Comparison of 1975, 1998 and 2007 Turkish Earthquake Codes on Selected RC Buildings

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Submitted to the
Institute of Graduate Studies and Research
in partial fulfilment of the requirements for the degree of

Master of Science
in
Civil Engineering

Eastern Mediterranean University
September 2017
Gazimağusa, North Cyprus

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ABSTRACT

Throughout the years Turkey has encountered several disastrous earthquakes that had done major losses in lives and in the economy. The loss was huge and that brought the attention of Ministry of Public Works in Turkey to revise and update their design codes. Turkish Earthquake Codes have been updated and improved to meet the safety levels that are needed for this seismic area. In this thesis, the 1975, 1998, and 2007 Turkish Earthquake Codes are compared. Six different case studies were chosen and designed with different elevations; four case studies containing different type of irregularities while the other two are regular designs. The non-linear static pushover analysis method presented in TEC-2007 was chosen for evaluating and understanding how these buildings behave under a seismic activity. Moreover, the performance, the cost and the damage percentages of the buildings in respect to each of the 1975, 1998, and 2007 Turkish Earthquake Codes were conducted. Subsequently each case was investigated to find out the performance of each code in the event of an earthquake. The study has identified that the 1975 Turkish Earthquake Code is dangerous to follow in seismic activity areas due to most cases not meeting the safety criteria. While on the other hand, the 1998 and 2007 Turkish Earthquake Codes has been identified as safe to follow with minor differences.

Keywords: Earthquake, Turkish Earthquake Code, non-linear static pushover, performance, cost, damage percentages.

ÖZ

Türkiye, yıllar içerisinde ekonomisine ağır zararlar veren ve büyük can kayıplarının yaşandığı depremlerle karşı karşıya kalmıştır. Kayıplar çok büyüktü ve bu Bayındırlık Bakanlığı'nın ilgisini, tasarım kodlarını gözden geçirip düzenlemeye yöneltti. Türk Deprem Yönetmelikleri güncellendi ve bu sismik bölgenin güvenlik ihtiyaçlarını karşılamak üzere geliştirildi. Bu çalışmada, 1975, 1998 ve 2007 Türk Deprem Yönetmelikleri karşılaştırılmıştır. Altı farklı vaka incelemesi seçildi ve farklı yüksekliklerde tasarlandı; dört vaka incelemesi birbirinden farklı düzensizlikler içerirken diğer iki vaka incelemesi düzensizlik içermiyordu. TEC-2007'de sunulan statik itme analiz metodu binaların sismik aktivite karşısında nasıl tepki verdiğini ölçmek ve davranışını anlamak için seçilmiştir. Ayrıca, bu binaların performans seviyeleri, maliyet ve hasar yüzdeleri 1975, 1998 ve 2007 Türk Deprem Yönetmeliklerine göre hesaplanıp karşılaştırılmıştır. Bu çalışmada seçilen, 1975 Türk Deprem Yönetmeliği ile tasarlanan çoğu binanın elde edilen yapısal performans seviyesine bağlı olarak güvenli olmadığı görülmüştür. Diğer yandan, 1998 ve 2007 Türk Deprem Yönetmeliklerine göre tasarlanmış vakaların güvenli olduğu saptanmıştır.

Anahtar Kelimeler: Deprem, Türk Deprem Yönetmeliği, statik itme analizi, performans, maliyet, hasar yüzdeleri.

DEDICATION

In Dedication to

My everything, my beloved Parents

To the Angel, my brother, Fadi 09/07/2017

To my Lovely Siblings

To my Dearest Family and Friends

ACKNOWLEDGMENT

I would first like to thank my advisor Assc. Prof. Dr. Giray Özay. The door to Prof. Özay office was always open whenever I ran into a trouble spot or had a question about my research or writing. He consistently allowed this thesis to be my own work, but steered me in the right direction whenever he thought I needed it. Without him it wasn't possible to accomplish this study.

I am also very grateful for Assoc.Prof. Dr. Serhan Şensoy, Chairman of the Civil Engineering Department. Also, many thanks to the Civil Engineering faculty of Eastern Mediterranean University for their kindness and support.

Finally, I must express my very profound gratitude to my parents Mr. Samir and Mrs. Rima. My siblings, Mahmoud, Fadi (Rest in Peace my beloved brother), and Farah, my Grandma Layla, my uncles, and my friends for providing me with unfailing support and continuous encouragement throughout my years of study and through the process of researching and writing this thesis. This accomplishment would not have been possible without them.

Mustapha Ayache

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LIST OF SYMBOLS

A(T)	Spectral Acceleration Coefficient
A_0	Effective Ground Acceleration Coefficient
A_g	Gross Section Area of Column.
A_p	Plane Area of Storey Building.
a_y, a_x	Length of Re-entrant Corners in x, y Direction
C	Base Shear Coefficient
C_0	The Seismic Zone Coefficient in TEC-1975
Di	Amplification Factor to be Applied in Equivalent Seismic Load Method
f_{ctd}	Design Tensile Strength of Concrete
g	Acceleration of Gravity (9.81 m/s^2)
I	Building Importance Factor
K	Structure Type Coefficient
L_x, L_y	Length of the Building at x, y Direction
n	Live Load Participation Factor
N	Total Number of Stories of Building from the Foundation Level
ρ	Transverse Reinforcement
q_i	Total Live Load at i'th Storey of Building
R	Structural Behavior Factor
$R_a(T)$	Seismic Load Reduction Factor
S	Dynamic Coefficient
S	The Dynamic Coefficient
$S(T)$	Spectrum Coefficient
TA, TB	Spectrum Characteristic Period [s]

T_o	The Effective Period of the Ground in Seconds
Vi	Storey Shear at i'th Storey of Building in the Earthquake Direction Considered
V_t	Total Seismic Load Acting on the Structure.
W	Total Weight of Building Calculated by Considering Live Load Participation Factor
η_{bi}	Torsionally Irregularity Factor Defined at i'th Storey of Building
η_{ci}	Strength Irregularity Factor defined at i'th Storey of Building
η_{ki}	Stiffness Irregularity Factor defined at i'th Storey of Building
Δi	Storey Drift of i'th Storey of Building

LIST OF ABBREVIATIONS

3F	Three Floors
5F	Five Floors
C	Collapse
CP	Collapse Prevention
CSM	Capacity Spectrum Method
DCH	High Ductility Building Member
DCL	Low Ductility Building Member
DCM	Displacement Coefficient Method
DCM	Medium Ductility Building Member
HDL	High Ductility Building Level
IO	Immediate Occupancy
LS	Life Safety
m	Meter
NDL	Nominal Ductility Building Level
RC	Reinforced Concrete
TEC-1975	1975 Turkish Earthquake Code
TEC-1998	1998 Turkish Earthquake Code
TEC-2007	2007 Turkish Earthquake Code
TS-500	Requirements for Design and Construction of Reinforced Concrete
USD	United State Dollars

Chapter 1

INTRODUCTION

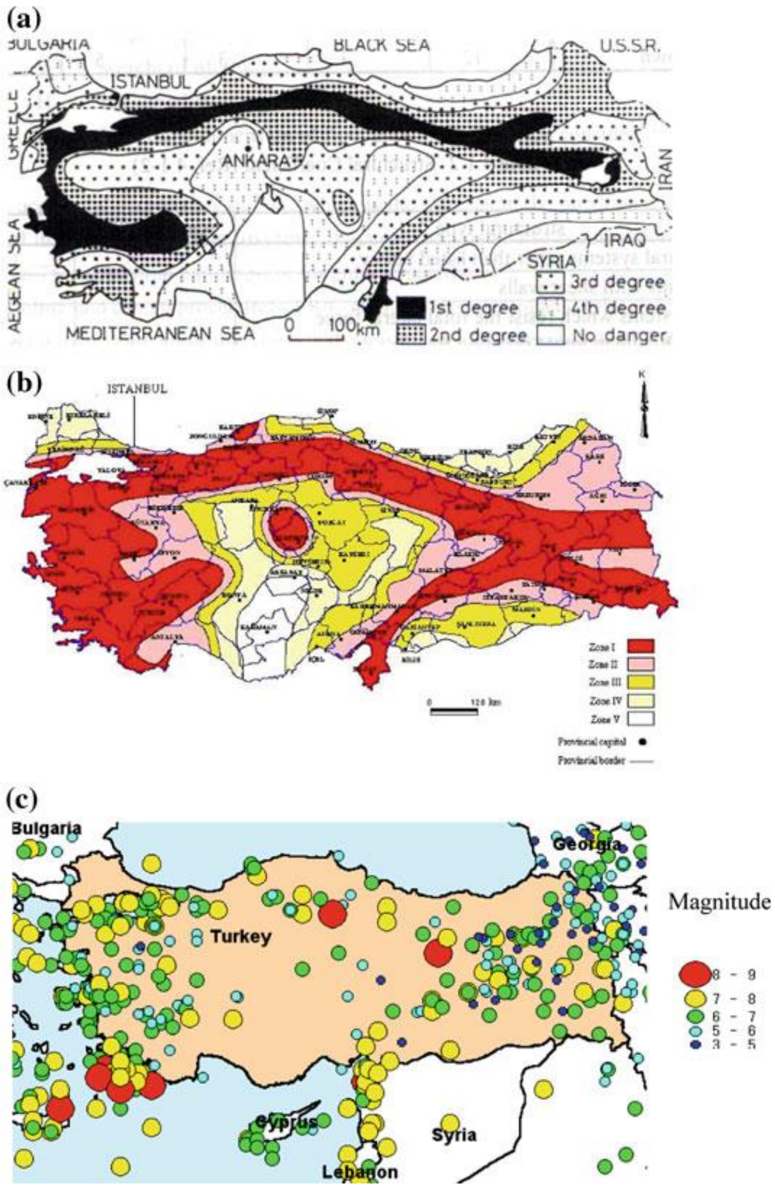
1.1 Introduction

Turkey is located between two tectonic plates, Eurasia and Arabia, which are crumbling into one another, north to south as shown in the Figure 1.1. The Turkish landmass is a modest tectonic plate, which is being pinched between the Eurasian and Arabian plates. This movement has created the North Anatolian fault. When the North Anatolian fault, which is also called a conservative margin, slip, it starts to cause the earthquakes. Many of Turkey's major cities are located along this fault.

Figure 1.1: Northern Anatolian Fault. Hartleb et al (2006)

Due to its location on the North Anatolian Fault, Turkey has been subjected to several disastrous earthquakes throughout the time which caused a huge loss in lives and economy. For example, Kocaeli (Ms= 7.4) and Duzce (Ms= 7.2) earthquakes, that happened in 1999 are the largest natural disasters of the 20th century in Turkey after 1939 Erzincan earthquake (Ms = 6.8). For the Kocaeli earthquake, the official death toll was more than 15,000, with approximately 44,000 people injured and thousands left homeless. A total of 330,000 residences were damaged; the shares of light, moderate, and severely damaged or collapsed units are 118,000, 112,000, and 100,000, respectively, Ilki, & Celep (2012).

The damage that happened to structures was more than expected considering the magnitudes of the events. That is due to the huge building stock were designed lacking the needed steel reinforcements for seismic activity. Most of these structures are far from processing qualities that would ensure satisfactory seismic performance. After the Erzincan earthquake in 1939, the Ministry of Public Works released the first set of precise legal board for earthquake-resistant designs in 1940, followed by another version in 1942 associated with a seismic zone map. The code was modified in 1949 and 1953 to reflect the change of the seismic zone map, Gulkan (2000). The next revisions in 1968 and 1975 introduced important enhancements to the seismic design and the international developments to the engineering society in Turkey. The concept of ductility was first time mentioned in the 1975 code. The principles of the capacity design were introduced by the 1998 code together with important detailing issues for seismic design. The latest version of the code released in 2007 has set a very important milestone towards safety in seismic design of existing buildings and new ones.



a) Seismic zone map in 1972
b) current seismic zone map
c) historical hazardous earthquakes around Turkey

Figure 1.2: Seismic Map by Ministry of Public Works , Ilki, & Celep (2012)

1.2 Previous Work Done

After outlining the performance of existing buildings in Turkey during recent earthquakes (particularly Kocaeli 1999 and Duzce 1999 Earthquakes), and by focusing on the observed common structural deficiencies, a brief summary of the evolution of the Turkish Seismic Design Code in the last decades is presented in this paper, Ilki &

Celep (2012). It is important to note that the poor seismic performance of existing buildings in Turkey outlined in this study is not directly related to the inefficiency of the relevant seismic design codes, but rather to extremely low-quality construction and the absence of a strict inspection system at the time of their construction. It should also be highlighted that the lessons learnt from the catastrophic consequences of recent earthquakes, revisions in the seismic design code and the developments in the material and workmanship characteristics have significantly improved the quality of newer constructions in Turkey in the last decade. In another study, Akgül (2007), 4 structural system models which represent the existing medium-rise reinforced concrete buildings in Turkey are chosen and designed in accordance with TEC-1975 and TEC-1998 regulations. Two of the models are 4 stories and the rest are 6 stories. Also, material properties and seismic zones of the models vary to represent the buildings in different seismic regions with different material characteristics. These models have a symmetric plan as commonly used. The seismic performances of these models are determined by using linear and nonlinear evaluation methods in TEC-2007. The main reason for damage in reinforced concrete buildings in Turkey is the non-ductile designs as discussed by Isler (2008). In the highlight of such findings, essentials given by Turkish Earthquake Regulation 2007 with respect to design shall is discussed particularly for buildings constructed after the date such regulation is put into effect, and seismic features of the earthquake is being commentated according to the data in connection with the strong ground motion obtained. As for the analysis methods, Equivalent Seismic Load Method, the Mode-Superposition Method and the Analysis Method in Time Domain, and their distinct outcomes are discussed in this paper, Dogangun & Livaoglu (2006). Lastly, methodologies and developing technologies for rapid condition assessment and structural evaluation of existing buildings in Turkey are discussed in this study by Gunes (2015) in order to identify and prioritize high-risk buildings and for guiding decisions on retrofitting or renewal.

1.3 Aim and Scope

Since there is no clear mention of a comparison between the 1975, 1998 and 2007 Turkish Earthquake Codes in the previous work done that's mentioned above, the aim of this thesis is to investigate the seismic performances of the 6 case studies that are; 2 regular types of buildings and 4 irregular types designed according to the 1975, 1998 and 2007 Turkish Earthquake Codes.

The scope of this study is to determine the seismic performance and seismic safety of each code. Compare the seismic performance between each code by using the Non-Linear Static Analysis Method (pushover analysis). And lastly, compare the efficiency of each code regarding economical values

1.4 Outline of the Thesis

Chapter 1 contains a brief introduction about the location of Turkey being on the Anatolian fault and how seismically active that fault is, hence states the objective of this study.

Chapter 2 contains a brief explanation and comparison between the 1975, 1998 and 2007 Turkish Earthquake Codes.

Chapter 3 explains the analysis methods briefly and which method was chosen for this study.

Chapter 4 gives details on the case studies chosen to develop the structural models for the analysis. Design parameters and software used to conduct this study.

Chapter 5 gives the results of the analysis.

Chapter 6 gives the conclusions drawn from this research along with the recommendations for future work.

Chapter 2

COMPARSION OF THE 1975, 1998 & 2007 TURKISH EARTHQUAKE CODES

2.1 Introduction

Throughout the years the Ministry of Public Works and Settlement Government of Republic of Turkey has been working on finding the perfect seismic regulations design code to minimize the risk of earthquakes. The first set of legal provisions was established in 1940 and continued to evolve until the last edition which was published in 2007.

This study is going to focus mainly on the 1975, 1998, and 2007 Turkish Earthquake Codes. In this chapter, the main seismic design rules and regulations will be discussed for each code briefly. And at the end, there will be a comparison between the three codes.

2.2 Turkish Earthquake Code 1975

Ductility and base shear force were first mentioned and used explicitly in this code. The 1975 TEC was valid for more than 20 years and many building stocks are designed according to its rules and regulations.

Main improvements on this code were:

- Involvement of comprehensive rules related to seismic-resistant buildings
- Involvement of technicalities about minimum cross-sectional dimensions and minimum reinforcement ratios for structural members

- Involvement of a significant shear design for beam–column joints
- Involvement of information about irregular buildings. Small details that were not sufficient

2.2.1 Seismic Design Regulations

The following equation is about how to calculate the base shear:

$$C = C_0\, K\, S\, I \geq \frac{C0}{2} \tag{2.1}$$

Where:
C_0: The seismic zone coefficient for each zone are: 0.10 for Zone I, 0.08 for Zone II, 0.06 for Zone III, and 0.04 for Zone IV
K: Structure Type Coefficient
S: Dynamic Coefficient
I: Building Importance Factor

The structure type coefficient K values are given in the table below:

Table 2.1: Structure Type Coefficient (K) values TEC(1975):

Structure Type	**K[1]**
Ductile frame[2]	(a) 0.60, (b) 0.80, (c) 1.00
Non-ductile frame[2]	(a) 1.20, (b) 1.50, (c) 1.50
Steel frames with bracing[2]	(a) 1.20, (b) 1.50, (c) 1.60
Shear wall-ductile frames[2.3]	(a) 0.80, (b) 1.00, (c) 1.20
Shear wall structures with frames	1.33
Masonry buildings	1.5
Other	1

- *1 The minimum value of K is 1.0 for one or two-story structures*
- *2 Having (a) reinforced concrete or reinforced masonry infill walls, (b) unreinforced masonry infill walls, (c) light weight or few infill walls, or prefabricated concrete infill walls*
- *3 The ductile frames should resist at least 25% of the lateral loads*

The dynamic coefficient is to be evaluated by the following equation:

$$S = \frac{1}{|0.8 + T - To|} \leq 1.0 \tag{2.2}$$

Where:

T and T_o: are the fundamental periods of the building and soil column, respectively.

T_o: the effective period of the ground in seconds.

The dynamic coefficient: should be assumed as 1.0 for one and two-story structures and all masonry buildings.

The building importance factor, I, is 1.0 for normal buildings, or 1.5 for important or populated buildings.

The values of live load reduction (*n*) in this code are shown in Table 2.2.

Table 2.2: Live load reduction (*n*) values TEC (1975)

Structure Type	**Live Load Reduction Factor (*n*)**
Storage type structures	0.8
Schools, theatres, concert halls, shops, dormitories	0.6
Residential buildings, offices, hospitals, hotels	0.3

2.2.2 Soil Types

Soil types were classified on the basis of blow counts or shear wave velocity, and values for were set for each type. Shear wave velocities for soil types I through IV were set at greater than 700 m/sec for I, 400 to 700 m/ sec for II, 200 to 400 m/sec for III, and less than 200 m/sec for IV. Figure 2.1 presents spectral shapes for soil types I through IV, respectively.

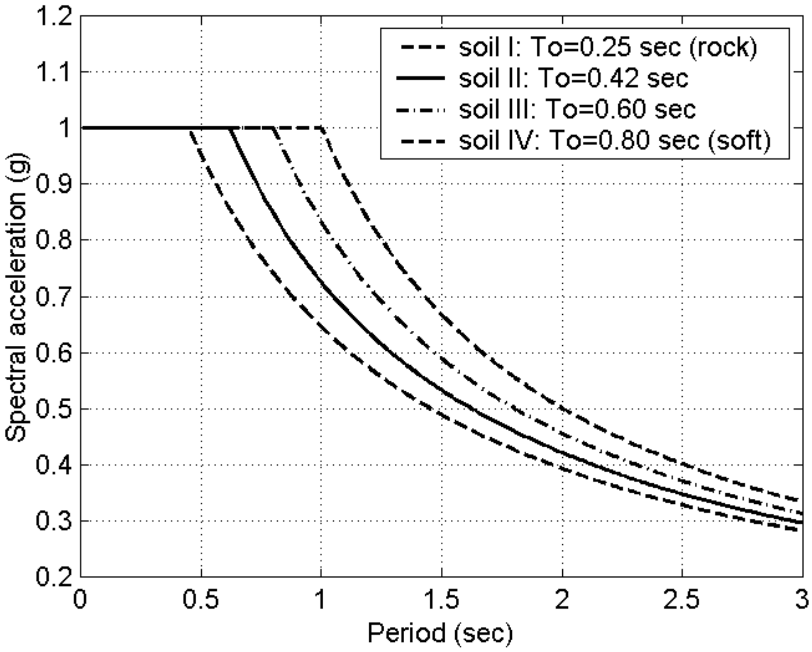


Figure 2.1: Spectral coefficients for soil types TEC (1975)

2.2.3 Geometry and Detailing Requirements

Geometry and detailing requirements for reinforced concrete components were modified in the1975 code. Minimum dimensions were specified for beams (200 mm x 300 mm [width times depth, = B x D]), columns (the smaller of 0.05 times the story height and 250 mm), and shear walls (0.05 times the story height and 150 mm). Minimum reinforcement ratios and sizes were set for beams (minimum stirrup diameter of 8 mm and minimum stirrup spacing of B or 0.5D) and shear walls (ρ = 0.0025, 0.0020 for horizontal and vertical reinforcement, respectively; maximum rebar spacing of 300 mm or 1.5 times the wall thickness). Figure 2.2 shows sample detailing requirements for beams and shear walls. Minimum floor slab thicknesses were set at 100 mm. Infilled joist slab construction (termed "asmolen" construction) was permitted only in buildings taller than 12 m if shear walls were used as the lateral force-resisting system.

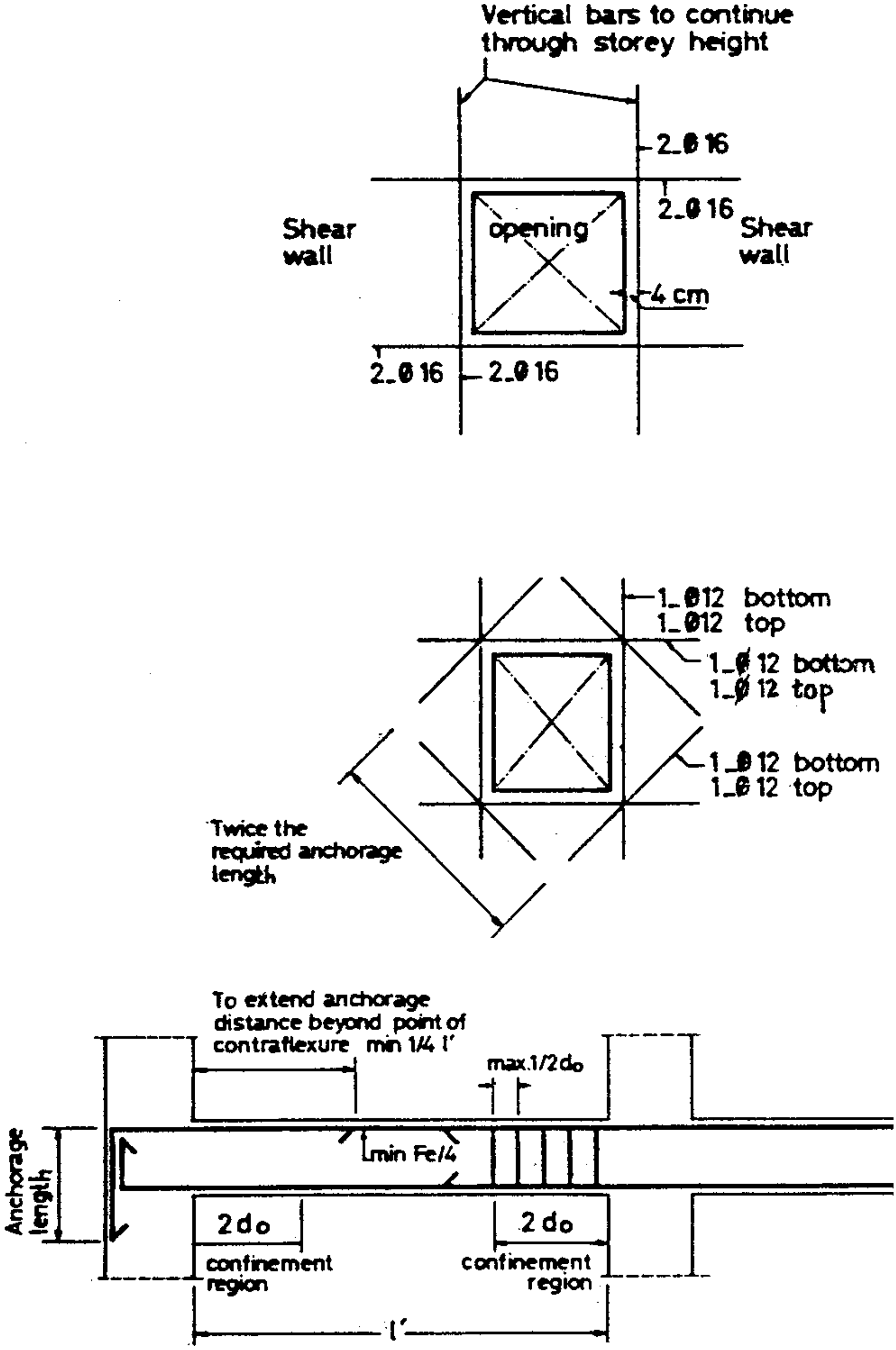


Figure 2.2: Detailing requirements for beams and shear walls TEC (1975)

The 1975 code provided much information on minimum details for columns. The minimum rectangular column dimension was limited to 250 mm or 0.05 times the story height; the maximum column width-to-depth ratio was 3.0. The minimum and maximum longitudinal rebar ratios were 0.01 and 0.035, respectively. Columns were divided into three regions as shown in Figure 2.3 confinement regions at each end of the column clear height, a middle region, and beam-column joint regions. The confinement region was defined as the distance not smaller than 0.167 times the column clear height or 450 mm, measured from the slab soffit or beam top surface. The volumetric ratio of transverse reinforcement, ρ, in this region was set at:

$$\rho = 0.12\,\frac{f_c'}{f_y} \tag{2.3}$$

Where:
f_c': Concrete compressive strength
f_y: Rebar yield strength

Hooks of 135° were required on ties in confinement regions; the minimum tie diameter was 8 mm, and the minimum and maximum tie spacings were 50 mm and 100 mm, respectively. In the middle region, tie sizes were based on gravity and earthquake forces calculated in Equation 2.1. The maximum tie spacing, in Figure 2.3, was the smaller of 200 mm and 12 times the diameter of the longitudinal rebar.

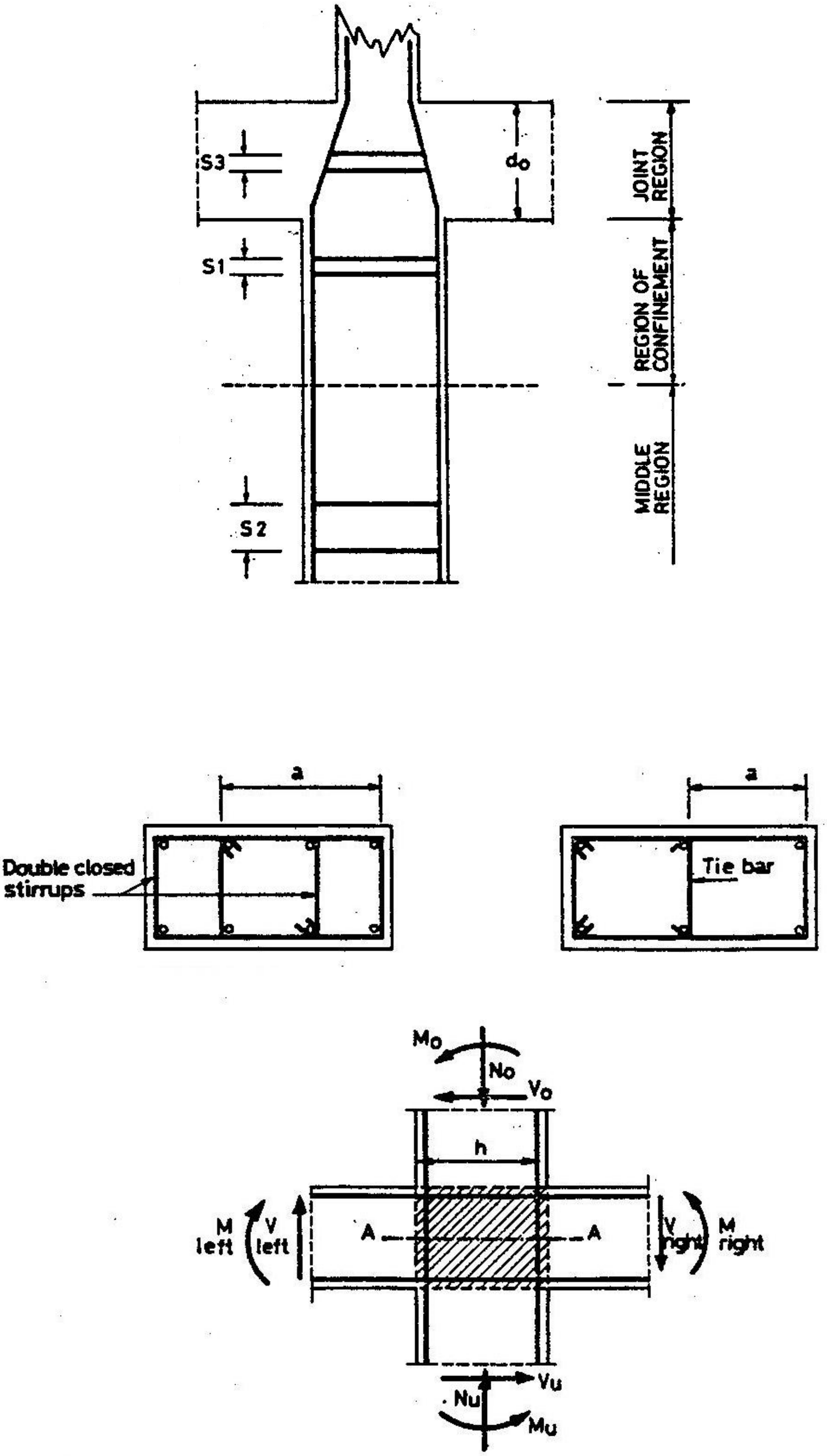


Figure 2.3: Detailing requirements for columns TEC (1975)

2.3 1998 Turkish Earthquake Code

After approximately 20 years, a new code came out with improved formulas and regulations to help maintain safer structures regarding seismic activity. The most important advances in this code are:

- Interpretation of the design earthquake in terms of incident chance
- Interpretation of the elastic design spectrum
- Interpretation of the seismic load reduction factor depending on the structural characteristics
- Involvement of demands on confinement and regulations for reinforcement design
- Perceptible definition of irregularities

Plastic hinges forming at beams should guarantee that the columns are stronger than beams constructed into the same joint, that's the main capacity design principle. Moreover, shear capacity should be higher than the bending capacity of beams, columns, and shear walls to make sure that the ductile failure is higher than that treated in seismic design in the case of seismic loads. In this code, ordinary buildings with an importance factor (I) of 1 are designed so that they counter earthquakes that correspond to 475 years, and has a probability of exceedance of 10% in 50 years. On the other hand, buildings with an importance factor of 1.5 should be designed to handle earthquakes that correspond to 2,475 years and probability of exceedance of 2% in 50 years.

The spectral acceleration coefficient *A(T)* is given by the equation 2.4:

$$A(T) = A_o I \, S(T) \tag{2.4}$$

A_o: Effective seismic acceleration coefficient to be considered respectively 0.40, 0.30, 0.20 and 0.10, for the seismic zones I, II, III and IV
I: Building importance factor (in this code, it got more revised and will be shown in Table 2.3)
S(T): Elastic spectrum coefficient (usually 5% damping ratio and is determined through Equations (2.5a, 2.5b, 2.5c) as a function of the fundamental period of the

building (T) and the characteristic spectrum periods (TA and TB depending on the ground type which is shown in Table 2.4)

Table 2.3: Building Importance Factor according to TEC (1998).

Purpose of occupancy or type of building	**Importance factor (I)**
1. Buildings to be utilized after the earthquake and buildings containing hazardous materials (a) Buildings required to be utilized immediately after the earthquake (hospitals, firefighting buildings, telecommunication facilities, transportation stations and terminals, power generation and distribution facilities, official administration buildings, etc.) (b) Buildings containing or storing toxic, explosive and flammable materials, etc.	1.5
2. Intensively and long-term occupied buildings and buildings preserving valuable goods (a) Schools, dormitories, military barracks, prisons, etc. (b) Museums	1.4
3. Intensively but short-term occupied buildings Sport facilities, cinema, theatre and concert halls, etc.	1.2
4. Buildings other than defined above (residential and office buildings, hotels, building-like industrial structures, etc.)	1.0

Table 2.4: Characteristic Spectrum Periods.

Local site class	**T_A (s)**	**T_B (s)**
Z1	0.10	0.30
Z2	0.15	0.40
Z3	0.15	0.60
Z4	0.20	0.90

$$S\ (T) = 1 + 1.5\,\frac{T}{TA}\ (0 \leq T \leq T_A) \tag{2.5a}$$

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

$$S\ (T) = 2.5\left(\frac{TB}{T}\right)^{0.8} (T \geq TB) \tag{2.5c}$$

Structural designs are classified with two types of ductility levels: normal or high. There are some factors that determine which type of ductility should be used in designs. First, the allowance of inelastic deformations demands that the lateral load be calculated by using the elastic design spectrum and should be minimized relying on the characteristics of the structural system by using seismic load reduction factor $R_a(T)$ given in Equations 2.6a, 2.6b.

$$R_a(T) = 1.5 + (R - 1.5)\frac{T}{TA} \qquad (0 \leq T \leq T_A) \tag{2.6a}$$

$$R_a(T) = R \qquad (T > T_A) \tag{2.6b}$$

The reduced base shear force (V_t) can be calculated by Equation 2.7, where W is the total weight of the building.

$$V_t = W\frac{A(T)}{Ra(T)} \geq 0.10 A_o\, I\, W \tag{2.7}$$

The building is said to be a high ductility system if it can possess those characteristics. The diversity of the spectral acceleration coefficient for different seismicity levels is shown in the Figure 2.4. Structural System Behaviour Factors are given in Table 2.5, to determine structures with high ductility and normal ductility.

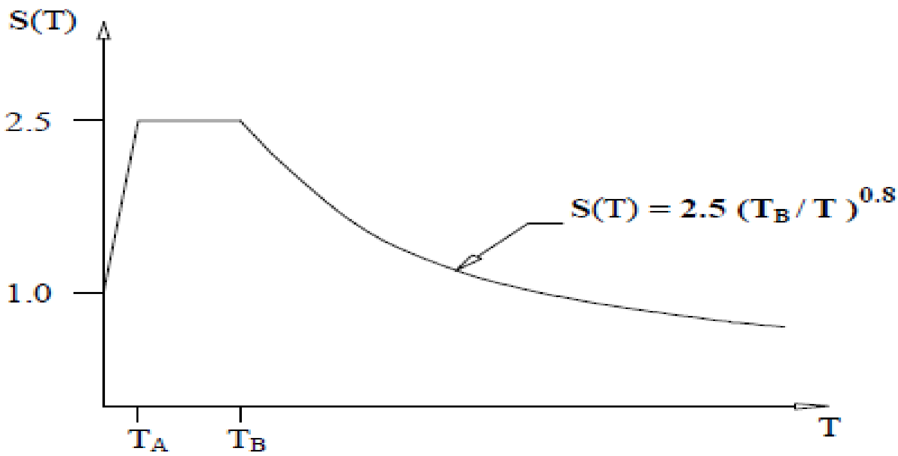


Figure 2.4: Special Design Acceleration Spectra TEC (1998)

Table 2.5: Structural System Behaviour Factors TEC (1998)

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 solid and / or coupled structural 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	6
(2.2) Single-story buildings in which seismic loads are fully resisted by columns with hinged upper connections	-	5
(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.	-	4
(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	5

2.3.1 Geometry and Detailing Requirements

Lateral load resisting structural systems of reinforced concrete buildings shall be classified with respect to their seismic behaviour into two classes defined below.

Reinforced concrete structural systems given below are defined as Systems of High Ductility Level:

- Frame type structural systems comprised of columns and beams dimensioned and reinforced
- Structural systems comprised of solid or coupled structural walls dimensioned and reinforced

- Frame - wall structural systems made of combining two systems defined above.

Reinforced concrete structural systems given below are defined as Systems of Nominal Ductility Level:

- Frame type structural systems comprised of columns and beams dimensioned and reinforced
- Structural systems comprised of solid or coupled structural walls dimensioned and reinforced
- Frame - wall structural systems made of combining two systems defined above.

Detailing requirements are more stringent for systems with high ductility. Transverse reinforcement requirements for beams are presented in Figure 2.5. These requirements apply for frames of both high and nominal ductility. The volumetric ratio of transverse reinforcement, ρ, in this region was set at:

$$\rho \geq \frac{fctd}{f_y} \tag{2.8}$$

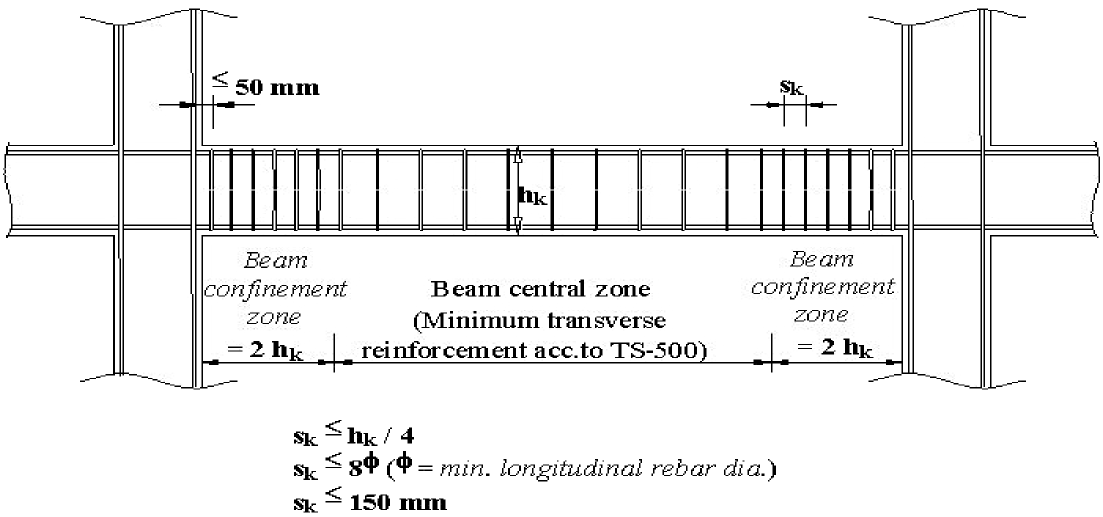


Figure 2.5: Transverse reinforcement requirements for beams TEC (1998)

The detailing requirements for columns of high and nominal ductility levels are most similar. The minimum cross-section dimensions are 250 mm by 300 mm. Information on the transverse reinforcement requirements along the height of a column are shown in Figure 2.6. All hoops must have 135° seismic hooks at both ends. Cross ties may have 90° hooks at one end. The sum of the column strengths at a beam-column joint must exceed 120% of the sum of the beam strengths at that joint. The shear strength of a column must exceed the shear force associated with the plastic moments in the column. The only major provision that is not applicable for columns of nominal ductility level is the spacing of transverse reinforcement along the confinement zones, which is required to be half the spacing in the column middle region. Lap splices of column longitudinal rebar should be made in the middle third of the column. If column rebars are spliced at the bottom of a column, the splice length is increased to 125% or 150% of the development length of the bar in tension, depending on the number of bars being spliced. For columns in frames of nominal ductility, the maximum spacing of the transverse reinforcement between the confinement zones is increased by a factor of 2 over the spacing shown in Figure 2.6. For shear walls, the minimum wall thickness is the smaller of 0.067 times the story height and 200 mm.

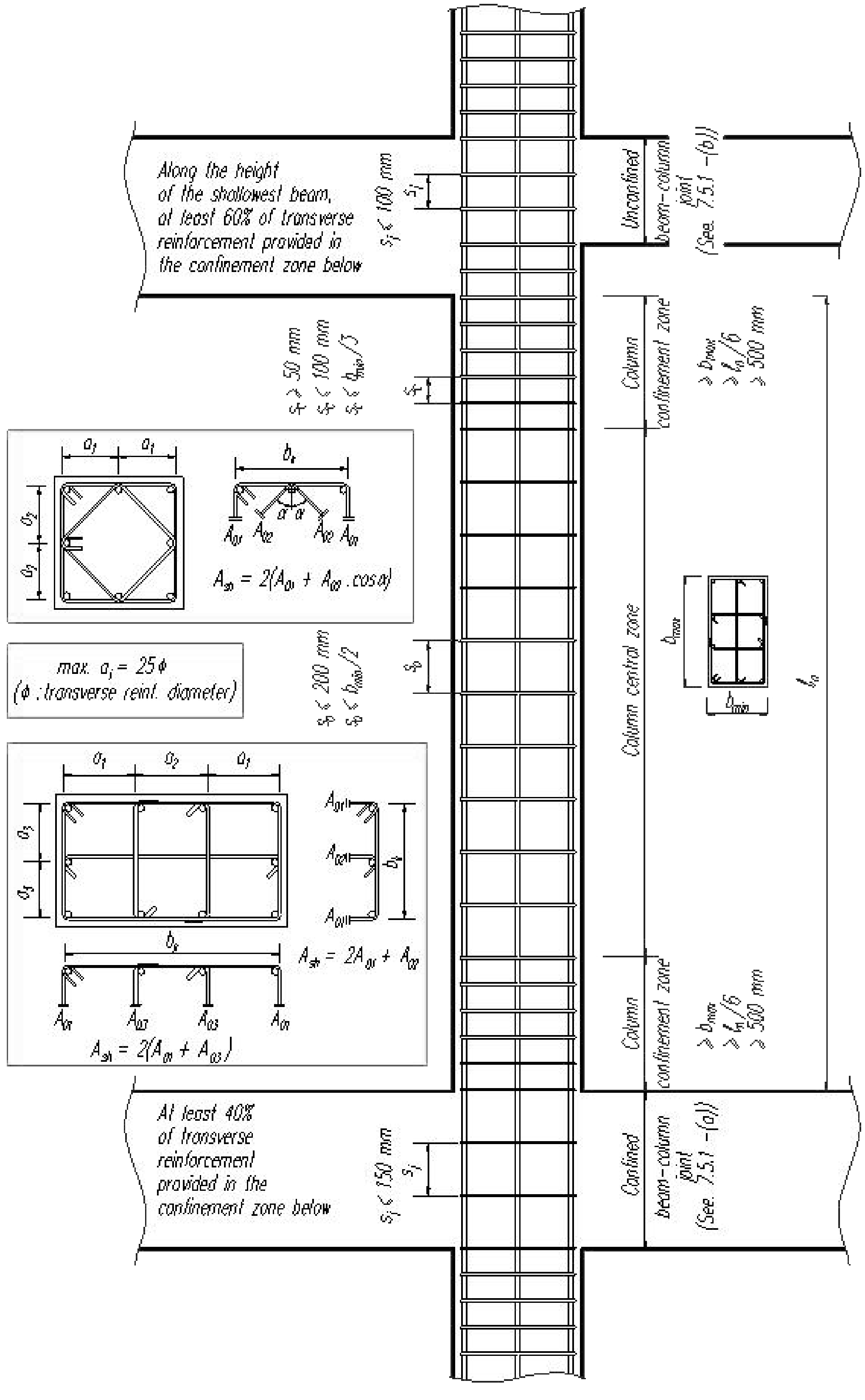


Figure 2.6: Column confinement zones and detailing requirements TEC (1998)

2.3.2 Irregular Buildings

Regarding the definition of irregular buildings whose design and construction should be avoided because of their unfavourable seismic behaviour, types of irregularities in plan and in elevation are given in Table 2.6a and Table 2.6b

Table 2.6a: Irregularities in Plan TEC (1998)

A – IRREGULARITIES IN PLAN
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 story drift at any story to the average story drift at the same story in the same direction.
Floor Discontinuities: **I -** The case where the total area of the openings including those of stairs and elevator shafts exceeds 1/3 of the gross floor area, **II** – The cases where local floor openings make it difficult the safe transfer of seismic loads to vertical structural elements, **III** – The cases of abrupt reductions in the in-plane stiffness and strength of floors.
Projections in Plan: The cases where projections beyond the re-entrant corners in both of the two principal directions in plan exceed the total plan dimensions of the building in the respective directions by more than 20%.
Nonparallel Axes of Structural Elements: The cases where the principal axes of vertical structural elements in plan are not parallel to the orthogonal earthquake directions considered.

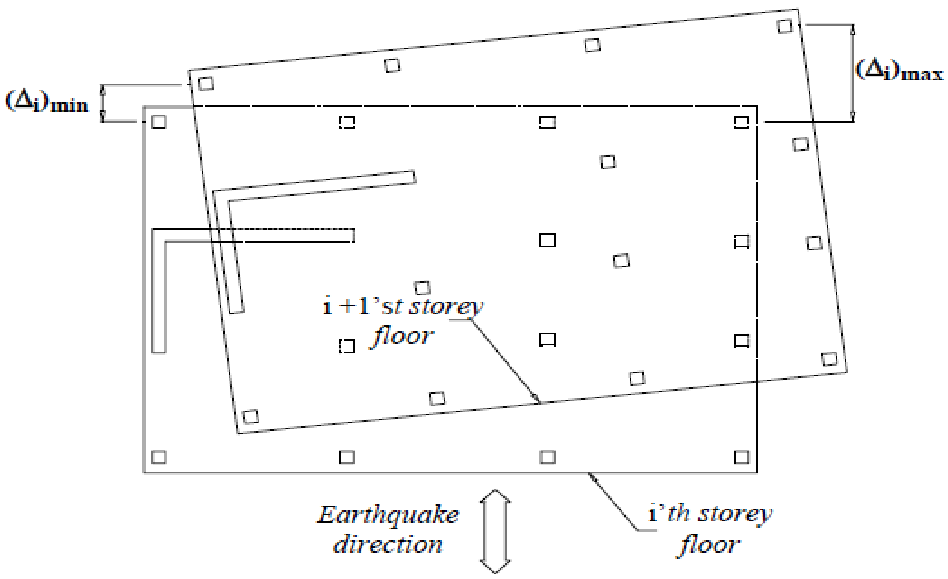


Figure 2.7: Torsional Irregularity TEC (1998).

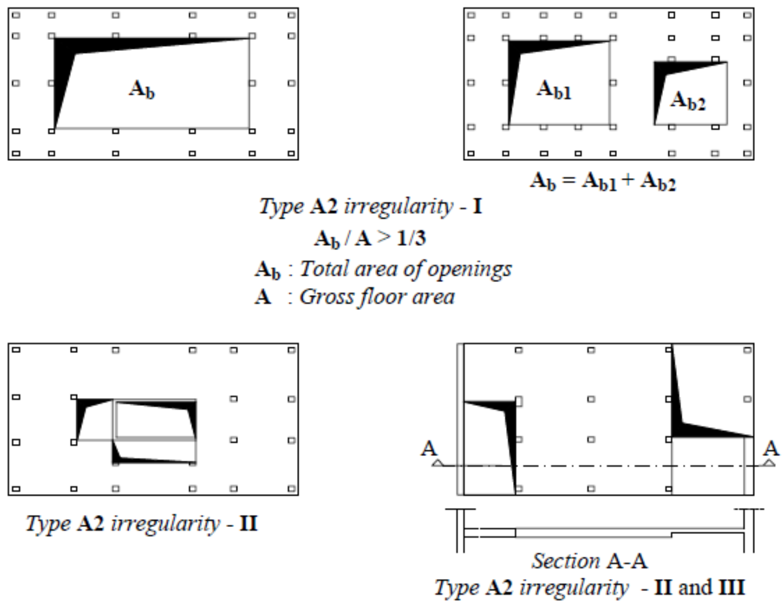


Figure 2.8: Floor Discontinuities TEC (1998).

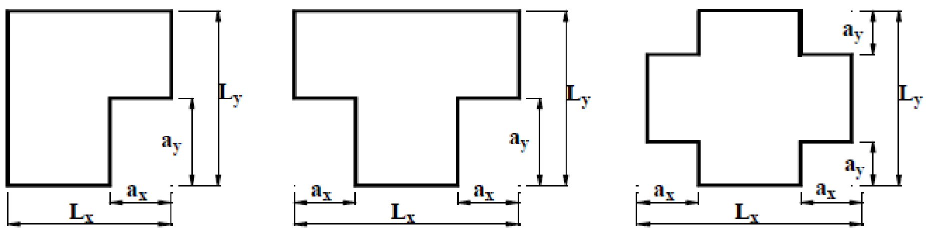


Figure 2.9: Projections in plan TEC (1998).

$$a_x > 0.2\ L_x \tag{2.9.a}$$
$$a_y > 0.2\ L_y \tag{2.9.b}$$

Where;
L_x, L_y : Length of the building at x, y direction
a_y, a_x : Length of re-entrant corners in x, y direction

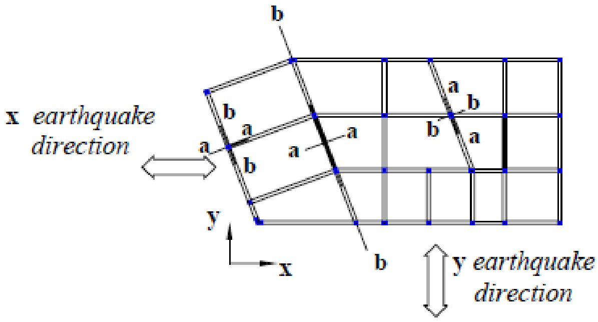


Figure 2.10: Nonparallel Axes of Structural Elements TEC (1998).

In buildings where type A4 irregularity exists, internal forces along the principal axes of structural elements shall be determined in accordance with equation 2.10:

$$B_a = \pm B_{ax} \pm 0.30\ B_{ay} \qquad (2.10a)$$
$$B_a = \pm 0.30\ B_{ax} \pm B_{ay} \qquad (2.10b)$$

The above operations shall be performed for both axis a and perpendicular axis b, by considering both x and y earthquake directions and senses to yield the most unfavourable results.

In the case where principle axes of some of the structural elements are not parallel to the orthogonal earthquake directions, directional combination rule shall be applied additionally to the internal forces of such elements combined in accordance with the following:

Rules to be applied for the statistical combination of non-simultaneous maximum contributions of response quantities calculated for each vibration mode, such as the base shear, storey shear, internal force components, displacement and storey drift, are specified in the following provided that they are applied independently for each response quantity:

- In the cases where natural periods of any two-vibration mode with $T_s < T_r$ always satisfy the condition $T_s / T_r < 0.80$, Square Root of Sum of Squares

(SRSS) Rule may be applied for the combination of maximum modal contributions.

- In the cases where the above given condition is not satisfied, Complete Quadratic Combination (CQC) Rule shall be applied for the combination of maximum modal contributions. In the calculation of cross correlation coefficients to be used in the application of the rule, modal damping factors shall be taken as 5% for all modes.

Table 2.6b: Irregularities in Elevation TEC (1998)

B- IRREGULARITIES IN ELEVATION
Interstory Strength Irregularity (*Weak Story*): In reinforced concrete buildings, the case where in each of the Orthogonal earthquake directions, *Strength Irregularity Factor* η_{ci} which is defined as the ratio of the *effective shear area* of any storey to the *effective shear area* of the storey immediately above, is less than 0.80. [$\eta_{ci} = (\Sigma Ae)i / (\Sigma Ae)i+1 < 0.80$]. *Definition of effective shear area in any story:* $\Sigma Ae = \Sigma Aw + \Sigma Ag + 0.15\ \Sigma Ak$
Interstory Stiffness Irregularity (*Soft Story*): 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 story drift at any story to the average storey drift at the story immediately above, is greater than 1.5. [$\eta_{ki} = (\Delta i)ort / (\Delta i+1)ort > 1.5$]

2.3.3 Materials

Concrete with strength less than C20 should not be utilized. In all seismic zones, it is important to use concrete produced with concrete quality control requirements mentioned in TS–500.

Ribbed bars and stirrups can be utilized with a strength of lower of S420 and making sure the rupture strain of reinforcement to be exceeding 10 % satisfying both of the conditions given by equation 2.11 and as pre-stressing steel in prefabricated buildings.

$$\sum A_g / \sum A_p \geq 0.002$$
$$Vt / \sum A_g \leq 0.5\, f_{ctd} \qquad (2.11)$$

Where;
A_g : Gross section area of column.

A_p : Plane area of story building.
V_t : Total seismic load acting on the structure.
f_{ctd} : Design tensile strength of concrete.

2.3.4 Soil Groups and Local Sites Classes

Soil groups and local site classes to be considered as the bases of determination of local soil conditions are given in Table 2.7 and Table 2.8, respectively.

Table 2.7: Soil Groups TEC (1998)

Soil group	**Description of soil group**	**Standard penetration (N/30)**	**Relative density (%)**	**Unconfined compressive strength (KPa)**	**Drift wave velocity (m/s)**
(A)	1. Huge volcanic stones, metamorphic stones, rigid cemented sedimentary stones	-	-	>1000	>1000
	2. Highly compressed sand, pebbles	>50	85-100	-	>700
	3. Hard clay and silty clay	>32	-	>400	>700
(B)	1. Soft volcanic stones like tuff and agglomerate, weathered cemented sedimentary stones with planes of discontinuity	-	-	500-1000	700-1000
	2. Compressed sand, pebbles	30-50	65-85	-	400-700
	3. Highly rigid clay, silty clay…	16-32	-	200-400	300-700
(C)	**1.** Highly weathered soft metamorphic rocks and cemented sedimentary rocks with planes of discontinuity	-	-	<500	400-700
	2. mildly compressed sand and pebbles	10-30	35-65	-	200-400
	3. Rigid clay and silty clay	-	-	100-200	200-300
(D)	**1.** Soft, deep alluvial layers with high ground water level	-	-	-	<300

	2. Loose sand	<10	<35	-	<200
	3. Soft clay and silty clay	<8	-	<100	<200

Table 2.8: Local Site Classes TEC (1998)

Local Site Class	***Soil Group according to Table 2.7 and Topmost Soil Layer Thickness* (h$_1$)**
Z1	Group **(A)** soils Group **(B)** soils with $h_1 \leq 15$ m
Z2	Group **(B)** soils with $h_1 > 15$ m Group **(C)** soils with $h_1 \leq 15$ m
Z3	Group **(C)** soils with $15 \text{ m} < h_1 \leq 50$ m Group **(D)** soils with $h_1 \leq 10$ m
Z4	Group **(C)** soils with $h_1 > 50$ m Group **(D)** soils with $h_1 > 10$ m

2.4 2007 Turkish Earthquake Code

After the disastrous earthquakes that happened in 1999, officials responsible took actions to evaluate buildings regarding seismic safety and retrofit some. Although the inadequacy of regulations regarding about seismic safety assessment and retrofitting made the design engineers life harder since they had no basis to rely and thus resulting in an inappropriate approach towards the matter. Therefore, the 2007 Turkish Earthquake Code was released emphasizing the matter of seismic assessment and retrofitting of existing buildings. The 2007 code has minor changes related to new reinforced concrete buildings. On the hand, the seismic safety regulations for steel structures are explained explicitly.

The significant changes in the code are:

- Involvement of a lengthy chapter on seismic safety assessment and retrofitting
- Involvement of a linear elastic method for seismic safety assessment considering the inelastic behaviour in terms of approximate allowable demand/capacity ratios given depending on the damage level

- Involvement of different levels of design earthquakes and performance levels to be considered for different structures
- Involvement of push-over analysis for seismic safety assessment
- Involvement of nonlinear time history analysis

As mentioned above, the 2007 Turkish Earthquake code only has minor revisions from the 1998 Turkish Earthquake code, so it’s worth to mention those minor changes.

2.4.1 Geometry and Detailing Requirements

Almost the same geometry and detailing requirements that are used in TEC-1998 are used in TEC-2007 with only this minor change in the volumetric ratio of transverse reinforcement equation:

$$\rho \geq 0.8\,\frac{fctd}{f_y} \qquad (2.12)$$

2.4.2 Structure Behaviour Factors

Structural System Behaviour Factors are given in Table 2.9, to determine structures with high ductility and normal ductility with new values.

Table 2.9: Structural System Behaviour 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 solid and / or coupled structural 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 **(2.2)** Single-story buildings in which seismic loads are fully resisted by columns with hinged upper connections **(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. **(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 - - 3	7 3 5 6

2.4.2 Irregular Buildings

Same definitions as in the TEC-1998 but the minor difference is how the TEC-2007 deal with few irregularities:

- Weak Storey: TEC-2007 states, when the value of $\eta ci < 0.8$ the behaviour factor should be multiplied by 1.25. While in TEC-1998 it states that: when the value of $\eta ci < 0.8$ the behaviour factor should be multiplied by 1.2.
- Torsional Irregularity: TEC-2007 states that the eccentricity should be multiplied by a factor $D_i = (\eta_{bi} / 1.2)^{0.5}$. While in TEC-1998 it states: multiply the eccentricity value by a factor $D_i = (\eta_{bi} / 1.2)^{2.}$
- Soft Storey: The cases where the ratio of the average floor drift at any floor to the average floor drift at the floor located directly atop or beneath, at each of the two-orthogonal direction of the earthquake under study, is higher than 2.0. (in TEC-1998 it was said to be soft storey if it was only higher than 1.5)
- A4 irregularity is no longer mentioned in TEC-2007. The rules that were related to that type of irregularity have been generalised for all cases.

Internal forces in Element Principal Axes:

Under the combined effects of independently acting x and y direction earthquakes to the structural system, internal forces in element principal axes a and b shall be obtained by equation 2.13 such that the most unfavourable results yield Figure. 2.11.

$$B_a = \pm B_{ax} \pm 0.30\, B_{ay} \quad \text{or} \quad B_a = \pm 0.30\, B_{ax} \pm B_{ay} \tag{2.13a}$$
$$B_b = \pm B_{bx} \pm 0.30\, B_{by} \quad \text{or} \quad B_b = \pm 0.30\, B_{bx} \pm B_{ay} \tag{2.13b}$$

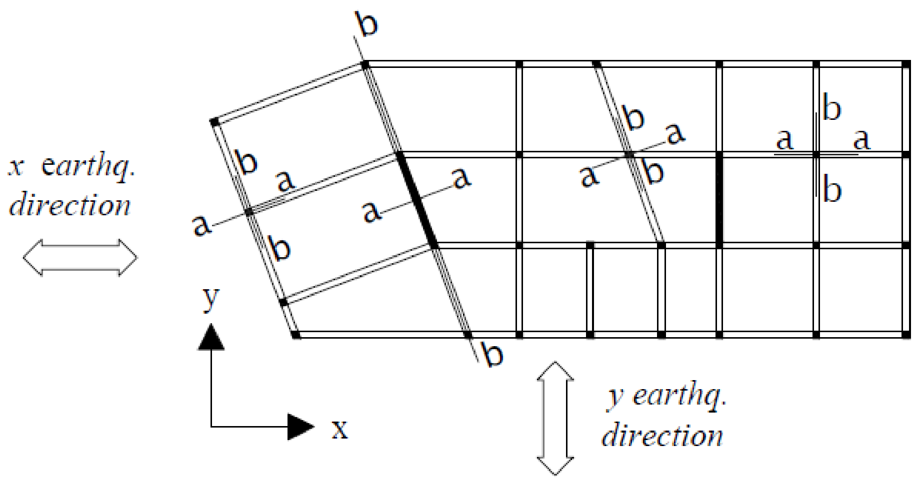


Figure 2.11: Internal forces in Element Principal Axes

Under the combined effects of independently acting x and y direction earthquakes to the structural system, the directional combination rule shall be additionally applied to the internal forces obtained in element principal axes a and b by modal combination according to the following:

Rules to be applied for the statistical combination of non-simultaneous maximum contributions of response quantities calculated for each vibration mode, such as the base shear, storey shear, internal force components, displacements and storey drifts, are specified in the following provided that they are applied independently for each response quantity:

- In the cases where natural periods of any two-vibration mode with $T_m < T_n$ always satisfy the condition $T_m / T_n < 0.80$, Square Root of Sum of Squares (SRSS) Rule may be applied for the combination of maximum modal contributions.

- In the cases where the above given condition is not satisfied, Complete Quadratic Combination (CQC) Rule shall be applied for the combination of maximum modal contributions. In the calculation of cross correlation coefficients to be used in the application of the rule, modal damping factors shall be taken as 5% for all modes.

2.4.3 Life Safety Performance Level

The buildings that satisfy the conditions mentioned below can be agreed to be in Life Safety Performance Level provided that the brittle damaged components, if any, are strengthened:

- As the result of the calculations made for each earthquake direction applies on each floor, at most 30 % of the beams except for the secondary ones (that does not take place in the horizontal load-bearing system) and at most the proportion of the columns defined in the next point can exceed the Advanced Damage Zone.
- The total contribution of the columns in the Advanced Damage Zone to the shear force that is borne by the columns in each floor should not exceed 20 %. For the top floor, the ratio of the total shear forces of the columns in the Significant Damage Zone to the total shear forces of all the columns at that floor can be at most 40 %.
- All other load - bearing components are in Minimum Damage Zone or Significant Damage Zone. However, the shear forces borne by the columns which exceeds the Minimum Damage Bound both in upper and lower sections for any floor should not be more than 30 % of the shear force borne by all columns of the floor.

The analysis methods and details about structure performance levels are discussed in details in Chapter 3.

2.5 Comparison between the 1975, 1998, and 2007 Turkish Earthquake Codes

Throughout the years The Turkish Earthquake codes have changed and improved to match the geographic location of the country. At first, codes were basic and not concerned in the seismic activity in the Anatolian Fault. The main improvement started in the 1975 code where the concept of ductility was mentioned for the first time at structural levels. The base shear force was also given as a function of structural ductility for the lateral load resisting system. The 1975 was valid for more than 20 years and many building stocks up to this day are still available with those rules and regulations of the code. The 1975 was a major improvement to previous codes because it included more detailed principles related to seismic-resistant detailing along with minimum reinforcements ratios for structural members. It also introduced irregularities in buildings although definitions weren't that detailed. Moreover, it talked about considering an additional eccentricity of 5% of the largest plan dimension of the design. But after over 20 years the 1998 Turkish Earthquake Code came out with improved formulas and regulations to maintain safer structures regarding seismic activity. The 1998 Code introduced the definition of design earthquake, acceptable structural performance under the design earthquake, elastic design spectrum, seismic load reduction load factor, and detailed definition of irregularities. Although those improvements were huge but weren't enough regarding the safety of the structures in the seismic activity areas especially after the 1999 earthquakes there were experienced, so the 2007 Turkish Earthquake Code was introduced. The 2007 Code include the issues on seismic safety assessment of existing buildings. It also has minor improvements related to newly designed reinforced concrete buildings from the 1998 Code. However, the seismic safety requirements for steel structures were thoroughly discussed in the 2007 Code, unlike old versions. It also introduced performance levels and different levels of design earthquakes. New analysis types were discussed to determine seismic safety assessment and retrofitting which include: Non-linear static push-over analysis and nonlinear time history analysis.

In summary, the Turkish Earthquakes Codes has developed and improved throughout the years to assess safety regarding seismic activity and disasters that the country has countered in the past. The 2007 Code is being used and referred to all infrastructures now in Turkey and Northern part of Cyprus because of how it addresses the issues and safety measurement.

Chapter 3

EARTHQUAKE ANALYSIS METHODS

3.1 Introduction

Methods to be used for the seismic analysis of buildings and building-like structures are, Equivalent Static Analysis, Linear Dynamic (Response Spectrum) Analysis, Nonlinear Static Pushover Analysis. These types analysis helps in the understanding of how structures behave under earthquakes. Nonlinear Static Pushover Analysis is chosen for this study because of its reliability for the design and evaluation of low rise buildings.

3.2 Equivalent Static Analysis

Equivalent static analysis comes handy when dealing with a displacement controlled structure which causes the natural frequencies of variation to be higher than the usual. Its use allows fast development of foundation loading and it also gives information about the final stiffness of the structure, Bourahla (2013).

3.3 Linear Dynamic (Response Spectrum) Analysis

For design purposes, response spectra serve as a common seismic analysis. It has the ability to cut through time and proved only the maximum response without really explaining it. A response spectrum is simply the diagram resulting from independent variable as the natural variation frequencies of a system and the dependant variable as the equivalent maximum response values, Chandak (2012).

3.4 Nonlinear Static Pushover Analysis

Nonlinear static analysis is the most used method to get the seismic performance of structures. This method is based on meeting the lateral force carrying capacity with the earthquake demand and to find the performance point of the related structure. In this analysis method material and geometric nonlinearities can be used to perform the nonlinear response of structures, CSI (2009).

3.4.1 Nonlinear Static Pushover Analysis According to TEC-2007

It is required that the effective mass calculated by considering first natural vibration mode of considered earthquake direction to total building mass shall not be less than 0.70. In addition, number of stories shall not be more than eight excluding the basement. Otherwise Incremental Equivalent Seismic Load Method can't be applied to the structural system.

In Incremental Equivalent Seismic Load Method, performance point of building is represented with base shear-roof displacement curve and modal capacity diagram. Roof displacement is the displacement calculated in each pushover step in x earthquake direction considered at center of mass at the top story of the building. Base shear force is the sum of equivalent earthquake loads in each step in x earthquake direction. Structural system is calculated under vertical loads and proportionally increasing earthquake loads to obtain pushover curve until the performance point is reached.

Modal capacity diagram obtained at the end of pushover analysis and elastic response spectrum are taken into consideration together.

Spectral displacement ratio, C_{R1}=1, in case of initial period $T_1^{(1)}$ is equal to or greater than T_B that is the characteristic period at acceleration spectrum. ($T_1^{(1)} \geq T_B$ or $(w_1^{(1)})^2 \leq w_B^2$).

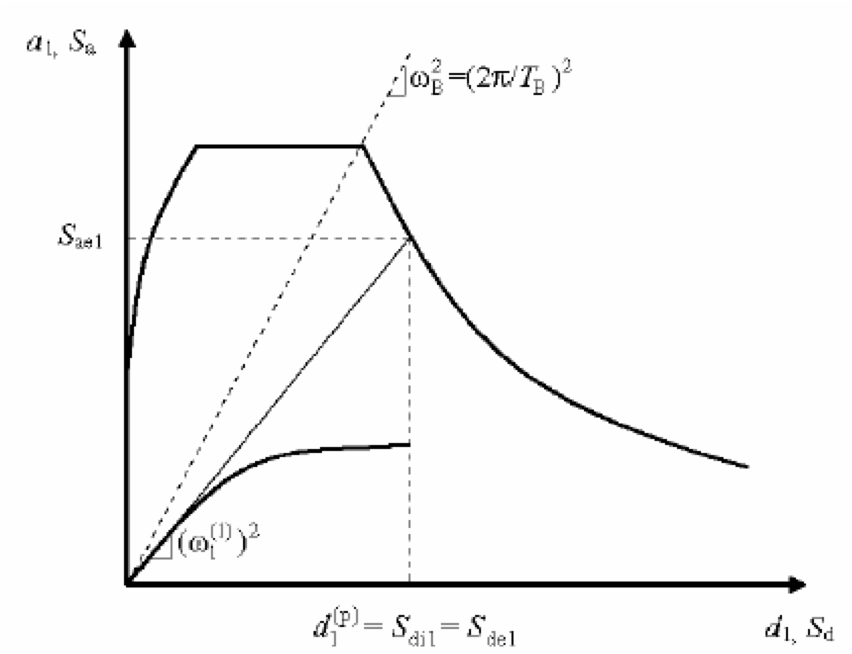


Figure 3.1: Determination of Performance Point $T_1^{(1)}) \geq T_B$. TEC-2007 (2007)

If Spectral displacement ratio, C_{R1}, in case of initial period $T_1^{(1)}$ less than T_B, then its calculated in the following in method:

1. Modal capacity diagram obtained at the end of pushover analysis is converted to a bi-linear diagram. In this diagram, the slope of the beginning line is taken as equivalent to value, $(w_1^{(1)})^2$ corresponding to the first mode the angle of line in first step (i=1) of pushover analysis ($T_1^{(1)} = 2\pi / w_1^{(1)}$).
2. In the first step of successive approximation method it is assumed that C_{R1}=1 and coordinates of equivalent yield point is determined by using equivalent areas rule, as shown in Figure 3.2.

3. Coordinates of equivalent yield point is determined again by using equivalent areas rule. Successive approximation method is completed when the results of two adjacent steps are approximately same, as shown in Figure 3.3.

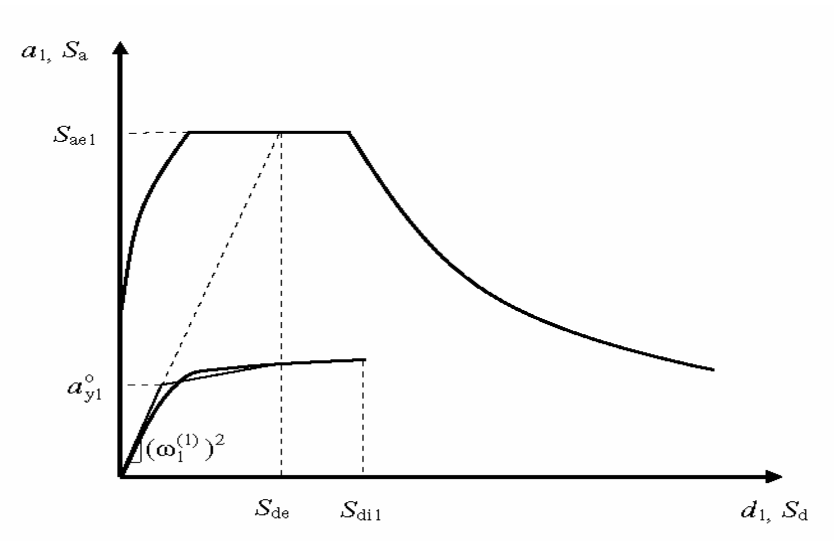


Figure 3.2: Determination of Performance Point $T_1^{(1)} < T_B$. TEC-2007 (2007)

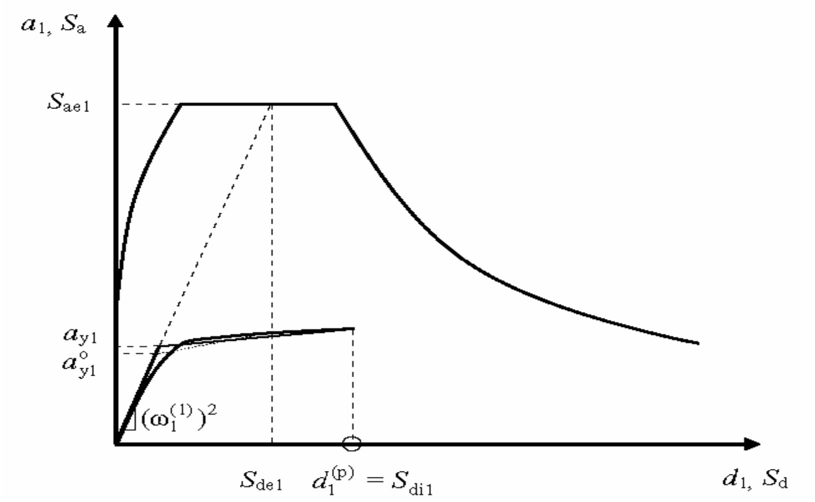


Figure 3.3: Determination of Performance Point $T_1^{(1)} < T_B$. TEC-2007 (2007)

3.5 Structure Performance Levels

The performance of a structure is directly linked to the damage level likely to appear in the structure under the influence of earthquake. Four categories of performance level are determined as show in Figure 3.4

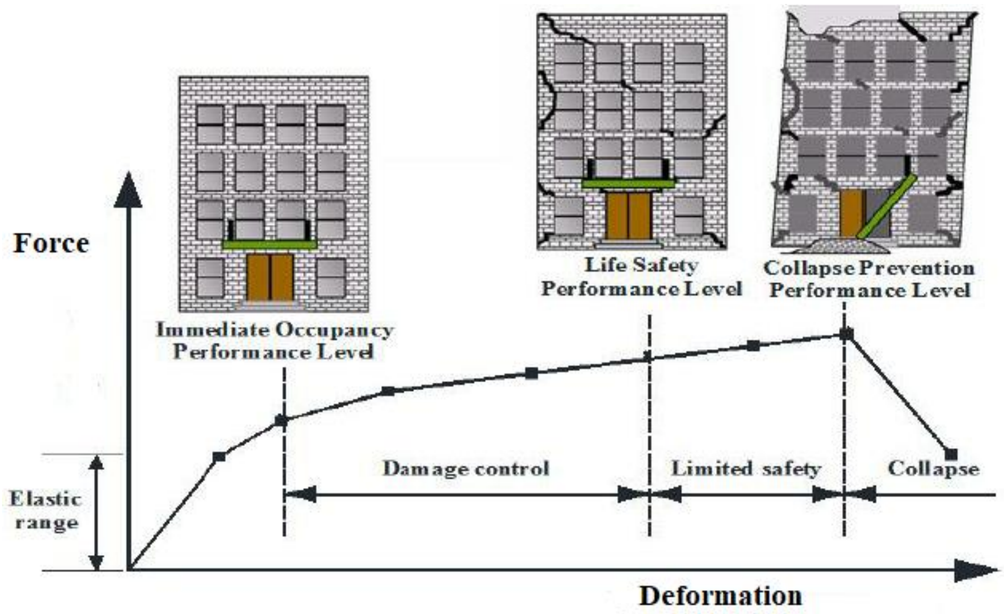


Figure 3.4: Performance Levels. (Abd-Elhamed, & Sayed 2012)

3.5.1 Immediate Occupancy Category (IO)

The building is classified in the Immediate Occupancy category if less than 10% of the beams in it exceed the Advanced Damage Zone while other assets of the building stay in the Minimum Damage Zone.

3.5.2 Life Safety Category (LS)

To achieve Life Safety category, building must meet the following:

- The damage of beams in any floor should be less than 30% in Marked Damage and in Advanced Damage region.
- The shear load supported by columns in the Advanced Damage region, should be less than 30% of the storey shear in any floor.

3.5.3 Collapse Prevention Category (CP)

To achieve Collapse Prevention Category, building must meet the following:

- Less than 20% of the total beams can be in the Collapse region, not counting the secondary ones
- The shear load supported by columns in the Minimum Damage region should not exceed 30% of the storey shear in any floor.

3.5.4 Collapse Category (C)

If the building fails to meet the conditions stated in the Collapse Prevention Category, then it falls in the Collapse Category.

Chapter 4

CASE STUDIES

4.1 Introduction

In this study, several case studies were designed according to TEC-2007, TEC-1998, and TEC-1975. Four types of irregular buildings were chosen along with 2 regular buildings and investigated upon, with an elevation of 5 and 3 stories. STA4CAD computer software was used for designing and performing the nonlinear static pushover analysis. Information about the buildings' geometry and the TEC-2007, TEC-1998, and TEC-1975 parameters for the seismic design chosen in this study are presented in this chapter.

4.2 Case Studies

4.2.1 Case Study 1 (Regular Building 1)

Typical building type with no irregularities found in the check.

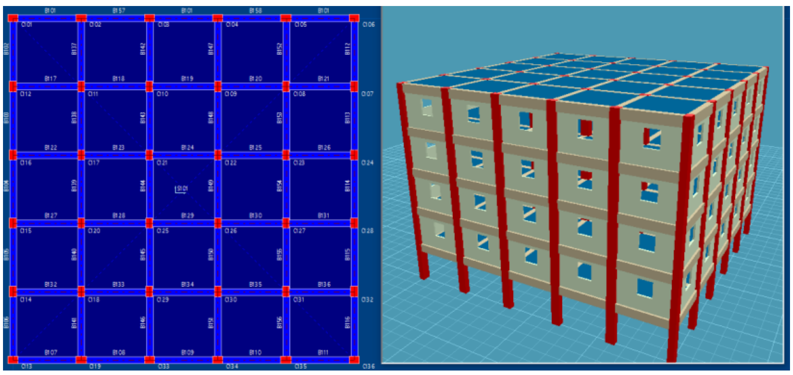


Figure 4.1: Two-Dimensional Plan & Three-Dimensional View of Case 1 (Regular Building 1).

Table 4.1: Case 1 (Regular Building 1) Building Specifications.

Case	**Case 1 (Regular Building 1)**	
Floor number	5F (15m)	3F (9m)
Columns	60x40 cm^2	40x30 cm^2
Beams	40x60 cm^2	30x40 cm^2
Slab	20 cm	20 cm

Report for Case 1 3F irregularity check:

```
IRREGULARITY CHECK UNDER SEISMIC ACTION
A1,B2  type irregularities
max(di/hi)=0.02  PDELTA ANALYSIS WITH CRACK SECTION. STORY DISPLACEMENTS IS MULTIPLYING WITH 0.4
1. sto X dtop = 0.0013513 + -0.0000001 × (.0 - 10.0)=0.0033806  (C101)
1. sto X dbot = 0 + -0.0000001 × (20.0 - 10.0)=0.0033757  (C113)
2. sto X top  = 0.0025824 + -0.0000002 × (.0 - 10.0) - 0.0033806 = 0.0030796  (C201)
2. sto X dbot = 0.0025824 + -0.0000002 × (20.0 - 10.0) - 0.0033757 = 0.003076  (C213)
```

X DIR. (+%5)

Story	ΔX dtop(m)	ΔX dbot(m)	ΔX ort	nbi	nki	R·Δx/h	θi	story type
3	0.0005042	0.0005041	0.0005042	1.00	0.00	0.00134	0.00182	Normal sto
2	0.0012318	0.0012304	0.0012311	1.00	1.44	0.00328	0.00507	Normal sto
1	0.0013522	0.0013503	0.0013513	1.00	1.10	0.00361	0.00662	Normal sto

X DIR. (-%5)

Story	ΔX dtop(m)	ΔX dbot(m)	ΔX ort	nbi	nki	R·Δx/h	θi	story type
3	0.0005042	0.0005041	0.0005042	1.00	0.00	0.00134	0.00182	Normal sto
2	0.0012318	0.0012304	0.0012311	1.00	1.44	0.00328	0.00507	Normal sto
1	0.0013522	0.0013503	0.0013513	1.00	1.10	0.00361	0.00662	Normal sto

Y DIR. (+%5)

Story	ΔY dlft(m)	ΔY drgt(m)	ΔY ort	nbi	nki	R·Δy/h	θi	story type
3	0.0006092	0.0006093	0.0006093	1.00	0.00	0.00162	0.00250	Normal sto
2	0.0014467	0.0014474	0.0014470	1.00	1.38	0.00386	0.00681	Normal sto
1	0.0017205	0.0017214	0.0017210	1.00	1.19	0.00459	0.00957	Normal sto

Y DIR. (-%5)

Story	ΔY dlft(m)	ΔY drgt(m)	ΔY ort	nbi	nki	R·Δy/h	θi	story type
3	0.0006092	0.0006093	0.0006093	1.00	0.00	0.00162	0.00250	Normal sto
2	0.0014467	0.0014474	0.0014470	1.00	1.38	0.00386	0.00681	Normal sto
1	0.0017205	0.0017214	0.0017210	1.00	1.19	0.00459	0.00957	Normal sto

```
TDY 2.3.2.1 A1 torsional irregularity:
nbi=1.001 <1.2 , ✓
TDY 2.19 requirement OK         .0046 < .02 ✓
TDY 2.20 requirement OK          max θi=.01 < 0.12 ✓
```

B1-Vertical irregularity check

Story	Aw	Agx	Agy	Akx	Aky	Σ Aex	Σ Aey	ncix	nciy	EXPLANATION
3	4.32	0.00	0.00	3.90	3.90	4.91	4.91	1.00	1.00	top sto ✓
2	4.32	0.00	0.00	3.90	3.90	4.91	4.91	1.00	1.00	Regular ✓
1	4.32	0.00	0.00	0.00	0.00	4.32	4.32	0.88	0.88	Regular ✓

Rerport for Case 1 5F irregularity check:

```
IRREGULARITY CHECK UNDER SEISMIC ACTION
    A1,B2  type irregularities
    max(di/hi)=0.02  PDELTA ANALYSIS WITH CRACK SECTION. STORY DISPLACEMENTS IS MULTIPLYING WITH 0.4
    1. sto X dtop = 0.0012935 + 0 × (.0 - 10.0)=0.0032338  (C127)
    1. sto X dbot = 0 + 0 × (20.0 - 10.0)=0.0032338  (C131)
    2. sto X top  = 0.0031792 + 0 × (.0 - 10.0) - 0.0032338 = 0.0047141  (C227)
    2. sto X dbot = 0.0031792 + 0 × (20.0 - 10.0) - 0.0032338 = 0.0047141  (C231)
```

X DIR. (+%5)

Story	ΔX dtop(m)	ΔX dbot(m)	ΔX ort	nbi	nki	R·Δx/h	θi	story type
5	0.0006987	0.0006987	0.0006987	1.00	0.00	0.00186	0.00255	Normal sto
4	0.0012420	0.0012420	0.0012420	1.00	1.78	0.00331	0.00553	Normal sto
3	0.0016858	0.0016857	0.0016858	1.00	1.36	0.00450	0.00868	Normal sto
2	0.0018856	0.0018856	0.0018856	1.00	1.12	0.00503	0.01126	Normal sto
1	0.0012935	0.0012935	0.0012935	1.00	0.69	0.00345	0.00912	Normal sto

X DIR. (-%5)

Story	ΔX dtop(m)	ΔX dbot(m)	ΔX ort	nbi	nki	R·Δx/h	θi	story type
5	0.0006987	0.0006987	0.0006987	1.00	0.00	0.00186	0.00255	Normal sto
4	0.0012420	0.0012420	0.0012420	1.00	1.78	0.00331	0.00553	Normal sto
3	0.0016858	0.0016857	0.0016858	1.00	1.36	0.00450	0.00868	Normal sto
2	0.0018856	0.0018856	0.0018856	1.00	1.12	0.00503	0.01126	Normal sto
1	0.0012935	0.0012935	0.0012935	1.00	0.69	0.00345	0.00912	Normal sto

Y DIR. (+%5)

Story	ΔY dlft(m)	ΔY drgt(m)	ΔY ort	nbi	nki	R·Δy/h	θi	story type
5	0.0006494	0.0006493	0.0006493	1.00	0.00	0.00173	0.00209	Normal sto
4	0.0010927	0.0010927	0.0010927	1.00	1.68	0.00291	0.00429	Normal sto
3	0.0014523	0.0014522	0.0014523	1.00	1.33	0.00387	0.00659	Normal sto
2	0.0015630	0.0015629	0.0015630	1.00	1.08	0.00417	0.00823	Normal sto
1	0.0009773	0.0009772	0.0009772	1.00	0.63	0.00261	0.00607	Normal sto

Y DIR. (-%5)

Story	ΔY dlft(m)	ΔY drgt(m)	ΔY ort	nbi	nki	R·Δy/h	θi	story type
5	0.0006494	0.0006493	0.0006493	1.00	0.00	0.00173	0.00209	Normal sto
4	0.0010927	0.0010927	0.0010927	1.00	1.68	0.00291	0.00429	Normal sto
3	0.0014523	0.0014522	0.0014523	1.00	1.33	0.00387	0.00659	Normal sto
2	0.0015630	0.0015629	0.0015630	1.00	1.08	0.00417	0.00823	Normal sto
1	0.0009773	0.0009772	0.0009772	1.00	0.63	0.00261	0.00607	Normal sto

```
TDY 2.3.2.1 A1 torsional irregularity:
nbi=1 <1.2    Seism. ecc.= % 5 used and code requirement OK ✓
TDY 2.3.2.1 B2  condition is not satisfied.  ✓
TDY 2.19 requirement OK          .005 < .02 ✓
TDY 2.20 requirement OK           max θi=.011 < 0.12 ✓
```

B1-Vertical irregularity check

Story	Aw	Agx	Agy	Akx	Aky	Σ Aex	Σ Aey	ncix	nciy	EXPLANATION
5	6.48	0.00	0.00	15.60	15.60	8.82	8.82	1.00	1.00	top sto ✓
4	6.48	0.00	0.00	15.60	15.60	8.82	8.82	1.00	1.00	Regular ✓
3	6.48	0.00	0.00	15.60	15.60	8.82	8.82	1.00	1.00	Regular ✓
2	6.48	0.00	0.00	15.60	15.60	8.82	8.82	1.00	1.00	Regular ✓
1	6.48	0.00	0.00	0.00	0.00	6.48	6.48	1.00	1.00	Regular ✓

4.2.2 Case Study 2 (Regular Building 2)

Typical building type with no irregularities found in the check.

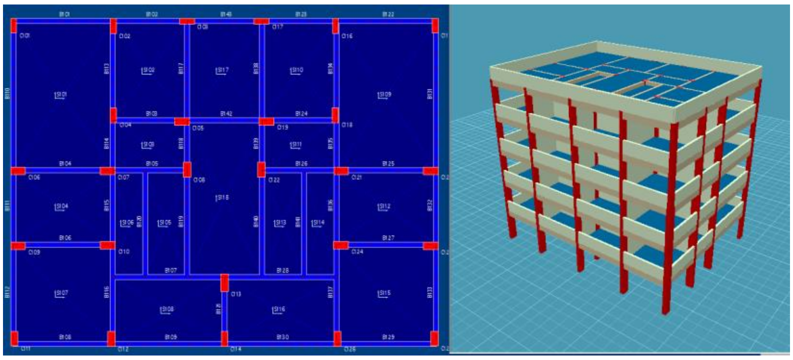


Figure 4.2: Two-Dimensional Plan & Three-Dimensional View of Case 2 (Regular Building 2).

Table 4.2: Case 2 (Regular Building 2) Building Specifications.

Case	**Case 2 (Regular Building 2)**	
Floor number	5F (15m)	3F (9m)
Columns	30x50 cm^2	30x40 cm^2
	25x50 cm^2	25x40 cm^2
	30x60 cm^2	30x60 cm^2
Beams	25x50 cm^2	20x40 cm^2
Slab	20 cm	20 cm

Report for Case 2 3F irregularity check:

```
IRREGULARITY CHECK UNDER SEISMIC ACTION
    A1,B2  type irregularities
    max(di/hi)=0.02  PDELTA ANALYSIS WITH CRACK SECTION. STORY DISPLACEMENTS IS MULTIPLYING WITH 0.4
    1. sto X dtop = 0.0021297 + 0.0000164 × (.1 - 6.21)=0.0050729  (CZ01)
    1. sto X dbot = 0 + 0.0000164 × (12.91 - 6.21)=0.0055992  (CZ12)
    2. sto X top  = 0.0043856 + 0.000033 × (.03 - 6.22) - 0.0050698 = 0.0053834  (C103)
    2. sto X dbot = 0.0043856 + 0.000033 × (12.91 - 6.22) - 0.0055992 = 0.0059153  (C112)

    X DIR.  (+%5)
```

Story	ΔX dtop(m)	ΔX dbot(m)	ΔX ort	nbi	nki	R·Δx/h	θi	story type
3	0.0008183	0.0009345	0.0008764	1.07	0.00	0.00249	0.00358	Normal sto
2	0.0021534	0.0023661	0.0022597	1.05	1.58	0.00631	0.01172	Normal sto
1	0.0020292	0.0022397	0.0021344	1.05	0.94	0.00597	0.01398	Normal sto

X DIR. (-%5)

Story	ΔX dtop(m)	ΔX dbot(m)	ΔX ort	nbi	nki	R·Δx/h	θi	story type
3	0.0008183	0.0009345	0.0008764	1.07	0.00	0.00249	0.00358	Normal sto
2	0.0021534	0.0023661	0.0022597	1.05	1.58	0.00631	0.01172	Normal sto
1	0.0020292	0.0022397	0.0021344	1.05	0.94	0.00597	0.01398	Normal sto

Y DIR. (+%5)

Story	ΔY dlft(m)	ΔY drgt(m)	ΔY ort	nbi	nki	R·Δy/h	θi	story type
3	0.0009511	0.0008978	0.0009245	1.03	0.00	0.00254	0.00331	Normal sto
2	0.0019594	0.0019972	0.0019783	1.01	1.14	0.00533	0.00899	Normal sto
1	0.0016106	0.0016193	0.0016150	1.00	0.82	0.00432	0.00927	Normal sto

Y DIR. (-%5)

Story	ΔY dlft(m)	ΔY drgt(m)	ΔY ort	nbi	nki	R·Δy/h	θi	story type
3	0.0009511	0.0008978	0.0009245	1.03	0.00	0.00254	0.00331	Normal sto
2	0.0019594	0.0019972	0.0019783	1.01	1.8	0.00533	0.00899	Normal sto
1	0.0016106	0.0016193	0.0016150	1.00	0.82	0.00432	0.00927	Normal sto

```
TDY 2.3.2.1 A1 torsional irregularity:
nbi=1.066 <1.2    Seism. ecc.= % 5 used and code requirement OK ✓
TDY 2.3.2.1 B2 type irregularity found, only response seismic code
TDY 2.19 requirement OK          .0063 < .02 ✓
TDY 2.20 requirement OK           max θi=.014 < 0.12 ✓
```

B1-Vertical irregularity check

Story	Aw	Agx	Agy	Akx	Aky	∑ Aex	∑ Aey	ncix	nciy	EXPLANATION
3	2.35	0.00	0.00	7.60	9.48	3.49	3.77	1.00	1.00	top sto ✓
2	2.54	0.00	0.00	7.60	11.00	3.68	4.19	1.05	1.11	Regular ✓
1	3.12	0.00	0.00	0.00	0.00	3.12	3.12	1.00	1.00	Regular ✓

Report for Case 2 5F irregularity check:

```
max(di/hi)=0.02  PDELTA ANALYSIS WITH CRACK SECTION. STORY DISPLACEMENTS IS MULTIPLYING WITH 0.4
1. sto X dtop = 0.0009988 + 0.000015 × (.03 - 6.17)=0.0022659  (CZ03)
1. sto X dbot = 0 + 0.000015 × (12.81 - 6.17)=0.0027466  (CZ12)
2. sto X top  = 0.0025584 + 0.0000392 × (.03 - 6.08) - 0.0022659 = 0.0035372  (C103)
2. sto X dbot = 0.0025584 + 0.0000392 × (12.86 - 6.08) - 0.0027485 = 0.0043114  (C114)
```

X DIR. (+%5)

Story	ΔX dtop(m)	ΔX dbot(m)	ΔX ort	nbi	nki	R·Δx/h	θi	story type
5	0.0005987	0.0008029	0.0007008	1.15	0.00	0.00214	0.00245	Normal sto
4	0.0010638	0.0014069	0.0012353	1.14	1.76	0.00375	0.00522	Normal sto
3	0.0014186	0.0018115	0.0016150	1.12	1.31	0.00483	0.00788	Normal sto
2	0.0014149	0.0017246	0.0015697	1.10	0.97	0.00460	0.00889	Normal sto
1	0.0009064	0.0010987	0.0010025	1.10	0.64	0.00293	0.00673	Normal sto

X DIR. (-%5)

Story	ΔX dtop(m)	ΔX dbot(m)	ΔX ort	nbi	nki	R·Δx/h	θi	story type
5	0.0005987	0.0008029	0.0007008	1.15	0.00	0.00214	0.00245	Normal sto
4	0.0010638	0.0014069	0.0012353	1.14	1.76	0.00375	0.00522	Normal sto
3	0.0014186	0.0018115	0.0016150	1.12	1.31	0.00483	0.00788	Normal sto
2	0.0014149	0.0017246	0.0015697	1.10	0.97	0.00460	0.00889	Normal sto
1	0.0009064	0.0010987	0.0010025	1.10	0.64	0.00293	0.00673	Normal sto

Y DIR. (+%5)

Story	ΔY dlft(m)	ΔY drgt(m)	ΔY ort	nbi	nki	R·Δy/h	θi	story type
5	0.0008519	0.0008223	0.0008371	1.02	0.00	0.00227	0.00287	Normal sto
4	0.0014108	0.0013602	0.0013855	1.02	1.66	0.00376	0.00574	Normal sto
3	0.0017889	0.0016713	0.0017301	1.03	1.25	0.00477	0.00828	Normal sto
2	0.0015478	0.0015768	0.0015623	1.01	0.90	0.00420	0.00867	Normal sto
1	0.0009627	0.0009735	0.0009681	1.01	0.62	0.00260	0.00637	Normal sto

Y DIR. (-%5)

Story	ΔY dlft(m)	ΔY drgt(m)	ΔY ort	nbi	nki	R·Δy/h	θi	story type
5	0.0008519	0.0008223	0.0008371	1.02	0.00	0.00227	0.00287	Normal sto
4	0.0014108	0.0013602	0.0013855	1.02	1.66	0.00376	0.00574	Normal sto
3	0.0017889	0.0016713	0.0017301	1.03	1.25	0.00477	0.00828	Normal sto
2	0.0015478	0.0015768	0.0015623	1.01	0.90	0.00420	0.00867	Normal sto
1	0.0009627	0.0009735	0.0009681	1.01	0.62	0.00260	0.00637	Normal sto

```
TDY 2.3.2.1 A1 torsional irregularity:
nbi=1.146 <1.2    Seism. ecc.= % 5 used and code requirement OK ✓
TDY 2.3.2.1 B2  condition is not satisfied.  ✓
TDY 2.19 requirement OK          .0048 < .02 ✓
TDY 2.20 requirement OK           max θi=.009 < 0.12 ✓
```

B1-Vertical irregularity check

Story	Aw	Agx	Agy	Akx	Aky	∑ Aex	∑ Aey	ncix	nciy	EXPLANATION
5	3.28	0.00	0.00	7.60	9.48	4.42	4.70	1.00	1.00	top sto ✓
4	3.28	0.00	0.00	7.60	9.48	4.42	4.70	1.00	1.00	Regular ✓
3	3.41	0.00	0.00	7.60	9.48	4.55	4.83	1.03	1.03	Regular ✓
2	4.19	0.00	0.00	7.60	11.00	5.33	5.84	1.17	1.21	Regular ✓
1	4.44	0.00	0.00	0.00	0.00	4.44	4.44	0.83	1.00	Regular ✓

4.2.3 Case Study 3 (Weak Storey)

When the strength irregularity factor η_{ci} is below 0.8 a building is having a weak storey irregularity according to the 1998 and 2007 Turkish Earthquake Codes.

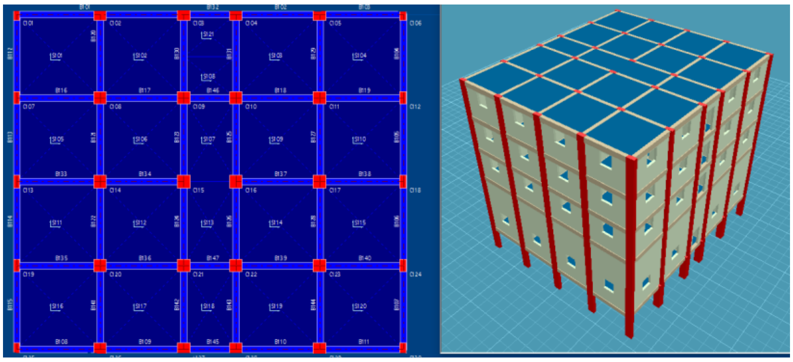


Figure 4.3: Two-Dimensional Plan & Three-Dimensional View of Case 3 (Weak Storey).

Table 4.3: Case 3 (Weak Storey) Building Specifications.

Case	Case 3 (Weak Storey)	
Floor number	5F (18m)	3F (12m)
Columns	60x60 cm^2	50x50 cm^2
	30x60 cm^2	30x40 cm^2
Beams	30x40 cm^2	30x40 cm^2
Slab	20 cm	20 cm

Irregularity check reports from STA4CAD shows that Case 3 is satisfying the irregularity type of weak storey since η_{ci} is below 0.8 in both reports.

Report of Case 3 3F for irregularity check:

```
IRREGULARITY CHECK UNDER SEISMIC ACTION
A1,B2  type irregularities
max(di/hi)=0.02  PDELTA ANALYSIS WITH CRACK SECTION. STORY DISPLACEMENTS IS MULTIPLYING WITH 0.4
1. sto X dtop = 0.0013407 + -0.0000071 × (.0 - 8.29)=0.0034991  (C101)
1. sto X dbot = 0 + -0.0000071 × (16.0 - 8.29)=0.003215  (C125)
2. sto X top  = 0.0023868 + -0.0000173 × (.0 - 8.53) - 0.0034991 = 0.0028372  (C201)
2. sto X dbot = 0.0023868 + -0.0000173 × (16.0 - 8.53) - 0.003215 = 0.0024287  (C225)
```

X DIR. (+%5)

Story	ΔX dtop(m)	ΔX dbot(m)	ΔX ort	nbi	nki	Δx/h	θi	story type
3	0.0007272	0.0005912	0.0006592	1.10	0.00	0.00142	0.00235	Normal sto
2	0.0011349	0.0009715	0.0010532	1.08	1.60	0.00221	0.00457	Normal sto
1	0.0013996	0.0012860	0.0013428	1.04	0.96	0.00205	0.00534	Normal sto

X DIR. (-%5)

Story	ΔX dtop(m)	ΔX dbot(m)	ΔX ort	nbi	nki	Δx/h	θi	story type
3	0.0007272	0.0005912	0.0006592	1.10	0.00	0.00142	0.00235	Normal sto
2	0.0011349	0.0009715	0.0010532	1.08	1.60	0.00221	0.00457	Normal sto
1	0.0013996	0.0012860	0.0013428	1.04	0.96	0.00205	0.00534	Normal sto

Y DIR. (+%5)

Story	ΔY dlft(m)	ΔY drgt(m)	ΔY ort	nbi	nki	Δy/h	θi	story type
3	0.0007151	0.0007141	0.0007146	1.00	0.00	0.00139	0.00284	Normal sto
2	0.0011966	0.0011949	0.0011957	1.00	1.67	0.00233	0.00579	Normal sto
1	0.0016499	0.0016478	0.0016488	1.00	1.03	0.00241	0.00725	Normal sto

Y DIR. (-%5)

Story	ΔY dlft(m)	ΔY drgt(m)	ΔY ort	nbi	nki	Δy/h	θi	story type
3	0.0007151	0.0007141	0.0007146	1.00	0.00	0.00139	0.00284	Normal sto
2	0.0011966	0.0011949	0.0011957	1.00	1.67	0.00233	0.00579	Normal sto
1	0.0016499	0.0016478	0.0016488	1.00	1.03	0.00241	0.00725	Normal sto

```
TDY 2.3.2.1 A1 torsional irregularity:
nbi=1.103 <1.2 , solved by modal analysis     ✓
TDY 2.3.2.1 B2  condition is not satisfied.  ✓
```

B1-Vertical irregularity check

Story	Aw	Agx	Agy	Akx	Aky	∑ Aex	∑ Aey	ncix	nciy	EXPLANATION
3	5.40	0.00	0.00	13.02	13.87	7.35	7.48	1.00	1.00	top sto ✓
2	5.40	0.00	0.00	13.02	13.87	7.35	7.48	1.00	1.00	Regular ✓
1	5.40	0.00	0.00	0.00	0.00	5.40	5.40	0.73	0.72	< .8 Irregula

```
nci < .8 Irregular; R= 8 > R= 5.85 to be solved   ✓

Ba=Bax+0.3×Bay, Ba=0.3×Bax+Bay :
Beams     , Columns   ; (Ba=Bax+0.3×Bay, Ba=0.3×Bax+Bay) calculated.
Exam.: C101 column;
Mx=-1.50+0.38+2.23+ 0.3 × 0.00=1.11
My=-0.46+0.14+9.37+ 0.3 × 0.28=9.15
```

Report of Case 3 5F for irregularity check:

```
IRREGULARITY CHECK UNDER SEISMIC ACTION
A1,B2  type irregularities
max(di/hi)=0.02  PDELTA ANALYSIS WITH CRACK SECTION. STORY DISPLACEMENTS IS MULTIPLYING WITH 0.4
1. sto X dtop = 0.0014716 + -0.0000088 × (.0 - 8.29)=0.0038605  (C101)
1. sto X dbot = 0 + -0.0000088 × (16.0 - 8.29)=0.00351  (C125)
2. sto X top  = 0.0027934 + -0.0000223 × (.0 - 8.53) - 0.0038605 = 0.0035986  (C201)
2. sto X dbot = 0.0027934 + -0.0000223 × (16.0 - 8.53) - 0.00351 = 0.0030564  (C225)
```

X DIR. (+%5)

Story	ΔX dtop(m)	ΔX dbot(m)	ΔX ort	nbi	nki	Δx/h	θi	story type
5	0.0005970	0.0004792	0.0005381	1.11	0.00	0.00116	0.00249	Normal sto
4	0.0009514	0.0007897	0.0008705	1.09	1.62	0.00186	0.00501	Normal sto
3	0.0012479	0.0010436	0.0011458	1.09	1.32	0.00243	0.00782	Normal sto
2	0.0014394	0.0012225	0.0013310	1.08	1.16	0.00281	0.01034	Normal sto
1	0.0015442	0.0014040	0.0014741	1.05	0.83	0.00226	0.00966	Normal sto

X DIR. (-%5)

Story	ΔX dtop(m)	ΔX dbot(m)	ΔX ort	nbi	nki	Δx/h	θi	story type
5	0.0005970	0.0004792	0.0005381	1.11	0.00	0.00116	0.00249	Normal sto
4	0.0009514	0.0007897	0.0008705	1.09	1.62	0.00186	0.00501	Normal sto
3	0.0012479	0.0010436	0.0011458	1.09	1.32	0.00243	0.00782	Normal sto
2	0.0014394	0.0012225	0.0013310	1.08	1.16	0.00281	0.01034	Normal sto
1	0.0015442	0.0014040	0.0014741	1.05	0.83	0.00226	0.00966	Normal sto

Y DIR. (+%5)

Story	ΔY dlft(m)	ΔY drgt(m)	ΔY ort	nbi	nki	Δy/h	θi	story type
5	0.0005750	0.0005739	0.0005745	1.00	0.00	0.00112	0.00289	Normal sto
4	0.0009539	0.0009522	0.0009530	1.00	1.66	0.00186	0.00598	Normal sto
3	0.0012660	0.0012637	0.0012649	1.00	1.33	0.00247	0.00952	Normal sto
2	0.0015093	0.0015066	0.0015079	1.00	1.19	0.00294	0.01293	Normal sto
1	0.0018076	0.0018049	0.0018062	1.00	0.90	0.00264	0.01299	Normal sto

Y DIR. (-%5)

Story	ΔY dlft(m)	ΔY drgt(m)	ΔY ort	nbi	nki	Δy/h	θi	story type
5	0.0005750	0.0005739	0.0005745	1.00	0.00	0.00112	0.00289	Normal sto
4	0.0009539	0.0009522	0.0009530	1.00	1.66	0.00186	0.00598	Normal sto
3	0.0012660	0.0012637	0.0012649	1.00	1.33	0.00247	0.00952	Normal sto
2	0.0015093	0.0015066	0.0015079	1.00	1.19	0.00294	0.01293	Normal sto
1	0.0018076	0.0018049	0.0018062	1.00	0.90	0.00264	0.01299	Normal sto

```
TDY 2.3.2.1 A1 torsional irregularity:
nbi=1.109 <1.2 , solved by modal analysis     ✓
TDY 2.3.2.1 B2  condition is not satisfied.  ✓
```

B1-Vertical irregularity check

Story	Aw	Agx	Agy	Akx	Aky	∑ Aex	∑ Aey	ncix	nciy	EXPLANATION
5	5.40	0.00	0.00	13.02	13.87	7.35	7.48	1.00	1.00	top sto ✓
4	5.40	0.00	0.00	13.02	13.87	7.35	7.48	1.00	1.00	Regular ✓
3	5.40	0.00	0.00	13.02	13.87	7.35	7.48	1.00	1.00	Regular ✓
2	5.40	0.00	0.00	13.02	13.87	7.35	7.48	1.00	1.00	Regular ✓
1	5.40	0.00	0.00	0.00	0.00	5.40	5.40	0.73	0.72	< .8 Irregula

```
nci < .8 Irregular; R= 8 > R= 5.85 to be solved   ✓

Ba=Bax+0.3×Bay, Ba=0.3×Bax+Bay :
Beams     , Columns    ; (Ba=Bax+0.3×Bay, Ba=0.3×Bax+Bay) calculated.
Exam.: C101 column;
Mx=-1.48+0.36+2.39+ 0.3 × 0.00=1.27
My=-0.45+0.14+10.01+ 0.3 × 0.34=9.80
```

4.2.4 Case Study 4 (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 relative storey drift at any i'th storey to the average relative storey drift at the storey immediately above or below, is greater than 2.0 TEC (2007) or greater than 1.5 TEC (1998), falls in the soft storey irregularity type of building according to the TEC-1998 and TEC-2007.

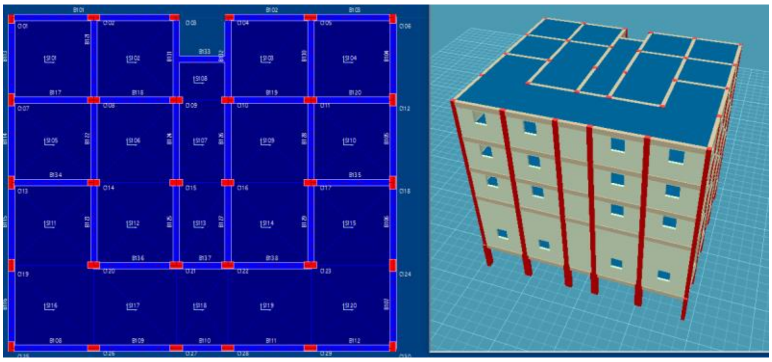


Figure 4.4: Two-Dimensional Plan & Three-Dimensional View of Case 3 (Weak Storey).

Table 4.4: Case 4 (Soft Storey) Building Specifications.

Case	**Case 4 (Soft Storey)**	
Floor number	5F (18m)	3F (12m)
Columns	30x60 cm^2	40x30 cm^2
	40x30 cm^2	30x30 cm^2
	30x30 cm^2	25x30 cm^2
Beams	30x40 cm^2	25x30 cm^2
Slab	20 cm	20 cm

Irregularity check reports from STA4CAD shows that Case 4 is satisfying the irregularity type of soft storey since η_{ki} is above 2.0 according to TEC-2007 in both reports.

Report of Case 4 3F for irregularity check:

```
IRREGULARITY CHECK UNDER SEISMIC ACTION
A1,B2  type irregularities
max(di/hi)=0.02  PDELTA ANALYSIS WITH CRACK SECTION. STORY DISPLACEMENTS IS MULTIPLYING WITH 0.4
1. sto X dtop = 0.0006671 + -0.0000106 × (.0 - 9.05)=0.0019084   (C101)
1. sto X dbot = 0 + -0.0000106 × (16.0 - 9.05)=0.0014828   (C125)
2. sto X top  = 0.0052293 + -0.0000804 × (.0 - 9.09) - 0.0019084 = 0.0129907   (C201)
2. sto X dbot = 0.0052293 + -0.0000804 × (16.0 - 9.09) - 0.0014828 = 0.0102019   (C225)
```

X DIR.   (+%5)

Story	ΔX dtop(m)	ΔX dbot(m)	ΔX ort	nbi	nki	Δx/h	θi	story type
3	0.0015921	0.0009845	0.0012883	1.24	0.00	0.00310	0.00731	Normal sto
2	0.0051963	0.0040808	0.0046385	1.12	1.80	0.00507	0.02328	Normal sto
1	0.0007633	0.0005931	0.0006782	1.13	0.29	0.00149	0.00670	Normal sto

X DIR.   (-%5)

Story	ΔX dtop(m)	ΔX dbot(m)	ΔX ort	nbi	nki	Δx/h	θi	story type
3	0.0015921	0.0009845	0.0012883	1.24	0.00	0.00310	0.00731	Normal sto
2	0.0051963	0.0040808	0.0046385	1.12	1.80	0.00507	0.02328	Normal sto
1	0.0007633	0.0005931	0.0006782	1.13	0.29	0.00149	0.00670	Normal sto

Y DIR.   (+%5)

Story	ΔY dlft(m)	ΔY drgt(m)	ΔY ort	nbi	nki	Δy/h	θi	story type
3	0.0008651	0.0008646	0.0008648	1.00	0.00	0.00169	0.00458	Normal sto
2	0.0051101	0.0051065	0.0051083	1.00	2.95	0.00498	0.02461	Normal sto
1	0.0006751	0.0006743	0.0006747	1.00	0.26	0.00132	0.00610	Normal sto

Y DIR.   (-%5)

Story	ΔY dlft(m)	ΔY drgt(m)	ΔY ort	nbi	nki	Δy/h	θi	story type
3	0.0008651	0.0008646	0.0008648	1.00	0.00	0.00169	0.00458	Normal sto
2	0.0051101	0.0051065	0.0051083	1.00	2.95	0.00498	0.02461	Normal sto
1	0.0006751	0.0006743	0.0006747	1.00	0.26	0.00132	0.00610	Normal sto

```
TDY 2.3.2.1 A1 torsional irregularity:
1.2< nbi=1.236 <2  , solved by modal analysis     ✓
TDY 2.3.2.1 B2 condition is not satisfied, dynamic solved by modal analysis     ✓
```

B1-Vertical irregularity check

Story	Aw	Agx	Agy	Akx	Aky	∑ Aex	∑ Aey	ncix	nciy	EXPLANATION
3	3.30	0.00	0.00	7.60	11.59	4.44	5.04	1.00	1.00	top sto ✓
2	3.30	0.00	0.00	7.60	11.59	4.44	5.04	1.00	1.00	Regular ✓
1	5.40	0.00	0.00	0.00	0.00	5.40	5.40	1.22	1.07	Regular ✓

```
Ba=Bax+0.3×Bay, Ba=0.3×Bax+Bay :
Beams      , Columns    ; (Ba=Bax+0.3×Bay, Ba=0.3×Bax+Bay) calculated.
Exam.: C101 column;
Mx=-4.18+0.56+1.72+ 0.3 × 0.00=-1.90
My=-1.37+0.27+5.77+ 0.3 × 0.88=4.93
```

Report of Case 4 5F for irregularity check:

```
IRREGULARITY CHECK UNDER SEISMIC ACTION
A1,B2  type irregularities
max(di/hi)=0.02  PDELTA ANALYSIS WITH CRACK SECTION. STORY DISPLACEMENTS IS MULTIPLYING WITH 0.4
1. sto X dtop = 0.0006735 + -0.000011 × (.0 - 9.02)=0.0019326  (C101)
1. sto X dbot = 0 + -0.000011 × (16.0 - 9.02)=0.0014913  (C125)
2. sto X top  = 0.0062073 + -0.0000989 × (.0 - 9.05) - 0.0019326 = 0.0158236  (C201)
2. sto X dbot = 0.0062073 + -0.0000989 × (16.0 - 9.05) - 0.0014913 = 0.0123077  (C225)
```

X DIR. (+%5)

Story	ΔX dtop(m)	ΔX dbot(m)	ΔX ort	nbi	nki	Δx/h	θi	story type
5	0.0010101	0.0006432	0.0008267	1.22	0.00	0.00197	0.00700	Normal sto
4	0.0014411	0.0009178	0.0011794	1.22	1.43	0.00281	0.01511	Normal sto
3	0.0021084	0.0013098	0.0017091	1.23	1.45	0.00411	0.02595	Normal sto
2	0.0063294	0.0049231	0.0056263	1.12	1.65	0.00617	0.04875	Normal sto
1	0.0007730	0.0005965	0.0006848	1.13	0.26	0.00162	0.01053	Normal sto

X DIR. (-%5)

Story	ΔX dtop(m)	ΔX dbot(m)	ΔX ort	nbi	nki	Δx/h	θi	story type
5	0.0010101	0.0006432	0.0008267	1.22	0.00	0.00197	0.00700	Normal sto
4	0.0014411	0.0009178	0.0011794	1.22	1.43	0.00281	0.01511	Normal sto
3	0.0021084	0.0013098	0.0017091	1.23	1.45	0.00411	0.02595	Normal sto
2	0.0063294	0.0049231	0.0056263	1.12	1.65	0.00617	0.04875	Normal sto
1	0.0007730	0.0005965	0.0006848	1.13	0.26	0.00162	0.01053	Normal sto

Y DIR. (+%5)

Story	ΔY dlft(m)	ΔY drgt(m)	ΔY ort	nbi	nki	Δy/h	θi	story type
5	0.0004923	0.0004918	0.0004921	1.00	0.00	0.00096	0.00427	Normal sto
4	0.0007825	0.0007815	0.0007820	1.00	1.59	0.00153	0.00984	Normal sto
3	0.0012689	0.0012670	0.0012679	1.00	1.62	0.00247	0.01767	Normal sto
2	0.0066281	0.0066191	0.0066236	1.00	2.61	0.00646	0.05145	Normal sto
1	0.0007013	0.0007002	0.0007008	1.00	0.23	0.00147	0.00961	Normal sto

Y DIR. (-%5)

Story	ΔY dlft(m)	ΔY drgt(m)	ΔY ort	nbi	nki	Δy/h	θi	story type
5	0.0004923	0.0004918	0.0004921	1.00	0.00	0.00096	0.00427	Normal sto
4	0.0007825	0.0007815	0.0007820	1.00	1.59	0.00153	0.00984	Normal sto
3	0.0012689	0.0012670	0.0012679	1.00	1.62	0.00247	0.01767	Normal sto
2	0.0066281	0.0066191	0.0066236	1.00	2.61	0.00646	0.05145	Normal sto
1	0.0007013	0.0007002	0.0007008	1.00	0.23	0.00147	0.00961	Normal sto

```
TDY 2.3.2.1 A1 torsional irregularity:
1.2< nbi=1.234 <2  , solved by modal analysis    ✓
TDY 2.3.2.1 B2 condition is not satisfied, dynamic solved by modal analysis    ✓
```

B1-Vertical irregularity check

Story	Aw	Agx	Agy	Akx	Aky	∑ Aex	∑ Aey	ncix	nciy	EXPLANATION
5	3.30	0.00	0.00	7.60	11.59	4.44	5.04	1.00	1.00	top sto ✓
4	3.30	0.00	0.00	7.60	11.59	4.44	5.04	1.00	1.00	Regular ✓
3	3.30	0.00	0.00	7.60	11.59	4.44	5.04	1.00	1.00	Regular ✓
2	3.30	0.00	0.00	7.60	11.59	4.44	5.04	1.00	1.00	Regular ✓
1	5.40	0.00	0.00	0.00	0.00	5.40	5.40	1.22	1.07	Regular ✓

```
Ba=Bax+0.3×Bay, Ba=0.3×Bax+Bay :
Beams    , Columns   ; (Ba=Bax+0.3×Bay, Ba=0.3×Bax+Bay) calculated.
Exam.: C101 column;
Mx=-4.27+0.56+1.91+ 0.3 × 0.00=-1.80
My=-1.36+0.26+6.66+ 0.3 × 1.02=5.86
```

4.2.5 Case Study 5 (Projection in Plan)

When the dimensions of projections in the two perpendicular directions in plan surpass the total plan dimensions of that storey of the building in the corresponding directions by at least 20%, falls in the projection in plan irregular type of building according to TEC-1998 and TEC-2007.

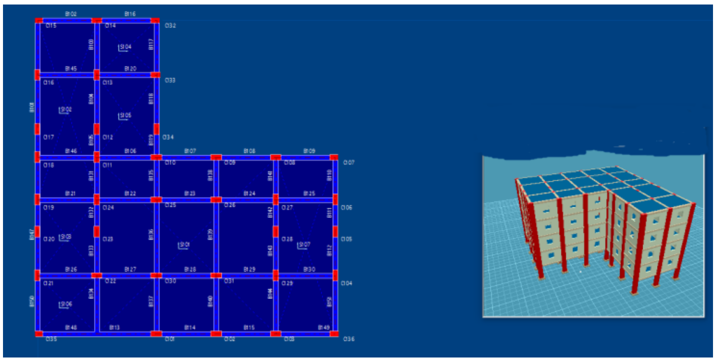


Figure 4.5: Two & Three-Dimensional Plan of Case 5 (Projection in Plan).

Table 4.5: Case 5 (Projection in Plan) Building Specifications.

Case	**Case 5 (Projection in Plan)**	
Floor number	5F (15m)	3F (9m)
Columns	80x40 cm^2	60x30 cm^2
Beams	25x40 cm^2	25x30 cm^2
Slab	20 cm	20 cm

Projection in plan irregularity in this case is achieved by satisfying the conditions of equation 2.9 a & b, where $a_x > 0.2\ L_x$ and $a_y > 0.2\ L_y$. In this case: $a_x = 4$m, $L_x = 10$m, $a_y = 6$m, $L_y = 12$m. Thus, resulting that 4 > 2 and 6 > 2.4; respectively.

4.2.6 Case Study 6 (Torsional Irregularity)

Torsional Irregularity occurs when the torsional irregularity factor η_{bi} is higher than 1.2 according to TEC-1998 and TEC-2007.

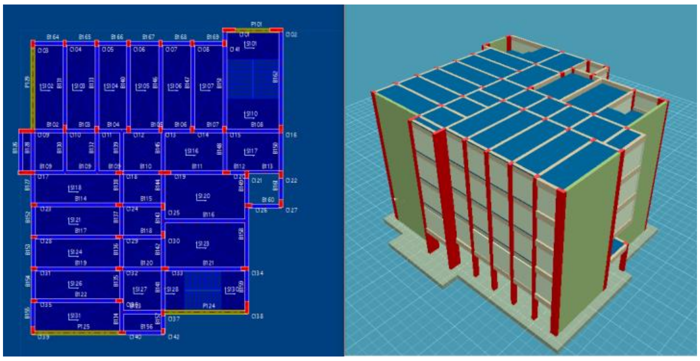


Figure 4.6: Two-Dimensional Plan & Three-Dimensional View of Case 6 (Torsional Irregularity).

Table 4.6: Case 6 (Torsional Irregularity) Building Specifications.

Case	**Case 6 (Torsional Irregularity)**	
Floor number	5F (16m)	3F (10m)
Columns	50x30 cm^2	40x30 cm^2
	60x30 cm^2	40x30 cm^2
	130x30 cm^2	100x30 cm^2
Beams	30x60 cm^2	30x40 cm^2
Slab	20 cm	20 cm

Irregularity check reports from STA4CAD shows that Case 6 is satisfying the irregularity type of torsional irregularity since η_{bi} is greater than 1.2 in both reports.

Report of Case 6 3F for irregularity check:

```
IRREGULARITY CHECK UNDER SEISMIC ACTION
A1,B2  type irregularities
max(di/hi)=0.02  PDELTA ANALYSIS WITH CRACK SECTION. STORY DISPLACEMENTS IS MULTIPLYING WITH 0.4
1. sto X dtop = 0.0005489 + -0.0000021 × (1.9 - 11.66)=0.0014223  (C101)
1. sto X dbot = 0 + -0.0000021 × (22.4 - 11.66)=0.001317  (C112)
2. sto X top  = 0.0012688 + 0.0000005 × (1.9 - 11.3) - 0.0014223 = 0.0017373  (C201)
2. sto X dbot = 0.0012688 + 0.0000005 × (22.4 - 11.3) - 0.001317 = 0.0018697  (C212)
```

X DIR. (+%5)

Story	ΔX dtop(m)	ΔX dbot(m)	ΔX ort	ηbi	ηki	R·Δx/h	θi	story type
3	0.0004788	0.0005837	0.0005312	1.10	0.00	0.00156	0.00133	Normal sto
2	0.0006949	0.0007479	0.0007214	1.04	1.36	0.00199	0.00225	Normal sto
1	0.0005689	0.0005268	0.0005479	1.04	0.76	0.00152	0.00218	Normal sto

X DIR. (-%5)

Story	ΔX dtop(m)	ΔX dbot(m)	ΔX ort	ηbi	ηki	R·Δx/h	θi	story type
3	0.0004788	0.0005837	0.0005312	1.10	0.00	0.00156	0.00133	Normal sto
2	0.0006949	0.0007479	0.0007214	1.04	1.36	0.00199	0.00225	Normal sto
1	0.0005689	0.0005268	0.0005479	1.04	0.76	0.00152	0.00218	Normal sto

Y DIR. (+%5)

Story	ΔY dlft(m)	ΔY drgt(m)	ΔY ort	nbi	nki	R·Δy/h	θi	story type
3	0.0001781	0.0003975	0.0002878	1.38	0.00	0.00106	0.00071	Normal sto
2	0.0001941	0.0005765	0.0003853	1.50	1.34	0.00154	0.00121	Normal sto
1	0.0001320	0.0004629	0.0002974	1.56	0.77	0.00123	0.00119	Normal sto

Y DIR. (-%5)

Story	ΔY dlft(m)	ΔY drgt(m)	ΔY ort	nbi	nki	R·Δy/h	θi	story type
3	0.0001781	0.0003975	0.0002878	1.38	0.00	0.00106	0.00071	Normal sto
2	0.0001941	0.0005765	0.0003853	1.50	1.34	0.00154	0.00121	Normal sto
1	0.0001320	0.0004629	0.0002974	1.56	0.77	0.00123	0.00119	Normal sto

TDY 2.3.2.1 A1 torsional irregularity:
1.2< nbi=1.556 <2 , solved by modal analysis ✓
TDY 2.3.2.1 B2 condition is not satisfied. ✓
TDY 2.19 requirement OK .002 < .02 ✓
TDY 2.20 requirement OK max θi=.002 < 0.12 ✓

B1-Vertical irregularity check

Story	Aw	Agx	Agy	Akx	Aky	∑ Aex	∑ Aey	ncix	nciy	EXPLANATION
3	5.58	2.35	2.94	18.79	24.45	10.75	12.19	1.00	1.00	top sto ✓
2	5.58	2.35	2.94	18.79	24.45	10.75	12.19	1.00	1.00	Regular ✓
1	5.58	2.35	2.94	0.00	0.00	7.93	8.52	0.74	0.70	< .8 Irregula

nci < .8 Irregular.; R=1.25 x nci x R= 6.99 to be solve ×

Ba=Bax+0.3×Bay, Ba=0.3×Bax+Bay :
Beams , Columns ; (Ba=Bax+0.3×Bay, Ba=0.3×Bax+Bay) calculated.
Exam.: C101 column;
Mx=-0.70+0.14+3.88+ 0.3 × 1.14=3.67
My=-0.18+0.05+0.29+ 0.3 × 0.06=0.19

Report of Case 6 5F for irregularity check:

IRREGULARITY CHECK UNDER SEISMIC ACTION
A1,B2 type irregularities
max(di/hi)=0.02 PDELTA ANALYSIS WITH CRACK SECTION. STORY DISPLACEMENTS IS MULTIPLYING WITH 0.4
1. sto X dtop = 0.0004932 + -0.0000016 × (1.9 - 11.65)=0.001272 (C101)
1. sto X dbot = 0 + -0.0000016 × (22.4 - 11.65)=0.0011902 (C112)
2. sto X top = 0.0013025 + 0.0000011 × (1.9 - 11.31) - 0.001272 = 0.0019581 (C201)
2. sto X dbot = 0.0013025 + 0.0000011 × (22.4 - 11.31) - 0.0011902 = 0.0020968 (C212)

X DIR. (+%5)

Story	ΔX dtop(m)	ΔX dbot(m)	ΔX ort	nbi	nki	Δx/h	θi	story type
5	0.0004547	0.0005538	0.0005043	1.10	0.00	0.00108	0.00129	Normal sto
4	0.0006392	0.0007370	0.0006881	1.07	1.36	0.00144	0.00207	Normal sto
3	0.0007729	0.0008637	0.0008183	1.06	1.19	0.00168	0.00285	Normal sto
2	0.0007832	0.0008387	0.0008110	1.03	0.99	0.00164	0.00325	Normal sto
1	0.0005088	0.0004761	0.0004924	1.03	0.61	0.00099	0.00232	Normal sto

X DIR. (-%5)

Story	ΔX dtop(m)	ΔX dbot(m)	ΔX ort	nbi	nki	Δx/h	θi	story type
5	0.0004547	0.0005538	0.0005043	1.10	0.00	0.00108	0.00129	Normal sto
4	0.0006392	0.0007370	0.0006881	1.07	1.36	0.00144	0.00207	Normal sto
3	0.0007729	0.0008637	0.0008183	1.06	1.19	0.00168	0.00285	Normal sto
2	0.0007832	0.0008387	0.0008110	1.03	0.99	0.00164	0.00325	Normal sto
1	0.0005088	0.0004761	0.0004924	1.03	0.61	0.00099	0.00232	Normal sto

Y DIR. (+%5)

Story	ΔY dlft(m)	ΔY drgt(m)	ΔY ort	nbi	nki	Δy/h	θi	story type
5	0.0003421	0.0004959	0.0004190	1.18	0.00	0.00097	0.00083	Normal sto
4	0.0003615	0.0007205	0.0005410	1.33	1.29	0.00140	0.00124	Normal sto
3	0.0003436	0.0008764	0.0006100	1.44	1.13	0.00171	0.00160	Normal sto
2	0.0002586	0.0008956	0.0005771	1.55	0.95	0.00175	0.00177	Normal sto
1	0.0001020	0.0006007	0.0003513	1.71	0.61	0.00117	0.00128	Normal sto

```
Y DIR.  (-%5)
```

Story	ΔY dlft(m)	ΔY drgt(m)	ΔY ort	nbi	nki	Δy/h	θi	story type
5	0.0003421	0.0004959	0.0004190	1.18	0.00	0.00097	0.00083	Normal sto
4	0.0003615	0.0007205	0.0005410	1.33	1.29	0.00140	0.00124	Normal sto
3	0.0003436	0.0008764	0.0006100	1.44	1.13	0.00171	0.00160	Normal sto
2	0.0002586	0.0008956	0.0005771	1.55	0.95	0.00175	0.00177	Normal sto
1	0.0001020	0.0006007	0.0003513	1.71	0.61	0.00117	0.00128	Normal sto

```
TDY 2.3.2.1 A1 torsional irregularity:
1.2< nbi=1.71 <2  , solved by modal analysis     ✓
TDY 2.3.2.1 B2  condition is not satisfied.  ✓

B1-Vertical irregularity check
```

Story	Aw	Agx	Agy	Akx	Aky	∑ Aex	∑ Aey	ncix	nciy	EXPLANATION
5	5.58	2.35	2.94	20.42	26.54	11.00	12.50	1.00	1.00	top sto ✓
4	5.58	2.35	2.94	20.42	26.54	11.00	12.50	1.00	1.00	Regular ✓
3	5.58	2.35	2.94	20.42	26.54	11.00	12.50	1.00	1.00	Regular ✓
2	5.58	2.35	2.94	20.42	26.54	11.00	12.50	1.00	1.00	Regular ✓
1	5.58	2.35	2.94	0.00	0.00	7.93	8.52	0.72	0.68	< .8 Irregula

```
nci < .8 Irregular; R= 8 > R= 5.85 to be solved   ✓

Ba=Bax+0.3×Bay, Ba=0.3×Bax+Bay :
Beams     , Columns    ; (Ba=Bax+0.3×Bay, Ba=0.3×Bax+Bay) calculated.
Exam.: C101 column;
Mx=-0.70+0.14+5.05+ 0.3 × 2.59=5.28
My=-0.24+0.06+0.30+ 0.3 × 0.09=0.15
```

4.3 Design and Earthquake Parameters

Common design and earthquake parameters used for TEC-1975, TEC-1998, and TEC-2007 are shown in the following tables.

Table 4.7: TEC-1975 Parameters

<table>
<tr><th colspan="2">Parameter</th><th>TEC-1975</th><th>Explanation</th></tr>
<tr><td colspan="2">Seismic zone</td><td>2</td><td>-</td></tr>
<tr><td colspan="2">Importance factor</td><td>1</td><td>Residential</td></tr>
<tr><td colspan="2">Structural Behavioural Factor K</td><td>1</td><td>DCH</td></tr>
<tr><td colspan="2">Response spectrum</td><td>1</td><td>High seismicity</td></tr>
<tr><td colspan="2">Site class</td><td>Z2</td><td>-</td></tr>
<tr><td colspan="2">Soil factor</td><td>1.2</td><td>Site Class Z2</td></tr>
<tr><td colspan="2">Seismic Zone Coefficient C</td><td>0.08</td><td>Seismic map of Cyprus</td></tr>
<tr><td rowspan="2">Periods</td><td>T_A</td><td>0.15</td><td rowspan="2">Type 1 spectrum, site class Z2</td></tr>
<tr><td>T_B</td><td>0.4</td></tr>
<tr><td colspan="2">Damping factor</td><td>5%</td><td>-</td></tr>
<tr><td colspan="2">Concrete type</td><td>C20</td><td>-</td></tr>
<tr><td colspan="2">Reinforcement type</td><td>420C</td><td>-</td></tr>
</table>

Method of design	Nonlinear static pushover analysis	-
Reinforced concrete design method	Ultimate limit state	-

Table 4.8: TEC-1998 and TEC-2007 Parameters

Parameter		**TEC-1998 & TEC-2007**	**Explanation**
Seismic zone		2	-
Importance factor		1	Residential
Behavior factor R		8	DCH
Response spectrum		1	High seismicity
Site class		Z2	-
Soil factor		1.2	Site Class Z2
A_0		0.3	Seismic map of Cyprus
Periods	T_A	0.15	Type 1 spectrum, site class Z2
	T_B	0.4	
Damping factor		5%	-
Concrete type		C20	-
Reinforcement type		420C	-
Method of design		Nonlinear static pushover analysis	-
Reinforced concrete design method		Ultimate limit state	-

Chapter 5

ANALYSIS RESULTS AND DISCUSSION

5.1 Introduction

This chapter presents the analysis results of the nonlinear static pushover analysis method obtained from the STA4CAD structural software to examine the behaviour and performance of selected reinforced concrete case studies against an earthquake. Comparison of maximum base shear and displacement, cost, and damage percentages are also discussed in this chapter.

5.2 Performance Level

In this section the performance level according to the three codes will be presented. TEC-2007 and TEC-1998 cases reached Life Safety category in both 5F and 3F elevations. TEC-1975 cases reached either Collapse or Collapse Prevention categories in most cases in both elevations except for Case 1 3F, it reached Life Safety category.

5.2.1 Case 1 (Regular Building 1)

5.2.1.1 TEC-1975 (5F)

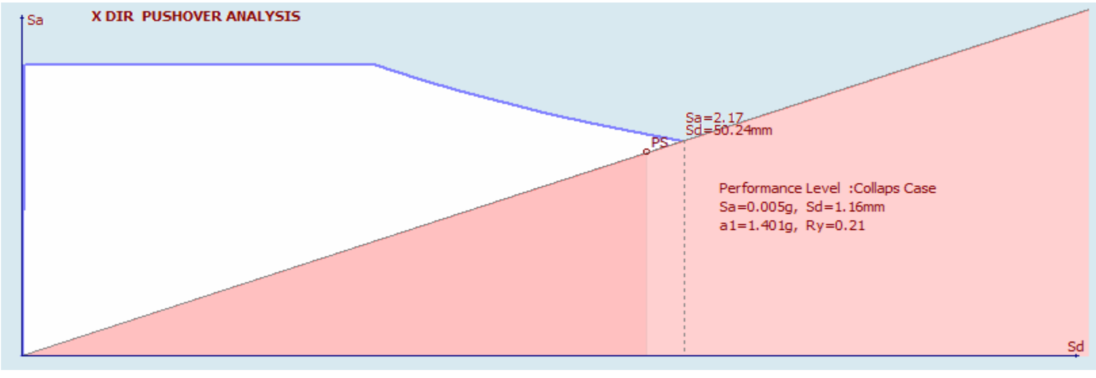


Figure 5.1: Performance Level of Case 1 (Regular Building 1 5F) TEC-1975.

5.2.1.2 TEC-1998 (5F)

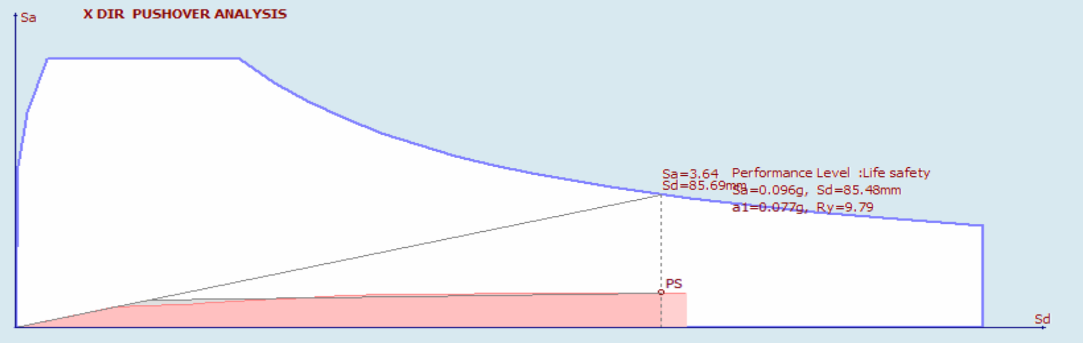


Figure 5.2: Performance Level of Case 1 (Regular Building 1 5F) TEC-1998.

5.2.1.3 TEC-2007 (5F)

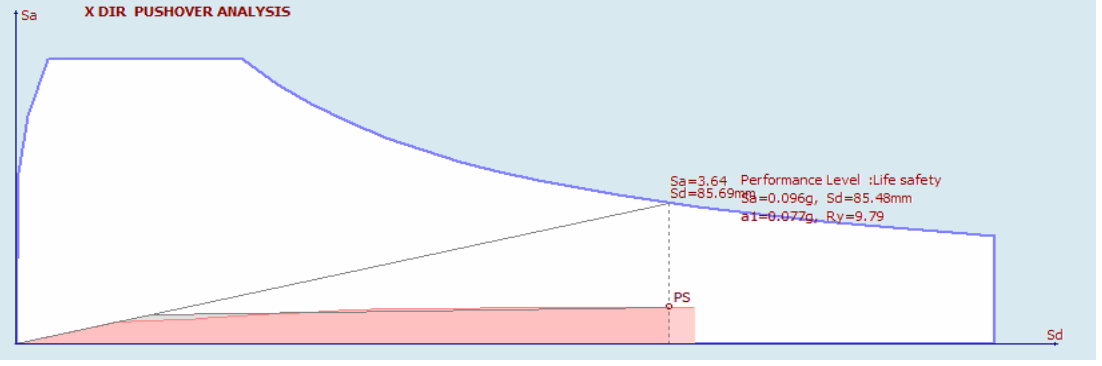


Figure 5.3: Performance Level of Case 1 (Regular Building 1 5F) TEC-2007.

5.2.1.4 TEC-1975 (3F)

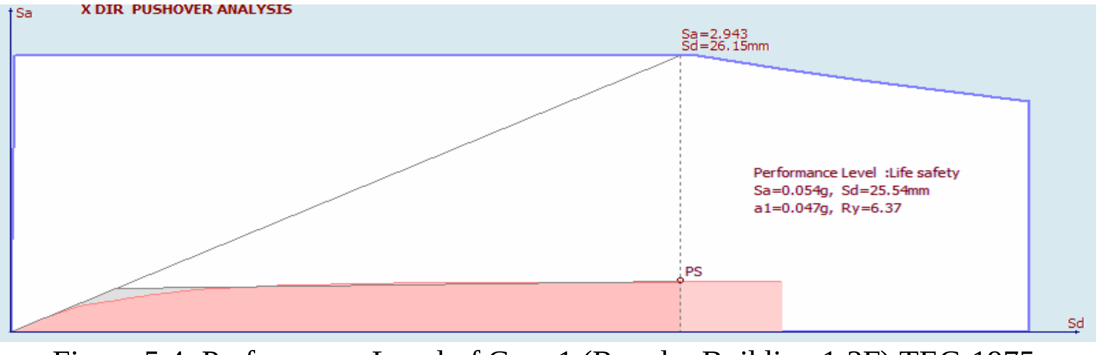


Figure 5.4: Performance Level of Case 1 (Regular Building 1 3F) TEC-1975.

5.2.1.5 TEC-1998 (3F)

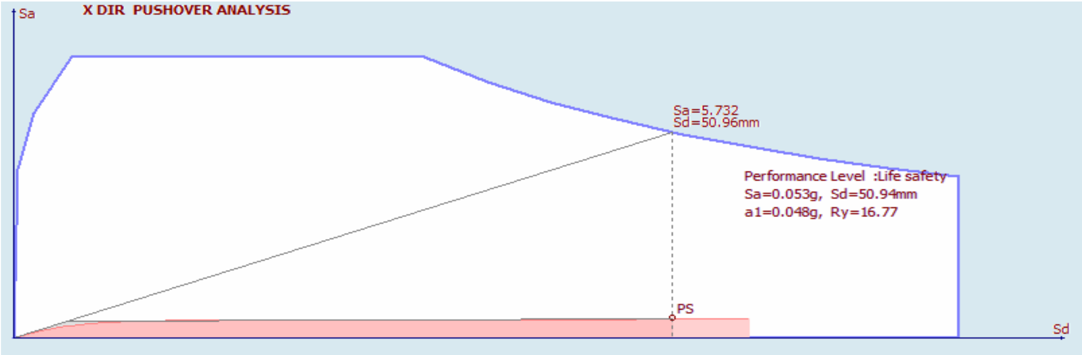


Figure 5.5: Performance Level of Case 1 (Regular Building 1 3F) TEC-1998.

5.2.1.6 TEC-2007 (3F)

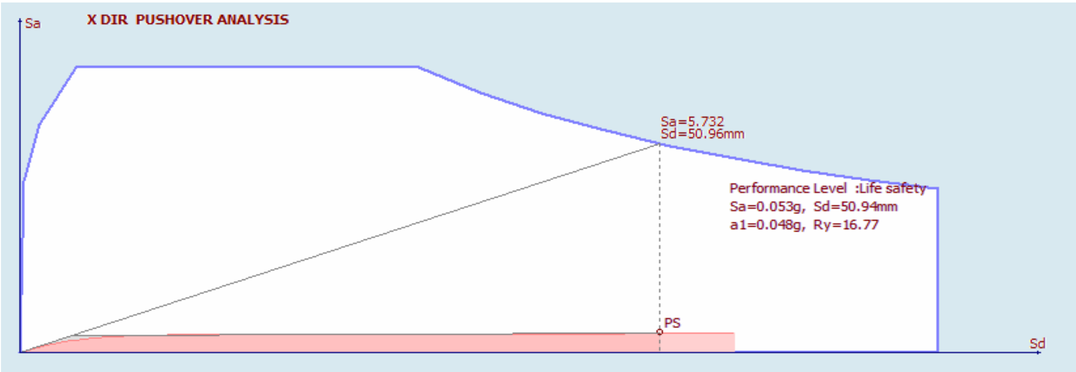


Figure 5.6: Performance Level of Case 1 (Regular Building 1 3F) TEC-2007.

5.2.2 Case 2 (Regular Building 2)

5.2.2.1 TEC-1975 (5F)

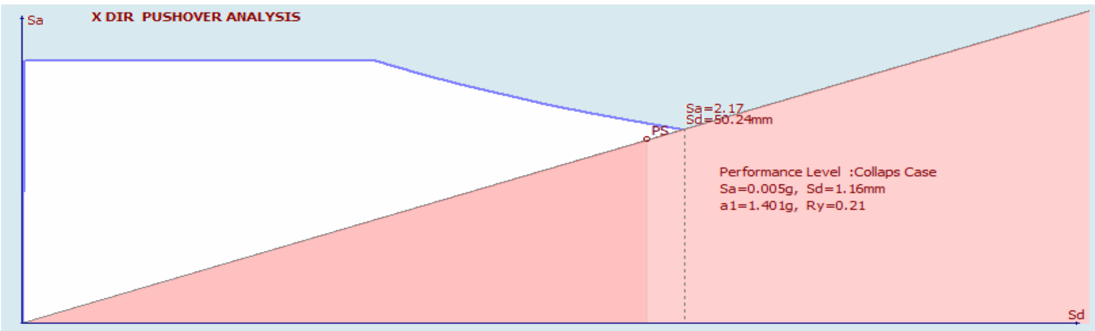


Figure 5.7: Performance Level of Case 2 (Regular Building 2 5F) TEC-1975.

5.2.2.2 TEC-1998 (5F)

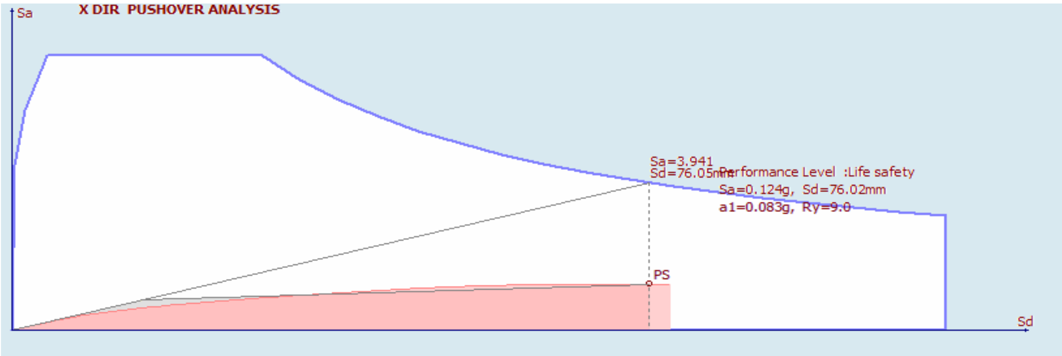


Figure 5.8: Performance Level of Case 2 (Regular Building 2 5F) TEC-1998.

5.2.2.3 TEC-2007 (5F)

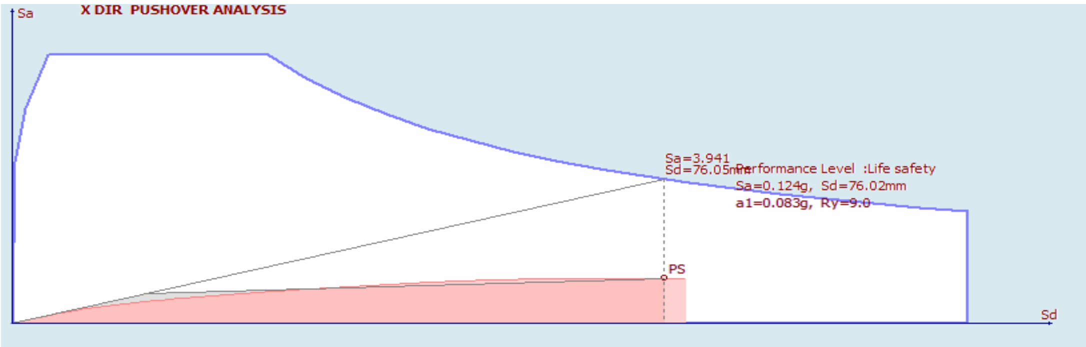


Figure 5.9: Performance Level of Case 2 (Regular Building 2 5F) TEC-2007.

5.2.2.3 TEC-1975 (3F)

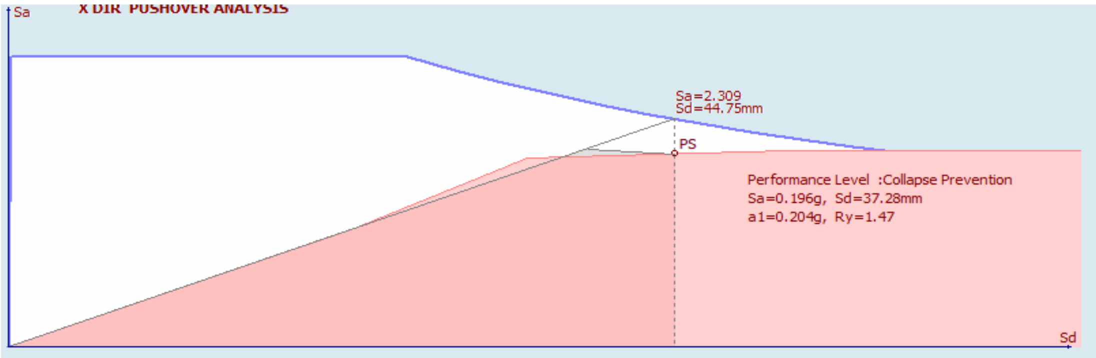


Figure 5.10: Performance Level of Case 2 (Regular Building 2 3F) TEC-1975.

5.2.2.4 TEC-1998 (3F)

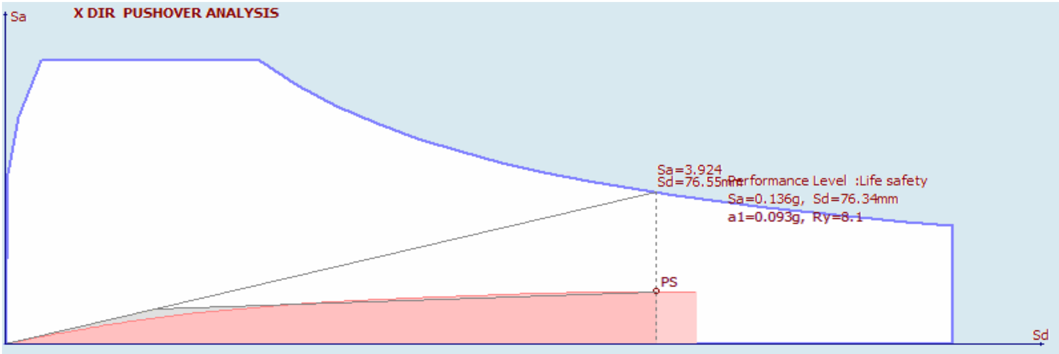


Figure 5.11: Performance Level of Case 2 (Regular Building 2 3F) TEC-1998.

5.2.2.6 TEC-2007 (3F)

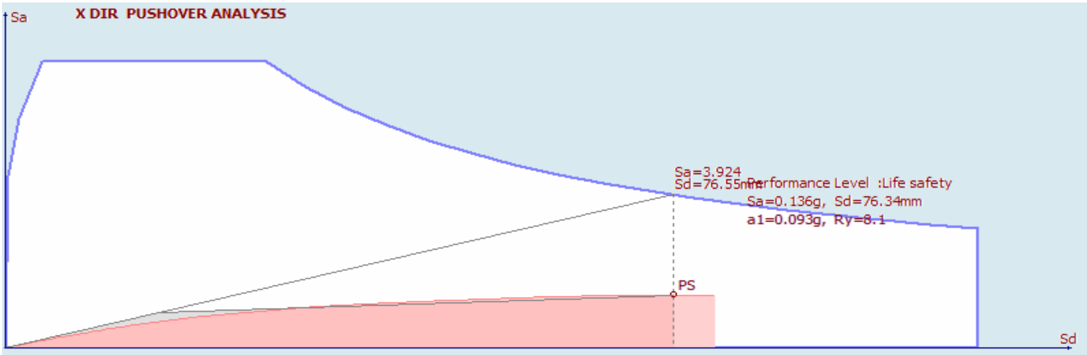


Figure 5.12: Performance Level of Case 2 (Regular Building 2 3F) TEC-2007.

5.2.3 Case 3 (Weak Storey)

5.2.3.1 TEC-1975 (5F)

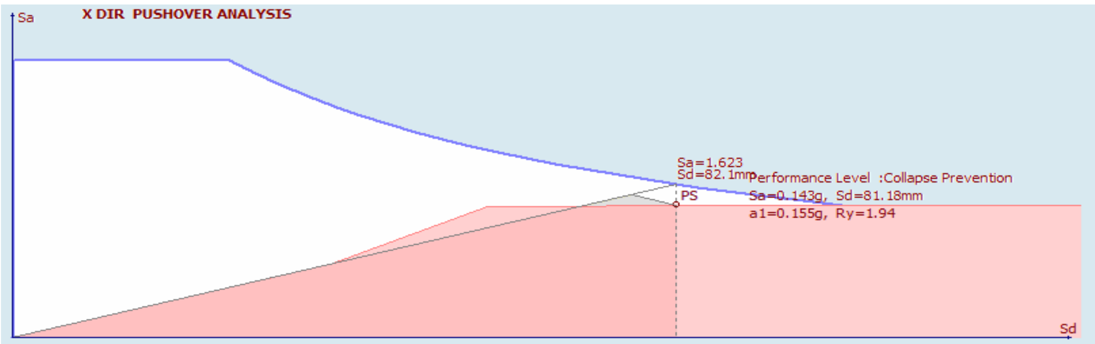


Figure 5.13: Performance Level of Case 3 (Weak Storey 5F) TEC-1975.

5.2.3.2 TEC-1998 (5F)

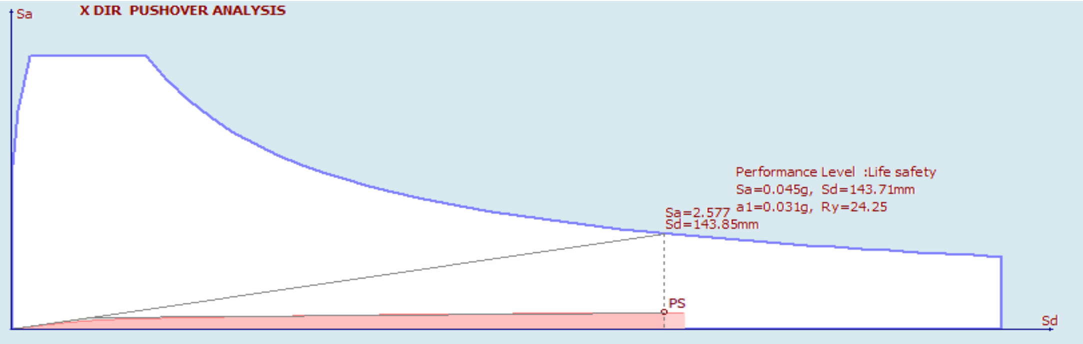


Figure 5.14: Performance Level of Case 3 (Weak Storey 5F) TEC-1998.

5.2.3.3 TEC-2007 (5F)

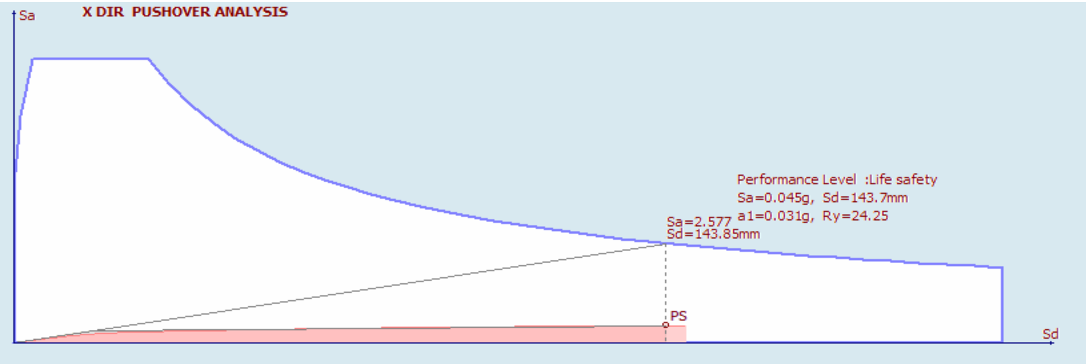


Figure 5.15: Performance Level of Case 3 (Weak Storey 5F) TEC-2007.

5.2.3.4 TEC-1975 (3F)

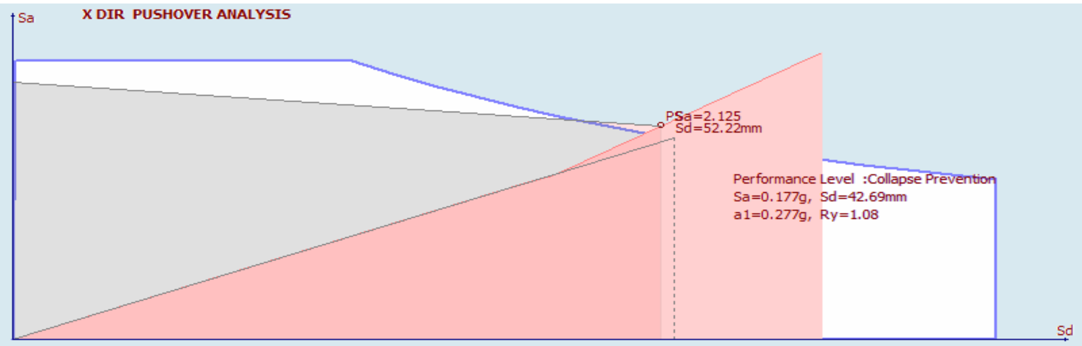


Figure 5.16: Performance Level of Case 3 (Weak Storey 3F) TEC-1975.

5.2.3.5 TEC-1998 (3F)

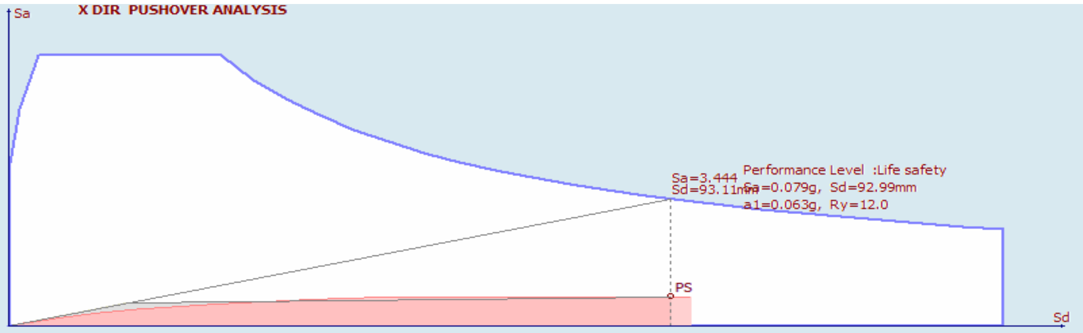


Figure 5.17: Performance Level of Case 3 (Weak Storey 3F) TEC-1998.

5.2.3.6 TEC-2007 (3F)

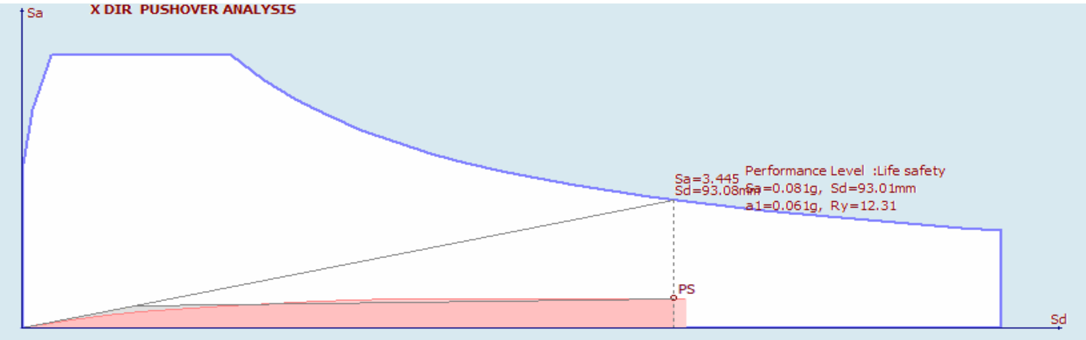


Figure 5.18: Performance Level of Case 3 (Weak Storey 3F) TEC-2007.

5.2.4 Case 4 (Soft Storey)

5.2.4.1 TEC-1975 (5F)

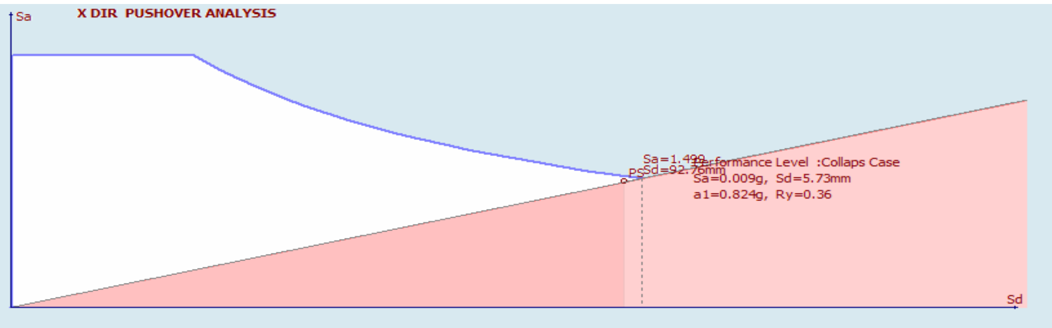


Figure 5.19: Performance Level of Case 4 (Soft Storey 5F) TEC-1975.

5.2.4.2 TEC-1998 (5F)

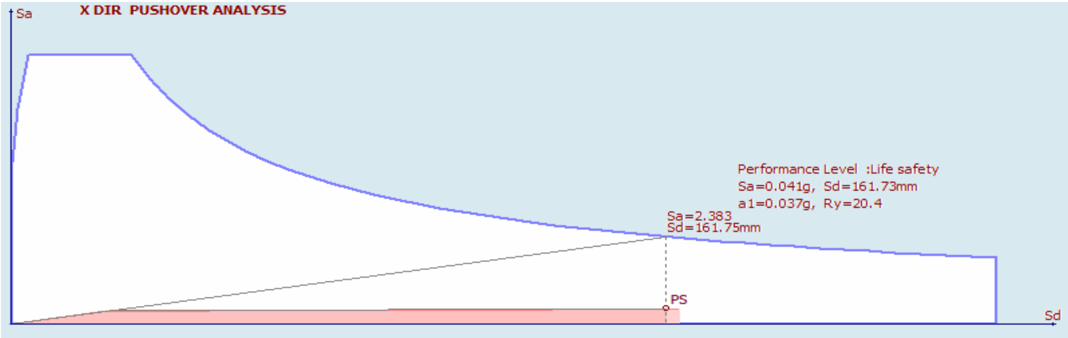


Figure 5.20: Performance Level of Case 4 (Soft Storey 5F) TEC-1998.

5.2.4.3 TEC-2007 (5F)

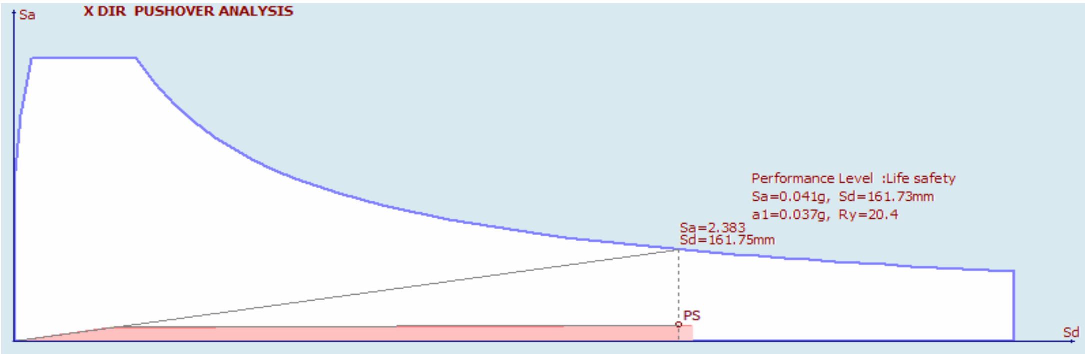


Figure 5.21: Performance Level of Case 4 (Soft Storey 5F) TEC-2007.

5.2.4.4 TEC-1975 (3F)

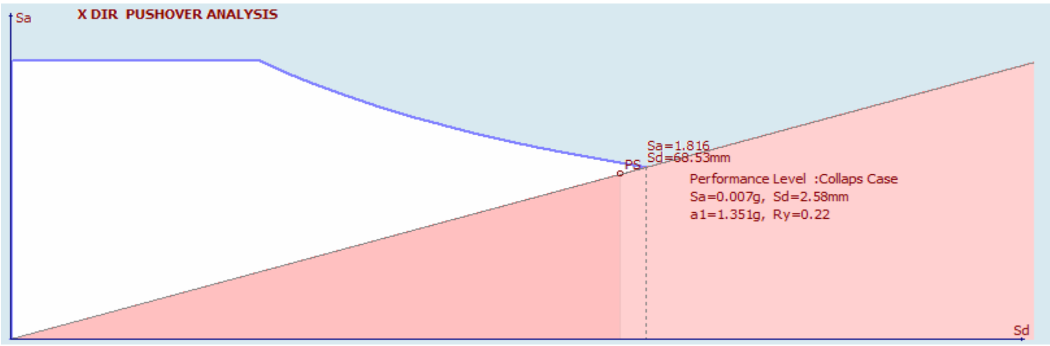


Figure 5.22: Performance Level of Case 4 (Soft Storey 3F) TEC-1975.

5.2.4.5 TEC-1998 (3F)

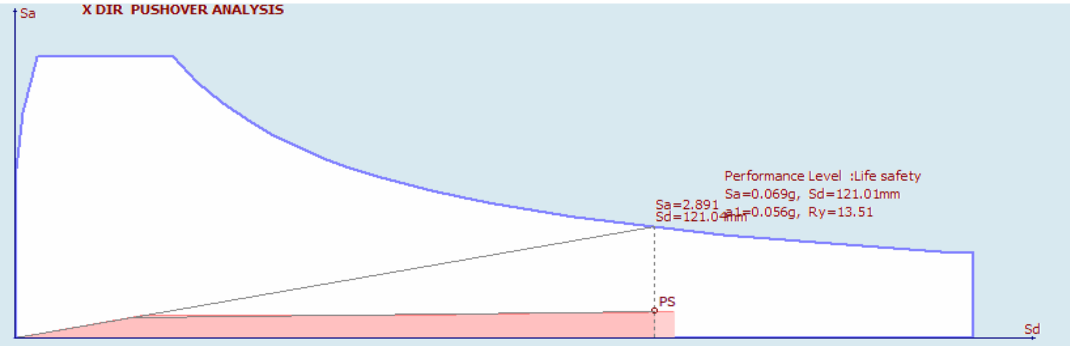


Figure 5.23: Performance Level of Case 4 (Soft Storey 3F) TEC-1998.

5.2.4.6 TEC-2007 (3F)

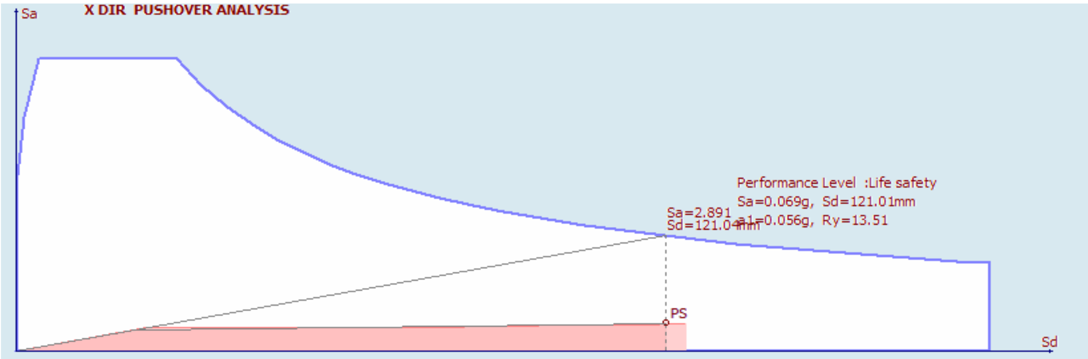


Figure 5.24: Performance Level of Case 4 (Soft Storey 3F) TEC-2007.

5.2.5 Case 5 (Projection in Plan):

5.2.5.1 TEC-1975 (5F)

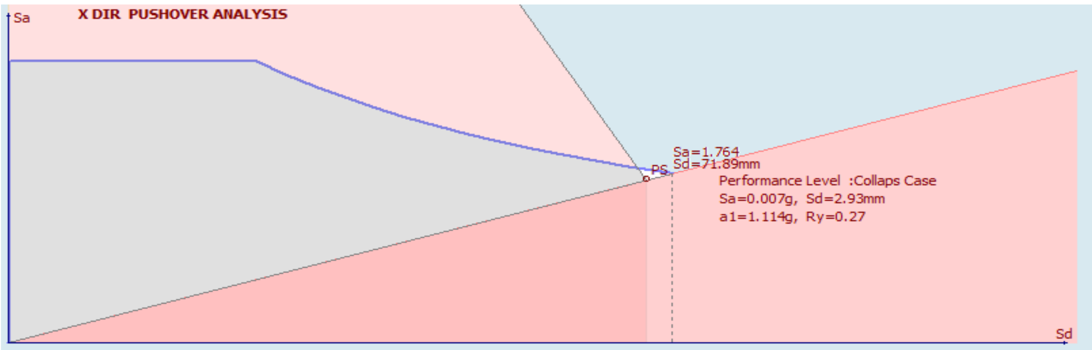


Figure 5.25: Performance Level of Case 5 (Projection in Plan 5F) TEC-1975.

5.2.5.2 TEC-1998 (5F)

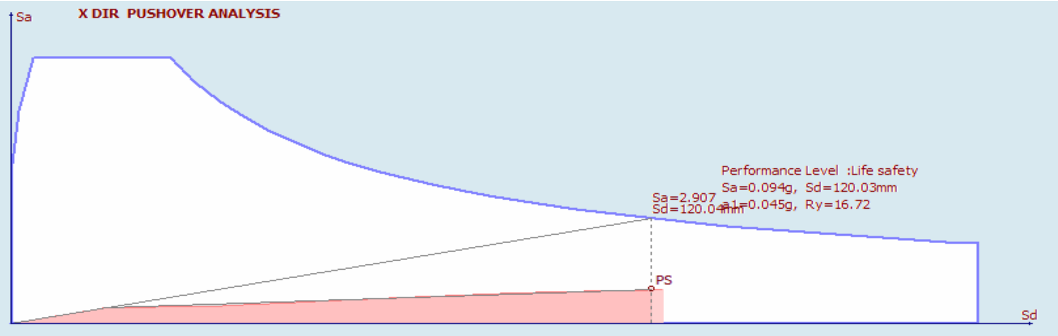


Figure 5.26: Performance Level of Case 5 (Projection in Plan 5F) TEC-1998.

5.2.5.3 TEC-2007 (5F)

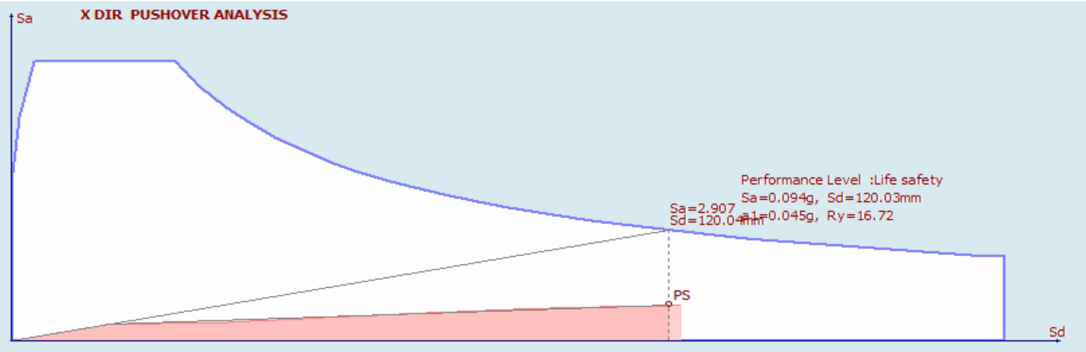


Figure 5.27: Performance Level of Case 5 (Projection in Plan 5F) TEC-2007.

5.2.5.4 TEC-1975 (3F)

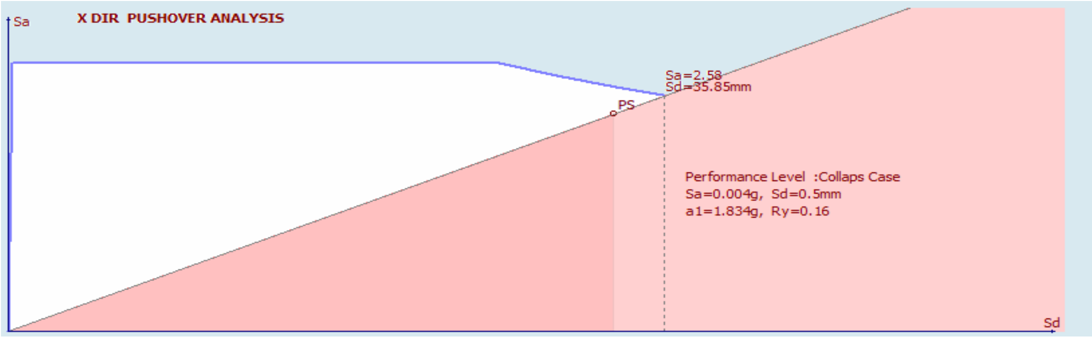


Figure 5.28: Performance Level of Case 5 (Projection in Plan 3F) TEC-1975.

5.2.5.5 TEC-1998 (3F)

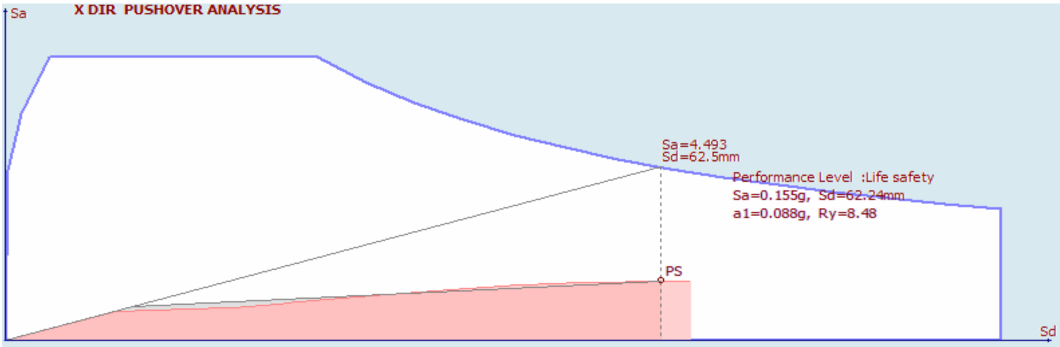


Figure 5.29: Performance Level of Case 5 (Projection in Plan 3F) TEC-1998.

5.2.5.6 TEC-2007 (3F)

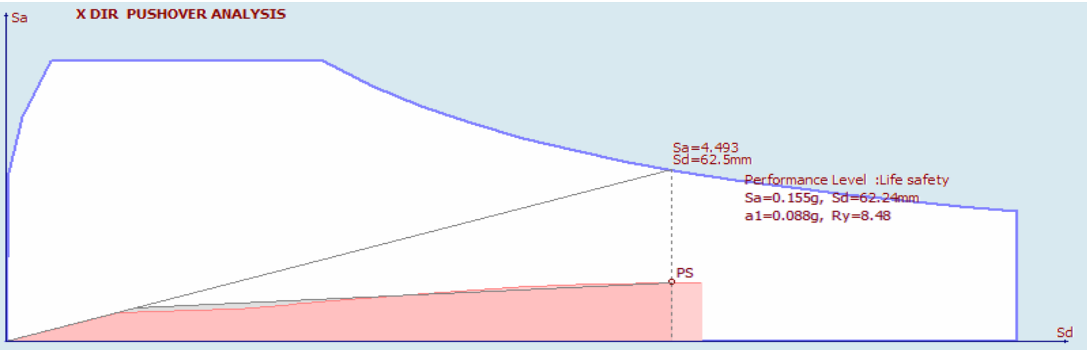


Figure 5.30: Performance Level of Case 5 (Projection in Plan 3F) TEC-2007.

5.2.6 Case 6 (Torsional Irregularity)

5.2.6.1 TEC-1975 (5F)

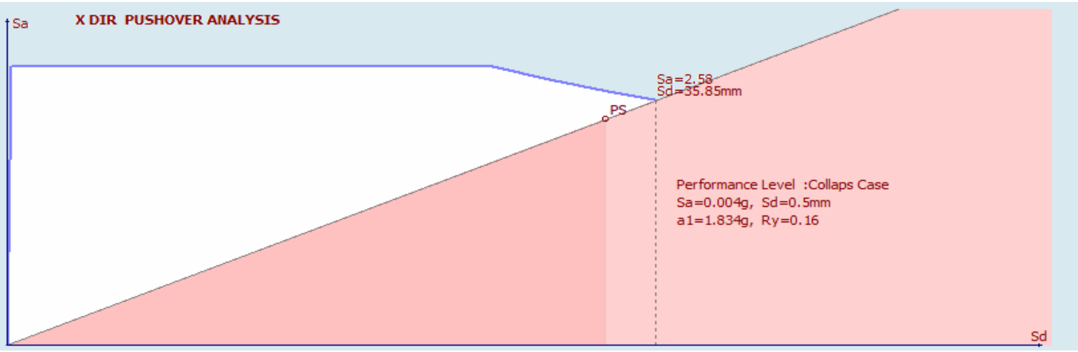


Figure 5.31: Performance Level of Case 6 (Torsional Irregularity 5F) TEC-1975.

5.2.6.2 TEC-1998 (5F)

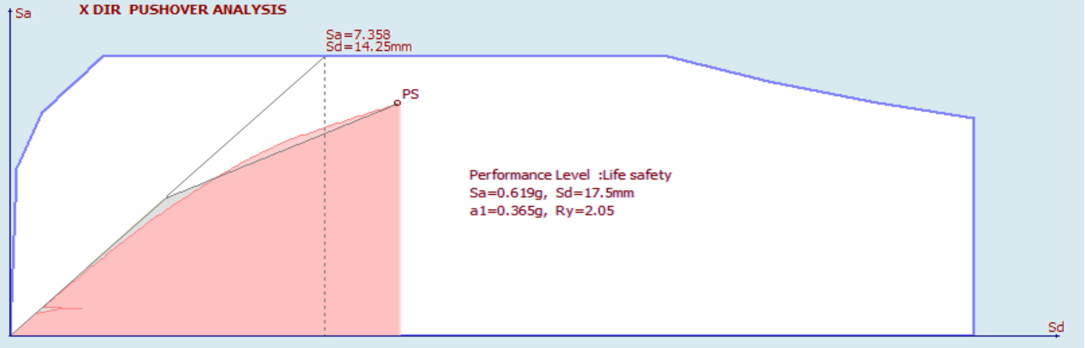


Figure 5.32: Performance Level of Case 6 (Torsional Irregularity 5F) TEC-1998.

5.2.6.3 TEC-2007 (5F)

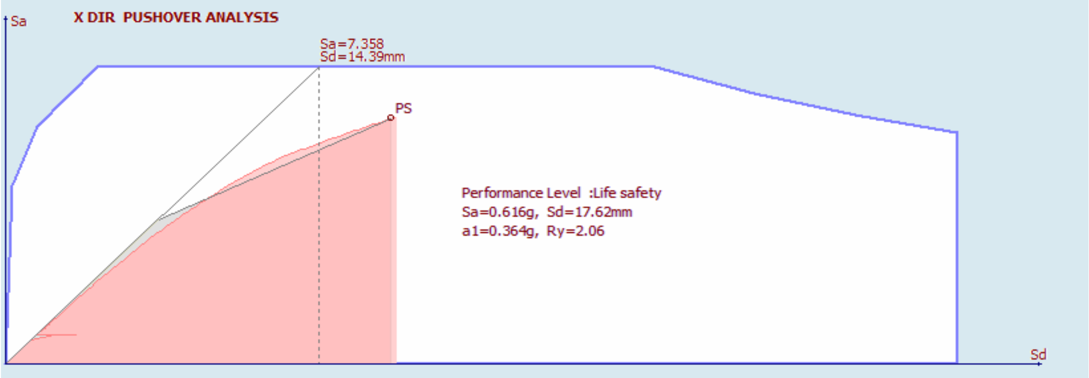


Figure 5.33: Performance Level of Case 6 (Torsional Irregularity 5F) TEC-2007.

5.2.6.4 TEC-1975 (3F)

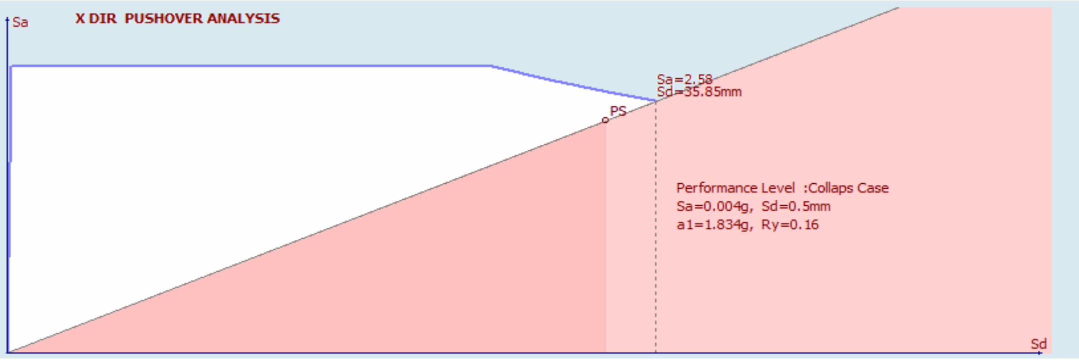


Figure 5.34: Performance Level of Case 6 (Torsional Irregularity 3F) TEC-1975.

5.2.6.5 TEC-1998 (3F)

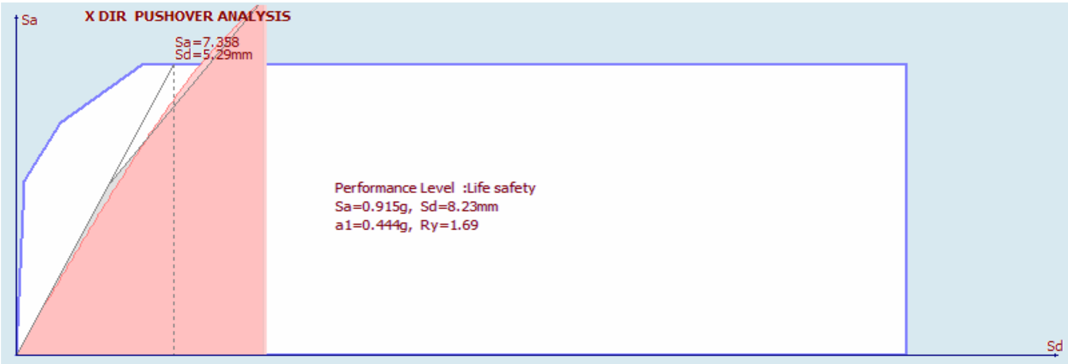


Figure 5.35: Performance Level of Case 6 (Torsional Irregularity 3F) TEC-1998.

5.2.6.6 TEC-2007 (3F)

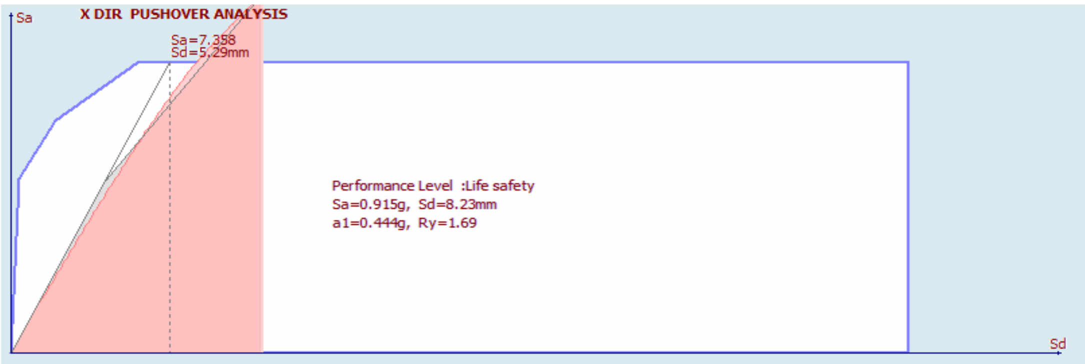


Figure 5.36: Performance Level of Case 6 (Torsional Irregularity 3F) TEC-2007.

5.3 Capacity Curves

Capacity curves were conducted up to performance point to see if any differences are available. The curves are shown in this section.

5.3.1 Case 1 (Regular Building 1)

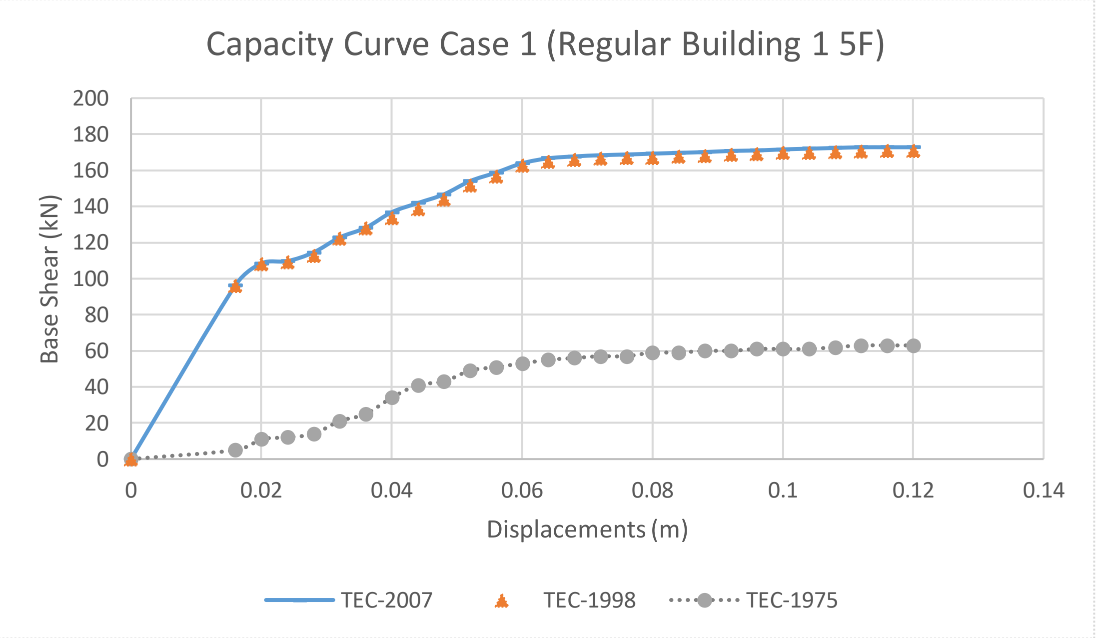


Figure 5.37: Capacity Curve Case 1 (Regular Building 1 (5F)).

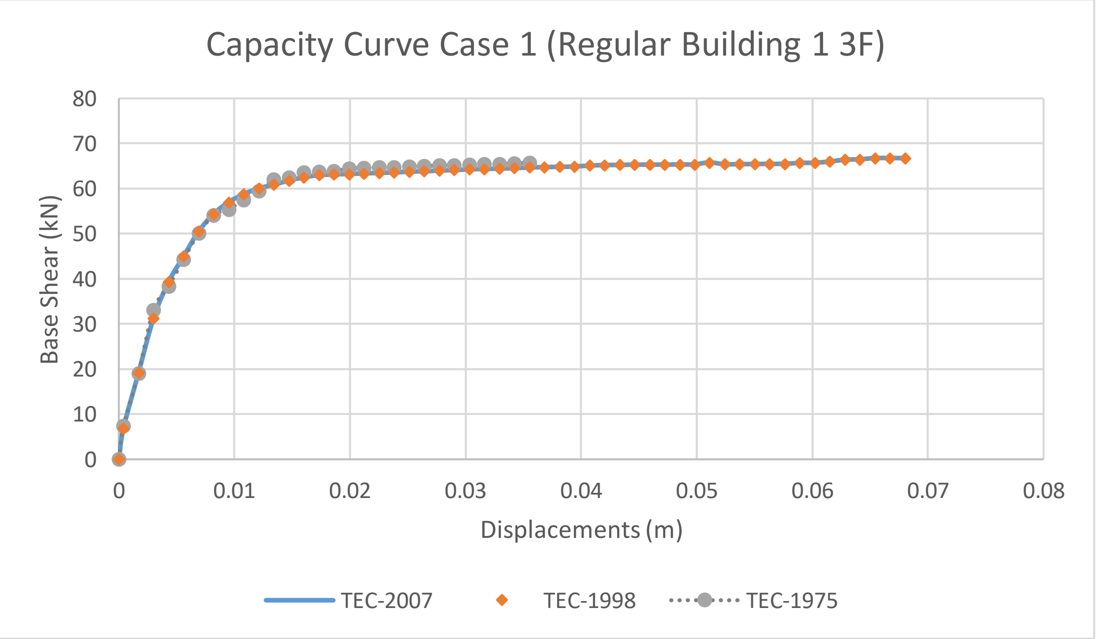


Figure 5.38: Capacity Curve Case 1 (Regular Building 1 (3F)).

- TEC-2007 and TEC-1998 cases reached Life Safety category in both 5F and 3F cases. TEC-1975 reached Collapse category in 5F case and Life Safety category in 3F case.

- TEC-2007 and TEC-1998 have the same capacity curves in both elevations.

- The following table shows the base shear and displacement when the structure reaches Immediate Occupancy levels and performance levels, as observed in the capacity curves.

Table 5.1: Base Shear and Displacement for Case 1

Code	Performance Level	Base Shear	Displacement
2007 5F	IO	101.300	0.0170140
	LS	172.783	0.1117350
2007 3F	IO	4.896	0.0004226
	LS	63.770	0.0679003
1998 5F	IO	101.300	0.0170140
	LS	172.783	0.1117350
1998 3F	IO	4.896	0.0004226
	LS	63.770	0.0679003
1975 5F	IO	14.122	0.0223147
	C	62.845	0.1124277
1975 3F	IO	17.273	0.0014901
	LS	65.754	0.0358976

5.3.2 Case 2 (Regular Building 2):

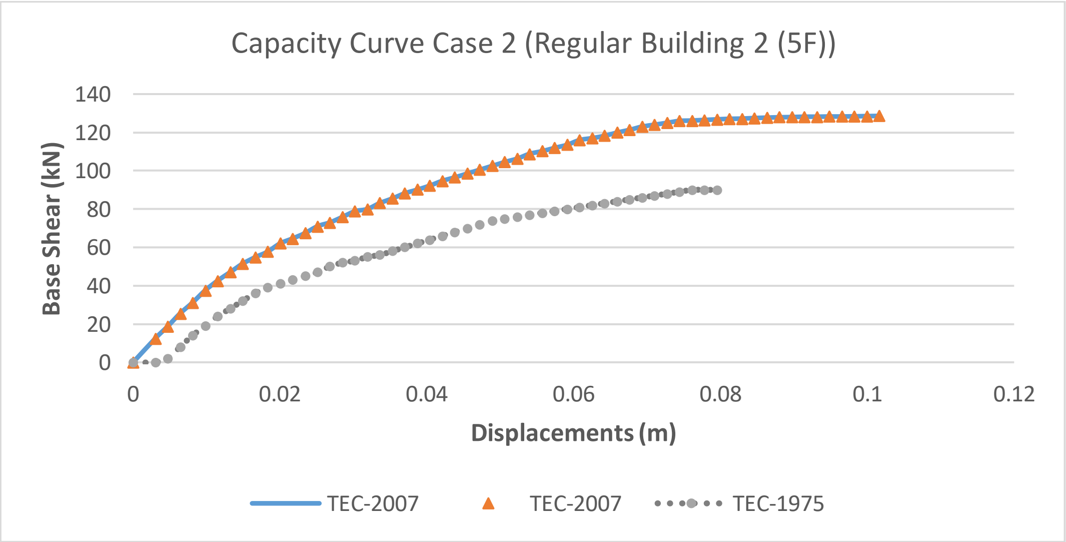


Figure 5.39: Capacity Curve Case 2 (Regular Building 2 (5F)).

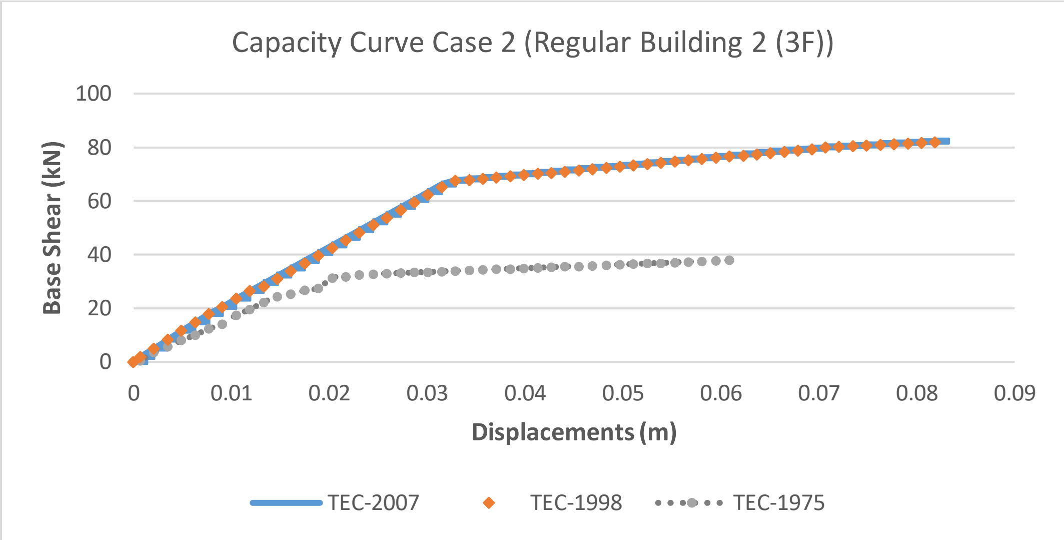


Figure 5.40: Capacity Curve Case 2 (Regular Building 2 (3F)).

- TEC-2007 and TEC-1998 cases reached Life Safety category in both 5F and 3F cases. TEC-1975 reached Collapse category in 5F case and Collapse Prevention category in 3F case.

- TEC-2007 and TEC-1998 have the same capacity curves in both elevations.

- The following table shows the base shear and displacement when the structure reaches Immediate Occupancy levels and performance levels, as observed in the capacity curves.

Table 5.2: Base Shear and Displacement for Case 2

Code	Performance Level	Base Shear	Displacement
2007 5F	IO	12.381	0.0029638
	LS	128.658	0.1001099
2007 3F	IO	1.925	0.0007211
	LS	85.196	0.0962125
1998 5F	IO	12.381	0.0029638
	LS	128.658	0.1001099
1998 3F	IO	1.925	0.0007211
	LS	85.196	0.0962125
1975 5F	IO	7.995	0.0047657

	C	62.845	0.1124237
1975 3F	IO	14.233	0.0091237
	CP	42.821	0.0934341

5.3.3 Case 3 (Weak Storey):

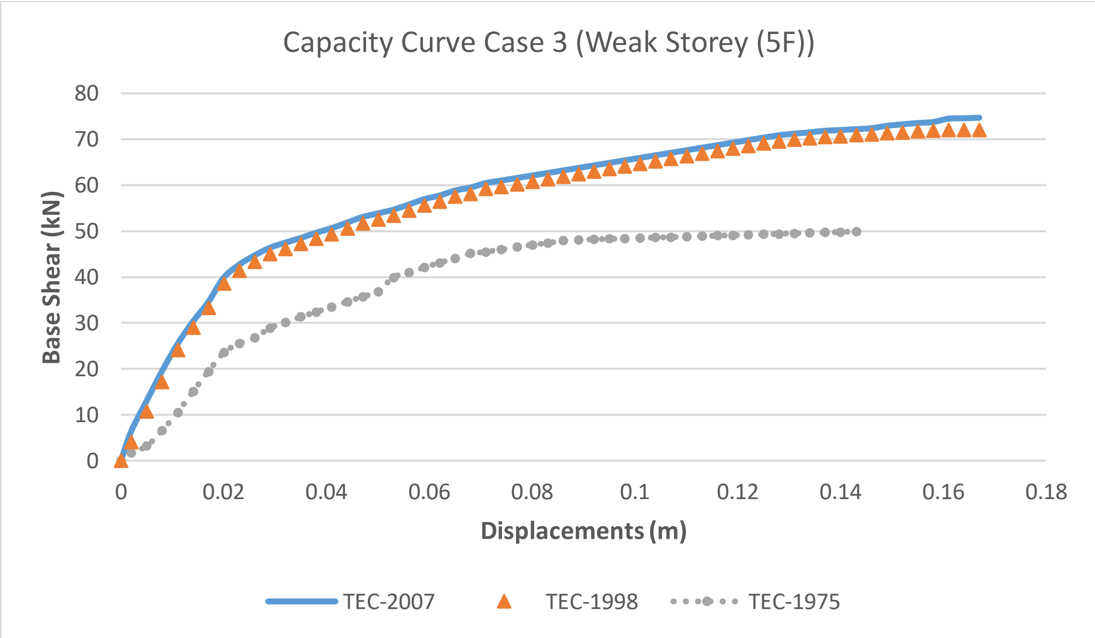


Figure 5.41: Capacity Curve Case 3 (Weak Storey (5F)).

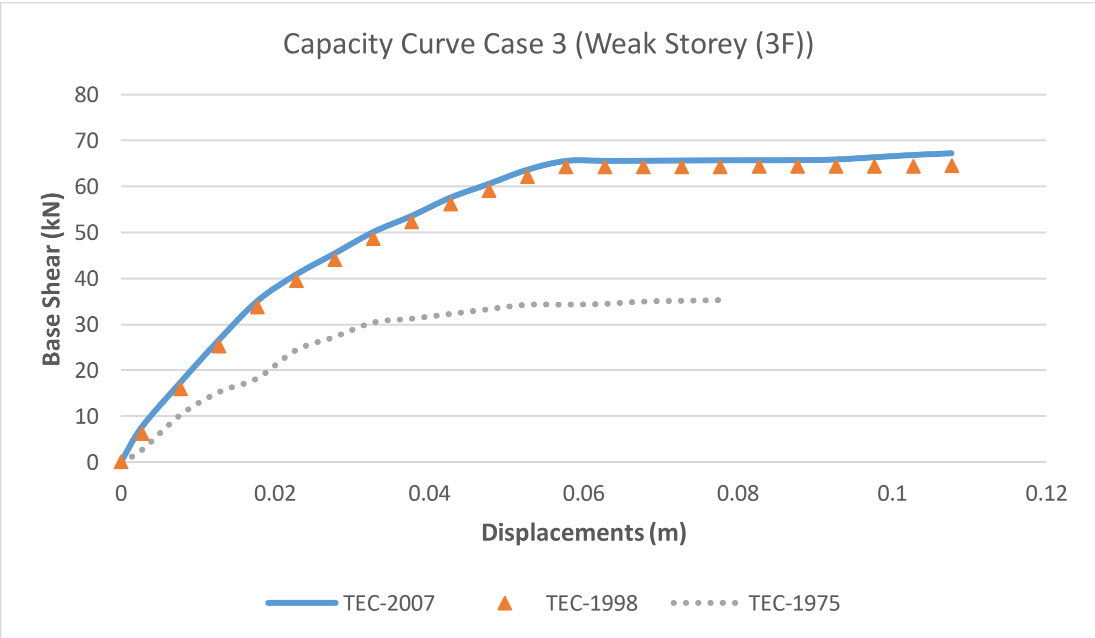


Figure 5.42: Capacity Curve Case 3 (Weak Storey (3F)).

- TEC-2007 and TEC-1998 cases reached Life Safety category in both 5F and 3F cases. TEC-1975 reached Collapse Prevention category in both 5F and 3F cases.

- TEC-2007 has different capacity curve than TEC-1998 due structural behaviour factor *(R)* being changed according how each code deal with weak storey irregularity. TEC-2007 states when the value of $\eta_{ci} < 0.8$ the behaviour factor should be multiplied by 1.25. While in TEC-1998 it states that when the value of $\eta_{ci} < 0.8$ the behaviour factor should be multiplied by 1.2. That can be noticed in both elevations.

- The following table shows the base shear and displacement when the structure reaches Immediate Occupancy levels and performance levels, as observed in the capacity curves.

Table 5.3: Base Shear and Displacement for Case 3

Code	**Performance Level**	**Base Shear**	**Displacement**
2007 5F	IO	9.967	0.0042812
	LS	74.696	0.1657133
2007 3F	IO	7.994	0.0028638
	LS	67.229	0.1060936
1998 5F	IO	9.967	0.0042813
	LS	73.696	0.1657121
1998 3F	IO	7.988	0.0028634
	LS	65.929	0.1065793
1975 5F	IO	3.221	0.0055414
	CP	49.994	0.1435424
1975 3F	IO	10.236	0.0077137
	CP	35.255	0.0773657

5.3.4 Case 4 (Soft Storey):

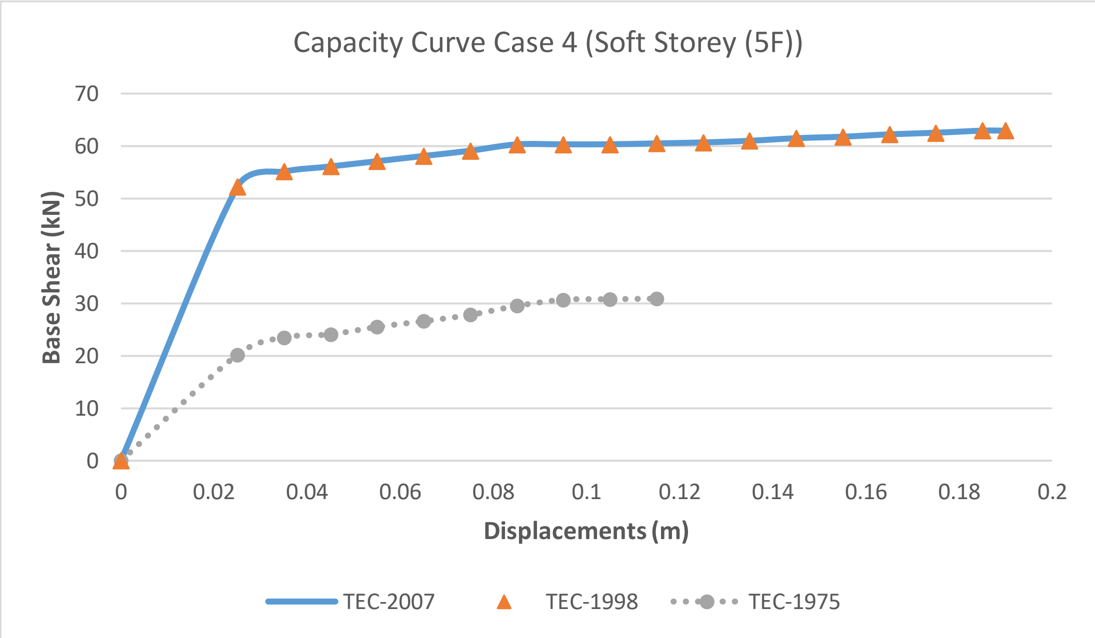


Figure 5.43: Capacity Curve Case 4 (Soft Storey (5F)).

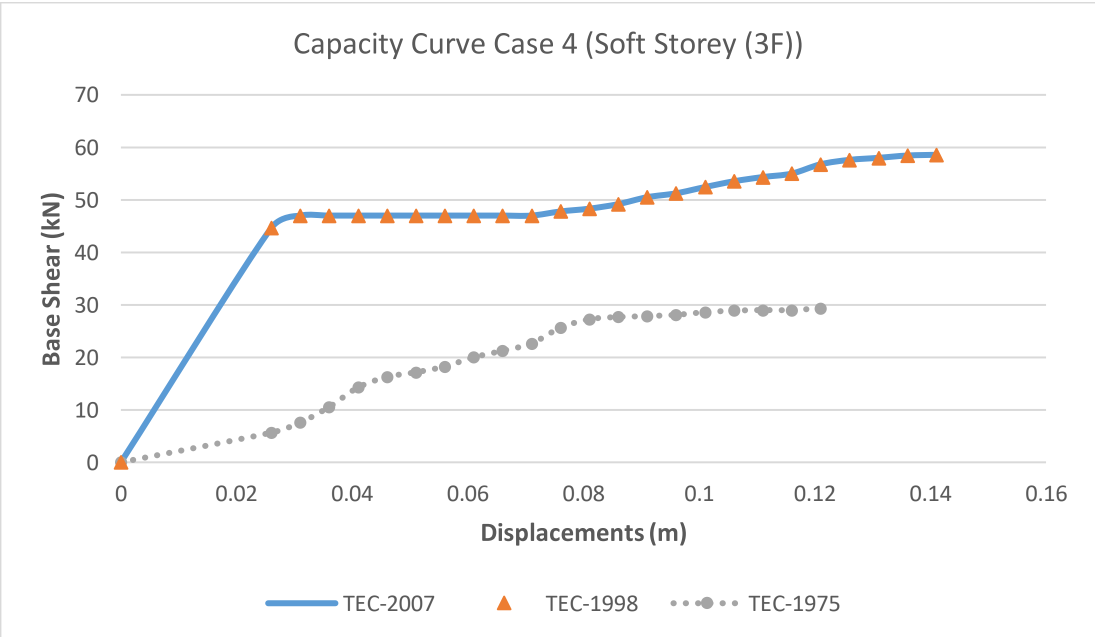


Figure 5.44: Capacity Curve Case 4 (Soft Storey (3F)).

- TEC-2007 and TEC-1998 cases reached Life Safety category in both 5F and 3F cases. TEC-1975 reached Collapse category in both 5F and 3F cases.

- TEC-2007 and TEC-1998 have the same capacity curve in both elevations.

- The following table shows the base shear and displacement when the structure reaches Immediate Occupancy levels and performance levels, as observed in the capacity curves.

Table 5.4: Base Shear and Displacement for Case 4

Code	Performance Level	Base Shear	Displacement
2007 5F	IO	53.918	0.0271013
	LS	63.424	0.1876469
2007 3F	IO	46.987	0.0275962
	LS	58.580	0.1399021
1998 5F	IO	53.918	0.0271013
	LS	63.424	0.1876469
1998 3F	IO	46.987	0.0275962
	LS	58.580	0.1399021
1975 5F	IO	20.233	0.0251532
	C	30.963	0.1846522
1975 3F	IO	7.584	0.0312454
	C	29.121	0.1213472

5.3.5 Case 5 (Projection in Plan):

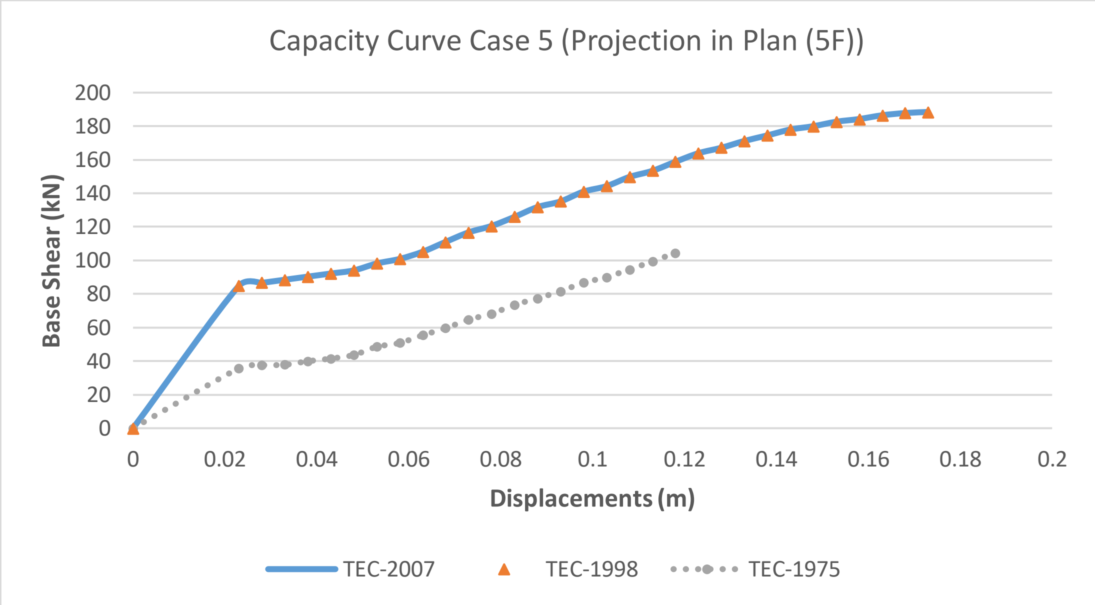


Figure 5.45: Capacity Curve Case 5 (Projection in Plan (5F)).

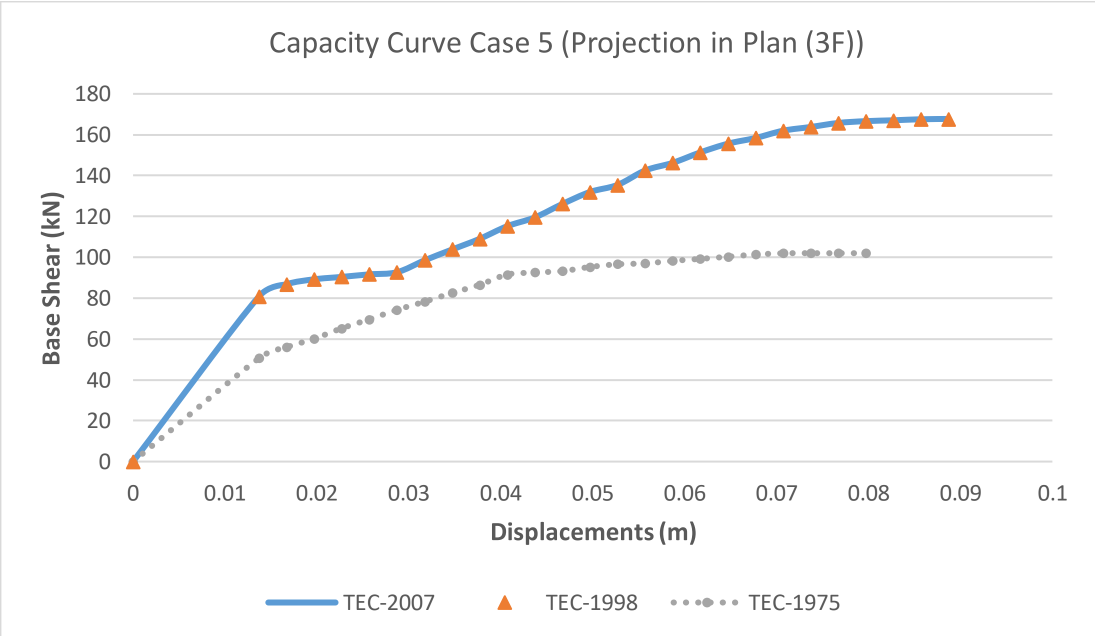


Figure 5.46: Capacity Curve Case 5 (Projection in Plan (3F)).

- TEC-2007 and TEC-1998 cases reached Life Safety category in both 5F and 3F cases. TEC-1975 reached Collapse category in both 5F and 3F cases.

- TEC-2007 and TEC-1998 have the same capacity curves in both elevations.

- The following table shows the base shear and displacement when the structure reaches Immediate Occupancy levels and performance levels, as observed in the capacity curves.

Table 5.5: Base Shear and Displacement for Case 5

Code	Performance Level	Base Shear	Displacement
2007 5F	IO	89.225	0.0259387
	LS	188.464	0.1705609
2007 3F	IO	83.325	0.0144381
	LS	167.124	0.0870029
1998 5F	IO	89.225	0.0259387
	LS	188.464	0.1705609
1998 3F	IO	83.325	0.0144381
	LS	167.124	0.0870029
1975 5F	IO	39.953	0.0382521
	C	104.523	0.1708592
1975 3F	IO	55.963	0.0167581
	C	101.964	0.7378975

5.3.6 Case 6 (Torsional Irregularity):

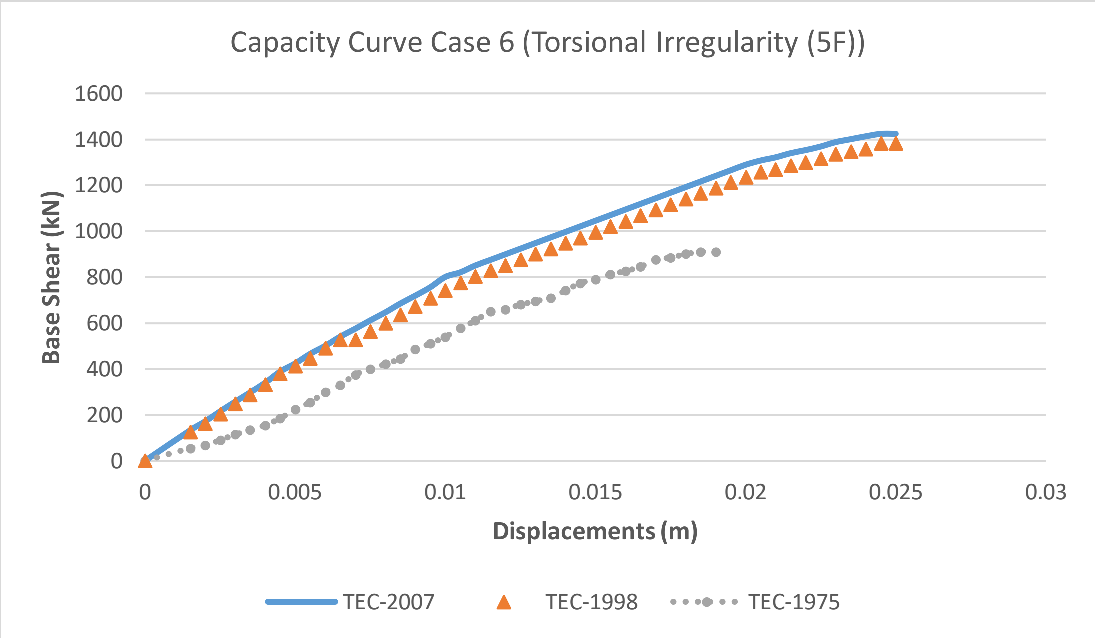


Figure 5.47: Capacity Curve Case 6 (Torsional Irregularity (5F)).

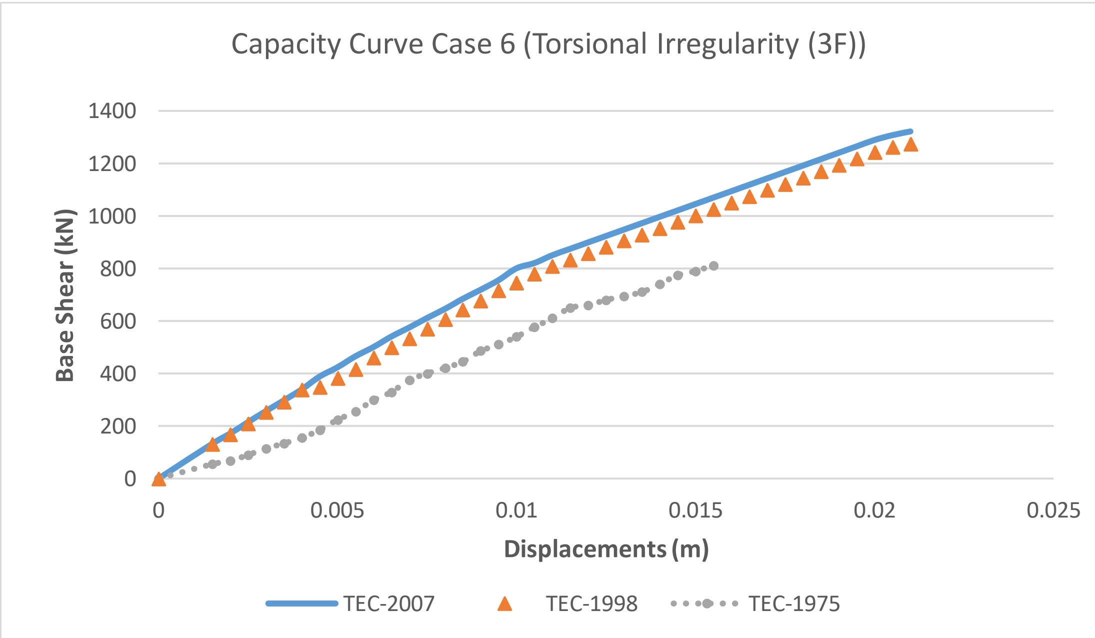


Figure 5.48: Capacity Curve Case 6 (Torsional Irregularity (3F)).

- TEC-2007 and TEC-1998 cases reached Life Safety category in both 5F and 3F cases. TEC-1975 reached Collapse category in both 5F and 3F cases.

- TEC-2007 has different capacity curve than TEC-1998 due eccentricity being changed according how each code deal with torsional irregularity. TEC-2007 states that the eccentricity should be multiplied by a factor Di= $(\eta_{bi} / 1.2)^{0.5}$. While in TEC-1998 it states that the eccentricity should be multiplied value by a factor Di= $(\eta_{bi} / 1.2)^{2}$. That can be noticed in both elevations.

- The following table shows the base shear and displacement when the structure reaches Immediate Occupancy levels and performance levels, as observed in the capacity curves.

Table 5.6: Base Shear and Displacement for Case 6

Code	Performance Level	Base Shear	Displacement
2007 5F	IO	132.612	0.0015221
	LS	1423.699	0.0243978
2007 3F	IO	159.274	0.0011353
	LS	1159.424	0.0107333
1998 5F	IO	130.040	0.0015069
	LS	1388.830	0.0247543
1998 3F	IO	156.274	0.0011353
	LS	1153.424	0.0107333
1975 5F	IO	224.423	0.0055275
	C	910.577	0.0185635
1975 3F	IO	163.041	0.0018295
	C	806.351	0.0073274

5.3.7 Discussion about Capacity Curves:

TEC-2007 and TEC-1998 cases have reached Life Safety category. It can be observed that their cases have also reached similar displacement and base shear values in Cases 1,2,4, and 5. In cases 3 and 6 there was a small difference in the values achieved due to how each code deal with those types of irregularities. In weak storey, TEC-2007 states, when the value of $\eta_{ci} < 0.8$ the behaviour factor should be multiplied by 1.25.

While in TEC-1998 it states that when the value of $\eta_{ci} < 0.8$ the behaviour factor should be multiplied by 1.2. In torsional irregularity, TEC-2007 states that the eccentricity should be multiplied by a factor Di= $(\eta_{bi} / 1.2)^{0.5}$. While in TEC-1998 it states that the eccentricity should be multiplied value by a factor Di= $(\eta_{bi} / 1.2)^{2}$. TEC-1975 cases displacements and base shear values were low due to falling into either Collapse or Collapse Prevention categories. It can be drawn that steel reinforcements, irregularity checks, seismic safety regulations in TEC-2007 plays a positive role in the seismic performance of the structure.

5.4 Damage Reports

The damage percentages sustained by cases under seismic action throughout the analysis are presented in this section.

5.4.1 Case 1 (Regular Building 1)

Table 5.7: Damage Report for Case 1.

Damage % of Case 1 (Regular Building 1)						
Seismic Code	**2007**		**1998**		**1975**	
Elevation	**Beams**	**Columns**	**Beams**	**Columns**	**Beams**	**Columns**
5F	17.9	0	17.9	0	100	78
3F	13.9	0	13.9	0	27.3	0

5.4.2 Case 2 (Regular Building 2)

Table 5.8: Damage Report for Case 2.

Damage % of Case 2 (Regular Building 2)						
Seismic Code	**2007**		**1998**		**1975**	
Elevation	**Beams**	**Columns**	**Beams**	**Columns**	**Beams**	**Columns**
5F	18.2	0	18.2	0	100	76
3F	16.4	0	16.4	0	100	66

5.4.3 Case 3 (Weak Storey)

Table 5.9: Damage Report for Case 3.

Damage % of Case 3 (Weak Storey)						
Seismic Code	**2007**		**1998**		**1975**	
Elevation	**Beams**	**Columns**	**Beams**	**Columns**	**Beams**	**Columns**
5F	30.0	0	30.4	0	100	78
3F	33.0	0	33.1	0	100	75

5.4.4 Case 4 (Soft Storey)

Table 5.10: Damage Report for Case 4.

Damage % of Case 4 (Soft Storey)						
Seismic Code	**2007**		**1998**		**1975**	
Elevation	**Beams**	**Columns**	**Beams**	**Columns**	**Beams**	**Columns**
5F	20.0	0	20.0	0	100	71
3F	18.0	0	18.0	0	100	70

5.4.5 Case 5 (Projection in Plan)

Table 5.11: Damage Report for Case 5.

Damage % of Case 5 (Projection in Plan)						
Seismic Code	**2007**		**1998**		**1975**	
Elevation	**Beams**	**Columns**	**Beams**	**Columns**	**Beams**	**Columns**
5F	33.3	0	33.3	0	100	77.1
3F	30.1	0	30.1	0	100	74

5.4.6 Case 6 (Torsional Irregularity):

Table 5.12: Damage Report for Case 6.

Damage % of Case 6 (Torsional Irregularity)						
Seismic Code	**2007**		**1998**		**1975**	
Elevation	**Beams**	**Columns**	**Beams**	**Columns**	**Beams**	**Columns**
5F	17.5	0	18.0	0	100	75
3F	14.5	0	15.0	0	100	73

5.4.7 Discussion about Damage Percentages

The cases 1,2,4 and 5 designed according to TEC-2007 and TEC-1998 have encountered the similar damage in both elevations. In cases 3 and 6 designed according to TEC-2007 and TEC-1998, the damage was slightly different due to how each code dealing with weak storey and torsional irregularities. While on the other hand most TEC-1975 cases were completely damaged. The difference in damage between TEC-2007 and TEC-1975 is due to the higher steel reinforcement used.

5.5 Cost

A report of quantity and cost of the materials needed for each case is presented in this section.

5.5.1 Case 1 (Regular Building 1)

Table 5.13: Cost of Case 1 (5F).

Type	Cost	Quantity			Costs (USD)			Total Cost (USD)		
	USD	TEC-2007	TEC-1998	TEC-1975	TEC-2007	TEC-1998	TEC-1975	TEC-2007	TEC-1998	TEC-1975
C20 factory concrete (m^3)	75	481	481	481	36,075	36,075	36,075	98,503.9	97,483.9	90,903.9
Plain surface concrete form in (m^2)	11	3509.9	3509.9	3509.9	38,608.9	38,608.9	38,608.9			
Reinforcement steel ф 8-12 mm in (tn)	600	28.1	26.4	20.7	16,860	15,840	12,420			
Reinforcement steel ф 14-50 mm in (tn)	600	11.6	11.6	6.4	6,960	6,960	3,840			

The difference in cost between TEC-2007 and TEC- 1998 is about 1.05%. While the difference in cost between TEC-2007 and TEC-1975 is about 8%.

Table 5.14: Cost of Case 1 (3F).

Type	Cost	Quantity			Costs (USD)			Total Cost (USD)		
	USD	TEC-2007	TEC-1998	TEC-1975	TEC-2007	TEC-1998	TEC-1975	TEC-2007	TEC-1998	TEC-1975
C20 factory concrete (m^3)	75	401.6	401.6	401.6	30,120	30,120	30,120	69,394.9	69,334.9	67,834.9
Plain surface concrete form in (m^2)	11	2435.9	2435.9	2435.9	26,794.9	26,794.9	26,794.9			
Reinforcement steel ϕ 8-12 mm in (tn)	600	13.1	13	10.5	7,860	7,800	6,300			
Reinforcement steel ϕ 14-50 mm in (tn)	600	7.7	7.7	7.7	4,620	4,620	4,620			

The difference in cost between TEC-2007 and TEC- 1998 is about 0.09%. While the difference in cost between TEC-2007 and TEC-1975 is about 2.27%.

5.5.2 Case 2 (Regular Building 2)

Table 5.15: Cost of Case 2 (5F).

Type	Cost	Quantity			Costs (USD)			Total Cost (USD)		
	USD	TEC-2007	TEC-1998	TEC-1975	TEC-2007	TEC-1998	TEC-1975	TEC-2007	TEC-1998	TEC-1975
C20 factory concrete (m^3)	75	232.5	232.5	232.5	17,437.5	17,437.5	17,437.5	68,075.4	67,175.4	58,595.4
Plain surface concrete form in (m^2)	11	2268.9	2268.9	2268.9	24,957.9	24,957.9	24,957.9			
Reinforcement steel ϕ 8-12 mm in (tn)	600	28.8	27.3	21	17,280	16,380	12,600			
Reinforcement steel ϕ 14-50 mm in (tn)	600	14	14	6	8,400	8,400	3,600			

The difference in cost between TEC-2007 and TEC- 1998 is about 1.33%. While the difference in cost between both cases and TEC-1975 is about 14.7%.

Table 5.16: Cost of Case 2 (3F).

Type	Cost	Quantity			Costs (USD)			Total Cost (USD)		
	TEC-2007	TEC-1998	TEC-1975	TEC-2007	TEC-1998	TEC-1975	TEC-2007	TEC-1998	TEC-1975	TEC-2007
C20 factory concrete (m^3)	75	124.8	124.8	124.8	9,360	9,360	9,360	36,967.9	36,667.9	33,667.9
Plain surface concrete form in (m^2)	11	1238.9	1238.9	1238.9	13,627.9	13,627.9	13,627.9			
Reinforcement steel ϕ 8-12 mm in (tn)	600	15.6	15.1	11.8	9,360	9,060	12,600			
Reinforcement steel ϕ 14-50 mm in (tn)	600	7.7	7.7	6	4,620	4,620	3,600			

The difference between TEC-2007 and TEC- 1998 is about 0.08%. While the difference between TEC-2007 and TEC-1975 is about 9.3%.

5.5.3 Case 3 (Weak Storey)

Table 5.17: Cost of Case 3 (5F).

Type	Cost	Quantity			Costs (USD)			Total Cost (USD)		
	USD	TEC-2007	TEC-1998	TEC-1975	TEC-2007	TEC-1998	TEC-1975	TEC-2007	TEC-1998	TEC-1975
C20 factory concrete (m^3)	75	426	426	426	31,950	31,950	31,950	87,868.1	86,788.7	76,064.1
Plain surface concrete form in (m^2)	11	2667.1	2667.1	2667.1	29,338.1	29,338.1	29,338.1			
Reinforcement steel ϕ 8-12 mm in (tn)	600	26.7	24.8	18.1	16,020	14,880	10,860			
Reinforcement steel ϕ 14-50 mm in (tn)	600	17.6	17.7	6.5	10,560	10,620	3,900			

The difference in cost between TEC-2007 and TEC- 1998 is about 1.23%. While the difference in cost between TEC-2007 and TEC-1975 is about 14.2%.

Table 5.18: Cost of Case 3 (3F).

Type	Cost	Quantity			Costs (USD)			Total Cost (USD)		
	USD	TEC-2007	TEC-1998	TEC-1975	TEC-2007	TEC-1998	TEC-1975	TEC-2007	TEC-1998	TEC-1975
C20 factory concrete (m^3)	75	254.7	254.7	254.7	19,102.5	19,102.5	19,102.5	52,786.8	52,606.8	44,026.8
Plain surface concrete form in (m^2)	11	1611.3	1611.3	1611.3	17,724.3	17,724.3	17,724.3			
Reinforcement steel ϕ 8-12 mm in (tn)	600	15.7	15.4	9	9,420	9,240	5,400			
Reinforcement steel ϕ 14-50 mm in (tn)	600	10.9	10.9	3	6,540	6,540	1,800			

The difference in cost between TEC-2007 and TEC- 1998 is about 0.34%. While the difference in cost between TEC-2007 and TEC-1975 is about 18%.

5.5.4 Case 4 (Soft Storey)

Table 5.19: Cost of Case 4 (5F).

Type	Cost	Quantity			Costs (USD)			Total Cost (USD)		
	USD	TEC-2007	TEC-1998	TEC-1975	TEC-2007	TEC-1998	TEC-1975	TEC-2007	TEC-1998	TEC-1975
C20 factory concrete (m^3)	75	396.3	396.3	396.3	29,722.5	29,722.5	29,722.5	88,824.7	88,384.7	73,985.1
Plain surface concrete form in (m^2)	11	2496.9	2496.9	2496.9	27,462.6	27,462.6	27,462.6			
Reinforcement steel ϕ 8-12 mm in (tn)	600	29.4	28.5	21	17,640	17,100	12,600			
Reinforcement steel ϕ 14-50 mm in (tn)	600	23.4	23.5	7	14,040	14,100	4,200			

The difference in cost between TEC-2007 and TEC- 1998 is about 0.5%. While the difference between in cost both cases and TEC-1975 is about 18%

Table 5.20: Cost of Case 4 (3F).

Type	Cost	Quantity			Costs (USD)			Total Cost (USD)		
	USD	TEC-2007	TEC-1998	TEC-1975	TEC-2007	TEC-1998	TEC-1975	TEC-2007	TEC-1998	TEC-1975
C20 factory concrete (m^3)	75	234.5	234.5	234.5	17,587.5	17,587.5	17,587.5	53,996.5	53,696.5	43,556.5
Plain surface concrete form in (m^2)	11	1499	1499	1499	16,489	16,489	16,489			
Reinforcement steel ϕ 8-12 mm in (tn)	600	18.4	17.9	12.8	11,040	10,740	7,680			
Reinforcement steel ϕ 14-50 mm in (tn)	600	14.8	14.8	3	8,800	8,800	1,800			

The difference in cost between TEC-2007 and TEC- 1998 is about 0.55%. While the difference in cost between TEC-2007 and TEC-1975 is about 21%.

5.5.5 Case 5 (Projection in Plan)

Table 5.21: Cost of Case 5 (5F).

Type	Cost	Quantity			Costs (USD)			Total Cost (USD)		
	USD	TEC-2007	TEC-1998	TEC-1975	TEC-2007	TEC-1998	TEC-1975	TEC-2007	TEC-1998	TEC-1975
C20 factory concrete (m^3)	75	625.2	625.2	625.2	46,890	46,890	46,890	152,951.5	152,171.5	130,051.5
Plain surface concrete form in (m^2)	11	3696.5	3696.5	3696.5	40,661.5	40,661.5	40,661.5			
Reinforcement steel ϕ 8-12 mm in (tn)	600	44.5	43.1	32.5	26,700	25,860	19,500			
Reinforcement steel ϕ 14-50 mm in (tn)	600	64.5	64.6	35	38,700	38,760	21,000			

The difference in cost between TEC-2007 and TEC- 1998 is about 0.51%. While the difference in cost between TEC-2007 and TEC-1975 is about 16%.

Table 5.22: Cost of Case 5 (3F).

Type	Cost	Quantity			Costs (USD)			Total Cost (USD)		
	USD	TEC-2007	TEC-1998	TEC-1975	TEC-2007	TEC-1998	TEC-1975	TEC-2007	TEC-1998	TEC-1975
C20 factory concrete (m^3)	75	328.1	328.1	328.1	24,607.5	24,607.5	24,607.5	79,818.7	79,578.7	64,278.7
Plain surface concrete form in (m^2)	11	2019.2	2019.2	2019.2	22,211.2	22,211.2	22,211.2			
Reinforcement steel ϕ 8-12 mm in (tn)	600	21.3	20.9	17.1	12,780	12,540	10,260			
Reinforcement steel ϕ 14-50 mm in (tn)	600	33.7	33.7	12	20,220	20,220	7,200			

The difference in cost between TEC-2007 and TEC- 1998 is about 0.31%. While the difference in cost between TEC-2007 cases and TEC-1975 is about 21.5%.

5.5.6 Case 6 (Torsional Irregularity)

Table 5.23: Cost of Case 6 (5F).

Type	Cost	Quantity			Costs (USD)			Total Cost (USD)		
	USD	TEC-2007	TEC-1998	TEC-1975	TEC-2007	TEC-1998	TEC-1975	TEC-2007	TEC-1998	TEC-1975
C20 factory concrete (m^3)	75	725	725	725	54,372.40	54,372.40	54,372.40	142,028.25	140,888.25	133,508.25
Plain surface concrete form in (m^2)	11	4505.1	4505.1	4505.1	49,555.85	49,555.85	49,555.85			
Reinforcement steel ϕ 8-12 mm in (tn)	600	47.1	45.1	42.1	28,260	27,060	25,260			
Reinforcement steel ϕ 14-50 mm in (tn)	600	16.4	16.5	7.2	9,840	9,900	4,320			

The difference in cost between TEC-2007 and TEC- 1998 is about 0.8%. While the difference in cost between TEC-2007 and TEC-1975 is about 6.2%.

Table 5.24: Cost of Case (3F).

Type	Cost	Quantity			Costs (USD)			Total Cost (USD)		
	USD	TEC-2007	TEC-1998	TEC-1975	TEC-2007	TEC-1998	TEC-1975	TEC-2007	TEC-1998	TEC-1975
C20 factory concrete (m^3)	75	436.1	436.1	436.1	32,709.36	32,709.36	32,709.36	85,229.16	84,149.16	79,289.16
Plain surface concrete form in (m^2)	11	2707.3	2707.3	2707.3	29,779.8	29,779.8	29,779.8			
Reinforcement steel ϕ 8-12 mm in (tn)	600	28.3	26.5	22	16,980	15,900	13,200			
Reinforcement steel ϕ 14-50 mm in (tn)	600	9.6	9.6	6	5,760	5,760	3,600			

The difference in cost between TEC-2007 and TEC- 1998 is about 1.27%. While the difference in cost between TEC-2007 and TEC-1975 is about 7.2%.

5.5.7 Discussion about Cost

The cases designed according to TEC-2007 are costing more than the cases designed according to TEC-1998 in the range of 0.08% to 1.3%. That is due to the amount of steel reinforcement used. Thus, yielding that the TEC-1998 is slightly more economical. The TEC-1975 cases are all costing less than the other codes that is due to lack of steel reinforcements in design.

Chapter 6

CONCLUSION AND RECOMMENDATION FOR FUTURE WORK

6.1 Conclusion

Turkey has been subjected to several disastrous earthquakes throughout the time which caused a huge loss in lives and economy. That is mainly due to its geographic location which happens to be on the Anatolian Fault. Since the disastrous 1939 Erzincan Earthquake, the first set of explicit legal provisions for earthquake-resistant design was established in 1940 by the Ministry of Public Works followed by many improvements and safety regulations up to the latest code which was published in 2007. In this study, three of Turkey's major seismic codes were compared, TEC-1975, TEC-1998, and TEC-2007; in respect to performance, capacity curves, damage percentage, and cost. There were some differences observed in the analysis results and the following has been concluded:

1. Performance curves: TEC-2007 and TEC-1998 cases have reached Life Safety category in both elevations. TEC-1975 cases were either in Collapse or Collapse Prevention category in both elevations. Except for Case 1 in 3 storey elevation, it achieved Life Safety category. It can be drawn that steel reinforcements, irregularity checks, seismic safety regulations in TEC-2007 plays a positive role in the seismic performance of the structure. The main reason observed in this study was the difference in steel reinforcement in TEC-2007 and TEC-1975. The difference in longitudinal reinforcements between TEC-2007 and TEC-

1975 is calculated in the range between 3% up to 7%, and in the range between 26% up to 51% in transverse reinforcements. That can be observed in the cost tables for all cases in both elevations.

2. Capacity curves: According to the analysis results, TEC-2007 and TEC-1998 have reached similar displacement and base shear values in Cases 1,2,4, and 5. In cases 3 and 6 there was a small difference in the values achieved due to how each code deal with those types of irregularities. In weak storey, TEC-2007 states, when the value of $\eta_{ci} < 0.8$ the behaviour factor should be multiplied by 1.25. While in TEC-1998 it states that when the value of $\eta_{ci} < 0.8$ the behaviour factor should be multiplied by 1.2. In torsional irregularity, TEC-2007 states that the eccentricity should be multiplied by a factor $Di= (\eta_{bi} / 1.2)^{0.5}$. While in TEC-1998 it states that the eccentricity should be multiplied value by a factor $Di= (\eta_{bi} / 1.2)^{2}$. TEC-1975 cases displacements and base shear values were low due to falling into either Collapse or Collapse Prevention categories. Concluding that reinforcing steel governs in determining seismic performance.

3. Damage percentage: According to the analysis results it can be concluded that the TEC-1975 damage percentages are high, that is due to most cases falling in the Collapse criteria. While on the other hand, the TEC-1998 and TEC-2007 both have similar damage percentages since all their cases fell in the Life Safety criteria. That is shown in the 5F and 3F elevations.

4. Cost: The higher reinforcements used in the TEC-2007 cases demand for more reinforcement steel which why it slightly more expensive in the range of 0.08% up to 1.3% higher than the TEC-1998 cases. It can be concluded that the TEC-

1998 is a little more economical than the TEC-2007 in both 3F and 5F cases. The TEC-1975 cases were all costing less in the range of 5% up to 21%, that is due to lack of needed steel reinforcements.

To sum it up, the analysis results obtained from conducting this study showed how the TEC-1975 is dangerous to be following in most designs nowadays, despite regular case of 3F being safe according to TEC-1975, all irregular buildings were in the collapse level of safety. TEC-2007 and TEC-1998 cases have reached similar results in both elevations. Both achieved life safety category with minor differences in Cases 3 and 6 that is due to how each code deal with weak storey and torsional irregularities. Although the TEC-1998 seemed more economical up 1.3% due to the volumetric ratio of transverse reinforcement equation being different in both codes. It also can be concluded that elevation plays a role in safety, and cost. The higher the elevation; higher cost is expected and higher safety risk. Keeping in mind the huge building stock in Turkey and Northern Cyprus is still designed according to TEC-1975, which are redeemed as not safe because of the seismic activity in the area. It is worth analysing them and maybe retrofit in order to save lives if a catastrophe happens.

6.2 Recommendation for Future Studies

Considering the work done in this study and the results obtained, these are future advices and proposal for future research work:

1. Building with high rise elevation can be investigated.
2. Working with different type of irregularities that weren't covered in this study.
3. Working on a different method of analysis
4. Working on a different seismic zone.

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