

- It is necessary the design axial force satisfies the condition

$$Nd \leq 0.1Acfck \tag{3.59}$$

Where:

N_d Is the axial force calculated under combined effect of seismic load and vertical loads multiplied with load coefficient

A_c Is the total cross sectional area of column

f_{ck} Is the characteristic compressive cylinder strength of concrete

2- Column

- Diameter of circular column shall be at least 300mm.
- In the rectangular section shorter dimension of columns shall not be less than 250mm, and section area shall not be less than 75000 mm^2 (TEC, 2007).
- All columns shall be satisfy equation below :

$$Ac \geq \frac{Ndm}{(0.50fck)} \tag{3.60}$$

Where:

A_c Is the total cross sectional area of column.

N_{dm} Is the maximum axial force caused by combine effects of gravitational and seismic loads

f_{ck} Is the characteristic compressive strength of a concrete

3- Ductile walls

1. Structural walls are the vertical elements of a structural system where the ratio of length to thickness into plan is higher than or equal to seven. By the exception of the special case given below. The thickness of wall shall not be less than 1/20 the maximum storey height and 200 mm.
   - In the buildings anywhere seismic loads are completely carry by structural walls along the full height of building. The thickness of wall shall not be less than 1/20 the maximum storey height and 150 mm. provided to both conditions given by Equation 3.61 and Equation 3.62 are satisfied (TEC, 2007).

$$Vt/\sum Ag \leq 0.5\, fcdt \qquad (3.61)$$

$$\sum Ag/\sum Ap \geq 0.002 \qquad (3.62)$$

These equations shall be apply on the ground floor level in buildings by stiff peripheral walls in basement stories, where it shall be applied on foundation top level for other buildings (TEC, 2007).

   - On the walls positioned during lateral direction by the elements to the length is equal to at least to (1/5) of storey length and contain storey length larger than 6m, the thickness of wall in the ground may be equal to at least (1/20) of horizontal length among the points wherever it situated in lateral direction. On the other hand this thickness should be equal or more than 300mm (TEC, 2007).

3.2.7.3. Reinforcement Conditions

1- Beam Reinforcement:

- **Longitudinal reinforcement requirement:**
- At beam support the minimum ratio of the top tension reinforcement (ρmin) shall be ≥ 0.8 fctd / fyd (TEC, 2007).

Where:

Pmin	Minimum tension reinforcement ratio.
Fctd	Design tensile strength of concrete.
Fyd	Design value of yield strength of steel.

- The maximum ratio of tension reinforcement along beam span and at support (ρmax) shall be ≤ 0.02 (TEC, 2007).
- Dia. of the longitudinal rebars should be equal or greater than 12mm.
- The no. of rebars at the top and bottom of the beam should be equal or greater than 2 rebars.
- For first and second seismic zone, bottom reinforcement at beam support shall be equal or greater than 50% of top reinforcement. But for third & fourth seismic zones decreased to 30% (TEC, 2007).
- ¼ of the maximum top support reinforcement shall be extended along the full beam length (TEC, 2007).
- If the beam height greater than ¼ clear span shall be web reinforcement providing along the beam height on both sides of the web (TEC, 2007).
- The diameter of web reinforcement shall not be smaller than 12 mm and spacing of web reinforcement shall be equal or smaller than 300 mm.
- If the beams at support are discontinued to the other side of column, top and bottom of the beam shall be extended up to face of other side of the confined core of column and then shall be bent 90° from inside the hoops, horizontal

part of the 90° bent shall be equal or greater than 0.4ℓb and vertical part shall be equal or greater than 12∅. As shown in Figure 3.13

Where:

ℓ_b Development length of tensile reinforcement as given in TS500.

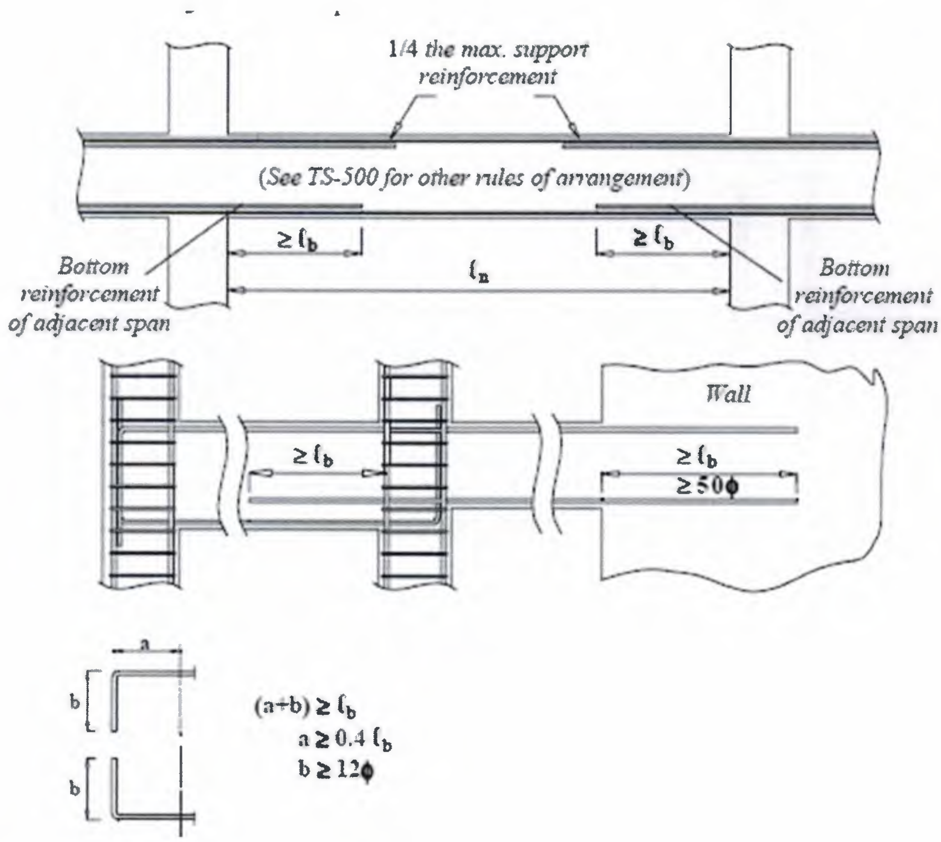


Figure 3.13: Longitudinal Reinforcement Requirements for Beams (TEC, 2007)

- Spacing of such hoops shall be equal or smaller than (0.25 the beam depth, 100mm) (TEC, 2007).
- Mechanical connections, welded lap splice shall be equal or greater than 600 mm (TEC, 2007).

- **Transverse reinforcement requirement:**
- Beam Confinement zone length equal twice the beam depth.
- Distance of first hoop from column shall be equal or smaller than 50 mm (TEC, 2007).
- Spacing of the hoops shall not be more than (0.25 the beam depth, 8 times the minimum diameter of longitudinal reinforcement, 150mm), as shown in Figure 3.14.

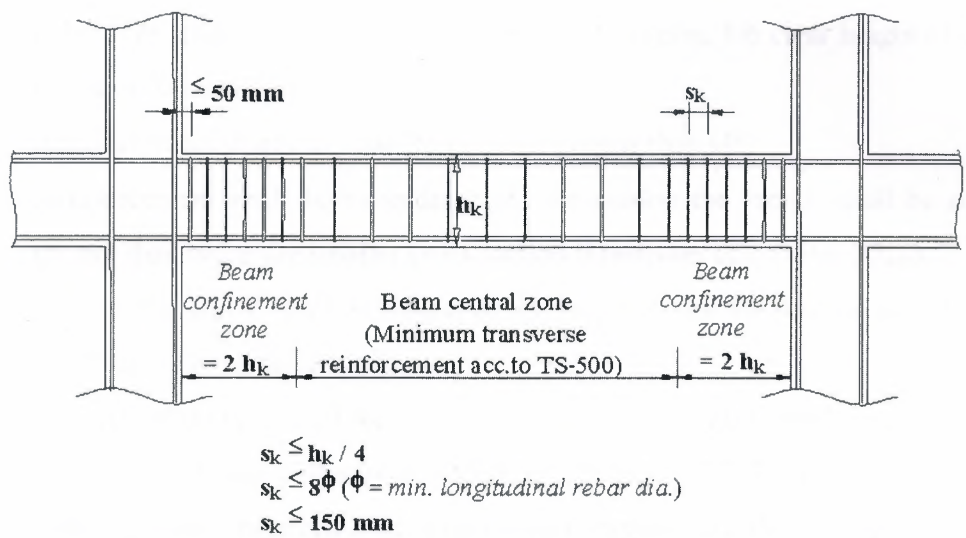


Figure 3.14: Transverse reinforcement requirements for beams (TEC, 2007)

2- Column Reinforcement Conditions

- **Longitudinal reinforcement:**
- Longitudinal column reinforcement shall be equal or greater than 0.01 and shall it be equal or smaller than 0.04 of gross section area (TEC, 2007).
- Minimum number of rebars for rectangular columns shall be 4Ø16 or 6Ø14, but for circular columns shall be 6Ø14 (TEC, 2007).
- At lap spliced sections the ratio of longitudinal reinforcement shall not exceed 0.06 (TEC, 2007).

- If lap splices are made at the bottom end of column longitudinal reinforcement the following conditions shall be met:
  - Minimum lap splice length shall be 1.25 times ℓb, if 50% of the longitudinal reinforcement or less is spliced at the bottom end of column.
  - Length of lap splice shall be at least 1.5.ℓb if more than 50% of the longitudinal reinforcement is spliced (TEC, 2007).

- **Transverse Reinforcement:**
- Column confined zone length shall be equal or greater than (diameter of circular column or smaller cross section dimension of column, 1/6 clear height of column (*hc*) and 500mm) (TEC, 2007).
- Reinforcement diameter shall be equal or greater than Ø8.
- Reinforcement shall be exceeding into foundation the length shall be equal or greater than twice the smaller cross section dimension of column (D_{min}).
- The reinforcement shall be continued the length inside the foundations shall be ≥ {25Dmax, 300} (TEC, 2007).
- Spacing of hoops shall be equal or smaller than (1/3 smaller cross section dimension (Dmin), 100mm) or shall it be ≥ 50mm (TEC, 2007).
- Lateral distance between legs of hoops and crossties (a), shall be equal or smaller than 25.Dhoop (TEC, 2007).

Where:

D_{hoop} Is the hoop diameter.

- Pitch of spirals shall be equal or smaller than ≤ [1/5 Dcore, 80mm] (TEC, 2007).

Where:

D_{core} Is the core diameter

- If Nd > 0.2 Ac fck In columns with hoops the minimum total area of transverse reinforcement to be used in confinement zones shall be calculated as seen in Equation 3.63. In this calculation, core diameter of column, bk, shall be considered separately for each direction, as seen in Figure 3.15 (TEC, 2007).

$$A_{sh} \geq 0.3\ s\ b_k\ [(A_c / A_{ck})-1]\ (f_{ck} / f_{ywk}) \tag{3.63a}$$

$$A_{sh} \geq 0.075\ s\ b_k\ (f_{ck} / f_{ywk}) \tag{3.63b}$$

But in columns with spirals the minimum volumetric ratio of transverse reinforcement to be used in confinement zones shall be calculated as seen in Equation 3.64 (TEC, 2007).

$$\rho_s \geq 0.45\ [(A_c/A_{ck})-1]\ (f_{ck} / f_{ywk}) \tag{3.64a}$$

$$\rho_s \geq 0.12\ (f_{ck} / f_{ywk}) \tag{3.64b}$$

- If $Nd \leq 0.2\ Ac\ fck$, at least 2/3 the transverse reinforcement given by Equation 3.63 and Equation 3.64 shall be used in column confinement zones.as a minimum transverse reinforcement (TEC, 2007).

Where:

Nd	The axial force calculated under combine effect of seismic loads and vertical loads multiplied with loads coefficient.
A_c	The gross area of column or wall zone.
f_{ck}	The characteristic compressive cylinder strength of concrete.
A_{ck}	Concrete core area within outer edges of confinement reinforcement.
f_{ywk}	The characteristic yield strength of transverse reinforcement.

A_{sh} Total area steel of hoop.

ρ_s Volumetric ratio of spiral reinforcement of column

b_k Core diameter of column.

- Diameter of transverse reinforcement in central column shall not be less than Ø8.
- Spacing of hoops at Central columns reinforcement shall not be more than 1/2 smaller cross section dimension of column (D_{min}) and 200mm (TEC, 2007).

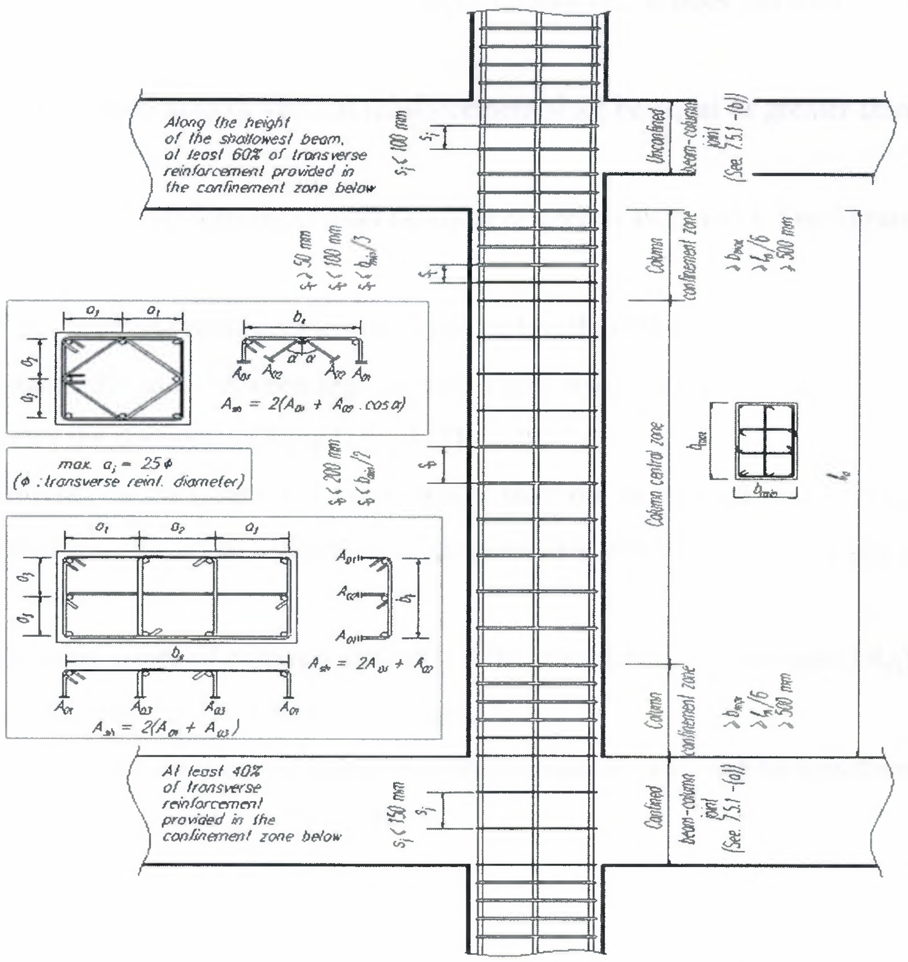


Figure 3.15: Column Confinement Zones and Detailing Requirements (TEC, 2007)

3- Ductile Shear-Wall Reinforcement Conditions

- **Wall end zones reinforcement:**
- If $H_w/l_w \leq 2.0$the section of web shall be considered as the full section of the wall (TEC, 2007).

Where:

H_w Total wall height measured from top foundation level or ground floor level.

l_w Length of wall or segment of coupled wall in plan

- The critical wall height shall not be less than l_w, and H_w, it does not exceed $2l$ (TEC, 2007).
- The ratio of the total area of vertical reinforcement shall be equal or greater than 0.001.
- Amount of vertical reinforcement shall be equal or greater than 4Ø14. See Figure 3.16.
- Diameter of transverse reinforcement shall not be less than 8mm.
- The horizontal distance between legs of hoops and cross sties (a) equal or less than 25 times the diameter of hoop (D_{hoop}) (TEC, 2007).
- Vertical spacing of the hoops and / or crossties shall not be more than 0.5 times the wall thickness (b_{web}) or 100mm or it does not less than 50mm. See Figure 3.16.
- For confinement zones of columns at least 2/3 the transverse reinforcement (A_{sh}) shall be provided along the critical wall height.
- Such reinforcement shall be extended into the foundation shall not be less than twice the wall thickness (b_{web}) (TEC, 2007).

- **Web reinforcement:**
- Total cross section area of Vertical and horizontal reinforcement ratio should not be less than 0.0025 of the gross section area of the wall web.
- Spacing of web reinforcement shall not be more than 250mm (TEC, 2007).

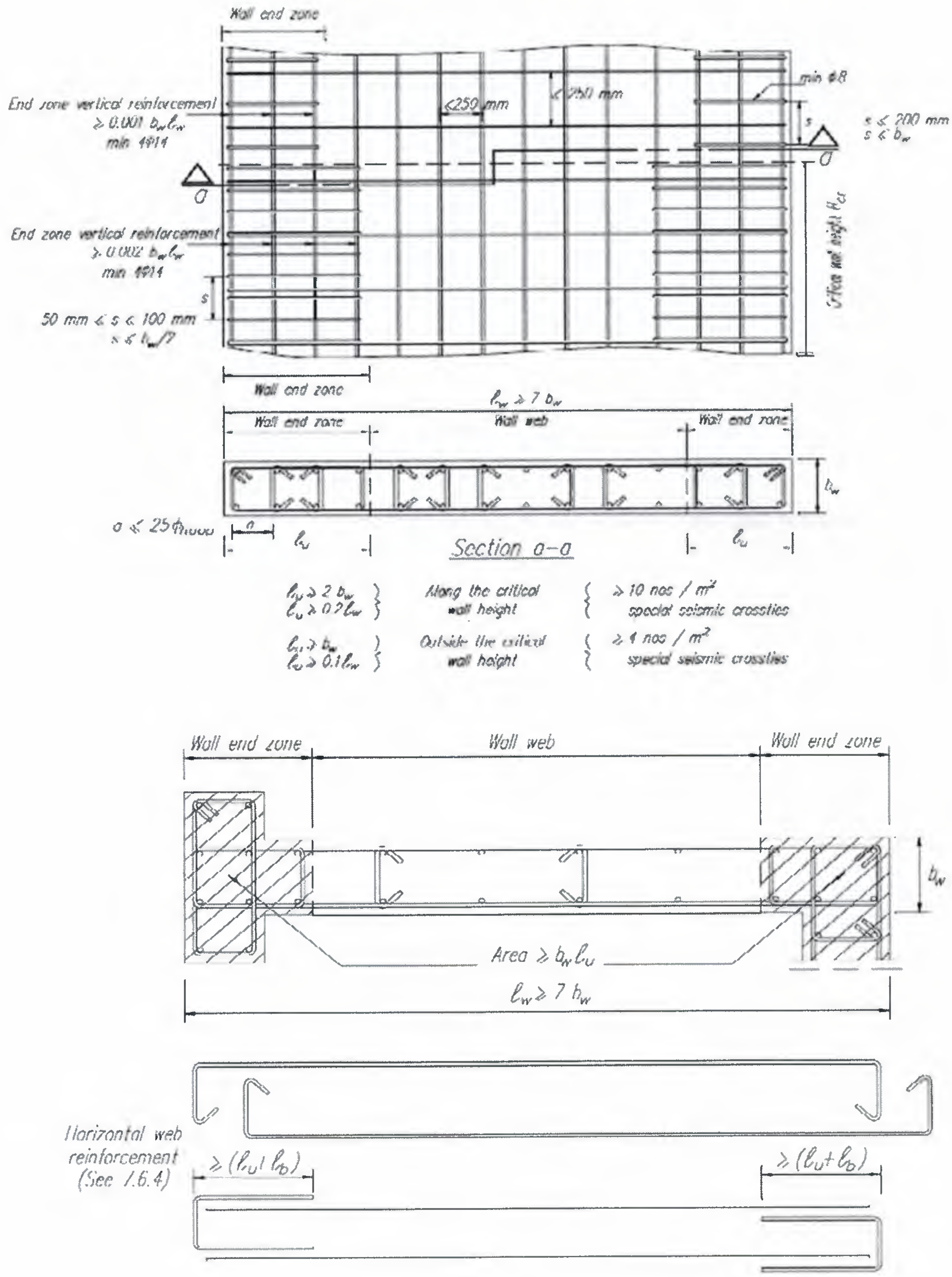


Figure 3.16: Ductile Wall Reinforcement Requirements (TEC, 2007)

3.2.8 Requirement for Foundation Tie Beams

Tie beams shall be providing in reinforced concrete to connect separate footings or pile caps in both directions or to connect continuous foundations at column or structural wall axes (TEC, 2007).

Tie beams can be created at any level between the bottom of the building foundation and the bottom of the column (TEC, 2007).

The spacing of hoop in tie beams shall be equal or less than 200 mm and their diameter shall be equal or greater than 8 mm.

Tie beams may be replaced by reinforced concrete slabs. In such a case slab thickness shall not be less than 150 mm (TEC, 2007).

The minimum requirements of tie beams depending on the soil groups and seismic zone of the building. As shown in Appendix 5

CHAPTER 4

COMPARISON BETWEEN OF TURKISH STANDARDS AND EUROCODE8

4.1 Comparison of Ground Condition

The Eurocode8 defines five types of soil classes and two special ground types with a soil factor (S) for each type, while Turkish Code consider four types of soil without soil factor.

Both codes consider the shear wave velocity to find soil types, but the Turkish seismic code gives more information about soil profile depending of topmost layer thickness of soil (h1).

Table 4.1 shows the divergence of ground types with shear wave velocity according to these codes.

Table 4.2: Ground types defined in EC8, TEC2007

Eurocode8		Turkish code	
Soil type	**Shear wave velocity**	**Soil type**	**Shear wave velocity**
A	V_s>800m/s	A	Volcanic rocks Vs >1000 m/s; Sand, gravel Vs >700 m/s; Hard clay, silty clay Vs >700 m/s
B	360m/s < Vs < 800m/s	B	Volcanic rocks Vs ≈ 700 – 1000; Dense and, gravel Vs ≈ 400 – 700 Stiff clay, silty clay Vs ≈ 300 – 700
C	180m/s < *Vs* < 360m/s	C	Volcanic rocks *Vs ≈ 400 – 700;* sand and gravel *Vs ≈200 – 400*; Stiff clay, silty clay *Vs ≈200 – 300*
D	*Vs*< 180m/s	D	Soft, deep alluvial layers with high water table *Vs < 200;* Loose sand *Vs < 200;* Soft clay, silty clay *Vs < 200*
E	*Vs*> 800 m/s		

4.2 Comparison of Elastic Response Spectrum

Eurocode8 defines two types of response spectra depending on the magnitude of earthquakes contributing most to the seismic hazard.

Type 1 of response spectra for medium and large magnitude earthquakes and type 2 of response spectra for low magnitude earthquakes, if a surface-wave magnitude, Ms More than 5.5, it is recommended that the Type 1 response spectrum is adopted, if not, Type 2 is recommended.

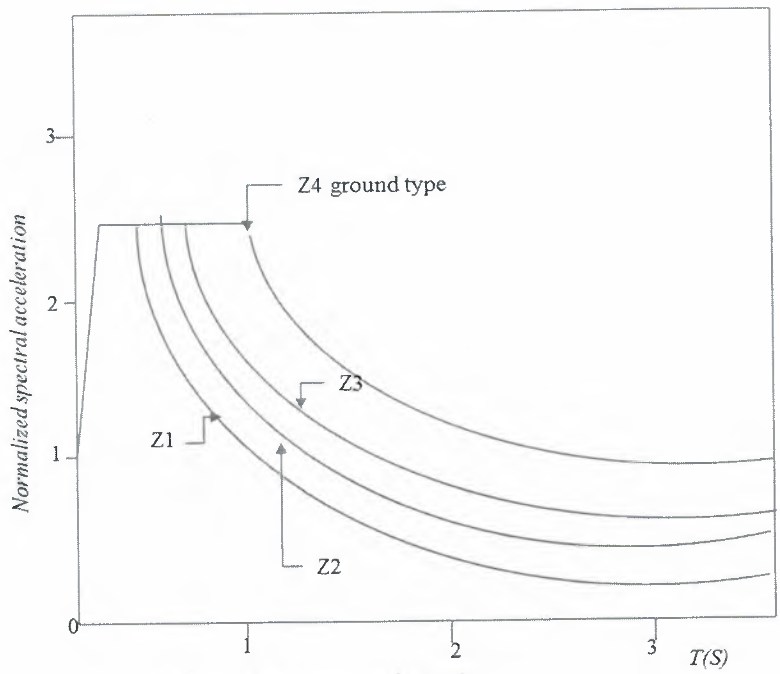
Turkish Code defines one type of response spectra, the spectrum coefficient dependent on the local site conditions and the building natural period, T, the period of elastic design spectra of these codes depend on soil type.

The ordinates of elastic design spectra (Se) and inelastic design spectra (Sd) for the reference return period defined by the earthquake codes can be determined using the expressions given in Table 4.2. In this table, for the horizontal design spectrum β shows the lower bound factor, the value of β is 0.2 and γ I shows importance factor.

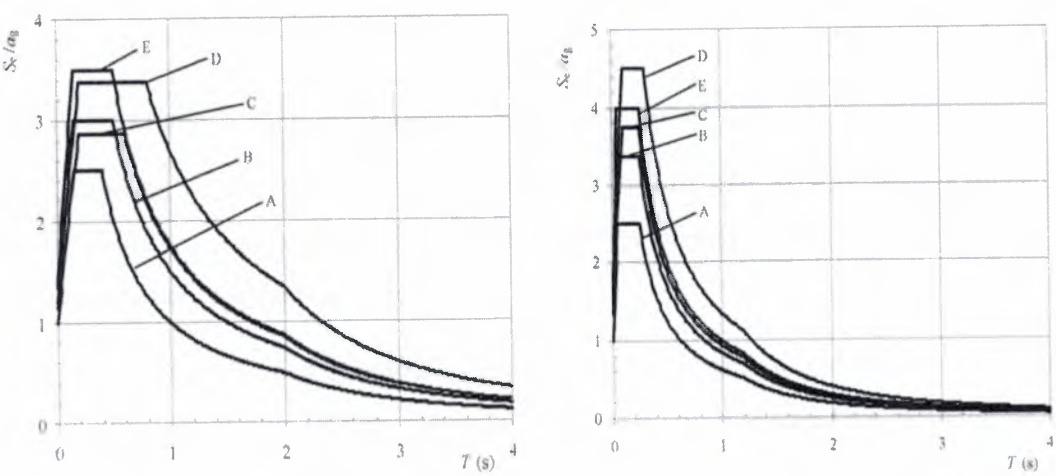
Table 4.2: Ordinates of Elastic and design spectra for EC8, TEC 2007

	$T \le T_B$	$T_B \le T \le T_C$	$T \ge T_C$
TEC	$S_e = a_{gR}[1 + 1.5\frac{T}{T_B}]$ $S_d = \frac{a_g}{R_a}[1 + 1.5\frac{T}{T_B}]$	$S_e = 2.5 \cdot a_{gR}$ $S_d = \frac{2.5 \cdot a_g}{R_a}$	$S_e = 2.5 \cdot a_{gR}[\frac{T_C}{T}]^{0.8}$ $S_d = \frac{2.5 a_g}{R_a}[\frac{T_C}{T}]^{0.8}$
EC8	$S_e = a_g \cdot S[1 + \frac{T}{T_B}(\eta 2.5 - 1)]$ $S_d = a_g S[\frac{2}{3} + \frac{T}{T_B}(\frac{2.5}{q} - \frac{2}{3})]$	$S_e = 2.5 \cdot a_g \cdot S \cdot \eta$ $S_d = \frac{2.5}{q} \cdot a_g \cdot S$	$T_C \le T \le T_D \rightarrow S_e = 2.5 a_g \cdot S \cdot \eta \cdot [\frac{T_C}{T}]$ $T_C \le T \le T_D \rightarrow S_d \begin{cases} = \frac{2.5}{q} a_g \cdot S \cdot [\frac{T_C}{T}] \\ \ge \beta \cdot a_g \end{cases}$ $T_D \le T \le 4s \rightarrow S_e = 2.5 a_g \cdot S \cdot \eta \cdot [\frac{T_C T_D}{T^2}]$ $T \ge T_D \rightarrow S_d = \frac{2.5}{q} a_g \cdot S \cdot [\frac{T_C T_D}{T^2}] \ge \beta \cdot a_g$

Turkish code for all ground types gives the same peak value, but Eurocode8 not like Turkish code gives the maximum peak values for ground types other than ground type A. The shapes of the elastic response spectrum of the Type 2 are more peaking for short period structure excepting for ground Type A. as seen from Figure 4.1.



(a) Elastic design spectra for TEC



(b) Elastic design spectra of type 1 and type 2 for EC8

Figure 4.1: Elastic Design Spectra for Ground Types according to TEC 2007 and EC8

4.3 Comparison Criteria for Structural Regularity

The design building of both EC8 and TEC are based on the two criteria which are regularity in plan and in elevation.

But in EC8 the criteria of regularity in plan should satisfy, with respect to two orthogonal axes the building structure should be symmetrical in plan. The slenderness λ= Lmax / Lmin of the building shall not be higher than 4, and the structural eccentricity (eo) shall not be more than 30% of the torsional radius (r).

While in TEC the irregularities in plan is classified into three type which are A1-Torsional Irregularity, A2-Floor Discontinuities, and A3- Projections in Plan.

Also in EC8 Criteria for Regularity in Elevation shall satisfy, From their foundations to the top of the building All lateral load resisting systems, such as frames, shall run without interruption, Both the lateral stiffness and the mass of the individual stories shall remain constant or reduce gradually, In framed buildings the ratio of the actual stories resistance to the resistance should not vary disproportionately between adjacent stories.

While irregularities in Elevations in TEC is divided into three types which are, B1- Inter storey Strength Irregularity (Weak Storey), B2- Interstorey Stiffness Irregularity (Soft Storey), and B3-Discontinuity of Vertical Structural Elements.

4.4 Comparison Design of Reinforced Concrete Structure

The concept of ductility estimates the capacity of the structural system and its components to deform prior to collapse, without a substantial loss of strength, but with an important energy amount dissipated.

The design of reinforced concrete structures to seismic action should provide sufficient information about the energy dissipation capacity of the structure without reducing substantially the strength in favour of horizontal and vertical loads. An adequate strength should be provided from the seismic combination for the structural elements and the non-linear deformations in critical areas should permit to obtain the total ductility considered in the design.

The structural design taking into account the ductility concept leads to an increase of the strength and the quantity of dissipated energy. It also ensures a global plastic mechanism of the structure before collapse.

The design of structures located in seismic areas uses the dissipative behavior principle. According to this substantially reduced seismic loads are used instead of those corresponding to the elastic response of the structures through the behavior factor. The reduction of the seismic design forces is realized based on the ductility, redundancy and the strength excess of the structure. Among these, the most significant reduction of the design forces is based on the ductility of the structure that depends on the chosen structural type and the material characteristics. Thus, the rigid structures are characterized by lower behavior factors.

The rigidity is related to serviceability limit state, for which the structural displacements must remain in some limits. This assures that no damage occurs in non-structural elements.

In the conventional practice for non-seismic loads, structures are designed only for two demands, strength and rigidity. While the new approach in the structural design under seismic loads, known as capacity design conception, requires the verifications of three demands, rigidity, strength and ductility, The ductility check is related to the control of weather the structure is able to dissipate the given quantity of seismic energy considered in structural analysis or not.

In Eurocode8, the reinforced concrete building elements are divided into three types according to their ductility level; low ductility (DCL), medium ductility (DCM) and high ductility level (DCH).

But in Turkish code, the reinforced concrete building elements are divided into two types according to their ductility level; high ductility building level (HDL), nominal ductility level (NDL).

Frame type take it into consideration by the EC8 and by TEC 2007, the EC8 accept the ductile and rigid ductile frame but TEC 2007 only considers rigid ductility in the frame.

4.4.1 Comparison material requirement

Material requirement are explained in the Table 4.3.

Table 4.3: Used Material Comparison (TEC 2007, EC8)

	TEC2007	EC8	
Ductility	**Medium and High**	**Medium**	**High**
Characteristic strength of Concrete	Fck ≥ C20	Fck ≥ C16/20	Fck ≥ C20/25
Type of the reinforcement	BÇI (220) BÇIIIa (420)	B or C type reinforcement	C type reinforcement
Characteristic strength of reinforcement	Fyk ≤ 420MPa	Fyk ≤ 600 (EC2)	Fyk ≤ 600 (EC2)
Minimum strain of reinforcement at maximum stress	10%	5% (EC2)	7.5% (EC2)
Concrete design code	TS-500	EC2	

4.4.2 Comparison of Geometrical restrictions

Comparison Geometrical restrictions for (beam, column and ductile walls) are explained in the Table 4.4.

Table 4.4: Comparison Geometrical restriction according to EC8 and TEC 2007

	EC8	TEC 2007
Beam	• The width of the beam should be greater than or equal 200 mm. • The beam web width to height ratio of primary seismic beams should satisfy the expression below $\left(\frac{lot}{b}\right) \leq \left(\frac{70}{(h/b)^{1/3}}\right)$ And $h\ /\ b \leq 3.5$	• The beam web width equal or greater than 250 mm. • The height of the beam shall not be less than 300 mm also three times slab thickness and more than 3.5 time web width or ¼ of clear span length.

	• The space between the centroidal axes of two beams is limited to less than *bc/4*.	• It is necessary the design axial force satisfies the condition $Nd \leq 0.1Acfck$
Column	• The cross sectional dimension of column shall not be less than 250 mm. • The cross sectional dimension of column must be at least 1/10 distance connecting the point of contra flexure and the ending of column, if the inter storey drift sensitivity coefficient ϴ is bigger than 0.1.	• In the rectangular section shorter dimension of columns shall not be less than 250mm, and section area shall not be less than 75000 mm2. Diameter of circular column shall be at least 300mm. • All columns shall be satisfy equation below : $Ac \geq \frac{Ndm}{(0.50fck)}$
Ductile walls	• The thickness of the web of wall (*bwo*) should be greater than of clear storey height (*hs*) divided by 20, or by a minimum of 150mm. • the width of the boundary element (b_w) should be higher than or equal (0.20)m.	• The thickness of wall shall not be less than 1 / 20 the storey height and 200 mm. • The thickness of wall shall not be less than 150 mm if seismic loads are completely carry by structural walls.

4.4.3 Comparison Reinforcement requirement

4.4.3.1 Comparison Beam Reinforcement requirement

Comparison Beam Reinforcement requirement are explained in Table 4.5

Table 4.5: Beam Reinforcement according to EC8 and TEC2007

EC8	TEC 2007
Longitudinal reinforcement:	**Longitudinal reinforcement:**
• Minimum ratio of tension side (ρ_{min}) shall be $\geq 0.5 f_{ctm} / f_{yk}$. • The ratio of tension reinforcement should not be exceed $\rho_{max} = \rho' + 0.0018 fcd / (\mu\varpi\ \varepsilon sy,\ d\ fyd)$	• At beam support the minimum ratio of the top tension reinforcement (ρ_{min}) shall be $\geq 0.8\, f_{ctd} / f_{yd}$. • The maximum ratio of tension reinforcement (ρ_{max}) shall be ≤ 0.02.
• Dia. of the longitudinal rebars should be $\geq 14mm$.	• Dia. of the longitudinal rebars should be $\geq 12mm$.
• The no. of rebars at the bottom and top of the beam should be ≥ 2 rebars.	• The no. of rebars at the bottom and top of the beam should be ≥ 2 rebars.
• Minimum bottom reinforcement at beam support shall be equal to 50% of top reinforcement.	• For first and second seismic zone, bottom reinforcement at beam support shall be $\geq$ 50% of top reinforcement. But for third & fourth seismic zones decreased to 30%.
• ¼ of the maximum top supports reinforcement shall be along the full beam length.	• ¼ of the maximum top support reinforcement shall be extended along the full beam length.
Transverse Reinforcement:	**Transverse Reinforcement:**

• For DCH the critical region length equal to 1.5 the height of the beam. But for DCM equal to the height of the beam.	• Beam Confinement zone length equal twice the beam depth.
• Distance of first hoop from column shall be < 50 mm.	• Distance of first hoop from column shall be ≤ 50 mm.
• For DCH spacing of the hoops shall be≤ min {6dbL, hw/4, 24bw, 175mm}, but for DCM shall be ≤ {8dbL, hw/4, 24bw, 225mm}.	• Spacing of the hoops shall be ≤ $(0.25\ b_d,\ 8D_{bar,min}, 150mm$

4.4.3.2 Comparison Column Reinforcement requirement

Comparison Column Reinforcement requirement are explained in Table 4.6.

Table 4.6: Column Reinforcement according to EC8 and TEC 2007

	EC8	TEC 2007
column	**Longitudinal reinforcement:**	**Longitudinal reinforcement:**
	• Longitudinal column reinforcement shall be ≥ *0.01* and shall it be ≤ *0.04* of gross section area. If the same cross section (symmetrical reinforcement) shall be $\rho=\rho'$ • The corner columns (rectangular) at least one intermediate bar shall be provided between corner bars along each column side.	• Longitudinal column reinforcement shall be ≥ *0.01* and shall it be ≤ *0.04* of gross section area. • Minimum number of rebars for rectangular columns shall be 4∅16 or 6∅14, but for circular columns shall be 6∅14.
	Transverse Reinforcement:	**Transverse Reinforcement:**
	• For DCH critical region length shall be max{$1.5h_c$, $1.5b_c$, 0.6m,$l_c/5$}, but for DCM shall be max{h_c, b_c,	• Column confined zone length shall be ≥ {D_{min}, 1/6 h_c, 500mm} • Reinforcement diameter shall be

0.45m, $l_c/6$}. • For DCH diameter of the hoops shall be ≥ {6mm, $0.4(f_{ydL}/f_{ywd})^{1/2} d_{bL,max}$}, but for DCM shall be ≥ {6mm, $d_{bLmax}/4$}. • For DCH spacing of hoops shall be min{$6d_{bL}$, $b_o/3$, 125mm}, but for DCM shall be min{$8d_{bL}$, $b_o/2$, 175mm} • For DCH spacing of hoops at central columns reinforcement shall be min{$20d_{bL}$, h_c, b_c, 400mm}, but for DCM shall be min{$12d_{bL}$, $0.6h_c$, $0.6b_c$, 240mm}.	≥ Ø 8 • Spacing of hoops shall be ≤ [1/3D_{min}, 100mm] or shall it be ≥ 50mm. • Spacing of hoops at Central columns reinforcement shall be ≤ 1/2 D_{min} and 200mm.

4.4.3.3 Comparison Ductile walls Reinforcement requirement

Comparison Ductile walls Reinforcement requirement are explained in Table 4.7.

Table 4.7: Ductile walls Reinforcement according to EC8 and TEC

	EC8	TEC
Ductile walls	**Wall end zones reinforcement:**	**Wall end zones reinforcement:**
	• The vertical reinforcement ratio shall not be less than 0.5% and it shall not be more than 4%.	• The total area of vertical reinforcement ratio shall be ≥ 0.001.
	• Diameter of hoops shall be ≥6mm, $0.4(f_{yd} / f_{ywd})^{1/2}$ d_{bL}.	• Diameter of transverse reinforcement shall be ≥ 8mm.
	• For DCH vertical spacing of hoops and cross ties should be ≤ $6d_{bL}$, $b_o/3$, 125mm, but for DCM ≤ $8d_{bL}$, $b_o/2$, 175mm.	• Vertical spacing of the hoops and or crossties shall be ≤ $0.5b_{web}$ or 100mm or ≥ 50mm.
	Web reinforcement:	**Web reinforcement:**
	• Minimum vertical reinforcement ratio should be 0.005 if compressive strain exceeds 0.002, but minimum horizontal reinforcement ratio should be 0.002.	• Total cross section area of Vertical and horizontal reinforcement ratio should not be smaller than 0.0025 of the gross section area of the web.
	• Spacing of web reinforcement should not be greater than 250mm or 25 times the diameter of bar.	• Spacing of web reinforcement shall be ≤ 250mm.

Table 4.8: General comparison between EC8 and TEC 2007

	EC8	TEC 2007
Ground condition	There are five types of soil class and two special ground types with a soil factor.	There are four types of soil class (A, B, C, D) without a soil factor.
Elastic response spectrum	Defines two types of response spectra type1 for medium and large magnitude earthquake type2 for low magnitude earthquake	Defines one type of response spectra
	The shape of elastic response spectrum different peak value depend on ground types	For all ground types gives the same peak value
General rules of earthquake resistant building	➤ **Criteria for regularity in plan**	➤ **Irregularities in plan**
	Should be symmetrically in plan	Torsoinal irregularity
	Slenderness ratio (λ)=Lmax/Lmin≤4	Floor Discontinuity
	Structural eccentricity≤ 30% torsional radius	Projections in Plan
	➤ **Regularity in elevation**	➤ **Irregularities in elevation**
	All horizontal load resisting systems, such as structural walls, frames, or cores, shall run without the interruption from their footings to the top of the structure.	Interstorey strength irregularity Interstorey stiffness irregularity Discontinuity of vertical structural elements
R.C structure	According to their ductility level divided for DCL, DCM and DCH.	According to their ductility level divided for NDL and HDL
Material requirement	Characteristic strength of Concrete for DCM shall be ≥ C16/20 but for DCH≥ C20/25.	Characteristic strength of Concrete for medium and high ductility shall be ≥ C20
	Characteristic strength of reinforcement shall be ≤ 600	Characteristic strength of reinforcement shall be ≤ 420MPa
Beam	Width of beam ≥ 200mm	Width of beam ≥ 250mm

geometrical restrictions	The beam web width to height ratio should satisfy the expression below $\left(\frac{lot}{b}\right) \leq \left(\frac{70}{(h/b)^{1/3}}\right)$ And $h\ /\ b \leq 3.5$	The height of the beam shall not be less than 300 mm also three times slab thickness.
Column geometrical restrictions	The cross sectional dimension of column shall not be less than 250 mm.	In the rectangular section shorter dimension of columns shall not be less than 250mm.
Ductile wall geometrical restrictions	The thickness of the web of wall (*bwo*) should be > of clear storey height divided by 20, or ≥ 150mm.	The thickness of wall shall not be less than 1 / 20 the storey height and 200 mm.
Beam reinforcement	Dia. of longitudinal rebars ≥ *14mm.*	Dia. of the rebars ≥ *12mm.*
	For DCH the critical region length equal to 1.5 the height of the beam.	Beam Confinement zone length equal twice the beam depth.
	For DCH spacing of the hoops shall ≤ min {6dbL, hw/4, 24bw, 175mm}.	Spacing of the hoops shall be ≤ $(0.25\ b_d,\ 8D_{bar,min},\ 150mm)$
Column reinforcement	For DCH critical region length shall be max $\{1.5h_c, 1.5b_c, 0.6m, l_c/5\}$.	Column confined zone length shall be ≥ $\{D_{min}, 1/6\ h_c, 500mm\}$
	For DCH dia. of the hoops shall be ≥ $\{6mm, 0.4(f_{ydL}/f_{ywd})^{1/2}\ d_{bL,max}\}$.	Reinforcement diameter shall be ≥ Ø 8
	For DCH spacing of hoops shall be $\min\{6d_{bL}, b_o/3, 125mm\}$.	Spacing of hoops shall be ≤ $[1/3D_{min}, 100mm]$.
Ductile walls reinforcement	Diameter of hoops shall be ≥6mm.	Diameter of hoop shall be ≥ 8mm
	In wall end zones, for DCH vertical spacing of hoops and cross ties ≤ $6d_{bL}$, $b_o/3$, 125mm.	Vertical spacing of the hoops and or crossties ≤ $0.5b_{web}$ or 100mm.
	Spacing of web reinforcement should not be greater than 250mm.	Spacing of web reinforcement shall be ≤ 250mm.

4.5 Discussion

In the comparison between the Eurocode8 and TEC 2007, it can be noticed that there are many difference and similarity between the two codes.

So The main differences of the two codes can be summarize as the width of beam and the thickness of wall in TEC 2007 greater than the width of beam and the thickness of wall in EC8. Also there are some differences in comparison of reinforcement requirement for beam, column and ductile wall, such as the diameter of stirrups according to TEC 2007 should be equal or higher than Ø 8 which is greater than the diameter of stirrups in EC8 which is equal or greater than Ø 6, As well as the space of stirrups in TEC 2007 smaller than the space of stirrups in EC8. So that it can be observe that these differences of both codes have effect on cost of construction and TEC 2007 is more costly than EC8 because TEC 2007 needs more material for construction buildings.

CHAPTER 5

SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

5.1 Summary and Conclusions

Generally the earthquake is a ground shaking which can be horizontally and vertically or in all directions caused by a sudden movement of rock on the crust of earth. It results in a sudden release of energy and creates seismic waves. Earthquake affects buildings indirectly, the ground shaking leads to the shaking of building structure and exerts inertia forces on them.

Earthquake hits every part of the world and cause many death and injuries and leaves a lot of structures large damages, because of their weakness to resist seismic attack in case of earthquake events due to poor detailing of seismic resisting building according to insufficient design codes. Thus many seismic codes were published in all around the world.

The most important causes of damage during earthquakes have to do with a 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 mistakes of the past.

The importance of earthquake resistant of building construction in developing country is to ensure the human life safety, to keep environment, and to keep the economy of these countries. In developing countries or nations, they can build less expensive earthquake proof buildings by using local existing materials by considering earthquake resistant building construction method.

In this study, the seismic construction recommended by European Union code (EC8) and Turkish Earthquake code are considered for comparison.

Based on the research performed in this study, the following conclusions were drawn:

- Seismic construction in turkey according to TEC2007 needs more material (used more material for construction building) and heavier collapse occurred in turkey because turkey is situated in a very active earthquake zone and more than 90% of its land areas is with in highly seismic regions.

- The Turkish earthquake code 2007 different from Eurocode8 because turkey is an earthquake zone strict. Turkey are all at higher risk of earthquake than many other in European. In Europe general there are not significant earthquake and even in some members of European Union such as Finland and Ireland there is no earthquake occurrences.
- The fundamental requirements of the Eurocode8 and Turkish Seismic codes are the same such as prevent the damage structural and nonstructural elements in minor shaking, limit the damage structural and nonstructural elements in moderate shaking, and prevent overall or partial collapse of building in major ground shaking, as well as their design approaches are very similar, both are aimed at designing safe and economic structures.
- Both codes consider the shear wave velocity to find soil types, but the Turkish seismic code gives more information about soil profile depending of topmost layer thickness of soil (h1).
- Turkish Seismic code for all ground types gives the same peak value, but Eurocode8 not like Turkish Seismic code, EC8 type 1 and type 2 specify different peak values depend on ground types. The shapes of the elastic response spectrum of the Type-2 are more peaking for short period structure excepting for ground type A, this difference can be explained by the fact that EC8 takes into account site effect by introducing site factor (S), and near and far field
- The elastic response spectrum for acceleration, which is used for computing elastic earthquake force, shows diversity, from one earthquake to another and it is affected by local ground conditions. Theoretically, seismic ground motions are shown by elastic acceleration spectrum in both building codes. Spectrum characteristic periods are defined due to the local site classes.
- Eurocode8 is an up to date standard and provides seismic zoning map for Rock conditions. The supplied soil factors (S) and different spectrum types can be used to determine the site specific spectrum. This has been enabled by provided soil period and amplification factors. However TEC 2007 provided Seismic Zoning Map specifies the soil amplification within the given ground shaking value and the code only allows the site period change by different spectrum types.

- EC8 specifies the values of the maximum allowable behavior factor depending on type of structural system, regularity in elevation and prevailing failure mode in the system with walls, whereas TEC specifies periods for structure and ground class (T and TB) dependent values of behavior factor in addition to structural system.
- The design building of both Eurocode8 and Turkish seismic code are based on the two criteria which are regularity in plan and in elevation.
- Earthquakes affect buildings as horizontal loads, an adequate number of shear walls should be placed to increase the lateral rigidity and decrease translation during the construction of a load bearing system.
- According to Eurocode8 and Turkish seismic code should be conduct soil test and investigate the soil nature to avoid soil liquefactions.
- The width of beam according to Eurocode8 should be equal or greater than 200mm. while the width of beam according to Turkish seismic code 2007 shall be equal or greater than 250mm.
- According to Eurocode8 and Turkish seismic code 2007, in the beam reinforcement the number of rebars at the bottom and top of the beam should be equal or greater than 2 rebars.
- According to Eurocode8 and Turkish seismic code 2007, in the beam reinforcement the distance of first hoop from column shall be smaller than or equal 50mm.
- The cross-sectional dimensions of column shall not be less than 250mm according to Eurocode8 and Turkish seismic code 2007.
- In the building construction, the many mistakes that cause of earthquake damage are: (soft story, irregularities in vertical and horizontal directions, poor workmanship, and low strength of materials, inadequate transverse reinforcement stirrup usage, and lack of control).

5.2 Recommendations

From the observations and conclusions made from this study, following recommendations are made on seismic construction according to European Union construction (Eurocode8) and Turkish seismic construction:

- Earthquake should be considered in design of building construction to certain permanence of structures and strength with satisfactory degree of protection against seismic waves and its intensity.
- The architects and structural engineers should work together in the planning and design stage to ensure that a proper pattern and design is selected for building construction.
- 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 to avoid the damage and collapse of building.
- Construction on ground prone to liquefaction should be avoided.
- In new reinforced concrete construction, smooth reinforcing bars should not be used other than for stirrups and ties.
- Careful attention should be paid to concrete mix design, quality control, and placement.
- Concrete shall be placed by using vibrator.
- Experienced supervisor should be employed to have good quality control at site.
- Avoid open ground (Soft storey) which is used for car parking.

5.3 Recommendations for Future Studies

More research should be carried out on the convergences and divergences in seismic construction based on European Union construction (EC8) and Turkish seismic construction consider the plan of building, calculating the various elements of construction of building (steel bar, concrete etc.), to find the most economical code for construction.

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APPENDIX 1

BEAM REINFORCEMENT FORMEDIUM DUCTILITYACCORDING TO EC8

5.4.3.1.2 Detailing for local ductility

(1)P The regions of a primary seismic beam up to a distance $l_{cr} - h_w$ (where h_w denotes the depth of the beam) from an end cross-section where the beam frames into a beam-column joint, as well as from both sides of any other cross-section liable to yield in the seismic design situation, shall be considered as being critical regions.

(2) In primary seismic beams supporting discontinued (cut-off) vertical elements, the regions up to a distance of $2h_w$ on each side of the supported vertical element should be considered as being critical regions.

(3)P To satisfy the local ductility requirement in the critical regions of primary seismic beams, the value of the curvature ductility factor μ_ϕ shall be at least equal to the value given in **5.2.3.4(3)**.

(4) The requirement specified in **(3)**P of this subclause is deemed to be satisfied, if the following conditions are met at both flanges of the beam.

a) at the compression zone reinforcement of not less than half of the reinforcement provided at the tension zone is placed, in addition to any compression reinforcement needed for the ULS verification of the beam in the seismic design situation.

b) The reinforcement ratio of the tension zone ρ does not exceed a value ρ_{max} equal to:

$$\rho_{max} = \rho' + \frac{0,0018}{\mu_\varphi \varepsilon_{sy,d}} \cdot \frac{f_{cd}}{f_{yd}} \tag{5.11}$$

with the reinforcement ratios of the tension zone and compression zone, ρ and ρ', both normalised to bd, where b is the width of the compression flange of the beam. If the tension zone includes a slab, the amount of slab reinforcement parallel to the beam within the effective flange width defined in **5.4.3.1.1(3)** is included in ρ.

(5)P Along the entire length of a primary seismic beam, the reinforcement ratio of the tension zone, ρ, shall be not less than the following minimum value ρ_{min}:

$$\rho_{\min} = 0{,}5\left(\frac{f_{\text{ctm}}}{f_{\text{yk}}}\right) \tag{5.12}$$

(6)P Within the critical regions of primary seismic beams, hoops satisfying the following conditions shall be provided:

a) The diameter d_{bw} of the hoops (in millimetres) shall be not less than 6.

b) The spacing, s, of hoops (in millimetres) shall not exceed:

$$s = \min\{h_{\text{w}}/4;\ 24d_{\text{bw}};\ 225;\ 8d_{\text{bL}}\} \tag{5.13}$$

where

d_{bL} is the minimum longitudinal bar diameter (in millimetres); and

h_{w} the beam depth (in millimetres).

c) The first hoop shall be placed not more than 50 mm from the beam end section (see Figure 5.6).

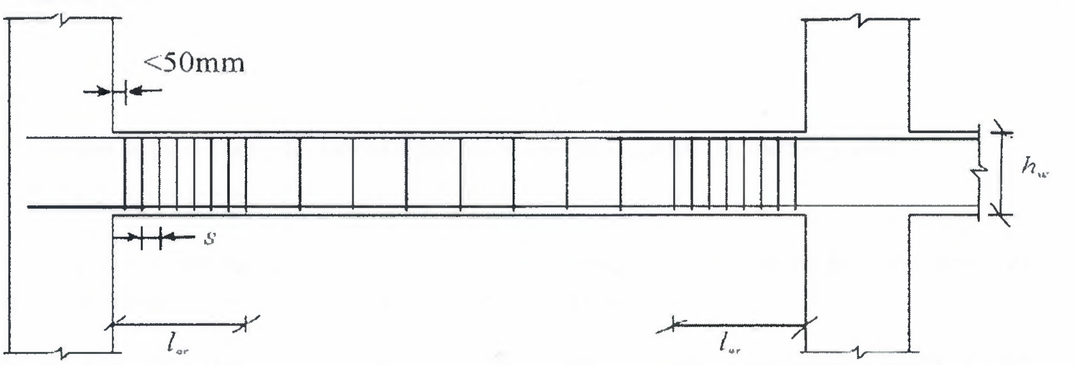

Figure 5.6: Transverse reinforcement in critical regions of beams

APPENDIX 2

COLUMN REINFOCEMENTFOR MEDIUM DUCTILITY ACCORDING TO EC8

5.4.3.2.2 Detailing of primary seismic columns for local ductility

(1)P The total longitudinal reinforcement ratio ρ_l shall be not less than 0,01 and not more than 0,04. In symmetrical cross-sections symmetrical reinforcement should be provided ($\rho = \rho'$).

(2)P At least one intermediate bar shall be provided between corner bars along each column side, to ensure the integrity of the beam-column joints.

(3)P The regions up to a distance l_{cr} from both end sections of a primary seismic column shall be considered as being critical regions.

(4) In the absence of more precise information, the length of the critical region l_{cr} (in metres) may be computed from the following expression:

$$l_{cr} = \max\{h_c;\ l_{cl}/6;\ 0{,}45\} \tag{5.14}$$

where

h_c is the largest cross-sectional dimension of the column (in metres); and

l_{cl} is the clear length of the column (in metres).

(5)P If $l_c/h_c<3$, the entire height of the primary seismic column shall be considered as being a critical region and shall be reinforced accordingly.

(6)P In the critical region at the base of primary seismic columns a value of the curvature ductility factor, μ_ϕ, should be provided, at least equal to that given in **5.2.3.4(3)**.

(7)P If for the specified value of μ_ϕ a concrete strain larger than ε_{cu2}=0,0035 is needed anywhere in the cross-section, compensation for the loss of resistance due to spalling of the concrete shall be achieved by means of adequate confinement of the concrete core, on the basis of the properties of confined concrete in EN 1992-1-1:2004, **3.1.9**.

(8) The requirements specified in **(6)**P and **(7)**P of this subclause are deemed to be satisfied if:

$$\alpha\omega_{wd} \geq 30\mu_{\varphi}\nu_{d} \cdot \varepsilon_{sy.d} \cdot \frac{b_c}{b_o} - 0{,}035 \qquad (5.15)$$

where

ω_{wd} is the mechanical volumetric ratio of confining hoops within the critical regions

$$\left[\omega_{wd} = \frac{\text{volume of confining hoops}}{\text{volume of concrete core}} \cdot \frac{f_{yd}}{f_{cd}}\right];$$

μ_{ϕ} is the required value of the curvature ductility factor;

ν_d is the normalised design axial force ($\nu_d = N_{Ed}/A_c{\cdot}f_{cd}$);

$\varepsilon_{sy.d}$ is the design value of tension steel strain at yield;

h_c is the gross cross-sectional depth (parallel to the horizontal direction in which the value of μ_{ϕ} used in **(6)**P of this subclause applies);

h_o is the depth of confined core (to the centreline of the hoops);

b_c is the gross cross-sectional width;

b_o is the width of confined core (to the centreline of the hoops);

α is the confinement effectiveness factor, equal to $\alpha = \alpha_n \cdot \alpha_s$, with:

a) For rectangular cross-sections:

$$\alpha_n = 1 - \sum_n b_i^2 / 6b_o h_o \qquad (5.16a)$$

$$\alpha_s = (1 - s/2b_o)(1 - s/2h_o) \qquad (5.17a)$$

where

n is the total number of longitudinal bars laterally engaged by hoops or cross ties; and

b_i is the distance between consecutive engaged bars (see Figure 5.7; also for b_o, h_o, s).

b) For circular cross-sections with circular hoops and diameter of confined core D_o (to the centreline of hoops):

$$\alpha_n = 1 \qquad (5.16b)$$

$$\alpha_s = (1 - s/2D_o)^2 \qquad (5.17b)$$

c) For circular cross-sections with spiral hoops:

$$\alpha_n = 1 \qquad (5.16c)$$

$$\alpha_s = (1 - s/2D_o) \qquad (5.17c)$$

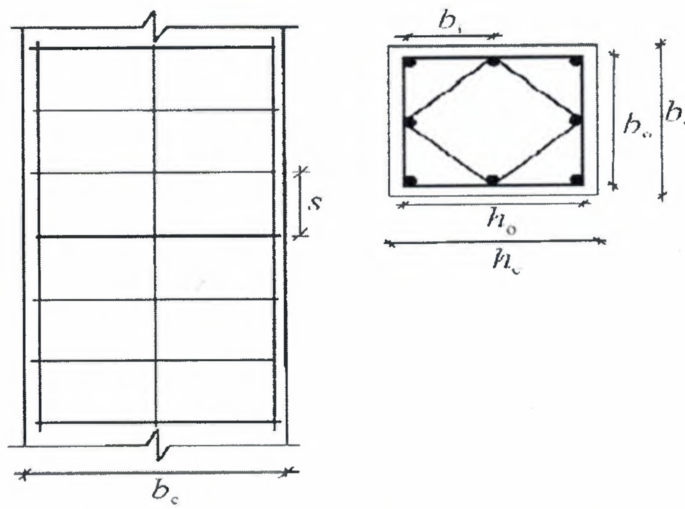

Figure 5.7: Confinement of concrete core

(9) A minimum value of ω_{wd} equal to 0,08 should be provided within the critical region at the base of the primary seismic columns.

(10)P Within the critical regions of the primary seismic columns, hoops and cross-ties, of at least 6 mm in diameter, shall be provided at a spacing such that a minimum ductility is ensured and local buckling of longitudinal bars is prevented. The hoop pattern shall be such that the cross-section benefits from the triaxial stress conditions produced by the hoops.

(11) The minimum conditions of **(10)**P of this subclause are deemed to be satisfied if the following conditions are met.

a) The spacing, s, of the hoops (in millimetres) does not exceed:

$$s = \min\{b_o/2; 175; 8d_{bL}\} \quad (5.18)$$

where

b_o (in millimetres) is the minimum dimension of the concrete core (to the centreline of the hoops); and

d_{bL} is the minimum diameter of the longitudinal bars (in millimetres).

b) The distance between consecutive longitudinal bars engaged by hoops or cross-ties does not exceed 200 mm, taking into account EN 1992-1-1:2004, **9.5.3(6)**.

(12)P The transverse reinforcement within the critical region at the base of the primary seismic columns may be determined as specified in EN 1992-1-1:2004, provided that the value of the normalised axial load in the seismic design situation is less than 0,2 and the value of the behaviour factor q used in the design does not exceed 2,0.

APPENDIX 3

Ductile Wall REINFORCEMENT FOR MEDIUM DUCTILITY ACCORDING TO EC8

5.4.3.4.2 Detailing for local ductility

(1) The height of the critical region h_{cr} above the base of the wall may be estimated as:

$$h_{\mathrm{cr}} = \max\left[l_{\mathrm{w}},\ h_{\mathrm{w}} / 6\right] \tag{5.19a}$$

but

$$h_{\mathrm{cr}} \leq \begin{cases} 2 \cdot l_{\mathrm{w}} \\ \begin{cases} h_{\mathrm{s}} & \text{for n} \leq 6 \text{ storeys} \\ 2 \cdot h_{\mathrm{s}} & \text{for n} \geq 7 \text{ storeys} \end{cases} \end{cases} \tag{5.19b}$$

where h_s is the clear storey height and where the base is defined as the level of the foundation or the top of basement storeys with rigid diaphragms and perimeter walls.

(2) At the critical regions of walls a value μ_ϕ of the curvature ductility factor should be provided, that is at least equal to that calculated from expressions (5.4), (5.5) in **5.2.3.4(3)** with the basic value of the behaviour factor q_o in these expressions replaced by the product of q_o times the maximum value of the ratio M_{Ed}/M_{Rd} at the base of the wall in the seismic design situation, where M_{Ed} is the design bending moment from the analysis; and M_{Rd} is the design flexural resistance.

(3) Unless a more precise method is used, the value of μ_ϕ specified in **(2)** of this subclause may be supplied by means of confining reinforcement within edge regions of the cross-section, termed boundary elements, the extent of which should be determined in accordance with **(6)** of this subclause. The amount of confining reinforcement should be determined in accordance with **(4)** and **(5)** of this subclause:

(4) For walls of rectangular cross-section, the mechanical volumetric ratio of the required confining reinforcement ω_{vd} in boundary elements should satisfy the following expression, with the -values of μ_ϕ as specified in **(2)** of this subclause:

(4) For walls of rectangular cross-section, the mechanical volumetric ratio of the required confining reinforcement ω_{wd} in boundary elements should satisfy the following expression, with the -values of μ_ϕ as specified in **(2)** of this subclause:

$$\alpha\omega_{wd} \geq 30\mu_\varphi(\nu_d + \omega_v)\varepsilon_{sy.d}\frac{b_c}{b_o} - 0{,}035 \qquad (5.20)$$

where the parameters are defined in **5.4.3.2.2(8)**, except ω_v, which is the mechanical ratio of vertical web reinforcement ($\omega_v=\rho_v f_{yd.v}/f_{cd}$).

(5) For walls with barbells or flanges, or with a section consisting of several rectangular parts (T-, L-, I-, U-shaped sections, etc.) the mechanical volumetric ratio of the confining reinforcement in the boundary elements may be determined as follows:

a) The axial force, N_{Ed}, and the total area of the vertical reinforcement in the web, A_{sv}, shall be normalised to $h_c b_c f_{cd}$, with the width of the barbell or flange in compression taken as the cross-sectional width b_c ($\nu_d=N_{Ed} / h_c b_c f_{cd}$, $\omega_v=(A_{sv}/h_c b_c)f_{yd} / f_{cd}$). The neutral axis depth x_u at ultimate curvature after spalling of the concrete outside the confined core of the boundary elements may be estimated as:

$$x_u = (\nu_d + \omega_v)\frac{l_w b_c}{b_o} \qquad (5.21)$$

where b_o is the width of the confined core in the barbell or flange. If the value of x_u from expression (5.21) does not exceed the depth of the barbell or flange after spalling of the cover concrete, then the mechanical volumetric ratio of the confining reinforcement in the barbell or flange is determined as in a) of this subclause (i.e. from expression (5.20), **5.4.3.4.2(4)**), with ν_d, ω_v, b_c and b_o referring to the width of the barbell or flange.

b) If the value of x_u exceeds the depth of the barbell or flange after spalling of the cover concrete, the general method may be followed, which is based on: 1) the definition of the curvature ductility factor as $\mu_\phi=\phi_u / \phi_y$, 2) the calculation of ϕ_u as $\varepsilon_{cu2.c} / x_u$ and of ϕ_y as $\varepsilon_{sy} / (d - x_y)$, 3) section equilibrium for the estimation of neutral axis depths x_u and x_y, and 4) the values of strength and ultimate strain of confined concrete, $f_{ck.c}$ and $\varepsilon_{cu2.c}$ given in EN 1992-1-1:2004, **3.1.9** as a function of the effective lateral confining stress. The required confining reinforcement, if needed, and the confined wall lengths should be calculated accordingly.

(6) The confinement of **(3)-(5)** of this subclause should extend vertically over the height h_{cr} of the critical region as defined in **5.4.3.4.2(1)** and horizontally along a length l_c measured from the extreme compression fibre of the wall up to the point where unconfined concrete may spall due to large compressive strains. If more precise data is not available, the compressive strain at which spalling is expected may be taken as being equal to ε_{cu2}=0,0035. The confined boundary element may be limited to a distance of $x_u(1- \varepsilon_{cu2}/\varepsilon_{cu2.c})$ from the hoop centreline near the extreme compression fibre, with the depth of the confined compression zone x_u at ultimate curvature estimated from equilibrium (cf. expression (5.21) for a constant width b_o of the confined compression zone) and the ultimate strain $\varepsilon_{cu2.c}$ of confined concrete estimated on the basis of EN 1992-1-1:2004, **3.1.9** as $\varepsilon_{cu2.c}$=0,0035+0,1$\alpha\omega_{wd}$ (Figure 5.8). As a minimum, the length l_c of the confined boundary element should not be taken as being smaller than $0,15 \cdot l_w$ or $1,50.b_w$.

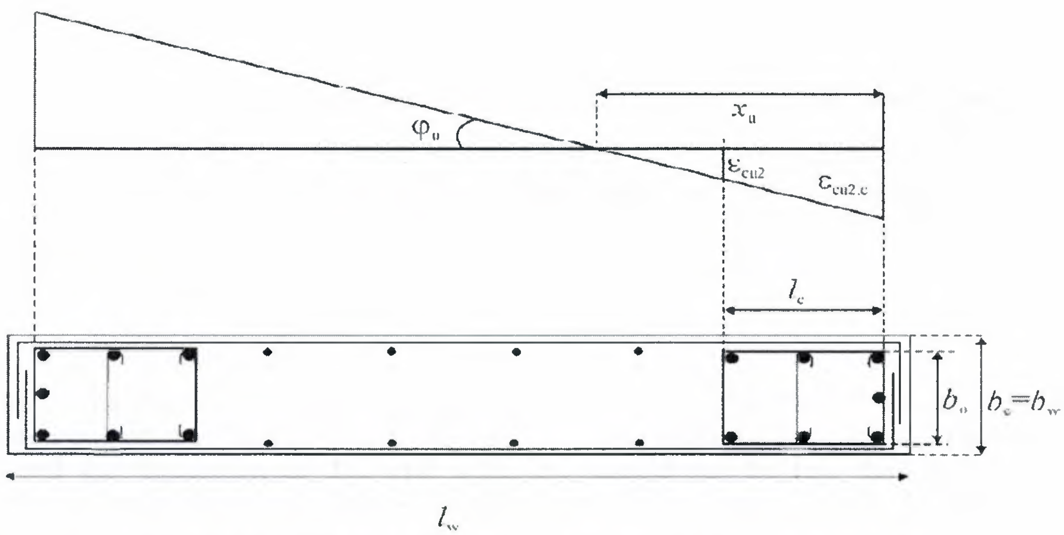

Figure 5.8: Confined boundary element of free-edge wall end

(top: strains at ultimate curvature; bottom: wall cross-section)

(7) No confined boundary element is required over wall flanges with thickness $b_f \geq h_s/15$ and width $l_f \geq h_s/5$, where h_s denotes the clear storey height (Figure 5.9). Nonetheless, confined boundary elements may be required at the ends of such flanges due to out-of-plane bending of the wall .

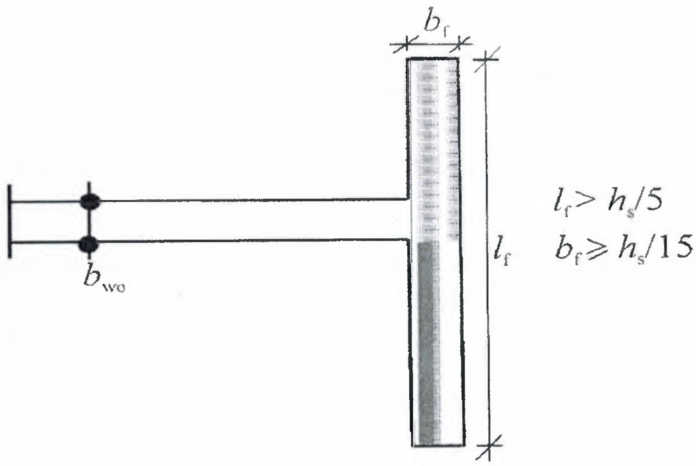

Figure 5.9: Confined boundary element not needed at wall end with a large transverse flange

(8) The longitudinal reinforcement ratio in the boundary elements should be not less than 0,005.

(9) The provisions of **5.4.3.2.2(9)** and **(11)** apply within the boundary elements of walls. Overlapping hoops should be used, so that every other longitudinal bar is engaged by a hoop or cross-tie.

(10) The thickness b_w of the confined parts of the wall section (boundary elements) should not be less than 200 mm. Moreover, if the length of the confined part does not exceed the maximum of $2b_w$ and $0,2l_w$, b_w should not be less than $h_s/15$, with h_s denoting the storey height. If the length of the confined part exceeds the maximum of $2b_w$ and $0,2l_w$, b_w should not be less than $h_s/10$ (See Figure 5.10).

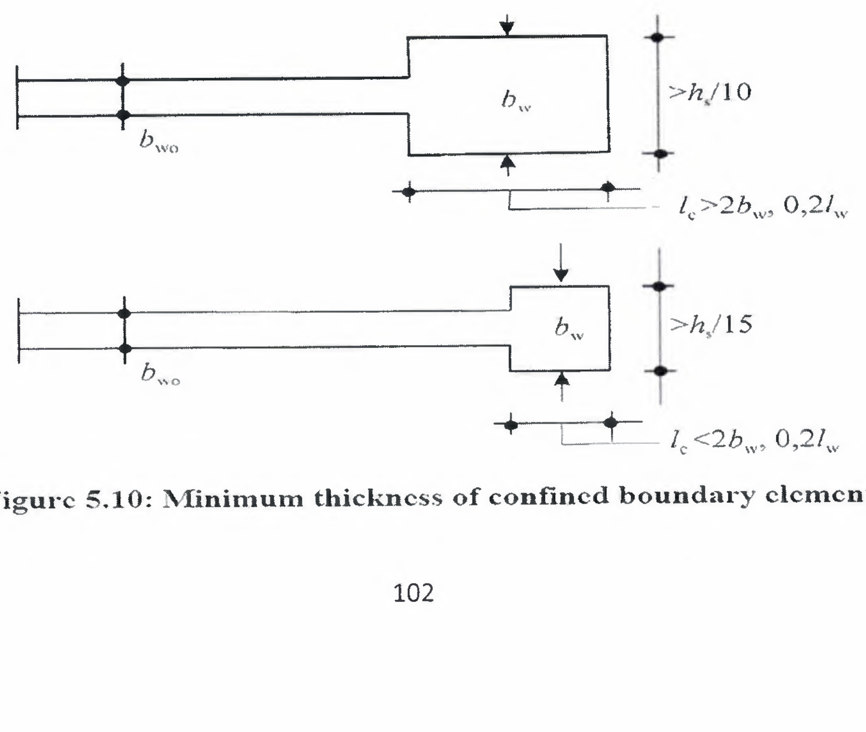

Figure 5.10: Minimum thickness of confined boundary elements

(11) In the height of the wall above the critical region only the relevant rules of EN 1992-1-1:2004 regarding vertical, horizontal and transverse reinforcement apply. However, in those parts of the section where under the seismic design situation the compressive strain ε_c exceeds 0,002, a minimum vertical reinforcement ratio of 0,005 should be provided.

(12) The transverse reinforcement of the boundary elements of **(4)-(10)** of this subclause may be determined in accordance with EN 1992-1-1:2004 alone, if one of the following conditions is fulfilled:

a) The value of the normalised design axial force ν_d is not greater than 0,15; or,

b) the value of ν_d is not greater than 0,20 and the q-factor used in the analysis is reduced by 15%.

APPENDIX 4

FOUNDATION TIE BEAMSACCORDING TO EC8

5.8.2 Tie-beams and foundation beams

(1)P Stub columns between the top of a footing or pile cap and the soffit of tie-beams or foundation slabs shall be avoided. To this end, the soffit of tie-beams or foundation slabs shall be below the top of the footing or the pile cap.

(2) Axial forces in tie-beams or tie-zones of foundation slabs in accordance with **5.4.1.2(6)** and **(7)** of EN 1998-5, should be taken in the verification to act together with the action effects derived in accordance with **4.4.2.6(2)**P or **4.4.2.6(3)** for the seismic design situation, taking into account second-order effects.

(3) Tie-beams and foundation beams should have a cross-sectional width of at least $b_{w,min}$ and a cross-sectional depth of at least $h_{w,min}$.

NOTE The values ascribed to $b_{w,min}$ and $h_{w,min}$ for use in a country may be found in its National Annex to this document. The recommended values are: $b_{w,min}$ = 0,25 m and $h_{w,min}$ = 0,4 m for buildings with up to three storeys, or $h_{w,min}$ = 0,5 m for those with four storeys or more above the basement.

(4) Foundation slabs arranged in accordance with EN 1998-5:2004, **5.4.1.2(2)** for the horizontal connection of individual footings or pile caps, should have a thickness of at least t_{min} and a reinforcement ratio of at least $\rho_{s,min}$ at the top and bottom.

NOTE The values ascribed to t_{min} and $\rho_{s,min}$ for use in a country may be found in its National Annex to this document. The recommended values are: t_{min} = 0,2 m and $\rho_{s,min}$ = 0.2%.

(5) Tie-beams and foundation beams should have along their full length a longitudinal reinforcement ratio of at least $\rho_{b,min}$ at both the top and the bottom.

NOTE The value ascribed to $\rho_{b,min}$ for use in a country may be found in its National Annex to this document. The recommended value of $\rho_{b,min}$ is 0.4%.

APPENDIX 5

FOUNDATION TIE BEAMS ACCORDING TO TEC 2007

12.3.4. Foundation Tie Beams

12.3.4.1 – In reinforced concrete and structural steel buildings, tie beams shall be provided to connect individual footings or pile caps in both directions or to connect continuous foundations at column or structural wall axes. Tie beams may be omitted or their numbers may be reduced on soils classified as Group (A) in Table 12.1.

12.3.4.2 – Consistent with the foundation excavation, tie beams may be constructed at any level between the bottom of the foundation and the bottom of the column.

12.3.4.3 – The minimum requirements to be satisfied by tie beams are given in **Table 12.3** depending on the seismic zone of the building and the soil groups defined in **Table 12.1.**

TABLE 12.3 – MINIMUM REQUIREMENTS FOR TIE BEAMS

DESCRIPTION OF REQUIREMENT	*Seismic Zone*	*Soil Group* (A)	*Soil Group* (B)	*Soil Group* (C)	*Soil Group* (D)
1. Minimum axial force of tie beam (*)	1, 2 3, 4	%6 %4	%8 %6	%10 %8	%12 %10
2. Minimum cross-section dimension (cm) (**)	1, 2 3, 4	25 25	25 25	30 25	30 25
3. Minimum cross-section area (cm^2)	1, 2 3, 4	625 625	750 625	900 750	900 750
4. Minimum longitudinal reinforcement	1, 2 3, 4	4∅14 4∅14	4∅16 4∅14	4∅16 4∅16	4∅18 4∅16

(*) *As a percentage of the greatest axial force of columns or structural walls connectedby tie beams*

(**) *The minimum cross-section dimension shall not be less than 1/30 of the clear span ofthe tie beam.*