A Comparison between the 2007 Turkish Earthquake Code and the Eurocode 8 for Sample Buildings

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ABSTRACT

Earthquake are a natural phenomenon caused by the shifting of tectonic plate in the crest layer of the earth. Based on its magnitude it can cause a catastrophic effect on structures which expose people to losses in lives and money.

The 2007 Turkish Earthquake Code and the Eurocode 8 are among many design codes that are concerned in the safety of buildings from future earthquakes. In this thesis, the 2007 Turkish Earthquake Code and Eurocode 8 are compared. Five different cases were chosen and designed, with each case study containing different type of irregularies. For the sake of evaluating the designed structure with regards to earthquake, the non-linear static pushover analysis method presented in TEC-2007 was chosen for 3 floor and 5 floor buildings. Finally the performance, the cost and damage percentage of each Eurocode 8 case with its 2007 Turkish Earthquake Code counterpart have been compared using three different analysis cases which represent different combination of spectrum, A_0 and behavior factor. At the end each case was compared to find out the performance of each code in the event of an earthquake.

Keywords: Earthquake, Eurocode 8, 2007 Turkish Earthquake Code, non-linear static pushover, performance, cost, damage percentage.

ÖZ

Deprem yer kabuğu içindeki kırılmalar sebebi ile ani olarak ortaya çıkan titreşimlerin dalgalar halinde yayılarak yer yüzeyini sarsması olayıdır. Büyüklüğüne göre, yapılar üzerindeki yıkıcı etkisinden dolayı can ve mal kaybına sebep olabilmektedir.

2007 Türk Deprem Yönetmeliği ve Eurocode 8 diğer deprem yönetmeliklerinde olduğu gibi yapıların depremden kaynaklanan gelecekteki güvenliği için tasarlanmıştır. Bu çalışmada 2007 Türk Deprem Yönetmeliği ile Eurocode 8 karşılaştırılmıştır. Bu maksatla her biri farklı yapısal düzensizliğe sahip beş yapı seçilmiştir. Tasarlanan yapıların deprem açısından yapısal performanslarının değerlendirilebilmesi için 2007 Türk Deprem Yönetmeliğinde sunulan statik itme analiz yöntemi kullanılmıştır. Sonuç olarak yapıların performansları, hasar yüzdesi ve maliyetleri 2007 Türk Deprem Yönetmeliği ve Eurocode 8 yönetmelikleri kullanılarak farklı tasarım spektrumu, etkin yer ivmesi katsayısı, taşıyıcı sistem davranış katsayısı ve farklı kat sayıları kombinasyonlarına göre karşılaştırılmıştır.

Anahtar kelimeler: Deprem, Eurocode 8, 2007 Türk Deprem Yönetmeliği, statik itme analizi, performans, maliyet, hasar yüzdesi.

DEDICATION

In Dedication to

My Parents for nursing me with affections and love

To my lovely Siblings

To my Precious Fiancée

To my Dearest Friends

For their Affection, Encouragement and Everlasting Faith in Me

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LIST OF ABBREVIATIONS

EC8	Eurocode 8 (design of structure for earthquake resistance)
TEC-2007	2007 Turkish Earthquake Code 2007
TS-500	requirements for design and construction of reinforced concrete Buildings.
DCH	High ductility building member
DCM	Medium ductility building member
DCL	Low ductility building member
NDL	Nominal ductility building level
HDL	High ductility building level
3F	Three floor buildin
5F	Five floor building
Floor D.	Floor discontinuity
USD	United State Dollars
m	Meter
tn	Tons

LIST OF SYMBOLS

c_u	Undrained shear strength of soil.
$v_{s\,30}$	Average value of propagation velocity of S waves in the upper 30 m of the soil. Profile at shear strain of 10–5 or less.
N_{SPT}	Standard penetration test blow-count.
T	Vibration period of a linear single degree of freedom system.
T_B	Lower limit of the period of the constant spectral acceleration branch.
T_C	Upper limit of the period of the constant spectral acceleration branch.
T_D	Value defining the beginning of the constant displacement response range of the spectrum.
S	Soil factor.
a_g	Design ground acceleration on type A ground.
a_{gR}	Reference peak ground acceleration on type A ground.
Se(T)	Elastic response spectrum.
h	Damping correction factor with a reference value of η=1 for 5% viscous damping.
ξ	Viscous damping ratio of the structure, expressed as a percentage.

$S_{De}(T)$ Elastic displacement response spectrum.

$Sve(T)$ Elastic vertical ground acceleration response spectrum.

$Sd(T)$ Design spectrum (for elastic analysis).

a_{vg} Design ground acceleration in the vertical direction.

γ_I Importance factor.

Ms Magnitude.

q Behavior factor.

β Lower bound factor for the horizontal design spectrum.

G_{kj} Characteristic value of dead loads.

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

ψ_{2i} Characteristic value of live load.

Q_{ki} Combination coefficient for variable action I.

λ Slenderness.

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

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

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

r_x Square root of the ratio of the Torsional stiffness to the lateral stiffness in the y direction (Torsional radius).

l_s Radius of gyration of the floor mass in plan (square root of the ratio of

(a) the polar moment of inertia of the floor mass in plan with respect

(b) to the center of mass of the floor to (b) the floor mass).

I Building importance factor.

$A(T)$ Spectral acceleration coefficient.

A_0 Effective ground acceleration coefficient.

$S(T)$ Spectrum coefficient.

$S_{ae}(T)$ Elastic spectral acceleration.

g Gravity coefficient.

T_A, T_B Spectrum characteristic periods

E_d Load Combinations.

G Dead load.

Q Live load.

g_i Total live load at ith floor of the building.

q_i Total dead load at ith floor of the building.

n Live load participation factor.

N Number of floor in the structure.

Δ_{bi} Torsional irregularity factor determmined at ith floor of the structure.

$\Delta(i)_{ort}$ Average storey drift of i,th floor of the structure.

$\Delta(i)_{max}$	Maximum storey drift of i,th floor of the structure.
$\Delta(i)_{min}$	Minimum storey drift of i,th floor of the structure.
A_b	Total area of openings.
A	Gross floor area.
L_x, L_y	Length of the building at x, y direction.
a_y, a_x	Length of re-enter corners in x, y direction.
A_e	Effective shear area.
A_w	Effective of web area of column cross sections.
A_g	Section areas of structural elements at any storey.
A_k	Infill wall areas.
η_{ki}	Stiffness irregularity factor defined at i,th floor of the structure.
Δ_i	Storey drift of ith floor of the structure.
h_i	Height of ith floor of the structure [m].
h_w	Height of wall or cross-sectional depth of beam.
f_{ctm}	Mean value of tensile strength of concrete.
fyk	Characteristic yield strength.
ρ	Tension reinforcement ratio.
ρ_{min}	Minimum tension reinforcement ratio.
ρ_{max}	Maximum tension reinforcement ratio.
ρ'	Compression steel ratio in beams.

ε_{sy} Design value of steel strain at yield.

h_c Cross-sectional dimension of column.

d_{bw} Diameter of hoops.

d_{bL} Longitudinal bar diameter.

b_{wo} Thickness of web.

l_c Clear Column Length.

s Spacing.

ω_{wd} Ratio of the volume of confining hoop to that of confined core to the centerline of the parameter hoop, times f_{yd}/f_{cd} .

b_c Cross sectional-dimension of column.

b_o Width of confined core in a column or in the boundary element of a wall (to centerline of hoops).

ρ_w Shear reinforcement ratio.

ρ_v Reinforcement ratio of vertical web bars in a wall.

N_{Ed} Axial force from the analysis for the seismic design situation.

A_c Gross section area of column.

f_{cd} Design value of concrete compressive strength.

l_{ot} the length between Torsional restraints.

b Total depth of beam in central part of lot..

h	Width of compression flange.
b_w	Width of primary seismic beam.
h_w	Depth of beam.
h_s	Clear storey height in meter.
N_d	Axial force determined under the mixed effect of earthquakes and vertical forces. Multiplied with load coefficients.
f_{ck}	Typical compressive cylinder strength of concrete.
A_g	Gross sections area of or wall end zone.
A_p	Plane area of Storey building.
V_t	Total seismic load acting on a building.
f_{ctd}	Design tensile strength of concrete.
N_{dmax}	Greater of the axial pressure forces calculated under the mixed effect of earthquakes and vertical forces.
f_{ctm}	Main value tensile strength of concrete.
f_{yk}	Characteristic yield strength.
μ_φ	Value of the curvature ductility factor.
μ_ϖ	Design value of steel at yield.
f_{yd}	Design value of yield strength of steel.
v_d	Column axial load ratio.

A_c — Column cross-section area.

l_c — Length of the column.

h_c — Biggest cross-sectional dimension of the columns (in meters).

h_o — Depth of confined core in a column (to centerline of hoops).

b_o — Core with length

l_w — Long side of the rectangular wall section.

H_w — Total height of the wall.

h_s — Storey height

v_d — Wall axial load ratio

D_{bar} — Diameter of longitudinal rebars.

D_{min} — Smallest dimension of beam cross-section

N_d — Axial load determined under the mixed effect of earthquakes and vertical forces multiplied with loads coefficients

A_{ck} — Concrete core area within outer edges of confinement reinforcement.

f_{ywk} — Characteristic yield strength of transverse reinforcement.

A_{sh}	Total area steel of hoop

Chapter 1

INTRODUCTION

1.1 General

Earthquake is a natural phenomena generated by the discharge of the elastic strain power which is located within the tectonic plate, when the rocky material in the crust layer of the earth reaching its strength, thus causing sudden movement. The area in which the movement takes place is called the fault and these sudden movements (slip) are the causes of the earthquake [1].

An earthquake can cause violent vibration depending on its magnitude. Thus, a structure found within this area will encounter vibration at its foundation. The first rule concerning movement developed by Newton, also known as motion's law, declare that when the bottom of any infrastructure moves, the top part tends to move along with it. In the case of a building, the bottom parts and the top parts tend to move with it because the bottom and top parts are connected with columns [2].

Disastrous earthquakes hits all over the globe leading to many causalities and collateral damages. Until the end of the 20th century, earthquakes were regarded as natural disasters that cannot be avoided or contained. At the beginning of the 21st century, devastating earthquakes claimed the lives of over 500,000 people. This large number shows that earthquakes alone can no longer be held responsible for these causalities, since many research and interpretations found that the cause of all the lethal casualties and collateral damages are due to the inadequate seismic resistance of the buildings stock, lifelines and industry, which built according to incompetent design, codes [3]. This led to several seismic codes being published to hinder the effects of earthquakes and make them less threatening to lives and properties [4].

In general the codes for structural design are legal documents representing the minimum requirement for building a safe structure. These codes were put by knowledgeable people with a high sense of responsibility and with a lot of experience in engineering. While it has not necessarily depicted the best practice, but it generally gives structural engineers a way to design and build a safe structure while avoiding costly and grave mistakes. Safety and economy in general cannot be characterized without one another. Thus, for a structure to be considered successful in the engineering field it should be safe and economical [4].

The European Directive Construction Products issued a study in 1989, which consists of preconditions that should be met regarding the strength, stability and fire resistance of structure's construction [12]. The reason behind the publication of Eurocodes, was in order to opening the boundaries in between and to create a harmonious technical requirements between the European countries. Eurocodes are specialized regulations, agreed upon by the European nations, which has one objective: assure the realization of these preconditions. They are comprised of many standards assembled into ten codes. To validate the reliability of structures, Eurocode take a semi-probabilistic advance depending on partial coefficient applied to actions, covering the flaws in the analysis models, the properties of the material used and constructed [6]. As mentioned before, the structural Eurocode consist of 10 standards, which gives design rules and requirements, for every situation possible to help in the design of whole structures (concrete, steel) and (or) any element found in nature. Eurocode, after setting up a set of main rules and preconditions, defines the fundamentals of structural design. Eurocode 1 provides a guide for structural design of buildings, thus, provide the basics references for the structural Eurocodes [5].

Eurocode 8 responsible for the design of infrastructures in seismic areas. It does not set new rules, instead it implements the other Eurocodes and in addition to them, it adds more rules into those rules. It is essential to obtain a seismic zone map, its related info refines peak ground accelerations and spectral form of the region that want to use the Eurocode 8 in it. This set of information are received in the National Annex of each European Countries [5].

Turkey has always been in constant threat from different types of catastrophes, and earthquakes are the most prominent of these types. Turkey's geographical location is one of the most critical seismic action zone in the globe. This is the reason that raised the awareness of engineers to study and improve the designing code to counter such seismic activities. The great Erzincan Earthquake, which was the most destructive earthquake to hit Turkey in the 20th century (1939), led to the publication of the first Turkish seismic design code. After each critical catastrophe, new laws and regulations are added and the old designing codes are adjusted to implement these new laws and regulations. The last adjustments were made after the earthquake that hit Marmara in 1999, which lead to the revision of the 1998 code in 2007 and the new regulations were introduced under the title: Specification for Buildings to be constructed in Earthquake Regions [6].

1.2 Previous Work Done

To achieve a more accurate study, a brief review of many papers, journal articles and conference papers on Earthquake Engineering published in the past ten years concerning the comparison between EC8 and TEC-2007 was written as part of this study. A brief summary is given below:

- Dogangun and R. Livaoglu [7], in his study, the design methods advised by Turkish Earthquake Code, UBC, Eurocode 8, and IBC are compared. The main aim of the research was to compare the distinctions that could happen if distinct codes were to be used, the dynamic analysis was chosen to analyse various sample of structures based at code determined distinct locations.

- Dogangun and R. Livaoglu [8], tests the dissimilarities in outcomes acquired by the usage of the Equivalent Seismic Load Method, the Mode-Superposition Method and the Analysis Method in Time Domain. The outcome from these distinct methods for structures have been compared.

- E. Toprak, F. Gülten Gülay and P. Ruge [9], used the linear static method to compare the performance level of a single existing building according to EC8 and TEC-2007, while applying on it the parameters of the earthquake that hit Adana Ceyhan in 1998. It has been found that both codes reach the same performance level of collapse.

- Bayhan and P. Gülkan [11], This study aims to investigate the correctness of existing assessment procedures using data collected from an actual structure tested in the laboratory. The procedures outlined in FEMA-356, EC8 and

TEC-2007 are applied to a full-size, three-story, non-symmetric reinforced concrete building analyzed at the ELSA lab at JRC/Ispra under the SPEAR project. Therefore in order to do that, a three dimensional model of the building is subjected to the records used in the experimental phase and deformation demands are computed according to the procedures described in the guidelines that are being assessed for their correctness. The performance of the structure is evaluated at member level and the accuracy of the considered procedures is rated through comparisons with measurements and observations made after the experiments. The study shows that the major distinction between the procedures stem from different performance-based limit values and the characterizing phrases that are used to qualify them. It appears necessary that a harmonization should be agreed upon before universal application of these procedures. Otherwise, the conflicting acceptability criteria among different procedures are likely to create confusion among engineers.

- Rami Subhi Atiyah [12], the TEC-2007 and EC8 design principles are studied and compared. One case study has been chosen and designed with two different height of five and seven floor using STA4-CAD V12.1 computer software. He concluded that Eurocode 8 and the 2007 Turkish Earthquake Code deliver similar result for the cost of the building.

1.3 Aim and Scope

In this thesis, the TEC-2007 and EC8 are compared. The non-linear static pushover analysis method was chosen to evaluate the designed structure, that each one contain a different type of irregularities, with regards to earthquake, thus, allowing the performance check of the building. Finally, by comparing the performance, cost and damage percentage of each Eurocode 8 case with its 2007 Turkish Earthquake Code counterpart, it can determined which design code is more efficient in the given cases.

1.4 Thesis Outline

Chapter one: presents a brief explanation of earthquake and its effect on human lives and property, while providing a brief history of both the 2007 Turkish Earthquake Code and Eurocode 8, in addition to stating the aim and objectives of this study while giving a summary on the previous studies concerning the topic of this thesis.

Chapter two: present a brief summary of the 2007 Turkish Earthquake Code and Eurocode 8 for concrete designing of buildings, while giving a detailed comparison at the end.

Chapter three: present the methodology used in this study to develop the structural models for analysis, while citing the software and design parameters.

Chapter four: present the results of the analysis.

Chapter five: present the conclusion and recommendation for future studies.

Chapter 2

SUMMARY AND COMPARISON OF EC8 & TEC-2007

2.1 Introduction

Major verdict chosen at the primary steps of designing a structure plays an important part in deciding how the structure reaches its performance goals during a seismic action. This section present how Eurocode 8 and the 2007 Turkish Earthquake Code arrange these verdicts, regarding foundation design, the site of the structure and preference for superstructure. It is notable to mention that for the circumstances of this study, the rules and restrictions defined by the two codes are taken directly from its corresponding context. Not only the analytical value but also the basic rules and principles are taken into consideration when comparing the 2007 Turkish Earthquake Code and EC8. Hence, all distinct sub-clauses from Eurocode 8 and 2007 Turkish Earthquake Code are attributed with it corresponding codes.

2.2 Basic Requirements and Principles

2.2.1 Eurocode 8

Eurocode 8 define basic rules and requirements that all structures built or to be built in seismic regions should be met, each with a competent level of accuracy:

- No collapse specification: the structure should hold its entire vertical bearing capacity, residual lateral tenacity and rigidity to preserve lives during and after the seismic events. Although, the structure could be considerably damaged, also may have mild everlasting drifts [8].

- Damage limitation specification: a building must be modeled and built in a way that minimize the collateral damage, and to reduce the absolute constraint of structural in addition to the non-structural damage in an earthquake that has a bigger chance to occur. Its component should not show perpetual deformations, it should maintain its full strength and rigidity without the need of a repair. Although, non-structural components may undergo some minor damages that can be fixed effortlessly and economically [8].

To be able to meet the basic requirements of seismic design mentioned in EC8, it is necessary to check the following limit states:

- Ultimate limit state: the limits related to collapse and other patterns of structural malfunction, which might menace people's well being.

The structure should be designed to grant a competent protection and energy dissipation volume stated in the appropriate sections of EC8. The merit of behavior factor *q* along with the corresponding rigidity, both presented in Eurocode 8 are what define the parity between energy dissipation and protection. Moreover, the stability of the entire building under the design seismic reaction should be checked in both the overturning and the sliding stability. Also, the probable impact of second order effects on the earthquake's results should also be taken into consideration while performing the study.

- Damage limitation state: the limits related to the deterioration after which the detailed service conditions are not allowed.

An appropriate amount of authenticity opposed to undesirable deterioration have to be assured by meeting the deformation limits or different appropriate limits presented in Eurocode 8. The structural system has to be assured in structures essential for civil preservation (power plants, hospital, prisons), by meeting the deformation limit or different appropriate limits presented in Eurocode 8.

2.2.2 2007 Turkish Earthquake Code

The basic rule for designing against earthquake according to TEC-2007 is to protect structural in addition to the non-structural components of a structure against any kind of deterioration in low level of seismic action, to alter and narrow the deterioration in both structural and non- structural component to a fixable margin in mild level of seismic action, to avoid the total or limited breakdown of structures in high level of seismic action, and to prevent casualties.

Table 2.1: Importance Factor of Structures [13]

Purpose of occupancy	Importance factor (I)
1. Structures that should be used immediately following the seismic action and structures having dangerous substances inside them (a) Structures needed to be used directly following the seismic actions such as: hospitals, firefighting buildings, energy-producing stations and power allocation stations, governmental sector structures, etc.) (b) Structures having or keeping poisenous, munation, bombs and burnable substances, etc.	1.5
2. Extensively and constantly inhabited	

structures and structures maintaining precious equipements (a) Univerities, dorms, army's garissons, jails, etc. (b) Exhibitions	1.4
3. Extensively however temporarly inhabited structures Arenas, movie houses, play houses and sports field, etc.	1.2
4. Different structures Additional structures that were not mentioned previously (houses, appartement buildings, offices, inns, industrial buildings, etc.)	1.0

A building ought to be designed to resist the earthquake load as one body; in addition, each structural component of it must have enough rigidity, balance and strength to assure no interruption to occur while securing the transmission of seismic loads to the soil foundation.

2.3 Specific Measure in Design

2.3.1 Eurocode 8

Eurocode 8 states that a structure should have a normal and simple form in both plan and elevation.

Premature development of shaky structural mechanism should be prevented, to prevent the occurrence of total dissipative and ductile behavior. For this reason, the capacity steps that are utilized to achieve the order of resistance of the distinct architectural elements and failing class essential for assuring suitable plastic mechanism and for preventing brittle type of failure, have to be attributed where needed.

The depicting of links between architectural component and dangerous zones, where non- linear actions are predictable, have to obtain special attention while designing. In addition, non-structural component prospect along with soil deformity must be taken into account while performing the test, like the existence of a neighboring building. The rigidity of the basis has to be suitable for transferring the forces received from the structure to the soil as constant as feasible.

2.3.2 2007 Turkish Earthquake Code

Designing and building of irregular structures, which are explaned in appropriate sections of TEC-2007, should not be allowed. The different types of irregularities will be cited later in this thesis.

The rules of the ductile design presented in appropriate sections of the 2007 Turkish Earthquake Code have to be conducted, in order to deplete a major chunk of the earthquake load sustained by the architectural system.

To guarantee the transmission of the seismic forces to the foundation safely and without interruption, each structural components of the structure along with the structure as a whole system should be afforded with appropriate rigidity, strength and balance. Considering these aspects, it is important that the storey plan hold appropriate rigidity and durability to assure the transmission of the x-direction earthquake force amidst the components of the whole structure safely.

2.4 Soil Conditions

2.4.1 Eurocode 8

The identification of the local ground conditions have to be take into consideration over competent geotechnical inspections, on site and/or at the laboratory. Additional supervision relating to ground inspection and categorization is presented in Eurocode 8 section-5.

The nature of the upholding foundation along with the place of the construction should be relieved from hazards relating to slope imbalance, ground split and permanent settlements induced by the increase in density and liquefaction in the event of a seismic action.

Moreover, EC8 assert that, relying on the importance level of the building along with the specific conditions of the design; ground investigation and/or geological analysis have to be implemented to determine the seismic action.

By determining the types of the ground with distinctive mechanical characteristic, the impact of the local soil conditions on the seismic response of the structure may be defined. Five type has been picked to classify the profiles of the soil as seen below. Although, it is important to mention that the ground classification plan might vary depending on the country and these information can be found in its National Annex.

Table 2.2: Ground Types Defined By Eurocode 8.

Ground type	**Depiction of stratigraphic profile**	**Parameters**		
		$V_{s,30}$ (m/s)	**N_{SPT} (blows/30 m)**	**C_u (kPa)**
A	Stone or other stone-like geological composition, containing at most 5m of fragil substance at the top.	>800	-	-
B	Deposit of highly compressed sand, pebbles, or highly rigid clay, no lower than few tens of meters in width, identified by a continious raise of mechanical characterestic with the increase in depth.	360-800	>50	>250

C	Rooted deposits of compressed or mildly compressed sand pebbles or rigid clay with width starting with few tens to numerous hundreds of meters.	180-360	15-50	70-250
D	Deposit of loose-to-medium frictional soil (with or without a few soft cohesive layers), or of mostly soft-to-hard cohesive soil.	<180	<15	<70
E	A soil profile made up of a top alluvium layer with v_s merit of type C or D and a changing width in the interval of around 5 to 20m, dominated by rigider substance with v_s merit higher than 800 m/s.	-	-	-
S_1	Deposits made up of, or having a layer that at least 10m in width, of soft clay/silts with a high Atterberg limit (PI>40) and larger water capacity	<100 (indicative)	-	10-20
S_2	Deposits of soluble soils, of fragile clays, or any other soil type excluded from types A-E or S_1	-	-	-

Where;
$V_{s,\,30}$ *(m/s)* : average shear wave velocity.
N_{SPT} : number of blows evaluated with the standard penetration test.
C_U : undrained cohesive resistance.

The average shear velocity could be calculated using the following equation:

$$V_{s,\,30} = \frac{30}{\sum_{i-1,N} \frac{hi}{v_i}} \qquad (2.1)$$

Where;
h_i : the thickness (m)
v_i : shear-wave velocity (at a shear strain level of 10^{-5} or less) of the *i*- th layer in a total of N, located in the top 30 m

It is important to mention that, the site category as stated by EC8 should be subjected on the merit of average shear wave velocity, $\nu_{s,30}$, if they were obtainable. If not, the values of N_{SPT}, must be utilized.

Specific research for the definition of the earthquake are necessary, for sites having the special ground types of S1 or S2. For these types, and especially in the case of *S2*, so that to attain a better outcome, studying the probability of soil breakdown affected by an earthquake is crucial.

2.4.2 2007 Turkish Earthquake Code

According to TEC-2007 soils, types can be classified depending on two factors: the types of soil and the local site classes, which are presented in two table in the 2007 Turkish Earthquake Code. However it is important to note that the merit of soil specifications presented in the formentioned table are to be taken as initial values solely for the purpose of guidance while defining the soil types.

In the primary and secondary seismic zones, disregarding the structure elevation, soil inspections related on appropriate site and laboratory test are mandatory along with relevant report and attached design documents, for buildings with factor of importance of $I = 1.5$ and $I = 1.4$ and with a height of 60 m or beyond.

In buildings that are not falling in the mentioned criteria, within the primary and secondary seismic zones, to classify the soil groups and soil classes, accessible local reports or observations results and/or issued sources must be incorporated and cited in the earthquake report.

Moreover, a group (D) soil having a water table 10m below the soil surface have to be inspected and the result of the inspection shall be included in a documented report to establish the probable existence of liquefaction, by utilizing adequate analytical methods build upon on site and laboratory tests.

Table 2.3: Soil Groups According to TEC-2007

Soil group	Description of soil group	Standard penetration (N/30)	Relative density (%)	Unconfined compressive strength (KPa)	Drift wave velocity (m/s)
(A)	**1.** Huge volcanic stones, unweathered sound metamorphic stones, rigid cemented sedimentary stones	-	-	>1000	>1000
	2. Hihjly compressed sand, pebbles	>50	85-100	-	>700
	3. Hard clay and silty clay				
(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 2.4: Local Site Classes

Local site class	**Soil group according to soil groups and top most layer width (h_1)**
Z_1	Group **(A)** soils Group **(B)** soils with $h_1 \leq 15$ m
Z_2	Group **(B)** soils with $h_1 > 15$ m Group **(C)** soils with $h_1 \leq 15$ m
Z_3	Group **(C)** soils with 15 m $< h_1 \leq 50$ m Group **(D)** soils with $h_1 \leq 10$ m
Z_4	Group **(C)** soils with $h_1 > 50$ m Group **(D)** soils with $h_1 > 10$ m

**Note:* in the circumstances where the width of the topmast soil layer under the foundation is less than 3 m, the layer below may be considered as the topmost soil layer indicated in the table above.

Unlike Eurocode 8, TEC-2007 represents a Local site class that is associated with soil groups and uppermost layer breadth, which allow the determination of the earthquake with respect to the ground conditions. The formula (2.1) shows that Eurocode 8 take the soil thickness into consideration.

Moreover, the 2007 Turkish Earthquake Code present some statements to monitor and increase the quality of the construction for the worry relating the poor supervision at the construction site and the design documents.

2.5 Seismic Design

2.5.1 Seismic Action as Stated by Eurocode 8

National Authorities should split National regions into seismic areas, relating on the local hazard. The danger is given as a sole parameter, the merit of the reference peak ground acceleration on type A ground, $a_{gR.}$ Also, the danger within each area is considered constant.

In EC8, the seismic action at a specified point on the exterior is defined as elastic response spectrum, $S_e(T)$.The form of the elastic response spectrum is identical for both category of seismic actions the no-collapse conditions and the damage limitation conditions.

$S_e(T)$ can be obtained from the formula below, for the horizontal elements of earthquake:

$$S_e(T)= a_g \,.\, S \,.[1 +(T/T_B).(\eta.\ 2.5 – 1)] \qquad (0 \leq T \leq T_B) \qquad (2.2)$$

$$S_e(T)= a_g.\ S.\ \eta.\ 2.5 \qquad (T_B \leq T \leq T_C) \qquad (2.3)$$

$$S_e(T)= a_g.\ S.\ \eta.\ 2.5\ (T/T_C) \qquad (T_C \leq T \leq T_D) \qquad (2.4)$$

$$S_e(T)= a_g.\ S.\ \eta.\ 2.5\ (T_C \,.\, T_D / T^2) \qquad (T_D \leq T \leq 4s) \qquad (2.5)$$

Where;
$S_e(T)$: Elastic response spectrum.
T : Vibration period of a linear single-degree-of-freedom system.
a_g : Design ground acceleration on type A ground.
T_B : Lower limit of the period of the constant spectral acceleration branch.
T_C : Upper limit of the period of the constant spectral acceleration branch.
T_D : Value defining the beginning of the constant displacement response range of The spectrum.
S : Soil factor.
η : Damping correction factor with a reference value of η=1 for 5% viscous damping.

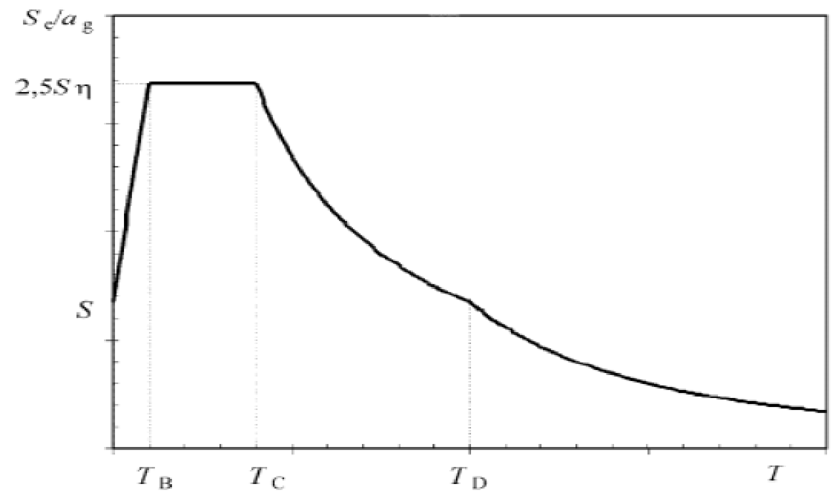


Figure 2.1: Form of $S_e(T)$

The values of the periods T_B, T_C and T_D and the soil factor S defining the form of $S_e(T)$ are built upon the ground types.

The values for each of these periods must be utilized by a nation that defines them in its National Appendix. Eurocode 8 ignores the deep geology and suggest two categories of spectra: Type 1 and Type 2.

In the cases where the earthquake that is majorly responsible for the determined seismic danger on the area, for the aim of probabilistic danger estimation has an adequate-wave magnitude, Ms, no more than 5.5, it is advised that Type 2 spectrum is used. The advised values of the periods S, T_B, T_C and T_D for the spectra's categories, Type 1 and Type 2, are stated in the tables below. Different spectra can be located in the National Annex.

Table 2.5: The Merit Of The Periods Advised By The Type 1 $S_e(T)$ [12]

Soil type	***S***	**T_B (s)**	**T_C (s)**	**T_D (s)**
A	1	0.15	0.4	2.0
B	1.2	0.15	0.5	2.0
C	1.15	0.2	0.6	2.0
D	1.35	0.2	0.8	2.0
E	1.4	0.15	0.5	2.0

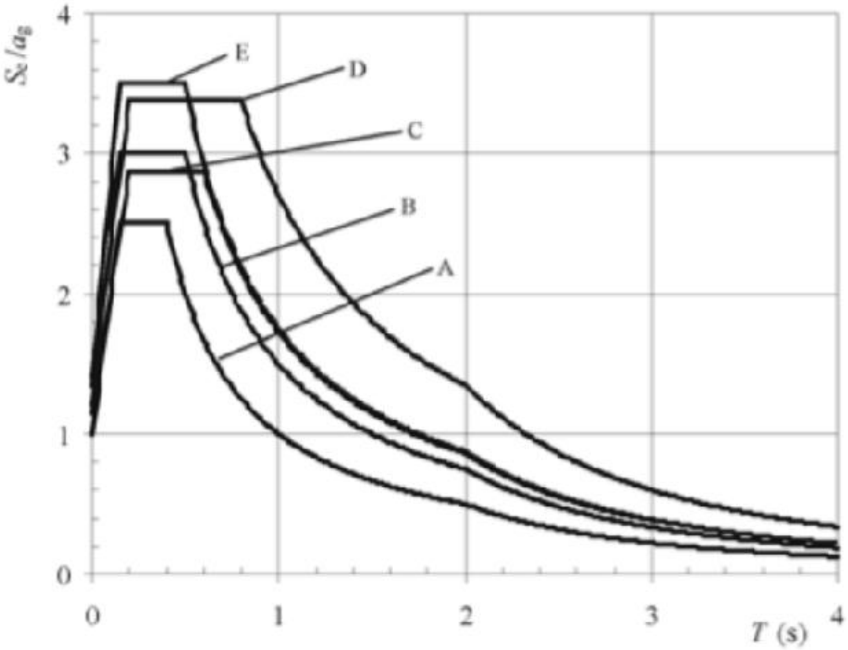


Figure 2.2: Recommended Type 1 $S_e(T)$ For Ground Types A To E (5% Damping)

Table 2.6: The Values Of The Periods Advised By The Type 2 $S_e(T)$ [12].

Soil Type	***S***	***T_B (s)***	***T_C (s)***	***T_D (s)***
A	1	0.05	0.4	1.2
B	1.35	0.05	0.5	1.2
C	1.5	0.10	0.25	1.2
D	1.8	0.10	0.30	1.2
E	1.6	0.05	0.25	1.2

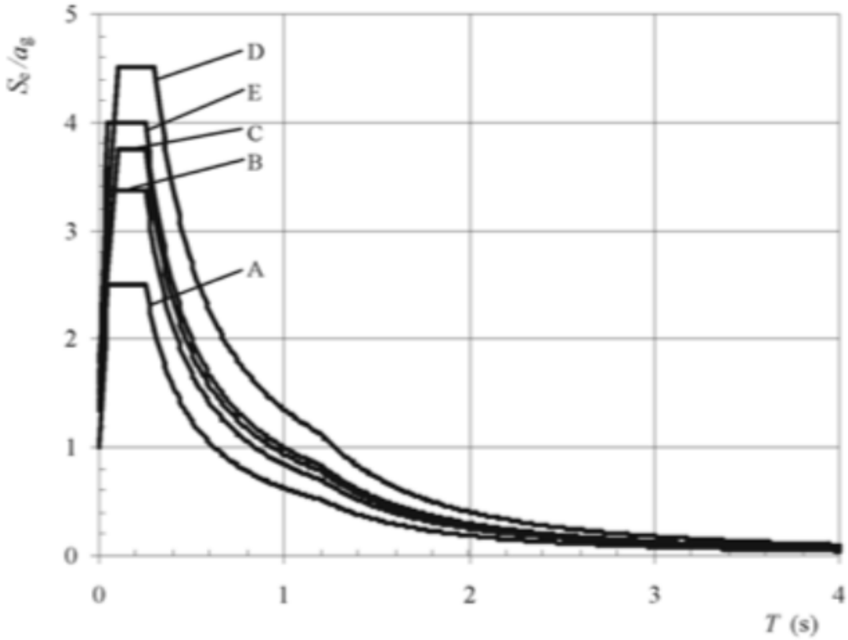


Figure 2.3: Type 2 Recommended $S_e(T)$ For Ground Types A To E (5% Damping).

The merit of the damping correction factor (η) can be obtained by using the following equation:

$$\eta = \sqrt{10}/(5+\xi) \geq 0.55 \qquad (2.6)$$

Where;
ξ : Viscous damping ratio of the building, given as a percentage.

The elastic displacement response spectrum, $S_{De}(T)$, can be determined by direct alteration of $S_e(T)$, by utilizing the equation below:

$$S_{De}(T)= S_e(T) \left[\frac{T}{2\pi}\right]^2 \qquad (2.7)$$

It is noted to mention that this equation should be used in vibration periods below 4.0 second. As for buldings with vibration periods higher than that, a more acuurate solution for $S_{De}(T)$ can be obtained.

The vertical element of the earthquake also known as the vertical elastic response spectrum, S_{ve}, may be acquired through the derivative of the formulas below:

$$S_{ve}(T)= a_{vg} \,.\, [1 +(T/T_B).(\eta.\ 3\ - 1)] \qquad (0 \leq T \leq T_B) \qquad (2.8)$$

$$S_{ve}(T)= a_{vg} \,.\eta \,.\, 3 \qquad (T_B \leq T \leq T_C) \qquad (2.9)$$

$$S_{ve}(T)= a_{vg} \,.\, \eta \,.\, 3.0\ (T_C/T) \qquad (T_C \leq T \leq T_D) \qquad (2.10)$$

$$S_{ve}(T)= a_{vg} \,.\, \eta \,.\, 3.0\ [T_C \,.\, T_D / T^2]^2 \qquad (T_D \leq T \leq 4s\ ) \qquad (2.11)$$

The values that are to be assigned to T_B, T_C, T_D and a_{vg} for every form of $S_{ve}(T)$ that must be utilized in a country it has been specified in its National Annex. However, these advised values cannot be applied for the other ground class S_1 and S_2.

Table 2.7: Advised Values of Periods Expressing The $S_{ve}(T)$ [13]

Spectrum	a_{vg}/a_g	T_B **(s)**	T_C **(s)**	T_D **(s)**
Type A	0,9	0,05	0,15	1,0
Type B	0,45	0,05	0,15	1,0

It can clearly be noticed that, in Eurocode 8 the vertical elastic response does not altered with the soil conditions directly.

The design ground displacement, d_g, relating to the design ground acceleration, a_g, can be obtained by utilizing the equation below:

$$d_g = 0{,}025 \; . \; a_g \, . \, S. \; T_C \, . \; T_D \tag{2.12}$$

Moreover, special investigations should be concluded to calculate the design ground displacement for a given construction site.

Design spectrum for elastic analysis: The ability of the building to distribute energy, primarily throughout ductile behavior or throughout its components and/or through different way, should be considered by carrying out an elastic test established on a minimized response spectrum (obtained from the behavior factor *q*) regarding the elastic one.

EC8 defined *q* as an assumption ratio of the earthquake loads that can be experienced by the structure in the case where the response was entirely elastic with 5% viscous damping, to the earthquake loads that may be utilized within the building framework, with a typical elastic model, also assuring a sufficient response from the building.

The merits of *q*, are defined for several elements and architectural plans with relation to appropriate ductility types in several sections of Eurocode 8.

Table 2.8: The Merits of the Behavior Factor q.

Type of Building	**DCM**	**DCH**
Frame system, dual system, coupled wall system	3,0 α_U/α_1	4,5 α_U/α_1
Uncoupled wall system	3,0	4,0 α_U/α_1

Torsionally flexible system	2,0	3,0
Inverted pendulum	1,5	2,0

Table 2.9: Approximate Values of Multiplication Factor αU/α1

Structural system	**α_U/α_1**
1. Frames or frame- equivalent dual systems	
a. one-floor structures	1,1
b. multistorey, one-bay frames	1,2
c. multistorey, multi-bay frames	1,3
2. Wall or wall-equivalent dual systems	
a. wall systems with only uncoupled walls per horizontal direction	1,0
b. other uncoupled wall systems	1,1
c. wall-equivalent dual, or coupled wall systems	1,2

For the horizontal elements of the earthquake the design spectrum, $S_d(T)$, can be determined by using the equations below:

$$S_d(T) = a_g\ S\ \left[\frac{2}{3} + (T/T_B)\left(\frac{2.5}{9} + \frac{2}{3}\right)\right] \qquad (0 < T < T_B) \qquad (2.13)$$

$$S_d(T) = a_g\ S\ \frac{\mathbf{2.5}}{\boldsymbol{q}} \qquad (T_B \leq T \leq T_C) \qquad (2.14)$$

$$Sd(T) = \begin{cases} a_g\ S\ \frac{2.5}{q}\ [\frac{Tc}{T^2}] \\ \geq\ \beta\ a_g \end{cases} \qquad (\ T_B \leq T \leq T_C\ ) \qquad (2.15)$$

$$Sd(T) = \begin{cases} a_g\ S\ \frac{2.5}{q}\ [\frac{Tc\ T_D}{T^2}] \\ \geq\ \beta\ a_g \end{cases} \qquad (\ T_D \leq T\ ) \qquad (2.16)$$

Where;
a_g, S, T_C and T_D are as previously explained in $S_e(T)$ equations: equations 2.2 to 2.5.
$S_d\ (T)$: Design spectrum.
q : Behavior factor.
β : Lower bound factor for the horizontal design spectrum.

The value of the lower bound factor β to be used in a certain country can obtained from its National Appendex. Generally, the advised value for β is 0.2.

For the vertical components of the earthquake, $S_d\ (T)$ can be obtained by using the same equations mentioned above, also with a little alteration to the equation with the design ground acceleration in the vertical direction, a_{vg} replacing a_g, and the value for S must be 1 and the recommended highest value for q must be 1.5 for all materials and architectural plans.

In the cases where a value for q is taken higher than 1.5 in the vertical direction an adequate analysis must be conducted.

Eurocode 8 classifies buildings into four categories according to their signinficance on the public well being, civil security and on the consequences on human life, if a failure in the whole structural to happen during or after the earthquake. The advised merit of the importance factor (Υ_I) for the various importance classes are mentioned in the table below:

Table 2.10: Merits of YI for Important Classes [12].

Importance classes	**Type of Structure**	**The recommended value of Y**
I	structure of smaller priority for public well being. e.g. agricultural structures	0.8
II	Typical structures, that aren't included in the other types	1.0
III	Structures in which their resistance against earthquake is significant in the cases where a breackdown is to occur, e.g. universities, meeting halls, libraries etc.	1.2
IV	Structure in which their integrity throughout the earthquake present a crucial priority for civil security, e.g. emergency rooms, clinic, police stations, power stations, etc.	1.4

2.5.2 Seismic Action According to TEC-2007

Earthquake loads acting on buildings according to TEC-2007 are established on Spectral Acceleration Coefficient *A(T),* and Seismic Load Factor. Earthquake loads must be presumed to behave independently throughout the vertical axes of the structure in the horizontal plane. Although it is presumed that, the earthquake forces and the wind force do not act at the same time, the most unfavorable response value as a result of the wind should be accounted in the design of each structural component.

For evaluating earthquake loads subjected on a structure, *A(T),* should be calculated using the following equation:

$$A(T) = A_o, I \,.\, S(T) \tag{2.17}$$

Where;
A(T): the coefficient for spectral acceleration
A_o : the effective coefficient for ground acceleration
I : the structure importance factor
S(T): the spectrum coefficient.

The following equation could be utilized to calculate The elastic spectral acceleration, $S_{ac}(T)$, determined as the ordinate of elastic acceleration spectrum for 5% damping ratio:

$$S_{ac}(T) = A(T) \cdot g \tag{2.18}$$

Where;
$S_{ac}(T)$:the elastic spectral acceleration;
A(T) : spectrum coefficient;
g : gravity (9,81 m/s^2).

The values for A_0 (Effective coefficient Ground Acceleration), mentioned in the equation 2.17 is determined in the table below:

Table 2.11: Values for A_0 for the Different Seismic Zone [13].

Seismic Zone	A_0
1	0.4
2	0.3
3	0.2
4	0.1

The Spectrum Coefficient, *S(T),* mentioned in the equation 2.17 above should be calculated using the following equations:

$$S(T) = 1 + 1.5\frac{T}{T_A} \qquad (0 \le T \le T_A) \tag{2.19}$$

$$S(T) = 2.5 \qquad (T_A \leq T \leq T_B) \qquad (2.20)$$

$$S(T) = 2.5\left[\frac{T_B}{T}\right]^{0.8} \qquad (T_B < T) \qquad (2.21)$$

Where;
T: the structure natural period;
T_A,T_B: the spectrum characteristic periods.

T_A and T_B are given in the table below, depending on local site classes which were defined earlier:

Table 2.12: Spectrum Characteristic Periods [13].

Local site class	**T_A (s)**	**T_B (s)**
Z_1	0.10	0.3
Z_2	0.15	0.4
Z_3	0.15	0.6
Z_4	0.20	0.9

In certain circumstances, the elastic acceleration spectrum is allowed to be determined via special inspections by taking into consideration the site and local seismic conditions. Although, the obtained values of the spectral acceleration coefficients should not be in any case less than those defined by the formula (2.17) which is based upon the periods determined in table (2.12).

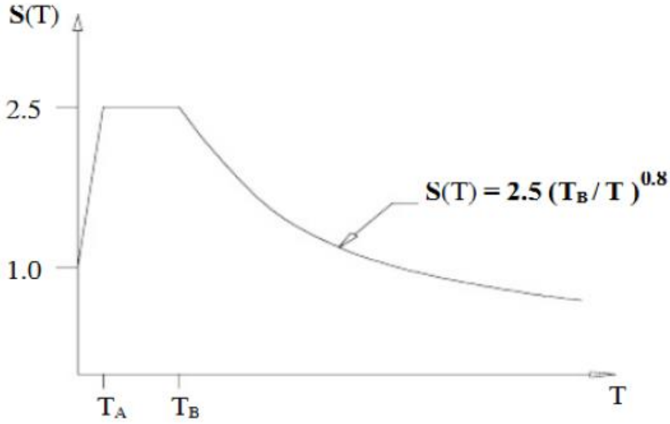


Figure 2.4: Design Acceleration Spectrums [13].

The Seismic Load Reduction Factor can be obtained by using either of the equations below.

$$Ra\ (T) = 1.5 + (R - 1.5)\frac{T}{T_A} \qquad (0 \le T \le T_A) \qquad (2.22)$$

$$Ra\ (T) = R \qquad (T_A < T) \qquad (2.23)$$

Where;
R: structural system behavior factor
T: natural vibration period

Table 2.13: Structural System Behavior Factor R [13].

BUILDING STRUCTURAL SYSTEM	***Systems of Nominal Ductility Level***	***Systems of High Ductility Level***
(1) CAST-IN-SITE REINFORCED CONCRETE BUILDINGS		
(1.1) Structure where the earthquake forces are completly resisted by frames…...	4	8
(1.2) Structures where the earthquake forces are completly resisted by coupled structural walls…………………………	4	7
(1.3) Structures where the earthquake forces are completly resisted by solid structural walls…………………………….	4	6
(1.4) Structures where the earthquake forces are at the same		

time resisted by frames and solid and/or coupled structural walls………………………………………………………	4	7
(2) PREFABRICATED REINFORCED CONCRETE BUILDINGS (2.1) Structures where the earthquake forcess are completly resisted by frames with joints able of cycling moment transmit……………………………………………………..	3	7
(2.2) Single-storey structures where the earthquake forces are completly resisted by columns with hinged upper joints.	-	3
(2.3) Prefabricated structures with hinged frame joints where the earthquake forces are completly resisted by prefabricated or cast – in – site solid structural walls and/or coupled structural walls……………………………………	-	5
(2.4) Structures where the earthquake force are resisted at the same time by frames with joints able of cycling moment transmit and cast – in – situ solid and/or coupled structural walls…………………………………………………….	3	6
(3) STRUCTURAL STEEL BUILDINGS (3.1) Structures where the earthquae forces are completly resisted by frames………………………………………	5	8
(3.2) Single - storey structures where the earthquake forces are completly resisted by columns with joints hinged at the top…………………………………………………………	-	4
(3.3) Structures where the earthquake forces are completly resisted by braced frames or cast -in-site reinforced concrete structural walls.		
(a) Frames that are braced at the center………..……….	4	5
(b) Eccentrically braced frames...…………………………………………………….	-	7
(C) Reinforced concrete structural walls……………………………………………………..	4	6
(3.4) Structures where the earthquake forces are resisted at the same time by structural steel braced frames or cast-in-site reinforced concrete structural walls		
(a) Frames that are braced at the center…………………….	5	6
(b) Eccentrically braced frames……………………………………………………….	-	8
(c) Reinforced concrete structural walls…………………………………………………………	4	7

2.6 Load Combination

2.6.1 Eurocode 8

Defined below is the design value *Ed* that has directly affects the behaviour of seismic design:

$$Ed = \Sigma\, G_{kj} + \gamma\, A_{Ed} + \Sigma\, \psi 2i\, Q_{ki} \qquad (2.24)$$

Where;
γI : Importance factor as seen in table 2.6.
G_{kj} : Characteristic value of dead loads.
A_{Ed} : Design value of return period of specific earthquake motion ;
$\psi_2 i$: Combination coefficient of live load.
Q_{ki} : Characteristic value of live load.

Eurocode 8 has reduced the value of variable actions that should be added to the design of the earthquake above the level of a single member. Although, below that level the value of variable actions can be used locally for the verification of members and sections [8].

$$\sum G_{k,j} + \sum \psi_{E,i}\ .\ Q_{k,i} \qquad (2.25)$$

Where;
$\phi_{E,i}$ *:* present the combination coefficient for variable actions.

As stated by the Eurocode 8, the imposed loads $Q_{k,i}$ are not following the whole structure when and earthquake strikes. Thus Eurocode 8 states that the use of the coefficient $\psi_{E,i}$ will help in making that happen. Also it is recommended that the coefficient $\psi_{E,i}$ to be used when there's a reduced participation of masses in the motion of the building because of the flexible links among them.The combination coefficients mentioned above may be formulated by utilizing the equation below:

$$\Psi_{Ei} = \phi \cdot \psi_{2i} \tag{2.26}$$

Where;
ϕ_{2i} : is the combination coefficient (for the quasi-permanent value of variable action *q(i)* for the design of structures);
ϕ : is a factor that can be found in the National Appendex.

Table 2.14: Values Of ϕ For Calculating Ψ_{ei}

Type of variable action	**Storey**	ϕ
Categories A-C	Roof…………………………… Floors with correlated inhabitancy …………………... Separetly inhabited floors..........	1,0 0,8 0,5
Categories D-F and Archives		1,0

The table below present the recommended values of the combination coefficient, ϕ_{2i} defined in corresponding parts of Eurocode:

Table 2.15: Advised Values of the Factor Ψ for Buildings

Actions	Ψ_0	Ψ_1	Ψ_2
Category A: private, inhabitual regions	0,7	0,5	0,3
Category B: work locations (offices)	0,7	0,5	0,3
Category C: public regions	0,7	0,7	0,6
Category D: market places	0,7	0,7	0,6
Category E: depository regions	1,0	0,9	0,8
Category F: traffic regions Vehicle weight ≤ 40 kN	0,7	0.7	0,6
Category G: traffic regions 30 kN < Vehicle weight < 160 kN	0,7	0,5	0,3
Category H: ceilings	0	0	0

2.6.1 TEC-2007

The total weight considered as the seismic weight, *W*, can be determined using the formula below:

$$W = \sum_{i-1}^{N} w_i \tag{2.27}$$

Where;
w_i : represent the weight of the storey at the ith floor.

The story weight mentioned above may be determined using the following formula:

$$wi = g_i + n\, q_i \tag{2.28}$$

Where;
g_i: represent the dead (gravity) loads;
q_i: represent the live (imposed) loads;
n: represent the live participation factor

Live participation factor, n, mentioned above are presented in the table below:

Table 2.16: Live Load Participation Factor (N)

Reason For Inhabitancy of the Structure	**n**
Depot, storehouse, etc.	0.8
Universities, dorms, sport facility, movie houses, play houses, auditorium, parking lot, cafeterias, markets, etc.	0,6
Homes, business places, Inns, hospital, appartements etc.	0.3

2.7 Irregularities

2.7.1 Irregularities According to Eurocode 8

EC8 defines two types of regularities for the design building that should be accomplished, which are in plan and in elevation.

2.7.1.1 Characteristic for Regularity in Plan

So that a structure is said to be characterized as being regular in plan, the following conditions must be fulfilled:

- The building must be nearly symmetrical in plan with reference to the two orthogonal axes also corresponding to the lateral rigidity and mass allocation.

- The arrangement of the plan have to be compact for example each storey must be delimited by a contour line.

- In order for the diformation of the floor to have minimal impact on the allocation of the loads between the perpendicular structural components, the in-plan rigidity of the stories should be adequately higher when compared to the lateral rigidity of the perpendicular structural components.

- The slenderness $\lambda = L_{max} / L_{min}$, in plan, of the designed building must not be bigger than 4, where L_{max} and L_{min} are the taller and shorter in plan measurements of the structure respectively, determined in the vertical line.

- For every direction and at every level of analysis *x* and *y*, the structural eccentricity (e_o) and the torsional radius *r* must be determined with respect to the requirements below for the direction of analysis *y*:

$$e_{ox} \leq 0.3 \ . \ r_x \qquad\qquad r_x \geq l_s \qquad (2.29)$$

Where;
e_{ox}: Distance between the center of rigidity and the center of mass, determined throughout the x axix, that is normal to the direction of the analysis taken into account.
r_x : Square root of the ratio of the torsional rigidity to the lateral rigidity in the y direction (torsional radius).
l_s : Radius of gyration of the floor mass in plan (square root of the ratio of (a) the polar moment of inertia of the floor mass in plan with respect to the center of mass of the floor to (b) the floor mass).

2.7.1.2 Characterestic for Regularity in Elevation

So that a structure is said to be characterized as being regular in elevation, the requirements below must be fulfilled:

- All lateral load-resisting systems: like walls and beams, must be able work without interference from the lowest point to the highest point of the structure or, if setbacks at distinct elevations are occuring, to the peak of the important area of the building.

- The individual masses of the floors and the lateral rigidity have to stay consistent or steadily minimized, without sudden shift, from the lowest point to the highest point of the structure.

- The resistance ratio to the resistance determined by the analysis, of the actual floors in framed buildings should not differ asymmetrically amid adjacent stories.

2.7.2 Irregularities According to TEC-2007

The 2007 Turkish Earthquake Code define two criteria in which should be avoided in the construction and designing of a structure, for their dangerous seismic behavior: the irregularities in plan stated as A-type irregularities and the irregularities in elevation stated as B-types irregularities. The relevant requirement of these two type are given in this section.

2.7.2.1 Characterestic for Regularity in Plan

A1-Torsional Irregularity

A1-type irregularity: the first form of structural irregularity defined by TEC-2007. It occur where the Torsional irregularity factor (η_{bi}) is higher than 1.2. This factor represent the ratio of the maximum relative floor drift to the mean relative floor drift at the same plane in a structure, for the two direction of the earthquake [13].

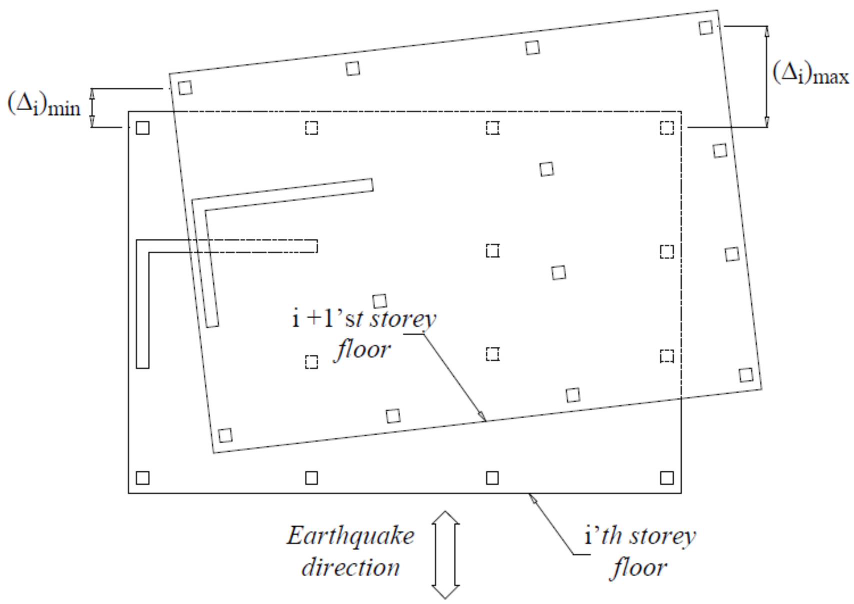


Figure 2.5: Type A1- Torsional Irregularity [13].

η_{bi}, can be calculated by using the following formulas:

$$\eta_{bi} = (\Delta_i)_{max} / (\Delta_i)_{ort} > 1.2 \tag{2.30}$$

$$(\Delta_i)_{avg} = ½ [(\Delta_i)_{max} - (\Delta_i)_{min}] \tag{2.31}$$

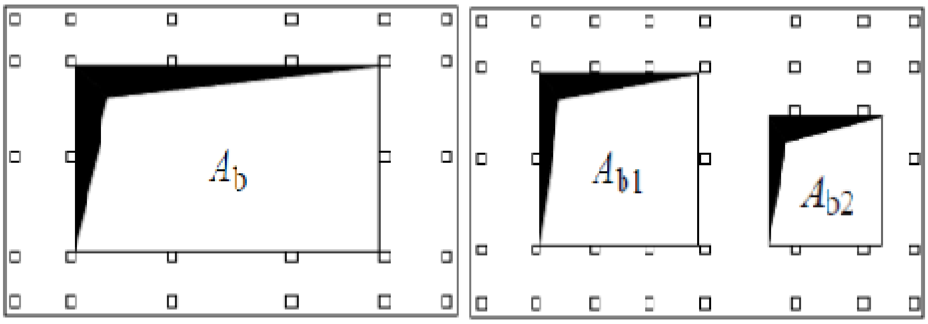
Where;
η_{bi} : Torsional irregularity factor determined at the i'th floor.
$(\Delta_i)_{avg}$: Average floor drift at the i'th floor.
$(\Delta_i)_{max}$: Maximum floor drift at the i'th floor.
$(\Delta_i)_{min}$: Minimum floor drift at the i'th floor.

It is important to mention that the storey drift of the structure must be obtained with regard to the ± 5% impact of additional eccentriceties.

A2-Floor Discontinuities

There exist three cases for the occurrence of such type:

- The case where the area of the slab hole for instance the elevator shafts, and stairs are higher than 1/3 of the total area of the floor, as shown below.



(a) A_2-Irregularity-I (b) A_2-Irregularity-I
Figure 2.6: Type A2- First Cases [13].

$$A_b = A_{b1} + A_{b2} \quad (2.32.a)$$

$$A_b / A > 1/3 \quad (2.32.b)$$

Where;
A_b : Total area of the hole.
A : Total floor area.

- The cases where the safe transfer of earthquake forces to vertical architectural components is being interrupted by the gaps in the local, as shown in the figure below:

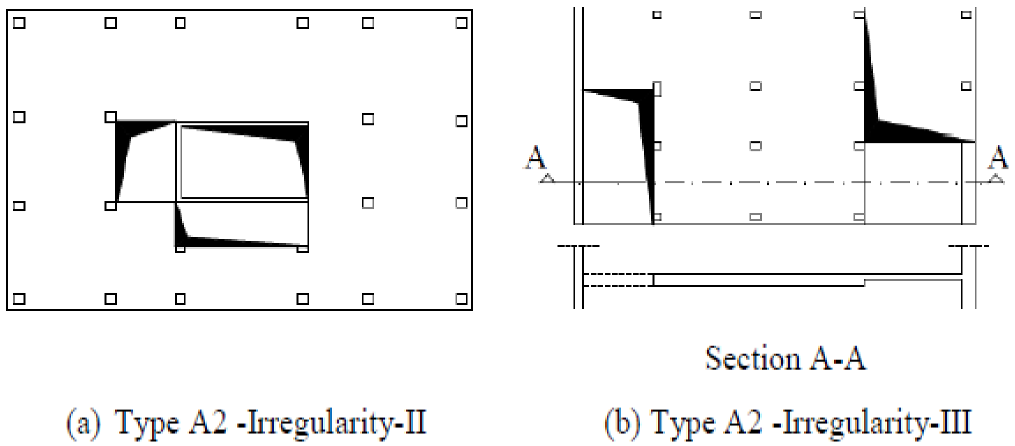


Figure 2.7: Type A2- Irregularity Second and Third Cases [13].

- The cases of sudden depletion in the in-plane stiffness and durability of the storeys. As shown in the figure above.

A3- Projections in Plan:

A3-type of irregularity: The cases in which 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% [13].

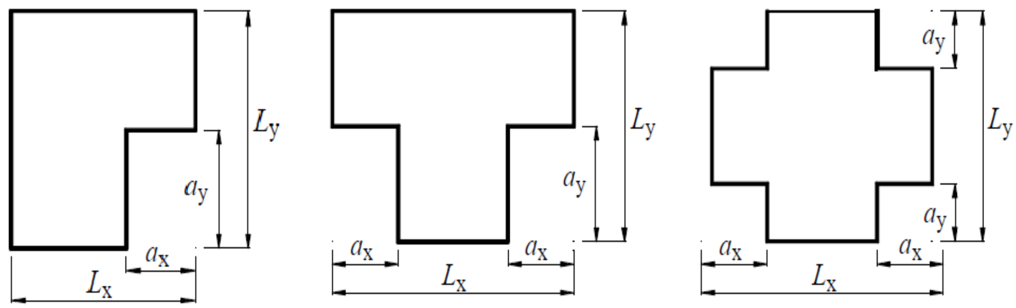


Figure 2.8: Type A3- Irregularity [13].

$$a_x > 0.2\ L_x \tag{2.33.a}$$

$$a_y > 0.2\ L_y \tag{2.33.b}$$

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

2.7.2.2 Characteristic for regularity in Elevations

B1- Interstorey Strength Irregularity (Weak Storey):

B1-type of irregularity: in the cases where the strength irregularity factor η_{ci} is below 0.8. This factor can be determined at any floor as the ratio of the effective shear area in it to the effective shear area of the floor located directly atop of it [13].

η_{ci} can be calculated using the following formula :

$$\eta_{ci} = (\sum A_e)_I \ / \ (\sum A_e)_I + 1 \ < 0.8 \tag{2.34}$$

Where;
A_e : Effective shear area.

The effective shear area in any floor can be determined using the equation below:

$$\sum A_e = \sum A_w + \sum A_g + 0.15 \ \sum A_k \tag{2.35}$$

Where;
A_w : Effective of web area of column cross sections.(leaving out the projection in the orthogonal direction to the seismic action)
A_g : Section areas at any floor of structural components.
A_k : Infill wall areas.

Moreover, in the case where the total infill area defined in the equation above at any floor is higher than the infill area of the floor located above it, the infill walls must not taken into consideration while defining the strength irregularity factor η_{ci}.also when the value of the strength irregularity factor is In the range between 0.6 and 0.8 ($0,60 \leq (\eta_{ci})_{min} < 0,80$) , the structure behavior factor has to be multiplied by 1.25. The minimum strength irregularity factor $(\eta_{ci})_{min}$ that to be subjected to the whole structure in both earthquake directions, should not in any circumstance be taken less than 0.6. If it taken less than 0.6, the durability and the rigidity of the weak storey must be raised plus the redoing of the seismic design becomes a necessity.

B2- Interstorey Stiffness Irregularity (Soft Storey):

B2-Type of irregularity : 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 . This ratio is stated as stiffness irregularity factor and can be obtained using the equation below:

$$\eta_{ki} = (\Delta_i/h_i)_{ave} / (\eta_{i+1} / h_{i+1})_{ave} > 2.0 \tag{2.36}$$

$$\eta_{ki} = (\Delta_i/h_i)_{ave} / (\eta_{i-1} / h_{i-1})_{ave} > 2.0 \tag{2.37}$$

Where;

η_{ki} : Stiffness irregularity factor defined at i’th floor.

Δ_i : Storey drift of the i’th floor.

h_i : Height of the i’th floor [m].

It is necessary to take the effects of ±%5 additional eccentricities into consideration, while determining the Floor drifts.

B3-Discontinuity of Vertical Structural Components:

The cases in which beams or columns are used to brace the structural walls of upper floors, or the cases where the vertical structural components (structural walls, columns etc.) are absent. The following are the conditions related to this type of irregularity:

a) Whenever the location of columns is at any floor of the structure, it must not be allowed to be braced by a cantilever or to be located at the very top of the gussets of the columns located directly below it. That is to be applied in all seismic area.

b) In the circumstances in which a column rests on a beam which is supported at both ends, all internal loads elemetns resulted from the joint impact of vertical forces and earthquake forces must be raised by 50 % at all sections of the beam also the adjacent column to it, in the direction of the seismic action taken into account, must be raised by 50% as well [13].

c) Structural walls loacated in upper floors are not allowed to be braced by the columns located beneath them.

d) Structural walls, in their own Plane, are also not allowed to be supported by the beam span at any floor of the structure.

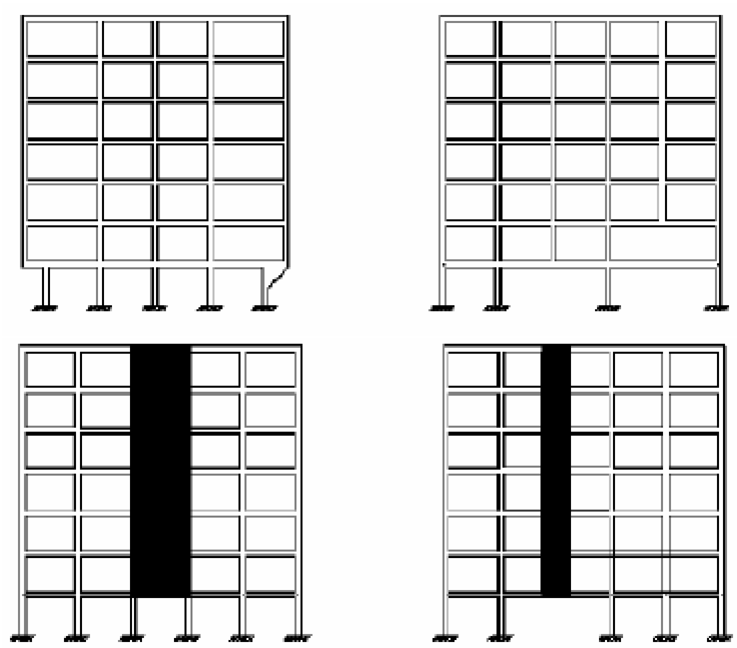


Figure 2.9: Type B3- Discontinuities of Vertical Structural Elements [13].

Article 2.3.2.4 in TEC-2007 regulations, defines these items which are shown in figure above.

2.8 Special Design Rules For Reinforced Concrete Buildings

2.8.1 Material Conditions

2.8.1.1 Eurocode 8

The following conditions should be implemented in the fundamental seismic components of the reinforced concrete building

- For DCM and DCH reinforced concrete components, the lowest concrete class than can be taken is C16/20 and C20/25 respectively.

- Excluding closed stirrups and crossties, in critical regions of essential earthquake components of DCM and DCH type of building, only ribbed bars are allowed to be used as reinforcing steel.

- In the critical regions of the main earthquake components, reinforcing steel of grade B or C, determined by Eurocode 2, should be implemented for DCM type of building but for DCH type of building only reinforcing steel of grade C should be implemented. The table below shows the properties of reinforcing steel classes according to Eurocode 2.

Table 2.17: Characteristic of Reinforcement [12].

Product form	**Bars and de-coiled rods**			**Wire fabrics**			**Requirement of quantile value (%)**
Classes	A	B	C	D	E	F	-
Characteristic yield strength f_{yk} or $f_{0,2k}$ *(Mpa)*	400 to 600						5.0
Minimum value of $k = (f_t / f_y)_k$	≥1.05	≥1.08	≥ 1.15 < 1.35	≥ 1.05	≥1.08	≥ 1.15 < 1.35	10
Characteristic strain at maximum force, ε_{uk} (%)	≥ 2.5	≥ 5.0	≥ 7.5	≥ 2.5	≥ 5.0	≥ 7.5	10
Bendability	Bend/Rebind test			-			
Shear strength	-			0,3 . $A f_{yk}$ (*A* is area of wire)			Minimum
Maximum Nominal deviation bar From size (mm) nominal mass ≤8 (individual bar ≤8 of wire (%)	± 6.0 ±4.5						5.0

Note: The value for the fatigue stress range with an upper limit of β f_{yk} and for the Minimum relative rib area that to be used bya nation can be taken from its National Appendex.

- **For DCM reinforced concrete components, welded wire meshes can be utilized if they fullfil the design conditions.**

2.8.1.2 TEC-2007

The following material conditions should be implemented when designing and building are reinforced in seismic areas:

- Concrete with strength lower than C20 should not be utilized. Additionally, the concrete that should be utilized should meet the quality control requirements defined in Turkish standard (TS-500).

- Ribbed bars along with stirrups and crossties can be utilized as a reinforcement steels with strength lower than S420 and with a rupture strain of 10% or higher.

$$\sum A_g \,/ \sum A_p \geq 0.002 \qquad (2.38)$$

$$V_t \,/ \sum A_g \leq 0.5\, f_{ctd} \qquad (2.39)$$

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.

2.8.2 Geometric Conditions

2.8.2.1 According to Eurocode 8

a) **Beam**

- The thickness of the primary seismic beams should not be taken lower than 20 cm, for the building constructed according to the DCH reinforced concrete beam design. In addition, the ratio of the width respect to the height should satisfy the equation below [13]:

$$(\, l_{ot} /b \,) \leq 70 \,/\, (\, h /b \,)^{1/3} \quad \text{and} \quad h /b \leq 3.5 \qquad (2.40)$$

Where:
l_{ot} :present the Distance between torsional restraints.
b : present the Total depth of beam in central part of l_{ot}.
h : present the thickness of compression flange.

- The length located between the centroid axes of the two members have to be higher than $b_c/4$, for buildings using the DCM and DCH reinforced concrete beam design. b_c is defined as the highest cross-sectional dimension of the column perpendicular to the longitudinal axis of the beam.

- To make use of the favorable effect of column compression on the bond of horizontal bars passing through the joint, the thickness b_w of a primary seismic beam should fulfill the equation below:

$$b_w \leq \min \{ b_w + h_w \ ; 2b_c \} \tag{2.41}$$

Where;
h_w: present the depth of the beam
b_c: present cross-sectional dimension of column.

b) Column

- For DCH reinforced concrete column design, the cross-sectional sides of primary seismic columns, h, must not be taken lower than 25 cm.

- 1/10 of the bigger length amidst the point of contra flexure and the ends of the pillar should be lower than the value that can be taken for the cross-sectional dimensions of primary seismic columns in DCM and DCH reinforced concrete column design, unless θ, which is Interstorey drift sensitivity coefficient ≤ 1.0, for bending in a plane adjacent to the column dimension that is taken into consideration.

c) Ductile Shear-Wall Geometric Conditions

- the thickness of the web, b_{wo}, (in meters) in the case of DCM and DCH reinforced concrete ductile shear walls design should meet the requirement defined by the equation below:

$$b_{wo} \geq \max \{ 0.15 \text{ or } h_s / 20 \} \tag{2.42}$$

Where;
h_s: present the clear floor elevation in meters.

Random holes, not systematically arranged to form coupled walls, must be refrained from in primary seismic shear walls, except for the cases where their impact is negligible or taken into consideration while analyzing, dimensioning and depecting the structure.

2.8.2.2 According to TEC-2007

a) Beam Geometric Conditions

Dimension of beams cross sections should meet the following requirements:

- $b_w \geq 250\ mm$
- $b_w \leq h_b + b_c$
- $b_h \geq \{ 3\ t_w\ ;\ 300mm \}$
- $b_h \leq \{ 3{,}5\ b_w\ ;\ 1/4\ l\ ^* \}$

Where,
b_w: width of beam web,
h_b: height of beam,
b_c: diameter of the supporting column in the orthogonal direction to the beam's axis,
L_n: clear span.

* Special cases where the conditions mention above are not fullfiled, the bracing of the web should be provided along the elevation of the beam on the two sides of the web.

Only if the design of the axial load of the beam meets the consecutive requirement, it must be redesigned as a column.

$$N_d \leq 0,1\ A_c\ f_{ck} \tag{2.43}$$

Where;
N_d: is the factor axial force formulated under concurent action of vertical forces and earthquake forces,
A_c: is the gross section area,
f_{ck}: typical compressive cylinder strength of concrete.

b) Column Geometric Conditions

- The lowest height of the shorter length of columns with rectangular section that is allowed to be taken is 2.50cm and the section area should not be lower than 7.5cm^2. Moreover, the diameter of circular columns must be taken no lower than 3cm.

- the gross section area of column (A_c) should be taken as the biggest one of axial pressure strengths determined under the joint effect of N_{dm} vertical forces and earthquake forces, for that to happen it should meet the requirements below:

$$A_c \geq N_{dmax} / (0.50\ f_{ck}) \tag{2.44}$$

Where:
N_{dmax} : highest of the axial pressure forces calculated under combined effect of seismic loads and vertical loads.

c) Ductile Shear-Wall Geometric Conditions

- The architectural walls present the vertical components of the architectural system in cases in which the length to thickness ratio in plan is equivalent to seven or more. Except in the following cases defined below:

1. In structures in which the earthquake forces are completly supported by walls throughout the whole height of the structure, wall thickness must not be lower than 0.05 of the longest height of the floor or 15cm, as long as both requirements presented by equations (2.1) and (2.2), moreover, these equations must be implemented at the base storey level in structures with rigid peripheral walls in basement storey, whereas it should be implemented at the top level of the foundation for different structural buildings.

2. In walls that located in the lateral direction of the components that has a length equal to at least to 0.2 of the floor length while having a floor length higher than 6m, wall thickness in the ground can be equal to minimum of 0.05 of horizontal length between the points where it's located in lateral direction. Although, the value of the thickness should not be lower than 3cm.

3. Other than the two cases before, the wall thickness must not be taken lower than 0.05 of the floor elevation or 2cm.

2.8.3 Reinforcement Conditions

2.8.3.1 Reinforcement Conditions According to EC8

1- Beam Reinforcement Conditions

Table 2.18: Beam Reinforcement Conditions Defined By EC8 [12].

	DCH	DCM
critical region length(1)	1.5 h_w	h_w
-Longitudinal bars (L) :		
$\rho_{min,}$ tension side(2)	$0.5\ f_{ctm} / f_{yk}$	
$\rho_{max,}$ critical regions(3)	$\rho' + 0.018\ f_{cd} / (\mu_{\varphi}\ \varepsilon_{sy,d}\ f_{yd})$	
$A_{s,min}$ top and bottom	2Ø14 (308 mm^2)	-
$A_{s,min}$ critical regions	$0.5A_{s,top}$	-
$A_{s,min}$ top – span	$A_{s,top-support} / 4$	
$A_{s,min,}$ supports bottom	$A_{s,bottom-span} / 4$	-
d_{bL}/h_c – bar crossing interior joint(4)	$\leq \frac{6.25(1+0.8v_d)}{(1+0.75\frac{\rho'}{\rho_{max}})} \frac{f_{ctm}}{f_{yd}}$	$\leq \frac{7.5(1+0.8v_d)}{(1+0.5\frac{\rho'}{\rho_{max}})} \frac{f_{ctm}}{f_{yd}}$
d_{bL}/h_c – bar crossing exterior joint(4)	$\leq 6.25(1+0.8v_d) \frac{f_{ctm}}{f_{yd}}$	
-Transverse bars (w):		
I-Outside critical regions(5)		
Spacing s_w	$\leq 0.75d$	
ρ_w	$0.08\sqrt{\frac{f_{ck}(Mpa)}{f_{yk}(Mpa)}}$	
II-in critical regions(5)		
d_{bw} (6)	$\geq 6mm$	
Spacing s_w	$\leq \min\{6d_{bL}, h_w/4, 24b_w, 175mm\}$	$\leq \min\{8d_{bL}, h_w/4, 24b_w, 225mm\}$

(1) For beams upholding cut-off vertical compenents, the critical length should be $2h_w$, where h_w: is the depth of the beam [12].

(2) f_{ctm} is the main value tensile strength of concrete, and f_{yk} is the typical yield strength [12].

(3) f_{cd} is the design merit of concrete compressive strength, $\mu\phi$ present the amount of the curvature ductility factor that is related to the primary amount, q_o, of the behavior

factor utilized in the framework of the structure as: $\mu_\varphi=2q_o-1$ if $T \geq T_C$ or $\mu_\varphi=1+2(q_o-1)T_C/T$ if $T<T_C$. $\varepsilon_{sy,d}$ is the design value of steel at yield, and f_{yd} is the design merit of yield strength of steel [12].

(4) h_c is the column depth in the direction of the bar, d_{bL} is the diameter of the longitude bars and $v_d = N_{Ed} / A_c f_{cd}$ is the column axial force ratio, for the analyticaly lowest merit of the axial force because of the design seismic action plus the simeltanious gravity [12].

(5) The first hoop shall be ≥ 50mm from the first beam end section [12].

(6) d_{bw} is the diameter of hoops [12].

2- Column Reinforcement Conditions

Table 2.19: Columns Reinforcement Requirements Defined By To EC8 [13]

	DCH	**DMC**
"Critical regions" Length is$^{(1)}$	max{$1.5h_c$, $1.5b_c$, 0.6m, $l_c/5$}	max{h_c, b_c, 0.45m, $l_c/6$}
Longitudinal bars (L) :		
ρ_{lmin}	0.01	
ρ_{lmax}	0.04	
Symmetrical cross-sections	$\rho=\rho'$	
At the corners$^{(2)}$	One bar along each column side	
Spacing between confined bars	≤15cm	≤20cm
Distance of unconfined bar from nearest confined	≤15cm	
Transverse bars (w)		
Outside critical regions		
Spacing s	min{$20d_{bL}$, h_c, b_c, 40cm}	min{$12d_{bL}$, $0.6h_c$, $0.6b_c$, 24cm}
Inside critical regions:		
$d_{bw}^{(4)}$	≥{6mm, $0.4(f_{ydL}/f_{ywd})^{0.5}$, d_{bLmax}}	≥{6mm, $d_{bLmax}/4$}
Spacing s	min{$6d_{bL}$, $b_o/3$, 12.5cm}	min{$8d_{bL}$, $b_o/2$, 17.5cm}
$\omega_{wd.min}^{(5)}$	0.08	-
$\alpha\omega_{wd}^{(6)}$	$30\mu_\phi v_d \varepsilon_{sy,d} b_c/b_o - 0.035$	
In critical region at		

column base:		
$\omega_{wd.min}^{(5)}$	0.12	0.08
$\alpha\omega_{wd}$	$30\mu_{\phi}v_d\varepsilon_{sy,d}b_c/b_o$ - 0.035	-

(1) If l_c/h_c <3, the entire length of the column must be treated as a critical regions and must be reinforced as such. Where l_c is the length of the column, h_c is the largest cross-sectional dimension of the columns (in meters), b_c is the cross-sectional dimension of column [12].

(2) At least one middle bar must be placed between corner bars throughout the two side of the column, to assure the stability of the beam-column joints [12].

(3) d_{bw} is the diameter of the hoops [12].

(4) ω_{wd} the ratio of the volume of confining hoops to the volume of the confined core to the centerline of the perimeter hoop, times f_{yd}/f_{cd} [12].

(5) α is the confinement effectiveness factor, determined as $\alpha = \alpha_s.\alpha_n$; where α_s=(1-s/2b_o) for hoops and α_s=(1-s/2b_o) for spirals : α_n=1- $\{b_o/((n_h-1)h_o)+h_o/((n_b-1)b)\}/3$ for rectangular hoops with n_b legs parallel to the side of the core with length b_o and n_h legs parallel to the one with length h_o [12].

(6) Index c stand for the completely concrete section and index o the restrained core to the central of the perimeter hoop; b_o is the shorter side of this core [12].

3- Ductile Shear-Wall Reinforcement Conditions:

Table 2.20: Ductile Shear-Wall Reinforcement Conditions Defined By EC8 [13]

	DCH	**DCM**
critical regions length(1)	$\geq$ max (l_w, $H_w/6$) $\leq$ min ($2l_w$, h storey) if$\leq$ 6 storey $\leq$ min ($2l_w$, 2h storey) if > 6 storey	
boundary elements:		
a)In critical regions		
-length of l_c from the edge $\geq$	$0.15l_w$, $1.5b_w$, length over which ε_c>0.0035	
-thickness b_w over $\geq$	0.2m; $h_{st}/15$ if $l_c \leq$ max ($2b_w$, $l_w/5$), $h_{st}/10$ if l_c>max($2b_w$, $l_w/5$)	
-vertical reinforcement:		
$\rho_{w,min}$	0.5 %	
$\rho_{w,max}$	4 %	
Confining hoop (w)(2):		
$d_{bw}\geq$	6mm, $0.4(f_{yd}/f_{ywd})^{0.5}d_{bL}$	6mm
Spacing $s_w\leq$	$6d_{bL}$, $b_o/3$, 12.5cm	$8d_{bL}$, $b_o/2$, 17.5cm
$\omega_{wd}\geq$	0.12	0.8
$\alpha\omega_{wd}\geq$(3)	$30\mu\phi(v_d+\omega_v)\varepsilon_{sy,d}b_c/b_o$ - 0.035	
b)Over the rest of the elevation of the wall	In parts of the section where ε_c>0.2%: $\rho_{v,min}$=0.5%; Elsewhere 0.2% In parts of the section where ρ>2%: - length of unstrained bar in the compression region from closest restrained bar$\leq$15cm; -hoops with $b_w\geq$ max (6mm, $d_{bL}/4$) & spacing $s_w\leq$ min ($20d_{bL}$, b_{wo}, 40cm) outside the limit of this length.	
Web:-		
Vertical bars (v) :		
$\rho_{v,min}$	Where in the section ε_c> 0.2%: 0.5%; elsewhere 0.2%	
$\rho_{v,max}$	4%	
$d_{bv}\geq$	8mm	-
$d_{bv}\leq$	$b_{wo}/8$	
Spacing s_v	min($25d_{bw}$, 25cm)	min($3b_{wo}$, 40cm)
horizontal bars (h) :		
$\rho_{h,min}$	0.2%	max(0.1%, 25 ρ_v)
$d_{bh}\geq$	8mm	-
$d_{bh}\leq$	$b_{wo}/8$	-
Spacing $s_h\leq$	min{$25d_{bh}$, 25cm}	40cm
Axial load ratio v_d N_{Ed}/A_cf_{cd}	$\leq$ 0.035	$\leq$ 0.4

(1) l_w present the large dimension of the rectangular wall section or rectangular section there ; H_w is the total elevation of the wall; h_{storey} is the elevation of the floor [12].

(2) For DCM; The DCL conditions are satisfied and can be adapted to the confining reinforcment of boundary components if $v_d = N_{Ed}/A_c f_{cd}$ fullfil $v_d \leq 0.15$. It's also worth mentioning that it can be applied if $v_d \leq 0.2$ and that the merit of q utilized in the design is less than or equal to 85% of the merit permitted when the DCM confining rienforcement is utilized in boundary components [12].

(3) Curvature ductility factor, determined as: $\mu\varphi=2q_o-1$ if $T \geq T$C or $\mu\varphi=1+2(q_o-1)T_C/T$ if $T<T_C$, to the output of the basic merit qo times the merit of the ratio M_{Edo} / M_{Rdo} at the lowest point of the wall. $\varepsilon_{sy,d}= f_{yd}/E_s$,$\omega_{vd}$ is considered as the mechanical ratio of the vertical web reinforcement [12].

(4) M_{Edo} represent the moment at the lowest point of the wall from the analysis for the seismic design situation; M_{Rdo} is the design merit of the flexural capacity at the lowest point of the wall for the axial force N_{Ed} from the analysis for the same seismic design situation [12].

2.8.3.2 Reinforcement Conditions According to TEC-2007

1- Beam Reinforcement Conditions:

Table 2.21: Generals Rules of TEC-2007 Beams Reinforcement Design [13].

	HDL	**NDL**
Longitudinal reinforcement:		
$\rho_{min}^{(1)}$	$\geq 0.8\ f_{ctd}/f_{yd}$	
ρ_{max}	≤ 0.02	
$D_{bar}^{(2)}$	≥ 12 mm	

- For first & second seismic zone(1) $A_{s,bottom-support}$	$\geq 0.5 A_{s,top-supports}$
- For third & fourth seismic zone(1) $A_{s,bottom-support}$	$\geq 0.3 A_{s,top-supports}$
s_b	$\leq$ 30cm
$A_{s,extended}$	0.25 $A_{s,top-beam}$
Beam-column	
$A_{s,extended}$	
For 90° reinforce bent inside of column	
- horizontal	$\geq$ 0.41 b
- vertical	$\leq$ 12 Ø
Spacing s_{hoops}	$\leq$ 0.25 b_d(4), 10cm
Transfer reinforcements:	
Mechanical connections, welded lap splice	$\geq$ 60cm
Confinement zone length	$2b_d$
s_{hoop}	$0.25b_d$, $8D_{bar,min}$, 150mm

(1) The minimum ratio of top tension reinforcements at beams support [13].

(2) D_{bar} is the diameter of longitudinal rebars [13].

2- Column Reinforcement Conditions:

Table 2.22: Column Reinforcement Conditions Defined by the TEC-2007 [13]

	HDL	**NDL**
Longitudinal reinforcement		
ρ_{min}	$\geq$ 0.01	
ρ_{max}	$\leq$ 0.04	
Min number of rebars	- Rectangular sections: 4Ø16 or 6Ø14 - Circular sections: 6Ø14	
- lap splices section ρ_{max}	0.06	
bottom end – column(1) $\leq$ 50% > 50%	$\geq 1.25 l_b$ $\geq 1.5 l_b$	
Transfer reinforcements:		
confined zone length(2)	$\geq$ min{D_{min}, 1/6 h_c, 500 mm}	
Reinforcement diameter	$\geq$ Ø 8	
extended(3)	$\geq 2D_{min}$	
continued(4)	$\geq$ {$25D_{max}$, 300}	
spacing s_{hoop}	$\leq$ [1/3 D_{min}, 100 mm] $\geq$ 50 mm	

$a^{(5)}$	$\leq D_{hoop}$
pitch of spirals	$\leq$ [1/5 D_{core} , 80mm]
-If $N_d > 0.2\ A_c\ f_{ck}^{(6)}$:	
columns with hoops	$A_{sh} \geq 0.3\ s\ b_k\ [(A_c/A_{ck}) - 1]\ (f_{ck} / f_{ywk})$ $A_{sh} \geq 0.075\ s\ b_k\ (f_{ck} / f_{ywk})$
columns with spirals	$\rho_s \geq 2/3\ 0.12\ (f_{ck} / f_{ywk})$
Central columns$^{(6)}$ reinforcement	
- Transfer reinforcement	$\leq$ Ø8
s_{hoops}, crossties	$\leq 1/2\ D_{max}$
$a^{(5)}$	25 D_{hoop}

(1) In the circumstances where lap joins of column longitudinal reinforcement are created at the bottom end [13].

(2) D_{min} is the smallest dimension of beam cross-section, h_c is clear height of the column [13].

(3) Reinforcement shall be exceeding into foundation [13].

(4) The reinforcement shall be continued the length inside the foundations [13].

(5) a, is the lateral length between legs of hoops and crossties [13].

(6) N_d present the axial load formulated under the combine effect of seismic forces and vertical forces multiplied with loads coefficients, Ac is the gross area of column or wall zone, f_{ck} present the characteristic compressive cylinder strength of concrete, A_{ck} present the concrete core area within outer edges of confinement reinforcement, f_{ywk} present the characteristic yield strength of the transverse reinforcement [13].

3- Ductile Shear-Wall Reinforcement Conditions

Table 2.23: Ductile Shear Wall Reinforcement Conditions Defined by the TEC-2007 [13].

	HDL	**NDL**
critical wall height(1)		
Web reinforcement:		
$\rho_{min}^{(2)}$	≥ 0.0025	
spacing s	≤ 25cm	
Or: -		
$\rho_{min}^{(3)}$	0.0015	
spacing (s)(3)	≤ 30cm	
$\rho_{min}^{(4)}$	0.002	
Wall end zones reinforcement:		
$\rho_{min}^{(5)}$	≥ 0.001	
As_{min}	$\geq 4Ø\ 14$	
Transfer reinforcement:		
D_{min}	≥ 8mm	
a(5)	$\leq 25\ D_{hoop}$	
spacing (s)(7)	$\leq 0.5\ b_{web}$ or 10cm ≥ 5cm	
confinements zones(8)	$\geq 2/3\ A_{sh}$	
extended steel(9)	$\geq 2b_{web}$	

(1) If $H_w/l_w \leq 2.0$ web section should be taken as the whole section of the wall, where H_w is the wall height measured from level that reduce more than 20% of the length of the wall in plan or from the top of the ground [13].

(2) Total cross section area of each the vertical and the horizontal web reinforcement on the two sides of the wall [13].

(3) If equations 2.38 and 2.39 are satisfied [13].

(4) For critical wall height zones [13].

(5) The ratio will be increase to $2x10^{-3}$ throughout the elevation of the critical wall as determined before [13].

(6) The lateral distance between legs of hoops and crossties [13].

(7) Vertical spacing of the hoops and / or crossties [13].

(8) For confinement zones of columns $\geq$ 2/3 A_{sh} must be applied along the elevation of the critical wall [13].

(9) Such reinforcement shall be extended into the foundations [13].

2.9 Comparison of EC8 & TEC-2007

Both TEC-2007 and EC8 were written while putting into account the safety of the residences within the structure, the usage of the structure and the limitation of collateral damages to the structure after the occurrence of an earthquake. The limit states and recommendations has been set respecting these criteria almost in same manner in both the TEC-2007 and the EC8.

However with that being said, the two codes try to make an estimation of site-specific design ground motions and represent them by an equivalent static force applied to the structure. That may be acquired by taking a probabilistic seismic hazard analysis approach. Concerning this matter, the ground types were put according to the geographic properties for each region for both the Eurocode 8 and the 2007 Turkish Earthquake Code. Determining the parameters for the elastic response spectra, TEC-2007 determine the local site class by relating to soils class and the width of the layer. On the other hand, the Eurocode 8 take the width of the layer into consideration for the ground type via an expression of average shear wave velocity.

Regarding the probabilistic approach, both codes depends on the seismic zone maps to acquire the hazard of the earthquake as a single parameter. Thus determining their importance factors based on the reference peak ground acceleration factor, which is, ought to be in buildings with 50 years periods and 10% exceedance probability. In the case of the 2007 Turkish Earthquake Code, it implies that the factor of importance should always be equal to one or higher which is slightly safer than the Eurocode 8 (TEC-2007 does not use a factor lower than 1 however EC8 use 0.8 as its lowest factor).

This sufficient that, by utilizing the zones maps which display the level of the anticipated ground motions within the area of application of the seismic code, and then by using the represented parameters for these zones along with a categorization of the near-surface geology; elastic response spectrum can be formulated at any given location [6]. In this regard, both seismic codes define their own elastic response with spectra curves with the same 5% damping ratio while using a slightly diverse approach from one another: for distinctive ground types, Eurocode 8 characterize two distinctive model of elastic response spectra, type 1 for higher seismicity region and type 2 for mild seismicity region. These two models are being used by various European countries with distinctive seismicity. The elastic response spectra is formulated: by leaning on ground types, the soil factors, the periods that are resolved with extra criterion such as design ground acceleration (usually taken from the seismic zones maps and multiplied by the factor of importance), and damping correction factor. On the other hand, the 2007 Turkish Earthquake Code define only one spectra depending on the local site class; periods are resolved, then the spectra is drawn without taking the design ground acceleration into consideration. Hence, the elastic spectral acceleration is formulated by relating upon the ground acceleration and the factor of importance (after defining the spectra's ordinate for the relevant natural period of the structure). Although there is an obvious difference in calculation order both the 2007 Turkish Earthquake Code and Eurocode 8 deliver the same result.

To minimize the seismic needs of the structure which that the elastic spectra do not take into consideration. Ground motions usally cause non-linear behavior in buildings. For this reason both codes formulate their method by making an alternation to the elastic spectra by adding factors that provide a substitution for the ability of the building to deplete the seismic energy over inelastic dislocations, which are called behavior factors.

The behavior factors determined in TEC-2007 and EC8 have minor distinctions. The 2007 Turkish Earthquake Code define behavior factors for different architectural systems and ductility classes. Nonetheless, it acknowledge substituting the factor of behavior if it is essential, also it particularly supervise the usage of structural walls frame systems of high ductility level. The Eurocode 8 define behavior factors as constant values along with extra specifications to get more precise and authentic results, for system with different type of structures and ductility's classes. In addition, it permit the determination of these specifications by using the non-linear analysis. We can say that the factors that are determined in TEC-2007 are higher than the factors defined in the Eurocode 8 by comparison.

Concerning the classification of structure's ductility, Eurocode 8 define three classes for ductility high, mild and low. It allow the designing process of infrastructure with non-dissipative behavior (low ductility) in area with low seismicity corresponding to the standard for concrete's design of building defined in Eurocode 2 along with an extra condition for the usage of steel reinforcing of classes C and B. They depict two distinct combination of ductility and strength also; they give an additional choice to exchange them in the design. DCH buildings are permitted to take a higher values of factors of behavior than DCM, also to take more demanding condition for members, in addition to arrange a higher safety margins related to capacity design computation to guarantee the ductile global behavior. The 2007 Turkish Earthquake Code take on the same approach, by defining two classes of ductility: high and nominal. Nevertheless it employ some strict supervision to pick between them while considering the class's importance, the building's elevation etc. thus in some cases, it is not proposed as an option rather than a definition. Due to being located in a higher seismic zone, there is no lower class described in the 2007 Turkish Earthquake Code.

Relating to the approach of both the 2007 Turkish Earthquake Code and Eurocode 8, it can be said that the combination of the seismic action with other actions in the 2007 Turkish Earthquake Code is more on the conservative side comparing to Eurocode 8. In Eurocode 8, at the time of the earthquake not all the live loads are being shown. However, the 2007 Turkish Earthquake code exercise live load participation factor as a normal approach.

Even though both the 2007 Turkish Earthquake Code and Eurocode 8 have somehow the same guideline for conceptual design, there is some distinction concerning the application of the code. At first, concerning the regularity of structures both the 2007

Turkish Earthquake Code and Eurocode 8 defines several criteria for structural regularities for the sake of categorizing structures in two groups: regular and non-regular. This differentiation has suggestion for some aspect such as simplifying the planar model (the structural model), the lateral force (analyzing procedure) and the amount of behavior factor (remain the same in the 2007 Turkish Earthquake Code). As stated in Eurocode 8, it requires the usage of model analysis to make a categorization of irregularity in elevation, which leads to minimizing the q factor. Similarly, a categorization of irregularities in plan requires the usage of a 3D structural model. Due to the irregularity in plan, it is requires the combination of the earthquake’s effect in the two principle directions of the structure. The 2007 Turkish Earthquake Code has similar approach with some distinction like requirements concerning the seismic zone and the height of buildings. In certain circumstances where Torsional irregularity in plan are apparent in structure which are located in the seismic zone 1 or 2,the lateral force approach can be applied exclusively to structures with height less than 25 meter. Also, in the cases were a soft story irregularity in elevation is apparent in the same structure, the height of the building should be less than 60 meter so that a lateral analysis can be applied. Irregular configurations are being allowed while penalizing the design specifications.

While considering the regularities in buildings, both the 2007 Turkish Earthquake Code and Eurocode 8 use distinct approach. The structural regularities in TEC-2007 are more direct and more specified than in its counterpart. For instance, they mentioned as irregularities in elevation and irregularities in plan and symbols were assigned for each one so that they can be differentiate. In addition to this, each category contains a set of sub-clauses in it: for example, irregularities in plan has 3 sub-clauses the Torsional irregularities (A1), Floor discontinuity (A2), and projections in plan. On the other hand, in Eurocode 8 there exist a set of requirement which a building have to comply with so that it can be classified as regular, and even if a building fail to meet even one of these requirement it will be classified as irregular: requirement concerning strength, stability and fire resistance of structures. Although these requirements are fewer in comparison with the ones in the 2007 Turkish Earthquake Code, they grant designers with more design opportunities. To wrap things up, we can say for sure that the two codes underline the same critical requirement, which are crucial for the seismic design.

Finaly as for the analysis procedure, both codes define the same procedure for relevant structure with a slight distinction in the calculation procedures. Both implement ±5 % extra eccentricity in every direction so that they can clarify the skepticism in the region of mass in the earthquake. Nevertheless, in the lateral force method of analysis, Eurocode 8 take into consideration the incidental Torsional effects by multiplying with a factor the actions effects in the individual load resisting system, that if no more accurate method is used to define the accidental eccentricity.

Chapter 3

METHODOLOGY

3.1 Introduction

In this section several case studies were designed according to TEC-2007 and Eurocode 8. Five types of irregular building were chosen and investigated upon, with two different elevations: three floor and five floor buildings. STA4CAD computer software was used for designing and performing the non-linear static pushover analysis. Moreover, this chapter contain detailed information about the building's geometry and the TEC-2007 and EC8 parameters for the seismic design chosen in this study. Also for the sake of making a better comparison, three cases for analysis were selected:

- First Analysis Case: where the spectrum, A_0 and the behavior factor where taken according to the advised value defined by the codes (each case according to its corresponding code).

- Second Analysis Case: where the ATC-3 normalized spectrum was used for both the TEC-2007 and the EC8 cases, while using the same value for A_0 and the behavior factor for both cases (0.3 and 6 respectively).

- Third Analysis Case: where the ATC-3 normalized response spectrum is used for both cases, the TEC-2007 & the EC8 case, but with the advised value of A_0 and behavior factor that are defined according to their corresponding code.

3.2 Common Design Parameter

In this section the common design parameter used in this study can be found in their corresponding table.

Table 3.1: EC8 Parameters for the First Analysis Case.

<table>
<tr><th colspan="2">Parameter</th><th>EC8</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>residental</td></tr>
<tr><td colspan="2">Behavior factor q</td><td>5.85</td><td>DCH, frame, multi bay</td></tr>
<tr><td colspan="2">Response spectrum</td><td>1</td><td>High seismicity</td></tr>
<tr><td colspan="2">Ground type</td><td>B</td><td>-</td></tr>
<tr><td colspan="2">Soil factor</td><td>1.2</td><td>Ground type B</td></tr>
<tr><td colspan="2">A_0</td><td>0.25</td><td>Seismic map of Cyprus</td></tr>
<tr><td rowspan="2">Periods</td><td>T_B</td><td>0.15</td><td rowspan="2">Type 1 spectrum, ground type B</td></tr>
<tr><td>T_C</td><td>0.5</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>
<tr><td colspan="2">Method of design</td><td>Nonlinear static pushover analysis</td><td>-</td></tr>
<tr><td colspan="2">Reinforced concrete</td><td>Ultimate limit</td><td></td></tr>
</table>

design method	state	-

Table 3.2: TEC-2007 Parameters for the First Analysis Case.

Parameter		TEC-2007	Explanation
Seismic zone		2	-
Importance factor		1	residental
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	-

Table 2.3: EC8 & TEC-2007 Parameters for the Second Analysis Case.

Parameter	**EC8 & TEC-2007**
Seismic zone	2
Importance factor	1
Behavior factor	6
Response spectrum	ATC-3
Ground type	B
Soil factor	1.2
A_0	0.3
Damping factor	5%
Concrete type	C20
Steel type	420C
Method of design	Nonlinear static pushover analysis
Reinforced concrete design method	Ultimate limit state

Table 2.4: EC8 Parameters for the Third Analysis Case.

Parameter	**EC8**
Seismic zone	2
Importance factor	1
Behavior factor	5.85
Response spectrum	ATC-3
Ground type	B
Soil factor	1.2
A_0	0.25

Damping factor	5%
Concrete type	C20
Steel type	420C
Method of design	Nonlinear static pushover analysis
Reinforced concrete design method	Ultimate limit state

Table 3.5: TEC-2007 Parameters for the Third Analysis Case.

Parameter	**TEC-2007**
Seismic zone	2
Importance factor	1
Behavior factor	8
Response spectrum	ATC-3
Ground type	B
Soil factor	1.2
A_0	0.3
Damping factor	5%
Concrete type	C20
Steel type	420C
Method of design	Nonlinear static pushover analysis
Reinforced concrete design method	Ultimate limit state

3.3 Case Studies

3.3.1 Case Study 1 (Weak Storey):

A building that has a weak storey is a building, where the strength irregularity factor η_{ci} is below 0.8

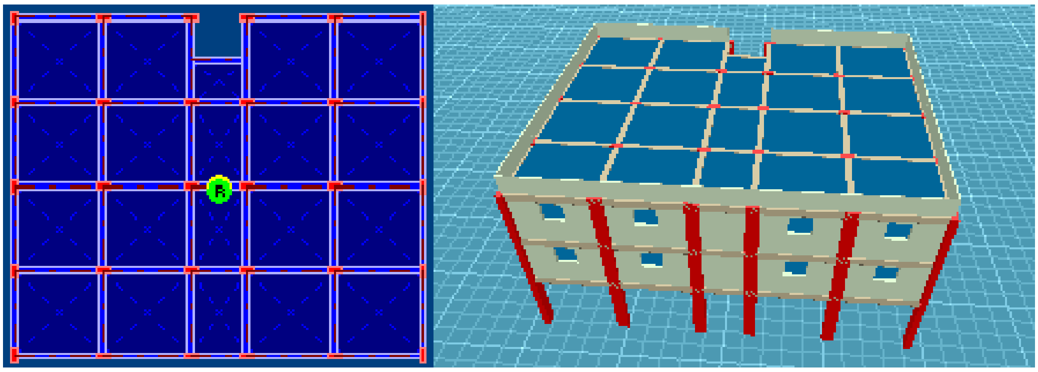


Figure 3.1: Two & Three Dimensional Plan of Case 1 (Weak Storey).

Table 3.6: Case 1 (Weak Storey) Building Specifications.

Case	**Case 1 (Weak Storey)**	
Floor number	**3F (10m)**	**5F (16m)**
Columns	60x30 cm^2	60x30 cm^2
Beams	30x40 cm^2	30x40 cm^2
Slab	20 cm	20 cm

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 3.1: Irregularity Check of Case 1 (Weak Storey) 3F.

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

Report 3.2: Irregularity Check of Case 1 (Weak Storey) 5F.

3.3.2 Case Study 2 (Soft Storey):

A building is classified, as having a soft storey, is a building that has one storey with a 70% less stiffness than the storey located directly above it.

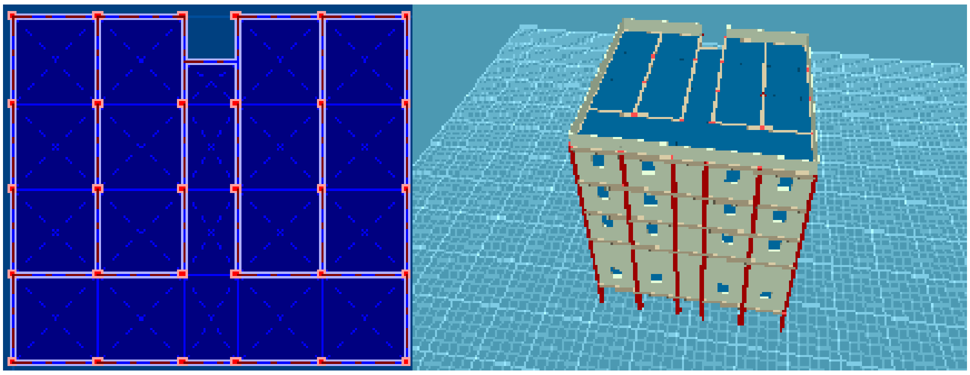


Figure 3.2: Two & Three Dimensional Plan of Case 2 (Soft Storey).

Table 3.7: Case 2 (Soft Storey) Building Specifications

Case	**Case 2 (Soft Storey)**	
Floor number	**3F (12m)**	**5F (18m)**
Columns	60x30 cm^2(1st F)	60x30 cm^2 (1st F)
	40x30 cm^2(2nd F)	40x30 cm^2(2nd F)
	30x30 cm^2(other F)	30x30 cm^2(other F)
Beams	30x40 cm^2	30x40 cm^2
Slab	20 cm	20 cm

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	Σ Aex	Σ 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 3.3: Irregularity Check of Case 2 (Soft Storey) 3F.

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	Σ Aex	Σ 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

Report 3.4: Irregularity Check of Case 2 (Soft Storey) 5F.

3.3.3 Case Study 3 (Projection in Plan):

The cases in which 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%.

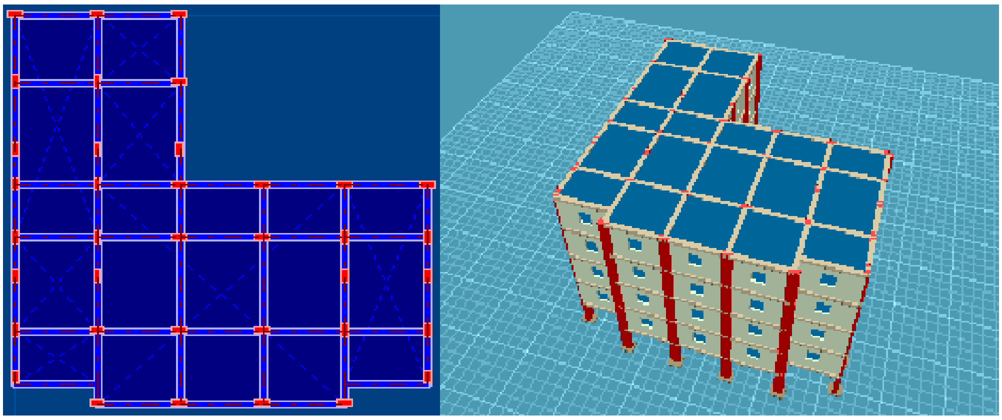


Figure 3.3: Two & Three Dimensional Plan of Case 3 (Projection in Plan).

Table 3.8: Case 3 (Projection in Plan) Building Specifications.

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

```
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 = 0.000647 + 0.000002 × (.0 - 13.38)=0.001549  (C117)
1. sto X dbot = 0 + 0.000002 × (23.6 - 13.38)=0.0016697  (C102)
2. sto X top  = 0.0021006 + 0.0000061 × (.0 - 13.46) - 0.001549 = 0.0034968  (C217)
2. sto X dbot = 0.0021006 + 0.0000061 × (23.6 - 13.46) - 0.0016697 = 0.0037365  (C202)
```

X DIR. (+%5)

Story	ΔX dtop(m)	ΔX dbot(m)	ΔX ort	nbi	nki	Δx/h	θi	story type
3	0.0015340	0.0016229	0.0015785	1.03	0.00	0.00316	0.00521	Normal sto
2	0.0013987	0.0014946	0.0014467	1.03	0.92	0.00291	0.00698	Normal sto
1	0.0006196	0.0006679	0.0006437	1.04	0.48	0.00140	0.00432	Normal sto

X DIR. (-%5)

Story	ΔX dtop(m)	ΔX dbot(m)	ΔX ort	nbi	nki	Δx/h	θi	story type
3	0.0015340	0.0016229	0.0015785	1.03	0.00	0.00316	0.00521	Normal sto
2	0.0013987	0.0014946	0.0014467	1.03	0.92	0.00291	0.00698	Normal sto
1	0.0006196	0.0006679	0.0006437	1.04	0.48	0.00140	0.00432	Normal sto

Y DIR. (+%5)

Story	ΔY dlft(m)	ΔY drgt(m)	ΔY ort	nbi	nki	Δy/h	θi	story type
3	0.0013645	0.0011742	0.0012693	1.07	0.00	0.00266	0.00368	Normal sto
2	0.0012997	0.0011925	0.0012461	1.04	0.98	0.00253	0.00507	Normal sto
1	0.0005936	0.0005730	0.0005833	1.02	0.50	0.00124	0.00336	Normal sto

Y DIR. (-%5)

Story	ΔY dlft(m)	ΔY drgt(m)	ΔY ort	nbi	nki	Δy/h	θi	story type
3	0.0013645	0.0011742	0.0012693	1.07	0.00	0.00266	0.00368	Normal sto
2	0.0012997	0.0011925	0.0012461	1.04	0.98	0.00253	0.00507	Normal sto
1	0.0005936	0.0005730	0.0005833	1.02	0.50	0.00124	0.00336	Normal sto

```
TDY 2.3.2.1 A1 torsional irregularity:
nbi=1.075 <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	11.84	0.00	0.00	15.96	18.68	14.23	14.64	1.00	1.00	top sto ✓
2	11.84	0.00	0.00	15.96	18.68	14.23	14.64	1.00	1.00	Regular ✓
1	11.84	0.00	0.00	0.00	0.00	11.84	11.84	0.83	0.81	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=-0.95+0.30+4.21+ 0.3 × 0.04=3.58
My=0.69+0.26+14.47+ 0.3 × 0.50=15.56
```

Report 3.5: Irregularity Check of Case 3 (Projection in Plan) 3F.

```
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 = 0.0007214 + 0.0000021 × (.0 - 13.37)=0.0017342  (C117)
1. sto X dbot = 0 + 0.0000021 × (23.6 - 13.37)=0.0018567  (C102)
2. sto X top  = 0.0024543 + 0.0000064 × (.0 - 13.44) - 0.0017342 = 0.0041857  (C217)
2. sto X dbot = 0.0024543 + 0.0000064 × (23.6 - 13.44) - 0.0018567 = 0.0044423  (C202)
```

X DIR. (+%5)

Story	ΔX dtop(m)	ΔX dbot(m)	ΔX ort	nbi	nki	Δx/h	θi	story type
5	0.0019019	0.0019665	0.0019342	1.02	0.00	0.00383	0.00809	Normal sto
4	0.0020586	0.0021481	0.0021034	1.02	1.09	0.00419	0.01475	Normal sto
3	0.0020369	0.0021430	0.0020900	1.03	0.99	0.00418	0.01938	Normal sto
2	0.0016743	0.0017769	0.0017256	1.03	0.83	0.00346	0.01746	Normal sto
1	0.0006937	0.0007427	0.0007182	1.03	0.45	0.00155	0.00861	Normal sto

X DIR. (-%5)

Story	ΔX dtop(m)	ΔX dbot(m)	ΔX ort	nbi	nki	Δx/h	θi	story type
5	0.0019019	0.0019665	0.0019342	1.02	0.00	0.00383	0.00809	Normal sto
4	0.0020586	0.0021481	0.0021034	1.02	1.09	0.00419	0.01475	Normal sto
3	0.0020369	0.0021430	0.0020900	1.03	0.99	0.00418	0.01938	Normal sto
2	0.0016743	0.0017769	0.0017256	1.03	0.83	0.00346	0.01746	Normal sto
1	0.0006937	0.0007427	0.0007182	1.03	0.45	0.00155	0.00861	Normal sto

Y DIR. (+%5)

Story	ΔY dlft(m)	ΔY drgt(m)	ΔY ort	nbi	nki	Δy/h	θi	story type
5	0.0014955	0.0011272	0.0013113	1.14	0.00	0.00292	0.00530	Normal sto
4	0.0016954	0.0013686	0.0015320	1.11	1.17	0.00331	0.00955	Normal sto
3	0.0017444	0.0014848	0.0016146	1.08	1.05	0.00340	0.01289	Normal sto
2	0.0014764	0.0013207	0.0013985	1.06	0.87	0.00288	0.01254	Normal sto
1	0.0006259	0.0005880	0.0006069	1.03	0.46	0.00131	0.00660	Normal sto

Y DIR. (-%5)

Story	ΔY dlft(m)	ΔY drgt(m)	ΔY ort	nbi	nki	Δy/h	θi	story type
5	0.0014955	0.0011272	0.0013113	1.14	0.00	0.00292	0.00530	Normal sto
4	0.0016954	0.0013686	0.0015320	1.11	1.17	0.00331	0.00955	Normal sto
3	0.0017444	0.0014848	0.0016146	1.08	1.05	0.00340	0.01289	Normal sto
2	0.0014764	0.0013207	0.0013985	1.06	0.87	0.00288	0.01254	Normal sto
1	0.0006259	0.0005880	0.0006069	1.03	0.46	0.00131	0.00660	Normal sto

TDY 2.3.2.1 A1 torsional irregularity:
nbi=1.14 <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	11.84	0.00	0.00	15.96	18.68	14.23	14.64	1.00	1.00	top sto ✓
4	11.84	0.00	0.00	15.96	18.68	14.23	14.64	1.00	1.00	Regular ✓
3	11.84	0.00	0.00	15.96	18.68	14.23	14.64	1.00	1.00	Regular ✓
2	11.84	0.00	0.00	15.96	18.68	14.23	14.64	1.00	1.00	Regular ✓
1	11.84	0.00	0.00	0.00	0.00	11.84	11.84	0.83	0.81	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=-0.93+0.28+4.55+ 0.3 × 0.07=3.92
My=0.68+0.26+14.85+ 0.3 × 0.51=15.94

Report 3.6: Irregularity Check of Case 3 (Projection in Plan) 5F.

3.3.4 Case Study 4 (Floor Discontinuity):

Is a type of building, where there is an openings which is higher than 1/3 of the gross floor area.

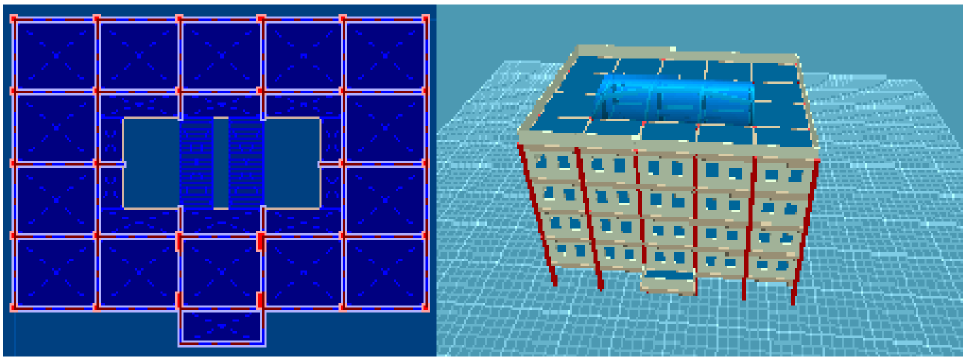


Figure 3.4: Two & Three Dimensional Plan Case 4 (Floor Discontinuity).

Table 3.9: Case 4 (Floor Discontinuity) Building Specifications.

Case	**Case 4 (Floor Discontinuity)**	
Floor number	**3F (9m)**	**5F (15m)**
Columns	40x40 cm^2	40x40 cm^2
Beams	25x50 cm^2	25x50 cm^2
Slab	20 cm	20 cm

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 = 0.0008895 + -0.0000044 × (.0 - 8.73)=0.0023192 (C101)
1. sto X dbot = 0 + -0.0000044 × (16.0 - 8.73)=0.0021443 (C123)
2. sto X top = 0.0020142 + -0.0000064 × (.0 - 8.43) - 0.0023192 = 0.0028507 (C201)
2. sto X dbot = 0.0020142 + -0.0000064 × (16.0 - 8.43) - 0.0021443 = 0.0027704 (C223)

X DIR. (+ě5)

Story	ΔX dtop(m)	ΔX dbot(m)	ΔX ort	nbi	nki	Δx/h	θi	story type
3	0.0006881	0.0006916	0.0006899	1.00	0.00	0.00135	0.00197	Normal sto
2	0.0011403	0.0011082	0.0011242	1.01	1.63	0.00222	0.00403	Normal sto
1	0.0009277	0.0008577	0.0008927	1.04	0.79	0.00181	0.00412	Normal sto

X DIR. (-ě5)

Story	ΔX dtop(m)	ΔX dbot(m)	ΔX ort	nbi	nki	Δx/h	θi	story type
3	0.0006881	0.0006916	0.0006899	1.00	0.00	0.00135	0.00197	Normal sto
2	0.0011403	0.0011082	0.0011242	1.01	1.63	0.00222	0.00403	Normal sto
1	0.0009277	0.0008577	0.0008927	1.04	0.79	0.00181	0.00412	Normal sto

Y DIR. (+ě5)

Story	ΔY dlft(m)	ΔY drgt(m)	ΔY ort	nbi	nki	Δy/h	θi	story type
3	0.0007160	0.0007172	0.0007166	1.00	0.00	0.00140	0.00176	Normal sto
2	0.0009919	0.0009889	0.0009904	1.00	1.38	0.00193	0.00310	Normal sto
1	0.0006749	0.0006747	0.0006748	1.00	0.68	0.00132	0.00278	Normal sto

Y DIR. (-%5)

Story	ΔY dlft(m)	ΔY drgt(m)	ΔY ort	nbi	nki	Δy/h	θi	story type
3	0.0007160	0.0007172	0.0007166	1.00	0.00	0.00140	0.00176	Normal sto
2	0.0009919	0.0009889	0.0009904	1.00	1.38	0.00193	0.00310	Normal sto
1	0.0006749	0.0006747	0.0006748	1.00	0.68	0.00132	0.00278	Normal sto

TDY 2.3.2.1 A1 torsional irregularity:
nbi=1.039 <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	Σ Aex	Σ Aey	ncix	nciy	EXPLANATION
3	5.04	0.00	0.00	11.82	8.24	6.81	6.28	1.00	1.00	top sto ✓
2	5.24	0.00	0.00	12.24	8.96	7.08	6.58	1.04	1.05	Regular ✓
1	5.44	0.00	0.00	0.00	0.00	5.44	5.44	0.77	0.83	< .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=-0.63+0.18+3.94+ 0.3 × 0.00=3.49
My=-0.27+0.09+2.81+ 0.3 × 0.27=2.71

Report 3.7: Irregularity Check of Case 4 (Floor Discontinuity) 3F.

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 = 0.0009701 + -0.0000049 × (.0 - 8.73)=0.0025322 (C101)
1. sto X dbot = 0 + -0.0000049 × (16.0 - 8.73)=0.0023362 (C123)
2. sto X top = 0.0023704 + -0.0000076 × (.0 - 8.44) - 0.0025322 = 0.0035538 (C201)
2. sto X dbot = 0.0023704 + -0.0000076 × (16.0 - 8.44) - 0.0023362 = 0.0034467 (C223)

X DIR. (+%5)

Story	ΔX dtop(m)	ΔX dbot(m)	ΔX ort	nbi	nki	Δx/h	θi	story type
5	0.0005288	0.0005329	0.0005309	1.00	0.00	0.00104	0.00204	Normal sto
4	0.0009276	0.0009139	0.0009207	1.01	1.73	0.00181	0.00450	Normal sto
3	0.0012477	0.0012237	0.0012357	1.01	1.34	0.00243	0.00718	Normal sto
2	0.0014215	0.0013787	0.0014001	1.02	1.13	0.00277	0.00933	Normal sto
1	0.0010129	0.0009345	0.0009737	1.04	0.70	0.00198	0.00750	Normal sto

X DIR. (-%5)

Story	ΔX dtop(m)	ΔX dbot(m)	ΔX ort	nbi	nki	Δx/h	θi	story type
5	0.0005288	0.0005329	0.0005309	1.00	0.00	0.00104	0.00204	Normal sto
4	0.0009276	0.0009139	0.0009207	1.01	1.73	0.00181	0.00450	Normal sto
3	0.0012477	0.0012237	0.0012357	1.01	1.34	0.00243	0.00718	Normal sto
2	0.0014215	0.0013787	0.0014001	1.02	1.13	0.00277	0.00933	Normal sto
1	0.0010129	0.0009345	0.0009737	1.04	0.70	0.00198	0.00750	Normal sto

Y DIR. (+%5)

Story	ΔY dlft(m)	ΔY drgt(m)	ΔY ort	nbi	nki	Δy/h	θi	story type
5	0.0006034	0.0006052	0.0006043	1.00	0.00	0.00118	0.00204	Normal sto
4	0.0009375	0.0009376	0.0009375	1.00	1.55	0.00183	0.00404	Normal sto
3	0.0011983	0.0011952	0.0011967	1.00	1.28	0.00234	0.00614	Normal sto
2	0.0012559	0.0012526	0.0012542	1.00	1.05	0.00245	0.00739	Normal sto
1	0.0007548	0.0007571	0.0007559	1.00	0.60	0.00148	0.00521	Normal sto

Y DIR. (-%5)

Story	ΔY dlft(m)	ΔY drgt(m)	ΔY ort	nbi	nki	Δy/h	θi	story type
5	0.0006034	0.0006052	0.0006043	1.00	0.00	0.00118	0.00204	Normal sto
4	0.0009375	0.0009376	0.0009375	1.00	1.55	0.00183	0.00404	Normal sto
3	0.0011983	0.0011952	0.0011967	1.00	1.28	0.00234	0.00614	Normal sto
2	0.0012559	0.0012526	0.0012542	1.00	1.05	0.00245	0.00739	Normal sto
1	0.0007548	0.0007571	0.0007559	1.00	0.60	0.00148	0.00521	Normal sto

TDY 2.3.2.1 A1 torsional irregularity:
nbi=1.04 <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	5.04	0.00	0.00	11.82	8.24	6.81	6.28	1.00	1.00	top sto ✓
4	5.24	0.00	0.00	11.82	8.24	7.01	6.48	1.03	1.03	Regular ✓
3	5.24	0.00	0.00	11.82	8.24	7.01	6.48	1.00	1.00	Regular ✓
2	5.24	0.00	0.00	12.24	8.96	7.08	6.58	1.01	1.02	Regular ✓
1	5.44	0.00	0.00	0.00	0.00	5.44	5.44	0.77	0.83	< .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=-0.63+0.18+4.19+ 0.3 × 0.01=3.75
My=-0.27+0.08+3.05+ 0.3 × 0.30=2.96

Report 3.8: Irregularity Check of Case 4 (Floor Discontinuity) 5F.

3.3.5 Case Study 5 (Torsional Irregularity):

In the circumstances where the torsional irregularity factor (η_{bi}) is higher than 1.2.

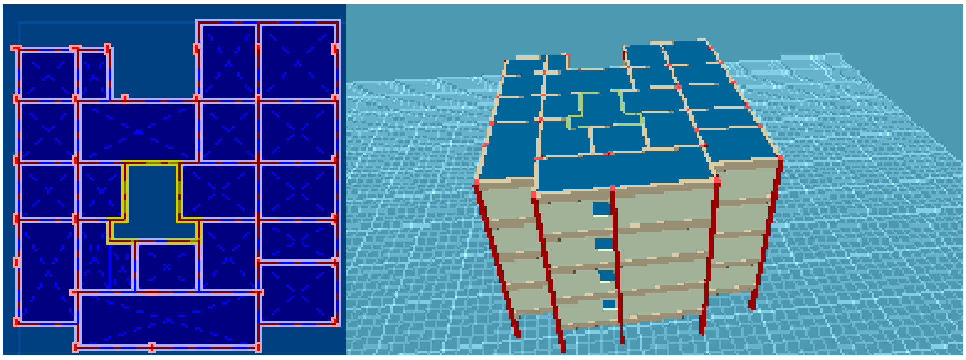


Figure 3.5: Two & Three Dimensional Plan of Case 5 (Torsional Irregularity).

Table 3.10.: Case 5 (Torsional Irregularity) Building Specifications.

Case	Case 5 (Torsional Irregularity)	
Floor number	**3F (9m)**	**5F (15m)**
Columns	60x30 cm^2	30x60 cm^2
Beams	30x60 cm^2	30x60 cm^2
Slab	20 cm	20 cm

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	nbi	nki	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	nbi	nki	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 3.9: Irregularity Check of Case 5 (Torsional Irregularity) 3F.

```
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
```

Report 3.10: Irregularity Check of Case 5 (Torsional Irregularity) 5F.

3.4 Nonlinear Static Pushover Analysis According to TEC-2007

When acquiring the performance level, for evaluating the demands, it is essential to consider the inelastic behavior of the building. Pushover analysis is implemented to classify the seismic hazards, to help with the pick of performance levels and the targets of the design performance. The pushover analysis present a connection between the base shear and the lateral displacement, while specifying the lateral load capacity in addition to the maximum amount of inelasticity that can be handled by the structure. The shift of the gradients of the shear vs displacement curve present an evidence of the yielding of numerous elements of the structure. The central goal of the pushover analysis is to conclude the member forces along with the universal and local deformation capacity of a building. The curves stand for base shear-weight ratio versus story level displacements for load dispensation. Shear V is formulated by adding all executed lateral loads over the ground level, and the weight of the building W is the total amount of the weights of all storeys. Other than that, those curves stand for the loss of lateral loads resisting capacity and shear failure of the column at a displacement level. The differentiations in gradient of these curves give a demonstration of yielding various structural elements: first yielding of beam, then yielding of column and shear failure in the members.

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.

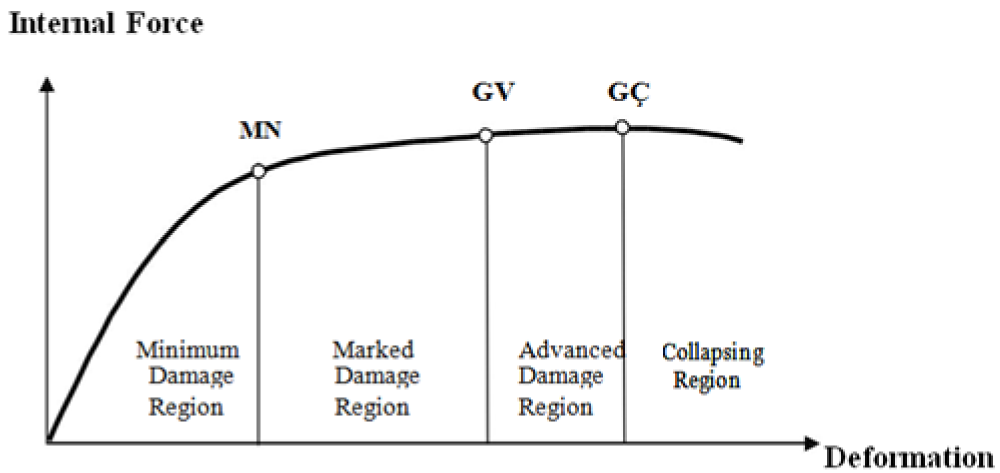


Figure 3.6: Member Damage Levels and Member Performance Regions on Capacity Curve [12].

3.5.1 Immediate Occupancy Category

The structure is categorized in the Immediate Occupancy level if no more than 10% of the beams in it surpass the Advanced Damage Zone with all the other components of structure stays in the Minimum Damage Zone.

3.5.2 Life Safety Category

The structure that can meet the following conditions can be put in the Life Safety category:

- The damage rate of beams in any floor must not be more than 30% in Marked Damage and in Advanced Damage region.

- The columns located in the Advanced Damage region, must not support more than 20% of the shear load subjected on the floor. As for the top storey, the columns located in the Advanced Damage region should not be subjected to more than 40% of the shear load applied on floor.

- The shear load supported by columns in the Advanced Damage region, must not surpass 30% of the story shear in any floor.

3.5.3 Collapse Prevention Category

The structure that meets the following conditions can be put in the Collapse Prevention category:

- After getting the outcome of the computation of all earthquakes subjected on any floor. No more than 20% of the total beams can be in the Collapse region, excluding the secondary ones (that which are not located in the horizontal load-bearing system).

- The shear load supported by columns in the Minimum Damage region must not surpass 30% of the storey shear in any floor.

3.5.4 Collapse Category

If the structure fails to meet the conditions stated in the Collapse Prevention Category, then it is said to be in the Collapse Category. Structures existing in the Collapse Category shall not be allowed to be used.

3.6 Pushover Curve

The pushover curve which is the main output of the pushover analysis method, represent a relation between the base shear (summation of horizontal forces) and the lateral displacement at a certain point at the top storey of a structure. Also including all phases of horizontal force displacement.

The pushover curve can be transformed into a curve representing the relation between the acceleration and the displacement response spectrum, in which it portrays the seismic capacity of the structure. It is feasible to incorporate within the same graph the seismic demand, in order to check if the capacity accommodates the demand.

The accommodation of the seismic capacity and the demand is referred to as performance point.

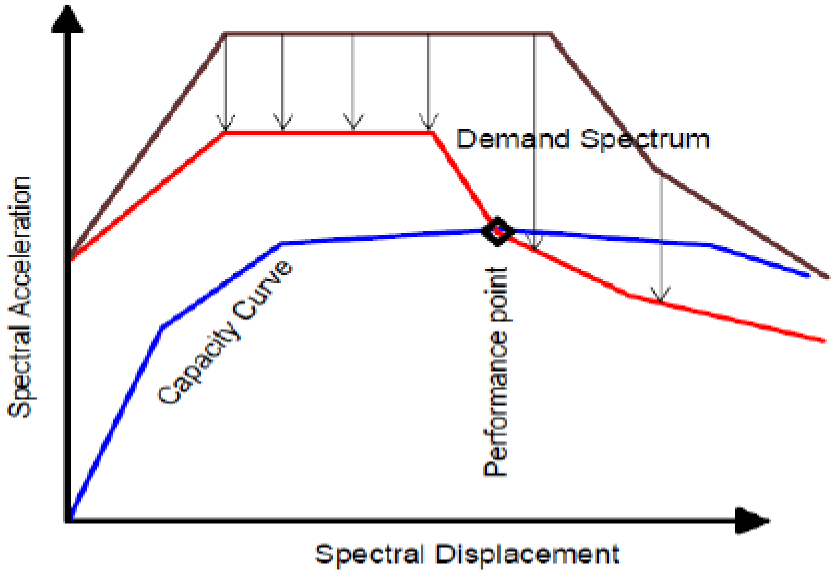


Figure 3.7: Capacity Curve, Demand Spectrum and Performance Point

Chapter 4

PERFORMANCE CHECK AND DISCUSSION

4.1 Introduction

This section present the analysis result done and taken from the STA4CAD structural software. The purpose of this chapter is to examine the behavior and performance of selected reinforced concrete structures, three and five storeys, against earthquake while implementing the pushover analysis method of TEC-2007. Comparison of maximum base shear and displacement, cost, and damage report of three and five story reinforced concrete buildings where investigated and reported in this section.

4.2 Performance Level

In this section the performance level of the cases cases can be found in this section.

4.2.1 Case 1 (Weak Storey):

4.2.1.1 3F Eurocode 8

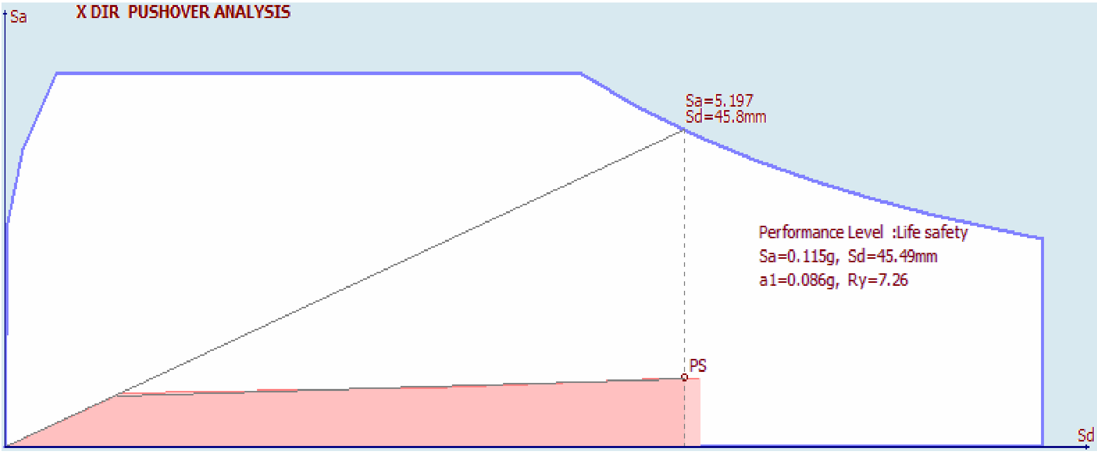


Figure 4.1: Performance Level of Case 1 (Weak Storey) 3F EC8 Case: First Analysis Case.

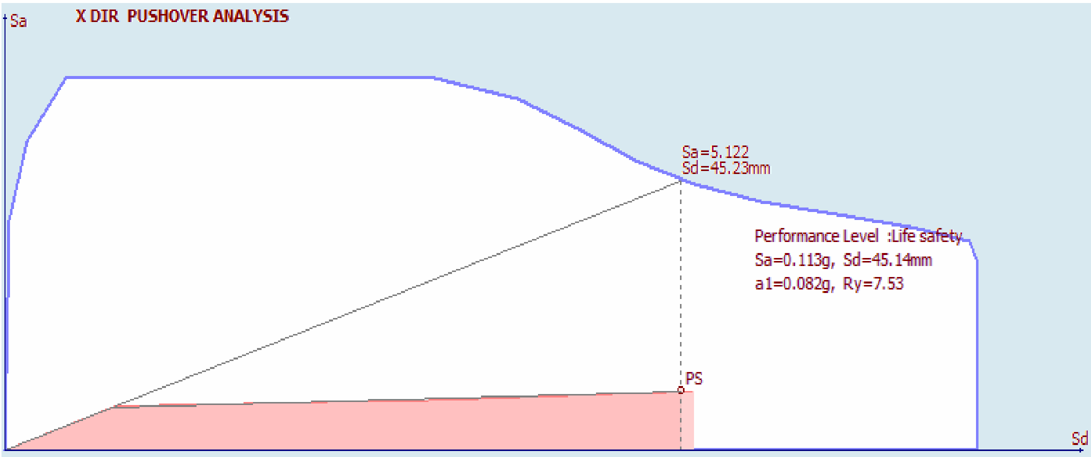


Figure 4.2: Performance Level of Case 1(Weak Storey) 3F EC8 Case: Second Analysis Case.

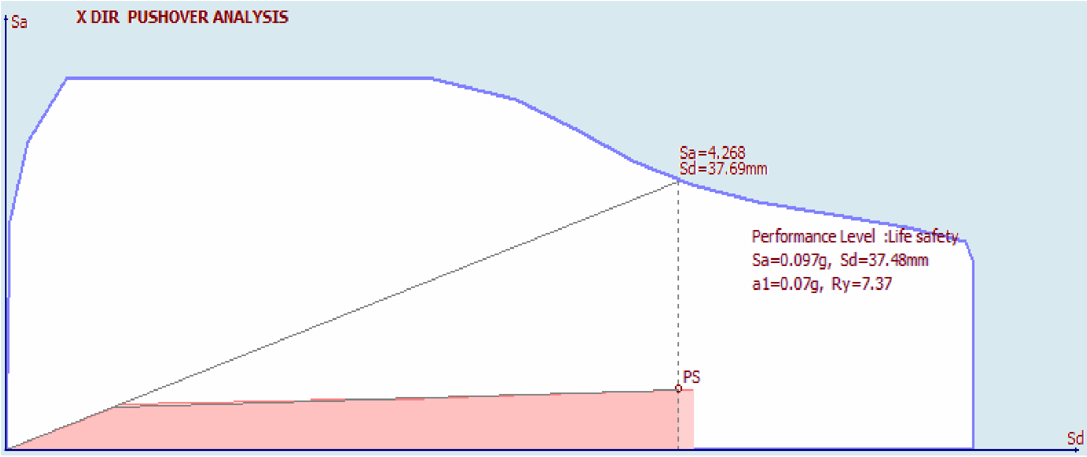


Figure 4.3: Performance Level of Case 1 (Weak Storey) 3F EC8 Case: Third Analysis Case.

4.2.1.2 3F TEC-2007:

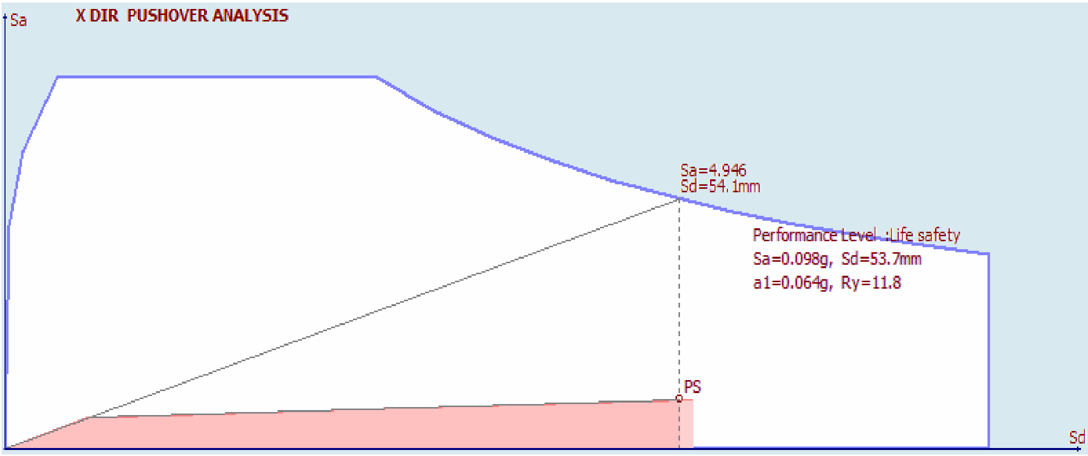


Figure 4.4: Performance Level of Case 1 (Weak Storey) 3F TEC-2007 Case: First Analysis Case.

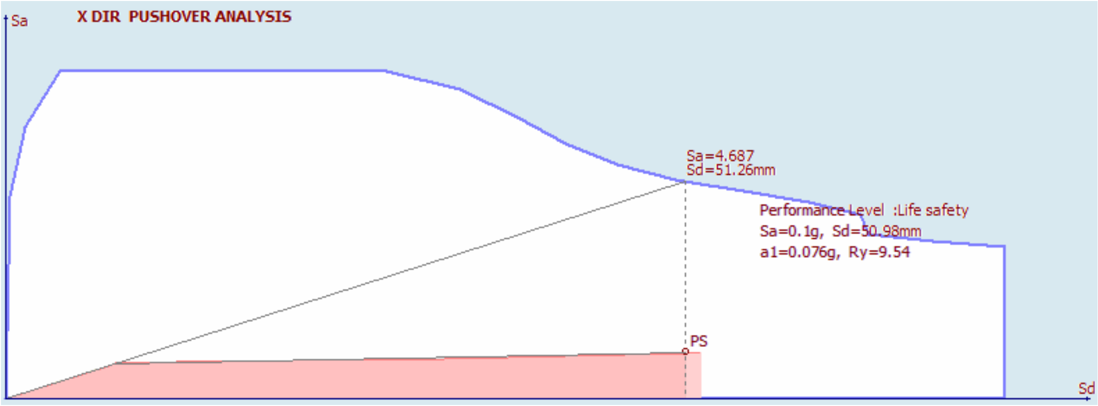


Figure 4.5: Performance Level of Case 1 (Weak Storey) 3F TEC-2007 Case: Second Analysis Case.

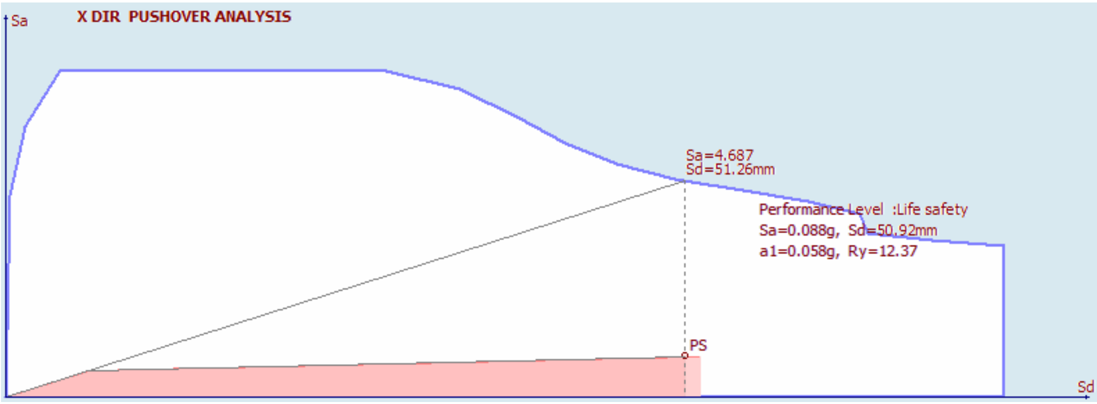


Figure 4.6: Performance Level of Case 1 (Weak Storey) 3F TEC-2007 Case: Third Analysis Case.

4.2.1.3 5F Eurocode 8:

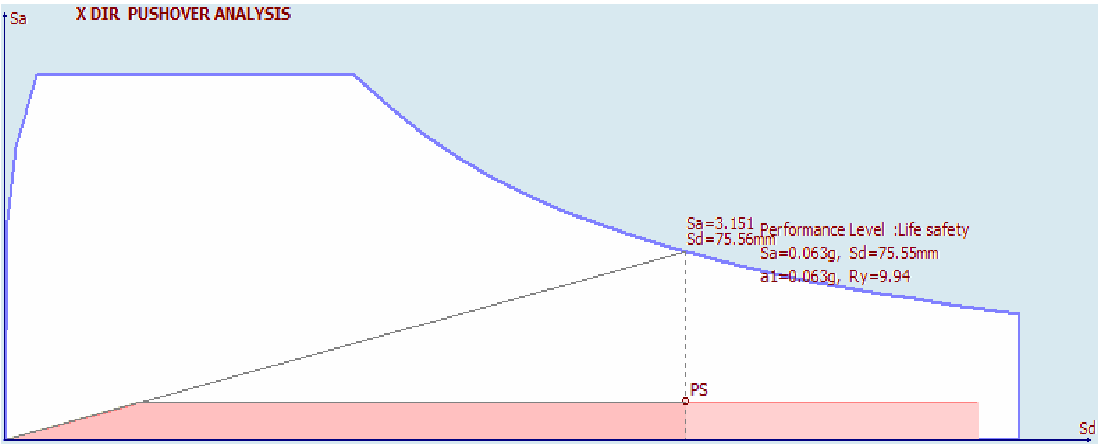


Figure 4.7: Performance Level of Case 1 (Weak Storey) 5F EC8 Case: First Analysis Case.

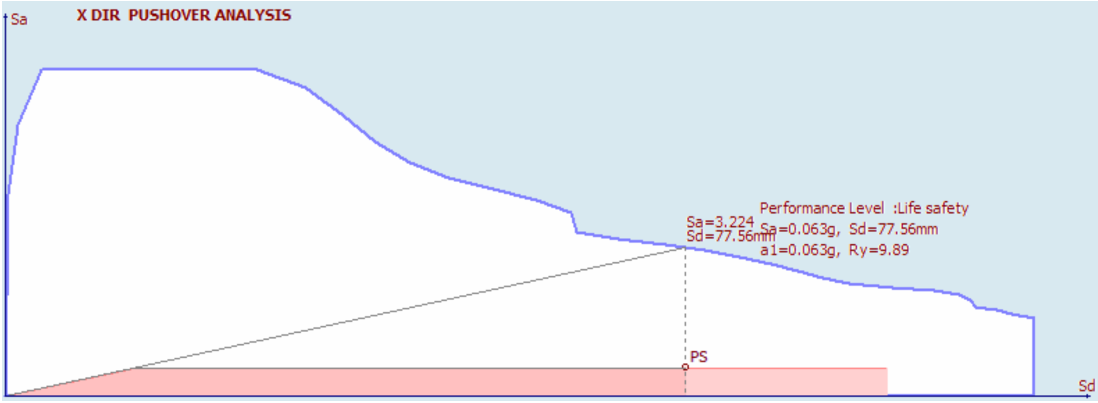


Figure 4.8: Performance Level of Case 1 (Weak Storey) 5F EC8 Case: Second Analysis Case.

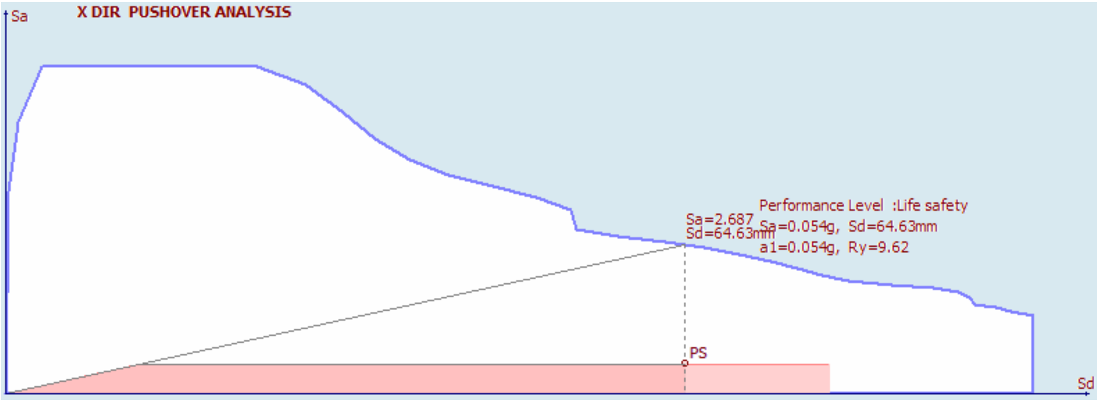


Figure 4.9: Performance Level of Case 1 (Weak Storey) 5F EC8 Case: Third Analysis Case.

4.2.1.4 5F TEC-2007:

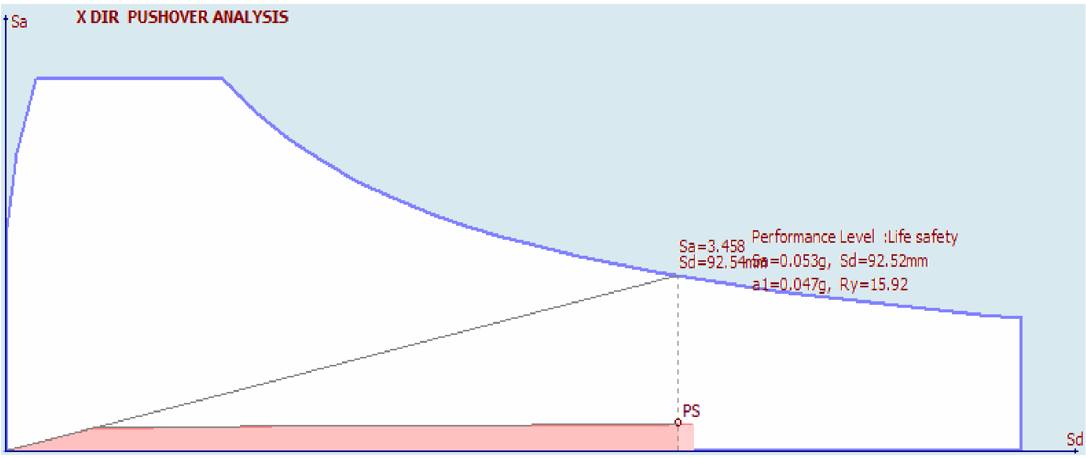


Figure 4.10: Performance Level of Case 1 (Weak Storey) 5F TEC-2007 Case: First Analysis Case

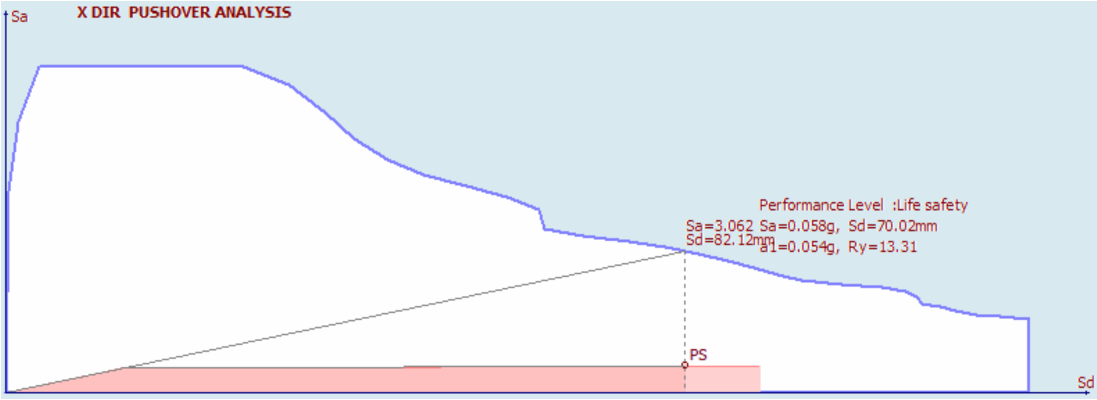


Figure 4.11: Performance Level of Case 1 (Weak Storey) 5F TEC-2007 Case: Second Analysis Case.

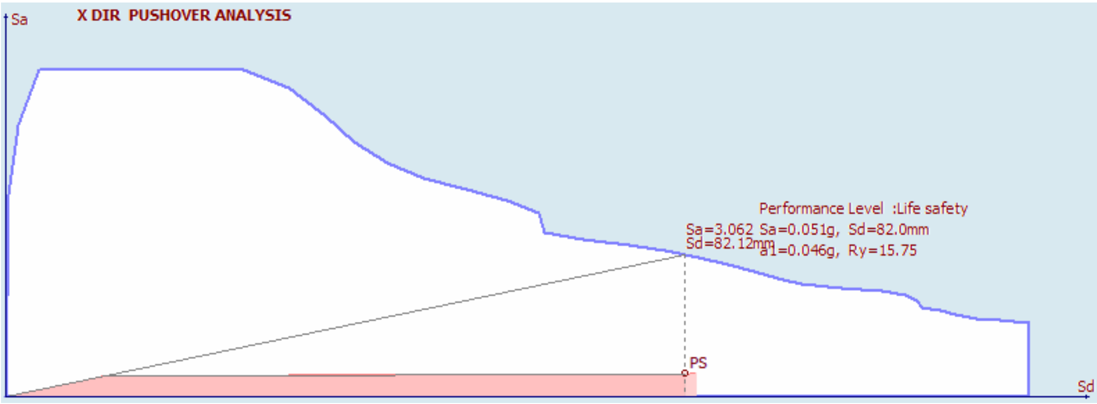


Figure 4.12: Performance Level of Case 1 (Weak Storey) 5F TEC-2007 Case: Third Analysis Case.

4.2.2 Case 2 (Soft Storey) Case

4.2.2.1 3F Eurocode 8:

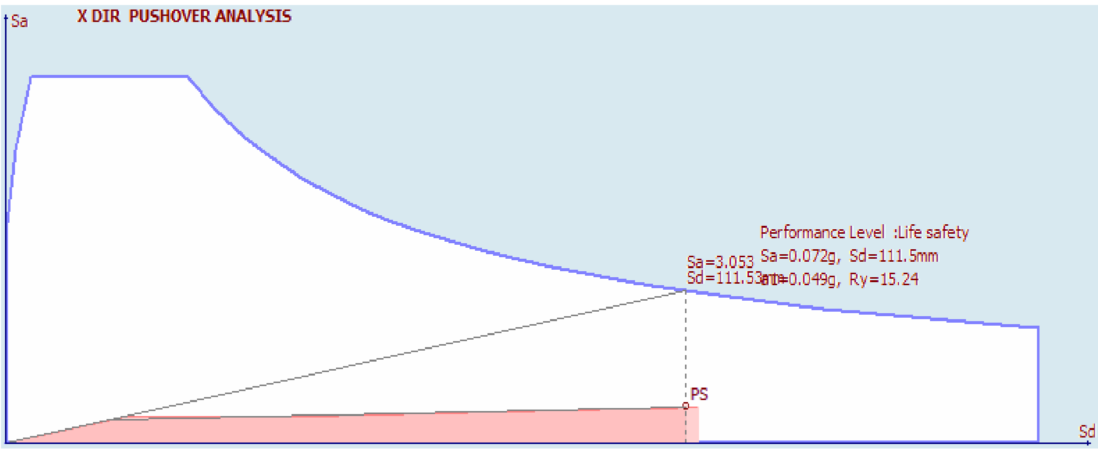


Figure 4.13: Performance Level of Case 2 (Soft Storey) 3F EC8: First Analysis Case.

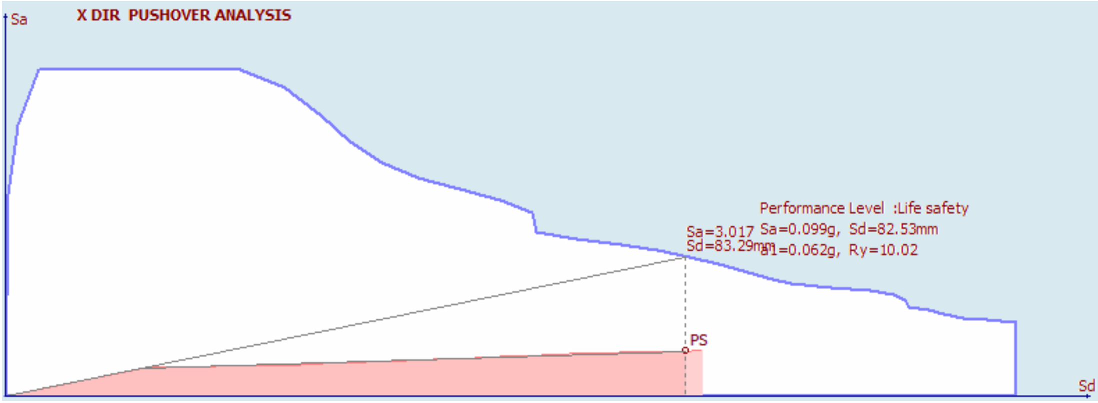


Figure 4.14: Performance Level of Case 2 (Soft Storey) 3F EC8 Case: Second Analysis Case.

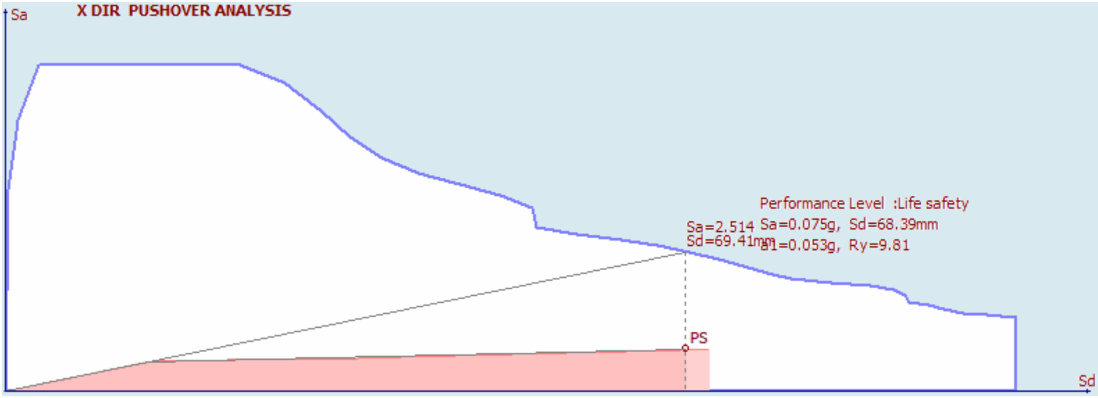


Figure 4.15: Performance Level of Case 2 (Soft Storey) 3F EC8 Case: Third Analysis Case.

4.2.2.2 3F TEC-2007:

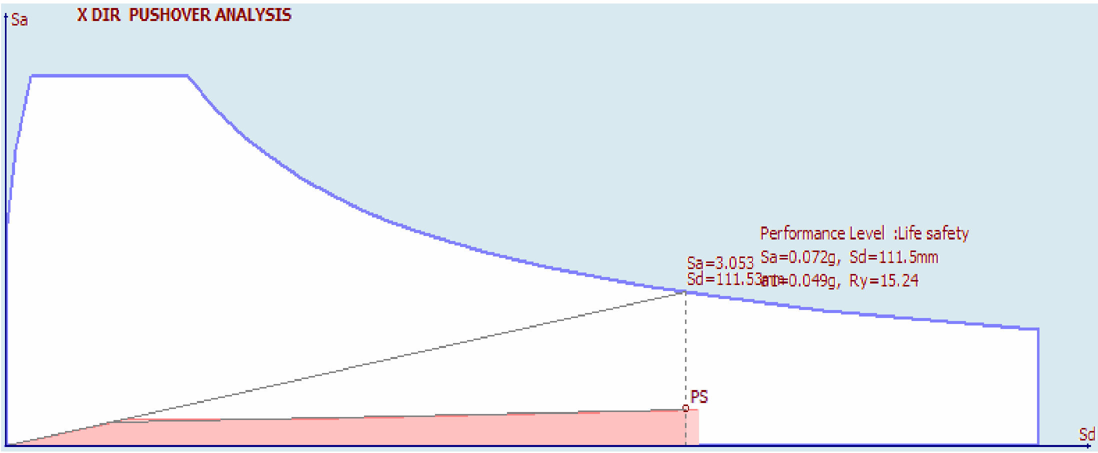


Figure 4.16: Performance Level of Case 2 (Soft Storey) 3F TEC-2007 Case: First Analysis Case.

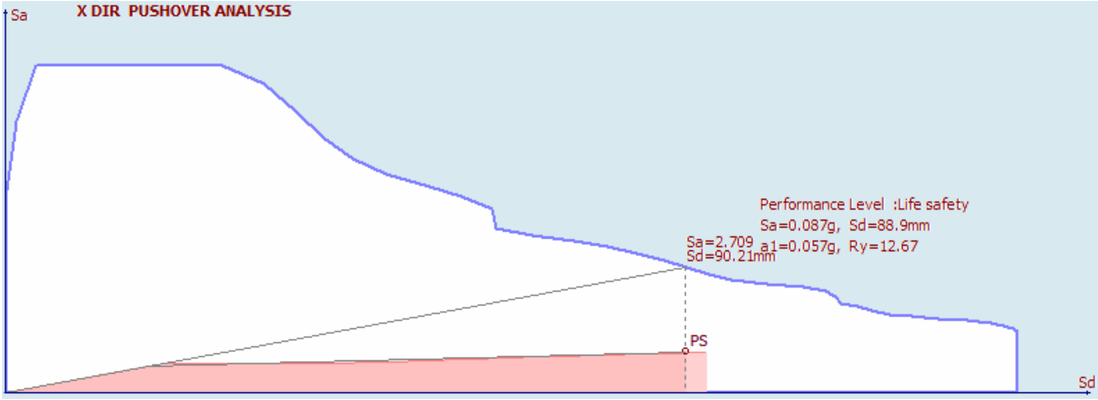


Figure 4.17: Performance Level of Case 2 (Soft Storey) 3F TEC-2007 Case: Second Analysis Case.

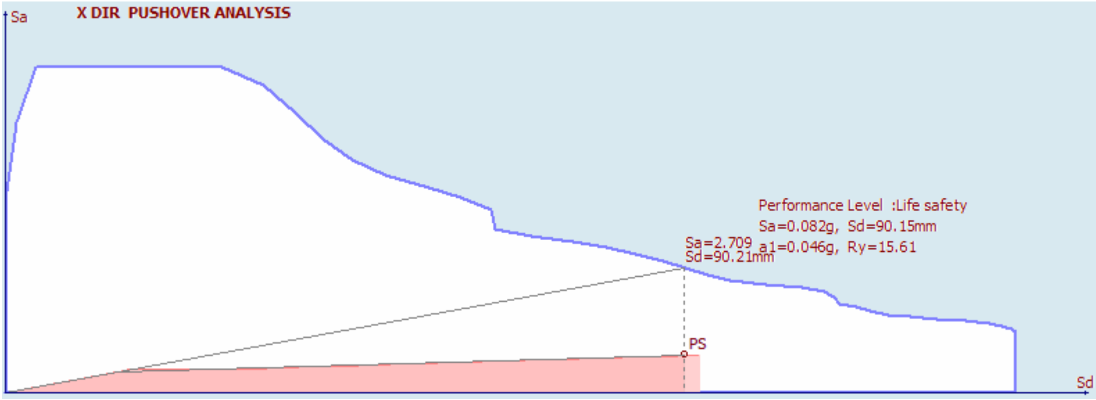


Figure 4.18: Performance Level of Case 2 (Soft Storey) 3F TEC-2007 Case: Third Analysis Case.

4.2.1.3 5F Eurocode 8

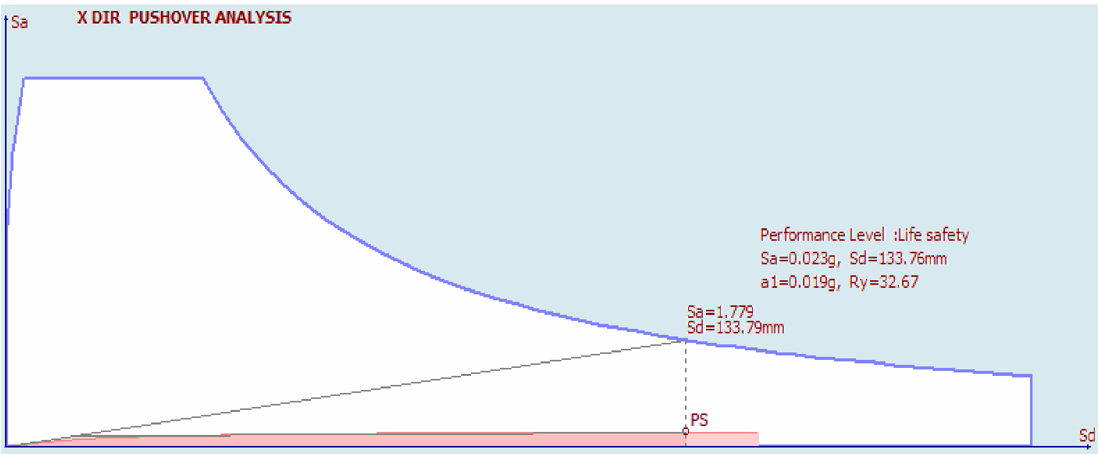


Figure 4.19: Performance Level of Case 2 (Soft Storey) 5F EC8: First Analysis Case.

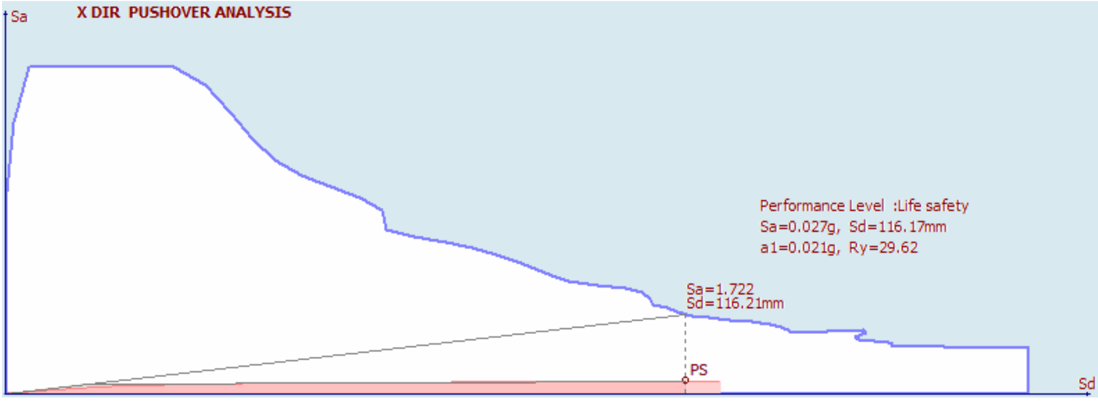


Figure 4.20: Performance Level of Case 2 (Soft Storey) 5F EC8 Case: Second Analysis Case.

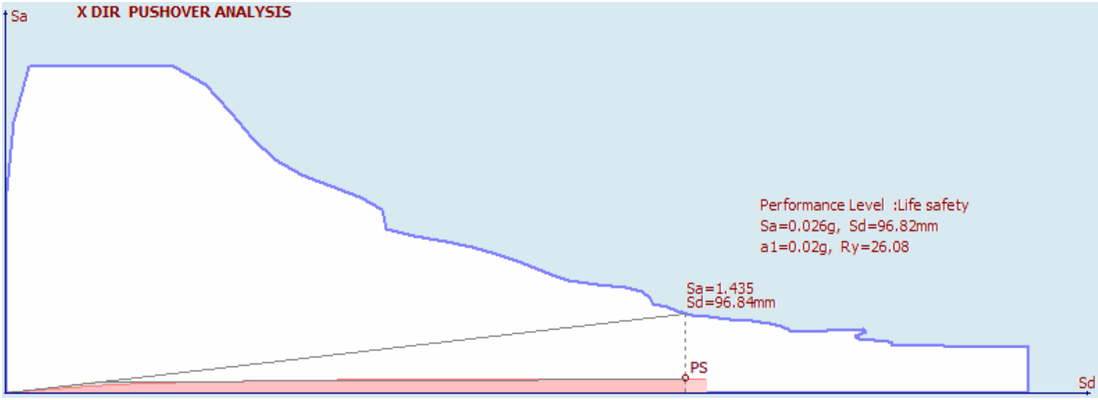


Figure 4.21: Performance Level of Case 2 (Soft Storey) 5F EC8 Case: Third Analysis Case.

4.2.2.4 5F TEC-2007:

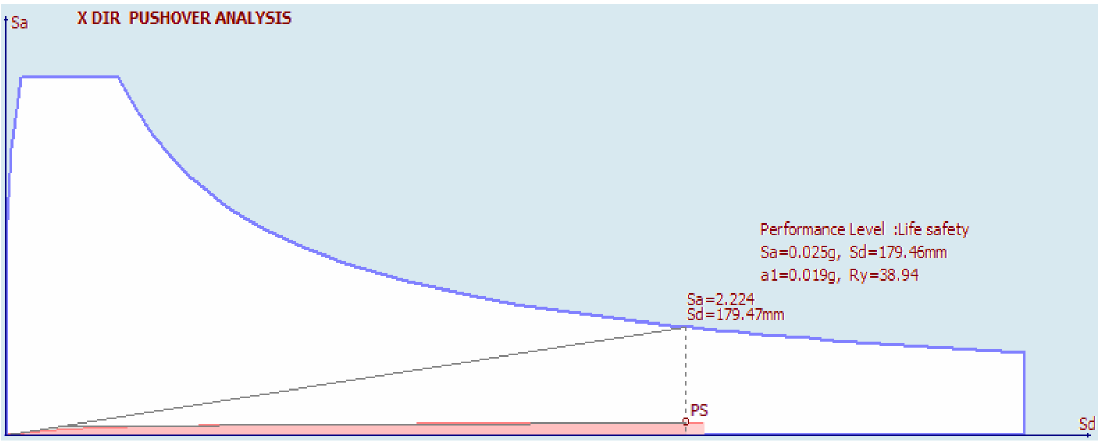


Figure 4.22: Performance Level of Case 2 (Soft Storey) 5F TEC-2007 Case: First Analysis Case.

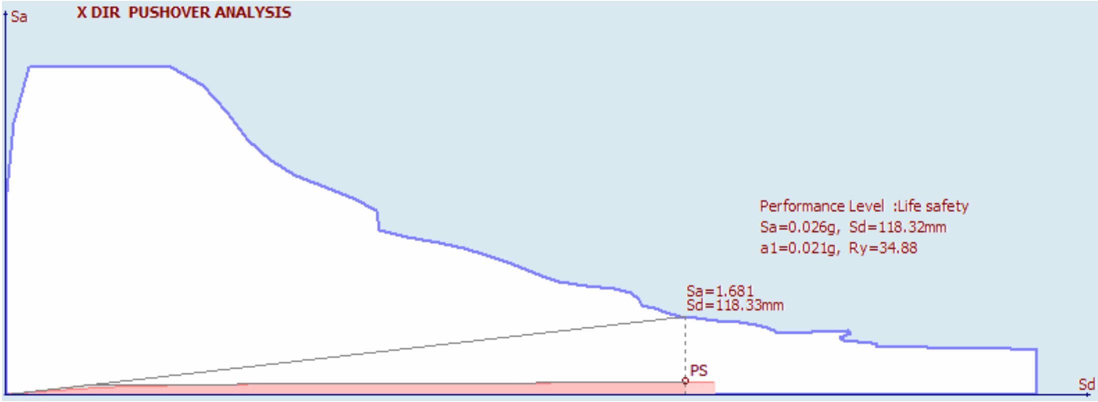


Figure 4.23: Performance Level of Case 2 (Soft Storey) 5F TEC-2007 Case: Second Analysis Case.

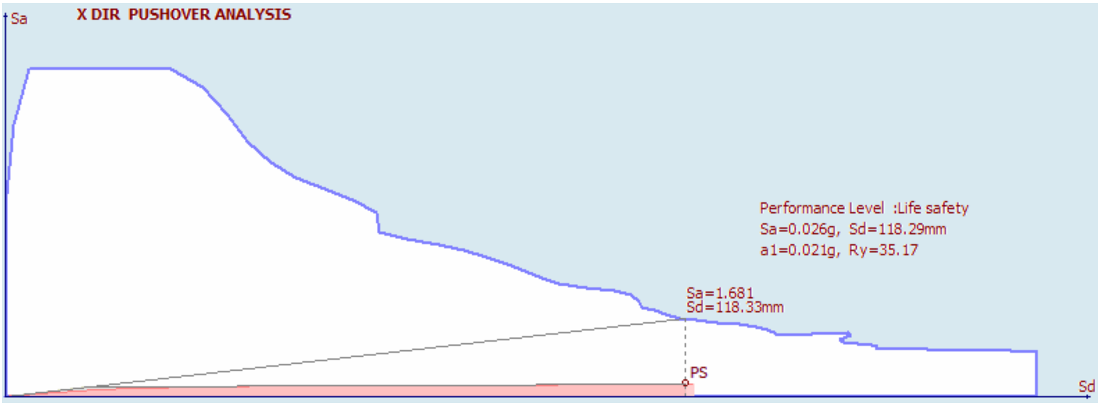


Figure 4.24: Performance Level of Case 2 (Soft Storey) 5F TEC-2007 Case: Third Analysis Case.

4.2.3 Case 3 (Projection in Plan)

4.2.3.1 3F Eurocode 8:

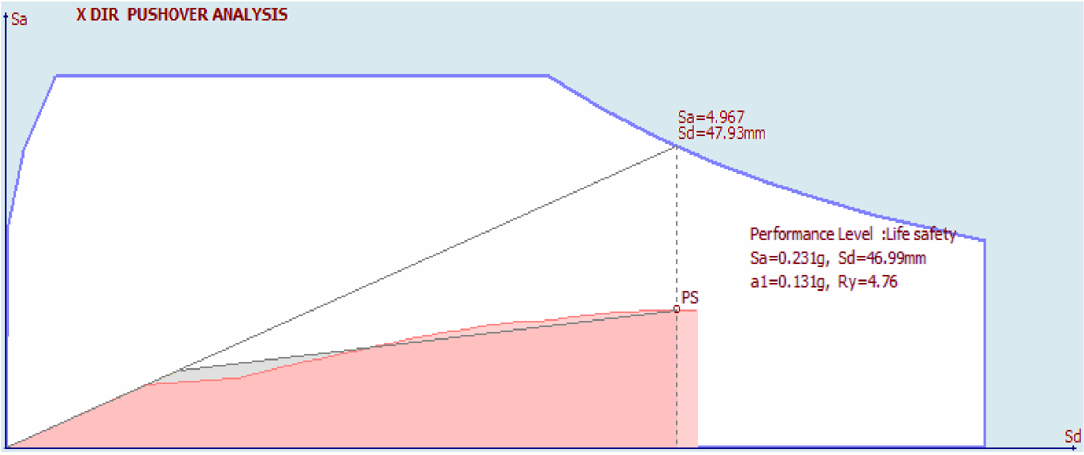


Figure 4.25: Performance Level of Case 3 (Projection in Plan) 3F EC8 Case: First Analysis Case.

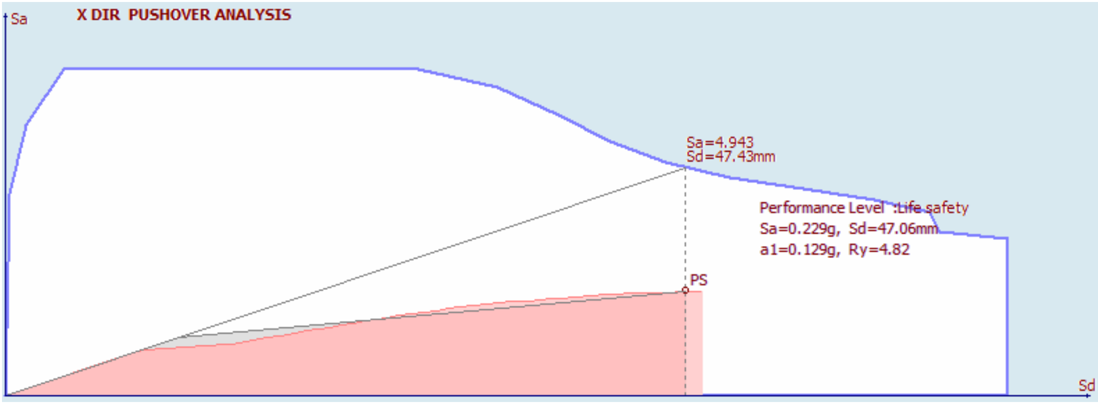


Figure 4.26: Performance Level of Case 3 (Projection in Plan) 3F EC8 Case: Second Analysis Case.

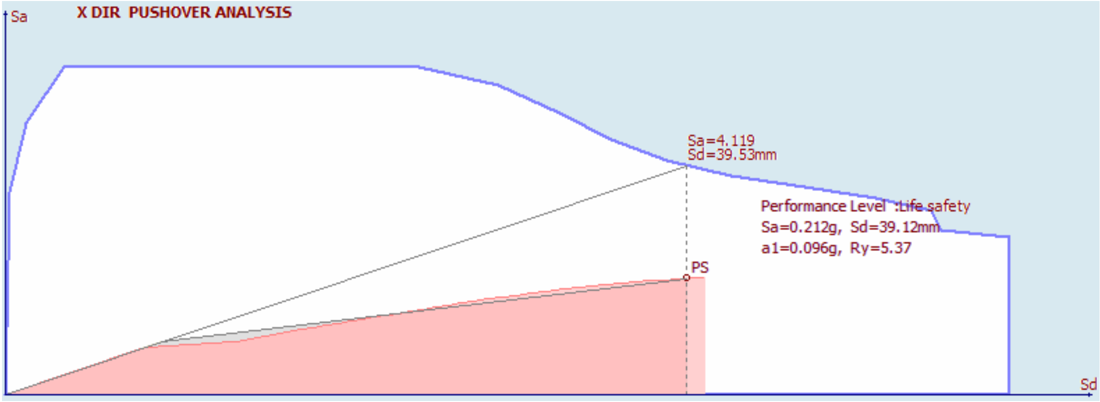


Figure 4.27: Performance Level of Case 3 (Projection in Plan) 3F EC8 Case: Third Analysis Case.

4.2.3.2 3F TEC-2007:

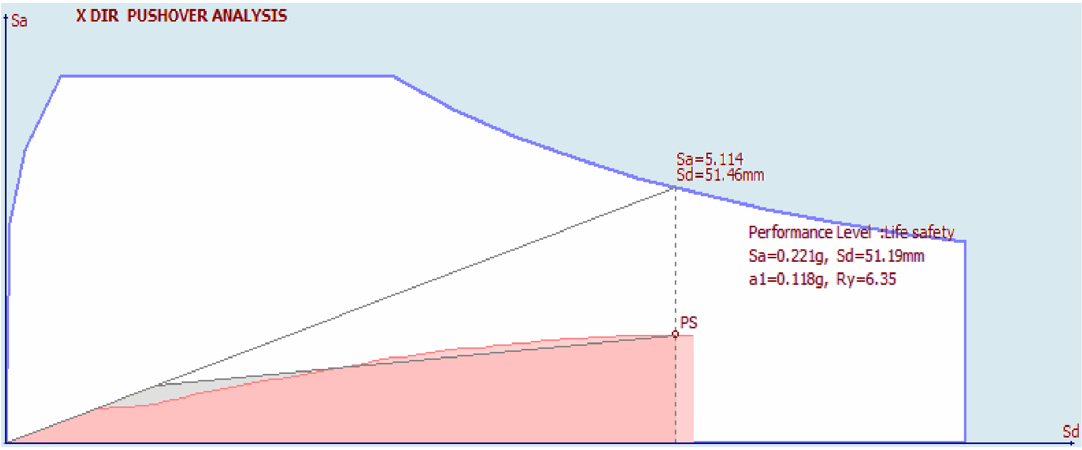


Figure 4.28: Performance Level of Case 3 (Projection in Plan) 3F TEC-2007 Case: First Analysis Case.

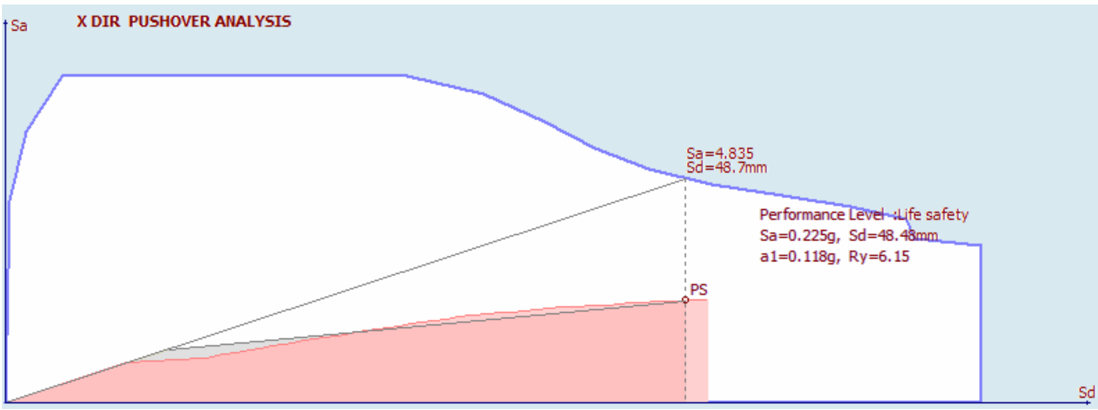


Figure 4.29: Performance Level of Case 3 (Projection in Plan) 3F TEC-2007 Case: Second Analysis Case

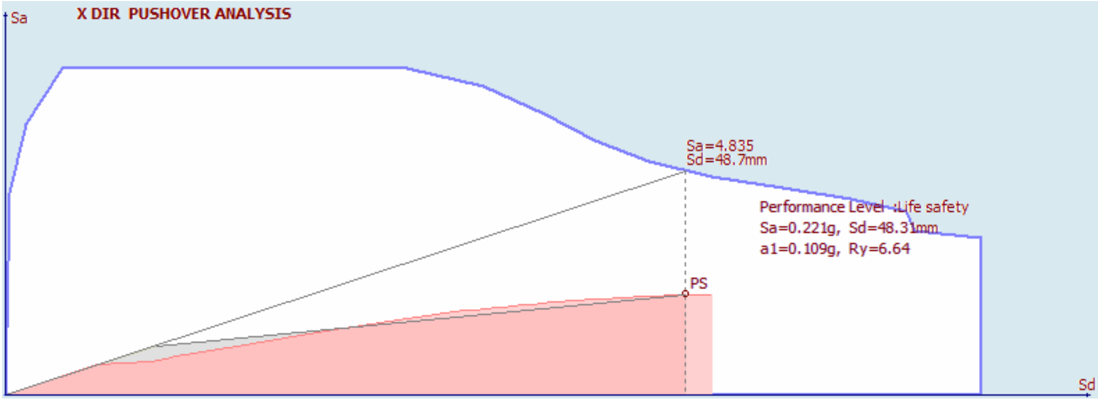


Figure 4.30: Performance Level of Case 3 (Projection in Plan) 3F TEC-2007 Case: Third Analysis Case.

4.2.3.3 5F Eurocode 8:

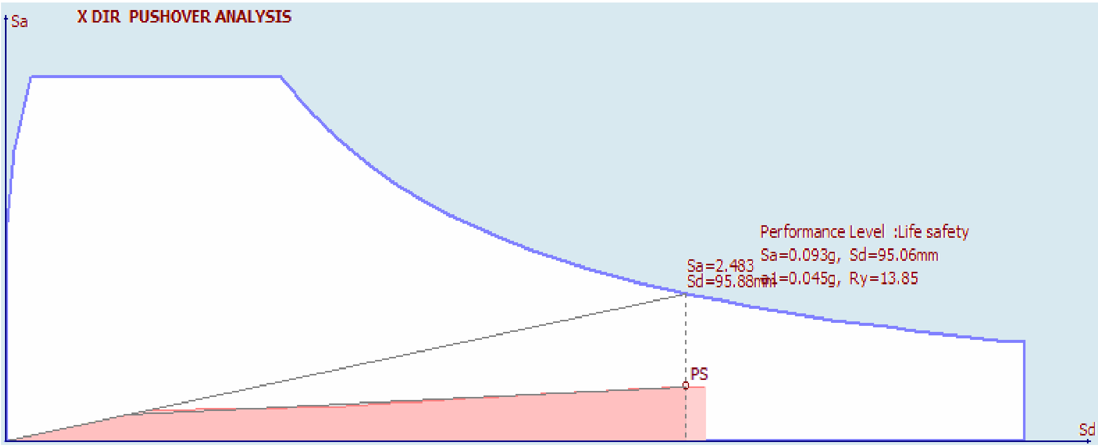


Figure 4.31: Performance Level of Case 3 (Projection in Plan) 5F EC8 Case: First Analysis Case.

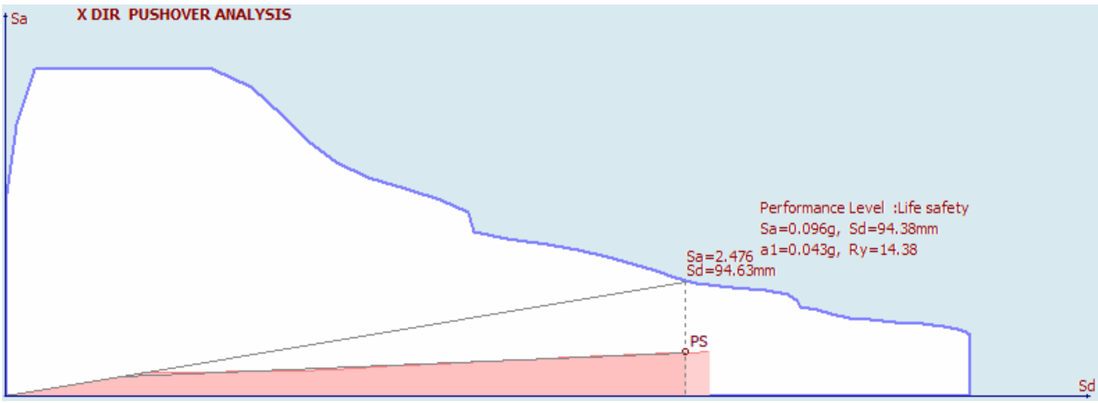


Figure 4.32: Performance Level of Case 3 (Projection in Plan) 5F EC8 Case: Second Analysis Case.

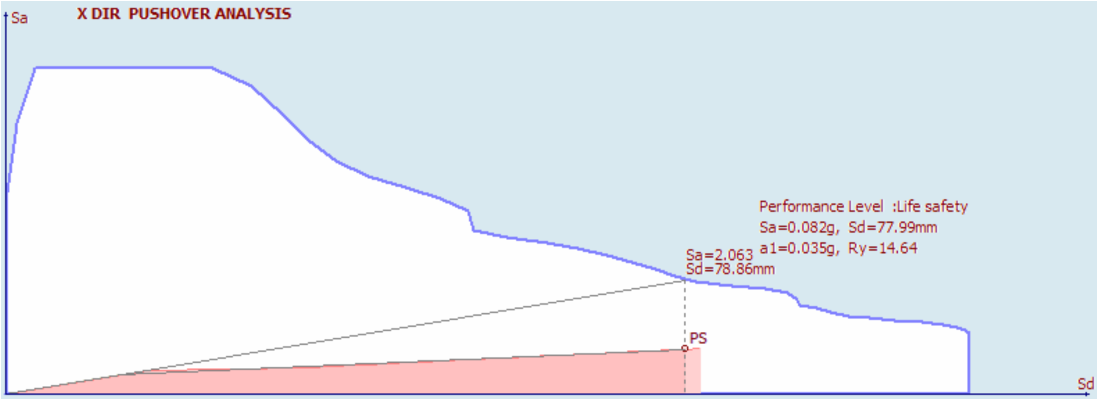


Figure 4.33: Performance Level of Case 3 (Projection in Plan) 5F EC8 Case: Third Analysis Case.

4.2.3.4 5F TEC-2007:

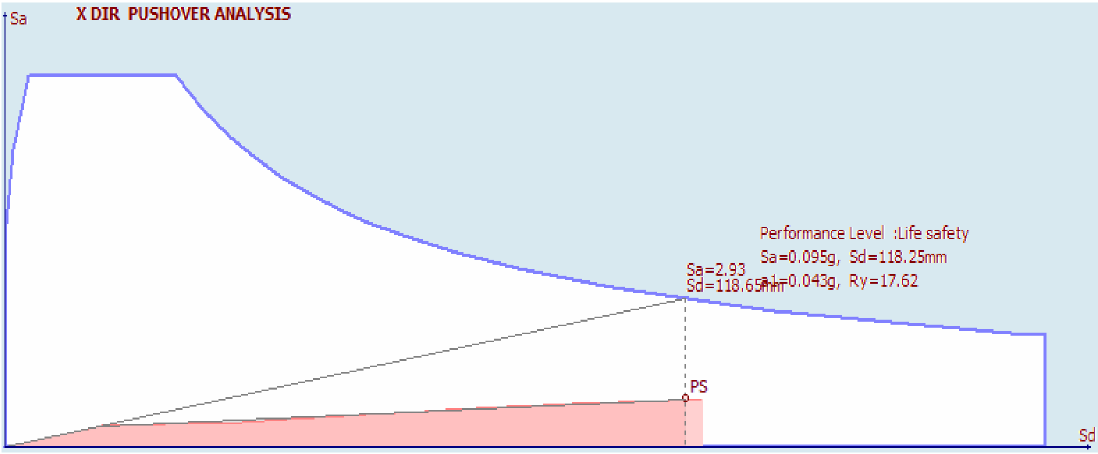


Figure 4.34: Performance Level of Case 3 (Projection in Plan) 5F TEC-2007 Case: First Analysis Case.

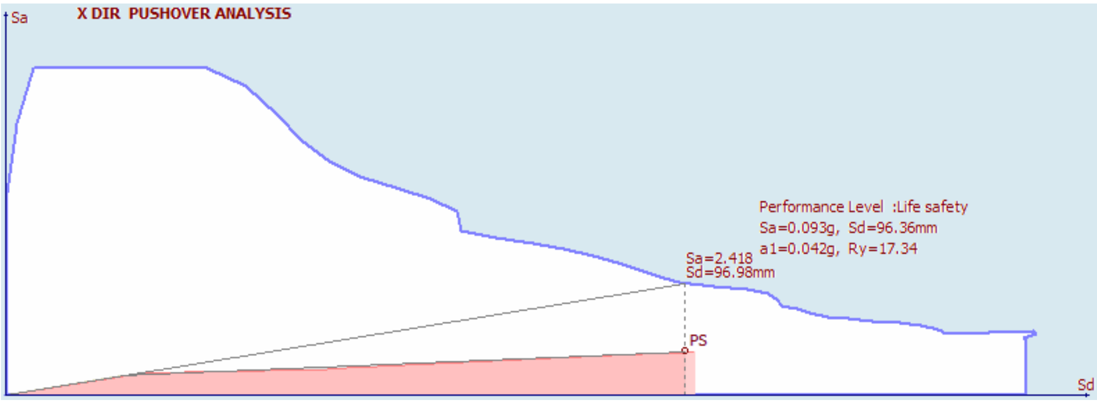


Figure 4.35: Performance Level of Case 3 (Projection in Plan) 5F TEC-2007 Case: Second Analysis Case.

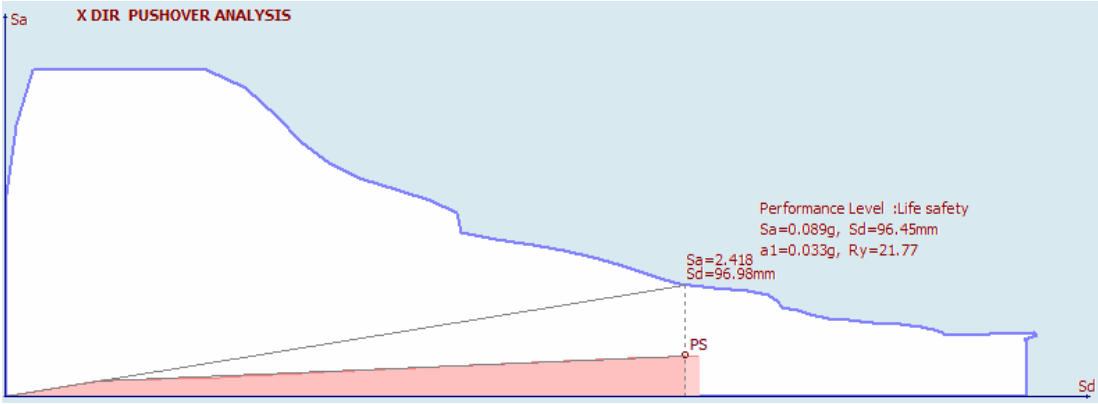


Figure 4.36: Performance Level of Case 3 (Projection in Plan) 5F TEC-2007 Case: Third Analysis Case.

4.2.4 Case 4 (Floor Discontinuity)

4.2.4.1 3F Eurocode 8:

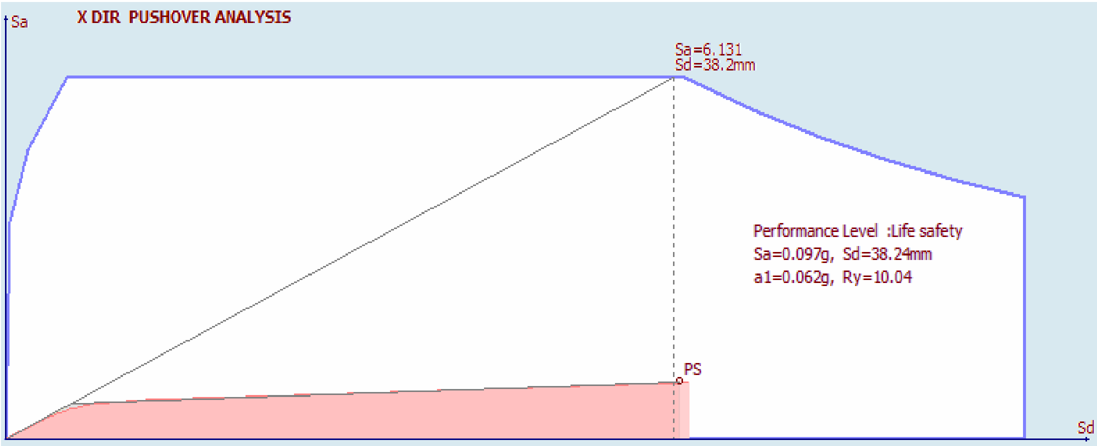


Figure 4.37: Performance Level of Case 4 (Floor D.) 3F EC8 Case: First Analysis Case.

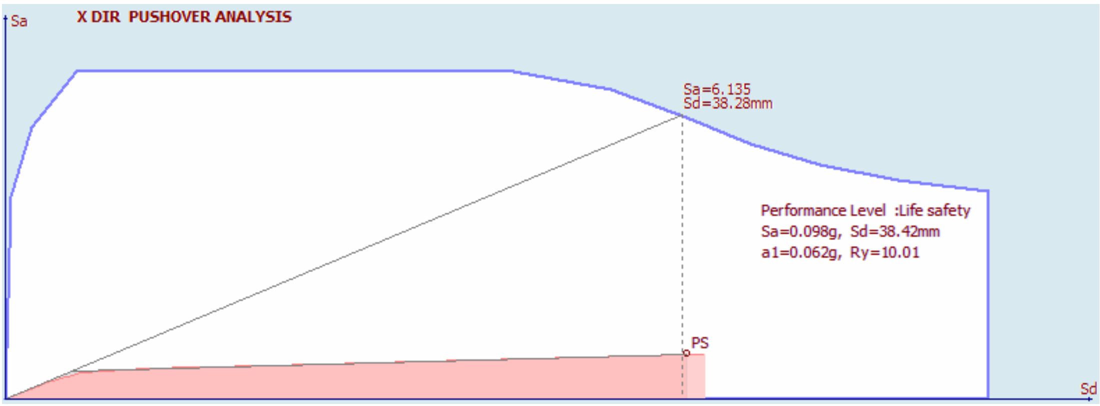


Figure 4.38: Performance Level of Case 4 (Floor D.) 3F EC8: Second Analysis Case.

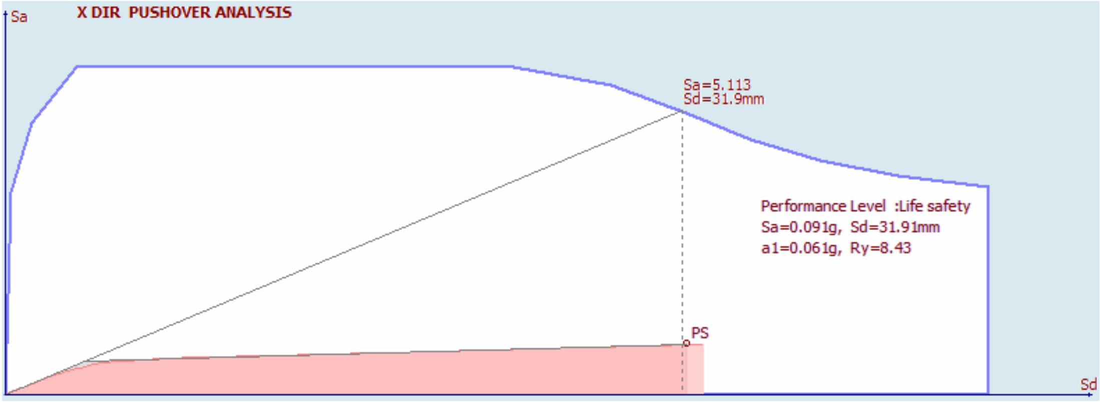


Figure 4.39: Performance Level of Case 4 (Floor D.) 3F EC8 Case: Third Analysis Case.

4.2.4.2 3F TEC-2007:

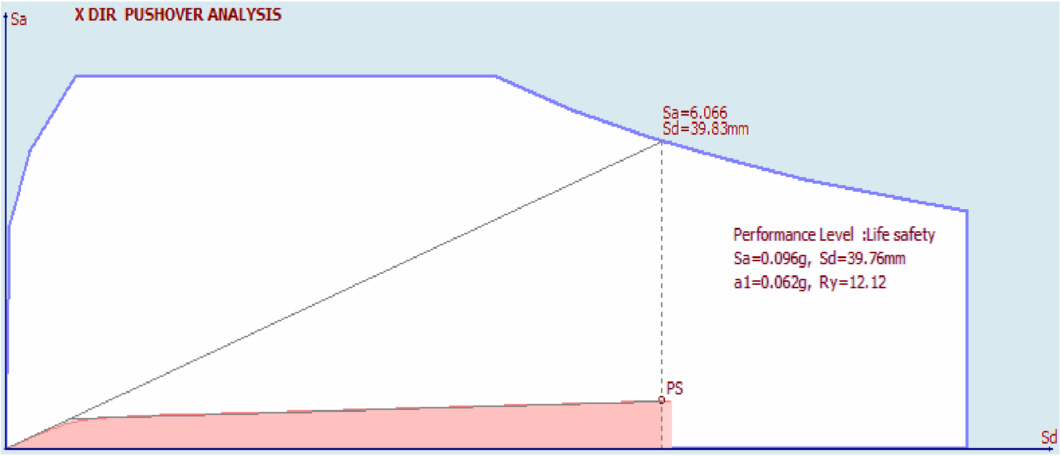


Figure 4.40: Performance Level of Case 4 (Floor D.) 3F TEC-2007 Case: First Analysis Case.

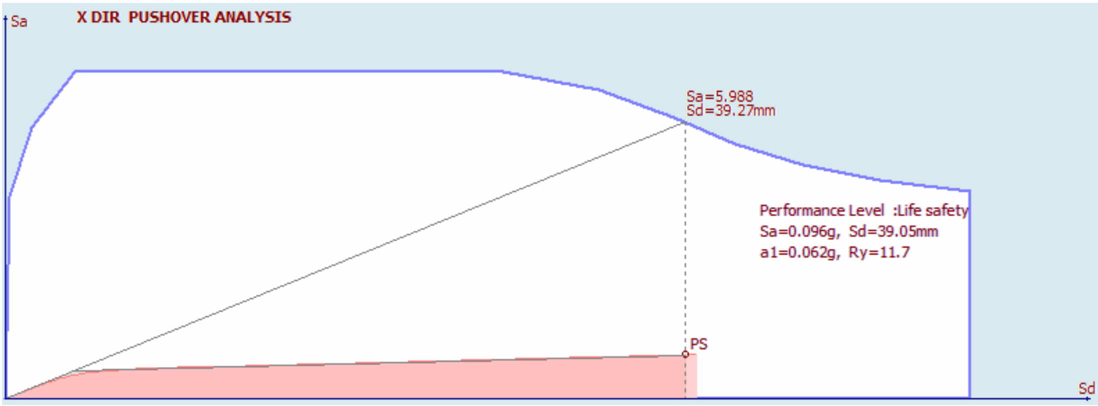


Figure 4.41: Performance Level of Case 4 (Floor D.) 3F TEC-2007 Case: Second Analysis Case

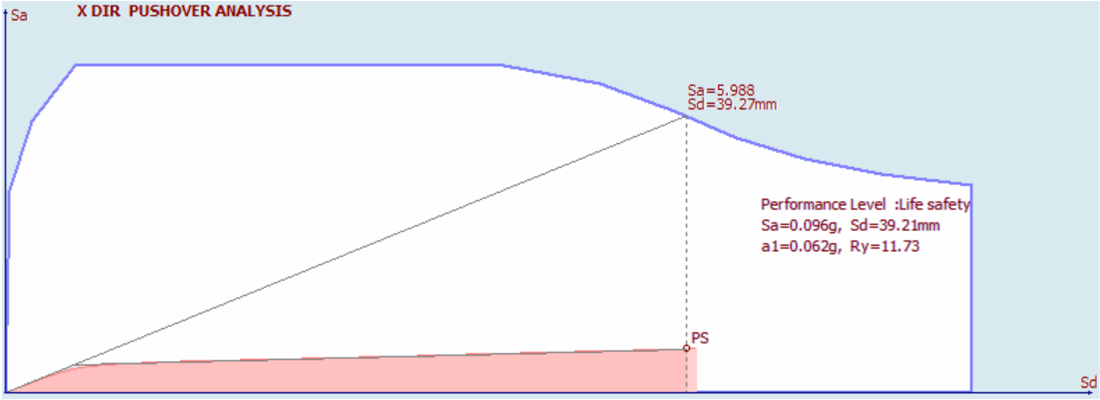


Figure 4.42: Performance Level of Case 4 (Floor D.) 3F TEC-2007 Case: Third Analysis Case.

4.2.4.3 5F Eurocode 8:

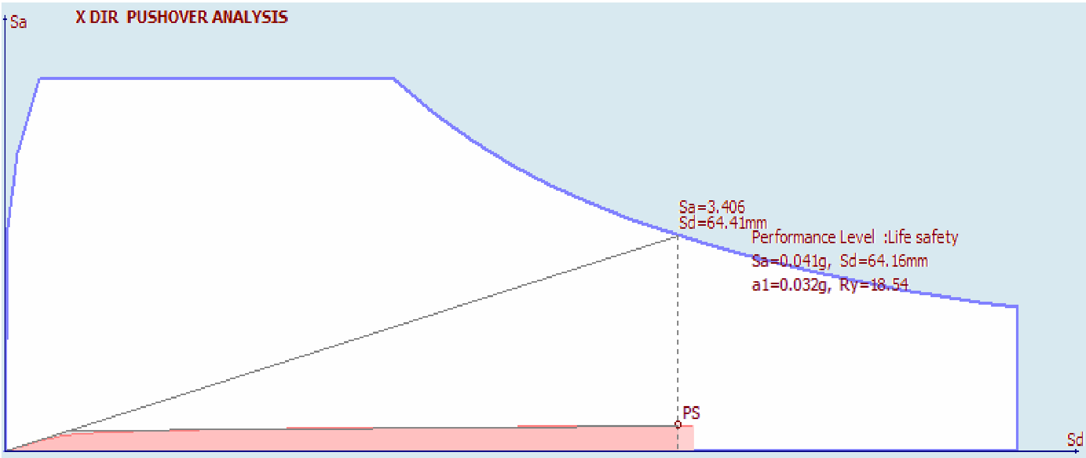


Figure 4.43: Performance Level of Case 4 (Floor D.) 5F EC8 Case: First Analysis Case.

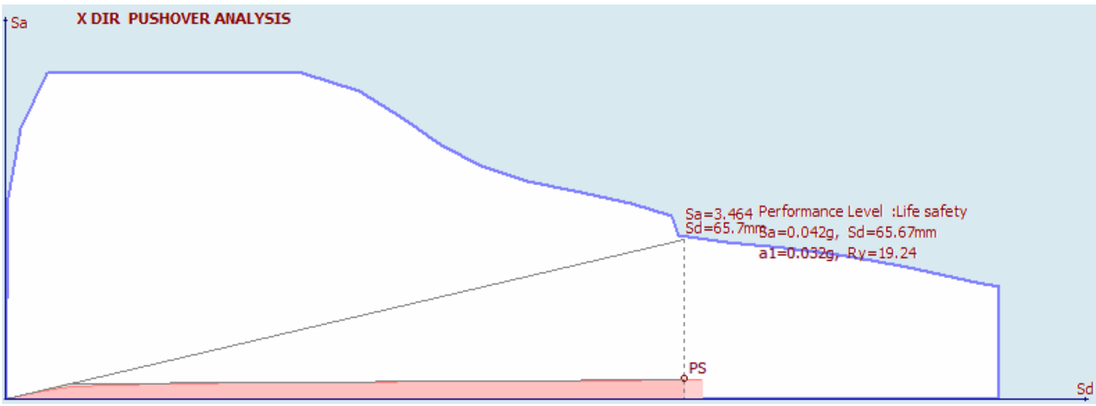


Figure 4.44: Performance Level of Case 4 (Floor D.) 5F EC8 Case: Second Analysis Case.

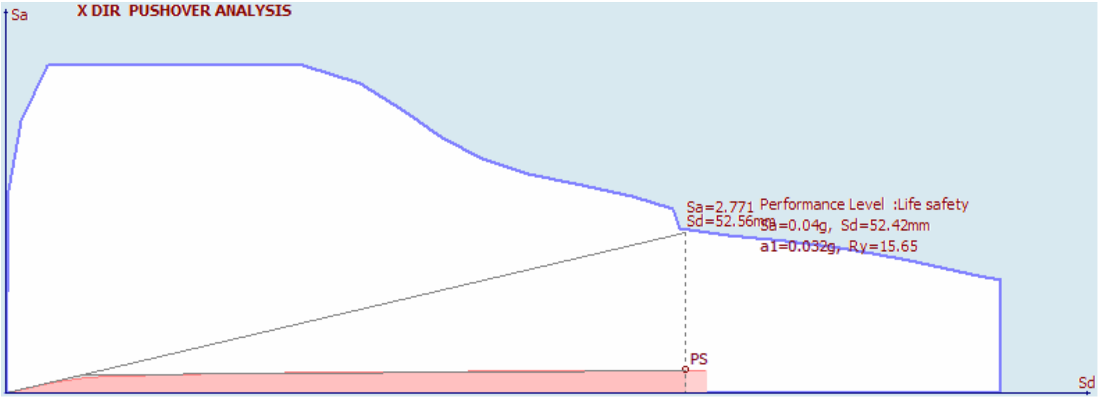


Figure 4.45: Performance Level of Case 4 (Floor D.) 5F EC8: Third Analysis Case

4.2.4.4 5F TEC-2007:

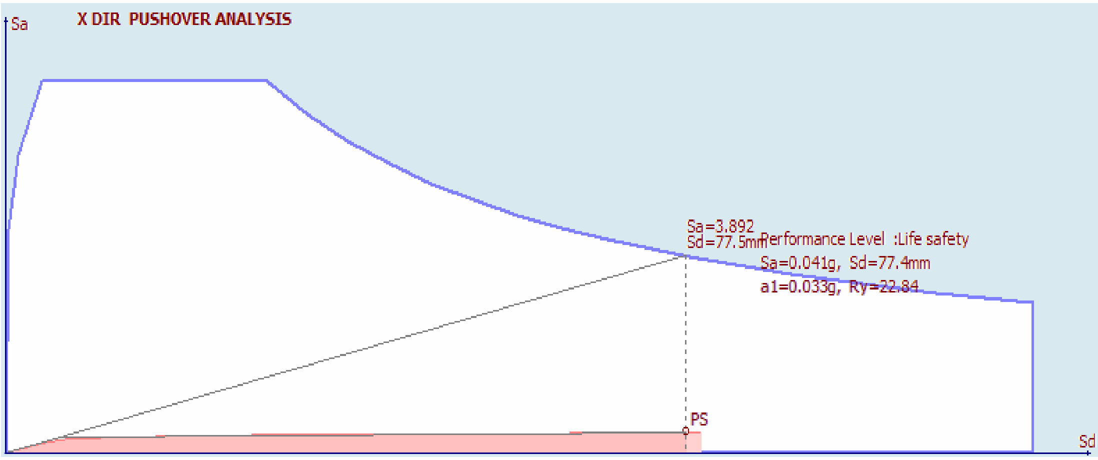


Figure 4.46: Performance Level of Case 4 (Floor D.) 5F TEC-2007 Case: First Analysis Case.

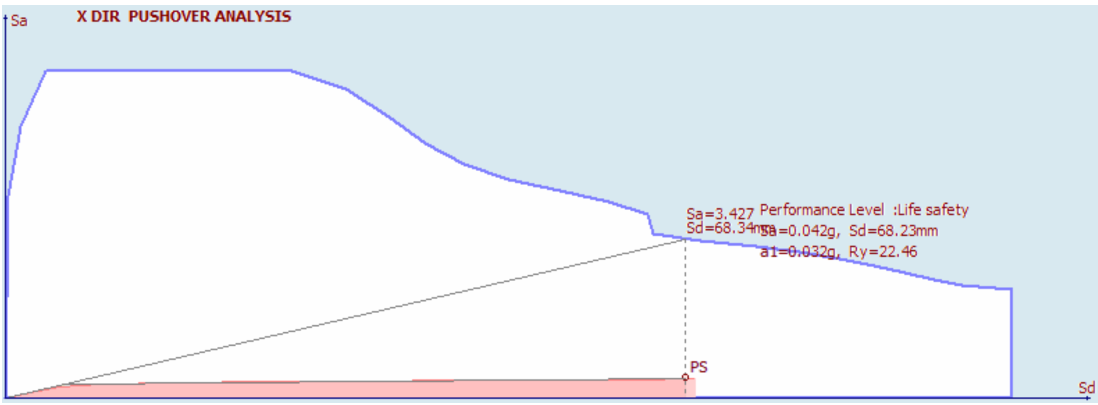


Figure 4.47: Performance Level of Case 4 (Floor D.) 5F TEC-2007 Case: Second Analysis Case.

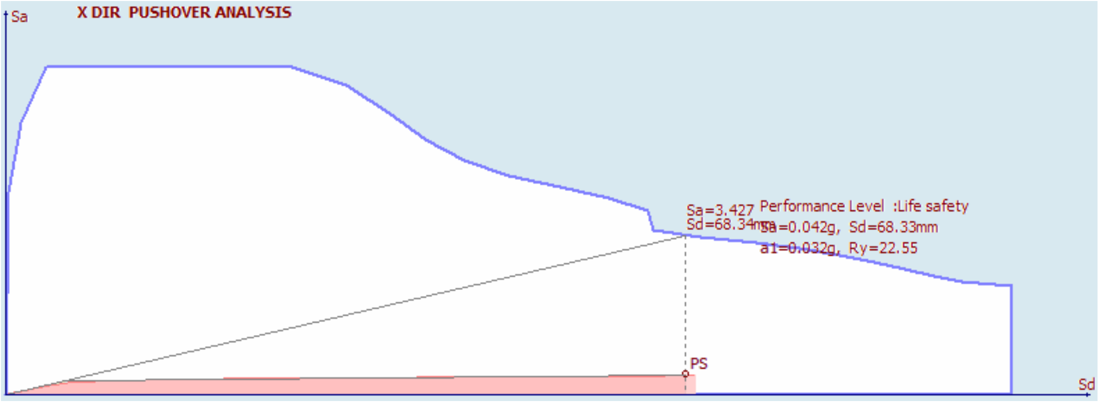


Figure 4.48: Performance Level of Case 4 (Floor D.) 5F TEC-2007 Case: Third Analysis Case.

4.2.5 Case 5 (Torsional Irregularity)

4.2.5.1 3F Eurocode 8:

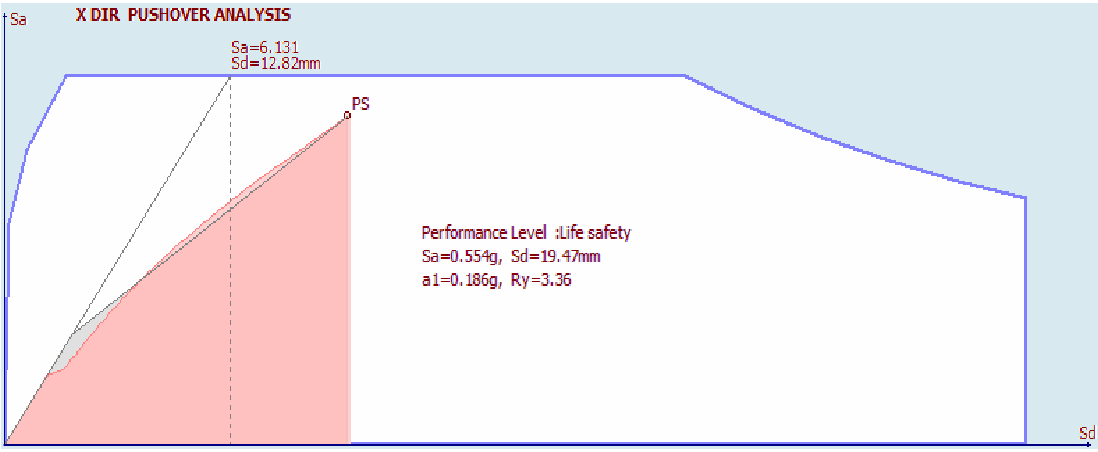


Figure 4.49: Performance Level of Case 5 (Torsional Irregularity) 3F EC8 Case: First Analysis Case.

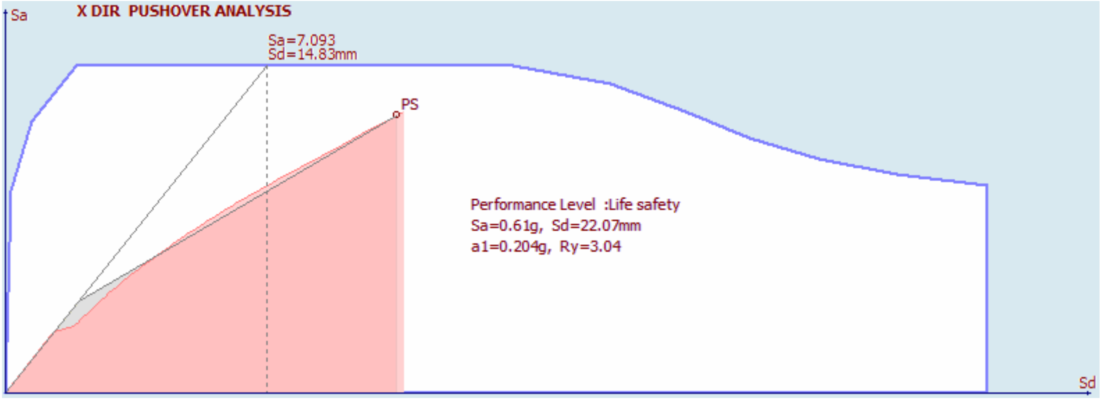


Figure 4.50: Performance Level of Case 5 (Torsional Irregularity) 3F EC8 Case: Second Analysis Case.

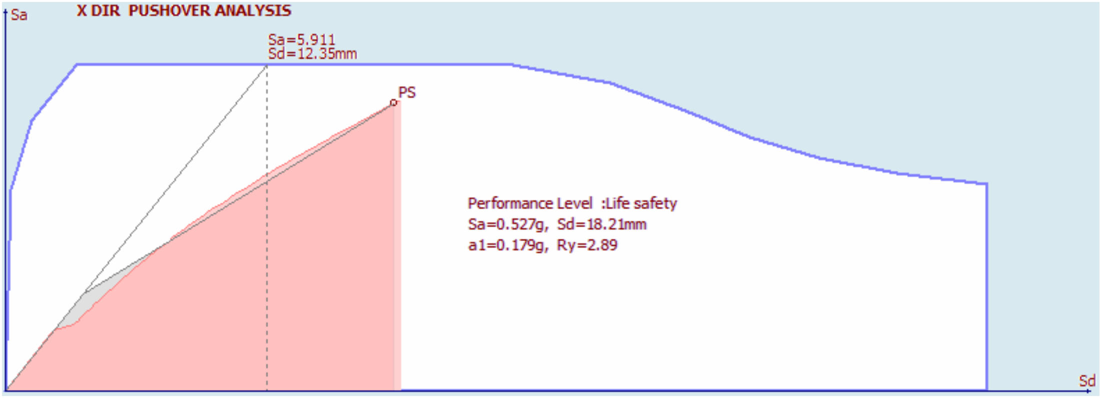


Figure 4.51: Performance Level of Case 5 (Torsional Irregularity) 3F EC8 Case: Third Analysis Case.

4.2.5.2 3F TEC-2007:

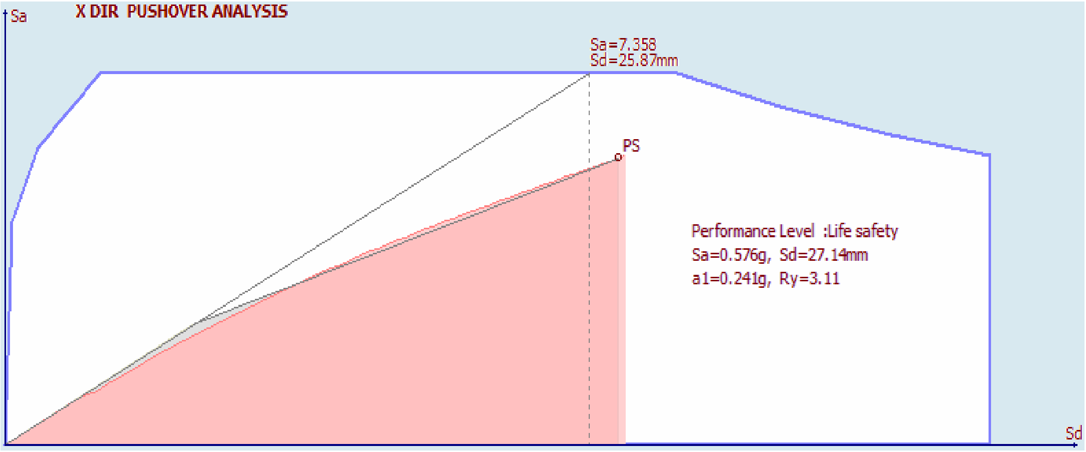


Figure 4.52: Performance Level of Case 5 (Torsional Irregularity) 3F TEC-2007 Case: First Analysis Case.

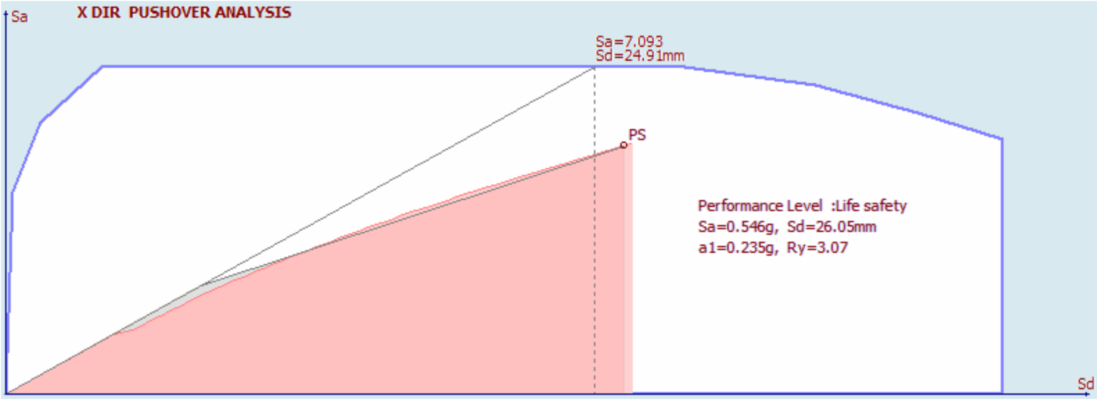


Figure 4.53: Performance Level of Case 5 (Torsional Irregularity) 3F TEC-2007 Case: Second Analysis Case.

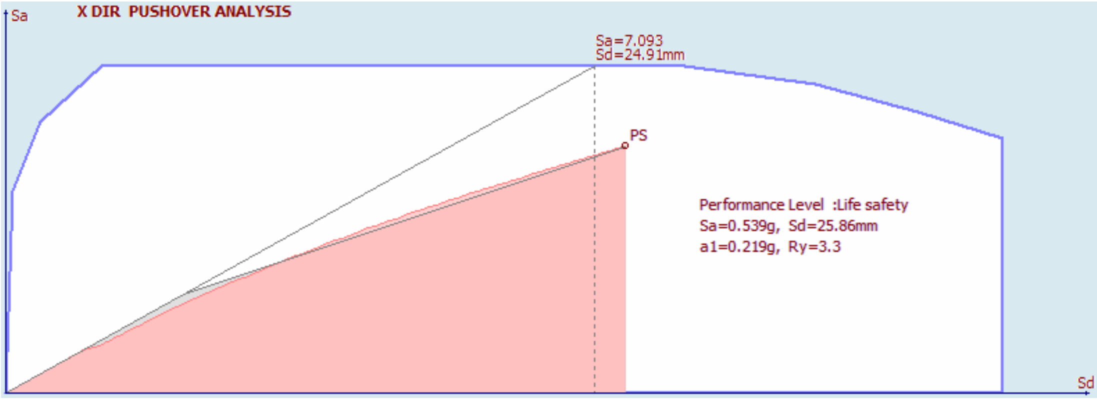


Figure 4.54: Performance Level of Case 5 (Torsional Irregularity) 3F TEC-2007 Case: Third Analysis Case.

4.2.5.3 5F Eurocode 8:

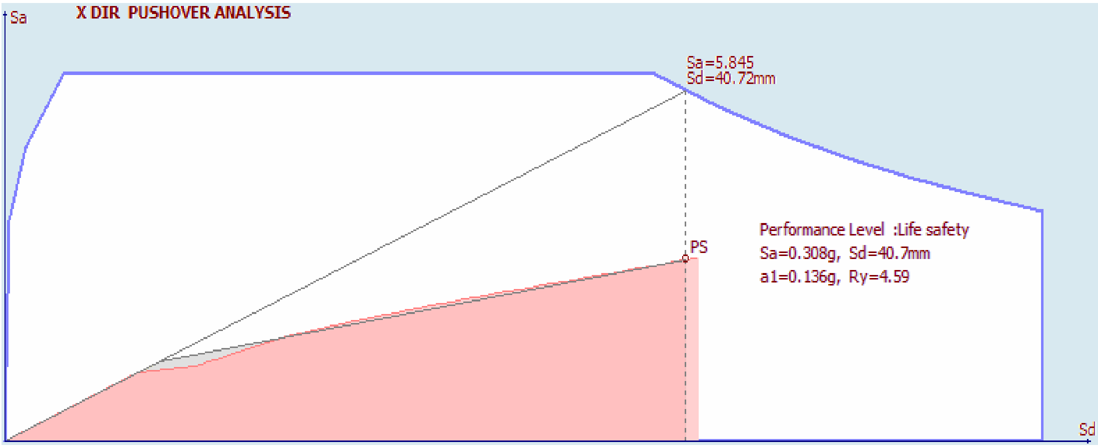


Figure 4.55: Performance Level of Case 5 (Torsional Irregularity) 5F EC8 Case: First Analysis Case.

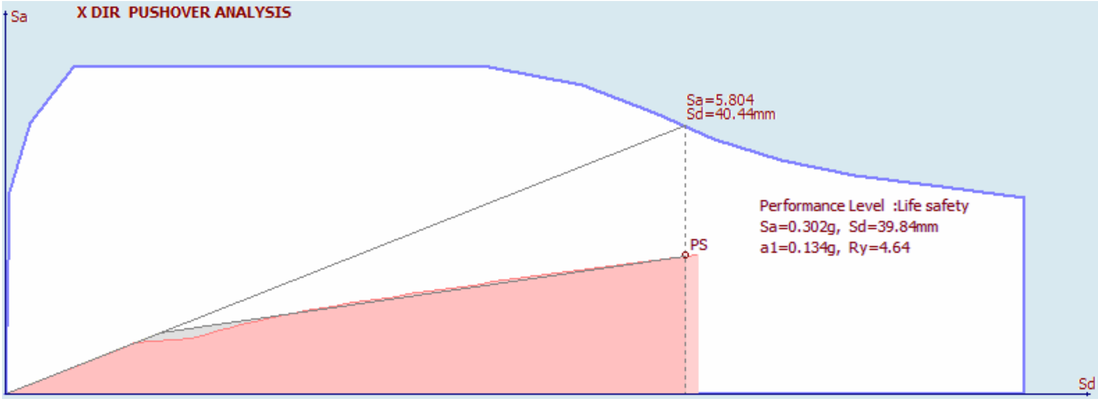


Figure 4.56: Performance Level of Case 5 (Torsional Irregularity) 5F EC8 Case: Second Analysis Case.

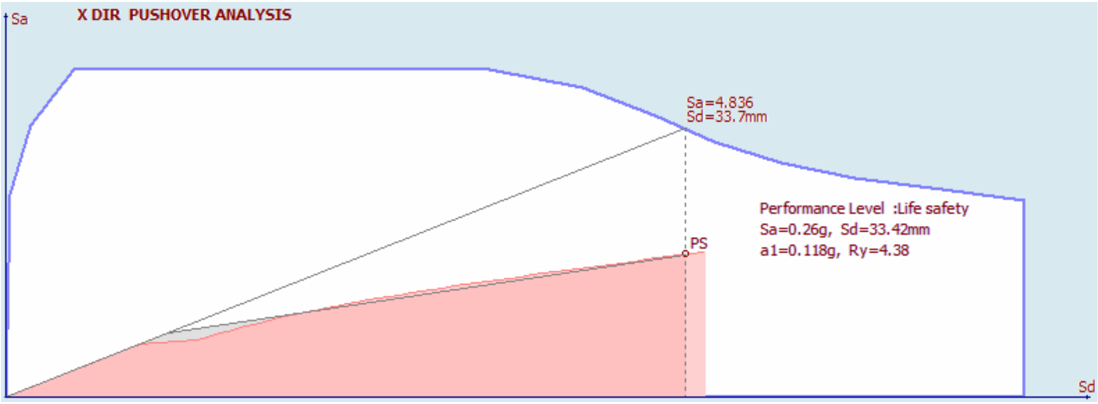


Figure 4.57: Performance Level of Case 5 (Torsional Irregularity) 5F EC8 Case: Third Analysis Case.

4.2.5.4 5F TEC-2007:

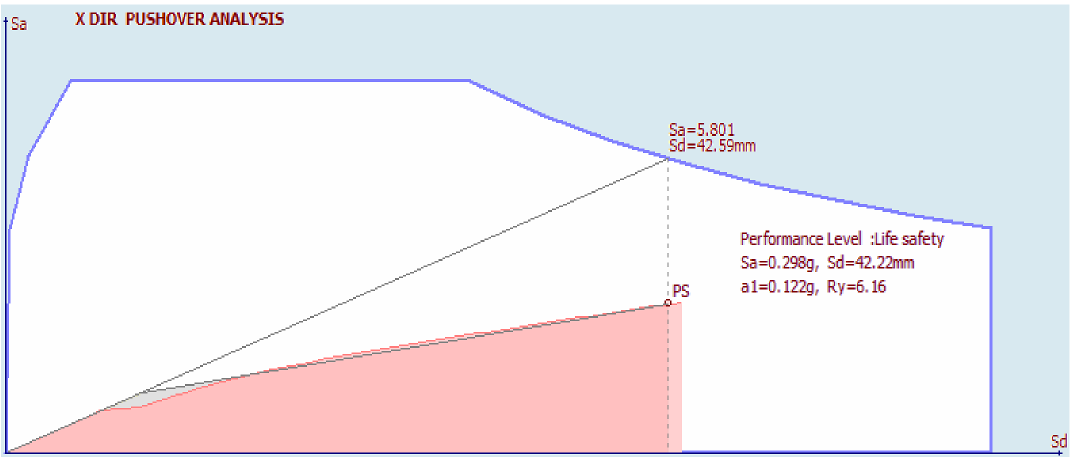


Figure 4.58: Performance Level of Case 5 (Torsional Irregularity) 5F TEC-2007 Case: First Analysis Case.

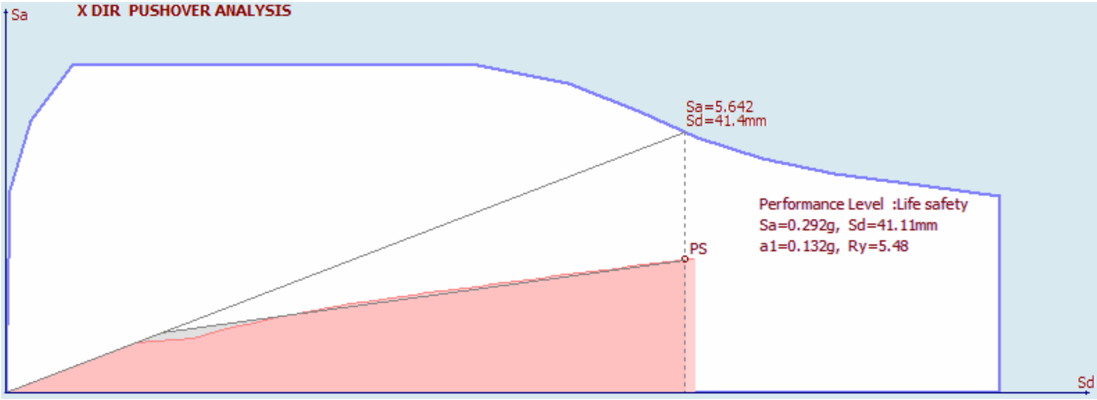


Figure 4.59: Performance Level of Case 5 (Torsional Irregularity) 5F TEC-2007 Case: Second Analysis Case.

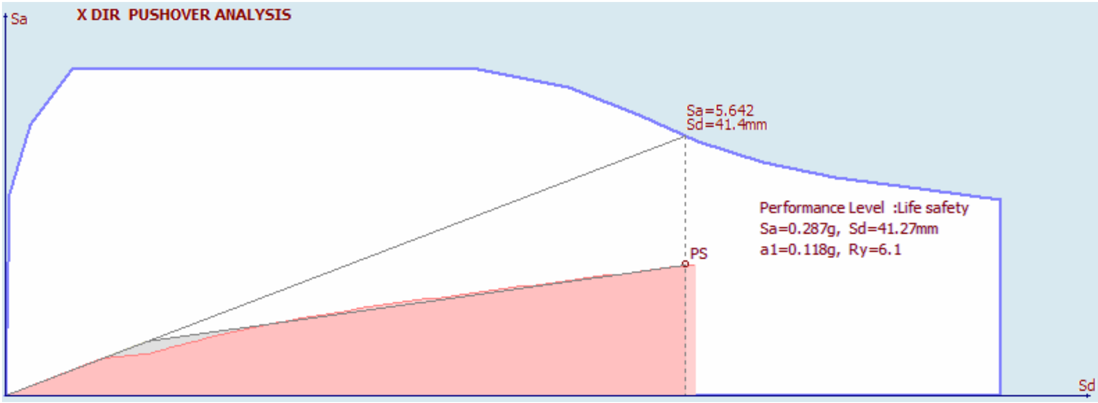


Figure 4.60: Performance Level of Case 5 (Torsional Irregularity) 5F TEC-2007 Case: Third Analysis Case.

4.3 Capacity Curves

Although all cases have reached the life safety performance level the following distinction has been observed in the capacity curves which are presented In this section.

4.3.1 Case 1 (Weak Storey):

4.3.1.1 3 Floor:

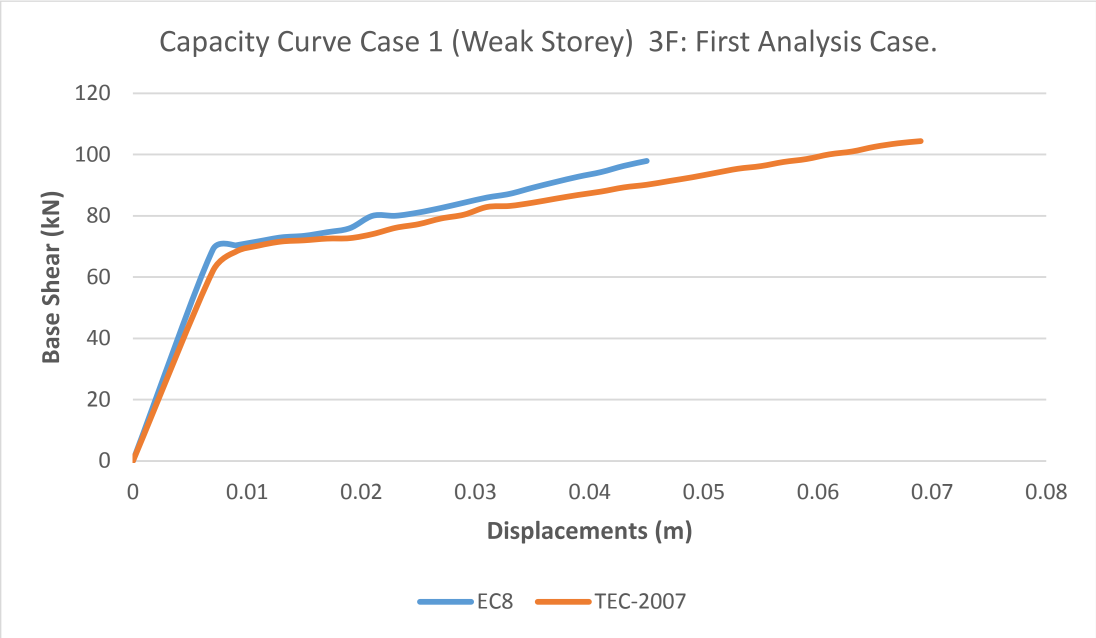


Figure 4.61: Capacity Curve Case 1 (Weak Storey) 3F: Spectrum, A_0 & Behavior Factor According to Code.

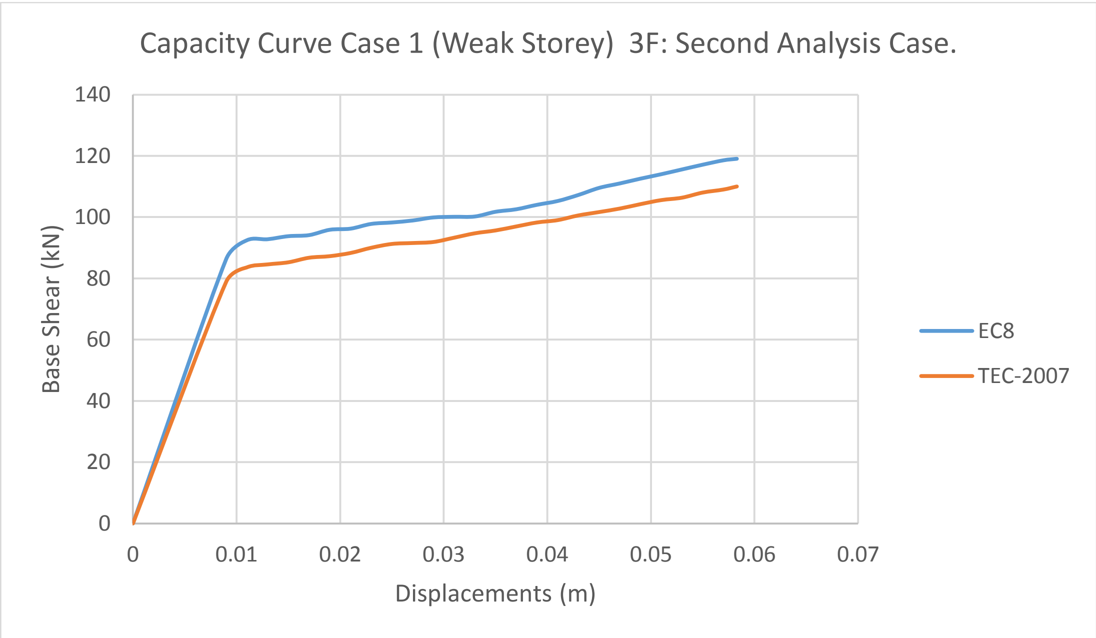


Figure 4.62: Capacity Curve Case 1 (Weak Storey) 3F Case: Second Analysis Case.

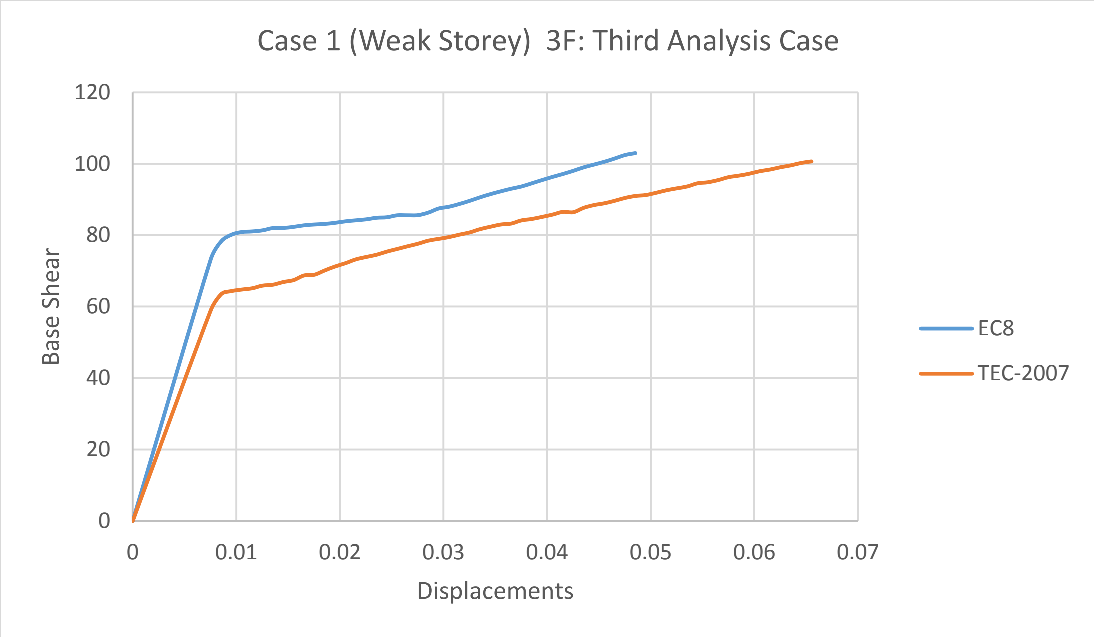


Figure 4.63: Capacity Curve Case 1 (Weak Storey) 3F Case: Third Analysis Case.

4.3.1.2 5 Floor:

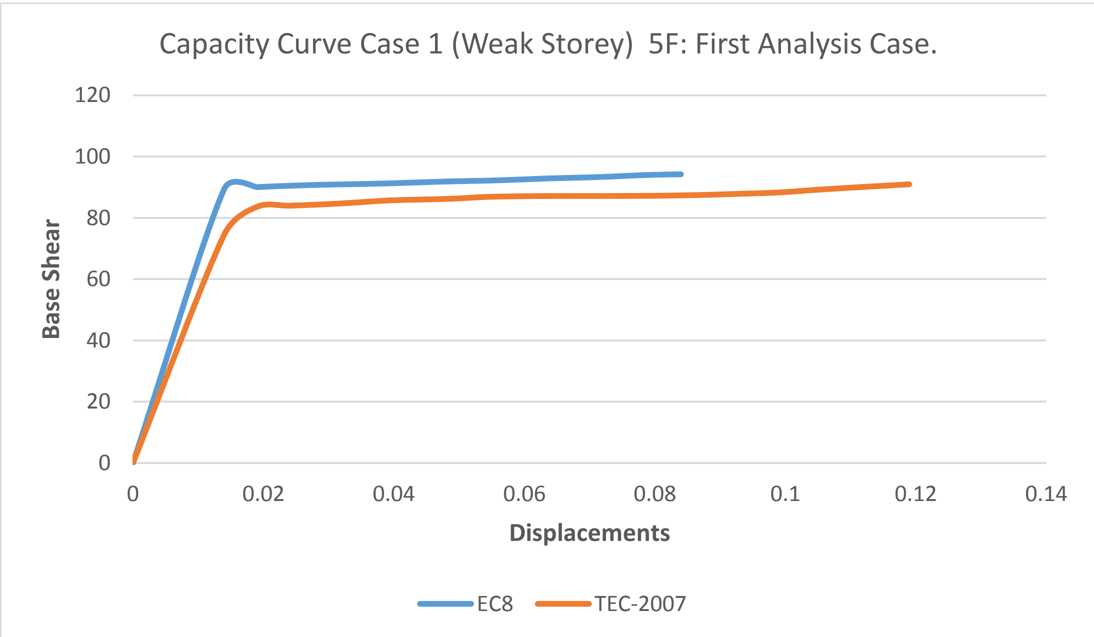


Figure 4.64: Capacity Curve Case 1 (Weak Storey) 5F: First Analysis Case.

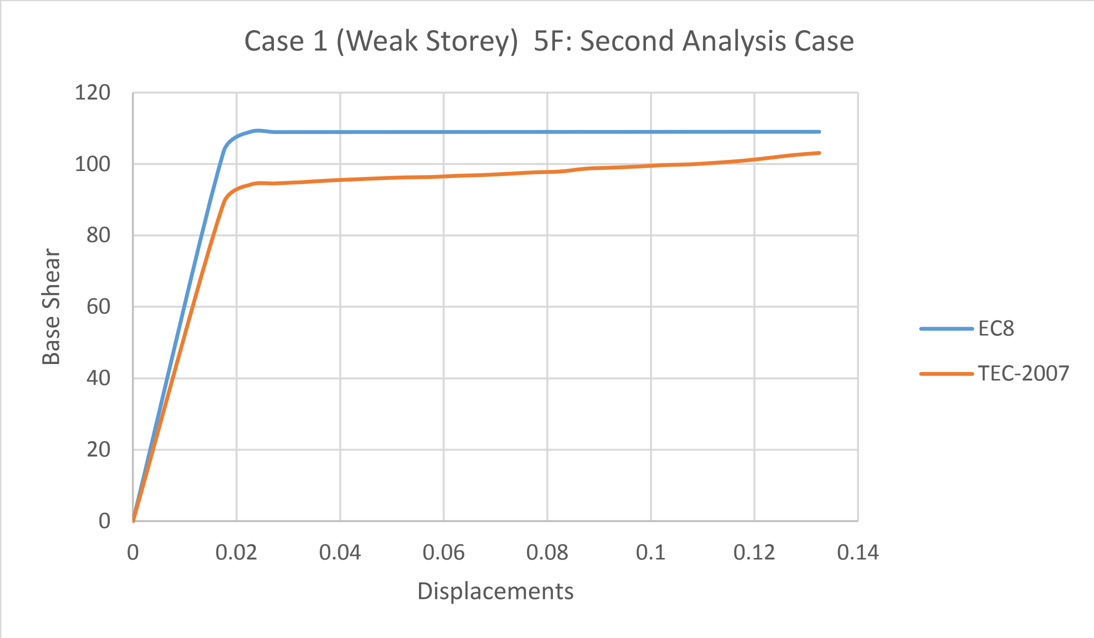


Figure 4.65: Capacity Curve Case 1 (Weak Storey) 5F: Second Analysis Case.

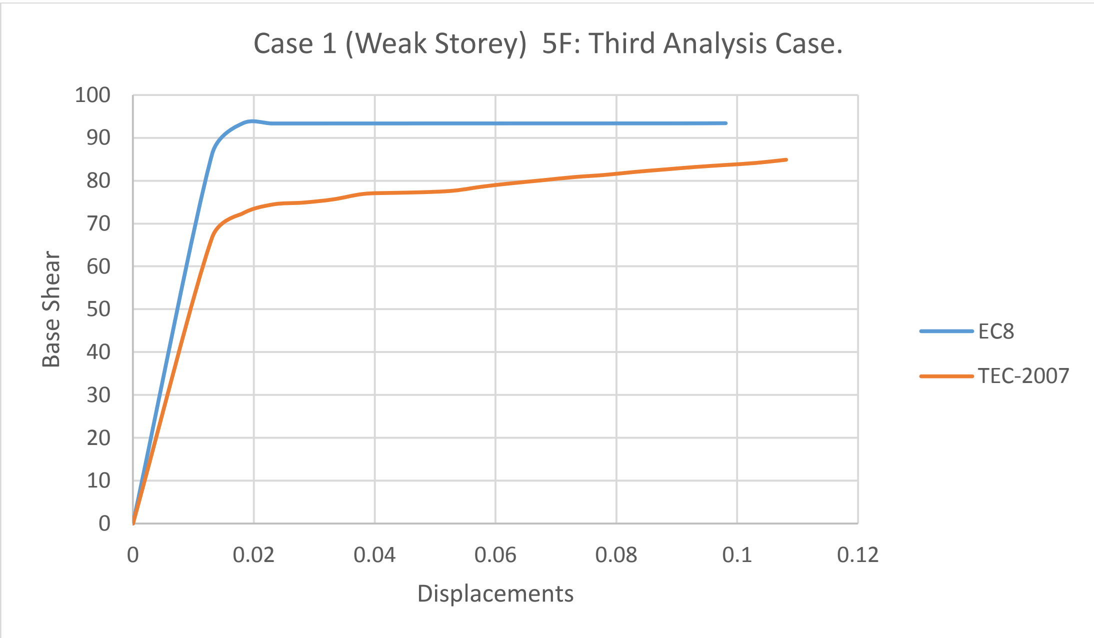


Figure 4.66: Capacity Curve Case 1 (Weak Storey) 5F Case: Third Analysis Case.

In Case 1 (Weak Storey) the following has been observed in the capacity curves:

1. In the first analysis where the spectrum, A_0 and the behavior factor where taken according to the advised value defined by the codes:

- The displacement demand in the TEC-2007 cases was higher than the displacement demand reached by the EC8 cases. That can be observed in both the 3F & 5F cases.

- The capacity demand of the EC8 cases are higher than the capacity demand of the TEC-2007 cases. That can be observed in the 3F & 5F cases.

- It has been observed that, with the increase in height of the building the capacity demand decreases, however the displacement demand reached by the 5F cases were higher than the displacements demand reached by the 3F cases for both codes.

2. In the second analysis where the ATC-3 normalized spectrum was used for both the TEC-2007 and the EC8 cases, while using the same value for A_0 and the behavior factor for both cases (0.3 and 6 respectively):

    - The displacement demand in the TEC-2007 cases reached the same value as the displacement demand reached by the EC8 cases. That can be observed in both the 3F & 5F cases.

    - The capacity demand of the EC8 cases are higher than the capacity demand of the TEC-2007 cases. That can be observed in the 3F & 5F cases.

- It has been observed that, with the increase in height of the building the capacity demand decreases, however the displacement demand reached by the 5F cases were higher than the displacements demand reached by the 3F cases for both codes.

3. In the third analysis where the ATC-3 normalized response spectrum is used for both cases, the TEC-2007 & the EC8 case, but with the advised value of A_0 and behavior factor that are defined according to the code :

    - The displacement demand in the TEC-2007 cases was higher than the displacement demand reached by the EC8 cases. That can be observed in both the 3F & 5F cases.

    - The capacity demand of the EC8 cases are higher than the capacity demand of the TEC-2007 cases. That can be observed in the 3F & 5F cases.

    - It has been observed that, with the increase in height of the building the capacity demand decreases, however the displacement demand reached by the 5F cases were higher than the displacements demand reached by the 3F cases for both codes.

This results is due to the difference in the analysis approach while dealing with this type of irregularity in both codes:

- EC8 when dealing with this irregularity type reduce the behavior factor by 20%
- TEC-2007 when dealing with this type of irregularity, when the value η_{ci} < 0.8 the behavior factor should be multiplied by 1.25x η_{ci}.

4.3.2 Case 2 (Soft Storey):

4.3.2.1 3 Floor:

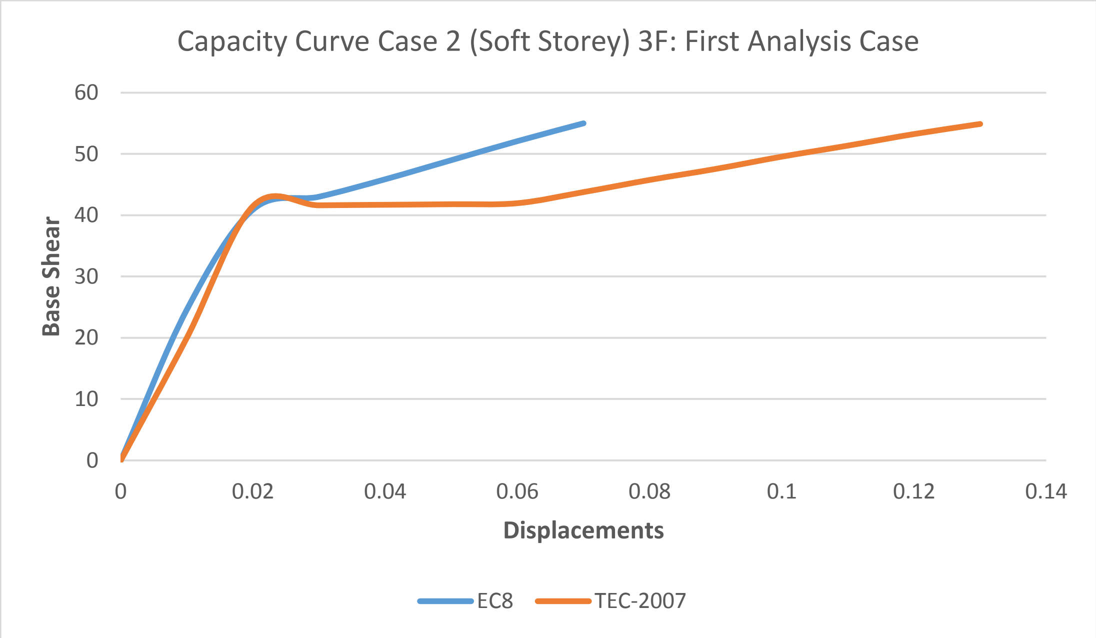


Figure 4.67: Capacity Curve Case 2 (Soft Storey) 3F Case: First Analysis Case.

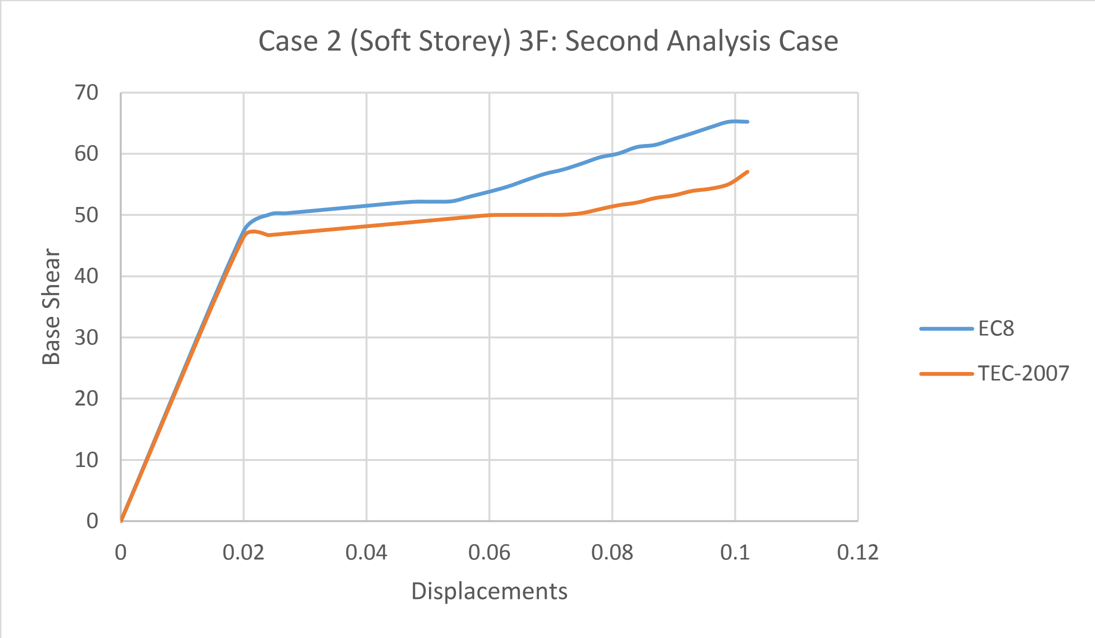


Figure 4.68: Capacity Curve Case 2 (Soft Storey) 3F Case: Second Analysis Case.

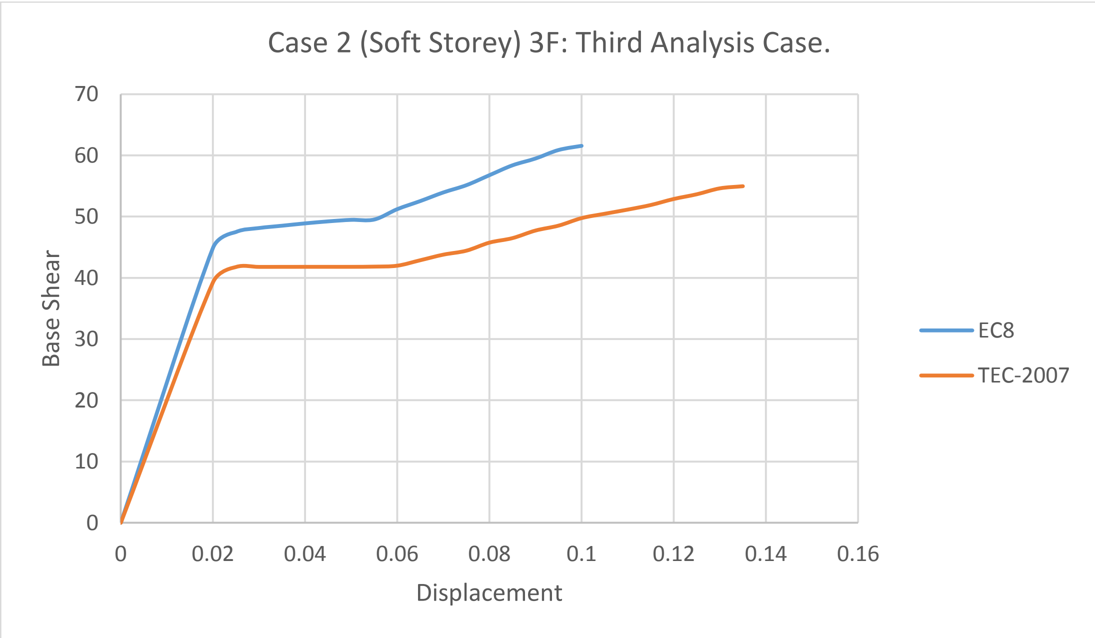


Figure 4.69: Capacity Curve Case 2 (Soft Storey) 3F Case: Third Analysis Case.

4.3.2.2 5 Floor:

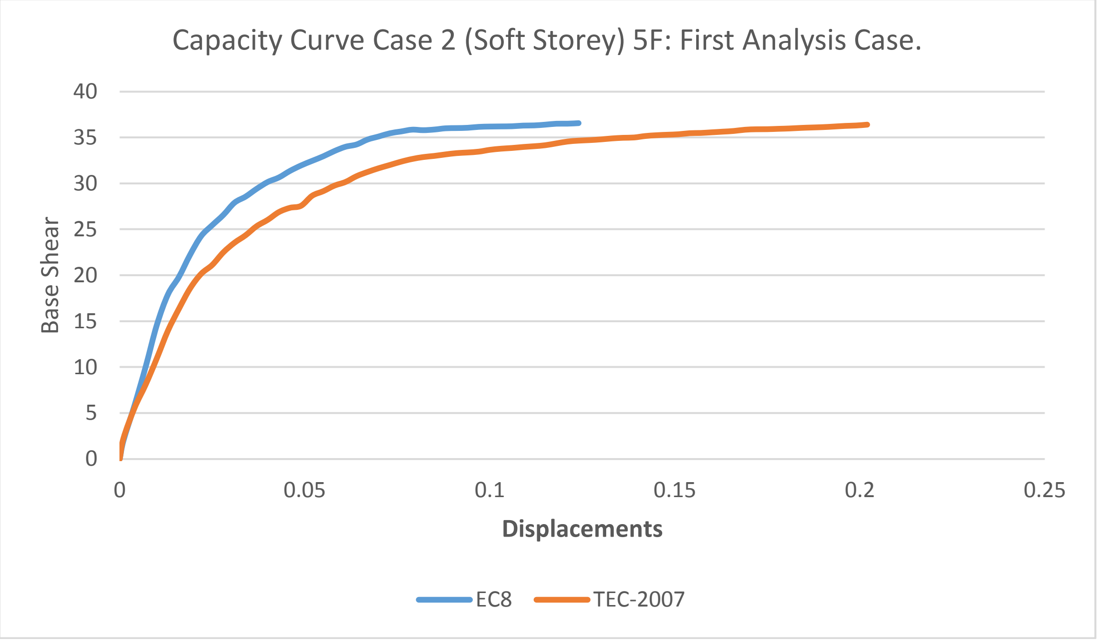


Figure 4.70: Capacity Curve Case 2 (Soft Storey) 5F Case: First Analysis Case.

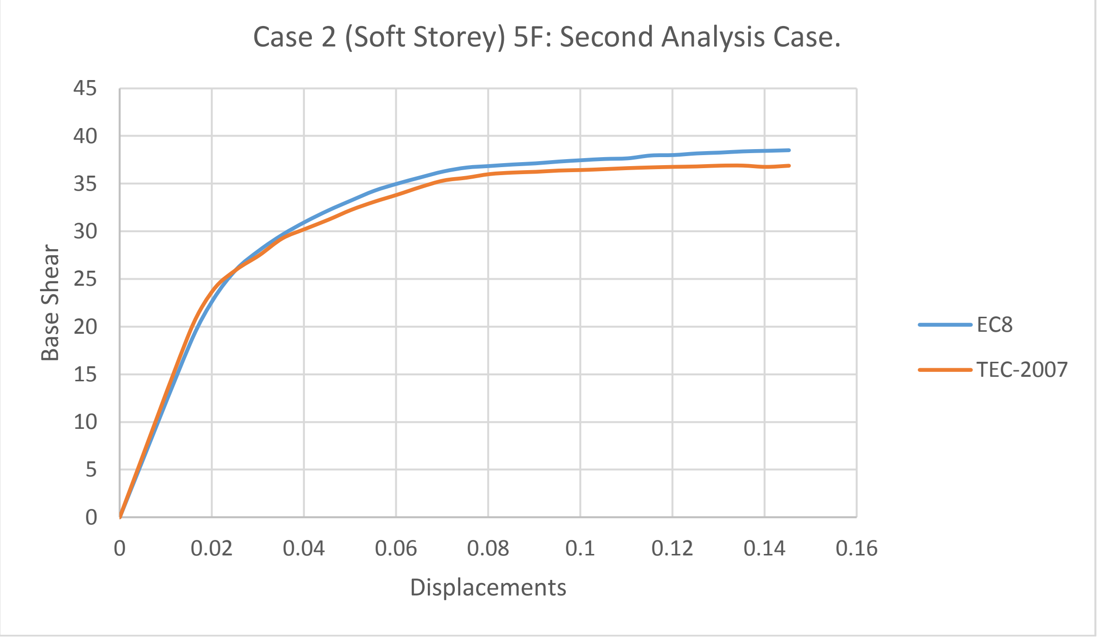


Figure 4.71: Capacity Curve Case 2 (Soft Storey) 5F Case: Second Analysis Case.

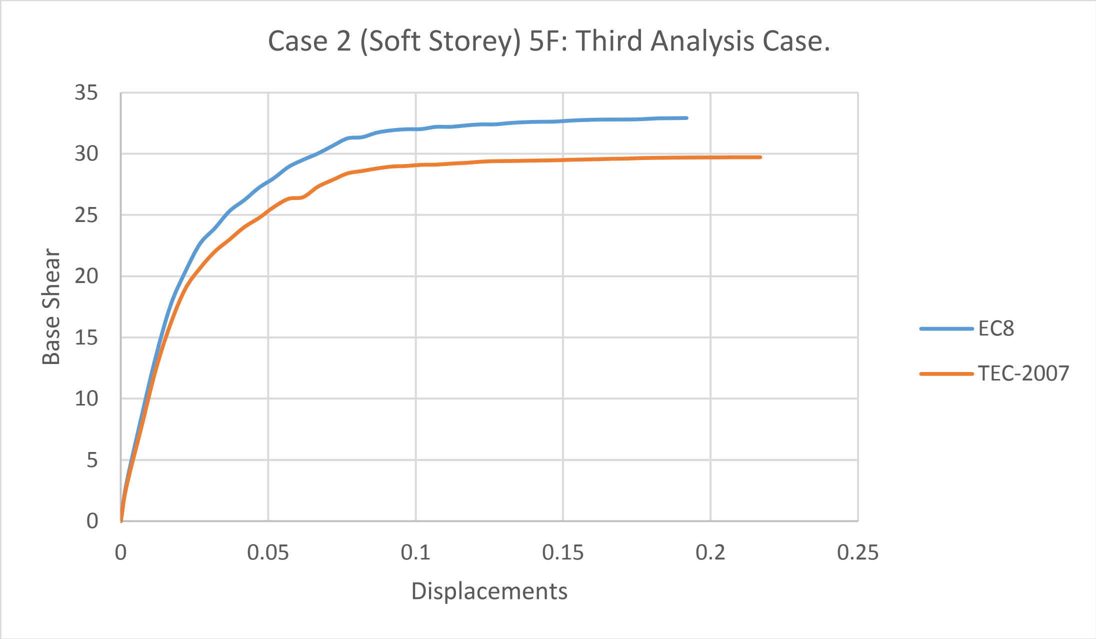


Figure 4.72: Capacity Curve Case 2 (Soft Storey) 5F Case: Third Analysis Case.

In Case 2 (Soft Storey) the following has been observed in the capacity curves:

1. In the first analysis where the spectrum, A_0 and the behavior factor where taken according to the advised value defined by the codes:
   - The displacement demand in the TEC-2007 cases was higher than the displacement demand reached by the EC8 cases. That can be observed in both the 3F & 5F cases.

   - The capacity demand of the EC8 cases are higher than the capacity demand of the TEC-2007 cases. That can be observed in the 3F & 5F cases.

   - It has been observed that, with the increase in height of the building the capacity demand decreases, however the displacement demand reached by the 5F cases were higher than the displacements demand reached by the 3F cases for both codes.

2. In the second analysis where the ATC-3 normalized spectrum was used for both the TEC-2007 and the EC8 cases, while using the same value for A_0 and the behavior factor for both cases (0.3 and 6 respectively):

   - The displacement demand in the TEC-2007 cases reached the same value as the displacement demand reached by the EC8 cases. That can be observed in both the 3F & 5F cases.

   - The capacity demand of the EC8 cases are higher than the capacity demand of the TEC-2007 cases. That can be observed in the 3F & 5F cases.

   - It has been observed that, with the increase in height of the building the capacity demand decreases, however the displacement demand reached by the 5F cases were higher than the displacements demand reached by the 3F cases for both codes.

3. In the third analysis where the ATC-3 normalized response spectrum is used for both cases, the TEC-2007 & the EC8 case, but with the advised value of A_0 and behavior factor that are defined according to the code :
   - The displacement demand in the TEC-2007 cases was higher than the displacement demand reached by the EC8 cases. That can be observed in both the 3F & 5F cases.

- The capacity demand of the EC8 cases are higher than the capacity demand of the TEC-2007 cases. That can be observed in the 3F & 5F cases.

- It has been observed that, with the increase in height of the building the capacity demand decreases, however the displacement demand reached by the 5F cases were higher than the displacements demand reached by the 3F cases for both codes.

This results is due to the difference in the analysis approach while dealing with this type of irregularity in both codes:

- EC8 when dealing with this irregularity type reduce the behavior factor by 20%
- TEC-2007 when dealing with this type of irregularity, when the value $\eta_{ci} < 0.8$ the behavior factor should be multiplied by $1.25x\ \eta_{ci}$.

4.3.3 Case 3 (Projection in Plan):

4.3.3.1 3 Floor:

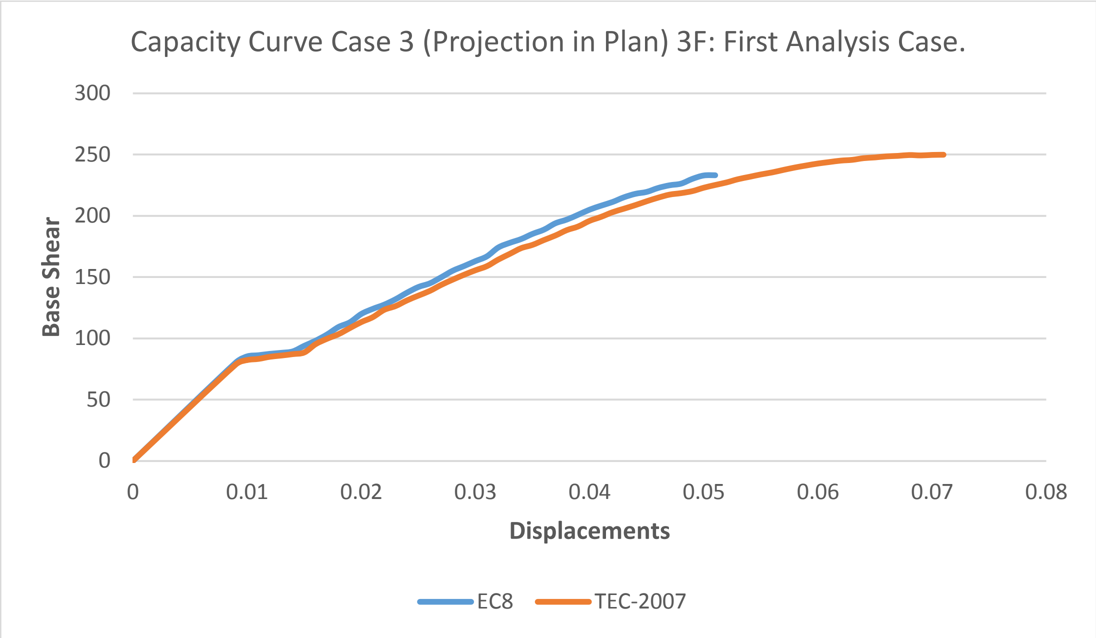


Figure 4.73: Capacity Curve Case 3 (Projection in Plan) 3F Case: First Analysis Case.

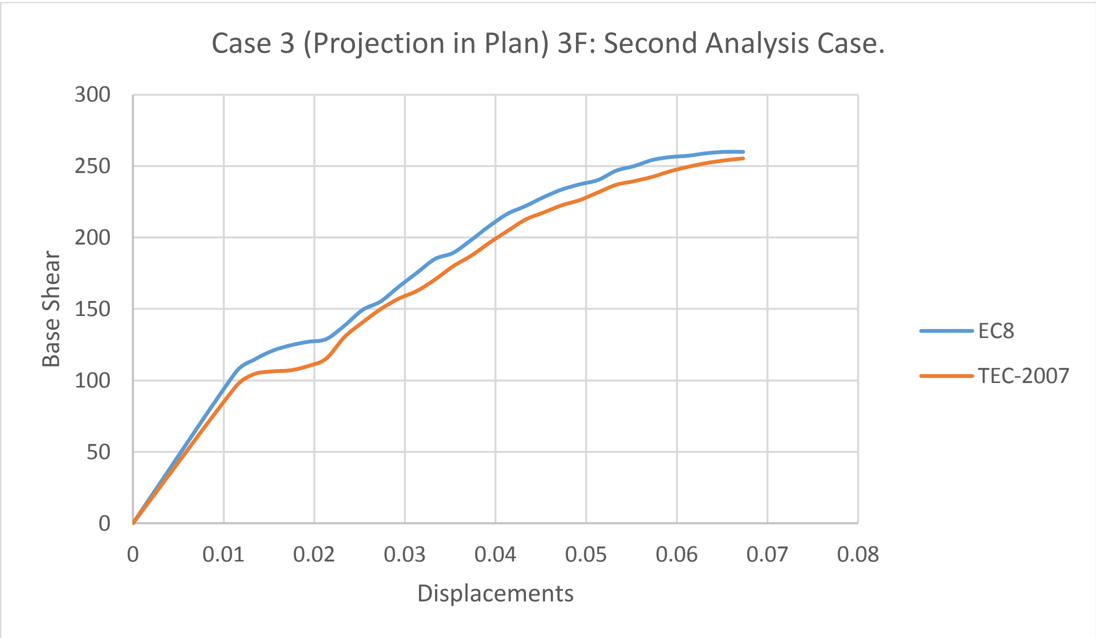


Figure 4. 74: Capacity Curve Case 3 (Projection in Plan) 3F Case: Second Analysis Case.

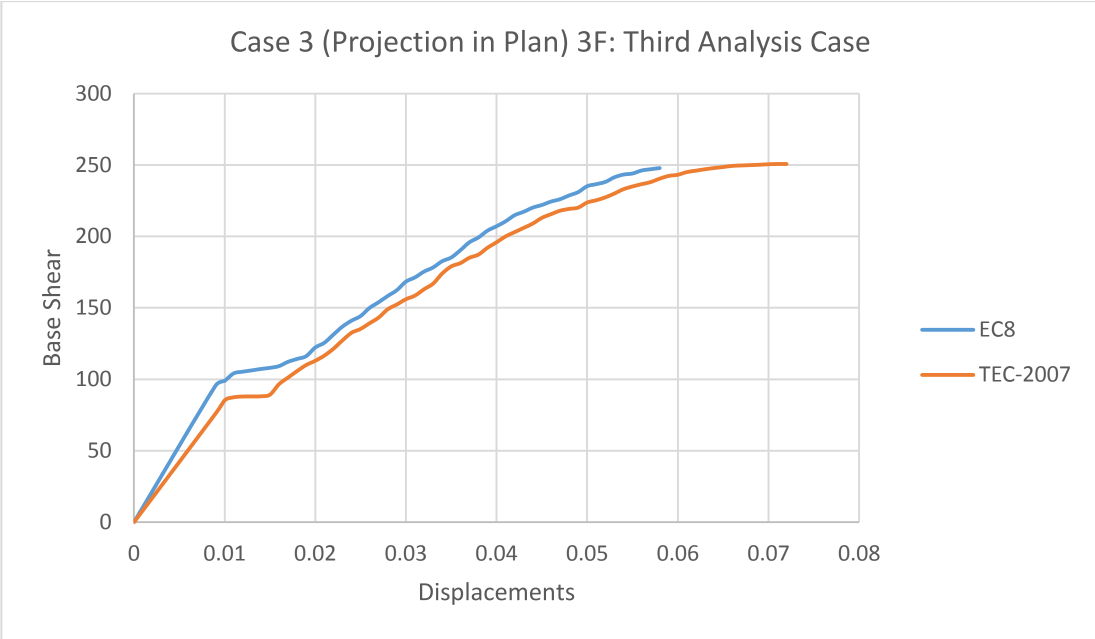


Figure 4.75: Capacity Curve Case 3 (Projection in Plan) 3F Case: Third Analysis Case.

4.3.3.1 5 Floor:

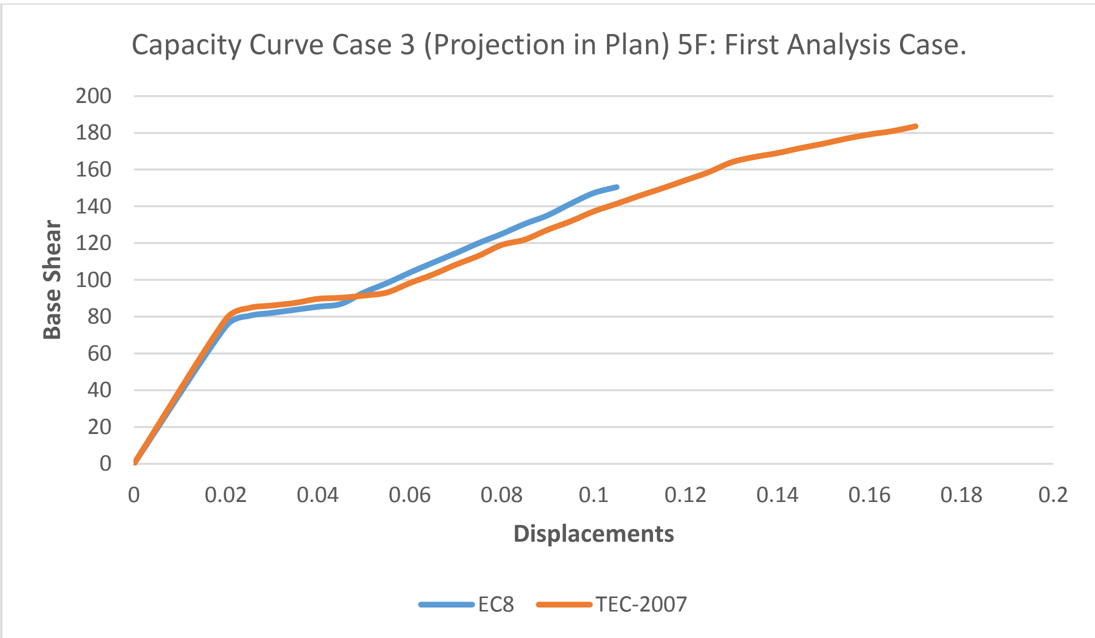


Figure 4.76: Capacity Curve Case 3 (Projection in Plan) 5F Case: First Analysis Case.

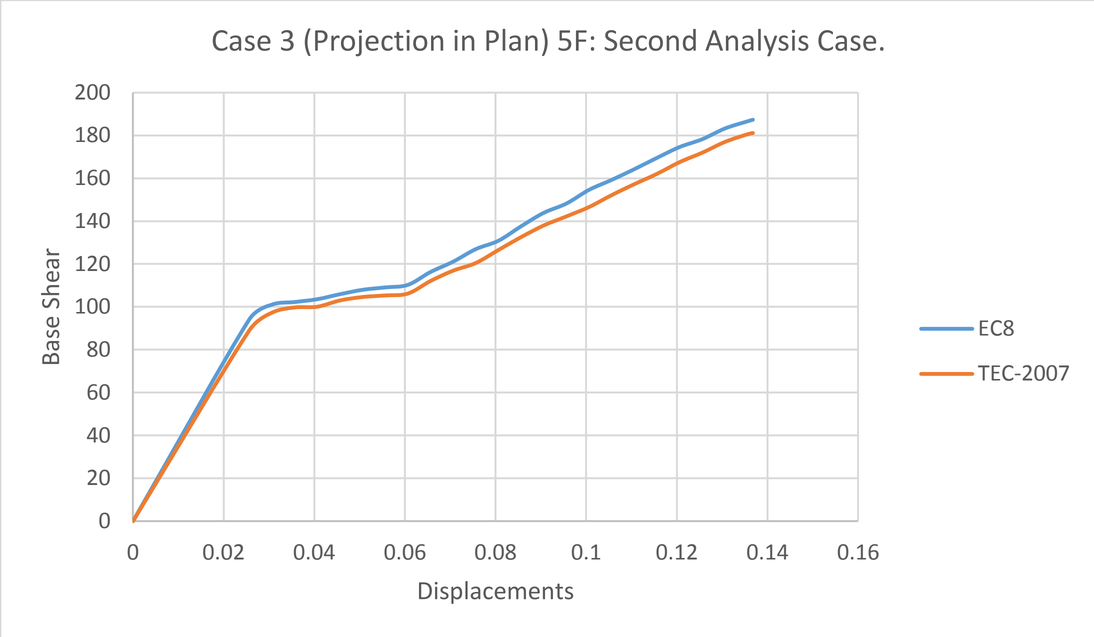


Figure 4.77: Capacity Curve Case 3 (Projection in Plan) 5F Case: Second Analysis Case.

In case 3 (Projection in Plan) the following has been observed in the capacity curves:

1. In the first analysis where the spectrum, A_0 and the behavior factor where taken according to the advised value defined by the codes:

   - The displacement, at the performance point, reached by the TEC-2007 cases was higher than the displacement reached by the EC8 cases. That can be observed in both the 3F & 5F cases.

   - The EC8 cases have higher capacity than the TEC-2007 cases. That can be observed in the 3F & 5F cases.

   - It has been observed that, with the increase in height of the building the base shear decreases, however the displacement reached by the 5F cases were higher than the displacements reached by the 3F cases for both codes.

2. In the second analysis where the ATC-3 normalized spectrum was used for both the TEC-2007 and the EC8 cases, while using the same value for A_0 and the behavior factor for both cases (0.3 and 6 respectively):

    - The displacement demand in the TEC-2007 cases reached the same value as the displacement demand reached by the EC8 cases. That can be observed in both the 3F & 5F cases.

    - The capacity demand of the EC8 cases are higher than the capacity demand of the TEC-2007 cases. That can be observed in the 3F & 5F cases.

    - It has been observed that, with the increase in height of the building the capacity demand decreases, however the displacement demand reached by the 5F cases were higher than the displacements demand reached by the 3F cases for both codes.

3. In the third analysis where the ATC-3 normalized response spectrum is used for both cases, the TEC-2007 & the EC8 case, but with the advised value of A_0 and behavior factor that are defined according to the code :

- The displacement demand in the TEC-2007 cases was higher than the displacement demand reached by the EC8 cases. That can be observed in both the 3F & 5F cases.

- The capacity demand of the EC8 cases are higher than the capacity demand of the TEC-2007 cases. That can be observed in the 3F & 5F cases.

- It has been observed that, with the increase in height of the building the capacity demand decreases, however the displacement demand reached by the 5F cases were higher than the displacements demand reached by the 3F cases for both codes.

This results is due to the difference in the analysis approach while dealing with this type of irregularity in both codes:

- EC8 when dealing with this irregularity type reduce the behavior factor by 20%

4.3.4 Case 4 (Floor Discontinuity):

4.3.4.1 3 Floor:

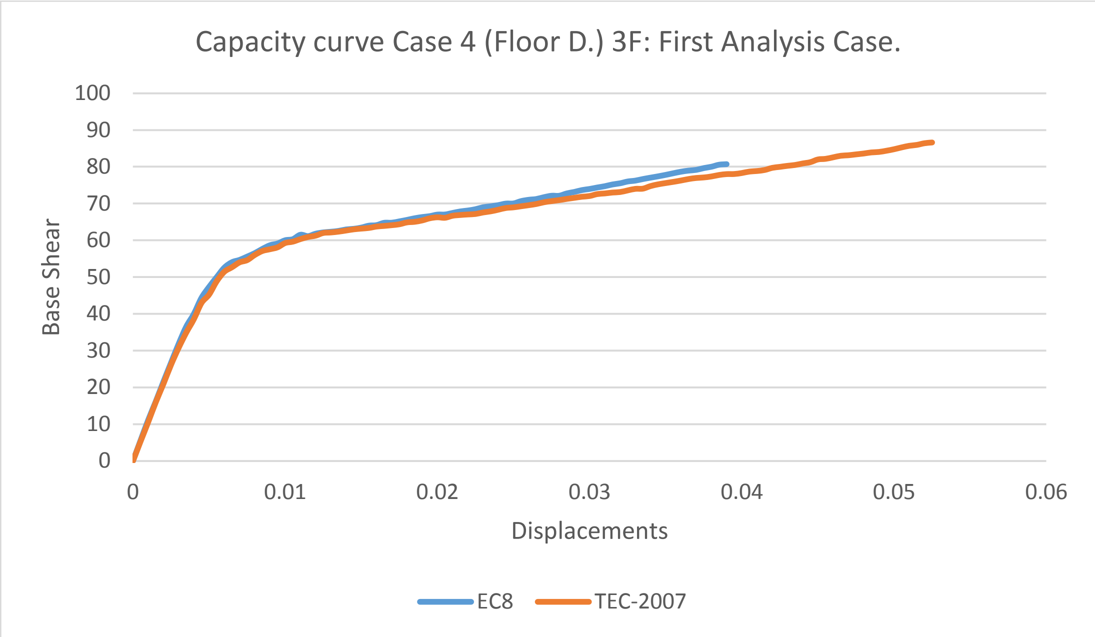


Figure 4.78: Capacity Curve Case 4 (Floor D.) 3F Case: First Analysis Case.

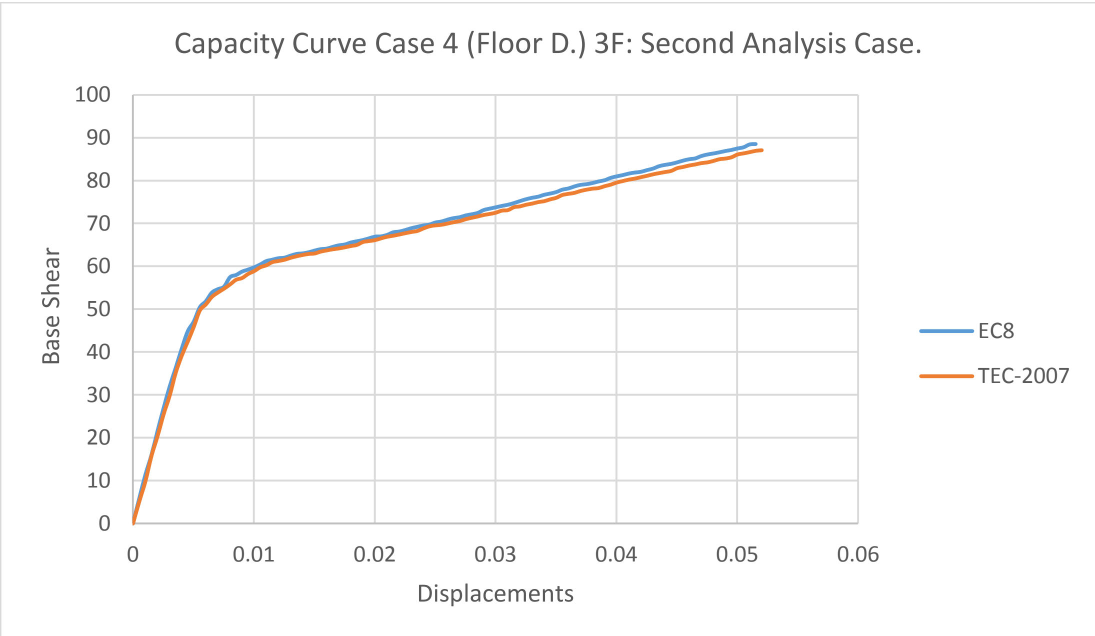


Figure 4.79: Capacity Curve Case 4 (Floor D.) 3F Case: Second Analysis Case.

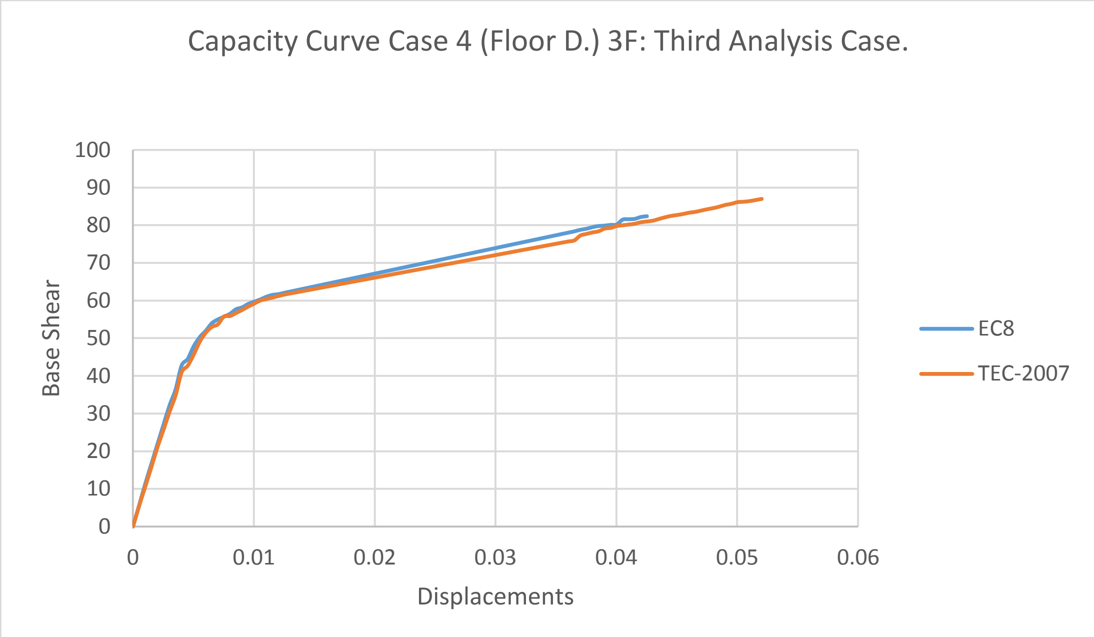


Figure 4.80: Capacity Curve Case 4 (Floor D.) 3F Case: Third Analysis Case.

4.3.4.2 5 Floor:

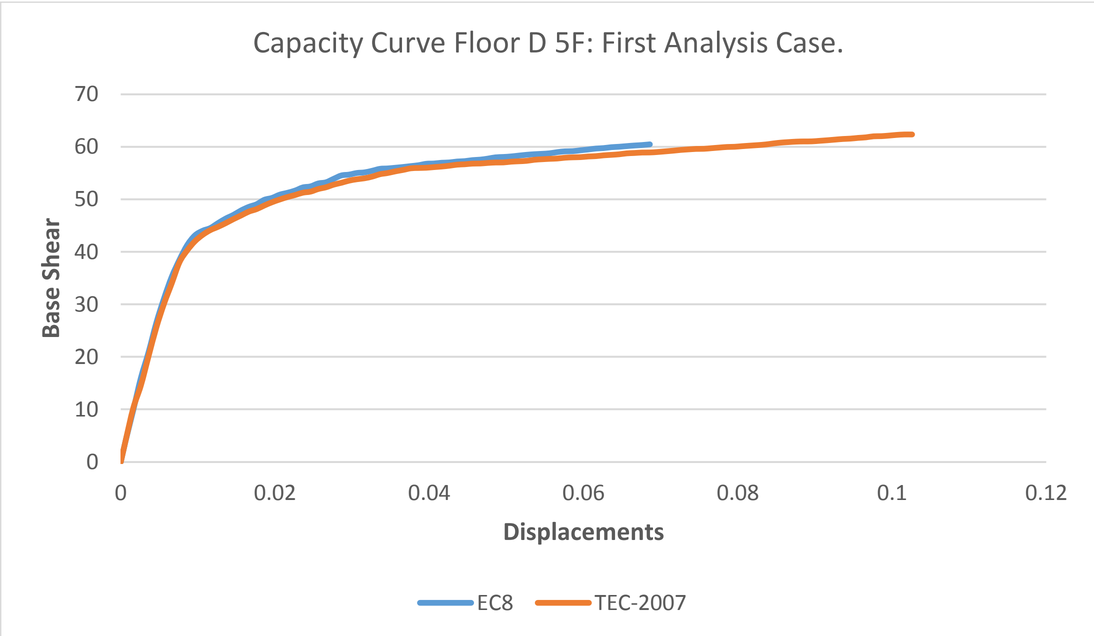


Figure 4.81: Capacity Curve Case 4 (Floor D.) 5F Case: First Analysis Case.

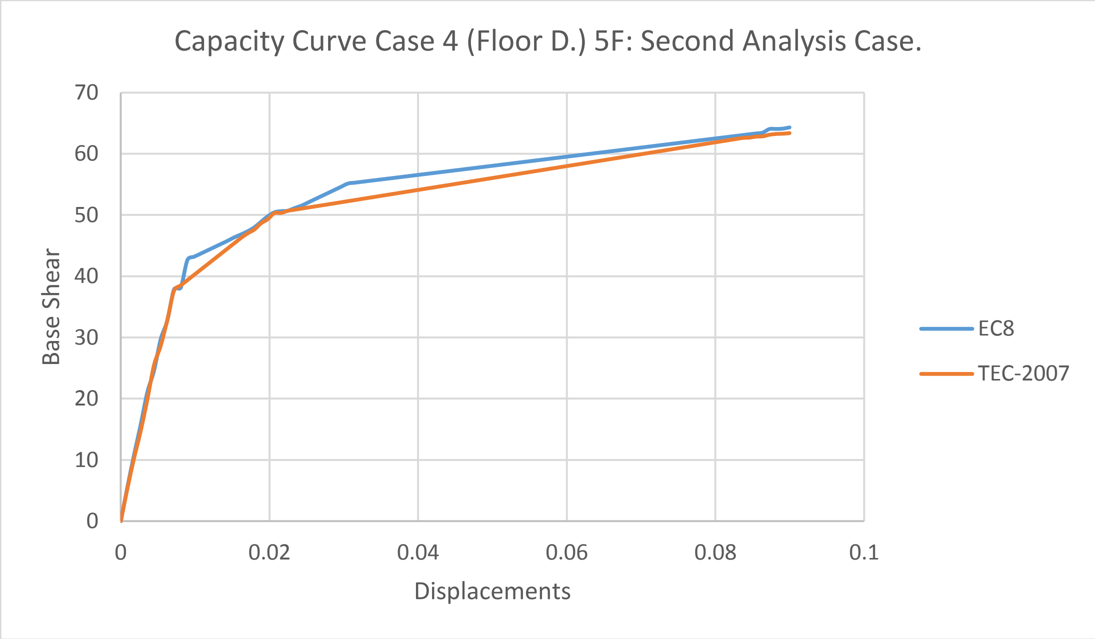


Figure 4.82: Capacity Curve Case 4 (Floor D.) 5F Case: Second Analysis Case.

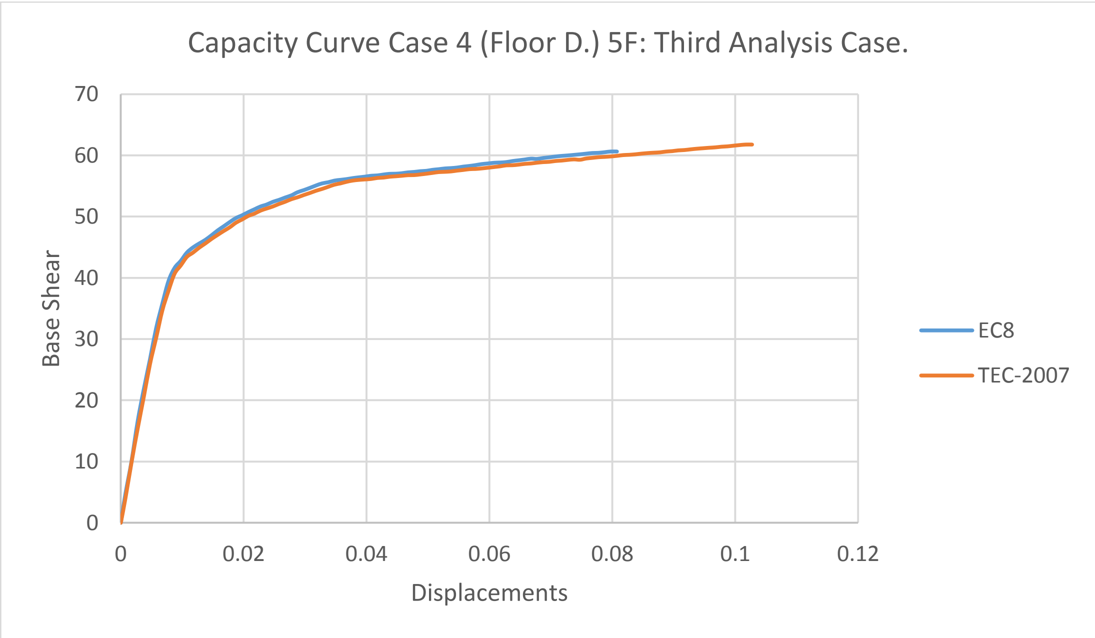


Figure 4.83: Capacity Curve Case 4 (Floor D.) 5F Case: Third Analysis Case.

In case 4 (Floor Discontinuity) the following has been observed in the capacity curves:

1. In the first analysis where the spectrum, A_0 and the behavior factor where taken according to the advised value defined by the codes:

    - The displacement, at the performance point, reached by the TEC-2007 cases was higher than the displacement reached by the EC8 cases. That can be observed in both the 3F & 5F cases.

    - The EC8 cases have higher capacity than the TEC-2007 cases. That can be observed in the 3F & 5F cases.

    - It has been observed that, with the increase in height of the building the base shear decreases, however the displacement reached by the 5F cases were higher than the displacements reached by the 3F cases for both codes.

2. In the second analysis where the ATC-3 normalized spectrum was used for both the TEC-2007 and the EC8 cases, while using the same value for A_0 and the behavior factor for both cases (0.3 and 6 respectively):

    - The displacement demand in the TEC-2007 cases reached the same value as the displacement demand reached by the EC8 cases. That can be observed in both the 3F & 5F cases.

    - The capacity demand of the EC8 cases are higher than the capacity demand of the TEC-2007 cases. That can be observed in the 3F & 5F cases.

- It has been observed that, with the increase in height of the building the capacity demand decreases, however the displacement demand reached by the 5F cases were higher than the displacements demand reached by the 3F cases for both codes.

3. In the third analysis where the ATC-3 normalized response spectrum is used for both cases, the TEC-2007 & the EC8 case, but with the advised value of A_0 and behavior factor that are defined according to the code :

    - The displacement demand in the TEC-2007 cases was higher than the displacement demand reached by the EC8 cases. That can be observed in both the 3F & 5F cases.

    - The capacity demand of the EC8 cases are higher than the capacity demand of the TEC-2007 cases. That can be observed in the 3F & 5F cases.

    - It has been observed that, with the increase in height of the building the capacity demand decreases, however the displacement demand reached by the 5F cases were higher than the displacements demand reached by the 3F cases for both codes.

This results is due to the difference in the analysis approach while dealing with this type of irregularity in both codes:

- EC8 when dealing with this irregularity type reduce the behavior factor by 20%

4.3.5 Case 5 (Torsional Irregularity):

4.3.5.1 3 Floor:

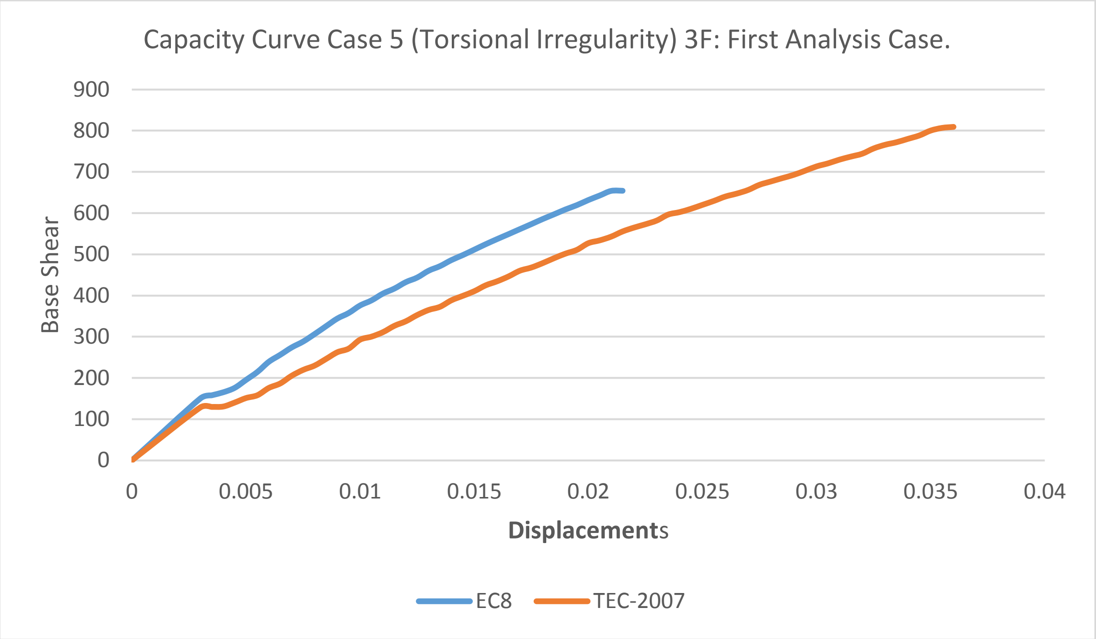


Figure 4.84: Capacity Curve Case 5 (Torsional Irregularity) 3F Case: First Analysis Case.

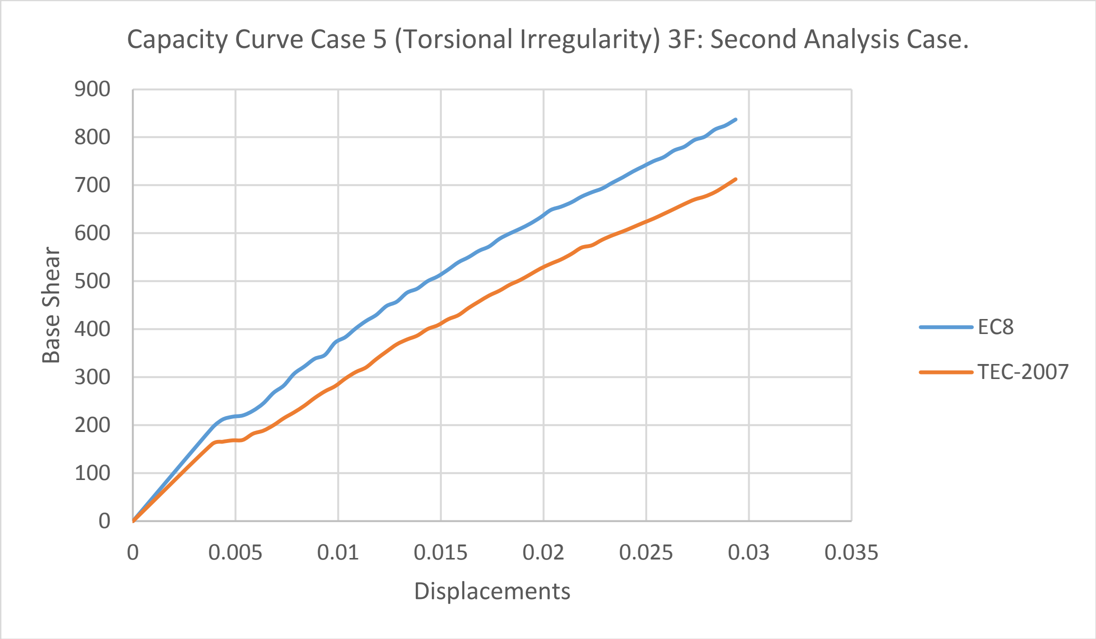


Figure 4.85: Capacity Curve Case 5 (Torsional Irregularity) 3F Case: Second Analysis Case.

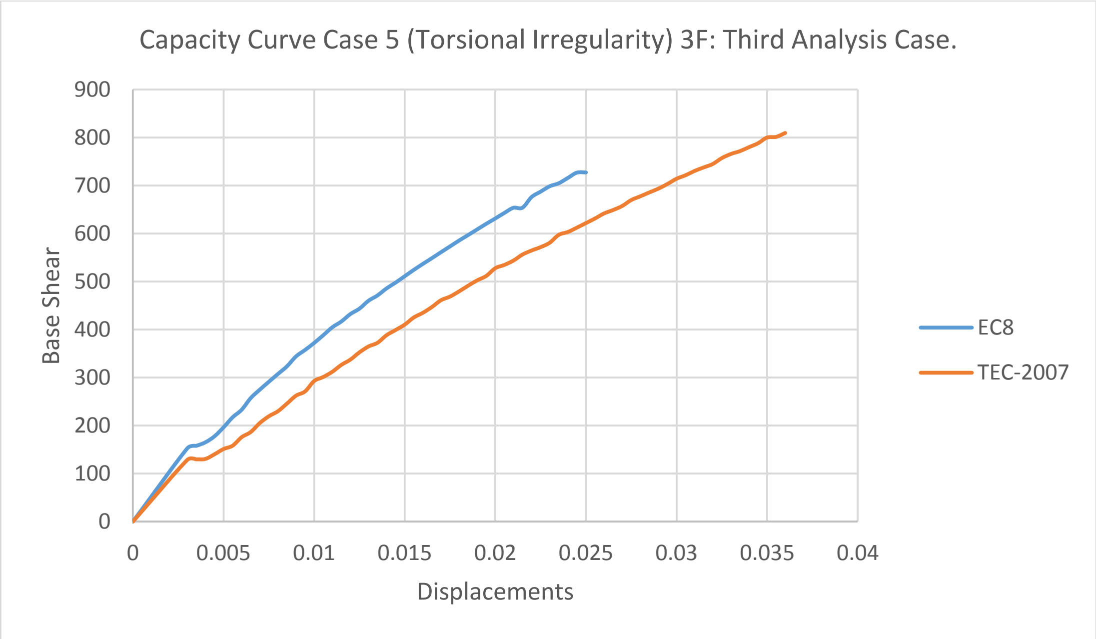


Figure 4.86: Capacity Curve Case 5 (Torsional Irregularity) 3F Case: Third Analysis Case.

4.3.5.2 5 Floor:

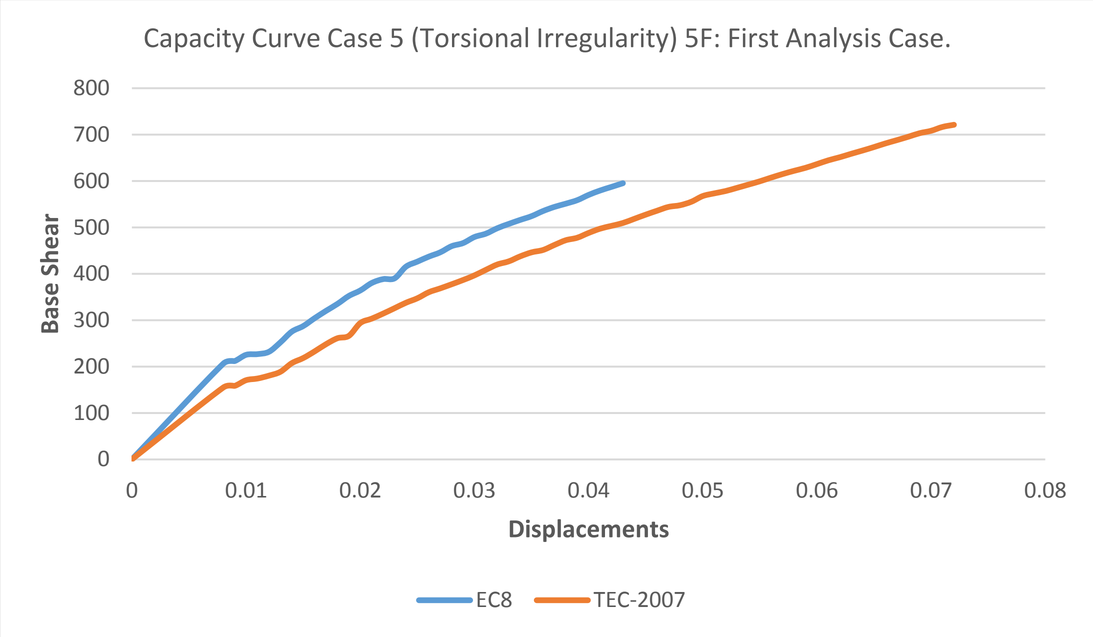


Figure 4.87: Capacity Curve Case 5 (Torsional Irregularity) 5F Case: First Analysis Case.

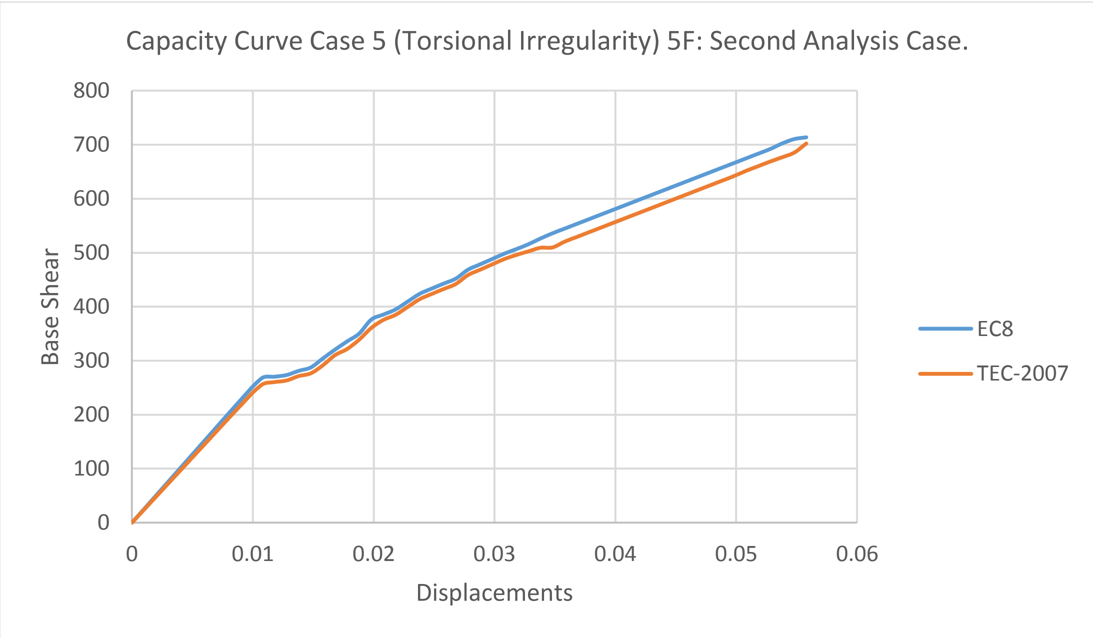


Figure 4.88: Capacity Curve Case 5 (Torsional Irregularity) 5F Case: Second Analysis Case.

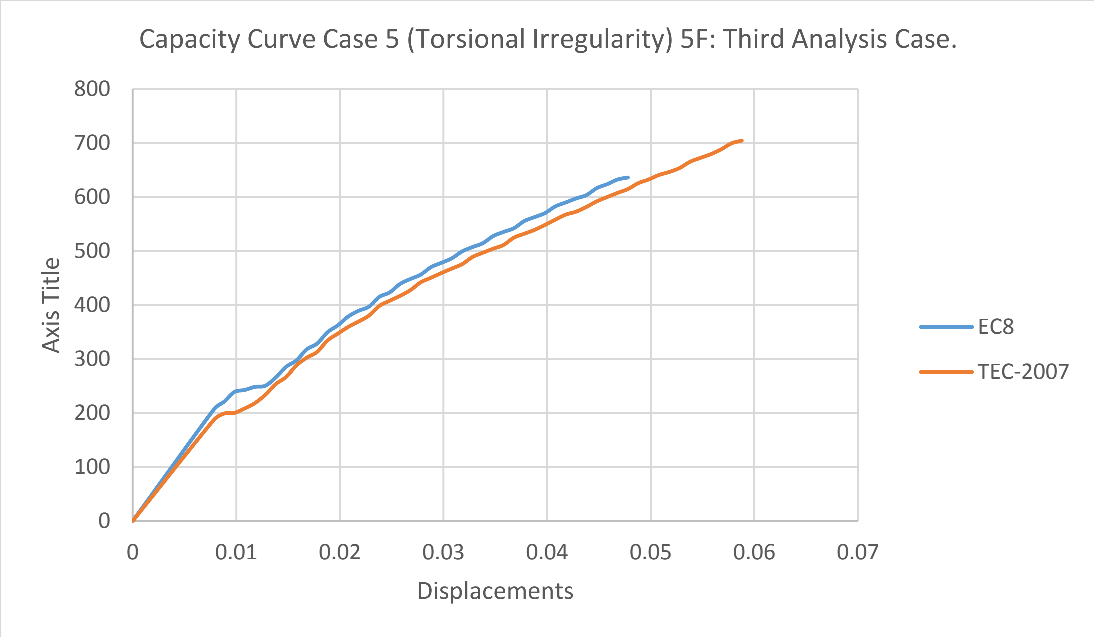


Figure 4.89: Capacity Curve Case 5 (Torsional Irregularity) 3F Case: Third Analysis Case.

In case 5 (Torsional Irregularity) the following has been observed in the capacity curves:

1. In the first analysis where the spectrum, A_0 and the behavior factor where taken according to the advised value defined by the codes:

    - The displacement, at the performance point, reached by the TEC-2007 cases was higher than the displacement reached by the EC8 cases. That can be observed in both the 3F & 5F cases.

    - The EC8 cases have higher capacity than the TEC-2007 cases. That can be observed in the 3F & 5F cases.

    - It has been observed that, with the increase in height of the building the base shear decreases, however the displacement reached by the 5F cases were higher than the displacements reached by the 3F cases for both codes.

2. In the second analysis where the ATC-3 normalized spectrum was used for both the TEC-2007 and the EC8 cases, while using the same value for A_0 and the behavior factor for both cases (0.3 and 6 respectively):

    - The displacement demand in the TEC-2007 cases reached the same value as the displacement demand reached by the EC8 cases. That can be observed in both the 3F & 5F cases.

    - The capacity demand of the EC8 cases are higher than the capacity demand of the TEC-2007 cases. That can be observed in the 3F & 5F cases.

    - It has been observed that, with the increase in height of the building the capacity demand decreases, however the displacement demand reached by the 5F cases were higher than the displacements demand reached by the 3F cases for both codes.

3. In the third analysis where the ATC-3 normalized response spectrum is used for both cases, the TEC-2007 & the EC8 case, but with the advised value of A_0 and behavior factor that are defined according to the code :

- The displacement demand in the TEC-2007 cases was higher than the displacement demand reached by the EC8 cases. That can be observed in both the 3F & 5F cases.

- The capacity demand of the EC8 cases are higher than the capacity demand of the TEC-2007 cases. That can be observed in the 3F & 5F cases.

- It has been observed that, with the increase in height of the building the capacity demand decreases, however the displacement demand reached by the 5F cases were higher than the displacements demand reached by the 3F cases for both codes.

This results is due to the difference in the analysis approach while dealing with this type of irregularity in both codes:

- EC8 when dealing with this irregularity type reduce the behavior factor by 20%

- TEC-2007 when dealing with this type of irregularity, multiply the eccentricity value by a factor $D_i = (\eta_{bi} / 1.2)^{0.5}$

4.4 Damage Report

The damage percentage sustained by the building because of the earthquake forces are presented in this section.

4.4.1 Case 1 (Weak Storey) Case:

Table 4.1: Damage Report for Case 1 (Weak Storey): First Analysis Case.

Case	Case 1 (Weak Storey): First Analysis Case			
Seismic Code	Eurocode 8		TEC-2007	
	Beams	Columns	Beams	Columns
Damage(%) 3F	16.7	0	12.5	0
Damage(%) 5F	20.8	0	16.7	0

Table 4.2: Damage Report for Case 1 (Weak Storey): Second Analysis Case.

Case	Case 1 (Weak Storey): Second Analysis Case			
Seismic Code	Eurocode 8		TEC-2007	
	Beams	Columns	Beams	Columns
Damage(%) 3F	16.7	0	16.7	0
Damage(%) 5F	20.8	0	25	0

Table 4.3: Damage Report for Case 1 (Weak Storey): Third Analysis Case.

Case	Case 1 (Weak Storey): Third Analysis Case			
Seismic Code	Eurocode 8		TEC-2007	
	Beams	Columns	Beams	Columns
Damage(%) 3F	16.7	0	12.5	0
Damage(%) 5F	16.7	0	16.7	0

4.4.2 Case 2 (Soft Storey) Case:

Table 4.4: Damage Report for Case 2 (Soft Storey): First Analysis Case.

Case	Case 2 (Soft Storey): First Analysis Case			
Seismic Code	Eurocode 8		TEC-2007	
	Beams	Columns	Beams	Columns
Damage(%) 3F	20	0	25	0
Damage(%) 5F	30	0	30	0

Table 4.5: Damage Report for Case 2 (Soft Storey): Second Analysis Case.

Case	Case 2 (Soft Storey): Second Analysis Case			
Seismic Code	Eurocode 8		TEC-2007	
	Beams	Columns	Beams	Columns
Damage(%) 3F	20	0	25	0
Damage(%) 5F	30	0	30	0

Table 4.6: Damage Report for Case 2 (Soft Storey): Third Analysis Case.

Case	Case 2 (Soft Storey): Third Analysis Case			
Seismic Code	Eurocode 8		TEC-2007	
	Beams	Columns	Beams	Columns
Damage(%) 3F	15	0	25	0
Damage(%) 5F	30	0	30	0

4.4.3 Case 3 (Projection in Plan):

Table 4.7: Damage Report for Case 3 (Projection in Plan): First Analysis Case.

Case	Case 3 (Projection in Plan): First Analysis Case			
Seismic Code	Eurocode 8		TEC-2007	
	Beams	Columns	Beams	Columns
Damage(%) 3F	29.2	0	29.2	0
Damage(%) 5F	29.4	0	30	0

Table 4.8: Damage Report for Case 3 (Projection in Plan): Second Analysis Case.

Case	Case 3 (Projection in Plan): Second Analysis Case			
Seismic Code	Eurocode 8		TEC-2007	
	Beams	Columns	Beams	Columns
Damage(%) 3F	29.2	0	29.2	0
Damage(%) 5F	30	0	30	0

Table 4.9: Damage Report for Case 3 (Projection in Plan): Third Analysis Case.

Case	Case 3 (Projection in Plan): Third Analysis Case			
Seismic Code	Eurocode 8		TEC-2007	
	Beams	Columns	Beams	Columns
Damage(%) 3F	20.8	0	29.2	0
Damage(%) 5F	29.2	0	30	0

4.4.4 Case 4 (Floor Discontinuity):

Table 4.10: Damage Report for Case 4 (Floor D.) Case: First Analysis Case.

Case	Case 4 (Floor Discontinuity): First Analysis Case			
Seismic Code	Eurocode 8		TEC-2007	
	Beams	Columns	Beams	Columns
Damage(%) 3F	20.8	0	20.8	0
Damage(%) 5F	16.7	0	20	0

Table 4.11: Damage Report for Case 4 (Floor D.): Second Analysis Case.

Case	Case 4 (Floor Discontinuity): Second Analysis Case			
Seismic Code	Eurocode 8		TEC-2007	
	Beams	Columns	Beams	Columns
Damage(%) 3F	20.8	0	20.8	0
Damage(%) 5F	20	0	20	0

Table 4.12: Damage Report for Case 4 (Floor D.): Third Analysis Case.

Case	Case 4 (Floor Discontinuity): Third Analysis Case			
Seismic Code	Eurocode 8		TEC-2007	
	Beams	Columns	Beams	Columns
Damage(%) 3F	20.8	0	20.8	0
Damage(%) 5F	12.5	0	20	0

4.4.5 Case 5 (Torsional Irregularity) Case:

Table 4.13: Damage Report for Case 5 (Torsional Irregularity): First Analysis Case.

Case	**Case 5 (Torsional Irregularity): First Analysis Case**			
Seismic Code	**Eurocode 8**		**TEC-2007**	
	Beams	**Columns**	**Beams**	**Columns**
Damage(%) 3F	1.2	0	1.2	0
Damage(%) 5F	3.4	0	3.4	0

Table 4.14: Damage Report for Case 5 (Torsional Irregularity): Second Analysis Case.

Case	**Case 5 (Torsional Irregularity): Second Analysis Case**			
Seismic Code	**Eurocode 8**		**TEC-2007**	
	Beams	**Columns**	**Beams**	**Columns**
Damage(%) 3F	1.2	0	1.2	0
Damage(%) 5F	3.4	0	3.4	0

Table 4.15: Damage Report for Case 4 (Floor D.): Third Analysis Case.

Case	**Case 5 (Torsional Irregularity): Third Analysis Case**			
Seismic Code	**Eurocode 8**		**TEC-2007**	
	Beams	**Columns**	**Beams**	**Columns**
Damage(%) 3F	1.2	0	1.2	0
Damage(%) 5F	3.4	0	3.4	0

4.5 Cost

4.5.1 Case 1 (Weak Storey):

The quantity and cost of the different material used. Allong with the total cost of the buildings are presented in this section.

4.5.1.1 3 Floor:

Table 4. 16: Cost of Case 1 (Weak Storey) 3F: First Analysis Case.

Type	Cost	Quantity		Costs (USD)		Total Cost (USD)	
	USD	EC8	TEC-2007	EC8	TEC-2007	EC8	TEC-2007
C20 factory concrete (m^3)	75	262.2	262.2	19,664.1	19,664.1	49,864.54	50,067.72
Plain surface concrete form in (m^2)	11	1632.8	1632.8	17,960.91	17,960.91		
Reinforcement steel ϕ 8-12 mm in (tn)	600	14.284	14.344	8,570.32	8,606.43		
Reinforcement steel ϕ 14-50 mm in (tn)	600	6.1	6.4	3,669.21	3,836.28		

Table 4.17: Cost Case 1 (Weak Storey) 3F Case: Second Analysis Case.

Type	Cost	Quantity		Costs (USD)		Total Cost (USD)	
	USD	EC8	TEC-2007	EC8	TEC-2007	EC8	TEC-2007
C20 factory concrete (m^3)	75	262.2	262.2	19,664.1	19,664.1	49,989.37	50,163.39
Plain surface concrete form (m^2)	11	1632.8	1632.8	17,960.91	17,960.91		
Reinforcement steel ϕ 8-12 mm (tn)	600	14.2	14.4	8,537.71	8,632.77		
Reinforcement steel ϕ 14-50 mm (tn)	600	6.4	6.5	3,826.65	3,905.61		

Table 4.18: Cost Case 1 (Weak Storey) Case 3F: Third Analysis Case.

Type	Cost	Quantity		Costs (USD)		Total Cost (USD)	
	USD	EC8	TEC-2007	EC8	TEC-2007	EC8	TEC-2007
C20 factory concrete (m^3)	75	262.2	262.2	19,664.1	19,664.1	49,911.39	50,052.05
Plain surface concrete form (m^2)	11	1632.8	1632.8	17,960.91	17,960.91		
Reinforcement steel ϕ 8-12	600	14.2	14.4	8,537.71	8,628.19		

mm (tn)							
Reinforcement steel ϕ 14-50 mm (tn)	600	6.2	6.3	3,749.38	3,798.85		

4.5.1.2 5 Floor:

Table 4.19: Cost of Case 1 (Weak Storey) 5F: First Analysis Case.

Type	Cost	Quantity		Costs (USD)		Total Cost (USD)	
	USD	EC8	TEC-2007	EC8	TEC-2007	EC8	TEC-2007
C20 factory concrete (m^3)	75	433.4	433.4	32,503.50	32,503.50	82,368.39	82,839.68
Plain surface concrete form (m^2)	11	2685.3	2685.3	29,538.85	29,538.85		
Reinforcement steel ϕ 8-12 mm (tn)	600	23.5	23.6	14,127.76	14,176.59		
Reinforcement steel ϕ 14-50 mm (tn)	600	10.3	11.0	6,198.28	6,620.74		

Table 4.20: Cost Case 1 (Weak Storey) 5F: Second Analysis Case.

Type	Cost	Quantity		Costs (USD)		Total Cost (USD)	
	USD	EC8	TEC-2007	EC8	TEC-2007	EC8	TEC-2007
C20 factory concrete (m^3)	75	433.4	433.4	32,503.5	32,503.5	82,670.62	82,997.32
Plain surface concrete form (m^2)	11	2685.3	2685.3	29,538.85	29,538.85		
Reinforcement steel ϕ 8-12 mm (tn)	600	23.6	23.8	14,176.62	14,292.56		
Reinforcement steel ϕ 14-50 mm (tn)	600	10.8	11.1	6,451.65	6,662.41		

Table 4.21: Cost Case 1 (Weak Storey) 5F: Third Analysis Case.

Type	Cost	Quantity		Costs (USD)		Total Cost (USD)	
	USD	EC8	TEC-2007	EC8	TEC-2007	EC8	TEC-2007
C20 factory concrete (m^3)	75	433.4	433.4	32,503.5	32,503.5	82,441.77	82,826.46
Plain surface concrete form (m^2)	11	2685.3	2685.3	29,538.85	29,538.85		
Reinforcement steel ϕ 8-12 mm (tn)	600	23.5	23.6	14,077.43	14,166.52		
Reinforcement steel ϕ 14-50 mm (tn)	600	10.5	11.0	6,321.99	6,617.59		

4.5.2 Case 2 (Soft Storey) Case:

4.5.2.1 3 Floor:

Table 4.22: Cost of Case 2 (Soft Storey) 3F Case: First Analysis Case.

Type	Cost	Quantity		Costs (USD)		Total Cost (USD)	
	USD	EC8	TEC-2007	EC8	TEC-2007	EC8	TEC-2007
C20 factory concrete (m^3)	75	245.1	245.1	18,383.62	18,383.62	53,518.56	54,439.07
Plain surface concrete form (m^2)	11	1568	1568	17,253.88	17,253.88		
Reinforcement steel ϕ 8-12 mm (tn)	600	15.3	15.7	9,159.77	9,449.55		
Reinforcement steel ϕ 14-50 mm (tn)	600	14.5	15.6	8,721.29	9,352.02		

Table 4.23: Cost Case 2 (Soft Storey) 3F Case: Second Analysis Case.

Type	Cost	Quantity		Costs (USD)		Total Cost (USD)	
	USD	EC8	TEC-2007	EC8	TEC-2007	EC8	TEC-2007
C20 factory concrete (m^3)	75	245.1	245.1	18,383.62	18,383.62	53,747.08	54,229.2
Plain surface concrete form (m^2)	11	1568.5	1568.5	17,253.88	17,253.88		

Reinforcement steel ϕ 8-12 mm (tn)	600	15.5	15.3	9,270.90	9,163.86		
Reinforcement steel ϕ 14-50 mm (tn)	600	14.7	15.7	8,838.68	9,427.84		

Table 4.24: Cost Case 2 (Soft Storey) Case 3F: Third Analysis Case.

Type	Cost	Quantity		Costs (USD)		Total Cost (USD)	
	USD	EC8	TEC-2007	EC8	TEC-2007	EC8	TEC-2007
C20 factory concrete (m^3)	75	245.1	245.1	18,383.62	18,383.62	53,580.03	54,265.52
Plain surface concrete form (m^2)	11	1568.5	1568.5	17,253.88	17,253.88		
Reinforcement steel ϕ 8-12 mm (tn)	600	15.3	15.6	9,166.33	9,359.31		
Reinforcement steel ϕ 14-50 mm (tn)	600	14.6	15.4	8,776.2	9,268.71		

4.5.2.2 5 Floor:

Table 4.25: Cost of Case 2 (Soft Storey) 5F Case: First Analysis Case.

Type	Cost	Quantity		Costs (USD)		Total Cost (USD)	
	USD	EC8	TEC-2007	EC8	TEC-2007	EC8	TEC-2007
C20 factory concrete (m^3)	75	397	397	29,773.57	29,773.57	86,364.76	88,239.34
Plain surface concrete form (m^2)	11	2497.9	2497.9	27,476.51	27,476.51		
Reinforcement steel ϕ 8-12 mm (tn)	600	25	25.9	14,989.79	15,541.02		
Reinforcement steel ϕ 14-50 mm (tn)	600	23.5	25.7	14,124.89	15,448.24		

Table 4.26: Cost Case 2 (Soft Storey) 5F Case: Second Analysis Case.

Type	Cost	Quantity		Costs (USD)		Total Cost (USD)	
	USD	EC8	TEC-2007	EC8	TEC-2007	EC8	TEC-2007
C20 factory concrete (m^3)	75	397	397	29,773.57	29,773.57	86,452.25	87,687.71
Plain surface concrete form (m^2)	11	2497.9	2497.9	27,476.51	27,476.51		
Reinforcement steel ϕ 8-12 mm (tn)	600	25.0	25.2	15,011.00	15,125.57		
Reinforcement steel ϕ 14-50 mm (tn)	600	23.7	25.5	14,191.17	15,312.06		

Table 4.27: Cost Case 2 (Soft Storey) Case 5F: Third Analysis Case.

Type	Cost	Quantity		Costs (USD)		Total Cost (USD)	
	USD	EC8	TEC-2007	EC8	TEC-2007	EC8	TEC-2007
C20 factory concrete (m^3)	75	397	397	29,773.57	29,773.57	86,394.9	88,056.98
Plain surface concrete form (m^2)	11	2497.9	2497.9	27,476.51	27,476.51		
Reinforcement steel ϕ 8-12 mm (tn)	600	25.0	25.8	15,011.00	15,494.88		
Reinforcement steel ϕ 14-50 mm (tn)	600	23.6	25.5	14,133.82	15,312.06		

4.5.3 Case 3 (Projection in Plan):

4.5.3.1 3 Floor:

Table 4.28: Cost of Case 3 (Projection in Plan) 3F Case: First Analysis Case.

Type	Cost	Quantity		Costs (USD)		Total Cost (USD)	
	USD	EC8	TEC-2007	EC8	TEC-2007	EC8	TEC-2007
C20 factory concrete (m^3)	75	373.8	373.8	28,036.54	28,036.54	89,922.6	90,956.02
Plain surface concrete form	11	2209.4	2209.4	24,303.67	24,303.67		

(m^2)							
Reinforcement steel ϕ 8-12 mm (tn)	600	23.8	23.6	14,301.16	14,182.84		
Reinforcement steel ϕ 14-50 mm (tn)	600	38.8	40.7	23,281.23	24,432.97		

Table 4.29: Cost Case 3 (Projection in Plan) 3F Case: Second Analysis Case.

Type	Cost	Quantity		Costs (USD)		Total Cost (USD)	
	USD	EC8	TEC-2007	EC8	TEC-2007	EC8	TEC-2007
C20 factory concrete (m^3)	75	373.8	373.8	28,036.54	28,036.54	90,060.57	91,013.03
Plain surface concrete form (m^2)	11	2209.4	2209.4	24,303.67	24,303.67		
Reinforcement steel ϕ 8-12 mm (tn)	600	23.8	23.7	14,284.20	14,204.06		
Reinforcement steel ϕ 14-50 mm (tn)	600	39.1	40.8	23,436.16	24,468.76		

Table 4.30: Cost Case 3 (Projection in Plan) 3F: Third Analysis Case.

Type	Cost	Quantity		Costs (USD)		Total Cost (USD)	
	USD	EC8	TEC-2007	EC8	TEC-2007	EC8	TEC-2007
C20 factory concrete (m^3)	75	373.8	373.8	28,036.54	28,036.54	89,964.39	90,951.92
Plain surface concrete form (m^2)	11	2209.4	2209.4	24,303.67	24,303.67		
Reinforcement steel ϕ 8-12 mm (tn)	600	23.9	23.6	14,317.13	14,181.78		
Reinforcement steel ϕ 14-50 mm (tn)	600	38.8	40.7	23,307.05	24,429.93		

4.5.3.2 5 Floor:

Table 4.31: Cost of Case 3 (Projection in Plan) 5F Case: First Analysis Case.

Type	Cost	Quantity		Costs (USD)		Total Cost	
	USD	EC8	TEC-2007	EC8	TEC-2007	EC8	TEC-2007
C20 factory concrete (m^3)	75	618.7	618.7	46,401.97	46,401.97	148,004.38	149,501.95
Plain surface concrete form (m^2)	11	3672.5	3672.5	40,397.58	40,397.58		
Reinforcement steel ϕ 8-12 mm (tn)	600	42.4	42.2	25,438.94	25,341.35		
Reinforcement steel ϕ 14-50 mm (tn)	600	59.6	62.7	35,765.89	37,361.05		

Table 4. 32: Cost Case 3 (Projection in Plan) 5F Case: Second Analysis Case.

Type	Cost	Quantity		Costs (USD)		Total Cost (USD)	
	USD	EC8	TEC-2007	EC8	TEC-2007	EC8	TEC-2007
C20 factory concrete (m^3)	75	618.7	618.7	46,401.97	46,401.97	148,280.09	149,919.08
Plain surface concrete form (m^2)	11	3672.5	3672.5	40,397.58	40,397.58		
Reinforcement steel ϕ 8-12 mm (tn)	600	42.5	42.3	25,475.53	25,373.96		
Reinforcement steel ϕ 14-50 mm (tn)	600	60	62.9	36,005.01	37,745.57		

Table 4.33: Cost Case 3 (Projection in Plan) 5F: Third Analysis Case.

Type	Cost	Quantity		Costs (USD)		Total Cost (USD)	
	USD	EC8	TEC-2007	EC8	TEC-2007	EC8	TEC-2007
C20 factory concrete (m^3)	75	618.7	618.7	46,401.97	46,401.97	148,057.73	149,718.86
Plain surface concrete form (m^2)	11	3672.5	3672.5	40,397.58	40,397.58		
Reinforcement steel ϕ 8-12	600	42.4	42.2	25,435.57	25,339.88		

mm (tn)							
Reinforcement steel ϕ 14-50 mm (tn)	600	59.7	62.6	35,822.61	37,576.43		

4.5.4 Case 4 (Floor Discontinuity):

4.5.4.1 3 Floor:

Table 4.34: Cost of Case 4 (Floor D.) 3F Case: First Analysis Case.

Type	Cost	Quantity		Costs (USD)		Total Cost (USD)	
	USD	EC8	TEC-2007	EC8	TEC-2007	EC8	TEC-2007
C20 factory concrete (m^3)	75	215	215	16,122.94	16,122.94	62,797.35	63,329.59
Plain surface concrete form (m^2)	11	2056.6	2056.6	22,622.48	22,622.48		
Reinforcement steel ϕ 8-12 mm (tn)	600	22.8	20.3	13,357.03	12,195.52		
Reinforcement steel ϕ 14-50 mm (tn)	600	17.8	20.6	10,694.9	12,388.65		

Table 4.35: Cost Case 4 (Floor D.) 3F Case: Second Analysis Case.

Type	Cost	Quantity		Costs (USD)		Total Cost (USD)	
	USD	EC8	TEC-2007	EC8	TEC-2007	EC8	TEC-2007
C20 factory concrete (m^3)	75	215	215	16,122.94	16,122.94	63,144.11	64,206.74
Plain surface concrete form (m^2)	11	2056.6	2056.6	22,622.48	22,622.48		
Reinforcement steel ϕ 8-12 mm (tn)	600	22.8	21.9	13,686.34	13,138.94		
Reinforcement steel ϕ 14-50 mm (tn)	600	17.8	20.5	10,682.35	12,322.38		

Table 4.36: Cost Case 4 (Floor D.) 3F: Third Analysis Case.

Type	Cost	Quantity		Costs (USD)		Total Cost (USD)	
	USD	EC8	TEC-2007	EC8	TEC-2007	EC8	TEC-2007
C20 factory concrete (m^3)	75	215	215	16,122.94	16,122.94	62,888.33	62,972.25
Plain surface concrete form (m^2)	11	2056.6	2056.6	22,622.48	22,622.48		
Reinforcement steel ϕ 8-12 mm (tn)	600	22.9	20.3	13,754.32	12,182.44		
Reinforcement steel ϕ 14-50 mm (tn)	600	17.3	20.1	10,388.59	12,044.39		

4.5.4.2 5 Floor:

Table 4.37: Cost of Case 4 (Floor D.) 5F Case: First Analysis Case.

Type	Cost	Quantity		Costs (USD)		Total Cost (USD)	
	USD	EC8	TEC-2007	EC8	TEC-2007	EC8	TEC-2007
C20 factory concrete (m^3)	75	357.2	357.2	26,793.75	26,793.75	102,920.02	103,698.2
Plain surface concrete form (m^2)	11	3362.7	3362.7	36,989.53	36,989.53		
Reinforcement steel ϕ 8-12 mm (tn)	600	39.5	36.2	23,695.82	21,714.1		
Reinforcement steel ϕ 14-50 mm tn)	600	25.7	30.3	15,440.92	18,200.82		

Table 4.38: Cost Case 4 (Floor D.) 5F Case: Second Analysis Case.

Type	Cost	Quantity		Costs (USD)		Total Cost (USD)	
	USD	EC8	TEC-2007	EC8	TEC-2007	EC8	TEC-2007
C20 factory concrete (m^3)	75	357.2	357.2	26,793.75	26,793.75	102,682.49	104,065.82
Plain surface concrete form (m^2)	11	3362.7	3362.7	36,989.53	36,989.53		
Reinforcement steel ϕ 8-	600	39.5	38	23,686.78	22,823.28		

12 mm (tn)							
Reinforcement steel ϕ 14-50 mm (tn)	600	25.4	29.1	15,212.43	17,459.26		

4.5.5 Case 5 (Torsional Irregularity):

4.5.5.1 3 Floor:

Table 4.39: Cost of Case 5 (Torsional Irregularity) 3F Case: First Analysis Case.

Type	Cost	Quantity		Costs (USD)		Total Cost	
	USD	EC8	TEC-2007	EC8	TEC-2007	EC8	TEC-2007
C20 factory concrete (m^3)	75	436.1	436.1	32,709.36	32,709.36	84,586.13	85,227.18
Plain surface concrete form (m^2)	11	2707.3	2707.3	29,779.8	29,779.8		
Reinforcement steel ϕ 8-12 mm (tn)	600	31.1	28.3	18,661.12	17,007.84		
Reinforcement steel ϕ 14-50 mm (tn)	600	5.7	9.6	3,435.85	5,730.18		

Table 4.40: Case 5 (Torsional Irregularity) 3F Case: Second Analysis Case.

Type	Cost	Quantity		Costs (USD)		Total Cost (USD)	
	USD	EC8	TEC-2007	EC8	TEC-2007	EC8	TEC-2007
C20 factory concrete (m^3)	75	436.1	436.1	32,709.36	32,709.36	84,609.11	85,246.43
Plain surface concrete form (m^2)	11	2707.3	2707.3	29,779.80	29,779.80		
Reinforcement steel ϕ 8-12 mm (tn)	600	31.1 tn	28.4	18,675.42	17,025.87		
Reinforcement steel ϕ 14-50 mm (tn)	600	5.7 tn	9.6	3,444.53	5,731.4		

Table 4.41: Cost Case 5 (Torsional Irregularity) 3F: Third Analysis Case.

Type	Cost	Quantity		Costs (USD)		Total Cost (USD)	
	USD	EC8	TEC-2007	EC8	TEC-2007	EC8	TEC-2007
C20 factory concrete (m^3)	75	436.1	436.1	32,709.36	32,709.36	84,609.11	85,227.05
Plain surface concrete form (m^2)	11	2707.3	2707.3	29,779.80	29,779.80		
Reinforcement steel ϕ 8-12 mm (tn)	600	31.1 tn	28.3	18,675.42	17,006.68		
Reinforcement steel ϕ 14-50 mm (tn)	600	5.7 tn	9.6	3,444.53	5,731.21		

4.5.5.2 5 Floor:

Table 4.42: Cost of Case 5 (Torsional Irregularity) 5F Case: First Analysis Case.

Type	Cost	Quantity		Costs (USD)		Total Cost (USD)	
	USD	EC8	TEC-2007	EC8	TEC-2007	EC8	TEC-2007
C20 factory concrete (m^3)	75	725.0	725.0	54,372.40	54,372.40	140,842.04	141,991.52
Plain surface concrete form (m^2)	11	4505.1 m^2	4505.1 m^2	49,555.85	49,555.85		
Reinforcement steel ϕ 8-12 mm (tn)	600	51.6 tn	47.1 tn	30,988.83	28,235.30		
Reinforcement steel ϕ 14-50 mm (tn)	600	9.9	16.4	5,924.96	9,824.97		

Table 4.43: Case 5 (Torsional Irregularity) 5F Case: Second Analysis Case.

Type	Cost	Quantity		Costs (USD)		Total Cost (USD)	
	USD	EC8	TEC-2007	EC8	TEC-2007	EC8	TEC-2007
C20 factory concrete (m^3)	75	725.0 m^3	725.0	54,372.40	54,372.40	140,975	141,956
Plain surface concrete form (m^2)	11	4505.1 m^2	4505.1 m^2	49,555.85	49,555.85		
Reinforcement steel ϕ 8-12 mm (tn)	600	51.6 tn	47.0 tn	30,987.89	28,215.58		
Reinforcement steel ϕ 14-50 mm (tn)	600	10.1 tn	16.4 tn	6,058.83	9,812.14		

Table 4.44: Cost Case 5 (Torsional Irregularity) 3F: Third Analysis Case.

Type	Cost	Quantity		Costs (USD)		Total Cost (USD)	
	USD	EC8	TEC-2007	EC8	TEC-2007	EC8	TEC-2007
C20 factory concrete (m^3)	75	725.0	725.0	54,372.40	54,372.40	140,860.96	141,857.43
Plain surface concrete form (m^2)	11	4505.1	4505.1	49,555.85	49,555.85		
Reinforcement steel ϕ 8-12 mm (tn)	600	51.6	47.1	30,932.31	28,247.91		
Reinforcement steel ϕ 14-50 mm (tn)	600	10.0	16.1	6,000.40	9,681.27		

4.5.6 Verdict on Cost

It can be clearly seen that around all the cases the buildings designed according to Eurocode 8 are slightly more economical, compared to the ones designed according to TEC-2007 up to 1%. This small difference is due to the larger amount of steel utilized for reinforcement in the cases done according to TEC-2007.

Chapter 5

CONCLUSION AND RECOMMENDATION FOR FUTURE WORK

5.1 Conclusion

Since long ago, the central objective of engineers has been to establish a set of rules and criteria to counter the harmful effect on structures that an earthquake can have. In this study two well-known codes that are being used today are compared, the Eurocode 8 and TEC-2007, in terms of performance, cost and damage percentage. From the three analysis cases that were applied to the sixty building case, all of the cases, whether they were designed according to TEC-2007 or EC8, reached the same performance level of life safety, but there were some differences in the analysis results. The following has been concluded:

1. Capacity curves: from three different analysis cases that have been used, it was concluded that the ductility plays a major role in the outcome of the capacity curves: since high ductility level leads to lower capacity demand and a higher displacement demand throughout all the cases.

2. Damage percentage: throughout the three analysis cases it has been found that:

a. In 36 building cases the damage sustained by the TEC-2007 cases was equal to the damage sustained by the EC8 cases in the following cases:

    - First analysis case: 3 floor cases of case 3, 4 and 5. Also, the 5 floor cases of case 2 and 5.

    - Second analysis case: 3 floor cases of case 1, 2, 3, 4 and 5. Also, the 5 floor cases of case 3, 4, 5.

    - Third analysis case: 3 floor cases of case 4 and 5. Also, the 5 floor cases of case 1, 2 and 5.

b. In 18 building cases, the damage sustained by TEC-2007 cases was higher than the damage sustained by the EC8 cases in the following cases:

    - Fist analysis case: 3 floor cases of case 2. Also, 5 floor cases of case 3 and 4.

    - Second analysis case: 3 floor cases of case 2. Also, 5 floor case of case 1.

c. In six building cases, the damage sustained by the TEC-2007 cases was lower than the damage sustained by the EC8 cases in the following cases:

- First analysis case: 3 & 5 floor cases of case 1.

- Third analysis case: 3 floor cases of case 1.

3. Cost: there is no major difference in cost throughout all the cases that have ben studied. However, it is worth to mention that because of the higher ductility in the TEC-2007 cases the demand for more reinforcement steel was needed in the TEC-2007 cases compared with the EC8 cases. However, the low cost of the reinforcement steel result in the small difference in cost (lower than 1%).

The analysis results obtained throughout this study lead to the conclusion that both EC8 and TEC-2007 deal with the earthquake almost in the same manner with little difference in the analysis approach and some of the conditions and requirements.

5.2 Recommendation for Future studies

Taking the work performed in this study into consideration, these are future advices and proposal for future research work:

- Building with higher elevation can be investigated.

- Working on a different method of analysis, time history for instance.

- Working on a different seismic zone.

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